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Cost Optimization of Concentric Braced Steel Building Structures

T. Balogh and L. G. Vigh

Abstract**—**Seismic design may require non-conventional concept, due to the fact that the stiffness and layout of the structure have a great effect on the overall structural behaviour, on the seismic load intensity as well as on the internal force distribution. To find an economical and optimal structural configuration the key issue is the optimal design of the lateral load resisting system. This paper focuses on the optimal design of regular, concentric braced frame (CBF) multi-storey steel building structures. The optimal configurations are determined by a numerical method using genetic algorithm approach, developed by the authors. Aim is to find structural configurations with minimum structural cost. The design constraints of objective function are assigned in accordance with Eurocode 3 and Eurocode 8 guidelines. In this paper the results are presented for various building geometries, different seismic intensities, and levels of energy dissipation.

Keywords**—**Dissipative Structures, Genetic Algorithm, Seismic Effects, Structural Optimization.

I. Introduction

THE cost is an important component of the structural design. Engineer's aim is to design economical buildings providing sufficient safety against collapse. In the conceptual design phase selecting the most economical structural configuration or bracing system topology, internal forces and minimal cross section sizes need to be estimated. Approximate methods can often be successfully applied with sufficient precision. Commercial – usually finite element analysis based – design softwares can be also invoked; however, this normally leads to an iterative try-and-error process to find suitable solution. Advanced optimization methods can be effective tool for engineers to find the most cost-effective solution for a specific design situation or new, previously unknown structural solutions.

Research on structural optimization is a developing area. A number of books, journals and conferences are related to this topic. In the literature many studies can be found on the optimization of various structures. For example, Chen and Rajan [1] optimize a roof truss and rigid moment resistant frame for weight using genetic algorithm. Jármai et al. [2] presents a detailed cost calculation method for steel structures and optimizes a multi-storey steel frame for structural costs considering seismic effects. The structural costs contain the fabrication costs of the connections. Hayalioglu and Degertekin in [3] study genetic algorithm based cost optimization of steel frame structures with semi-rigid connections. Further examples can be mentioned: cost optimization of industrial frame structures in Kravanja et al. [4], cost optimization of a welded box beam and stiffened plate in Jármai et al. [5] and optimization of three-dimensional truss structures in Kaveh et al. [6].

Developing proper design concept is essential in extreme design situations such as seismic design situation due to the extreme consequences and due to the fact that the load intensity and distribution depends on the structural response. In case of multi-storey concentric braced steel structures the most important variables are the type, number stiffness and topology of the bracing system. Furthermore, the seismic loads can be effectively reduced by dissipative design. In dissipative seismic design the energy absorbing capacity of the structure is provided by concentrating ductility in plastic hinges. The ductile behaviour need to be ensured in material, cross-section, element and global level. From the dissipative design point of view the structural and bracing system configuration (type, topology, stiffness, level of energy dissipation) are also very important.

To define the most suitable configuration using try and error method can be difficult and time-consuming because of the large number of variables and the nonlinearity of the structural behaviour (material and geometrical nonlinearity, loss of stability, etc.), the seismic loads (response spectra) as well as the design constraints (resistance of centrically and eccentrically loaded columns, limited damage criteria, global stability, capacity design rules, etc.).

The purpose of the research presented in this paper is to develop an optimization algorithm for building structures subjected to seismic effects. This paper discusses a genetic algorithm based optimization method and summarizes the results of optimization of multi-storey steel buildings.

II. Optimization Problem

A. Investigated Structures

In this paper multi-storey, regular, concentric braced steel frame (CBF) structures (Fig. 1) are studied. The function of the investigated structures is domestic area. The vertical loads are transferred to the steel structure by reinforced concrete slabs. The design constraints are derived from the design

T. Balogh is PhD Student at the Department of Structural Engineering of the Budapest University of Technology and Economics, Budapest, H-1111 Műegyetem rkp 3-9., Hungary (phone: +36-1-463-17-38; e-mail: baloghtamas@epito.bme.hu).

L. G. Vigh is Associate Professor at the Department of Structural Engineering of the Budapest University of Technology and Economics, Budapest, H-1111 Műegyetem rkp 3-9., Hungary (phone: +36-1-463-17-79; e-mail: geri@vbt.bme.hu).

criteria (ultimate limit state, serviceability limit state and seismic limit state checks) according to MSZ EN 1993-1-1:2009 (EC3-1-1, [7]) and MSZ EN 1998-1:2008 (EC8-1, [8]). Due to the discussed specialities of the problem a simple cross-section optimization of structural elements for costs is not enough, the process shall cover bracing topology and layout optimization.

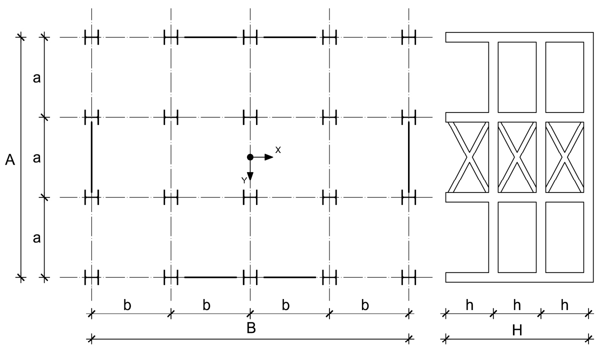


Fig. 1 Schematic plan of the studied structures

The presented research incorporates the study of different building configurations therefore the optimization process is performed by various building cases. As the Table I shows, the effects of building's height are studied by using three different storey numbers. The optimal solutions are determined by two different level of energy dissipation capacity (quasi-elastic and DCM – Ductility Class Medium – structures with q=1.5 and q=4.0 behavior factors) to compare the elastic and dissipative design cases. This research is extended to different seismicity areas assuming three different peak ground accelerations.

B. Parameters

During the optimization, the geometric parameters of height (h), number of stories (n_s), bay widths (a, b) and number of bays in each direction (n_a, n_b) are fixed. In addition, the steel grade, the gravity and wind loads, the seismic intensity (peak ground acceleration, behaviour factor, ground type, response spectra) and the bracing type (elastic or dissipative concentric X-bracing) are also invariant.

C. Variables

The following variables are considered as optimization variables: the cross section of columns (European HEA, HEB, HEM profiles are considered), girders (European IPE sections are considered) and bracing (any cross-section type) as well as the bracing layout (number of braced bays, bracing configuration).

III. Problem Formulation

A. Objective Function

Aim is to find the most economical configuration resulting minimum structural cost. The objective function shall represent the structural cost; the most economical configuration is adjusted to the global optima of the objective function. The optimization problem can be described as follows:

$$\begin{aligned} &min\ K(\mathbf{x}) \\ &g_i(\mathbf{x}) \le 1.0 \quad i = 1...\mathrm{ng} \end{aligned} \tag{1}$$

where $K(\mathbf{x})$ is the cost function, $\mathbf{x}$ is the variable matrix. The elements of $\mathbf{x}$ refer to indexes adjusted with the feasible profiles stored in separate vectors. The $g_i(\mathbf{x})$ are the inequality design constraints, ng is the number of the design constraints. The cost of the structure is calculated in the following way:

$$K = K_{\mathrm{fo}} + K_{\mathrm{mat}} + K_{\mathrm{fab}} \tag{2}$$

where K_{fo} is the cost of foundation (including the cost of excavation, material and construction of simple pad footing and piling). The cost of hot-rolled sections, plates, stiffener plates and bolts are covered by the material cost of the steel structure (K_{mat}). The K_{fab} is the fabrication cost of the steel structure including the costs of preparation, cutting, welding, painting and drilling of bolt holes. The material costs are based on commercial prices. The fabrication costs are defined by literature recommendations [5] (see Section IV).

Inequality design constraints correspond to design criteria/checks required by EC3-1-1 and EC8-1:

- Ultimate Limit State checking of elements, including strength, global and local stability checks,
- Serviceability Limit State checking: girder deflection, building sway deformation under characteristic load combination,
- Seismic strength check of non-dissipative elements, including overstrength (for 475-year return period seismic event),
- Dissipative element design and capacity design: bracing member check, local ductility of brace members, ensuring global mechanism (for 475-year return period seismic event),
- Inclusion or limitation of P-Δ effects at seismic event,
- Limited damage criteria (for 95-year return period seismic event).

Typically, a utilization factor expressing the ratio of design demand value to capacity (e.g. design bending moment to bending resistance) is defined for each design check. The utilization factor of any elements shall not exceed 1.0.

$$\begin{aligned} &\frac{N_{\mathrm{Ed},i}}{\chi_{\mathrm{y},i}\dfrac{N_{\mathrm{Rk},i}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{yy},i}\frac{M_{\mathrm{y,Ed},i}}{\chi_{\mathrm{LT},i}\dfrac{M_{\mathrm{y,Rk},i}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{yz},i}\frac{M_{\mathrm{z,Ed},i}}{\dfrac{M_{\mathrm{z,Rk},i}}{\gamma_{\mathrm{M1}}}} \le 1.0 \\ &\frac{N_{\mathrm{Ed},i}}{\chi_{\mathrm{z},i}\dfrac{N_{\mathrm{Rk},i}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{zy},i}\frac{M_{\mathrm{y,Ed},i}}{\chi_{\mathrm{LT}.i}\dfrac{M_{\mathrm{y,Rk},i}}{\gamma_{\mathrm{M1}}}} + k_{\mathrm{zz},i}\frac{M_{\mathrm{z,Ed},i}}{\dfrac{M_{\mathrm{z,Rk},i}}{\gamma_{\mathrm{M1}}}} \le 1.0 \end{aligned} \tag{3}$$

where $N_{\mathrm{Ed},i}$, $M_{\mathrm{y,Ed},i}$, $M_{\mathrm{z,Ed},i}$ are the design axial force and the design bending moments in the ith column. As an example, stability check of an eccentrically loaded column can be formulated as (3). In case of the Serviceability Limit State checks the inequality constraints are written:

$$w_i \le w_{\lim,i} \quad i = 1...\mathrm{nb} \tag{4}$$

$$u_j \le u_{\lim,j} \quad j = 1...\mathrm{ns} \tag{5}$$

$$u \le u_{\lim} \tag{6}$$

where w_i is the deflection of the ith girder, nb is the number of girders, u_j is the interstorey drift of the jth storey and ns is the number of storeys. The u is the overall horizontal displacement over the building height (shall not exceed H/500, where H is the total height of the building). As per the limited damage criteria the interstorey drift ratios need to be smaller than 0.5% (non-structural elements of brittle materials attached to the structure) for 95-year return period earthquake event:

$$d_{\mathrm{eDL},j} = q_{\mathrm{d}} \cdot d_j \le 0.005h \quad j = 1...\mathrm{ns} \tag{7}$$

where d_j is the jth storey relative drift based on the design response spectra and q_{d} is the displacement behaviour factor.

IV. Cost Calculation

There are many examples for structural mass optimization in the literature, however more and more researcher uses cost objective function by solving an optimization problem [2], [3], [4], [5]. In this study the cost of the structure is calculated by summarizing the foundation cost and the cost of the steel structure. The structure cost incorporates the price of the hot-rolled members, plates, connections and the fabrication costs, but the transportation, erection, maintains and amortization costs are ignored in this calculation. The cost of a steel member can be calculated based on the recommendations of Jármai and Farkas [5] in the following way:

$$K = K_{\mathrm{mat}} + K_{\mathrm{fab}} = k_{\mathrm{m}} \cdot \rho \cdot A \cdot l + k_{\mathrm{f}} \cdot \sum_i T_i \tag{8}$$

where K_{mat} is the material cost, K_{fab} is the fabrication costs. The k_{m} and k_{f} constants are representing the material and fabrication cost factor, ρ is the density, A is the cross-section area and l is the length of the element. T_i represents the time the ith manufacturing process (assembly, welding, drilling bolt holes, painting, etc.). For example the welding time including additional fabrication times can be calculated as

$$T_2 + T_3 = 1.3 \sum_i C_{2i}\, a_{\mathrm{w},i}^{1.5}\, L_{\mathrm{w},i} \tag{9}$$

where C_{2i} constants consider the different welding technologies, $a_{\mathrm{w},i}$ is the weld size and $L_{\mathrm{w},i}$ is the weld length. The constants and difficulty parameters are defined on the basis of Jármai and Farkas [5]. For further details refer to Jármai et al. [5] and [10].

The value of the k_m material cost factor is defined on available common commercial prices. The k_{f} factor is highly dependent on the economy environment of the different countries and regions. Some recommendations for the value of fabrication cost factor can be found in [10]. In this study the factor k_{f} is defined as 0.7 \$/min considering Hungarian circumstances.

Contribution of the foundation cost to the overall building cost is usually significant. The loads on the foundation also depend on the stiffness, configuration and the level of energy dissipation capacity of the structure. For the studied structures pad and pile foundation types are considered. Where a bracing is connected to the base, pile foundation is typically chosen due to the significant shear forces. In other cases, simple pad foundation is normally sufficient. The resistance of the foundation is calculated assuming soil type of medium-dense sand soil (C ground type for response spectra). The cost of foundation includes the cost of excavation, material and construction of pad footing and piling. The cost factors were defined by consultation with Hungarian geotechnical engineers, and thus represent Hungarian circumstances.

V. Numerical Algorithm

A. Algorithm Components

The optimality problem is discrete, non-convex and highly nonlinear due to the large number of variables, the nonlinearity of structural behaviour, seismic loads and design constraints. Furthermore the characteristic of the problem implies the existence of local optima. For the optimization problem a numerical algorithm is developed in MATLAB [11] using an effective heuristic search method which can handle the discussed specialities. The algorithm incorporates the following modules:

- simplified global static (linear static) and seismic (modal response spectra) structural analysis,
- design checks including resistance verifications, serviceability checks and capacity design checks,
- fitness function evaluation,
- optimization framework: genetic algorithm.

B. Optimization Algorithm

A number of heuristic optimization methods can handle the specialties of objective function (high degree of nonlinearity, discrete, non-convex, large number of local optimum, etc.) for example genetic algorithm, particle swarm optimization, harmony search method. These methods do not guarantee finding the global optima, but the solutions can be very close to the optimum with appropriate settings. This makes them suitable for solving practical optimization problems and finding nearly optimal structural configurations.

In this study genetic algorithm based optimization is used because of its beneficial characteristics: the genetic algorithm is able to handle high degree of non-linearity, different optimal solutions in parallel and discrete objective functions, can scan a very large search space and its operation is stable. Its applicability to similar problems is confirmed by examples from the published literature [1], [3]. The genetic algorithm is a widely used heuristic optimum search method. It essentially imitates the biological evolution, thus most of the technical terms are biological. This method is excellent for non-linear and discrete design problems, especially in case large number of local optimum exists.

At the beginning of the searching process the algorithm generates a so-called initial population with a certain number of individuals which are different structural configurations. In this study to improve and accelerate convergence of the optimization process the initial population is constrainedly chosen as to include feasible solutions only, instead of full random generation. The method incorporates preliminary analysis for the given topology as well as design of girders subjected to pure bending and columns subjected to axial compression (the resistance is limited to 60% with respect buckling problems). Starting from the initial population (which is usually randomly created) the genetic algorithm is seeking the solution by changing genotype (genetic makeup, properties) of the individuals. The genotypes are stored in bit sequences, chromosome-like data structures (vectors) or matrices (in special cases). Any of the individuals can be the optimal solution, the algorithm handles them simultaneously and improves them with genetic operators (recombination, mutation) through the iteration steps (generations). So this searching method not only relies on coincidence, as it gradually improves the individuals of the search space. In this study integer coding is used for the variable representation, the integer variables are stored in three dimensional matrices instead of chromosome-like data structure (vector), physically indicating the member location in space. Due to the integer coded variables the objective function is interpreted just in discrete points of the search space.

When number of braced bays and bracing layout are optimization variables so-called multi population method is applied due to the fact that the mutation and recombination of structures with different bracing system configuration is not practical. For further details refer to [11].

C. Constraints Handling

In the present study a genetic algorithm based optimization method was used. Because of the genetic algorithm is not capable to handle constrained optimization problems, the objective function need to be transformed in unconstrained format by using penalty function. For this purpose static, dynamic and adaptive penalty function can be used. In this paper a dynamic penalized objective function (10) is presented. Using dynamic penalization the penalty is increasing along the iteration. In the beginning of the optimization process the algorithm can handle various solutions from search space. Towards the end of the optimization the unfeasible configurations are neglected because of the large penalties. The used dynamic penalty function (11) is based on the Joines and Houck's method [9]. The unconstrained objective function and the optimization problem can be written in the following form:

$$min\ F(\mathbf{x}) = K(\mathbf{x}) + P(\mathbf{x}, q) \tag{10}$$

where $F(\mathbf{x})$ is the "new" objective function (so-called fitness function), $K(\mathbf{x})$ is the cost function and the $P(\mathbf{x},q)$ the penalty function. Accordingly, individuals are not ranked directly on the basis of structural cost. Fitness value is calculated for each individual, using the following penalty function:

$$\begin{aligned} P(\mathbf{x},q) &= \sum_{n}\sum_{m} r_{\mathrm{q},n}^{\alpha}(\mathbf{x},q)\cdot d_{n,m}^{\beta}(\mathbf{x}) \\ r_{\mathrm{q},n}(\mathbf{x},q) &= K_n(\mathbf{x})\cdot \mathrm{c}\cdot q \\ d_{n,m}(\mathbf{x}) &= \begin{cases} 0 & \text{if } g_{n,m}(\mathbf{x}) \text{ is feasible} \\ \Delta g_{n,m}(\mathbf{x}) = g_{n,m}(\mathbf{x}) - 1 & \text{otherwise} \end{cases} \end{aligned} \tag{11}$$

where $K_n(\mathbf{x})$ is the cost of the nth member, q is the number of generation (the number of current iteration step) and c is a constant. The value of c is defined by parametric analysis. In this study c is equal 2, the α and β parameters are set to 1, meaning linear penalization.

D. Simplified Structural Analysis

For the structural analysis, finite element method is applied. Floor slab is considered rigid in its plane, resulting in rigid diaphragm action. Due to the rigid diaphragm action and the building regularity, the three-dimensional problem can be transformed to two-dimensional problem by using approximate numerical model (Fig. 2 (a)). To reduce the size of stiffness matrix and thus reducing the computational demand in modal analysis, the model is further transformed to an MDOF beam model (Fig. 2 (b)) [12].

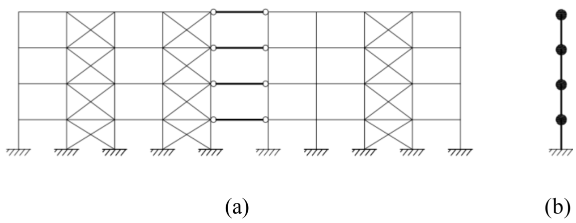


Fig. 2 Numerical model (a) approximate 2D model (b) MDOF beam model

Internal forces and deformations for ultimate limit state checks are calculated by linear static analysis. Springs representing the foundation stiffness are assumed to be relatively rigid. Modal response spectrum analysis is used for the seismic load calculation. Combination of modal responses and combination of *X*- and *Y*-direction seismic actions are completed by the SRSS and 30% rules, respectively.

Design checks are completed in accordance with standards EC3-1-1 and EC8-1. Design of floor beams and girders is typically governed by dominant vertical bending computed in ULS. Column design check is normally controlled by flexural and lateral torsional buckling verifications. Critical buckling load for global buckling (used in ULS verifications) is determined by the approximate formula recommended by

EC3-1-1:

$$\alpha_{\text{cr}} = \left(\frac{H_{\text{Ed}}}{V_{\text{Ed}}}\right)\left(\frac{h}{\delta_{\text{H,Ed}}}\right) \tag{12}$$

where H_{Ed} is the design value of shear force at the bottom of level, V_{Ed} is the design value of vertical load, $\delta_{\text{H,Ed}}$ is the drift and h is the storey height. Using the critical buckling load factor (α_{cr}) the buckling length L_{cr} and the relative slenderness of column members can be calculated in the following way:

$$L_{\text{cr}} = \lambda \cdot \text{i} = \lambda_1 \cdot \overline{\lambda} \cdot \text{i} \quad \overline{\lambda} = \sqrt{\frac{\alpha_{\text{ult,k}}}{\alpha_{\text{cr}}}} \quad \text{és} \quad \lambda_1 = \pi\sqrt{\frac{\text{E}}{\text{f}_\text{y}}} \tag{12}$$

where λ is the slenderness, i is the radius of inertia, $\alpha_{\text{ult,k}}$ is the minimum load amplifier of the design loads to reach the characteristic resistance of the structural component, E is the modulus of elasticity and f_y is the yield strength. The ULS verification of the bracing members includes strength and stability checks where relevant (i.e. it shall be completed for conventional bracing, but irrelevant for slender conventional bracing and buckling restrained braces).

Global instability phenomena are covered by P-Δ effects, while local (in-storey) buckling of column members shall be separately checked. In case of middle and high ductility class (DCM and DCH, respectively) dissipative bracing elements are checked according to capacity design rules. Overstrength in design of non-dissipative members (columns, beams, connections, foundations) is considered in accordance to EC8-1 (Fig. 3) preventing non-desired failure of non-dissipative elements in advance to formation of plastic mechanism. In case of the concentric braced, dissipative frame structures the tensioned bracing bars are the plastic hinges, where the plastic deformations are allowed.

TABLE I

STRUCTURAL PROPERTIES AND PARAMETERS

Material	Grade
Steel grade	S235
Loads	**Intensity**
Dead load (g_k)	6.5 kN/m^2
Live load (q_k)	2.0 kN/m^2
Geometrical parameters	
Storey height (h)	4 *m*
Bay width (b)	6 *m*
Frame width (a)	6 *m*
Number of storeys (n_s)	2 – 4 – 6
Number of bays (n_x)	5
Number of frames (n_y+*1*)	6
Seismic and other parameters	
Peak ground acceleration (a_{gR})	0.08g – 0.15g – 0.3g
Importance factor (γ_I)	1.0
Behaviour factor (q)	1.5 – 4.0
Soil type	*C*
Response spectra	Type I
Damping ratio	5%
Terrain category (for wind load)	I
Altitude above sea level (for snow load)	400 *m*

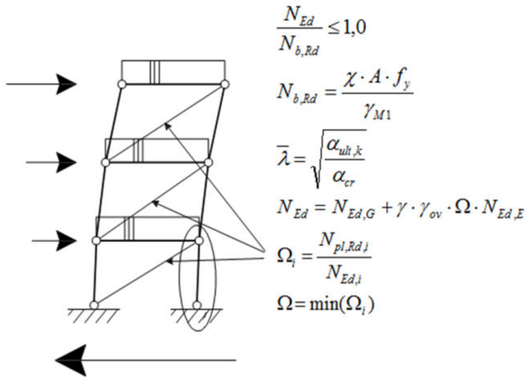


Fig. 3 Seismic design check of non-dissipative elements: calculation of overstrength

Where $N_{\text{Ed,G}}$ and $N_{\text{Ed,E}}$ are the design axial forces in the column from the non-seismic actions included in the combination of actions for the seismic design situation and from the design seismic action. $N_{\text{Ed},i}$ is the design tension force in the ith bracing element from the design seismic action. The $N_{\text{pl,Rd},i}$ is the design value of yield resistance in tension of the gross cross-section of ith bracing member. The γ and γ_{ov} are a partial factor and the material overstrength factor.

In our calculations the accidental eccentricity is indirectly taken into account by factoring the obtained internal forces in accordance to (13) [12].

$$F = F_{\text{EQ}}\frac{1}{n}\left(1 + 0.3\frac{L}{l_\text{y}}\frac{n-1}{n+1}\right) \tag{13}$$

where F is the load on the outer bracing elements from seismic effects, F_{EQ} is the total earthquake load; n is the number of braced frames; L is the width of the building; l_y is the distance between the outer braced frames.

VI. PARAMETRIC STUDY: PROGRAMME

In the current study, optimal configuration of domestic building structures is investigated in the framework of parametric study. Based on the parametric study, optimal topology of bracing elements, structural costs, and the effect of storey numbers, seismicity, stiffness and energy dissipation capacity are analyzed. Parameters describing the investigated cases are illustrated in Fig. 1 and listed in Table I. Altogether 18 cases are optimized. Building plan geometry and certain load and strength parameters are kept constant among the various cases. Storey numbers, level of energy dissipation (expressed in terms of behaviour factor analogous to the response modification factor) and seismic intensity (expressed in terms of peak ground acceleration) are varied. Two different bracing types are considered: conventional "quasi-elastic" bracing (behaviour factor of q=1.5) and concentric slender X-braced frame (CBF, q=4). Beam-to-column connections are rigid in the "main" direction (X) and hinged in direction Y. Column base connections are also considered hinged.

VII. Results of the Parametric Study

A. Bracing System Layouts

In case of concentrically braced frame structures the shear forces from the horizontal loads (wind and seismic effects) are transferred to the foundation by the bracing elements. According to engineering considerations, where the seismic loads are lower (lower storey number or seismicity), less number of bracings (two in each direction) may be sufficient for the studied structure. Increasing the seismicity more bracing elements (four or six in each direction) need to be used.

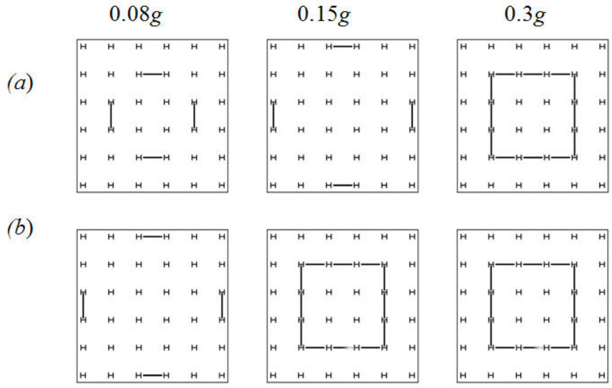
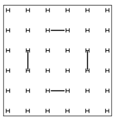
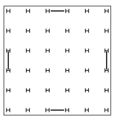
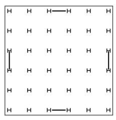
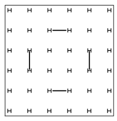
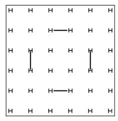
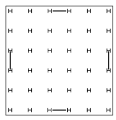
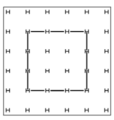
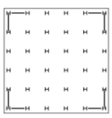
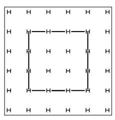
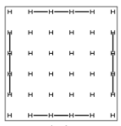
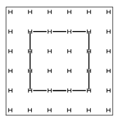
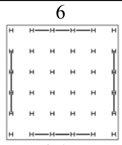


Fig. 4 Bracing systems topologies (q=1.5) (a) two-storey high building (b) four storey high building

TABLE II
Various Bracing System Layouts of Quasi-Elastic (q=1.5) Structures

Storey number	2	2	4	4	6	6
Topology						
Peak ground acc. (g)	0.08	0.08	0.08	0.08	0.08	0.08
Total cost (€)	95 570	96 030	190 230	191 230	290 670	292 570
Foundation cost (€)	16 070	17 470	23 600	23 330	32 130	31 930
Steel structure cost (€)	75 740	74 790	159 830	160 470	247 670	248 930
Bracing cost (€)	3 760	3 770	6 800	7 430	10 870	11 690
Storey number	2	2	4	4	6	6
Topology						
Peak ground acc. (g)	0.3	0.3	0.3	0.3	0.15	0.15
Total cost (€)	109 230	111 970	241 530	244 700	334 570	340 470
Foundation cost (€)	20 030	23 330	33 030	39 560	37 730	42 140
Steel structure cost (€)	75 430	77 600	165 200	164 570	256 370	260 330
Bracing cost (€)	13 770	11 030	43 300	40 630	40 470	38 000

To use four bracing elements is not necessarily beneficial by the studied ground-plan because the global bending stiffness is much lower than using six bracings, thus the axial forces in the columns and on the foundations are grown. Our results confirm the above-discussed engineering expectations. As an example, Fig. 4 illustrates the results of optimal bracing system layouts for two- and four-storey, quasi-elastic buildings at different peak ground accelerations.

It might be surprising that in some of the optimal solutions, bracing is not located at the perimeter, but at the adjacent inner frames (Fig. 4), which is seemingly in contradiction with the expectations that perimeter bracing is beneficial due to the larger torsional rigidity provided. This "mutation" of the optimum solution is resulted by the fact that normally internal column is initially designed with larger section the outer ones due to the larger tributary area associated with the gravity loads. Where the bracings are located at the adjacent inner frames the internal forces from the gravity loads are higher, thus the increase in section sizes, foundation and the additional costs can be smaller. Furthermore the column slenderness depends on the buckling length and the radius of inertia, therefore, the larger HEA, HEB and HEM sections can be effectively utilized. The higher seismic effects raise the axial forces in the columns and on the foundations. Saving can be earned in the cost of foundation and bracing when a column member or a footing is used in X- and Y-direction simultaneously (Fig. 4).

In cases where the bracings are located at the adjacent inner frames the torsional rigidity of structure is lower. Due to the torsional effects from the 5% accidental eccentricity prescribed by the design code EC8-1, the internal forces arising from the seismic effects increase. Substituting in Eq. (13), the increment in the internal forces is approximately 7%. This equation also shows that placing the bracings at the adjacent inner frames can be beneficial alternative in case of large buildings where the increase in internal forces is relatively small. Note that, however, architectural and functional requirements often govern the selection of bracing layout and position.

Although observable savings (in foundation cost: ~10-15 %) resulted from the proper position of the bracing can be realized in case of high seismicity, this tendency is less observable by lower storey numbers and lower seismic loads. Corresponding results are shown in Table II summarizing the first- and second best solutions.

B. Energy Dissipation Capacity

Seismic loads can be drastically reduced by improving the energy absorbing capacity of the structure, applying dissipative structural systems. Fig. 5 compares the calculated base shear forces for conventional bracing and CBF systems of various building heights. The figure confirms the drastic decrease in the resulting seismic effects.

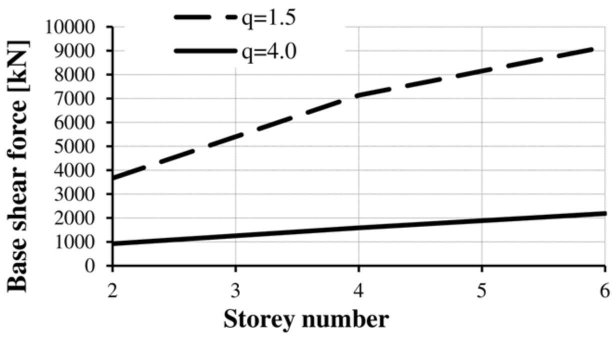


Fig. 5 Base shear forces (0.15*g*)

The source of seismic load reduction is two-fold: 1) development of plastic mechanism itself provides global load reduction (expressed by the behavior factor in the design response spectra); 2) reduction of member sizes leads to lower global stiffness, yielding to longer fundamental period of the structure and resulting in smaller design acceleration read from the design response spectra. Because of these two effects, the optimality problem is characterized with great nonlinearity (Fig. 5). It is clearly visible on Fig. 5 that the rate of change in seismic load intensity exceeds the quotient of behavior factors.

Fig. 6 shows the total structural costs of the optimized buildings as a function of peak ground acceleration. Through the decrease in seismic loads, significant savings can be realized; the major sources of the cost saving is the decrease of the bracing and foundation costs.

(a)

(b)

(c)

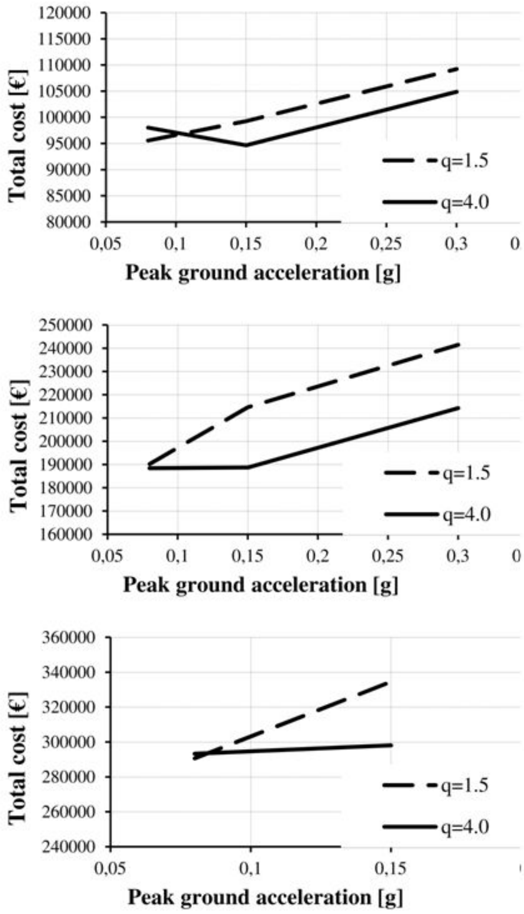


Fig. 6 Total costs of (a) two- (b) four- (c) six-storey buildings

As expected, remarkable savings can be earned by dissipative seismic design, mostly in high seismicity regions (Fig. 6). However, at lower seismicity, due to the relatively low horizontal loads, the lateral stiffness of the resulting bracing system is relatively small when using dissipative design, and the limited-damage criteria and the second-order effects often govern the design (refer to Section *F*). This observation explains that dissipative and quasi-elastic design concepts results in the same solution. (Note that the European profiles included in the database and considered in the optimization procedure were not sufficient to resist the high seismic loads arised in the case of the six-storey building and thus the algorithm did not converge. In the further research stronger cross-sections will be used.)

C. Savings in Bracing and Footing Costs

The bracing and foundation costs represent a significant portion of the total cost. As Fig.7 illustrates, approximately 60-70% cost saving can be earned on the bracing elements, even at moderate seismicity. If the erection costs were considered, the difference could be even higher.

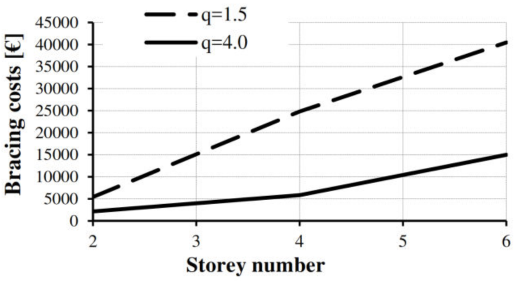


Fig. 7 Bracing costs (0.15*g*)

By reducing the base shear force (Fig. 5), the maximum shear and axial forces at the foundation can be drastically reduced, leading to significant cost savings at the foundation structures and construction works (Fig. 8).

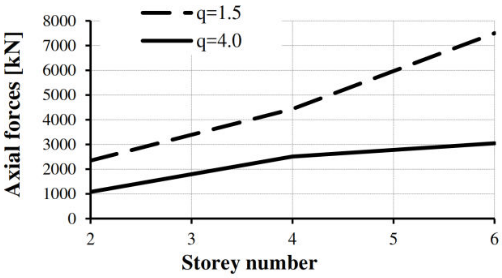


Fig. 8 Maximum axial forces at the foundation (0.15*g*)

D.Building's Height

It is straightforward that the larger seismic mass of taller buildings results in larger global seismic loads. As demonstrated in Table II, in case of six-storey building, bracing in two bays is sufficient in each direction at low seismicity, while six braced bays are necessary in moderate seismicity regions. The relation of height and base shear force, however, is non-linear (Fig. 5), due to the fact that the longer fundamental period of taller buildings (Fig. 9) yields to decreased spectral accelerations.

Overturning bending moment is also increasing with ascending building height, resulting in rise of the axial forces in the columns and on the foundations (Fig. 8), significantly raising the bracing and footing costs. The results also confirm that bracing in adjacent bays is preferable over the separate bays (compare first and second best layouts in Table II). Since overturning bending moment is dominantly resisted by the outer columns as chord members of the braced cantilever, the increased lever arm between these columns of the joint braced bays provides larger resistance with the same cross-section than at separate bracings.

Our results imply that for the studied configurations dissipative design can be economical in high and moderate seismicity regions as well, disregarding the number of storeys. Needless to say, however, the achieved savings are relatively smaller at shorter buildings.

E. Lateral Stiffness

Compared to quasi-elastic design concept, lower lateral stiffness is resulted with dissipative design, in accordance to the lower seismic loads. This is well represented by the comparison of the fundamental periods and deformations of the different cases in Figs. 9 and 10, respectively. Approximately doubled fundamental period is obtained with dissipative structure (Fig. 9), meaning about four times lower lateral stiffness, which is apparent in the calculated deformations, too (Fig. 10).

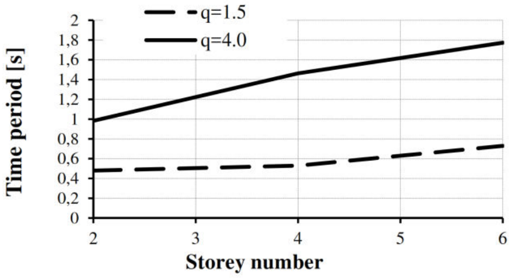


Fig. 9 Fundamental period in *X* direction (0.15*g*)

F. Interstorey Drifts and P-Delta Effects

According to EC8-1, limited damage criterion is checked through the limitation of interstorey drifts calculated for a 95-year return-period earthquake. Limited damage criteria – and similarly, P-Delta effects – may often govern the design of dissipative structures due to their relatively low stiffness, as demonstrated in Fig. 10 for the four-storey building.

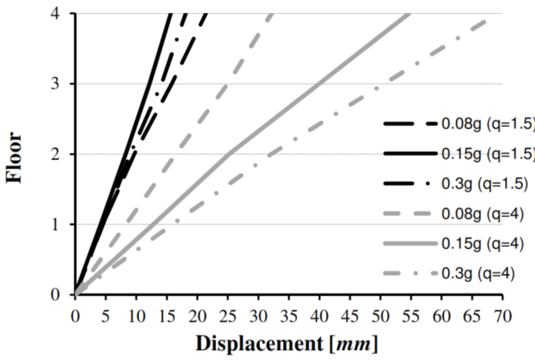


Fig. 10 Deformations of four-storey buildings in *X* direction (95-year return period seismic event)

It is observed that the deformation pattern of the studied buildings can be characterized as global shear deformations (Fig. 10). Although at taller buildings one may expect the increased dominancy of the overturning bending moment, the calculated optimum configurations (Table II) come with relatively wide bracing system, decreasing the bending deformations.

In EC8-1, significance of *P-Δ* effect is expressed in terms of the θ ratio, as follows:

$$\theta = \frac{P_{\text{tot}} \cdot d_{\text{r}}}{V_{\text{tot}} \cdot h} \qquad (14)$$

where P_{tot} is the total gravity load at and above the storey considered in the seismic situation, d_{r} is the design interstorey drift (at the calculation of deformations the response spectra shall not be reduced with behaviour factor), V_{tot} is the total seismic storey shear and h is the interstorey height. According to the code, the factor θ cannot exceed 0.3. In the range of 0.1 and 0.2, second-order effects can be taken into account by simply factoring the horizontal loads [8]. In our study, in order to simplify the numerical calculations, the factor θ was limited to 0.2.

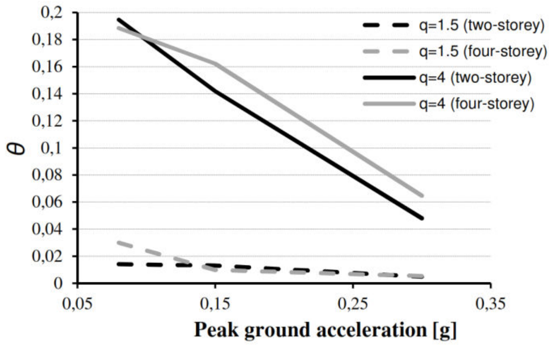


Fig. 11 Second order effects in *Y* direction (two- and four-storey building)

For the optimum configurations of two- and four-storey buildings, Fig. 11 compares the calculated θ factor in relation to the design ground acceleration and behavior factor. According to the expectations, it is found that significance of P-Δ effects is increased in case of dissipative structures (Fig. 11.). With ascending seismic intensity, second-order effects become smaller because of the higher lateral stiffness. The figure well confirms that second-order effects may govern the design at low seismicity when applying dissipative structure.

G. Overstrength

In order to prevent premature failure of the non-dissipative members, overstrength design is necessary in the case of $q = 4$. In EC8-1, overstrength applied to the seismic internal forces calculated from the modal response spectra analysis is normally expressed by the product $1.1 \cdot \gamma_{ov} \cdot \Omega$, where γ_{ov} is the ratio of the actual mean value and design value of the yield strength (as per EC8-1, it can be assumed as 1.25), and Ω is the minimum of the strength to calculated internal load ratios of the dissipative members.

The calculated Ω factors can be seen in Fig. 12. The value of Ω is relatively high in case of low seismicity. Although the lower calculated seismic load would allow smaller cross-sections for the bracing members of the lateral load resisting system, it is highly influenced by the limited damage criteria and the high second-order effects. As a result, dissipative members cannot be highly utilized.

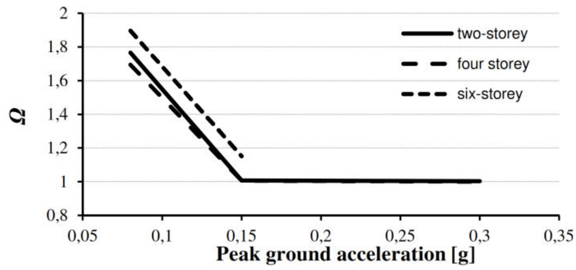


Fig. 12 Calculated Ω factors in *X* direction (two-, four- and six-storey building)

The quotient of the behavior factor and the "overstrength factor" ($q/1.1 \cdot \gamma_{ov} \cdot \Omega$ hereafter referred as effective energy dissipation ratio) well characterizes the effectiveness of the dissipative system, i.e. one can measure the actual reduction in seismic loads due to the energy dissipation.

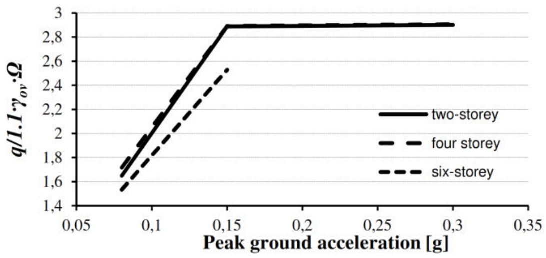


Fig. 13 The quotient of the behavior factor and the overstrength factor in X direction (two-, four- and six-storey building)

As Fig. 13 confirms by lower seismicity the effective energy dissipation ratios are relatively low (near to the quasi-elastic design) due to the low utilization of the bracing elements. It can be thus concluded that among the studied buildings benefits of dissipative design may not be realized in low seismicity regions because of the high overstrength factors.

VIII. Conclusions

Parametric study on optimum structural configurations of regular, concentric braced, multi-storey steel building structures is discussed in this paper varying the storey numbers, seismic intensities and energy dissipation capacities. Through the genetic algorithm based optimization process a dynamic penalized cost objective function was used.

The structural cost, optimal bracing system layout and the lateral stiffness of the optimized structures are analyzed. Furthermore the interstorey drifts, P-Δ effects and the effect of the building's height and energy dissipation capacity are also discussed. The following conclusions can be drawn related to the results:

- In most of the investigated cases the optimal location of the bracings is at the adjacent inner frames. However normally the second best solution provides similarly good solution.
- The bracing and foundation costs represent a significant

portion of the total cost, thus these cost components need to be considered by a cost optimization.
- Through the decrease in seismic loads due to the dissipative design significant savings can be realized. The major sources of the cost saving is the decrease of the bracing and foundation costs.
- Comparing different energy dissipation levels the rate of change in seismic load intensity exceeds the quotient of behaviour factors.
- At lower seismicity the limited-damage criteria and the second-order effects may often govern the design in the dissipative seismic design due to the relatively low horizontal loads and lateral stiffness.
- The benefits of dissipative design may not be realized in low seismicity regions because of the high overstrength factors resulted by the governing limited-damage criteria and second-order effects.
- Our results imply that for the studied configurations dissipative design can be economical in moderate seismicity regions (saving in bracing cost is ~60-80%, in total cost ~15%) as well by different number of storeys, although the achieved savings are relatively smaller at shorter buildings.
- Based on the results can be declared that the developed algorithm is numerically stable and suitable for cross-section and bracing system topology optimization of multi-storey, concentric braced steel building structures.

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