CHAPTER 2

REVIEW OF LITERATURE

2.1 GENERAL

Research work done in the last twenty years in the area of transmission line tower failures, X-braced panels, K-braced panels, single angle compression members, behaviour of bolted connections, dynamic behaviour of towers, local buckling in angle sections have been reviewed in this chapter and is broadly classified into two phases namely, analytical studies on cross bracing systems, analytical studies on the failure of transmission line towers in the field and experimental investigations on cross bracing systems and on the transmission line towers in the laboratory.

2.2 ANALYTICAL STUDIES ON CROSS BRACINGS

Picard and Beaulieu (1987) conducted theoretical studies to determine the transverse stiffness offered by the tension diagonal in cross bracing systems and evaluation of the effect of this stiffness on the out-of-plane buckling resistance of the compression diagonal. The proposed theory was supported by seven transverse stiffness tests and fifteen buckling tests. In the double diagonal cross bracing, the tension diagonal acts as elastic spring at the point of intersection of the compression diagonal. If the spring stiffness is zero, then the effective length factor is one and if the spring stiffness is infinitive then the effective length factor is 0.5. If the force in the tension member becomes zero then, the spring stiffness is equal to flexural stiffness.
of the tension diagonal. Therefore spring stiffness is never equal to zero and the effective length factor is always smaller than one. The compression to tension C/T ratio is larger than 1.6, the effective length factor increases. ‘K’ reaches its maximum value when the C/T ratio is equal to infinity, corresponding to minimum spring stiffness, equal to the flexural stiffness of the tension diagonal. The equations presented for effective length factor ‘K’ are valid for continuous diagonals which are connected at their intersection point. If one diagonal is interrupted at the intersection point, constant stiffness is not maintained and the connection at this point may become a weak link when the interrupted diagonal is compressed.

Using the energy approach, stability criteria for X-bracing systems are formulated and verified by Stoman (1988). By employing the Raleigh-Ritz method of stationary potential energy, simple closed-form analytic relationships for directly evaluating the transverse stiffness provided by the tension brace to the compression brace and for evaluating the compression brace critical load, for any ratio of (T/C) tension / compression brace axial force. The criteria addressed the general case where the two diagonals are different in section, material properties and lengths. Only hinged-hinged and built-in boundary conditions are considered in the evaluation. As a design aid, curves are generated for the effective length factor as a function of the ratios of the axial load in the tension compression brace respectively. The formulated analytical expressions for the effective length factor are simple and very amenable to design application.

Stoman (1989) has formulated the effective length criteria for cross-bracing system. The criteria entail closed-form analytic relationship for evaluating compression brace critical load and effective length factor ‘K’ with due consideration of the relative stiffness and end conditions of the tensile diagonal. Stability criteria formulated for cross-bracing systems based on
Raleigh-Ritz method of stationary potential energy by the author in the earlier studies, are expanded to address bracing systems with diagonals of different stiffness and boundary conditions. Closed form equations are derived for evaluating the critical load and the effective length factor for built in brace ends, pinned brace ends, only tension diagonal ends built and only compression diagonal ends built-in cases. Effective length spectra curves are generated to elucidate the criteria and to facilitate design application. Results indicate that the effective length is very sensitive to the relative stiffness of the two interconnected diagonals and depending on brace end conditions, the effective length factor values range from 0.35 to 1.0. The proposed solution agreed well with the available experimental data.

Wang and Boresi (1992) have formulated stability criteria for X-bracing systems using theoretical approach. The effective length factor K for X-bracing system is formulated and verified for identical members. Simple closed-form relationships for calculating the critical compressive load with two different end constraints are presented. As a design aid, curves for the effective length factor versus the ratio T/C, where T and C are tension and compression brace axial forces, respectively are given. Criteria are formulated for the general case where the tension and compression braces may have different material and geometrical properties. The ratio T/C increases as the effective length factor K decreases, after a certain value at which the compression brace buckles in the second mode, will remain unchanged. As the flexural rigidity of the tension brace increases, the ratio T/C at which buckling occurs in the second mode decreases and the effective length factor K also decreases. It is possible for the compression brace to snap through in the first buckling mode due to total loss of the tension brace flexural rigidity. However, when the tension brace flexural rigidity reaches a certain value, the compression brace buckles in the second mode. Results of this study are compared with other solutions available in the literature.
Issa and Richard Avent (1995), used discrete-field analysis techniques to investigate the critical buckling load of various three dimensional X-braced lattice configurations and geometries. Regardless of number of joints in the system, the solution for the critical buckling load is reduced to solving for the Eigen values of a 5×5 matrix. The formulation derived is used to develop a series of curves showing the critical buckling load for various X-braced lattice configurations, which allows an engineer to determine critical buckling loads without lengthy and cumbersome calculations. Studies on the buckling load for the planar lattice indicated that the modified Euler-buckling load approximations may vary upto 30% in comparison to results obtained by the discrete-field-analysis method. The results obtained from this study indicate a similar trend for the three dimensional X-braced lattice.

Elastic stability analysis of X-bracing system with special attention on the effect of mid-span connection on the out-of-plane buckling load is performed by Ali Davaran (2001). Commonly, in practice, one of the diagonal members is discontinuous at the mid-span intersection and a gusset plate is used to connect the members. The centre connection is modelled as partly pinned or semi-rigid. From these studies, closed-form relationships are obtained for the evaluation of effective length factor of X-bracing with pinned end connections and either pinned or semi rigid mid-connection. The tension and compression diagonals are assumed to have different section properties and axial loading. The results are graphically displayed for some practical cases. Three types of double angle X-bracing with different detailing for mid connection are compared. The numerical results show that the discontinuous center connection can affect the buckling load and effective length factor of diagonal members. Further it is concluded that to increase the load carrying capacity of X-bracing systems, the rotational stiffness of center connection should be increased via suitable detailing of intersection connection. When
pair of angles is used as diagonal members, the out of plane rotational stiffness of the diagonal at the intersection and load carrying capacity of the compression member can be increased by cutting one section from each diagonal and continuing the other through the intersection.

Several analytical studies have been conducted on the out-of-plane buckling of X-bracing systems without considering the effects of mid-span connection and inelastic behaviour. Jiho Moon et al (2008) have conducted analytical studies to suggest approximate solutions for the effective length factors to predict the out-of-plane buckling load of X-bracing systems with discontinuous diagonals. The suggested factors are for general case where the tension and compression diagonals have different section properties, lengths and axial loads. If the tension diagonal has a sufficient flexural stiffness, the tension diagonal resists completely the out-of-plane displacement and makes the compression diagonal to buckle in the second mode. The second-mode buckling occurs when the equations suggested are satisfied for the X-bracing systems with a discontinuous tension and compression diagonal respectively. For identical diagonals, the second mode buckling occurs when tension/compression ratio is more or equal to one for the X-bracing system with discontinuous tension diagonal and the ratio more or equal to 0.57 for the system with discontinuous compression diagonal. Based on the studies conducted on inelastic out-of-plane buckling of X-bracing systems with discontinuous diagonals, inelastic buckling loads were calculated from the suggested effective length factor for X-bracing systems with discontinuous diagonal and the column buckling curves form the Korean Standard and AISC. The finite element modelling using ABAQUS is verified with the experimental results from DeWolf and Pelliccione (1979). Results were compared with those from non-linear finite element analysis taking into consideration the effect of large deformations, material inelasticity, initial imperfections and residual stresses. The column buckling design curves over-
estimated the inelastic out of plane buckling load of the X-bracing system when the stress level of the tension diagonal was relatively high.

Ali Davaran and Nader Hoveidae (2009) studied the buckling and post buckling behaviour of single story, single span models of cross braced systems with a common and specific mid connection details (with pair of cover plate on both sides of discontinuous diagonal members) using an extensive finite element analysis. Results showed that the centre connection detail of cross bracing can affect the overall behaviour of the braced system. For a cross bracing consisting of built-up sections with a gusset plate at mid-connection, it was revealed that details of the connection at intersection point could affect the effective length factor of the members in out of plane buckling. The buckling load of cross-bracing with suggested mid-connection is 20% greater than the buckling load of the bracings with common mid-connection detail when larger cross sections are assigned for the bracing. The push over-analysis showed that the response modification factor and strength of braced system with suggested mid connection details are 28% and 5% more than those of the system with common mid connection detail.

To improve the existing cross-bracing system with common center connection, it is possible to add a pair of cover plate on both sides of discontinuous diagonal members in some built-up sections used for braces.

2.3 EXPERIMENTAL STUDIES ON CROSS BRACINGS

Elgaaly et al (1992) conducted experiments on three dimensional trusses. The trusses were designed such that the target angle would fail first without causing significant deformations in the remaining members of truss. Following each test, the target angle alone was replaced, allowing multiple tests to be conducted in the same setting. Fifty single angle members with equal angle were also tested as part of the truss. The results indicated six
modes of buckling due to the coupling effect of local, flexural, torsional and torsional flexural modes. Most of the members failed in local buckling which occurred near the bolt hole. However some local buckling occurred away from the connections near the middle of the member. Failure mode was governed by the slenderness ratio of the member, breadth of the leg, width to thickness ratio, the end conditions and the eccentricity of the applied load. Experimental failure loads were compared with the predicted loads as given in ASCE Manual No: 52 for Steel Transmission Towers and the AISC Specifications for steel buildings. It was concluded that the current design method for non slender single angle members were not adequate and the nominal load predicted using Manual No: 52 closely agreed with the experimental results.

Cannon and Nickerson (1993) as a part of Transmission Line Mechanical Research Centre (TLMRC) research program on the strength of X-bracing in Transmission towers, conducted sixty-six tests on X-braced frames with parallel posts and forty-five tests on X-braced frames with battered posts. A two dimensional test frame was constructed of hot rolled angle section with nominal yield strength of 2.48 kPa. For connections 15.9 mm diameter ASTM bolts were used. The X-brace members were attached directly to the leg members at both ends using end connections ranging from one bolt to four bolts. To load the diagonal independently, the tension member was slotted at the top so that it could be free to elongate in the direction of applied tension load. The test frame was designed to allow separate application of tension and compression force to the respective members. The X-brace member sizes and the corresponding leg member sizes were selected as representative of typical combinations found in transmission towers. The X-bracing systems were chosen in such a way that the bracing slenderness ratio varies from 60 to 280. Strain measurements were taken to monitor the forces in the tension and compression members. The X-braces
were loaded to failure maintaining the tension member load at a selected percentage varying from 0 to 100% of the force in the compression member. The tension and compression loads were increased uniformly. A recommendation for effective slenderness ratios of continuous tension/compression (T/C) X-bracing members without intermediate redundant bracing was proposed. The variation of T/C (tension/compression) ratio was having minimal effect on the strength of the compression member in an X-brace. Whereas in the X-brace with unequal leg angles framed on their long legs showed 40% difference in their compression load capacity for T/C range of 0 to 1.0 for specimens with parallel posts. The effect of the end connection stiffness on the compression capacity was found to be substantial. If the end condition of the compression member is significantly different then the result may show greater influence of T/C. It was concluded that the ASCE Manual No. 52 results are very conservative for slenderness ratio greater than 120 and it is deficient for slenderness ratio less than 120.

Closed-form relationships are formulated for the direct evaluation of the critical loads of cross-bracings with semi-rigid ends for any value of relative stiffness of the end connections and any ratio of the dimensionless parameters of tension and compression members by Fred Segal et al (1994). Stability criteria for cross-bracing with semi-rigid ends are formulated. The criteria are formulated for general cases such as tension and compression braces having different sectional properties, lengths and axial loading. Parametric solutions are graphically displayed to clarify distinct behaviour including the boundary separating symmetrical and anti-symmetrical modes of buckling. Design aids in the form of curves were generated for the effective length factor as a function of the relative stiffness of the end connections and the ratio of the dimensionless parameters of tension and compression diagonals. In practice, cross-bracing members always experience some eccentricity. Analytical expressions for eccentrically loaded cross bracings
with simple end connections are derived with examples clearly indicating that the eccentricity reduces the load carrying capacity and the tensile force does not contribute substantially to the load-carrying capacity of the bracing system.

Kemp and Behncke (1998) conducted series of tests with slenderness ratio ranging from 102 to 160 on cross bracing system with bolted end connections widely used in transmission line towers and concluded that due to the dominant effect of end eccentricity, the secant formulation can be used to provide a design formulation for cross bracing in these range of slenderness ratios. Finite element program ABAQUS was used to predict the theoretical behaviour of cross bracing system. The variables considered in the experimental program include slenderness ratio of the bracing, inclination of the main legs and bracings, number of bolts in connections and relative stiffness of the main leg to the bracing member. Test results have been compared with flexibility-based analysis, three dimensional finite element analysis, ASCE Standards and European manuals. It was found that the flexibility-based analysis closely predicted the experimental values. The ASCE and the European manuals do not adequately consider reduction in strength due to end eccentricities. For the Euler range of slenderness ratio of cross bracings between 100 and 160, the cross over joints acted as an effective restraint to the buckling of the strut. Cross bracing members connected to main legs by one flange of each bracing member significantly influences the displacement within the bracing system. As a result, the intersection joint of tension, compression bracings system deflects out of plane even at low loads and bending moments are generated. By changing the number of bolts in the end connection from one to two the ultimate strength in tests was increased by 17%.
2.4 ANALYTICAL INVESTIGATIONS ON THE FAILURE OF TRANSMISSION TOWERS AND PANELS

A nonlinear analytical technique for predicting and simulating the ultimate structural behaviour of self supporting transmission towers under static load condition was presented by Al-Bermani and Kitipornchai (1992). The proposed method considered both the geometric and material nonlinear effects and treated the angle members in the tower as general asymmetrical thin-walled beam column elements. Modelling of material non linearity for angle members was based on the assumption of lumped plasticity coupled with the concept of a yield surface in force space. In the computer software package AK TOWER, a formex formulation is used for the automatic generation of topology, geometry, load and constraint conditions of transmission tower structure. The formex is a mathematical entity that consists of an arrangement of integers. Formex algebra is a mathematical system that consists of a set of abstract objects, known as formices, and a number of rules according to which of these objects may be manipulated.

The developed software was used to predict the ultimate behaviour of two 275 kV full-scale towers tested in Australia. Since most of the tower connections are multiple-bolted end connections offering some degree of restraint, it was assumed that the restraint offered by a connection relative to the moments induced in the tower members is large enough to regard the connection as rigid. The effect of joint flexibility can also be incorporated in the proposed technique provided, information on joint flexibility is known. Modelling of material non-linearity based on the assumption of lumped plasticity, coupled with the concept of yield surface in force space, provided a compact and practical method for modelling non-linear global structural behaviour.
The solution to the nonlinear response is obtained as a sequence to linearized solutions in which either the tangent stiffness is modified to reflect the extent of the development of plastic flow, or the load is modified by a residual force to maintain equilibrium. The stress resultants in the cross-section interact to produce yielding of the section. Any plastic behaviour is deemed to be concentrated at the familiar generalized plastic hinges located at the two extremities of an element. Plastic hinges are assumed to be elastic prior to the full plastification so that the initial stiffness of the complete element corresponds to that of the elastic beam. As the stress resultants at the ends of the element increase, the hinges yield resulting in a reduction in the element stiffness. For a steel section the hinges are assumed to become fully elastic again upon unloading. Since transmission towers are constructed of angle sections, a single equation representing the stress-resultant yield surface for angle sections under a combination of axial force and bi-axial moments is used. The analysis of towers involves the solution of several simultaneous equations and for this purpose, an out-of-core solution scheme is used.

The analytical predictions of ultimate loads, the nodal deflections at various points and the tower failure deflected shapes are compared with the experimental results. The developed software predicted the collapse load of heavy suspension tower subjected to flexural loads accurately. The predicted collapse load of tension tower subjected to torsional loads was 16% lower than the test collapse load.

Al-Mashary et al (1992) investigated six 132 kV tangent towers that failed in a transmission line in Al-Qassim region, owned by Saudi Consolidated Electrical Company. Two towers failed by bending of cross arms and three towers failed at their base. The governing specifications of ASCE Manual No.52-1971, were followed. The laboratory tests on tensile
specimens were satisfactory. A three dimensional analysis of the tower, employing the frame-members for the main legs, showed high localized bending moments in legs causing 30 to 40% over stress. These bending moments were neglected in the original design calculations. These moments although considered as secondary and neglected in common design practice, are significantly high at certain locations and lead to unexpected failures.

Al-Bermani and Kitipornchai (1993) reviewed the current design practices of transmission line towers and presented a nonlinear analytical technique for accurate simulation and prediction of the ultimate strength and behaviour of towers under static load conditions. Geometry and material non-linearities, joint flexibility and the effects of large deflection are accounted for the analysis.

The effect of nonlinearity is treated as effective load in conjunction with the applied loads for general equilibrium. This reduces the need for continuous updating of the tangent stiffness matrix and results in substantial savings in computation time. More stable algorithm can be achieved when the predictor part of the algorithm (tangent stiffness) is kept as simple as possible while a strict force recovery procedure is employed. Arc length method with simple solution advancement control is used as a solution strategy. The solution advancement control has been achieved using the arc-distance from the previous cycle and the maximum volume of the yield function in the least two cycles to extrapolate a maximum arc-distance for the present cycle. This brings the force-point gradually to the yield surface and guards against excessive deviation from the surface. Nonlinear incremental solution method is adopted. Within each loading cycle several iterations are performed to account for the effect of the variation in stress and strains during the cycle. This iteration process is terminated whenever equilibrium as defined by a
certain convergence criteria is satisfied and the force point for any element which has entered the plastic stage returns to yield surface. An out-of-balance force convergence criterion has been adopted using the Euclidean norm measure with a convergence tolerance set to 5%. Predictions of the ultimate loads and the failure deflected shapes have generally been very good, considering the complexity of the towers.

Alam and Santhakumar (1994) studied extensively the system reliability of the transmission line towers. An attempt has been made to implement the system reliability theories and methods with respect to the transmission line tower using second-order bounding and equivalent linear safety margin approaches. Important issues related to the non-linear force-deformation behaviour of the tower members under buckling failure were also discussed. The critical safety margins are identified by automatic failure mode generation by matrix formulation with the aid of the member replacement technique. A complete computer program has been automated to compute the reliability of transmission towers. Practical towers used to assess the reliability and illustrate the effectiveness of the methods lead to the adaptation of a system-based reliability analysis procedure.

Sanjeev Gupta et al (1994) presented the nonlinear structural analysis performed on a portion of the failed transmission line using ETADS software. The analysis included the characterization of the likely storm loads and their application to the transmission system. Heavy rain fall under freezing temperature with 32 to 38 mm radial ice formation and average wind speed of 5.4 m/s with peak wind gust of 13.9 m/s led to the failure. Investigations are carried out to determine the most likely cause of the failure. The line contained three bundled conductors corresponding to three phases of electric current and two shield wires. The structural analysis was limited to a
portion where the failure was speculated to have initiated based on the field observations.

Dynamic broken insulator analysis was performed by using a fuse element with only axial force capability and whose stiffness can be set to zero by the user at any time during the analysis. First 38 mm of radial ice was applied statically and the solution was carried out till convergence. In next step, the stiffness of the fuse elements were set to zero and dynamic analysis was carried out. The first load step resulted in a 200 kN longitudinal force imbalance at one of the structure. It was concluded that the sudden release of these forces on the system due to insulator breakage may be beyond the load carrying capacity of the other structure.

A dynamic and buckling analysis using a single structure model was performed to investigate the possibility of line failure under conductor galloping loads. Software using an algorithm developed by Baenziger (1991) and modified by Li Li and Anjam (1991) was used to idealize galloping loads by equivalent cyclic loads along with 38 mm radial ice. Galloping loads were expressed as net horizontal and vertical time varying forces at the insulator attachment points. Dynamic analysis for one full cycle of sinusoidal galloping loads for six seconds duration was performed by applying the loads at all three conductor locations. The analysis results showed that if at 38 mm of radial ice, the conductors in adjacent spans had galloped out of phase, it is possible that the buckling failure of the structure could have initiated the transmission line failure.

Natarajan and Santhakumar (1995) conducted studies on reliability based optimization of transmission line towers. Four independent computer programs for component reliability, reliability analysis, optimization and automation of failure mode generation were developed and linked together.
This has enabled more economical design of towers and ensured a particular level of chosen reliability. The weight of optimal tower accounting for reliability as a constraint for tangent tower is only 3 to 4% heavier than the tower designed using conventional method.

Faisal Abdullah and Al-Mashary (1999) investigated the non-linear behaviour of transmission towers including the effect of secondary members, geometric non-linearity and the inelastic behaviour. A 132 kV tangent tower of 45 m height with a square base which failed in site was used for the case study. The tower was analysed as a three dimensional structure with main legs as beam column members. A Finite element program ABAQUS was used for the analysis and a two node linear beam element B310S was used. Numerical integration over the section is carried out through nine points. The material stress strain curve was assumed to be elastic perfectly plastic. From the non linear analysis it was concluded that the secondary members have a local effect on the axial forces and bending moments of the main members. The presence of secondary members in the analytical model highly affects the bending moment and therefore recommended to be included in the analytical model. The geometric non linearity can magnify the axial forces and bending moments significantly. An amplification of 21% and 40% in the axial forces and the total bending moments respectively was the upper limit of the geometric non linearity effect for the case study tower. Inelastic analysis of the case study tower shows a uniform level of axial force and tower displacement cut off which means that no clear sign of force redistribution does exist before collapse.

Menezes et al (2000) (Working group 22.08) conducted the studies on the variability of mechanical properties of materials for transmission line steel towers. For the correct use of reliability based methods, knowledge of
the distribution of appropriate design and material random variables are required. Two different steel types ASTM A36 and ASTM A572 which are most commonly used for transmission line towers are considered. The survey was carried out at the inventory of company in Brazil. The inventory contains the results of numerous tests of steel specimens used for the bulk production of test towers. The test results were grouped according to the thickness and compared. It was observed that the mean value of the yield point for A36 steel shows an increase of 39.5% when compared to the minimum specified value and for the A572 steel the increase was 24.2%. For the ultimate tensile stress the mean value is 22.7% and 22.1% respectively. Additional information collected from the survey showed that the galvanization process increases the yield point by 10%, ultimate tensile stress by 4% and decreases the elongation by 30% for both the steel types. A 138 kV double circuit tower was analyzed based on the deterministic approach with and without considering the variation in the mechanical properties of the material as well as geometrical properties. The tower strength increased 10% when the variation in the properties is considered.

Prasad Rao et al (2001) developed Non-linear FEM models for the analysis of panels of latticed towers by calibration with test results. It was found that the current methods of design of main leg members based on the forces obtained from a linear analysis are not consistent with the test results. In this study, non-linear analysis of angle compression members and the single panel of angle planar as well as three-dimensional lattice frames, as in typical lattice towers, are carried out using MSC-NASTRAN. The member eccentricity, local deformation as well as rotational rigidity of joints, beam column effects and material non-linearity are taken into account. The analytical models are calibrated with test results. The results obtained using non-linear analyses compared well with the test results. Using this model, full
scale tower analysis was done to obtain more accurate results including secondary bracing forces prior to failure and the strength of the tower. Based on this study it is recommended that the secondary bracing member designs should meet both strength and stiffness requirements to perform their functions adequately.

McClure and Lapointe (2003) presented a macroscopic modelling approach to study the dynamic response of line sections subjected to exceptional unbalanced loads due to ruptured conductors. This approach is easily adoptable to study other problems like sudden ice-shedding effects from conductors or the sudden failure of other line components such as tower members suspensions strings etc., Line dynamic analysis is conducted to capture the shock propagation loads in the conductor and verified through case studies of two tower failures, which occurred due to conductor breakage during an ice storm.

Accurate structural analysis of transmission line towers is complicated because of its high redundancy and the tower is a space structure with angle sections connected eccentrically. The influence of geometric and material nonlinearities plays a dominant role in determining the ultimate behaviour of the structure. A non-linear analytical technique to simulate and assess the ultimate structural response of latticed transmission towers was presented by Al-Bermani and Kitipornchai (2003). It was claimed that the technique developed may be used to verify new tower designs and reduce or eliminate the need for full scale tower testing. The method has been calibrated with results from full scale tower tests with good accuracy both in terms of the failure load and the failure mode and hence can be used to assess the strength of the existing towers or to upgrade old and ageing towers.
Battista et al (2003) presented a new analytical-numerical model for structural analysis of transmission line towers under wind action. 3D-Finite element model was constructed for analysing the dynamic coupled behaviour of transmission lines and tower under the action of wind. The suspension rods formed by the chains of insulators were identified as the most important component of the system in the analysis of wind flow and tower lines coupled model interactive dynamic behaviour and response. The tower structure and all cables were discretized with spatial frame elements. A 230kV transmission line with delta type towers was used for the study. The soil structure interaction was also performed taking in to account two types, medium sand and clay soils. Linear elastic springs and rigid elements were used to simulate the soil and concrete footing. The study of the structural dynamic characteristics has shown that, whichever is the soil type, the first 10 lower natural oscillation frequencies do not change.

A tower strength improvement was investigated by adding a series of diaphragm bracings at mid-height of the slender diagonal members by Al-Bermani et al (2004). Analytical studies showed that considerable strength improvements could be achieved using diaphragm bracings. The effect of different types of diaphragm bracings including joining of internal nodes of diaphragm members and their location was investigated. Experimental results from the studies conducted on a tower sub structure strengthened with different types of diaphragm bracing under bending and torsional loads agreed reasonably well with the analytical predictions and showed that simple diaphragm bracing systems can be very effectively used in the upgrading of old towers. From the experimental results the most efficient diaphragm bracing system was predicted and the same system was implemented on an existing 105 m high TV tower. This scheme used less steel than the replacement of the existing diagonal bracings, was easier to implement in
practice, and led to improved tower performance. Although no dynamic assessment of tower retrofitted with diaphragm bracings was conducted, it is expected that such retrofitting will improve the tower’s dynamic response since it enhances the stiffness without too much increase in mass.

Murtagh et al (2004) have proposed an approximate method to determine the natural frequencies and mode shapes of towers. Modal characteristics of the tower-mass model are at first obtained by employing lumped mass method. These characteristic are subsequently compared to the corresponding modal properties of the system, predicted from finite element analysis. The system is approximated as a cantilever beam with a mass at its free end and the fundamental natural frequency and mode shapes are extracted by analytical formulation. Two simple approximate methods are used to obtain the natural frequencies and mode shapes. In the lumped mass method, the tower- top mass system is discretized into several convenient degrees of freedom. The second method involves approximating the tower by a cantilever beam with a concentrated mass at its free end. Frequencies and mode shapes are determined from finite element analysis using ANSYS software. The tower members are modelled using truss elements. The proposed approximate methods yielded accurate estimates of the natural frequency and mode shapes of lattice tower with mass at the top.

Prasad Rao et al (2005) conducted detailed studies on failures of cross arms. Four different towers are analysed using nonlinear Finite Element Analysis. Cross arm including part of tower body is modelled using beam and plate elements and analysed using finite element program NE-NASTRAN. The effect of inner triangulation formed by redundant patterns on the cross arm performance and the effect of staggering of secondary bracings in the elevation and in the plan of the cross arm and detailing of cross arm tips
loading points) are studied. The finite element nonlinear analysis results are compared with the test results. The secondary bracing members did not offer the desired restraint to the bottom main chord member of the cross arm if the top tie member is not strained in tension. The tip of cross arm has been checked for bending stresses. For the stability of cross-arms, the redundant pattern provided in plan and elevation should complete the requirement of inner triangulation. A detailed nonlinear finite element analysis is needed to obtain correct estimates of failure loads and failure patterns in the presence of eccentricities.

Silva et al (2005) proposed an alternative structural analysis modelling strategy for the steel tower design considering all the structural forces and moments combining three dimensional beam and truss finite elements. Three analysis methodologies, spatial truss finite element, beam finite element and combined beam and truss finite elements were compared. The studies were conducted on two towers of 40m and 75m heights actually built. In the combined beam and truss finite element modelling, beam elements are used for the main structure and truss elements are used for the bracing system. The stress values are 30 and 47% higher in the combined model as compared to the truss model. No difference was observed between the beam element model and combined model. The predicted lateral displacements were almost same in all the models. The dynamic analysis was performed using all the three methods based on free vibration strategy. The tower fundamental frequencies are 20% higher in truss and beam models as compared to the combined model.

Bracing failure associated with leg member of a 220 kV double circuit transmission line tower due to lightning was studied by Zinnia Nair et al (2005). Lightning strike caused flow of high inductive currents in the bracings and leg member of the tower, the tower metal being electrically
connected to a single earth. The lightning currents caused heating of the relevant tower members, particularly due to the skin effect of the members. The bracing sustained extensive damage and there was visible bending in the leg member and all these members sustained blotches. Except for severe lightning, the weather was normal. Investigations into the causes of failure of the tower revealed that the tower members failed due to temperature induced forces, the temperature rise being due to lightning currents flowing through the members. Analyses of temperature stresses in failed members and their buckling strength are studied. The restraints at the ends of bracings and leg members prevented the expansion and thereby caused axial compression. The axial compressive force induced in the bracing member due to the temperature rise was several times more than its capacity. Whereas in the case of the leg member the compressive stresses developed due to temperature rise was less than its capacity. Hence the damage in leg member was less. Such type of failures can be averted by providing a ring type earthing and connecting all four leg members of the tower evenly to the earth bus, and by designing the members so that the buckling strength will be more than the induced compressive forces.

Li et al (2005) presented simplified models for the coupled system of transmission lines and their supporting towers under the action of earthquake excitations through the proofs of experiments and numerical analysis. The equations of motion are derived and the computer program is compiled to obtain the earthquake responses of the coupled system. The transmission lines and their supporting towers were modelled as the lumped mass system. One-span conductor in the case of out-of-plane vibration was taken as catenaries represented by a cluster of masses. The time-history method with Rayleigh damping is adopted to calculate the seismic responses of the transmission tower system by compiling the program based on the
Matlab. To verify the rationality of the computational model presented, the model tests were carried out and compared with the numerical results. Due to the limitation of shaking-table, the small-scaled model was designed for the coupled tower-line system, in which the tower was modelled by steel bar and the steel chains were used for the conductors. The model system consists of three towers and two layer lines with two conductors for each layer. Two steel boxes were used as lumped masses attached to the top and middle of tower so that the acceleration sensors could be installed in them. Two steel solid blocks were attached to the middle and bottom of each bar. To investigate the accuracy of the proposed theoretical method, seismic responses of the model towers on different site conditions were obtained from the shaking-table experiments. Based on these studies, a simplified analysis method is proposed to make the seismic response calculation of coupled tower–conductor system faster and more effective.

A numerical model for simulating the ultimate behaviour of lattice steel towers was developed by Phill-Seung Lee and Ghyslaine McClure (2007). The elasto- plastic large deformation analysis of a steel lattice tower using finite element analysis was presented and the numerical results are compared with the full scale destruction tests. A two noded three dimensional L-Section beam finite element was proposed. This element can consider eccentricities of loading, boundary condition as well as material and geometrical nonlinearities. The full scale tower section of 10 m height with eight panels and one cross arm was modelled using the proposed two noded three dimensional beam elements and non linear static analysis was performed. The same full scale tower section was tested for bending and flexure - torsion tests and their behaviour was compared with the numerical results. Three different numerical models were used considering the rigidity and eccentricity of connections. In Model -I, one bolt connection was
considered as pinned and two or more bolt connections were modelled as rigid. Based on the position of the bolts, eccentricities at beam ends were also modelled using the new general three-dimensional L-section beam finite element. The Model-II is same as Model-I but the members are connected at bending centres that is eccentricities are not considered. In Model-III eccentricity is modelled as in Model-I but all connections are considered as rigid.

The analysis results are compared with the results obtained from a commercial computer program, USFOS. In the linear elastic range, the load displacement curve of Model-I matched well with the experimental results but Model-II and Model-III showed larger stiffness. Model-I gives the best approximation of the experimental load-bearing capacity; Model-III shows a collapse load almost twice as large while Model-II gives a load about 10% higher. The results of USFOS gives 30% higher collapse load. The numerical model including the three dimensional L-section beam finite element and the model of connections are very useful in predicting the ultimate behaviour of the lattice steel tower structures.

Al-Bermani et al (2009) presented a non linear analytical technique accounting for both material and geometric non linearity to predict transmission tower failure. In the proposed nonlinear analysis technique, the tower is modelled as an assembly of beam-column and truss elements. Linear geometric and deformation stiffness matrices are used to describe the behaviour of a general thin walled beam column element in an updated Lagrangian frame work. This approach greatly reduces the number of elements required for accurate modelling of the nonlinear structural response. A lumped plasticity approach coupled with the concept of a yield surface in force space is adopted for modelling the material nonlinearity. This method
was calibrated with the results from full scale tower test conducted on 275 kV D/C transmission tower. The technique showed a good accuracy in terms of failure load and failure mode.

Hemant Patil et al (2010) conducted failure analysis on 400 kV S/C horizontal configuration tower by conducting nonlinear Finite Element analysis using NE-NASTRAN software. Both geometric and material nonlinearities have been included in the analysis. It was predicted that the nonlinear analysis forces are higher compared to linear analysis force. Further the remedial measures have been suggested for the instability encountered in the structure.

2.5 EXPERIMENTAL INVESTIGATIONS ON THE FAILURE OF TRANSMISSION TOWERS AND PANELS

Knight and Santhakumar (1993) conducted experimental and analytical studies on a full scale quadrant of the lower most panel of a transmission line tower designed as a pin-jointed truss based on Indian standard for normal load condition. The test results established that the secondary stresses could be enough to cause failure of the leg members even under normal working load conditions. Transmission tower with secondary braces would be lighter than one designed without these braces. Joint effects shall be considered since it may produce premature failure. Analysis results correlated well with those observed if joints effects are considered. Failure surface shall be considered in predicting the collapse load even though the structure may be considered principally a space truss.

Sub-assemblage test of a half-scaled transmission tower was carried out to estimate its performance against wind load by Byoung-Wook Moon et al (2009). A 154 kV B-2 type transmission tower designed for 300 m
span was chosen for the study. Two panels in the middle of the tower were modelled for the experiment with a half scale in length and cross sectional area reduced to 25% of those of the prototype structure. The height of the test specimen is 3 m and plan dimensions are 2 m and 1.5 m square at bottom and top. The load applied to the test specimen was determined following the similarity law. As the wind load is proportional to the surface area of the structure, the lateral load imposed on the test specimen is reduced to 25% of the design load of the prototype structure. Two hydraulic actuators are used to impose both bending moment and lateral shear force to simulate the loads transmitted from the removed upper structure through a triangular jig. For preliminary analysis the specimen was modelled by beam elements and all elements are assumed to be rigidly connected. The numerical modelling and analysis of the model structure were carried out using the finite element analysis program MIDAS-Civil. Nonlinear analysis of the specimen was carried out using the finite element analysis program ANSYS. Nonlinear material property was found from the coupon test results. The triangular upper parts of the jig and angle member were modelled by BEAM188. The bolt connection was modelled by COMBIN7 with capabilities of joint flexibility, friction, damping and certain control features. The load was imposed on the test specimen gradually by displacement control. Local buckling occurred at one of the columns at the actuator displacement of 37 mm. Lateral displacement obtained in experiment was similar to the displacement computed from nonlinear analysis. The axial forces of leg members subjected to design load computed by numerical analysis corresponded to 80-90% of the buckling loads. The axial forces of bracing members turned out to be less than 13% of the buckling loads, which implies that adding additional braces may not increase stiffness and the strength of a transmission tower significantly. From the experiment it was observed that local buckling occurred at the two leg members subjected to compression. The local
buckling occurred as a result of the bending moment caused by unbalanced deformation and associated local buckling.

Perez et al (2009) presented the results of a test program to determine the compression strength of single angle members subjected to short duration axial load. Lattice steel transmission towers are largely constructed of single angle members. When a conductor breaks, the load on the tower is of large magnitude but short duration. The main focus of the study is to determine the behaviour of the angle sections used in transmission towers which have higher compressive strength subjected to short duration axial loads. The parameters of the short duration axial load to simulate a broken conductor applied to the angles, as a function of the time, were determined using CIGRE B2-308 (2004). This study discusses a full-scale test of a broken conductor and insulator on a transmission line. Time history plots of transmission line component loads are presented and used to develop the short duration load function.

2.6 STUDIES ON BOLT SLIP AND DEFORMATION OF TOWER

Kitipornchai et al (1994) theoretically investigated the effect of bolt slippage on the deflection and ultimate-strength response of lattice structures. Two idealized bolt-slippage models were presented. In Model 1, it is assumed that the two ends of a member, under either tension or compression will slip relative to one another by an amount when the axial force in the member exceeds the loading needed to initiate slip. The member length after slipping may increase or decrease depending on the type of force in the member. In this model, once a member starts to slip, no additional load is carried by the member until the assumed slip is completed. In Model II, slippage is assumed to be a continuous process from the onset of loading. A Ramberg-Osgood
type of model, which is generally used in describing the non-linear stress-strain curve of material, is used in this study. A transmission tower subassembly of 8m in height with square base of 3.7 m is studied by applying a combination of vertical, transverse and longitudinal loads at top. The responses of the tower with no bolt slippage and with bolt slippage, using slippage Models I and II are compared. In this example, a maximum bolt slippage of 1mm and slip load of 10% of yield load was allowed to occur in all members of the tower. It was concluded that, while slippage of bolts may have some effect on deflection, it does not significantly influence the ultimate strength of the structures. The study showed that the bolt slippage adds to the uncertainties in estimating structural deflection.

Ungkurapinan et al (2003) developed mathematical expressions to describe joint slip and load-deformation behaviour. Joint slip is the relative displacement of a bolted joint under shear. It is greater in transmission towers as bolt diameters are small, members joined are thin, bearing type joints with a lower clamping force are normally used, and the coefficient of friction of galvanized faying surfaces is low. 36 lap joint tests were conducted on bolted angle sections under compression by varying the bolt hole clearance from 1.6 to 3.2 mm, number of 16 diameter bolts from one to four and the edge distance of angle section from 25 to 51 mm. Each bolt was tensioned to 114.27 kN prior to testing. All the specimens were instrumented with electrical displacement transducers to measure the relative movement at a bolt location, relative deformation of an angle member between bolt holes and total joint deformation. Joint slip defined as early deformation that takes place at an approximately constant load increased from no construction clearance, normal construction clearance to maximum clearance.
Theoretical study was conducted for computing the individual bolt loads using Fisher and Rumpf theory. This was based on deformation measurements made in individual bolts and these are made in portions of angle sections between bolt holes. It was found that the outer bolts carry highest proportions of the loading. Greater care is needed in tightening the outer bolts to reduce the induced tension during initial tightening and it can be accomplished by initially tightening the middle bolts followed finally by the outer bolts in a joint. Joint slip is significant and is considerably more in friction type joints and takes place during service loads. Joint deformation should be incorporated in analytical methods dealing with transmission line towers. It was concluded that the current practice of using a construction clearance of 1.6 mm is satisfactory in spite of its influence on joint slip generation.

The analytical deflections computed for towers using computer software are less than those from test results. Prasad Rao et al (2004) conducted studies for deriving a relationship between the ratio of test to theoretical deflection and a non-dimensional parameter to serve as an index for monitoring the structural displacements during testing. Currently, structural dynamic evaluation plays an insignificant role in the design of towers. Using the fundamental frequency of a tower, the peak response of the tower to gusty wind and the impact force caused by conductor breakage can be evaluated. Both theoretical and experimental studies have been carried out to evaluate the natural frequencies of the towers tested at Tower Testing and Research Station, Structural Engineering Research Centre at Chennai. Based on these data an equation was derived using the tower geometry and test/theoretical deflection ratios, which enables us to predict the natural frequency of the tower in a way closer to its actual value. The non-dimensional parameter suggested form this study can be implemented in
analysis programs for predicting the correct deflections, while accounting for the increased deformation due to joint slips and rotations. The natural frequencies determined from the analysis programmes are higher than the experimental values. An equation was derived for modifying the analytical frequency of the tower if the analytical and experimental deflections are given.

Ju et al (2004) have studied the structural behaviour of the butt-type steel bolted joints by using three dimensional elasto-plastic finite element methods. The bolt clearance, the bolt head, the washer, the deformable bolt and the friction were included in order to simulate the actual structural behaviour of bolted connection. Studies are conducted to evaluate the plastic-strain field in the local area, nominal applied force of the bolted connection and the crack behaviour of the A36 steel plate. The numerical results are compared with AISC specification and found to be satisfactory despite the complication of stress and strain fields during the loading stages. When the steel reaches the nonlinear behaviour, the bolt nominal forces obtained from the finite element analyses are almost linearly proportional to the bolt number arranged in the connection. To calculate the ultimate load for the bolt shear failure of the connection with appropriate bolt spacing and end distance, it is appropriate to neglect the plate deformation and the bolt bending effects, which are major assumptions in the AISC specification for the bolted joints. Linear elastic fracture mechanics can still be applied to the bolted joint problem for the major part of the loading, even though this problem reveals highly non-linear structural behaviour.

Mohan Gupta and Gupta (2004) have investigated the stress distribution in bolted steel angles under tension. The stress distribution in the vicinity of connections in a bolted steel angle is non-uniform because of the
coupled effects of connection eccentricity, shear lag and stress concentrations. Although, some researchers have attempted finite element analysis, stipulations in various codes and specifications regarding the design of angle tension members are primarily based on the experimental studies. Only a couple of previous studies have included geometric as well as material non-linear effects in such finite element analysis. The study presented the state-of-the-art review of finite element techniques used in modelling the angle tension members with bolted connections. This study is followed by a non-linear finite element analysis so as to obtain the stress distributions in the vicinity of connections, at design loads. This stress distribution is then evaluated to draw several realistic conclusions. The magnitude and distribution of stresses at critical section for three bolts and four bolt connections is almost same. The resulting stress distribution justifies the use of area along gross shear plane in block shear strength prediction equation. The distribution and concentration of Von Mises stresses indicates that block shear failure may occur in a two bolts connection, and net section failure may occur in three and four bolts connection.

Discrete based clamping model consisting of a rigid bolt and nut connected with a discrete spring element, and a Stress-based clamping model consisting of pre-stressing a bolt defined with deformable solid elements and a model with deformable washers using non-linear finite element analysis was developed by Reid and Hiser (2005). Both techniques were used to simulate physical testing of a bolted joint undergoing slippage. Testing was carried out on a test fixture having a typical slip base geometry and surface characteristics to test the slip behaviour of a bolted joint. Testing was intended to provide the shear characteristics of bolted connection of an actual slip base mechanism, as well as provide a means of validating the modelling techniques. In the finite element model, the clamping forces are achieved with
a single centrally located discrete spring element. In the Stress-based clamping model, deformable solid elements with the material properties of steel, which when stretched through an initial deflection, would produce the desired preload in a manner consistent with an actual bolt was used. Parametric studies were conducted by varying the static and dynamic coefficients of friction between 0 - 0.4. Both techniques were used to simulate physical testing of a bolted joint undergoing slippage. Simulation results compared fairly well with the test results.

Nah et al (2009) conducted experiments for determining the slip coefficient of faying under static tension loading and to determine the loss of clamping force. The shear capacity of the connections and the relaxation of the clamping force under service load were evaluated. Splice plates of 12 mm thickness with three kinds of high strength bolts with zinc coating was used. Each bolt was clamped by a calibrated wrench method. Rusted surface and shot blasted surface exhibited slip coefficients of 0.61, 0.5 respectively. For red lead surface, the slip coefficient of 0.2 is considered and for zinc primer a slip coefficient of 0.4 is considered. The coating thickness greatly affects the loss of clamping force due to the creep behaviour of the coating. From the relaxation test it was observed that the first one week governs 85% of the total relaxation.

Xiaohong Zhang (2009) presented a detailed study on the post-elastic response of latticed towers combining advanced finite element analysis and full-scale dynamic testing of four tower section prototypes. USFOS, (Ultimate Software package for nonlinear static and dynamic analysis for Offshore Structures) a commercial software, developed for post-elastic analysis of offshore platform structures, was used to perform the nonlinear static and transient dynamic analysis of transmission towers. The eccentricity
of connection of the diagonal with respect to the horizontal is represented by an offset vector that can be defined in USFOS. Other offset vectors are defined to model the out-of-plane eccentricities with respect to the centroidal axis of the main leg. The offset vectors provide a simple way to model members with complex connection eccentricity details. To the real joint coordinates defined when building the model geometry, “virtual joints” are added with the offset vectors, and rigid links are automatically introduced between the pair of joints. With these rigid links, the bolted connection is assumed free to rotate (ideal pin) within the plane defined while the translations are linked between the master nodes and slave nodes. With rigid links representing the joint eccentricities, the USFOS analysis proceeds with the assumption that failure is governed by the members and no connection failure will occur. For the loading case of combined torsion and flexure on the tower section prototype, the structure was modelled with and without those eccentricities and a capacity 10 to 15% higher was obtained when the eccentricities were ignored. For the tower under bending only, the difference was not significant.

Fernandez et al (2010) proposed a method based on a combination of finite element method and data mining techniques to set up prediction models that can be used to calculate bolted connections. Based on the results of a finite element model validated by test, a number of finite element simulations are developed varying the thickness, bolt diameter, friction etc. The results of these simulations are used to generate a data base which can be used to create prediction models. Bolted connection was simulated in ABACUS. Butt joint with preloaded bolts was subjected to shear. The middle plates are prevented from sliding until the frictional force is exceeded and the connection starts to work on bearing basis. The definition of general contact in ABACUS is used for simulation of contacts between the plates and
between plates and bolts. The normal behaviour of the bolt shank with regards to the walls of the hole is modelled by the hard contact property. The friction coefficient is varied between 0.2 - 0.5 according to the treatment of the contact surfaces. The effect of bolt preloading is simulated by cooling the bolt shank with a dummy thermal dilation coefficient along the length. 144 finite element simulations are carried-out to obtain the sliding force, maximum sliding strength, and the maximum force that the connection can withstand. Data base is created with five input parameters and three output parameters and it was used to train the various prediction models.