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Optimum design of steel floor system: effect of floor division number, deck thickness and castellated beams

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Abstract. Decks, interior beams, edge beams and girders are the parts of a steel floor system. If the deck is optimized without considering beam optimization, finding best result is simple. However, a deck with higher cost may increase the composite action of the beams and decrease the beam cost reducing the total cost. Also different number of floor divisions can improve the total floor cost. Increasing beam capacity by using castellated beams is other efficient method to save the costs. In this study, floor optimization is performed and these three issues are discussed. Floor division number and deck sections are some of the variables. Also for each beam, profile section of the beam, beam cutting depth, cutting angle, spacing between holes and number of filled holes at the ends of castellated beams are other variables. Constraints include the application of stress, stability, deflection and vibration limitations according to the load and resistance factor (LRFD) design. Objective function is the total cost of the floor consisting of the steel profile cost, cutting and welding cost, concrete cost, steel deck cost, shear stud cost and construction costs. Optimization is performed by enhanced colliding body optimization (ECBO), Results show that using castellated beams, selecting a deck with higher price and considering different number of floor divisions can decrease the total cost of the floor.

Keywords: structural optimization; steel floor optimization; composite castellated beams; enhanced colliding bodies optimization; floor division number

1. Introduction

Structural design optimization is an important problem that has been studied for the past few decades. One of the major challenges in structural design optimization is to introduce new meta-heuristic algorithms with higher potential and simpler usage. Popular meta-heuristic algorithms are Particle Swarm Optimization (PSO) (Eberhart and Kennedy 1995), Ant Colony Optimization (ACO) (Dorigo *et al*. 1996), Big Bang-Big Crunch (BB-BC) (Erol and Eksin 2006), Charged System Search (CSS) (Kaveh and Talatahari 2010), Ray Optimization (RO) (Kaveh and Khayatazad 2012), and Dolphin Echolocation Optimization (DEO) (Kaveh and Farhoudi 2013). Successful applications of meta-heuristic algorithms in structural optimization problems have been reviewed by Saka and Geem (2013).

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Recently, Sadollah *et al*. (2015) developed Water Cycle, Mine Blast and improved mine blast algorithms, Gonçalves *et al*. (2015) presented Search Group Algorithm, and Mirjalili developed the Ant Lion Optimizer (2015). Whale Optimization Algorithm (WOA) was presented by Mirjalili and Lewis (2016), and Water Evaporation Optimization (WEO) was developed by Kaveh and Bakhshpoori (2016), Some other applications of the metaheuristic algorithms can be found in the work of Kaveh and Zolghadr (2014), Kaveh *et al*. (2015),

Steel floor optimization is an important part of structural optimization. Decks, interior beams, edge beams and girder beams are parts of a steel floor system. This system is also determinate and its analysis can easily be performed without using the finite element packages. However, it is difficult to obtain the best result because of many numbers of variables and constraints.

At first, many researchers tried to optimize simple, composite and castellated beams. Morton and Webber (1994) used relatively straight-forward exhaustive search method to optimize composite beams. Klanšek and Kravanja (2007) utilized the non-linear programming (NLP) approach to optimize composite beams according to Euro-code 4 and conditions of both ultimate and serviceability limit states. Senouci and Al-Ansari (2009) optimized composite beams by genetic algorithms according to AISC-LRFD. They also tried to find the effect of span and loading on the optimum result by a parametric study.

Cost optimization of floor system is studied first by Adeli and Kim (2001). They utilized neural network and mixed integer non-linear programming according to the LRFD criteria. They also employed floating-point genetic algorithms to find the best results. Platt (2006) used the evolver (genetic algorithm solving program) to parametric optimization of floor. She considers the combination of configuration, size, topology and spacing of truss girders and beams. Kaveh and Abadi (2010) used an improved harmony search (HS) algorithm. They optimized a composite floor system consisting of reinforced concrete slab and steel I-beams according to AISC-LRFD rules. Poitras *et al*. (2011) considered a complete floor system and utilized particle swarm optimization (PSO) for optimization. They found composite action can be as economical as non-composite action depending on some conditions and they used formed steel deck instead of normal concrete deck. Kaveh and Ahangaran (2012) employed the social harmony search and found this new variant of HS to be better than other variants of it. Kaveh and Massoudi (2012) optimized floors by ant colony optimization (ACO),

The main objective of the present study is to optimize the cost of the steel floor elements and finding the effect of the number of floor divisions, concrete thickness and using castellated beams. This paper is organized as follows: In section 2, the design of structural elements of floor is introduced. Section 3 defines the problem and identify variables, constraints and objective function of the optimization problem. The optimization algorithm is discussed in section 4. Some numerical examples are introduced in section 5. Finally, conclusions are extracted in section 6.

2. Structural floor design

Structural elements are designed according to AISC-LRFD 10. Thus the load combination W for stress and stability check is (ASCE 1994):

$$W = 1.2DL + 1.6LL$$

where DL is the dead load and LL is the live load, and the load combination for serviceability criteria (deflection and vibration) is:

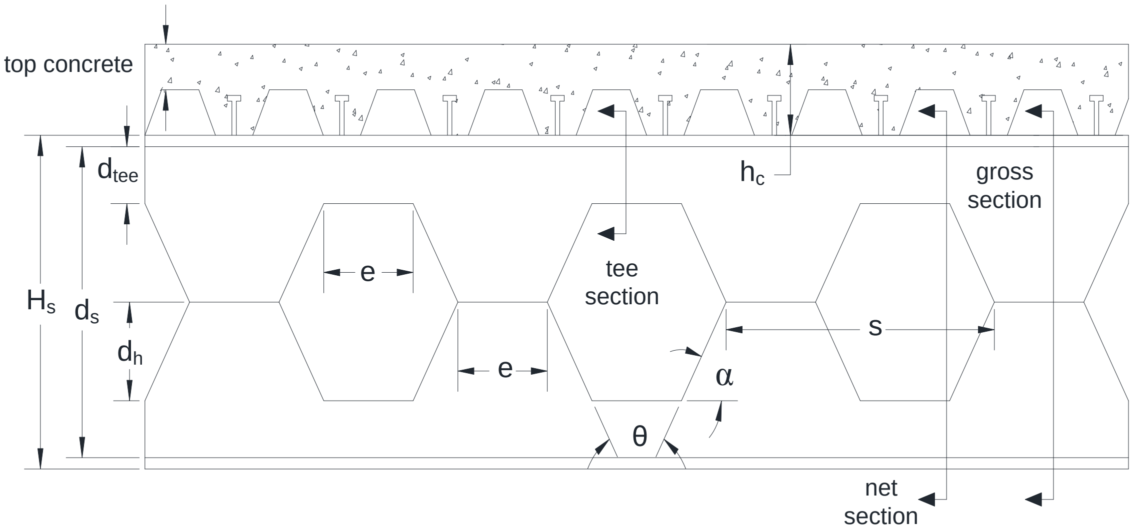

Fig. 1 Details of a composite castellated beam and steel deck

$$W_{def} = DL + LL$$

where W_{def} is the total loading for deflection calculation

A composite castellated beam and steel deck section (in perpendicular condition) is presented in Fig. 1. The deck should be designed independent of the beam as follow:

2.1 Deck design

Deck span is the distance between two beams (B) and, deck width is taken as one meter for the design. In this study, composite steel deck is used so that its section shape can guarantee the composite action between steel deck rolled form and concrete. Also the shrinkage and temperature effects of the concrete are controlled by rebar. Due to the complex effect of rolled form steel decks, the partial composite action and different section from one company to another, company catalogues are the sources of determining their capacity.

2.2 Castellated composite beam design

Castellated beams are produced by cutting rolled profile beam in special shape and welding them together in order to increase moment of inertia and moment capacity. Hexagonal cutting shape is one of the most popular cutting methods. But it is necessary to avoid keen corners because of stress concentration effects. Web openings of this beam, produce some secondary effects and filling end holes can controlled these issue.

Composite beams are produced by composite interaction between concrete and steel. This composite action can help to increase the moment capacity of the beams. For designing this type of beam, first the effective width of the concrete slab should be calculated for interior beams, edge beams and girders according to span and beam spacing (AISC 2010). Second, for the composite section centre line must be calculated. For interior and edge beams, deck ribs are perpendicular to the beam axis and top concrete (Fig. 1) must be considered only. But for girders, deck ribs are parallel to the beam axis and the entire concrete can be considered (AISC 2010). In this study, the centre line, moment of inertia and moment capacity of the composite section are determined by the superposition of elastic stresses. For some stresses, stability, deflection and vibration criteria must be checked as follows:

2.2.1 Stress criteria

In this study, the un-braced length ratio of all beams is considered as zero. This is because the top flange of the beam is controlled by concrete slab.

The ultimate moment calculated for load combinations must be smaller than the nominal moment (AISC 2010)

$$M_u < \phi_b M_n = \phi_b \times \min(M_{n-con}, M_{n-st}) = \phi_b \times \min(0.7F_c Z_{net-com-top}, F_y Z_{net-com-bot}) \tag{1}$$

where M_n is the nominal moment capacity of beam; $M_{n\text{-}con}$ is the nominal moment capacity (Concrete limit); $M_{n\text{-}st}$ is the nominal moment capacity (Steel limit); $Z_{net\text{-}com\text{-}bot}$ the plastic modules at bottom of composite net section; $Z_{net\text{-}com\text{-}top}$ is the plastic modules at top of composite net section; φ_b is the bending reduction factor; F_c is the compressive strength of concrete; F_y is the yield strength of steel.

Also the Vierendeel effect at unfilled holes produces secondary moment and these two moments must satisfy the following equations

$$m_u = \frac{V_u \times e}{4} \tag{2}$$

$$\frac{M_u}{Z_{net-com-bot}} + \frac{m_u}{Z_{tee}} < \phi_b F_y \tag{3}$$

where m_u is the secondary shear ultimate moment; V_u is the ultimate shear force; e is the web post length; M_u is the ultimate moment; $Z_{net\text{-}st}$ is the plastic modules of steel net section; Z_{tee} is the plastic modules of steel tee section. ϕ_b for concrete and steel are considered to be equal to 0.9 (AISC 2010).

For composite section, steel beams must resist shear forces alone (AISC 2010) as described in the following

$$A_W = d_s \times t_w \tag{4}$$

$$V_u < \phi_v V_{n-w} = \phi_v \times 0.6 F_y A_W C_v \tag{5}$$

where A_w is the area of the net section web; t_w is the thickness of the web; d_s is the internal castellated beam height; V_u is the ultimate shear force; $V_{n\text{-}w}$ is the nominal web shear capacity of net section; φ_v is the shear reduction factor; C_v is the web shear coefficient.

Also vertical shear capacity of tee beams, must be controlled by (AISC 2010)

$$A_{tee} = d_{tee} \times t_w \tag{6}$$

$$\frac{V_u}{2} < \phi_v V_{n-tee} = \phi_v \times 0.6 F_y A_{tee} C_v \tag{7}$$

where A_{tee} is the area of each tee section; $V_{n\text{-}tee}$ is the nominal web shear capacity of tee section;

Horizontal shear between holes in castellated beams must be checked as follows

$$A_{he} = e \times t_w \tag{8}$$

$$V_h = \frac{V_u \times Q_{com}}{I_{com}} \times s < \phi_v V_{n-p} = \phi_v \times 0.6 F_y A_{he} C_v \tag{9}$$

where V_h is the horizontal shear at post web; Q_{com} and I_{com} are the first and second moment of inertia of composite section, respectively; s is the spacing between the holes (Fig. 1); $V_{n\text{-}p}$ is the nominal shear capacity of post web; ϕ_v and C_v are equal to 1 (AISC 2010).

When steel deck is used in a perpendicular position, Q_{com} and I_{com} must be considered for two conditions, because each choice may produce greater shear and worst conditions:

(a) Considering the whole thickness of the concrete

(b) Considering the top thickness of the concrete

2.2.1 Stability criteria

Horizontal shear may cause web plate buckling in the castellated beam (Kerdal and Nethercot 1984). According to the Structural Stability Research Council (SSRC), in-plane stress at the unfilled web must satisfy the following equations

$$\begin{aligned}
&L_b = 2d_h \\
&r_T = \frac{t_w}{\sqrt{12}} \\
&C_b = 1.75 + 1.05\frac{M_1}{M_2} + 0.3(\frac{M_1}{M_2})^2 < 2.3 \\
&C_c = \frac{2\pi^2 E_s}{F_y} \\
&f_{rb} = \frac{3}{4}\frac{V_h \tan\theta}{t_w \theta^2 e} < \phi_b F_{rb} = \left[1 - \frac{(\frac{L_b}{r_T})^2}{2{C_c}^2 C_b}\right]\phi_b F_y
\end{aligned} \tag{10}$$

where θ, e and d_h are the cutting angle, hole pure distance and cutting depth of castellated beam (Fig. 1), respectively; t_w is the thickness of the web; M_1 and M_2 are the moment at each beam end; E_s is steel modules of elasticity; ϕ_b is equal to 0.9 similar to the moment equation.

2.2.2 Deflection criteria

Beam deflection can be calculated by standard equations of structural analysis. For interior and edge beams, bending deflection (def_b) can be calculated by

$$def_b = \frac{5W_{d1}{L_T}^4}{384E_s I_n} + \frac{5W_{d2}{L_T}^4}{384E_s I_{def}} \tag{11}$$

where W_{d1} and W_{d2} are the pre-composite and post-composite loads, respectively. L_T is total beam length. I_{def} and I_n are the effective moment of inertia for deflection of composite beam and steel net section moment of inertia, respectively.

Concrete weight must be resisted by steel section only(pre-composite level) and other dead and live load must be sustained by composite section (post-composite level),

Deflection of girders is related to the number floor division (beam spacing) and the number of interior beams.

Unlike the standard composite beam, the shear deflection of the composite beam with web opening is significant. Thus researchers have developed experimental-based equation for calculating the shear deflection (def_s) as follow (Benitez *et al*. 1998)

$$def_s = def_b \times \left(1 + \frac{1}{5}(\frac{HEW}{L_T})(\frac{I_{com}}{I_{com-g}} - 1)(3 \times (\frac{HEW}{L_T})^3 - 4 \times (\frac{HEW}{L_T})^2 - 6 \times (\frac{HEW}{L_T}) + 12) \right) \tag{12}$$

where I_{com} and I_{com-g} are the net and gross composite section moment of inertia, respectively. This equation based on a rectangular shape holes and hexagonal shapes must be considered as rectangular shapes with effective width

$$HEW = e + d_h \times \cot(\alpha) \tag{13}$$

and def_s identify the effect of one hole. For web opening with width to height ratio lower than 2, maximum deflection of beam is independent of the holes location. Thus total shear deflection can be obtained from number of unfilled holes (N_{uh}) times to def_s and total beam deflection is calculated as follows:

$$def = def_b + def_s \times N_{uh}$$

Also for considering the effect of differential shrinkage and creep on a composite steel-concrete structure, effective width (or concrete modulus of elasticity) can be divided by 3 (Roll 1971).

Also, allowable deflection (def_{all}) under live and dead loads is presented by (AISC 2010) as

$$def < def_{all} = \frac{L_T}{240} \tag{14}$$

2.2.3 Vibration criteria

Portion of the live load (between 10% to 25%) that is used for calculating deflection is used for calculating vibration (def_{vib}) (Murray *et al*. 2003). Combining the effect of interior beam deflection (def_{int}), the girder beam deflection (def_{gir}) and column deflection (def_{col}) for calculating frequency is described as follows (Naeim 1991)

$$def_{vib} = \frac{def_{\text{int}} + def_{gir}}{1.3} + def_{col} \tag{15}$$

To take into account the difference between the frequency of simply supported beam with distributed mass and concentrated mass at mid-span, deflection is divided by 1.3 ($\frac{4}{\pi}$), (Murray *et al*. 2003).

Because of small compression deflection of column, def_{col} is considered as zero. Also, 0.2 times of the live load is considered in calculating the deflection.

For considering greater stiffness of concrete on the metal deck under the dynamic as compared to static loading, it is assumed that concrete modulus of elasticity 1.35 times the normal concrete modulus of elasticity. The effect of differential shrinkage and creep on a composite steel-concrete structure is not considered for vibration calculations.

$$f = \frac{1}{2\pi}\sqrt{\frac{Stiffness}{Mass}} = \frac{1}{2\pi}\sqrt{\frac{\frac{W}{def_{vib}}}{\frac{W}{g}}}\sqrt{\frac{g}{def_{vib}}} = \frac{1}{2\pi}\sqrt{\frac{g}{def_{vib}}} \tag{16}$$

where W and g are the load and gravity acceleration, respectively.

In order to consider the effect of frequency of all parts of the floor, the total frequency of floor (f_t) is determined by

$$\frac{1}{f_t} = \frac{1}{f_{\text{int}}} + \frac{1}{f_{gir}} + \frac{1}{f_{col}} \tag{17}$$

where f_{int}, f_{int} and f_{int} are the interior, girder and column frequencies, respectively.

Due to the large axial stiffness of the column in comparison to the bending stiffness of beams, column frequency is considered infinity.

The maximum initial amplitude (inch) of the beam (A_o) is determined as (Naeim 1991)

$$A_{ot} = (DLF)_{\max} \times \left(\frac{0.6(L_T \times 0.393)^3}{48(E_s \times 14.22 \times 10^{-3})(I_{def} \times 0.393^4)} \right) \tag{18}$$

$$h_{c-eff} = \frac{Actual\,Slab Weight}{Concrete\,Weight} \tag{19}$$

$$N_{eff} = 2.97 - 0.0578 \times \left(\frac{S_b}{h_{c-eff}} \right) + 2.56 \times 10^{-8} \times \left(\frac{L_T{}^4}{I_{def}} \right) + 0.0001 \left(\frac{L_T}{S_b} \right)^3 \tag{20}$$

$$A_0 = \frac{A_{ot}}{N_{eff}} \tag{21}$$

where S_b is beam spacing. *(DLF)*$_{max}$ values for various natural frequencies are presented in design practice to prevent floor vibrations (Naeim 1991). Effective concrete height ($h_{c\text{-}eff}$) is not equal to the concrete height in the steel deck floor. Required damping ratio (D_{req}) for specified amplitude and frequency must be lower than allowable damping ratio (D_{all}) and it is determined by (Naeim 1991)

$$D_{req} = 35A_o f + 2.5 < D_{all} = 0.035 \tag{22}$$

2.3 Shear stud design

For desired composite action between steel and concrete, shear studs are required. The shear capacity of these must be smaller than the maximum shear forces that composite beam will experience. Steel Headed Stud Anchor is considered in current research. Its diameter is considered as 19mm and 1, 2 or 3 studs can be installed at each rib.

$$Q_u = \min(0.85F_c b_e h_c, A_s F_y) < N_c \phi_v Q_n = N_c \times \phi_v \times 0.5 A_{sa} \sqrt{F_c E_c} \le R_g R_p A_{sa} F_{u-ss} \tag{23}$$

where F_c and E_c are compression strength and elasticity modules of concrete, respectively. b_e and h_c are the effective width and height of concrete, respectively. A_s and A_{sa} are the steel section area and steel headed shear stud area, respectively. $F_{u\text{-}ss}$ is the ultimate stress of shear stud. R_g and R_p are the group and position effect factor for shear stud, respectively. Considering linear shear diagram, N_c is half of the total number of shear stud and ϕ_v is equal to 0.75 (AISC 2010).

3. Problem definition

3.1 Cost function

The cost for each beam is considered as the sum of the profile steel beam cost, welding procedure cost, cutting procedure cost and shear stud cost. The cost for steel deck is the sum of the steel deck concrete cost, steel deck steel plate cost and steel deck application cost. Initial cost is the sum of the beam costs and steel deck cost.

Each sub-cost is determined by multiplying the corresponding weight, length, volume or area by appropriate coefficients. Cost of filling end holes by plates is considered by the cost of the added weights, cutting and welding to the total cost.

3.2 Variables

In this study, five variables are used for optimal design of each beam, consisting of the profile section, cutting depth (d_h), cutting angle (α), hole spacings (s) and number of filled end holes of the castellated beams. The number of beams at floor width and concrete thickness are two other variables that are changed. The minimum and maximum magnitudes of the variables must be known for avoiding non-acceptable results and for fast convergence to the global optimum. Profile section is the sequence number of the hot rolled steel profiles. Cutting angle is limited between 40^o to 64^o. Other limits on the variables are presented as the constraints.

3.3 Constraints

Castellated beam application constraints (g_1 to g_5) and steel beam design constraints (g_6 to g_{14}) are considered as follows

$$g_1 = d_h - \frac{3}{8}(H_s - 2t_f) \tag{24}$$

$$g_2 = (H_s - 2t_f) - 10(d_t - t_f) \tag{25}$$

$$g_3 = \frac{2}{3} d_h \cot(\alpha) - e \tag{26}$$

$$g_4 = e - 2d_h \cot(\alpha) \tag{27}$$

$$g_5 = 2d_h \cot(\alpha) + e - 2d_h \tag{28}$$

$$g_6 = M_u - \phi_b M_n \tag{29}$$

$$g_7 = \frac{M_u}{Z_{net-com-bot}} + \frac{m_u}{Z_{tee-com}} - \phi_b F_y \tag{30}$$

$$g_8 = V_u - \phi_v V_{n-w} \tag{31}$$

$$g_9 = \frac{V_u}{2} - \phi_v V_{n-tee} \tag{32}$$

$$g_{10} = V_h - \phi_v V_{n-p} \tag{33}$$

$$g_{11} = f_{rb} - \phi_b F_{rb} \tag{34}$$

$$g_{12} = def - def_{all} \tag{35}$$

$$g_{13} = D_{req} - D_{all} \tag{36}$$

Some design constraints for the steel decks are as follows

$$g_{14} = B - L_{sd-\max} \tag{37}$$

where L_{sd-max} is the maximum length of the un-shored steel deck.

For comparison and for comparing the sum of constraints with each other, these are normalized.

3.4 Penalty function

Optimization algorithms are also designed for unconstraint problems and an external procedure should be defined for avoiding unacceptable regions. Penalty functions increase the objective function cost, and the optimization algorithm automatically avoids this area. In this study, penalty function is expressed as the function of positive (unacceptable) values of the constraint functions

$$NAC = sum(g_i > 0) \tag{38}$$

$$PF = 10^{NAC} \tag{39}$$

$$Cost_{fin} = Cost_{ini} \times PF \tag{40}$$

where $Cost_{fin}$ and $Cost_{ini}$ are the final and initial cost, respectively. The value of 10 is chosen by experience for the current problem and it can be changed for other problems.

4. Optimization algorithm

Interior beam optimization, edge beam optimization, girder optimization and deck optimization are four sub optimizations of this problem. Each of the first three problems has 5 variables according to the explanation given in the previous section. Deck optimization has one variable and the number of floor division is another variable. Thus, there are 17 optimization variables for this problem. Optimizing these variables simultaneously, decreases the convergence rate. In order to solve this problem, and to observe the conditions around the optimum result, the following approach is adopted.

4.1 Sub optimization approach

If the deck is optimized without considering beam optimization, finding the best result is simple. But other decks with higher costs can increase composite action of the beams and decrease the beam cost, hence reducing the total cost. Thus, after finding the best deck independently (by sorting deck choices from lower to highest cost and selecting first acceptable choice), some other near acceptable results are considered and optimum result of other parts of the floor are calculated for the entire system.

The range for the number of divisions of the floor is limited for different examples. To observe the impact of increasing the number of division, different values are considered and the results of optimization are obtained.

In order to optimize each beam, the following meta-heuristic algorithm is used:

4.2 Meta-heuristic optimization algorithm

The meta-heuristic algorithms finding the best result by iterative manner. They have an initial population and evaluate the objective function value of them. The algorithm produces next generation from initial population in order to have better chance to find the best result. So increasing the number of population and iteration number can increase chance of finding the optimum result.

Colliding body Optimization is one of the recently developed meta-heuristic algorithms. The efficiency of this algorithm for structural optimization is validated by researchers (Kaveh and Mahdavi 2014). The CBO is simple in concept and depends on no internal parameter.

In this technique, one object collides with other objects and they move towards a minimum energy level. Each colliding body (CB) has a specified mass (m_k) related to the fitness function as

$$m_k = \frac{\frac{1}{fit(k)}}{\sum_{i=1}^{n} \frac{1}{fit(i)}}, \quad k = 1,2,...,n \tag{41}$$

where *fit* and *n* are the fitness function and number of CB, respectively. In order to select pairs of objects for the collision, CBs are sorted according to the magnitudes of their mass in a decreasing order and these are divided into two equal groups: a stationary group and a moving group. Moving objects collide to stationary objects to improve their positions and push stationary objects towards better positions by change in their velocity. Initial velocity of the moving objects (v_1) is defined as a distance between their positions and destination of the stationary object. Initial velocity of stationary objects is considered as zero. Next velocity of stationary (v_{sta}) and moving (v_{mov}) groups are calculated as follows

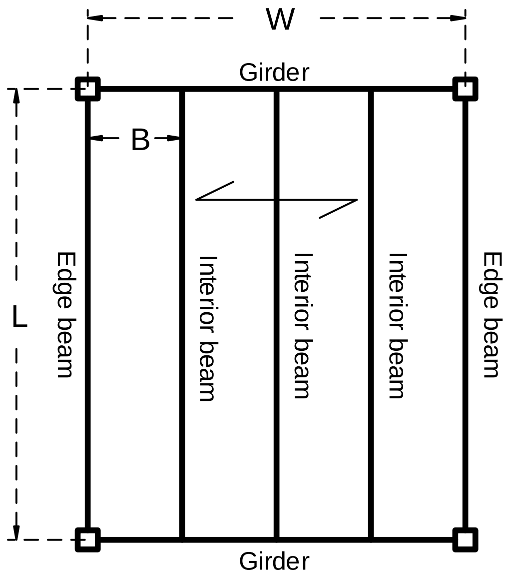

Fig. 2 Floor system configuration for the floor division number is equal to 4

$$v_{mov} = \frac{(m_1 - \varepsilon m_2)v_1}{m_1 + m_2} \tag{42}$$

$$v_{sta} = \frac{(m_1 + \varepsilon m_2)v_1}{m_1 + m_2} \tag{43}$$

where m_1, m_2, v_1 and v_2 are the mass and velocity of each pair of moving and stationary objects. Also, ε is defined as follow

$$\varepsilon = 1 - \frac{iter}{iter_{\max}} \tag{44}$$

where *iter* and $iter_{\max}$ are the current iteration number and maximum iteration number, respectively. Next position of each CB is its last position plus a random ratio of velocity.

In order to improve the CBO to get faster and more reliable solutions, Enhanced Colliding Bodies Optimization (ECBO) has been developed which uses a memory to save a number of historically best CBs and also utilizes a mechanism to escape from local optima (Kaveh and Ghazaan 2014). Utilizing this improvement requires to identify the colliding memory size (CMS) and the random parameter (RP),

5. Numerical examples

In order to study the effect of parameters on the optimum cost of the floor, two examples are studied. MATLAB software is used for modeling the optimization process. This software is also

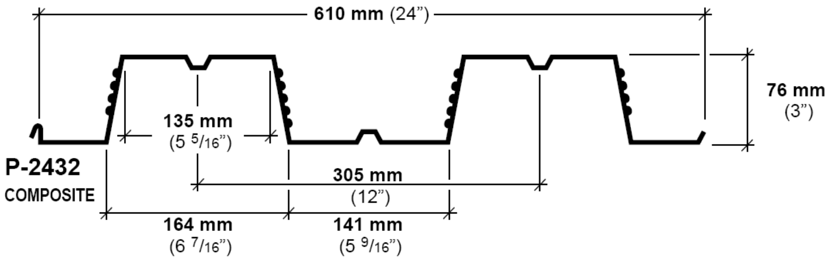

Fig. 3 Details of a steel deck from Canam® Steel catalogue

Table 1 Cost coefficients

Component	Price ($)	Unit
Steel profile	2.86	$ per each kg
Welding beams	1	$ per each m
Cutting beams	0.8	$ per each m
Shear studs	2.4	$ per each kg
Concrete	131	$ per each m^3
Steel deck	2.25	$ per each kg
Application	10.8	$ per m^2

used for the analysis and checking design criteria. The design results are also double-checked with ETABS software.

In both examples, floor systems with two girders, two edge beams and some interior beams are considered as shown in Fig. 2 and all connections are assumed pinned connections.

For algorithm adjustments, the population size and the iteration number are 40 and 60, respectively. Also, CMS and RP are considered to be 4 and 0.3, respectively.

5.1 Example 1

At first example, the span and width of the floor system are 10 m and 8 m, respectively. Interior beams are affected by live and dead area loads. Edge and girder beams are affected by live and dead uniform distributed load (in order to influence of adjacent bay and wall load), Girder beam also affected by end reaction of interior beam as a point load.

Full composite action is considered, since partially composite action, is very sensitive to construction and installation conditions of shear studs and it has a large amount of uncertainty.

In order to have a comparison with other references examples (Poitras *et al*. 2011), the steel deck choices were taken from the Canam® Steel catalogue as presented in Fig. 3. According to P-2434 (composite type of this catalogue) deck thicknesses values are considered as 0.76 mm, 0.91 mm and 1.21 mm. Slab thickness values are taken as 125 mm, 140 mm, 150 mm, 165 mm, 190 mm and 200 mm. Maximum span for each combination of deck and steel thickness is determined and the load resistance for each span is calculated. It is assumed that each span has adjacent span in the start and end (triple span condition), Shoring decks are not considered.

The profile sections are chosen by the Canadian Handbook of Steel Construction, starting from

Table 2 Problem type description and costs (Example 1)

Type	Description	Cost ($)	%
1	Poitras *et al*. (2011) best results	14832	0.96
2	Checking Poitras *et al*. (2011) results	15523	1.00
3	Optimizing composite beams	14097	0.91
4	Optimizing composite castellated beams	12796	0.82

Table 3 Critical constraints (Example 1)*

Type	Girders		Edge beams			Interior beams			
1	-	-	-	-	-	-	-	-	-
2	FM	CC	VB			FM			
3	FM	CC	FM	De	VB	FM	De	VB	
4	FM	BU	HS	De	VB	HS	FM	De	VB

*HS: Horizontal shear, RM: Radial moment, DE: Deflection, FM: Flexural Moment, VB: Vibration, BU: Buckling web, CC: Concrete compression

Table 4 Results (Example 1)

Type	Girders	Edge beams	Interior beams		Concrete floor	
	Section	Section	Section	Number	Steel thickness(mm)	Depth(mm)
1	W530×82	W460×60	W460×60	2	0.76	140.00
2	W530×82	W460×60	W460×60	2	0.76	140.00
3	W460×60	W410×39	W410×46	3	0.91	125.00
4	W460×52	W410×39	W410×39	3	0.76	125.00

W410×39 and ending with W690×289. The steel yielding stress, steel module of elasticity and concrete compression capacity are 3550 kg/cm^2, 2050000 kg/cm^2 and 200 kg/cm^2, respectively.

The values of the cost coefficients are determined by other researchers (Poitras *et al*. 2011) and engineering experiences. Cost coefficients are given in Table 1.

Poitras *et al*. (2011) did not consider the effect of shrinkage and temperature as discussed below. In order to compare the results of this study with their results, shrinkage and temperature effects are not considered in Example 1. They also used the S16 standard requirements (CSA 2009). Penalty factors in their work was considered constant and this assumption decreased the convergence rate.

For comparing results with other researchers and presenting effect of castellated beams, 4 problem types are assumed and they are defined at Table 2. Also final Costs of each type are presented in this table.

Critical constraints (over 80% demand capacity ratio) is shown in Table 3. Also, detailed results include the section profile of each beam as presented in Table 4.

The results of Example 1 are shown for comparison and 4% difference is observed between the results of Poitras *et al*. (2011) and the checked values. It should be mentioned that they considered 75% for composite action and our study considers full composite action. Thus, the number of shear studs are lower than our study.

Table 5 Effect of the floor division number and deck section on the total cost (Example 1)

Floor division number	Deck Price					
	Composite beams			Composite castellated beams		
	Low	Medium	High	Low	Medium	High
2	20197	17851	17207	19762	16350	17484
3	14892	14170	14323	14422	13969	14446
4	14399	14097	14639	12796	13312	13842
5	14208	14274	14444	13781	15440	14057

Table 6 Critical constraints (Example 2)*

Floor division number	Deck price	Girders			Interior and edge beams			
3	Low	HS	FM	RM	HS	FM	RM	VS
	Medium	HS			HS	FM	RM	VS
	High	HS	FM	RM	HS	FM	RM	VS
4	Low	HS	FM	RM	HS	FM	RM	
	Medium	HS			HS	FM	VS	DE
	High	FM			HS	FM	RM	
5	Low	HS	RM		HS	RM	FM	VS
	Medium	HS	FM	RM	HS	FM	VS	DE
	High	HS	FM	VS	HS	FM	VS	

*HS: Horizontal shear, RM: Radial moment, DE: Deflection, FM: Flexural moment, VS: Vertical shear

Table 7 Results (Example 2)

Floor division number	Deck Price	Girder					Edge and interior beam					Deck result	
		Section	Cut depth (cm)	Cut angle (d)	Hole spacing (cm)	Filled hole	Section	Cut depth (cm)	Cut angle (d)	Hole spacing (cm)	Filled hole	Steel thicknes (cm)	Concrete height (cm)
3	Low	IPE500	13.93	62.48	37.19	0	IPE240	7.95	63.8	23.07	0	12.5	0.91
	Medium	IPE500	13.93	62.48	37.19	0	IPE240	7.95	63.8	23.07	0	12.5	0.91
	High	IPE450	9.93	63.67	29.49	0	IPE240	5.88	63.7	17.04	0	14	0.91
4	Low	IPE360	10.96	63.98	31.05	0	IPE220	8.07	63.8	17.69	0	12.5	0.76
	Medium	IPE400	8.16	62.69	21.79	0	IPE240	11.7	63.2	23.96	0	14	0.76
	High	IPE300	17.46	63.78	50.54	3	IPE220	6.82	63.5	15.79	0	12.5	0.91
5	Low	IPE330	9.81	63.80	26.81	0	IPE200	8.66	59.9	19.38	0	12.5	0.76
	Medium	IPE300	8.62	62.88	25.51	0	IPE180	6.37	64	13.66	0	14	0.76
	High	IPE330	12.61	63.16	30.47	0	IPE200	4.32	59.4	11.1	0	12.5	0.91

By changing floor division numbers and deck sections, a parametric study is performed for composite and composite castellated beams and it is presented in Table 5.

Table 8 Effect of floor division number and deck section on the total cost (Example 2)

Floor division number	Total cost ($)		
	Low	Medium	High
3	7514.7	8532.2	7711.6
4	6871.3	7275.5	6732.8
5	6810.4	6207.9	6795.3

Table 9 Average ratio of hole spacing to cutting depth (Example 2)

Section	Average s/d_h
IPE180	2.14
IPE200	2.40
IPE220	2.25
IPE240	2.69
IPE300	2.93
IPE330	2.57
IPE360	2.83
IPE400	2.67
IPE450	2.97
IPE500	2.67

5.1 Example 2

This example is similar to example 1. Span and width are 6m and 7m, respectively. The profile sections are chosen by the IPE steel section, starting from IPE140 and ending with IPE600. The steel yielding stress, steel module of elasticity and concrete compression capacity are 2400 kg/cm^2, 2039000 kg/cm^2 and 250 kg/cm^2, respectively. The effects of shrinkage and temperature is considered. There is no uniform distributed load on edge beams and girders. In order to simulate adjacent bay conditions, they also resist 2 times of typical load of the exiting bay. Because the same loading was used on the interior and edge beams, their results are presented together. Other parameters of Example 2 are similar to those of Example 1.

Critical Constraints (over 80%), detailed results and costs of the choices are shown in Table 6, Table 7, and Table 8, respectively. Also, holes spacing for cutting depths are extracted from detailed results for beams and the average of these ratios are calculated and presented in Table 9.

6. Conclusions

Optimization and parametric studies of steel floor systems with composite and castellated beams and steel decks are performed in this study. The objective function is the floor cost where 17 variables and parameters are considered. The stress, stability, deflection and vibration criteria are all discussed. Results indicate that:

1. Using the high price decks in order to amplify the composite action can improve the results and decrease the cost between 5% to 10% in composite beams and composite castellated

beams. It seems there is little chance for the chosen higher price decks to result in the best result. So considering the first three acceptable decks is a good assumption.

2. Considering different number of divisions, can decrease total cost between 10% to 20%.
3. Using composite castellated beams improves the results by about 14% compared to the composite beams.
4. The optimum degree of castellated cutting angle is about 63^o.
5. Average ratio of hole spacing to cutting depth is between 2 and 3. (This ratio on commercial castellated beams is 3),

The results show that the utilized optimization algorithm, ECBO, performs quite well, and it has reliable and accurate solution. The fast-converging feature of the standard CBO is generally preserved in ECBO, whereas the modifications of the latter algorithm improve the exploration capabilities of the CBO. One can conclude that ECBO algorithm is competitive with the other available optimization methods. For an extensive comparative study of ECBO, when applied to different structural optimization problems, one can refer to Kaveh (2014),

Some useful suggestions for future improvements consist of the following:

1. Considering live load reduction as discussed in steel design codes may improve the accuracy of the results.
2. Considering camber, the cost of producing negative deflection and its effect on the reduction of deflection can improve the results for which the critical constraints are deflection or vibration.
3. Different deck types can change the results of optimization.
4. Rotating beams and decks or altering the division number of interior beams can lead to better results.
5. Using combination of 2 beam types with different yield capacity to construct castellated beams, or considering the cutting centerlines as inclined lines, to produce variable height castellated beams, can decrease the total cost of the floor.

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