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OPTIMUM DESIGN OF ONE-BAY PORTAL STEEL FRAMES

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ABSTRACT

The paper presents optimum design of statically indeterminate two-hinged steel portal frames under multiple loadings. An explicit formulation of the analysis equations using the Virtual Work Method is developed. Loading cases include both gravity loads and wind loads. Design equations involving local buckling, lateral torsional buckling, shear buckling, combined stresses and deflection constraints, as provided by the latest Egyptian Code of Practice for Steel Construction and Bridges, are included. The objective function is chosen as the minimum weight of the structure. The design variables are the cross-sectional dimensions of the built-up sections for rafters and columns. The design constraints cover all cases of discontinuity for compact prismatic sections. Ordinary mild steel and high tensile steel cases are considered. The optimization technique adopted in this research is the Modified Method of Feasible Directions. Several examples are presented to validate the efficiency of the formulation and to prove that the designs obtained in this work are more economical than those provided by other classical design approaches. Savings up to fifty percent of the weight of the frame are achieved for some cases.

KEYWORDS: Optimization; steel frames; virtual work; design codes; feasible directions.

1. INTRODUCTION

Optimum design of structures has been an active area of research for more than four decades [1]. Numerous publications are now available covering different aspects of this topic. However, periodic updating of design codes, failure of some optimization formulations to capture all design requirements, and reluctance of design firms to adopt optimization techniques on the practical level, necessitate more research efforts in this direction.

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An optimality criterion approach is developed for optimum structural design of steel frames on a parallel machine [2]. The optimum design of a planar steel framed structure subjected to a single load case, using a single design variable for each cross-section, is studied [3]. A large space frame steel structure subjected to realistic AISC-specified stress, displacement, and buckling constraints is optimized on supercomputers [4]. An algorithm which takes into account the non-linear response of the frame due to the effect of axial loads, sway constraints, and combined stress limitations is developed [5]. An algorithm for preliminary minimum weight design of moment frames for lateral loading, maintaining the least possible drift for the given loading and geometry is introduced [6]. A new method for the optimum design of frames with stress, stiffness and stability constraints is presented [7]. An algorithm is used for the optimum design of steel frames composed of tapered members having an I-section. The displacements at joints are considered as constraints [8].

The goal of this research is to develop an efficient algorithm for minimum weight design of one-bay two-hinged portal steel frames composed of prismatic built-up sections, as compared to other classical design approaches. Local buckling, lateral torsional buckling, shear buckling, combined stresses, and deflection constraints, as given by ECP'01 [9], are included. Gravity loads and wind loads are considered. The Modified Method of Feasible Directions (MMFD) optimization technique [10] is used to solve the problem. To this end, the rest of this work is organized as follows. First, the structural problem is posed and the method of analysis together with the resulting straining actions is presented. Next, the design variables, the objective function, and the constraints are identified. Then, three examples are introduced to validate the efficiency of the developed algorithm. Finally, several conclusions are drawn.

2. PROBLEM STATEMENT

Figure 1 outlines a plane one-bay two-hinged portal/gable steel frame of span L, height H, and rafter slope angle ϕ. The cross-sections of the column and rafter are made of built-up sections as shown in the figure. The design variables t_1, b_1, t_2, and h_1 define the thickness and width of the column flange and web, respectively. Similarly, t_3, b_2, t_4, and h_2 define the same dimensions for the rafter's section.

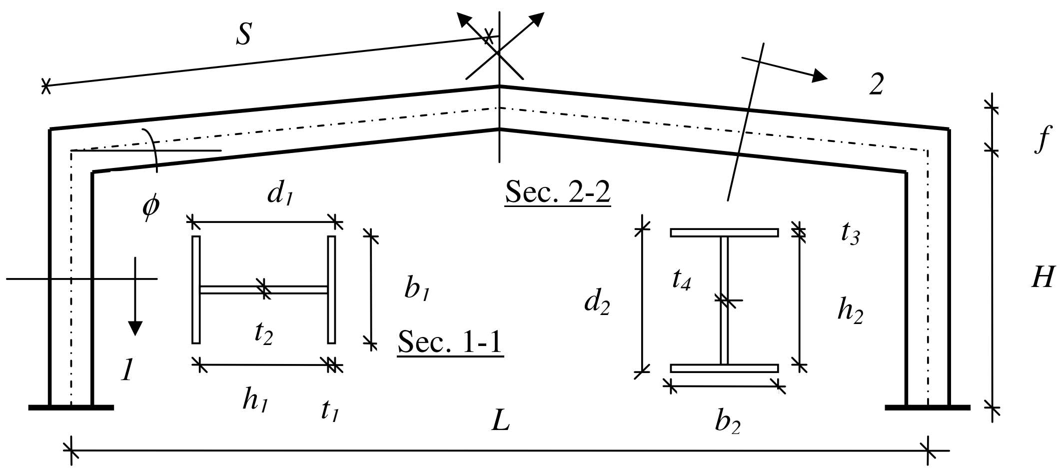

Fig.1. Layout of Portal Frame and Design Parameters

Vertical dead and live loads and lateral wind loads, as per ECP'93 [11], are considered in this work. Using the Method of Virtual Work [12], closed form solutions for the normal forces, shearing forces, and bending moments at different frame locations are derived. The internal forces values are given in terms of the design variables shown in Fig.1, the parameters given in the list of symbols at the end of the paper, and the following formulae:

$$\psi = \frac{8H + 5f}{\frac{H^3 K}{S} + 3H^2 + 3Hf + f^2} \tag{1}$$

$$\eta = \frac{3H + 2f}{8H + 5f} \tag{2}$$

$$K = \frac{Ix_2}{Ix_1} \tag{3}$$

The internal force diagrams for both gravity loads and lateral loads are illustrated in Figs. 2 and 3, respectively. Values of the straining actions at different joints are also shown in these figures. It should be mentioned here that, the live load

Fig.2. Straining Actions for Gravity Loads

values provided by ECP'93 should be transformed from the horizontal projection to the inclined span length of the rafter before using the formulae given in Fig.2. Furthermore, the direction of bending moment M_3 shown in Fig. 3 may be reversed to the outside of the frame depending upon the dimensions of the rafter span and the column height.

3. OPTIMIZATION FORMULATION

Using the results of the aforementioned analysis for defining the constraints for compact built-up sections, as given by the latest version of the Egyptian Code of Practice for Steel Construction and Bridges ECP'01, the following optimization formulation is obtained:

Minimize:

$$W_t = 2(A_1 \times H + A_2 \times S)\gamma_s \tag{4}$$

subject to:

$$\frac{(h_1 - 2S_w)}{t_2} - \frac{699 / \sqrt{F_y}}{13\alpha_1 - 1} \leq 0 \quad or$$

$$\frac{(h_1 - 2S_w)}{t_2} - \frac{58}{\sqrt{F_y}} \leq 0 \tag{5}$$

$$\frac{(b_1 - t_2 - 2S_w)}{2t_2} - \frac{15.3}{\sqrt{F_y}} \leq 0 \tag{6}$$

$$\frac{H}{b_1} - \frac{20}{\sqrt{F_y}} \leq 0 \tag{7}$$

$$\frac{H(h_1 + 2t_1)}{b_1 t_1} - \frac{1380C_b}{F_y} \leq 0 \tag{8}$$

$$\frac{h_1}{t_2} - \frac{105}{\sqrt{F_y}} \leq 0 \tag{9}$$

$$\frac{K_b H}{ix_1} - 180 \le 0 \quad and \quad \frac{H'}{iy_1} - 180 \le 0 \tag{10}$$

$$\frac{WS}{(2b_1t_1 + h_1t_2)F_c} + \frac{WSLH\psi}{32F_{cb}Ix_1} d_1 A'_1 - 1 \le 0 \quad or$$

$$\frac{WS + 0.55W'L - R_1}{(2b_1t_1 + h_1t_2)F_c} + \frac{WSLH\psi + 16M_3}{32F_{cb}Ix_1} d_1 A'_1 - 1.20 \le 0 \tag{11}$$

$$\frac{WSL\psi}{16h_1t_2} - q_b \le 0 \tag{12}$$

$$\frac{(h_2 - 2S_w)}{t_4} - \frac{699/\sqrt{F_y}}{13\alpha_2 - 1} \le 0 \quad or$$

$$\frac{(h_2 - 2S_w)}{t_4} - \frac{58}{\sqrt{F_y}} \le 0 \tag{13}$$

$$\frac{(b_2 - t_4 - 2S_w)}{2t_3} - \frac{15.3}{\sqrt{F_y}} \le 0 \tag{14}$$

$$\frac{S_p}{b_2} - \frac{20}{\sqrt{F_y}} \le 0 \tag{15}$$

$$\frac{S_p(h_2 + 2t_3)}{b_2t_3} - \frac{1380C_b}{F_y} \le 0 \tag{16}$$

$$\frac{h_2}{t_4} - \frac{105}{\sqrt{F_y}} \le 0 \tag{17}$$

$$\frac{S}{ix_2} - 180 \le 0 \quad and \quad \frac{S_p}{iy_2} - 180 \le 0 \tag{18}$$

$$\frac{W\left(\frac{L^2\psi}{32} + f\right)}{F_c(h_2t_4 + 2b_2t_3)} + \frac{WSLH\psi}{32F_{cb}Ix_2} d_2 A''_1 - 1 \le 0 \quad or$$

$$\frac{W\left(\frac{L^2\psi}{32} + f\right) + (0.55W'L - R_1)\frac{f}{S} - [(0.80H - 0.10f)W' - X_1]\frac{L}{2S}}{F_c(h_2t_4 + 2b_2t_3)}$$

$$+ \frac{WSLH\psi + 16M_3}{32F_{cb}Ix_2} d_2 A''_1 - 1.20 \le 0 \tag{19}$$

$$\frac{WL^2\psi}{32(h_2t_4+2b_2t_3)}\times\frac{1}{F_c}+\frac{WSL\left(1-\frac{\psi(H+f)}{4}\right)}{8F_{cb}Ix_2}d_2A''_1-1\leq 0 \qquad (20)$$

$$\frac{WL(1-\frac{f\psi}{8})}{2h_2t_4}-q_b\leq 0 \qquad (21)$$

$$\frac{WLf\psi}{16h_2t_4}-q_b\leq 0 \qquad (22)$$

$$\left\{\begin{array}{l}\frac{W_LSL^2\psi^2H^3\eta}{192EIx_1}+\frac{W_LS^2LH\psi}{16EIx_2}\left[\frac{\psi\eta L}{8}(3H+2f)-\frac{L}{12}\right]\\-\frac{W_LS^2L}{4EIx_2}\left[1-\frac{\psi(H+f)}{4}\right]\left[\frac{\psi\eta L}{8}(3H+2f)-\frac{L}{6}\right]\\-\frac{W_LS^2L}{12EIx_2}\left[\frac{\psi\eta L}{8}(H+\frac{f}{2})-\frac{L}{8}\right]-\frac{L}{300}\leq 0\end{array}\right\} \qquad (23)$$

$$\left\{\begin{array}{l}\frac{W_LSL^2\psi^2H^3\eta}{192EIx_1}+\frac{W_LS^2LH\psi}{16EIx_2}\left[\frac{\psi\eta L}{8}(3H+2f)-\frac{L}{12}\right]\\-\frac{W_LS^2L}{4EIx_2}\left[1-\frac{\psi(H+f)}{4}\right]\left[\frac{\psi\eta L}{8}(3H+2f)-\frac{L}{6}\right]\\-\frac{W_LS^2L}{12EIx_2}\left[\frac{\psi\eta L}{8}(H+\frac{f}{2})-\frac{L}{8}\right]\end{array}\right\}\frac{2f}{\sqrt{4f^2+L^2}}-\frac{H}{150}\leq 0 \qquad (24)$$

$$0\leq t_1,t_2,t_3,t_4,h_1,h_2,b_1,b_2\leq 100000 \qquad (25)$$

The correspondence between the constraint numbers, as given in this work, and those stated by ECP'01 is shown in Table (1). It is assumed that a knee bracing is utilized to connect the compression flange in the lower side of the rafter with the purlins. Consequently, the laterally unsupported length of the rafter is taken equal to the spacing between purlins, S_p. It should be noted that Eqs. (5-24) are given in MKS metric units; i.e. kgf or tonf is used as the unit of forces and cm is used as the unit of lengths. However, all dimensions and results that presented in this paper are given in SI units.

Table 1. Types of Constraints as per ECP' 01

Element	Constraint Type	Equation Number	ECP' 01
Column	Local buckling	5	Table 2.1a, Page 9
		6	Table 2.1c, Page 11
	Lateral buckling	7	Equation 2.18, Page 16
		8	Equation 2.18, Page 16
		10	Table 4.1, Page 51
	Shear buckling	9	Equation 2.3, Page 14
	Shear stress	12	Equations 2.7-2.10, Pages 14 and 15
	Combined normal stresses	11	Equation 2.35, Page 25
	Horizontal deflection	24	Table 9.1, Page 132
Rafter	Local buckling	13	Table 2.1a, Page 9
		14	Table 2.1c, Page 11
	Lateral buckling	15	Equation 2.18, Page 16
		16	Equation 2.18, Page 16
		18	Table 4.1, Page 51
	Shear buckling	17	Equation 2.3, Page 14
	Shear stress	21,22	Equations 2.7-2.10, Pages 14 and 15
	Combined normal stresses	19,20	Equation 2.35, Page 25
	Vertical deflection	23	Table 9.1, Page 132

The optimization formulation stated by Eqs. (4-25) is coded in Fortran77 programming language and linked to the MMFD optimization technique. The flow chart summarized the optimization method used in this work is shown in Fig. 4. Generally speaking, in the optimization of steel structures the built-up sections have an advantage than hot-rolled sections since the rounded approximation in the calculated results are very close to the optimal solution. Consequently, in this paper the obtained design variables are rounded in order to be utilized in the practical applications. The following section outlines the cases for which the formulation is experimented.

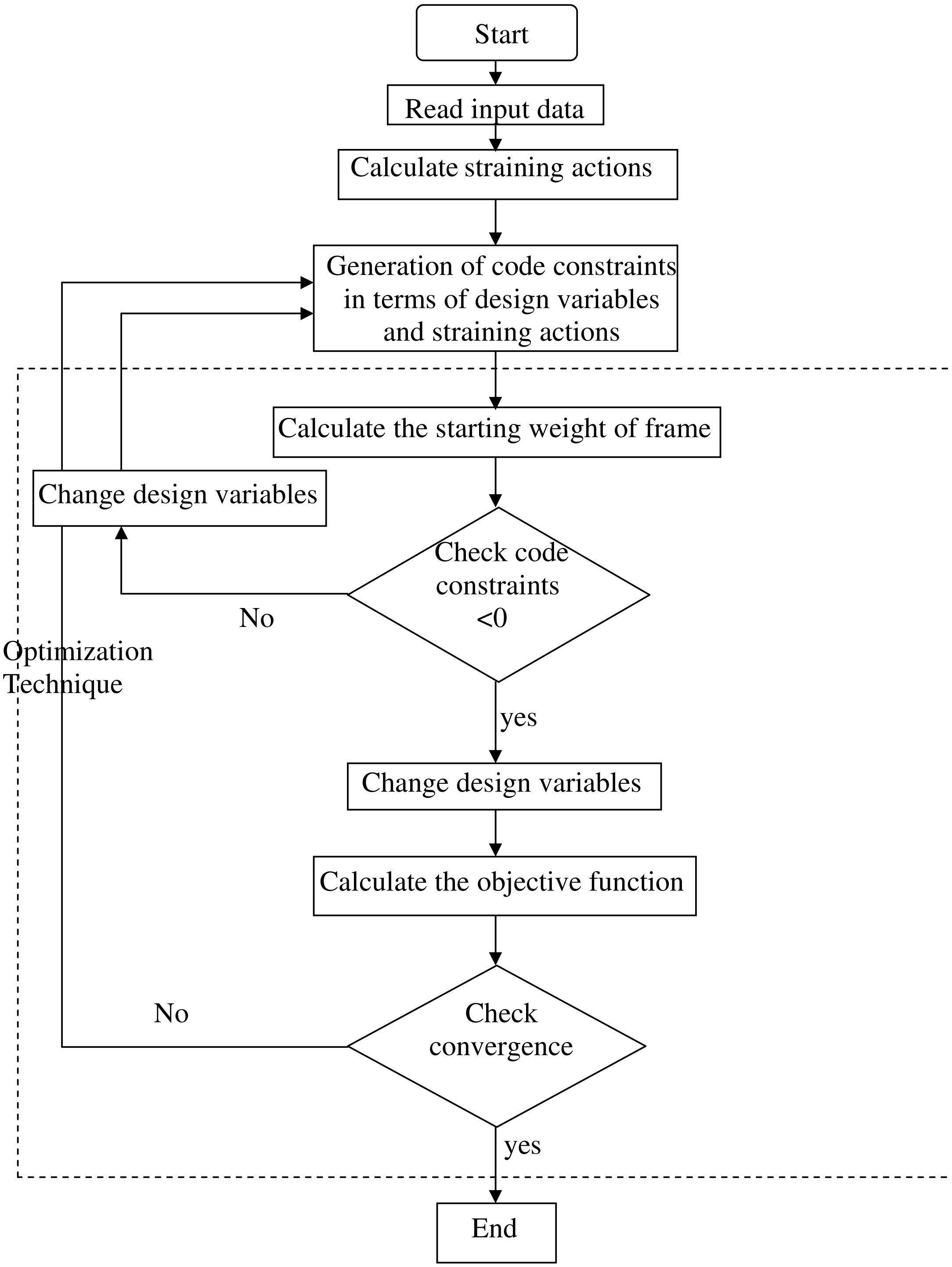

Fig.4. Flow Chart of the Optimization Method

4. APPLICATIONS

Three examples are presented in this work. Spans ranging from *9000* mms to *30000* mms are considered. Normal mild steel and high tensile steel cases are investigated. Comparisons are done for hot rolled sections and built-up sections. Each of the three case studies is summarized hereafter.

4.1 Example 1

The first example is a frame constructed in Ras Ghareb, Gulf of Suez. The frame span L is *9000* mms; the eave height H is *4500* mms; the rafter slope in *1:20* (refer to Fig.1). The steel grade used in this frame is high strength steel *(360/520)*. The cross-sectional dimensions of the built-up sections used for this frame are listed in Table 2. The heights of the tapered sections for the compared example are also indicated. The weight of the constructed frame is *3.7* KN.

Table 2. Design History for Example 1

Design Variables	Iteration Number								Compared Example
	0	5	10	15	20	25	Final-1	Final-2	
t_1(mm)	100	38.1	18.9	13.8	11.3	6.8	7.1	6.9	5.0
t_2(mm)	100	68.5	18.6	7.4	4.9	1.8	1.9	5.0	5.0
t_3(mm)	100	13.5	14.2	11.6	10.4	7.3	6.9	6.9	5.0
t_4(mm)	100	67.6	15.0	9.9	8.0	4.0	3.6	5.0	5.0
h_1(mm)	1000	903.4	786.7	410.5	271.2	98.9	106.5	104.0	200/300
h_2(mm)	1000	921.3	828.8	550.2	441.0	219.2	198.6	196.6	200/300
b_1(mm)	1000	530.3	339.6	246.7	202.9	128.2	132.5	132.6	150
b_2(mm)	1000	301.8	259.9	212.9	191.4	137.8	131.7	131.8	150
W_t (KN)	424.2	122.1	33.4	14.3	9.5	3.4	3.3	3.6	3.7

Using a starting point of $t_1 = t_2 = t_3 = t_4 = 100$ mms, $h_1 = h_2 = b_1 = b_2 = 1000$ mms and a starting weight of *424.2* KN, an optimal solution of *3.3* KN is achieved after *29* iterations. The iteration histories for the objective function and the design variables are given in Table 2 (Final-1). An improvement of about *10.8%* between the optimized weight found in this work and the actual constructed structure is achieved.

Figure 5 outlines the iteration history for the weight of the frame. The active constraints at optimality are those defined by Eqs. (6), (8), (9), (11), (14), (16), (17), and (23) in this work.

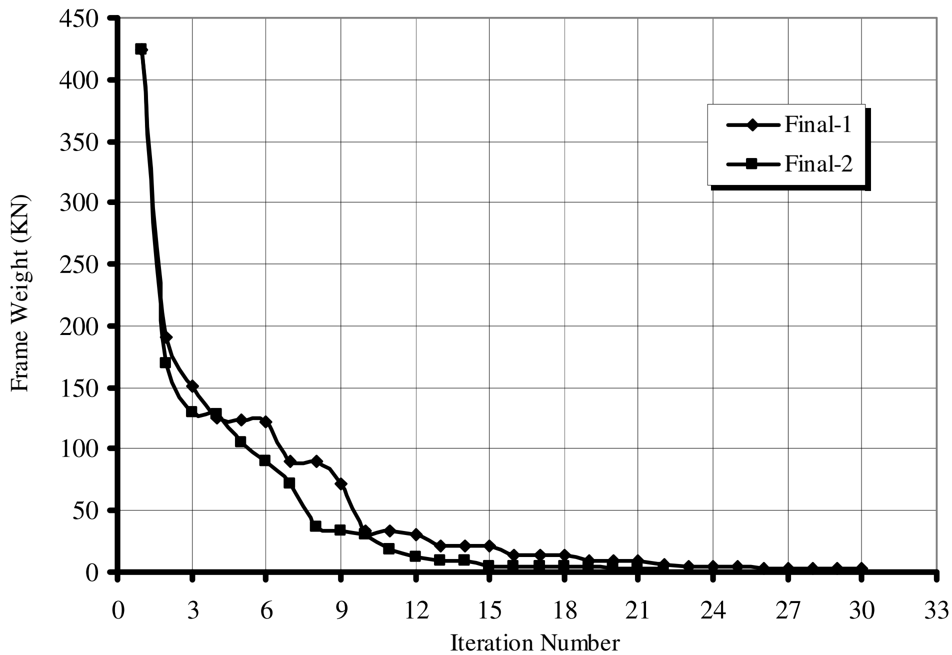

Fig.5. Iteration History for Example 1

However, *Section 7.1* of the ECP'01 requires that the minimum thickness for built-up sections is *5* mms. If this side constraint is enforced, a minimum weight of *3.6* KN is achieved after *21* iterations (Final-2). Specifics of this solution are also given in Table 2. The iteration history for this case is also shown in Fig. 5. If the calculated design variables of Final-2 are rounded to eliminate the fractions of mms a minimum weight of *3.66* KN is obtained. To this end, it is worthwhile noting that the cross-sections used for the constructed example do not satisfy ECP'01 design constraints.

4.2 Example 2

The second example is given in [13]. The span of the frame *L* is *22000* mms, the height of the eave *H* is *6000* mms and the angle ϕ of the rafter is *5.7˚*. Steel grade is normal mild steel (*240/350)*. The cross-sections for the compared example are prismatic hot rolled ones. The cross-sectional dimensions and weight of the frame are given in Table 3.

Table 3. Design History for Example 2

Design Variables	Iteration Number						Ref. [13]
	0	5	10	15	Final		
					Calculated	Rounded	
t_1(mm)	100	17.0	11.7	11.4	9.5	10.0	20.0
t_2(mm)	100	54.6	28.5	11.3	6.2	7.0	12.0
t_3(mm)	100	17.0	10.0	7.5	7.8	8.0	20.0
t_4(mm	100	15.8	11.0	8.8	7.8	8.0	12.0
h_1(mm)	1000	888.2	847.0	762.8	422.2	423.0	240.0
h_2(mm)	1000	784.6	743.2	593.6	530.2	531.0	240.0
b_1(mm)	1000	374.9	299.2	276.3	229.2	230.0	280.0
b_2(mm)	1000	403.6	244.2	187.9	194.7	195.0	280.0
W_t (KN)	803.3	103.0	51.9	27.9	19.1	19.8	38.4

Using the same unrealistic starting point of the previous example, a minimum weight of *19.1* KN is reached after *19* iterations with an improvement of *50%* between the weight of the frame and the one given in the stated reference. The iteration histories of the objective function and the design variables are listed in Table 3. Furthermore, the rounded design variables that give a total frame weight equal to *19.8* KN are shown in Table 3.

Two other starting points, one of which is infeasible and the other is an extremely overdesign, are used to demonstrate the robustness of the formulation presented in this work. Iteration histories are shown in Fig. 6. All starting points converged to the same optimal solution. The active constraints at optimality are those defined by Eqs. (6), (9), (11), (14), (17), and (19).

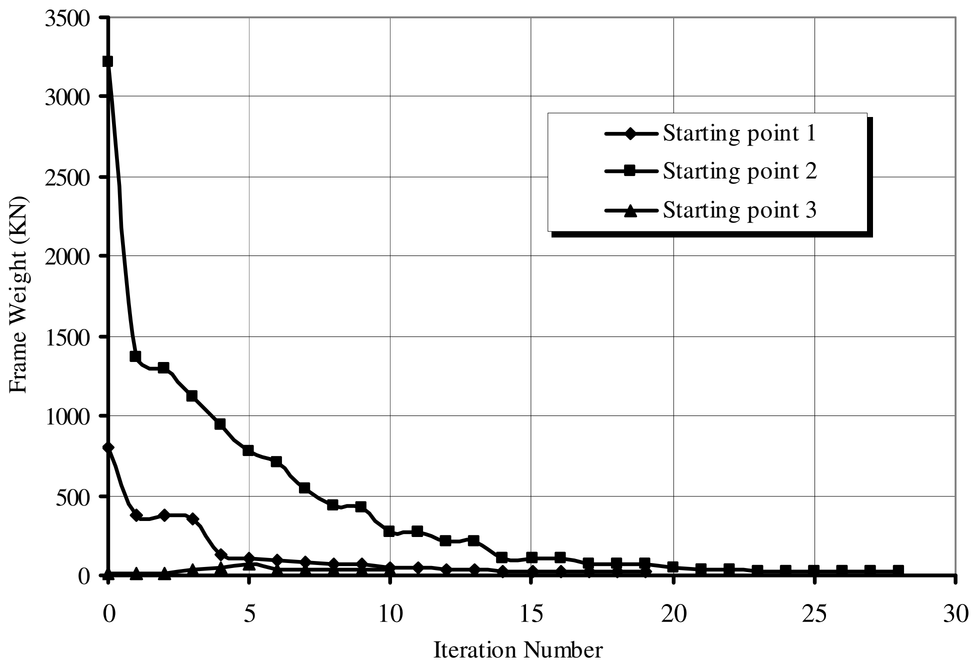

Fig.6. Iteration History with Different Starting Points for Example 2

4.3 Example 3

A third and final example is given to illustrate the versatility of the formulation presented in this research. The span L of the frame is *30000 mms*, the eave height H is *6000 mms* and the angle of the rafter ϕ is 5.7 degrees. Normal mild steel *(240/350)* is used for this example. The rafter and columns are prismatic members composed of built-up cross-sections. The initial dimensions of cross-sections, as well as the weight of the frame, are given in Table 4. Also, design histories are included. A minimum weight of *34.8* KN is achieved after *20* iterations. On the other hand, the total frame weight for the rounded design variables is *35.14* KN (see Table 4).

Table 4. Design History for Example 3

Design Variables	Iteration Number					
	0	5	10	15	Final	
					Calculated	Rounded
t_1(mm)	100	40.8	32.4	14.6	12.9	13.0
t_2(mm)	100	63.5	12.1	10.5	7.4	8.0
t_3(mm)	100	19.0	9.6	9.6	11.0	11.0
t_4(mm)	100	11.3	10.7	10.4	9.9	10.0
h_1(mm)	1000	903.0	817.5	708.8	498.8	499
h_2(mm)	1000	764.2	722.7	703.0	672.3	673
b_1(mm)	1000	868.6	667.9	314.7	265.4	266
b_2(mm)	1000	402.8	215.4	209.5	175.7	176
W_t (KN)	992.6	177.5	78.1	42.4	34.8	35.14

Another irrational over design is investigated. Values of $t_1 = t_2 = t_3 = t_4 = 200$ mms, $h_1 = h_2 = b_1 = b_2 = 2000$ mms, and a weight of *3970* KN, are used as the other starting point. A convergence to a minimum weight of *34.8* KN is reached after *25* iterations. Iteration histories for the two starting points are shown in Fig. 7. The active constraints at optimality are those defined by Eqs. (9), (11), (16), (17), and (23) in this work.

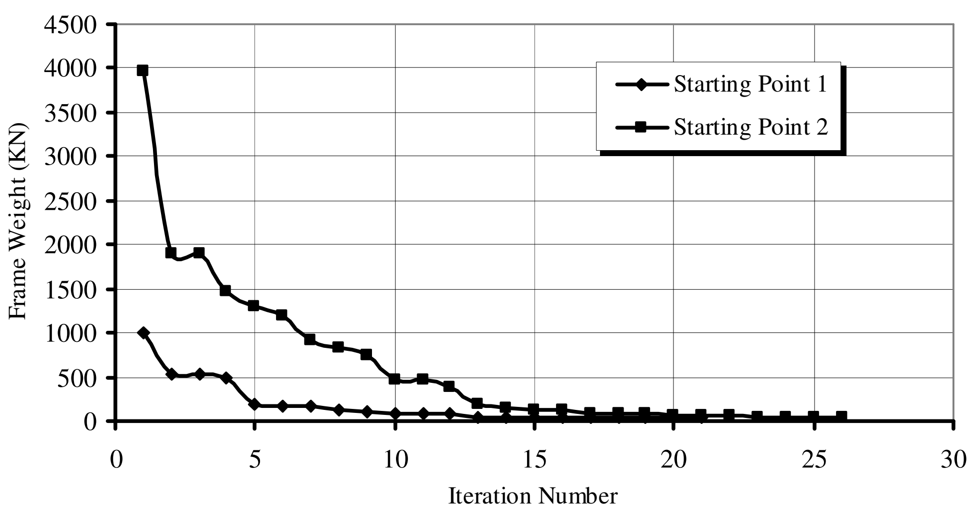

Fig.7. Iteration History for Example 3

Two other optimization methods; namely the Method of Feasible Directions (MFD) [10] and Davidon-Fletcher-Powell (DFP) variable metric method [14] are used to illustrate the efficiency and fastness of the MMFD. A convergence to the optimal weights of *40.799* and *67.27* KN are reached after *40* and *42* iterations for MFD and DFP, respectively. Figure 8 shows the iteration histories for the three optimization techniques.

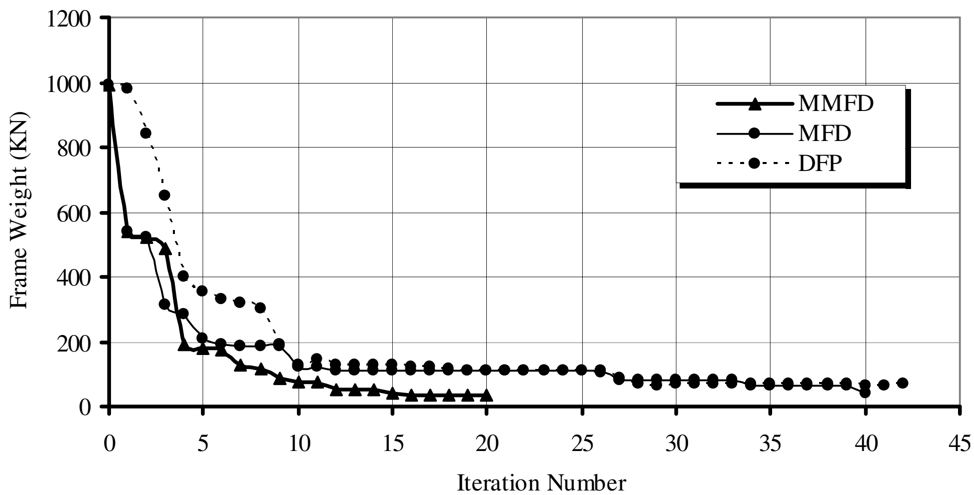

Fig.8. Iteration Histories for Different Optimization Methods

5. CONCLUSIONS

A robust algorithm is developed for optimum design of one-bay two-hinged portal/gable steel frames under gravity and lateral wind loads. The frames are composed of prismatic compact built-up sections. An explicit closed-form formulation for the straining actions is developed using the Virtual Work Method. The objective function is represented by the weight of the frame and the design variables by the cross-sectional dimensions. All ECP'01 constraints for stresses, stability, and deformations are incorporated. Different spans and different steel grades are included. The optimization technique adopted in this work is the MMFD. Three examples are presented to demonstrate the validity of the formulation. All results indicate the efficiency, practicality, and versatility of this approach over other conventional design approaches. Savings up to 50% in designs are achieved.

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NOTATIONS

The following symbols are used in this paper:

A_1	Cross-sectional area of column.
A_2	Cross-sectional area of rafter.
A'_1, A''_1	Code factor, ECP'01, Eq. 2.35.
b_1	Flange width of column section.
b_2	Flange width of rafter section.

C_b	Code coefficient, ECP'01, Eq. (2.28) & Table 2.2.
d_1	Total height of column section.
d_2	Total height of rafter section.
E	Modulus of elasticity of steel (210 GPa).
f	Difference of frame height at column and at mid-span (ridge).
F_y	Yield stress of steel.
F_c	Allowable stress in axial compression.
F_{cb}	Allowable stress in bending.
H	Column height.
H'	Out-of plane buckling length of column.
h_1	Height of web of column section.
h_2	Height of web of rafter section.
Ix_1	Moment of inertia of column section about X-axis.
Ix_2	Moment of inertia of rafter section about X-axis.
Iy_1	Moment of inertia of column section about Y-axis.
Iy_2	Moment of inertia of rafter section about Y-axis.
ix_1	Radius of gyration for column section abut X-axis.
ix_2	Radius of gyration for rafter section abut X-axis.
iy_1	Radius of gyration for column section about Y-axis.
iy_2	Radius of gyration for rafter section about Y-axis.
K_b	Buckling length factor.
L	Frame span.
q_b	Buckling shear stresses.
S	Rafter length.
S_w	Size of weld.
S_p	Spacing between purlins.
t_1	Thickness of flange of column section.
t_2	Thickness of web of column section.
t_3	Thickness of flange of rafter section.
t_4	Thickness of web of rafter section.
W	Gravity loads (D.L.+L.L.)
W'	Basic wind load.
W_L	Live load.
W_t	Total weight of frame.
α_1, α_2	Code factor, ECP'01, Table 2.1a.
γ_s	Specific weight of steel (7800 Kg/m^3).
ϕ	Slope angle of the rafter.

الحلول المثلى للإطارات المعدنية

يُقدم هذا البحث الحلول المثلى للإطارات المعدنية ذات البحر الواحد تحت تأثير الأحمال المتعددة. وتُستخدم طريقة الشغل الافتراضي في تحليل المنشأ وصياغة المسألة. وتشمل حالات التحميل الأحمال الرأسية وأحمال الرياح وفقاً للكود المصري للأحمال لسنة 1993، ويتم التصميم طبقاً لجميع إشتراطات الكود المصري للمنشآت المعدنية والكباري لسنة 2001 الخاصة بالإنبعاج والإجهادات والترخيم، ويمثل دالة الهدف في هذا العمل وزن المنشأ وتمثل متغيرات التصميم أبعاد القطاعات المعدنية سواء للأعمدة أو الكمرات كما تمثل إشتراطات الكود قيود التصميم، ويمكن إستخدام أنواع حديد عادي المقاومة أو عالي المقاومة في المسألة كما تستخدم طريقة الاتجاهات الممكنة لإيجاد التصميم الأمثل. وقد تم تطبيق الطريقة البحثية المقدمة في هذا العمل على ثلاثة أمثلة و مقارنة النتائج مع تصميمات أخرى وذلك لبيان مدى كفاءة الأسلوب المقدم وقدرته على تحقيق تصميمات أكثر اقتصادية بنسب وصلت إلى خمسين بالمائة.