NUMERICAL SIMULATION OF STRUCTURAL BEHAVIOUR OF TRANSMISSION TOWERS

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ABSTRACT

Transmission towers are a vital component and management needs to assess the reliability and safety of these towers to minimise the risk of disruption to power supply that may result from in-service tower failure. Latticed transmission towers are constructed using angle section members which are eccentrically connected. Towers are widely regarded as one of the most difficult form of lattice structure to analyse. Factors such as fabrication errors, inadequate joint details and variation of material properties are difficult to quantify. Consequently, proof-loading or full-scale testing of towers has traditionally formed an integral part of the tower design. Stress calculations in the tower are normally obtained from a linear elastic analysis where members are assumed to be axially loaded and, in the majority of cases to have pinned connections. In practice, such conditions do not exist and members are detailed to minimize bending stresses. Despite this, results from full-scale tower test often indicated that bending stresses in members could be as high as axial stresses. EPRI (1986) compared data from full-scale tests with predicted results using current techniques and concluded that the behaviour of transmission towers under complex loading condition cannot be consistently predicted using the present techniques. They found that almost 25% of the towers tested failed below the design loads and often at unexpected locations. Furthermore, available test data showed considerable discrepancies between member forces computed from linear elastic truss analysis and the measured values from full-scale tests. The paper describes a nonlinear analytical technique to simulate and assess the ultimate structural response of latticed transmission towers. The technique may be used to verify new tower design and reduce or eliminate the need for full-scale tower testing. The method can also be used to assess the strength of existing towers, or to upgrade old and aging towers. 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. The method has been employed by electricity utilities in Australia and other countries to: (a) verify new tower design; (b) strengthen existing towers, and (c) upgrade old and aging towers.

KEYWORDS

Angle member, computer simulation, lattice structure, full-scale test, nonlinear analysis, numerical method, thin-walled structure, transmission tower, ultimate strength

INTRODUCTION

Overhead transmission lines play an important role in the operation of a reliable electrical power system. Transmission towers are a vital component and management needs to assess the reliability and safety of these towers to minimise the risk of disruption to power supply that may result from in-service tower failure. One of the problems facing tower designers is the difficulty in estimating wind loads as they are based on a probabilistic approach. Another is tower strength which in contrast, could be deterministic provided a proven-reliable analytical tool is available for the specified design load conditions. In practice, factors such as fabrication errors, inadequate joint details and variation of material properties are difficult to quantify and they are often used to justify the use of full-scale tower testing. Strictly speaking, however, test results are only valid for the particular tower under the particular test loading conditions, and they may not predict exactly how a tower may behave in practice under different loading conditions.

This paper describes a computer simulation technique for predicting the ultimate structural behaviour of self-supporting and guyed latticed transmission towers under static loading. The technique can predict accurately the failure load and the failure mode of towers, and may thus be used to replace or reduce the need to carry out full-scale tower testing. The method has been employed by electricity utilities in Australia and other countries to: (a) verify new tower design; (b) strengthen existing towers, and (c) upgrade old and aging towers.

Three case studies are presented: (i) a calibration case study, (ii) a case study involving the strengthening of existing towers, and (iii) a case study involving upgrading old towers. For commercial reasons, ownership of the towers will not be revealed.

CURRENT ANALYTICAL TECHNIQUES

Latticed transmission towers are constructed using angle section members which are eccentrically connected. Towers are widely regarded as one of the most difficult form of lattice structure to analyse. Consequently, proof-loading or full-scale testing of towers has traditionally formed an integral part of the tower design. Stress calculations in the tower are normally obtained from a linear elastic analysis where members are assumed to be axially loaded and, in the majority of cases to have pinned connections. In practice, such conditions do not exist and members are detailed to minimize bending stresses. Despite this, results from full-scale tower test often indicated that bending stresses in members could be as high as axial stresses (Roy et al, 1984). EPRI (1986) compared data from full-scale tests with predicted results using current techniques and concluded that the behaviour of transmission towers under complex loading condition cannot be consistently predicted using the present techniques. They found that almost 25% of the towers tested failed below the design loads and often at unexpected locations. Furthermore, available test data showed considerable discrepancies between member forces computed from linear elastic truss analysis and the measured values from full-scale tests.

NONLINEAR ANALYTICAL TECHNIQUES

In the proposed nonlinear analytical technique, the tower is modelled as an assembly of beam-column 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 framework. This approach greatly reduces the number of elements required (Albermani and Kitipornchai, 1990a; 1992) 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 (Albermani and Kitipornchai 1990b) for modelling the material nonlinearity. The formex algebra approach (Albermani et al, 1992) is used for automatic generation of data necessary for the analysis.

All of the members in the tower are modelled in the analysis, including secondary bracing members. The technique accounts for both geometric and material nonlinearity. The geometric nonlinearity accounts for the effects of the accumulated stresses on the structural stiffness of the elements and the effect of the continuing changes in the geometry as the applied load is increased. Buckling of structural members can be detected during the load application. The material nonlinearity accounts for the effect of combined stresses on the plastification of the element cross-section. Stress-resultant yield surfaces and a lumped plasticity approach are used for this purpose (Albermani and Kitipornchai, 1990b). The analysis can also incorporate other nonlinear effects due to joint flexibility, bolt slippage (Kitipornchai et al, 1994) and differential support settlement.

In the analysis process, an incremental-iterative predictor-corrector solution strategy is used. Loads are applied in small increments. At each load increment several iterations are performed to satisfy equilibrium and the structural geometry is constantly updated. The solution method is equipped with a number of numerical strategies that enable prediction of any buckling or instability as well as tracing the nonlinear load-deflection path.

The described numerical simulation technique has been used to analyse self-supporting and guyed towers (Albermani, 1997) under specified loading conditions. Some of the towers modelled have subsequently been tested to failure. Predicted failure loads and failure modes are in good agreement with those obtained from tests. Over the past 10 years, this technique has been employed by a number of electricity utilities in Australia and in some other countries.

CASE STUDIES

Case Study 1: Verifying New Tower Design

A new 330 kV double circuit suspension tower was designed and tested to failure in Australia. The nonlinear analysis was used to verify the design and plan the test sequence prior to the full-scale testing. The tower is shown in Figure 1. It has a square base of 12.68mx12.68m and a height of 53.4m. The self-weight of the tower was 132 kN. Eight loading conditions were specified for the tower. The tower response was described in terms of a load factor, λ, which is the ratio of the applied load to the specified design load for the particular load case.

The tower was modelled using 1557 elements and 790 nodal points. This gave a total of 4740 degrees-of-freedom to model the tower response. The ultimate load factor obtained for the eight loading conditions varied from 1.10 to 1.78, indicating that the tower would have no difficulty in passing the full-scale test. Results from the nonlinear analysis, assuming nominal yield stresses, indicated that failure modes of the tower were due to the spread of plasticity for the loading conditions. The tower was full-scale tested to 100% ultimate design load, using the recommended test sequence and, as expected, the tower passed the test under all loading conditions.

Load Case 8A (microburst wind on full tower, loads kept at 100% design loads, except for transverse conductor and earthwire loads) was chosen as the loading condition to test the tower to failure. The predicted ultimate load factor for this loading condition was 1.20 with failure due to the spread of plastic hinges.

Tensile tests were conducted on various members of the tower and actual yield stresses were found to be at least 10% higher than the assumed nominal yield stresses. The tower was then re-analysed under Load Case 8A, assuming higher yield stresses. The nonlinear analysis predicted that the tower would fail by buckling in the compression legs (see Figure 2(a)) at load factor 1.307. The tower was tested to failure and reported failure at load factor 1.30 by buckling of the compression face of the tower (see Figure 2(b)), virtually identical to the prediction. This demonstrated that the nonlinear analysis was capable of predicting accurately both the failure load and the failure mode of the tower.

Case Study 2: Strengthening Existing Towers

A 220 kV transmission line with about 1300 suspension towers was constructed in the early 1980's. The line experienced two major tower failures at approximately seven years apart. The cause of failure was thought to be the higher than expected wind loads. The investigation involved analysing the tower for higher wind loads and identifying weak areas in the tower so that suitable schemes could be devised to strengthen the tower to reduce the potential risk of a future failure.

The geometry of a typical tower along this line is shown in Figure 3. The tower has a rectangular base of 6.5m in the transverse direction, 3.55m in the longitudinal direction and a height of about 39.0m. The self-weight of the tower was 36 kN. Nine new revised loading conditions were used to evaluate the *as-built* tower response.

The tower was modelled using 1100 elements and 730 nodal points. This gave a total of 4380 degrees-of-freedom. The ultimate load factors obtained for the nine revised loading conditions varied from 0.59 to 1.55 for the *as-built* tower. Results obtained from the nonlinear analysis revealed that the tower would collapse under three of the nine loading conditions. The collapse was due to either spread of plasticity or premature buckling. Figure 4(a) shows the tower response under one such loading condition where the tower collapsed at a load factor of 0.74. Figure 4(b) shows the magnified deflected shape of the tower at collapse. Under this loading condition, plastic hinges initiated at a load factor of 0.69 in the compression leg at the lower part of the Common Body and spread downward in this leg.

Results from the nonlinear analysis indicated that the *as-built* tower would fail at loads significantly below the revised ultimate design loads. A number of possible modification schemes to strengthen the tower were investigated. The first scheme involved adding a diaphragm at two locations with some horizontal and secondary bracing members. The addition of these new members was introduced in stages referred to as Upgrades 1 to 4 as shown in Figure 5. With these upgrades, the tower response improved but still did not reach the revised ultimate design loads under two loading conditions.

The second modification scheme involved using stay wire to strengthen the *as-built* tower. Four stay wires (19/3.25mm) tensioned to 25kN were attached to the tower as shown in Figure 6(a). In addition, four ring members (75x75x5 MS) were added to the tower at the stay attachment level to help transferring loads from the stay to the tower as shown in the same figure.

The stayed tower was re-analysed under the same nine loading conditions. A significant improvement in the tower response was observed. However, the tower still failed to reach the ultimate design loads under one loading condition where plastic hinges formed at the compression leg just below the stay attachment level. In order to prevent this failure, further modifications to the stayed tower were made. These include increasing the pretension force in the stay from 25 to 50kN and the addition of a horizontal diagonal member at the stay attachment level and a set of ring members just below the attachment level. These modifications are shown in Figure 6(b). With these modifications, the tower was re-analysed and passed successfully for all the nine revised ultimate design loading conditions with the lowest load factor being 1.03.

Case Study 3: Upgrading Old Towers

An existing 400 kV transmission line was designed and constructed almost 45 years ago. The line has performed its function and suffered no tower failure. The nonlinear analysis was conducted to determine if the capacity of the towers in this line could be upgraded to carry larger and heavier conductor loads, and to devise appropriate practical upgrading schemes for the towers.

One of the towers analysed is shown in Figure 7(a). It has a square base of about 9.5mx9.5m and a height of about 49.0m. The tower self-weight was calculated to be 127kN. Seven loading conditions were specified based on the revised wind and incorporating larger conductor loads. The tower was modelled using 1245 elements, 660 nodal points and 3960 degrees-of-freedoms. In the nonlinear analysis, the vertical loads were applied first up to 100% of their specified values, followed by the incremental application of the transverse and the longitudinal loading.

Results from the nonlinear analysis indicated that the tower did not reach the new ultimate design loads in four out of the seven loading conditions. Load factors at failure ranged from 0.87 to 1.32. Based on results from the nonlinear analysis including the failure pattern, the tower was upgraded by adding a horizontal diaphragm as shown in Figure 7(b). The upgraded tower was re-analysed using the same seven loading conditions. With this modification, the tower was able to carry the increased loads without any difficulty with the lowest load factor being 0.99.

CONCLUSION

The paper describes a nonlinear analytical technique for simulating the ultimate structural response of latticed transmission towers. Accurate structural analysis of towers is complicated because the structure is three-dimensional and comprised of angle section members eccentrically connected. The influence of geometric and material nonlinearities plays a very important role in determining the ultimate behaviour of the structure. The proposed technique may be used for verifying new tower design thereby reducing or eliminating the need for full-scale tower testing, in addition to providing a degree of design *confidence*. It can also be used for assessing strength of existing towers, or upgrading old and aging towers.

Three case studies have been reported. In the first case, the technique was used to verify design prior to full-scale tower testing which included loading the tower to failure. The method has been shown to predict accurately both the failure load and the failure mode. In the second case, the technique was used to strengthen existing towers and in the third case, the technique was employed to upgrade the capacity of towers that were 45 years old. In Case Studies 2 and 3, proof-loading of the towers would have been very difficult, if not impossible.

ACKNOWLEDGEMENTS

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REFERENCES

Albermani, F.G.A. (1997), Design verification of guyed transmission tower using nonlinear analysis, International Journal of Space Structures, **12, 1,** 43-50.

Albermani, F.G.A. and Kitipornchai, S. (1990a), Nonlinear analysis of thin-walled structures using least elment/member, Journal of Structural Engineering, ASCE, **116, 1,** 215-234.

Albermani, F.G.A. and Kitipornchai, S. (1990b), Elasto-plastic large deformation analysis of thin-walled structures, Engineering Structures, **12, 1,** 28-36.

Albermani, F.G.A., Kitipornchai, S. and Chan, S.L. (1992), Formex formulation of transmission tower structures, International Journal of Space Structures, **7, 1,** 1-10.

Albermani, F.G.A. and Kitipornchai, S. (1992), Nonlinear analysis of transmission towers, Engineering Structures, **14, 3,** 139-151.

Electric Power Research Institute (1986), Structural development studies at the EPRI transmission line mechanical research facility, Interim Report No. 1: EPRI EL-4756, Sverdrup Technology Inc., Tullahoma, Tennessee.

Kitipornchai, S., Albermani, F.G.A. and Peyrot, A.H. (1994), Effect of bolt slippage on the ultimate strength of latticed structures, Journal of Structural Engineering, ASCE, **120, 8,** 2281-2287.

Roy, S., Fang, S. and Rossow, E. (1984), Secondary stresses on transmission tower structures, Journal of Energy Engineering, ASCE, **110, 2**, 157-172.

LIST OF FIGURE CAPTIONS

Figure 1: Case Study 1 – 330 kV DC Suspension Tower

Figure 2: Predicted Failure and Test Failure of 330 kV DC Suspension Tower

Figure 3: Case Study 2 – 220 kV As-built Tower

Figure 4: Load-Deflection Curve and Tower Deflected Shape of 220 kV As-Built Tower

Figure 5: First Modification Scheme to Strengthen 220 kV Tower

Figure 6: Second Modification Scheme to Strengthen 220 kV Tower

Figure 7: Case Study 3 – Upgrading 400 kV Suspension Tower

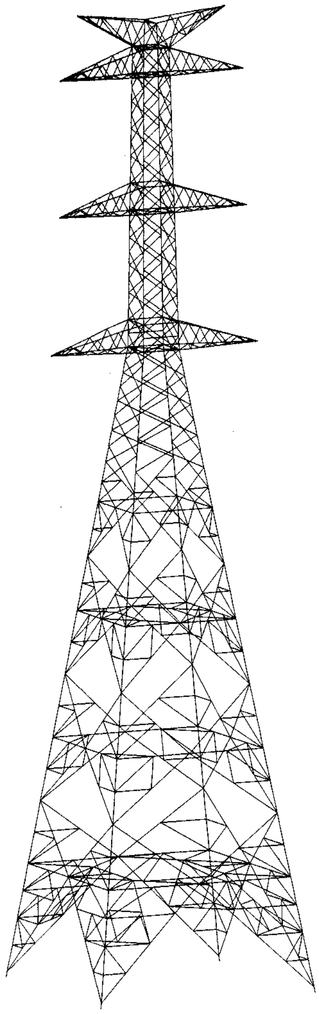

Fig 1

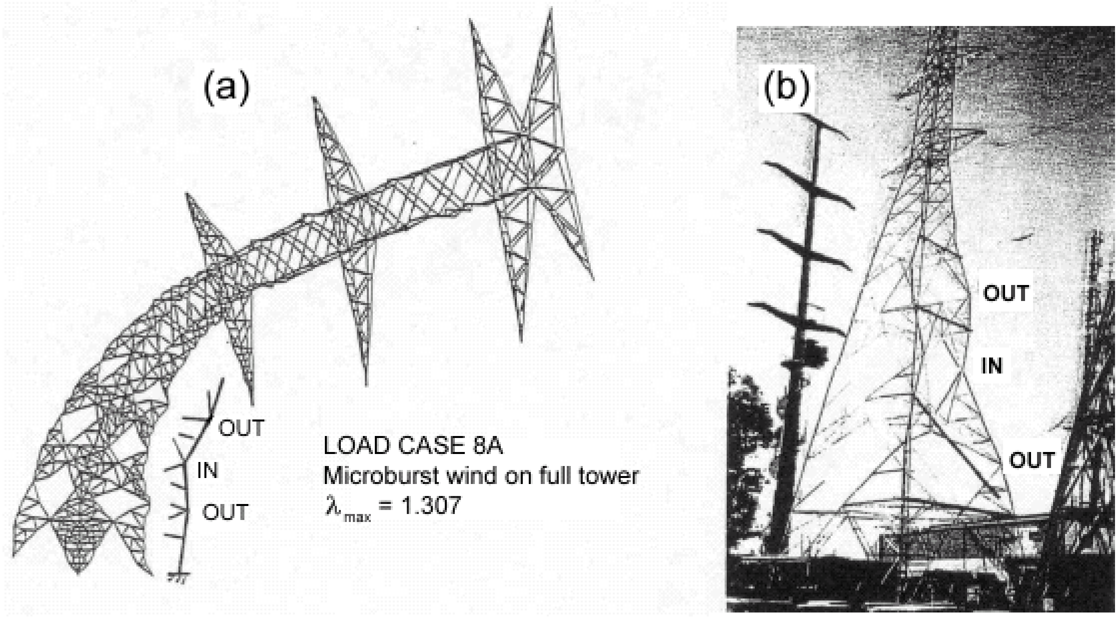
(a)
OUT
IN
OUT
LOAD CASE 8A
Microburst wind on full tower
λ_{max} = 1.307
(b)
OUT
IN
OUT

Fig 2

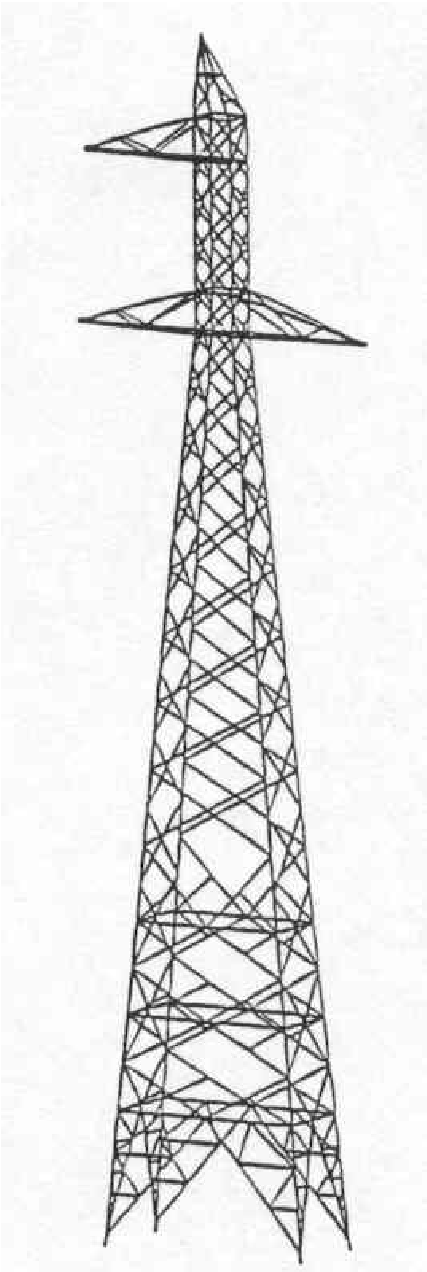

Fig 3

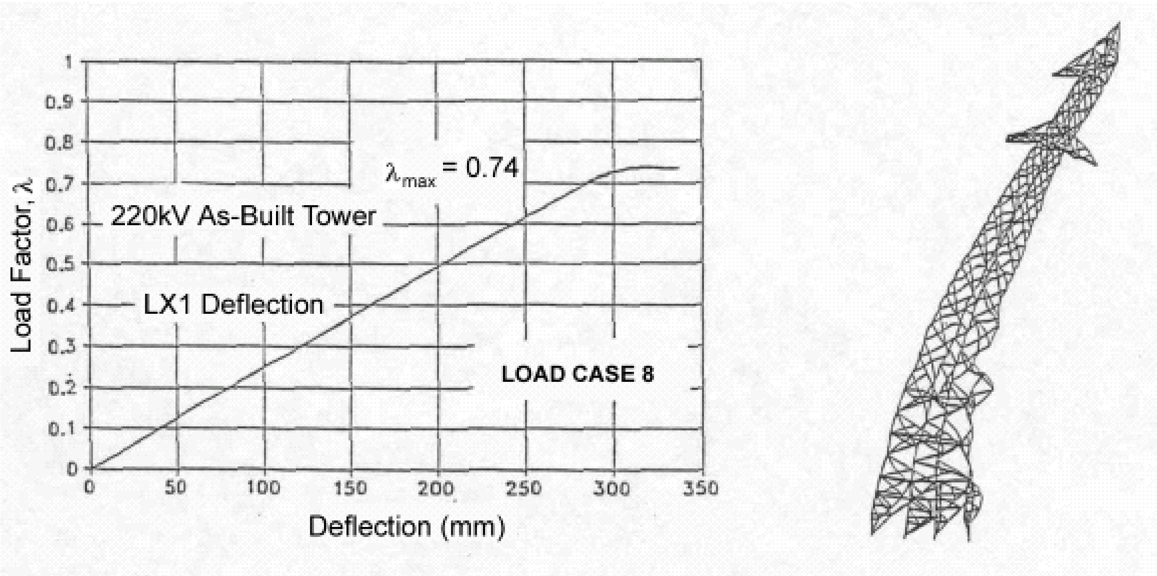
Load Factor, λ
λmax = 0.74
220kV As-Built Tower
LX1 Deflection
LOAD CASE 8
Deflection (mm)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0
50
100
150
200
250
300
350

Fig 4

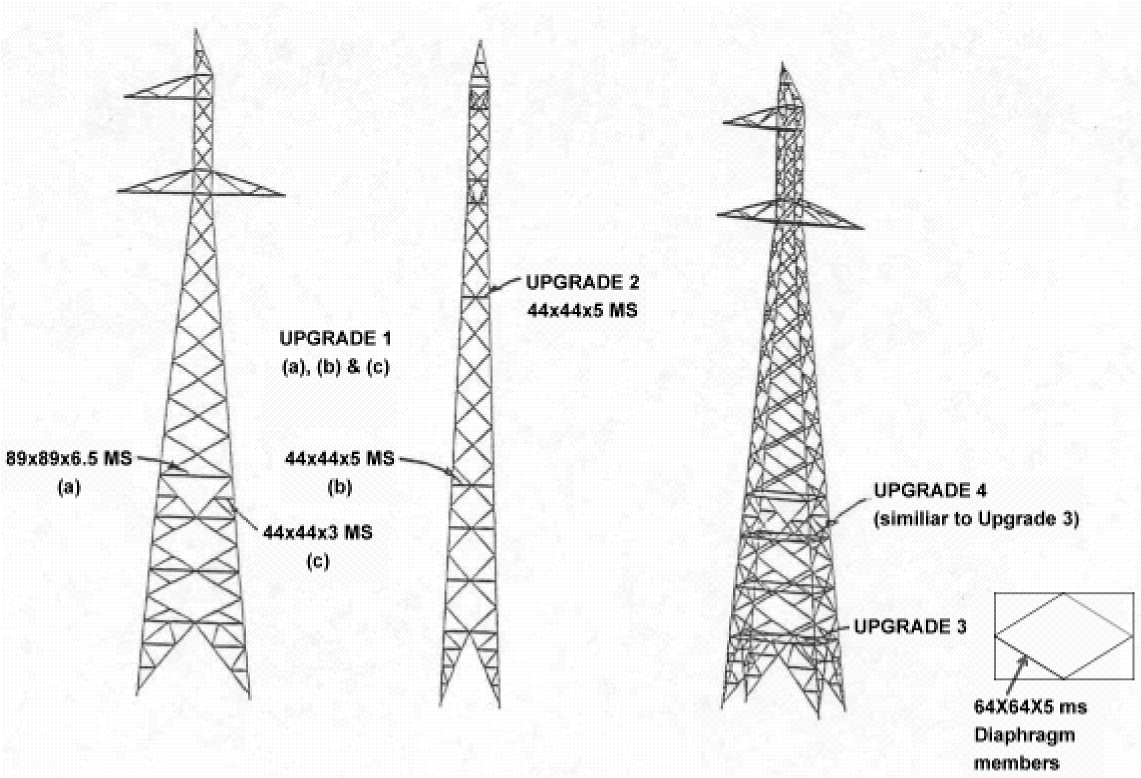
UPGRADE 2
44x44x5 MS
UPGRADE 1
(a), (b) & (c)
89x89x6.5 MS
(a)
44x44x5 MS
(b)
44x44x3 MS
(c)
UPGRADE 4
(similiar to Upgrade 3)
UPGRADE 3
64X64X5 ms
Diaphragm
members

Fig 5

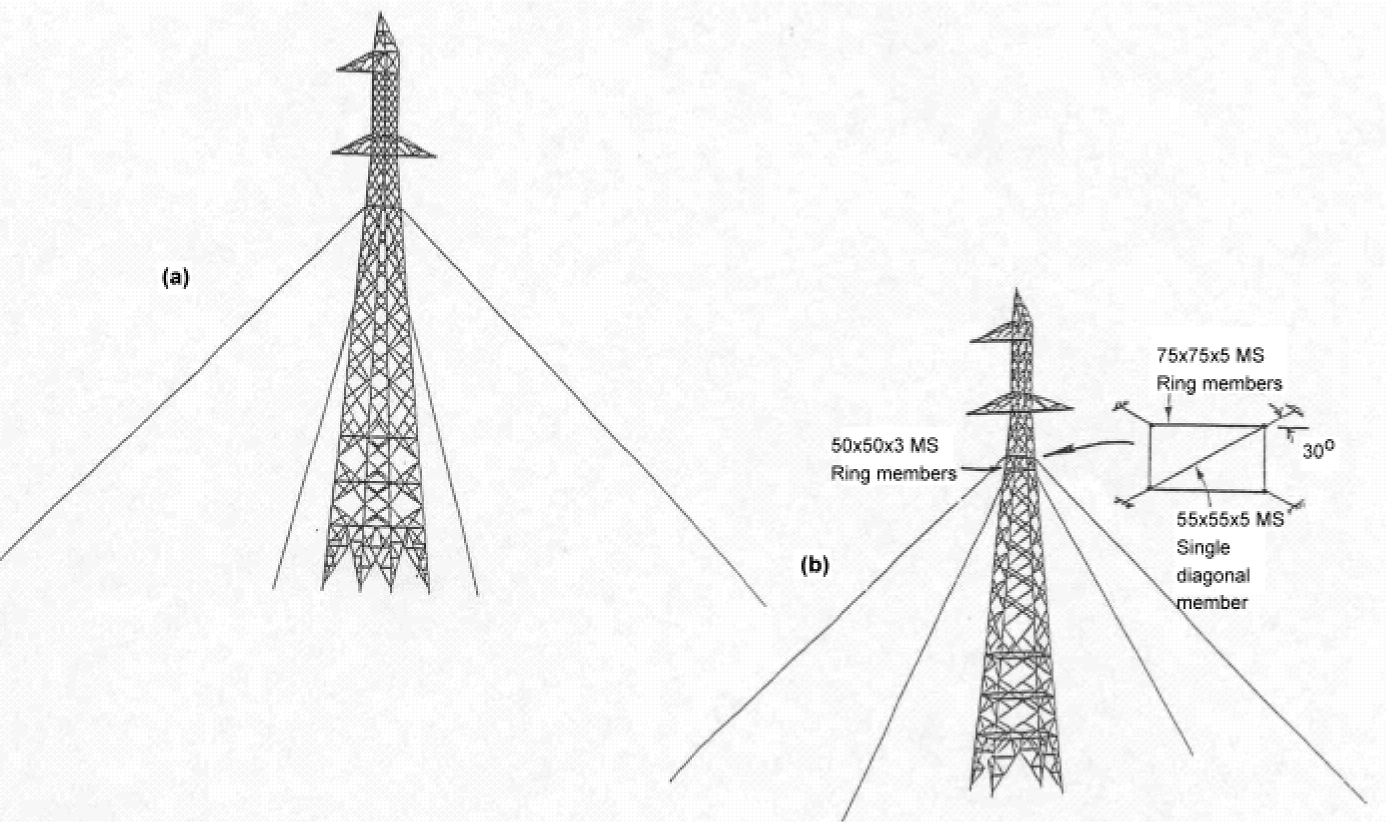
(a)
(b)
75x75x5 MS
Ring members
30°
50x50x3 MS
Ring members
55x55x5 MS
Single
diagonal
member

Fig 6

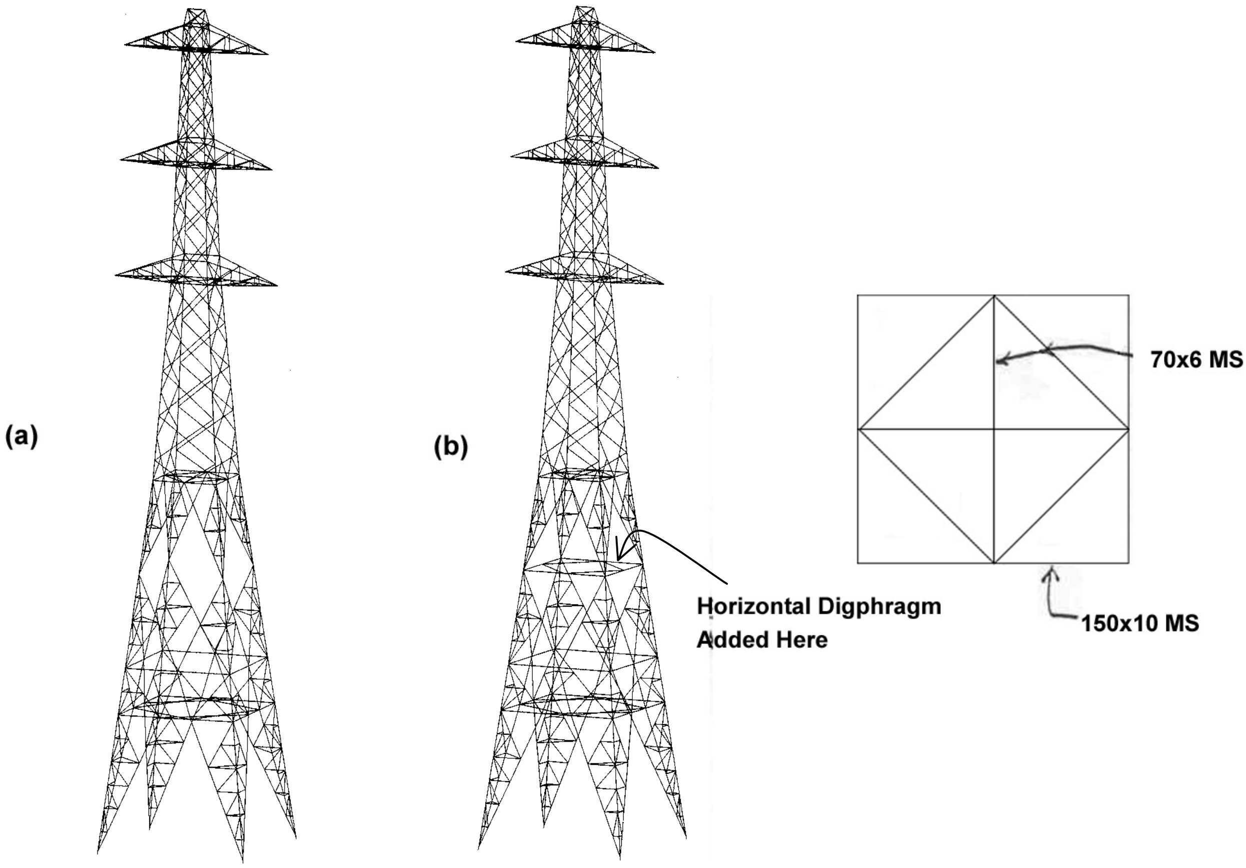

Fig 7