CHAPTER 6

MULTIOBJECTIVE OPTIMIZATION FOR PERFORMANCE-BASED SEISMIC DESIGN

Abstract: In the context of performance-based seismic design, an automated procedure is presented in this chapter for optimized member sizing of regular plane steel special moment-resisting frame structures. Multiple merit measures, which aim to reflect either the present capital investment or the future seismic risk for code-compliant design solutions, are treated simultaneously as separate objective functions other than stringent constraints. Specifically, the initial capital costs are accounted for by the steel material weight as well as the number of different standard steel section types, the latter roughly quantifying degree of design complexity related additional construction expenses; the seismic risk objective is considered by maximum interstory drift demands at two hazard levels with exceedance probabilities being 50% and 2% in 50 years, respectively. A genetic algorithm is used as the search engine for the presently posed multiobjective optimization problem. The resulting large pool of optimized tradeoff alternative designs provides much flexibility for structural engineers to select the final structural design with the most desirable balance between initial investment and seismic performances.

6.1 Introduction

In contrast to traditional prescriptive seismic code provisions that typically use empirical formulations for structural design, emerging performance-based seismic design methodologies require explicit evaluation of seismic demands on structural systems at predefined performance levels so that the resulting designs are expected to achieve predictable performances when subject to future seismic events. Recent guidelines such as Vision 2000 (SEAOC 1995), ATC-40

(1996), and FEMA-273 (1997) outlined analysis procedures to evaluate seismic performance of building structures and provided both qualitative and quantitative definitions for structural and component performance levels. In terms of overall structural and nonstructural damage levels, FEMA-350 (2000) described Immediate Occupancy (IO) and Collapse Prevention (CP) as two performance levels paired with seismic hazard levels of exceedance probabilities being 50% and 2% in 50 years (briefly denoted hereafter as 50/50 and 2/50 hazard levels), respectively, which correspond to average return periods of 72 and 2475 years, respectively. The IO level implies very light damage with minor local yielding and negligible residual drifts, while the CP level is associated with extensive inelastic distortion of structural members with little residual strength and stiffness.

Predicted structural performance parameters at each hazard level are usually checked against deterministic criteria resulting from a combination of analytical analysis and engineering judgment. Probabilistic procedures also become available to consider various sources of uncertainty and randomness in estimating both seismic demand and capacity as well as in seismic excitations. For example, FEMA-350 presented procedures that evaluate seismic performances in terms of the confidence level that a building would provide desired performance levels at specified seismic hazards. Using these multilevel multicriteria performance-based design procedures, structural engineers are hopefully able to directly and explicitly control seismic performances of a design project, indicating structural as well as nonstructural damages are expectedly reduced to an acceptable level.

In addition to satisfying conventional code requirements, it is of particular interest to seek for a design solution with more economical use of resources. An optimized seismic design is obtained when it can achieve balanced minimization of two general competing objectives: the present capital investment and future seismic risk. Multiple merit measures exist in design practice and can be used to assess the quality of a design candidate. Structural material usage is one of such merit objectives. Besides, degree of design complexity may also be used as another merit measure because it may affect the labor-related construction cost significantly. Unlike stringent code specifications that a valid design has to comply with, these various merit objectives are not restrictive in nature and their actual values are subject to structural engineers' choice. This is especially true for performance-based seismic design where multiple seismic performance objectives need to be appropriately achieved. Acceptable structural performance parameters recommended in established guidelines are best interpreted as indicative of performance ranges that a structure may sustain when responding at different performance levels. Structural engineers are expected to make judgmental decisions on what these performance measures should actually be for code-compliant seismic designs.

Traditionally there have been two widely used approaches to handling multiple (usually conflicting) merit objectives in optimal structural design by solving single-objective based optimization problems: the weighted sum approach and the ε-constraint approach (Section 4.1.2). Much of existing research on optimal seismic design in the literature is single-objective based with the structural material usage (weight or cost) as the mostly cited objective function while treating seismic risk related performance merits as constraints for checking design validity only.

When facing a single optimized design solution obtained using the above approaches, structural engineers do not have a broad view of how other alternative code-compliant designs behave in terms of relevant merit objectives under consideration. Sometimes a design solution other than the single optimized one resulting from material usage minimization may be preferred in order to emphasize structural performance in terms of another (perhaps composite) merit measure. Rather than accepting or rejecting a single design solution using predetermined relative importance ranks of merit measures, structural engineers may be more interested in actively selecting a final design from among a group of design candidates, which are obtained without a priori merit weighting, that exhibit diverse characteristics in terms of different merit measures.

Therefore, a more natural way is to treat all relevant merit measures separately as well as simultaneously in structural design optimization, which leads to the formulation of a multiobjective optimal design problem and thus a distribution of design solutions that establishes optimized tradeoff among all selected conflicting objectives. Structural engineers are then expected to compare these alternative designs and choose with much freedom the one that compromises different competing merit aspects in the most preferred manner.

A complete design process comprises conceptual design, structural component selection, detailing, construction consideration, and so on. Automated optimization is responsible for the part of design process that is repetitive in nature. One such example that will be studied in this chapter is member-sizing optimization for performance-based seismic design of regular plane steel special moment-resisting frame (SMRF) structures with a predefined geometrical layout.

Research on design optimization of civil structural systems in the context of performance-based seismic design has appeared only recently. Beck et al. (1999) presented a multi-criteria optimal design framework for performance-based design of structural systems, using a decision theoretic approach based on aggregation of preference functions for the multiple conflicting design criteria. Li et al. (1999) proposed a multiobjective and multilevel procedure for optimizing seismic steel frames; total structural strain energy and total structural weight were considered as two objectives at the system level and member weight was considered as a single objective at the element level. Ganzerli et al. (2000) minimized overall material cost for a simple reinforced concrete portal frame with performance constraints on beam/column plastic rotations. Foley (2002) summarized state-of-the-art of the performance-based design for building structures and discussed application of structural optimization techniques in such a design framework. Grierson et al. (2002) investigated performance-based seismic design of steel framework structures with two identified objectives: the structural cost (weight) and uniform height-wise interstory ductility demand. Liu et al. (2003) considered life cycle costs in multiobjective design optimization of seismic steel SMRF structures using a series of structural performance (damage) levels in terms of maximum interstory drift ratios.

6.2 A multiobjective design optimization framework

An automated structural design optimization procedure typically comprises three general components: (1) relevant constraints that define the feasible design space, (2) appropriate objective functions based on which merits of different valid designs are assessed and compared, and (3) suitable numerical algorithms that guide the optimization search process. In the present study, constraints that check the validity of each design candidate come from current seismic design codes for steel structures; multiple objective functions are considered to assess different merits of code-compliant designs regarding either initial capital costs or seismic structural performances; a recently emerged evolutionary algorithm is adopted herein to solve the posed multiobjective structural optimization. Figure 6.1 shows a schematic flowchart for the present automated design procedures.

Practical objective functions are essential for an automated structural design procedure to produce optimized designs that are viable in a real-world practice. In the performance-based seismic design optimization, merit objectives are defined such that they either address the immediate economic concern or reflect future seismic risks in terms of structural performance parameters at predefined hazard levels. In this chapter, the initial capital costs are taken into account by two separate objective functions: steel material weight and degree of design complexity in terms of the number of different standard steel section types. Seismic structural performances are represented by maximum interstory drift ratios computed at 50/50 and 2/50 hazard levels, respectively.

6.2.1 Steel material weight

A code-compliant structural design with reduced use of resources is appealing to the civil engineering profession, which has been most often implemented with a minimum weight criterion in the structural optimization community when dealing with homogenous material such as steel. Weight minimization of steel moment frames has been fruitfully studied in the literature using various optimization techniques (e.g., Pezeshk 1998; Foley and Schinler 2003). It should be pointed out that, due to price differences for different section types as well as for different strength grades, the steel material cost may be a more accurate measure than the steel material weight when describing actual steel material usage. In order to make an easy comparison to existing minimum weight steel frame design procedures, however, steel material weight will still be used as a merit objective function throughout the present study.

6.2.2 Number of different steel section types

Steel material weight alone, however, cannot completely reflect the total initial capital investment, as has often been observed in the real-world design practice. The reason is that total initial costs are dependent not only upon steel material usage but also upon the associated degree of design/construction complexity. Consequently, an ‘optimal’ design resulting from material weight minimization alone may not necessarily correspond to the one with minimum total initial monetary costs. In order to roughly account for the extra construction costs due to complexity of the design, fabrication, and erection stages, the number of different commercially available standard steel beam/column section types is used in this study as another merit objective subject to simultaneous minimization. A reduced number of section types will facilitate the construction process by duplicating column splices as well as beam-to-column connections. This objective has also been used in structural optimization by other researchers (Sarma and Adeli 2000; Greiner et al. 2001).

6.2.3 Seismic structural performance indices

Suitable structural performance indices are needed to reflect appropriately structural behaviors at designated seismic hazard levels, based on which merits of different design solutions can be compared. Both direct and derived quantities have been used in the literature that reveal (inelastic) structural responses and/or imply seismic damages sustained by structures. Some of the widely used performance indices are described as below.

Peak roof drift ratio

Peak roof drift ratio is defined as the peak lateral roof displacement at a particular hazard level, Δ_u, normalized by the building height *H*. This parameter reflects the height-wise average deformation severity and overall damage to nonstructural elements. It also reflects degree of vulnerability of a building structure to P-delta effects due to cumulative interior gravity loads above a story level acting on the deflected shape; a large peak roof drift ratio indicates possible loss of stability and potential collapse (Aschheim and Black 2000).

Peak interstory drift ratio

Peak interstory drift ratio is the ratio of the transient peak story displacement obtained at a particular hazard level to the respective story height. This localized response parameter reveals structural instability and collapse resistance due to P-delta and it is an excellent measure of both structural and non-structural damages due to its close relationship to plastic rotation demands on individual beam-column connection assemblies (FEMA-350 2000). FEMA-273 provided suggested peak interstory drift ratio threshold values for different structural performance levels: 0.7% transient and negligible permanent drift ratio at Immediate Occupancy, 2.5% transient and 1% permanent drift ratio at Life Safety, and 5% transient or permanent drift ratio at Collapse Prevention performance level, respectively. A single scale quantity, i.e., the maximum interstory drift ratio is often used in structural earthquake engineering research, which is taken as the largest value of height-wise peak interstory drift ratios.

System displacement ductility

Based on an SDOF system analogy, system displacement ductility μ is defined as the ratio of the peak displacement Δ_u to the yield displacement Δ_y of a control node (usually the roof node). In the context of the static pushover analysis, the yield displacement corresponds to the intersection point of a bilinear idealization of the original pushover curve (Figure 3.3). System ductility approximately measures the extent of structural damage due to post-yield inelastic deformation in a global sense. A steel design practice usually relies on component ductility to dissipate seismic energy imparted on the structural system. On the other hand, sufficient detailing is necessary to ensure enough deformation capacity for the structural system in order to accommodate a large ductility demand, which will in turn require additional construction efforts.

Other performance indices

In addition to the above displacement-related seismic demands, excessive cumulative deformation/damage demands through cyclic load reversals may also be significant in terms of hysteretic energy dissipation (e.g., Park and Ang 1985). There are other types of structural response parameters such as permanent or residual interstory drifts, plastic hinge rotations, column compressive demands, column splice tensile demands, and so on (Gupta and Krawinkler 1999; FEMA-350 2000). In the present study, the maximum interstory drift demands calculated at 50/50 and 2/50 hazard levels are selected, in accordance with FEMA-350, as the primary seismic performance indices for the multiobjective optimization of steel SMRF structures.

6.3 Numerical Examples

The present multiobjective optimization procedure is now applied for seismic design of the plane five-story four-bay steel SMRF described in Section 3.2, using seismic inputs provided in Section 3.4. DRAIN-2DX is used to obtain the fundamental period, based on which the design base shear and nominal design drift ratios are determined, in accordance with 2000 NEHRP provisions. Relevant MATLAB programming is provided in Appendix F.

6.3.1 Primary and secondary merit measures

The merit objective functions used in the present multiobjective structural optimization are steel material weight, number of different sections types, and maximum interstory drift ratios at both 50/50 and 2/50 hazard levels. All of these four objectives are subject to simultaneous minimization by the genetic algorithm (GA) and are hence referred to as *primary* merit measures. In contrast, parameters that assess other merit aspects of alternative designs but are not used in formulating the present optimization problem are called *secondary* merit measures, including (1) nominal design drift ratios that are calculated by the codified equivalent lateral force procedure, (2) system yield coefficient S_y that is defined as the ratio of the system yield force V_y to the participating building seismic weight W (Figure 3.3), and (3) peak roof drift ratio and system ductility measures at 50/50 and 2/50 hazard levels, respectively.

6.3.2 Distribution of optimized designs with respect to primary merit measures

Using the present GA-based design optimization procedure, a total of 1,560 optimized tradeoff designs are obtained at the 400th generation. This set of designs exhibits a broad distribution over the four primary merit measures, as depicted in Figure 6.2. Steel material weight stays in a range of 128,160 lbs (570.0 KN) and 397,050 lbs (1,765.9 KN); the number of different steel section types, which varies from 2 to 10, indicates optimized designs with diverse degrees of design complexity. FEMA-273 suggested deterministic thresholds for median maximum interstory drift demands for different performance levels; for steel SMRF structures, 0.7% at the IO level and 5% at the CP level are considered acceptable. From Figure 6.2, it is seen that most of the designs satisfy the drift ratio limit at the IO level (paired with 50/50 hazard level) and all designs satisfy the drift ratio limit at the CP level (paired with 2/50 hazard level). Note that drift demands plotted in Figure 6.2 are obtained by the static pushover analysis.

FEMA-350 presented a probabilistic procedure that evaluates structural performances in terms of confidence levels for specified performance levels, taking account the uncertainty and randomness in both structural demand and capacity estimation as well as in seismic excitations. Median interstory drift capacities are 2% and 10% for global behaviors at respective IO and CP performance levels. Minimum recommended confidence levels, in terms of global behavior limited by interstory drifts, are 50% at IO level and 90% at CP level, respectively. Calculation shows that most of the present optimized structural designs have much higher confidence levels of satisfying both IO and CP performance levels than those recommended in FEMA-350, which is in agreement with Lee and Foutch (2002) regarding post-Northridge steel SMRF designs complying with new NEHRP provisions.

Note that seismic performance evaluation in this study is based on a simple structural model. It has been demonstrated that structural responses could be significantly affected by modeling assumptions made in the analysis (Gupta and Krawinkler 1999). The general multiobjective design optimization procedure developed in this chapter, however, can be equally used when a more refined structural model is constructed instead.

6.3.3 Distribution of optimized designs with respect to secondary merit measures

Figure 6.3 shows the dispersion of all 1,560 designs with respect to each secondary merit parameter. It is observed that these designs have maximum nominal design drift ratios well below the 2% threshold per 2000 NEHRP provisions for a codified steel SMRF design investigated in this study, indicating many conservative designs are present in the optimized design population. It is known that drift ratios other than strength requirements usually control the design of seismic structures in practice. Structural designs with very low nominal drift ratios are often revised/discarded in order to achieve a better present economy. In this study, these conservative design solutions are selectively retained (in an optimized design sense) in the present multiobjective optimization due to two reasons: (1) they increase diversity in alternative designs during the optimization process, which makes it more likely for designs to evolve toward better ones; (2) conservative designs in the final results provide additional candidates and their performance merits can be compared with other traditional design candidates, based on which the final structural design will be strived for with more preferred compromise among different objectives.

The system yield coefficient S_y obtained form the static pushover analysis indicates actual structural lateral strength against system yielding and can roughly measure the system overstrength ratio, which is defined as the ratio of ultimate strength to the codified nominal design strength, if additional strength due to positive system strain hardening is ignored. S_y in this study ranges from 0.20 to 0.81 for all these optimized designs, as observed in Figure 6.3. The codified nominal design base shear coefficient for the present five-story frame, using the rationally computed fundamental period with an codified upper bound, is between 0.092 and 0.141 after being increased by 5% to consider effects of accidental torsion, from which one obtains an estimate of system overstrength ratio varying between 2.23 to 5.74. The overstrength ratio of a typical steel SMRF structure is about between 3.3 to 4.5 (Lee and Foutch 2000), which largely results from satisfying stringent drift limitations. In this study, the fact that much stronger than code-required designs exist with nominal design drift ratios far below the 2% threshold explains the presence of large overstrength ratios.

It is noted, however, that structural designs with high yield strength levels usually incur large roof and floor accelerations that decrease occupant comfort during mild excitations and imply potential damages to mounted nonstructural systems when subject to strong ground motions; the associated large base shears also increase internal forces in column members at the base, which imposes difficulty on foundation design. All these concerns should be taken into due consideration when selecting the desirable alternative designs.

As shown in Figure 6.3, system (displacement) ductility measures at the 50/50 hazard level are all equal to unity, indicating that all optimized designs remain effectively elastic; At the 2/50 hazard level, however, system ductility values are more scattered, which implies that these structures will encounter varied inelastic damage severities.

Use of maximum interstory drift ratio as the sole deformation measure may lose information on height-wise drift variation. It may be desirable to design a building structure that has relatively uniform deformation demands over the height in order to avoid soft story mechanisms where drift demands are concentrated in one or only a fewer stories. Since the peak roof drift ratio describes the average height-wise drift demand, a "drift uniformity ratio", which is defined as the ratio of maximum interstory drift ratio to the peak roof drift ratio, may be used to roughly address the concern of how severe the drift concentration is and hence to help selection of design solutions with more desirable deformation patterns. Plotted in Figure 6.4 are such uniformity measures for all 1,560 optimized designs at 50/50 and 2/50 seismic hazard levels, respectively. The fact that a majority of drift uniformity ratios are close to one indicates that drift demands are satisfactorily evenly distributed over the building height for these structural designs.

6.3.4 Minimum material weight designs with varied section type numbers

As discussed before, traditional optimal seismic design of steel structures uses material weight as the sole objective function; degree of design complexity, which leads to additional construction costs, is not usually taken into explicit account. To achieve an economical design regarding the overall initial costs, degree of design complexity in terms of the number of different section types needs to be considered appropriately.

For a given number of different section types, the design with the minimum material weight is identified from the 1,560 designs. Nine designs are thus obtained with the section type number being 2 to 10, respectively. Tables 6.1 and 6.2 provide member sizes and detailed information of these designs, respectively. The relationship between the steel material weight and the number of different section types is plotted in Figure 6.5. It is observed that a tradeoff between these two merit measures exists except for the optimized design with ten different section types. This is because the GA based optimization procedure has not produced lighter designs with more than nine different section types up to the 400th generation.

It is also observed in Figure 6.5 that the minimum material weight is very sensitive to the number of section types when the number is small while it becomes insensitive when the number is relatively large. Specifically, the minimum material weight associated with the optimized design of two section types is 166.6 k-lbs. By simply introducing one, two, and three net new section types, the material weight is reduced by 9.8%, 14.3%, and 19.1%, respectively. However, the reduction rate in material weight becomes very slow when the number of section types is larger than five. For instance, designs with six and nine section types reduce the material weight of the design with two section types only by 20.2% and 23.1%, respectively; the entailed extra construction cost due to the aggravated degree of design complexity in terms of different section type numbers could very likely overwhelm these slight extra savings in the material usage.

By comparing merit measures in Table 6.2 and in Figures 6.2 and 6.3, it is clear that these minimum weight designs are among those with the least seismic resistances. It is also interesting to notice that optimized designs in Table 6.2 are consistently heavier than those in Table 5.1 that were obtained using material weight and number of section types as two objectives. This may indicate that more GA evolution is needed for the present design problem where another set of ‘continuous’ objectives (drift ratios) exists, leading to much more tradeoff designs accordingly.

A design process that simply complies with seismic code provisions does not explicitly consider actual seismic performance or damage implication and is likely ended up with a final design solution based on material usage reduction only. The essence of performance-based seismic design methodology is that realistic seismic structural behaviors are explicitly considered during the design process, which will be illustrated in the following text.

6.3.5 Optimized designs with the same number of different section types

Numerous design solutions exist for a particular number of different section types. For illustration purposes, Figure 6.6 plots all 199 optimized designs with five different section types, which form a subset of the 1,560 alternative designs. There are clear tradeoffs between steel material weight and maximum interstory drift demands. Note that maximum interstory drift ratios at 50/50 and 2/50 hazard levels are more complementary than competing, since they both describe seismic demands on structures.

A structural engineer now has much flexibility in choosing a preferred design other than the minimum weight design with five section types. For example, another alternative design with a material weight of 150,504 lbs (669.4 KN) is selected, which is about 11.7% heavier than the minimum weight design with five different section types (134,786 lbs or 599.5 KN in Table 6.2). As a reward, the maximum interstory drift demands at 50/50 and 2/50 hazard levels are reduced by 20.2% and 22.1%, respectively. Whether or not this particular tradeoff is beneficial will be judged by experienced engineers.

6.3.6 Optimized designs with close material weights

Another design situation is considered where the steel material weight assumes a relatively fixed value, say, $150,000 \pm 2,000$ lbs (667.1 ± 8.9 KN). For a given section type number, the design solution with the lowest maximum interstory drift ratio at the 2/50 hazard level is identified among the 1,560 alternative designs. Six structural designs are thus obtained with member sizes and detailed information in Tables 6.3 and 6.4, respectively. Drift ratio demands vs. number of section types is plotted in Figure 6.7, from which one may visually find that the best compromise design could be, for example, the one with four or five different section types.

6.3.7 Time history analysis of two alternative designs

Two alternative designs are selected herein for detailed time history analysis using sets of SAC ground motion records representative of 50/50 and 2/50 hazard levels, respectively (Somerville et al. 1997). *Design I* is the one with the overall minimum material weight of 128,160 lbs (570.0 KN) with nine different section types (refer to Tables 6.1 and 6.2), and *Design II* has steel material weight close to 150 k-lbs (specifically 151,796 lbs or 675.1 KN) with the lowest maximum interstory drift demands among all optimized designs of four different section types (refer to Tables 6.3 and 6.4).

For each design, time history analysis is performed using twenty ground motion records at each hazard level. The peak absolute interstory drift ratio for each story from a single time history analysis is selected and, as a result, there are in total twenty peak interstory drift ratios for each story. Assuming a lognormal distribution of each interstory drift demand, the sample median (50th percentile) drift demand is calculated as the exponent of sample mean of the natural logarithmic of drift demands from all time history analyses, i.e., $D_{median}=\exp\left[\left(\sum_{i=1}^{N}\ln D_i\right)/N\right]$; the 84th and 95th percentile sample drift demands are obtained by multiplying the sample median value by the exponent of one and two times sample standard deviation of the natural logarithmic of drift demands from all time history analyses, respectively, i.e., $D_{84th}=D_{median}\exp(\beta)$ and $D_{95th}=D_{median}\exp(2\beta)$ with $\beta=\left[\left(\sum_{i=1}^{N}\left(\ln D_i-\ln D_{median}\right)^2\right)/(N\text{-}1)\right]^{1/2}$.

Height-wise nominal design drift ratio profiles based on codified elastic analysis and normalized static pushover curves for these two alternative designs are plotted in Figures 6.8 and 6.9, respectively. Figure 6.10 shows the median, 84th percentile, and 95th percentile of peak interstory drift ratio demand profiles that are obtained using time history analyses for these two designs at 50/50 and 2/50 hazard levels, respectively; the respective maximum drift demands are excerpted and reported in Figure 6.11.

It is observed that Design II consistently has a higher seismic capacity than Design I. In terms of median maximum interstory drift demands at 50/50 and 2/50 hazard levels, for example, the values for Design II are 0.88% and 3.15%, respectively, which represent 21.4% and 20.5% less than the median maximum interstory drift demands of 1.12% and 3.96% for Design I, respectively. These observations imply that Design II would perform better seismically and therefore will incur less potential seismic damages than Design I. Combing this ascertained knowledge of structural performances with the fact that Design II is 23,636 lbs (105.1 KN) or 18.4% heavier than Design I while Design I has five more section types than Design II, a structural engineer is then expected to make decision on the choice of the final structural design.

6.4 Summary

Design of economical seismic structures necessitates a balanced minimization of two general competing objectives: the present capital investment and the future seismic risk. Many of the existing optimal seismic design procedures are single-objective based with structural material usage as the sole objective function while imposing constraints from code specifications and possibly relevant structural performance consideration. In contrast to stringent constraints in traditional prescriptive seismic codes, acceptable performance parameters recommended in recent performance-based seismic design guidelines are best interpreted as indicative of performance ranges that a structure may sustain when responding at different performance levels. For performance-based seismic design optimization, a natural approach is therefore to consider structural performance indices and other applicable merit measures as objective functions other than constrains.

An automated procedure has been presented in this chapter that combines performance-based seismic design methodology and genetic algorithms (GAs) for optimized member sizing of steel special moment resisting frames in accordance with the 2000 NEHRP seismic structural design criteria, AISC-LRFD steel design criteria, and AISC seismic design criteria. Merits of a code-compliant design are assessed by multiple objective functions, which reflect either initial expenses in terms of steel material weight and number of different steel section types or future seismic risks in terms of interstory drift demands at selected seismic hazard levels.

By treating all selected objective functions simultaneously as well as separately, the present GA based multiobjective design optimization procedure produces a wide distribution of alternative designs that establishes optimized tradeoff among these merit objectives. Consequently, structural engineers have a much broader vision of the entire optimized valid design space other than a single structural design that is obtained from traditional single objective based structural optimization. Through an explicit tradeoff analysis of design candidates that are preliminarily selected from the optimized design pool, engineers are able to conveniently determine the final compromise design that has desirable seismic behavior with balanced initial expenses.

This study has focused on development of general multiobjective optimization procedures for performance-based seismic structural designs and a simple centerline structural model is used herein for seismic performance evaluation. A more refined structural model incorporating, for example, panel zone deformation, realistic connection representation, strength and stiffness participation of the gravity frames, and other contributing factors will improve the accuracy of seismic structural response estimates and hence the quality of optimized tradeoff designs.

Table 6.1 Member sizes for minimum weight designs with varied section type numbers

Section Group ID	# of Section Types	2	3	4	5	6	7	8	9	10
	C1	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X257	W14X257	W14X257
	C2	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342
	C3	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342
	C4	W14X342	W14X132	W14X145	W14X145	W14X145	W14X132	W14X132	W14X132	W14X145
	C5	W14X342	W14X342	W14X233	W14X211	W14X211	W14X233	W14X233	W14X233	W14X233
	C6	W14X342	W14X342	W14X342	W14X342	W14X342	W14X257	W14X257	W12X230	W14X283
	B1	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W33X130	W33X130	W30X124
	B2	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W33X118
	B3	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X108
	B4	W30X99	W30X99	W30X99	W24X68	W24X68	W24X68	W24X68	W24X68	W24X68
	B5	W30X99	W30X99	W30X99	W24X68	W18X50	W18X55	W18X55	W18X55	W18X55

Table 6.2 Merit measures of minimum weight designs with varied section type numbers

Number of section types	Minimum material weight (lbs)	System yield coeff.	Max. nominal design drift ratio (%)	50/50 hazard level				2/50 hazard level			
				Max. interstory drift ratio (%)	Peak roof drift ratio (%)	Drift uniformity ratio	System ductility	Max. interstory drift ratio (%)	Peak roof drift ratio (%)	Drift uniformity ratio	System ductility
2	166,570	0.239	1.78	0.82	0.59	1.39	1.00	3.31	2.23	1.48	2.83
3	150,190	0.235	1.73	0.79	0.59	1.34	1.00	3.35	2.25	1.49	2.83
4	142,702	0.240	1.82	0.80	0.61	1.31	1.00	3.48	2.35	1.48	2.69
5	134,786	0.223	1.84	0.78	0.65	1.21	1.00	3.33	2.46	1.35	2.79
6	132,986	0.210	1.82	0.75	0.65	1.16	1.00	3.00	2.45	1.23	2.98
7	130,873	0.215	1.85	0.78	0.66	1.18	1.00	3.14	2.50	1.25	2.87
8	129,213	0.228	1.87	0.77	0.64	1.21	1.00	3.16	2.44	1.29	2.78
9	128,160	0.228	1.92	0.79	0.64	1.23	1.00	3.21	2.46	1.30	2.76
10	133,441	0.249	1.77	0.71	0.63	1.13	1.00	3.07	2.42	1.27	2.69

Table 6.3 Member sizes for designs of steel material weight close to 150 k-lbs with lowest maximum interstory drift demands and varied section type numbers

	# of Section Types	3	4	5	6	7	8
Section Group ID	C1	W14X342	W14X342	W14X342	W14X257	W14X257	W14X257
	C2	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342
	C3	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342
	C4	W14X132	W14X159	W14X159	W14X145	W14X193	W14X159
	C5	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342
	C6	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342
	B1	W30X99	W33X118	W33X118	W33X130	W33X130	W33X130
	B2	W30X99	W33X118	W33X118	W33X130	W33X130	W36X135
	B3	W30X99	W33X118	W33X118	W33X118	W33X118	W33X118
	B4	W30X99	W24X68	W27X84	W24X68	W27X84	W30X99
	B5	W30X99	W24X68	W21X50	W24X68	W21X50	W12X50

Table 6.4 Merit measures for designs of steel material weight close to 150 k-lbs with lowest maximum interstory drift demands and varied section type numbers

Number of section types	Minimum material weight (lbs)	System yield coeff.	Max. nominal design drift ratio (%)	50/50 hazard level				2/50 hazard level			
				Max. interstory drift ratio (%)	Peak roof drift ratio (%)	Drift uniformity ratio	System ductility	Max. interstory drift ratio (%)	Peak roof drift ratio (%)	Drift uniformity ratio	System ductility
3	150,190	0.235	1.73	0.79	0.59	1.34	1.00	3.35	2.25	1.49	2.83
4	151,796	0.281	1.56	0.63	0.58	1.09	1.00	2.72	2.26	1.20	2.58
5	151,596	0.279	1.48	0.63	0.56	1.13	1.00	2.58	2.18	1.18	2.67
6	148,344	0.294	1.57	0.63	0.58	1.09	1.00	2.60	2.25	1.16	2.53
7	151,888	0.293	1.44	0.61	0.56	1.09	1.00	2.45	2.17	1.13	2.62
8	151,236	0.308	1.57	0.60	0.57	1.05	1.00	2.44	2.22	1.10	2.49

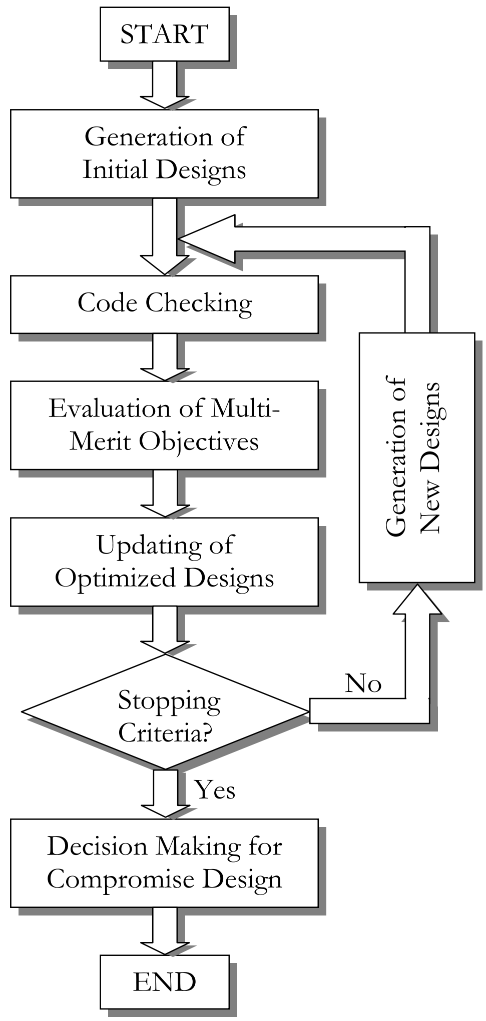
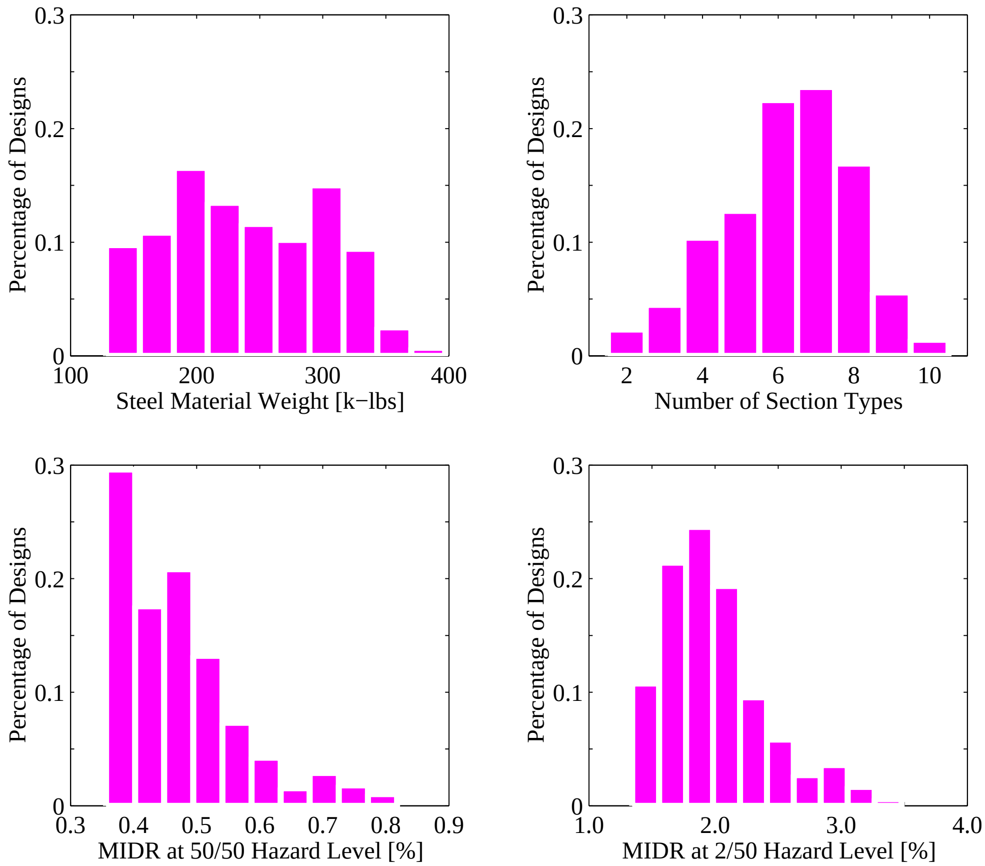


Figure 6.1 Flowchart of the automated design procedure



MIDR = maximum interstory drift ratio

Figure 6.2 Distribution of all optimized tradeoff designs at the 400th generation with respect to each primary merit measure

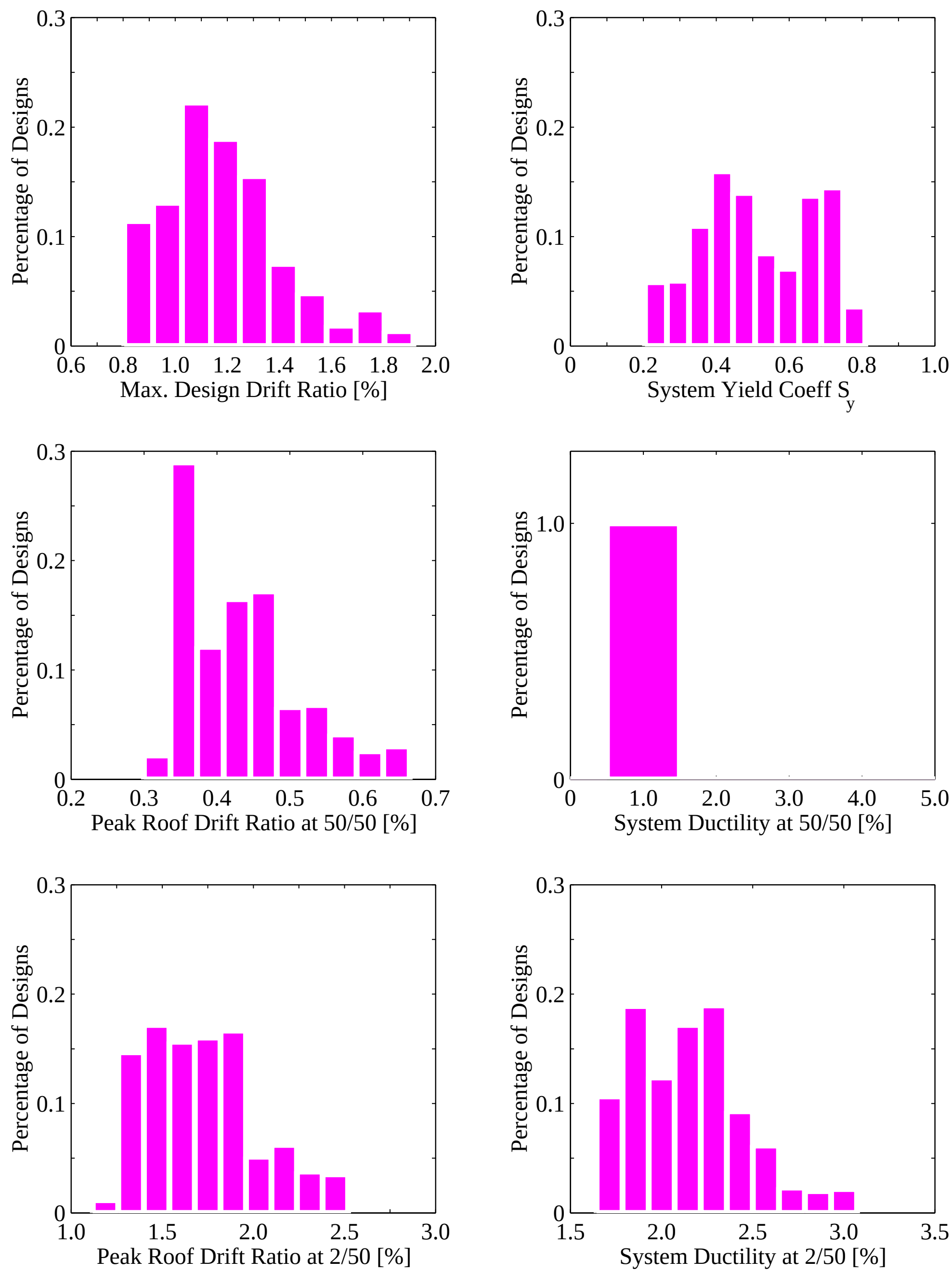


Figure 6.3 Distribution of all optimized tradeoff designs at the 400th generation with respect to each secondary merit measure

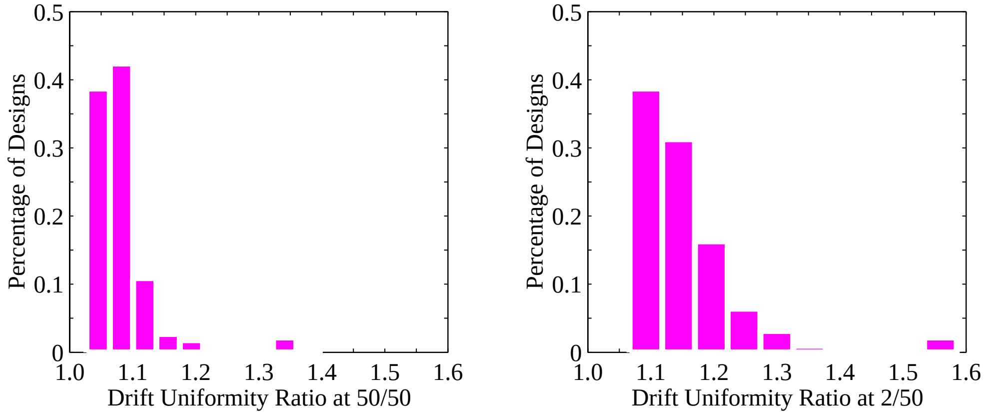


Figure 6.4 Drift uniformity ratios of all optimized tradeoff designs at the 400th generation

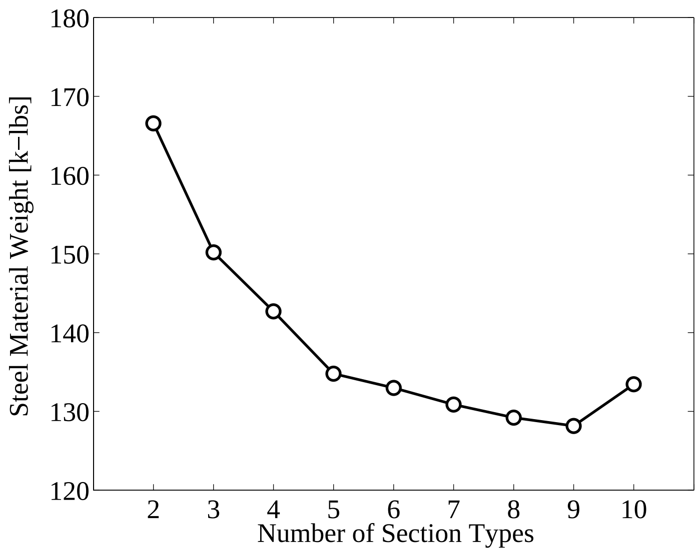
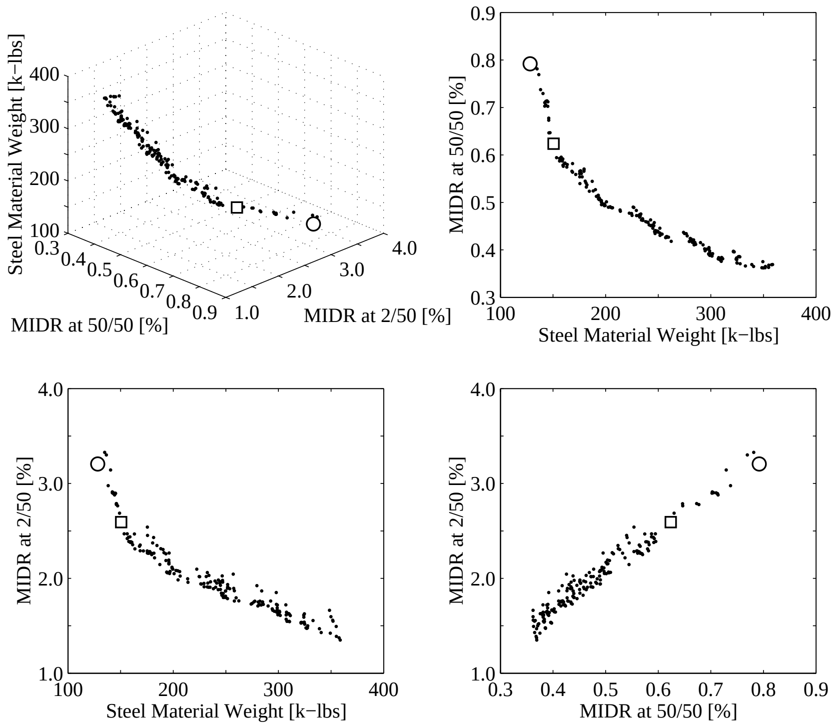


Figure 6.5 Minimum weight designs with varied degrees of design complexity at the 400th generation



MIDR = Maximum Interstory Drift Ratio
○ = Design with minimum steel material weight □ = An alternative design

Figure 6.6 Optimized tradeoff designs with five section types at the 400th generation

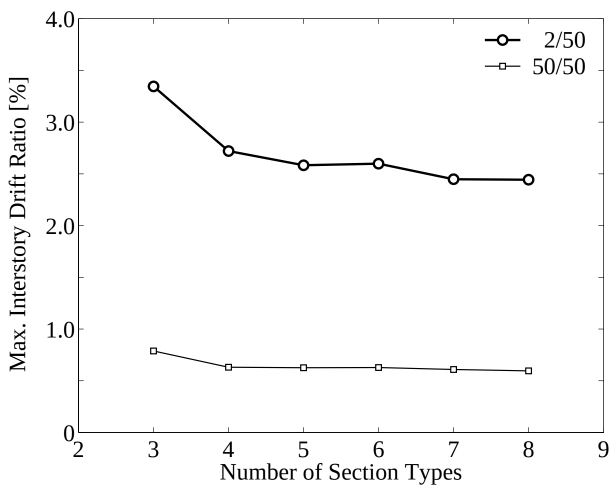


Figure 6.7 Comparison of designs of steel material weight close to 150 k-lbs with lowest maximum interstory drift demands and varied section type numbers

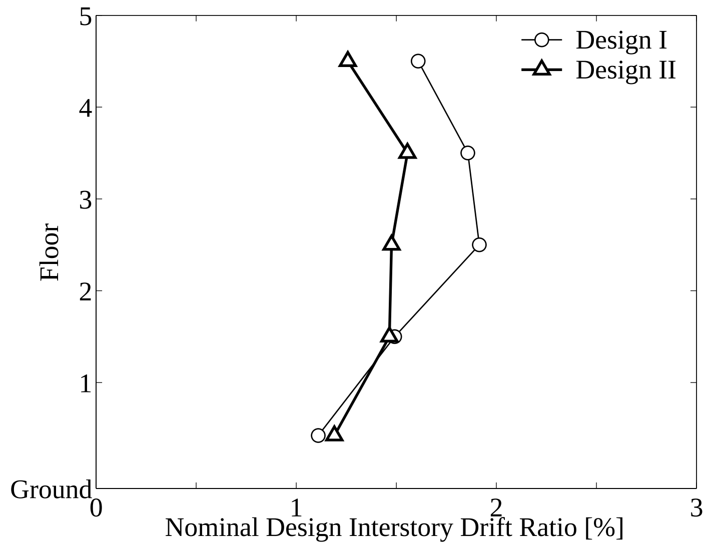


Figure 6.8 Nominal design drift ratio profiles for two alternative designs

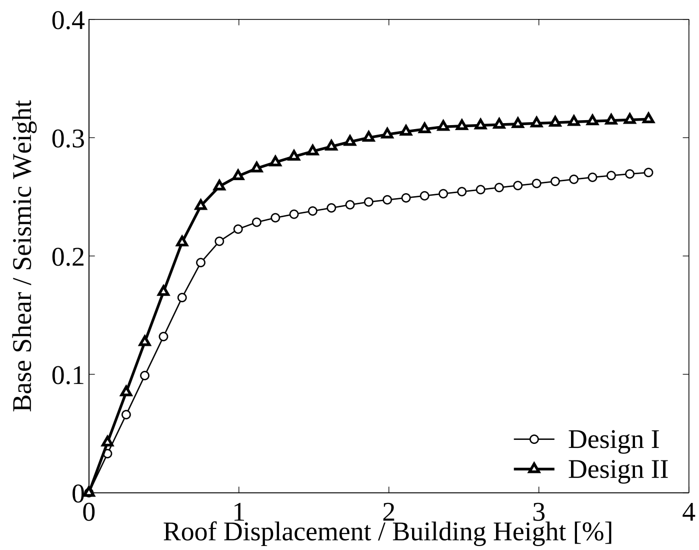


Figure 6.9 Normalized static pushover curves for two alternative designs

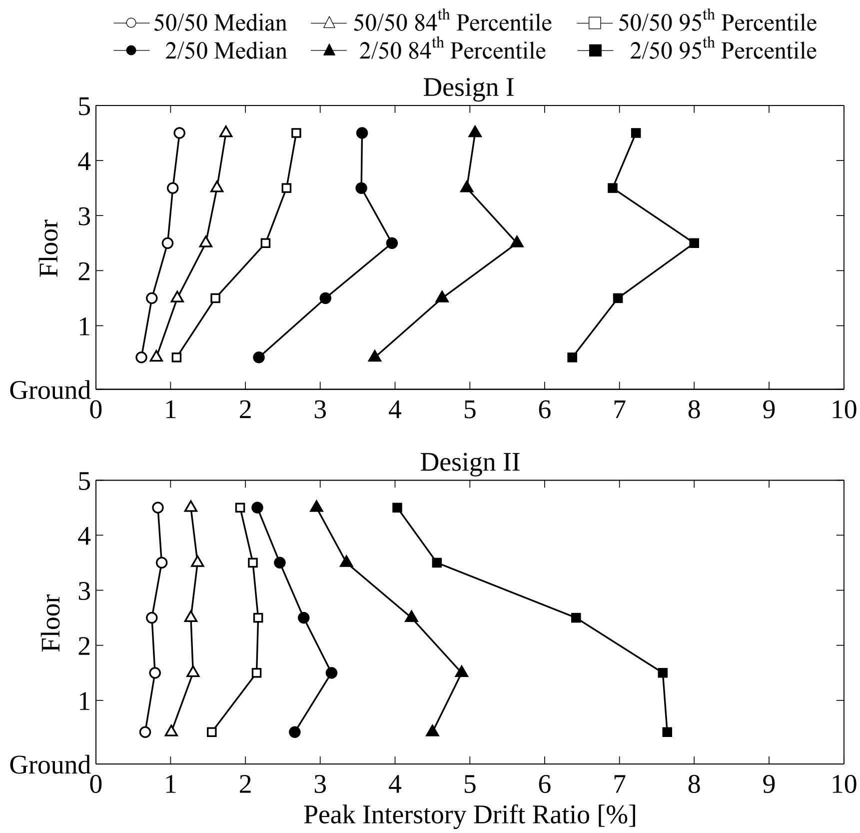


Figure 6.10 Peak interstory drift demand profiles at different hazard levels for two alternative designs by time history analysis

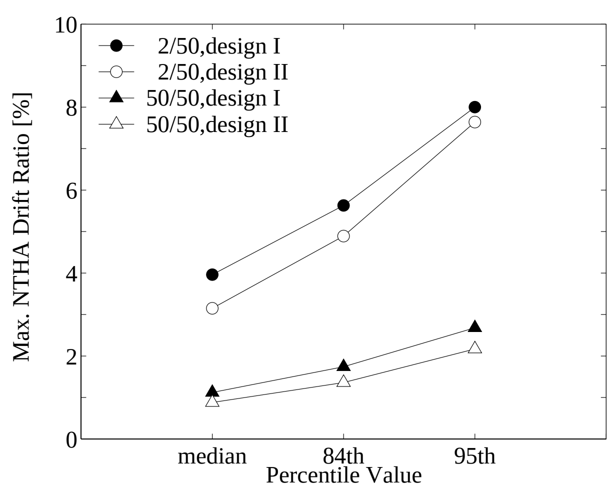


Figure 6.11 Maximum interstory drift demands at different hazard levels for two alternative designs by time history analysis