SUPERPOSITION OF STATIC AND DYNAMIC WIND ACTIONS ON STEEL TOWERS BY FEM ANALYSIS

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UDK: 624.042.41
DOI: 10.14415/konferencijaGFS2021.17

Summary: *Wind actions can be dissolved into two components: steady or mean action, and gust or variable action. The first component produces static load on structure, but the second produces dynamic load. The dynamic component depends on meteorological data for gusts in certain region, and on internal characteristics of the structure. Dominant design approach is to treat wind action generally as a static load. This may be unjustified for high, slender, and flexible structures like steel towers. The paper is analysing the response of a specific steel tower structure on wind actions using static approach first, and then superimposing static and dynamic load. FEM was used as a method of analysis. Results and recommendations for further treatment of similar structures are given as a conclusion.*

Keywords: *steel towers, static and dynamic analysis, FEM*

1. INTRODUCTION

Steel towers are regularly exposed to severe wind actions, which can be dissolved into two components: steady or mean action, and gust or variable action. The Eurocode standard for towers and masts [1] prescribes analysis of these structures in case of gust wind actions and the vibrations that arise due to it. However, the referent standard does not provide procedures for such analysis, but only the equivalent gust wind load as a static action:

$$F_{T,W}(z) = F_{m,W}(z)\, [1 + (1 + 0.2\, (z_m/h)^2)\, (1 + 7\, I_v(z_e))\, c_s\, c_d - 1] / c_0(z_m)] \qquad (1)$$

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Common practice in tower design is to limit the analysis domain to static calculations, since dynamic analyses are often complex and tedious. The aim of this research is to present a comparative analysis of a concrete tower structure under wind action, using different analysis approach, static and dynamic, linear and non-linear, using FEM and advanced engineering software. The results of the research should serve as a guidance for design of the structures of this class. Special attention was addressed to gust wind load and its dynamic nature.

2. TOWER STRUCTURE MODELLING

The selected structure was taken from [2], and its purpose was to serve as a watchtower for fires in mountainous and forest regions. The tower has height of 20 m, and it is equipped with a platform on top. The structure is modular, easy dismantling, with modules that can be all packed into one transport piece (Fig. 1).

a) b)

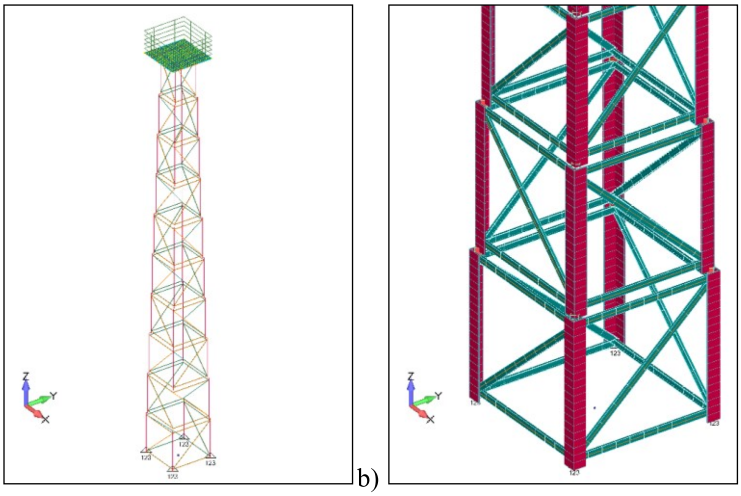


Figure 1. Tower structure; a) disposition; b) detail

Wind mean load (WM) for the analysed tower structure was calculated according to [3], and wind gust load (WG) according to [1], all acting in X-direction (Fig. 2).

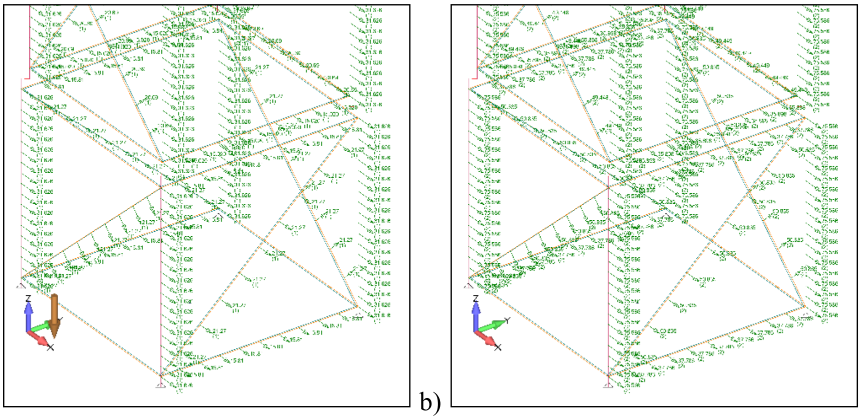
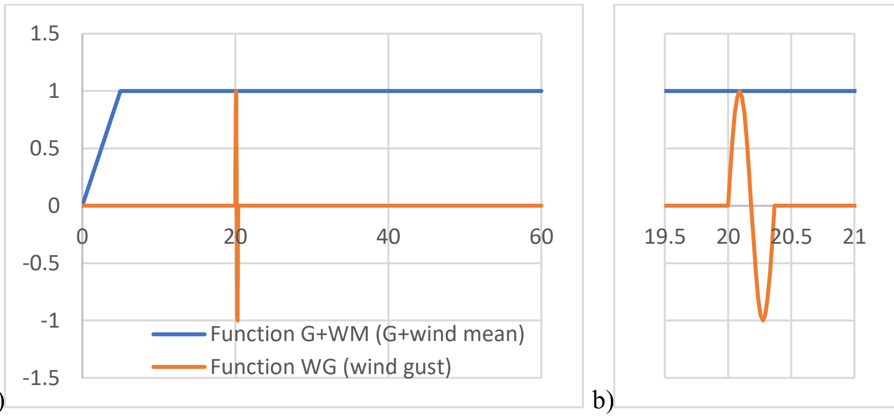

a) b)

Figure 2. Wind load detail; a) wind mean; b) wind gust

In order to analyse the dynamic action of the wind gust combined with the static wind mean action, specific load functions were created (Fig. 3). The function that represents the static wind action and the tower self weight was bilinear time-dependent function. Here full wind action rises from t_0=0 to t_1=5 s, and remains constant. From t_1=5 s to t_2=20 s the structure should relieve from possible oscillations. The gust wind action was adopted as sine function. From t_0=0 s to t_2=20 s it has a zero value; from t_2=20 s the sine function begins, and it lasts for one oscillation period (T_c). After one oscillation period, the function takes zero value up to t_3=60 s, in order to analyse damping of the structure vibration.
The methodology developed in this research enables that wind gust load, as a transient phenomenon, can act on a structure that is already deformed under wind mean action. That way, the design engineer may perceive the cumulative acting of the two wind components and to detect potentially critical situations for the tower structure.



a) b)

Figure 3. Load functions; a) full view; b) wind gust function detail

The period (T_c) depends on the excitation frequency of the wind gust, and in this research it was varied below and above the natural frequency of the structure, in order to investigate the possible resonant behaviour.

The overall structural damping factor (G) was included into all dynamic analyses. Its value was obtained according to the expression:

$$G = 2\xi = 2 * 0.05 = 0.10 \quad (2)$$

where:

$\xi =$ 0.05 relative damping value recommended for steel structures.
The system damping frequency *W3* [5] was taken as the value of the frequency of the 1st mode of oscillation of the structure, W3 = ν1 = 3.620 Hz.

3. ANALYSIS RESULTS AND DISCUSSION

The selected tower structure was subdued to a series of FEM analyses (Table T1). The displacement values of the tower top in wind direction and extreme stresses in members were chosen as output values. Here it must be remarked that all safety factors were omitted in order to obtain a more general approach to the problem. Significant values are given in bold letters.

Table T1. Review of the models, analyses, and results

No.	Model	Load combination	WM character	WG character	Analysis domain	Max. UX [mm]	Max. stress [MPa]	Min. stress [MPa]	Various
1	M11	G+WM	Static	Static	Linear	27	83	-80	
2	M11	G+WM	Static	Static	Non-linear	27	89	-80	
3	**M11**	**G+WM+WG**	**Static**	**Static**	**Linear**	**90**	**280**	**-270**	
4	**M11**	**G+WM+WG**	**Static**	**Static**	**Non-linear**	**105**	**1400**	**-160**	
5	M11	G+WM+WG	Static	Static	Linear Buckling	-	-	-	**P_{cr}=0.49**
6	M11	-	-	-	Eigenvalue(1)	-	-	-	**ν1=3.62**
7	MF150	G+WM+WG	Static	Dynamic	Linear	35	98	-101	f12=5.430
8	MF115	G+WM+WG	Static	Dynamic	Linear	54	132	-147	f11=4.163
9	MF110	G+WM+WG	Static	Dynamic	Linear	58	143	-158	f10=3.801
10	MF105	G+WM+WG	Static	Dynamic	Linear	61	150	-164	f9=3.801
11	**MF100**	**G+WM+WG**	**Static**	**Dynamic**	**Linear**	**65**	**154**	**-174**	**f1=3.620**
12	MF095	G+WM+WG	Static	Dynamic	Linear	67	164	-178	f2=3.440
13	MF090	G+WM+WG	Static	Dynamic	Linear	69	168	-182	f3=3.258
14	MF085	G+WM+WG	Static	Dynamic	Linear	78	190	-205	f4=3.077
15	MF080	G+WM+WG	Static	Dynamic	Linear	84	206	-221	f5=2.896
16	**MF075**	**G+WM+WG**	**Static**	**Dynamic**	**Linear**	**101**	**252**	**-267**	**f6=2.715**
17	MF070	G+WM+WG	Static	Dynamic	Linear	87	213	-228	f7=2.534
18	MF065	G+WM+WG	Static	Dynamic	Linear	77	189	-204	f8=2.353
19	**MFN075**	**G+WM+WG**	**Static**	**Dynamic**	**Non-linear**	**129**	**629**	**-508**	**f6=2.715**

Legend: G = self weight; WM = wind mean action; WG = wind gust action;
ν1 = natural frequency of the structure; f1...f12 = load frequency

The first group of analyses implies static character for both the wind mean action (WM) and the wind gust action (WG) (No. 1-5), and this is the simplest and the most common approach to tower analysis. In case of WM action only (No. 1-2), the displacements and stresses were quite low, and using of non-linear analysis with large displacements did not show any changes.
Including the WG load significantly increased the total load and the influences. Further, application of non-linear analysis notably increased the displacements and stresses, hinting the stability problems. This was confirmed by linear buckling analysis that revealed the critical load factor value: $P_{cr} = 0.49$ (Fig. 4).

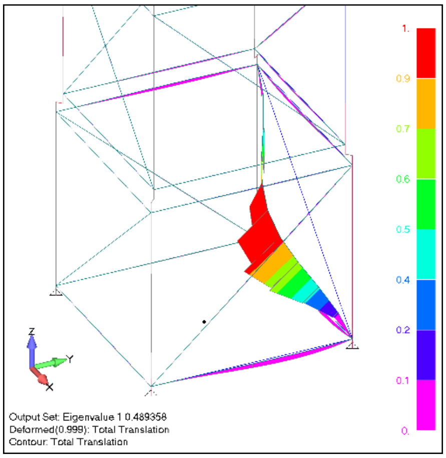


Figure 4. Model M11; Load G+WM+WG – Linear buckling analysis; a) total displacements [relative]

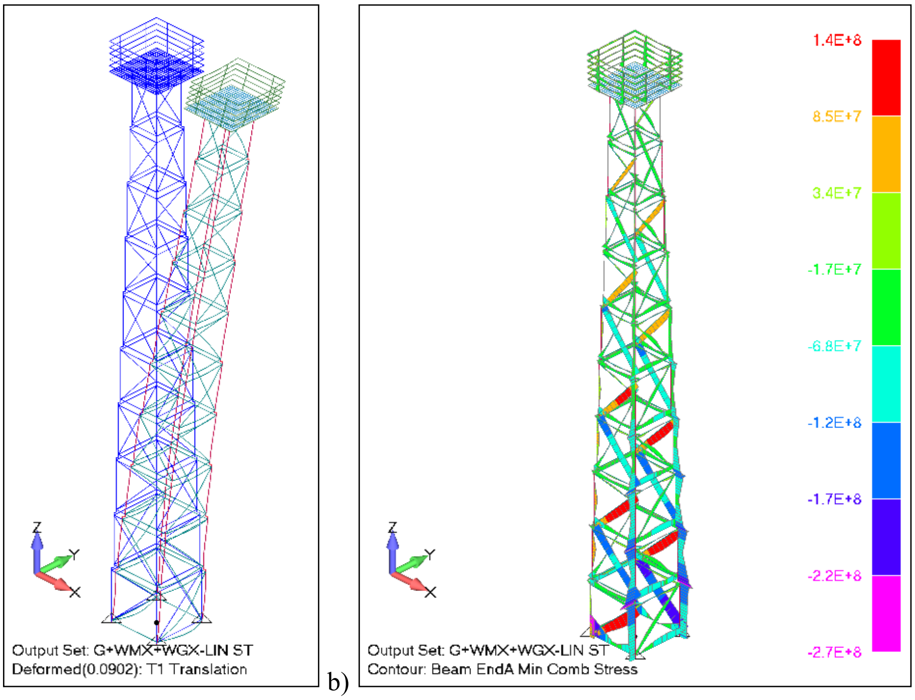
It can be seen that even static approach can give disperse results, and that application of linear analysis was not on the safe side. In addition, involving of gust wind action significantly changed the behaviour of the tower structure.
Since the true nature of the gust wind action is dynamic, the following group of analyses (No. 6-19) were conducted treating the WM load as static load, and superimposing the WG load on it as a dynamic one, using the functions described in Section 2. In order to formulate the excitation derived from the WG load, an eigenvalue analysis has been done first (No. 6). The load frequency equal to the natural frequency of the structure should be the resonant load, but the research encompassed a series of different excitation loads with frequencies ranging from 65-150%, of the resonant load in order to check thoroughly the structural behaviour. Note that the models for dynamic analyses are labelled as *MF* following with a number, which denotes the percent of the excitation load related to the natural frequency of the tower. Thus, the label *MF075* means that the excitation load frequency was 75 % of the *v1*.
From the conducted analyses one may see that for the nominally "resonant" load the displacement of the tower top was 65 mm, and the extreme stresses were σ = +154/-174 MPa. However, by decreasing the excitation load frequency to 95, 90, 85 %, etc., the displacements and stresses were rising until the value of 75 %, when they have reached the maximal values (displacements of 88 mm and stresses σ = +214/-229 MPa). After

further decreasing of the excitation load frequency, the output values started to drop. On the other hand, increasing the excitation load frequency to 105, 110...150 %, showed decreasing of the output values.

One may conclude that in this case 75 % of the resonant load frequency was a critical value. This may not stand for other structures of this type. Namely, in the research [4], which investigated a tower 110 m high, such analysis revealed that the critical load frequency was 90 % of the “resonant” load, but again somewhat below it. These results point out that a dynamic analysis of a tower should involve a wider spectrum of load frequencies, in order to obtain a safe structure. The methodology presented in this paper can easily enable such check.

Finally, for the critical load frequency a non-linear dynamic analysis with large displacements was performed. Similarly to the static approach, the non-linear domain of analysis here also gave significantly higher values. The displacements were 129 mm (increase of 28 %), and the stresses were σ = +629/-508 MPa (increase of +250/-190 %).

In Fig. 5-10 are presented characteristic analysis models and their output values. For dynamic analyses are also given diagrams that illustrate structural displacements vs. loading time (Fig. 7 and 9). Here it must be noted that the time axis divided into time steps Δt = 0.1 s.



a) b)

Figure 5. Model M11; Load G+WM+WG – Linear static;
a) UX displacements [m]; b) min. streses [Pa]

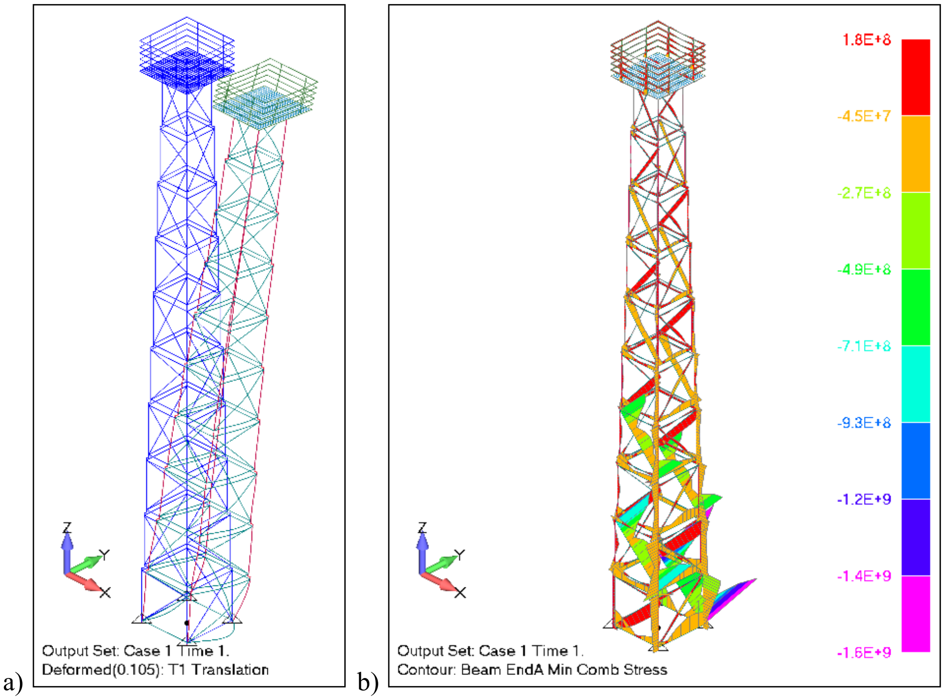


Figure 6. Model M11; Load G+WM+WG – Non-linear static;
a) UX displacements [m]; b) min. stresses [Pa]

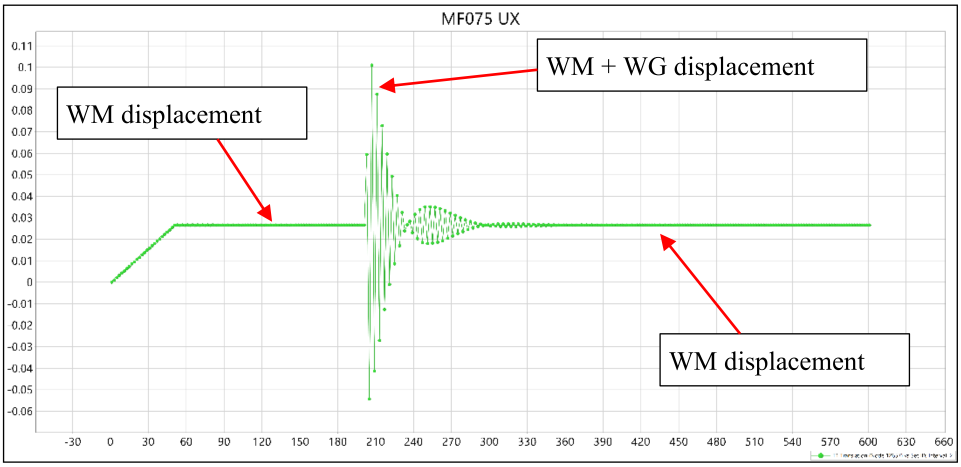


Figure 7. Model MF075; Load G+WM+WG – Linear dynamic;
UX displacements [m] vs. time

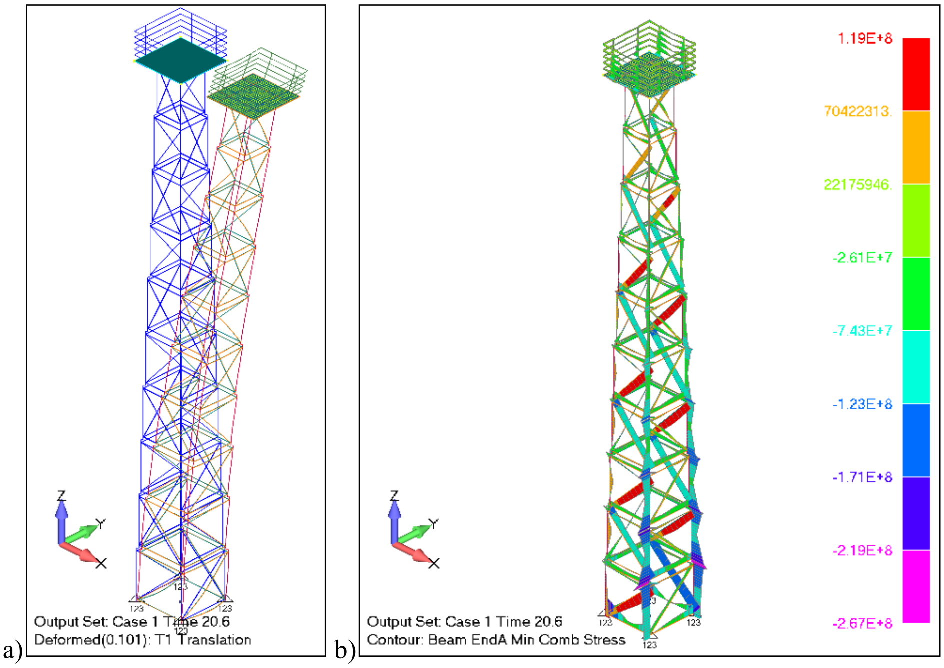


Figure 8. Model MF075; Load G+WM+WG – Linear dynamic;
a) UX displacements [m]; b) min. stresses [Pa]

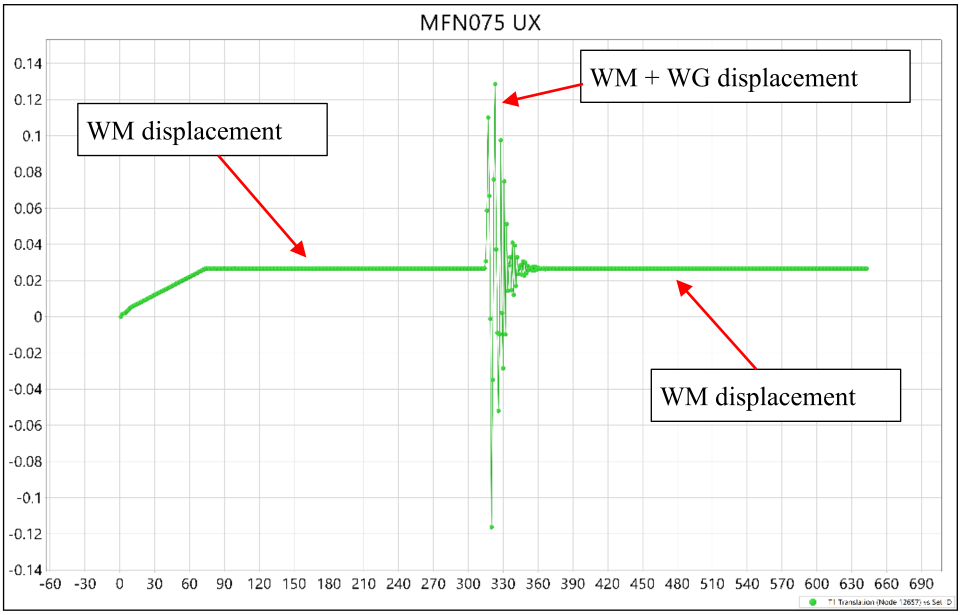


Figure 9. Model MF075; Load G+WM+WG – Non-linear dynamic;
UX displacements [m] vs. time

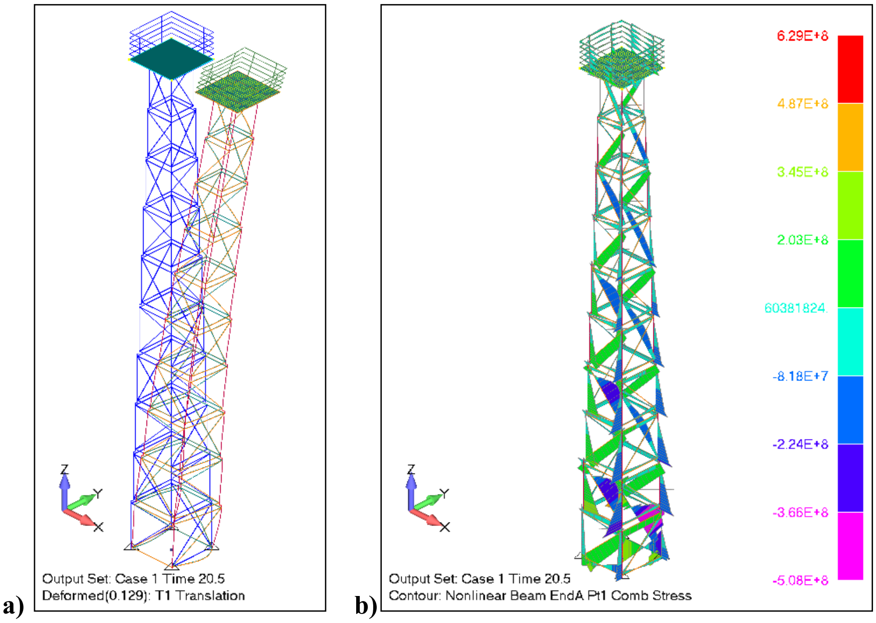


Figure 10. Model MFN075; Load G+WM+WG – Non-linear dynamic; a) UX displacements [m]; b) min. stresses [Pa]

4. CONCLUSIONS

Tall and slender structures are very sensitive to horizontal loads like wind and earthquake. Wind loads mostly act as a steady load during relatively long time periods, ranging from several hours to several days, sometimes weeks. However, during those periods, extreme wind loads may occur during short intervals lasting only for seconds, and causing vibrations. These extreme loads, known as gusts, are treated in the relevant Eurocode standards in simplified way, as static loads.

Advanced engineering analysis methods based on FEM give possibility to analyse wind gusts maintaining their dynamic character. This possibility is shown on a concrete example, by developing a numerical model in which wind steady load and gust load can act simultaneously.

Varying of the load frequency showed that every particular structure needs special care in order to reveal the critical load case regarding vibrations. Thereat, the design process must lay on reliable meteorological data that encompass occurrence of gusts and their characteristics. This may prevent collapse hazards, which are present at tower structures.

Another aspect of the advanced engineering software is the capability of including the large displacements into analysis, enabling the Second Order Theory effects, by applying the non-linear analysis, which can largely replace the stability check procedures. In this research, it was proved that non-linear analyses showed greater displacement and stress values, as in static, as well as in dynamic approach to the problem, when the gust load is present.

The available analysis methodology and the numerical model developed in this research suggest implementing of new strategies for steel tower design into current codes.
Further investigations should broaden this research by examining other parameters that affect tower structures under gust loads, like lasting of the load, damping factors, structure height, etc.

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СУПЕРПОЗИЦИЈА СТАТИЧКОГ И ДИНАМИЧКОГ ДЕЈСТВА ВЕТРА НА ЧЕЛИЧНИ ТОРАЊ ПУТЕМ МКЕ

Резиме: *Дејство ветра се може разложити на две компоненте: равномерно или осредњено дејство и налет, или промењиво дејство. Прва компонента изазива статичко деловање на конструкцију, а друга динамичко. Динамичка компонента зависи од метеоролошких података за налете ветра у одређеном подручју и од унутрашњих карактеристика конструкције. Преовлађујући пројектантски приступ је генерални третман ветра као статичког оптерећења. Ово може бити неоправдано за високе, витке и флексибилне конструкције попут челичних торњева. У раду се анализира одговор конкретне конструкције челичног торња на дејство ветра најпре пименом статичког приступа, а затим суперпозицијом статичког и динамичког оптерећења. За анализу је коришћена МКЕ. Резултати и препоруке за будући третман сличних конструкција су дати као закључак рада.*

Кључне речи: *челични торњеви, статичка и динамичка анализа, МКЕ*