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Seismic Analysis of Guyed Mast Towers in Sri Lanka

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Abstract: With the rapid development of the telecommunication sector in the country, telecommunication/broadcasting towers play a vital role in telecommunication and broadcasting sectors. There are many structural forms available for towers and guyed mast is one such type commonly seen in country sides of Sri Lanka where land is available for cheaper price. Moreover, in the case of very tall tower is needed, guyed mast is more economical solution than self-supporting towers. The failure of a guyed tower especially under a disaster situation such as earthquake is a major concern in many ways. One is the failure of communication/broadcasting may hamper the communication needs to carry out rescue and other essential operations. Further, failure of a tower may itself cause a considerable economic loss as well as damages to human life. Therefore, checking of structural performance tower under seismic and other extreme weather effects is quite vital.

Even though, Sri Lanka was believed to have no seismic threats, presently a strong argument is going on amongst the professionals regarding the seismic condition of our country with the reported earth tremors in recent times. Hence, evaluating the structural performance of existing telecommunication/ broadcasting towers under seismic loads is utmost important since almost all existing towers have not been designed considering seismic forces due to traditional belief that Sri Lanka will not be subjected to earthquakes of appreciable magnitudes.

Considering the above situation, assessment of structural performance of exiting Guyed mast towers (which were not initially designed considering earthquake loading) under possible earthquake loading was selected as the objective of this study. Accordingly, behavior of existing Guyed mast towers under seismic loading using ANSI/TIA-222-G tower design code was studied and results, observations and conclusions based on this analysis are presented.

Keywords: Guyed mast, seismic loading. Telecommunication towers

1. Introduction

Telecommunication and broadcasting sectors of the country has developed exponentially over last few decades and, a large number of telecommunication /broadcasting towers are available in the country to facilitate wireless data and signal transmissions. Guyed mast is one of the economical structural forms available for taller towers.

A failure of tower will itself cause a considerable economic loss as well as possible loss of lives. So extreme care should be given to the design of these structures especially guyed mast towers which show nonlinear response because of guy assembles. However, almost all of these towers were designed only considering wind loading since Sri Lanka was considered as a country free from earthquake. However, after Tsunami that is caused by an earthquake and with the recent tremors reported in the country, now most of the structural designers and professionals are aware of the importance of considering seismic effects for their designs. Therefore, it is worth to study performance of guy mast towers too under possible seismic effects.

2 Objective

The objective of this research is assessing the performance of exiting guy mast towers (which were not initially designed considering earthquake loading) under possible earthquake loading.
Various types of telecommunication towers with different structural forms are available in the country and this study has been limited to analysis of guyed mast towers since seismic performance on Greenfield self-supporting lattice towers, which are the most common type of telecommunication towers in this country, have been studied in the previous researches. Guyed mast is second most commonly used structural form for telecommunication towers next to self-supporting towers in Sri Lanka.

3. Methodology

For analysis of guyed towers under earthquake loading, equivalent static method given in ANSI/TIA-222-G-2005was used since there are no time history data are available for Sri Lankan context.(this is further discussed in chapter 4.2) Seismic analyses were also carried out under different seismic conditions relevant some other selected countries for comparison purpose.

Two towers having different tower heights of 35 m, and 55 m were selected for this analysis. These towers have been designed for wind speed of 50 m/s (180km/h), which is slightly above the recommended design wind speed for Zone 1 for normal structures .

ANSI/TIA-222-G-2005 Structural Standard for Antenna Supporting Structures and Antennas, which is highly appreciated and very commonly used code of practice by both local and foreign tower designers, was used for the structural analysis and design of towers under both wind and seismic loadings.

3D computer models for each tower were prepared using SAP2000 structural analysis software and analysis of towers under both wind and earthquake loads were carried out using such models. Finally, the results of analyses under wind and earthquake loads were compared.

4. Loading

4.1 Wind loads

Calculation of wind loads on towers were carried out according to ANSI/TIA-222-G-2005 for the design wind speed of 50 m/s (180 km/h) , which is close to the recommended design wind speed for Zone 1 Normal structures condition. Wind loads were also calculated for the wind speed of 33.5 m/s, which is the lowest allowable design wind speed that can be used for structural design in Sri Lanka, for the purpose of comparison of results.

4.2 Seismic loads

For the calculation of seismic loads on towers, four methods are given in the ANSI/TIA-222-G -2005. Those methods are

- Equivalent lateral force
- Equivalent modal analysis
- Modal analysis
- Time history analysis

According to the code, only equivalent static method and time history method are applicable for guyed mast towers. First method is a static method and next is a dynamic method.

4.2.1 Equivalent Static Method

For the calculation of seismic shear, Maximum considered earthquake spectral response acceleration at short period (S_S) and Maximum considered earthquake spectral response acceleration at 1.0 second (S_1) are required. These are site specific acceleration coefficients and these values for countries other than USA have not been given in ANSI/TIA-222-G-2005. Further, recommended seismic acceleration parameters are not locally available, since code of practice for seismic design is not available in Sri Lanka yet. So values used in the previous researches were considered.

It was decided to calculate S_s and S_1 using the approximate method given in USGS website[14] based on Peak Ground Acceleration of 0.1g, which is value used by Gunathilaka eta el of their study on Greenfield towers[01] . Accordingly, S_s and S_1 were calculated as 0.5 and **0.2** respectively. These two values are quite close to the recommended values for south India and cities in Australia in USGS, where similar type of seismological condition exists when compared with Sri Lanka. This would correspond to minor to moderate damage condition.

In order to compare the seismic performance of towers under higher earthquake magnitudes, another set of site-specific acceleration coefficient were also considered. Accordingly, site specific acceleration coefficients for Pakistan as S_s = 1.22 and S_1 = 0.49 given in USGS website was selected. The above values applicable to Pakistan represent severe seismic condition.

For the calculation of fundamental natural frequency of a tower, a formula has been given in ANSI/TIA-222-G-2005 [14]. However, to obtain better accuracy, natural frequencies were obtained from the modal analysis performed using SAP 2000 model and calculated fundamental natural frequencies for 35 m and 55m towers are 3.67 Hz and 2.96 Hz. The formula given in ANSI/TIA-222-G [14] gave values of 3.47 Hz and 2.48 Hz for 35m and 55m respectively and those are close range with the SAP 2000 values.

To compensate for mass of antennas and other ancillaries (such as ladders , feeder cables, platforms, etc) material density of member materials were modified by a factor individually calculated for each model based on ratio of pure weight of tower members and actual weight of tower including all ancillaries.

Consideration of masses of all ancillaries is important since mass of such items could contribute significantly for seismic force generation of a tower under an earthquake as the weight of ancillaries including antennas takes considerable portion of overall self-weight of an actual tower**.**

4.2.2 Time history analysis

For carrying out time history analysis of towers, guidelines given in ANSI/TIA-222-G should be adopted. There are no recorded events which represent the past earthquake occurred in Sri Lanka. So it was decided to use only the static analysis procedures for our research, as response spectrum method is not suitable for guyed mast towers according to ANSI/TIA-222-G-2005 [14].

(Whole procedure of calculations under Equivalent Static Method is described in Appendix "A".)

5. 3d modeling

3D finite element truss models were prepared for both towers (35 m & 55 m). The towers are modeled as elastic three-dimensional truss model where individual members of the mast are modeled as straight members connected at joints producing only axial forces in the members.

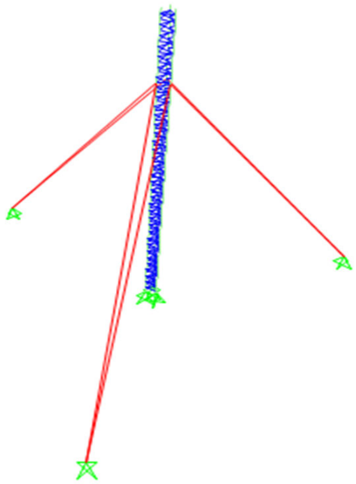

Figure 1 3D model of 35m tower

Vertical members are modeled by using sixty angle sections and cross bracing members are modeled by using L angle section.

Cable is modeled by using frame element. In addition, bending stiffness of the frame elements are reduced by scale multiplier. In addition, compression limits of those frame elements are set to zero to idealize the structural characteristics of cables.

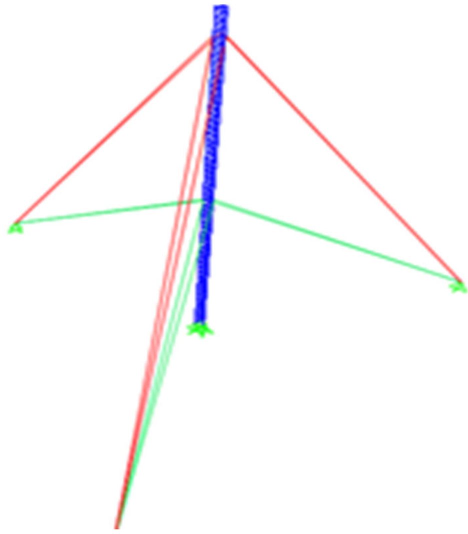

Figure 2: 3D model of 55m tower

6. Analysis and results

Each of the towers was subdivided to panels according to geometries of towers and wind and earthquake load under static equivalent analysis approach were separately calculated for each panel. The calculated wind and earthquake loads for each panel were assigned as nodal loads for respective tower models.

As per ANSI/TIA-222-G-2005 (14) specifications, following load cases given in Table 1 were considered in this study.

Supports reactions, maximum axial forces in leg members and maximum base reaction, Maximum joint displacement, Maximum guy tension of each tower for the load combinations described above were obtained from SAP 2000 analysis results of respective tower models.

As expected, maximum uplift reactions, , tension in guys and members in each and every case are observed when dead load has a factor of safety of 0.9, while maximum downward and horizontal reactions and compression forces in members are observed when dead load has a factor of safety of

1.2. Only critical load combinations in respective structural actions are shown in respective graphs.

Table 1: Considered load combinations for static analysis

Load case	Case Name	Remarks
1	0.9D+1.0Dg+1.6W33.5	Under 33.5m/s wind speed
2	0.9D+1.0Dg+1.6W50	Under 50m/s wind speed
3	1.2D+1.0Dg+1.6W33.5	Under 33.5m/s wind speed
4	1.2D+1.0Dg+1.6W50	Under 50m/s wind speed
5	0.9D +1.0Dg+ 1.0Emod.	Earthquake load under Appropriate condition for SL
6	0.9D+1.0Dg+1.0Esev.	Earthquake load under severe seismicity
7	1.2D+1.0Dg+1.0Emod	Earthquake load under Appropriate condition for SL
8`	1.2D+1.0Dg+1.0Esev.	Earthquake load under severe seismicity condition

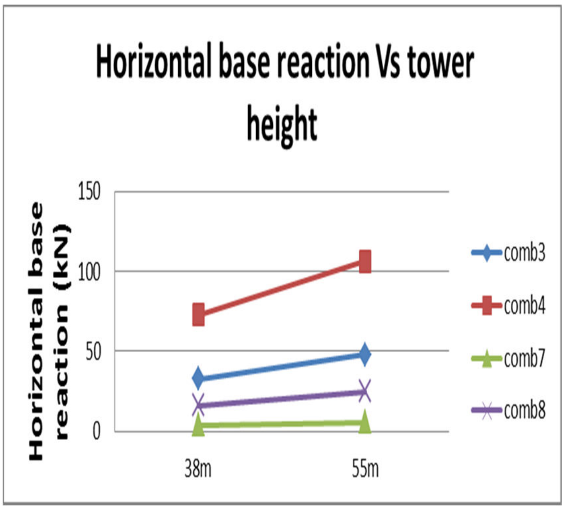

Figure 2: Variation of maximum horizontal base reaction

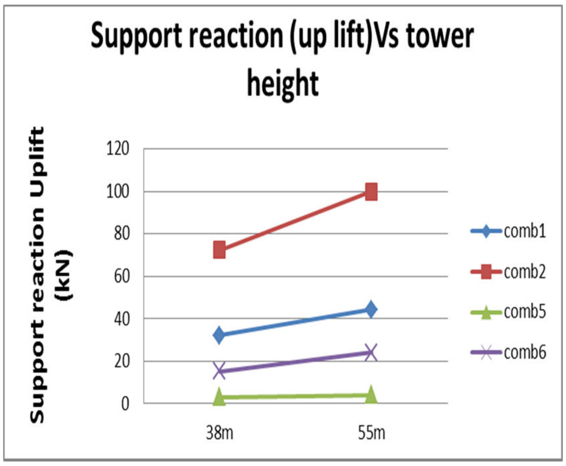

Figure 3: Variation of maximum base uplift

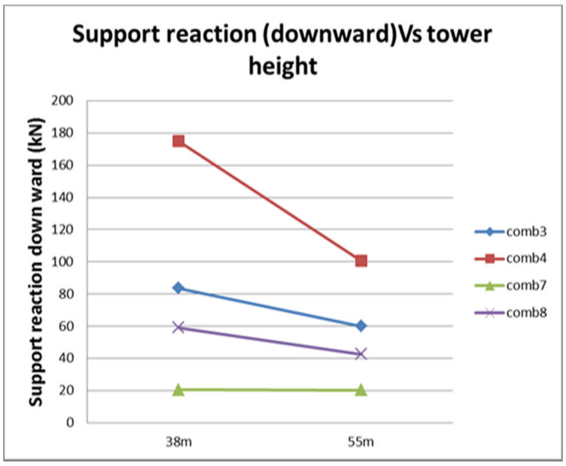

Figure 4: Variation of maximum downward base reaction

According to results of the graphs, support reactions under assumed earthquake loading condition for Sri Lanka are very much less than the support reaction under design wind loading, even if for design wind speed of 33.5m/s. The gap between reactions (uplift and horizontal) under wind loading and earthquake loading increases with the increase of the tower height. But when it considers about downward support reaction, when the tower height increases the gap reduces.

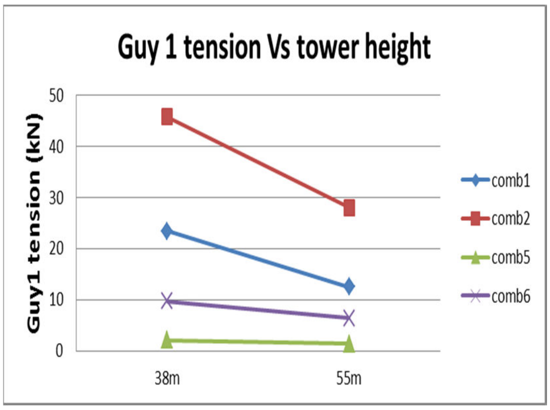

Figure 5: Variation of maximum axial tension (lower guy)

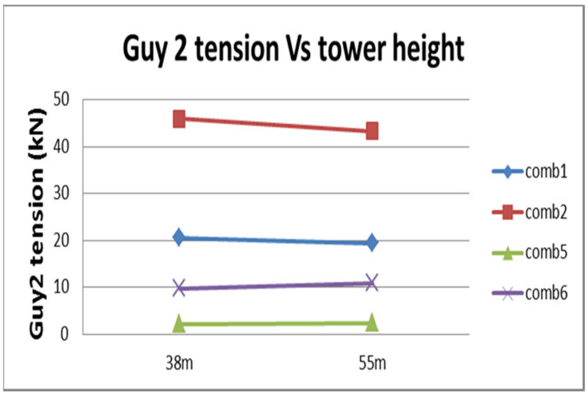

Figure 6: Variation of maximum axial tension in top guy

When it considers the results obtained for axial tension in cables (Figure 6 and 7)), they also exhibit a similar variation like support reactions (uplift and horizontal).

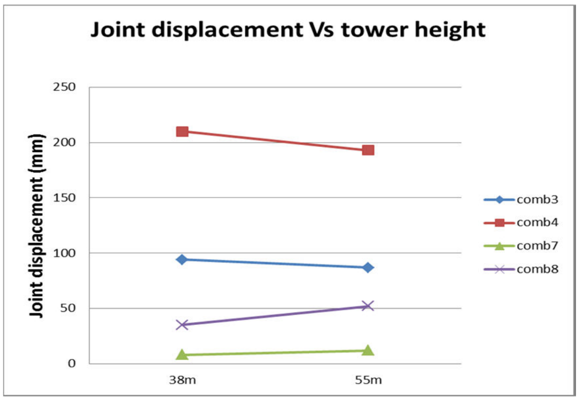

Figure 7: Variation of maximum joint displacement

Figure 8 shows the variation of maximum joint displacement of towers with respect to the considered load combinations. It is also very clear that tower deflection under assumed earthquake loading condition for Sri Lanka is far below the deflections under wind loading conditions. However, earthquakes could induce higher deflections due to dynamic nature of forces and dynamic analysis will require to verify it fully.

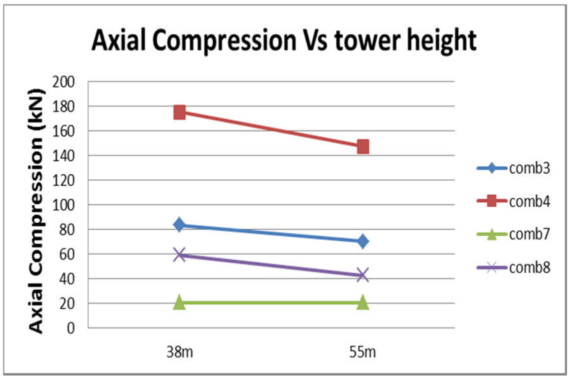

Figure 8: Variation of maximum axial compression in members

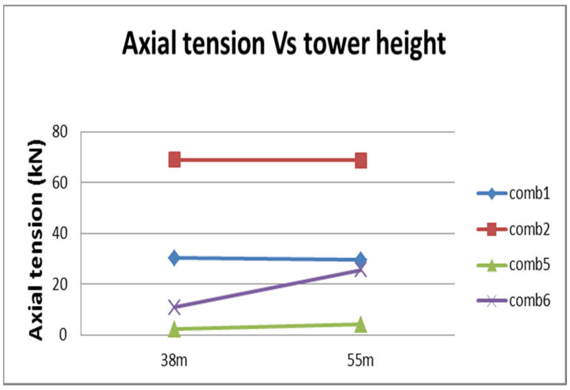

Figure 9: Variation of maximum axial tension in members

Maximum axial forces (both compression and tension) in leg members vary in same way as in support reactions. In other words this means, member stresses developed under assumed earthquake loading for Sri Lanka is not critical compared with member stresses under design wind load condition of towers. However, under earthquake loading calculated based on very severe seismicity condition, axial tension forces of leg members has almost reached the design values under 33.5m/s wind load (Figure 10). So when the height of the tower increases there is a possibility for the dominance of earthquake forces compared to wind forces.

The results obtained from this study match with the results of previous studies [1],[13] carried out in Sri Lanka for other types of towers. There are some other researches [4] done on this topic in different countries gave similar kind of results. Also, ANSI/TIA-222-G-2005[1] in itself has specified that analysis under earthquake loading for normal towers are not required if S_s is less than or equal to 1.00. This has also been proved by this analysis.

7. Conclusion

As per the objective of this study, selected guyed mast towers were analyzed using static equivalent method given in ANSI/TIA-222-G-2005 (14) to assess the structural performance of guyed mast towers under seismic loading and compared with wind load analysis. Accordingly, some interesting findings were seen as described below

• Structural actions (member forces, support reactions deflections ,etc) developed in all selected guy towers under most probable type of seismic loads relevant to Sri Lanka are very low compared with same under design wind loads when seismic analysis done as per Static Equivalent method given in ANSI/TIA-222-G-2005 (14) . Hence, it can be expected that existing towers in this height range will survive without any problem under a minor to moderate earthquake, which is the most probable type of earthquake that can be expected to a country like Sri Lanka. Further, even under a considerable major earthquake, structural actions in towers will not be greater than structural actions under design wind loads in all selected towers. Hence, it cannot expect a major problem in guy towers in this height range even under major earthquake.

• As there are no suitable time history data available for Sri Lanka, only static analysis procedure was adopted in this study. As per the ANSI/TIA-222-G-2005 (14), it cannot use other approaches as Response spectrum method too for guy mast analysis. Therefore, further studies regarding these towers using appropriate time histories obtained from other counties is recommended to verify the results obtained in this study since Static Equivalent method is only a simple conservative analytical tool for seismic analysis.

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Appendix A

Calculation of equivalent static load for 35m and 55m towers

The following equation is given in ANSI/TIA-222-G [14] to calculate total seismic shear V_s under equvalant static method and it was used for the calculation of earthquake loading of towers.

$$V_s = \frac{S_{DS} W I}{R}$$

Alternatively, for ground supported structures, V_s need not be greater than

$$V_s = \frac{f_1 S_{D1} W I}{R}$$

When the alternative equation for V_s is used , V_s shall not be less than $0.044 S_{DS} WI$ and for sites where S_1 equals or exceeds 0.75 , V_s using the alternative equation shall not be less than

$$V_s = \frac{0.5 S_1 W I}{R}$$

$S_{DS} = 2/3\ S_S$

$S_{D1} = 2/3\ S_1$

Where;

S_{DS}- Design spectral response acceleration at short period

S_{D1}- Design spectral response acceleration at period of 1.0 second

S_1 - Maximum considered earthquake spectral response acceleration at 1.0 second

S_s- Maximum considered earthquake spectral response acceleration at short period

f_1 - Fundamental frequency of the structure

W- Total weight of structure including appurtenances

I - Importance factor

R - Response modification coefficient equal to 3.0 for lattice self supporting structures

V_s- Total seismic shear

The vertical distribution of seismic force was done according to following formula given in ANSI/TIA-222-G[14].

$$F_{sz} = \frac{W_z h_z^{ke}}{\sum_{i=1}^{n} W_i h_i^{ke}}$$

Where;

F_{sz}= Lateral seismic force at level Z

W_z= Portion of total gravity load assigned to level under consideration

W_i= Portion of total gravity load assigned to level i

h_z = Height from the base of the structure to level under consideration

h_i = Height from the base of the structure to level i

k_e= seismic force distribution exponent (taken as 2.0 is it can set as 2.0 for any structure)