Reliability Assessment of an OVH HV Power Line Truss Transmission Tower Subjected to Seismic Loading

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Abstract. The study focuses on the reliability of a transmission tower OS24 ON150 + 10, an element of an OVH HV power line, under seismic loading. In order to describe the seismic force, the real-life recording of the horizontal component of the El Centro earthquake was adopted. The amplitude and the period of this excitation are assumed random, their variation is described by Weibull distribution. The possible space state of the phenomenon is given in the form of a structural response surface (RSM methodology), approximated by an ANOVA table with directional sampling (DS) points. Four design limit states are considered: stress limit criterion for a natural load combination, criterion for an accidental combination (one-sided cable snap), vertical and horizontal translation criteria. According to these cases the HLRF reliability index β is used for structural safety assessment. The RSM approach is well suited for the analysis – it is numerically efficient, not excessively time consuming, indicating a high confidence level. Given the problem conditions, the seismic excitation is shown the sufficient trigger to the loss of load-bearing capacity or stability of the tower.

INTRODUCTION

The economical usage of electric power requires a fully operational extended power system. In the power grid, the overhead power transmission lines – the focus of the paper – are decisive and universal elements of the infrastructure. The power system is essential for the citizens and their safety, hence the supporting structures of the overhead high voltage (OVH HV) lines meet a number of strict technical requirements. Their required quality and reliability is the highest design priority.

This type of structures distinguishes wind and icing the most unfavorable of all loads covered by the Polish design codes. Taking any seismic loads into account is not mandatory in standard analysis of the supporting towers, although there actions really occur in mining or mountainous areas [1].

The efficiency of the power grid after seismic activity or motions induced by mining exploitation is an important element of security ensuring in the area endangered or affected by such an incident. The lack of electricity severely hampers the rescue operations, exposes the civilians to the risk of electrocution, intensifies chaos, and increases panic. In addition, the reparation or reinstallation of the line is very expensive [2].

COMPUTATIONAL MODEL AND NUMERICAL ANALYSIS OF THE TOWER

The overhead transmission line supporting tower OS24 ON150+10, presented in Fig. 1, is the case of the paper. The computational model of the tower, implemented in Autodesk Robot Structural Analysis software [3], was supplied by SAG Elbud Gdansk S.A. company. The passage tower, 32,28m high, is situated on a straight line, and sustains 6 AFL-6 240 conductors and 2 AFL-1,7 70 earth wires, of a 280m span. The structure is an assembly of 14 different L-shape profiles, made of S235 JR steel.

A spatial frame model is assumed for the tower. It consists of 375 nodes and 879 beam elements. The nodal joints are assumed quasi-rigid, with parameters diversified to match the assembly schemes for the elements (e.g. columns). The columns are pin-supported (fixed accordingly in all three dimensions) on a solid ground foundation.

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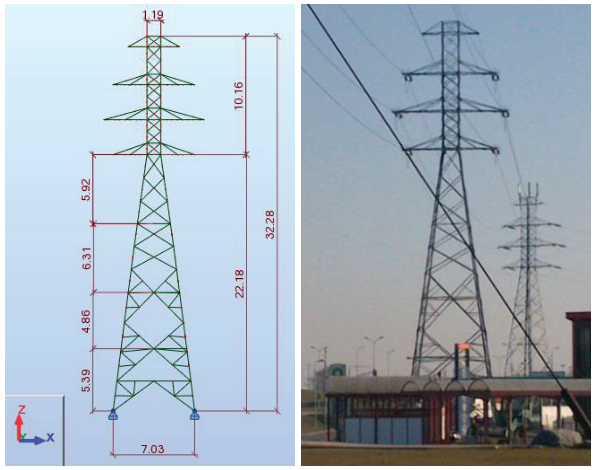


(a) (b)

FIGURE 1. A visualization of the tower in Autodesk Robot Structural Analysis software, with all the dimensions given (a), and a photograph of an erected, fully operational tower in Gdansk, Poland (b).

Seventeen load cases are considered, based on PN-EN 1993-3-1: 2008 [4], taking into account: dead load, equipment load, wind action, severe icing and different cases of cable tensioning. Moreover, ten different ULS combinations are proposed.

The most unfavorable load case corresponds to a case of all the conductors snapped on one side (equivalent to a collapse of a neighboring tower), leading to a maximum stress of 228.55MPa (which implies a low, 2.8% reserve of bearing capacity). However it should be noted that a one – sided cable pull is a rare, extreme event (an accidental combination), so a more probable combination is frequently assumed for the design guidelines. A combination neglecting the cable snap phenomena (a natural combination) also stands as a computation basis throughout the paper. This combination results in a maximum stress of 178.25MPa, yielding a 24.1% bearing capacity margin.

SEISMIC LOADING

A non-linear dynamic study carried out in the paper takes the actual earthquake recording into consideration. The El Centro earthquake (1940) records have been used in this work and the EW component (PGA=2.1 m/s^2) of the ground motion have been applied to the structure. Those records were the first examples of such earthquake recording made close to a main surface rupture by a strong-motion seismograph during a major seismic motion, thus a relevant description. The recording is presented in Fig. 2.

The El Centro earthquake took place in May 18, 1940 in the Southern California. The seismic motion caused a significant building damage in the nearest towns, it was observed that about 80% buildings in the Imperial Valley were partially damaged [5]. As a near – surface quake of a high potential, this tremor is a suitable pattern for possible excitations in Poland, as mining and technological activities may induce excitations of a similar range and scale. In the southern Poland region of Podhale in 2004 an earthquake similar to the El Centro tremor (yet relatively weak) caused a swift failure of the power grid, resulting in a day – long blackouts. The impact of the shocks on the supporting structures of overhead power lines was very diverse – in some towers additional tensional forces in the cables were observed, in the other support structures the horizontal deformations were observed too [6].

The occurrence of seismic waves caused by earthquakes is also a complex stochastic problem, the propagation analysis of vibration takes a considerable effort. The numerical solution of the problem requires a vast knowledge of the phenomenon – it is necessary to adopt the correct and detailed geometry of the structure, identify the material characteristics and topography of the subsoil and to select a relevant description of the dynamic force. Thus, analysis of a single excitation simplifies the problem – it is radically different depending on the location of shocks or of the power line. The verification of the obtained results is also problematic, due to the interpretation of the results [7].

Summarizing, the solution presented in this paper is an exact solution of a single problem, but it is not suggested as a general procedure. However, simple random methods, involving discrete FEM application are possible to properly estimate the structural response and the variation of its reliability.

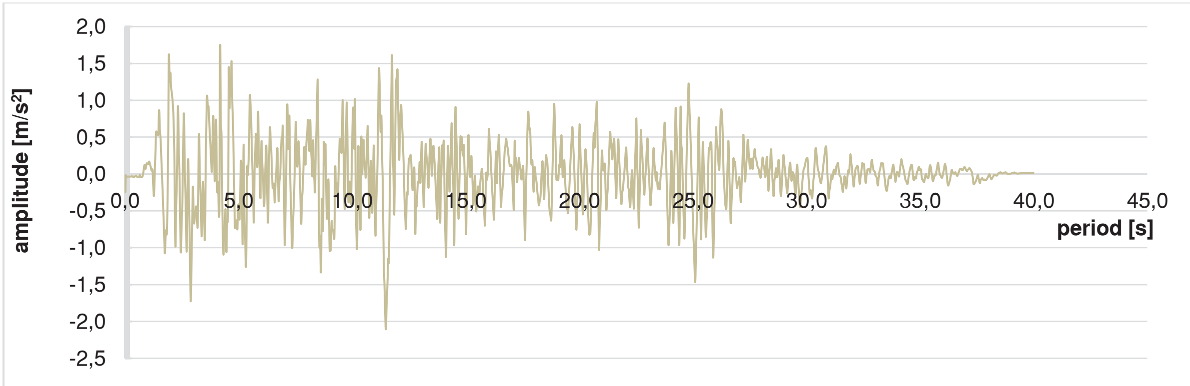


FIGURE 2. The EW component (PGA=2.1 m/s²) of the El Centro ground motion applied in the paper.

Random Description of El Centro Excitation Amplitude and Period Change

In the paper, the amplitude and the period of the El Centro earthquake were assumed as random task parameters, as suggested in [8]. Their random variation follows the Weibull distribution (1):

$$f(x;\lambda;k)=\begin{cases}\dfrac{k}{\lambda}\left(\dfrac{x}{\lambda}\right)^{k-1} e^{-\left(\frac{x}{\lambda}\right)^{k}}, & for\, x\geq 0\\ 0, & for\, x<0\end{cases} \tag{1}$$

This random model is widely used in case of accidental loading, e.g. the wind, snow, rainfalls and earthquakes. Its positive – only distribution function reflects the fact that the load is unable to have values lesser than zero.

The distribution factors are assumed as follows: k = 1.6 for shape and λ = 0.8 for scale. Those values were adopted a priori on the basis of available information on the medium-strength earthquakes in Poland. Thus the non – zero concise Weibull formula is expressed by (2):

$$f(x;\lambda;k)=\frac{1.6}{0.8}\left(\frac{x}{0.8}\right)^{0.6} e^{-\left(\frac{x}{0.8}\right)^{1.6}}=2.28653\cdot x^{0.6}e^{-1.42908\cdot x^{1.6}} \tag{2}$$

The Weibull probability distribution with the factors: k = 1.6 for shape and λ = 0.8 for scale is shown in Fig. 3.

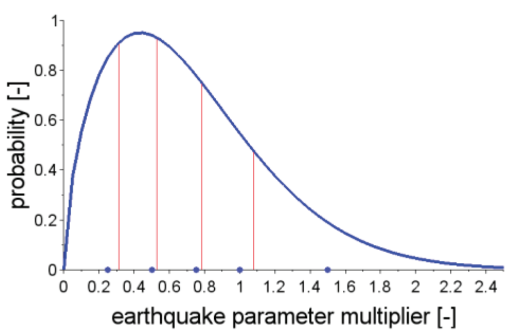


FIGURE 3. The Weibull probability distribution adapted in the paper (k = 1.6, λ = 0.8).

Adopting the distribution form (3) for both marginal distributions, a spatial joint probability density function is created. Afterwards, according to the Directional Sampling (DS) approach, the obtained space state of the phenomena is divided into a number of strata with equal probability (space state division into $5\times 5=25$ subspaces). Division of a Weibull random variable into strata is shown in Fig. 3 for a one – dimensional case.

A target sample is obtained from each subset, closest to a representative percentile of the strata. Based on this approach a number of 25 target samples is generated in the form of pairs of given discrete realizations of the marginal probability distributions, namely: $x_1,x_2\in\{0.25;0.5;0.75;1;1.5\}$, as also shown in Fig. 3.

Both the amplitude and the period distributions are bound to perfectly represent the El Centro excitation when $x_1, x_2 = 1$. Thus a set of motions weaker than the El Centro earthquake greatly exceeds the set of motions stronger than the base excitation – about 75% of the generated sample space is within the range where $x_1, x_2 \in \langle 0,1 \rangle$, a fact that also reflects the Poland – oriented adaptation of the El Centro ground motion.

RSM – BASED RELIABILITY INDEX ESTIMATION

One of the most frequently used means of structural reliability assessment is the reliability index β, a factor derived from the structural response probability space [9]. The index can be interpreted as the shortest distance from the neutral point of the analysis to the limit state function, described in the sample space of the phenomena. Simplified graphical outline of the reliability index determination procedure is given in Fig. 4.

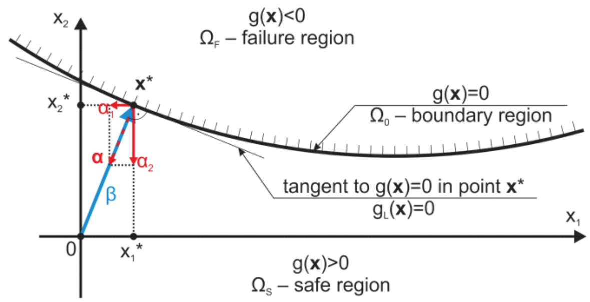


FIGURE 4. A two–dimensional, non-linear limit state function g (x) = 0 in random space state, the linearization of the limit state equation, the HLRF index determination and the unitary sensitivity normal vector α to the interface.

Calculating the index on the basis of real–life structural response is a considerable task, due to enormous complexity of the phenomenon [10]. This is a motivation to express the index as a function of an approximate response surface (linear, at best, if only possible). The approximation is done by means of dedicated software, RSM–Win [11] (a Fortran F90 source code), which is using the tabular analysis of variance (ANOVA table) method.

It should be pointed out, that adopting the neutral point is not a straightforward issue – in this analysis the starting point defines a lack of seismic excitation scenario. Thus, the limit state function of the task bounds a set of scenarios of a given seismic excitation leading to a failure of a structural tower member or to a sudden global collapse of the line.

For the structural response approximation applied in the task, a second order polynomial with cross terms is used, expressed by formula (3), where the β_i and β_{ij} are slope factors of the surface obtained from a minimal fit – ratio calculations, different for each of the presented criteria:

$$\bar{y}(x_1, x_2) = \beta_{11}x_1^2 + \beta_{22}x_2^2 + \beta_{12}x_1x_2 + \beta_1x_1 + \beta_2x_2 + \beta_0 \tag{3}$$

First of all, every following approximation of the response surface was performed for a two-dimensional probabilistic space of the phenomena. Taking the above – mentioned DS approach assumptions into account, a number of 26 calculation points (samples) is used in every approximation – the 25 samples defined in the previous chapter are refined by adding one sample representing the neutral point $(x_1, x_2) = (0,0)$, a case where no earthquake occurs. This is proved very useful due to a scenario of a high probability of structural failure.

While this type of approximation is chosen, the second order Hasofer–Lind–Rackwitz–Fiessler (HLRF) reliability index, known from previous investigations [11] to map even complex structural responses properly, is used in the analysis. It is also assessed by means of RSM–Win software. The HLRF index may also incorporate directly the β_i and β_{ij} factor values derived from the ANOVA table approximation.

It is worth mentioning, that other types of approximation formulas and other variants of reliability index estimation were tested using the dedicated software, however the results were either of unsatisfactory accuracy and slow convergence or more time–consuming.

In order to broaden the criterion analysis, the probability of failure is computed as the integral of each estimated response surface over the failure area. This comes from a highly nonlinear and mathematically complex form of a limit state function, for which the standard computations cannot be conducted, as it happens in the domain of Gaussian variables.

STRUCTURAL RELIABILITY ASSESSMENT UTILIZING SLS AND ULS CRITERIA

Four design limit states are considered in the paper: stress limit criterion for a natural load combination, stress limit criterion for an accidental combination (which assumes a complete cable snap on one side of the tower) and two translation criteria – for the vertical (u) and horizontal (r) displacements, both employing the natural load combination presuppositions.

The engineering approach considers each criterion essential – the first emulates the ULS, the third and fourth criteria emulate two basic SLS requirements. The second one, extending the standard ULS requirements, is recommended for further consideration as it is known to be crucial in structural failure prediction.

Stress Limit Criterion for a Natural Load Combination – ULS

First of all, a stress limit criterion for a natural load combination is analyzed. Based on the S235 JR steel bearing capacity, a limit is set on the value of 235 MPa. Taking into account a maximum stress level of 178.25MPa, it gives a safety margin of the neutral point equal to 56.75 MPa.

In this criterion, the extreme von Mises stresses, numerically computed in all 26 samples, are taken as the surface base nodes values. For the approximation the safety margin is calculated in [MPa] in every point. In this case of analysis, the second order response surface approximation gives a set of slope factors values for the base Eq. (3), leading to the Eq. (4):

$$\bar{y}(x_1,x_2)=25.732\cdot x_1^2+11.494\cdot x_2^2-12.682\cdot x_1x_2-36.770\cdot x_1-47.097\cdot x_2+67.647 \tag{4}$$

The results for the obtained approximate structural response surface of phenomena for the first criterion are shown in Fig. 5. Subsequently: the topography of the space state of the phenomena, a projection of the surface on the two-dimensional (x_1,x_2) probabilistic space, and the accompanying safe state (area marked in blue) are shown.

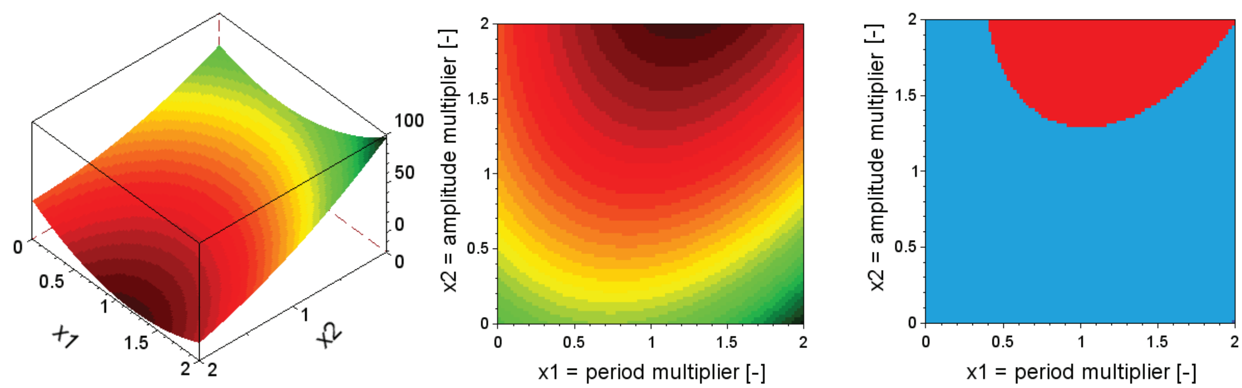


FIGURE 5. Approximate structural response surface (first criterion) – the topography of the space state (a), the two-dimensional projection of the surface (b) and the accompanying range of safe state (c).

On the basis of the surface given by formula (4), the HLRF reliability index is calculated: $\beta_{\mathrm{HLRF,C1}}=3.0059$. Such a value plainly shows that the probability of failure of the tower due to the loss of capacity under an earthquake is negligible, if the tower is subjected to a typical set of loads only, thus it is not a major threat either to the structure or to the overall OVH power line. The corresponding probability of failure of the structure is estimated to 10% only.

It should be pointed out, that this analysis is performed on a straight line typical tower – a separate ULS analysis should be made for different types of towers and for the towers of various functions in the line (apart from the tangent suspension structure, the angle suspension, tangent strain, angle strain, tangent dead–end and angle dead–end towers should be also considered), as the extreme von Mises stresses may vary significantly for different power line structures.

Moreover it is shown, that special attention should be paid to FE modelling of cross-bars and arresters elements. In a number of computational series the refinement of the starting numerical model of these members was required in order to carry out the calculations successfully.

Stress Limit Criterion for an Accidental Load Combination – ULS Extended

A more strict criterion concerns a stress limit for an accidental load combination. This load case introduces a scenario of a one-sided cable snap occurring coincident with the excitation with the highest amplitude of the motion. This scenario complements the natural ULS combination, analyzed in the previous section.

Again, a limit is set on the value of 235 MPa, so the maximum stress level of 228.55MPa results in the safety margin of the neutral point equal to 6.45 MPa only.

In this criterion, the accidentally heightened extreme von Mises stresses, numerically computed in all 26 samples, are taken as the surface base nodes values. As in previous approximation, the safety margin is computed in [MPa] in every point. For the adopted case of analysis, the second order response surface approximation gives a different set of slope factors for the Eq. (3), resulting in the Eq. (5):

$$\bar{y}(x_1, x_2) = 25.732 \cdot x_1^2 + 11.494 \cdot x_2^2 - 12.682 \cdot x_1 x_2 - 36.770 \cdot x_1 - 47.097 \cdot x_2 + 17.347 \quad (5)$$

It should be noted, that the Eq. (5) resembles the form (4), only the zeroth order term is modified. This is due to the fact, that a certain defined stress acts upon a group of elements already bearing the highest strain under a natural combination of loads. Thus a stationary set increment of the stress level causes the approximate surface to drop almost parallel to its previous position.

The obtained approximate structural response surface for the second criterion is shown in Fig. 6, similarly to the first part of the analysis.

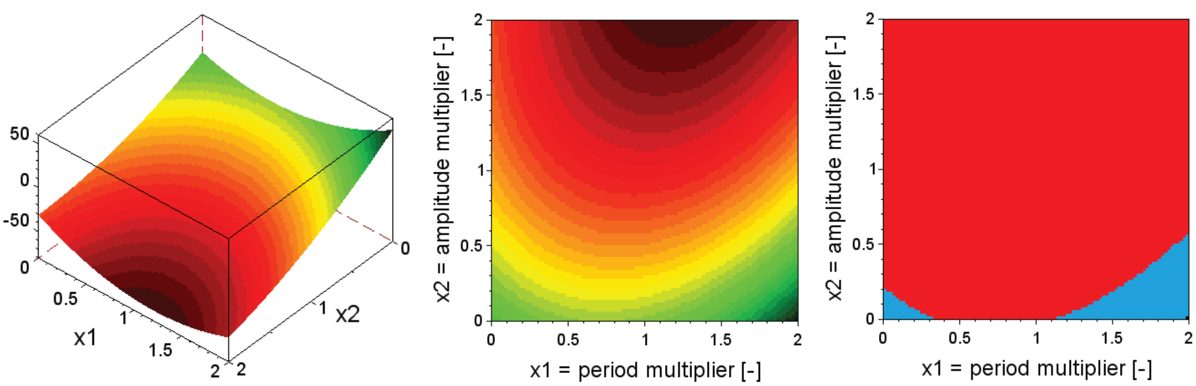


FIGURE 6. Approximate structural response surface (second criterion) – the topography of the space state (a), the two-dimensional projection of the surface (b) and the accompanying range of safe state (c).

On the basis of the surface defined by Eq. (5), the HLRF reliability index is calculated: $\beta_{\mathrm{HLRF,C2}} = 0.0761$. Reliability decreases dramatically between the two analyzed cases incorporating a stress limit state criterion.

The second scenario assumes an incident of conductors snapping on one side of the tower (or a collapse of an adjacent tower, as mentioned before) and it indicates, that the probability of structural failure due to the loss of bearing capacity or stability is very large, meaning a serious threat not only to the transmission tower, but to the entire OVH power line too. Such a low reliability index corresponds to the enormous, 99% failure probability.

If a complete cable snap on one side of the tower occurs during an earthquake, the impact of seismic excitation may be the sole and sufficient reason to the loss of the tower load-bearing capacity or stability. In such a situation any further reliability analysis does not pay, what is reflected in the real–life observations. This scenario may induce a cascade failure mechanism – an additional loading imposed on the neighboring towers due to the collapse of the analyzed one swiftly exhausts their safety margins. This is hazardous for the tangent towers, but far less probable for dead–end and angle dead–end towers.

It is worth mentioning, that the previous analysis, given in [12], clearly indicates, that the single-sided damage of the conductors produces a similar failure of the power line when the wind load or the icing level greatly exceeds the design code values. Additional set of cable snap load cases should be of grave importance in the design of towers, these restrictions should take priority over the standard dead load – wind – icing load cases. Such a situation is taken under consideration in the numerical model of the SAG Elbud Gdansk S.A. company.

Vertical Displacement Criterion for a Natural Load Combination – SLS in z–Axis

Next, the limit displacements generated by a natural load combination are analyzed. Vertical displacements are analyzed first, as the ground vertical motion is induced.

This time, the limit is imposed according the SLS regulation, set on the value of 20 cm, so maximum displacement of 12.8 cm results in a safety margin of the neutral point equal to 7.2 cm.

Due to the criterion, the extreme vertical displacements (u) values, computed numerically in all 26 samples, are taken as surface base nodal values. Again, a safety margin, this time in [cm], is computed for every point. In this case of analysis, the second order response surface approximation gives a new set of slope factors for the Eq. (3), resulting in the Eq. (6):

$$\bar{y}(x_1, x_2) = 0.225 \cdot x_1^2 + 0.052 \cdot x_2^2 - 0.136 \cdot x_1 x_2 - 0.290 \cdot x_1 - 0.164 \cdot x_2 + 0.259 \tag{6}$$

The results for the obtained approximate structural response surface for the third criterion are shown in Fig. 7.

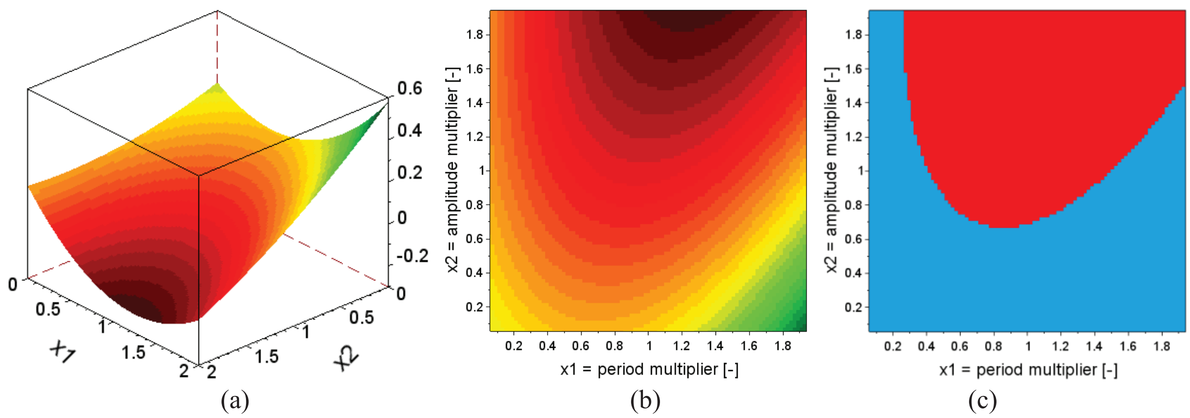


FIGURE 7. Approximate structural response surface (third criterion) – the topography of the space state (a), the two-dimensional projection of the surface (b) and the accompanying range of safe state (c).

On the basis of the surface given by formula (6), the HLRF reliability index is calculated: $\beta_{\mathrm{HLRF,C3}} = 0.7473$. It is relatively low, compared to other results of the natural load case. Thus the probability of failure of the tower due to an excessive vertical displacement (u) during the earthquake is low, if the tower is subjected to a typical set of loads only, hence it is a possible but minor threat to the transmission tower structure. The relevant probability of structural failure (this time an SLS case) is about 30%.

However, it should be stated, that in this case holding the SLS regulations is a complex problem. First, introduction of the limit value is heavily dependent on the type of the tower. Next, at different levels of seismic excitation and in different points in time the extreme displacements are observed in various nodes of the tower, a situation shown in Fig. 8.

Furthermore, numerical measurement of the tower motion continues for just 60 seconds. After that time the excitement almost disappears, but certain nodes of the tower, especially in the top regions of the columns or on crossbars, show a significantly longer return period to the initial position, even although including the material damping in the model. This effect is also presented in Fig. 8.

Also, the conductors flickering effect induced by the excitation of neighboring towers is not taken into account.

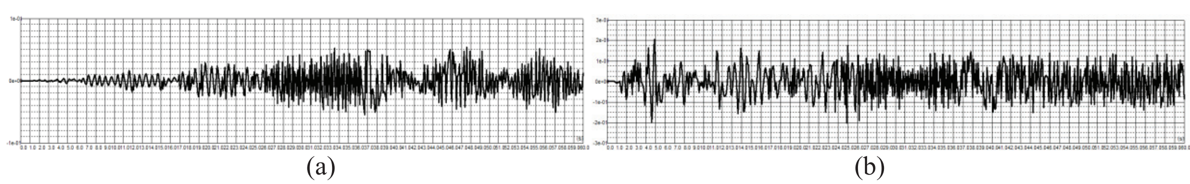


FIGURE 8. Diagrams of vertical displacement for two neighboring nodes of the tower for the standard load combination, supplemented with a El Centro – exact excitation – the node on the top of the column (a) and the furthest placed node on the adjacent top crossbar (b).

Horizontal Displacement Criterion for a Natural Load Combination – SLS in xy–Plane

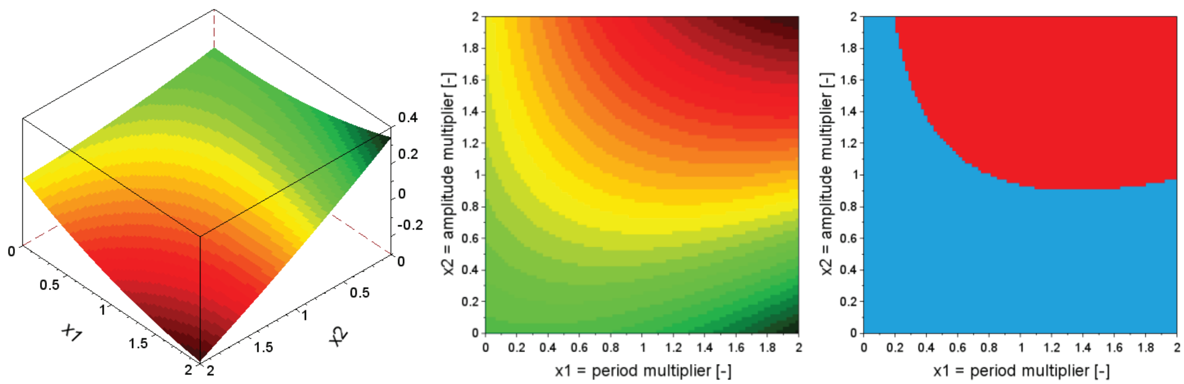
Finally, horizontal displacements are analyzed, with a joint approach to both transversal directions, in order to investigate maximum displacement independent of direction, a resultant vector in a transverse plane.

Again, the limit is set on the value of 20 cm, thus a maximal displacement value of 2.8 cm results in a safety margin of the neutral point equal to 17.2 cm. However it should be pointed out, that even the slightest excitation decreases this margin radically, suggesting that the neutral point will not prove useful in this approximation.

In this criterion, the extreme horizontal displacements (v) values, computed numerically in all 26 samples, are taken as surface base nodal values. Again, the safety margin, in [cm], is calculated for every point. In the adopted case of analysis, the second order response surface approximation gives a set of slope factors for the base Eq. (3), leading to the Eq. (7):

$$\bar{y}(x_1, x_2) = 0.054 \cdot x_1^2 + 0.021 \cdot x_2^2 - 0.128 \cdot x_1 x_2 - 0.030 \cdot x_1 - 0.102 \cdot x_2 + 0.189 \qquad (7)$$

The results for the obtained approximate structural response surface for the fourth criterion are shown in Fig. 9.



(a) (b) (c)

FIGURE 9. Approximate structural response surface (fourth criterion) – the topography of the space state (a), the two-dimensional projection of the surface (b) and the accompanying range of safe state (c).

On the basis of the surface defined by Eq. (7), the HLRF reliability index is calculated: $\beta_{\mathrm{HLRF,C4}} = 1.4095$. Such a value shows that the probability of failure of the tower due to an excessive horizontal displacement (v) during an earthquake is low, if the tower is subjected to a typical set of loads only, hence, not yielding a structural threat. The relevant failure probability of the structure (an SLS case again) is about 20%.

First of all, it should be noted, that the conclusions summarizing the previous section are also relevant here.

Also, it should be pointed out, that the two latter indices are highly affected by the SLS requirements – the more harsh the regulations the lower the indices. Thus the obtained values indicate that the PN-EN regulations are conservative and cautious – for a natural load combination their values are significantly lower than the value of stress limit criterion, so the SLS requirements act prior to ULS regulations. It also means, that the cross-sections of certain elements of the tower may be designed in an overly safe manner, considering a regular usage.

Moreover, an interesting effect is observed in the inflection of the structural response surface to a hyperbolic paraboloid. It is probably the cause of mixing both transversal directions in one series of calculations. The divided analysis in both directions separately is bound to remove this unfavorable form of the surface – a saddle of the surface in the vicinity of the neutral point may cause apparent convergence problems of the assessed reliability index or impose a low confidence level of the obtained results of the analysis. The latter problem is often accented in literature, e.g. [13].

Convergence Analysis of the Estimated Reliability Indices

It should be proved that the undertaken RSM – based structural reliability analysis is numerically efficient and trustworthy. Thus an additional convergence analysis is carried out for every assessed reliability index, independently of the order of surface approximation and the limit state scenario. The best convergence is observed for a HLRF second order reliability calculations, so this series of results is chosen for presentation in the paper.

The example of convergence analysis is provided for the most strict scenario, employing an incident of HV conductors snapping on one side of the tower as a result of earthquake action. The result of the second order surface – based HLRF reliability index estimation in each iteration (for accidental stress limit analysis) is shown in Fig. 10.

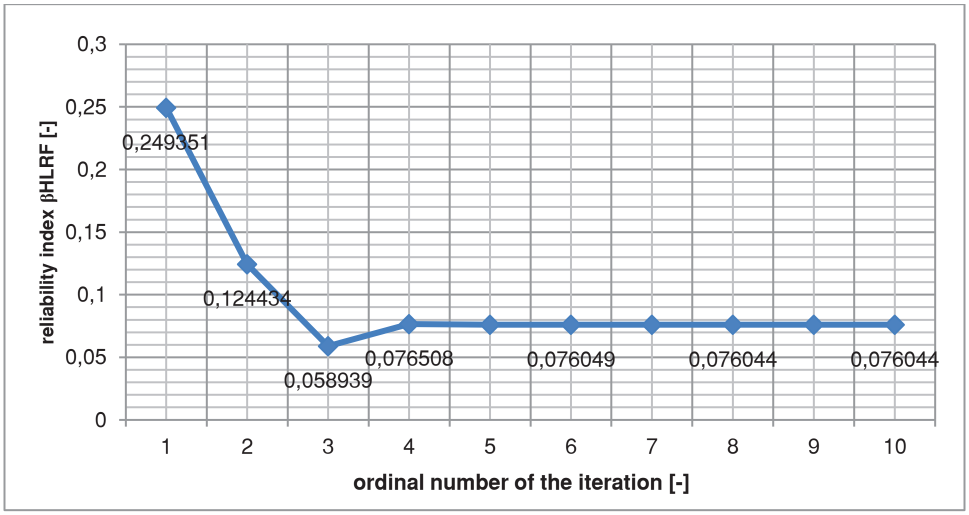


FIGURE 10. The second order surface – based HLRF reliability index estimation in each iteration of the analysis (for the accidental stress limit case).

The presented results state that the response surface methodology is numerically efficient and computational time saving, it also indicates a high confidence level. The application of a second order reliability index is justified given a shown significant curvature of the structural response surface and its non-monotonic topography. Higher order approximations are also possible, yet the ratio of the result precision versus the computational time decreases. The first order reliability analysis (the Cornell indices applied) give greater safety margins for the analyzed load cases, thus should not be taken into account in the design of transmission towers in the areas of seismic activity.

CONCLUSIONS AND FURTHER WORK RECOMMENDATIONS

It is clearly shown that the probabilistic assessment of structural reliability using standard techniques (the Monte Carlo Method – derived Directional Sampling technique and the Response Surface Methodology) is a trustworthy means. The reliability index is a sufficient measure of structural safety – the index estimation is much easier than computing the exact probability of failure, the latter requires computing complicated multi – dimensional integrals, in straight dependence on the dimensions of the space state.

Moreover, it is proven that a relatively small amount of computations is required to acquire results accurate enough – thus it is possible to match this approach with a conventional engineering design process. Even though some Monte Carlo Method variance reduction techniques are proved more effective than the directional sampling (DS) approach (in the way that using even a smaller number of samples may produce results of the same numerical resolution), the simplicity of the generation is viewed as the key reason to its application. Moreover the DS approach is known to be the most efficient while the number of random variables is limited.

On the other hand, a distinct FE model uncertainty is introduced, as shown in [14]. The parameters of the model should be verified with the data from existing OHV power line structures.

The occurrence and magnitude of seismic waves induced by earthquakes are crucial in the analysis. First of all, describing the earthquakes with only two identical random variables (the case presented in the paper) form an oversimplified strategy, though providing a sufficient measure of the structural reliability level for standard design process calculations.

In the case of a more complex analysis, first of all the relevant random model should not be built up on a set subsoil motion (like the El Centro recording), but the used data should reflect the local region, where the transmission towers are erected. A vast knowledge about the phenomenon is also necessary. A full time – variant stochastic process analysis is recommended, mostly due to different magnitudes and return periods of the earthquakes, as suggested in [15].

The support truss structures are erected in a generally safe manner, yet the high structural safety margin incorporates the unexpected, accidental loads [16]. However, if a failure occurs to the transmission lines, the reliability of the structures of the line is almost brought to zero and the towers may be destroyed independently on the loads acting on a structure at the moment of its failure.

It should be noted, that the presented analysis was carried out on an ideal model. The time and environment – induced imperfections are separate factors to be further considered. It was proved that both geometric irregularities and material parameter degradation of the tower elements can strongly reduce the load-bearing capacity of transmission towers, and the reliability indices. In case of cable–snap scenario, the second order geometric effects and the weakening of the material can lead to the failure of the tower even without any seismic excitation, such a situation is shown in [17].

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