IMPACT OF THE WIND LOAD PROBABILITY DISTRIBUTION AND CONNECTION TYPES ON THE RELIABILITY INDEX OF TRUSS TOWERS

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The paper concerns the analysis of reliability of three truss towers performed with the system approach. The first stage of the reliability analysis involved determination of the reliability index for trusses while assuming the same reliability of elements. In the second stage, assessment of the reliability was made according to Eurocodes. The impact of the wind load probability distribution and connection types in towers on their reliability was analysed. In the capacity function, the following random variables were taken into account: cross-sectional area, yield strength, modulus of elasticity, minimum moment of inertia, and length of the element.

Keywords: steel truss towers, load distribution, system approach, reliability index

1. Introduction

Steel truss towers are pin-jointed multi-element self-supporting structures. Towers are characterized by being rigidly fixed to the base. The towers come in a variety of types, the classification is based, among others, on use (Rykaluk, 2005). In many cases, steel truss towers provide the best structural option due to many factors, including low material consumption and a simple erection process. In the design of such structures, reliability assessment is an issue of key importance. The overall level of safety is largely determined by interaction of structural elements in the load-carrying system, and by safety of individual elements. The issues related to truss tower failure have been discussed in many papers (Błaszczyński *et al.*, 2014; Paczkowska and Paczkowski, 2013; Skwarek *et al.*, 2013; Skwarek and Hulimka, 2011; Davies, 2011).

The reliability analysis of truss structures has been dealt with by many researchers. Four groups of methods can be used for assessing the reliability of a structure: I level (semi--probabilistic) methods, II level (approximation and simulation) methods, III level (fully probabilistic) methods and the system approach (serial, parallel and mixed systems). In each of the methods, a reliability index (Woliński and Wróbel, 2001) is usually employed as a measure of the overall structural reliability. In (Kamiński and Szafran, 2010), the reliability index of a steel telecommunication truss tower was determined using a generalized stochastic perturbation method and the response function technique. In (Dudzik, 2017), the reliability index for an aluminium truss was determined with the FORM approximation method, and also the Monte Carlo and Importance Sampling simulation methods. The reliability index of an aluminium truss tower was also determined in (Winkelmann and Oziębło, 2015). The authors used three different probabilistic methods: the Monte Carlo method, the Point Estimate method and the Response Surface Method. Of all the reliability methods, the system approach is definitely the most comprehensive one. The system approach includes both determination of reliability models and reliability analysis. This type of the reliability analysis was used, among others, in (Biegus, 1977; Thoft-Christensen and Baker, 1982; Woliński and Wróbel, 2001; Park *et al.*, 2004;

Kłosowska *et al.*, 2017; Kubicka *et al.*, 2019; Mochocki *et al.*, 2018a,b; Mochocki and Radoń, 2019).

In addition to the choice of the reliability analysis method, it is very important to assign the right type of the probability distribution to the right types of loads. The distribution of the maximum (or minimum) values is adequate for variable loads such as wind, snow and ice. To describe some random variables, it is necessary to determine the distribution of extreme values, e.g. extreme wind load in a given time period. The most often used extreme value distributions include the Gumbel, the Weibull and the Frechet distributions (Gwóźdź and Machowski, 2011; Śniady, 2000; Murzewski, 1999; Nowak and Collins, 2000). To carry out the reliability analysis, it is necessary to replace a given distribution with another one having required characteristics. For approximation of the distributions, the following are most commonly applied: the Method of Moments and the Point Collocation Method (Gwóźdź and Machowski, 2011).

The paper reports the reliability analysis of steel truss towers, in which the system approach was employed. Three types of towers with different topology were discussed. Reliability models were determined for these structures, and next the reliability analysis was carried out. Two types of load distribution (normal and Gumbel) were used. In calculations, due to the transformation from extreme to normal distribution, two approximation methods were applied, namely the Method of Moments and the Point Collocation Method.

2. Materials and methods

In the paper, the system approach was used to analyse the reliability of steel truss towers. This method takes into account interaction of individual elements of the whole structure in the load-carrying system. The system approach consists of two stages. In the first stage, a reliability model is defined, and in the second – the reliability index is calculated.

2.1. Reliability model

To define the reliability model, which is the structure eigenvalue, kinematically admissible failure mechanisms (KAFMs), which contain minimal critical sets of elements (MCSEs), are specified. Failure of all elements included in the MCSE leads to transformation of the safe structural system into a mechanism – a system of incomplete fixity. To determine the reliability model, spectral analysis (Kłosowska *et al.*, 2017; Mochocki *et al.*, 2018b) of the linear stiffness matrix was used

$$(\mathbf{K}_L - \lambda\mathbf{I})\mathbf{q} = \mathbf{0} \tag{2.1}$$

where $\mathbf{K}_L$ is the linear stiffness matrix, λ is the eigenvalue and $\mathbf{q}$ is the displacement vector. To identify all possible non-repeatable combinations of removed rods transforming the truss into a mechanism, a program based on the application of the Finite Element Method was created.

2.2. Reliability of the system

The reliability analysis of the system leads to calculation of the reliability index β, which is a measure of safety. The reliability index depends on the reliability of the system R, which is determined for each reliability model applied. For the trusses of concern, three types of reliability models were obtained, i.e. the serial system, the parallel-serial system and the serial-parallel system (Fig. 1). The reliability R of these models was calculated as follows:
— for the serial system

$$R = \prod_{i=1}^{k} R_i \tag{2.2}$$

— for the parallel-serial system

$$R = \prod_{j=1}^{m} \Big[1 - \prod_{i=1}^{k}(1 - R_{ji}) \Big] \tag{2.3}$$

— for the serial-parallel system

$$R = 1 - \prod_{i=1}^{k} \Big(1 - \prod_{j=1}^{m} R_{ji} \Big) \tag{2.4}$$

After calculating the reliability of the system, the reliability index was obtained as

$$\beta = -\Phi^{-1}(1 - R) \tag{2.5}$$
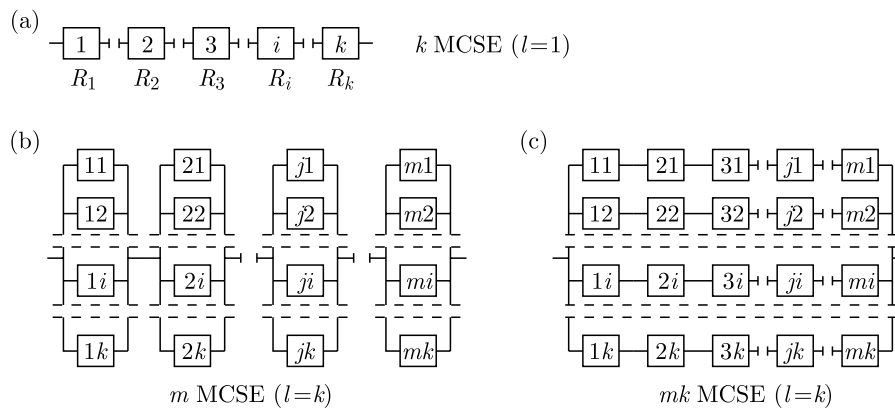
where Φ is the Laplace function.

Fig. 1. Types of reliability models: (a) serial system, (b) parallel-serial system, (c) serial-parallel system

The reliability of system (2.2)-(2.4) depends on the reliability of individual elements R_i (or R_{ji}), and on the number of system elements k, m or mk (Kubicka *et al.*, 2019; Mochocki *et al.*, 2018a,b; Mochocki and Radoń, 2019). The first step in the determination of reliability of individual elements R_i is to determine the expected value μ, standard deviation σ of the load effect $E_i(\mu_{E_i}, \sigma_{E_i})$ and capacity $N_i(\mu_{N_i}, \sigma_{N_i})$. This is the main part of the reliability analysis. The way of calculating these parameters is described in Sections 2.2.1 and 2.2.2, respectively. After computing the load effect and capacity, the standard deviation σ_{Z_i} and the expected value μ_{Z_i} of the safety margin Z_i were calculated

$$\sigma_{Z_i} = \sqrt{\sigma_{N_i}^2 + \sigma_{E_i}^2} \qquad \mu_{Z_i} = \mu_{N_i} - \mu_{E_i} \tag{2.6}$$

Finally, the reliability index β_i, and next the reliability R_i of a single element were estimated from the following formulas

$$\beta_i = \frac{\mu_{Z_i}}{\sigma_{Z_i}} \qquad R_i = 1 - \Phi(-\beta_i) \tag{2.7}$$

2.2.1. Effect of action

The expected value μ_{E_i} and standard deviation σ_{E_i} (first and second probabilistic moments) of the load effect E_i depend on the applied probability distribution of a random variable. To describe the behaviour of random variables representing structure loads only a few probability distributions can be used, e.g.: the normal (Gaussian), the log-normal, the Gumbel, the Weibull

and the Frechet distributions (Gwóźdź and Machowski, 2011; Śniady, 2000; Murzewski, 1999; Nowak and Collins, 2000).

In the paper, the analysed towers were designed according to the standards (PN-EN 1990:2004; PN-EN 1991-1-4; PN-EN 1993-1-1; PN-EN 1993-3-1). The main loads on these structures were the permanent load (self-weight), and also the wind load. Probability distributions for the wind load and for the permanent load differed significantly. To describe the permanent load, the normal distribution was used. As regards wind, the nature of load needs be taken into account. The wind load is idealized as a random (stochastic) process. In calculations, it is necessary to take into account the estimated lifetime of buildings. The characteristic wind load is determined from basic values of wind velocity or wind pressure. These values are identified based on long-term measurements taken by weather stations. In the measurement dataset, it is possible to find values that occurred the most often and very rarely. These empirical data can be approximated by a probability distribution. The best model used to describe the behaviour of wind load are the Gumbel, and Frechet and Weibull distributions. In the paper, the Gumbel distribution was applied.
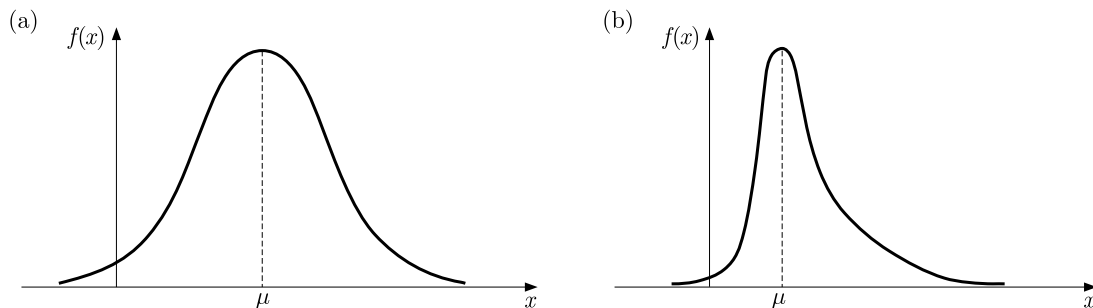

Fig. 2. Probability density function: (a) normal distribution, (b) Gumbel distribution

The normal distribution is characterized by the following density function

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \qquad x \in (-\infty, \infty) \tag{2.8}$$

an example of which is shown in Fig. 2a. Function (2.8) is defined by two parameters, namely by the mean value

$$\mu = \int_{-\infty}^{\infty} x f(x)\, dx \tag{2.9}$$

and by the variance

$$\sigma^2 = \int_{-\infty}^{\infty} (x-\mu)^2 f(x)\, dx \tag{2.10}$$

The Gumbel distribution is characterized by the following density function (Fig. 2b)

$$f(x) = \alpha \exp\{-\alpha(x-u) - \exp[-\alpha(x-u)]\} \qquad x \in (-\infty, \infty) \tag{2.11}$$

where α, u are parameters of the Gumbel distribution. The mean value and standard deviation are equal: $\mu = u + 0.5772/\alpha$ and $\sigma = \pi/2.4495\alpha$, respectively.

The method of the reliability analysis selected for the paper required that the normal distribution should be used for all random variables. That made it necessary to transform the Gumbel distribution into the normal distribution. To achieve that, two approximation methods,

namely the Method of Moments and the Point Collocation Method were used (Murzewski, 1989; Gwóźdź and Machowski, 2011).

As regards the two-parameter distributions employed in the paper, the method of probabilistic moments is based on the use of conformity conditions of the mean value and the variance. For the two-parameter distributions A and B, from the conformity conditions of the mean value and the variance, the following equations were developed

$$\int_{-\infty}^{\infty} x f_A(x)\, dx = \int_{-\infty}^{\infty} x f_B(x)\, dx \qquad \int_{-\infty}^{\infty} (x - \mu_A)^2 f_A(x)\, dx = \int_{-\infty}^{\infty} (x - \mu_B)^2 f_B(x)\, dx \quad (2.12)$$

To increase accuracy of calculations, the Point Collocation Method can be used. This method is based on the comparison of the cumulative distribution $F(x)$ and the probability density $f(x)$ at a properly selected collocation point x_d, which can be expressed by the following formulas

$$f_A(x_d) = f_B(x_d) \qquad F_A(x_d) = F_B(x_d) \tag{2.13}$$

This method can be used for any two-parameter distributions A and B for two types of the collocation point x_d, i.e. collocation at the central point and collocation at the limit point. In the paper, the location of the collocation point defines the parameter k. It specifies the probability that the realization of x_d of the variable X will not be exceeded with the probability k. A special type of the Point Collocation Method is the approach in which the approximating distribution is always a normal distribution (Rackwitz and Flessler, 1978). This approach is used in the paper. The relationships between the parameters of the approximating and approximated distribution have the following form

$$\frac{1}{\mu_x}\varphi\Big(\frac{x_d - \mu_x}{\sigma_x}\Big) = f_B(x_d) \qquad \Phi\Big(\frac{x_d - \mu_x}{\sigma_x}\Big) = F_B(x_d) \tag{2.14}$$

where φ is the function of the normal standard distribution.

An alternative to the presented analytical methods is provided by the graphical method based on the use of probabilistic grids (Gwóźdź and Machowski, 2011; Nowak and Collins, 2000).

2.2.2. Effect of capacity

The expected value μ_{N_i} and standard deviation σ_{N_i} of the effect of capacity N_i depend on whether the element is in tension or compression. Generally, the standard deviation of the capacity is a function of some random variables (Bołotin, 1968; Kubicka and Radoń, 2018)

$$\sigma_{N_i} \approx \sqrt{\sum_{i=1}^{n}\Big(\frac{\partial f}{\partial X_i}\Big)^2 \sigma_{X_i}^2} \tag{2.15}$$

where f is the function of uncorrelated variables, X_i is the single random variable and n is the number of variables. In the case of elements under tension, random variables are the cross-sectional area A and the yield strength f_y. The coefficients of variation for these variables are 6% and 8%, respectively (JCSS, 2001; Gwóźdź and Machowski, 2011). Based on approximation (2.15), the standard deviation of the capacity is written as

$$\sigma_{N_{t,Rd}} \approx \sqrt{\Big(\frac{\partial N_{t,Rd}}{\partial A}\Big)^2 \sigma_A^2 + \Big(\frac{\partial N_{t,Rd}}{\partial f_y}\Big)^2 \sigma_{f_y}^2} \tag{2.16}$$

which gives the following formula

$$\sigma_{N_{t,Rd}} \approx \sqrt{f_y^2 \sigma_A^2 + A^2 \sigma_{f_y}^2} \tag{2.17}$$

In the case of compressed elements, random variables include the cross-sectional area A, yield strength f_y, modulus of elasticity E, minimum moment of inertia J_{min}, and element length L. The respective coefficients of variation for the variables listed above are: 6% for A, J_{min} and L, 8% for f_y and 5% for E (JCSS, 2001; Gwóźdź and Machowski, 2011). The standard deviation of capacity (2.15) is written as follows

$$\sigma_{N_{b,Rd}} \approx \sqrt{\Big(\frac{\partial N_{b,Rd}}{\partial \chi}\Big)^2 \sigma_\chi^2 + \Big(\frac{\partial N_{b,Rd}}{\partial A}\Big)^2 \sigma_A^2 + \Big(\frac{\partial N_{b,Rd}}{\partial f_y}\Big)^2 \sigma_{f_y}^2} \tag{2.18}$$

where χ is the buckling coefficient, and σ_χ is the standard deviation of the buckling coefficient

$$\sigma_\chi \approx \sqrt{\Big(\frac{\partial \chi}{\partial A}\Big)^2 \sigma_A^2 + \Big(\frac{\partial \chi}{\partial f_y}\Big)^2 \sigma_{f_y}^2 + \Big(\frac{\partial \chi}{\partial E}\Big)^2 \sigma_E^2 + \Big(\frac{\partial \chi}{\partial J_{min}}\Big)^2 \sigma_{J_{min}}^2 + \Big(\frac{\partial \chi}{\partial L}\Big)^2 \sigma_L^2} \tag{2.19}$$

3. Results

The paper reports the reliability analysis of steel truss towers, in which the system approach was used. Three types of towers with different topology were discussed (Fig. 3). The towers had the same height ($h = 16$ m) and the same number of segments (8 segments), but they differed in geometry and number of elements l_e. Reliability models were determined for these structures, and next the reliability analysis was carried out. For the wind load, two types of distribution (normal and Gumbel) were used. The impact of connection types and load distributions on the reliability index was analysed. In calculations, to transform from the extreme to normal distribution, two approximation methods (Moments and Point Collocation) were applied. Calculations were carried out using the author-developed programs in the Mathematica environment.

In the first stage, the reliability model was defined. For the first case, tower T1 (Fig. 3a) ($l_e = 40$), the number of KAFMs is 528. They are parallel-serial and serial-parallel mechanisms. The number of causative elements for MCSEs varies from 2 to 9 (Fig. 4). For tower T2 (Fig. 3b) ($l_e = 33$), the number of KAFMs is also 528. All mechanisms are parallel-serial and the number of causative elements for all MCSEs is 2 (Fig. 5). The third case, tower T3 (Fig. 3c) ($l_e = 32$) is a statically determinate structure. It is a serial model with 32 causative elements (Fig. 6).

The second stage involved calculation of the reliability index β. The impacts of the reliability of individual elements and that of the load probability distribution on the reliability index were analysed. In the first case, the reliability index was calculated assuming the same reliability of all elements ($R_i = R$). Three cases of the reliability of individual elements were considered. The results are shown in Table 1. In the second instance, the towers were designed according to Eurocodes. In calculations, the normal and Gumbel distribution of the wind load were examined. The Method of Moments (MoM) and the Point Collocation Method (PCM) were used for transition from the extreme to normal distribution. The results are shown in Table 2.

Table 1. Reliability index β of trusses depending on the reliability of individual elements

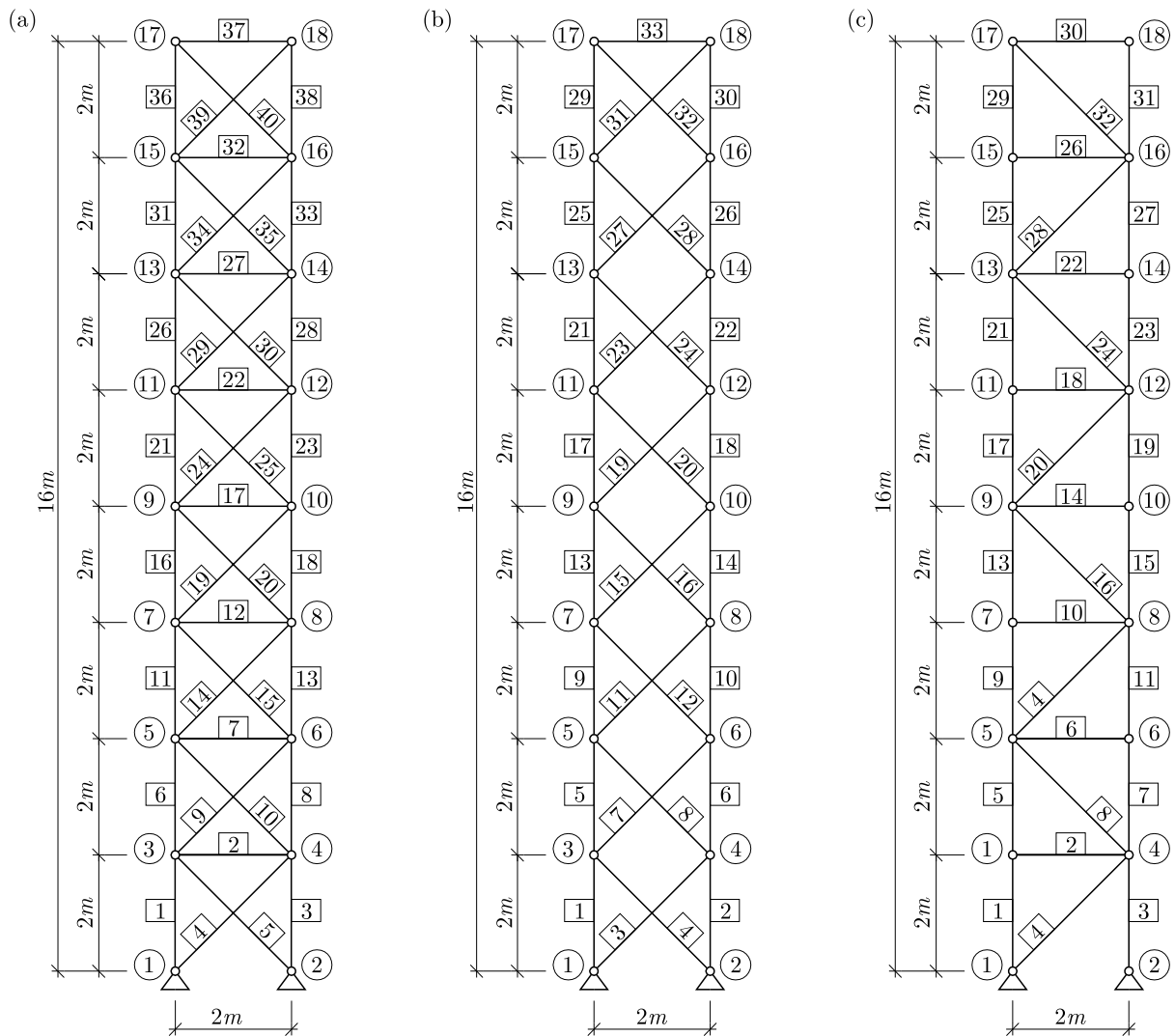| Reliability of individual elements	T1	T2	T3
$R = 0.999$	3.881	3.275	1.859
$R = 0.9999$	4.884	4.405	2.727
$R = 0.99999$	5.724	5.317	3.414

Fig. 3. Truss towers: (a) T1, (b) T2, (c) T3

Table 2. Reliability index β of trusses depending on types of load distribution

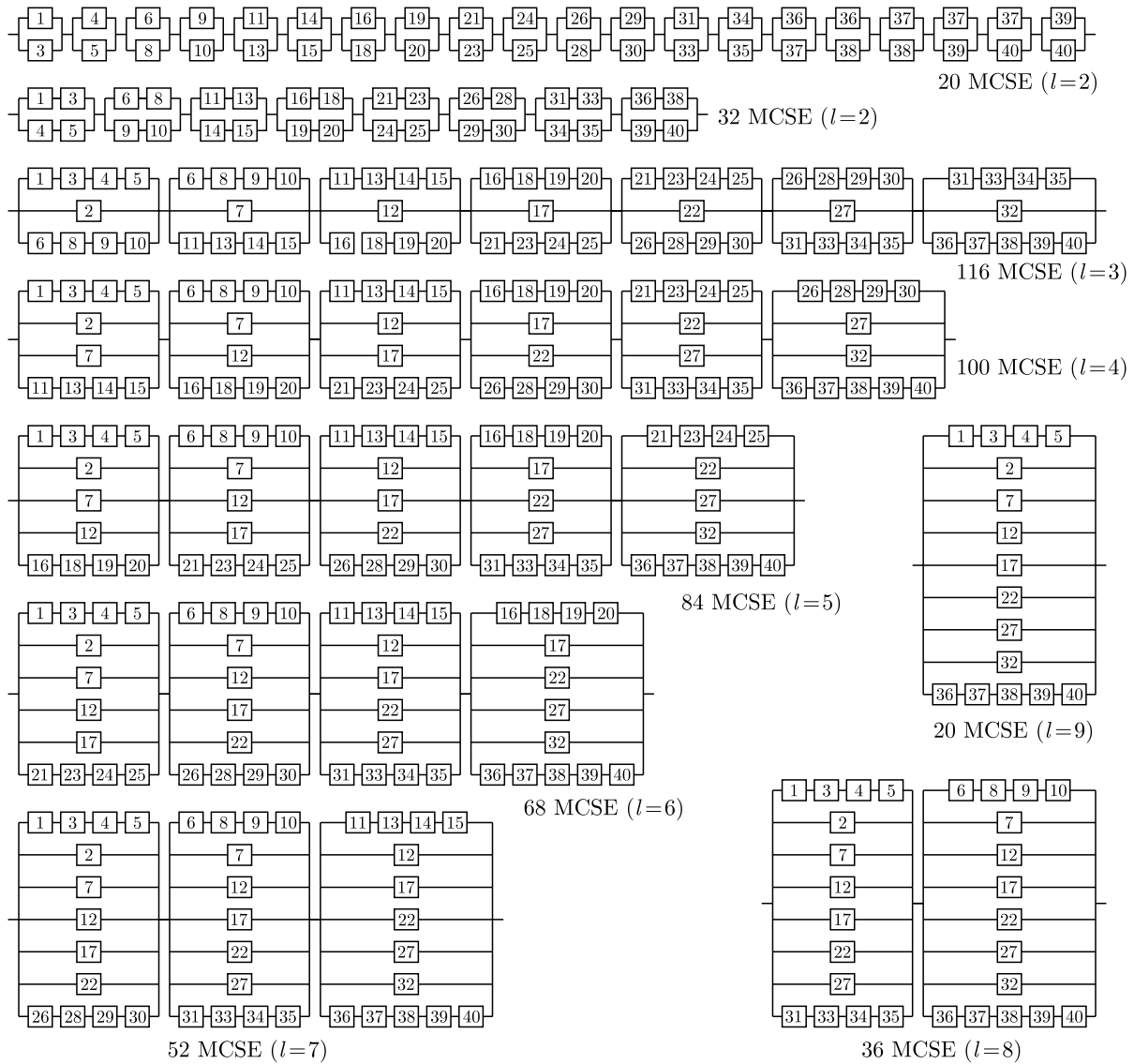| Load distribution $(\mu; \sigma)$	T1	T2	T3
Normal (27.891; 5.578)	3.896	3.927	2.550
Gumbel (27.891; 5.578) $\xrightarrow{MoM}$ Normal (31.106; 7.154)	3.334	3.256	2.056
Gumbel (27.891; 5.578) $\xrightarrow{PCM;k=0.5}$ Normal (26.974; 5.01)	4.083	4.146	2.710
Gumbel (27.891; 5.578) $\xrightarrow{PCM;k=0.6}$ Normal (29.69; 5.487)	3.706	3.712	2.396
Gumbel (27.891; 5.578) $\xrightarrow{PCM;k=0.75}$ Normal (35.123; 6.412)	2.995	2.870	1.781
Gumbel (27.891; 5.578) $\xrightarrow{PCM;k=0.9}$ Normal (45.494; 8.054)	1.800	1.314	0.661

Fig. 4. Kinematically admissible failure mechanisms for tower T1

4. Conclusions

The system approach is the most comprehensive reliability analysis because it allows taking into account structural arrangement. With the system approach, two relevant concepts that are closely related and often equated with each other, namely the reliability model and structural safety, can be distinguished.

The reliability model can be characterized by the structure eigenvalue which depends on its geometry and boundary conditions. To define the reliability model, kinematically admissible failure mechanisms (KAFMs) which contain minimal critical sets of elements (MCSEs) should be specified. Exhaustion of the capacity of all elements included in the causative MCSE leads to transformation of the safe structural system into a system of incomplete fixity (mechanism). Tower T3 is statically determinate. The reliability model for this structure is a serial system. It should be noted that for such a system, failure of one element is equivalent to failure of the whole structure. The minimal critical set of elements contains only one element, and the number of kinematically admissible failure mechanisms is equal to the number of causative elements. Consequently, Tower 3 is the most unreliable. An increase in the safety of the serial
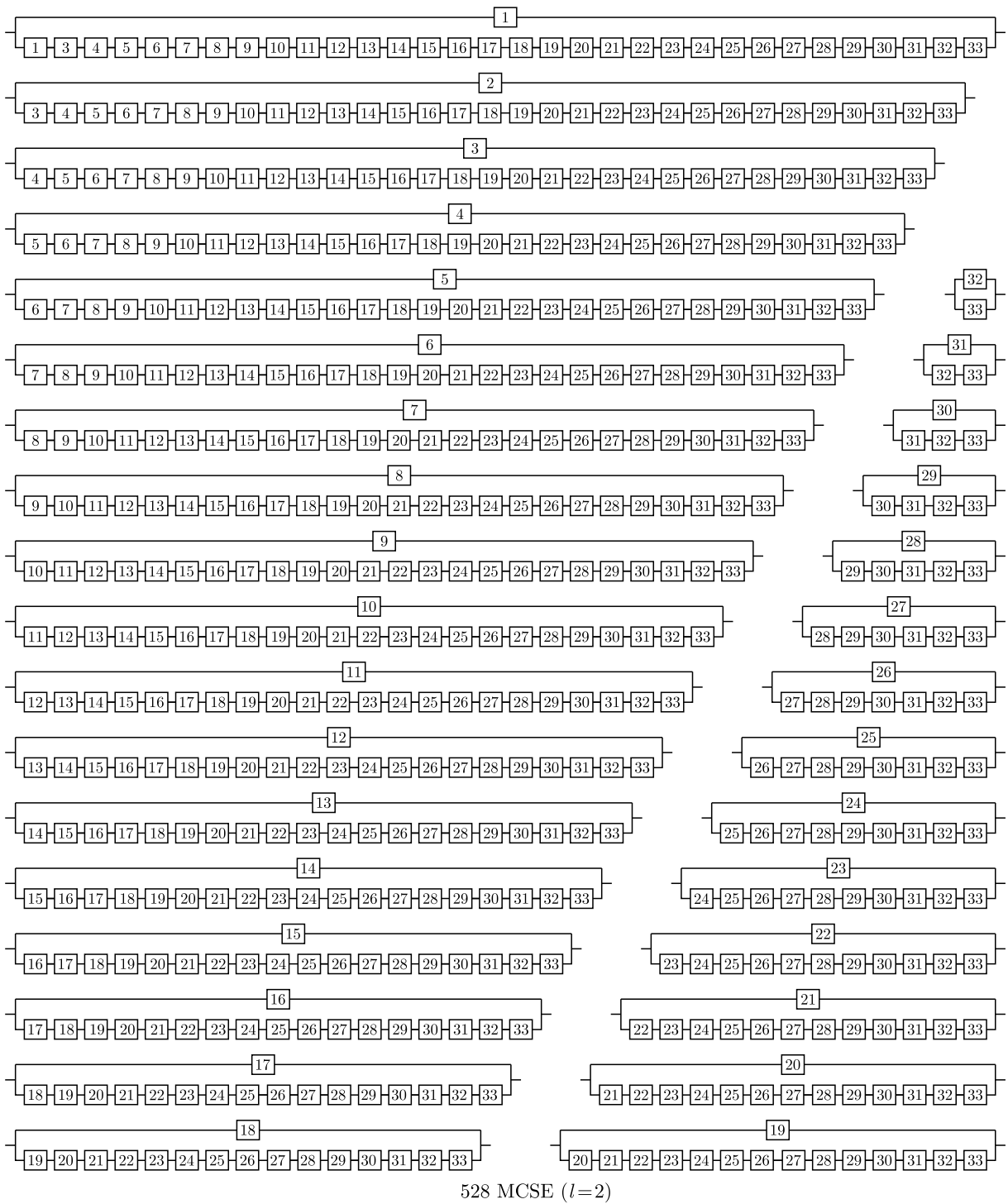
528 MCSE (l=2)

Fig. 5. Kinematically admissible failure mechanisms for tower T2
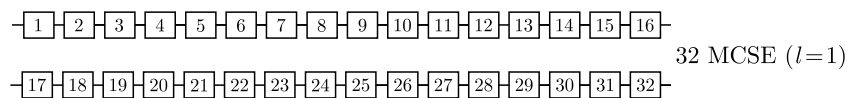

32 MCSE (l=1)

Fig. 6. Kinematically admissible failure mechanisms for tower T3

system can be obtained by two means, namely by increasing the safety of the weakest element of the system, or by limiting the number of elements that are linked in series. In the paper, Table 1 shows the increase in the reliability of the structure along with the increase in the reliability of the elements. Reliability of individual elements change from 0.999 to 0.99999. We assume here that the reliability of all components is the same. Towers T2 and T1 are statically indeterminate structures. These structures correspond to the mixed reliability model. Statically indeterminate structures have many kinematically admissible failure mechanisms, in which common elements are found. The same elements are components of more than one critical set. Then, estimation of the safety of the structure is a difficult task that gets even more complex as the numbers of MCSE and common causative elements increase. As a result, simplified estimation, i.e. the lower estimate of the structure safety is applied. That involves separation of the minimal critical sets of elements that have common causative elements. After MCSEs are separated, it can be assumed that all kinematically admissible failure mechanisms defined for a given structure are connected to one another in series. In the study, the assumption holds that MCSEs are unseparated. The reliability models of towers T2 and T1 differ fundamentally. The reliability model of tower T2 contains 528 KAFMs. All mechanisms are parallel-serial, and the number of causative elements for all MCSEs is 2. The reliability model of the tower T1 contain also 528 KAFMs, but the number of causative elements for MCSEs ranges from 2 to 9. They are parallel-serial and serial--parallel mechanisms. The failure of such a structure is definitely less likely. Tower T1 is the most reliable structure.

The wind load probability distribution has significant influence on the reliability index. In the paper, two types of load distribution (normal and Gumbel) were applied. The method of the reliability analysis selected for the paper required that the normal distribution should be used for all random variables. That made it necessary to transform the Gumbel distribution into the normal distribution. To achieve that, two approximation methods, namely the Method of Moments and the Point Collocation Method were used. The Method of Moments is simpler and relatively fast allows one to obtain satisfactory results. After transformation of the Gumbel distribution to the normal distribution the reliability index was reduced to 24%. The Point Collocation Method is more accurate but more computationally complex and, therefore, also more time-consuming. In this method, the most important issue is the choice of the parameter k. In the paper, calculations were performed for four cases of the parameter k ($k : 0.5, 0.6, 0.75, 0.9$). In the case of $k = 0.5$, the reliability index is overestimated compared to values obtained for the normal distribution, whereas in the case of $k = 0.9$ is significantly lower (to 286%). Much more reliable results were obtained for values $k = 0.6$ and $k = 0.75$. The advantage of the Point Collocation Method is the possibility of choosing the value of the parameter k depending on the design situation (uncertainty of the load model, structures with unusual loads). It allows obtaining the adequate safety margin.

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