Optimum Performance Based Design of Concentric Steel Braced Frames

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ABSTRACT :
Earthquake and structural engineering challenge of creating optimized, reliable and cost effective structures leads to the combination of optimization and performance based seismic design theory. The prime goal is to automate the design of the structure on the basis of performance based design and also considering the inherent uncertainties. In this study automating the design process of concentric steel braced frames is performed by use of genetic algorithms. The optimal design of structure minimizes the structural weight subjected to performance constraints on axial deformations of braces and plastic hinge rotation of beam-columns and also the force interactions relationships for them. Nonlinear static analysis (pushover) is implemented by considering the effect of post-buckling in compression brace elements and the performance based criteria is derived from the FEMA-356 (2000). The developed software in this study is capable of automating the design of braced Frames with different spans and stories for a prescribed performance objective, with the limitation of usage for structures in which the first mode is dominant. It is found that a wide range of valid design alternatives exists, from which a decision maker selects the one that balances and optimizes different objectives in the most preferred way.

KEY WORDS :
performance-based design, optimization, pushover analysis, genetic algorithm

1. INTRODUCTION :

During the past decade, significant progress has been made in performance-based engineering methods, which are rapidly becoming widely accepted in professional practice. The growing acceptability of the performance-based design approach is reflected by a number of documents regarding seismic rehabilitation of existing buildings (ATC-40, 1996; FEMA-273, 1997; FEMA-356, 2000). The concepts and principles laid out in these publications for seismic rehabilitation can also be applied for new building construction in the context of performance-based design (Krawinkler, 1998; Ganzerli, 2000; FEMA-350, 2000; Alimoradi, 2003; zou, 2004).

This design method involves a set of procedures by which a building structure is designed in a controlled manner such that its behavior is ensured at predefined performance levels under earthquake loading (performance objective). A nonlinear analysis tool is required to evaluate earthquake demands at the various performance levels. Pushover analysis is widely adopted as the primary tool for such nonlinear analysis because of its simplicity compared with dynamic procedures.

Earthquake and structural engineering challenge of creating optimized, reliable and cost effective structures leads to the combination of optimization and performance based seismic design theory. As the process of performance-based design of structural systems can be suitably formatted into an optimization problem and hence help the designer to make better decisions in an automated design environment, researchers have shown interest in this field and a number of applications considering some criteria of the existing codes are presented. (Ganzerli, 2000; Alimoradi, 2003; zou, 2004).

The design process is an iterative procedure, in which the initial design is modified repeatedly through the results of structural analysis to meet designer specified requirements. The pushover analysis considering both the geometric and material nonlinearities is a computationally costly practice and this would be intensified if it is combined with the design optimization of building frameworks. An efficient algorithm is yet needed to incorporate pushover analysis together with optimal performance-based design constrained with detailed criteria.

The goal of this study is to optimally automate the design process of concentric steel braced frames, as they are very efficient structural systems in steel for resisting lateral forces due to earthquakes. This is done by developing software (SnapGA) which utilizes Genetic algorithm optimization method and nonlinear analysis software.

2. PERFORMANCE BASED DESIGN :

The promise of performance-based design (PBD) is to produce structures with predictable seismic performance (Naeim, 2001). In order to implement it, a performance objective which consists of two major components must be selected: a stated maximum level of expected damage (performance level) and a level of seismic hazard. In general, performance objectives can be defined quantitatively or qualitatively. They may be expressed in a deterministic manner (FEMA-356) or in a reliability-based probabilistic approach (FEMA-350) (Alimoradi 2003). A performance level is a statement of the desired building behavior while experiencing earthquake demands of specified severity. Four building performance levels are defined in the literature, namely, Operational (OP), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) levels (FEMA-356). For each level, the qualitative description of the building performance is implied by structural response parameters, such as target displacement, axial deformation, plastic hinge rotation, etc.

A performance objective also, requires the assigning of particular earthquake intensities to maintain a satisfactory performance. Four probabilistic hazard levels related to earthquakes having 50%, 20%, 10% (BSE-1) and 2% (BSE-2) probability of exceeding in 50 years (mean return periods of 72, 225, 474, and 2475 years) are defined in FEMA-356. All possible performance levels and corresponding earthquake intensities noted are considered by this research study.

The estimation of demands can be accomplished using a variety of available procedures. Using the nonlinear static procedure, the inelastic behavior of the structure as a whole can be captured by a static pushover curve. The pushover curve gives an accurate description of the structural behavior, compared to the dynamic analysis, at least for a structure that has a low number of participating modes. The advantage in utilizing a pushover analysis relies in the fact that it can be used in most practical cases. On the other hand, dynamic inelastic time history analyses are often difficult to implement. The practical objective of inelastic seismic analysis procedures is to predict the expected behavior of the structure in future earthquake shaking. This has become increasingly important with the emergence of performance-based engineering (PBE) as a technique for seismic evaluation and design (FEMA-440, 2005).

The Coefficient Method is the primary nonlinear static procedure presented in FEMA-356. This approach modifies the linear elastic response of the equivalent SDOF system by multiplying it by a series of coefficients C_0 through C_3 to generate an estimate of the maximum global displacement (elastic and inelastic), which is termed the target displacement. The process begins with an idealized force-deformation curve (i.e., pushover curve) relating base shear to roof displacement. An effective period, T_e, is generated from the initial period, T_i, by a graphical procedure that accounts for some loss of stiffness in the transition from elastic to inelastic behavior. The effective period represents the linear stiffness of the equivalent SDOF system. When plotted on an elastic response spectrum representing the seismic ground motion as peak acceleration, S_a, versus period, T, the effective period identifies a maximum acceleration response for the oscillator. The assumed damping, often five percent, represents a level that might be expected for a typical structure responding in the elastic range (FEMA-440, 2005). The target displacement is established from the following equation:

$$\delta_t = c_0 c_1 c_2 c_3 s_a \left[\frac{T_e^2}{4\pi^2} \right] g \qquad (2.1)$$

Where δ_t is the target displacement, C_0 is a modification factor relating spectral displacement estimated for an equivalent SDOF system to the likely roof displacement of a multistory structure and C_1, C_2 and C_3 are modification factors to account for the effects of inelastic system degradation and $P-\Delta$ effects respectively. The effective fundamental period T_e is determined from

$$T_e = T_i \sqrt{K_i / K_e} \qquad (2.2)$$

Where T_i is the elastic fundamental period, K_i is the elastic lateral stiffness, and K_e is the secant stiffness at 60% of the yield strength of the building.

Assessing the performance of the building is the last to be done. Response quantities estimated from the analysis of the building model need to be compared to acceptable or allowable limits that are termed "acceptance criteria". These limits can be specified both at the global level in terms of interstory drift limits and at the local level in terms of component demand limits. The limits are a function of performance levels.

3. FORMULATION OF THE DESIGN OPTIMIZATION PROBLEM :

Structural optimization seeks optimal values of design variables that achieve the best outcome of a given objective while satisfying code or designer-specified criteria. An objective function, often known as a cost or performance criterion, is expressed in terms of the design variables and serves as a guide for the decision maker. The optimal design is the one providing the best value for the objective function while satisfying all the constraints; thus, the selection of an appropriate objective function is extremely important.

Automating the design process has been a subject of some recent studies. Ganzerli et al 2000. incorporated PBD concept and pushover analysis in the optimal design of portal reinforced concrete frame (one story, one bay) constrained with plastic hinge rotations of columns and beams under the criteria of FEMA-273. Zou, 2004. Combined pushover analysis together with numerical optimization procedure to automate the pushover drifts performance design of the reinforced concrete buildings. Alimoradi, 2003. tried to automate the design process considering the criteria of FEMA-350 guidelines. The aim of this study is to automate the design process of concentric braced frame regarding the detailed criteria of PBD. In order to achieve this goal, Genetic Algorithm optimization method (Goldberg, 1989), constrained with the criteria of FEMA-356 is used. The developed software is applicable for the optimal design of concentric braced frames with any number of stories and spans and with any location of bracings.

The optimal design of structure minimizes the structural weight subjected to performance constraints on axial deformations of braces and plastic hinge rotation of beam-columns and also the force interactions relationships for them. Assuming that the cost of a member is proportional to its material weight, that the unit material cost for each member is the same, and that the member has a prismatic section throughout its length, the least-cost design can be interpreted as the least-weight design of the structure, and the weight objective function (OBJ) to be minimized can be formulated as:

$$OBJ = \sum_{i=1}^{n_e} \rho_i \, l_i \, A_i \qquad (3.1)$$

Where: n_e is the number of members; ρ_i is the material mass density; and l_i and A_i are the fixed length and variable cross-section area of member i, respectively. The design variables are chosen as the cross-sectional areas of the columns and braces and the constrains can be summarized as

$$\Delta_p^{pl} \leq \overline{\Delta^{pl}} \qquad (3.2)$$

Where Δ_p^{pl} is the bracing axial deformation and $\overline{\Delta^{pl}}$ is its allowable amount based on Table 5-7 of FEMA-356 guidelines.

Flexural loading of columns, with axial loads at a target displacement less than 50% of P_{cl} (compression strength of the column), shall be considered deformation-controlled and maximum permissible plastic rotation demands on columns, in radians, shall be as indicated in Tables 5-6 of FEMA-356 guidelines, dependent on the axial load present and the compactness of the section.

$$\theta_p^{pl} \leq \overline{\theta^{pl}} \qquad (3.3)$$

Where θ_p^{pl} is the column plastic hinge rotation, and $\overline{\theta^{pl}}$ is maximum permissible amount based on Table 5-6 of FEMA-356 guidelines.

Flexural loading of columns, with axial loads at the target displacement greater than or equal to 50% of P_{cl}, shall be considered force-controlled and shall conform to

$$\frac{P_{UF}}{P_{cl}} + \frac{M_{UF}}{M_{CL}} \leq 1 \qquad (3.4)$$

Where P_{UF}, M_{UF}, M_{CL} are axial load, bending moment and the flexural strength of the member.

The base shear at the target displacement, V_t, shall not be less than 80% of the effective yield strength of the structure, V_y which is calculated using results of the Pushover for the idealized nonlinear force displacement curve developed for the building.

$$V_t \geq 0.8 V_y \qquad (3.5)$$

4. METHODOLOGY

SNAP-2DX analytical platform (Rai et. al., 1996) is used for pushover analysis. It is a structural, nonlinear analysis program for two-dimensional models. This program has a library of nonlinear elements including an element which is consistent with the phenomenological model proposed by Jain and Goel (1978), and also presented in FEMA-274 (1997). It is used for modeling nonlinear behavior of braces as shown in Figure1. The post-buckling residual compression force is set to be 20% of the buckling load as given in Tables 5–7 of FEMA-356. All connections in the braced frame are assumed to be ideally pinned. It is also assumed that the braces bypassed each other. Both the beam and column elements are modeled by the beam–column element, which uses lumped plasticity to model inelastic behavior. The post-yield stiffness of the beams and columns was assumed to be 3% of the initial stiffness.

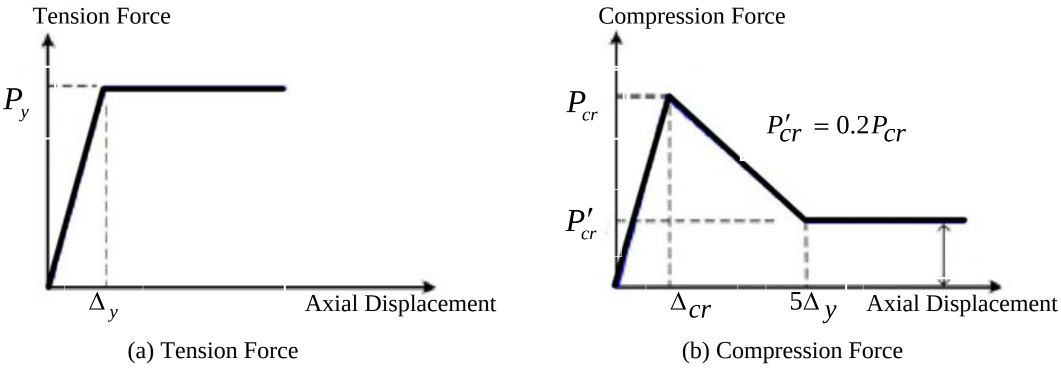


Figure 1 Nonlinear behavior of braces

Genetic algorithms which are a part of evolutionary computing, and have recently been in the focus of attention for automated structural design, are implemented as the optimization medium. They have proven themselves as reliable computational search and optimization procedures for complex objectives involving large number of variables. In the process of structural design optimization, one may ignore the discrete nature of variables and use approaches that deal with continuous variables and then the nearest discrete variable are chosen as the perfect design. This may be applicable when discrete variables are relatively close to each other but usually a decision maker has to select design variables from commercial sections which are not so close to each other. Genetic algorithms have overcome this problem by the ability of optimizing discrete variables.

SnapGA, the developed software in this study, combined the outstanding features of genetic algorithms with the noble idea of performance based design to optimally automate the process of design cost effective and reliable structures. The step by step procedure can be summarized as follows:

1. The input file (SnapGA.INP) which consists the geometry of framework, loading condition, performance objective, section properties, initial assignments and genetic algorithm specifications, is developed.
2. For each generation of genetic algorithm the following steps are applied:
   a. SnapGA.INP is used to produce the input file for SNAP-2DX program for each individual of the population.
   b. Regarding the desired performance objective, the properties of equivalent single degree of freedom structure, including T_e and K_e are calculated.
   c. Target displacement is calculated based on an iterative procedure.
   d. Processing the output file of SNAP-2DX, fitness function for each individual is calculated.
3. If the convergence criteria are met, optimization procedure is finished. Otherwise step 2 is repeated.
4. With regards to the optimized design an output file is generated which contains performance based design information including structure's demands, acceptance criteria, etc. and the optimization procedure.

In the next session the procedure is illustrated by a numerical example.

5. NUMERICAL EXAMPLE :

A three-story, three-bay planar frame is used to illustrate the proposed optimal design method. The geometry of the example is given in Figure 2.

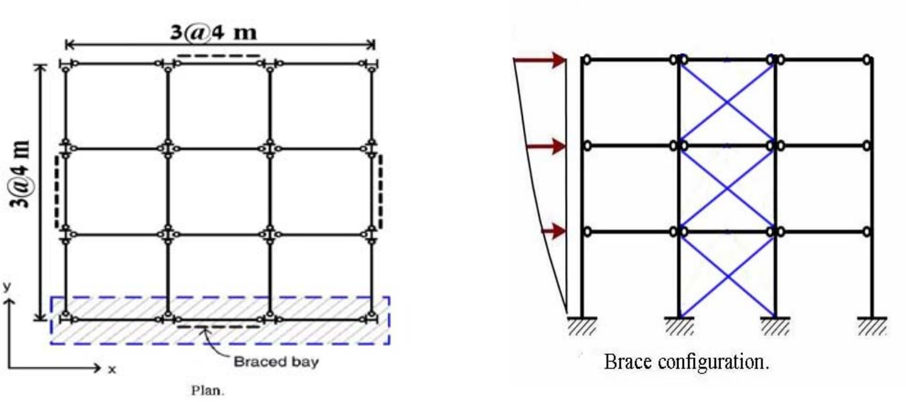


Figure 2 Configuration of model structure

The analysis method is based upon nonlinear static procedures. The demands are estimated using coefficient method of FEMA-356. The loads considered in the pushover analysis are lateral seismic and vertical gravity loads. While the lateral loads are incrementally applied, the gravity loads are maintained unchanged during the nonlinear pushover analysis process. It should be mentioned that two patterns of lateral loading is used for push over analysis.

The gravity loads at each story are as follow: D.L. = 30 kg/cm, L.L.= 12 kg/cm. Performance objective for the example is chosen as the combination of Life Safety (LS) performance level and the Basic Safety Earthquake-2 (BSE-2). The general design spectrum can be constructed at the site where the structure is located; the spectrum can be subdivided into three zones based on the period of the structure which reflects level of uncertainty in source, path, and site effects.

$$S_a = \begin{cases} 0.6 + 11.25T & 0 < T \le 0.2T_0 \\ 1.5 & 0.2T_0 \le T \le T_0 \\ 0.6/T & T > T_0 \end{cases} \tag{5.1}$$

It is assumed that T_0=0.4 sec.

Beam sections are considered constant trough the optimization as story floors are rigid. Column and brace sections which are used for the optimization are summarized in Table 1.

Table 1 Available column and brace sections for optimization

Columns	IPE 22	IPE 24	IPE 27	IPE 30	IPE 33	IPE 36
	IPE 40	IPE 45	IPE 50	IPE 55	IPE 60	IPE 75×147
Braces	2L 8	2L 10	2L 12	2L 15	2L 18	2L 20

An initial population of 20 individuals and total number of 30 generations are used for genetic algorithm optimization. Convergence criteria are set as both active and inactive, which are the fitness and maximum number of population. The properties of the optimized structure are as follows:

Table 2 Optimum Column and brace sections

Story	Interior column	Exterior column	Brace	Beam
1	IPE60	IPE 75×147	2L15	IPE27
2	IPE45	IPE40	2L15	IPE27
3	IPE30	IPE27	2L12	IPE27

Coefficient method parameters for the optimized design are summarized in Table 3.

Table 3 Coefficient method parameters

	Lateral loading proportional to the fundamental mode shape	Uniform lateral loading proportional to the total mass at each level
T_e	0.25998	0.2598
C_0	1.2	1.2
C_1	1.095247	1
C_2	1.193462	1.193462
C_3	1	1
S_a	1.5	1.5
δ_t (cm)	3.946445	3.603246

Pushover curves for the optimum design are presented in Figure 3 and Figure 4. The gradual movement of the design to the optimum point is shown in Figure 5.

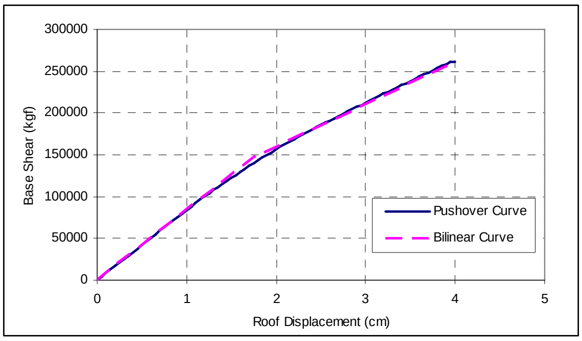


Figure 3 Pushover curves with lateral loading proportional to the fundamental mode shape

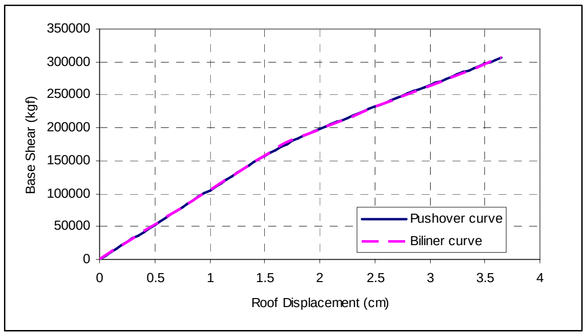


Figure 4 Pushover curves with uniform lateral loading proportional to the total mass at each level

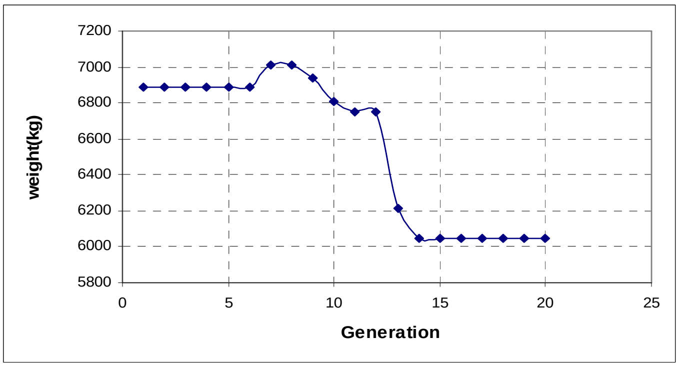
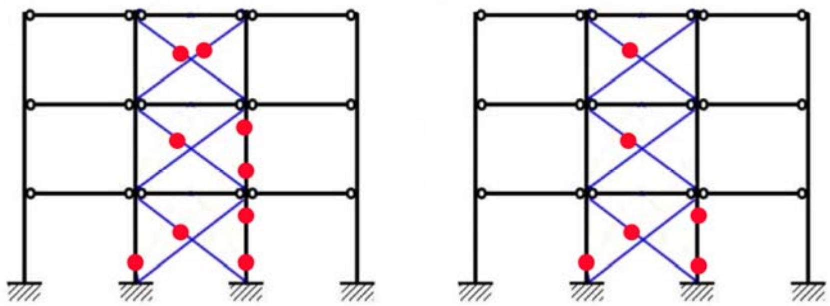


Figure 5 Gradual movement to the best frame weight

Figure 6 shows the formation of nonlinear hinges in the members of the frame in two lateral loading conditions.



a. Lateral loading proportional to the fundamental mode

b. Uniform lateral loading proportional to the total mass at each level

6. CONCLUSION

This study has demonstrated that performance-based criteria could be implemented not only for retrofitting existing structures but also for design of new buildings. The incorporation of performance-based criteria in the design process allows the designer to design a structure for a specific safety level. By adopting these criteria, the designer has a better control of the project, and can determine the expected structural behavior in the case of other earthquakes, which may occur in the life of the structure. Two-dimensional concentric steel braced frames were optimized for achieving minimum weight .Design variables included the cross-sectional of the columns and braces. Performance-based constraints were implemented in terms of axial deformations of braces and plastic hinge rotation of columns and also the force interactions relationships for them. It is shown by this study that the developed program SnapGA is capable of motivating the decision maker to choose the best design from a wide range of valid alternatives with regards to any performance objective.

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