CHAPTER 5

CONSTRUCTION-CONSCIOUS MINIMUM WEIGHT SEISMIC DESIGN

Abstract: The minimum weight criterion, which has been widely adopted in the literature for optimal design of civil structural systems, is inadequate to fully reflect the initial capital investment due to its lack of consideration of additional construction cost resulting from varied degree of design complexity such as different member sections and splice/connection types. In this chapter, design optimization of seismic steel moment frames involve simultaneous minimization of two competing criteria: the steel material weight and an approximate measure of design complexity in terms of the number of different standard steel member section types. The present code-compliant seismic structural design follows the equivalent lateral force procedure of the 2000 NEHRP seismic provisions in conjunction with AISC-LRFD seismic steel design criteria. A genetic algorithm is used for the posed bi-objective structural optimization problem to produce a set of design solutions establishing optimized tradeoff between the two selected merit objectives. A minimum weight design equipped with appropriate degree of design complexity is expected to achieve initial investment economy with more accuracy.

5.1 Introduction

A code-conforming structural design with reduced use of resources is appealing to the civil engineering profession. To implement this design philosophy, a minimum weight criterion has been most commonly adopted in structural optimization community to deal with design problems with homogenous material such as steel. In particular, weight minimization of steel moment frame structures subject to different loading scenarios has been fruitfully investigated by many researchers using various numerical techniques based on either optimality criteria/mathematical programming (e.g., Cameron et al. 1992; Pezeshk 1998) or evolutionary computation such as simulated annealing (Balling 1991) and genetic algorithms (GAs) (e.g., Pezeshk et al. 2000; Hayalioglu 2001). For design optimization of earthquake-resistant structures, seismic loads may be represented either by accelerograms (Papadrakakis et al. 2001) or by equivalent lateral forces prescribed by code provisions (Memari and Madhkhan 1999). In addition, semi-rigid connections have also been considered in optimal steel design problems (Xu et al. 1995; Foley and Schinler 2003).

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 cost is dependent not only upon the amount of material usage but also upon the degree of design complexity for a particular steel frame system. Consequently, an ‘optimal’ design solution resulting from minimizing material weight alone may not necessarily correspond to the one with the realistic least total initial monetary costs. Rather, in order to strive for the minimum weight objective, a variety of different member sections tend to be employed, which usually necessitates more column splices, design detailing, and other labor-intensive construction operations. All of these factors will undoubtedly increase the initial investment. In addition, welded column splices are very prone to fracture when subjected to large tensile seismic loads, which may lead to costly column damage retrofitting efforts in order to prevent potential safety consequences (FEMA-352 2000). Furthermore, it is sometimes worthwhile to use heavier members than necessary when expensive connection details (e.g., doubler plates and stiffeners) can be avoided as a result (Carter et al. 2000).

According to Carter et al. (2000), the labor cost associated with constructing a steel frame building framework has grown in recent decades to where it is now approximately 60% of the total construction cost; nearly half of this labor cost involves shop labor, such as prefabricating subassemblies and painting, and the other half involves erection labor. Costs from material usage have dropped from 40% of the total cost two decades ago to 26% nowadays (Carter et al. 2000). This observation suggests that degree of complexity of the design, fabrication, and erection stages of the building project be appropriately accounted for in order to prevent excessive entailed construction costs.

The most accurate approach could be direct quantification of total initial monetary cost that covers all possible expenses from material usage to construction stage. Material cost is relatively easy to quantify based on well-documented sources (e.g., Nucor 1999). The related construction costs, however, are difficult to calculate and are therefore largely estimated by engineering experiences. Some attempts have been made in the literature to address this issue in an approximate manner. For example, Carter (1999) presented empirical equations to convert use of doublers and stiffeners into equivalent steel usage; Xu et al. (1995) used beam weight modification factors to account for beam-to-column connections of varied rigidities.

Alternatively, degree of design complexity for steel frame structures may be reflected directly albeit roughly by the number of different standard beam/column section types (Sarma and Adeli 2000; Greiner et al. 2001). There are several advantages to minimize the number of different member sizes specified in a structural steel design: (1) larger quantities of a single member size become more economical, particularly if mill order quantities (typically 20 tons) can be achieved; (2) the complexity of inventory control, fabrication, and erection is reduced when fewer different sections are being handled, having a direct impact on labor cost; (3) the probability of erection error is reduced, avoiding costly rework; and (4) more connections can be duplicated and column splices can be simplified or even eliminated.

Design optimization of steel moment frame structures will be investigated in this chapter that considers two competing merit objective functions: the steel material weigh and an approximate design complexity measure in terms of the number of different standard member section types. Due to unit price discrepancies for different section types and for different strength grades (Nucor 1999), a steel material cost measure would be more appropriate than the steel material weight measure for describing the actual total steel usage. In this study, however, the steel material weight measure will still be considered in conjunction with the design complexity measure so that a direct extension of the existing minimum weight design procedures will be easily made and distinct features of the present study will be clearly identified.

5.2 Problem statement

The present study focuses on optimized member sizing for seismic design of plane steel special moment-resisting frames (SMRFs) with a given geometric layout while simultaneously minimizing both steel material weight and number of different steel section types. The posed multiobjective structural optimization problem can be conceptually stated as

Goal	To obtain a set of optimized SMRF designs with respect to both steel material weight and number of different steel section types.
Subject to	2000 NEHRP seismic structural design criteria;
	AISC-LRFD steel design criteria;
	AISC seismic design criteria.

Detailed design requirements from the above code provisions have been provided in Chapter 2, where the 2000 NEHRP equivalent lateral force procedure is used to design the SMRF building structure and the relevant design constraints include nominal design drift ratio limit, strength limit due to axial force and bending moment interaction, strong-column-weak-beam criterion, and slenderness ratio limits for steel member sections. The plane five-story four-bay steel SMRF design described in Section 3.2 will be used as the example structure in this numerical example. Information on seismic inputs has been discussed in Section 3.4. The multiobjective GA presented in Section 4.3 is used as the primary tool to solve the present SMRF design optimization problems. Note that for the current bi-objective design optimization, no unique design solution exists that can optimize both objectives at the same time; instead, a set of structural designs are to be obtained that exhibits an optimized tradeoff with respect to these conflicting merit objectives.

5.3 Numerical examples

5.3.1 Analysis of optimized tradeoff structural designs

The GA based automated design procedure is now used for optimized member sizing of the plane five-story four-bay SMRF structure. In this section, the fundamental period is obtained from a natural vibration analysis with DRAIN-2DX to determine design base shear level (with an upper bound on the calculated period) as well as to calculate the nominal design drift ratios (without upper bound on the calculated period). Relevant MATLAB (2001) programming is provided in Appendix F.

Figures 5.1 and 5.2 show the evolution of the number of total and new optimized tradeoff designs, respectively, as the generation progresses. It is observed that most of the optimized designs are obtained within the first 100 generations, after which the evolution is practically insignificant and the resulting optimized designs may be satisfactorily accepted as the final tradeoff solutions. This statement is again illustrated in Figure 5.3 that provides a clear generation-wise evolution history of the optimized tradeoff curve.

Eight final optimized design solutions are thus obtained at the 150th generation and are sketched in Figure 5.4 where member thickness is plotted proportional to the respective member sectional modulus. For each number of section types, the present optimized structural design owns the least material weight among all designs with the same particular section type number. Table 5.1 provides detailed member size information for these structural designs. It is apparent that, for optimized designs with fewer different section types, the total steel material weight is relatively sensitive to the number of section types. For example, the material weight associated with the optimized design of two section types is 166,570 lbs; by employing one and two more section types, the material weight is reduced by 12.8% and 17.5%, respectively. In contrast, for optimized designs with more than four different section types, dependence of material weight on the section type member becomes very weak. For example, optimized designs with five and nine different section types have a material weight of 130,037 lbs and 125,433 lbs, respectively, which indicate only 5.4% and 8.7% reduction, respectively, of the material weight of the optimized design with four different section types.

In the traditional minimum weight design procedures, degree of design complexity is usually not taken into explicit account. As a result, the final single optimized design is most likely composed of many different section types. As shown in Table 5.1, the optimized design with nine different section types has the overall least material weight of 125,433 lbs. The tradeoff analysis made in the above paragraph indicates that it is possible to have a slightly heavier optimized design solution while the number of different section types, i.e., the approximate measure of design complexity can be significantly reduced, which will expectedly lead to reasonable savings in construction expenses. Whether or not this construction cost reduction outweighs the increase in steel material usage would be judged by experienced structural engineers in order to seek for an overall economy in construction of new steel SMRF structures.

It should be emphasized that the present optimized tradeoff structural designs are obtained using a particular member-grouping pattern (Figure 3.2). Different sets of optimized designs would be expected if other grouping patterns compatible with different construction scenarios were used instead. Unless these constraints on member sizing are considered explicitly in structural optimization, it is unlikely that final optimized designs will automatically satisfy such specific construction constraints. If constraints are enforced after the optimization process is terminated, for instance, by averaging member sections in the same group to the nearest available discrete steel section type, by no means would it guarantee a code-compliant design solution, let alone an optimized one.

5.3.2 Comparison of optimized designs with different period calculation

The fundamental structural period was computed by rational analysis in Section 5.3.1. The 2000 NEHRP also permits use of empirical equations of the fundamental period to determine design base shear level as well as nominal design drift ratios. The design optimization is now performed using empirical fundamental periods for steel SMRF structures. All GA parameters are the same as used in Section 5.3.1.

The generation-wise evolution histories of the number of total and new optimized designs are plotted in Figures 5.5 and 5.6, respectively. Figure 5.7 presents generation-wise updating of the optimized tradeoff curve. Similar trend as in Section 5.3.1 can be observed, i.e., most of the satisfactory optimized designs are obtained within early generations, especially for designs with smaller number of different section types.

Eight final optimized design solutions are thus obtained at the 200th generation with detailed information provided in Table 5.2 and are sketched in Figure 5.8. Compared to optimized designs based on rationally calculated fundamental periods (Table 5.1), the present optimized designs are consistently heavier than their counterparts, as is also shown in Figure 5.9. This is because the empirical period is in general smaller than the rationally calculated period and hence a higher design base shear level is resulted; consequently, stronger and thus heavier optimized design solutions are naturally obtained.

5.4 Summary

It has been shown in this chapter that traditional optimal seismic steel frame design procedures based on a minimum weight (or more precisely minimum material cost) criterion alone very likely lead to a single final structural design solution that involves a large number of different member section types. The associated high degree of design complexity implies that more labor-intensive construction operations/costs are required and, as a result, a traditional minimum weight SMRF design may be far from the one with the minimum total initial capital cost.

By introducing the other separate merit objective function, the number of different standard steel section types that roughly reflects the degree of design complexity, the present genetic algorithm based bi-objective optimization procedure produces a set of structural designs that exhibit optimized tradeoff between steel material weight and design complexity. Structural engineers then play an active role in selecting a compromise design that balances these two competing objectives in the preferred manner. For instance, a reasonably heavier (than minimum weight) design could be the more economical design solution from an easy-to-build perspective.

The goal of the present study is to integrate construction concerns into the design optimization process so that the resulting optimized structural designs will be more likely viable in a real-world civil engineering practice. Future research in this direction is warranted. For example, it would be ideal if design/construction complexity could be quantified by monetary values with degree of accuracy comparable to that of material weight quantification. This effort will lead to a more convenient and straightforward unified minimum initial cost design formulation.

It is worth noting that construction-conscious design considerations often imply further explicit constraints on member diversity of the structural system. For example, one may require that all beam depths be limited to, say, 30” and all columns be designed as, say, 14” wide flange sections. To further ease the construction, all beams on some neighboring floors may be linked with the same member section. All these constraints can be incorporated into the present optimization procedure without any difficulty.

Table 5.1 Optimized tradeoff design solutions using rationally calculated periods

	# of Section Types	2	3	4	5	6	7	8	9
Section Group ID	C1	W14X342	W14X342	W14X342	W14X342	W14X311	W14X311	W14X311	W14X283
	C2	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342
	C3	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342	W14X342
	C4	W14X342	W14X233	W14X132	W14X132	W14X132	W14X132	W14X132	W14X132
	C5	W14X342	W14X233	W14X233	W14X233	W14X233	W14X233	W14X233	W14X233
	C6	W14X342	W14X233	W14X233	W14X233	W14X233	W14X233	W14X233	W14X233
	B1	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X108
	B2	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99	W30X99
	B3	W30X99	W30X99	W30X99	W30X99	W30X99	W27X84	W27X84	W27X84
	B4	W30X99	W30X99	W30X99	W24X62	W24X62	W24X62	W24X62	W24X62
	B5	W30X99	W30X99	W30X99	W24X62	W24X62	W24X62	W21X50	W18X55
Steel Material Weight [lbs]		166,570	145,315	137,437	130,037	128,301	126,801	125,601	125,433

Table 5.2 Optimized tradeoff design solutions using empirical periods

	# of Section Types	2	3	4	5	6	7	8	9
Section Group ID	C1	W14X455	W14X455	W14X455	W14X455	W14X455	W14X342	W14X398	W14X342
	C2	W14X455	W14X455	W14X455	W14X455	W14X455	W14X455	W14X455	W14X455
	C3	W14X455	W14X455	W14X455	W14X455	W14X455	W14X455	W14X455	W14X455
	C4	W14X455	W14X159	W14X311	W14X176	W14X159	W14X159	W14X159	W14X159
	C5	W14X455	W14X455	W14X311	W14X342	W14X311	W14X311	W14X311	W14X311
	C6	W14X455	W14X455	W14X311	W14X342	W14X311	W14X311	W14X311	W14X311
	B1	W33X118	W33X118	W33X118	W33X118	W33X118	W33X141	W33X118	W33X141
	B2	W33X118	W33X118	W33X118	W33X118	W36X135	W33X141	W36X135	W36X135
	B3	W33X118	W33X118	W33X118	W33X118	W33X118	W33X118	W33X118	W33X118
	B4	W33X118	W33X118	W33X118	W33X118	W33X118	W33X118	W27X84	W27X84
	B5	W33X118	W33X118	W18X50	W24X55	W18X50	W18X50	W24X55	W24X55
Steel Material Weight [lbs]		213,925	190,837	179,045	172,642	168,889	165,461	162,797	161,961

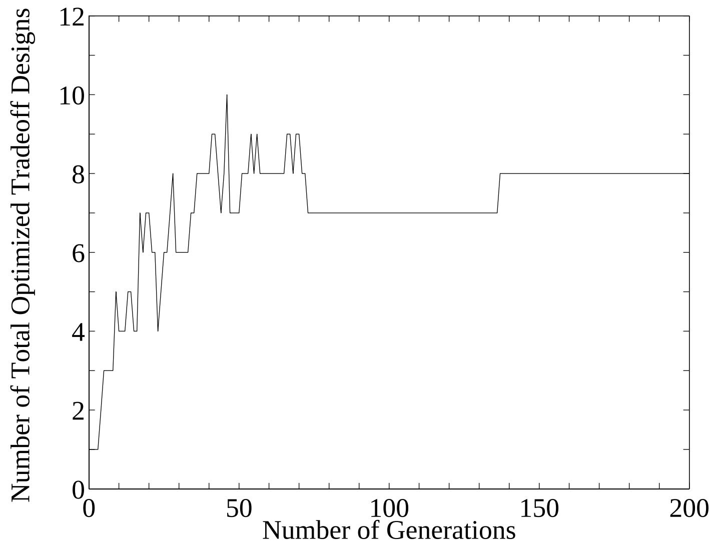

Figure 5.1 Generation-wise evolution of the number of total optimized tradeoff designs using rationally calculated fundamental periods

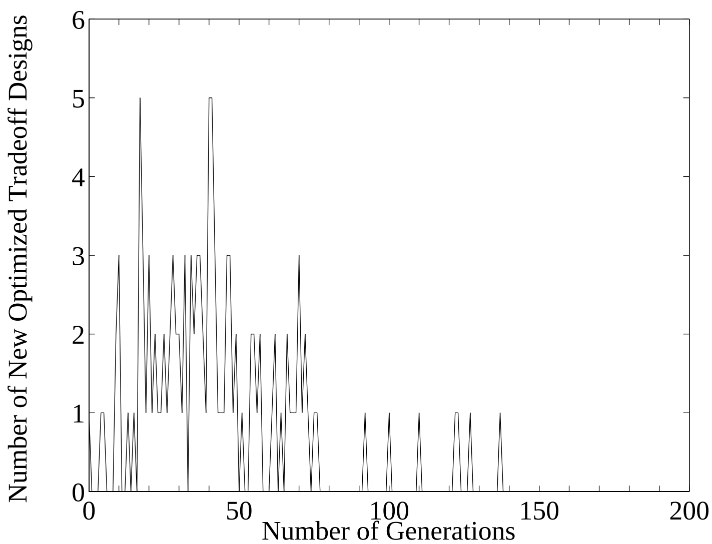

Figure 5.2 Generation-wise evolution of the number of new optimized tradeoff designs using rationally calculated fundamental periods

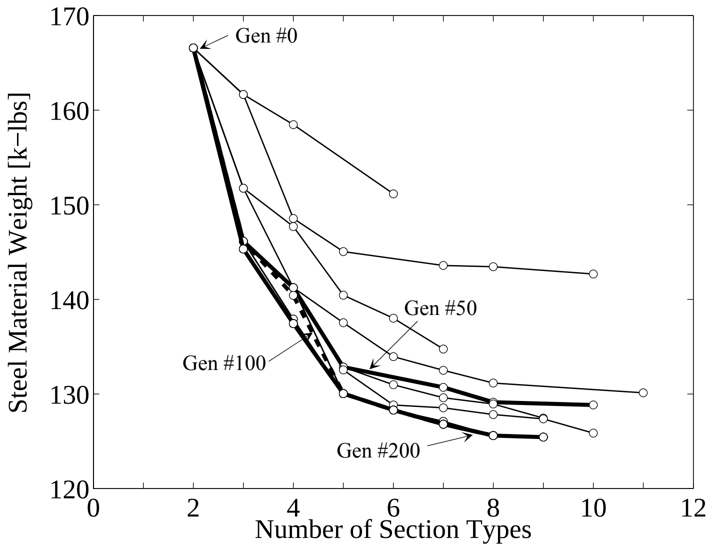

Figure 5.3 Generation-wise evolution of optimized tradeoff designs using rationally calculated fundamental periods

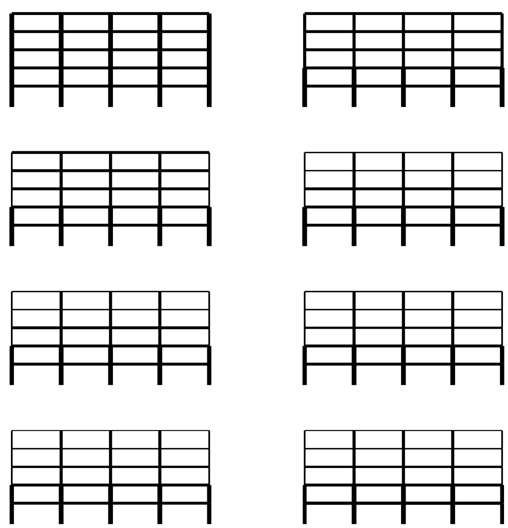

Figure 5.4 Sketch of optimized tradeoff designs using rationally calculated fundamental periods

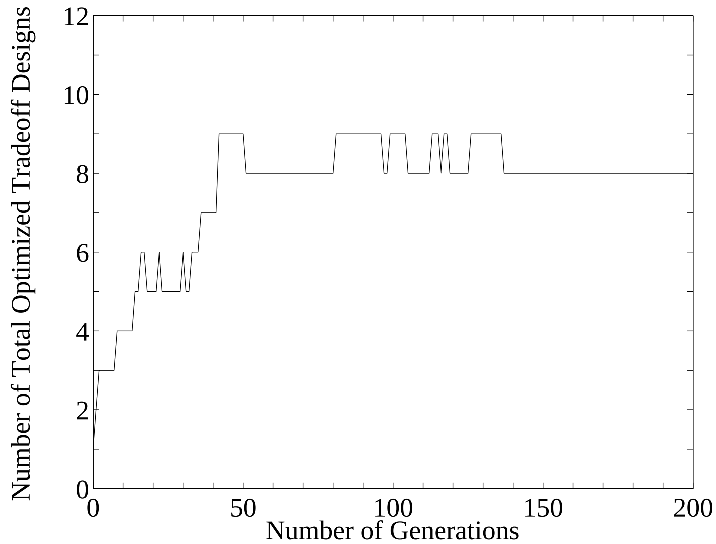

Figure 5.5 Generation-wise evolution of the number of total optimized tradeoff designs using empirical fundamental periods

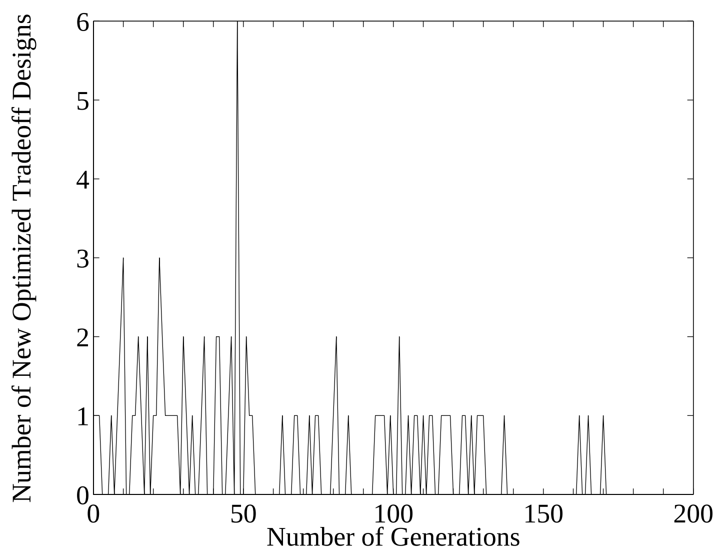

Figure 5.6 Generation-wise evolution of the number of new optimized tradeoff designs using empirical fundamental periods

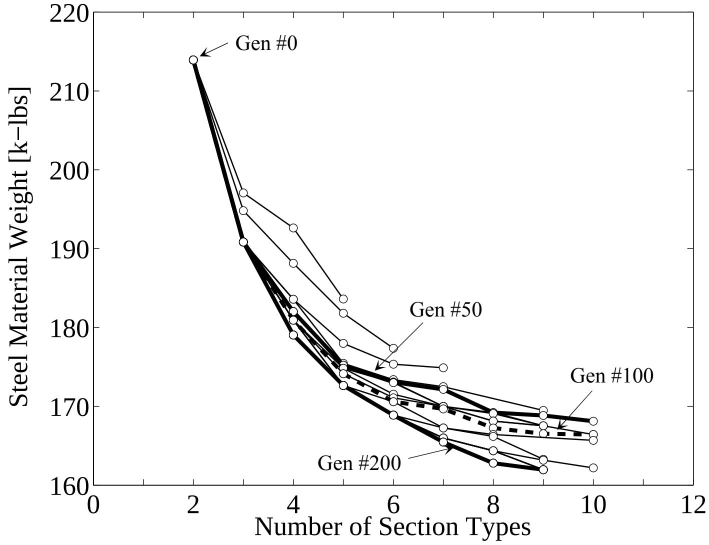

Figure 5.7 Generation-wise evolution of optimized tradeoff designs using empirical fundamental periods

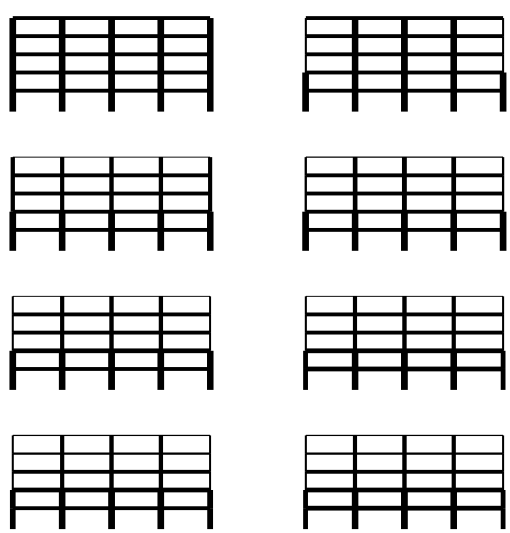

Figure 5.8 Sketch of optimized tradeoff designs using empirical fundamental periods

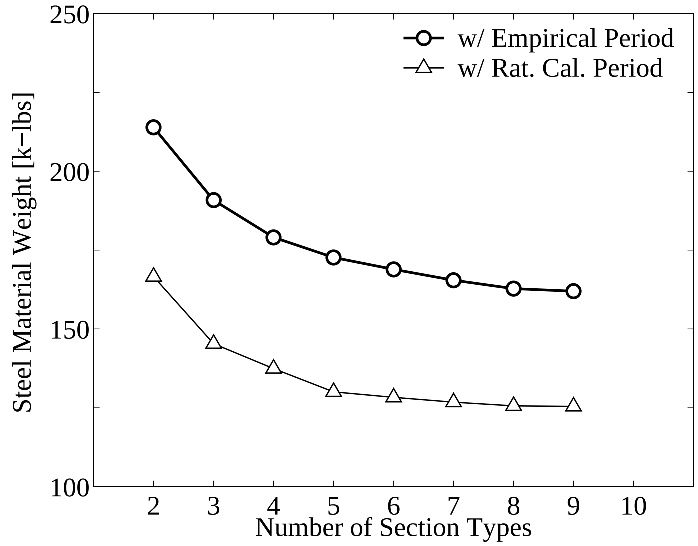

Figure 5.9 Comparison of optimized tradeoff designs obtained using different fundamental period calculations