Response of Historic Transmission Towers – Analysis, Renovation and Strengthening Issues

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Introduction

Response of historic transmission line towers are examined in this paper. Strengthening and renovation of such systems are studied herein with material data collected from structural testing to assess tangent modulus as well as some fatigue data. Assessment of overload capacities of elements is done using material data for tangent modulus is also completed using PLS-CADD software system. Methods of renovation of these towers and strengthening to carry such overloads are also examined.

The overload factors for buckling loads of existing historic towers are of the order of 1.1 to 1.7 after nonlinear analysis using finite element models, including geometric and material nonlinearity. This may be used in design of transmission line systems in electric utility companies, using ASCE 2007, Canadian 2005 and NESC Standards.

Towers in this paper refer specifically to steel lattice structures. It does not include wood or steel pole structures.

Historic Transmission Towers

Historic transmission towers were designed and built 60 or more years ago and are still in service. Some elements of these towers have deteriorated due to corrosion, fatigue and overloading. Some members have buckled and some connections have failed. The main objectives of this study are to examine such events and to suggest some methods of renovation and strengthening. A method of strengthening a tower is to install pre-tensioned guy wires at midpoint and at one quarter heights to provide additional stiffness. Another way of strengthening is to insert additional horizontal diagonal elements at every one quarter height of the tower to create diaphragms. Failed connections can also be strengthened by using new stiffeners and bolts at failed joints. The strength of existing towers can be predicted by doing some failure analysis.

Nonlinear Response of Transmission Towers

Historic transmission towers built 60 or more years ago have deteriorated due to corrosion, fatigue and loads due to extreme events, such extreme winds, snow and ice. Prior to reaching their limit resistance in buckling and plastic collapse, these structures will exhibit significant nonlinear response, leading to buckling of elements and collapse due to significant plastic deformations. Connections may also have failed due to fatigue and dynamic loads. It is useful to assess overload factors due to such events, by computing a ratio of maximum stresses versus allowable stress used in earlier design. Figures 1 and 2 show typical dead-end towers.

Figures 3 and 4 show typical suspension towers. The nonlinear response is of significance, for determining renovation and strengthening aspects of these towers. Analysis (determination of forces and displacements) and design (proportioning of members and connections to resist the determined effects) become very useful in renovation and strengthening of these towers.

Nonlinear Analysis by Matrix Methods

Linear elastic analysis is often performed first using elastic stiffness matrix [K] and load matrix {P} as shown in the following.

$$[\mathbf{K_e}]\ \{\mathbf{x}\} = \{\mathbf{P}\} \tag{1}$$

In the nonlinear analysis to be performed next as the displacements become nonlinear and inelastic, we solve the following incremental matrix equation.

$$[\mathbf{K_t}]\ \{\boldsymbol{\delta}\mathbf{x}\} = \{\boldsymbol{\delta}\mathbf{P}\} \tag{2}$$

Herein, the matrix $[\mathbf{K_t}]$ is the tangent stiffness matrix, and $\{\boldsymbol{\delta}\mathbf{x}\}$ is the incremental nodal displacements and $\{\boldsymbol{\delta}\mathbf{P}\}$ is the incremental loads and reactions matrix. The matrix $[\mathbf{K_t}]$ will include different levels of nonlinearity, including material inelastic deformations. In the second order elastic analysis the equation becomes

$$[\mathbf{K_e} + \mathbf{K_g}]\ \{\boldsymbol{\delta}\mathbf{x}\} = \{\boldsymbol{\delta}\mathbf{P}\} \tag{3}$$

The matrix $[\mathbf{K_g}]$ represents change in stiffness that results from these effects. $[\mathbf{K_g}]$ is formulated by using the change in geometry at each step of the analysis, including the effects of axial deformations on flexural stiffness.

The first order inelastic analysis is accomplished using the geometry of the un-deformed structure. This can be written as

$$[\mathbf{K_e} + \mathbf{K_m}]\ \{\boldsymbol{\delta}\mathbf{x}\} = \{\boldsymbol{\delta}\mathbf{P}\} \tag{4}$$

Here the matrix $[\mathbf{K_m}]$ is termed the plastic reduction matrix, which includes plastic deformations from inelastic excursions. In the second order inelastic analysis both geometric and material nonlinearity are included. The equations of equilibrium then become

$$[\mathbf{K_e} + \mathbf{K_g} + \mathbf{K_m}]\ \{\boldsymbol{\delta}\mathbf{x}\} = \{\boldsymbol{\delta}\mathbf{P}) \tag{5}$$

In computing buckling loads of the system, we use the generalized eigenvalue formulation of the equations of equilibrium.

$$[\mathbf{K_e} + \boldsymbol{\lambda} * \mathbf{K_g}]\ \{\mathbf{x}\} = \{\mathbf{0}\} \tag{6}$$

Here $\boldsymbol{\lambda}$ is an eigenvalue of the system. This gives the buckling load, as a load factor, with reference to $\{\mathbf{Pref}\}$ and eigenvector $\{\mathbf{x}\}$ is the buckled shape. This equation is used compute inelastic critical loads also.

$$1/\boldsymbol{\lambda}\ \{\mathbf{xf}\} = -[\mathbf{K_{ef}}]\ -1\ [\mathbf{K_{gf}}]\ \{\mathbf{x_f}\} \tag{7}$$

In which the subscript f refers to only the free degrees of freedom and not restraints. Thus the matrices in Eqn.7 would be non-singular and hence an inverse exists. An iterative procedure is used to find the eigenvalues first and the eigenvectors or buckled shape next. The buckling loads of the towers are determined using this procedure. Software systems Mastan2 and PLS-CADD use these schemes. This was indeed done for the towers in New England and New York states. Summary of these results is shown in Table 1.

Solution of Nonlinear Equations

The simplest approach is an Euler type solution, in which the tangent stiffness matrix is assembled for given vector of forces, and increments of either load or displacement are specified, and the equations are solved for the corresponding unknowns. The element forces are computed from these new displacements. The tangent stiffness matrix is reformulated using these forces and deformed shape. This process is repeated for the next increment of loads. Total loads, element forces and displacements at the end of each increment are determined by summing these quantities over all increments. Equilibrium between external loads and internal forces is not checked, and hence the computed displaced shape can drift from the true load-displacement curve. However, if small incremental steps are used, fairly good accuracy can be achieved for most civil engineering systems such as transmission line towers.

A higher level of computational sophistication may be achieved by using a modified approach, in which this analysis is done in a series of incremental steps, where in each step, the solution proceeds in a series of tangent stiffness matrices parallel to true load-displacement curve. The imbalance between the applied loads at the end of each linear step and the element end forces are calculated from the results of that step. Each scheme is used to correct, in an iterative fashion, for the imbalance between the linear approximation and the actual response. This is meant to reduce this imbalance to an acceptable level.

Numerical convergence, detection of limit points and tracing post-limit behavior, detection of states of bifurcation, stable to unstable one, with little numerical warnings, are always included in these techniques. Determinant is used to detect instability or the suddenness of its appearance. Detection of the onset of material yielding is also made, by checking any changes in the constitutive relationship equations. Mastan2 and PLS-CADD utilize most of these techniques.

Nonlinear Analysis of Towers – New England and New York

Nonlinear analysis of transmission towers were subjected to gravity and lateral loads with incremental analysis as explained in the previous sections. Software system PLS-CADD was used in this study. The results are listed in Table 1. The overload ratios are in the range of 1.1 to 1.7 for diagonal and chord elements of these towers. The strut elements have ratios of 1.1 to 1.2. The material data was determined by using test coupons from diagonal, chord and strut elements. Modulus of Elasticity E and Tangent Modulus Et were determined, in addition to Fy and yield strain ey. Romberg-Osgood type stress-strain curve and bilinear curve are used. This enables buckling loads. The reduction in areas of elements is also included. to account for rust and other forms of deterioration with time. This ratio ranges from 1 to 2 percent. This analysis was also done using Mastan2 (McGuire, Gallagher and Ziemien, 1,2)

Table 1

Chord elements	**Diagonal elements**	**Strut elements**	**Max stress ratio**	**Max stress ratio**
1.2 to 1.7	1.4 to 1.7	1.1 to 1.2	1.7	1.7

Analysis of Results

The nonlinear response of historic transmission line systems were studied in this paper. The overload ratios are in the range of 1.2 to 1.7 for main chord elements, while they are 1.4 to 1.7 for diagonals and 1.1 to 1.2 for strut elements, which are horizontal. Methods of strengthening these towers are as follows:

1. Use vertical guys at top and middle height
2. Use horizontal tie elements to form a diaphragm at top and middle
3. Replace buckled diagonals by new elements
4. Replace connections where they have failed, by using bolted ones.

The first option of using guys has some limitations, if right of way restrictions exist next to the towers in open farmlands or nearby homes. The utility has to rent land or buy adjacent land to install guy wire stays and their footings.

Conclusions

The renovation of historic towers requires a non-linear analysis, and redesign of connections, and replacement of some elements. Guy wires can be used to create horizontal diaphragms at two different levels, top and middle. Buckled elements could be replaced with new connections could also be replaced, where they have failed. Horizontal tie members will help form diaphragms at top and middle height of towers.

The non-linear analysis shows the response of transmission towers to gravity and lateral loads. Renovation and strengthening of these towers is enabled by this type of analysis. Right of way issues may have to be addressed if guy cables are used to strengthen these historic towers. In the nonlinear analysis, the guy cables will have to be included, to assess revised buckling loads. The material testing of coupons cut from typical chord, diagonal and strut elements are used to assess Tangent Modulus, which is then used in nonlinear analysis.

References

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Figure 1.

Figure 2.

Figure 3

Figure 4