3rd International Conference on Steel & Structure
11-12 December 2018
Olympic Hotel, Tehran, Iran

Analytical Shape Optimization of Metal Deck with respect to Bending Capacity

*Nader Fanaie[1], Payman Rajaeian[2]

Abstract

Metal decks are commonly used in building construction as well as bridge industry as formwork for the wet concrete slab. They have ribbed profile with embossments designed to interlock with concrete slab creating a reinforced concrete slab serving the dual purpose of permanent form and positive reinforcement. The in-plane stiffness and strength of the metal forms are widely relied upon for stability bracing in buildings. There have been a number of previous investigations focused on metal decks. This paper aims to determine optimum rib height of the section so as to achieve maximum bending capacity. To this end, by keeping the whole length of the plate in a period being constant, its elastic section modulus is extremized, which results in $h_r = 3W_r$ as optimum condition, where h_r and W_r are rib height and rib width, respectively. To the authors' knowledge this is the first time that such analytical calculations are being performed.

Keywords: metal deck; rib height; bending capacity; elastic section modulus; optimization

Introduction

Over the past few decades, steel and concrete have been widely used in construction, especially composite floor consisting of a concrete deck systems poured on top of corrugated steel sheets. Steel-concrete composite structures are those which have the merits of both used materials. A merit of such structures is their high bending strength. Being strong, lightweight, cost-effective, and easy to install, metal decks are commonly utilized in building construction as well as engineering practice. As shown in Fig. 1, metal deck is made by cold forming of structural steel sheet into a repeating pattern of parallel ribs (SDI, 3rd edition). The concrete thickness above the top of the steel deck shall not be less than $50\,\text{mm}$, nor that required by any applicable fire resistance ratings requirements. Minimum concrete cover for reinforcement shall be in accordance with ACI 318 (ANSI, 2011). Fig. 2 illustrates steel headed stud anchors, as

*1 .Corresponding author, K.N. Toosi University of Technology, Civil Engineering Department, No. 1346,Vali-Asr Street, P.O. Box. 15875-4416, 19697 Tehran, Iran, Tel.: +98 21 8877 9623, Email: fanaie@kntu.ac.ir

2. Iran University of Science and Technology, Tehran, Iran, Email: Payman.civil2012@gmail.com

required by the specification, to project a minimum of 38 mm above the deck flutes (AISC, Chicago, IL, 2016). This is intended to be the minimum in-place projection, and stud length prior to installation should account for any shortening of the stud that could occur during the welding process. The minimum specified cover over a steel headed stud anchor of 13 mm is after installation is intended to prevent the anchor from being exposed after construction is complete (AISC, Chicago, IL, 2016).

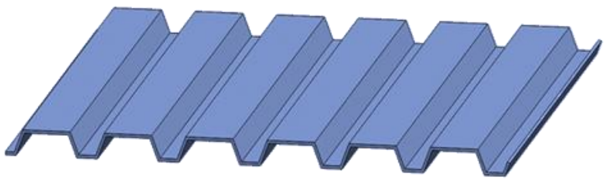

Figure 1. Metal deck

The design rules for composite construction with formed steel deck are based upon a study (Grant et al., 1977) of the then available test results (AISC, Chicago, IL, 2016).

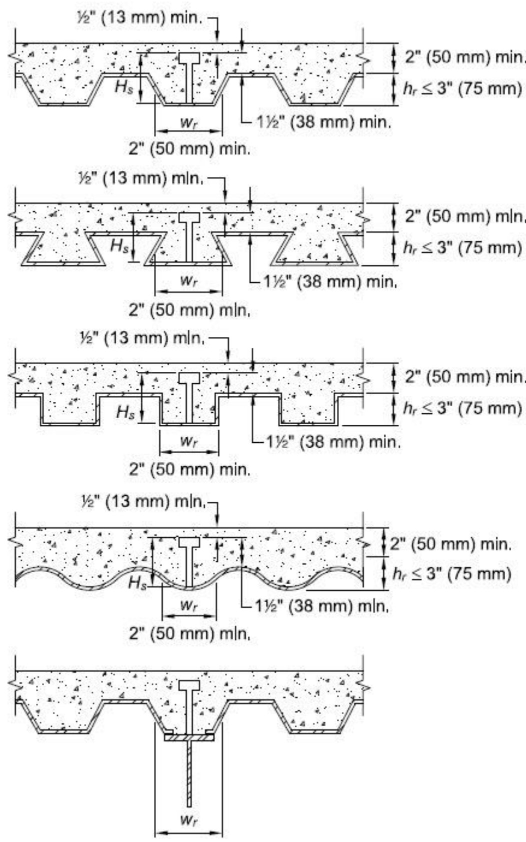

Figure 2. Steel deck limits

Shear connection at the interface of a concrete slab and supporting steel members is an assembly consisting of the connector, typically a steel headed stud anchor, its weld to the steel member, and the surrounding concrete with a specific deck flute geometry. Shear connection deforms when subjected to shear at the interface. Its ability to deform without fracturing is known as slip capacity or ductility of the shear connection. It is important to note that the term ductility does not merely relate to the ductility of the connector itself, but to the ductility of the overall shear connection assembly (AISC, Chicago, IL, 2016). Flexural strength of a composite section based on plastic stress distribution is the most typical manner for calculating the member strength. It assumes sufficiently ductile steel and concrete components capable of developing a fully plastic stress block across the depth of the composite section. This analysis also assumes a sufficiently ductile shear connection allowing for the shear at the interface to be evenly shared among the connectors located between the points of zero and maximum moment (AISC, Chicago, IL, 2016). An implicit assumption of the plastic stress distribution methodology is that the shear demands at the interface can be uniformly distributed over the shear span because the connectors are ductile and can redistribute the demands (Veist et al., 1997). Even when shear connections possess adequate ductility to accommodate interfacial slip, excessive slip demand at the interface will cause excessive discontinuities in the strain diagram at the interface of the concrete slab and cause early departure from the elastic behavior, and as a consequence invalidate the design approach. It is therefore important to limit the shear connection ductility demand at the interface. Composite beam tests in which the longitudinal spacing of steel anchors was varied according to the intensity of the static shear, and duplicate beams in which the anchors were uniformly spaced exhibited approximately the same ultimate strength and the same amount of deflection at nominal loads (AISC, Chicago, IL, 2016). Under distributed load conditions, only a slight deformation in the concrete near the more heavily stressed anchors is needed to redistribute the horizontal shear to other less heavily stress anchors (AISC, Chicago, IL, 2016). In computing the available flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam within the effective width of the slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, steel anchors are required to transfer the ultimate tensile force in the reinforcement from the slab to the steel beam. When steel deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. These create trenches that completely or partially replace sections of the concrete slab above the deck. These trenches running parallel to or transvers to a composite beam may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange (AISC, Chicago, IL, 2010). When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench. Trenches oriented transvers to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as noncomposite (AISC, Chicago, IL, 2010). A number of investigations have been conducted on metal decks. Thondel et al. (2012) studied the behavior of steel-concrete deck composite beam with high ribbed transversally oriented with the aim of exploring what influence a total height of the waves will have on beam deflection. Ezzat H. Fahmy et al. (2008) investigated the development of longitudinal cracking phenomenon in composite beams with ribbed metal deck. According to their research report, existence of the metal deck and height of the metal deck were identified as having effect on the longitudinal cracking of concrete slabs. Azmi et al. (1975) studied experimentally the longitudinal cracking phenomenon in composite beams with ribbed concrete slab on 76mm deep metal deck. They reported that the deck contributes with the transverse reinforcement of the slab in resisting the longitudinal cracking. The results showed that the presence of the ribbed metal deck improved the longitudinal cracking behavior of the composite beam in the case of single point loading. In addition, increasing the height of the ribbed metal deck improved the longitudinal cracking behavior of the composite beam. The metal deck has an important role when the composite beam is subjected to concentrated load applied over the steel beam. Elkelish et al. (1986) presented the results of an analytical investigation of the longitudinal cracking in composite beams with ribbed metal deck using the layered finite element technique. They concluded that longitudinal cracking occurs first at the bottom of the slab when beam was subjected to point loading applied over the steel beam while it starts at the top of the slab when the beam was subjected to a uniformly distributed load over the slab area. Ozgur Egilmez et al. (2012) studied buckling behavior of steel bridge I-girders braced by permanent metal deck forms. Ozgur et al. (2009) investigated lateral stiffness of steel bridge I-girders braced by metal deck forms. Hatefi et al. (2018) conducted research on cost optimization of a composite metal floor deck using harmony search metaheuristic algorithm. In the current study the authors aim to determine optimum rib height so as to achieve maximum bending capacity.

Details of calculations

A typical section of the metal deck is illustrated in Fig. 3. As it can be seen, the section follows a periodic pattern. A period of this section named as plate wave length, as depicted in Fig. 4, is considered in the following calculations.

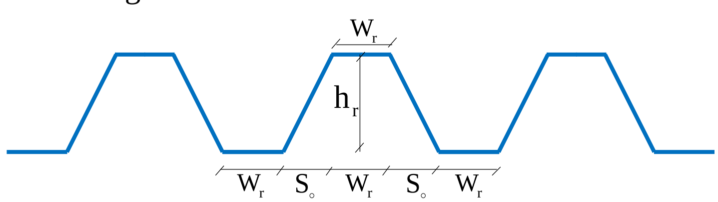

Figure 3. Typical metal deck section

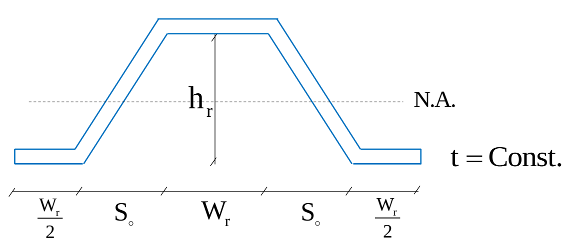

Figure 4. Plate wave length

Being a symmetrical section, the neutral axis passes through its mid height. Employing strength of materials concepts, section moment of inertia and elastic section modulus are calculated. For simplicity of calculation an equivalent section, as shown in Fig. 5, is considered. It should be noted that in this method, cross sectional area does not change. Equating the cross sectional area of main section with that of equivalent section, the corresponding thickness, t', is determined as follows.

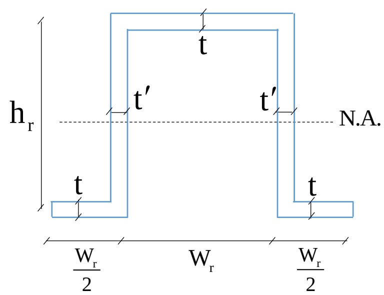

Figure 5. Equivalent section

$$A_1 = A_2 \rightarrow \sqrt{h_r^2 + S_\circ^2} \times t = h_r \times t' \rightarrow t' = \frac{\sqrt{h_r^2 + S_\circ^2}}{h_r} t \rightarrow t' = \sqrt{1 + (\frac{S_\circ}{h_r})^2}\, t \qquad (1)$$

A_1 and A_2 are the cross sectional area of each oblique and vertical wall respectively.

$$I = W_r t(\frac{h_r}{2})^2 + 2 \times \frac{W_r}{2} t(\frac{h_r}{2})^2 + 2 \times \frac{t' h_r^3}{12} = \frac{W_r t h_r^2}{2} + \frac{1}{6} t' h_r^3 = \frac{W_r t h_r^2}{2} + \frac{1}{6}\sqrt{1 + (\frac{S_\circ}{h_r})^2}\, t h_r^3$$

$$I = \frac{t h_r}{2}[W_r + \frac{h_r}{3}\sqrt{1 + (\frac{S_\circ}{h_r})^2}] \qquad (2)$$

$$S = \frac{I}{C} = \frac{I}{\frac{h_r}{2}} = \frac{\frac{t h_r^2}{2}[W_r + \frac{h_r}{3}\sqrt{1 + (\frac{S_\circ}{h_r})^2}]}{\frac{h_r}{2}} = t h_r[W_r + \frac{h_r}{3}\sqrt{1 + (\frac{S_\circ}{h_r})^2}]$$

$$S = t h_r[W_r + \frac{h_r}{3}\sqrt{1 + (\frac{S_\circ}{h_r})^2}] \qquad (3)$$

It is assumed that metal decks with various geometric properties have the same allowable stress resulting in bending capacity to be proportional to the elastic section modulus. In addition, it is rational to assume metal deck thickness to be constant.

$$\sigma_{max} = \frac{M_{max}}{S} \xrightarrow{\sigma_{max}=\sigma_{all}} M_{all} = S \times \sigma_{all} \tag{4}$$

Changing section parameters W_r , $S_\circ$, and h_r, the whole length of the plate as well as its weight change. To determine the most cost-effective metal deck, different plates with constant whole length need to be investigated. By keeping the whole length of plate being constant, the elastic section modulus is extremized. The whole length of plate in a period is as follows.

$$(W_r)_{tot} = \frac{W_r}{2} + \sqrt{h_r^2 + S_\circ^2} + W_r + \sqrt{h_r^2 + S_\circ^2} + \frac{W_r}{2} = 2W_r + 2\sqrt{h_r^2 + S_\circ^2} = \text{Const.} \tag{5}$$

If the constant parameter in the above mentioned equation is denoted by $2C$, where C is constant, it yields:

$$W_r + \sqrt{h_r^2 + S_\circ^2} = C \rightarrow W_r = C - \sqrt{h_r^2 + S_\circ^2} \tag{6}$$

$$S = th_r[W_r + \frac{h_r}{3}\sqrt{1+(\frac{S_\circ}{h_r})^2}] = th_r[C - \sqrt{h_r^2 + S_\circ^2} + \frac{h_r}{3}\sqrt{1+(\frac{S_\circ}{h_r})^2}]$$

$$= th_r[C - \sqrt{h_r^2 + S_\circ^2} + \frac{1}{3}\sqrt{h_r^2 + S_\circ^2}] = th_r[C - \frac{2}{3}\sqrt{h_r^2 + S_\circ^2}] \tag{7}$$

To determine the optimum rib height of the metal deck, its elastic section modulus needs to be extremized with respect to its height. According to the equation (7), in order to maximize the elastic section modulus it implies that the parameter $S_\circ$ needs to be considered as zero. This point is included in the following calculations.

$$S = th_r[C - \frac{2}{3}\sqrt{h_r^2 + S_\circ^2}]$$

$$\frac{dS}{dh_r} = 0 \rightarrow t[C - \frac{2}{3}\sqrt{h_r^2 + S_\circ^2}] + th_r \times \frac{-\frac{2}{3} \times 2h_r}{2\sqrt{h_r^2 + S_\circ^2}} = 0 \rightarrow t[C - \frac{2}{3}\sqrt{h_r^2 + S_\circ^2}] - \frac{\frac{2}{3}th_r^2}{\sqrt{h_r^2 + S_\circ^2}} = 0$$

$$(C - \frac{2}{3}\sqrt{h_r^2 + S_\circ^2})\sqrt{h_r^2 + S_\circ^2} - \frac{2}{3}h_r^2 = 0 \rightarrow C\sqrt{h_r^2 + S_\circ^2} - \frac{2}{3}h_r^2 - \frac{2}{3}S_\circ^2 - \frac{2}{3}h_r^2 = 0 \tag{8}$$

$$\rightarrow C\sqrt{h_r^2 + S_\circ^2} = \frac{2}{3}(S_\circ^2 + 2h_r^2) \xrightarrow{S_\circ=0} Ch_r = \frac{4}{3}h_r^2 \rightarrow h_r = \frac{3}{4}C$$

$$h_r = \frac{3}{4}(W_r + \sqrt{h_r^2 + S_\circ^2}) \xrightarrow{S_\circ=0} h_r = \frac{3}{4}(W_r + h_r) \rightarrow h_r = \frac{3}{4}W_r + \frac{3}{4}h_r \rightarrow \frac{1}{4}h_r = \frac{3}{4}W_r \rightarrow h_r = 3W_r$$

Elastic section modulus function in terms of h_r is a monotonically increasing function, up to $h_r = 3W_r$ (The derivative of elastic section modulus with respect to h_r is positive). By increasing in the rib height up to $h_r = 3W_r$, elastic section modulus increases. According to the code limitations, the rib height is not permitted to be greater than three times the rib width. Based on the requirement of the Iranian National Building Regulations, Part 10, nominal rib height of the metal decks should not exceed 75mm. The average rib width filled with concrete should not be less than 50mm. Theoretical relationships derived are not applicable to metal deck construction since if the parameter $S_\circ$ is considered to be zero, it results in the oblique walls of the section to be vertical causing the concrete cohesion to be less effective. Code provisions has made no mention of the slope of oblique walls, $S_\circ$. Average rib width is defined as $W_r + \frac{S_\circ}{2} + \frac{S_\circ}{2} = W_r + S_\circ$

According to code limitations previously mentioned it concludes that

$$h_r \leq 75\text{mm} \quad , \quad W_r + S_\circ \geq 50\text{mm}$$

According to the aforementioned calculations, the deeper is the rib height; the optimum elastic section modulus is achieved. Based on the code provisions, if the section rib height, h_r, is assumed to be 75mm as its maximum value, considering vertical walls, maximum elastic section modulus is achieved.

The variation of moment of inertia and elastic section modulus with respect to rib height are illustrated in Figs. 6 and 7.

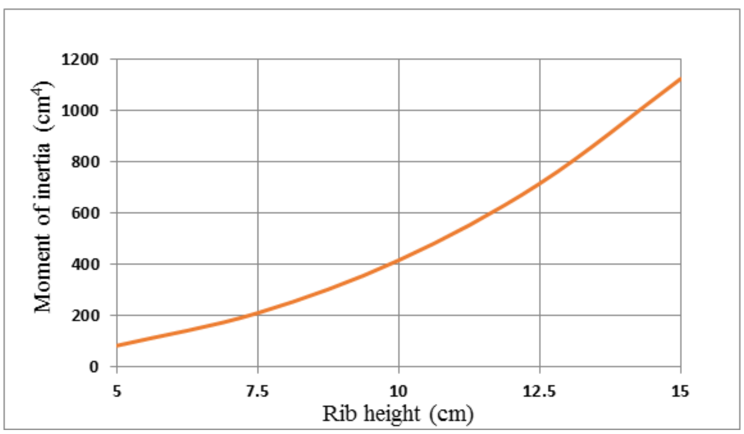

Figure 6. Variation of moment of inertia

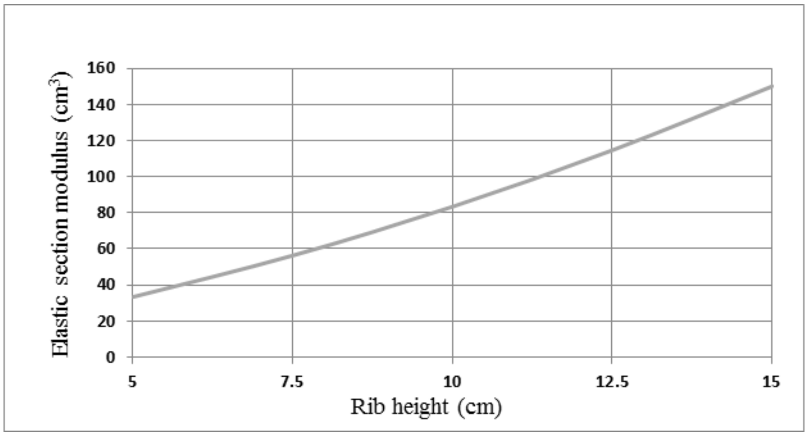

Figure 7. Variation of elastic section modulus

In what follows, a couple of examples are presented. It should be mentioned that the rib height is assumed to be 75mm in the calculations.

$$1)\ W_r = 50\,\text{mm}, S_{\circ} = 0 \rightarrow S = th_r[W_r + \frac{h_r}{3}\sqrt{1+(\frac{S_{\circ}}{h_r})^2}\,] = 56.25t$$

$$2)\ W_r = 40\,\text{mm}$$

$$W_r + S_{\circ} \geq 50\,\text{mm} \rightarrow S_{\circ} \geq 10\,\text{mm}$$

$$W_r + \sqrt{h_r^2 + S_{\circ}^2} = \text{Const} = \underbrace{50+75}_{\text{Case: } S_{\circ}=0} = 125 \rightarrow 40 + \sqrt{75^2 + S_{\circ}^2} = 125$$

$$\rightarrow \sqrt{75^2 + S_{\circ}^2} = 85 \rightarrow S_{\circ} = \sqrt{85^2 - 75^2} = 40\,\text{mm} > 10\,\text{mm}\ ,\ W_r + S_{\circ} = 40 + 40 = 80 > 50\,\text{mm}$$

$$S = th_r[W_r + \frac{h_r}{3}\sqrt{1+(\frac{S_{\circ}}{h_r})^2}\,] = t \times 75[40 + \frac{75}{3}\sqrt{1+(\frac{40}{75})^2}\,] = 51.25t$$

$$3)\ W_r = 30\,\text{mm}$$

$$W_r + S_{\circ} \geq 50\,\text{mm} \rightarrow 30 + S_{\circ} \geq 50\,\text{mm} \rightarrow S_{\circ} \geq 20\,\text{mm}$$

$$W_r + \sqrt{h_r^2 + S_{\circ}^2} = \text{Const} = \underbrace{50+75}_{\text{Case } S_{\circ}=0} = 125\,\text{mm}$$

$$\rightarrow 30 + \sqrt{75^2 + S_{\circ}^2} = 125 \rightarrow \sqrt{75^2 + S_{\circ}^2} = 95 \rightarrow S_{\circ} = \sqrt{95^2 - 75^2} = 58.3\,\text{mm} > 20\,\text{mm}$$

$$S = th_r[W_r + \frac{h_r}{3}\sqrt{1+(\frac{S_{\circ}}{h_r})^2}\,] = t \times 75[30 + \frac{75}{3}\sqrt{1+(\frac{58.3}{75})^2}\,] = 46.25t$$

The above mentioned calculations reveal that the case $S_{\circ} = 0$ yields the optimum condition.

Conclusions

The most important results obtained from this research are as follows:

1. As calculations show, by increasing the rib height up to $h_r = 3W_r$, elastic section modulus increases.
2. Obtaining the optimum condition for metal decks, maximum rib height needs to be used. Iranian National Building Code, Part 10, limits the rib height to 75mm which can be utilized as the most efficient one.
3. According to the obtained results, the case $S_\circ = 0$ brings about the optimum condition.
4. If the Construction Engineering Organization allows vertical walls to be used in metal decks, it is by far the most efficient, otherwise maximum slope for the walls needs to be used. In addition, the criterion $W_r + S_\circ \geq 50\text{mm}$ should be taken into account.

References

SDI Manual of Construction with Steel Deck, 3rd edition – Steel Deck Institute.

ANSI – American National Standards Institute/Steel Deck Institute, C – 2011 Standard for Composite Steel Floor Deck-Slabs.

AISC, Specification for Structural Steel Building, AISC/ANSI 360-16, American Institute of Steel Construction (AISC), Chicago, IL, 2016.

AISC, Specification for Structural Steel Building, AISC/ANSI 360-10, American Institute of Steel Construction (AISC), Chicago, IL, 2010.

Thondel, S., Studnicka, J., 2012. Behaviour of steel-concrete composite beam with high ribbed deck. Procedia Eng., 40, 457-462.

Ezzat H. Fahmy, Taalat F. Abu-Amra, 2008. Longitudinal cracking of concrete slabs in composite beams with ribbed metal deck. Journal of constructional steel research, 64, 670-679.

Azmi, H., 1975. Strength of composite beams incorporating 3-in cellular metal deck. Thesis submitted in partial fulfillment of the requirements for the M. Eng. Degree. Hamilton (Ontario, Canada): McMaster University.

Elkelish MS., Robinson, H., 1986. Longitudinal cracking of composite beams with ribbed metal deck. Canadian Journal of Civil Engineering, 13(6), 733-740.

Ozgur Egilmez, O., Todd A. Helwig, M.ASCE, and Reagan Herman, M.ASCE., 2012. Buckling behaviour of steel bridge I-Girders braced by permanent metal deck forms. Journal of bridge Eengineering, 17, Issue 4.

Ozgur Egilmez, O., Reagan S. Herman, Todd A. Helwig, 2009. Lateral stiffness of steel bridge I-Girders braced by metal deck forms. Journal of bridge engineering, 14, Issue 1.

Hatefi, H., Karamodin, A., Baygi, S., 2018. The cost optimization of a composite metal floor deck by harmony search metaheuristic algorithm. International journal of optimization in civil engineering, 8(1), 101-111.