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MINIMUM WEIGHT DESIGN OF ONE-BAY TWO-HINGED PORTAL STEEL FRAMES ACCORDING TO ECP'01

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ABSTRACT

This paper presents an 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 involve local buckling, lateral torsional buckling, shear buckling, combined stresses and deflection constraints, as provided by Egyptian Code of Practice for Steel Construction and Bridges (ECP'01 2001) 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.

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INTRODUCTION

Optimum design of structures has been an active area of research for more than four decades (Burns 2002). 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.

An optimality criterion approach is developed for optimum structural design of steel frames on a parallel machine (Adeli and Kamal 1993). However, design constraints are simplified to a great extent in their work. 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 by Pezeshk and Hjelmstab (Pezeshk 1994). Adeli and Soegiarso (Adeli 1997) optimize large space frames steel structures subjected to realistic AISC-specified stress, displacement, and buckling constraints on supercomputers. 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 by Saka and Kameshki (Saka 1998). Grigorian (1998) introduces an algorithm for preliminary minimum weight design of moment frames for lateral loading, maintaining the least possible drift for the given loading and geometry. A simple and new method for the optimum design of frames with stress, stiffness and stability constraints is presented by Manickarajah et al. (Manickarajah 2000).

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, are included. Gravity loads and wind loads are considered. The Modified Method of Feasible Directions optimization technique (Vanderplaats 1984) is used to solve the problem.

The rest of this research 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.

PROBLEM STATEMENT

Figure 1 outlines a plane one-bay two-hinged portal 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.

Vertical dead and live loads and lateral wind loads, as per ECP'93 (1993) are considered in this work. Using the Method of Virtual Work (Parnes 2001), closed form solutions for the normal forces, shearing forces, and bending moments at different frame locations are derived.

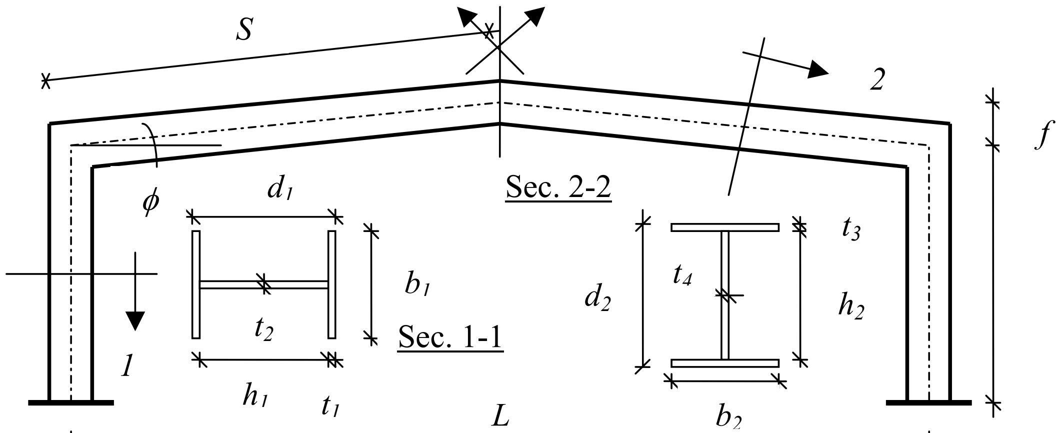

FIG.1 Layout of Portal Frame and Design Parameters

The internal forces values are given in terms of the design variables shown in Fig.1, parameters given in the list of symbols at the end of the paper, and the following formulas:

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

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

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

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 that, the live load values provided by ECP'93 should be transformed from the horizontal projection to the inclined span length of the rafter before using the formulas 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 column height.

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:

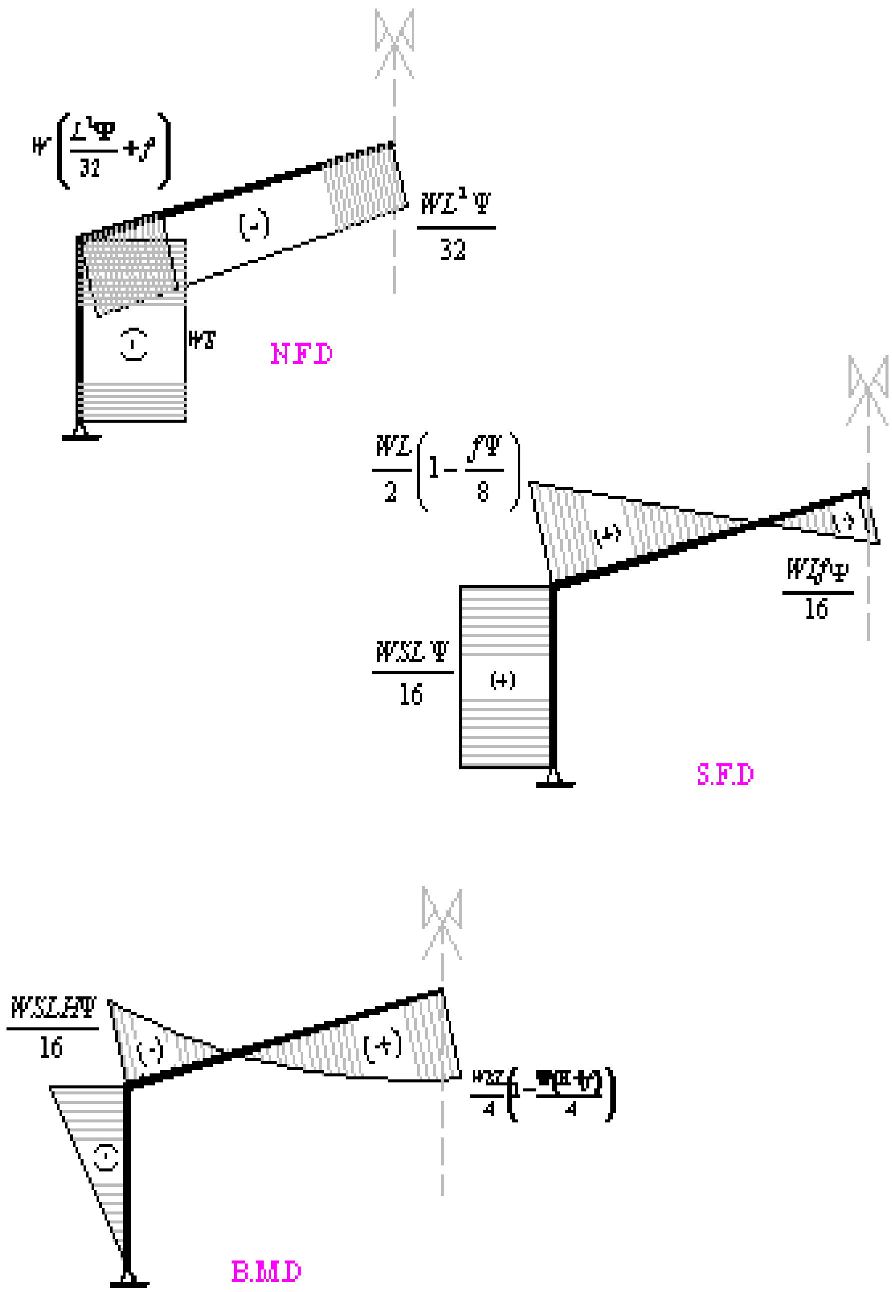

Fig. (2) Straining Actions for Gravity Loads

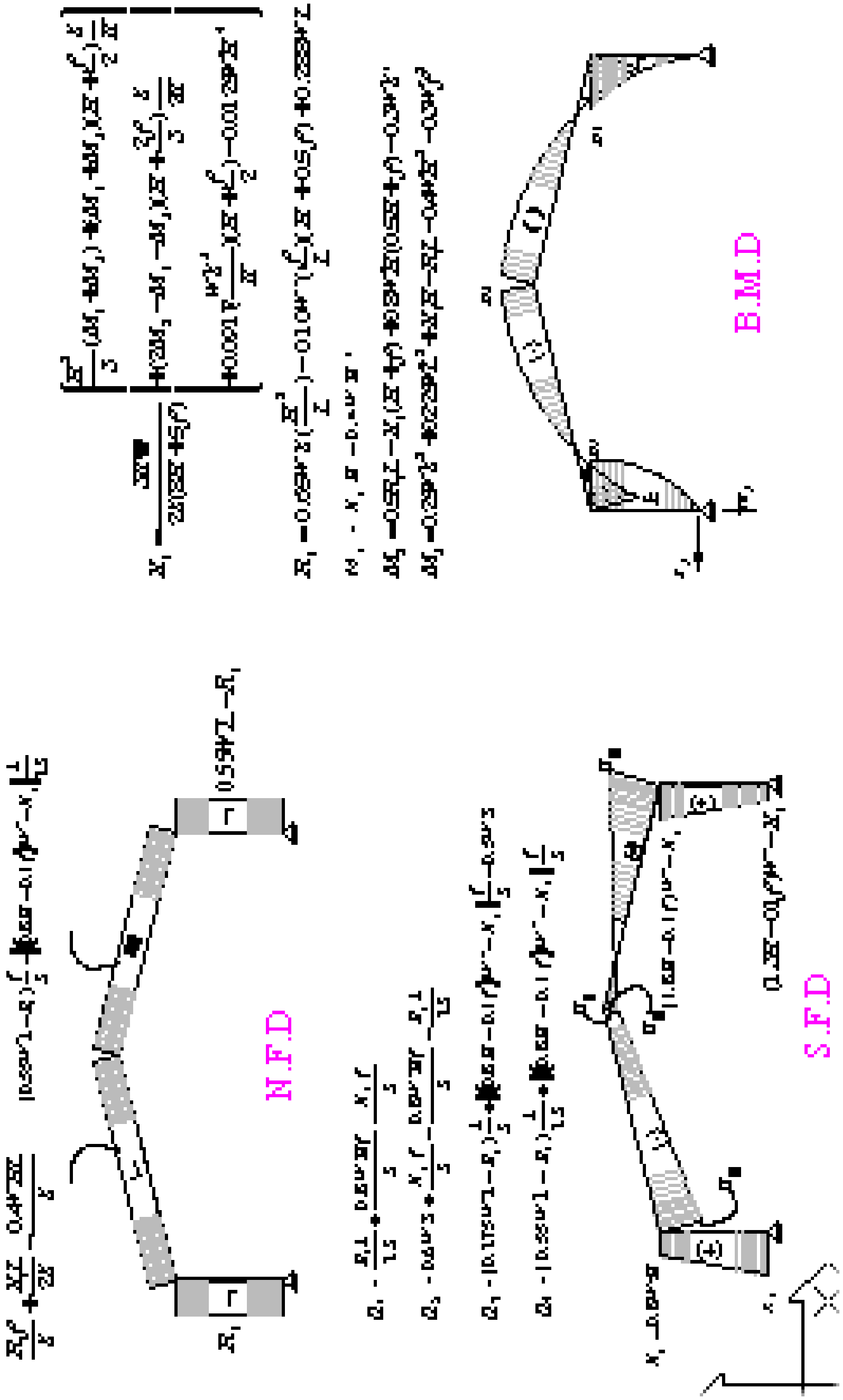

FIG. (3). Straining Actions for Wind Loads

Minimize:

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

subject to:

$$\frac{(h_1 - 2S_w)}{t_2} - \frac{699/\sqrt{F_y}}{13\alpha_1 - 1} \le 0$$
$$or \ \frac{(h_1 - 2S_w)}{t_2} - \frac{58}{\sqrt{F_y}} \le 0 \tag{5}$$

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

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

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

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

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

$$\frac{WS + 0.55W'L - R_1}{(2b_1 t_1 + h_1 t_2)F_c} + \frac{WSLH\psi + 16M_3}{32F_{cb} Ix_1} d_1 A'_1 - 1.20 \le 0 \tag{11}$$

$$\frac{WSL\psi}{16h_1 t_2} - q_b \le 0 \tag{12}$$

$$\frac{(h_2 - 2S_w)}{t_4} - \frac{699/\sqrt{F_y}}{13\alpha_2 - 1} \le 0$$
$$or \ \frac{(h_2 - 2S_w)}{t_4} - \frac{58}{\sqrt{F_y}} \le 0 \tag{13}$$

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

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

$$\frac{S_p(h_2 + 2t_3)}{b_2 t_3} - \frac{1380 C_b}{F_y} \leq 0 \tag{16}$$

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

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

$$\frac{W(\frac{L^2\psi}{32} + f)}{F_c(h_2 t_4 + 2b_2 t_3)} + \frac{WSLH\psi}{32 F_{cb} Ix_2} d_2 A''_1 - 1 \leq 0 \tag{19}$$

$$\frac{WL^2\psi}{32(h_2 t_4 + 2b_2 t_3)} \times \frac{1}{F_c} + \frac{WSL\left(1 - \frac{\psi(H + f)}{4}\right)}{8 F_{cb} Ix_2} d_2 A''_1 - 1 \leq 0 \tag{20}$$

$$\frac{WL(1 - \frac{f\psi}{8})}{2h_2 t_4} - q_b \leq 0 \tag{21}$$

$$\frac{WLf\psi}{16 h_2 t_4} - q_b \leq 0 \tag{22}$$

$$\left\{\begin{aligned} &\frac{W_L SL^2\psi^2 H^3\eta}{192 EIx_1} + \frac{W_L S^2 LH\psi}{16 EIx_2}\left[\frac{\psi\eta L}{8}(3H + 2f) - \frac{L}{12}\right] \\ &- \frac{W_L S^2 L}{4 EIx_2}\left[1 - \frac{\psi(H + f)}{4}\right]\left[\frac{\psi\eta L}{8}(3H + 2f) - \frac{L}{6}\right] \\ &- \frac{W_L S^2 L}{12 EIx_2}\left[\frac{\psi\eta L}{8}(H + \frac{f}{2}) - \frac{L}{8}\right] - \frac{L}{300} \leq 0 \end{aligned}\right\} \tag{23}$$

$$\left\{\begin{array}{l} \dfrac{W_L S L^2 \psi^2 H^3 \eta}{192 E I x_1} + \dfrac{W_L S^2 L H \psi}{16 E I x_2}\left[\dfrac{\psi \eta L}{8}(3H+2f)-\dfrac{L}{12}\right] \\ -\dfrac{W_L S^2 L}{4 E I x_2}\left[1-\dfrac{\psi (H+f)}{4}\right]\left[\dfrac{\psi \eta L}{8}(3H+2f)-\dfrac{L}{6}\right] \\ -\dfrac{W_L S^2 L}{12 E I x_2}\left[\dfrac{\psi \eta L}{8}(H+\dfrac{f}{2})-\dfrac{L}{8}\right] \end{array}\right\} \dfrac{2f}{\sqrt{4f^2+L^2}} - \dfrac{H}{150} \le 0 \qquad (24)$$

$$0 \le t_1, t_2, t_3, t_4, h_1, h_2, b_1, b_2 \le 10000 \qquad (25)$$

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 knee bracing is utilized to connect the compression flange in the lower side of the rafter with 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.

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 2.6, Page 14
	shear stress	12	Equations 2.7-2.10, Pages 14 and 15
	combined normal	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 2.6, Page 14
	Shear stress	21,22	Equations 2.7-2.10, Pages 14 and 15
	combined normal	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 Modified Method of Feasible Directions optimization technique. The following section outlines the cases for which the formulation is experimented.

APPLICATIONS

Three examples are presented in this work. Spans ranging from *9* ms to *30* ms are considered. Normal mild steel and high tensile steel cases are investigated. Comparisons are done for rolled sections and built-up sections. Each of the three case studies is summarized hereafter.

Example 1

The first example is a frame constructed in Ras Ghareb, Gulf of Suez. The frame span L is *9* ms; the eave height H is *4.5* ms; the rafter slope in *1:20* (refer to Fig.1). The steel grade used in this frame is high strength steel *(36/52)*. The cross-sectional dimensions of the built-up sections used for this frame are listed in Table 2. The dimensions of the tapered sections used in the actual frame are also indicated. The weight of the constructed frame is 0.37 tons.

Using a starting point of $t_1 = t_2 = t_3 = t_4 = 10$ cms, $h_1 = h_2 = b_1 = b_2 = 100$ cms and a starting weight of *42.42* tons, an optimal solution of 0.33 ton 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.

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(cm)	10	3.81	1.89	1.38	1.13	0.68	0.71	0.69	0.50
t_2(cm)	10	6.85	1.86	0.74	0.49	0.18	0.19	0.50	0.50
t_3(cm)	10	1.35	1.42	1.16	1.04	0.73	0.69	0.69	0.50
t_4(cm)	10	6.76	1.50	0.99	0.80	0.40	0.36	0.50	0.50
h_1(cm)	100	90.34	78.67	41.05	27.12	9.89	10.65	10.40	20/30
h_2(cm)	100	92.13	82.88	55.02	44.10	21.92	19.86	19.66	20/30
b_1(cm)	100	53.03	33.96	24.67	20.29	12.82	13.25	13.26	15
b_2(cm)	100	30.18	25.99	21.29	19.14	13.78	13.17	13.18	15
W_t (ton)	42.42	12.21	3.34	1.43	0.95	0.34	0.33	0.36	0.37

Figure 4 outlines the iteration history for the weight of the frame. The optimization technique used for this example is the Modified Method of Feasible Directions. The active constraints at optimality are those defined by Eqs. (6), (8), (9), (11), (14), (16), (17), and (23) in this work.

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 *0.36* ton is achieved after *21* iterations (Final-2). Details of this solution are also given in Table 2. The iteration history for this case is also shown in Fig. 4. To this end, it is worthwhile noting that the sections used for the constructed frame do not satisfy ECP'01 design constraints.

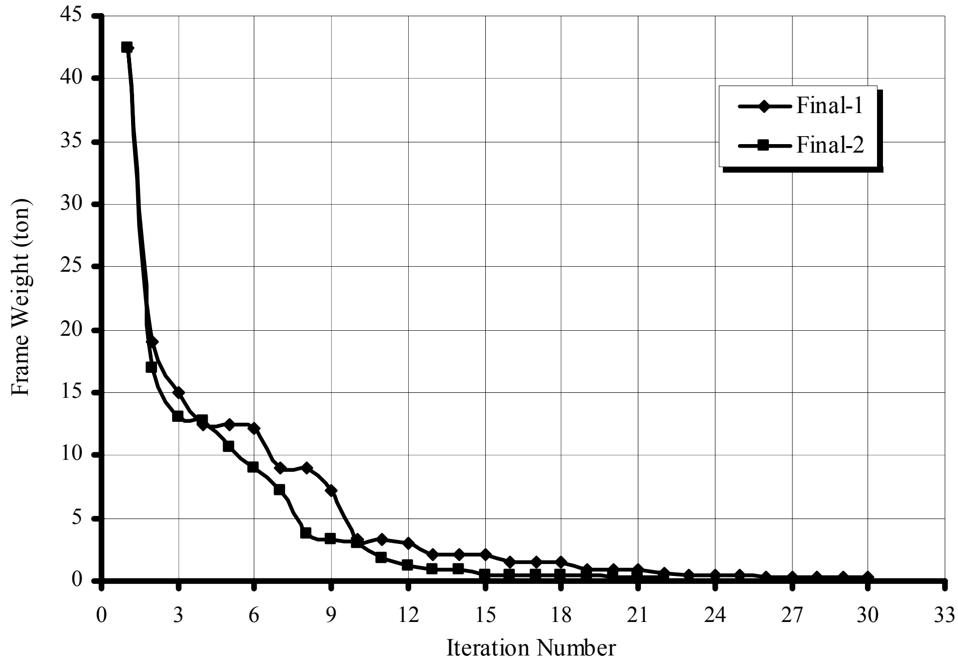

FIG. (4) Iteration History for Example 1

Example 2

The second example is given in (Machaly 2001). The span of the frame *L* is *22* ms, the height of the eave *H* is *6* ms and the angle ϕ of the rafter is 5.7°. Steel grade is normal mild steel (*24/37)*. The cross-sections are prismatic 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					Machaly (2001)
	0	5	10	15	Final	
t_1(cm)	10	1.70	1.17	1.14	0.95	2.00
t_2(cm)	10	5.46	2.85	1.13	0.62	1.20
t_3(cm)	10	1.70	1.00	0.75	0.78	2.00
t_4(cm)	10	1.58	1.10	0.88	0.78	1.20
h_1(cm)	100	88.82	84.70	76.28	42.22	24.00
h_2(cm)	100	78.46	74.32	59.36	53.02	24.00
b_1(cm)	100	37.49	29.92	27.63	22.92	28.00
b_2(cm)	100	40.36	24.42	18.79	19.47	28.00
W_t (ton)	80.33	10.30	5.19	2.79	1.91	3.84

Using the same unrealistic starting point of the previous example, a minimum weight of *1.91* ton 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. The optimization method adopted in this case is the Modified Method of Feasible Directions.

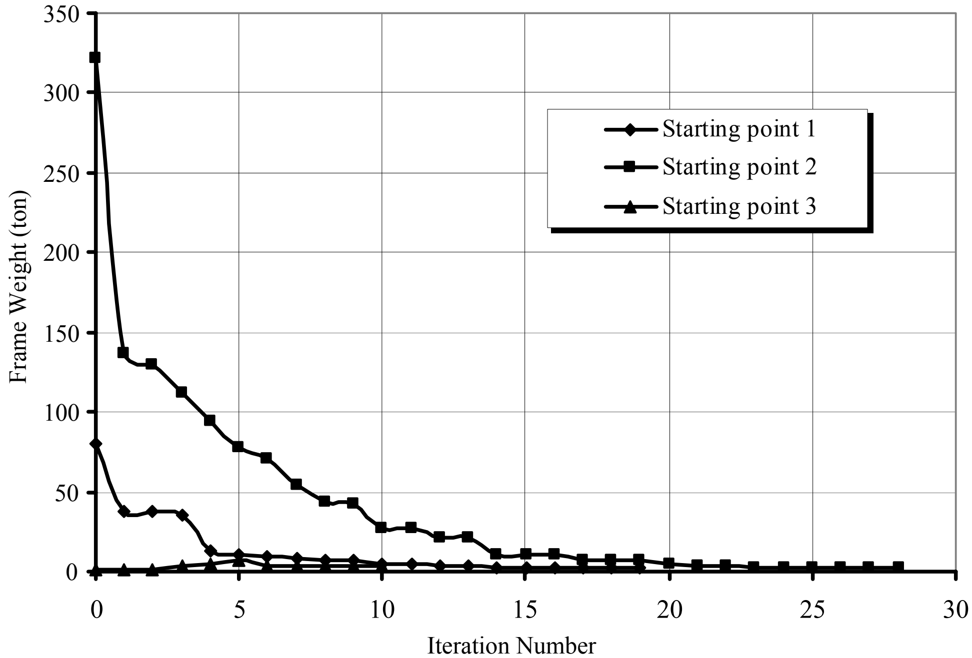

FIG. (5) Iteration History with Different Starting Points for Example 2

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. 5. 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).

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 *30 ms*, the eave height H is *6 ms* and the angle of the rafter ϕ is 5.7 degrees. Normal mild steel *(24/37)* 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 *3.48* ton is achieved after *20* iterations.

Another irrational over design is investigated. Values of $t_1 = t_2 = t_3 = t_4 = 20$ cms, $h_1 = h_2 = b_1 = b_2 = 200$ cms, and a weight of *397* tons, are used as the other starting point. A convergence to a minimum weight of *3.48 ton* is reached after *25* iterations. Iteration histories for the two starting points are shown in Fig. 6. The optimization technique used in this example is the Modified Method of Feasible Directions and the active constraints at optimality are those defined by Eqs. (9), (11), (16), (17), and (23) in this work.

TABLE (4) Design History for Example 3

Design Variables	Iteration Number				
	0	5	10	15	Final
t_1(cm)	10	4.08	3.24	1.46	1.29
t_2(cm)	10	6.35	1.21	1.05	0.74
t_3(cm)	10	1.90	0.96	0.96	1.10
t_4(cm)	10	1.13	1.07	1.04	0.99
h_1(cm)	100	90.30	81.75	70.88	49.88
h_2(cm)	100	76.42	72.27	70.30	67.23
b_1(cm)	100	86.86	66.79	31.47	26.54
b_2(cm)	100	40.28	21.54	20.95	17.57
W_t (ton)	99.26	17.75	7.81	4.24	3.48

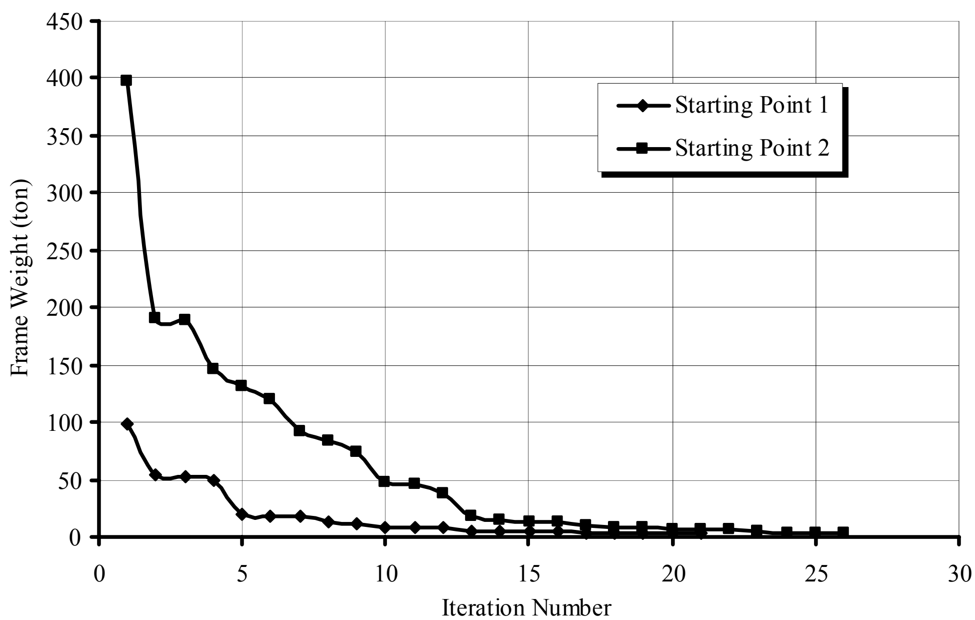

FIG. (6) Iteration History for Example 3

CONCLUSIONS

A robust algorithm is developed for optimum design of one-bay two-hinged portal 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 Modified Method of Feasible Directions. Three examples to demonstrate the validity of the formulation are presented. All results indicate the efficiency, practicality, and versatility of this approach over other conventional design approaches. Savings up to 50% in designs are achieved.

REFERENCES

ECP'01 (2001) "Egyptian Code of Practice for Steel Construction and Bridges", Ministerial Decree No. 279-2001.

Burns, S.A., *Editor*, (2002) "Recent Advances in Optimal Structural Design", Committee Report, American Society of Civil Engineer, ISBN 0-7844-0636-7.

Adeli, H., and Kamal, O., (1993) "Parallel Processing in Structural Engineering", Elsevier Applied Science, London, ISBN 1-85861-003-6.

Pezeshk, S., and Hjelmstab, K. D. (1994) "Optimal Design of Steel Frames to Enhance Overall Stability", Engineering Journal, Second Quarter, pp. 48-56.

Adeli, H., and Soegiarso, R. (1997) "Optimization of Large Space Frame Steel Structures", Engineering Journal, Second Quarter, pp. 54-60.

Saka, M. P. and Kameshki, E. S. (1998) "Optimum Design of Unbraced Rigid Frames", Computers & Structures 68, pp. 433-442.

Grigorian, M. and Grigorian, C. (1998) "Preliminary Minimum Weight Design of Moment Frames for Lateral Loading", Engineering Journal, Fourth Quarter, pp. 129 – 136.

Manickarajah, D., Xie, Y. M., and Steven, G. P. (2000) "Optimum Design of Frames with Multiple Constraints using an Evolutionary Method", Computers & Structures 74, pp. 731 – 741.

Vanderplaats, G. N. (1984) "An Efficient Feasible Directions Algorithm for Design Synthesis", AIAA Journal, Vol. 22, No. 11, pp. 1633-1640.

ECP'93 (1993) "Egyptian Code for Calculating Loads and Forces in Structural and Building Works", Ministerial Decree No. 45-1993.

Parnes, R., (2001) "Solid Mechanics in Engineering", John Wiley & Sons, New York, ISBN 0-471-49300-7.

Machaly, E. B. (2001) "Behavior, Analysis, and Design of Structural Steel Elements", Vol. 1, 4th Edition, Chapter 6, pp. 427-437.

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 (2100 t/cm^2).
f	Difference of frame height at column and at mid-span.
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.
ix_1	Radius of gyration for column section abut X-axis.
iy_1	Radius of gyration for column section about Y-axis.
ix_2	Radius of gyration for rafter section about X-axis.
iy_2	Radius of gyration for rafter section about Y-axis.
K_b	Buckling length factor.
L	Frame span.
q_b	Actual 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 (7.85 t/m^3).

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الحلول المثلى للآطارات المعدنية وفقا للكود المصرى للمنشآت المعدنية

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ملخص

يُقدم هذا البحث الحلول المثلى للإطارات المعدنية ذات البحر الواحد تحت تأثير الأحمال المتعددة. تُستخدم 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.

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