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Application of Cold-formed Steel Sections to Primary Members of Small Steel Buildings

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APPLICATION OF COLD-FORMED STEEL SECTIONS TO PRIMARY MEMBERS OF SMALL STEEL BUILDINGS

by

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SUMMARY

In the greater part of Mexico there is no provision in the building codes for snow load; consequently, a low live load is generally used, and light cold formed steel members can be employed for primary members, reducing the cost per square meter of construction with these structures. A comparison is given on the weight and cost of several types of structures in actual use in Mexico against those proposed. An analysis of the ratio of labor and steel cost trend is presented.

A simplification of the design procedures is proposed, adapted to hand design with a desk calculator, using available sections in Mexico. Possibility of use of variable sections is viewed, and results of this investigation given.

A brief study of fabrication procedure is presented with the idea of marketing through dealers to prospective customers. Erection procedures are studied with a "do-it-yourself" thought in mind.

Conclusions and future trends are derived from ideas presented.

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INTRODUCTION

There exists in Mexico a need for economical steel structures that are easy to design, fabricate and erect. If a customer needs a simple shed or small structure, he has to contract a steel fabricator who will custom design and build to his needs, with the consequent delays in time and high costs. If a simple economical mass produced structure could be furnished for self erection with interchangeable parts to make it more versatile, this approach could solve the problem.

This article was written to make available to every engineer or fabricator the basic knowledge to be able to design and build such structures without any previous experience in designing cold-formed steel section structures, using for this purpose simple general formulae for designing cold-formed steel structures and a general criteria for utilizing existing sections in a economical way. Cold-formed steel structures are ideal to solve the problem and the normal complicated procedure of design can be simplified with little loss of economy, so that they can be designed by an engineer having a basic knowledge of design, without the use of costly equipment such as computers.

The formulae used in this article are adapted to the metric system. In Appendix number 1, a procedure for translating English system formulae to the metric system is given.

STEEL STRUCTURE DESIGN TRENDS IN MEXICO

In order to reduce the cost of steel structures and to be on a competitive basis in this era of increasing labor and steel costs, the obvious solution of steel fabricators is to reduce the weight per sq. m. of the steel structure. To achieve that, there are two possible solutions:

(a) To reduce sections by means of a detailed design of conventional structures. This procedure or solution, while reducing the cost by reducing the weight, also increases the cost by the necessity of utilizing more engineering time.

(b) Using some other type of material that can have, with a lower weight, the necessary mechanical requirements. This can be cold-formed steel sections. The changeability of types of sections, plus the high ratio of section properties, such as moment of inertia, section modulus, etc. against weight per meter, make it ideal for the solution of this problem. The only drawback of this solution is the time consuming procedure of design, which can be avoided by the simplification of design methods, which will produce a slightly heavier section with little loss of economy. Another factor to be considered in this solution is the reduction of labor cost for fabrication, as the manpower or labor required is very much lower than that used for conventional structures, and these can be mass-produced with very simple equipment. The reduction on weight also simplifies the erection procedure, making it possible to self-erect by the customer.

LIVE LOADS

The minimum live load in Mexico is 25 kg/m^2(5.12 psf), which is very much below the normal live load in the U.S. According to the Metal Building Manufacturers Association (MBMA), the minimum roof live load should be 59 Kg/m^2(12 psf). This reduction is justified because of the lack of snow in most of the country.

Designing a conventional rigid frame with such a low live load presents some erection problems. A typical rigid frame structure will be so light that twisting and buckling of the members will be the major problem during the erection process. So, to prevent this, normally a heavier and more rigid structure will be built, with the consequent loss of economy.

Using cold-formed steel members, the structure can be designed for the low live loads, obtaining a substantial savings in weight, and at the same time having a rigid structure which will present no problems during the erection process.

The loads adopted for this example are the following:

Live Load	25 Kg/m^2	5.12 psf
Dead Load	15 Kg/m^2	3.07 psf
Wind Load	70 Kg/m^2	14.34 psf
Seismic Load	0.12 times the total load	

Loading conditions:

(a) Live Load plus Dead Load

(b) Dead Load plus 1/2 Live Load (one side)

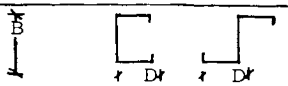
(c) Dead Load plus Wind Load

(d) Dead Load plus Seismic Load

AVAILABLE SECTIONS, TYPES OF COLD-FORMED ELEMENTS

To achieve the maximum economy and to insure the availability of delivery, only standard sections available in the local market were used, and only in special cases was a different section employed.

The available cold-formed sections in the local market are:



D		B	
mm.	in.	mm.	in.
102	4	51	2
127	5	51	2
152	6	64	2 1/2
178	7	70	2 3/4
203	8	76	3
229	9	83	3 1/4
254	10	89	3 1/2
305	12	89	3 1/2

All these sections in the following gages:

#10	3.42 mm
#12	2.66 mm
#14	1.90 mm
#16	1.52 mm

The type of steel used is A-375, with these characteristics:

Minimum yield point or yield strength = 3515 Kg/cm^2 (50 KSI)

Minimum ultimate strength = 4920 Kg/cm^2 (70 KSI)

Minimum elongation in 51 mm (2") 22%

Basic allowable tension $fb = 2100\ Kg/cm^2$

Modulus of elasticity $E = 2{,}100{,}000\ Kg/cm^2$

Expansion coefficient per C°: $\Delta = 0.000012$

ADVANTAGES OF USING COLD-FORMED STEEL SECTIONS AS PRIMARY ELEMENTS

Another factor to be considered in the design of primary elements is that using conventional hot rolled sections a long list of sections is necessary in order to achieve economy. This limits the flexibility of the design and increases weight of the structure, reducing the economy.

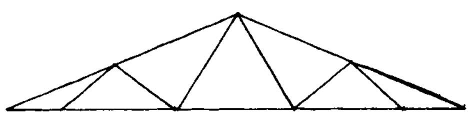
With the low loads specified, cold-formed steel sections can be used for primary elements, obtaining the advantages of flexibility of design, low weight, with consequent economy, availability of sections in the market, simplified design procedure, and ease of erection.

The use of cold-formed sections in outdoor structures has a disadvantage considering the actual code, which limits the thickness of material to be used in unprotected sections. The chemical composition of high strength steels used in cold rolled sections reduces the corrosion factor, and using protective painting this problem can be solved.

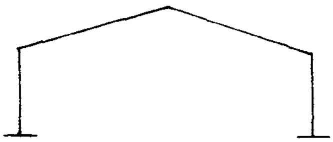
Using cold-formed C sections, water down-spouts can be hidden inside the columns with better unclustered construction.

TYPES OF STRUCTURES IN USE IN MEXICO

The most common and widely used type of roof steel structure in Mexico is the truss type, with members made of hot rolled angles.



Secondly, rigid frames with Welded I sections formed of three plates.



Concrete is also widely used, precast, cast in place, and prestressed sections. All these require skilled labor and equipment to erect.

ECONOMIC ASPECT

Undoubtedly, the economic aspect is one of the most important in every engineering project, and in this particular problem was one of the major considerations to affect the type of structure and design procedure.

COST PER SQUARE METER, WEIGHT, COMPARISON

The average weight of typical structures for roofs per square meter is:

Conventional Steel Roof Structure	20 Kg/m^2 (4.10 psf)
Reinforced Concrete Roof Structure	375 Kg/m^2 (76.81 psf)
Cold-formed Steel Structures	12-15 Kg/m^2 (2.46-3.07 psf)

These savings in weight produce substantial savings in the following items:

(a) Cost of Structure

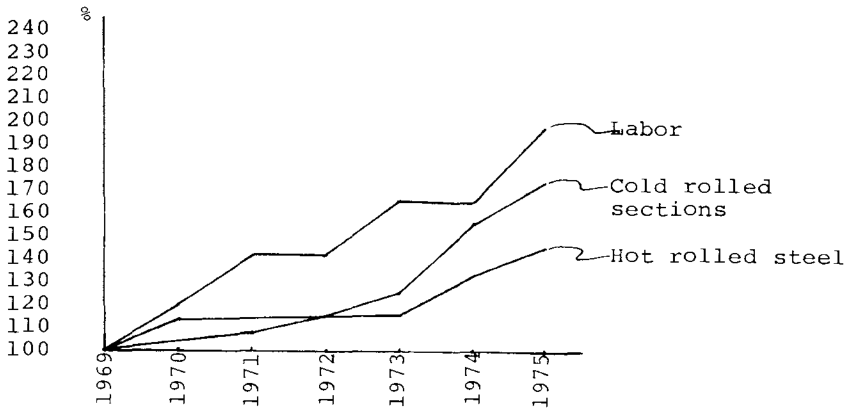
(b) Transportation

(c) Erection

(d) Foundations

RATIO OF STEEL TO LABOR COSTS, VARIATIONS

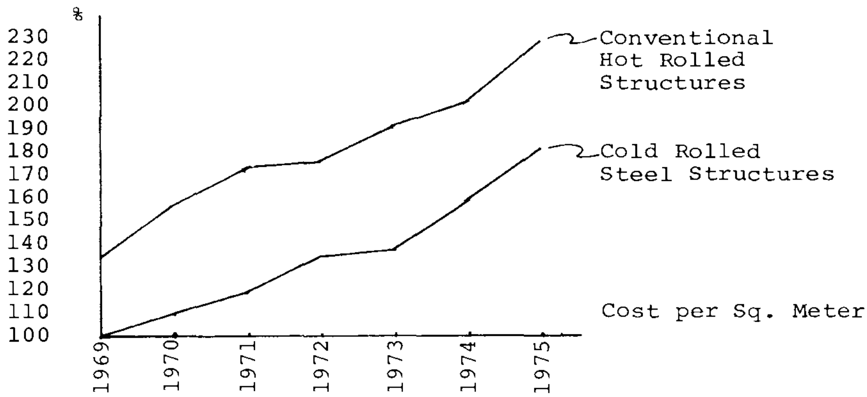
The trend of raising costs in the past years in Mexico is shown in the following graph:



Considering the proportion of labor cost to steel, cost in 1969 was:

Labor 55%

Steel 45%



DESIGN PROCEDURE

The simplified design procedure proposed consists of using for design the basic formulae for bending shear and axial load, or a combination of both.

(a) Bending

(b) Axial Load

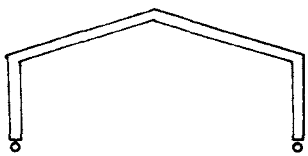
(c) Combined Bending and Axial Load

(d) Check for Shear

TYPES OF STRUCTURE

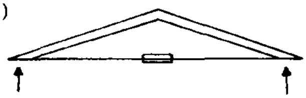
The types of structure proposed to be designed with cold-formed steel sections are:

(a)



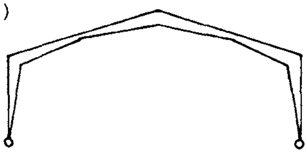
Rigid frame with pin column supports, constant section

(b)



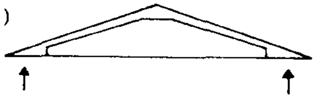
Open truss with tension rod, simply supported with constant section

(c)



Rigid frame with Pin Column Support, variable section

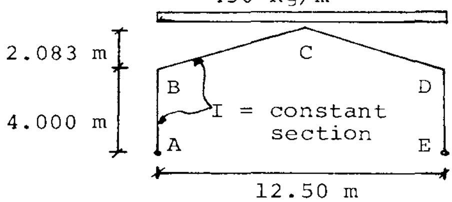
(d)

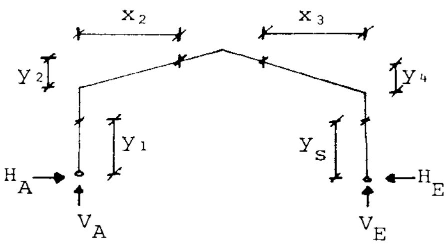


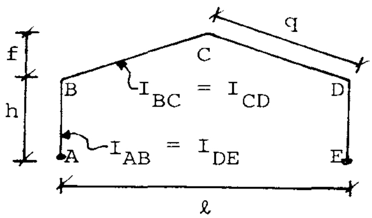
Open truss with tension rod, simply supported with variable section

The procedure of design is simplified by the use of formulae such as those given by V. Leontovich[1] Example #1 - Find maximum moments and reactions of the given rigid frame with hinged supports and constant section, subject to loads specified:

ω = dead + live load
= 450 Kg/m







$$q = \sqrt{\left(\frac{\ell}{2}\right)^2 + f^2} = 6.588 \text{ m}$$

General Frame Constants:

$$\phi = \frac{I_{AB}}{I_{BC}} \times \frac{q}{h} = \frac{1}{1} \times \frac{6.588}{4.00} = 1.65$$

$$\psi = \frac{f}{h} = \frac{2.083}{4.00} = 0.521$$

$$A = 4(3 + 3\psi + \psi^2 + 1/\phi)$$

$$A = 4(3 + (3 \times .521) + .521^2 + (1/1.65)) = 21.764$$

$$B = 2(2 + 2\psi) = 2(3 + (2 \times .521)) = 8.083$$

$$C = 2(2 + \psi + (2/\phi))$$

$$C = 2(2 + .521 + (2/1.65)) = 9.470$$

$$V_A = V_E = \omega \ell/2 = W/2$$

$$V_A = V_E = (450 \times 12.50)/2 = 2812.50 \text{ Kg}$$

$$H_A = H_D = \frac{\omega \ell^2}{A8h} (2 + B + \psi)$$

$$H_A = H_D = \frac{450 \times 12.5^2}{21.764 \times 8 \times 4} (2 + 8.083 + .52)$$

$$H_A = H_D = 1070.598 \text{ Kg}$$

$$M_B = M_D = -hH = -1070.598 \times 4.00$$

$$M_B = M_D = -4282.392 \text{ Kg} \cdot \text{m}$$

$$M_C = \frac{\omega \ell^2}{8} - Hh(1 + \psi)$$

$$M_C = \frac{450 \times 12.5^2}{8} - 1070.598 \times 4(1 + .521)$$

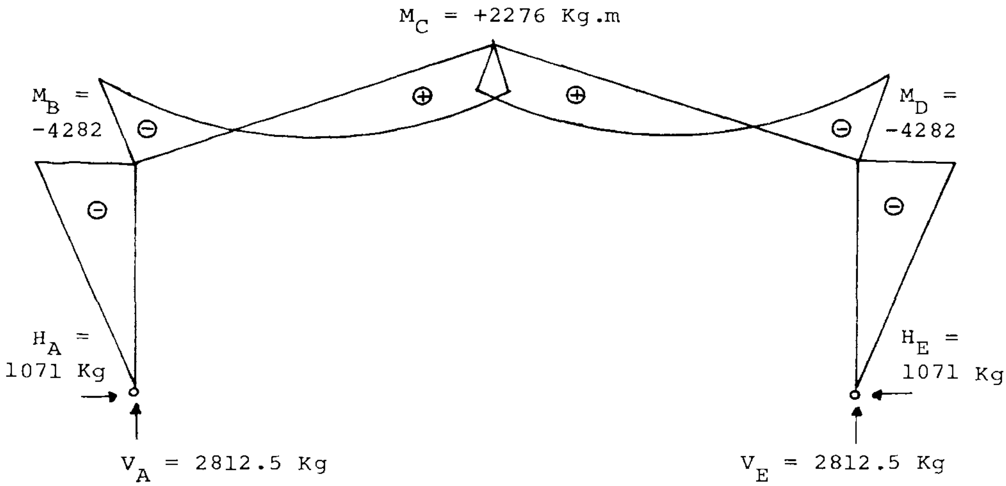
$$M_C = 2276.258 \text{ Kg} \cdot \text{m}$$

$$M_{y_1} = M_B(y_1/h) = \frac{-4282.392}{4.00} y_1 = 1070.598 y_1$$

$$M_{x_2} = (M_B + \frac{\omega \ell x_2}{4})(1 - \frac{2x_2}{\ell}) + M_C(\frac{2x_2}{\ell})$$

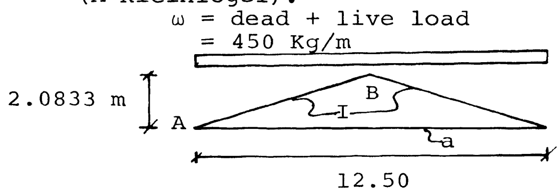
$$M_{x_2} = (-4282.392 + \frac{450 \times 12.5x_2}{4})(1 - \frac{2x_2}{12.5}) + 2276.258 \times \frac{2x_2}{12.5}$$

$$M_{x_2} = 2455.634x_2 - 225x_2^2 - 4282.392$$



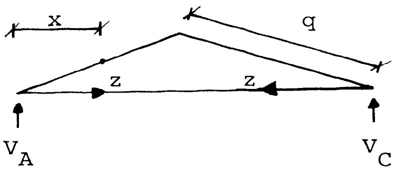
This calculation can be very much simplified by the use of a programmable pocket calculator, such as HP-55 or HP-65 (See Appendix 2).

Example #2. Symmetrical triangular constant section, rigid frame with tie rod. Simply supported. Uniform Load (A Kleinlogel).[2]



a = cross section area of the tie rod (cm^2)

I = Moment of inertia (cm^4)



$$q = \sqrt{\left(\frac{\ell}{2}\right)^2 + h^2}$$

$$= 6.588 \text{ m}$$

Tie rod and member should have the same modulus of elasticity E (Kg/cm^2).

$$V_A = V_C = \frac{\omega\ell}{2} = \frac{450 \times 12.50}{2} = 2812.5 \text{ Kg}$$

Suppose a trial value for I and a. considering a box section of two 25.4 cm x 14 Ga "C" sections, and a tie rod of #7. (7/8") $I = 1635.42 \text{ cm}^4$ $a = 3.87 \text{ cm}^2$

$$N_z = \frac{I \times \ell \times 3 \times 10^{-4}}{h^2 \times a \times q} + 2$$

$$N_z = \frac{1635.42 \times 12.5 \times 3 \times 10^{-4}}{2.083^2 \times 3.87 \times 6.588} + 2 = 2.055$$

$$Z = \frac{5\omega\ell^2}{16hN_z} = \frac{5 \times 450 \times 12.5^2}{16 \times 2.083 \times 2.055} = 5131.248 \text{ Kg}$$

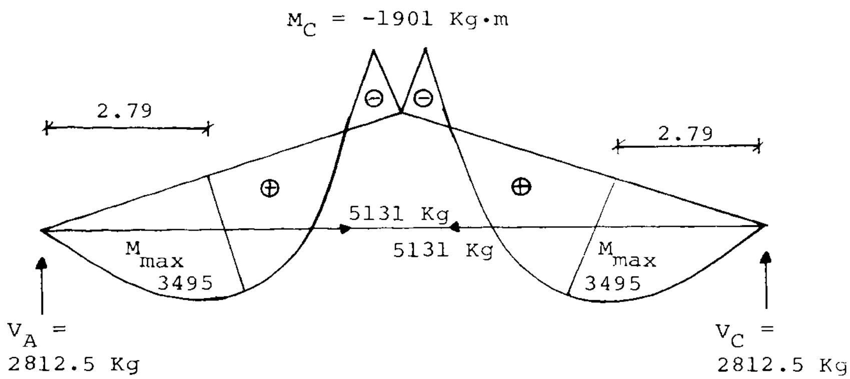
$$\frac{Z}{a} = \frac{5131.248}{3.87} = 1325.9 \text{ Kg/cm}^2 \quad \text{(tension stress on tie rod)}$$

$$M_B = \frac{\omega\ell^2}{8}\left(1 - \frac{5}{2N_z}\right) = \frac{450 \times 12.5^2}{8}\left(1 - \frac{5}{2 \times 2.055}\right)$$
$$= -1901.038 \text{ Kg·m}$$

$$\chi = \frac{\ell}{4} + \frac{M_B}{\omega\ell} = \frac{12.5}{4} + \frac{-1901.038}{450 \times 12.5} = 2.787 \text{ m}$$

χ = distance from A to maximum positive moment

$$M_{x_{max}} = \chi\left(V + \frac{2M_B}{\ell}\right) - \omega x^2 = 3495.410 \text{ Kg·m}$$



PURLINS

Purlins - simplified design procedure.

Purlins are designed as simply supported beams with lateral bracing. The necessary steps for designing purlins are:

Given data: ω = total load Kg/m on purlin

L = span in m

S = maximum allowable deflection

DESIGN PROCEDURE

(a) Pure bending. Calculate maximum bending moment, and obtain a preliminary $S_x = \frac{M}{2100}$. Choose a convenient section with the required S_x. This can be done by previously calculating a tabulation of available sections with their proper S_x, reduced and full, according to the criteria:

1. if $\frac{w}{t} \leq \left(\frac{w}{t}\right)_{lim}$ where $\left(\frac{w}{t}\right)_{lim} = \frac{1434}{\sqrt{f}} = 31.29$

   use full section

2. if $\frac{w}{t} > \left(\frac{w}{t}\right)_{lim}$ reduce w to b

Following formulae:

$$b = 46.28t - \frac{468.64t^2}{w}$$

which, reduced to the available gages:

for gage #10 $b_{10} = 158.29 - \frac{5481.40}{W}$

for gage #12 $b_{12} = 123.12 - \frac{3315.91}{W}$

for gage #14 $b_{14} = 87.94 - \frac{1691.79}{W}$

for gage #16 $b_{16} = 70.35 - \frac{1082.75}{W}$

with this reduced W to b, calculate S_x reduced and design with this.

Check deflection, maximum allowable $= \frac{\ell}{180}$

$$\delta = \frac{5}{384}\,\frac{\omega \ell^4}{EI_x}$$

ℓ = span in cm
ω = total uniform load in Kg/m
$E = 2{,}100{,}000$ Kg/cm^2
I_x = Moment of inertia, x axis

Check Shear

Actual shear stress $= \frac{V}{\text{Area of Web}} = \frac{V}{hxt}$

Maximum allowable shear stress F_v

(a) for $\frac{h}{t} \leq \frac{3186}{\sqrt{fy}}$ or $\frac{h}{t} \leq 53.74$ use $F_v = .4F_y$

$F_v = 1260$ Kg/cm^2

(b) if $\frac{3186}{\sqrt{fy}} < \frac{h}{t} < \frac{4586}{\sqrt{fy}}$ or $53.74 < \frac{h}{t} < 77.35$

use $F_v = \frac{1275\sqrt{fy}}{h/t} = \frac{75591}{h/t}$

(c) for $\frac{h}{t} \geq \frac{4586}{\sqrt{fy}}$ or $\frac{h}{t} \geq 77.35$

use $F_v = \frac{5{,}877{,}662}{(h/t)^2}$

Calculate distances of maximum lateral supports (sag-rods) for z sections:

$$F_b = \frac{2}{3}F_y - \frac{F_y^{\,2}}{2.7\pi^2 EC_b}\left(\frac{L^2 S_{x_c}}{dI_{y_c}}\right)$$

using: $F_y = 3515$ Kg/cm²

$E = 2{,}100{,}000$ Kg/cm²

$C_b = 1$ (conservative)

$$L = 12.27\sqrt{\frac{d^2 I_y}{S_x}}$$

I_y = moment of inertia, y axis

S_x = section modulus, x axis

d = depth of purlin in cm

Example:

$\ell = 9.82$ m

$\omega = 52$ Kg/m

$$\delta_{max} = \frac{982}{180} = 5.46 \text{ cm}$$

$$M = \frac{\omega\ell^2}{8} = \frac{9.82^2 \times 52}{8} = 626.81 \text{ Kg·m}$$

$$S_{x_{req}} = \frac{626.81 \times 100}{2109} = 29.72 \text{ cm}^3$$

/

Section chosen: z section 22.9 cm - 14 Ga

$S_x = 53.19 \text{ cm}^3$

$\frac{w}{t} = 32.68 \qquad \therefore \frac{w}{t} > \left(\frac{w}{t}\right)_{\text{lim}}$ so b has to be reduced

$b_{14} = 87.94 - \frac{1691.79}{w} = 59.84 \text{ mm}$

Calculating S_x reduced = 51,81 cm^3

Check deflection:

$$\delta = \frac{5}{384}\frac{\omega \ell^4}{EI} = \frac{5 \times .52 \times 982^2}{384 \times 2.10 \times 10^6 \times 607.91} = 4.93 \text{ cm}$$

acceptable

Check shear:

$$V = \frac{9.82 \times 52}{2} = 255.32 \text{ Kg} \qquad F_v = \frac{255.32}{.19 \times 20.62}$$

$$= 65.17 \text{ Kg/cm}^2$$

Allowable F_v

$$\frac{h}{t} = \frac{20.62}{.19} = 108.53$$

$$F_v = \frac{5{,}877{,}662}{(h/t)^2} = \frac{5{,}877{,}662}{(108.53)^2} = 499.04 \text{ Kg/cm}^2$$

F_v allowable > F_v real OK

Calculate distance between lateral supports

$$L = 12.27\sqrt{\frac{d^2 I_y}{S_x}} = 12.27\sqrt{\frac{(20.62)^2 \times 99.49}{51.81}} = 351 \text{ cm}$$

Locate at $\frac{\ell}{3} = \frac{982}{3} = 327$ cm

See Appendix 3.

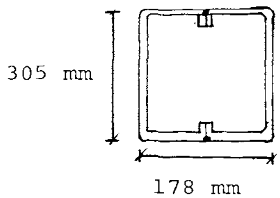
DESIGN OF BEAM-COLUMNS

Combined axial compression and bending stresses:

Data:	M (Moment)	Example:	M = 4282 Kg.m
	P (Axial Load)		P = 2812.5 Kg
	K		K = 2.0
	ℓ (Length)		ℓ = 4.00

(a) Design for compression (axial load) only:

1. Suppose a section; box section formed by two "C" 30.5 cms x 12 Ga and obtain:



$r = 7.35$

$S_{x_{efec}} = 234.18\ cm^2$

$Q = 0.66$

$A = 26.48\ cm^2$

$F_y = 3515\ Kg/cm^2$

$E = 2,100,000\ Kg/cm^2$

2. Calculate

$$\frac{K\ell}{r} = \frac{2 \times 400}{7.35} = 108.84$$

3. Calculate

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2 \times \pi^2 \times 2.10 \times 10^6}{3515}} = 108.60$$

4. Calculate

$$\frac{C_c}{\sqrt{Q}} = \frac{108.60}{\sqrt{.66}} = 133.68$$

5. if $\frac{K\ell}{r} < \frac{C_c}{\sqrt{Q}}$ $\qquad \frac{K\ell}{r} = 108 < 133.68 = \frac{C_c}{\sqrt{Q}}$

Calculate

$$F_{a_1} = 1834.83Q - .0787\left(\frac{QK\ell}{r}\right)^2$$

$$= 1834.83 \times .66 - .0787(.66 \times 108.84)^2$$

$$= 804.88 \text{ Kg/cm}^2$$

6. if $\frac{K\ell}{r} > \frac{C_c}{\sqrt{Q}}$

Calculate

$$F_{a_1} = \frac{10,680,089}{\left(\frac{K\ell}{r}\right)^2}$$

7. Calculate

$fa = \frac{P}{A}$ $\qquad fa = \frac{2812.5}{26.48} = 106.21 \text{ Kg/cm}^2$

check $\qquad$ Check OK for compression only

$F_{a_1} \geq fa$ $\qquad F_{a_1} = 804.88 > 106.21 = fa$

(b) Design for compression and bending:

8. Calculate

$\frac{fa}{F_{a_1}}$ $\qquad \frac{fa}{F_{a_1}} = \frac{106.21}{804.88} = 0.132$

9. Calculate

$fb = \frac{M \times 100}{S_x}$ $\qquad fb = \frac{4282 \times 100}{234.18} = 1828.51 \text{ Kg/cm}^2$

$F_b = 0.6F_y$ $\qquad F_b = .6 \times 3515 = 2109 \text{ Kg/cm}^2$

10. if $\frac{fa}{F_{a_1}} > 0.15$ go to 11 $\quad \frac{fa}{F_{a_1}} = 0.132 < 0.15$ go to 13

if $\frac{fa}{F_{a_1}} < 0.15$ go to 13

11. Calculate

$$F^1ex = \frac{12\pi^2 E}{23\left[\frac{K\ell}{r}\right]^2} = \frac{10,813,654}{23\left[\frac{K\ell}{r}\right]^2}$$

12. Check ($C_m = 0.85$)

$$\frac{fa}{F_{a_1}} + \frac{C_m fb}{\left(1 + \frac{fa}{F^1ex} F_b\right)} \leq 1$$

13. Check

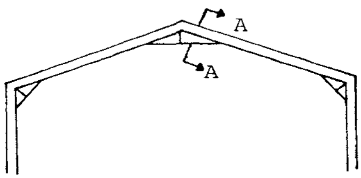
$$\frac{fa}{F_{a_1}} + \frac{fb}{F_b} \leq 1 \qquad 0.132 + \frac{1828.51}{2109} = 0.999 \leq 1$$

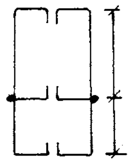
Section chosen is OK.

STRUCTURES DESIGNED WITH VARIABLE SECTION MEMBERS

In order to increase the economy of the structures, the possibility of using variable sections is now in study. These sections complicate the problem of simplified design, as the shape and the moment of inertia of each of the members have a pronounced influence on the magnitude of redundants. The basic approach to the solution of this problem has been, following the procedure proposed by V. Leontovich(1), to establish three basic parameters which would give the elastic properties of the members. These parameters would be only dependent upon the shape of the members, and to develop load constants dependent upon the shape of the member and manner of loading. This analysis would be a trial and error procedure without the use of a computer. Much work is still required to reach a satisfactory method.

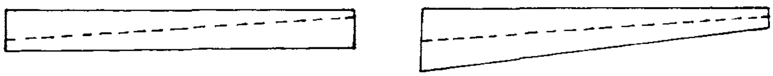
The fabrication of variable section members also produces some complications. One approach is to reinforce the critical or maximum moment points. This can be accomplished by using a constant section member, and welding an additional element to increase the mechanical properties:





Section A•A

Another approach is to shear diagonally a constant section, welding afterwards in an inverted position:



Still another approach would be to form the members with a variable section, using a press-brake operation. This is a more simple solution, but limits the speed of fabrication.

EASE OF FABRICATION AND ERECTION

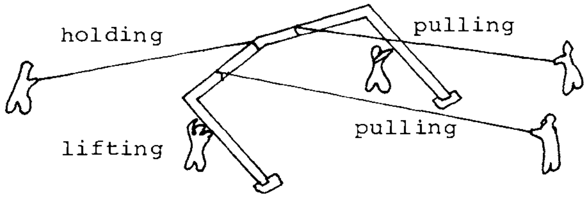
As has been previously noted in this paper, very light structures present some problems in erection. These problems consist basically of twisting during the erection process. C-type cold rolled sections are sufficiently rigid to prevent this. Fabrication is very much simplified, as the basic problem consists in welding base plates and connection plates.

FABRICATION PROCEDURE

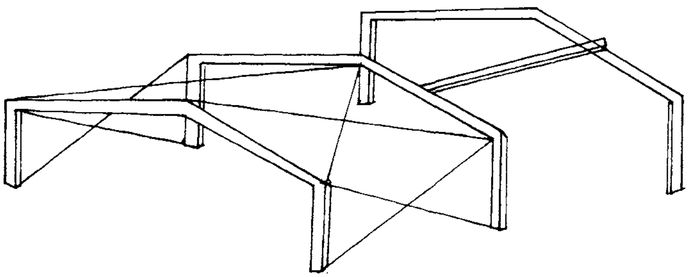
Structures designed with cold-rolled sections have the advantage of being able to be produced in a continuous fashion. "In-line" production reduces cost and time of fabrication. Waste is reduced to a minimum, as sections can be ordered to specific lengths. One problem that should be avoided in box sections formed by two C sections is the twisting, due to residual stress in welding. With a simple jig, this can be avoided.

DO-IT-YOURSELF APPROACH

Due to the light weight and small spans of the proposed structures, they can be very easily erected by the customer without any special equipment or previous experience. The frame is assembled on the ground, and only unskilled labor will be required to erect the frame.



After two frames have been erected, they will be wind-braced to plum line with the proper roof purlins attached.



MARKETING

This approach to self-erection by the customer allows the structures to be sold in hardware stores or any similar distribution media. The structures are furnished with an erection manual and the necessary wrenches and bolts to erect the structure.

Selling through distributors will allow non-skilled people to sell and promote the product.

Publicity on the ease of erection, flexibility, and interchangeability of elements, plus the low cost of transportation to the job site, will be a major selling point. Due to the small size of the members, these have been erected in unaccessible areas, with transportation done by very primitive means, such as mules.

CONCLUSIONS

Mexico is a virgin country for development of uses of cold-formed sections. These can be used not only in buildings, but in many other fields as well. The potential of this material is tremendous in a developing country, and more widespread use will most certainly be achieved in the near future.

Simplifying design methods and making this information available to the layman will certainly increase a prompt and wider use.

APPENDIX #1

CONVERSION OF FORMULAE FROM ONE SYSTEM TO ANOTHER

In the practical day-to-day work of an Engineer, a problem commonly arises when the formula for the solution of a certain problem is given using a different system from that commonly in use. A classical example is when using a reference book, the system used is the English system, but the available data and desired results are in the Metric system.

There are two ways to solve this problem: one is to change all the input data to the English system, using the formula and changing the result back to the metric system; unfortunately, this procedure increases the possibility of error, and forces us to change input and output data every time we use the formula.

The second way is to "translate" the formula, changing or including new constants which will allow us to use the input data in the desired system, and obtain the result also in the same system. This method is more easily understood with a practical example:

Change the following formula, given in the English system, to the Metric system:

$$\frac{W}{t} = \frac{171}{\sqrt{f}}$$

W in inches
t in inches
f in Kips per square inch

First Step: Investigate the units of the constant:

$$171 = \frac{W\sqrt{f}}{t} = \frac{in\ \sqrt{Kips/in^2}}{in} = \frac{\sqrt{Kips}}{in}$$

$\therefore$ 171 is given in $\frac{Kips}{in}$

Second Step: Find the value of the constant X in the metric system

$$X = \frac{171\sqrt{Kips}}{in}$$

1 Kip = 453.592 Kg
1 in = 2.54 cm

$$\therefore \quad X = \frac{171\sqrt{453.592}}{2.54} = 1433.82$$

Third Step: Substitute this value (X) in the original formula

$$\frac{W}{t} = \frac{1433.82}{\sqrt{f}}$$

W in cm
t in cm
f in Kg/cm^2

HP-55 Program Form

Appendix # 2.

Title RIGID FRAMES WITH UNIFORM VERTICAL LOAD Page 1 of 2

Press BST in RUN mode, switch to PRGM mode. Then key in the program.

DISPLAY		KEY ENTRY	X	Y	Z	T	COMMENTS
LINE	CODE						
00.							
01.	34	RCL					
02.	01	1					
03.	34	RCL					
04.	02	2					
05.	71	×					
06.	02	2					
07.	81	÷					
08.	84	R/S					V
09.	04	4					
10.	81	÷					
11.	34	RCL					
12.	02	2					
13.	71	×					
14.	33	STO					
15.	08	8					
16.	34	RCL					
17.	06	6					
18.	81	÷					
19.	34	RCL					
20.	03	3					
21.	81	÷					
22.	34	RCL					
23.	05	5					
24.	34	RCL					
25.	07	7					
26.	61	+					
27.	02	2					
28.	61	+					
29.	71	×					H
30.	84	R/S					
31.	42	CHS					
32.	34	RCL					
33.	03	3					
34.	71	×					− MB
35.	84	R/S					
36.	34	RCL					
37.	05	5					
38.	01	1					
39.	61	+					
40.	71	×					
41.	34	RCL					
42.	08	8					
43.	61	+					MC
44.	−00	GTO−00					
45.							
46.							
47.							
48.							
49.							

REGISTERS	
R_0	f
R_1	w
R_2	l
R_3	n
R_4	C
R_5	ψ
R_6	A
R_7	B
R_8	$\frac{wl}{8}$
R_9	
$R_{.0}$	
$R_{.1}$	
$R_{.2}$	
$R_{.3}$	
$R_{.4}$	
$R_{.5}$	
$R_{.6}$	
$R_{.7}$	
$R_{.8}$	
$R_{.9}$	

9320-2940

HP-55 User Instructions

Title RIGID FRAMES WITH UNIFORM VERTICAL LOAD Page 2 of 2

Programmer Alejandro L. de la Barra

STEP	INSTRUCTIONS	INPUT DATA/UNITS	KEYS				OUTPUT DATA/UNITS
1	Enter program						
2	Store coefficients	f	STO	0			
		h	STO	3			
		l	STO	2	2	÷	l/2
			RCL	0	g	→P	q
			x⇄y	CLX			
		I_{AB}	×				
		I_{BC}	÷	RCL	3	÷	
			1/x	RCL	0	RCL	
			3	÷	STO	5	ψ
			↑	g	x^2	x⇄y	
			3	×	+	+	
			3	+	4	×	A
			STO	6	RCL	5	
			2	×	3	+	
			2	×	STO	7	B
			R↓	R↓	+	RCL	
			5	+	3	+	
			2	×	STO	4	C
		w	STO	1			
3	Calculate V		BST	R/S			V
4	Calculate H		R/S				H
5	Calculate M_B		R/S				$M_B = M_D$
6	Calculate M_C		R/S				M_C

HP-55 Program Form

Appendix # 3

Title DESIGN OF Z COLD - ROLLED PURLINS Page 1 of 2

Press BST in RUN mode, switch to PRGM mode. Then key in the program.

DISPLAY LINE	DISPLAY CODE	KEY ENTRY	X	Y	Z	T	COMMENTS
00.							
01.	34	RCL					
02.	05	5					
03.	32	g					
04.	42	x^2					
05.	34	RCL					
06.	04	4					
07.	71	×					
08.	34	RCL					
09.	01	1					
10.	81	÷					
11.	84	R/S					
12.	34	RCL					
13.	00	0					
14.	71	×					
15.	34	RCL					
16.	05	5					
17.	71	×					
18.	84	R/S					
19.	34	RCL					
20.	05	5					
21.	71	×					
22.	34	RCL					
23.	06	6					
24.	81	÷					
25.	34	RCL					
26.	03	3					
27.	81	÷					
28.	84	R/S					
29.	13	1/X					
30.	34	RCL					
31.	05	5					
32.	71	×					
33.	84	R/S					
34.	34	RCL					
35.	08	8					
36.	32	g					
37.	42	x^2					
38.	34	RCL					
39.	07	7					
40.	71	×					
41.	34	RCL					
42.	09	9					
43.	81	÷					
44.	31	f					
45.	42	x					
46.	34	RCL					
47.	02	2					
48.	71	×					
49.	-00	GTO-00					

REGISTERS	
R_0	1.8829
R_1	168.72
R_2	12.27
R_3	1.80
R_4	W (Kg/m)
R_5	l (m)
R_6	Ix (cm^4)
R_7	Iy (cm^4)
R_8	d (cm)
R_9	Sx (cm^3)
$R_{.0}$	
$R_{.1}$	
$R_{.2}$	
$R_{.3}$	
$R_{.4}$	
$R_{.5}$	
$R_{.6}$	
$R_{.7}$	
$R_{.8}$	
$R_{.9}$	

9320-2940

HP-55 User Instructions

Title DESIGN OF Z COLD-ROLLED PURLINS Page 2 of 2

Programmer A. L. de la Barra

STEP	INSTRUCTIONS	INPUT DATA/UNITS	KEYS				OUTPUT DATA/UNITS
1	Enter Program						
2	Store Coefficients	1.8829	STO	0			
		168.72	STO	1			
		12.27	STO	2			
		1.80	STO	3			
		w(Kg/m)	STO	4			
		l (m)	STO	5			
3	Calculate Sx_{req}		BST	R/S			Sx_{req}
4	Calculate Ix_{req}		R/S				Ix_{req}
5	Store Proposed data	$Ix(cm^4)$	STO	6			
		$Iy(cm^4)$	STO	7			
		d(cm)	STO	8			
		$Sx(cm^3)$	STO	9			
6	Re-start		BST				
7	Calculate Sx_{req} (repeat)		R/S				Sx_{req}
8	Calculate Ix_{req} (repeat)		R/S				Ix_{req}
9	Calculate deflection		R/S				δ
10	Calculate $^x/100$		R/S				$^x/100$ *
11	Calculate L		R/S				L

Formulas used: $E = 2{,}100{,}000\ Kg/cm^2$

$$M = \frac{wl^2}{8} \qquad Sx_{req} = \frac{M}{Fb} = \frac{M}{2109} = \frac{wl^2}{168.72}$$

$$Ix_{req} = \frac{5}{384}\,\frac{wl^4}{E\,\delta} = .01116\ wl^4 = Sx_{req} \times 1.8829 \times l \qquad \left(\delta\ max = \frac{l}{180}\right)$$

$$\delta = \frac{5}{384}\,\frac{wl^4}{E\ Ix} = \frac{Ix_{req} \times l}{1.8 \times Ix_{req}}$$

$$x = \frac{l \times 100}{\delta}$$

* Amount given in step 10, should be x 100

$$L = 12.27\sqrt{\frac{d^2\ Iy}{Sx}}$$ (distance between lateral supports)

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