Journal of Constructional Steel Research 128 (2017) 380–396

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

Cost efficiency analyses of steel frameworks for economical design of multi-storey buildings

Oğuzhan Hasançebi

Middle East Technical University, Department of Civil Engineering, Ankara, Turkey

ARTICLE INFO

Article history:
Received 8 February 2016
Received in revised form 27 August 2016
Accepted 14 September 2016
Available online xxxx

Keywords:
Cost efficiency analysis
Multi-storey steel buildings
Steel frameworks
Beam-column connections
Bracing configurations
Structural optimization
Parallel evolution strategy

ABSTRACT

In the present study cost efficiencies of various steel frameworks are investigated for economical design of multi-storey buildings. A total of thirteen steel frames that incorporate various types of beam-column connection and bracing configuration are considered for detailed and comparative cost analyses. The three multi-storey buildings consisting of 10, 20 and 30 floors are stiffened according to each of the thirteen steel frameworks to yield thirty-nine test frames for numerical applications. First design optimizations are carried out using an evolution strategy (ES) integrated parallel optimization algorithm to minimize the total member weight in each test frame. An extensive cost analysis is then carried out on the optimized design of each test frame to calculate its estimated construction cost using a cost model that itemizes costs of all production stages including material, manufacturing, erection and transportation. Cost-efficient frameworks are identified for the three steel buildings by comparing estimated costs of the test frames. Furthermore, the variations in cost efficiencies of the steel frameworks versus the storey number (or building height) are scrutinized. The results collected are utilized to reach certain recommendations regarding the selection of economically feasible frames for design of multi-storey steel buildings.

1. Introduction

The design of tall steel buildings is usually governed by lateral loads especially as the height-to-width ratio of a structural system increases. Hence, steel buildings must be designed and detailed to have sufficient rigidity and stability to resist lateral loads safely. One way to provide lateral stability in such structures is to have moment-resisting (rigid) type connections between beams and columns. In such rigidly connected systems the lateral loads are resisted through flexural stiffness of beams and columns. Another way is to stiffen a structural framework with a full-bracing system that behaves like a vertical truss throughout the height of the building to transmit lateral forces to the ground. In general, a bracing system can be arranged in a variety of different topological configurations depending on structural and architectural requirements, and the most common ones used in practice are cross (X), diagonal, Z, V and eccentric V-bracings, etc. It should be noted that at times when all the beam-column connections are of moment-free type (i.e. pin connections) in a steel frame, an integrated bracing system is necessary to provide lateral stability of the structure. A bracing system can also be used to stiffen rigidly connected frameworks to increase the lateral stability of such structures.

Several studies in the literature have investigated design weight efficiencies of steel frames featuring various types of beam-column connection and bracing configuration. In Memari and Madhkhan [1], optimum design weights of various two-dimensional braced and unbraced steel frames were examined under gravity and lateral seismic forces. The optimization problem was formulated as the minimum weight design of a frame subject to a number of behavioural constraints including combined bending and axial stress, shear stress, compression buckling, tension slenderness and drift ratio according to AISC-ASD (American Institute of Steel Construction – Allowable Stress Design) [2] specifications. Several rigid and pin-jointed planar frames stiffened with a bracing system were sized optimally using a feasible directions optimization method. Amongst various bracing configurations considered in their studies, the frame with a V-bracing yielded the lightest design weight, whereas the one with an X-bracing led to the heaviest design weight. Kameshki and Saka [3] investigated efficiencies of X, V and *Z*-bracings in pin-jointed frames as well as rigidly connected frames without any bracing system. They employed a genetic algorithm for the optimum sizing designs of the planar frames according to BS 5950 [4] specifications. Considering the design optimization of a 3-bay 15-storey frame, it was demonstrated that the X-bracing system yielded the lightest design weight for the frame. Further, it was concluded that in rigidly connected frames as well as in pin-jointed frames with V or *Z*-bracings, inter-storey drift constraints were dominant criteria in the design process. Other studies in the literature have attempted to find optimal distribution of bracing members in steel frames. In Liang et al. [5], the optimum topology design of bracing systems was searched for planar steel frames using a performance based design optimization

E-mail address: oguzhan@metu.edu.tr.

http://dx.doi.org/10.1016/j.jcsr.2016.09.002
0143-974X/

method. In their approach a continuum design domain was used to stiffen a multi-storey building and inefficient materials were removed gradually from this domain until a performance based index of the bracing system was maximized. Recently, Kaveh and Farhoodi [6] studied the problem of layout optimization for X-bracing of steel frames according to specifications of IBC2006 [7].

The literature survey reveals that efficiencies of various steel frameworks have been so far investigated by comparing the design weights of the resulting structures when different framing systems are adopted. In these studies, a steel frame has been modeled as a planar structure and its design weight is associated only with the total member weight in the frame, i.e. connection weights are not included. On the other hand, it is common to anyone that the minimum design weight of a steel frame only ensures the least material cost for members, yet cannot guarantee the lowest construction cost on the whole. In particular, connection designs may appreciably affect the manufacturing cost of a steel frame since the fabrication cost of joints can be in excess of 30% of the total fabrication cost of a structure [8]. Therefore, it is extremely important that design efficiencies of different steel frameworks are evaluated based on construction costs of the resulting structures, rather than design weights only. In this regard, employing a precise and realistic cost model is required to determine estimated costs of the generated designs. Pavlovcic et al. [9] performed a cost function analysis in design optimization of steel frames, and developed a cost model that include all essential fabrication and erection activities of steel frames. Unlike usual practice in structural optimization where a simple cost function is developed by multiplying some geometrical properties by suitable weights representing cost coefficients, their cost function itemizes all stages of production including welding, cutting, drilling, surface preparation, assembly, flange aligning, painting as well as steel and bolting material costs, transportation, and erection, etc.

The present study is concerned with investigating cost efficiencies of various steel frameworks for economical design of multi-storey buildings. In this context a total of thirteen steel frames that incorporate various types of beam-column connection (i.e. rigid or pin) and bracing configuration (i.e. X, Z, V, eccentric V-bracings) are considered for detailed and comparative cost analyses. The numerical examples are performed using three multi-storey buildings consisting of 10, 20 and 30 floors. It is assumed that the buildings are subjected to gravity loads as well as lateral wind loads according to ASCE 7-05 [10] code of practice. Each building is modeled as a space frame stiffened according to each of the thirteen steel frameworks, resulting in thirty-nine test frames for numerical applications. Each test frame is first sized using standard hot rolled sections to attain minimum weight design of its members subject to stress, stability and displacement limitations in accordance with AISC-ASD [11] specifications. The design optimization is performed using an evolution strategy (ES) integrated parallel optimization algorithm developed earlier by the author [12] for optimizing very large steel structures, especially high-rise buildings, in a timely manner. An extensive cost analysis is then carried out on the optimized design of each test frame using the cost model developed by Pavlovcic et al. [9] to accurately estimate its construction cost. Cost-efficient frameworks are identified for the three steel buildings by comparing the estimated costs of the test frames. In addition, the variations in cost efficiencies of the steel frameworks versus the storey number (or building height) are examined. Based on the numerical investigations, certain recommendations are reached regarding the selection of economically feasible frames for design of multi-storey steel buildings. The following sections of the paper are organized as follows. The second section presents optimum design formulations of space steel frames according to AISC-ASD [11]. The ES integrated parallel optimization algorithm is briefly overviewed in the third section. The thirteen steel frameworks incorporating various types of beam-column connection and bracing configuration are introduced in the fourth section. The fifth section summarizes the cost model utilized in the study for estimating construction costs of the steel frames. The numerical examples, sizing optimizations of the test frames, connection designs, and in-depth cost analyses are presented in the sixth section. Finally, the concluding remarks are outlined in the last section of the paper.

2. Formulation of design optimization problem

For a steel frame consisting of N_m members that are collected in N_d design groups (variables), the optimum design problem according to AISC-ASD [11] specifications yields the following discrete programming problem, if the design groups are selected from standard sections.

The objective is to find a vector of integer values $\boldsymbol{I}$ (Eq. (1)) representing the sequence numbers of steel sections assigned to N_d member groups

$$\boldsymbol{I}^T = [I_1, I_2, \ldots, I_{N_d}] \tag{1}$$

to minimize the weight (W) of the frame

$$W = \sum_{i=1}^{N_d} \rho_i A_i \sum_{j=1}^{N_t} L_j \tag{2}$$

where A_i and ρ_i are the length and unit weight of a steel section adopted for the member group i respectively, N_t is the total number of members in group i, and L_j is the length of the member j which belongs to the group i.

The members subjected to a combination of axial compression and flexural stress must be sized to meet the following stress constraints:

$$if\ \frac{f_a}{F_a} > 0.15;\ \left[\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}\right] - 1.0 \le 0 \tag{3}$$

$$\left[\frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \tag{4}$$

$$if\ \frac{f_a}{F_a} \le 0.15;\ \left[\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \tag{5}$$

If the flexural member is under tension, then the following formula is used instead:

$$\left[\frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \tag{6}$$

In Eqs. (3)–(6), F_y is the yield stress of steel, and $f_a = (P/A)$ represents the computed axial stress, where A is the cross-sectional area of the member. The computed flexural stresses due to bending of the member about its major (x) and minor (y) principal axes are denoted by f_{bx} and f_{by} respectively. F'_{ex} and F'_{ey} denote the Euler stresses about principal axes of the member that are divided by a safety factor of 23/12. It should be noted unlike tension members for which the safety factor is given as 5/3, the AISC-ASD [11] employs a higher safety factor for compression members under Euler buckling. The reason for this is to account for the P-delta magnification effect. F_a stands for the allowable axial stress under axial compression force alone, and is calculated depending on elastic or inelastic bucking failure mode of the member using Formulas 1.5-1 and 1.5-2 given in AISC-ASD [11]. The allowable bending compressive stresses about major and minor axes are designated by F_{bx} and F_{by}, which are computed using the Formulas 1.5-6a or 1.5-6b and 1.5-7 given in AISC-ASD [11]. It is important to note that while calculating allowable bending stresses, a newer formulation (Eq. (7)) of the moment gradient coefficient c_b given in ANSI/AISC 360-05 [13] is employed in