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# CAPACITY DESIGN OPTIMIZATION OF STEEL BUILDING FRAMEWORKS THROUGH NONLINEAR DYNAMIC ANALYSIS

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**ABSTRACT:** This paper presents two unified capacity design optimization formulations for various steel building frameworks through adopting a nonlinear response history analysis procedure as the evolution tool for the structures under one-level or multi-level earthquake hazards. The explicit design objectives include minimum weight, minimum earthquake input energy and collapse prevention under an extreme earthquake. The design constraints include local plastic deformation limits on both fuse and non-fuse frame members, global deformation limit on inter-story drifts, and optional constraints on fundamental period and floor accelerations. The building damage under low to moderate earthquakes can be controlled by imposing appropriate constraint limits. The design optimization method employs a multi-objective genetic algorithm to search for optimal member section sizes from among commercially available steel section shapes. The design formulation is illustrated numerically for an eccentrically-braced steel frame.

## 1. Introduction

Current engineering practice, such as *National Building Code of Canada* (NRCC, 2010), adopts elastic analysis procedures to obtain structural demands under earthquake loading. However, building structures are generally designed to undergo large inelastic deformations under a major earthquake. To overcome this inconsistency, seismic provisions generally adopt force reduction factors to account for the ductility and overstrength of a structure. Furthermore, the inelastic deformation of the structure under earthquake loading is estimated through introducing displacement amplification factors. To confine material yielding to fuse members, capacity design principles must be employed. To date, for each type of steel frame, a specific set of rules has been developed to enforce the capacity design principle, such as Clause 27 in CSA/S16-09 (CSA, 2009).

Though it has been quite successful, this elastic-analysis-based seismic design methodology has several shortcomings: 1) an elastic analysis is unable to warrant a fully valid seismic design since it cannot always accurately capture the actual inelastic behavior of a structure (e.g., Tremblay, 2003); 2) capacity design principle is usually implemented by hand calculations since an elastic analysis cannot predict the structural demands for non-fuse members; and 3) design approaches have been problem dependent, since various steel frames have different ductility- and overstrength-related force modification factors.

To overcome the aforementioned shortcomings of an elastic-analysis-based design procedure, two unified optimal capacity design formulations are proposed in this paper through adopting a nonlinear response history (NRH) analysis as the evaluation tool for the structural demands under earthquakes (thus, the so-called inelastic-analysis-based design). The “unified” approaches herein are referred to that one design formulation fits all types of steel frames. Specifically, the approaches are “unified” in the following aspects: 1) material plastification and structural overstrength are directly accounted for in the analysis procedure. Thus, the ductility- and overstrength-related factors, and the classification of steel frames according to ductility capacity, are unnecessary in the design formulations; 2) multiple earthquake levels can be considered simultaneously such that the design formulations are suitable for the conventional one-level seismic design or the multi-level performance-based seismic design.

Inelastic-analysis-based seismic design methodologies represent a major direction of the development of seismic provisions. For examples, Los Angeles Tall Buildings Structural Design Council published *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region (2011 and 2014 editions)*, and the City of San Francisco Department of Building Inspection published *Recommended Administrative Bulletin on the Seismic Design and Review of Tall Buildings Using Non-Prescriptive Procedures (2007)*. In these two publications, nonlinear dynamic analysis is mandatory for the evaluation of tall building structures.

Due to its complexity and computational burden, nonlinear dynamic analysis was mainly used by researchers to check the behaviors of existing structures, and has been rarely used in engineering design office. So far, very limited work has been done concerning the seismic design optimization using a nonlinear response history analysis procedure. Some pioneering works in this field included Foley et al. (2007) and Gong et al. (2012; 2013). Other relevant studies included the earlier works conducted by the writer and co-workers (Gong et al., 2005 and 2006; Grierson et al., 2006; Xu et al., 2006) on the optimal design of steel frameworks using nonlinear pushover analysis procedures.

This paper is a continued effort of the previous works (Gong et al., 2012 and 2013). First, two unified design formulations, which satisfy the aforementioned two characteristics, for various steel building frameworks are proposed. In these formulations, the seismic design objectives include preventing a building from collapse under an extreme earthquake hazard and controlling building damage under minor or moderate earthquake levels. One design example is used to illustrate the strategy and practicability of the design formulations.

## 2. Unified Design Formulations

The design task is defined as to proportion the member sizes of a seismic force resisting system (SFRS). It is assumed that the layout of the structure is predefined and fixed throughout a design process. For the design solution to be practical, steel member sections will be selected from among commercially available section shapes unless otherwise specified. Since a nonlinear dynamic analysis adopts an assembly of ground motions, a statistical value (e.g., average) of the structural demands should be used in a design process.

The framework of the first unified optimization formulation is described as follows:

Minimize: $Obj = \{O_1 \quad O_2 \quad O_3\}^T$ or $Obj = \{f_1 \quad f_2 \quad f_3\}^T$ (1)

Subject to:

$$\theta_{l,e} \le \theta^0_{l,e} \qquad (l = 1, 2, \ldots n_f;\, e = 1, \ldots n_e) \tag{2}$$

$$\varphi_{k,e} \le \varphi^0_{k,e} \qquad (k = 1, 2, \ldots n_{nf};\, e = 1, \ldots n_e) \tag{3}$$

$$\delta_{s,e} \le \delta^0_{s,e} \qquad (s = 1, 2, \ldots n_s;\, e = 1, \ldots n_e) \tag{4}$$

$$A_j \in C_j \qquad (j = 1, 2, \ldots n_x) \tag{5}$$

and optional constraints:

$$a_s \le a^0_s \qquad (s = 1, 2, \ldots n_s) \tag{6}$$

$$T_1 \le T^0_1 \tag{7}$$

where $n_e$ is the number of earthquake levels considered and $e$ is earthquake level index. For example, $n_e$=2 for such a two-level design problem: collapse prevention under 2%/50-year events and immediate occupancy under 50%/50-year events. The objective vector *Obj* consists of three design criteria $O_1$, $O_2$ and $O_3$ or their corresponding normalized functions $f_1$, $f_2$ and $f_3$. The first design objective is to minimize structural weight, i.e., $O_1 = \sum_{i=1}^{n_x} \rho L_i A_i$ , where ρ is material mass density; $A_i$ is the cross-sectional area of member group $i$ (for a particular commercial steel section, $A_i$ is associated with a set of other sectional properties such as moment of inertia and plastic modulus); $L_i$ is the sum of the lengths of all members of group $i$; $n_x$ is the number of design variables. The weight/cost objective is normalized as $f_1 = \left( \sum_{i=1}^{n_x} \rho L_i A_i \right) \Big/ W_0$ , where $W_0$ is the largest weight among a group of design solutions. To maximize the survival odds of a structure during an earthquake, the second design objective is to minimize seismic input energy $E_i$ to the SFRS, i.e., $O_2$=$E_i$. The input energy objective is normalized as $f_2 = E_i / E_{imax}$ where $E_{imax}$ is the largest input energy among a group of designs. The third design objective is to warrant a desirable plastic mechanism (i.e., to prevent collapse) once the structure enters post-elastic stage under a large earthquake. The objective is implemented herein as to maximize the hysteretic energy $E_{hf}$ of fuse members. To this end, a ratio $\beta = E_{hf} / E_h$ is defined where $E_h$ is the total hysteretic energy of the entire SFRS. The third objective is required to be re-written as a minimization function $O_3$ = $f_3$ = (1 – β) in the design formulation. Objective $f_3$ plays a major role in the capacity design, since the pursuit of maximizing the hysteretic energy of fuse members drives the design algorithm to obtain solutions with more plastic deformations occurring within the fuses (thus, the desirable plastification mechanism). The normalized objective functions $f_1$ to $f_3$ ranges from 0 to 1.0. More rationales for choosing the objective functions can be found in the previous study (Gong et al., 2013).

Equation 2 is the plastic deformation constraints on fuse members, where $n_f$ is the number of fuse members. $\theta_{l,e}$ represents the plastic deformation of fuse member $l$ under earthquake level $e$. For the beams in a moment-resisting frame, θ is plastic-hinge rotation. For the shear links in an eccentrically-braced frame, θ is shear link rotation. For the braces in a concentrically-braced frame, θ is the plastic axial deformation of the braces. $\theta_{l,e}^0$ is the allowed plastic deformation corresponding to earthquake level $e$. The values of $\theta_{l,e}^0$, such as given in FEMA-356 (2000), are dependent upon sectional classifications and axial force ratios (average value of axial forces under an assembly of ground motions should be used). Equation 3 is the plastic deformation constraints on non-fuse members, where $n_{nf}$ is the number of non-fuse members. $\varphi_{k,e}$ represents the plastic deformation of non-fuse member $k$ under earthquake level $e$. $\varphi_{k,e}^0$ are the allowed plastic deformation, whose values reflect the anticipated level of protection to the non-fuse members (e.g., some minor plasticization is generally allowed as it has little detrimental impact). Equation 4 is inter-story drift constraints where $\delta_{s,e}$ is the inter-story drift demand at story $s$ under earthquake level $e$. $\delta_{s,e}^0$ and $n_s$ are the allowed inter-story drift and the number of stories, respectively. Equation 6 is floor acceleration constraints, where $a_s$ is the acceleration response at floor level $s$ and $a_s^0$ is the allowed acceleration limit. Under minor to medium level earthquakes, in the context of performance-based design, inter-story drift and acceleration constraints are used to control building damages. For a major earthquake, inter-story drift constraints are also considered as a means to prevent instability of the structure. Equation 7 imposes an upper limit $T_1^0$ on the first period $T_1$ of the vibration modes, which is often a requirement to account for the uncertainties associated with mathematical modeling of a structure (e.g., negligence of the participation of nonstructural components in seismic response might lead to the underestimation of earthquake demands). $T_1^0$ is often specified empirically in building codes. For example, *National Building Code of Canada* (NRCC, 2010) stated that $T_1$ of a braced frame shall not be taken greater than 2.0×0.025($h_n$), where $h_n$ is the height of the frame in meters.

Equations 2 and 3 are local behavior constraints since the constraints are concerned with the local performance of individual members. Equations 4 and 6 are global behavior constraints, as the inter-story drifts and floor accelerations reflect overall structural stiffness. Equations 2 to 4 are imposed either on single (i.e., conventional collapse prevention level) or multiple earthquake levels, while Equation 6 is usually imposed on minor or medium earthquake levels only. When considering multi-level earthquakes, the allowed plastic deformation limits and inter-story drift limits should be also multiple-valued to match the performance expectations at each hazard level. Note that a designer can choose to impose all or some of the constraints to a structural design. Also note that the second and third design objectives are enforced only at Collapse Prevention level, since the plastic mechanism is of primary concern at this level.

Equation 5 is size constraints. The design variable $A_j$ must be chosen from among a set of pre-selected commercially available steel sections. Note that $n_x$ is not the number of frame members. Due to grouping (i.e., the members having the same cross section are linked as one design variable), the number of design variable is less than the number of frame members. The selection of candidate sections in set $C_j$

should reflect design constraints such as section availability and local buckling prevention, which may not be explicitly included in the design formulation.

The framework of the second unified optimization formulation can be obtained as follows through modifying slightly the first formulation:

Minimize: $Obj = \{O_1,\ O_2\}^T$ or $Obj = \{f_1,\ f_2\}^T$ (8)

Subject to: $(\beta = E_{hf}/E_h) \geq \beta_0$ (9)

in addition to the constraints of Equations 2 to 7.

In the second formulation, the third design objective $f_3$ is removed. Instead, a lower bound $\beta_0$ is imposed to the energy ratio β. For example, $\beta_0$ is taken as 0.9, which requires that fuse members dissipate at least 90% of the total hysteretic energy. Thus, Equation 9 is used to carry out the capacity design principle by enforcing the formation of a desirable plastic mechanism of the SFRS under extreme earthquake events.

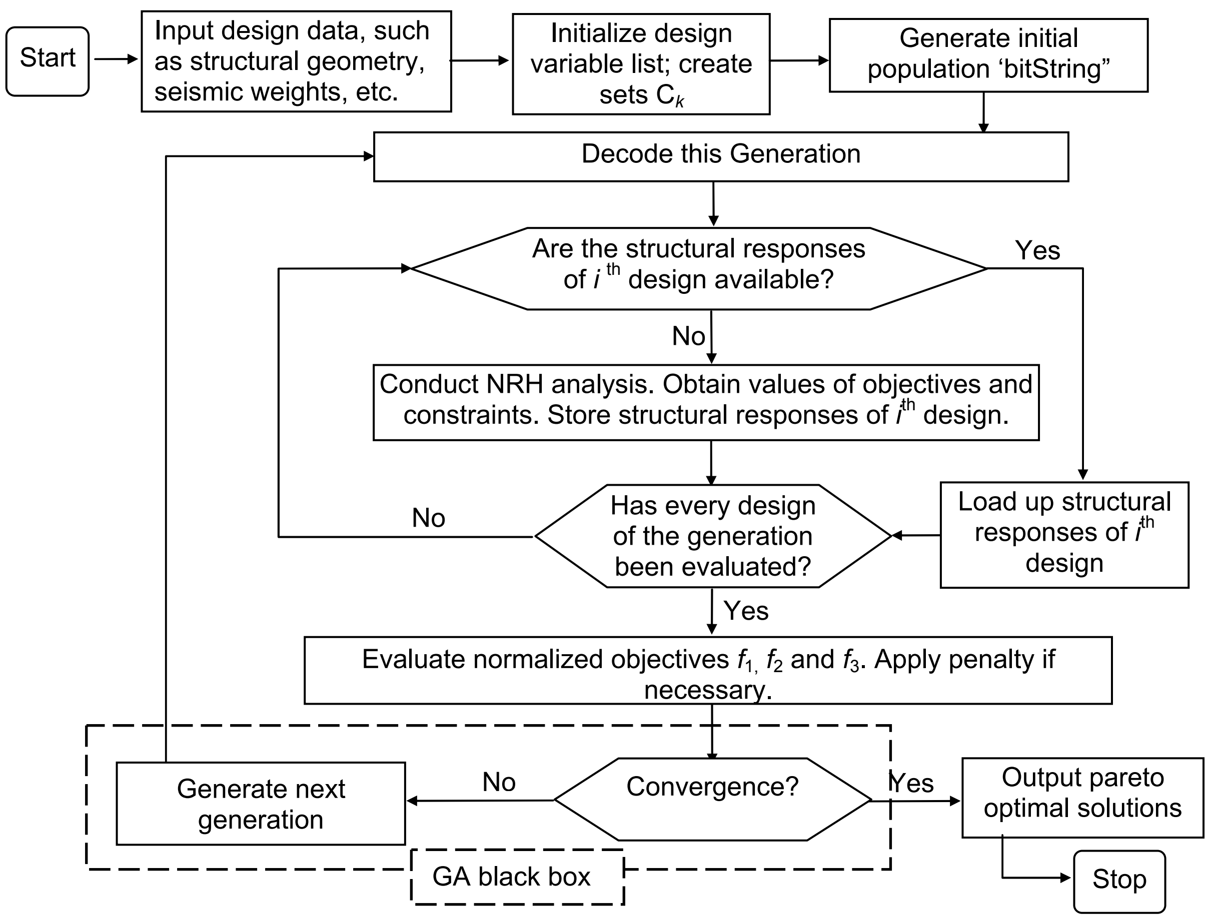


**Fig. 1 – Flowchart of Proposed Design Procedure**

In the first formulation, pursuing the maximum value of β will often result in getting optimal designs with a value of β close to 1.0 (if exist); thus, some good designs with a lower β value (say β=0.90) might be lost. The second formulation provides a remedy to this glitch. However, the second formulation cannot replace the first formulation in all cases, since the value of lower bound $\beta_0$ is determined empirically and also there is no guarantee that an optimal design exists for an arbitrarily chosen $\beta_0$ value.

In these two inelastic-analysis-based design formulations, the code-specified member strength check is not included because the adopted nonlinear response analysis must account for both material yielding and destabilizing effect of gravitational loads (i.e., the so-called advanced analysis). During the analysis, if a design is found to collapse or become unstable, the design is deemed infeasible. Thus, strength limit state is realized at the structural system level. Since member strength checking is an integral part of the analysis process, to be consistent with the limit state design philosophy, load and resistance factors should be included in the analysis procedure to provide certain safety margin for member design.

It can be seen that the underlying analysis method plays a vital role in the design formulation. The structural responses, such as hysteretic energy and plastic deformations, are only available through an inelastic response analysis. For an elastic-analysis-based design methodology, such a design formulation is impossible.

A multi-objective genetic algorithm, which is available in MATLAB software package (MathWorks Inc., 2009), is employed to search for 'Pareto-optimal' solutions. The overall design procedure is illustrated by the flowchart in Fig. 1. The genetic algorithm generates the first generation of designs randomly. Then, the later generations are obtained through reproduction, crossover and mutation. Each design is evaluated by a nonlinear analysis procedure under $n_g$ ground motions. The values of the objective functions and design constraints are obtained based on the nonlinear analysis results (average responses under $n_g$ ground motions are used in this study).

## 3. Numerical Example

### 3.1. Design Problem Statements

This section is to design the seismic force resisting system of a three-story office building located in Vancouver, Canada. All the floors have the same framing plan, and the symmetric plan layout is shown in Fig. 2. The bay size is 9.14 m center-to-center and all three stories are each 3.96 m high. Eccentrically braced frames (EBF) are adopted (Fig. 2 and Fig. 3. There are 4 one-bay frames per direction) as the seismic force resisting system. Lean-on gravity columns are used to account for the seismic weight and the destabilizing effects of the gravity loads of the interior simple frames. The gravity columns are modeled as continuous along the height to account for their bending effects. The sectional properties of the lean-on columns, including moment of inertia and cross-sectional area, are taken to be the sum of the corresponding values of all the tributary gravity columns [single column is HSS254×254×13, whose sectional properties can be found in CISC (2010)].

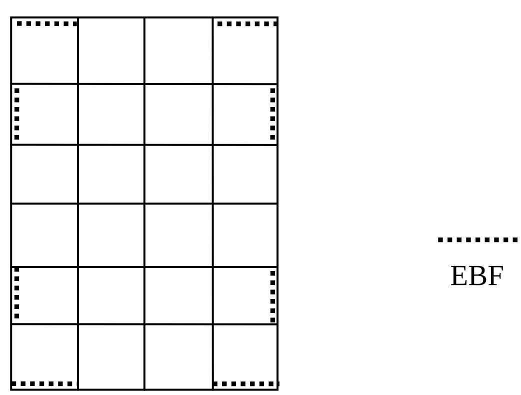


**Fig. 2 – Floor Structural Plan of the Three-Story Building**

This example considers only one level of earthquake hazard, i.e., the collapse-prevention level under 2%/50 year earthquakes. The design spectrum for Vancouver region specified by the *National Building*

*Code of Canada* (NRCC, 2010) is taken as the datum for scaling ground motions. To mitigate calculation burden, only three ground motion histories are adopted as input earthquake hazards, though more ground motions are desirable [e.g., FEMA-450 (BSSC, 2004) requires seven ground motions in order to establish average values of the structural responses]. The selected ground motion time-histories need to be scaled such that their response spectrum is compatible with the design spectra. The names of the adopted ground motions and their response spectra are shown in Fig. 4. Note that these response spectra are equal to or greater than the design spectrum value throughout the period ranging from 0.3 sec to 1.25 sec which is estimated to cover the first period of all the possible design solutions.

In a limit state design, member strength equation is generally written as $\sum \alpha_i S_i \leq \phi R_n$ where $\phi$ and $\alpha_i$ are resistance factor and load factors, respectively; $S_i$ are load effects; and $R_n$ is nominal resistance. To be consistent with the limit state design philosophy, a resistance factor needs to be included in the analysis technique. For an inelastic-analysis-based design method, it is more convenient to move the resistance factor to the left side of the strength equation, i.e., to amplify the load effects by $1/\phi$. Such obtained load effects are called required nominal strength for members. For this example, the ground motion time-histories are amplified by $1/\phi$ ($\phi$ is taken as 0.9 herein).

For the eccentrically-braced frames, the brace-to-column and beam-to-column connections are assumed to be pinned. But brace-to-beam and link connections are designed as rigid. The axial deformation of the beams and links are included in the analysis model since large axial force may develop in these members.

The load combination, 1.0D+1.0E+0.5L+0.25S (as suggested in *National Building Code of Canada*), is considered for design purpose, where D, E, L and S are dead, earthquake, live and snow loads, respectively. The seismic weights D and W, which are used to calculate earthquake load E, are given in Fig. 3b. The accompanying gravity loads are given in Fig. 3c where $Q_1$ to $Q_3$ are the gravity loads directly applied to the frame members and $P_1$ to $P_3$ are the gravity loads of interior frames which generate destabilizing effects on the EBF. Note that the load factors for the limit state design philosophy are already included in the computation of the seismic weights and the gravity loads.

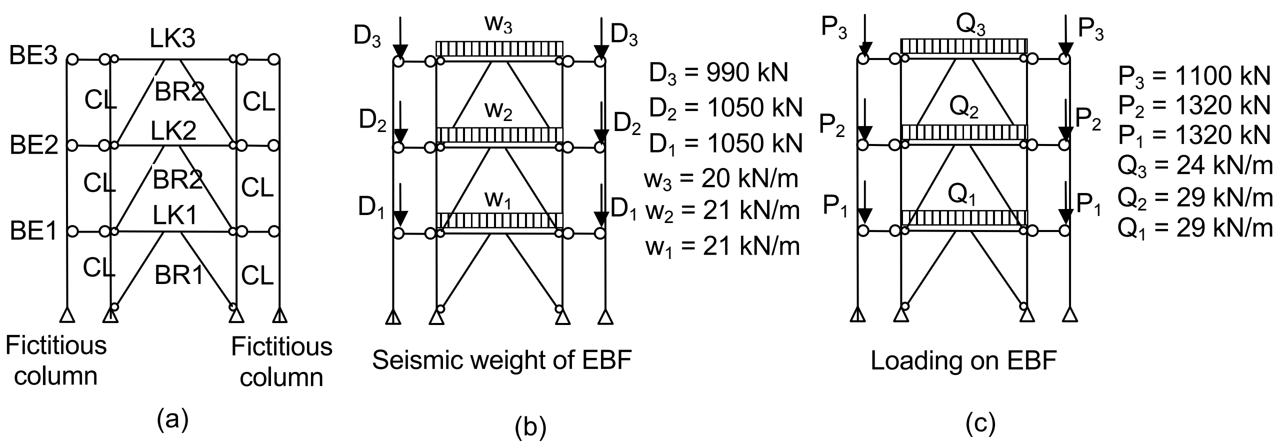


**Fig. 3 – EBFs: (a) Member Numbering, (b) Seismic Weight Distribution; (c) Accompanying Gravity Loads**

## 3.2. Design Results

For the illustration of this example, six design variables are employed (Fig. 3a). All the columns are designed to have the same section and thus grouped as the first variable (i.e., CL) and is chosen among W310 sections (CISC, 2010). For each floor, the link and the beams outside the link are designed as one component and thus are treated as one variable (if necessary, the beams outside of a link will be reinforced with flange cover plates), which gives design variables 2 (LK1), 3 (LK2) and 4 (LK3), and the beams are chosen among W360 or W410 sections (CISC, 2010). The length of all the links are 780 mm, and this length meets the shear hinge requirement. The two braces at story 1 are grouped as the fifth variable (i.e., BR1) while the braces at stories 2 and 3 are grouped as the sixth variable (i.e., BR2), and the braces are chosen among hollow structural sections (HSS). This member grouping technique not only reduces the number of design variables (comparing with the number of frame members) but also reflects the actual construction practice. The classification requirements are accounted for in the selection of candidate sections. For example, compact sections should be used for the columns and braces according to Canadian standard (CSA, 2009).

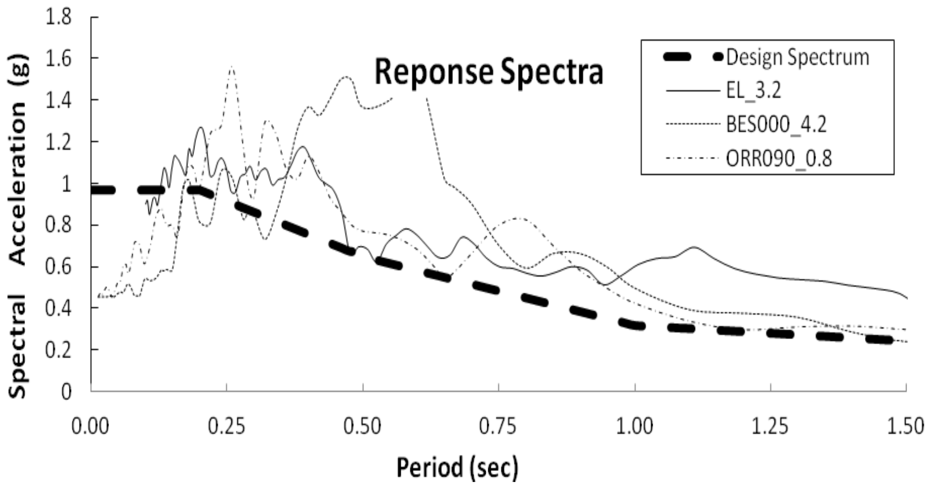


**Fig. 4 – Input Earthquake Histories**

The inter-story drift constraint limit is taken as 2.5% of story height. The link rotation capacity or link rotation limit is taken as 0.08 radians. The plastic rotation limits for columns, beams, and braces are taken as 0.005, 0.005, and 0.010 radians, respectively, as these non-fuse members are allowed to undergo some minor plastification. The first period constraint, which is treated as an optional constraint for the purpose of parametric study, is taken as $T_1 \leq 0.6$ sec based on the sentence 4.1.8.11.3(d)(ii) of *National Building Code of Canada* (NRCC, 2010).

This study adopts OpenSees software (PEER, 2008) to conduct nonlinear dynamic analysis. Stress-strain relationship with 3% strain hardening is adopted. The interaction of axial force and bending moment on a cross section is explicitly considered. Each non-fuse member is modeled by two beam-column elements. An initial imperfection of 1/500 of the member length is considered for braces and columns.

Since capacity design principle requires that non-fuse members are proportioned with respect to the actual resistance of the links, the material strength of links should use probable or expected yield strength in the dynamic analysis. The expected yielding strength of links is taken as 385 MPa. Since frame beams use the same component of links, the strength of the beams is also taken as 385 MPa. The material strength of columns and braces is taken as the specified value of 350 MPa, as required by limit state design philosophy. Damping ratio is taken as 0.05.

More details about the analysis technique and energy calculation can be found in Gong et al. (2012 and 2013). The first design formulation is implemented only, and the obtained pareto optimal solutions through an Generic Algorithm are provided in Tables 1 and 2.

The design process illustrates that cover plates are needed for the beams outside of a link in order to control the plastification of these beams. This example uses cover plate 12 mm by 100 mm for all beams.

Table 1 shows that the first period constraint play a critical role for this example. If the first period constraint were imposed, then the only solution would be Design #1. If the first period constraint were removed, the Pareto solutions have a period ranging from 0.597 to 0.703 seconds. The next constraint that is more likely to govern the design solution is link rotation constraint (Column 5 of Table 2). The #8 is the design solution with the least weight ($O_1$ = 40 kN), and the #1 is the design having the minimum seismic input energy ($O_2$ = 743 kN-m). All the Pareto-optimal solutions have a near 100% β value indicating that the plastic deformations almost exclusively occur within links (note that the plastic rotations allowed for non-fuse members are very small).

It can be seen that design objective $O_3$ drive the genetic algorithm to search for solutions with desired plastification mechanism, thus to achieve the collapse prevention objective under the design earthquake hazard. Since all the Pareto-optimal design solutions have the same plastic mechanism, it is reasonable to say that a design with a smaller seismic input energy will has a smaller risk of structural failure during a future earthquake hazard.

The numerical example also illustrates that design criteria $O_1$ and $O_2$ are generally competing with each other among the Pareto optimal set. But $O_3$ or β was not competing with either $O_1$ or $O_2$, because the yielding of fuse members in a structure is not dependent on its overall structural weight or seismic input energy but rather the relative strength between fuse and non-fuse members.

**Table 1 – Search results of multi-objective genetic algorithm**

| Solution # | CL (W) | LK1 (W) | LK2 (W) | LK3 (W) | BR1 (HSS) | BR2 & BR3 (HSS) | First Period (sec) |
|---|---|---|---|---|---|---|---|
| 1 | 310x52 | 410x100 | 410x46 | 360x33 | 203x203x13 | 178x178x8.0 | 0.597 |
| 2 | 310x52 | 410x74 | 410x46 | 360x33 | 203x203x13 | 178x178x8.0 | 0.606 |
| 3 | 310x52 | 410x74 | 410x46 | 360x33 | 203x203x9.5 | 178x178x8.0 | 0.623 |
| 4 | 310x52 | 410x60 | 410x46 | 360x33 | 203x203x13 | 178x178x8.0 | 0.616 |
| 5 | 310x52 | 410x46 | 360x39 | 360x39 | 203x203x13 | 178x178x8.0 | 0.638 |
| 6 | 310x52 | 410x46 | 360x39 | 360x33 | 203x203x13 | 178x178x8.0 | 0.641 |
| 7 | 310x52 | 410x46 | 360x39 | 360x33 | 178x178x8.0 | 178x178x8.0 | 0.684 |
| 8 | 310x52 | 410x46 | 360x39 | 360x33 | 178x178x8.0 | 178x178x6.4 | 0.703 |

Note: The beams outside of a link is formed by welding a cover plate of 12 mm × 100 mm to each flange of the chosen section.

**Table 2 – Structural responses of Pareto-optimal designs**

| Solution # | $O_1$ (kN) | $O_2$ (kN-m) | β (%) | Max. Link Rotation (% radians) | Max. Story Drift (% radians) | Max. Beam Plastic rotation (% radians) | Max. Brace Plastic rotation (% radians) |
|---|---|---|---|---|---|---|---|
| 1 | 51 | 743 | 99 | 5.8 (0.73) | 0.96 (0.38) | 0.029 | 0.19 |
| 2 | 48 | 757 | 99 | 5.8 (0.72) | 0.95 (0.38) | 0.029 | 0.16 |
| 3 | 47 | 790 | 98 | 5.7 (0.71) | 0.95 (0.38) | 0.031 | 0.18 |
| 4 | 47 | 805 | 99 | 6.5 (0.82) | 0.82 (0.33) | 0.018 | 0.12 |
| 5 | 46 | 820 | 99 | 6.9 (0.86) | 0.93 (0.37) | 0.041 | 0.05 |
| 6 | 45 | 822 | 99 | 6.6 (0.82) | 0.90 (0.36) | 0.038 | 0.05 |
| 7 | 42 | 834 | 98 | 7.0 (0.88) | 0.98 (0.39) | 0.046 | 0.19 |
| 8 | 40 | 810 | 97 | 7.0 (0.87) | 1.04 (0.42) | 0.056 | 0.37 |

Note: The number inside parenthesis is the ratio of demand to capacity.

## 4. Conclusions

This paper presented two unified multi-objective optimal capacity design formulations for various steel building frameworks subjected to earthquake ground motions. A design example of eccentrically-braced frame was used to illustrate the design procedure. It was illustrated that the unified design approach is made possible by using an inelastic analysis tool, which is nonlinear response history analysis procedure for this study.

A common misconception is that the current elastic-analysis-based design rules (EABD) can be equally applied to inelastic-analysis-based design methods (IABD). The following lists some major differences between the two design approaches as noticed by the author in this series of studies: 1) member strength checking is not required for an IABD approach, while it is required for an EABD approach; 2) plastic deformation limits on individual members are required for an IABD approach, while they are generally not required for an EABD approach; 3) safety factors need to be included in the analysis procedure for an IABD approach, while they are directly included in member strength checking for an EABD approach; 4) the limit state design philosophy is realized in structural system level for an IABD approach, while it is realized in member level for an EABD approach; 5) a set of ground motion histories are needed for an IABD approach, while a design response spectrum is required for an EABD approach; 6) ductility- and overstrength-related force modification factors are not required for an IABD approach, while they are needed for an EABD approach.

The explicit design objectives included the least structural weight, the minimum seismic input energy, and the maximum hysteretic energy of fuse members. The implicit design objective of controlling earthquake damage is implemented through imposing inter-story drift constraints and floor acceleration constraints. A multi-objective Genetic Algorithm was adopted to search for Pareto optimal design solutions, which allowed the designer to choose the final design solution based on his/her preference.

The selection of the design objectives and the design constraints fully reflects the strength of the adopted analytical tool. The earthquake input energies, plastic deformation of members and floor accelerations are the structural response information which can only be obtained through a nonlinear dynamic analysis. Comparing with displacement or force responses, an energy response has the advantage of not only reflecting the duration of ground motions but also accounting for the accumulative effect of inelastic deformations. While the inter-story drift constraints govern the overall structural performance, the plastic deformation constraints ensure the satisfactory local behavior of individual members. Furthermore, if the plastic deformation capacity of a member could be directly calculated as a function of its width-to-thickness ratios and internal forces, the classification requirement of the section could be waived in design code.

The fundamental period of the design solutions of the example appeared to be much greater than the empirical value given by *National Building Code of Canada* (NRCC, 2010). This entails further research to compare the design results of the proposed design method with traditional elastic-analysis-based design methods (especially when an empirical period is used) in order to develop a guideline for the inelastic-analysis-based design method. The long computer hours for the design example also demonstrated the necessity in future endeavor to develop strategies to mitigate computational burden. Future researches also include integrating the automatic selection of an assembly of ground motions into the design synthesis such that the chosen motion histories match the design spectrum as closely as possible.

In summary, the proposed design formulations are generally applicable to various steel frameworks such that the current a-set-of-rules-for-each-type-of-SFRS approach is not necessary. Since the analytical tool is most accurate, it helps to obtain a structural design solution with a better performance under ground motions. Furthermore, the capacity design principle will be implemented by computer instead of arduous hand calculations.

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