

SEISMIC RELIABILITY BASED OPTIMIZATION OF STEEL PORTAL FRAME STRUCTURES

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Abstract: The paper deals with the complex structural optimization of steel structures subjected to seismic loading. The developed algorithm adopts state-of-the-art design and analysis tools with respect to the seismic performance assessment and to the optimization methods. The performance assessment is based on complex and comprehensive reliability analysis directly incorporating the uncertainties of the seismic effects and the response of the structure. Due to the large number of variables and the high nonlinearity of the problem, genetic algorithm is invoked for the optimization. The developed algorithm is applied for low-rise steel frame industrial hall structures. Based on parametric study, influence of various components on the optimal solution are quantified, and conclusions on optimal design are drawn.

Introduction

In both moderate and high seismicity areas, the seismic resistant design is an important issue not just because of the life safety requirement, but also in order to avoid significant losses caused by a seismic excitation. The high nonlinearity in demand and capacity induces a time-consuming, trial and error design process, wherewith cost-effective, economical configurations may be hardly found. Furthermore, using prescriptive codes, combined with linear analysis methods, does not give an overall picture from the seismic performance and reliability of the structures. The economic and other losses and the insurance costs can be reduced by designing buildings associated with higher reliability index. Because of these considerations, the designers may need concepts and recommendations, wherewith better performing, more economical configurations can be achieved and the iterative design process can be shortened. To reach proper design recommendations, structural optimization techniques can be effectively utilized. Available results in the literature are normally derived from simplified calculations: either the number of incorporated optimization variables are small or the performance assessment is simplified. This fact motivated the authors to develop a complex and comprehensive structural optimization algorithm, first introduced in (Balogh and Vigh, 2013) and (Balogh et al., 2014). The algorithm is now improved by adopting state-of-the-art analysis, computational and assessment tools: the optimization algorithm is based on Performance Based Design concept, where the performance of the structure is defined as the reliability of the structure. The reliability assessment of steel portal frames is very complex problem, characterized by high degree of nonlinearity and needs high computational capacity to solve. Monte Carlo analysis often applied for non-normal distribution problems is not practical since thousands of nonlinear numerical analyses have to be carried out. In this study, a First Order Reliability Method (FORM) (Choi et al, 2006) based numerical algorithm is adopted to assess the probability of failure of steel structures subjected to seismic effects.

The paper adopts the introduced algorithm and focuses on low-rise industrial halls with steel portal frames (Fig. 5). There is lack of information in the literature related to the achievable reliability of steel portal frames. In most cases the researchers analyse multi-storey Moment Resisting Frames (Liu et al., 2014; Lin et al., 2010; Lagaros et al. 2008). However, it is possible in higher seismicity areas that the seismic effects become the leading action even in

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case of a light-weight structure. Results on such structures are hardly available in the literature. The influence of stressed skin design and dissipative design concepts on the optimal structural configurations shall be also studied.

Optimization problem and solution methodology

In case of extreme effects (i.e. effects that are extreme in nature or consequences), a solution with the minimum weight or initial cost may not necessarily be the optimum solution when the whole life cycle of the structure is considered. It is conceivable that slightly increasing in the cost of the bracing system can mean a significant change in the structural reliability.

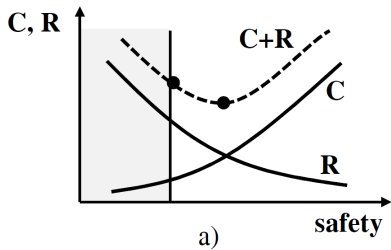


Figure 1. Optimal design: life cycle design optimum

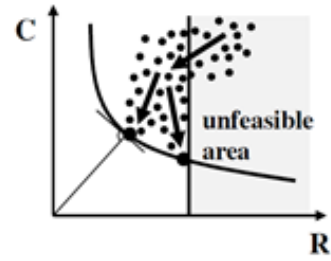
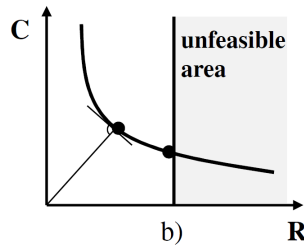


Figure 2. Genetic algorithm: solution strategy

In Fig. 1a the lower black point indicates the optimal configuration having the sum of cost (C) and risk (R) minimum, the upper one shows feasible optimum according to the standard. The dashed line (C+R) is the so-called life cycle cost. The risk as a cost component is strongly dependent on the function of the facility, the value of the stored materials and the economical losses due to the failure of the building. For this reason, general definition of the life cycle cost cannot be given. In case of minimum initial cost design the optimality problem can be formulated as follows:

$$\min! f(\mathbf{x}) \cdot P(\mathbf{x}), \tag{1}$$

where $P(\mathbf{x})$ is the penalty function, which expanded:

$$\min! K(\mathbf{x}) \cdot f_{EQ}(\mathbf{x}) \cdot g_{SLS}(\mathbf{x}) \cdot g_{ULS}(\mathbf{x}) \tag{2}$$

$$f_i(\mathbf{x}) = \begin{cases} 1 & p_{f,i}(\mathbf{x}) \leq p_{lim,i} \\ \left(\frac{p_{f,i}(\mathbf{x})}{p_{lim,i}} \right)^2 & p_{lim,i} < p_{f,i}(\mathbf{x}) \end{cases} \quad g_i(\mathbf{x}) = \begin{cases} 1 & \eta_i(\mathbf{x}) \leq \eta_{lim,i} \\ \eta_{lim,i} < \eta_i(\mathbf{x}) \end{cases}, \tag{3}$$

where $K(\mathbf{x})$ is the cost of the structure, $f_{EQ}(\mathbf{x})$ is the penalty function related to the reliability constraint. The $p_{lim,i}$ is the allowed probability according to the i^{th} failure mode. $g_{SLS}(\mathbf{x})$ and $g_{ULS}(\mathbf{x})$ are the discrete constraints related to the serviceability and ultimate limit states, respectively. The $\eta_i(\mathbf{x})$ and $\eta_{lim,i}$ are the utilization ratio in the i^{th} discrete limit state and the allowable utilization, which is defined as unity.

The performance objective is defined by the target value of the reliability index computed from complete reliability analysis. Assuming that the relative cost of increasing the safety of structures subjected to seismic loading is high and considering moderate consequences, the selected target reliability index value is 1.98 (for 50 years of reference period) in accordance to the Probabilistic Model Code of Joint Committee on Structural Safety (JCSS, 2000)

The probability of failure and the reliability index are calculated by the help of FORM analysis. To solve the optimization problem, a Genetic Algorithm (GA) based numerical algorithm has been developed in Matlab, as detailed in (Balogh and Vigh, 2012). During the seeking the optimal solution, GA examines a series of potential solutions (Fig. 2).

Failure functions

FORM analysis requires the estimation of the failure function in each iteration step. Flowcharts of the adopted failure function evaluation methods are presented in Figs. 3 and 4 for elastic and dissipative seismic design concepts, respectively.

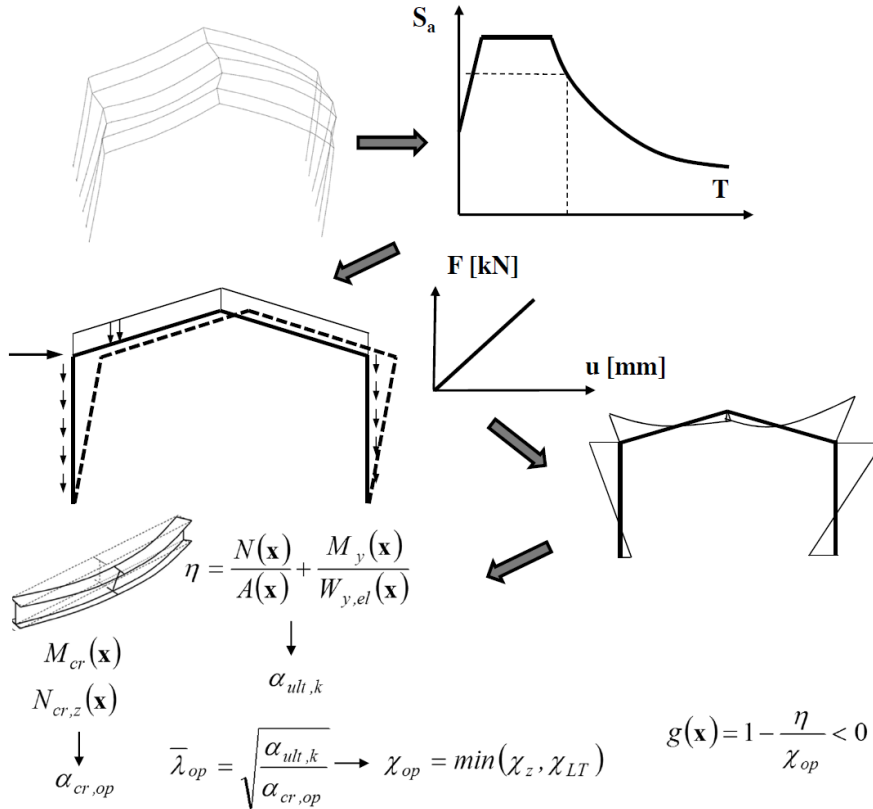


Figure 3. Failure function for elastic seismic design.

In case of elastic design, modal response spectrum analysis in accordance to Eurocode 8 Part 1 (EC8-1) (CEN, 2008) is invoked for the global seismic analysis. The analysis is completed on a 3D numerical model (refer to section of modelling). Based on the fact that the structure mainly vibrates in the first mode, the numerical analysis is carried out in both principal directions with a load distribution according to the first vibration mode. Strength and stability verification is completed in accordance to General Method of Eurocode 3 Part 1-1 (EC3-1-1) (CEN, 2005). According to the method, in-plane stability failures are considered via geometrically nonlinear analysis on imperfect model, while out-of-plane stability modes are handled by the reduction factor method (Fig. 3).

In case of dissipative design, the global response of the structure is calculated by geometrical and material non-linear static analysis (pushover analysis) and the seismic verification is completed at the target displacement level (Fig. 4). Strength and stability of elastic (non-dissipative) members are verified in the same way as above, while dissipative zones are verified in terms of deformation capacity (ductility). Plastic hinges are considered at beam ends and in the side bracing (CBF or BRB are considered). The deformation capacity and post-elastic stiffness of plastic hinges in the beam and of BRBs are selected on the basis of the provisions of FEMA 356 (FEMA, 2000) and on the basis the experimental studies detailed in (Zsarnóczy, 2013).

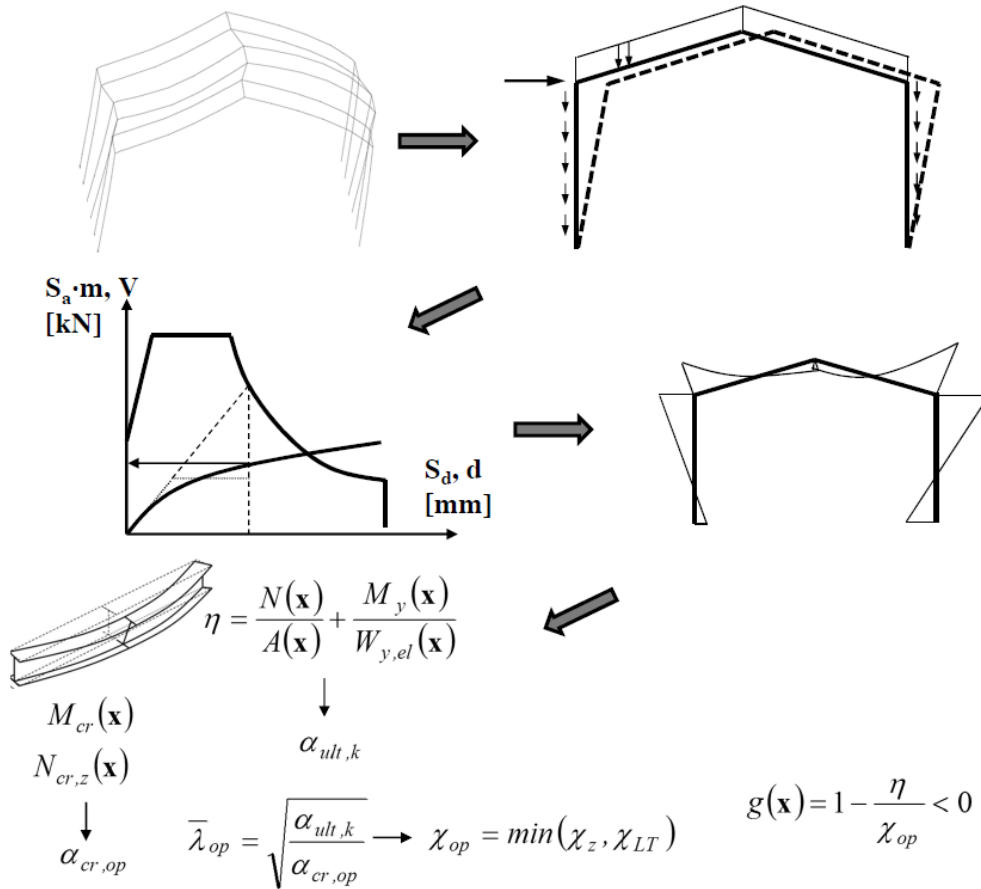


Figure 4. Failure function for dissipative seismic design.

Reference structure

Application of the developed methodology and algorithm is illustrated through the optimization of a low-rise (single storey) steel industrial hall. The reference structure is shown in Fig. 5.

The building consists of steel moment resisting frames with a span of 19,0 m and height of 9,36 m. Height of the tapered column is 5,9 m, the distance between the frames is 6,0 m. The girder is also tapered in the vicinity of the column-girder corner. The applied steel grade is S355. Trapezoidal sheeting supported by thin-walled Z-profile purlins is applied for the envelope of the building. Solid circular (tension-only) elements are used for the wind bracing. The side bracing can be composed of either X bracing with tension-only rods, conventional X-bracing or buckling restrained braces (BRB).

Ground snow load and wind load are represented by the distributed load values of 1,25 kN/m² and 0,59 kN/m², respectively. Beside the structural self-weight and the weight roof and side wall layers, an additional permanent load of machinery with varying values is also considered. As for the site-specific seismic parameters, Soil Type C and Type 1 spectra are considered in the analysis of two sites with different peak ground acceleration values (Komárom, Hungary and Râmnicu Sărat, Romania).

During each optimization case, the above parameters are considered as fixed. The optimization variables are the cross-section dimensions (height, flange width, flange and web thicknesses) of the main frame, cross-section of the wind bracing elements and cross-section of the side bracing.

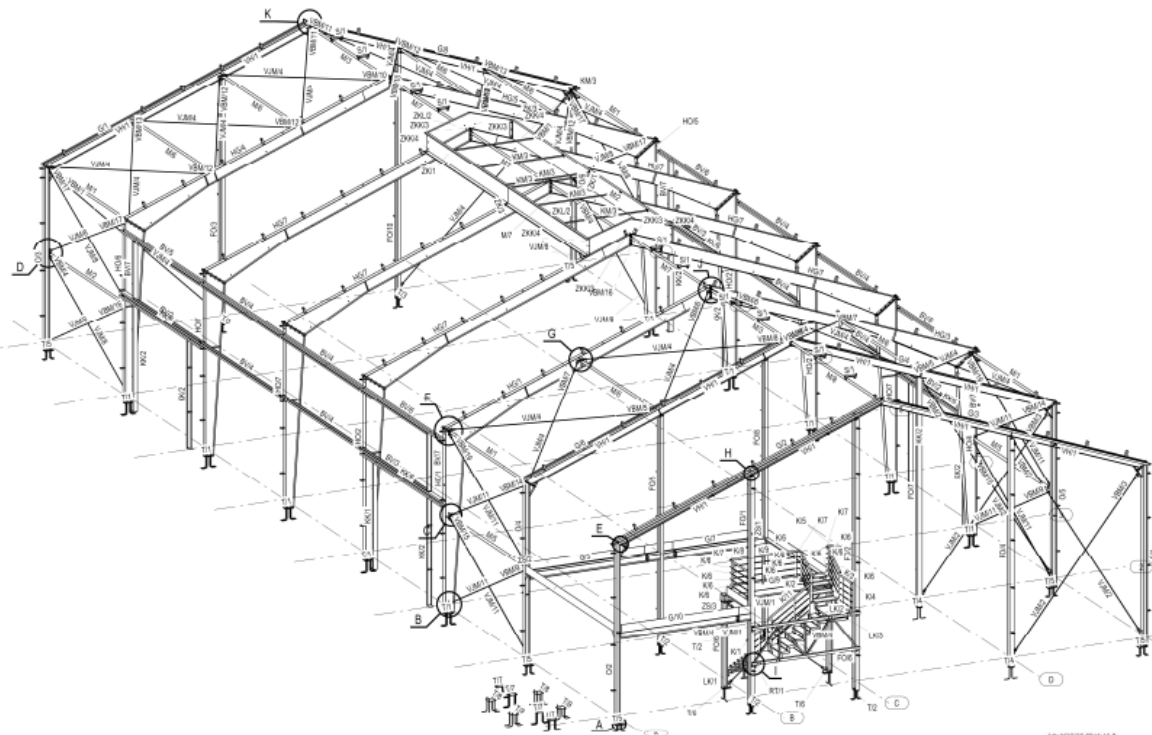


Figure 5. Reference building.

Parametric study

In the framework of a parametric study, influence of the following parameters on the optimum solution are investigated:

- rigidity of trapezoidal sheeting and purlins (i.e. inclusion of stressed skin design),
- seismic mass by varying the permanent service load (weight of machinery),
- seismic intensity (peak ground acceleration),
- span,
- elastic or dissipative design.

The studied cases and the corresponding parameters are listed in Table 1.

Table 1. Parametric study – Studied cases.

#	Permanent service load [kN/m ²]	Site	Peak ground acceleration [g]	Span [m]	Stressed skin design	Design concept
RLTP4502	0,2	Râmnicu Sărat	0,3	19	yes, LTP45	elastic
KLTP4502	0,2	Komárom	0,15	19	yes, LTP45	elastic
Rn02	0,2	Râmnicu Sărat	0,3	19	no	elastic
Kn02	0,2	Komárom	0,15	19	no	elastic
RLTP4510	1,0	Râmnicu Sărat	0,3	19	yes, LTP45	elastic
Rn10	1,0	Râmnicu Sărat	0,3	19	no	elastic
RLTP4515_1	1,5	Râmnicu Sărat	0,3	12	yes, LTP45	elastic and dissipative
RLTP4515_2	1,5	Râmnicu Sărat	0,3	12	yes, LTP45	elastic

Random variables

Table 2 summarizes the random variables. Most of the random variables are associated with distribution types and distribution parameters derived from literature review.

The uncertainties in the global stiffness are summarized into three random variables. In transverse direction the uncertainty in the stiffness of bracing system and sheeting and the uncertainty caused by connections, frames and foundations are separated. The effect of the spatial variation in stiffness of bracing elements, sheeting and frames are analysed. The results confirm that the applied uncertainty factors cover this issue.

The uncertainties in the seismic action are given by two random variables. To determine the distribution parameters, seismic hazard analysis is completed for the given sites, using the database of European Facility for Earthquake Hazard and Risk (EFEHR).

Table 2. Random variables and their distribution parameters

Random variable	μ	CoV	Distribution	Reference
Yield strength [MPa]	388	0.07	Lognormal	JCSS (2000)
a_{gR} [g]	Komárom	0.092	Lognormal	Calculation + EFEHR (2015)
	Râmnicu Sărat	0.168		
Permanent service load [kN/m ²]	0.2/1.0	0.2	Normal	
Beam section modulus factor	1	0.05	Normal	JCSS (2000)
Column section modulus factor	1	0.05	Normal	JCSS (2000)
Global stiffness factor in transverse direction – sheeting and bracing	1	0.2	Normal	
Global stiffness factor in transverse direction – other	1	0.1	Normal	
Global stiffness factor in longitudinal direction	1	0.25	Normal	
Effect model uncertainty	1	0.1	Lognormal	Calculation + JCSS (2000)
Resistance model uncertainty	1	0.2	Lognormal	
Model uncertainty – connection resistance	1.25	0.15	Lognormal	JCSS (2000)
Model uncertainty – roof bracing resistance	1	0.1	Lognormal	
Model uncertainty – wall bracing	1	0.1	Lognormal	
Intensity and record-to-record uncertainty	Komárom	1	Lognormal	Calculation + EFEHR (2015), FEMA (2009)
	Râmnicu Sărat	1		
Deformation capacity factor of plastic hinges	1	0.1	Lognormal	
Hardening factor of plastic hinges	1	0.1	Lognormal	
BRB $\omega\beta$	1.85	0.075	Normal	Zsarnóczyay (2013)
BRB yield stress	$f_y \cdot 1.13$	0.04	Normal	Zsarnóczyay (2013)

Numerical model

For the structural analysis a 3D numerical model as illustrated in Fig. 7 is developed in OpenSees (McKenna, 2012). Beam elements with 6 DOF per node are used for the frame and purlin members. Wind and side bracing are modelled by truss elements. The rigidity of the sheeting is represented by spring elements connecting the corresponding nodes of adjacent frames. Depending on the design concept and the applied analysis method (elastic modal response spectrum analysis or pushover analysis), elastic or bilinear elasto-plastic material model is associated to the frame members and the side bracing elements.

The numerical model is validated by full-scale test results; for details refer to (Dunai et al, 2015).

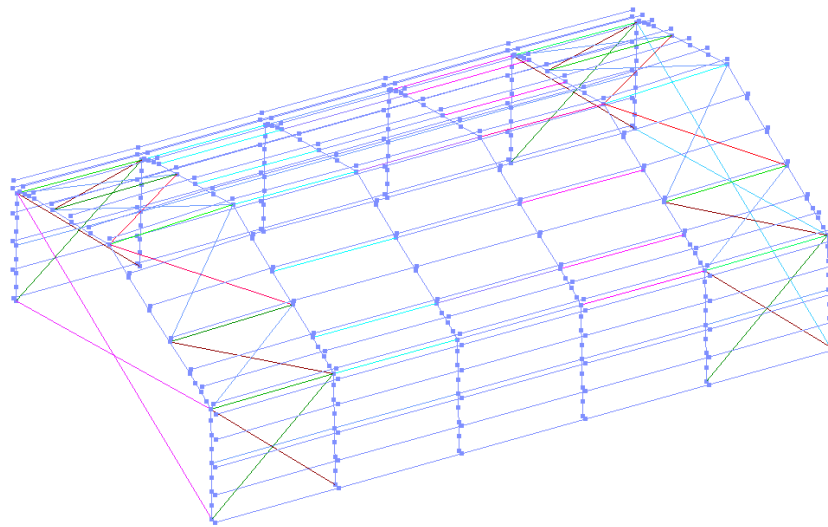


Figure 7. Numerical model of the structure.

Results of the optimization

Fig. 8 shows the cost of the best solutions in each iteration steps. The figure well represents the convergence of the optimization procedure. It can be observed that the optimal solution is normally found in 30-50 iteration steps. It is thus stated that the procedure is numerically stable and achieves reasonable convergence rate.

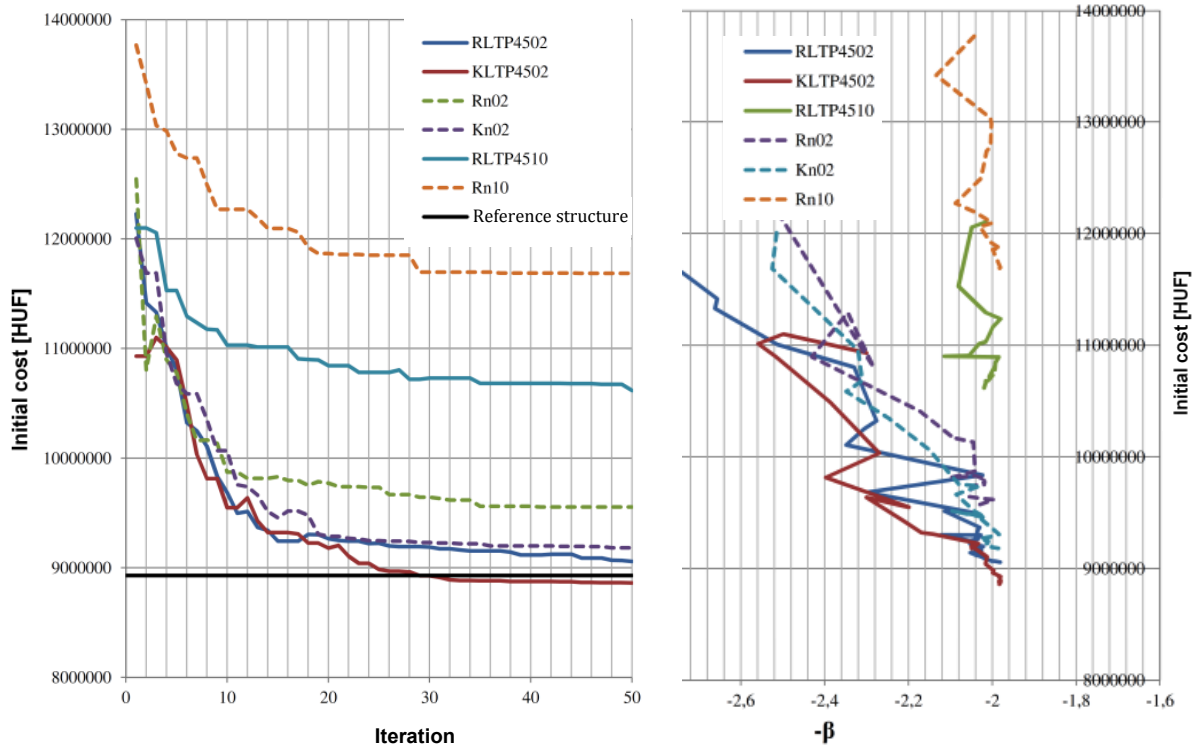


Figure 8. Convergence of optimization, cost and reliability index of the optimized structures.

Typical optimal solutions are shown in Fig. 9: the figure indicates the obtained cross-section dimensions both for the main frame and the bracing members. Comparing Fig. 9/a to 9/b and Fig. 9/c to 9/d, the beneficial effect of the sheeting rigidity is clearly observed: both the main frame and the bracing sections can be decreased. As a result, the overall initial cost of the structure can be decreased by 10-15%, which is also confirmed by Fig. 8.

The above conclusion is not straightforward and thus requires further explanation. Adding the rigidity of the sheeting to the structure increases the overall rigidity, which in general yields to larger seismic forces: in the cases of Fig. 9/a-b, the difference in the base shear forces is 24%. However, the sheeting connects the frames and thus a beneficial “diaphragm” effect develops, which causes the redistribution of the forces among the frames. This is illustrated in Fig. 10: although the total base shear force in the transverse direction is larger, the internal frames are subjected to lower seismic loads.

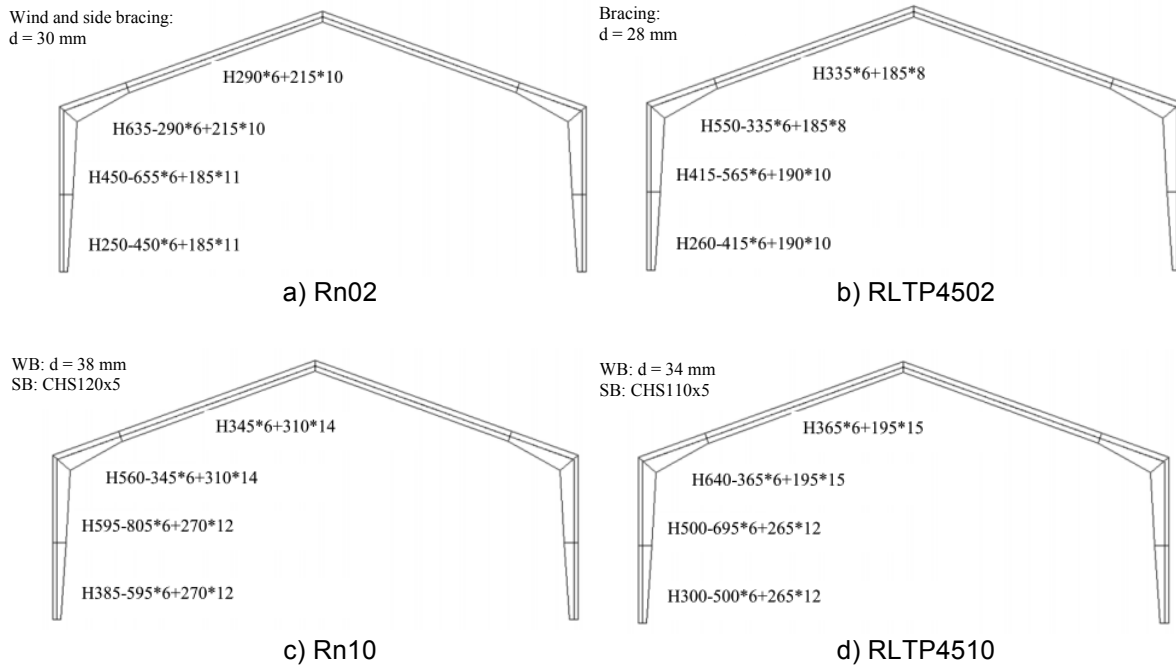


Figure 9. Optimal main frames for Râmnicu Sărat.
 (R: Râmnicu Sărat; n: sheeting rigidity not considered; LTP45: sheeting rigidity considered; 02 and 10: permanent service load of 0.2 and 1.0 kN/m², respectively)

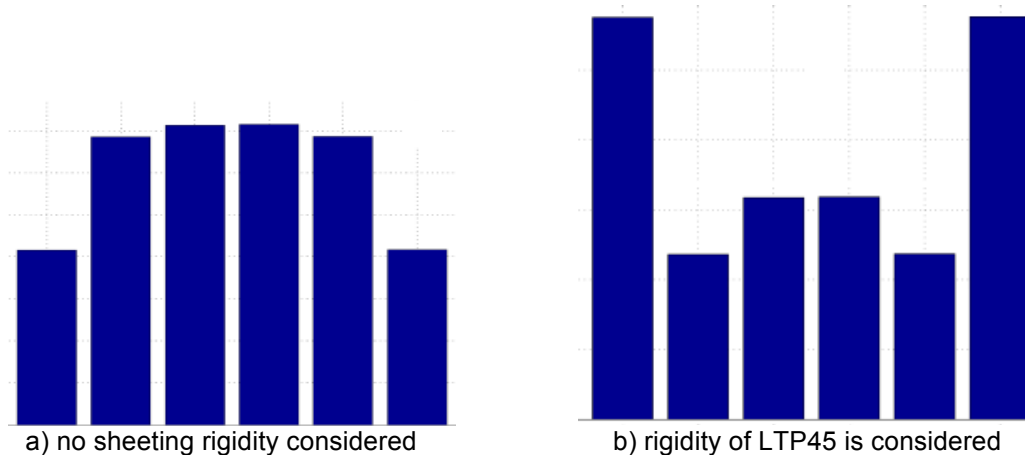


Figure 10. Distribution scheme of base shear forces among frames (Râmnicu Sărat, 0.2 kN/m²).

As Fig. 8 confirms, we can achieve nearly the same initial cost as of the reference structure in one case only. The reference structure was designed in accordance to EC8-1: the corresponding results are summarized in Fig. 11. The structure is verified by the code for the higher seismicity (Râmnicu Sărat). However, using the analysis developed by the authors indicates that the reliability index of this structure is $\beta = 1,45$, thus it does not reach the prescribed target reliability index (1.98). The authors believe that the primary reason for this is the fact that – unlike the developed algorithm – the design value of the peak ground

acceleration is based on the median hazard curve and does not consider the relatively large variance of the intensity.

In order to characterize the effectiveness of the optimization procedure, the reference structure is redesigned (Fig. 12) to achieve the target reliability index values. Comparing the obtained configurations with the optimal solutions, it is stated that the optimization provides approximately 5% cost saving.

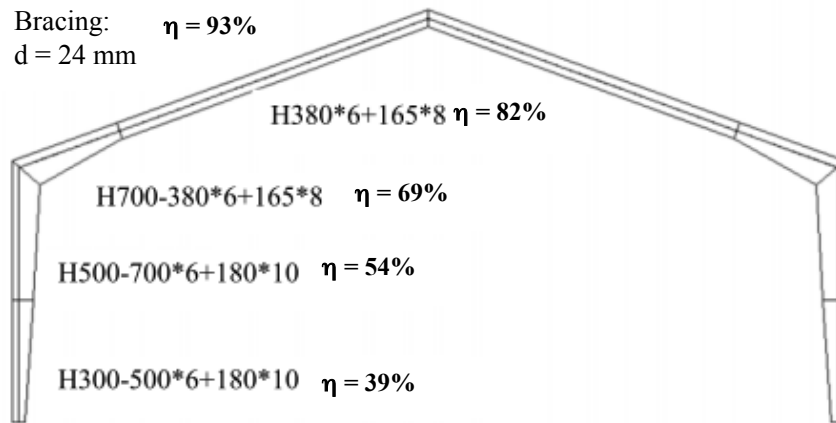


Figure 11. Reference structure: cross-sections and utilization factors (demand-to-capacity ratios) calculated from EC8-1, for Râmnicu Sărat (sheeting rigidity not considered).

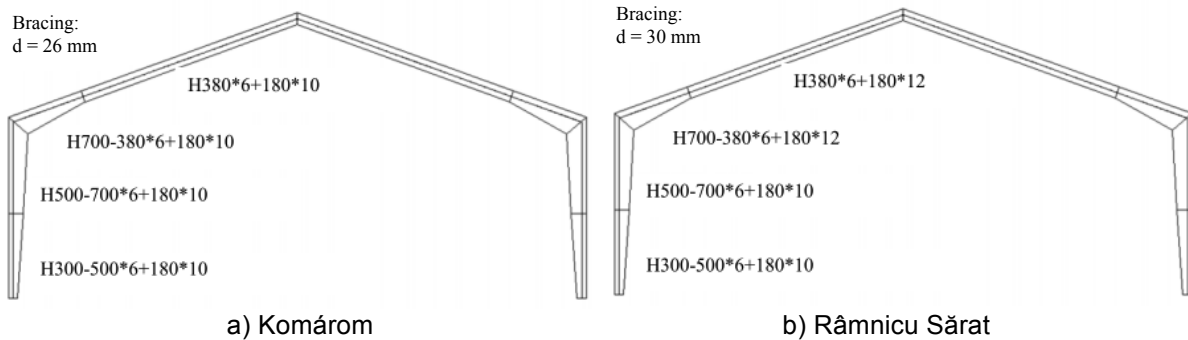


Figure 12. Strengthened structures for $\beta = 2.0$ (sheeting rigidity not considered).

By assessing the results of the case RLTP4515_1, the elastic and dissipative design concepts can be compared. It is observed that in the studied case (relatively light structure, no heavy floor system included), large ductility can be utilized in the longitudinal direction only. As the main frame contributes to the lateral resistance, the achieved reduction in the main frame cost is 2% only. On the contrary, in the longitudinal direction the cost of the seismic resistant system may increase, depending on the applied dissipative system. Note that, however, the base shear forces are decreased in both directions: by 20% and 70% in the transverse and longitudinal directions, respectively, which indicates large cost saving on the foundations. Also note that proper dissipative design concept normally provides higher reliability as well.

Concluding remarks

The paper introduced the development of a general optimization framework for steel structures subjected to seismic effect. The developed algorithm incorporates state-of-the-art design, analysis and assessment tools (performance based design, reliability assessment) as well as advanced optimization procedures (genetic algorithm). The performance objectives are expressed in terms of target reliability index values. Unlike common reliability analysis methods for seismic performance assessment, the developed method directly

considers the uncertainties of both the effect and the resistance sides.

Effectiveness of the developed algorithm is illustrated on a low-rise industrial hall. It is confirmed that the optimization process is numerically stable and effective.

Completing parametric study, the beneficial effect of stressed skin design is proven: in the studied cases 5-15% reduction of the overall steel structural costs can be achieved. It is also shown that although dissipative design does not necessarily yield to cost saving of the steel superstructure in case of relatively light structures, foundation costs may be drastically decreased.

The developed analysis and optimization tool is further extended and applied to fire loading. Details of the research development and the results of the above frames are available in (Dunai et al, 2015).

ACKNOWLEDGEMENT

This paper was supported by the *János Bolyai Research Scholarship of the Hungarian Academy of Sciences*. The presented results are part of the “*HighPerFrame*” R&D project Nr. GOP-1.1.1-11-2012-0568, supported by the *Új Széchenyi Terv*.

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