Structural Engineering and Mechanics, Vol. 43, No. 1 (2012) 31-51
DOI: http://dx.doi.org/10.12989/sem.2012.43.1.031

The effects of construction related costs on the optimization of steel frames

Byoung-Han Choi*[1], Abhinav Gupta[2a] and John W. Baugh Jr.[2a]

[1]*Korea Rural Research Institute, Ansan 426-908, Korea*
[2]*Civil Engineering, North Carolina State Univ. Raleigh, NC 27695-7908, USA*

(Received June 22, 2011, Revised April 2, 2012, Accepted May 17, 2012)

Abstract. This paper presents a computational study that explores the design of rigid steel frames by considering construction related costs. More specifically, two different aspects are investigated in this study focusing on the effects of (a) reducing the number of labor intensive rigid connections within a frame of given geometric layout, and (b) reducing the number of different member section types used in the frame. A genetic algorithm based optimization framework searches design space for these objectives. Unlike some studies that express connection cost as a factor of the entire frame weight, here connections and their associated cost factors are explicitly represented at the member level to evaluate the cost of connections associated with each beam. In addition, because variety in member section types can drive up construction related costs, its effects are evaluated implicitly by generating curves that show the trade off between cost and different numbers of section types used within the frame. Our results show that designs in which all connections are considered to be rigid can be excessively conservative: rigid connections can often be eliminated without any appreciable increase in frame weight, resulting in a reduction in overall cost. Eliminating additional rigid connections leads to further reductions in cost, even as frame weight increases, up to a certain point. These complex relationships between overall cost, rigid connections, and member section types are presented for a representative five-story steel frame.

Keywords: construction related costs; optimization of moment resisting steel frames; connection cost; member section types

1. Introduction

Numerous optimization approaches have been employed over the years for economical design of moment-resisting steel frames. Historically, many of these efforts focused on minimum weight designs because of the high material cost associated with structural steel. In recent years, however, steel material costs have experienced a significant decline whereas labor costs associated with construction aspects such as production and erection have experienced a significant increase. Consequently, there is greater need to consider construction related costs in a design formulation. Ideally, construction costs should be included explicitly within the objective function by considering respective contributions from detailing, production, erection, finishing, etc. Development of a

*Corresponding author, Senior Researcher, E-mail: han1020@hotmail.com
[a]Professor

complete yet generalized cost function, however, may be impractical for a variety of reasons, including (a) local site conditions such as tight urban settings, which may govern the use of certain production and erection techniques, (b) geographical variations in the available labor and transportation costs, and (c) difficulty in quantification of labor estimates for welded connections because connection detailing is performed only at the final design stage. As an alternative, simplified yet more realistic cost functions may be developed wherein only the major construction cost contributors are considered. For most moment resisting frames, total cost can be represented in terms of three main contributors: (a) cost of each individual structural member in the frame, (b) cost of welded connections, and (c) cost associated with the use of different numbers of section types within a frame. The first of these is the most straightforward since member weight is often an appropriate surrogate for cost, especially in a frame of a specified geometric layout. Quantifying the remaining costs can be more challenging.

With respect to welded connections, costs are governed by onsite labor requirements. A reasonable estimate of man-hours for onsite welding can be made only after the completion of connection detailing, which requires professional experience and is not performed until the final design stage. Therefore, it is impractical to consider man-hours explicitly in a cost formulation of welded connections. One widely popular but simplified representation of this cost is to express it as a factor of material weight, especially if all the connections within a moment resisting frame are considered to be welded. Unfortunately, such a representation of the connection cost cannot be employed for exploring and evaluating design alternatives in which only a few connections within a moment resisting steel frame are considered to be welded. The concept can however be quite useful if cost factors are employed within the context of each individual member depending upon whether or not it has a welded connection at one or both ends. A certain factor of the individual member cost can then be attributed to the cost of corresponding connections. This formulation would be even more realistic if (i) the individual member costs are evaluated by separating the material, production, coating, and erection costs, and (ii) appropriate (possibly different) connection cost factors are used for each of these items.

Although difficult to quantify, other costs are incurred as the variety in member section types increases, due to an increase in detailing and production requirements. In addition, frames with more homogeneity require fewer column splices, which leads to better safety since splices are prone to fracture under lateral loads. Unless explicitly taken into account, greater variety in member sections may result from optimization-based approaches, yielding solutions that may not be cost optimal. This effect, however, can be addressed implicitly by exploring the trade-off between other costs and the number of different section types in the frame. A similar trade off may also be evaluated for variations in total cost with different numbers of welded connections within a frame.

In related work, Liu *et al*. (2006) present a genetic algorithm based formulation for the design of moment-resisting rigid steel frames in which the cost contributions of construction aspects are considered implicitly within a multi-objective design formulation. Construction costs are considered to be directly related to the number of different member section types used in the design. While a minimum weight design formulation typically results in a frame with several different member section types, a GA-based formulation is used to develop a Pareto-optimal front for the steel material weight versus the number of different member section types. In another study, Carter *et al*. (2000) outlines the reduction in construction costs achieved by avoiding cost-intensive connection design, detailing, production, and erection. This is particularly the case for moment-resisting rigid connections which require labor intensive onsite welding. Since it is difficult to quantify these costs, some of the recent studies represent connection related construction costs in terms of weight modification factors (Xu and Grierson 1993). One common theme of these studies is related to the number of rigid connections considered within a frame of a given geometric layout. These studies follow conventional practice wherein all connections within a moment-resisting frame are considered to be welded (rigid or semi-rigid). In many cases, however, it may actually be possible to reduce the number of welded connections within the frame of a given geometric layout. Kripakaran (2006) present one such study in which connection cost is represented in terms of a fixed cost per connection. They evaluate the trade-off between total cost and the number of connections within a frame but do not consider the number of column splices or the number of different member section types. The GA-based framework in their study, however, can be used effectively to understand the relative significance of connection related construction costs and material costs of the entire frame if a realistic cost function is considered.

A number of more realistic cost models have been presented in recent studies (Ferm and Yeo 1990, Simos 1996, Farkas and Jarmai 1997, Jarmai and Farkas 1999). While the details provided by Carter *et al*. (2000) are useful, they cannot be extended directly to quantify the various costs in a moment resisting steel frame. Pavlovicic *et al*. (2004) present a detailed cost function that considers production, erection, and finishing costs in addition to material costs. This cost function is based on the fundamental form presented by Jarmai and Farkas (1999). Consideration of this cost function is fairly straightforward for a frame of given geometrical layout but not for evaluating the costs associated with a rigid connection, which requires estimation of man hours needed for erecting and welding; these are easier to estimate at the final design stage once connection detailing is performed, but not within an optimization framework since connection detailing would change at each stage of the search process, particularly if a trade-off between the number of connections and total cost is desired. Evers and Maatje (2000) present connection cost factors for rigid connections which, if included within the cost functions of Pavlovicic *et al*. (2004), can be used effectively and efficiently within an optimization framework.

In this paper, we present a cost function that is derived by combining the cost functions of Pavlovicic *et al*. (2004) and the connection cost factors of Evers and Maatje (2000). The combination requires additional modification in order to study the trade-off between total cost and the number of rigid connections within a frame. The modified form of the cost function is then implemented within the GA-based framework presented in Kripakaran (2006) to study the relative significance of connection related construction costs versus material costs. For simplicity, connections are considered either fully-rigid or hinged. Semi-rigid behavior is not considered in this study. The study is extended further by evaluating the trade-off between the total cost and the number of different member section types to evaluate the impact of reducing the number of section types. To do so, the framework is further modified to include this additional objective using a multi-objective constraint method. The effect of constraints on the number of column splices within the frame is also examined.

2. Formulation of design problem

In this section, we formulate a mathematical model for the frame optimization problem. Consider an m-story, n-bay frame that consists of mn beams leading to $2mn$ beam-column connections and $2mn+m$ structural members in the frame. Formulation of the optimization problem requires two sets of decision variables, one to represent the type of each beam-column connection and the other to represent the type of product for each structural member. Let c_i be a decision variable corresponding to the presence or absence of a rigid connection at location i, $i = 1\ldots.2mn$, such that the presence of a hinged connection at location i is represented by c_i=0 and the presence of a rigid connection by c_i= 1. Consequently, a binary string $<c_1, c_2, \ldots, c_{2mn}>$ represents the decision variables for the entire frame. In many case, the number of connection decision variables may be less than $2mn$ because of consideration of symmetry in the frame.

Similarly, an additional set of decision variables can be used to represent the product types p_j for all the $2mn + m$ structural members in the frame. Let us represent this set by an integer string $<p_1, p_2, \ldots, p_{2mn+m}>$. In addition to decision variables, formulation of the optimization problem requires a representation of the objective function in terms of the total construction cost of the moment-resisting steel frame. In this study, we represent the total cost C_T as

$$C_T = C_m + C_p + C_c + C_e \tag{1}$$

in which C_m, C_p, C_c and C_e correspond to the material, production, coating, and erection costs, respectively. A detailed discussion of the cost model and individual contributors to the total cost is presented in the next section.

Constraints for the design problem are dictated by strength and serviceability requirements. The various load combinations, strength requirements, and serviceability limits considered in this study conform to the specifications of the Manual of Steel Construction, Load and Resistance Factor Design (AISC 2001). The following load combinations are considered for strength evaluation

$$1.4D \tag{2}$$

$$1.2D + 1.6L + 0.5L_r \tag{3}$$

$$1.2D + 1.6W + 0.5L + 0.5L_r \tag{4}$$

where D, L, W, and L_r represent dead loads, live loads, wind loads, and roof live loads respectively. It must be noted that effectively four load combinations exist because wind loads result in two different load cases that correspond to the wind blowing in either direction. The serviceability requirements are evaluated for only two load cases corresponding to the lateral wind loads acting on either side of the frame. The members in the frame must conform to the following design equations for each load case.

$$\text{For} \quad \frac{P_u}{\phi P_n} \geq 0.2, \quad \frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_u}{\phi_b M_n}\right) \leq 1.0 \tag{5}$$

$$\text{For} \quad \frac{P_u}{\phi P_n} \leq 0.2, \quad \frac{P_u}{2\phi P_n} + \left(\frac{M_u}{\phi_b M_n}\right) \leq 1.0 \tag{6}$$

P_u is the factored axial load (tensile or compressive) in the member, P_n is the nominal (tensile or compressive) strength in the member, and ϕ is the corresponding resistance factor. For tension, $\phi = 0.9$ in Eq. (5) and $\phi = 0.75$ in Eq. (6). For compression, $\phi = 0.85$ in both equations. M_u and M_n are the required and nominal flexural strengths for major axis bending, respectively. ϕ_b is the resistance factor for flexure and is equal to 0.9. P_n is calculated using the area of the member and either the yield stress or the buckling stress, depending upon the nature of the axial force in the member, i.e., tension or compression. M_n is calculated using certain equations that evaluate the ability of the member to resist the following buckling possibilities: local flange buckling, local web buckling, and lateral torsional buckling. These equations are described in detail in AISC-LRFD (2001) and are not provided in this paper, to maintain brevity. The serviceability constraint given below governs the maximum side sway δ for any story of height H in the frame.

$$\delta \leq \frac{H}{400} \tag{7}$$

As discussed earlier, an additional constraint considered in the design formulation relates to the number of column splices. In accordance with some existing studies, each column is considered to be spliced at only one location, the second floor level (Liu *et al*. 2006).

3. Formulation of the cost model

The cost formulation that forms the basis of Eq. (1) is derived from a combination of cost functions given by Pavlovicic *et al*. (2004) and the connection cost factors given by Evers and Maatje (2000). As stated earlier, a key objective of this study is to evaluate the trade-off between total cost and the number of rigid connections within a frame. Consequently, the two formulations cannot be combined directly. Instead, modifications are needed to adapt these formulations for consideration in the present study. Let us start by considering the cost function proposed by Pavlovicic *et al*. (2004). This cost function is quite complex in the sense that total cost is represented in terms of contributions from the structural members (material cost), welding, cutting, painting, surface preparation, flange aligning, connection, and erection. Various items such as welding and cutting are further evaluated in terms of contributions from material cost, production (or manufacturing) cost, and erection (or assembly) cost. In comparison, Evers and Maatje (2000) specify total cost is terms of four main contributors, which are material, production, coating, and erection costs. As a first step, we rearrange the various terms in the cost function of Pavlovicic *et al*. (2004) into the simplified form given in Eq. (1). Each individual item in this particular equation, however, is evaluated in detail by considering costs associated with welding, cutting, coating, surface preparation, etc. A key modification proposed in this process relates to the connection cost. The actual cost of a connection depends upon various factors including the type of connections, weld length, web stiffening and doubler plates. However, since it is impractical to evaluate the man-hours needed for onsite welding of a moment resisting welded connection at the design optimization stage, the connection cost is not considered separately in Eq. (1). We employ a cost formulation in which each of the four cost contributors in Eq. (1) has two parts, one related to the structural members and the other related to the connections. The contribution from rigid connections for each term is evaluated using connection cost factors. In order to determine the connection cost factors, detailed discussions were held with practicing engineers, steel fabricators and erectors. The cost of hinged beam-column connections is not included in this cost model as it is significantly smaller than the cost of rigid beam-column connections.

3.1 Material cost

In this framework, the material cost C_m is defined as

$$C_m = \sum_{i=1}^{2mn+m} C_{m,i} \tag{8}$$

$$C_{m,i} = C_{m,i}^{memb} + C_{m,i}^{conn} = C_{m,i}^{steel} + C_{m,i}^{add} + C_{m,i}^{conn} \tag{9}$$

where, $C_{m,i}$ is the material cost associated with the ith structural member, which in turn consists of contributions corresponding to the particular member $C_{m,i}^{memb}$ and that of the connection $C_{m,i}^{conn}$ at the two ends of the ith member. $C_{m,i}^{memb}$ can be calculated by considering the steel hardware cost $C_{m,i}^{steel}$ and the additional material cost $C_{m,i}^{add}$ needed in the preparation of each structural member. For brevity of discussion, evaluation of both these costs terms is described in detail in Appendix I. At this point, the cost model is modified to account for the connection cost $C_{m,i}^{conn}$ in terms of the connection cost factors. Eq. (9) is rewritten as

$$C_{m,i} = C_{m,i}^{steel} + C_{m,i}^{add} + \left(R_m \frac{N_i^r}{2}\right) C_{m,i}^{add} \tag{10}$$

in which the connection cost factor R_m represents the ratio of the $C_{m,i}^{conn}$ to the $C_{m,i}^{add}$ and is taken as $(40/60 = 2/3)$ based on the recommendation that about 40% of the additional material cost relates to the rigid connections. Each member may have either 0, 1, or 2 rigid connections at its two ends which is represented by the value of N_i^r.

3.2 Production cost

The production cost C_p is defined in a similar manner

$$C_p = \sum_{i=1}^{2mn+m} C_{p,i} \tag{11}$$

$$C_{p,i} = C_{p,i}^{memb} + C_{p,i}^{conn} \tag{12}$$

where $C_{p,i}$ is the production cost associated with the ith member, which in turn can be evaluated in terms of the production cost associated solely with the member $C_{p,i}^{memb}$ and with the connections at its two ends $C_{p,i}^{conn}$. The term $C_{p,i}^{memb}$ consists of costs related to welding $C_{p,i}^{weld}$, assembly $C_{p,i}^{assembly}$, cutting $C_{p,i}^{cut}$, handling $C_{p,i}^{handling}$, surface preparation $C_{p,i}^{surface-preparation}$, and flange aligning 012.000000 $C_{p,i}^{flange-aligning}$. Once again, these individual cost contributors are explained in detail in Appendix I.

$$C_{p,i} = C_{p,i}^{weld} + C_{p,i}^{assembly} + C_{p,i}^{cut} + C_{p,i}^{handling} + C_{p,i}^{surface-preparation} + C_{p,i}^{flange-aligning} + \left(R_p \frac{N_i^r}{2}\right) C_{p,i}^{memb} \tag{13}$$

In the above equation, the connection related cost $C_{p,i}^{conn}$ is evaluated using the cost factor R_p, the ratio of $C_{p,i}^{conn}$ to $C_{p,i}^{memb}$. In this study, R_p is taken as $(80/20 = 4)$ which is based on the value specified by Evers and Maatje (2000) and adjusting it for the year 2010 according to the historic rate of increase, as studied and proposed by Carter *et al*. (2000).

3.3 Coating cost

The coating cost C_c is defined as

$$C_c = \sum_{i=1}^{2mn+m} C_{c,i} \tag{14}$$

$$C_{c,i} = C_{c,i}^{memb} + C_{c,i}^{conn} \tag{15}$$

where $C_{c,i}$ is the coating cost associated with the ith structural member. The contributions corresponding to the particular member $C_{c,i}^{memb}$ are described in Appendix I. The connection $C_{c,i}^{conn}$ is evaluated in terms the connection cost factor R_c, the ratio of the $C_{c,i}^{conn}$ to the $C_{c,i}^{memb}$.

$$C_{c,i} = C_{c,i}^{memb} + \left(R_c \frac{N_i^r}{2}\right) C_{c,i}^{memb} \tag{16}$$

R_c is taken as (35/65 = 7/13) based on the recommendation that about 35% of the coating cost relates to the rigid connections.

3.4 Erection cost

Finally, the erection cost C_e is also defined in terms of erection cost corresponding to each member $C_{e,i}$.

$$C_e = \sum_{i=1}^{2mn+m} C_{e,i} \tag{17}$$

$$C_{e,i} = C_{e,i}^{memb} + C_{e,i}^{conn} \tag{18}$$

The contribution corresponding to a particular member $C_{e,i}^{memb}$ is evaluated as described in Appendix I. The connection related erection cost $C_{e,i}^{conn}$ is evaluated using the connection cost factor R_e. Therefore, we can write

$$C_{e,i} = C_{e,i}^{memb} + \left(R_e \frac{N_i^r}{2}\right) C_{e,i}^{memb} \tag{19}$$

in which R_e is taken as (80/20 = 4) based on the recommendation that about 80% of the erection cost relates to the rigid connections. In summary, the complete connection cost can be expressed as

$$C_{total}^{conn} = \sum_{i=1}^{2mn+m} \left\{ \left(R_m \frac{N_i^r}{2}\right) C_{m,i}^{add} + \left(R_p \frac{N_i^r}{2}\right) C_{p,i}^{memb} + \left(R_c \frac{N_i^r}{2}\right) C_{c,i}^{memb} + \left(R_e \frac{N_i^r}{2}\right) C_{e,i}^{memb} \right\} \tag{20}$$

4. GA-based optimization framework

Genetic algorithms (Goldberg 1989) have been successfully employed in many structural engineering design optimization problems, both discrete and continuous. Rajeev and Krishnamurthy

(1997), Deb and Gulati (2001) developed GA-based methodologies for optimizing trusses. Kameshki and Saka (2001), Ali *et al*. (2009), Foley *et al*. (2007) used a GA for the optimization of nonlinear steel frames with semi-rigid connections. Gupta *et al*. (2005) studied the application of GA for finding the optimal support locations for piping systems in safety-critical systems like power plants. In this study, GA-based framework developed by Kripakaran (2006), in which the crossover and mutation operators developed by Gupta *et al.* (2005) are used to perform trade-off studies between the number of rigid connections and the total cost, is modified to include the additional objective using constraint method. The framework is modified further to study the effect of a constraint on the number of column splices within the frame.

The decision variables for this study are characterized in terms of a binary string c_i that represents the type of connection at each location and an integer string p_j that represents the product type for each member. The formulation adopted in this study is similar in nature to that used by Kripakaran (2006) and consists of primarily two steps. First, it determines a minimum-weight frame starting with the assumption that all connections in the frame are rigid. The purpose is to find the product set q_j for the various members that minimizes the total frame weight. This set serves as an initial solution at the next step of the GA implementation. Second, this framework uses a GA to perform trade-off study between the number of rigid connections and the total cost. Trade-off study is performed by conducting a series of GA runs, where each GA run is aimed at finding the solution with minimal total cost for a specified number of r_{req} of rigid connections. During a GA run for a specified r_{req}, the GA explores the solution space with exactly r_{req} rigid connections. To evaluate the quality of a particular connection configuration, the GA analyzes the frame with the particular connection configuration and q_j as its products. Since the solution has rigid connections at only certain locations, the members in the frame may violate the sway and/or strength requirements. Solutions that violate these constraints are transformed into feasible solutions by using algorithms that change the product types to find the minimal cost feasible solution for that connection configuration. Interested readers are referred to Kripakaran (2006) for a comprehensive description of the optimization framework. The fitness of each solution is evaluated using the following fitness function

$$Z = -(C_T + P_s + P_d + P_p) \tag{21}$$

in which C_T is the total cost of the frame and P_s, P_d and P_p are the penalties that correspond to the violations of constraints for the strength, sway, and number of different member (product) types, respectively. These penalties are calculated as follows.

$$P_s = a\sum_{j=0}^{2mn+m} p_s \tag{22}$$

where p_s is 0 if the corresponding design equation is satisfied, and p_s is a value from the corresponding design equation if the member violates the design equation. A constant multiplier a is set equal to a high value of 100,000 to assign a high penalty for violation of strength requirements. Penalty cost term P_d is defined as

$$P_d = b\sum_{j=0}^{2mn+m} p_d \tag{23}$$

in which $p_d = 0$, if member j is a beam or $\delta_j - \delta_{\max} \le 0$ and $p_d = \delta_j - \delta_{\max}$, if member j is a column and $\delta_j - \delta_{\max} > 0$. δ_j is the floor sway for column j and is simply the absolute difference of the horizontal displacements at the two end nodes of the member. $\delta_{\max}$ is the maximum allowable sway as specified by Eq. (7). A constant multiplier b is set equal to a high value of 100,000 as a penalty for violation of serviceability requirements. The penalty cost P_p, for enforcing the number of different member section types is defined as

$$P_p = c \times p_p \tag{24}$$

Where, $p_p = 0$, if $p_T - p_{\max} = 0$ and $p_p = |p_T - p_{\max}|$, if $|p_T - p_{\max}| \neq 0$. A constant multiplier c is set equal to 100,000; p_T is the number of different section types in a given solution; and $p_{\max}$ is the specified number of different section types.

Since a particular GA run must consider only those solutions that have a specified number of rigid connections, r_{req}, strings in the GA must satisfy the following property, $\Sigma c_i = c_{req}$, where c_{req} is the number of decision variables c_i that must equal 1 for the string to represent exactly r_{req} number of rigid connections in the frame. If symmetry is not used to reduce the number of decision variables, $c_{req} = r_{req}$. Otherwise, $c_{req} < r_{req}$. The following seeding technique is used to ensure that all solutions in the initial population have exactly the specified r_{req} number of rigid connections.

$$c_i = \begin{cases} 1, & \text{for } i \in S \\ 0, & \text{for } i \notin S \end{cases} \tag{25}$$

S is a set of cardinality c_{req} with elements randomly chosen from the collection $\{1, 2, 3, \ldots, 2mn\}$ such that no two elements in S are equal.

In an attempt to improve the quality of solutions in a subsequent generation, the GA uses the crossover operator to produce two offspring by combining two parent strings. Conventional crossover operators can generate solutions in which the number of rigid connections is different from r_{req}, even though the seeding technique ensured that all the solutions at the start of the GA only have a total of r_{req} rigid connections. Such a situation is avoided by using the crossover operator proposed by Gupta *et al.* (2005). This crossover operator has the unique characteristic of generating offspring that have an equal number of ones when each of the parent strings also have the same equal number of ones. Let us consider two parent strings, which have exactly c_{req} number of ones, given by $<c_{a1}, c_{a2}, \ldots, c_{a2mn}>$ and $<c_{b1}, c_{b2}, \ldots, c_{b2mn}>$. Then, define the set $S = \{i : c_{ai} \neq c_{bi}\}$. Note that S, whose cardinality is always even, can be randomly divided into two sets of equal sizes S_1 and S_2. The offspring, $<c_{c1}, c_{c2}, \ldots, c_{c2mn}>$ and $<c_{d1}, c_{d2}, \ldots, c_{d2mn}>$, are then given by

$$c_{ci} = \begin{cases} c_{ai}, & \forall i \in S \\ 1, & \forall i \notin S_1 \end{cases} \tag{26}$$

$$c_{di} = \begin{cases} c_{ai}, & \forall i \in S \\ 1, & \forall i \notin S_2 \end{cases} \tag{27}$$

This crossover scheme effectively reduce the size of the search space, which is 2^{2mn} when using uniform crossover, to ${}^{2mn}\mathrm{C}_{c_{req}}$. A crossover probability of 0.75 is used for the study.

GAs use the mutation operator to make random changes to existing solutions in a population. The premise for mutation is that random changes may result in solutions that lie in those regions of the decision space that were not explored in the previous generations. Mutation probability is kept very small at 0.1% to avoid any significant interference with the GA convergence. For the trade-off study, the decision space consists of all solutions that have exactly r_{req} number of rigid connections, i.e., c_{req} number of ones. Therefore, we use the following mutation operator, which makes random changes but does not alter the number of ones in the string. Let $\langle c_1, c_2, \ldots, c_{2mn}\rangle$ represent the solution. Let $S_1 = \{i : c_i = 1\}$ and $S_2 = \{i : c_i = 0\}$ represent the locations that have rigid connections and hinged connections in the original solution. For mutation, set $c_k = 0$, where k is randomly chosen from S_1 and set $c_l = 1$, where l is randomly chosen from S_2.

Binary tournament selection with a selection probability of 0.75 is employed in the GA. The convergence criteria is determined based on the number of consecutive generations for which there is no change in the global optimal solution. If this number exceeds a specified value, then the GA is considered to have converged to the optimal solution. 10 generations is used as the criterion for convergence.

5. Description of representative steel frame

The proposed optimization study is conducted by using the 5-story, 5-bay frame shown in Fig. 1. This frame is one of two end frames of a commercial building and is primarily used to resist the lateral forces arising due to wind in one direction. The figure shows the numbering of the members and the joints in the frame. It consists of 25 beams and 30 columns, i.e 55 structural members in all. Each beam has two connections, one at each junction with the columns yielding a maximum of 50 rigid beam-column connection locations in the frame. The binary string $\langle c_1, c_2, c_3, \ldots, c_{25}\rangle$ represents the decision variables that correspond to the various beam-column connections in the frame. The other set of decision variables in the problem is related to the product type p_j for the various members in the frame. Let $\langle p_1, p_2, \ldots, p_{30}\rangle$ represent the decision variables that correspond to the product type for each of the members in the frame. Note that symmetry in the structure is

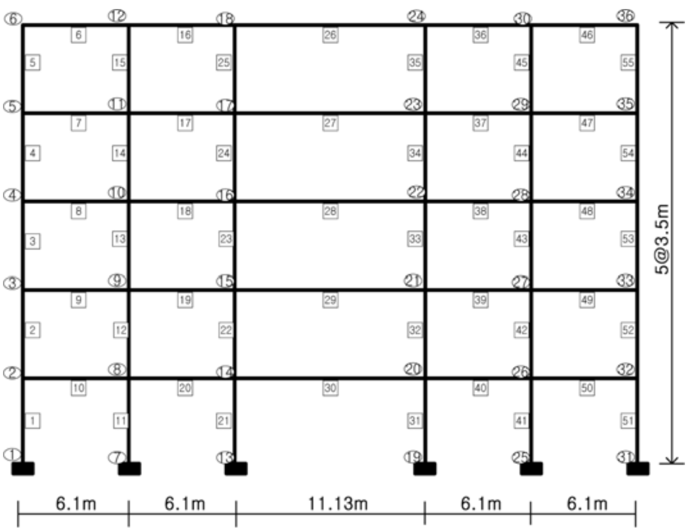

Fig. 1 Elevation of 5-bay 5-story frame

Table 1 Horizontal loads for frame given in Fig. 1

Load Type	Joint	Load (kN)
Wind Loads (W)	1 or 31	32.12
	2 or 32	64.19
	3 or 33	64.19
	4 or 34	64.19
	5 or 35	64.19
	6 or 36	32.12

Table 2 Vertical concentrated loads for frame given in Fig. 1

Load type	Joint	Load (KN)
Dead Loads (D)	6, 36	6.67
	12, 30	26.69
	18, 24	50.44
	2-5, 32-35	44.92
	8-11, 26-29	64.05
	14-17, 20-23	195.30
Live Loads (L)	2-5, 32-35	26.69
	8-11, 26-29	106.76
	14-17, 20-23	202.39
Roof Live Loads (L_r)	6, 36	17.08
	12, 30	68.32
	18, 24	129.53

Table 3 Vertical distributed loads for frame given in Fig. 1

Load type	Members	Load (KN/m)
Dead Loads (D)	7-10, 17-20, 37-40, 47-50, 27-30,6, 16, 36, 46, 26	16.30
	27-30	8.00
	6, 16, 36, 46	4.43
	26	1.20
Live Loads (L)	7-10, 17-20, 37-40, 47-50	17.77
	27-30	4.43
Roof Live Loads (L_r)	6, 16, 26, 36, 46	0.37

used to reduce the number of c_i from 50 to 25 and p_j from 55 to 30. Tables 1, 2, and 3 summarize all the loads considered in this study in accordance with the load combinations given in Eqs. (2)~(4). The W-shapes for the members of the frame are chosen from the shapes listed in the AISC Manual of Steel Construction for LRFD (AISC 2001). The W-shapes for structural members are selected based on following guidelines: (a) for beams, we select W-shapes with largest moment of Inertia (I_x)

Table 4 Product set for the beams

W 10 × 12	W 16 × 40	W 24 × 68	W 33 × 130
W 12 × 14	W 18 × 35	W 24 × 76	W 36 × 135
W 12 × 16	W 18 × 40	W 24 × 84	W 36 × 194
W 12 × 19	W 21 × 44	W 27 × 84	W 40 × 149
W 12 × 22	W 21 × 48	W 30 × 90	W 40 × 167
W 12 × 26	W 21 × 50	W 30 × 99	W 40 × 183
W 14 × 22	W 21 × 55	W 30 × 108	W 40 × 199
W 14 × 26	W 24 × 55	W 30 × 116	W 40 × 211
W 16 × 26	W 24 × 62	W 33 × 118	W 40 × 215
W 16 × 31			

Table 5 Product set for the columns

W 14 × 22	W 14 × 74	W 14 × 176	W 14 × 398
W 14 × 26	W 14 × 82	W 14 × 193	W 14 × 426
W 14 × 30	W 14 × 90	W 14 × 211	W 14 × 455
W 14 × 34	W 14 × 99	W 14 × 233	W 14 × 500
W 14 × 38	W 14 × 109	W 14 × 257	W 14 × 550
W 14 × 43	W 14 × 120	W 14 × 283	W 14 × 605
W 14 × 48	W 14 × 132	W 14 × 311	W 14 × 665
W 14 × 53	W 14 × 145	W 14 × 342	W 14 × 730
W 14 × 61	W 14 × 159	W 14 × 370	W 14 × 808
W 14 × 68			

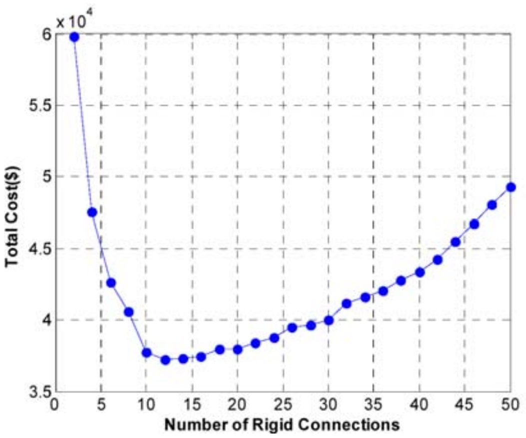

Fig. 2 Trade-off curve between the number of rigid connections and total cost

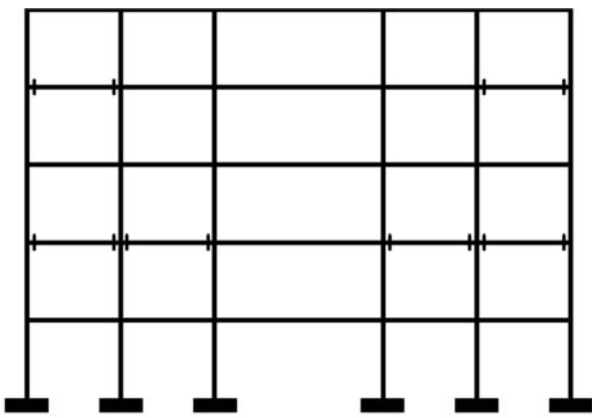

Fig. 3 Rigid connection locations in optimal solution

for minimal weight per unit length among groups of W-shapes with similar order of magnitude of I_x; and (b) for columns, we select W14 shapes such that all members that belong to a particular column have same web depth.

6. Results and discussion

6.1 Effect of reducing the number of rigid connections

To begin with, we evaluate the least-weight solution in which all the 50 connections in the frame are considered rigid. The weight of this solution is about 18 tonnes. The total cost of this frame using the cost model presented in our study is evaluated as $49,285. Next, we evaluate a trade-off curve that shows the variation in total frame cost with different numbers of rigid connections in the frame. Kripakaran (2006) evaluated a similar trade-off curve for this frame but considered a fixed cost for each moment connection. Since a detailed cost model has been considered in this study, the average connection cost is likely to vary due to a variation in the size of structural members with different numbers of connections. Fig. 2 gives the new trade-off according to which an optimal number of connections exist that would minimize the total cost. As seen in the figure, the total cost is minimum for the case of only 12 rigid connections in the frame, i.e., r_{opt} = 12 and its total cost is equal to $37,225, which is about 25% less than the cost for least weight design with 50 rigid connections. The optimal solution has a total frame weight of 31 tonnes compared to 18 tonnes for the least weight solution. Fig. 3 illustrates the location of the 12 rigid connections and Table 6 gives the W-shapes assigned to the various members in the optimal solution. It must be noted that several other solutions with 12 rigid connections are possible and which are also likely to be near optimal. Some of these "non-optimal" but "near-optimal" alternatives may be preferred by a decision maker (Kripakaran 2006). In this study, we did not evaluate these alternatives. Instead, we focused on

Table 6 W-shapes for optimal solution shown in Fig. 3

	W-shape	Members
Columns	W14 × 176	21,22,31,32
	W14 × 145	11,12,23,41,42,33
	W14 × 132	13,43
	W14 × 120	1,2,51,52
	W14 × 99	24,34
	W14 × 90	3,4,14,44,53,54
	W14 × 48	5,15,25,35,45,55
Beams	W24 × 62	19,39
	W21 × 55	7,9,27,47,49
	W21 × 48	17,37
	W21 × 44	8,18,28,29,38,48
	W18 × 35	30
	W16 × 31	10,20,40,50
	W12 × 19	6,16,26,36,46

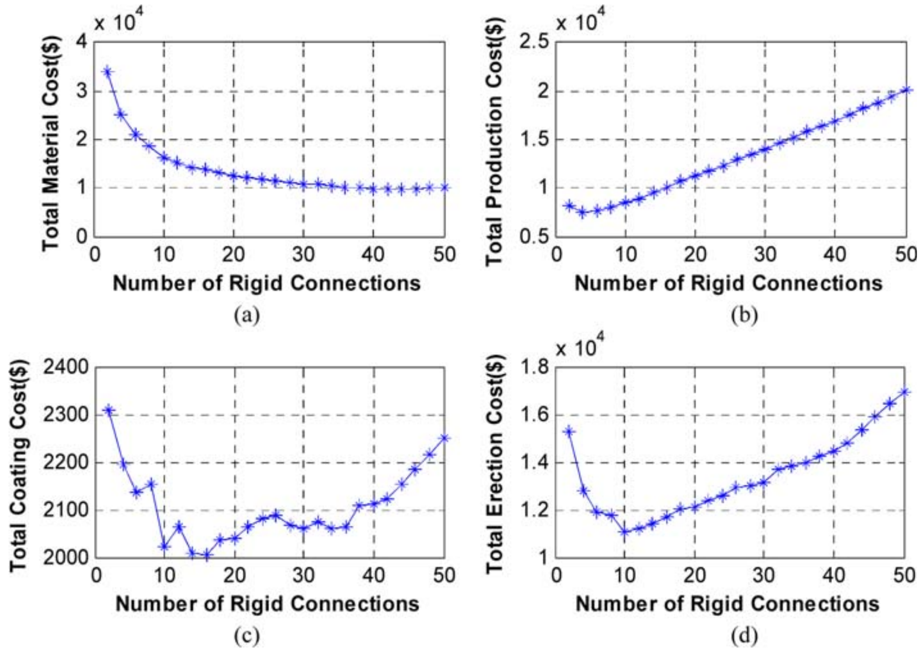

Fig. 4 Trade-off curves for the number of rigid connections and individual cost contributors

evaluating the effect of reducing welded connections in detail. To do so, we evaluated the trade-off curves for each of the four individual cost contributors in Eq. (1). Fig. 4 gives these trade-off curves. As seen in these figures, the material cost remains practically unchanged as the number of moment connections is reduced from a maximum of 50 to about 30. This is an interesting observation because it illustrates the excessive conservatism in a design that considers all 50 connections to be rigid, i.e., a designer can reduce the number of connections significantly without necessarily increasing the size of members. It can also be noted that the production and erection costs decrease significantly as the number of rigid connections decreases primarily because the production and erection costs are governed by the labor costs associated with rigid connections. As the number of rigid connections becomes very small, the production and erection costs increase with a decreasing number of rigid connections because of the additional costs associated with the production and handling of heavier members.

Next, we evaluate the effect of reducing the number of connections on the average connection cost. Fig. 5(a) gives the variation of total connection cost with the number of rigid connections and Fig. 5(b) the average connection cost for each case. Clearly, the average cost of rigid connections increases significantly when only a few rigid connections are considered because heavier members require additional cost for welding, cutting, preparation, etc., as can be inferred from the cost model described in Appendix I. To further illustrate the relative contribution of the production and erection costs, Fig. 6 compares the percent contribution of material, production (including coating), and erection costs. A comparison is given for two cases corresponding to 50 rigid connections and 12 rigid connections. This figure also gives the corresponding percentage values given by Carter *et al*. (2000). It can be seen that the relative contributions of material, production, and erection costs for the case of 50 rigid connections are similar to those given by Carter *et al*. (2000), and moreover,

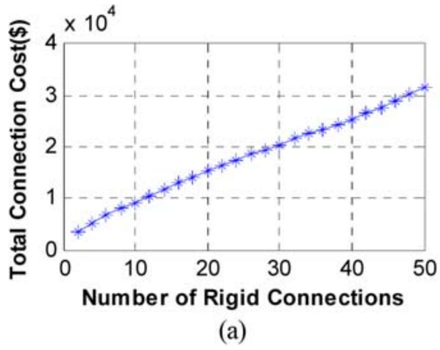

Fig. 5 (a) Variation of total connection cost, (b) Variation of average connection cost

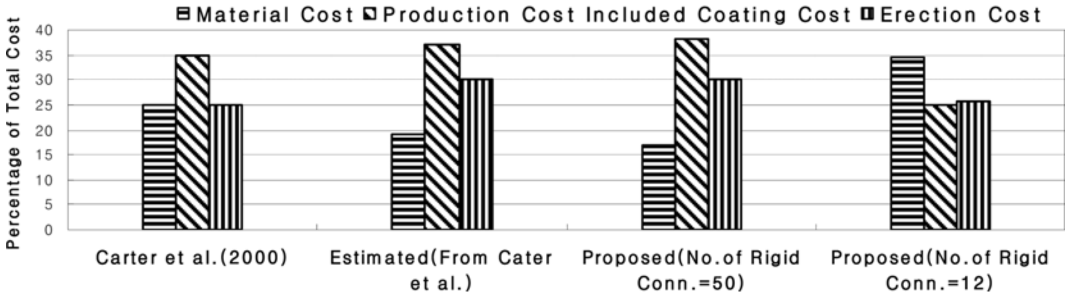

Fig. 6 Percent contributions of individual cost items

very close to those estimated in present year, according to the rates of variations which given by Cater *et al*. (2000). However, the relative contribution of material cost increases significantly for the case of 12 rigid connections.

6.2 Effect of reducing number of section types

As discussed earlier in this paper, the number of different section types used in a frame can implicitly affect the total cost. Liu *et al*. (2006) evaluated this effect in a rigid frame by considering all the connections to be rigid and by developing a trade-off curve to study the increase in total frame weight due to a decrease in the number of section types. We present similar trade-off curves, shown in Fig. 7, for four different cases corresponding to 50, 36, 24, and 12 rigid connections in the frame. For the case of 50 connections, the frame weight increases from 18 tones to 26 tonnes as the number of section types decrease from 10 to 2. This increase of 40% is similar to the corresponding percent change observed by Liu *et al*. (2006).

Fig. 7 also shows that as the number of section types decreases from 14 to 8, the rate of increase in frame weight is much greater for the cases of 36, 24, and 12 rigid connections. It must also be noted that these curves are calculated and shown for only up to 8 different section types. We could not find feasible solutions for smaller numbers of different section types for these three cases. If the total frame weight is used as a representation of the total cost, as is the case in Liu *et al*. (2006), Fig. 7 would indicate a preference for 50 connections because the total frame weight is much less for that case irrespective of the number of section types. However, such a conclusion would be

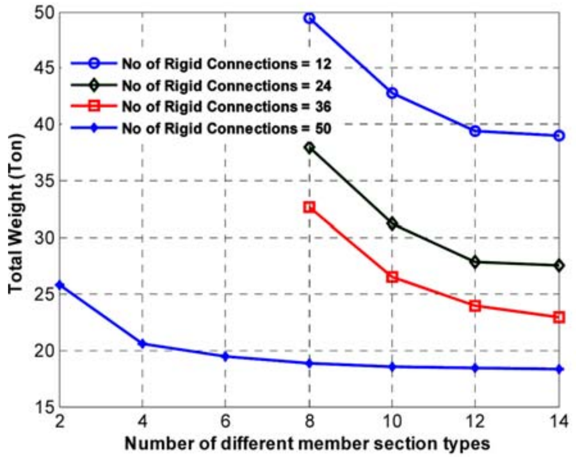

Fig. 7 Trade-off between the number of different section types and total frame weight

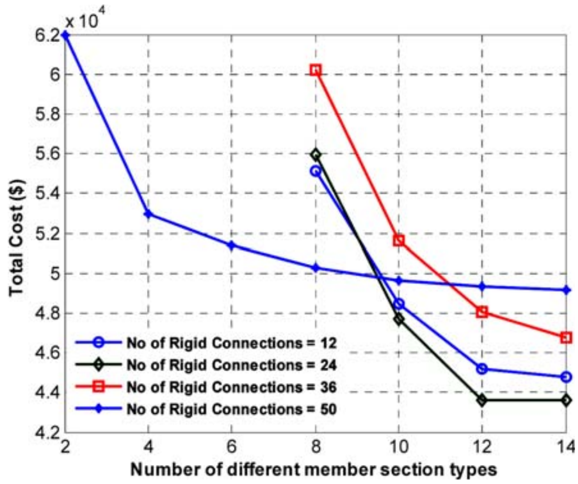

Fig. 8 Trade-off between the number of different section types and total frame cost

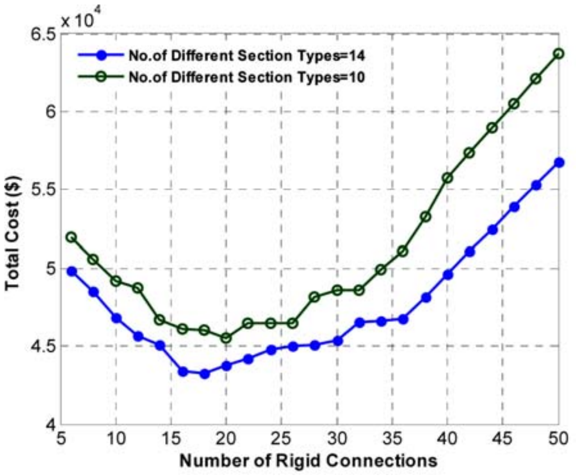

Fig. 9 Trade-off between the number of rigid connection and total cost with constraints on number of different section types, column splicing at second floor level for both curves

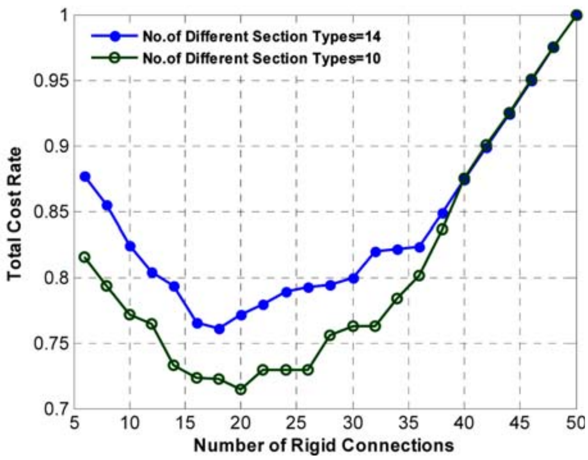

Fig. 10 Normalized Trade-off curves corresponding to Fig. 9

incorrect if the connection cost is also factored into the decision process as is the case in this study. Consequently, we also present an alternative set of trade-off curves that give the variation of total cost due to a change in the number of different section types. These curves are shown in Fig. 8 and show that the problem is quite complex in real life. As seen in this figure, the total cost is much lower for the case of 12, 24, and 36 connections compared to the case of 50 connections when the number of section types is 12 or 14. However, as the number of section types decreases to 8, the total cost for the 50 connection case is much smaller than for the other three cases. This observation leads us to reexamine Fig. 2. The optimal solution for 12 rigid connections in this figure consists of 14 different section types. Therefore, we develop the trade-off curve for variations in total cost with the number of rigid connections by considering exactly 10 different section types and by considering exactly 14 different section types. Fig. 9 shows these two curves, according to which not only does the total cost increase when only 10 section types are used but also the optimal number of rigid connections in the new curve increases to 20. In order to understand the change in the location of the optimal solution, we plot Fig. 9 using normalized costs such that the cost corresponding to 50 connections in each case is considered to be unity. The new sets of curves for normalized cost are shown in Fig. 10. This figure clearly shows a change in the optimal number of rigid connections from 18 to 20 if only 10 different section types are considered in the frame.

6.3 Effect of reducing connection cost factors

It is important to note that the results presented in this study are dependent upon the connection cost factors used in the cost model. As illustrated earlier in Fig. 4, the relative contributions of erection and production costs are higher as the number of rigid connections increases. This is particularly true because the connection cost factors for both the erection and production costs are taken as 80% ($R_p = R_e = 4$) as per the existing literature. Depending upon the location and the surroundings of a site, these connection cost factors may be lower. In such cases, the advantage of reducing the number of rigid connections within a frame is likely to be less significant or possibly nonexistent. To illustrate this dependency of our results on the connection cost factors, we reexamine Fig. 2 by considering a connection cost factor equal to only 60% ($R_p = R_e = 1.5$) for the erection and production costs. The new curve, given in Fig. 11, shows that the total frame cost is not significantly reduced for all possible numbers of rigid connections. Moreover, there is no well defined optimal number of rigid connections in the new curve. To evaluate the change in the nature of the optimal number of rigid connections, we plot these curves with respect to normalized costs, as shown in Fig. 12. It can be seen that the normalized cost decreases by about 10% when the number of rigid connections is reduced from 50 to 40. However, a further reduction in the number

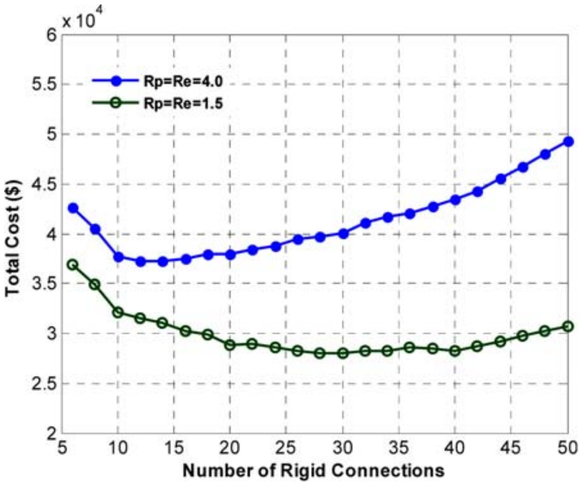

Fig. 11 Trade-off between the number of rigid connection and total cost for different connection cost factor

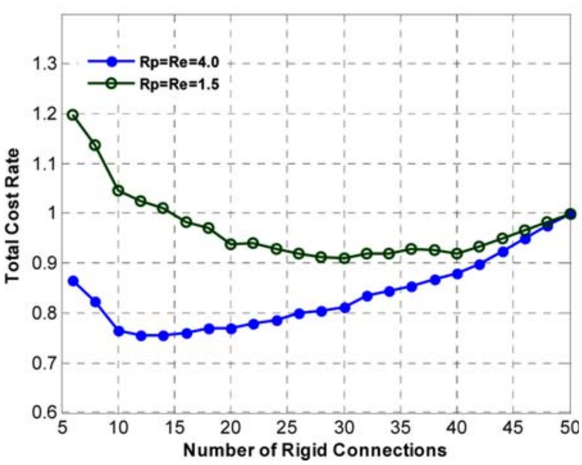

Fig. 12 Normalized Trade-off curves corresponding to Fig. 11

of rigid connections from 40 to 20 does not affect the cost much. Below 20 rigid connections, the cost increases with a decrease in the number of connections.

7. Conclusions

This paper presents the results from a computational study that focuses on the design of moment resisting rigid steel fames by considering construction costs in addition to material costs. Two main design features presented in this paper are related to the reduction in the number of cost intensive welded connections within a frame of given geometric layout and the reduction in the number of different structural section types used in the frame. A GA-based optimization framework is used to develop trade-off curves that give the variation of total cost with the number of rigid connections and with the number of different structural section types. A new cost function is developed based on existing cost formulations but modified to facilitate the development of trade-off curves, i.e., by employing connection cost factors at the member level to calculate the cost of connections at the two ends of each beam and not as a factor of the entire frame weight. Some of the key observations are the following:

- The number of rigid connections in the frame can be reduced from a maximum of 50 to about 30 without any appreciable increase in frame weight, which indicates excessive conservatism in the present design methodologies that consider all the connections within a frame to be rigid.
- The optimal number of rigid connections within the frame can be as low as 12 when there is no constraint on the number of different member section types employed. The overall frame cost with 12 rigid connections is about 25 percent lower than the cost with 50 rigid connections even though the frame weight for the 12 rigid connection design is about 72 percent higher than that of the 50 rigid connection design.
- The problem of designing rigid steel frames increases in complexity when a minimization of the number of member section types is included as another objective. It is observed that the total if the number of different member section types is limited to 10 compared to 14 in the originally calculated optimal solution, the optimal number of rigid connections increases from 12 to 20. Furthermore, the advantage of reducing rigid connections is lost if the number of section types is limited to 8, i.e., the overall cost of the frame design with 50 rigid connections is less than that with 12, 24, or even 36 rigid connections.
- It is also illustrated that the nature of the solution changes from a single optimal to a trade-off between cost and the number of rigid connections when the value of the connection cost factor is reduced. The overall cost remains about the same for frames with the number of rigid connections between 20 and 40.

References

Ali, N., Bel, H., Sellami, M., Cutting-Decelle, A.F. and Mangin, J.C. (2009), "Muti-stage production cost optimization of semi-rigid steel frames using genetic algorithms", *Eng. Struct.*, **31**(11), 2766-2778.

AISC (2001), *Manual of Steel Construction – Load Resistance Factor Design,* 3rd edition, AISC.

Carter, C.J., Murray, T.M. and Thornton, W.A. (2000), "Cost-effective steel building design", *Progr. Struct. Eng. Mater.,* **2**, 16-25.

Deb, K. and Gulati, S. (2001), “Design of truss-structures for minimum weight using genetic algorithms”, *Finite Elem. Analy. Des.*, **37**, 447-465.

Evers, H.G. . and Maatje, I.R.F. (2000), “Cost based engineering and production of steel constructions”, *Steel Design Codes -Fourth International Workshop on Connections in Steel Structures,* Roanoke, VA. 14-22.

Farkas, J. and Jarmai, K. (1997), *Analysis and Optimum Design of Metal Structures,* A.A. Balkema Publishers.

Fern, D.T. and Yeo, R.B.G. (1990), “Designing cost effective weldments”, *Welded Structures’90, International Conference,* Welding Institute, Abington Publications, London, 149-158.

Foley, C.M., Pezeshk, S. and Alimoradi, A. (2007), “Probabilistic performance-based optimal design of steel moment-resisting frames I: Formulation”, *J. Struct. Eng.*, **133**(6), 757-766.

Goldberg, D.E. (1989), *Genetic Algorithms in Search, Optimization and Machine Learning*, AddisonWesley Publishing Company.

Gupta, A., Kripakaran, P., Kumar, G. and Baugh Jr., J.W. (2005), “Genetic algorithm-based decision support for optimizing seismic response of piping systems”, *J. Struct. Eng.*, **131**(3), 389-398.

Jarmai, K. and Farkas, J. (1999), “Cost calculation and optimization of welded steel structures”, *J. Constr. Steel Res.,* **50**, 115-135.

Kameshki, E.S. and Saka, M.P. (2001), “Optimum design of nonlinear steel frames with semi-rigid connections using a genetic algorithm”, *Comput. Struct.*, **79**(17), 1593-1604.

Kartal, M.E. (2010), “Effects of semi-rigid connection on structural responses”, *Electronic J. Struct. Eng.*, **10**, 22-35.

Kripakaran, P. (2006), “Computational approaches for decision support in structural design and performance evaluation”, PhD Thesis, North Carolina State University.

Liu, M., Burns, S.A. and Wen, Y.K. (2006), “Genetic algorithm based construction-conscious minimum weight design of seismic steel moment-resisting frames”, *J. Struct. Eng.*, **132**(1), 50-58.

Papadrakakis, M. (2007), “Optimum performance-based reliability design of structures”, *Extreme Man-Made and Natural Hazards in Dynamics of Structures,* 137-159.

Pavlovcic, L., Krajnc, A. and Beg, D. (2004), “Cost function analysis in the structural optimization of steel frames”, *Struct. Multidis. Opt.,* **28**, 286-295.

Rajeev, S. and Krishnamoorthy, C.S. (1997), “Genetic algorithms-based methodologies for design optimization of trusses”, *J. Struct. Eng.*, **123**, 350-358.

Simoes, L.M.C. (1996), “Optimization of frames with semi-rigid connections”, *Comput. Struct.,* **60**(4), 531-539.

Xu, L. and Grierson, D.E. (1993), “Computer-automated design of semirigid steel frameworks”, *J. Struct. Eng.,* **119**(6), 1740-1760.

Notations

ρ_i : Weight per unit length of member i (ton/m)
A_{plate} : Area of plate (m^2)
a_w : Weld size = $0.4t_w$ (m)
f_{cut} : Cutting factor = 1.03
f_{weld} : Welding factor = 0.625
k_{align} : Cost factor for flange aligning = \$16.70/hour
$k_{asembly}$: Cost factor for assembling = \$13.31/hour
k_{erect} : Cost factor for erection = \$145.5/hour
$k_{handling\text{-}plate}$: Cost factor for plate handling = \$13.31/hour
$k_{m.steel}$: Cost factor of Steel material = \$400/ton
$k_{m,weld,wire}$: Cost factor for material consumption of $M_{weld,wire}$ = \$2.32/kg
$k_{m,weld,flux}$: Cost factor for material consumption of $M_{weld,flux}$ = \$1.88/kg
$k_{m,cut,propane}$: Cost factor for material consumption of $M_{cut,propane}$ = \$0.002/liter
$k_{m,cut,oxygenee}$: Cost factor for material consumption of $M_{cut,oxygene}$ = \$0.002/liter
$k_{m,paint1}$: Cost factor for material consumption of M_{paint} = \$5.4/liter
$k_{m,paint2}$: Cost factor for material consumption of M_{paint2} = \$4.5/liter
$k_{m,paint3}$: Cost factor for material consumption of M_{paint3} = \$4.5/liter
k_{paint} : Cost factor for painting = \$31.22/hour
$k_{surf\text{-}prep}$: Cost factor for surface preparation = \$47.67/hour
k_{weld} : Cost factor for welding = \$84.34/hour
k_{cut} : Cost factor for cutting = \$22.26/hour
L_i : Length of member i (m)
L_c : Cutting length (m)
L_w : Length of weld = $4L_{el}$ (m)
$M_{weld,wire}$: Material consumption (wire) = $0.97a_w^2 - 0.01a_w + 0.001$ (kg/m)
$M_{weld,flux}$: Material consumption (flux) = $0.51a_w^2 + 0.29a_w - 0.044$ (kg/m)
$M_{cut,propane}$: Material consumption (propane) = $2.171t_{pl} + 7.87$ (liter/m)
$M_{cut,oxygene}$: Material consumption (oxygene) = $1.645t_{pl}^2 + 56.644t_{pl} - 6.73$ (liter/m)
M_{paint1} : Material consumption (paint 1) = 0.13 (liter/m^2)
M_{paint2} : Material consumption (paint 2) = 0.173 (liter/m^2)
M_{paint3} : Material consumption (paint 3) = 0.15 (liter/m^2)
T_{weld} : Welding time = $2.62a_w^2 + 1.37a_w + 0.09$ (min/m)
$T_{handling}$: Material handling time = $-0.00000008(\rho_iV_i)^2 + 0.0015(\rho_iV_i) + 4.52$ (min)
T_{cut} : Cutting time = $-0.015t_{pl}^2 + 0.421t_{pl} + 1.43$ (min/m)
$T_{handling\text{-}plate}$: Plate handling time = $-0.000000014(\rho_iV_i)^2 + 0.001(\rho_iV_i) + 3.72$ (min)
$T_{surf\text{-}prep}$: Surface preparation time = 2.2 (min/m)
L_{pl} : Plate length (m)
L_{blast} : Length of blasting chamber = 300 (cm)
T_{align} : Flange aligning time = 0.66 (min/m)
L_{el} : Element length (m)
T_{paint} : Painting time = 7.0 (min/m^2)
T_{erect} : Erection time = 0.0014 (hour/kg)
t_w : Web thickness (m)
t_{pl} : Plate thickness (m)
V_i : Volume of member i (m^3)

Appendix I : cost formulations

The various terms in the following equations are explained at the end of this section.

Material cost of ith structural member: $C_{m,i}^{memb} = C_{m,i}^{steel} + C_{m,i}^{add}$

Steel material cost

$$C_{m,i}^{steel} = k_{m,steel}\rho_i L_i$$

Additional material cost

$$C_{m,i}^{add} = C_{m,weld,i} + C_{m,cut,i} + C_{m,coating,i}$$

$$C_{m,weld,i} = [k_{m,weld,wire}M_{weld,wire} + k_{m,weld,flux}M_{weld,flux}]L_w$$

$$C_{m,cut,i} = [k_{m,cut,propane}M_{cut,propane} + k_{m,cut,oxygen}M_{cut,oxygen}]L_c$$

$$C_{m,coating,i} = [k_{m,coat1}M_{coat1} + k_{m,coat2}M_{coat2} + k_{m,coat3}M_{coat3}]2A_{plate}$$

Production cost of ith structural member:

$$C_{p,i}^{memb} = C_{p,i}^{weld} + C_{p,i}^{assembly} + C_{p,i}^{cut} + C_{p,i}^{handling} + C_{p,i}^{surface-preparation} + C_{p,i}^{flange-aligning}$$

$$C_{p,i}^{weld} = k_{weld}[f_{weld}T_{weld}L_w]$$

$$C_{p,i}^{assembly} = k_{assembly}[T_{handling}]$$

$$C_{p,i}^{cut} = k_{cut}[f_{cut}T_{cut}L_c]$$

$$C_{p,i}^{handling} = k_{handling-plate}T_{handling-plate}$$

$$C_{p,i}^{surface-preparation} = k_{surf-prep}T_{surf-prep}(L_{pl} + L_{blast})$$

$$C_{p,i}^{flange-aligning} = k_{align}T_{align}L_{el}$$

Coating cost of ith structural member:

$$C_{c,i}^{memb} = k_{paint}T_{paint}2A_{plate}$$

Erection cost of ith structural member:

$$C_{e,i}^{memb} = k_{erect}T_{erect}\rho_i L_i$$