ON THE APPLICATION OF ROBUSTNESS CRITERIA TO STEEL LATTICE MASTS

Simos Gerasimidis, Evangelos Efthymiou
Charalampos C. Baniotopoulos

Institute of Metal Structures, Department of Civil Engineering, Aristotle University of Thessaloniki, GR–54124, Greece, e-mail: ccb@civil.auth.gr

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Abstract: The present paper deals with the application of robustness criteria to special structures used in telecommunications, the steel lattice masts. Although progressive collapse is considered a widely familiar term and most engineers have heard of well-known events of this type of failure, such as the accident at Ronan Point in London, or the collapse of the towers of the World Trade Center in New York, there is not much progress made towards the quantification of the problem. The goal of this paper is to apply a method, proposed by the authors, of quantifying robustness to steel lattice masts and investigate the behavior of the structure in view of robustness. For the purpose of this research activity a typical and very common type of steel lattice mast regarding its geometrical and structural characteristics is used, whereas all the parameters of the study are presented and thoroughly described.

Keywords: Progressive collapse, Steel lattice masts, Time history analysis, Robustness safety factor

1. Introduction

Progressive collapse can be described as the disproportionate result caused by a triggering event such as local action or local lack of resistance [1]. The difficulty in creating a system with high robustness, which is defined as the insensitivity to local failure, originates from the obscurity of the main parameters simulating the progressive collapse scenarios [2]. It is not only the selection of critical local failures, which are unique for any structural geometry, but also the uncertainty of the loads and load combinations in case of a local failure. In addition, the lack of clear guidelines from the codes, including the Eurocodes, creates a vague environment in which the engineer
must handle the issue on a case-by-case basis. Although there are many design methodologies used to generate structural systems able to resist progressive collapse, most of them are adapted to each specific conditions, without providing a general principle.

The present paper aims at applying a methodology proposed by the authors in a previous paper [3], towards the quantification of robustness, on steel lattice masts. Steel lattice masts belong to the general category of special steel structures used either for telecommunication needs or as systems to transfer energy. As these two industries become strategic and growing in today’s economy, their structural safety and stability is considered vital. One can easily think that the risk of a possible collapse of a lattice mast used to transfer energy, which would result to the shutdown of the power network, can be estimated to be as high as that of the collapse of big and generally considered important structures, like bridges or others.

In the present paper, a common type of steel lattice mast, in the Greek area, has been analyzed and classified following a specific robustness analysis, which focuses on a critical Robustness Safety Factor (RSF), taking into consideration, only the elastic response of the structure. The limit used to calculate this elastic RSF (e-RSF), has been the stresses in the frame elements not to exceed the yielding value of the material. During the procedure of studying this method, the importance of the duration of the local failure has been analyzed and highlighted.

2. On the steel lattice mast modeling and analysis

2.1. Description of the structure

Steel lattice masts are usually flexible and light structures and their common, cost effective practice is using an open lattice, lightweight but adequately stiff system, since a lattice morphology requires only half as much material as a free standing tubular structure, with similar stiffness. Although the majority of these masts include bolted angle profiles, their structural forms vary widely, depending on their geometrical constraints.

The structural behavior of steel lattice masts is typically challenged by loads of wind and earthquake. In most cases, these masts are particularly vulnerable to the effects of wind, since the remote places they are usually constructed at, experience high levels of wind loading (top of hills or mountains, top of buildings etc.). Earthquake on the other hand commonly leads to high loading values in countries like Greece, where local earthquake codes have become very detailed and severe for all kinds of buildings.

Let consider a steel lattice mast with a square-shaped cross-section located on the ground. The structure consists of bolted angles on both columns and the horizontal members. Such a typical structure is presented in Fig. 1 where both the architectural and the structural models are depicted.
2.2. Actions on the structure

The basic loads considered in the design of the tower were the dead loads of all the elements, the imposed live loads, the environmental loads and the earthquake action. Regarding the permanent actions on the steel mast, these include the dead load of the structure (the cross-sections used), the staircase, the antenna with a diameter 0.60 m and the working deck, located 6 m above the ground. As far as the imposed live loading is concerned, the calculation was carried out taking into account the variable loads of the staircase and the working deck.

As steel lattice masts are flexible structures, they are primarily affected by the environmental loading, therefore the effect of wind and ice consist the primary loading of these structures, especially at high altitude [4], [5]. For the purposes of this research, the determination of the wind loads was carried out on the basis of DIN 4131, which provides the methodology for all the relevant calculations [6]. It should be noted here that DIN 4131 - being a complete and extensively tested group of rules - gives very similar results to EC1, while it is generally compatible with the Eurocodes applied later in the analysis. The wind forces on the antennas and the internal forces and moments of the structure were calculated by means of special software, after the assessment of the wind velocity. The determination of the wind actions included the assessment of the wind pressure $q$, the effective area $A$, the dynamic coefficient $\Phi_B$ and the aerodynamic loading coefficient $c_f$ [7].

Thus, the wind loading corresponding to each reference area $A$ is equal to

$$W = c_f \Phi_B q A,$$  \hfill (1)
The aerodynamic coefficient \( c_f \) is related to the direction of the wind and is defined as
\[
\frac{c_f}{c_{fo}} = \psi,
\]
where \( c_{fo} \) is the basic value of the aerodynamic coefficient and \( \psi \) is a reduction coefficient having to do with the height / width ratio of the mast under investigation. According to DIN 4131 and its relevant monographs, the values for \( c_{fo} \) and \( \psi \) are provided as a function of the slenderness \( \lambda \) of the mast and of the \( \Phi \) ratio, which is the coverage ratio and is calculated by dividing the effective area \( A \), namely the net projected area of the panel elements on a vertical plane (including connection plates, gusset plates etc.) by the area of the convex contour of the shadow. This procedure is also dependent on the type of the mast (triangular or quadrilateral tower), and on the wind angle of action.

According to DIN 1055, the influence of ice is taken into account by increasing the cross-section of the members of the mast by a value of thickness varying from 1 cm to 10 cm, depending on the total height of the structure (Fig. 2). Thus, a distributed load along the members is applied, incorporating each member’s thickness and an overall unit weight of 7 kN/m³ [7]. In the case of simultaneous wind and ice loading, the load is reduced to 75% of the basic value and the action areas are increased by \( 2\alpha \), whereas \( \alpha \) is the thickness of the ice (0.06 m).

![Fig. 2. Influence of ice on angle cross-sections and antennas](image)

Regarding the seismic loading, it can be particularly important in structures with high masses at the top. In the herein investigated mast, the seismic analysis of the structure was based on a Spectral Response Analysis, which was performed according to the Codes, taking into account the special regional characteristics of the specific mast [8]. Therefore, the seismic factor is calculated as
\[
\Phi_d(T) = \frac{n \cdot A \cdot g \cdot \gamma_l \cdot \theta \cdot \beta_0}{q},
\]

where \( \gamma_l, \theta \) and \( n \) are parameters relevant with the significance of the structure, the foundation and the damping. The basic load combinations for which the internal forces and moments were calculated are shown in Table 1, where \( G \) denotes the dead load, \( Q \).
denotes the live load, $S$ denotes the snow load, $E$ denotes the earthquake load, $W_0$ denotes the wind load acting with $0^\circ$ angle on the mast and $W_{0S}$ denotes the wind load acting with $0^\circ$ angle on the mast including the ice effect. For the complete design of the members and the assessment of the connections, all the relative provisions of Eurocode 3 were applied, while for simplification reasons, temperature variations and aero-elastic effects were ignored.

Table I

<table>
<thead>
<tr>
<th>A/A</th>
<th>Load Combinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$1.35G+1.5Q$</td>
</tr>
<tr>
<td>2</td>
<td>$1.35G+1.5W_0$</td>
</tr>
<tr>
<td>3</td>
<td>$1.35G+1.5W_0+0.9Q$</td>
</tr>
<tr>
<td>4</td>
<td>$1.35G+1.5W_{0S}+1.5S$</td>
</tr>
<tr>
<td>5</td>
<td>$1.35G+1.5W_{0S}+1.5S+0.9Q$</td>
</tr>
<tr>
<td>6</td>
<td>$1.35G+1.5S+0.9Q$</td>
</tr>
<tr>
<td>7</td>
<td>$G+0.3S+0.3Q\pm E$</td>
</tr>
</tbody>
</table>

2.3. Numerical application

The steel lattice mast under investigation is a square-shaped mast located on the ground, with base dimensions of 0.50x0.50 m and height equal to 10 m. The structure consists of bolted angles, namely L60x6 for the columns, L60x6 for the horizontal members and L40x4 for the main bracings, whereas the material is Fe360. Regarding the connections between the members, these are utilized by means of M16 bolts, quality 6.8.

For the complete design of the members and the assessment of the connections, all the relative provisions of Eurocode 3 were applied, while for simplification reasons, temperature variations and aero-elastic effects were ignored [9].

3. Robustness analysis

3.1. Loading combinations

The Ultimate Limit State loading combinations and the corresponding Serviceability Limit State combinations that robustness validation will be performed with, are presented in Table II.

Another important point that needs to be mentioned here is that the wind action on the mast with an angle of $45^\circ$ (loading case named $W_{45}$), was chosen on the basis of causing compression on the diagonally remaining column of the mast (Fig. 3).
Table II

<table>
<thead>
<tr>
<th>Ultimate Limit State</th>
<th>Serviceability Limit State</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.35G+1.5W₀</td>
<td>G+W₀</td>
</tr>
<tr>
<td>1.35G+1.5W₄₅</td>
<td>G+W₄₅</td>
</tr>
<tr>
<td>1.35G+0.9(1.5W₀S₄₅+1.5S)</td>
<td>G+0.9(W₀S₄₅+S)</td>
</tr>
<tr>
<td>1.35G+0.9(1.5W₄₅S₄₅+1.5S)</td>
<td>G+0.9(W₄₅S₄₅+S)</td>
</tr>
</tbody>
</table>

3.2. Time history analysis

Any progressive collapse event is considered to be triggered by the sudden, local failure of an element of the structure. For the purpose of this research, the progressive collapse incident is defined to start with the sudden total failure of one of the four anchorages of the mast, see Fig. 4 (refer to [10] for this selection). According to [11] a way to simulate this failure is to apply a time history analysis through which the failure of the anchorage is described. The gradual ineffectiveness of the anchorage is expressed by the time history function $f_c(t)$, while the dead and environmental loads are expressed by the time history function $f_d(t)$, Fig. 5.

As it is shown in Fig. 5 the simulation of the two loads imposed on the structure starts from 0 at time $t = 0$ and climbs to 1 after 1 sec. At time $t = 2$ secs the failure of the column anchorage begins, until it completely dies after a critical time period $\Delta t$.

The failure duration, as proven later, is a very important factor affecting the overall response of the mast. So far, there are no fixed tables or information on the duration of a failure, so it is considered to be an unknown parameter. Nevertheless, the duration of the failure mainly depends on the nature of the failure. A blast analysis would probably require a very low $\Delta t$, while a ductile material failure would result to different durations.
For the purposes of this research, since the critical failure time period $\Delta t$, is an unknown, all the analyses include it as a main parameter varying from 0.005 to 0.8 secs.

3.3. Method of analysis

Throughout all the calculations, the method of Hilbert-Hughes-Taylor dynamic analysis was selected, fixing parameter \('a'\) at $-0.3$. As a result of that, Newmark’s parameters gamma and beta are calculated as follows

$$\gamma = \frac{1}{2}(1 - 2\alpha) = 0.8,$$  \hspace{1cm} (3)

$$\beta = \frac{1}{4}(1 - \alpha)^2 = 0.4225.$$  \hspace{1cm} (4)
This way, only the algorithmic damping is applied to the method, in order to eliminate the effect of the highest order eigenmodes, which cannot be taken into consideration during the numerical integration, due to the nature of the method [12].

In order to approach the response of the structure with an adequate accuracy, the output time step size was chosen to be $\Delta t/100$, for each different column anchorage death durations $\Delta t$. The number of output time steps was chosen such that the output for the response of the structure covers at least $7.5\Delta t$.

It was decided not to introduce numerical (physical) damping because of the fact that the scope of this research is to check and use the instantaneous maximum values of forces and displacements, which always appear in the beginning of the event, so the effect of damping was going to be very small.

Table III shows all the input of the dynamic analysis needed for the completion of the model. $\Delta t$ is the column anchorage death duration, and total time is the total time for which the phenomenon will be monitored. The software used for the analyses was SAP 2000 (v.11.0.8).

<table>
<thead>
<tr>
<th>$\Delta t$</th>
<th>Step size</th>
<th>Steps</th>
<th>Total time</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.800</td>
<td>0.008</td>
<td>750</td>
<td>6</td>
</tr>
<tr>
<td>0.700</td>
<td>0.007</td>
<td>785</td>
<td>5.5</td>
</tr>
<tr>
<td>0.600</td>
<td>0.006</td>
<td>833</td>
<td>5</td>
</tr>
<tr>
<td>0.500</td>
<td>0.005</td>
<td>900</td>
<td>4.5</td>
</tr>
<tr>
<td>0.400</td>
<td>0.004</td>
<td>1000</td>
<td>4</td>
</tr>
<tr>
<td>0.300</td>
<td>0.003</td>
<td>1166</td>
<td>3.5</td>
</tr>
<tr>
<td>0.250</td>
<td>0.0025</td>
<td>1300</td>
<td>3.25</td>
</tr>
<tr>
<td>0.200</td>
<td>0.002</td>
<td>1500</td>
<td>3</td>
</tr>
<tr>
<td>0.100</td>
<td>0.001</td>
<td>3000</td>
<td>3</td>
</tr>
<tr>
<td>0.050</td>
<td>0.0005</td>
<td>6000</td>
<td>3</td>
</tr>
<tr>
<td>0.010</td>
<td>0.0001</td>
<td>30000</td>
<td>3</td>
</tr>
<tr>
<td>0.005</td>
<td>0.00005</td>
<td>60000</td>
<td>3</td>
</tr>
</tbody>
</table>

4. Robustness safety factor

4.1. Definition of robustness safety factor (e-RSF)

According to the Eurocodes [13], [14], the loading used for accidental loading cases such as the sudden failure of a column anchorage, is $G+0.3Q$ or $G+W$ in case of wind, and so this will be the reference loadings for the calculation of e-RSF. The other critical loading that participates in the calculation of e-RSF is the one that makes the structure

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reach its elastic limit. For the purpose of the safety factor, the elastic limit of the structure is the loading that makes at least one element of the structure reach a stress, equal to the yielding stress of the material.

Given all the above, the elastic RSF is defined as the ratio of the critical elastic loading over the accidental loading

\[ e-RSF = \frac{\text{Critical Loading}}{\text{Corresponding Serviceability Loading Combination}}. \] (5)

The actual meaning of the e-RSF can be shown very easily with an example of a structure that achieves an e-RSF equal to 1. This means that in case of the local failure that the e-RSF refers to, the structure has the capability to elastically withstand the accidental loading according to Eurocode.

In extent, the structure is insensitive to local failure and robust enough to resist progressive collapse. Furthermore, since the lattice mast under consideration includes axial members only, measuring the axial forces of the members actually measures the demand to capacity ratios of the members and so the structure is adequately designed against progressive collapse.

4.2. Output assessment

The output extracted from the model regards all the elements’ forces. From the forces, the corresponding stresses are calculated and the element with the highest stress, tunes the evaluation of the e-RSF.

For the lattice mast under consideration, all the elements act as axial members (moment releases at both ends) and the stresses calculated and compared, incorporate buckling effects.

Due to the dynamic phenomenon, all the forces follow a time dependent function. In Fig. 6, the force of the critical column is presented as a function of time. The diagram can be divided in three time periods. The first one (from 0 to 1 sec), which includes the loading of the structure and depicts an increasing axial force value. The second time period (from 1 to 2 seconds) described as the equilibrium between imposed loadings and axial forces and the final third one (from 2 seconds to the end), which starts with the failure of the anchorage and initiates the phenomenon.

The value measured for the purposes of the e-RSF, is the instantaneous maximum value reached in the first oscillation of the structure (in Fig. 6, this value is tuned to the elastic limit of the column equal to 134.2 kN). As the phenomenon progresses damping forces the values of the forces to decrease their variations during each oscillation and reach a final value.

The critical column is the column that acts in the compression as a result of the collapse of the anchorage. Fig. 7 and Fig. 8, show the compression of this column.

One interesting conclusion on the behavior of the mast can be drawn in Fig. 7 and Fig. 8. Fig. 7 showing the axial forces of loading case G+W_45, indicates that there is one column under compression while the other two are acting almost equally under tension. However, Fig. 8 showing the axial forces of loading case G+W_0, indicates that there is
one column under compression and only one under tension. The mast acts like a 2d structure for this specific loading case.

![Fig. 6. Force of critical column as a function of time](image)

Fig. 6. Force of critical column as a function of time

![Fig. 7. Frame forces (G+W45)](image)

Fig. 7. Frame forces (G+W45)

![Fig. 8. Frame forces (G+W0)](image)

Fig. 8. Frame forces (G+W0)

4.3. Results

The results obtained by all the analyses are shown in Fig. 9. Evidently, the mast is designed not to enter the unsafe robustness zone and the values of the e-RSF are tuned in such a way that are above 1.

Another important finding drawn from Fig. 9 is the effect of the failure duration period. As a major conclusion, it is very important to justify and correctly simulate the
duration period of the local failure leading to progressive collapse. It is much different to simulate a blast than a crash of a vehicle or a fire. Although all of them lead to the eventual ineffectiveness of the anchorage, they have very different local failure durations, which greatly affect the response of the structure.

![Graph of e-RSF vs. ∆t](image)

Additionally, although the shortest failure time seems to be governing, the authors consider that further work needs to be done to generalize this assumption. It is indeed logical, however, that short time period failures lead to sudden and more violent structural responses.

5. Conclusive remarks

This paper investigates the introduction of a new method quantifying the complicated phenomenon of progressive collapse. The definition of an elastic robustness safety factor is a step towards this quantification. The e-RSF is the first comparable safety factor which can categorize and validate the performance of all structures against progressive collapse, regardless their geometry.

Several factors affect the calculation of e-RSF. The impact of the local failure event duration is proven to be very significant and the accuracy, with which it is simulated, plays a great role to the dynamic response of the structure. The lattice mast considered in this analysis, demonstrates a very satisfactory progressive collapse performance. The failure scenarios analyzed, are not proven enough to initiate its progressive collapse.

A work on analyzing more than 300 different geometries of lattice masts has been already completed and extracting conclusions and applying this method to other transmission towers of similar morphology or possibly with robustness safety factors of less than 1 has been planned. It is noticeable that in cases of lattice towers with cables
present in the structural system, the response is significantly affected specially under specific types of loading (combination of wind and ice).

References