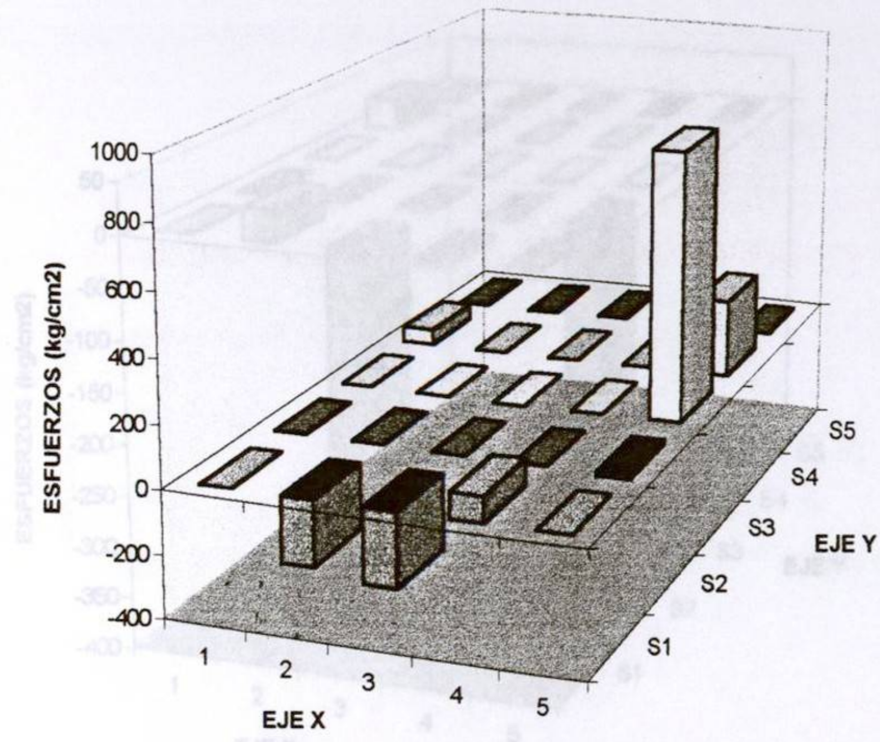
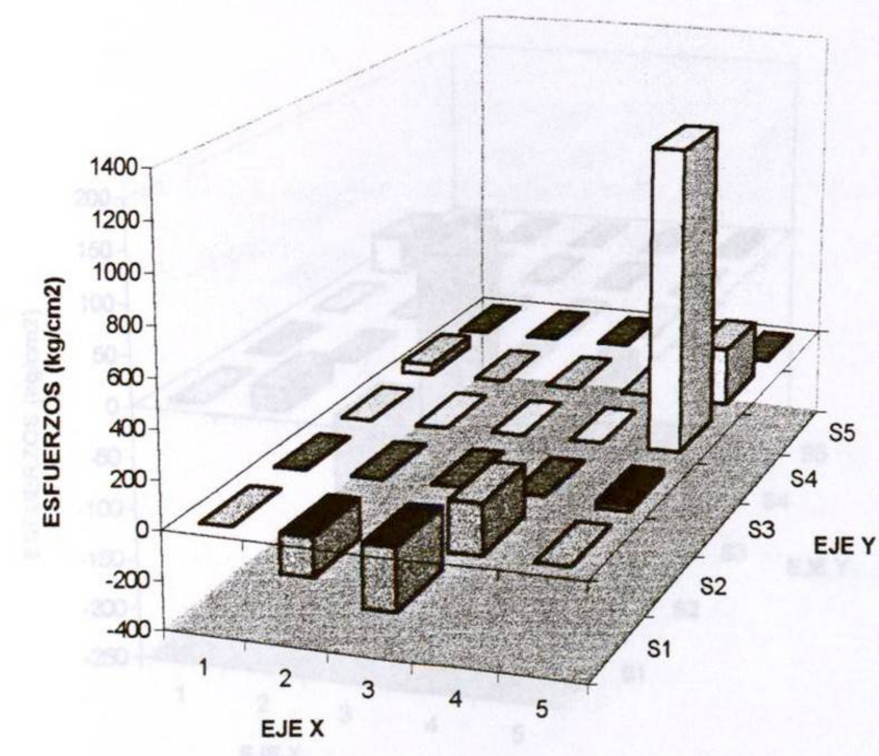
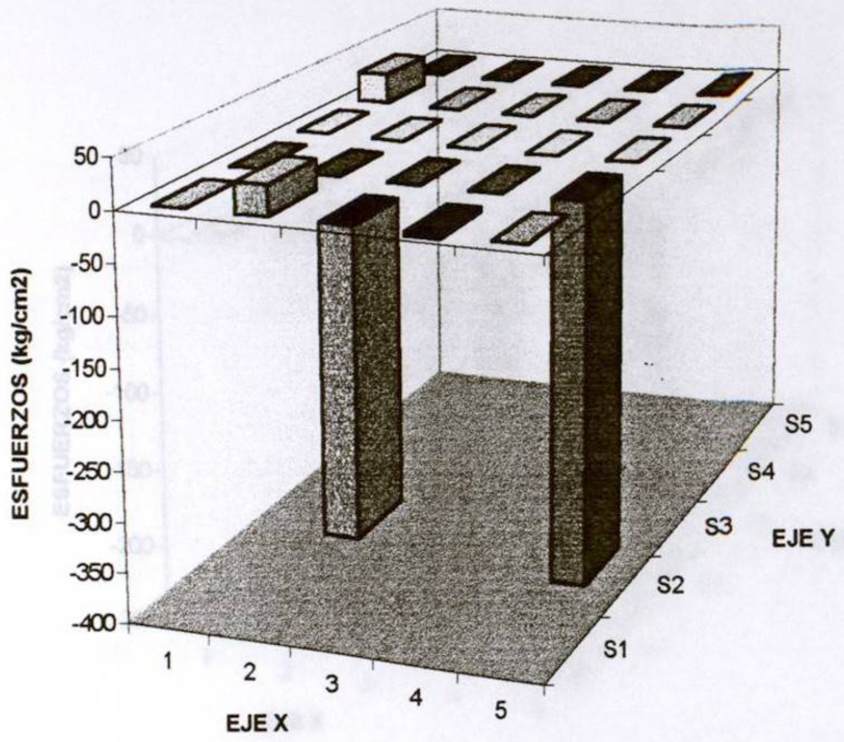
GRAFICA DE ESFUERZOS (kg/cm2) para Px=40 ton. , Mx=8.442 ton.-m , My=1.914 ton.-m



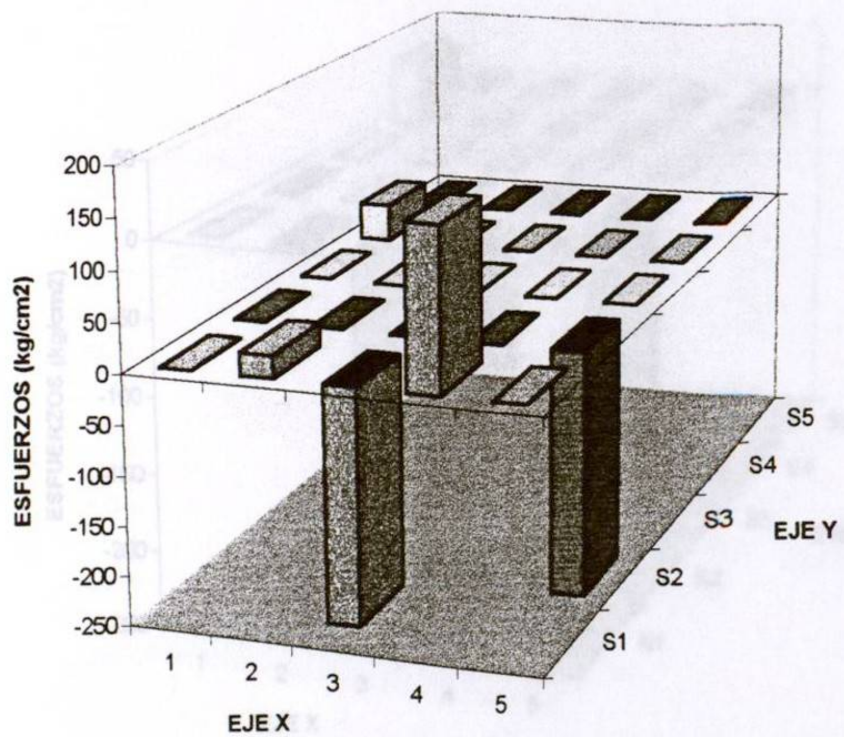
GRAFICA DE ESFUERZOS (kg/cm2) para Px=50 ton. , Mx=7.911 ton.-m , My=1.702 ton.-m



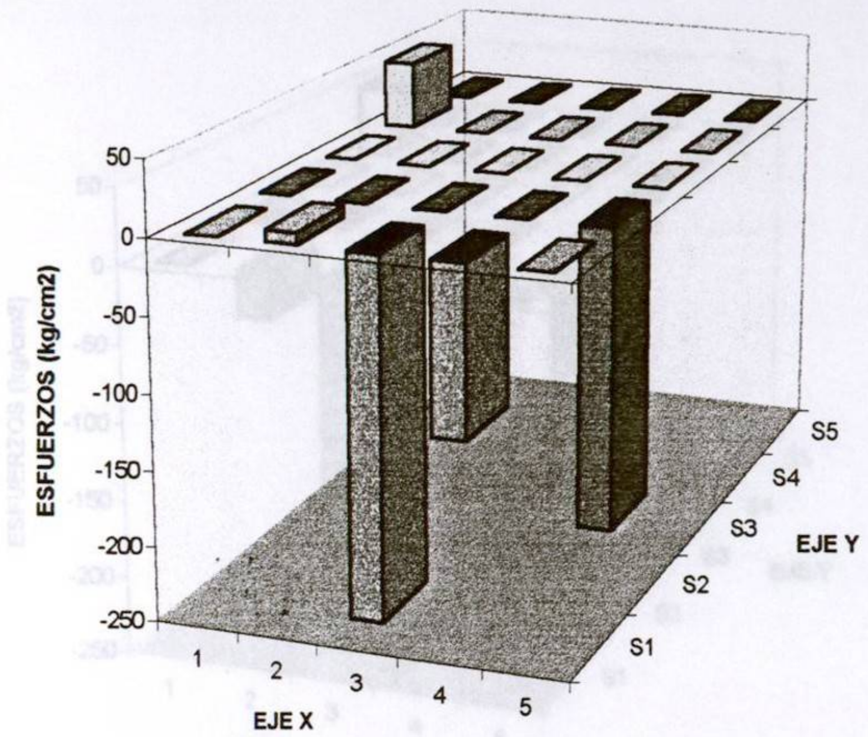
GRAFICA DE ESFUERZOS (kg/ cm2) para Px=48 ton. , Mx=3.885 ton.-m , My=0.835 ton.-m



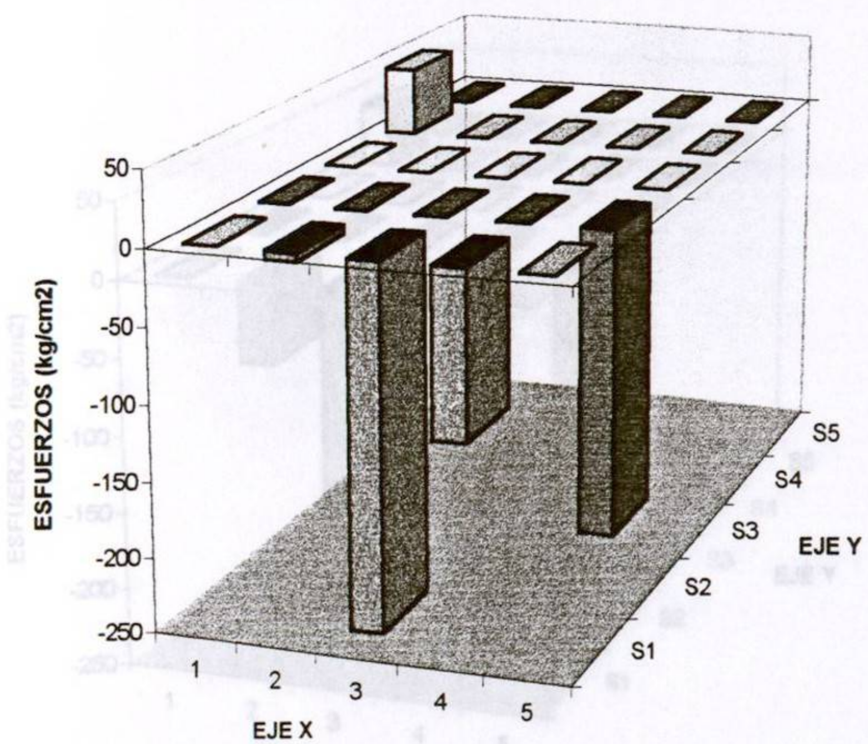
GRAFICA DE ESFUERZOS (kg/cm2) para Px=10 ton. , Mx=12.091 ton.-m , My=0.585 ton.-m



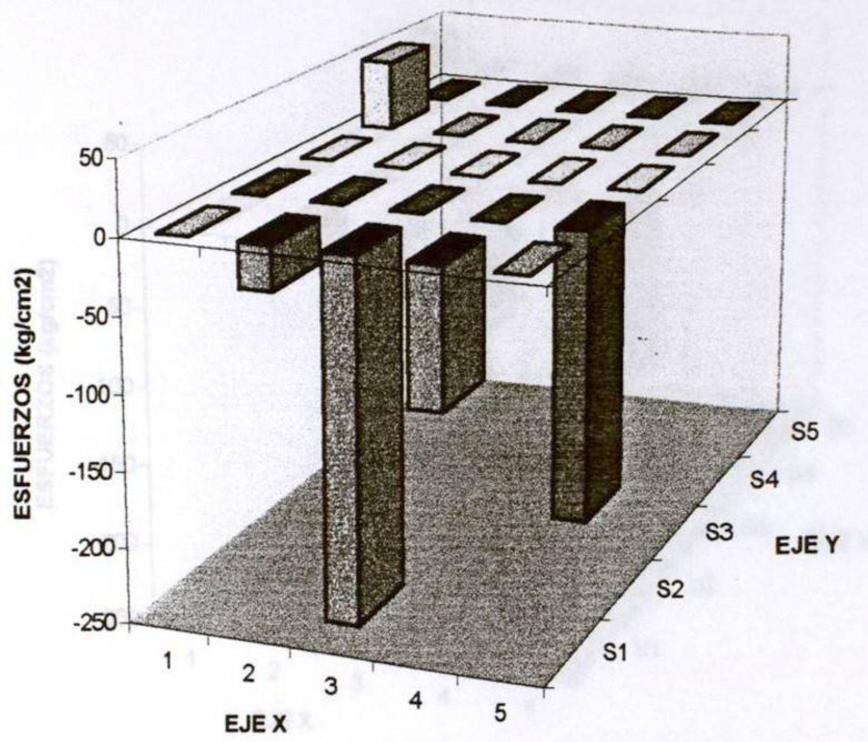
GRAFICA DE ESFUERZOS (kg/cm2) para Px=20 ton. , Mx=10.714 ton.-m , My=0.497 ton.-m



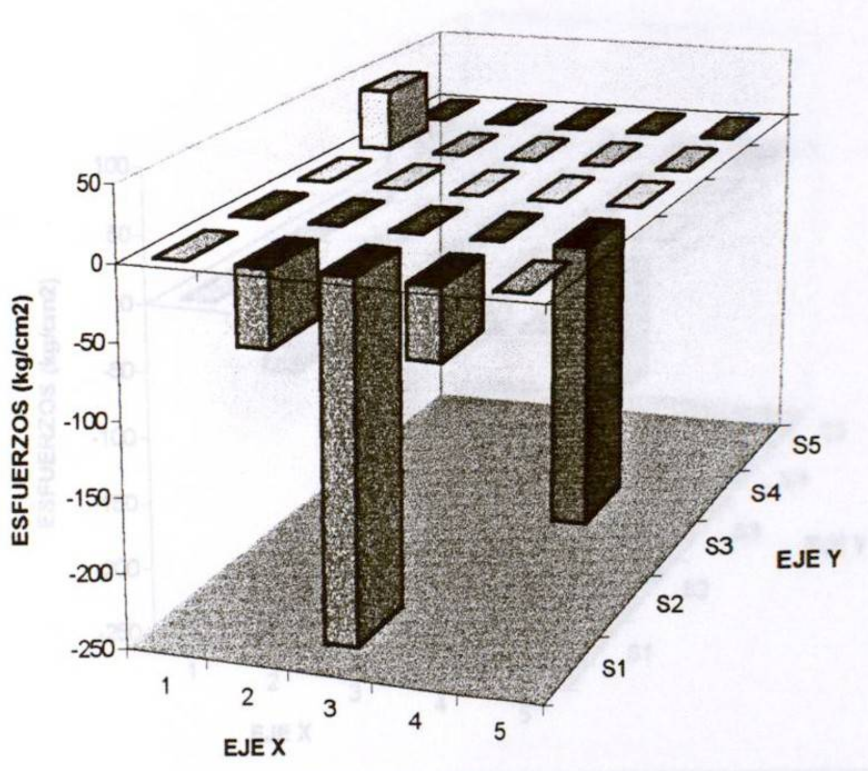
GRAFICA DE ESFUERZOS (kg/cm2) para Px=30 ton. , Mx=10.439 ton.-m , My=0.454 ton.-m



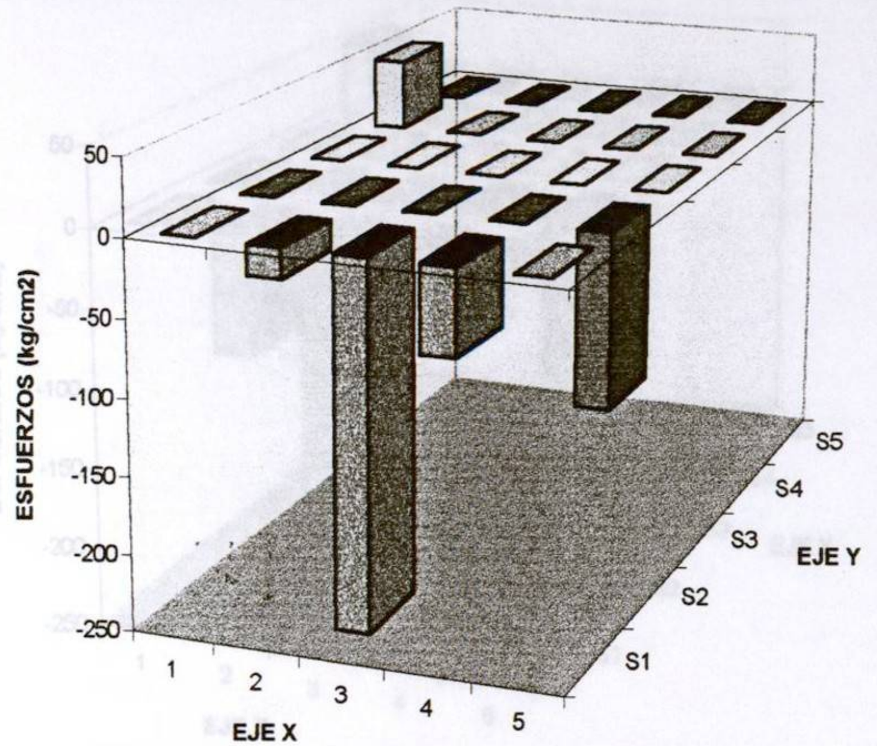
GRAFICA DE ESFUERZOS (kg/cm2) para Px=40 ton. , Mx=9.358 ton,-m , My=0.411 ton.-m



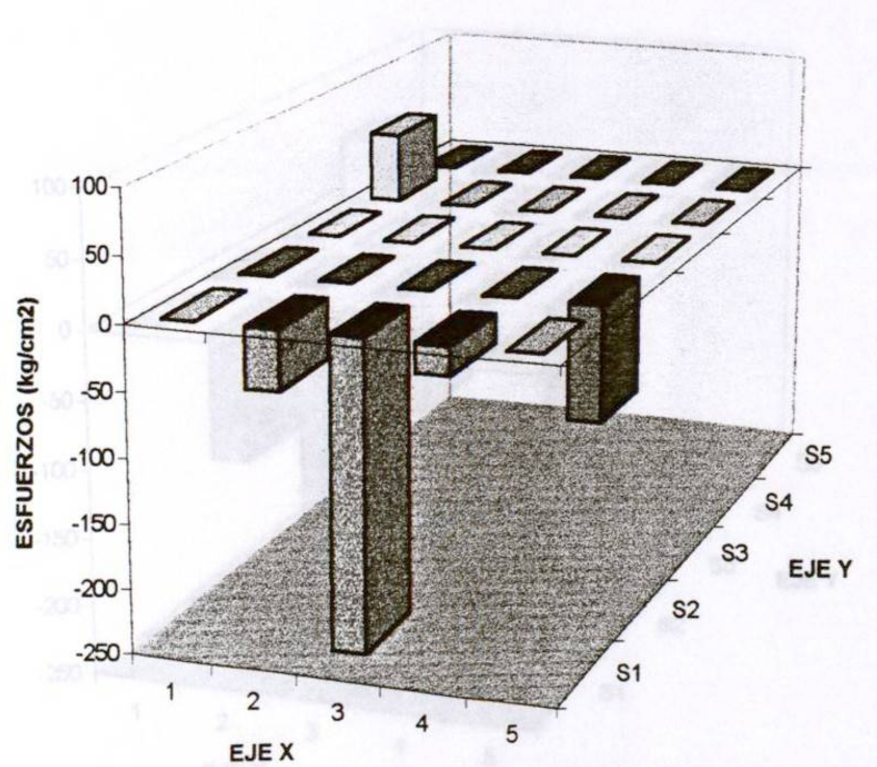
GRAFICA DE ESFUERZOS (kg/cm2) para Px=50 ton. , Mx=7.681 ton.-m , My=0.293 ton.-m



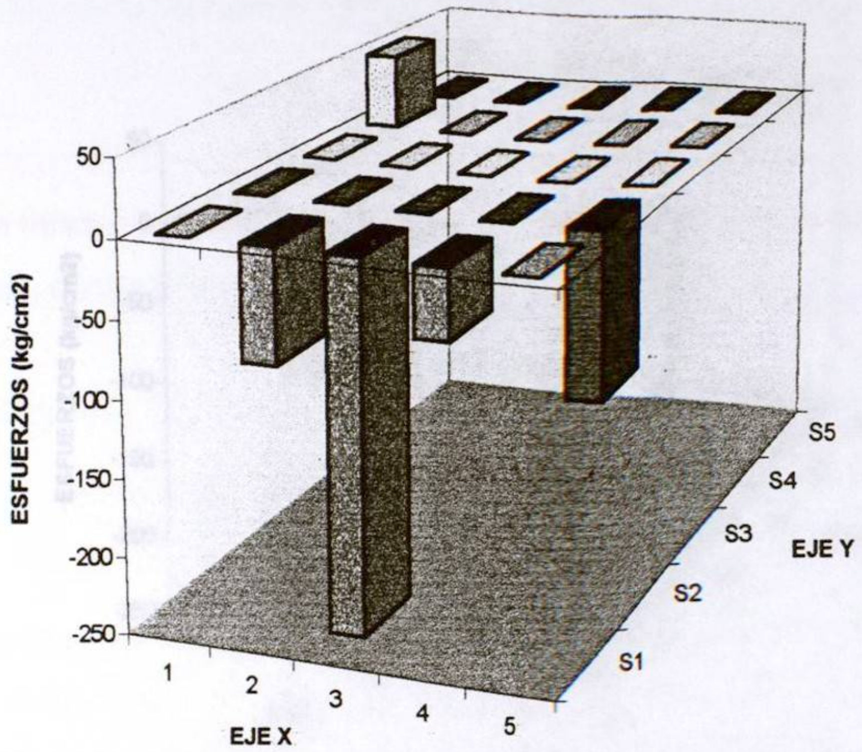
GRAFICA DE ESFUERZOS (kg/cm2) para Px=0 ton. , Mx=8.603 ton.-m , My=0.956 ton.-m



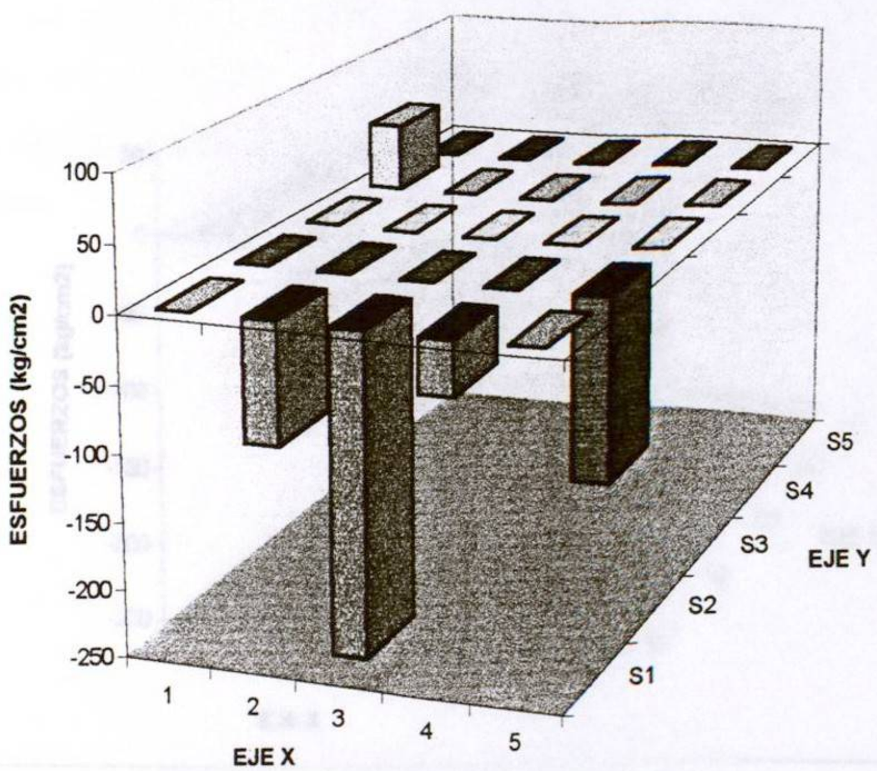
GRAFICA DE ESFUERZOS (kg/cm2) para Px=10 ton. , Mx=7.982 ton.-m , My=0.829 ton.-m



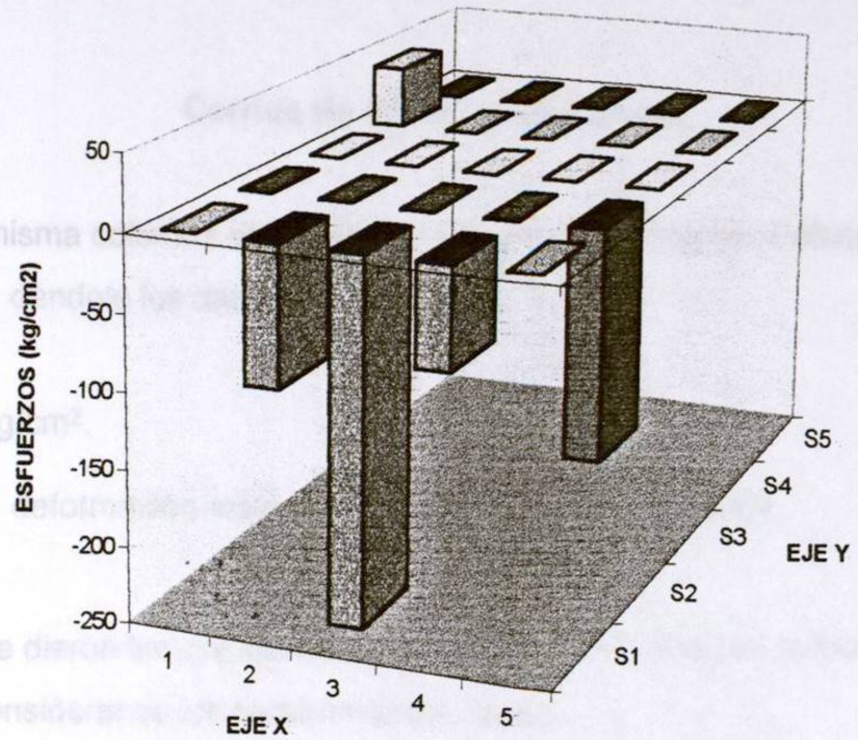
GRAFICA DE ESFUERZOS (kg/cm2) para Px=20 ton. , Mx=8,366 ton.-m , My=0.921 ton.-m



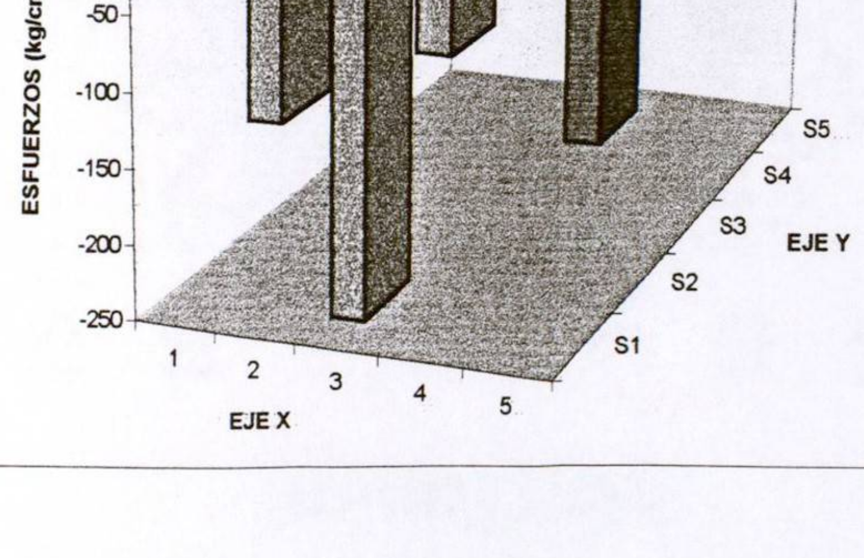
GRAFICA DE ESFUERZOS (kg/cm2) para Px=30 ton. , Mx=7.354 ton.-m , My=0.765 ton.-m



GRAFICA DE ESFUERZOS (kg/cm2) para Px=40 ton. , Mx=7.559 ton.-m , My=0.773 ton.-m



GRAFICA DE ESFUERZOS (kg/cm2) para Px=50 ton. , Mx=7 ton.-m , My=0.741 ton.-m



Corrida de Columna de Ensaye.

Para la misma columna del ensaye, se corrió en el programa diseñado en esta Tesis, dándole los datos de laboratorio:

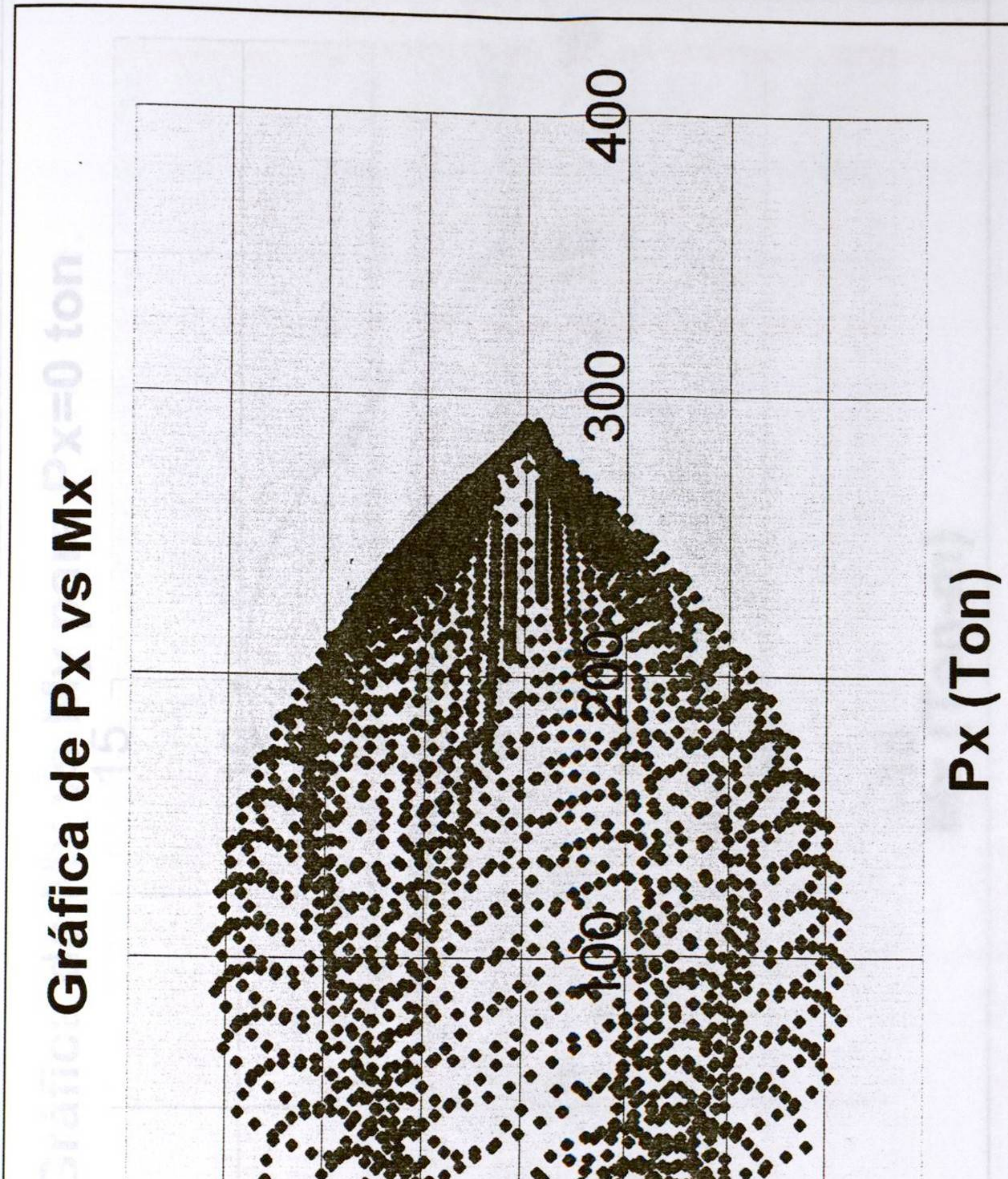
f´c = 250 kg/cm^2.

Ɛ = 0.001 deformación máxima que se permitió en el ensaye.

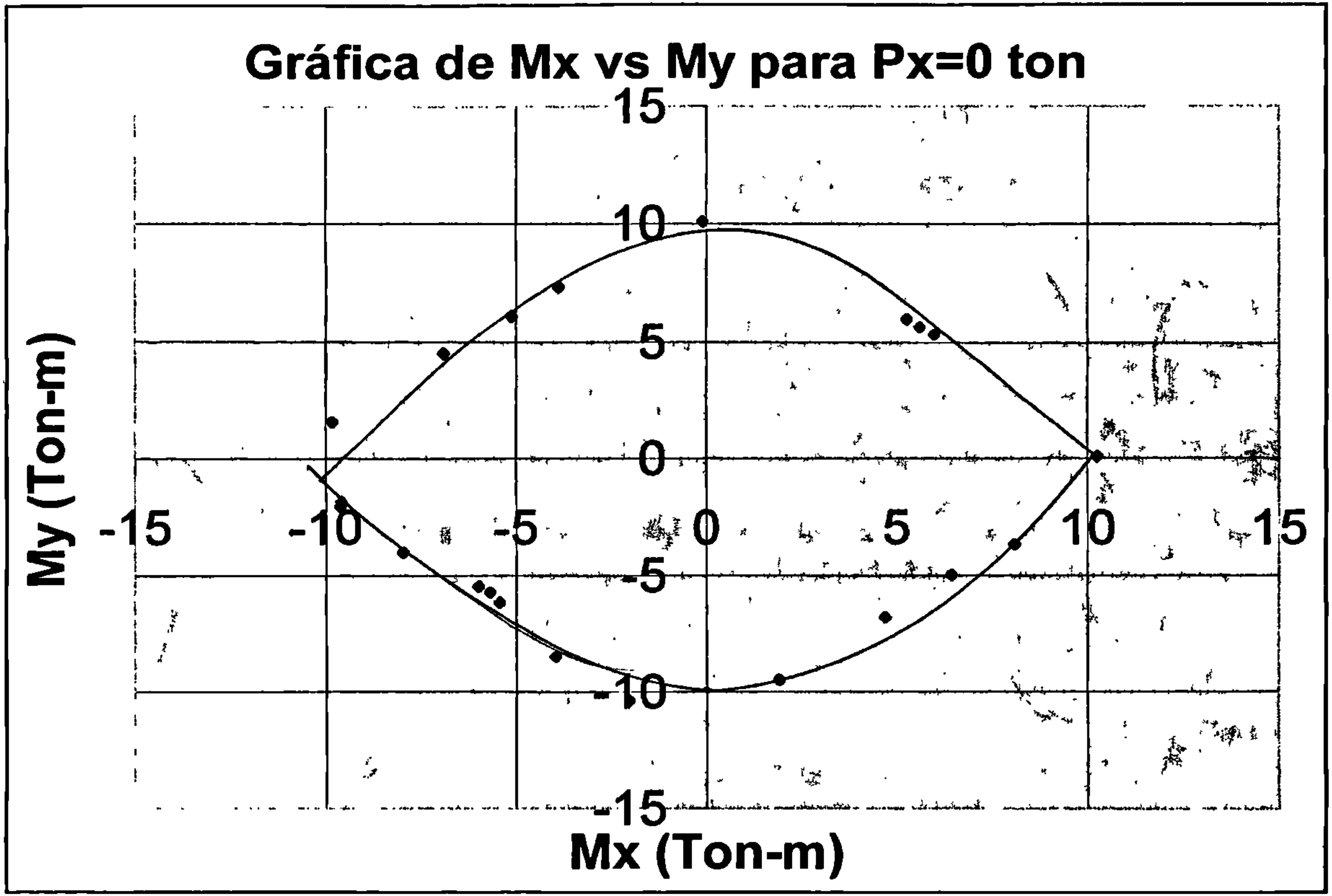
Se le dieron las coordenadas del acero de refuerzo, tal como quedó en el modelo, considerando los recubrimientos reales.

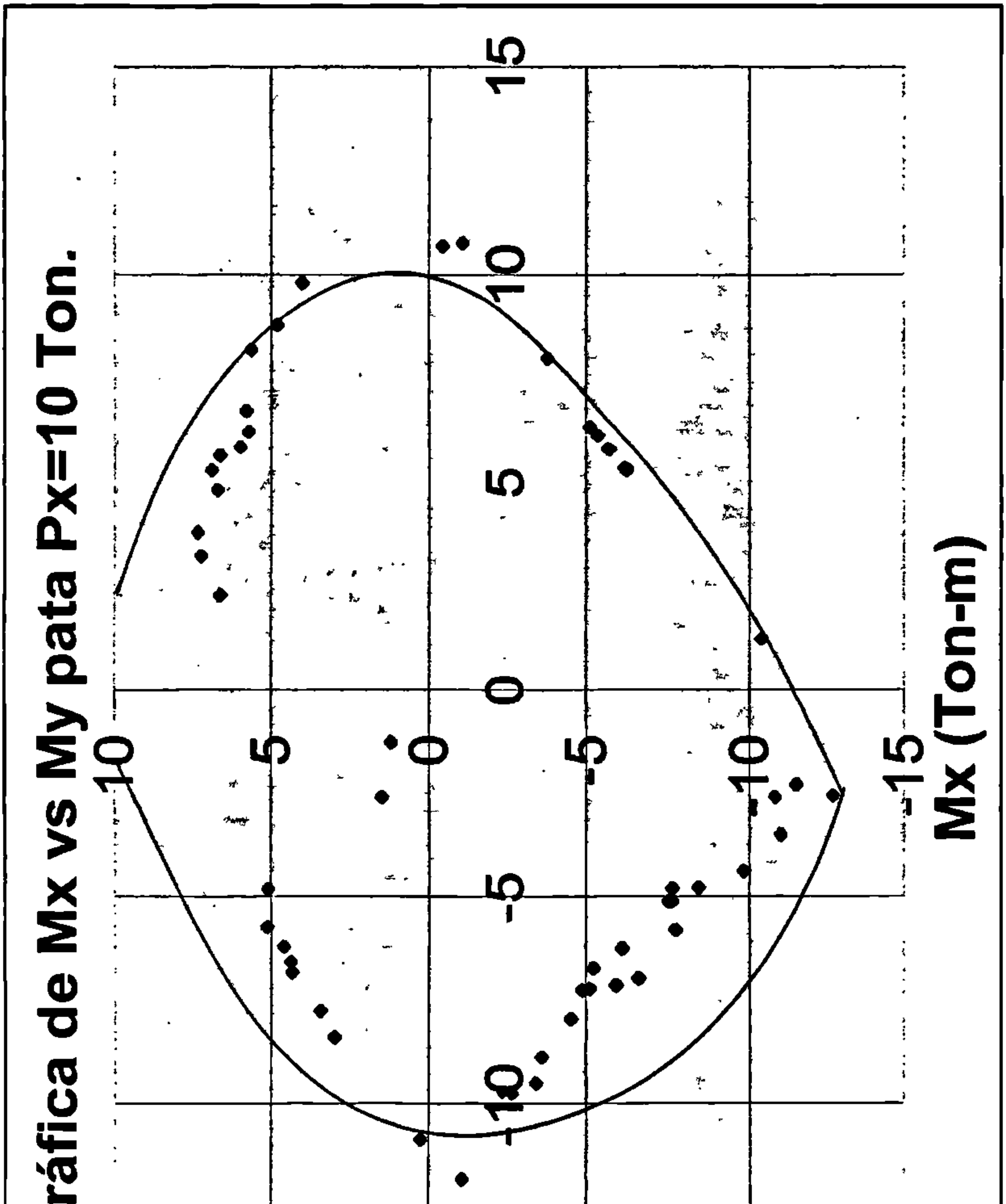
El módulo de elasticidad utilizado fue el proporcionado por el ensaye.

En la página 222 se representa la envolvente de falla de esta columna reportada por el programa y en las siguientes páginas, se obtuvieron las gráficas de Mx vs My para las cargas axiales con que se ensayó la columna espécimen.



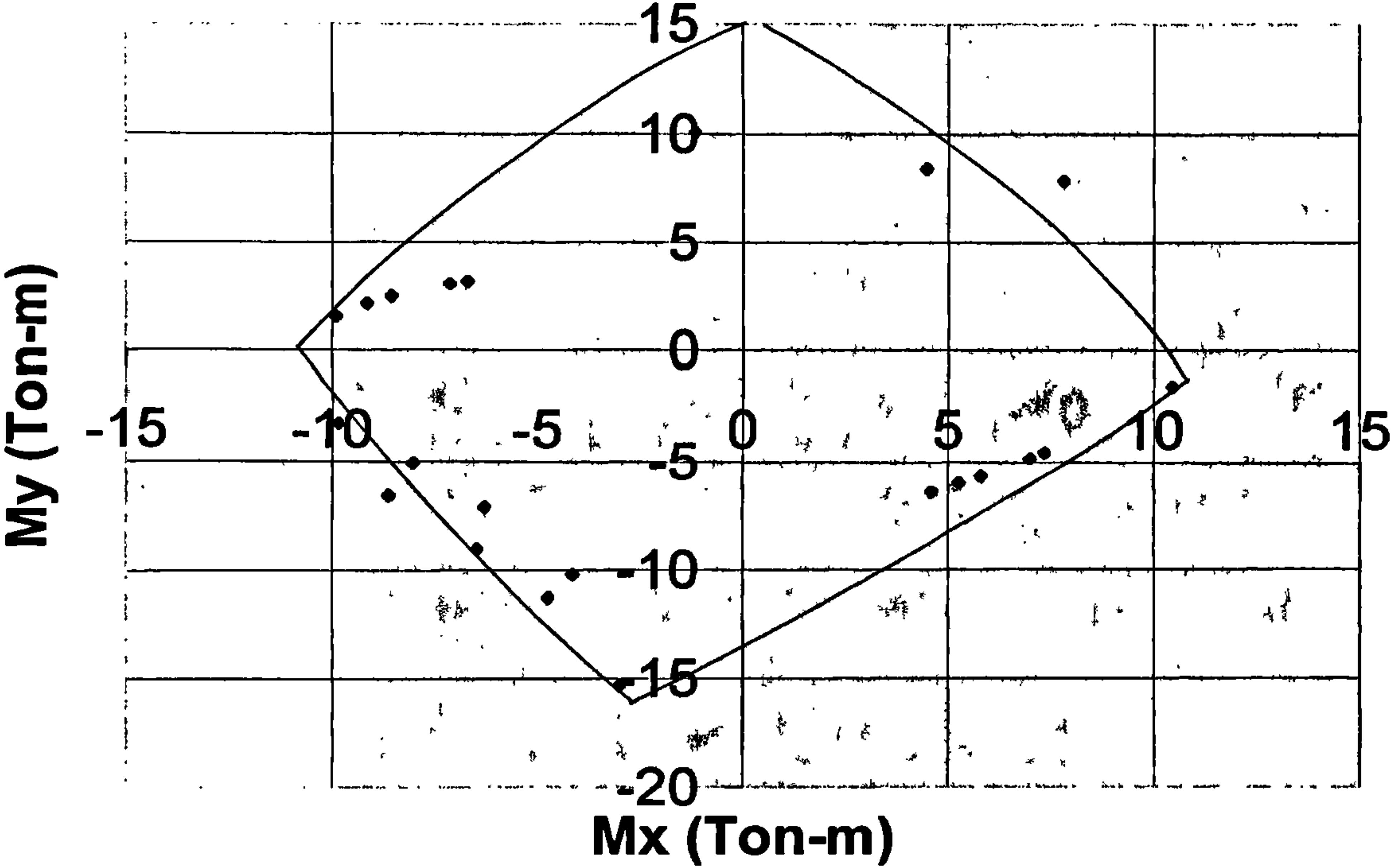
Gráfica de Px vs Mx
Mx (Ton-m)
Px (Ton)
20
15
10
5
0
-5
-10
-15
-20
0
100
200
300
400

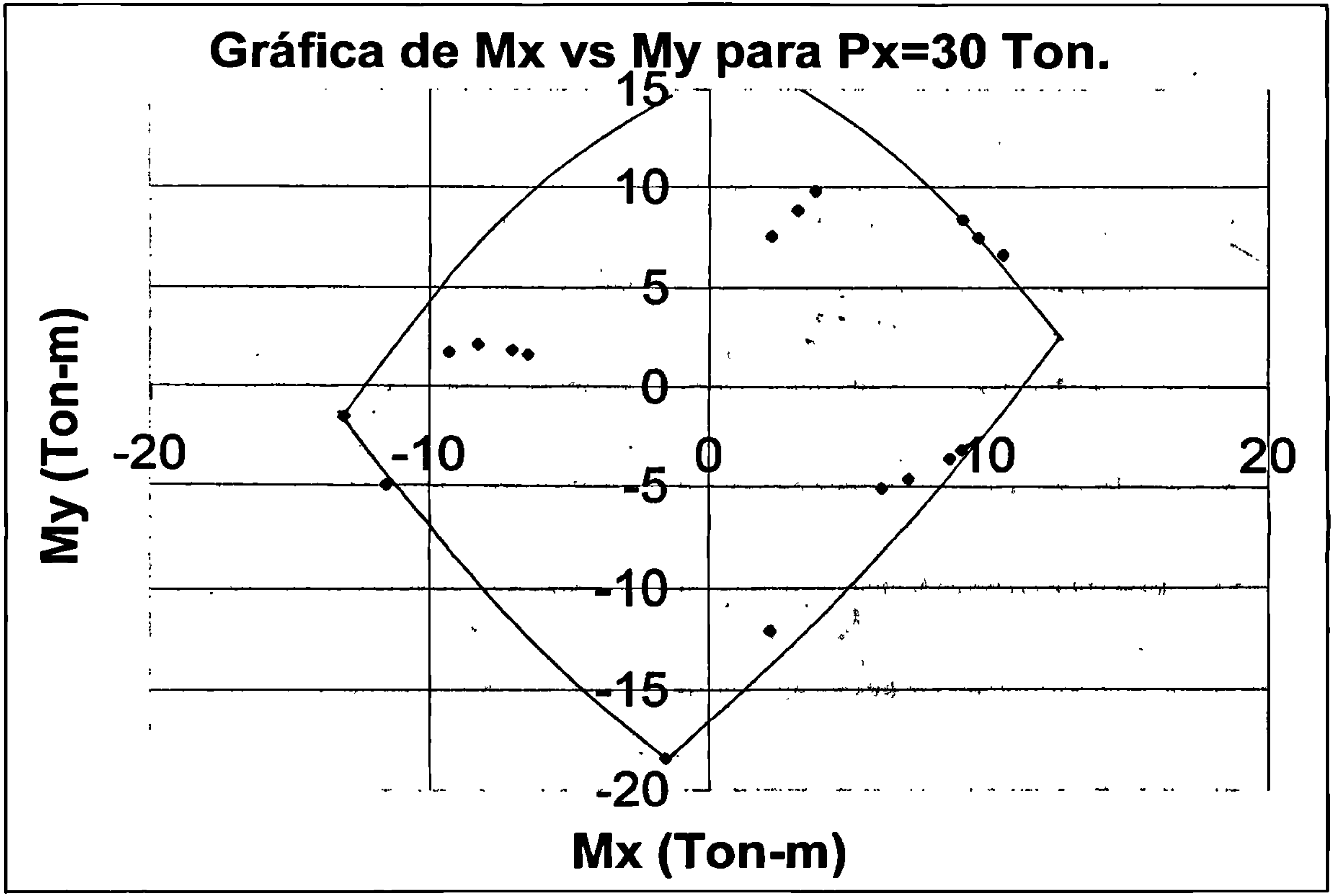

Gráfica de Mx vs My para Px=0 ton
My (Ton-m)
Mx (Ton-m)
15
10
5
0
-5
-10
-15
-15
-10
-5
0
5
10
15

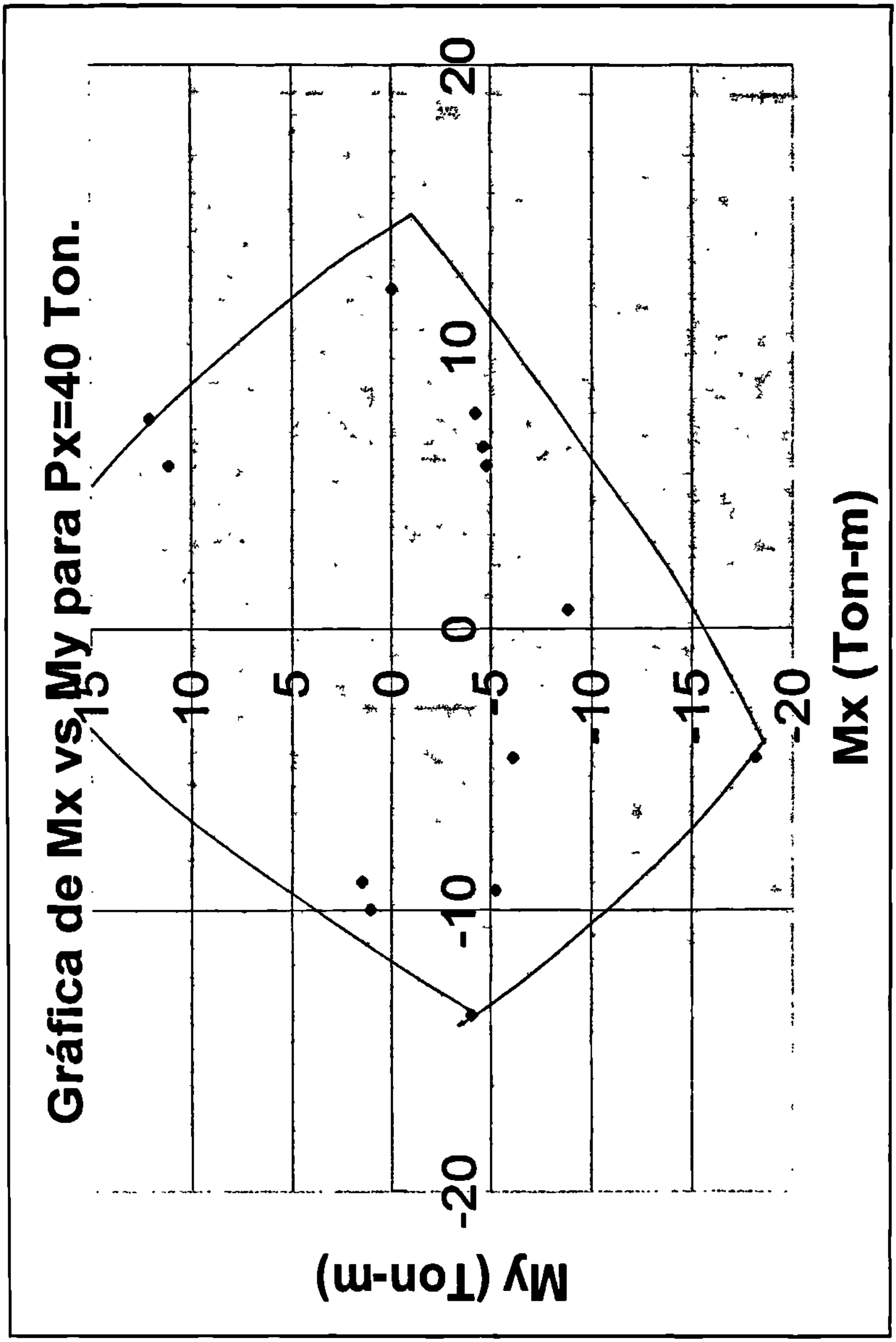


Gráfica de Mx vs My pata Px=10 Ton.
Mx (Ton-m)
My (Ton-m)
15
10
5
0
-5
-10
-15

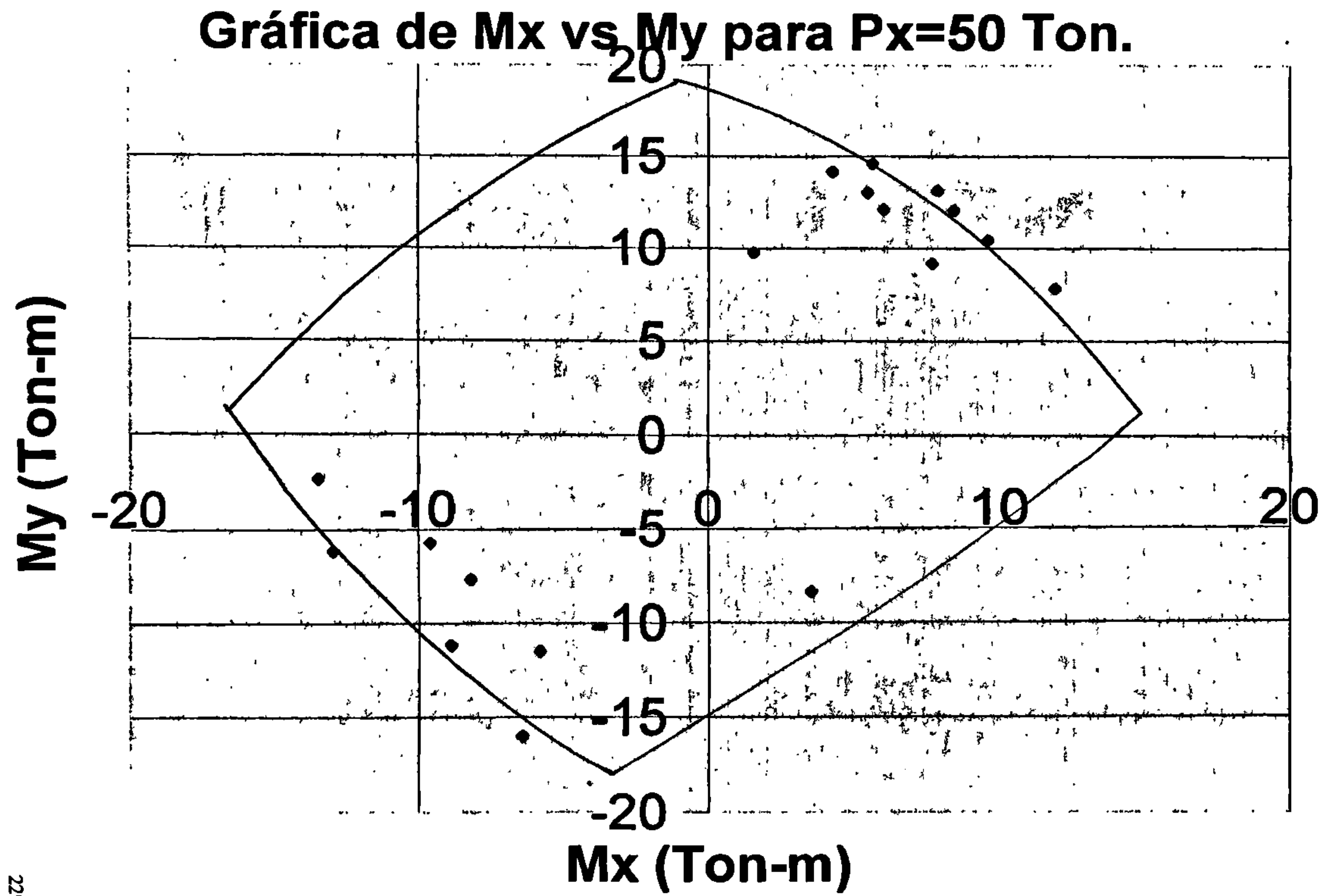
Gráfica de Mx vs My para Px=20 Ton.




Gráfica de Mx vs My para Px=30 Ton.
15
10
5
0
-5
-10
-15
-20
-20
-10
0
10
20
My (Ton-m)
Mx (Ton-m)



Gráfica de Mx vs. My para Px=40 Ton.
My (Ton-m)
Mx (Ton-m)
20
10
0
-10
-20
15
10
5
0
-5
-10
-15
-20


Gráfica de Mx vs My para Px=50 Ton.
My (Ton-m)
Mx (Ton-m)
20
15
10
5
0
-5
-10
-15
-20
-20
-10
0
10
20

CAPITULO 9

CONCLUSIONES.

Se estableció un algoritmo de uso general que permite obtener la resistencia última de columnas de cualquier sección transversal sujetas a carga axial y flexión biaxial. Hasta ahora, la literatura estructural de este tema solo incluye soluciones de ciertas formas de sección transversales como las cuadradas, rectangulares y circulares.

Se proporcionó el diagrama de flujo, con el fin de poder escribir la lógica en cualquier lenguaje, en este trabajo se escribió en "Basic". Es muy conveniente conjuntar las disciplinas de la Ingeniería Estructural y la de Ingeniería en Sistemas para lograr un avance mas pleno.

Con el fin de validar los resultados de este análisis se solucionaron seis iteraciones con secciones diferentes en los cuales se previó alguna complicación especial para el programa, con la intención de fallarlo, lo cual no sucedió y condujo a resultados exactos. Los resultados obtenidos provienen del análisis matemático exacto y no contemplan ningún factor de reducción.

Se corrió el programa para dos secciones conocidas, una sección cuadrada y otra circular. Paralelamente se analizó utilizando los diagramas de interación para momentos uniaxiales y el método aproximado de la carga inversa obteniendo resultados iguales en ambos casos adicionalmente; uno de ésos casos se comparó con el programa de la PCA y no se encontró concordancia. El programa de PCA muestra resultados factorizados y para la comparativa fueron desfactorizados, resultando aún así conservadores.

En el ensaye de la columna a escala natural, los primeros resultados del ensaye coinciden con la corrida del programa, y las deformaciones reportadas en los strain gages coinciden de forma significativa con el análisis; el hecho de que después se alejen del comportamiento reportado por el modelo obedece a la degradación de la columna por el ciclo de carga y descarga a la que fué sometida, de hecho, a las curvas que representan este comportamiento se les denomina "Diagramas de Comportamiento Histerético", por lo tanto, para proseguir, esta investigación, se precisa de muchos más especímenes y ser utilizados en una o dos ocasiones. Esos especímenes se pueden aprovechar para otra investigación sobre comportamiento histerético de columnas sujetas a cargas y momentos cíclicos biaxiales.

En este trabajo, no se hizo mucho hincapié en el análisis de resultados debido a que el modelo durante el ensaye se presentaron algunos problemas que ahora nos sirven para dar algunas sugerencias para las siguientes etapas de este proyecto de investigación

a) Se puede reducir la longitud del espécimen a un metro.
b) Los brazos para la aplicación de los momentos flectores pueden ser de acero.
c) Para lograr la transmisión del momento a la columna se sugiere una placa de acero soldando las varillas y rellenando con grout.
d) El nivel de deformaciones unitarias máximo que sea de 0.001 ya que más adelante el nivel de agrietamientos es muy alto y daña los strain gages del concreto.

e) Todas las grietas son paralelas entre si y perpendiculares a el eje axial, estas grietas en ocasiones están separadas mas de 10 cm, por lo cual se recomienda colocar los strain gages en varios niveles paralelos para evitar que todos se dañen al mismo tiempo al pasar una grieta en medio de ellos.

f) La sección de la columna deberá de tener una sección más pequeña con el fin de llevarla hasta su carga máxima.

g) Es muy conveniente contar con un equipo adecuado de adquisición de datos con capacidad para monitorear constantemente el comportamiento de los strain gages y no perder información valiosa.

APENDICE

RESUMEN DE PROPORCIONES Y DETERMINACIONES DE CONCRETO SIN ADITIVO.
CONCRETO FABRICADO DEL 04 AL 08 DE SEPTIEMBRE DE 1995 (1a SEMANA).
LOS MATERIALES ESTAN SECOS Y SE INCLUYE EL AGUA DE ABSORCION
CENIZA VOLANTE DE LA PLANTE CARBOELECTRICA DE RIO ESCONDIDO, AGREGADA EN ADICION AL CONSUMO DE CEMENTO.
RELACION A/CP = 0.35, PARA TODAS LAS BRIGADAS.

		BRIGADA 1	BRIGADA 2	BRIGADA 3	BRIGADA 4	BRIGADA 5
PROPORCIONAMIENTO (kg/m^3)						
AGUA	REACCION	219.6	211.1	206.9	203.3	195.3
	ABSORCION	15.4	14.9	14.6	14.3	13.7
CEMENTO PORTLAND I		399	384	376	369	355
CENIZA	CONSUMO	SIN	76.7	112.9	147.8	177.5
	PORCIENTO	SIN	20	30	40	50
AGREGADO GRUESO		1134	1090	1068	1049	1008
AGREGADO FINO		628	604	592	582	559
PROPIEDADES FISICAS DEL CONCRETO FRESCO						
REL. A/CE EN PESO		0.55	0.46	0.43	0.40	0.37
AIRE (%)		0.7	1.3	1.7	1.9	4.4
PESO VOL. TEORICO CONC. FRESCO (kg/m^3)		2396	2239	2449	2442	2292
PESO VOLUMETRICO CONC. FRESCO (kg/m^3)		2410	2237	2367	2324	2382
TEMP. CONCRETO FRESCO (°C)		32.0	33.5	35.0	37.0	38.0
REVENIMIENTO (cm)		20.0	10.0	8.0	5.0	3.0
VeBe (s)		no se tomó	no se tomó	3.60	7.80	8.80
CONDICIONES AMBIENTALES DEL CUARTO DE MEZCLADO						
TEMP. AMBIENTE (°C)		28.0	28.0	28.0	28.0	27.5
HUMEDAD RELATIVA (%)		92	92	85	92	92

ρ =	$M_E = 10^{-3}\, x$ tabulated factor			$M_I = 10^{-3}\, x$ tabulated factor		
	EXTERIOR COLUMNS			INTERIOR COLUMNS		
	f'_c = 4,000	f'_c = 5,000	f'_c = 6,000	f'_c = 4,000	f'_c = 5,000	f'_c = 6,000
0.005	4.13	4.56	4.96	3.10	3.42	3.72
0.01	4.37	4.78	5.15	3.28	3.58	3.87
0.02	4.86	5.21	5.55	3.64	3.91	4.16
0.03	5.34	5.65	5.95	4.01	4.23	4.46
0.04	5.83	6.08	6.34	4.37	4.56	4.76
0.05	6.31	6.51	6.71	4.74	4.89	5.05
0.06	6.80	6.95	7.14	5.10	5.21	5.35
0.07	7.28	7.38	7.53	5.47	5.54	5.65
0.08	7.79	7.81	7.93	5.83	5.86	5.95

Table 2-3 One-Step Combined Strength-Slenderness Design Factors M_E and M_I

Example 5. (Continued)

Interior Columns: $e = 1.6''$

$$\delta = \frac{1}{1 - 0.00364\,(36)} = 1.15 \text{ vs. } 1.10 \text{ from Ex. 3 corrected for } w = 145. \text{ See Table 2-2}$$

$e = (1.15)(0.10)(16) = 1.84''$
See page 3-65. For 16 x 16, 6-#6, read (*MA*) $P_u = 552k > $ (*MI*) $P_u = 542k >$ (2)(230). Use $\rho \geq 0.005$ (ACI 10.8.4). Final design: 16" x 16"; 4-#6.

BI-AXIAL BENDING

Introduction. Bi-axial bending of columns occurs when the column ing causes bending moment simultaneously about both principal axes. commonly encountered case of such loading is a corner column. The does not require that the moments resulting from use of the minimum e tricity be considered simultaneously (ACI 10.11.7).

Design for Bi-axial Bending Effects. Round columns, even wit minimum 4 bars, possess essentially polar symmetry; *bi-axial moment* simply be combined, $M_u = \sqrt{(M_x)^2 + (M_y)^2}$, *with no further complic*

For square and rectangular columns, the exact *solution for bi-axial* ing using only the design assumptions, page 2-3, is *too complicated for* ro design. The solution has been reduced to a *minimum of very simple* cal tions with coefficients obtained from *Figs. 2-13 and 2-14 following.* T coefficients are applied to *magnify* the *uniaxial applied design moment* which load capacity can be read *directly from tables in Chapter 3.*

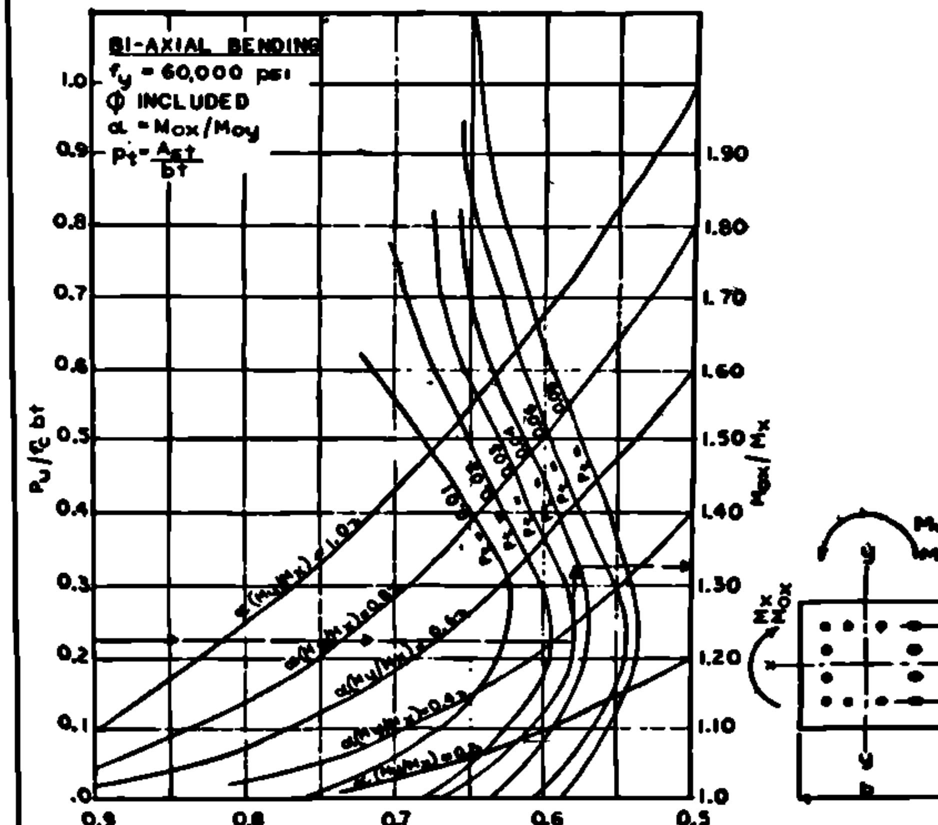


Fig. 2-13 Bi-axial Bending—Bars in 4 Faces

The abscissae, β, here and in Fig. 2-14 are identical. β values are shown for inform only and are not employed in using Figs. 2-13 and 2-14. (β is the ordinate to curves in 2-15.) See page 2-33 for mathematical derivation.

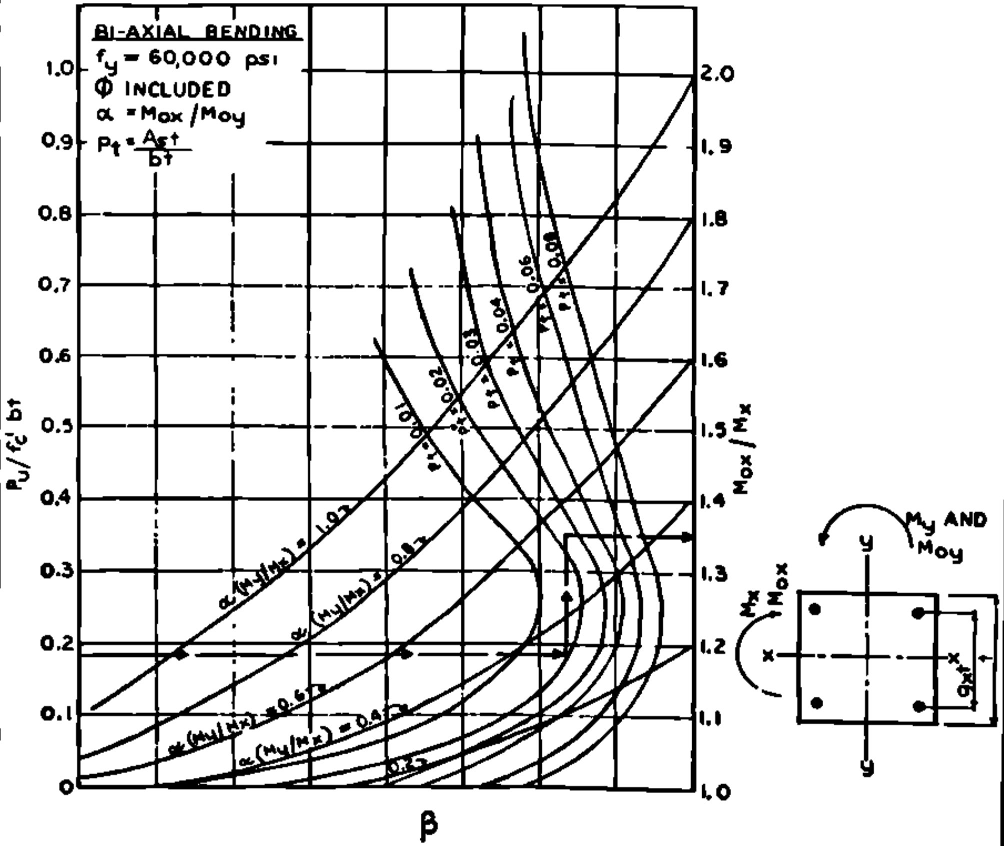


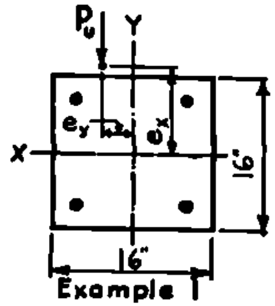
Fig. 2-14 Bi-axial Bending—4 Bars

Examples Using Charts (Figs. 2-13 and 2-14)

To illustrate the simplicity with which the biaxial bending capacity of a column can be obtained, three examples are presented in detail.

Example 1—Square Column

Given:



$f'_c = 4{,}000$ psi
$f_y = 60{,}000$ psi
$P_u = 196$ kips
$M_x = 134$ ft. kips
$M_y = 71$ ft. kips

Desired column size 16″ × 16″; 4 bars
To find area of steel required for biaxial bending
Solution:
Assume initially $p_t = 0.02$ $\alpha = M_{ox}/M_{oy} = 1.0$
Compute

$$\alpha \frac{M_y}{M_x} = \frac{71}{134} = 0.53$$

$$P_u/f'_c bt = \frac{196}{(4)(16 \times 16)} = 0.19$$

M_{ox} = Uniaxial bending capacity about X axis.
M_{oy} = Uniaxial bending capacity about Y axis.
M_x and M_y = design moments about X and Y axis, respectively.

With $P_u/f'_c bt = 0.19$, proceed horizontally in Fig. 2-14 to curve $p_t = 0.02$, then vertically to $M_y/M_x = 0.53$, read $M_{ox}/M_x = 1.34$ at right ordinate.
Hence, uniaxial design moment to supply biaxial bending resistance

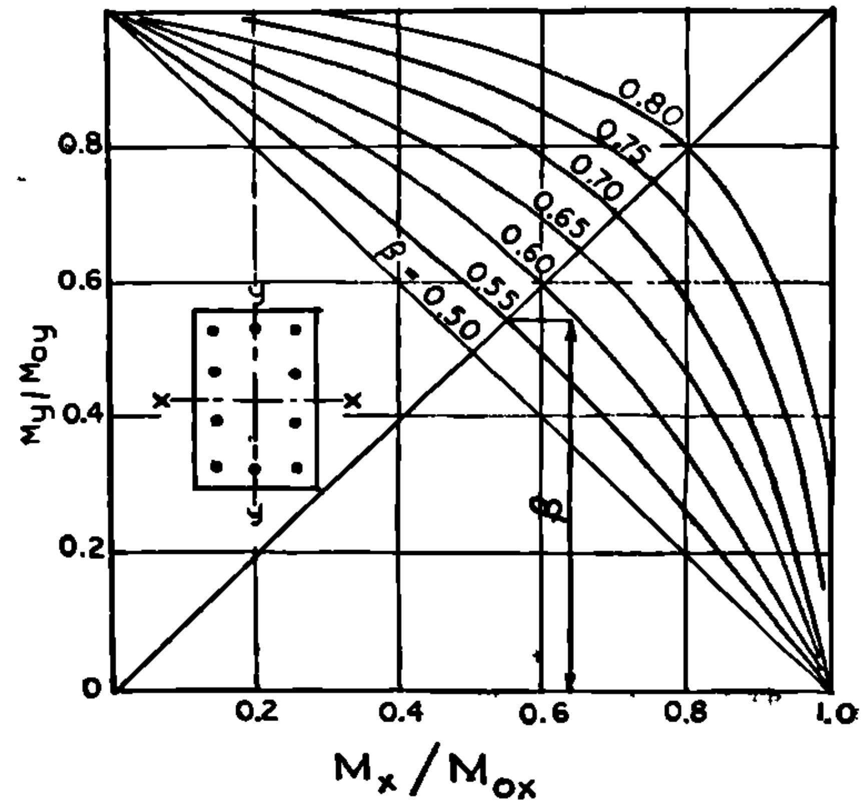
$$M_{ox} = 134 \times 1.34 = 180 \text{ ft.-kips}$$

$$M_u/P_u = \frac{(180)(12)}{196} = 11.00 \text{ inches}$$

From table page 3-20 with $e = 11.0$ in., interpolate for solution
Use: 4-#10 bars; $P_u = 203$ kips; $P_t = 0.0198$
By use of the linear variation of biaxial bending given by the formula:

$$\frac{M_y}{M_{oy}} + \frac{M_x}{M_{ox}} = 1;\ M_{oy} = M_{ox} = 134 + 71 = 205 \text{ ft.-kips}$$

For $P_u = 196\ k$ and $e_x = e_y = \frac{205 \times 12}{196} = 12.5$ in.; interpolate from table, 3-20, $P_u = 199\ k$ for 8-#9, which amounts to about 50 per cent more reinforcement that required by the more accurate solution.



$$\left(\frac{M_x}{M_{ox}}\right)^n + \left(\frac{M_y}{M_{oy}}\right)^n = 1 \qquad n = \frac{\log 0.5}{\log \beta}$$

Fig. 2-15 Bi-axial Bending Basic Relationships—Equation (2-1

Example 2—Square Column

Check the approximate solution of Example 3, Chapter 3, by the more exact methods here. Given:

$P_u = 400\ k$ $\quad f'_c = 4{,}000$ psi
$e_x = e_y = 2.25$ in. $\quad$ Bars — Grade 60

Check the approximate solution 16″ × 16″, 4 − #10, $p_t = 0.0198$

$M_x = e_x P_u = 900$ in.-kips
$M_y = e_y P_u = 900$ in.-kips

$\alpha = \frac{M_{ox}}{M_{oy}} = 1.0$ By symmetry about X-X and Y-Y.

$\alpha \frac{M_x}{M_y} = 1.0 \qquad \frac{P_u}{f'_c bt} = \frac{(400)}{(4)(16 \times 16)} = 0.39$

As in Example 1, enter Fig. 2-14 with 0.39, proceed right to $p_t = 0.02$, up to

$\alpha \frac{M_x}{M_y} = 1.0$, right to the ratio, $M_{ox}/M_x = 1.65$.

$M_{ox} = (1.65)(900) = 1{,}485$ in.-kips
$e_x = (1.65)(2.25) = 3.71$ in.

Enter load capacity tables, page 3-20, and read for 4-#8 $e = 3$ in., $P_u = 476$ kips, $e = 4$ in., $P_u = 408$ kips. Check: $P_t = 0.0123$. Enter Fig. 2-14, for $p_t = 0.0123$, read $M_{ox}/M_x = 1.59$.

Use: 16″ × 16″, 4-#8.
Note: The "straight-line" approximation of Example 3, Chapter 3, is always conservative. For a combined bending vector at 45 degrees ($M_x = M_y$) the difference between the straight line and more exact methods is, of course, maximum.

Example 3—Rectangular Section

Given:

$f'_c = 5{,}000$ psi
$f_y = 60{,}000$ psi
$P_u = 300$ kips
$M_x = 270$ ft.-kips
$M_y = 70$ ft.-kips
Column 12″ × 24″

Assume:

$p_t = 0.03$
$r_x = 0.67$
$r_y = 1.00$

Compute:

$\sqrt{q} = \sqrt{(0.03)(60)/5} = 0.6$

$\alpha \frac{M_y}{M_x} = (1.93)(70/270) = 0.50$

$P_u/f'_c bt = 300/(5)(12)(24) = 0.21$

$$\alpha = \frac{M_{ox}}{M_{oy}} = \frac{[1 - 0.7(1 - r_x)\sqrt{q}](1 + g_x)h_x}{[1 - 0.7(1 - r_y)\sqrt{q}](1 + g_y)h_y}$$

$$\alpha = \frac{[1 - (0.7)(1 - 0.67)(0.6)](1 + 0.8)(24)}{[1 - (0.7)(1 - 1.0)(0.6)](1 + 0.6)(12)} = 1.93$$

in which:

$q = p_t f_y/f'_c$
r_x = ratio of steel area in furthermost rows from x axis to total steel area
r_y = ratio of steel area in outermost rows from y axis to total steel area
h_x and h_y = height of member about x and y axis respectively
g_x and g_y = ratio of distance between outermost rows to total height about x and y axis

From Fig. 2-13 with $P_u/f'_c bt = 0.21$, $\quad p_t = 0.03$ and

$\alpha M_y/M_x = 0.50 \qquad M_{ox}/M_x = 1.32$

Hence: $M_{ox} = 1.32 \times 270 = 356$ ft.-kips

The determination of steel required for rectangular sections can best be pursued by utilizing tables with bars arranged in the same pattern as that for which the ratios, r_x and r_y, were assumed. This approach will be followed. With arrangement shown in sketch, and utilizing the table on page 3-105, enter with $P_u = 300$ kips and $e_x = \frac{356 \times 12}{300} = 14.24$ in.

For 6-#10, 3L-2S, read: $\quad P_u = 370\ k$ for $e_x = 12$ in.

$p_t = 0.0264 \qquad P_u = 267\ k$ for $e_x = 16$ in.

Interpolating for $e = 14.24$ in., $P_u = 314\ k > 300$.
Since the actual steel ratio is less than the assumed ratio, the selection of design should be checked with corrected constants. Note that r_x and r_y as assumed are unchanged since the pattern assumed is that selected, 3L-2S.

$\sqrt{q} = \sqrt{(0.0264)(60/5)} = 0.564$

$$\alpha = \frac{[1 - (0.7)(0.33)(0.564)]}{1}\left(\frac{1.8 \times 24}{1.6 \times 12}\right)$$
$$= 1.94 = 1.93$$

Actual M_{ox} and M_{oy} values are available in the load capacity tables of this book. Enter table with $P_u = 314^k$ and read for capacity about MI axis:

$P_u = 353\ k$ at $e_y = 6''$
$P_u = 269\ k$ at $e_y = 8''$

Interpolating for $P_u = 314^k$, $\quad e_y = 6.93'' \qquad M_{oy} = 2{,}170$ in.-kips.

$\alpha = \frac{(314 \times 14.24)}{2170} = 2.06$

$\alpha(M_y/M_x) = (2.06)(70/270) = 0.535$

Enter Fig. 2-13 with $\alpha(M_y/M_x) = 0.535$, $P_u/f'_c bt = 0.21$, and $p_t = 0.0264$.

$M_{ox}/M_x = 1.32$; selected design is O.K.

Use: 12″ × 24″, 6-#10, 3L-2S.

Derivations and Basis for Charts

The determination of the reinforcement required in a rectangular column to resist a prescribed set of biaxial bending moments with a specified axial load can be most simply achieved by determining the uniaxial capacities which will yield the prescribed biaxial resistance. It has been well established that even for rectangular sections with width to depth varying from one third to three, and different bar arrangements, that the biaxial bending capacity can be approximated by the formula:

$$(M_x/M_{ox})^a + (M_y/M_{oy})^a = 1 \ldots\ldots\ldots\ldots (2\text{-}1)$$

in which:

(continued)

M_x = applied moment about the x axis at P_u
M_y = applied moment about the y axis at P_u
M_{ox} = uniaxial moment capacity about x axis at P_u
M_{oy} = uniaxial moment capacity about y axis at P_u
n = exponent dependent on bar arrangement, f_y, p_t, f'_c, axial load and g.

It is apparent that when $n = 1$, M_x/M_{ox} varies linearly with respect to M_y/M_{oy}, as shown in Fig. 2-15. This response is correct for a homogeneous material behaving elastically. However, for reinforced concrete, due to the effect of cracking and inelastic response, n is always greater than 1. The relationship implied by equation (2-1) yields a series of curves sometimes labelled as sub- or superellipses as can be seen in Fig. 2-15.

For the purpose of design, it is more convenient to restate equation (2-1) as:

$$\left(\frac{M_x}{M_{ox}}\right)^n + \left(\frac{M_y}{M_x}\cdot\frac{M_x}{M_{ox}}\cdot\frac{M_{ox}}{M_{oy}}\right)^n = 1 \quad \text{(2-2a)}$$

Factoring and letting $M_{ox}/M_{oy} = \alpha$ we have

$$\left(\frac{M_x}{M_{ox}}\right)^n\left[1 + \left(\alpha\frac{M_y}{M_x}\right)^n\right] = 1 \quad \text{(2-2b)}$$

which on transposing yields

$$\frac{M_{ox}}{M_x} = \left[1 + \left(\alpha\frac{M_y}{M_x}\right)^n\right]^{1/n} \quad \text{(2-2c)}$$

It should be noted that the value of M_{ox}/M_x is expressed as the function of two variables $\left(\alpha\dfrac{M_y}{M_x}\right)$ and n. The value, M_y/M_x, is a given quantity. For a square section, with symmetrical reinforcement $\alpha = 1.0$. For rectangular sections, with reinforcement symmetrical about the bending axis, the value of α is closely approximated by the formula:

$$\alpha = \frac{M_{ox}}{M_{oy}} = \frac{[1 - .7(1 - r_x)\sqrt{q}](1 + g_x)h_x}{[1 - .7(1 - r_y)\sqrt{q}](1 + g_y)h_y} \quad \text{(2-3)}$$

in which:

$q = \dfrac{A_{st}f_y}{b\,t\,f'_c} = p_t\dfrac{f_y}{f'_c}$
r_x = ratio of steel area in furthermost rows from x axis to total steel area
r_y = ratio of steel area in outermost rows from y axis to total steel area
h_x and h_y = height of member about x and y axis respectively
g_x and g_y = ratio of distance between outermost rows to total height about x and y axis

For the average condition, it will be found that $\alpha = h_x/h_y$.

With the value of n known, it is apparent that a series of curves can be plotted relating M_{ox}/M_x to $\alpha(M_y/M_x)$. Although as previously stated, n is a function of six variables, through extensive computer calculations, it was found that n is affected primarily by four variables, $P_u/f'_c bl$, f_y, p_t and bar arrangement, with the two other variables having a negligible effect. Consequently, this made it possible to express the behaviour of biaxial bending in Figs. 2-13 and 2-14, with the results having an accuracy of about 4 per cent.

The value of n for any specific cross section can be obtained by calculation of the values of M_{ox}, M_{oy} and M_x associated with M_y using the general assumptions for design in the Code (ACI 10.3). From a computational point of view, it was more meaningful and expedient to relate n to a new variable. Let $\beta = M_x/M_{ox}$ when $M_x/M_{ox} = M_y/M_{oy}$, then equation (2-1) becomes:

$$2\beta^n = 1$$
$$\beta^n = 0.5$$
$$n \log\beta = \log 0.5$$
$$n = \log 0.5/\log\beta$$

It should be recognized that β is the ordinate to curves at the intersection of the diagonal and the curves in the sketch. While it may be of some interest to know the value of β, to avoid superfluous notation, its value is not shown in Fig. 2-13 and Fig. 2-14. Each of these figures represents the superposition of two graphs having identical abscissa. The first chart yields the value of β as abscissa with $P_u/f'_c\,bl$ as ordinate for various values of p_t. The second chart gives the value of M_{ox}/M_x as ordinate, for various values of $\alpha(M_y/M_x)$ with β as abscissa. For design, as shown in Fig. 2-13, enter the chart with a specified value, $P_u/f'_c\,bt$, horizontally to an assumed value of p_t. From this intersection, proceed vertically to the proper $\alpha(M_y/M_x)$ curve. The value of M_{ox}/M_x is read at the right ordinate. Since M_x is a specified value, M_{ox} is easily calculated.

With M_{ox} known, the designer must determine the percentage of reinforcement from uniaxial column charts. This must be followed by a check that the required p_t agrees with the assumed p_t.

Load capacity tables for columns under combined bending and axial loads are generally restricted to the case of steel at top and bottom only or uniformly spaced about the periphery. With other bar arrangements, a designer should interpolate between the two sets of values. If the data is restricted to solely the capacity of columns with steel at top and bottom, the capacity for other distribution of reinforcement can be closely approximated by the formula:

$$M_u = [1 - 0.7(1 - r_x)\sqrt{q}]M_{us} \quad \text{(2-4)}$$

where M_{us} is the moment capacity of the section with the steel confined to a top and bottom row.

SQUARE TIED COLUMNS 10" × 10"

Short columns; no sidesway [1] Bars symmetrical in 4 faces

ars
ade 60 — ncrete = 4,000 psi

P_u (kips)—Ultimate Usable Capacity; $M_u/P_u = e$ (in.) ($\phi = 0.70$)

$0.10f'_cA_g$ = 40 kips [2]

rs	p %	0	0.1t	2"	3"	4"	6"	8"	12"	16"	20"	24"	28"	OT [3] e (in.)	Balance e (in.)	Balance P_u (k)	40 k [2] e (in.)	O [3] M_u (k-ft.)
#5	1.24	287	226	174	131	101	60	0	0	0	0	0	0	1.83	4.75	86	7.89	21
#6	1.76	308	241	186	141	110	73	49	0	0	0	0	0	1.89	5.60	81	9.28	28
#7	2.40	333	259	201	152	120	83	60	0	0	0	0	0	1.95	6.77	75	10.98	36
#8	3.16	363	280	217	165	130	91	70	43	0	0	0	0	2.00	8.46	67	12.85	44
#9	4.00	396	303	234	177	141	99	76	51	0	0	0	0	2.04	10.93	57	14.82	53
#10	5.08	439	332	254	193	153	108	83	57	43	0	0	0	2.07	15.82	44	17.30	65
#11	6.24	485	361	265	198	157	110	85	58	44	0	0	0	2.07	36.44	20	17.96	73
#6	3.52	377	284	215	161	127	88	67	42	0	0	0	0	1.82	7.67	70	12.58	48
#7	4.80	428	317	239	179	142	99	76	51	0	0	0	0	1.84	10.51	59	15.06	58
#8	6.32	488	356	265	198	158	110	85	58	44	0	0	0	1.86	16.04	44	17.69	70

SQUARE TIED COLUMNS 12" × 12"

$0.10f'_c A_g$ = 57 kips

rs	p %	0	0.1t	2"	3"	4"	6"	8"	12"	16"	20"	24"	28"	OT e (in.)	Balance e (in.)	Balance P_u (k)	57 k e (in.)	O M_u (k-ft.)
#6	1.22	412	327	277	223	181	122	78	0	0	0	0	0	2.09	5.61	134	9.78	37
#7	1.66	438	345	294	238	195	138	95	0	0	0	0	0	2.18	6.42	129	11.36	48
#8	2.19	468	367	313	255	210	150	113	65	0	0	0	0	2.27	7.44	124	13.20	60
#9	2.77	501	391	334	274	226	163	126	77	0	0	0	0	2.35	8.67	118	15.18	73
#10	3.52	544	422	360	296	245	177	138	92	65	0	0	0	2.42	10.46	109	17.65	89
#11	4.33	590	453	385	315	257	186	145	100	73	0	0	0	2.49	13.25	91	19.70	103
#14	6.25	699	529	449	363	296	214	167	116	89	72	60	0	2.57	23.58	62	25.21	139
#7	3.33	533	406	341	274	225	161	124	78	0	0	0	0	2.12	8.24	121	15.76	86
#8	4.38	593	447	375	303	249	179	139	92	67	0	0	0	2.18	10.16	112	18.50	103
#9	5.55	660	492	412	333	274	198	154	107	78	61	0	0	2.22	12.76	101	21.44	122
#10	7.05	745	550	458	370	304	220	172	119	91	72	59	0	2.26	17.25	85	24.86	144

SQUARE TIED COLUMNS 14" × 14"

$0.10f'_c A_g$ = 78 kips

rs	p %	0	0.1t	2"	3"	4"	6"	8"	12"	16"	20"	24"	28"	OT e (in.)	Balance e (in.)	Balance P_u (k)	78 k e (in.)	O M_u (k-ft.)
#7	1.22	561	447	404	338	282	206	142	0	0	0	0	0	2.36	6.51	191	11.73	59
#8	1.61	592	469	425	357	301	223	166	92	0	0	0	0	2.46	7.32	188	13.48	75
#9	2.04	625	494	448	378	320	240	188	110	0	0	0	0	2.56	8.24	183	15.37	92
#10	2.59	668	526	477	404	344	261	205	130	89	0	0	0	2.67	9.47	177	17.77	114
#11	3.18	714	558	506	428	365	274	216	147	102	0	0	0	2.78	11.00	164	19.91	133
#14	4.59	823	636	577	489	420	317	251	177	134	103	83	0	2.94	15.17	143	25.32	182
#7	2.44	656	509	457	380	318	236	183	112	80	0	0	0	2.34	7.82	188	16.24	110
#8	3.22	717	551	495	412	347	260	205	132	95	0	0	0	2.43	9.12	182	18.93	140
#9	4.08	783	598	536	448	378	286	226	153	110	86	0	0	2.50	10.66	175	21.78	166
#10	5.18	869	658	589	492	417	315	251	176	129	101	83	0	2.57	12.86	165	25.31	197
#11	6.36	961	718	641	533	451	338	268	187	144	113	93	79	2.63	16.24	142	28.24	222

SQUARE TIED COLUMNS 16" × 16"

Short columns; no sidesway [1] Bars symmetrical in 4 faces

Bars Grade 60 — Concrete f'_c = 4,000 psi

P_u (kips)—Ultimate Usable Capacity; $M_u/P_u = e$ (in.) ($\phi = 0.70$)

$0.10f'_cA_g$ = 102 kips [2]

Bars	p %	0	0.1t	2"	3"	4"	6"	8"	12"	16"	20"	24"	28"	OT [3] e (in.)	Balance e (in.)	Balance P_u (k)	102 k [2] e (in.)	O [3] M_u (k-ft.)
4-#8	1.23	734	586	553	476	408	307	235	126	0	0	0	0	2.63	7.45	257	13.78	91
4-#9	1.56	768	612	578	499	430	327	261	150	0	0	0	0	2.74	8.23	255	15.53	112
4-#10	1.98	810	644	609	527	456	351	283	177	118	0	0	0	2.87	9.23	252	17.79	138
4-#11	2.43	856	677	640	554	481	372	300	199	135	0	0	0	3.00	10.37	241	19.84	163
4-#14	3.51	966	758	717	623	543	425	346	246	179	135	108	0	3.21	13.25	226	25.15	225
4-#18	6.25	1243	958	907	789	691	547	444	318	248	203	170	142	3.54	23.04	178	37.76	367
8-#6	1.37	749	589	554	472	400	296	220	128	0	0	0	0	2.43	6.95	261	14.35	101
8-#7	1.87	799	626	589	503	429	322	253	153	107	0	0	0	2.53	7.84	259	16.66	133
8-#8	2.46	860	669	630	538	461	350	280	179	127	0	0	0	2.63	8.90	256	19.35	170
8-#9	3.12	926	717	674	577	496	379	305	207	147	114	0	0	2.72	10.09	253	22.10	209
8-#10	3.96	1012	778	732	627	540	415	337	240	172	134	109	0	2.82	11.65	248	25.57	255
8-#11	4.87	1104	840	789	675	581	448	364	259	193	150	123	104	2.91	13.67	231	28.52	288
8-#14	7.03	1322	993	931	795	685	531	435	310	240	194	159	135	3.04	18.98	205	36.46	376
12-#10	5.95	1213	922	865	739	636	492	399	289	222	175	144	123	2.88	15.01	238	33.40	344
12-#11	7.31	1351	1015	950	809	696	538	436	314	243	196	162	138	2.95	18.52	213	37.27	393

SQUARE TIED COLUMNS 18" × 18"

$0.10f'_c A_g$ = 129 kips

Bars	p %	0	0.1t	2"	3"	4"	6"	8"	12"	16"	20"	24"	28"	OT e (in.)	Balance e (in.)	Balance P_u (k)	129 k e (in.)	O M_u (k-ft.)
4-#9	1.23	929	744	726	637	557	430	344	199	0	0	0	0	2.89	8.39	330	15.75	131
4-#10	1.56	972	777	758	667	586	457	369	233	152	0	0	0	3.03	9.30	328	17.87	162
4-#11	1.92	1018	811	791	697	613	481	391	262	175	0	0	0	3.18	10.24	322	19.89	193
4-#14	2.77	1128	893	872	771	681	541	445	326	230	171	136	0	3.43	12.55	314	24.96	267
4-#18	4.93	1405	1098	1072	951	844	680	565	419	329	267	215	179	3.85	19.28	279	37.13	442
8-#6	1.08	910	721	702	610	526	395	298	172	0	0	0	0	2.59	7.20	338	14.64	118
8-#7	1.48	961	758	738	643	557	424	336	203	140	0	0	0	2.69	8.00	336	17.10	156
8-#8	1.95	1021	802	781	681	593	456	366	236	165	0	0	0	2.80	8.95	334	19.62	200
8-#9	2.46	1088	850	828	723	631	489	396	270	191	147	0	0	2.91	10.00	331	22.37	247
8-#10	3.13	1173	912	888	776	679	531	433	308	222	171	139	0	3.02	11.36	327	25.66	306
8-#11	3.85	1265	976	950	829	725	568	465	338	249	193	157	133	3.14	12.83	319	28.62	361
8-#14	5.55	1484	1132	1101	961	841	664	544	401	315	247	203	171	3.32	16.52	306	36.33	470
12-#10	4.70	1375	1059	1030	898	786	618	504	368	282	223	183	156	3.12	14.12	321	33.43	416
12-#11	5.77	1513	1154	1122	976	853	671	549	401	316	251	206	175	3.23	16.39	309	37.48	485
16-#10	6.27	1576	1208	1175	1023	896	706	579	425	335	270	224	191	3.23	16.18	321	40.78	527

(1) See "Slender Columns, Capacity Reduction for", page 2-11.
(2) See "Control Points for Interaction Curves"; "Typical Interaction Curve", Fig. 3-11, page 3-18.
(3) "OT" is zero tension in bars on the tension side. Splices carry design compression only.

RESUMEN AUTOBIOGRAFICO

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Candidato para el Grado de
Maestro en Ciencias con Especialidad en
Ingeniería Estructural

Tesis: SOLUCIÓN ANALÍTICA DEL MODELO MATEMÁTICO DE COLUMNAS CON SECCIONES GEOMÉTRICAS ARBITRARIAS SOMETIDAS A LA COMBINACIÓN DE FLEXIÓN BIAXIAL Y CARGA AXIAL

Campo de Estudio: Ingeniería Estructural

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