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Joint stress based deflection limits for transmission line towers

B. Gayathri and Raghavan Ramalingam*

SASTRA University, Thanjavur - 613401, Tamil Nadu, India

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Abstract. Experimental investigations have revealed significant mismatches between analytical estimates and experimentally measured deflections of transmission towers. These are attributed to bolt slip and joint flexibility. This study focuses on effects of joint flexibility on tower deflections and proposes criterions for permissible deflection limits based on the stresses in joints. The objective has been framed given that guidelines are not available in the codes of practices for transmission towers with regard to the permissible limits of deflection. The analysis procedure is geometric and material nonlinear with consideration of joint flexibility in the form of extension or contraction of the cover plates. The deflections due to bolt slip are included in the study by scaling up the deflections obtained from analysis by a factor. Using the results of the analysis, deflection limits for the towers are proposed by limiting the stresses in the joints. The obtained limits are then applied to a new full scale tower to demonstrate the application of the current study.

Keywords: transmission line tower; permissible deflection; nonlinear analysis; joint deformations

1. Introduction

Transmission line towers are tall lattice structures that undergo large deformations similar to cantilever mode at their working loads. In traditional analysis of transmission line towers, truss elements are employed for the members. Axial forces predominate the structural action and design, and bending moments obtained even by modelling the tower with frame elements would be negligible. The reliability of transmission line towers comprises of both strength and serviceability aspects that pertain to the ultimate collapse and deflection behaviour of the tower respectively. Design standards and guidelines provide instruction on ensuring the strength of transmission line towers of any general loading or configuration. The deflection of the cross arms and groundwire peak influence the cable sag and tensions used in calculating the loads acting on the tower (Fedor and Czmoch 2016, Peyrot and Goulois 1979). The stake in serviceability of overhead transmission lines arises from (a) changes in cable sag and tension, (b) to control secondary effects from vertical eccentricities and (c) and cross-wind vibrations and other effects associated with slender structures (Deng *et al*. 2016, Li *et al*. 2012). A serviceability or deflection limit would serve largely as an indirect quantity for this ‘slenderness’ of the tower. However, design standards do not contain any instruction regarding the deflections that can be expected or permitted during the usage of the structure. The absence of deflection guidelines is generally attributed to difficulties in arriving at an accurate estimate of the deflection analytically. The large deflection study of transmission towers becomes essential in ultimate collapse predictions. In addition, phenomenon such as bolt slip and cover plate flexibility increase the tower deflections which are difficult to obtain analytically for design purposes.

Research has focused extensively on large deflection and nonlinear analysis of towers that have brought to light the influence of material and geometric nonlinearities on the ultimate collapse of transmission towers (Yang and Hong 2016) and also on the member force distributions for towers of various configurations (Rao *et al.* 2010). Nonlinear formulations adopting frame elements (as opposed to truss elements generally adopted during design) have also been attempted to incorporate both geometric and material nonlinearity (Al-Bermani and Kitipornchai 1992). Extensions of such analytical procedures have also been used to model and replicate full scale experimental tests of transmission towers (Albermani *et al.* 2009). Such nonlinear analytical studies have not been able to justify the deflections obtained from the analysis, even though they have been successful in capturing the ultimate collapse loads and failure patterns.

Studies on improving the analytical deflection of towers has largely comprised work on the inclusion of bolt slip deflection. Bolt slip is the relative slip between the elements connected by a bolt group. Indirect techniques to include the bolt slip displacement have been proposed by Kitipornchai *et al.* (1992) including the sudden and continuous slip models. Rao *et al.* (2012) measured the relative rotation at leg member joints due to bolt slip and used these measured rotations to modify the deflection results obtained from analysis. According to the results reported therein, the experimentally measured values of deflection through tests on full scale towers at 100% of the design loads are 1.3 to 1.8 times the deflection obtained through analysis without considering bolt slip. However, such a procedure would require bolt slip tests (refer, IS:802-

∗Corresponding author, Ph.D.
E-mail: raghavan.ramalingam@gmail.com

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1984 part 3) to be conducted on full-scale towers before modifications can be applied to the analytical results of similar towers. Further studies on the influence of bolt slip on transmission tower deformation have been published in recent years (Jiang *et al.* 2011, Baran *et al.* 2016). Bolt slip is a sudden occurrence, a rigid body motion, and no methodology for inclusion in the stiffness derivations exist in literature. This difficulty is offset by the finding that bolt slip has very little effect on the ultimate loads of transmission line towers (Kitipornchai *et al.* 1992, Kroeker 2000). Component level studies on bolt slip through finite element modelling or empirical load-slip displacement relations (Reid and Hiser 2005, Ungkurapinan *et al.* 2003) to understand the mechanics of bolt slip cannot be conveniently integrated into structural analysis, nor are they applicable to structures with different types of bolted joints. A robust finite element model for load distribution and slip in bolted joints has been presented by Gray and McCarthy (2011). Blachowski and Gutkowski (2016) studied the effects of bolt pretension on the deformation of communication towers with circular bolted flange connections. Xu and He (2017) discuss complexities in the modelling of such components.

The cover plates used in the leg member joints act as a semi-rigid connection owing to their flexibility. The structural response of frames with flexible moment connections are widely available in literature (Kartal *et al.* 2010, Sagiroglu and Aydin 2015). However, as stated earlier, the members of transmission towers predominantly carry loads as axial forces only owing to their truss or reticulated nature. Hence, modelling connections with rotational flexibility for transmission towers may be unnecessary given that the largest component of stresses in the leg member cover plates would be axial stresses. Semi-rigid analysis of reticulated (truss) structures has focused mainly on single-layer shells employing manufactured node connections (Ahmedizadeh and Maleek 2014, Ma *et al.* 2015, Cai *et al.* 2017).

In view of the above discussions on the effect of bolt slip and cover plate flexibility on tower deflections, it can be said that these two phenomenons need to be considered in the analysis in order to have closer estimates of tower deflection. It can also be said that any proposal for a deflection limit guideline will have to account for the effect of bolt slip and joint flexibility. This is crucial given that because of the complexities, a designer would still be at an error if the limit guidelines do not account for these effects. Among the two, the inclusion of bolt slip into the structural equations is more difficult than joint flexibility, given the discontinuous nature of bolt slip displacement. It is more convenient to search for suitable criteria for a deflection limit based on the more continuous part of the joint deformation, namely that due to joint flexibility. This joint deformation may be considered as axial deformations (extension or contraction) given that axial stresses are the primary stresses in transmission tower members. This study adopts the stresses in the leg member cover plate joints as the controlling parameter based on which certain allowable deflection limits are proposed for a limited height to base width range of transmission line towers.

2. Tower modelling

In design practice, transmission line towers are normally modelled for analysis in software using truss elements given their lattice type of structural geometry. Literature on transmission line tower studies also frequently use the nomenclature “lattice towers”. The use of frame elements have also been reported in literature for transmission tower members, but these are mainly to provide insights into material nonlinearities and study local effects. In modelling using truss elements, care must be taken with regard to the secondary bracings of the tower. The secondary bracing joints with the primary bracings, leg members and belt members are locations of planar joint instabilities. This is a situation where a low or zero stiffness exists in the joint perpendicular to the plane in which the secondary bracing lies. Rectification of this problem would require addition of weak springs or elements in the perpendicular direction. More conveniently, design procedures generally omit the secondary bracings from the tower model. Secondary bracings are separately designed to restrain (reduce slenderness) the leg members and primary bracings by designing them for 1.5-2.0% of the force in the leg members and primary bracings. In using a truss analysis, the nodes of the model are ideal fully released pins. Thus from conventional understanding, there is no semi-rigidity in the joint, no joint deformation and no joint stresses. In contrast, this study considers the extension or contraction of the cover plates as the flexibility of the joint and the associated stress as the joint stresses.

3. Methodology

The transmission towers are selected on the basis of commonly encountered tower heights and only small angle towers are considered. The height to base-width ratios also need to be within a range in which the deflected shapes can be expected to be similar. Two towers of significant height difference are chosen for this purpose. The first is a 33 kV AT-type tower reported Rao *et al.* (2012) of height 26.275 m and base width 6.8 m (Fig. 1) and the second is a double circuit tower of height 50 m and base width 8.5 m (Fig. 2). It is felt that this height range is a suitable one within which a majority of transmission line towers are contained and hence any limits proposed for towers in this height range would be of practical benefit.

The material properties of the members and joint cover plates used in all examples are as follows: Yield stress 250 N/mm^2, ultimate tensile stress 410 N/mm^2 and Young’s modulus of elasticity 2×10^5 N/mm^2. Linear analysis for the towers is performed using general structural analysis software. The linear analysis is done for verifying the structural adequacy of the members used in the tower. The analysis is done using a truss model with bottom supports assumed to be hinged connections. The deflection obtained from such linear analysis will be considerably lesser than the actual experimental deflections of the tower. Transverse and vertical loads are present but the transverse deflections are of primary interest as they are more critical given their larger magnitude. Experimental result of the tower transverse

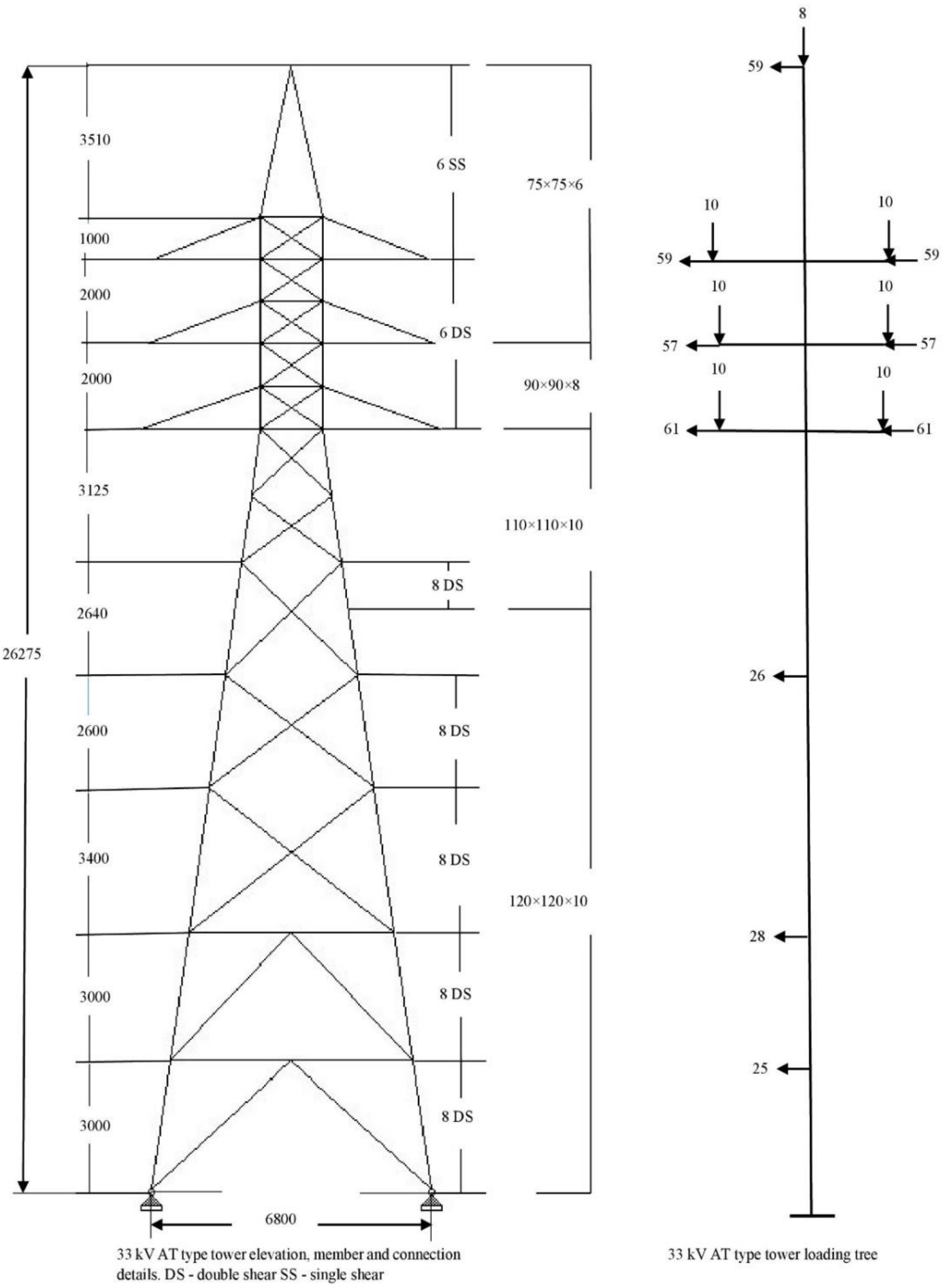

Fig. 1 33 kV AT Type tower elevation and loading tree (units: kN, mm)

Table 1 Comparison of groundwire transverse deflections

Description	AT tower	Double circuit tower
Experimental value	340 mm	Not available
Analytical value	245 mm	219mm
Error	95 mm	Not available

deflection for the AT-type tower (340 mm) is available in the reference. The transverse deflection at the groundwire location for both towers are shown in Table 1. Experimentally measured deflection is not available for the double circuit tower but similar error would be inherent in the analytical deflection estimate. The maximum deflection under the loading shown varies from maximum at the groundwire peak and successively lesser deflections will be present at the cross-arm levels. In practice, only transverse and longitudinal deflections of towers are measured in experimental tests, usually performed upon requests by clients for certification. Testing protocols and rigging arrangements are available in transmission line tower testing codes such as Indian Standard IS:802 part 3 and International Electrotechnical Commission IEC 60652 (2002). This includes the bolt slip test where loading is done for 50% of design load and then released, with bolt slip displacement being the difference between the displacements at the two load levels.

After structural adequacy of the members is ensured, next the adequacy of the cover plates used for the member connections is checked by estimating the state of stress existing in the cover plates. Under a specified set of loads

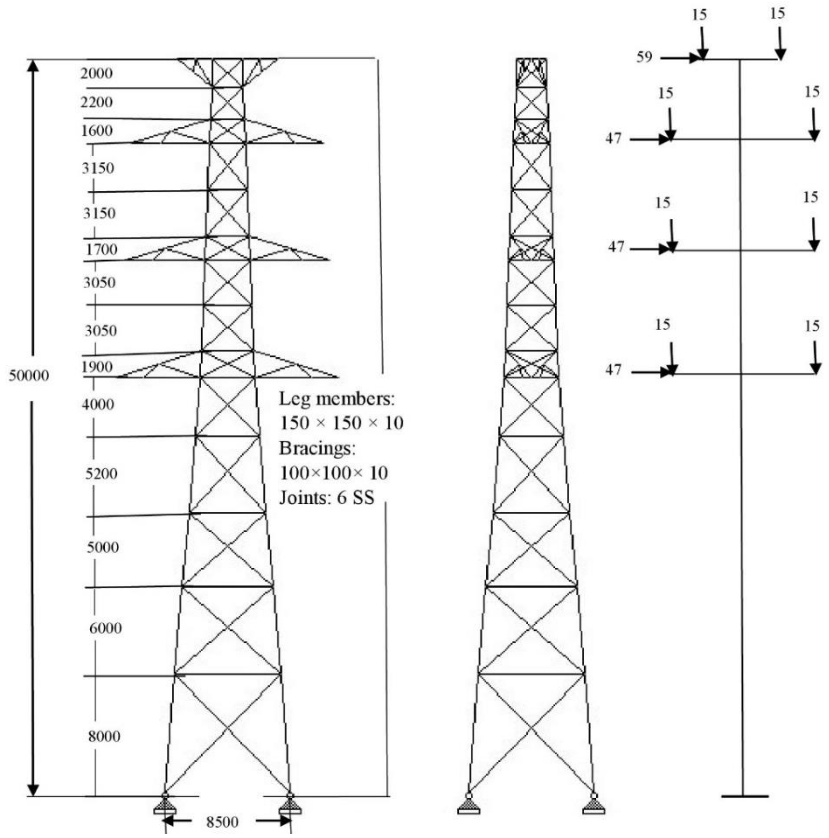

Fig. 2 Double circuit tower elevation and loading tree (units: kN, mm)

and resulting member forces, each of the joints connecting the various members of the tower are subjected to stresses in the plane of the connection. Under no circumstances can the connecting plates be allowed to undergo rupture under high tensile forces or excessive deformations under high compressive forces. In practice, only the leg members are connected with the cover plates and the other members connect directly to the leg members at a location sufficiently away from the cover plate to avoid local stress concentrations and also for ease of fabrication. However, in the present methodology, the joints are checked for adequacy under combined axial and shear stresses, with the forces coming from the nearest horizontal member taken for the consideration of shear on the connecting plate. Such a consideration, though not backed by actual construction practice, would only lead to conservative values for the final deflection limits obtained.

The joint stresses are evaluated using the results obtained for the member forces and the expression for combined axial and shear stresses given in Eq. (1). In the equation, P is the force acting in the leg members and Q is the force acting in the horizontal members connecting near the joint. A_1 and A_2 are the cross sections of the connecting plate in the direction of axial stress (direction of load P) and direction of shear stress (direction of load Q) respectively. The limiting stress assuming E250 grade steel plate is found to be 227 N/mm² with material safety factor γ_{m0} being 1.10. Information for the size of the cover plates for these towers is not found in the reference. Therefore, the thickness was conservatively taken to be the same thickness as the members that it connects. Information regarding the number of bolts and number of shear planes is found in the reference. This is used to arrive at appropriate lengths for the cover plates based on clauses given in the standard for steel structural design (IS:800-2007) for minimum pitch, edge and end distances. For instance, for the bottom-most panel, the plate lengths are calculated to be 160 mm for the AT-type tower and 150 mm for the double circuit tower assuming 10 mm diameter bolts.

$$\left(\sqrt{\left(\frac{P}{A_1}\right)^2 + 3\left(\frac{Q}{A_2}\right)^2}\right) \leq \frac{f_y}{\gamma_{m0}} \qquad (1)$$

It is important to note that this stress from Eq. (1) can be taken as the limiting value since the gross section yielding and net section rupture stresses are greater. The calculated values of joint stress are found to be maximum for the joint at the bottom most panel for both the towers. For the AT-type tower, this stress was found to be 303.7 N/mm². This is greater than the limiting value dictated by Eq. (1) to prevent failure of the joint under the combined stresses. In order to restrict the maximum joint stress, the load factor corresponding to the limiting stress 227 N/mm² is found. This is given by a load factor 227/303.7=0.74. For the double circuit tower, the maximum stress was found to be 270.23 N/mm². The load factor for limiting the joint stress is calculated to be 227/270.23=0.84. The design loads are recalculated corresponding to these load factors and the subsequent analysis are performed for these load values.

4. Nonlinearanalysis

Large deflection behaviour of the towers is studied by performing geometric nonlinear analysis including the effects of material nonlinearity and member buckling (Yang *et al.* 2012). The ultimate collapse loads of the tower can be determined as well as the load level of the initiation of the collapse. The current study adopts a corotated-updated Lagrangian (CR-UL) formulation for the geometric nonlinear analysis. The corotational approach involves decomposition of the total deformations in a body into rigid body motions and stress producing deformations (called natural deformations). The formulation is computationally efficient for structures composed of truss elements, since a truss element has five rigid body motions and just one stress producing deformation. The formulation is described in detail in the works of Mattiasson (1983) and has also been adopted by Jayachandran *et al* (2004) for truss structures. The global tangent stiffness matrix $\boldsymbol{K_G}$ is given by Eq. (2).

$$\boldsymbol{K_G} = \boldsymbol{A^T}(\boldsymbol{E^T K' E} + \boldsymbol{R_{x2} B})\boldsymbol{A}\delta\boldsymbol{p} \tag{2}$$

The terms in equations are as given by Mattiasson (1983). The transformation between various coordinate systems are represented by $\boldsymbol{A}$, $\boldsymbol{E}$ and $\boldsymbol{B}$, $\boldsymbol{R_{x2}}$ is the element force at the degree of freedom where the stress producing deformation is considered (natural force) and $\boldsymbol{p}$ is the global displacement vector. To derive the local (natural) tangent stiffness along the previously mentioned degree of freedom, an updated Lagrangian finite element equation given by Eq. (3) is applied. As seen from Eqs. (4)-(6), the displacement function φ_2', and incremental strain $\delta\mathbf{E}$ are in terms of the natural deformation $\boldsymbol{p_n}$.

$$\int_{\Omega} (\boldsymbol{\delta}\mathbf{E}\boldsymbol{C}\boldsymbol{\delta}\mathbf{E} + \boldsymbol{\delta p \sigma \delta p})d\boldsymbol{\Omega} = \int_{S} \boldsymbol{\delta}\mathbf{t}_{\mathbf{i}}\boldsymbol{\delta p}\mathrm{dS} + \int_{\Omega} \boldsymbol{\delta}\mathbf{f}_{\mathbf{i}}\boldsymbol{\delta p}\mathrm{d}\boldsymbol{\Omega} \tag{3}$$

$$\varphi_2'(x') = x'/L \tag{4}$$

$$\delta\boldsymbol{p_n}(x') = [\varphi_2'(x')][\delta\boldsymbol{r_{x2}}] \tag{5}$$

$$\delta\mathbf{E} = \delta\boldsymbol{p_{n,x}} + \frac{1}{2}\left(\delta\boldsymbol{p_{n,x}}\right)^2 \tag{6}$$

The above equations also involve the constitutive matrix $\boldsymbol{C}$, stress $\boldsymbol{\sigma}$, traction vector $\mathbf{t}$, body force vector $\mathbf{f}$ and $\boldsymbol{\Omega}$ and S are the volume and surface domains for integration respectively. Integration of Eq. (3) yields the natural tangent stiffness given in Eq. (7). The effects of member buckling and yielding are incorporated into the natural tangent stiffness expression. For member buckling, the lateral bowing deflection δ_c is found by assuming an initially bent member and employing an arc-length procedure to trace the load-deflection and modifying Eq. (7) to the form in Eq. (8). The reduced force carrying capacity of buckled members is found (Kondoh and Atluri 1985) (Eq. (10)) in terms of the Euler critical load P_{cr} and total strain $\mathbf{E}$.

$$\boldsymbol{K'} = \frac{A}{L}[\boldsymbol{C} + \boldsymbol{\sigma}] \tag{7}$$

$$\boldsymbol{K'} = \frac{1}{\left[\left(\dfrac{1}{\dfrac{A}{L}(\boldsymbol{C} + \boldsymbol{\sigma})}\right) + \dfrac{F_\Delta L}{\boldsymbol{R_{x2}}}\right]} \tag{8}$$

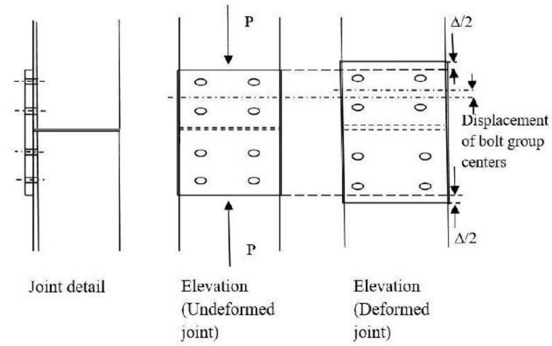

Fig. 3 Cover plate axial deformations joining leg members

where

$$F_\Delta = \left[1 - \frac{1}{1 + \frac{2}{3}\left(\frac{\delta_c}{L}\right)^2}\right] \tag{9}$$

$$R_{x2} = P_{cr}\left(1 - \frac{1}{2}\mathbf{E}\right) \tag{10}$$

A mixed hardening model for member inelasticity updates the total and plastic strains $\mathbf{E}'^{\boldsymbol{p}}$ in the members based on the level of yielding. The state determination procedure given by Bhatti (2005) concurrently with the yield criterion f given in terms of the changes in yield stress $\delta\boldsymbol{\sigma}_{\mathbf{y}}$ and mixed hardening parameter M are used to evaluate the updated stresses for use in Eq. (7). These are shown in Eqs. (11)-(13).

$$\delta\boldsymbol{\sigma}_{\mathbf{y}} = MH\delta\mathbf{E}'^{\boldsymbol{p}} \tag{11}$$

$$\delta\boldsymbol{\alpha} = (1 - M)H\delta\mathbf{E}'^{\boldsymbol{p}} \tag{12}$$

$$f = (\boldsymbol{\sigma} - \boldsymbol{\alpha}) - \boldsymbol{\sigma}_{\mathbf{y}} \tag{13}$$

The effects of the axial deformations or flexibility of the cover-plates on the tower behaviour are considered by modifying the nonlinear analysis to include the stiffness of the connections. The relative stiffness between the connection cover plates and the tower members comprises connection semi-rigidity. This amounts to a partial release for the deformation of the members and hence resulting final displacements would be larger. This semi-rigidity is modelled solely by considering axial deformations of the cover plate. A spring series is used where axial springs representing the deformations of the cover plates are at either end of the members. The natural tangent stiffness computed from Eq. (7) is modified with equivalent stiffness of this spring series as shown in Eq. (14). At the beginning of a new load increment, the deformations of the springs must be excluded from the deformations used for evaluating the strain increment (Eq. (6)) for calculating the stress $\boldsymbol{\sigma}$ in Eq. (7).

$$\boldsymbol{K'_{eq}} = \frac{\boldsymbol{K'}}{1 + \boldsymbol{K'}\left(\frac{1}{\boldsymbol{J_1}} + \frac{1}{\boldsymbol{J_2}}\right)} \tag{14}$$

The spring (cover plate) stiffnesses $\boldsymbol{J_1}$and $\boldsymbol{J_2}$ are related to the extension or contraction of the cover plate (Δ)

Table 2 Stiffnesses of cover plate for AT-type tower

Members	Cross section	Length of the plate (mm)	Stiffness (N/mm)	Shear Type
Up to height 12.26 m	120×120×10	160	6.30×10^6	Double
Up to height 18.16 m	110×110×10	160	5.76×10^6	Double
Up to height 20.16 m	90×90×8	100	6.04×10^6	Double
Up to height 26.70 m	75×75×5	80	2.36×10^6	Single

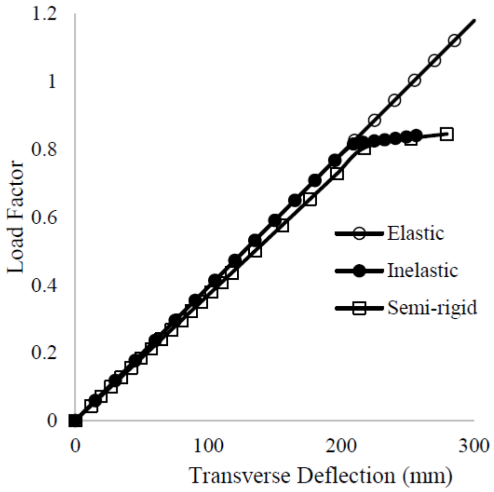

Fig. 4 Load-deflection of AT-type tower

along its length as shown in Fig. 3. Therefore, the spring stiffness is nothing but axial stiffness AE/L where L is the length of the plate assumed as stated in section 3. For double shear connections with two cover plates on both inner and outer faces of the tower leg members, the stiffness of the connection is twice that calculated for a single shear connection with one cover plate. The stiffness of cover plates for the AT-type tower at different height locations is shown in Table 2 and the stiffness for the double circuit tower for which a single member size has been assumed is 3.5×10^6 N/mm.

The nonlinear analysis is then performed for the two towers for 3 cases: (a) Elastic analysis without consideration of member inelasticity or buckling, (b) Analysis with member inelasticity and buckling and (c) Analysis with member buckling, member inelasticity and connection semi-rigidity. The resulting load-deflection plot are shown in Fig. 4 and Fig. 5 where the load factors are shown rather than the actual values of the loads itself. As seen from the figures, the failure of the AT-type tower occurs at load factor of 0.82 due to buckling of bottom leg members at a corresponding deflection of 234 mm. The double circuit tower, due to heavy sections taken for whole tower reaches its design load (load factor 1.0) without failure

5. Proposal of deflection limit criterion

As the AT-type tower experiences member failure at less

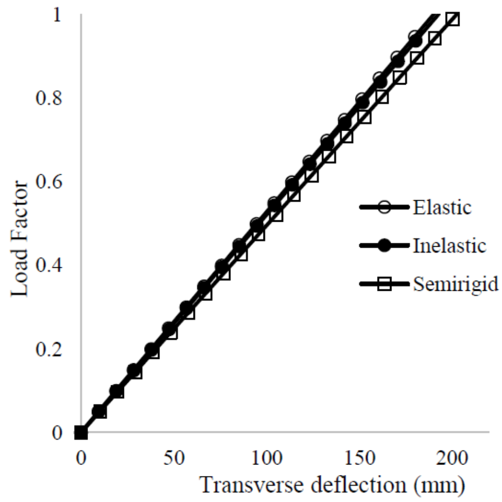

Fig. 5 Load-deflection of double circuit tower

than load factor 1.0 (Fig. 4), the loads acting on the tower should be scaled to this load factor. The maximum load factor that the tower can be permitted to attain in order to limit the joint stresses was calculated earlier as 0.74 even though member failure predictions from analysis was at load factor 0.82. The deflection corresponding to load factor 0.74 obtained from the nonlinear semi-rigid analysis is approximately 200 mm. For the double circuit tower, the deflection corresponding to the load factor for limiting joint stresses (0.84) is 170 mm. However, these deflections cannot be used directly. Bolt slip deflections constitute a major part of the additional deflections in transmission line towers. In this study, the total deflections including bolt slip is taken to be 1.5 times the deflections attained using the semi-rigid analysis. The multiplication factor 1.5 is assumed based on the findings reported in reference that experimentally obtained deflections are 1.3-1.8 times the analytical deflections. The factor 1.5 is particularly selected since it is the mean of the range 1.3 to 1.8 as well as the fact that, for the selected example of the AT Type tower, the experimentally measured deflection is close to 1.5 times the analytical estimate.

Thus the total deflections of the towers at the state of limiting joint stress is 200×1.5=300 mm for the AT-type tower (340 mm experimental deflection in reference) and 170×1.5=255 mm for the double circuit tower. The height to deflection ratios are 87.5 and 195 respectively and can be rounded to 80 and 200. By considering a transmission tower of given height as a cantilever, it is clear that the deflection depends on flexural rigidity of this cantilever. Thus, height alone is insufficient and a measure of the stiffness of this cantilever is required. The cross-sectional area of the leg members in the panels below the first cross-arm can be used to modify this height to deflection ratio for towers that employ other member sizes. Since the cover plates are assumed according to the sizes of the leg members, towers with larger leg member sections with consequently wider cover plates would deflect lesser for the same set of loads. Therefore, the height to deflection ratio can be reduced in proportion to the leg member cross-section sizes. The procedure is illustrated in Fig. 7. Height to deflection ratios

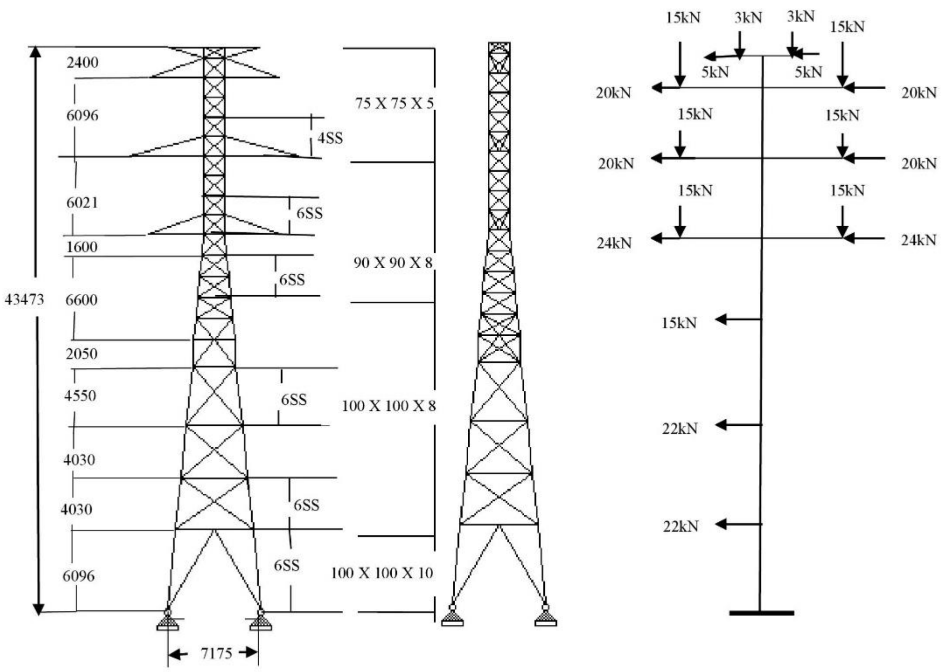

Fig. 6 240 kV K-type tower elevation and loading tree. DS - double shear SS - single shear (units: kN, mm)

are available for two height to base width ratios of 3.68 and 5.88 for the AT-type tower and double circuit tower respectively. Approximations of allowable height to deflection ratio for towers of other height to base width ratios will be based on that of these two towers. Application of the allowable height to deflection ratios should consequently be able to limit the joint stresses of the towers.

For evaluating the procedure described, the K-type tower in Rao *et al.* (2012) of height 43.65 m is chosen as an example. The tower is shown in Fig. 6. For the purpose of assessing the suitability of the procedure in regular design, nonlinear analysis is not performed for this tower and linear analysis using general software is used for obtaining the initial results. When member sizes as shown in Fig. 6 are used, the highest deflection was obtained as 455 mm (450 mm in reference) and the maximum stress in the joints under the combined stress state was found to be 355 N/mm^2. This is greater than the allowable stress under combined axial and shear. Thus a case for testing the deflection limit criteria exists to check whether the joint stresses can be brought within the limits.

The height to base width of this tower is 6.08 which is greater than that of the double circuit tower (5.88). To avoid extrapolation, the same height to deflection ratio as the double circuit tower, 200, is taken. This ratio is accordingly reduced to account for the smaller 100×100×10 member sections for the leg members. The final height to deflection ratio then reduces to 133, which is relaxed to 120. The reason for relaxing rather than tightening the allowable deflections is that shear stresses were taken into account in calculating joint stresses though these are negligible in practice. The current deflection value of 455 mm is greater than 1/120 of the tower height which is 363 mm. It is necessary here to remember that the height to deflection limits were obtained for the two towers after scaling for the

Table 3 *K*-type tower-original and revised member sections

Description	Original	Revised
Up to Height 6.15 m	100×100×10	150×150×12
Up to Height 22.61 m	100×100×8	150×150×10
Up to Height 35.20 m	90×90×8	130×130×10
Up to Height 43.65 m	75×75×5	110×110×10
Max Deflection of the tower (mm)	455	237
Max stress in cover plates (N/mm^2)	355	170

bolt slip displacements. Therefore, the allowable deflection of 363 mm for the current tower is inclusive of the correction for bolt slip. Thus the limit for the deflection obtained from the linear analysis is 363/1.5=243 mm. This is almost half the deflection that was obtained.

The current *K*-type tower is then revised with larger member sections (and larger cover plates) to achieve the target height to deflection ratio. The details of the member cross-sections for original tower *K*-type tower and revised tower are shown in Table 3. The load case is once again reliability with only transverse and vertical loads present

Table 3 shows that the application of the revision of the tower to achieve the deflection limit succeeded in limiting the joint stresses for the load case. Stresses in all members of the tower are checked and found to be within their strength capacities and the maximum stress in the cover plates is brought back within the limit of 227 N/mm^2. This method works using the joint stresses as the variable of interest. If one were to apply the deflection limit with the revised member sections, a more stringent limit would be obtained due to the heavier member sections adopted. This is contrary to the expectation that for a fixed height to base width ratio, the deflection limit should be more lenient with heavier member sections. The reason is that the cover plates are proportioned based on leg member sizes. As the member

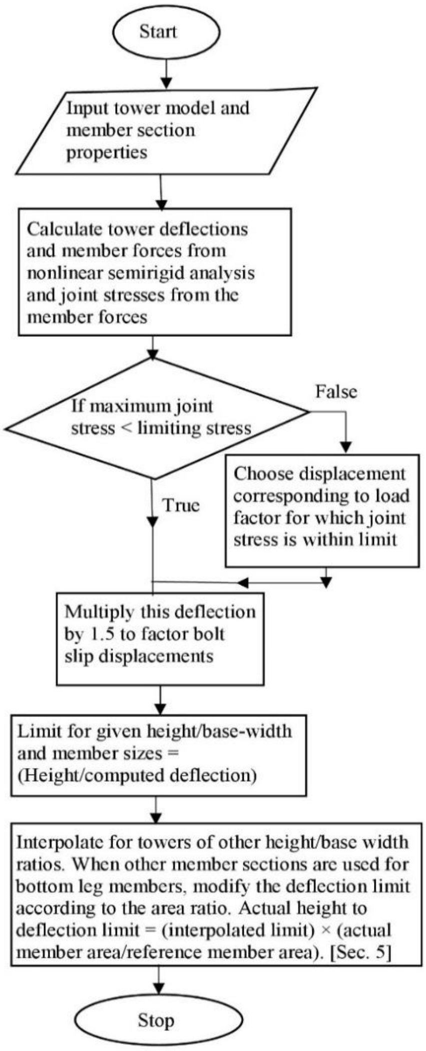

Fig. 7 Height to deflection limit determination procedure proposed in this study

sizes increase, the design loads for the tower members increase while increased structural stiffness causes limiting stresses of cover plates to be attained at lesser deflections (larger height to deflection ratios). In other words, increasing member sizes works out advantageously for joint stresses and disadvantageously for the height to deflection limits.

Therefore tower revisions can be carried out only once for any tower with large values of analytical deflections. The process cannot be applied iteratively as increasing the cover plate sizes has opposite effects on the joint stresses and deflection limit respectively. It is interesting to note that tower height to deflection limits given in the now obsolete Soviet design codes (Murthy and Santhakumar 1990) ranged from 70 to 120 respectively. The range 80 to 200 seen here is relatively more conservative.

6. Conclusions

A proposal for tower deflection limits was made wherein the joint strength is used as the basis for the allowable height to deflection ratio of transmission line towers. Two towers in a commonly encountered height range of 26-50 m were used as the reference towers in the study. The stresses in leg member joints was used as the criterion to limit the permissible deflection in towers-the permissible deflection being the one when the highest stress in any joint is just within its permissible stress. In order to predict the ultimate collapse loads of the two towers and also include the effects of the cover plate flexibility, nonlinear analysis with inclusion of member buckling and member yielding was used. The analysis was based on a corotated- updated Lagrangian (CR-UL) formulation. The cover plate semi-rigidity was modelled through axial springs in series with the member elements of the tower. The deflections results of the nonlinear semi-rigid analysis at the load level corresponding to permissible joint stresses was selected for the calculating the deflection limit. The deflection at this load level multiplied by a factor to account for bolt slip displacement was set as the maximum allowable deflection for the tower. This limit depended not only on the height to base width ratio of the tower but also the cross-sectional area of the leg members.

The height to deflection ratios obtained for the towers ranged were 80 and 200. When applied to a new tower satisfying member strength limits, the application of this deflection limit enabled to limit the joint stresses but cannot however be used iteratively.

The procedure also contains certain limitations, most importantly that the limits are based only on towers with two height to base width ratios. All the analysis were performed on reliability load conditions without longitudinal loads and hence the applicability of these limits is not known for broken wire conditions. However satisfying deflections will not be of primary importance in security load conditions. Nonetheless, the above demonstration shows that this procedure of basing the deflection limits on expectations of joint stresses could be a possible method for judging deflection limits for transmission line towers.

References

Ahmadizadeh, M. and Maleek, S. (2014), “An investigation of the effects of socket joint flexibility in space structures”, *J. Constr. Steel. Res.*, **102**, 72-81.

Al-Bermani, F.G.A. and Kitipornchai S. (1992), “Nonlinear analysis of transmission towers”, *Eng. Struct.*, **14**(3), 139-151.

Albermani, F., Kitipornchai, S. and Chan, R.W.K. (2009), “Failure analysis of transmission towers”, *Eng. Fail. Anal.*, **16**, 1922-1928.

Baran, E., Akis, T., Sen, G. and Draisawi, A. (2016), “Experimental and numerical analysis of a bolted connection in steel transmission towers”, *J. Constr. Steel Res.*, **121**, 253-260.

Bhatti, M.A. (2005), *Fundamental Finite Element Analysis and Applications: with Mathematica and Matlab Computations*, John Wiley & Sons, New York, USA.

Blachowski, B. and Gutkowski, W. (2016), “Effect of damaged circular flange-bolted connections on behaviour of tall towers, modelled by multilevel substructuring”, *Eng. Struct.*, **111**, 93-103.

Cai, J., Zhang, Q., Jiang, Y., Xu, Y., Feng, J. and Deng, X. (2017), "Nonlinear analysis of a radially retractable hybrid grid shell in the closed position", *Steel Compos. Struct.*, **24**(3), 287-296.

Deng, H.Z., Xu, H.J., Duan, C.Y., Jin, X.H. and Wang, Z.H. (2016), 'Experimental and numerical study on the responses of a transmission tower to skew incident winds", *J. Wind Eng. Ind. Aerodyn.*, **157**, 171-188

Fedor, K. and Czmoch, I. (2016), "Structural analysis of tension tower subjected to exceptional loads during installation of line conductors", *Procedia Eng*., **153**, 136-143.

Gray, P.J. and McCarthy, C.T. (2011), "A highly efficient user-defined finite element for load distribution analysis of large-scale bolted composite structures", *Compos. Sci. Technol.*, **71**(12), 1517-1527.

IEC 60652 (2002), Loading Tests on Overhead Lines Structures, International Electrotechnical Commission (IEC), Switzerland.

IS 802 Part 3 (1978), Code of Practice for Use of Structural Steel in Overhead Transmission Line Towers-Testing, Bureau of Indian Standards, India.

IS: 800 (2007), General Construction in Steel-Code of Practice, Bureau of Indian Standards, India.

Jayachandran, S.A., Kalyanaraman, V. and Narayanan, R (2004), "A corotation based secant matrix procedure for the elastic postbuckling analysis of truss structures", *Int. J. Struct. Stab. Dyn.*, **4**(1), 1-19.

Jiang, W.Q., Wang, Z.Q., McClure, G., Wang, G.L. and Geng, J.D. (2011), "Accurate modeling of joint effects in lattice transmission towers", *Eng. Struct.*, **33**(5), 1817-1827.

Kartal, M.E., Basaga, H.B., Bayraktar, A. and Muvafik, M. (2010), "Effects of semi-rigid connection on structural responses", *Elec. J. Struct. Eng.*, **10**, 22-35.

Kitipornchai, S., Al-Bermani, F.G.A. and Peyrot, A.H. (1992), "Effect of bolt slippage on ultimate behaviour of lattice structures", *J. Struct. Eng.*, ASCE, **120**(8), 2281-2287.

Kondoh, K. and Atluri, S.N. (1985), "Influence of local buckling on global instability: simplified large deformation post-buckling analysis of plane trusses", *Comput. Struct.*, **21**(4), 613-627.

Kroeker, D. (2000), "Structural analysis of transmission towers with connection slip modelling", MS Thesis, University of Manitoba, Manitoba.

Li, Q., Junjian, Y. and Wei, L. (2012), "Random wind-induced response analysis of transmission tower-line system", *Energy Procedia*, **16**, 1813-1821.

Ma, H., Fan, F., Wen, P., Zhang, H. and Shen, S. (2015), "Experimental and numerical studies on a single-layer cylindrical reticulated shell with semi-rigid joints", *Thin Wall. Struct.*, **86**, 1-9.

Mattiasson, K. (1983), "On the corotational finite element formulation for large deformation problems", Dr Ing Thesis, Chalmers University of Technology, Goteborg.

Murthy, S.S. and Santhakumar, A.R. (1990), *Transmission Line Structures*, McGraw Hill, Singapore.

Peyrot, A.H. and Goulois, A.M. (1979), "Analysis of cable structures", *Comput. Struct.*, **10**, 805-813.

Rao, N.P., Knight, G.M.S., Lakshmanan, N. and Iyer, N.R. (2010), "Investigation of transmission line tower failures", *Eng. Fail. Anal.*, **17**, 1127-1141.

Rao, N.P., Knight, G.M.S., Lakshmanan, N. and Iyer, N.R. (2012), "Effect of bolt slip in tower deformation", *Pract. Period. Struct. Des. Constr.*, ASCE, **17**(2), 65-73.

Reid, J.D. and Hiser, N.R. (2005), "Detailed modelling of bolted joints with slippage", *Finite Elem. Anal. Des.*, **41**, 547-562.

Sagiroglu, M. and Aydin, A.C. (2015), "Design and analysis of non-linear space frames with semi-rigid connections", *Steel Compos. Struct.*, **18**(6), 1405-1421.

Ungkurapinan, N., Chandrakeerthy, S.R.De.S., Rajapakse, R.K.N.D. and Yue, S.B. (2003), "Joint slip in steel electric transmission towers", *Eng. Struct.*, **25**, 779-788.

Xu, Y.L. and He, J. (2017), *Smart Civil Structures*, CRC Press, Boca Raton, Florida, USA.

Yang, F., Han, J., Yang, J. and Li, Z. (2012), "Study on the buckling behaviour of cold-formed angles in transmission towers", *Int. J. Steel Struct.*, **11**(4), 495-508.

Yang, S.C. and Hong, H.P. (2016), "Nonlinear inelastic responses of transmission-line system under downburst wind", *Eng. Struct.*, **123**, 490-500.

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