Genetic algorithm based optimization of regular steel building structures subjected to seismic effects

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SUMMARY
In either elastic or dissipative seismic design of structures, key issue is the optimal design of the lateral load resisting system. Beside the basic effort to provide economical structure, proper overall behaviour with global ductility cannot be achieved without optimized design of the seismic resistant system. In this paper, development of a numerical optimization algorithm is presented and – using the developed algorithm – results of structural optimization for various building cases are illustrated. Numerical algorithm is developed in Matlab, incorporating the simplified structural analysis (linear static and modal response spectrum analysis), the resistance and additional checking of the structure and optimization routine. In the study, bracing systems with different level of energy dissipation are used. Aim is find the optimal structural configuration resulting in minimum construction weight, with given building geometry, loading and seismic effects.

Keywords: Structural optimization, Genetic algorithm, Dissipative structures, Regular steel building

1. INTRODUCTION

During the preliminary design of a building one endeavours to design an optimal structure, which combines sufficient safety against collapse and economical configuration. The key issue in optimal structural design is the proper selection of bracing system and its layout. This especially applies for seismic design, because the stiffness and layout of structure affect the overall structural behaviour, the seismic load intensity as well as the internal force distribution. Dissipative design may considerably reduce the seismic effects even in moderate seismicity regions.
Type, number and layout of the bracing system and element member sizes are primary in dissipative seismic design. Optimal design of such system is complicated by the fact that the optimality problem is highly non-linear, as a result of: i) seismic load intensity is dependent on the overall and local stiffness, ductility and layout of the bracing elements; ii) structural behaviour and seismic response is non-linear (material and geometrical nonlinearity, i.e. ductile/dissipative structural elements, loss of stability, P-Delta effect, interaction of various failure modes). iii) design constraints are non-linear (resistance of centrally and eccentrically loaded columns, interaction of global rigidity and seismic loading, limited damage criteria, capacity design rules, global stability problems, etc.).
To achieve optimal solution, analysis shall cover bracing topology and layout optimization. This paper discusses the development of an optimization algorithm using genetic algorithm and simplified seismic analysis procedures. Special care is taken for the optimization parameter adjustment for the genetic algorithm. Using the developed algorithm, illustrative examples of structural optimization of various buildings are also presented.

2. OPTIMIZATION PROBLEM

2.1. Fundamental optimization problem

In this study, regular, centrically braced steel buildings (Fig. 2.1) are analyzed with different level of energy dissipation. The optimal solution is defined as the structure with the minimum weight, satisfying the design check criteria (ultimate limit state, serviceability limit state and seismic checks) according to Eurocode 3 Part 1-1 (EC3-1-1:2009) and Eurocode 8 Part 1 (EC8-1:2008).

At each individual structural optimization problem, fixed parameters are as follows:
- height and number of storeys (h, n_s),
- bay widths (a, b),
- number of bays in each direction (n_a, n_b),
- bracing type: ordinary (elastic) concentric bracing, dissipative X-bracing, BRB (Zsarnóczay et al., 2011),
- steel grade,
- gravity and wind loads,
- seismic weight,
- seismic intensity (peak ground acceleration, behaviour factor, ground type, response spectra).

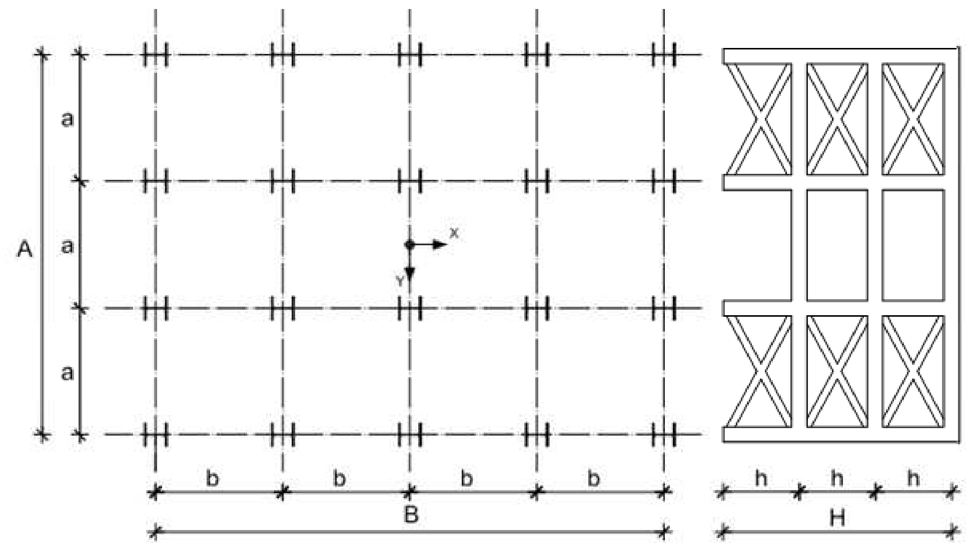


Figure 2.1. Geometric parameters of the examined structure

The optimization variables are discrete variables:
- column member sizes (European HEA, HEB, HEM profiles are considered),
- beam member sizes (European IPE sections are considered),
- bracing member sizes (any cross-section type),
- bracing layout (optional): number of braced bays, bracing configuration.

Because of the discrete variables, integer coding is used for the variable representation, stored in the matrix:

$$\boldsymbol{x} = \left[\boldsymbol{x}^{C} \quad \boldsymbol{x}^{B} \quad \boldsymbol{x}^{\boldsymbol{BR}}\right] \tag{2.1}$$

Elements of $\boldsymbol{x}^{\mathrm{C}}$ (column member profile), in $\boldsymbol{x}^{\mathrm{B}}$ (beam member profiles) and in $\boldsymbol{x}^{\mathrm{BR}}$ (bracing members) refer to indexes adjusted with the feasible profiles stored in separate vectors. The integer variables are stored in three dimensional matrices instead of chromosome-like data structure (vector), physically indicating the member location in space.
The fundamental optimization problem is written in the form:

$$\begin{gathered} \min\ f(\underline{x}), \quad \text{with} \\ 0 \le g_i(\underline{x}); i = 1,2,\ldots,k, \ \text{and} \\ 0 = h_i(\underline{x}); i = k+1,\ldots,m, \end{gathered} \tag{2.2}$$

The object function is the weight of the principal structure, calculated as the sum of the column, beam and bracing weights. Due to the fact that foundation design is always project specific, its cost is not represented in the object function. Similarly, connection detailing and its cost is not considered at this stage of the research.

The design constraints are as follows:

i) Ultimate Limit State checking of elements, including global and local stability checks;
ii) Serviceability Limit State checking: girder deflection, building sway deformation under characteristic load combination;
iii) Seismic strength check of non-dissipative elements, including overstrength; for the calculation of the seismic loads, simplified modal response spectra analysis is invoked;
iv) Dissipative element design and capacity design: bracing member check, local ductility of brace members, ensuring global mechanism;
v) Inclusion or limitation of P-Delta effects at seismic event;
vi) Limited damage criteria (95-year return period).

2.2. Optimization algorithm: Genetic algorithm

Firstly, the fact that the structural design constraints themselves are non-linear (local and global stability checks, seismic load calculation, capacity design, etc.) makes the optimization problem highly non-linear. Secondly, the characteristics of the object function and the large variable number imply the existence of local optima, especially when bracing layout/topology is variable. Thirdly, the problem is further complicated by the fact that the optimization variables are discrete, resulting in discontinuous object function. For such discrete non-linear optimization problem, genetic algorithm that can handle the discussed specialties may be efficiently applied.

Genetic algorithm is an effective, heuristic optimum search method (Goldberg, 1989). The algorithm is developed by J. H. Holland, in United States. During the iterations, it essentially imitates the biological evolution and excellent for non-linear and discrete design problems, especially in case large number of local optimum exists. This searching method not only relies on coincidence, as it gradually improves the individuals of the search space. The properties of individuals are stored in chromosomes which can be bit sequences, vectors or matrices (in special cases). Using the stored information the fitness or fitness rate (measurement of ‘goodness’) of the individuals is calculated. Starting from the initial population (which is usually randomly created) the genetic algorithm is seeking the solution by changing genotypes. In the methodology, iteration step is referred as generation. New specimens for the next generation can be created by recombination or mutation.

For the current study, application of genetic algorithm is beneficial as there is no restriction for the object function, the method is able to handle high degree of non-linearity, can scan a very large search space, its operation is stable and reliable (Naeim, Alimoradi, Pezeshk, 2004). Its applicability to similar problems is confirmed by examples from the published literature, (Chen, Rajan, 2000) (Hayalioghu, Degertekin, 2005).

2.3. Fitness function

The object function is converted into the so-called fitness function typically used for genetic-algorithm based solutions. The fitness rate of individuals is calculated on the basis of the so-called utilization ratio η (Eqn. 2.3.) of structural members:

$$\eta = \frac{E_d}{R_d} \tag{2.3}$$

where E_d is the design value of action effect on structural members; R_d is the design value of the structural member resistance. The $f_i(\boldsymbol{x})$ fitness rate for the i-th member is then calculated as follows

$$
\begin{aligned}
f_i(\boldsymbol{x}) &= (1-\eta_i(\boldsymbol{x}))\cdot(A_i\cdot\rho\cdot l_i)^2 && \text{if } \eta_i(\boldsymbol{x})\le 1{,}0 \\
f_i(\boldsymbol{x}) &= 1e+7 && \text{if } \eta_i(\boldsymbol{x})> 1{,}0
\end{aligned}
\tag{2.4}
$$

where $\eta_i(\boldsymbol{x})$ is the utilization ratio of the *i*-th member , A_i is the cross-section area, ρ is the density of steel (78,5 kN/m^3) and l_i is the length of the *i*-th member.

To prevent unfeasible solutions, global design constraints that cannot be practically adjusted with member utilization rate (e.g. limiting P-Delta effect) are considered by using penalty functions $g_i(\boldsymbol{x})$. If a design constraint that affects the entire building or a whole storey is not satisfied, the fitness rate is increased beyond 1e+7. The fitness function of the whole structure is thus calculated by simple summation:

$$
F(\boldsymbol{x}) = \sum_{i=1}^{n} f_i(\boldsymbol{x}) + \sum_{j=1}^{k} g_j(\boldsymbol{x}) \tag{2.5}
$$

where n is the number of structural members, k is the number of global design constraints.

3. NUMERICAL ALGORITHM

3.1. Overall algorithm structure

Numerical algorithm in Matlab is developed for the optimization problem incorporating:

- 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.

3.2. Simplified structural analysis

3.2.1. Numerical model and 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. Accordingly, the adjacent braced and unbraced frames can be linked in series by using rigid hinged bars (Zalka, 2009) (Fig. 3.1a). The unbraced frames can be substituted by leaning columns. The 2D problem implies beam elements with three degrees of freedom per node. In order to reduce the size of stiffness matrix and thus reducing the computational demand in modal analysis, the model is further transformed to multi-DOF beam model as illustrated in Fig. 3.1b. The conversion is completed on the basis of the storey stiffnesses, (Dulácska, Joó, Kollár, 2008).

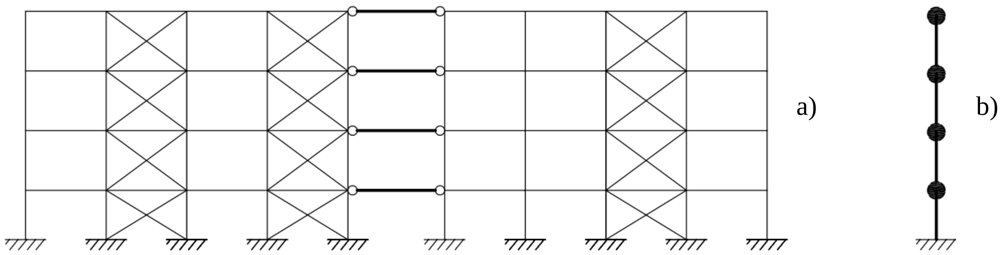


Figure 3.1. Numerical model: a) approximate two-dimensional model; b) multi DOF beam model

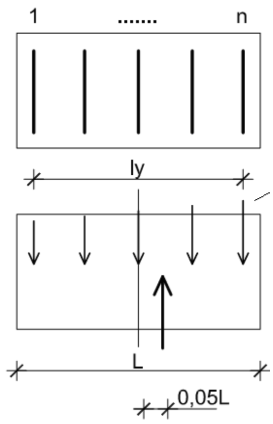


Figure 3.2. Approximation of accidental eccentricity

Internal forces and deformations for ultimate limit state checks are calculated by linear static analysis. 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.

In seismic design, EC8-1 prescribes an accidental eccentricity of seismic masses extending to 5% of the building dimensions. The accidental eccentricity is indirectly taken into account by factoring the obtained internal forces in accordance to (Eqn. 3.1.) (Dulácska, Joó, Kollár, 2008) (Fig. 3.2):

$$\frac{F}{n}\left(1+0{,}3\frac{L}{l_y}\frac{n-1}{n+1}\right) \tag{3.1}$$

where F 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.

Critical buckling load for global buckling (used in ULS verifications) is determined by the approximate formula recommended by EC3-1-1:

$$\alpha_{cr}=\left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{H}{\delta_{H,Ed}}\right) \tag{3.2}$$

where H_{Ed} is the design value of shear force at the bottom of level, V_{Ed} is the design value of vertical load, $\delta_{H,Ed}$ is the drift. The buckling length L_{cr} and the relative slenderness of column members can be obtained as:

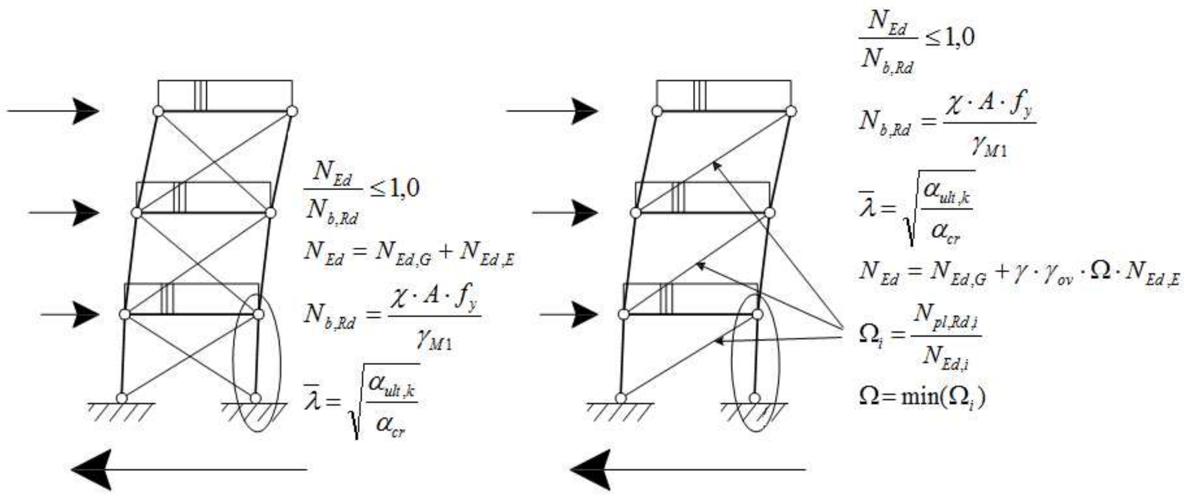
$$L_{cr}=\lambda\cdot i=\lambda_1\cdot\bar{\lambda}\cdot i \qquad \bar{\lambda}=\sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} \text{ and } \lambda_1=\pi\sqrt{\frac{E}{f_y}} \tag{3.3}$$

where λ is the slenderness, i is the radius of inertia, $\alpha_{ult,k}$ 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. In this way, the global buckling check in ULS verifications is handled by the reduction factor method using the computed critical buckling length or relative slenderness.

In seismic design situation, global buckling is considered via the inclusion of P-Delta effects (by calculating the stability index and corresponding load factor); accordingly, the buckling length of column member is equal to the storey height, to consider local buckling.

3.2.2. Member checks

Design checks are completed in accordance to EC3-1-1 and EC8-1. Design of floor girders and beams is typically governed by dominant vertical bending computed in ULS. Column design check is controlled by flexural and lateral torsional buckling verifications. In case of dissipative bracing systems, EC8-1 requires the calculation of overstrength in accordance to Fig. 3.3.



a) elastic structure   b) dissipative structure

Figure 3.3. Seismic design check of non-dissipative elements: columns

3.3. Optimization algorithm

3.3.1. Initial population

In order to improve and accelerate convergence of the optimization process, instead of full random generation, the initial population is constrainedly chosen as to include feasible solutions only. The method incorporates preliminary analysis for the given topology as well as design of beams subjected to pure bending and columns subjected to axial compression. In the latter case, utilization factor for cross-section strength check is limited to 60%, presuming that the member design in this way has sufficient resistance against buckling problems.

3.3.2. Crossover

Due to the nature of the problem, uniform crossover is applied, since the genome of the individuals cannot be represented by bit sequence. The uniform crossover mixes the genes of the parents with fixed – 50% – probability.

3.3.3. Mutation

The following mutations are applied for the individuals (except for the elites) in each generation:

- column section shift by one index (20% of all column members, randomly selected),
- beam section shift by one index (50% of all members, randomly selected),
- bracing member sections shift by one index (one level, randomly selected),
- member section change in accordance its utilization ratio (randomly selected beams or columns).

3.3.4. Testing and calibration of genetic algorithm parameters

The stability and convergence of the iteration is greatly influenced by the following algorithm parameters:

a) population size: number of evaluated individuals at each iteration step,
b) generation number: maximum number of iteration steps,
c) elite ratio: portion of population with “superior” fitness, that survives the generation with no alteration,

d) mutation ratio: number of individuals that are subjected to mutation in each iteration step,
e) recombination ratio: portion of population participating in selection for creating the new population.

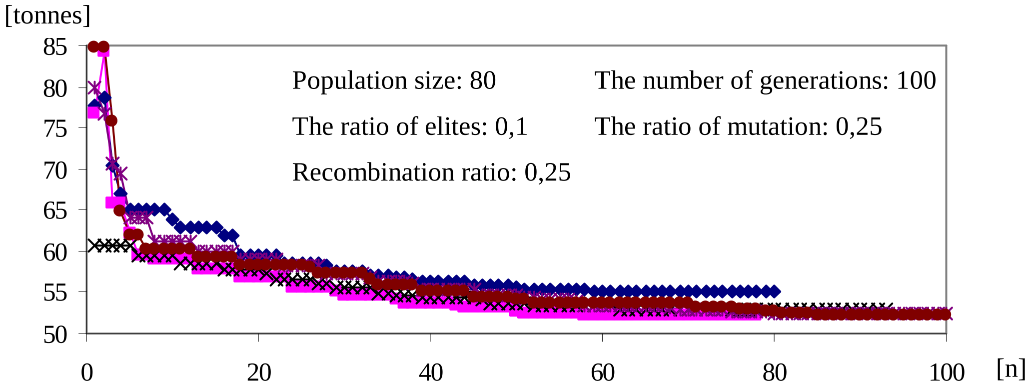


Figure 3.4. The results of parametric study in the last five cases

Calibration and verification of the algorithm and its parameters is completed by parametric study on a test – reference – four-storey building, with fixed bracing layout. Various combinations of the different parameters (population size: 20-40-80; mutation: 0.25-0.5; recombination ratio: 0.25-0.5; elite ratio: 0.1; generations: 100) are evaluated with respect to convergence rate, stability (in optimal solution finding) and computational demands.

As a result of the parametric study, a possible, efficient parameter setting is shown in Fig. 3.4. The convergence of the fitness function confirms functionality and stable operation of the algorithm. Note that population size was limited by the large computational demand and the available computing resources. In order to keep the algorithm efficient, the maximum considered population size is 80 in this study. In further studies, more extensive parameter-testing is necessary to find a more accurate value of mutation and recombination ratio with effective number of individuals.

3.3.5. Multi-population algorithm

When number of braced bays and bracing layout are optimization variables, the algorithm is extended with the so-called multi-population method. In the calibrated routine, the method applies parallel computations on 10 different populations (bracing systems) each having 20 individuals. At every 20th iteration steps the resulted individuals are compared ("competition") and the best alternatives are kept, while the worst 6 configurations are exchanged with new ones. The new bracing layouts are selected randomly, while the member sizes are obtained by crossover of the best individuals. Reaching the 60th generations, only the best two populations are kept.

For further details on the algorithm development, testing and calibration refer to (Balogh, 2011).

4. ILLUSTRATIVE OPTIMIZATION EXAMPLES

4.1. The studied parameter range

The developed algorithm is applied for optimization of several building configurations. In the following sections, the results are demonstrated through illustrative examples. Table 4.1 lists the data of the investigated building structures. The connections of the examined structure are rigid, except for the column base connection. In all cases, the importance factor is 1.0, the viscous damping ratio was 5%, Soil Type C and Type 1 response spectra are assumed.

In each analysis case, self weight, modal masses, the utilization ratios, base shear forces, relative displacements, periods, measurement of P-delta effects and ground forces are evaluated. Major results are discussed in the following. Note that due to the small number of building variations, general conclusions cannot be drawn yet.

Table 4.1. The data of analyzed cases.

	A [m]	a [m]	B [m]	b [m]	H [m]	h [m]	Self weight [kN/m^2]	Function	Acceleration [g]	Behaviour factor
1	30	6	30	6	12	4	6,5	domestic area	0,08	1,5
2	30	6	30	6	12	4	6,5	domestic area	0,15	1,5
3	30	6	30	6	12	4	6,5	domestic area	0,3	1,5
4	30	6	30	6	12	4	6,5	domestic area	0,08	4,0
5	30	6	30	6	12	4	6,5	domestic area	0,15	4,0
6	30	6	30	6	12	4	6,5	domestic area	0,3	4,0
7	30	6	30	6	12	4	6,5	domestic area	0,08	7,0
8	30	6	30	6	12	4	6,5	domestic area	0,15	7,0
9	30	6	30	6	12	4	6,5	domestic area	0,3	7,0
10	30	6	30	6	12	4	8,0	shopping area	0,08	1,5
11	30	6	30	6	12	4	8,0	shopping area	0,15	1,5
12	30	6	30	6	12	4	8,0	shopping area	0,3	1,5
13	30	6	30	6	12	4	8,0	shopping area	0,08	4,0
14	30	6	30	6	12	4	8,0	shopping area	0,15	4,0
15	30	6	30	6	12	4	8,0	shopping area	0,3	4,0
16	30	6	30	6	12	4	8,0	shopping area	0,08	7,0
17	30	6	30	6	12	4	8,0	shopping area	0,15	7,0
18	30	6	30	6	12	4	8,0	shopping area	0,3	7,0

4.2. Result discussion

4.2.1. Bracing system layouts

As expected, the results confirm that smaller seismic intensity comes with less number of braced bays, as illustrated in Fig. 4.1.

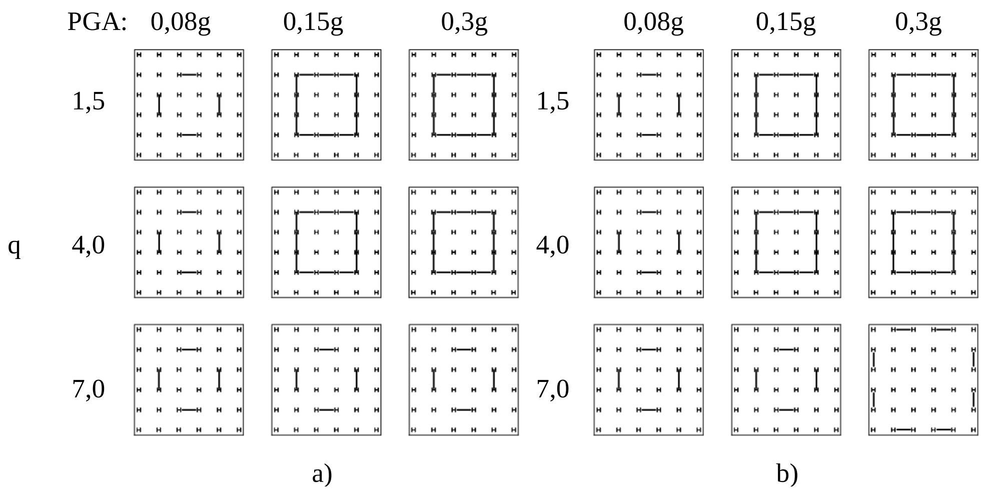


Figure 4.1. Layouts of bracing system: a) Domestic are; b) Shopping area

It might be surprising that in the optimal solutions, bracing is not located at the perimeter, but at the adjacent inner frames (Fig. 4.1.), which is seemingly in contradiction with the expectations that perimeter bracing is beneficial due to the larger torsional rigidity provided. The reason for this phenomenon is that the tributary area of a perimeter column is smaller than that of the inner one, thus leading to smaller required cross-section when considering gravity loads only. Adding the bracing to the perimeter frames would lead to a significant increase of these column sections, while applying inner bracing may less influence the column size.

4.2.2. Seismic load as function of behaviour factor

As typical example, Fig. 4.2. shows the relation of the applied behaviour factor vs. the resulting base shear force-to-seismic weight ratio, at different seismic intensity. As it is expected, seismic effects are efficiently reduced by using dissipative structures, also reducing the overall cost of the structure, especially in seismic-prone regions. Major benefits can be realized regarding cost of foundation and connections. The figure also confirms that application of dissipative design can be economical even in moderate seismicity regions. The results show that the rate of change in seismic load intensity exceeds the quotient of behaviour factors, provided that smaller seismic loads requires softer structure, where the resulting higher fundamental period further reduces the seismic loads.

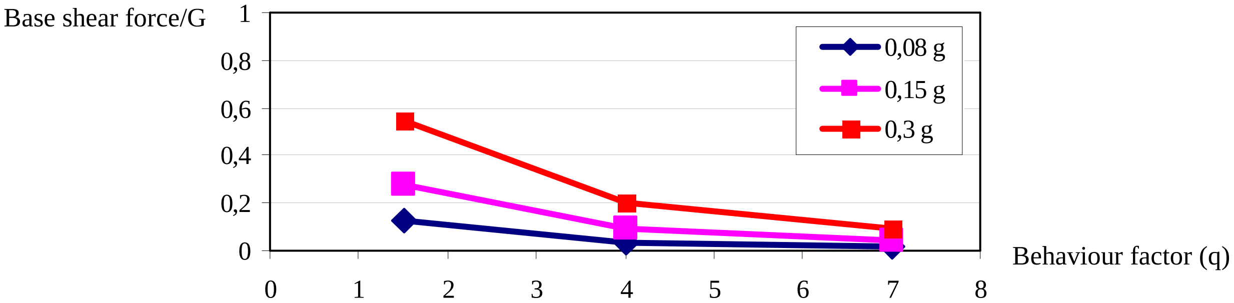


Figure 4.2. The change of base shear forces in case of smaller gravity loads (domestic area)

4.2.3. Quasi-elastic structure

Based on the results, it can be concluded that in case of low dissipative structures it may not necessarily be optimal and may not be crucial to achieve full utilization (utilization factor of 100%) for the bracing elements. In the investigated cases, the effective column buckling length in ULS verifications dominantly depends on the lateral stiffness of the structure, which is defined by the rigidity of bracing elements. The contribution of the bracing members to the overall weight is typically less than that of the columns; therefore it may be more advantageous to improve the lateral stiffness.

5. CONCLUDING REMARKS

In this paper, development of a numerical optimization algorithm for optimal design of regular steel buildings subjected to seismic actions is discussed. Parameters of the genetic algorithm based routine are tested and calibrated by parametric study; and stability and convergence of the developed routine is confirmed. It is found that genetic algorithm is capable for overall structural as well as bracing layout optimization. Competition (multi-population) algorithm is efficient in layout and/or topology optimization.

Through 18 illustrative examples, bracing systems with different level of energy dissipation (elastic concentric braced frame, dissipative concentric braced frame, buckling restrained brace frame) are analyzed and conclusions are drawn with respect to optimal structural configuration. The results confirm that dissipative design can be economical in moderate seismicity regions as well.

For further details on the presented algorithm development, analysis and its results refer to (Balogh, 2011).

ACKNOWLEDGEMENTS

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