RESPONSE ANALYSIS OF RC COOLING TOWER UNDER SEISMIC AND WINDSTORM EFFECTS

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ABSTRACT

The paper deals with the comparison of RC structure of cooling tower unit under seismic and stark wind loads. The calculated values of the envelopes of displacements and internal forces due to seismic loading states were compared with the envelope of the loading states due to the dead, operational and live loads, wind and temperature actions. The seismicity effect respects the seismic area of ground motion 0.3 g and takes into account ductility properties of the relatively rigid structure. The implementation of ductility is given by reduction of seismicity load. In this case the actions of wind pressure are higher than the seismicity effect under ductility correction. It was stated that seismicity effects, taking into account ductility properties of the structure, are lower than the actions of wind pressure. Other static loads, especially temperature action due to environment and surface insolation are very important for the design of structure.

KEYWORDS

Cooling tower, Wind load, Earthquake, Ductility, Dynamic response.

INTRODUCTION

The objective of this paper is execution of the basic static and dynamic calculation of twin cooling towers with the fans of propeller diameter 6 m for Oil Refinery. The basic static and dynamic analyses are based on the requirements of American standards for designing [1]. The structure sizing was executed for limiting combination of static loading states including the actions of wind pressure. The designed quantities due to the action of seismic load in introduction of the structure ductility are lower than the actions of wind pressure. The design loads are compared and the dominance of particular loading states is assessed according to response internal forces caused by these loads in the structure. From this comparison it is explicitly evident that the temperature effects exert in the resultant design load combination the biggest influence upon the structure of the towers. A share of these temperature effects in total stress of the structure can be estimated as approx. 50%.

DESCRIPTION OF THE STRUCTURE

The subject of analyses is twin reinforced-concrete cooling towers located next to each other on a common base plate. Each cooling tower has square plan. The cooling tower is from above terminated with a floor (ceiling) plate with a circular opening over which there is the diffuser of a cylindrical shape. The cooling towers are arranged in series in such a way that the towers have a common internal wall dividing them all along their height. On the edges of the twin towers in longitudinal direction there are transversal external walls reinforced by a triple of vertical reinforcing ribs of longitudinal orientation. These longitudinal stiffeners of thickness 400 mm exceed external surface of transversal external wall by 1 500 mm. The transversal external and internal walls of thickness 250 mm exceed external surface of longitudinal walls by 800 mm. In addition, internal stiffeners in the central section of transversal external walls reinforce each tower. In the centre of the towers there are columns on whose tops at the level of ceilings there are fans. The columns are strutted in two of their altitudinal levels to external walls by means of concrete girders and at the fan level by means of steel pipes into ceiling plates. The longitudinal external walls of the towers, in which there are suction inlets, are reinforced by vertical ribs and horizontal beams flanging the suction inlets. The basic altitudinal level of structure model ±0.0 corresponds with the centre line of the base plate. A spatial computational model of twin towers was created for calculation of the cooling towers. Its space and basic dimensions are stated in the Fig.1.

Twin cooling towers is designed as made of reinforced concrete where the used concrete is Grade 25 (equivalent to class of concrete B30) and the used reinforcement in concrete Grade 400 (equivalent to reinforcement 10 425 V). The stiffeners and supporting structure for technology inside the tower is designed as made of steel ASTM A36 or a similar one (equivalent to steel of series 37). The external walls, internal wall, floor plates, diffuser and vertical stiffeners were modelled by plate elements of the thickness corresponding with a modelled part of the structure. Internal columns, internal girders, columns/ribs in external walls, horizontal reinforcing of longitudinal walls over and under the shutter opening and horizontal stiffeners of the fan were modelled by beam elements. The used damping for dynamic loading states was 5% of critical damping.

In the computational model, the baseplate was supported by Winkler-Pasternak sub-base model. The sub-base parameters are automatically determined by the computational program according to a set sub-base structure in test pits. The starting values of Winkler-Pasternak constants were selected as follows: C_{1z} = 3.5 MN/m^3, $C_{1x} = C_{1y}$ = 2.0 MN/m^3, $C_{2x} = C_{2y}$ = 50 MN/m.

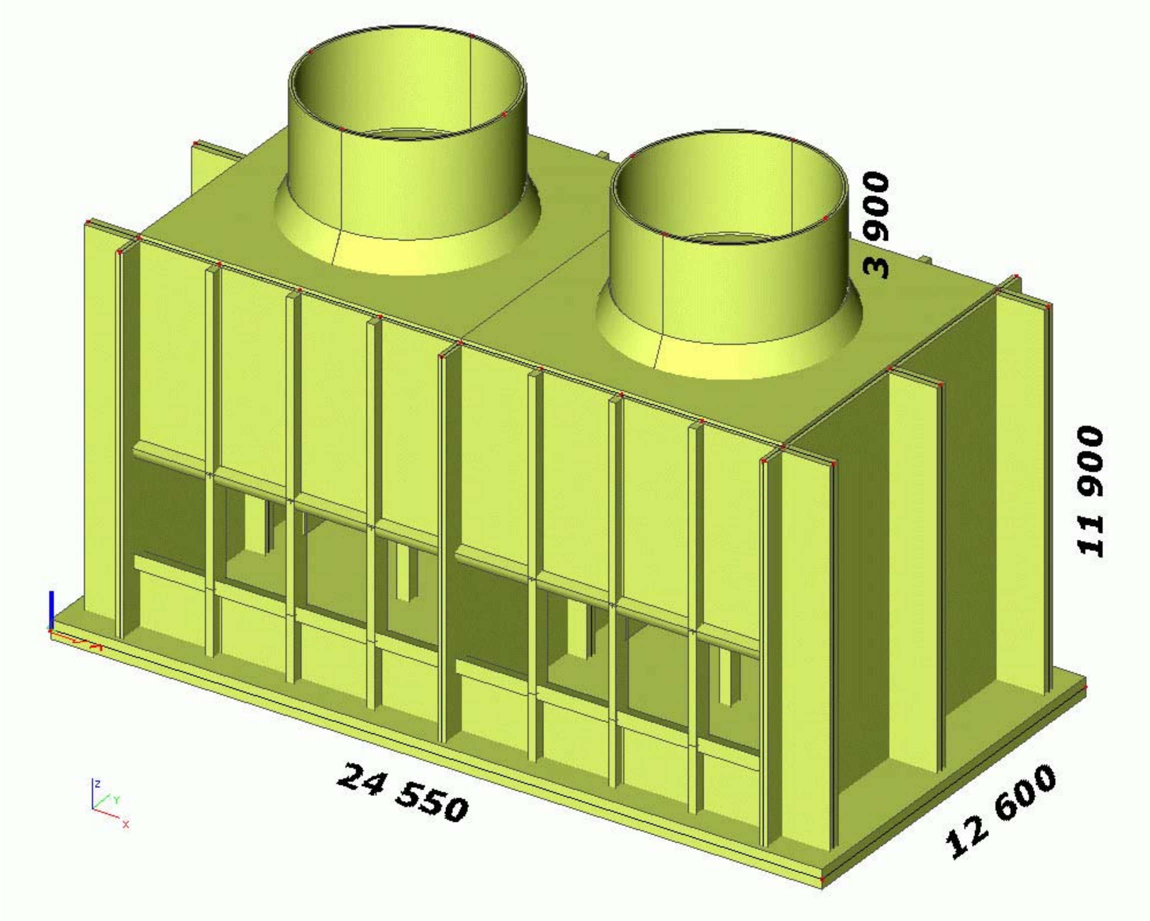

Figure 1 The basic dimensions of twin cooling tower unit

The calculation of improved values for combination of dead and operational loads are stated in Fig.2. The baseplate can slightly turn on the sub-base around horizontal axes and shift in a vertical direction. Horizontal shifting of the structure as a whole, i.e. on one longitudinal and one transversal edge of the baseplate, is prevented. Horizontal shifting on opposite edges is enabled in order not to hinder thermal expansion of the plate.

LOAD

The structure model was loaded with static and dynamic loads. The static loads include dead weight and permanent load due to process equipment, live load, operation load, temperature effects and actions of wind pressure. The dynamic loads include the effects due to the fans of the cooling towers and seismic effects.

Temperature load due to environment

Operating temperature of the towers is expected to comply with the temperature in the period of construction. External walls, ceiling plates and diffuser are loaded with non-uniform change in surface temperature due to a decrease or increase in ambient air temperature. The columns/ribs in longitudinal external walls and external stiffeners of the walls were loaded with temperature corresponding with centre line of the wall. Temperature of internal columns and girders at open-air temperature fluctuation in process operation is unchanged; therefore these members are left without temperature load. Internal stiffeners and diaphragm beam are subject to temperature load only at their edges where they contact external walls and ceiling. In these parts, contact sections of the width 725 mm are modelled that were loaded with the temperature corresponding with centre line of the wall or ceiling.

Environment temperature was adopted from national documents; determination of temperatures of the structure surface complies with the theory of heat propagation through solid medium. Temperature loads were only considered on a part of the structure above the formation level:
– Estimated mean temperature of the structure during construction … t_0 = +35.0 °C,
– Normal air operating temperature inside the structure … t_i = +35.0 °C,
– Minimum ambient air temperature in winter period … t_e = -5.0 °C,
– Maximum ambient air temperature in summer period … t_e = +55.0 °C.

These group of temperature loads includes: non-uniform decrease in temperature of surfaces in winter period, non-uniform increase in temperature of surfaces in summer period, surface insolation in perpendicular direction (for ceilings only) and inclined surface insolation under incidence of sunbeams at angle 45°

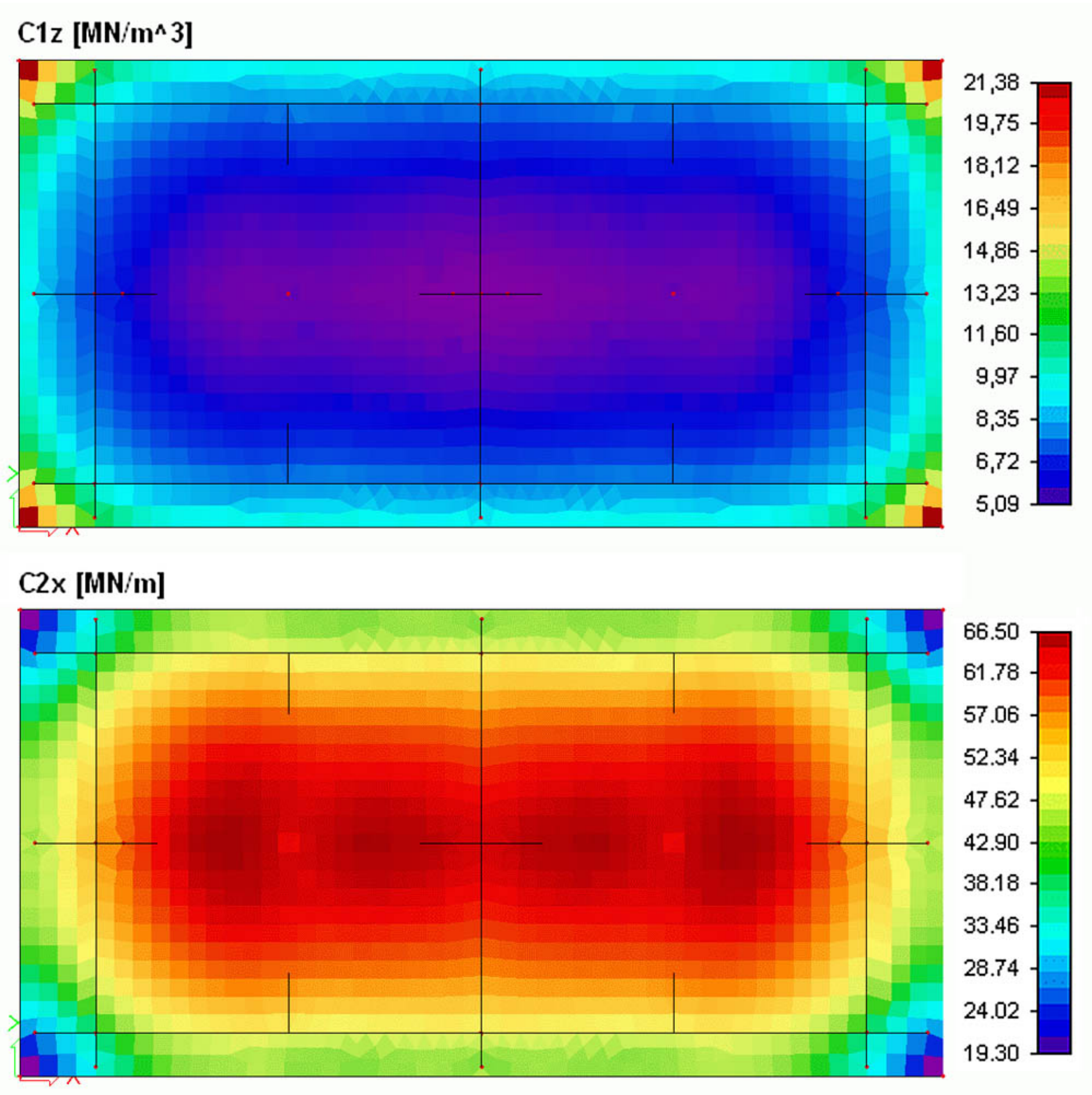

Figure 2 Winkler-Pasternak factors of elastic foundation

Wind load
The basic wind pressure values were adopted from [1] and the methodology of load determination including the elevation effect was adopted from [10]:
– Basic Wind Velocity ... $V = 50$ m/s,
– Topography Factor ... $S1 = 1.0$,
– Wind Pressure Variation with Height ... $S2 = 0.99$ (for $h < 15$ m),
– Statistical Factor ... $S3 = 1.0$,
– Design Wind Velocity ... $V_s = 49.5$ m/s,
– Dynamic Wind Pressure ... $q = 0.613 \times 49.5^2 = 1.502$ kPa.

Earthquake load
Seismic load parameters were determined according to [1] and [9]. Vertical acceleration component in comparison with horizontal component is according to [9] par 1631.2.5 determined by coefficient 0.667. Design elastic response spectrum is according to [9] par 1631.4.1:
– Coefficient of significance … $I = 1.25$,
– Seismic area … 1,
– Coefficient of seismic area ... $Z = 0.075$,
– Earth medium profile ... S_D and corresponding,
– Seismic waves propagation velocity (according to [9]) … $v_s = 360$ m/s,
– Seismic coefficient ... $C_v = 0.18$, $C_a = 0.12$,
– Damping ratio (according to [9] par 1631.2.2) ... $D = 5\%$,
– Maximum acceleration at the level of foundation plate … $2.5 \times C_v = 0.3$ g.

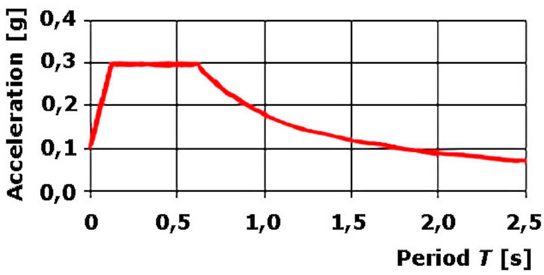

Figure 3 Design elastic response spectrum

Table 1 Natural frequencies of vibration [Hz]

(i)	$f_{(i)}$	Dominant vibrating part
1	2,19	Rotating vibration of both towers around axis x
2	2,57	Rotating vibration of both towers around axis y
3	3,28	Sliding vibration of towers on the subbase in the direction of axis z
4	3,31	Torsional vibration of walls around axis z
5	4,95	Higher bending mode of rotating vibration around axis x
6	5,25	Higher bending mode of rotating vibration around axis y
7	11,81	Bending vibration of longitudinal walls
8	12,22	Higher mode of bending vibration of longitudinal walls
9	13,48	Bending vibration of transversal walls
10	14,19	Bending vibration of lower cross and longitudinal walls
11	18,62	Higher mode of bending vibration of lower cross and tranversal walls
12	20,85	Torsional vibration of beam crosses

NATURAL VIBRATION

For the analyses of the dynamic structure response to seismic load it is necessary to know tuning of the structural system. For these purposes, calculation of natural vibration was executed – calculation of the lowest 100 natural modes of vibration and equivalent natural frequencies in frequency interval from approx. 0 Hz to 50 Hz. This separation is a sufficient one, as seismic excitation has substantial components approx. up to 32 Hz.

The lowest natural frequency and their modes approx. up to 20 Hz are important to determination of seismic response of the structure; the influence of higher modes in determination of total seismic response is due to a variable nature of these higher modes already very small (above 20 Hz approx. in the units up to tenths of per cent towards total response); approx. from frequencies above 30 Hz their influence to total seismic response achieves even lower values. The describing of the lowest 12 natural modes of vibration are included in Tab.1.

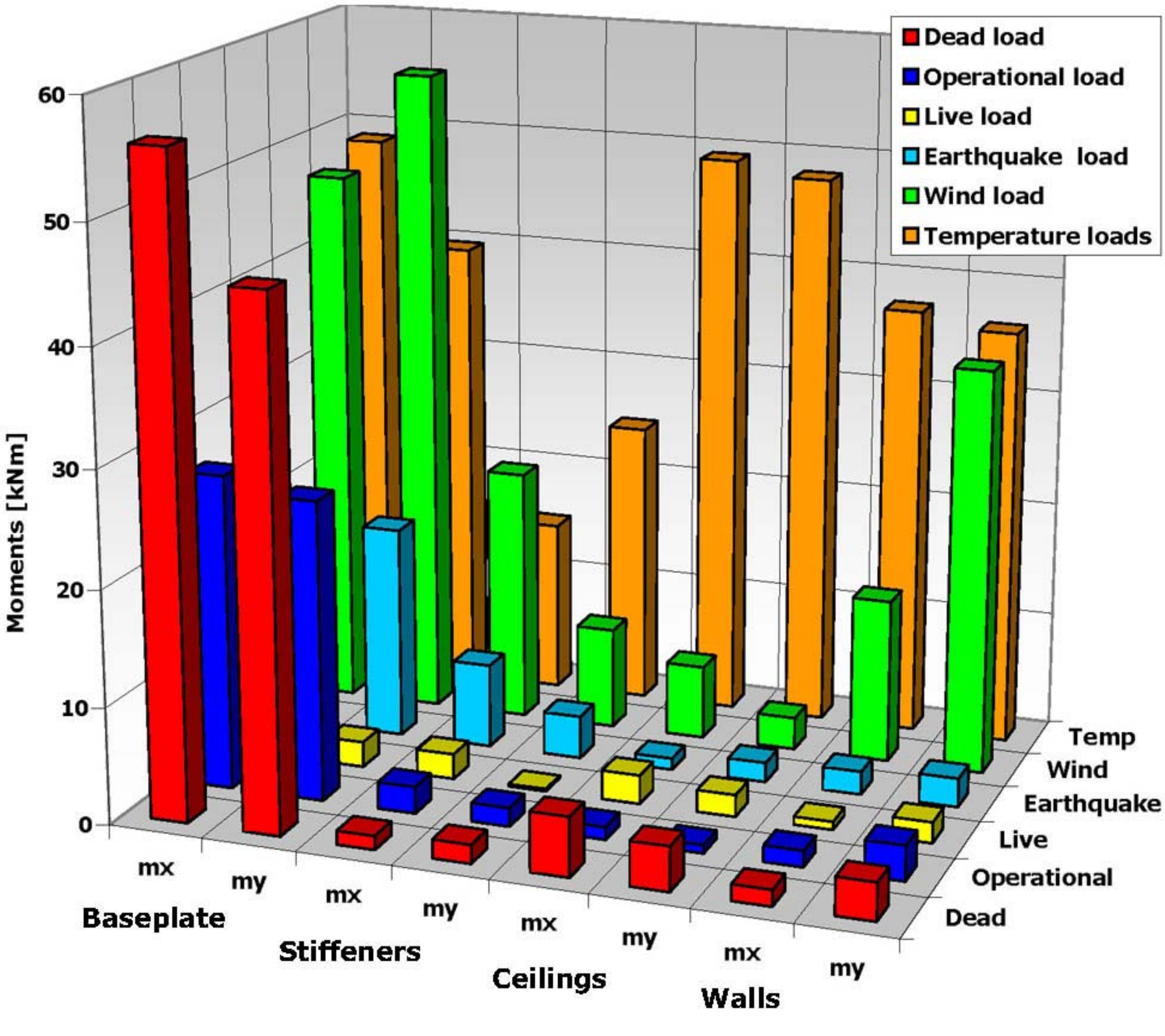

Figure 4 Comparison of design moments in plate elements for individual groups of loads

STRUCTURE RESPONSE

Seismic response were analysed by means of seismic load decomposition into natural modes of vibration (dynamic modal analysis according to [2]). Seismic analyses was executed for both horizontal directions *x* and *y* of load action taking into account vertical composite action of seismic excitation (ground motion in direction *z*) according to [2] par 1631.2.5. The envelope of the response values was formed based on both of these dynamic analyses states within seismic combination. The analyses of the structure response to seismic load determined the envelope of displacements and internal forces equivalent to the maximum and minimum branch of the envelope of the both load effects in directions $x + z$ and $y + z$. For sizing of the structure it is allowable to reduce earthquake load by coefficient of ductility *R* according to American standard. (to par 1631.5.4 [2]) that for the cooling towers (according to Tab.16 P in [2]) is determined R = 3.6. The calculated values of the envelopes of displacements and internal forces due to seismic loading states were compared with the envelope of the other loading states due to the dead, operational and live loads, wind and temperature actions, (Figs 4, 5, 6). It was stated that seismicity effects, taking into account ductility properties of the structure, are lower than the actions of wind pressure and other static load, especially temperature action due to environment and surface insolation.

Note: For using of reduction of earthquake load by ductility R it is necessary to satisfy condition of reasonable - sufficient way by shear reinforcement and spatial bending reinforcement e.g. with double-sided stressed reinforcement (into the armour plate cross) at both surfaces.

The results of response are determined in quantities of displacements and internal forces. The both characteristics are given separately for maximum and minimum branch of the envelope of the relevant combination. For a clear representation of the results of calculated response, the structure was divided into particular parts of plate elements and beam elements. Particular results are correlated with the coordinate systems as follows. Internal forces in plate elements, i.e. moments m_x and m_y, and axial forces n_x and n_y are determined in the middle plane of the plates and they are correlated with local axes of plate elements. In vertical members (in the walls, stiffeners and diffuser) local axis x has horizontal (global) direction, local axis y had vertical (global) direction. Local axis z has the direction normal to the middle plane of the element; internal forces in horizontal structures (ceilings, baseplate) have local axes x and y parallel to global axes. Axis z is normal to centre line of the member and it is directed upwards. In beam elements, local axis x is the axis of centre line of the member; axes z and y are the axes of cross section through beam member. Axis z is the axis in the direction as a rule of a longer dimension (height) of the section.

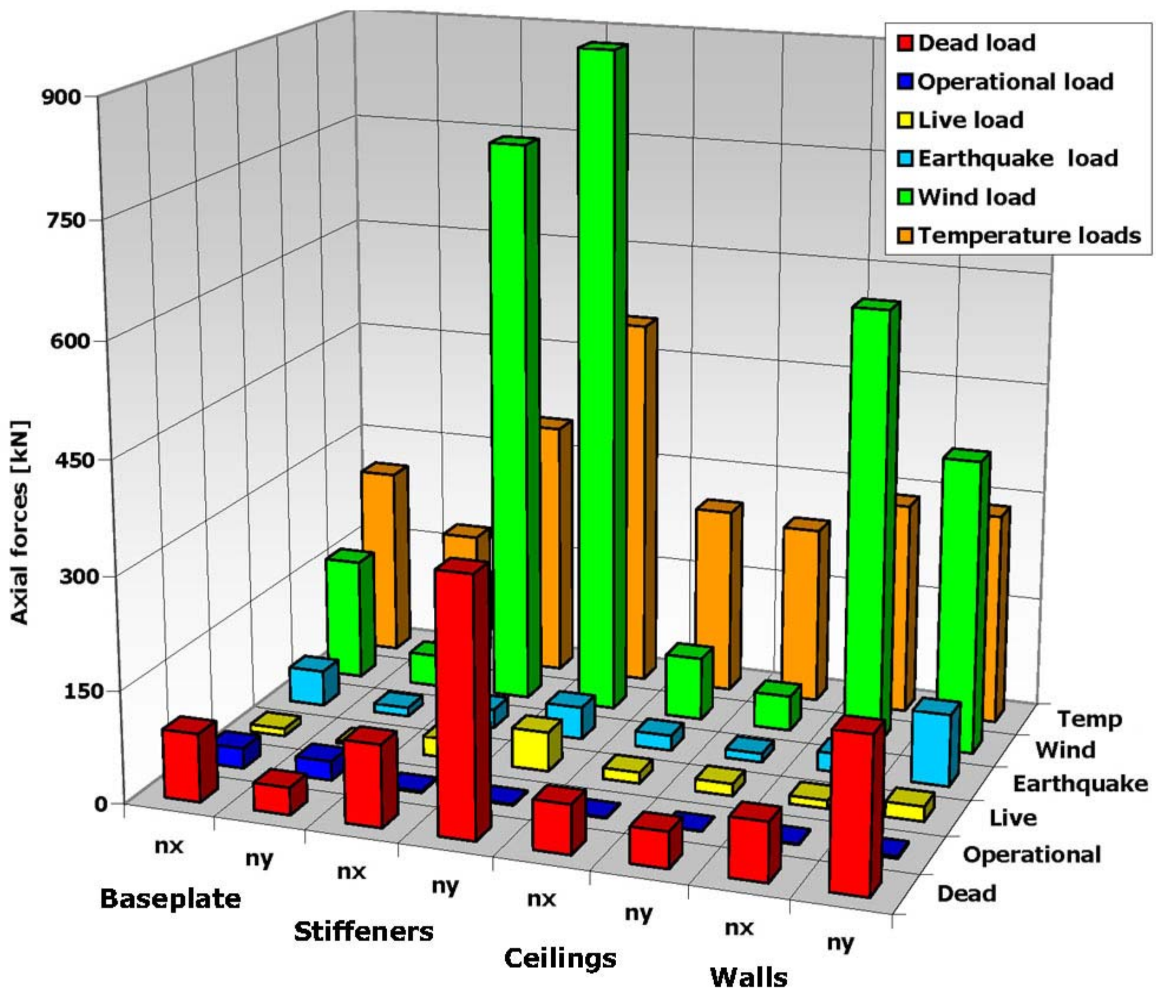

Figure 5 Comparison of design axial forces in plate elements for individual groups of loads

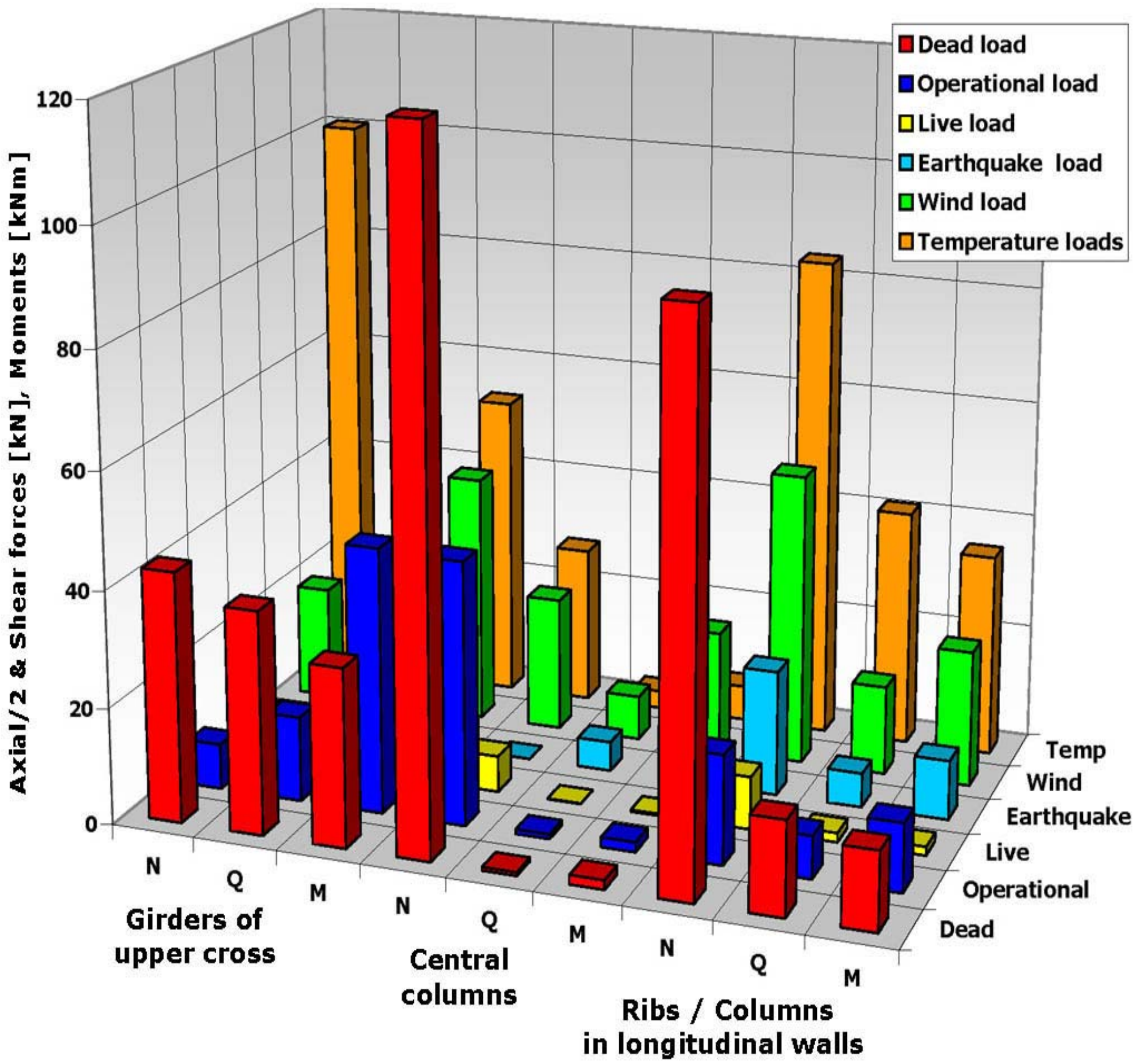

Figure 6 Comparison of design axial and shear forces and moments in beam elements for individual groups of loads

CONCLUSION

Using the example of a cooling tower unit, this paper analyses the influence of wind and natural seismicity and temperature effects beyond usual static load (dead, operational and live loads) on the static and dynamic response and compares the significance of these load types for the safety and reliability of the structure. The comparison has revealed that the dominant effect on the structure with reference to its safety (maximum displacements, extreme stress state in selected cross sections, etc.) is exercised by the temperature effect together with design wind load. The effects of natural seismicity (without reduction of this load by the ductility factor *R*) are comparable with the dynamic wind load within the interval of design wind velocities. However, technical seismicity may become dominant for the reliability of the structure in the case of vibrations of selected parts, such as joints, measuring probes installed in the structure for technological purposes, etc.

Therefore, for structure design, the combination of static loads with wind effect was used. In this case the wind effect in comparison with reduced seismic load is dominant. Reinforced concrete structure of the towers must have has also sufficient reserves for ductility strain at seismic load. In order to enable this ductility strain, the structure shall appropriately be reinforced especially by shear reinforcement – the principles of reinforcement are given in the Standard [4]. When use the ductility factor for sizing of structure dimensions, it must be submitted that after earthquake the structure shall be damaged and must be repaired (ductility strain assumes occurrence of cracks). Internal process equipment inside the towers shall probably be replaced.

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