DESIGN OF SINGLE COLUMN BENTS FOR 2-SPAN BOX GIRDER BRIDGES

Single column, two-span prestressed box girder bridge columns, which meet the following requirements, should be designed using the attached charts. If these requirements are met, it is not necessary to perform a STRUBAG, STRUDL, SEISAB or simplified analysis to determine the column reinforcement. The designer is cautioned to review the limitations to assure that the charts apply. Column sizes may be approximately round with a cross-sectional area of 20 sq. ft. (5 ft. dia.) or 28 sq. ft. (6 ft. dia.). Intermediate cross-sectional areas may be used by interpolating between the two charts. Columns may be prismatic or nonprismatic with a single diameter spiral core.

Limitations

Depth to span ratio	0.035 to 0.045
Column height	18 to 26 feet clear
Column area at base	20 to 28 sq. ft.(5 to 6 ft. dia.)
Span length	100 to 240 feet
Maximum skew	35 degrees
Maximum unbalance in spans	20%
Minimum radius of centerline	1000 ft.
Bridge width	32 to 46 feet
Column end conditions	"fixed" top and bottom
Column concrete	$f'_c = 3250$ psi
Column reinforcement	$f_y = 60{,}000$ psi

Column must be centered under cap

After the column reinforcement has been determined from the charts, run the YIELD program in check mode for Group I loads only. Column shear reinforcement shall be determined from the plastic load case. The footing is then designed for only the Group I and plastic load cases.

An example problem showing the design of a typical single column bent is attached.

Discussion

The load conditions which governed the design of the columns were P-loads and seismic.

Seismic loads generally governed the 6 foot diameter column designs, except the P-loads governed the design of narrow width structures in the 0.5 – 0.6g seismic level.

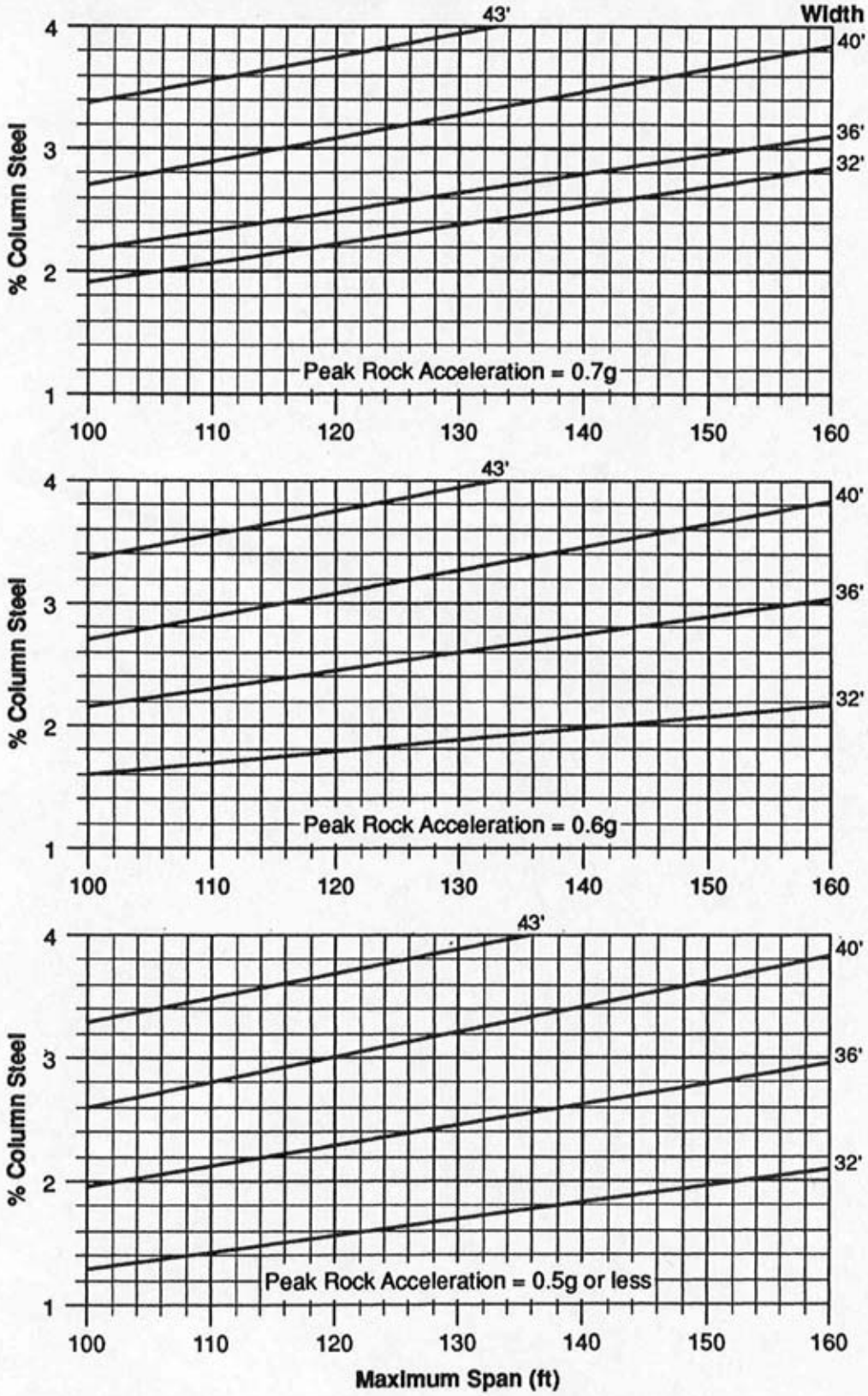
The design of the 5 foot diameter column was generally governed by P-loads, except for the narrow structures, which were governed by seismic loads. Note that the 5 foot diameter column generally requires a much larger percentage of steel than the equivalent 6 foot diameter column. The use of the 5 foot diameter column should be restricted to narrow structures in the low span length range.

Standard abutment and wingwall details are assumed (refer to applicable drawings). It is assumed that under the maximum credible earthquake the abutments will be damaged and undergo some movements. In these charts, the abutments were considered released in the transverse direction and soil support assumed in the longitudinal direction, therefore it is important that standard abutment heights and berms be used to preclude a collapse condition.

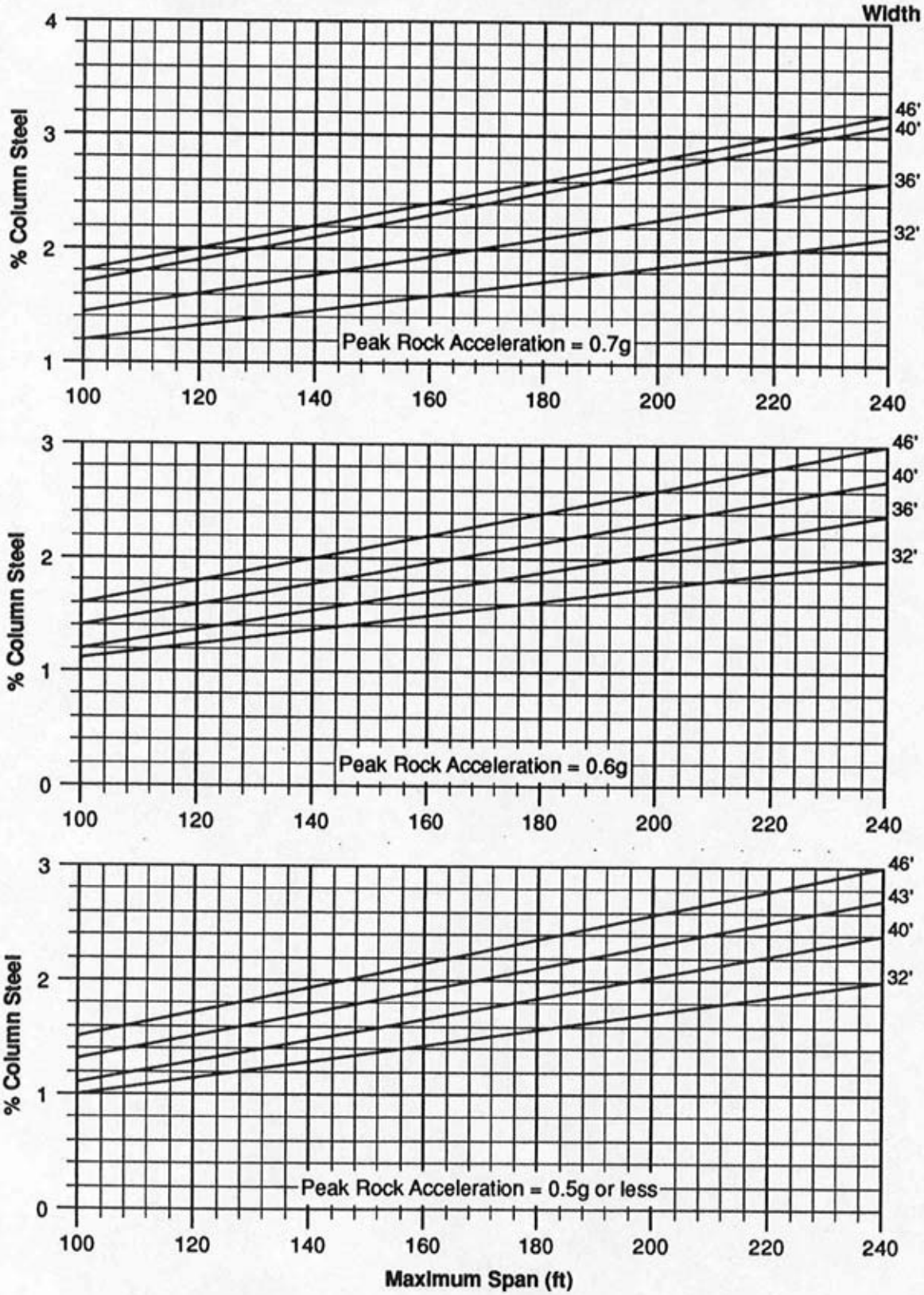
Abutment seat and key sizes shall be determined as outlined in Memo to Designers, Section 5.

It is hoped that this series of charts will be expanded to include other common bridge configurations. Please direct any comments or suggestions to J. Gates in the SASA section.

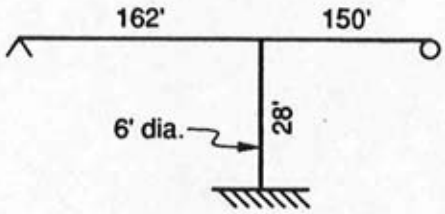
Column Reinforcement for Prestressed Box Girders
2 Spans, 1 Column Bent – 5' Diameter Column (20 Sq. Ft.)

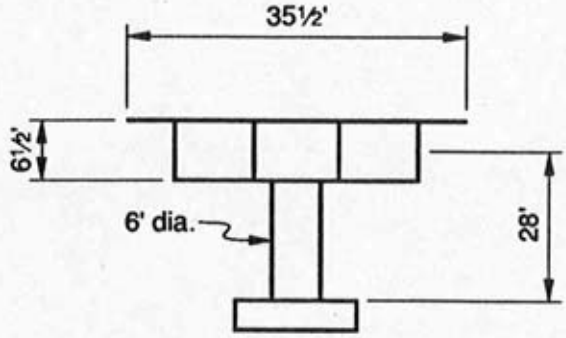


Column Reinforcement for Prestressed Box Girders
2 Spans, 1 Column Bent – 6' Diameter Column (28 Sq. Ft.)



2 Span – 1 Column
Example Column/Footing Design (using STD charts)





"A": Rock Acceleration = 0.6g

Verify Applicability of Charts

ITEM	ALLOWANCE	ACTUAL		
D/S	0.035 – 0.045	0.040	OK	✓
Column Clearance Height	18' – 26'	24.8'	OK	✓
Column Area	20 – 28 ft.^{2}	28.3 ft.^{2}	OK	✓
Maximum Span	100' – 240'	162.0'	OK	✓
Span Unbalance	0.20	0.08	OK	✓
Width	32' – 46'	35.5'	OK	✓
Alignment	1000' R	Straight	OK	✓
Skew	35°	0°	OK	✓

Determine Column Reinforcement from Charts

$$\left.\begin{array}{r} A = 0.6g \\ W = 35.5' \\ \text{Span} = 162' \\ \text{Column Area} = 28\ \text{ft.}^2 \end{array}\right\} = 1.7\%,\ A_s = 28.3\ (144)\ 0.017 = 69.3\ \text{in.}^2$$

Select Bar and Number	Approximate Spacing
#11 tot 44	4.8"
#14 tot 30	6.8" ← use
#18 tot 17	12.3"

Group Loads (Bottom of Column)

DL (From BDS)
(Bottom of Column) M_y(Trans) = 0
M_x(Long) = 57' k
P(Axial) = 1979' k

$$\text{Live Load Lanes on Bent} = \frac{35.5 - 3.5}{12} = 2.67 \quad \text{use 2 lanes}$$

BDS Output

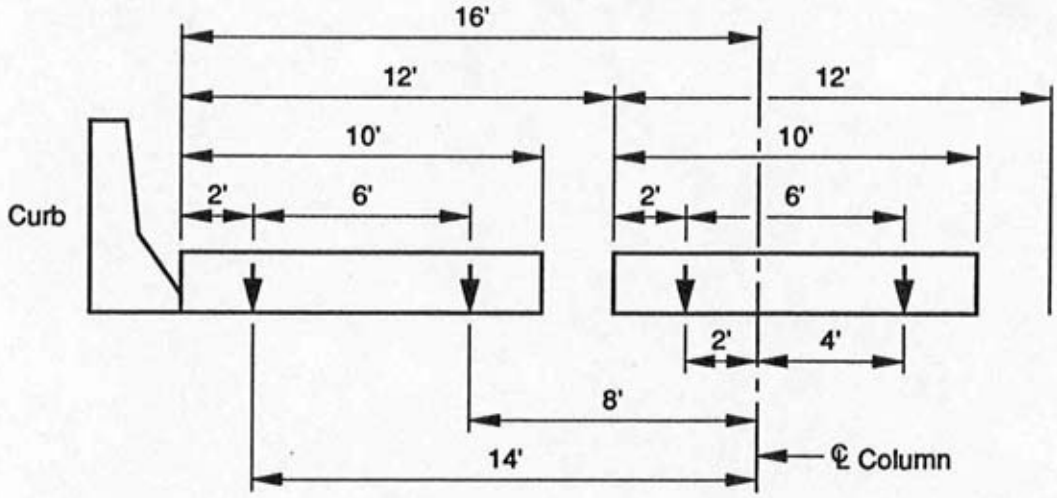
LL No. 1 (H Loads) Live Load Support Results (1 Lane — LL + I)

	Maximum Axial Load			Maximum Longitudinal Moment		
		Moment			Moment	
	Axial Load	Top	Bottom	Axial Load	Top	Bottom
Support Jt. 1						
Positive	80.2	0.0	0.0	0.0	0.0	0.0
Negative	-5.8	0.0	0.0	0.0	0.0	0.0
Member 3						
Positive	179.3	132.0	-66.0	94.8	1241.0	-620.0
Negative	0.0	0.0	0.0	87.8	-1092.0	546.0
Support Jt. 3						
Positive	77.9	0.0	0.0	0.0	0.0	0.0
Negative	-7.7	0.0	0.0	0.0	0.0	0.0

LL No. 4 (P Loads) Live Load Support Results (1 Lane — LL + I)

	Maximum Axial Load			Maximum Longitudinal Moment		
		Moment			Moment	
	Axial Load	Top	Bottom	Axial Load	Top	Bottom
Support Jt. 1						
Positive	222.0	0.0	0.0	0.0	0.0	0.0
Negative	-16.1	0.0	0.0	0.0	0.0	0.0
Member 3						
Positive	348.1	340.0	-170.0	260.2	3692.0	-1846.0
Negative	0.0	0.0	0.0	250.5	-3319.0	1660.0
Support Jt. 3						
Positive	214.5	0.0	0.0	0.0	0.0	0.0
Negative	-7.7	0.0	0.0	0.0	0.0	0.0

Live Loads



Live Load Position on Cap

H Loads – Case 1 (1 Lane)	(Maximum Transverse)	$M_y = 179.3(8 + 14)/2$	= 1972' k
		M_x	= 66' k
		P	= 179' k
H Loads – Case 2 (2 Lanes)	(Maximum Longitudinal)	$M_y = 94.8(8 + 14 + 2 - 4)/2$	= 948' k
		$M_x = 620(2)$	= 1240' k
		$P = 94.8(2)$	= 190' k
H Loads – Case 3 (2 Lanes)	(Maximum Axial)	$M_y = 179.3(8 + 14 + 2 - 4)/2$	= 1793' k
		$M_x = 66.0(2)$	= 132' k
		$P = 179.3(2)$	= 359' k
P Loads – Case 1 (1P)	(Maximum Transverse)	$M_y = 348.1(1.15)(8 + 14)/2$	= 4403' k
		$M_x = 170.0(1.15)$	= 196' k
		$P = 348.1(1.15)$	= 401' k
P Loads – Case 2 (1P + 1H Lane)	(Maximum Longitudinal)	$M_y = 260.2(1.15)(8 + 14)/2 + 94.8(2-4)/2$	= 3197' k
		$M_x = 1846(1.15) + 620$	= 2743' k
		$P = 260.2(1.15) + 94.8$	= 394' k
P Loads – Case 3 (Same as P Loads – Case 1 plus 1H Load)	(Maximum Axial)	$M_y = 348.1(1.15)(8 + 14)/2 + 179.3(2 - 4)/2$	= 4223' k
		$M_x = 170.0(1.15) + 66$	= 262' k
		$P = 348.1(1.15) + 179.3$	= 580' k

(Note: These numbers may also be obtained from a BENT analysis run.)

Use YIELD program to:

(a) Determine Column Shear Reinforcement
(b) Generate Footing Loads

YIELD Input	72" Diameter Column Check Mode Generate Footing Loads Out-Out Spiral = 72" - 4" = 68" (#6) (#14) Use #14 Tot 30 Loop Radius = 68/2 - 0.875 - 1.875/2 = 32.2" Impact 17.5% from BDS Clear Length = 24.8'

YIELD Output – Longitudinal Plastic Case Governs

Maximum Center-to-Center Spacing (Pitch) of Lateral

Bar Size	Case A Confinement	Case B Applicable Shear	Case C Minimum Shear
4	2.613	1.290	7.059
5	4.042	1.999	10.941
6	5.722	2.838	15.529

Minimum Spacing → Bar Size	1.0" Aggregate Minimum Pitch (inches)	Shear Strength (kips)
4	2.000	913
5	2.125	1168
6	2.250	1444

Note: Always use 1" Aggregate

Use #6 @ 2¾" Pitch

Footing – Use loads from YIELD with FOOT program.

COLUMN INTERACTION GRAPHS FOR STANDARD ARCHITECTURAL SIZES

Column Type **Pages**

Round – 4'ø, 5'-6"ø and 7'ø 13-11

Octagonal – 4'ø, 5'-6'ø and 7'ø 13-14

Hexagonal – 4'ø, 5'-6"ø and 7'ø 13-17

Square – 4', 5'-6" and 7' 13-20

4'-0" × 6'-0" 13-24

5'-6" × 8'-3" 13-40

7'-10" × 10'-6" 13-52

The graphs may be used to determine the nominal strength of column as required to design substructures and foundations by the load factor design procedure.

Assumptions:

- Specified Concrete Compressive Strength $f'_c = 3250$ psi
- Specified Yield Strength of Reinforcement $f_y = 60.0$ ksi

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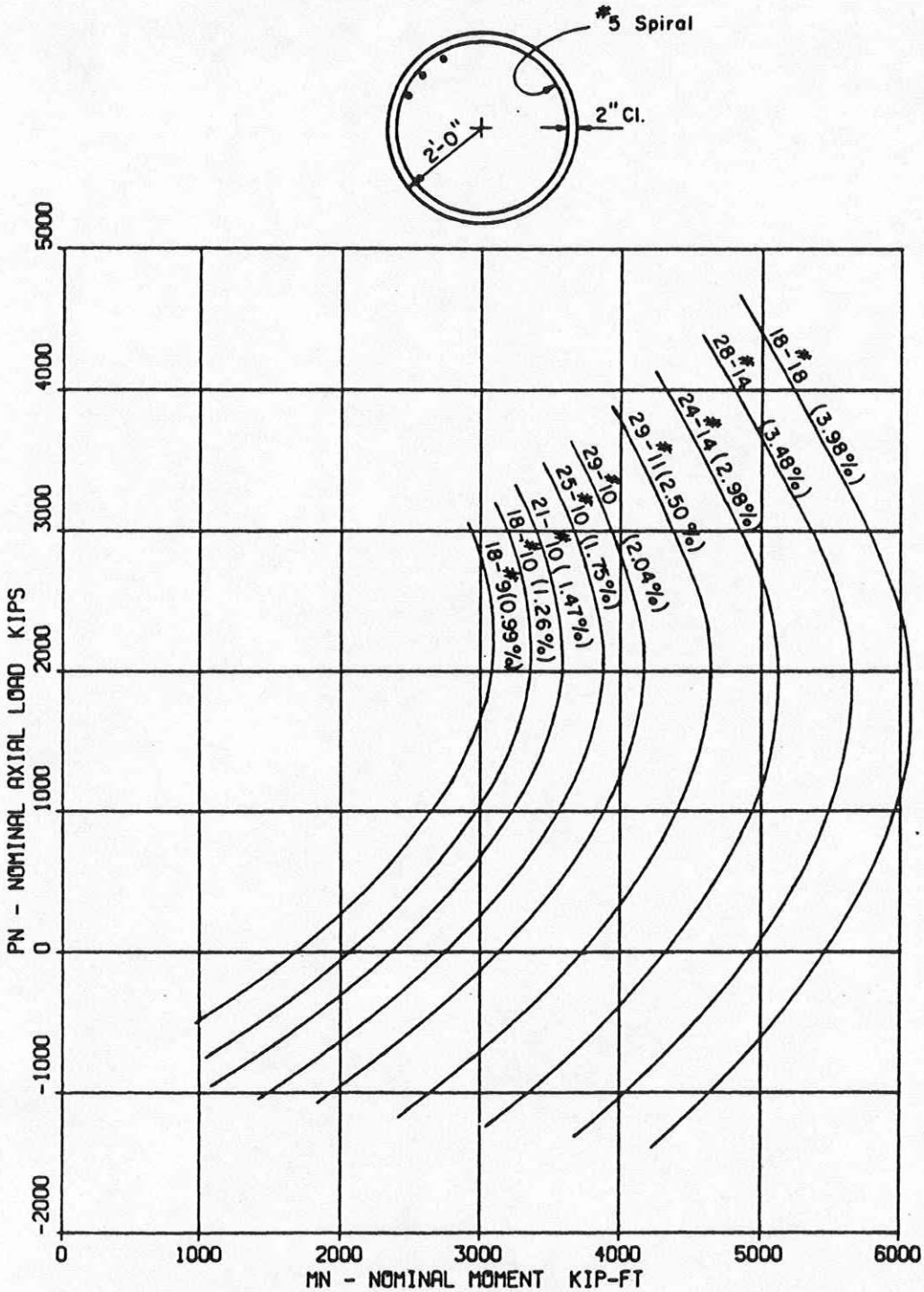
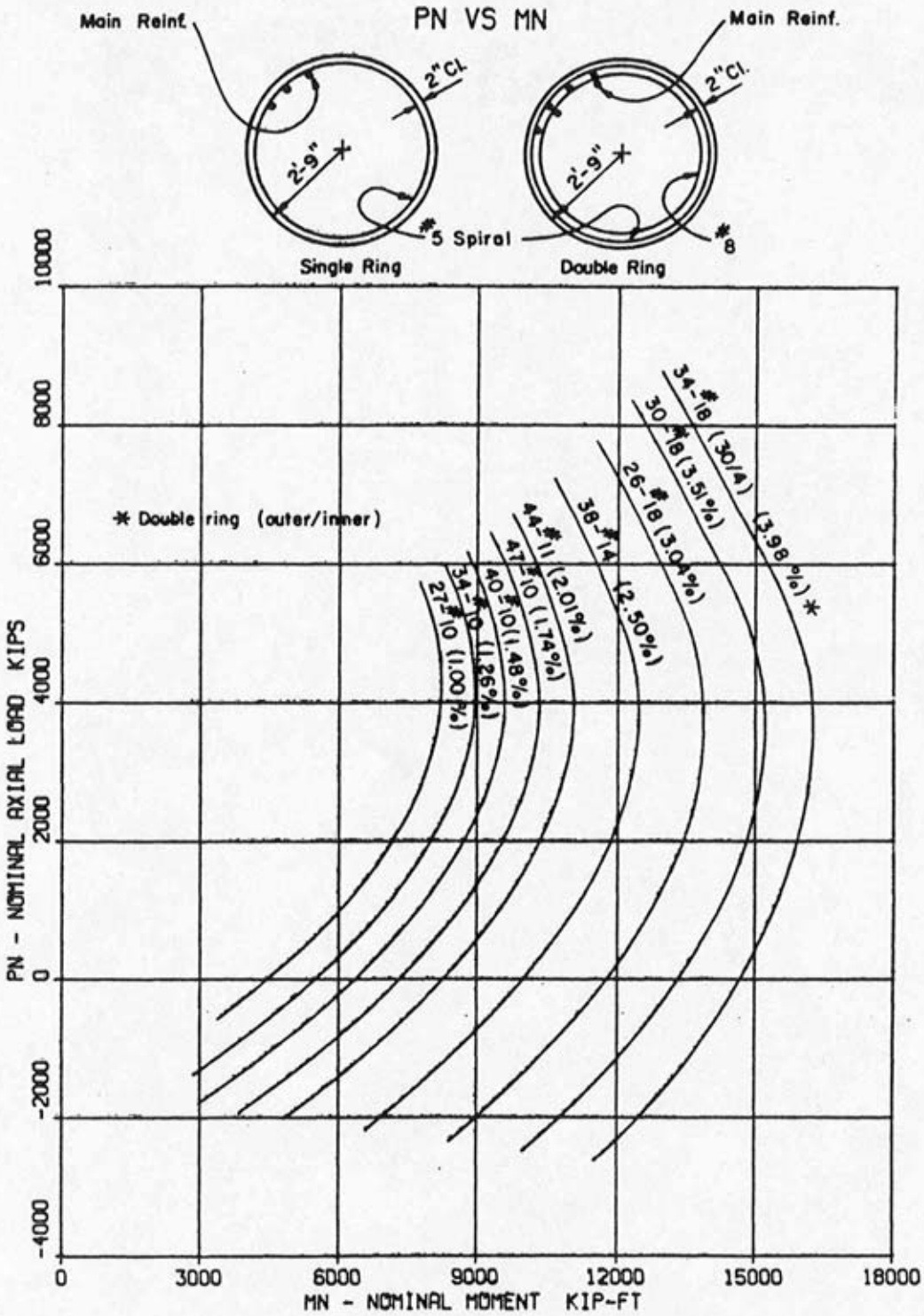
PN VS MN

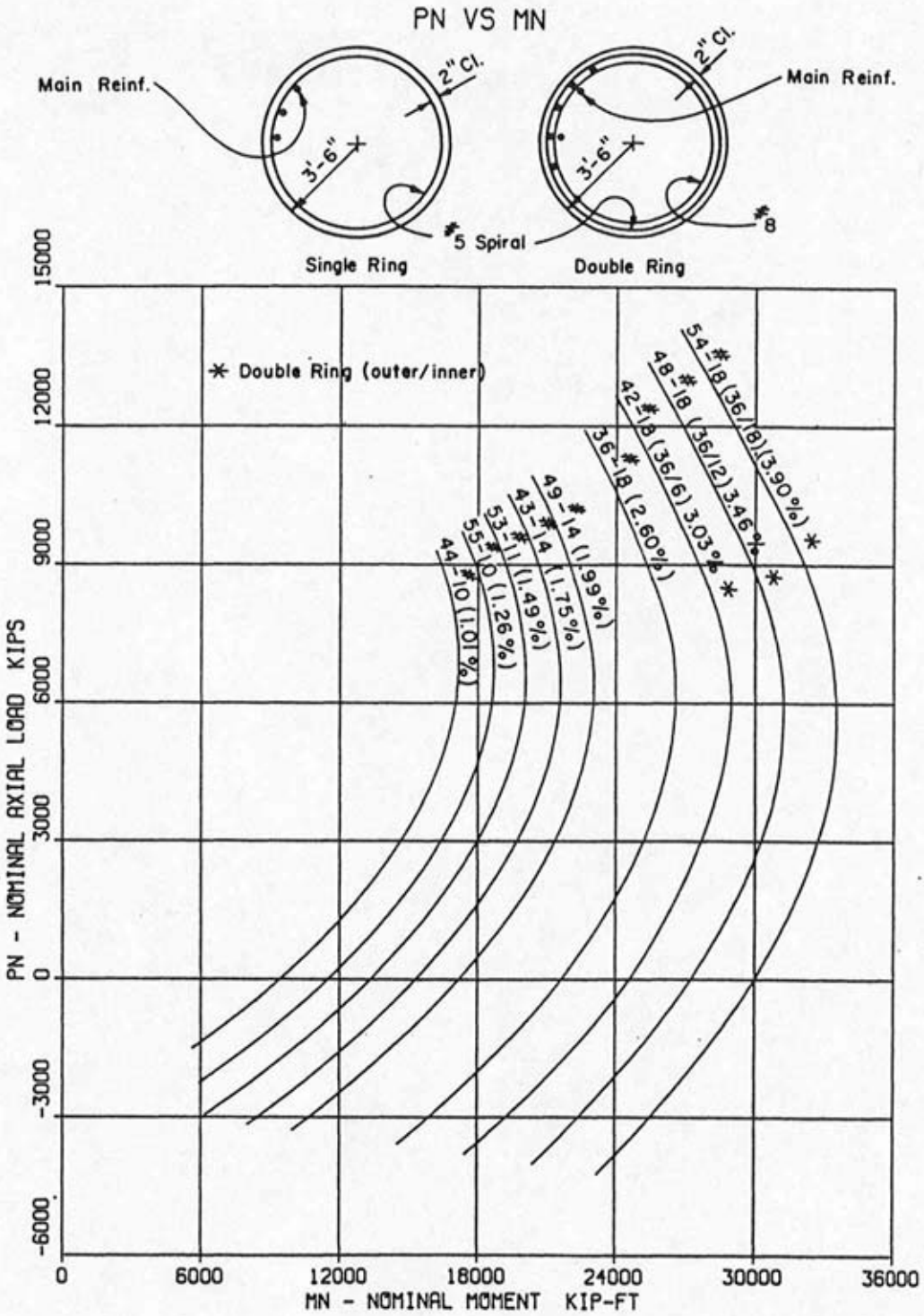


A = 12.6 ft^2
I = 12.6 ft^4
I_z = 25.1 ft^4

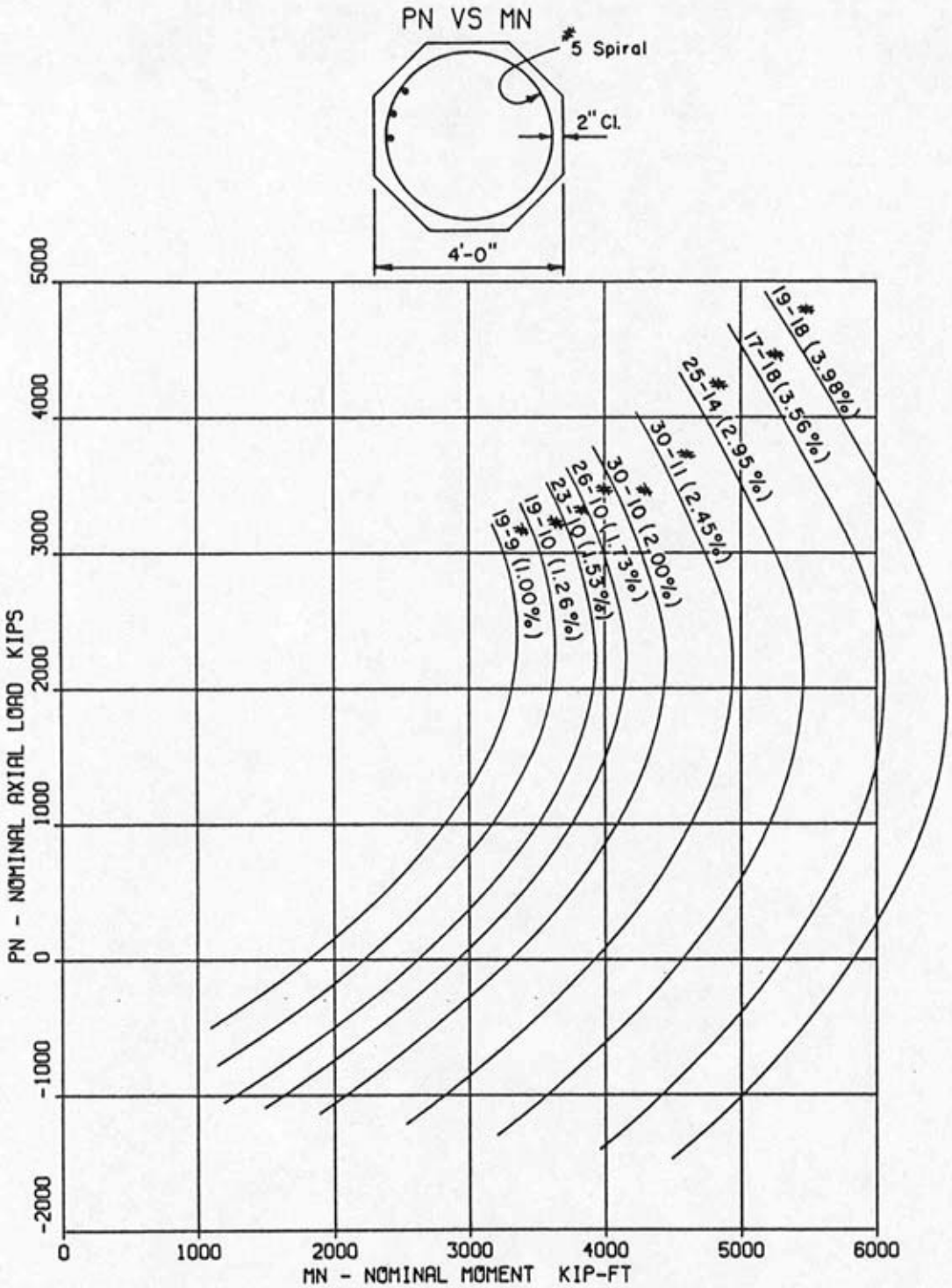
z = Local member longitudinal axis



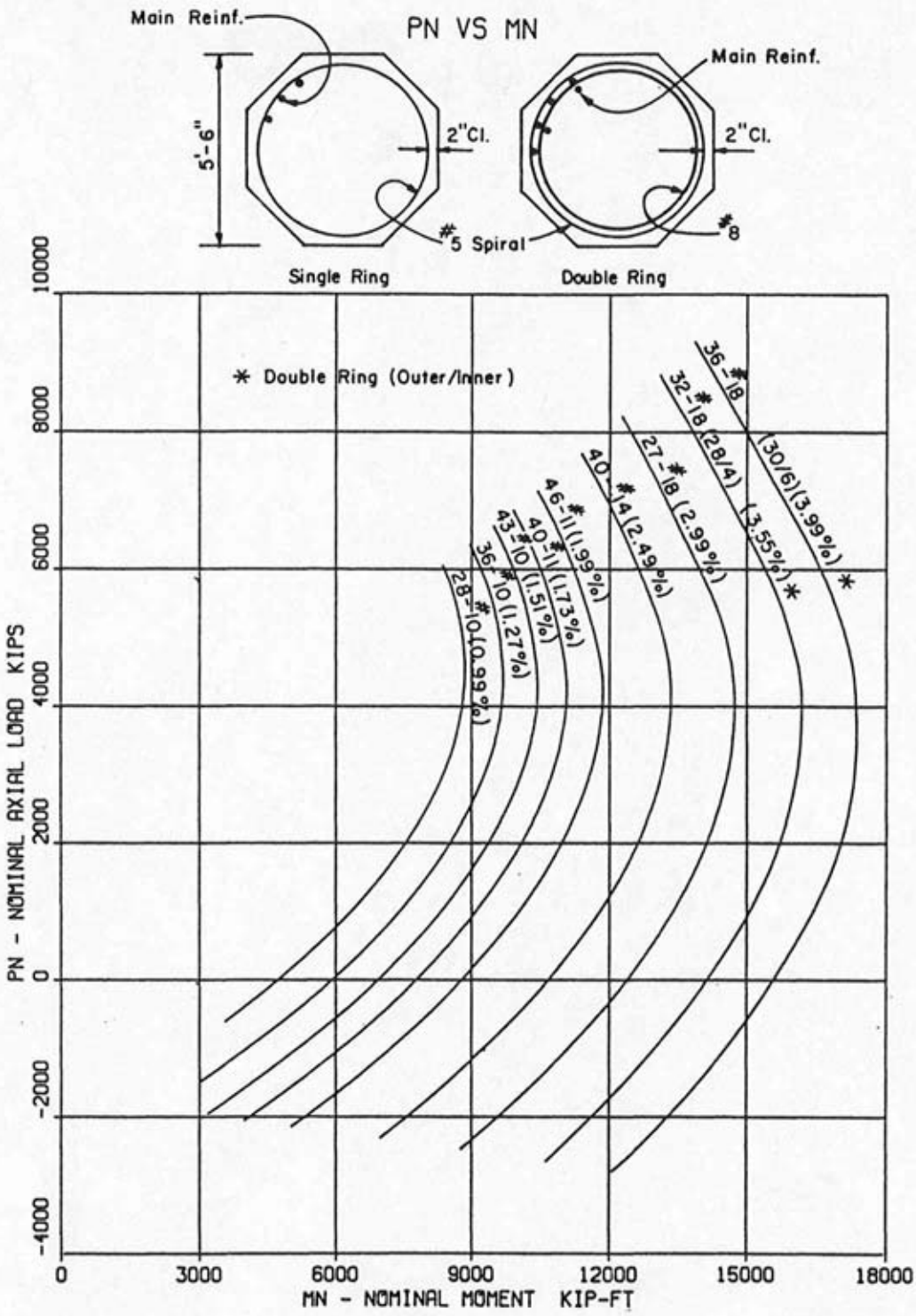
A = 23.8 ft²
I = 44.9 ft⁴
I_z = 89.8



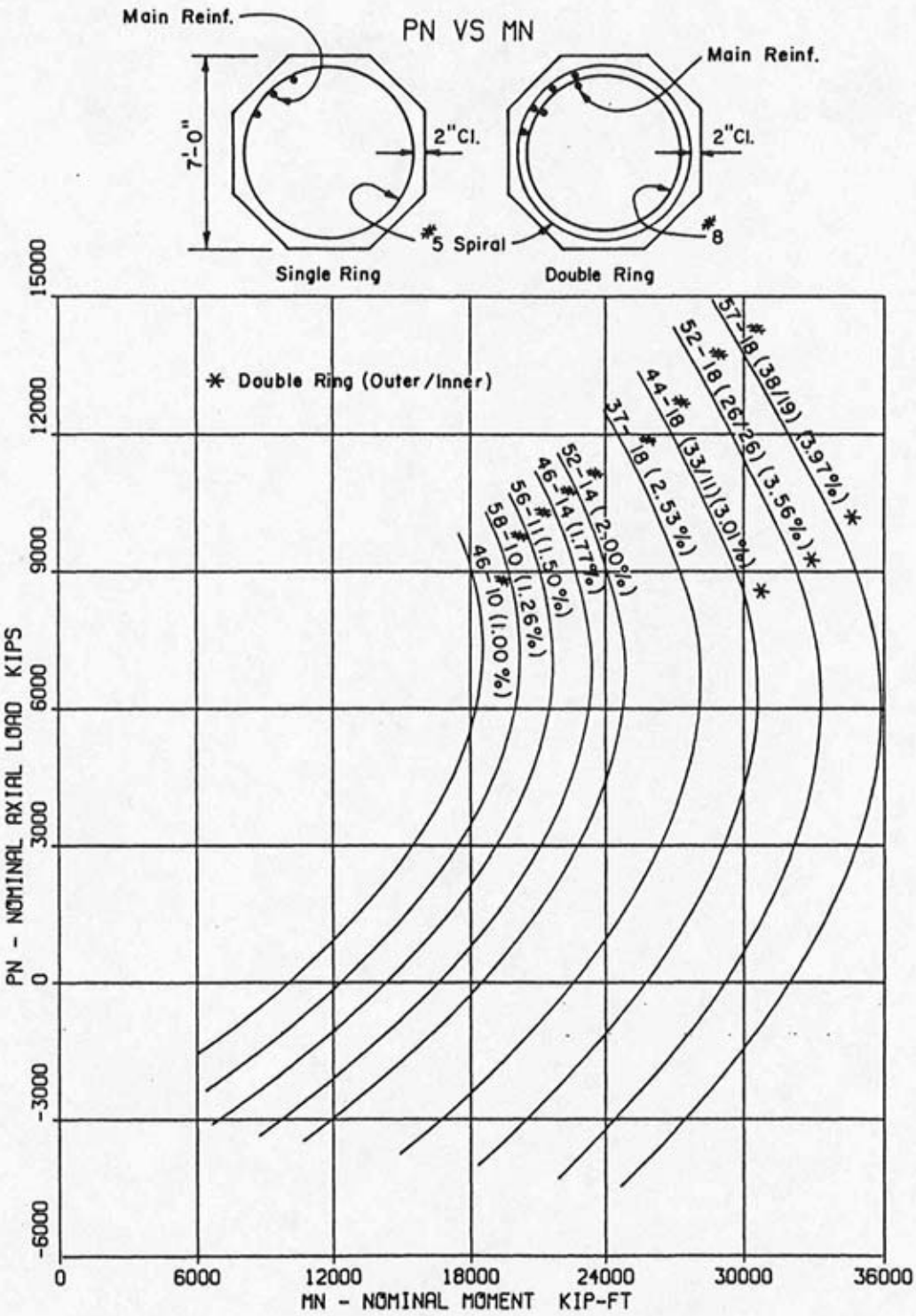
$A = 38.5\ ft^2$
$I = 117.8\ ft^4$
$I_z = 235.7\ ft^4$



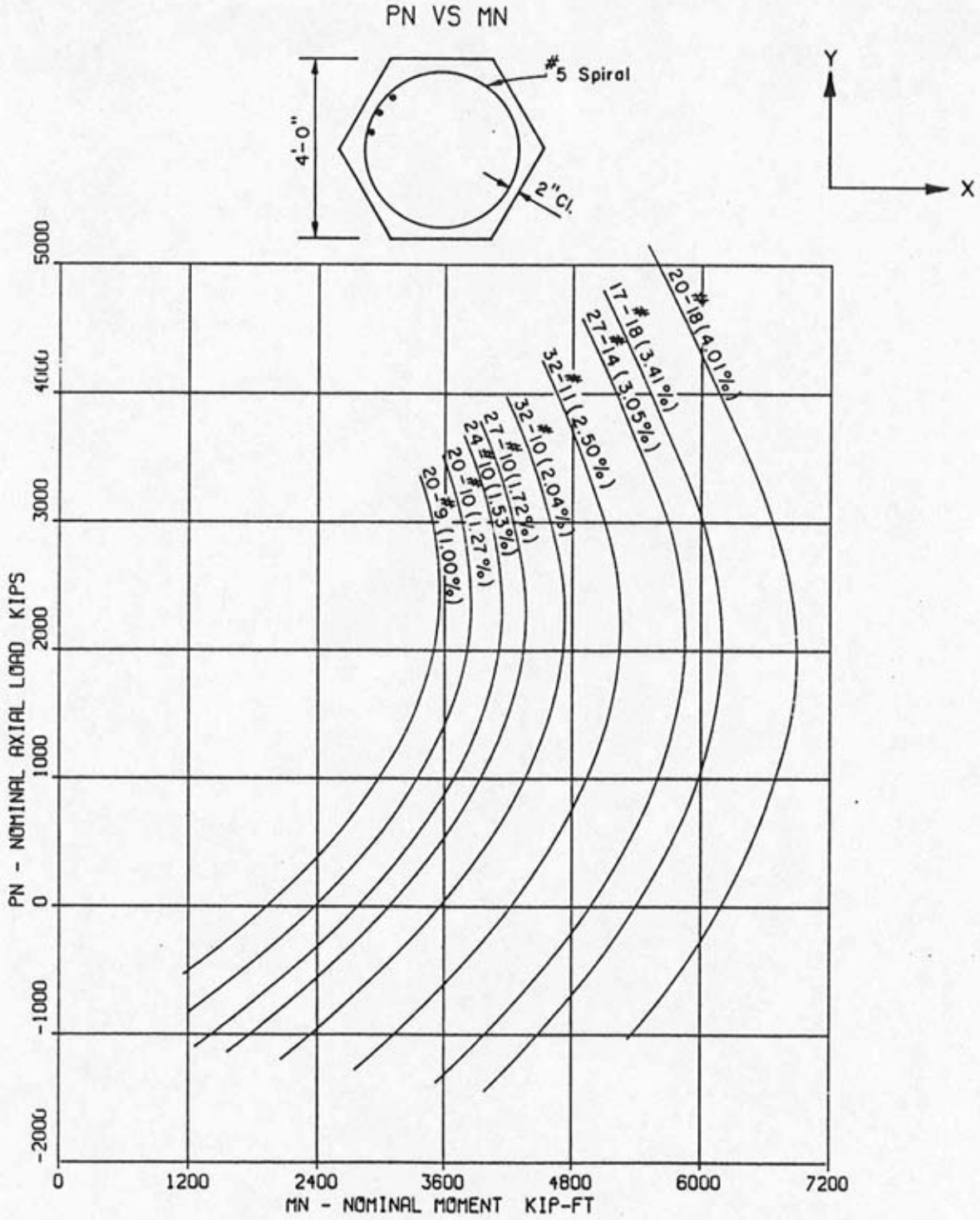
A = 13.3 ft²
I = 14.0 ft⁴
I_z = 28.1 ft⁴



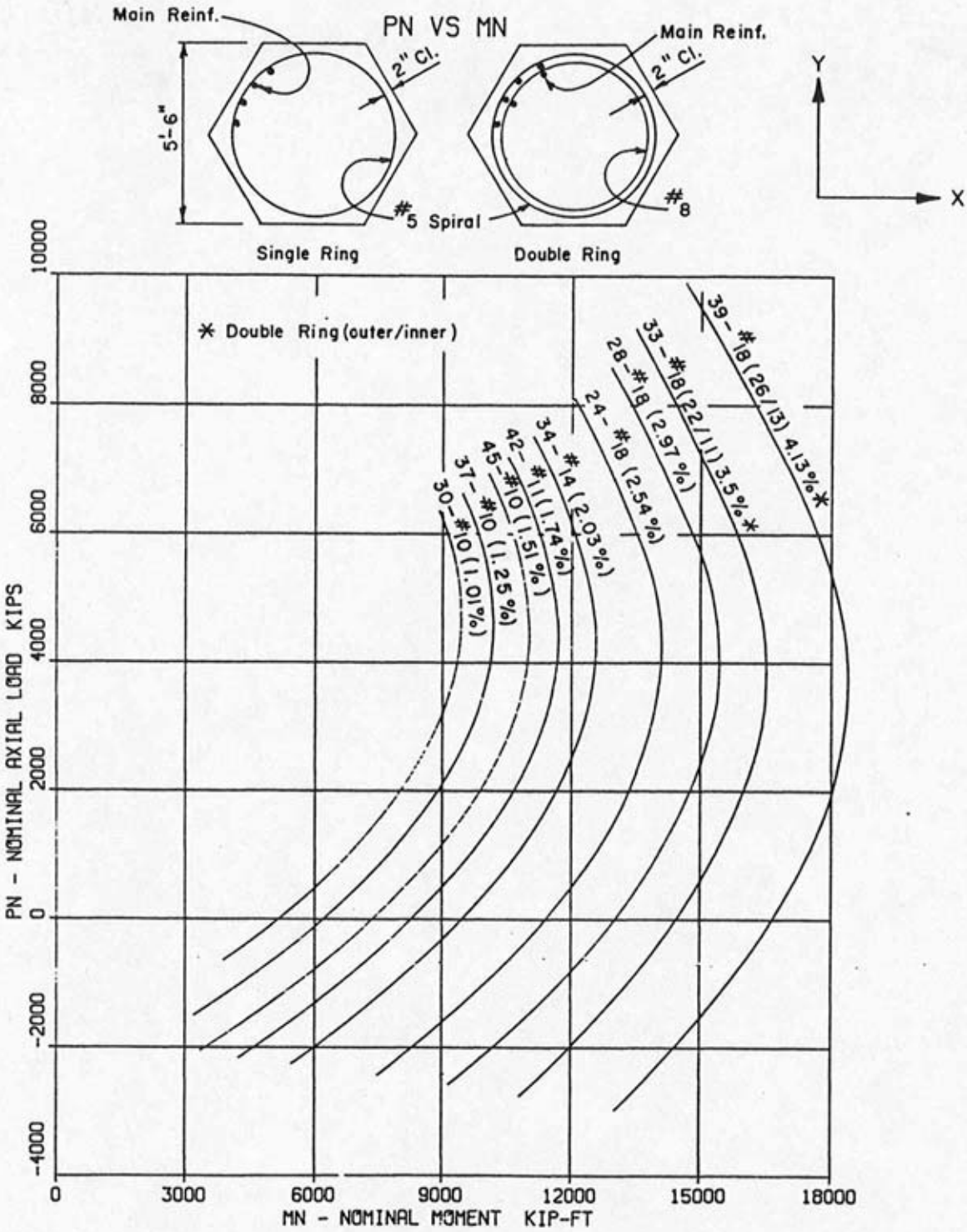
$A = 25.1\ ft^2$
$I = 50.1\ ft^4$
$I_z = 100.2\ ft^4$



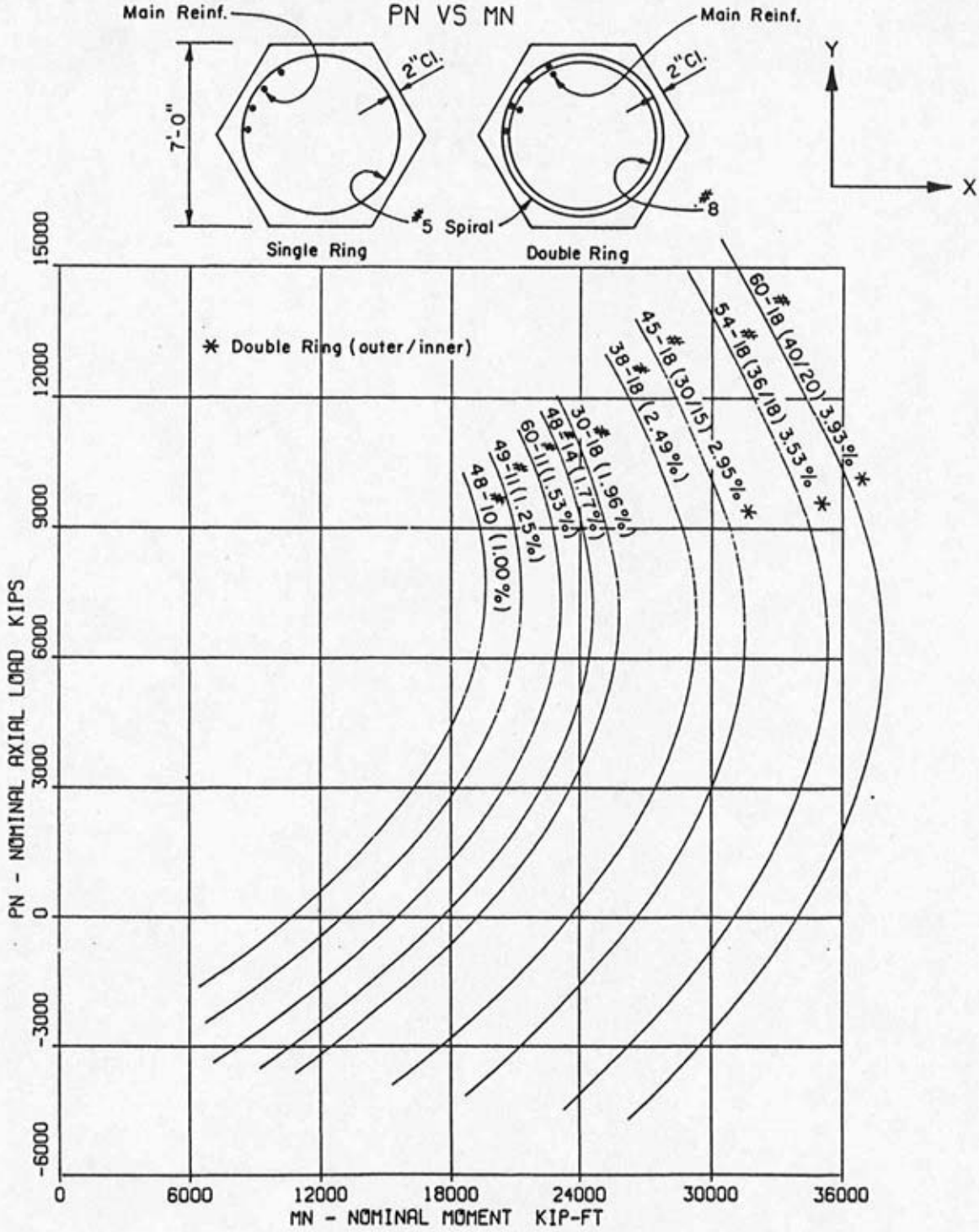
$A = 40.6\ ft^2$
$I = 131.4\ ft^4$
$I_z = 262.9\ ft^4$



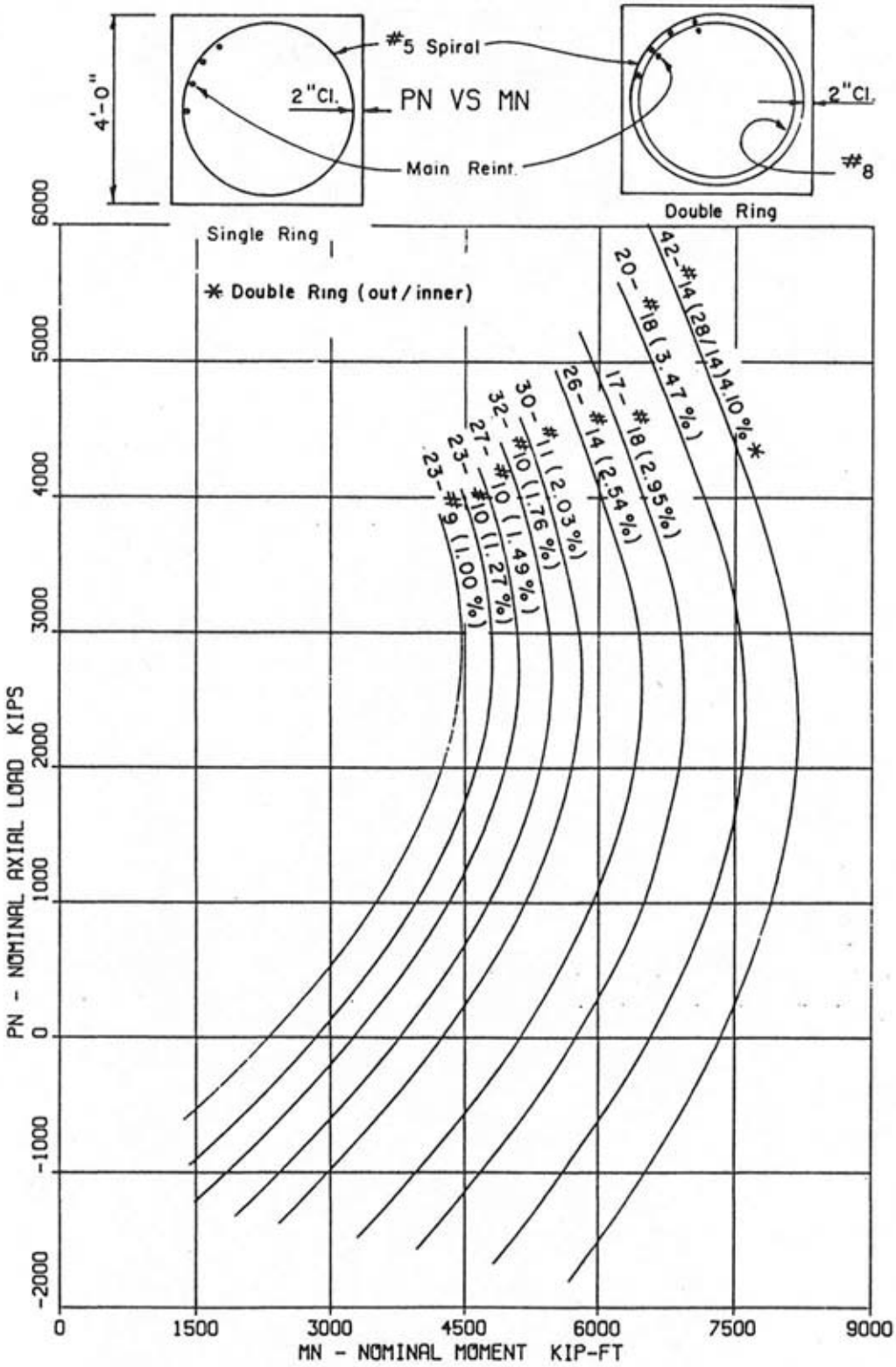
$A = 12.0\ ft^2$
$I_x = 13.3\ ft^4$
$I_y = 10.0\ ft^4$
$I_z = 23.3\ ft^4$



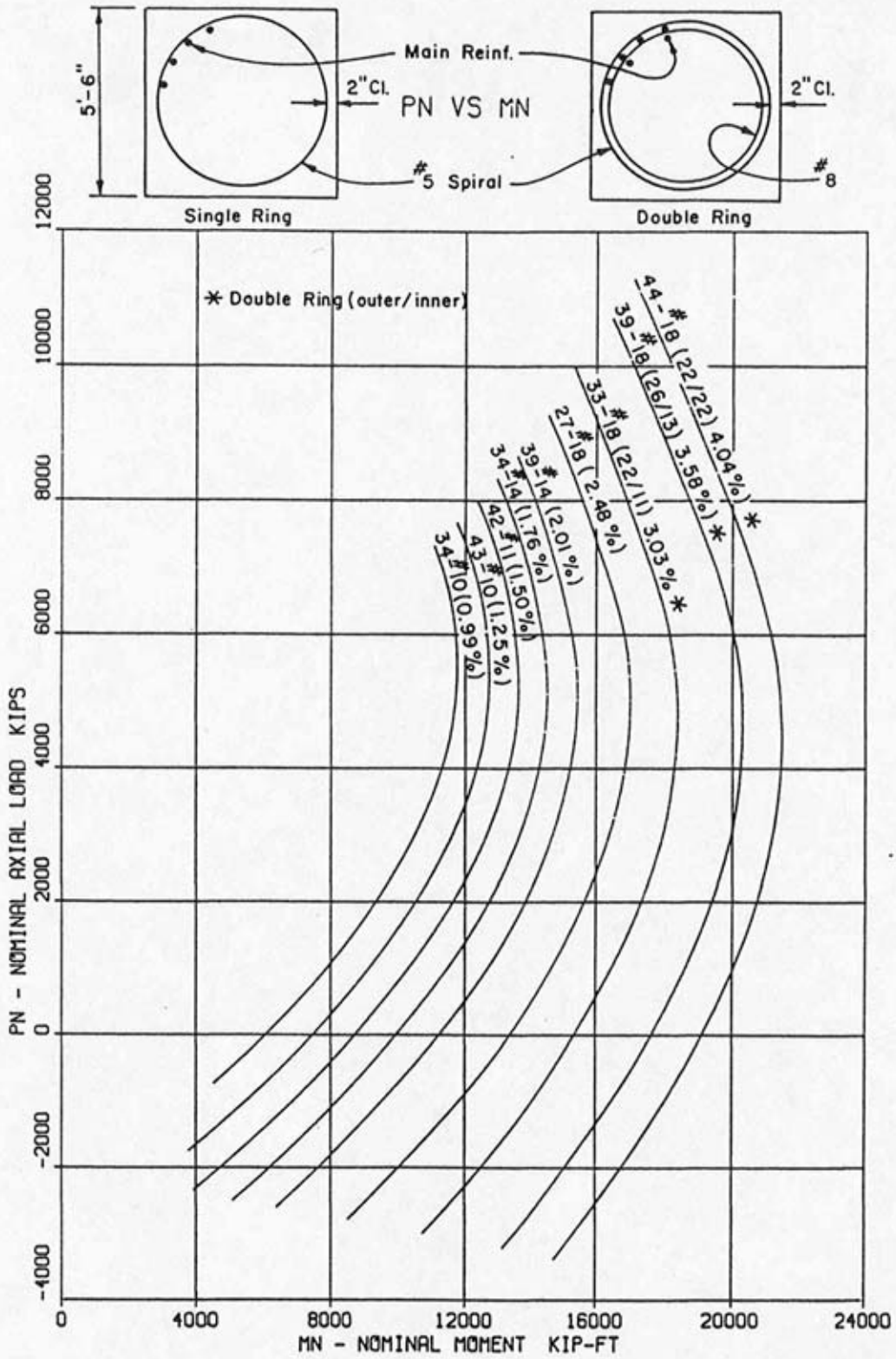
$A = 22.7\ ft^2$
$I_x = 47.7\ ft^4$
$I_y = 35.7\ ft^4$
$I_z = 83.4\ ft^4$



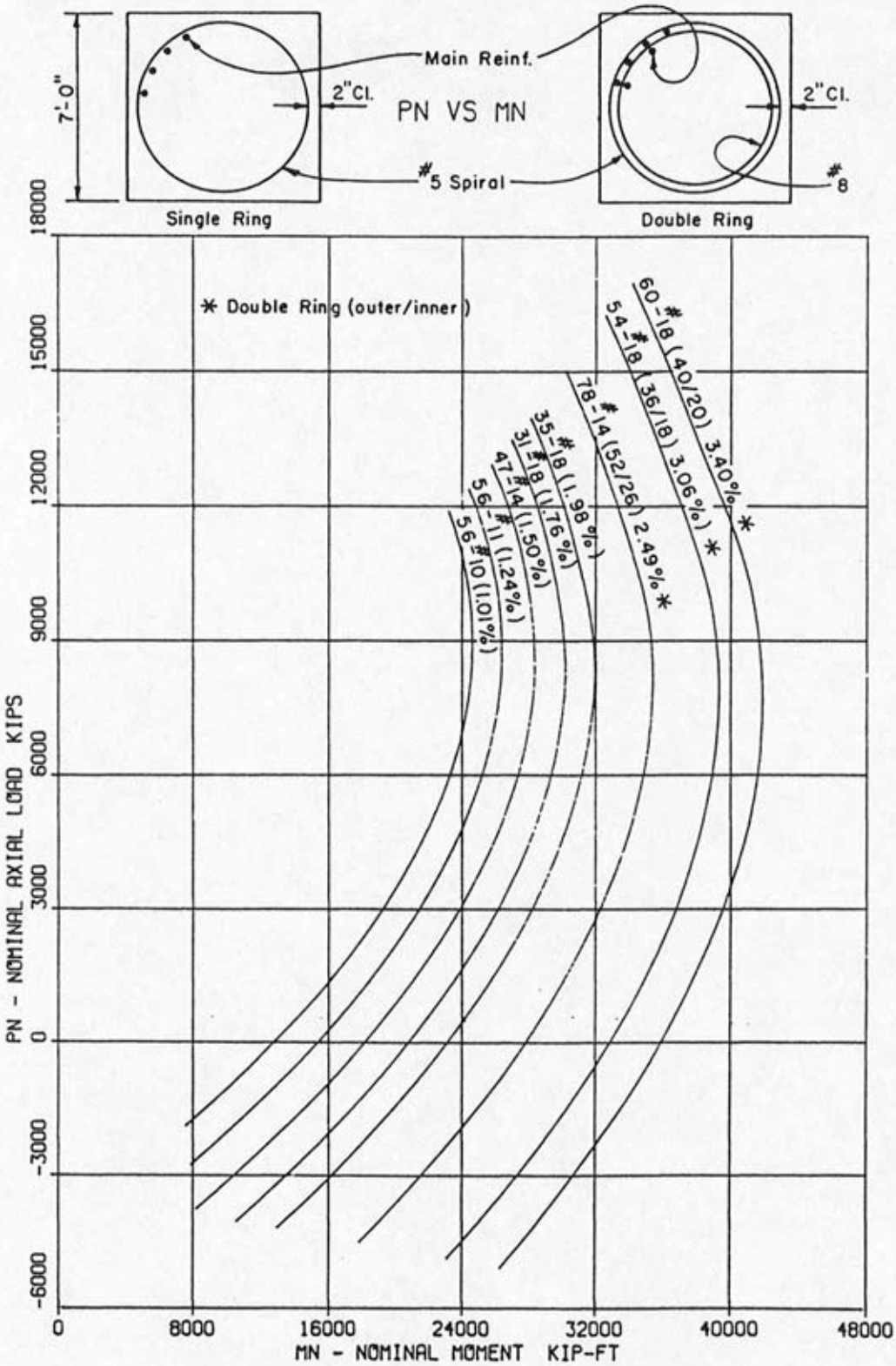
$A = 36.8\ ft^2$
$I_x = 125.1\ ft^4$
$I_y = 93.8\ ft^4$
$I_z = 218.8\ ft^4$



A = 16.0 ft²
I = 21.3 ft⁴
I_z = 42.7 ft⁴



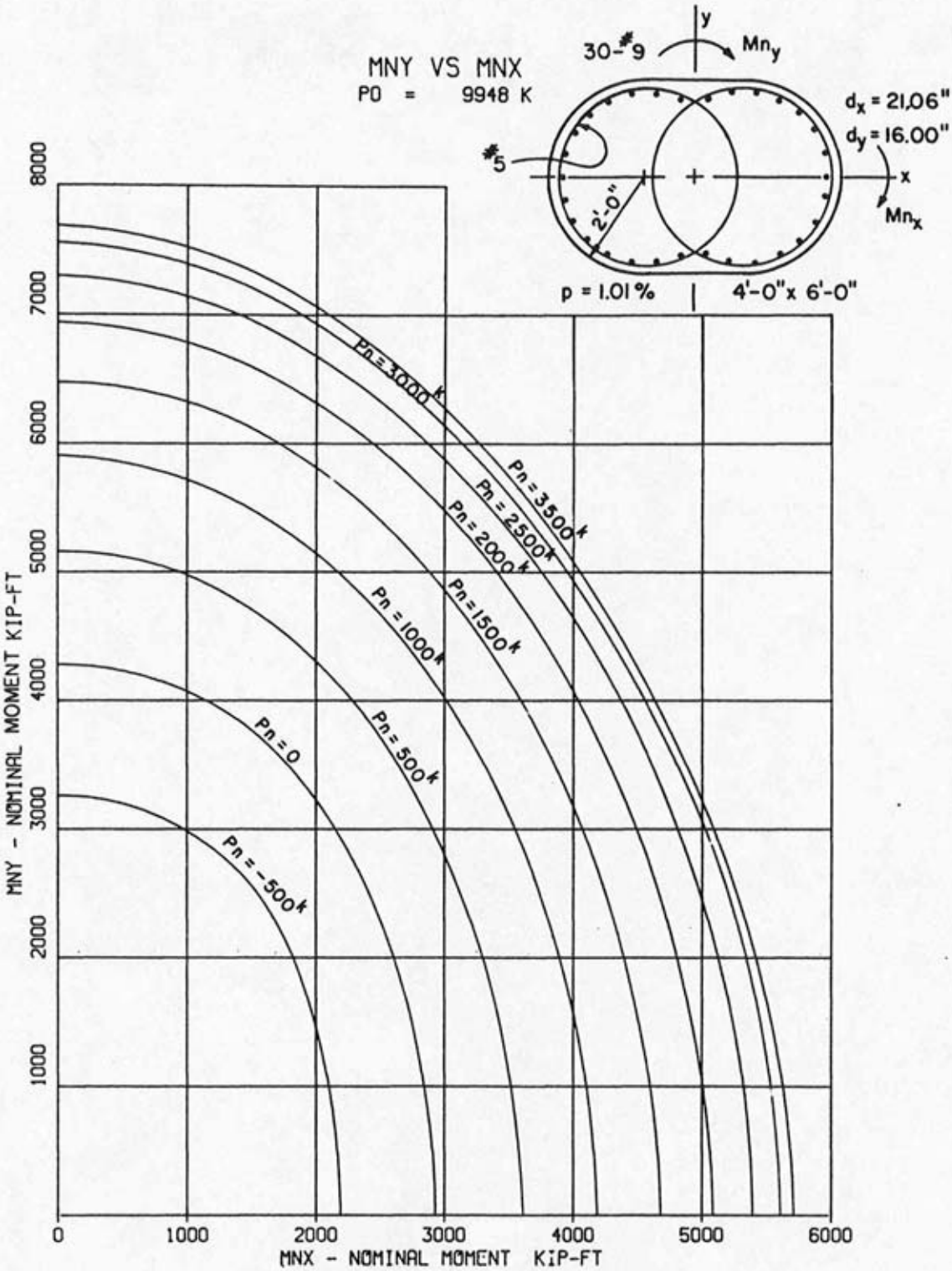
$A = 30.3\ ft^2$
$I = 76.3\ ft^4$
$I_z = 152.5\ ft^4$



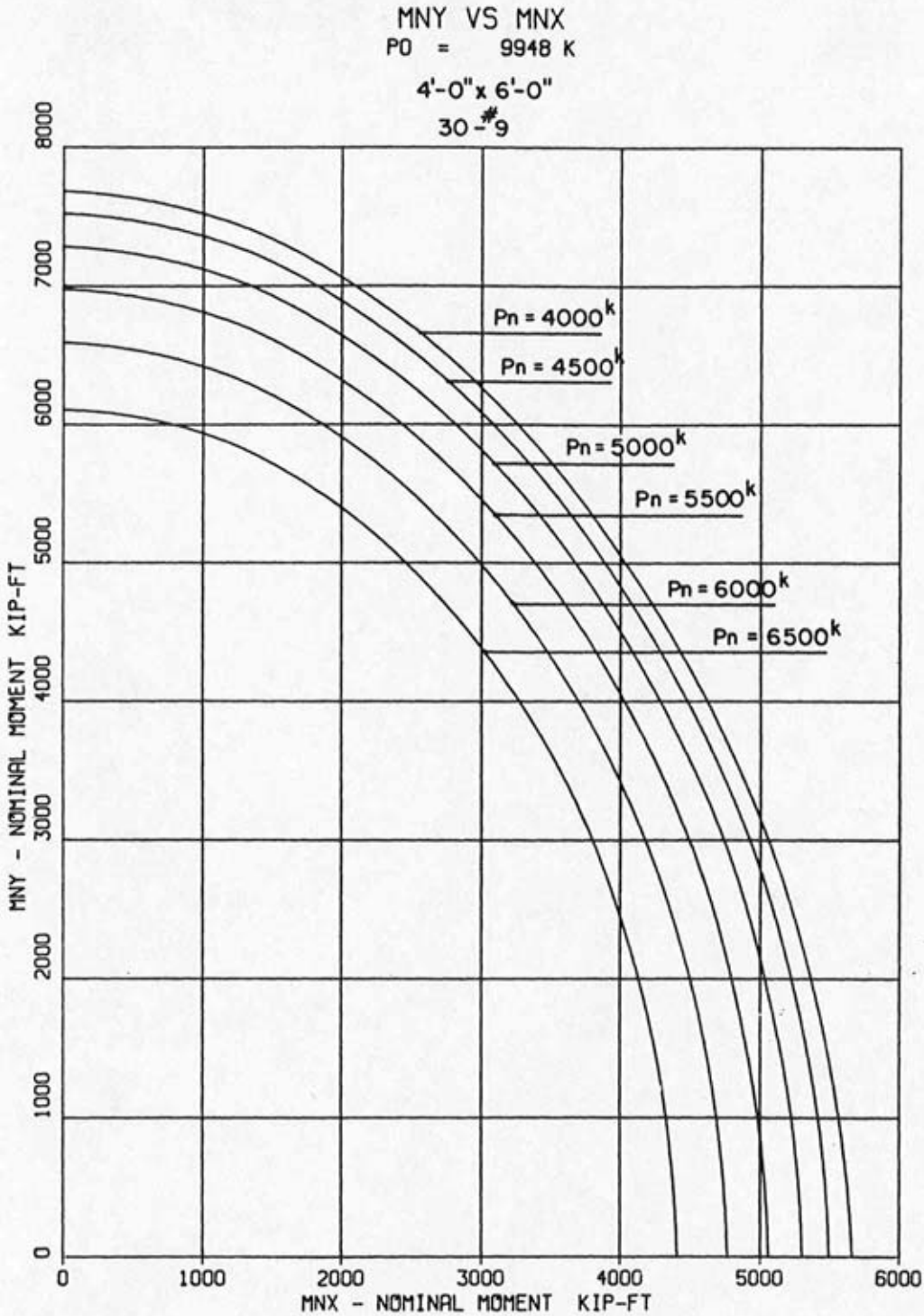
A = 49.0 ft²
I = 200.1 ft⁴
I_z = 400.2 ft⁴

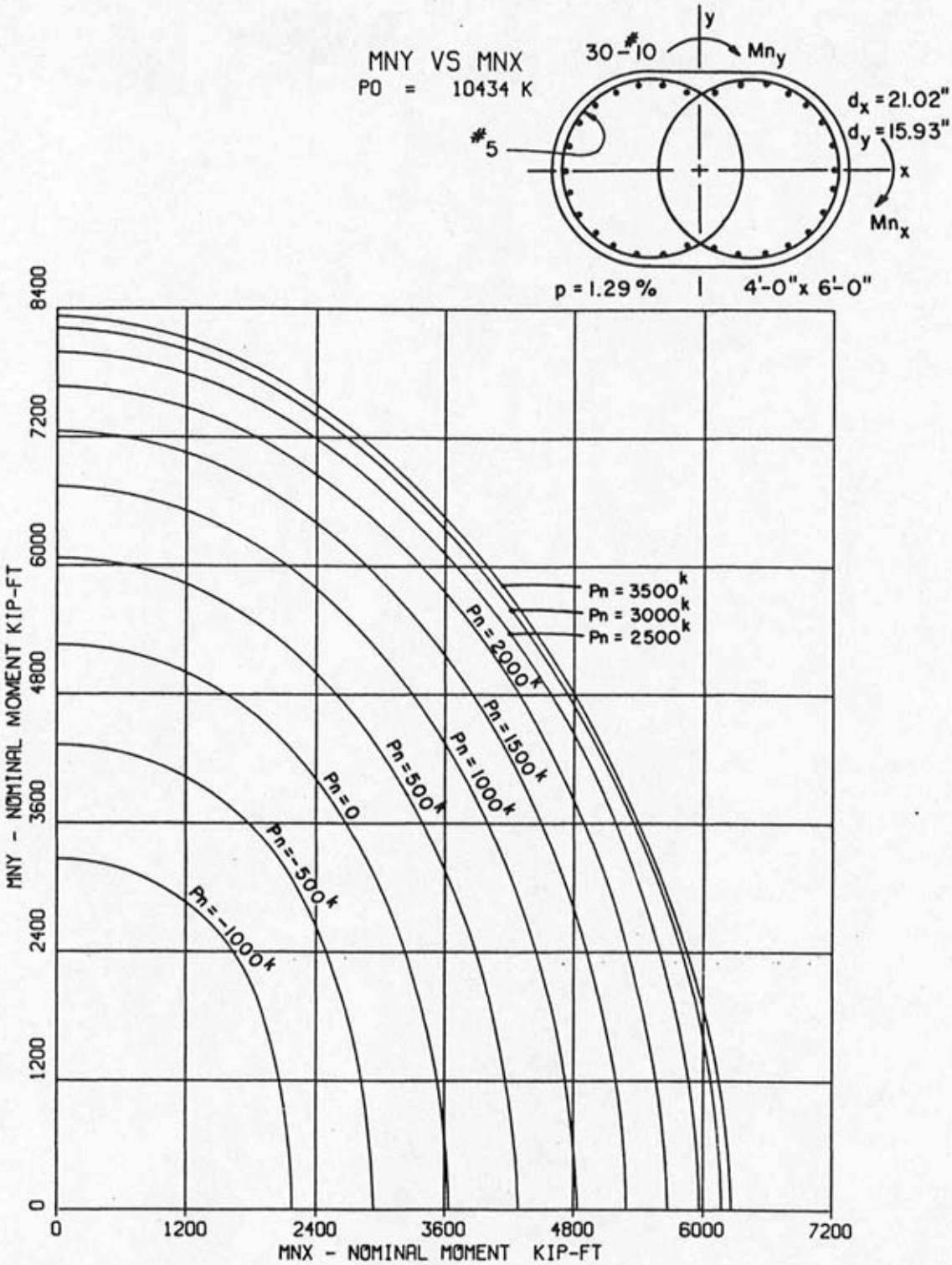
Column Types

4'-0" × 6'-0"
5'-6" × 8'-3"
7'-10" × 10'-6"



A = 20.6 ft²
I_x = 23.2 ft⁴
I_y = 49.1 ft⁴
I_z = 72.4 ft⁴


MNY VS MNX
PO = 9948 K
4'-0"x 6'-0"
30-#9
Pn = 4000k
Pn = 4500k
Pn = 5000k
Pn = 5500k
Pn = 6000k
Pn = 6500k
MNY - NOMINAL MOMENT KIP-FT
MNX - NOMINAL MOMENT KIP-FT
0
1000
2000
3000
4000
5000
6000
7000
8000

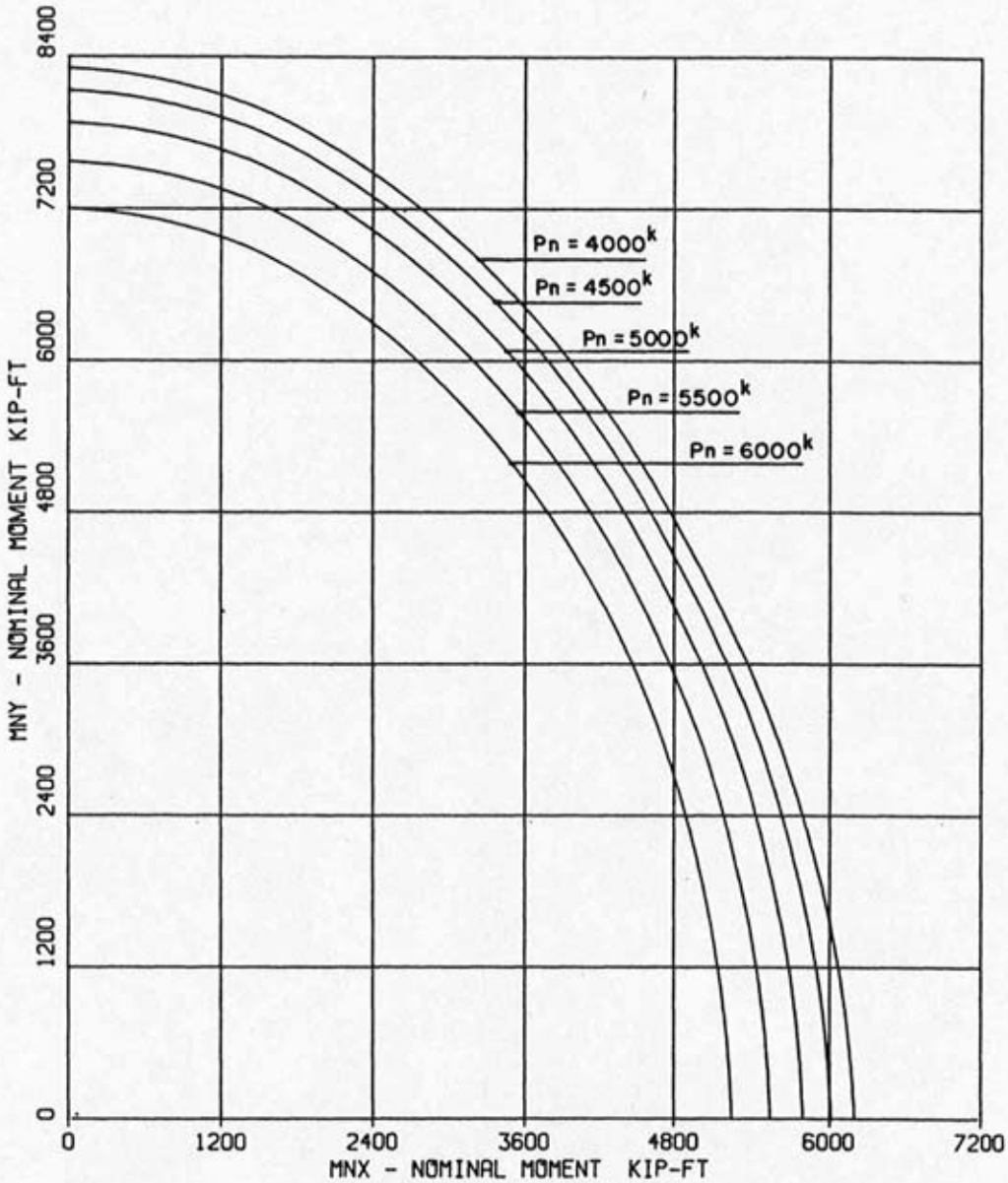


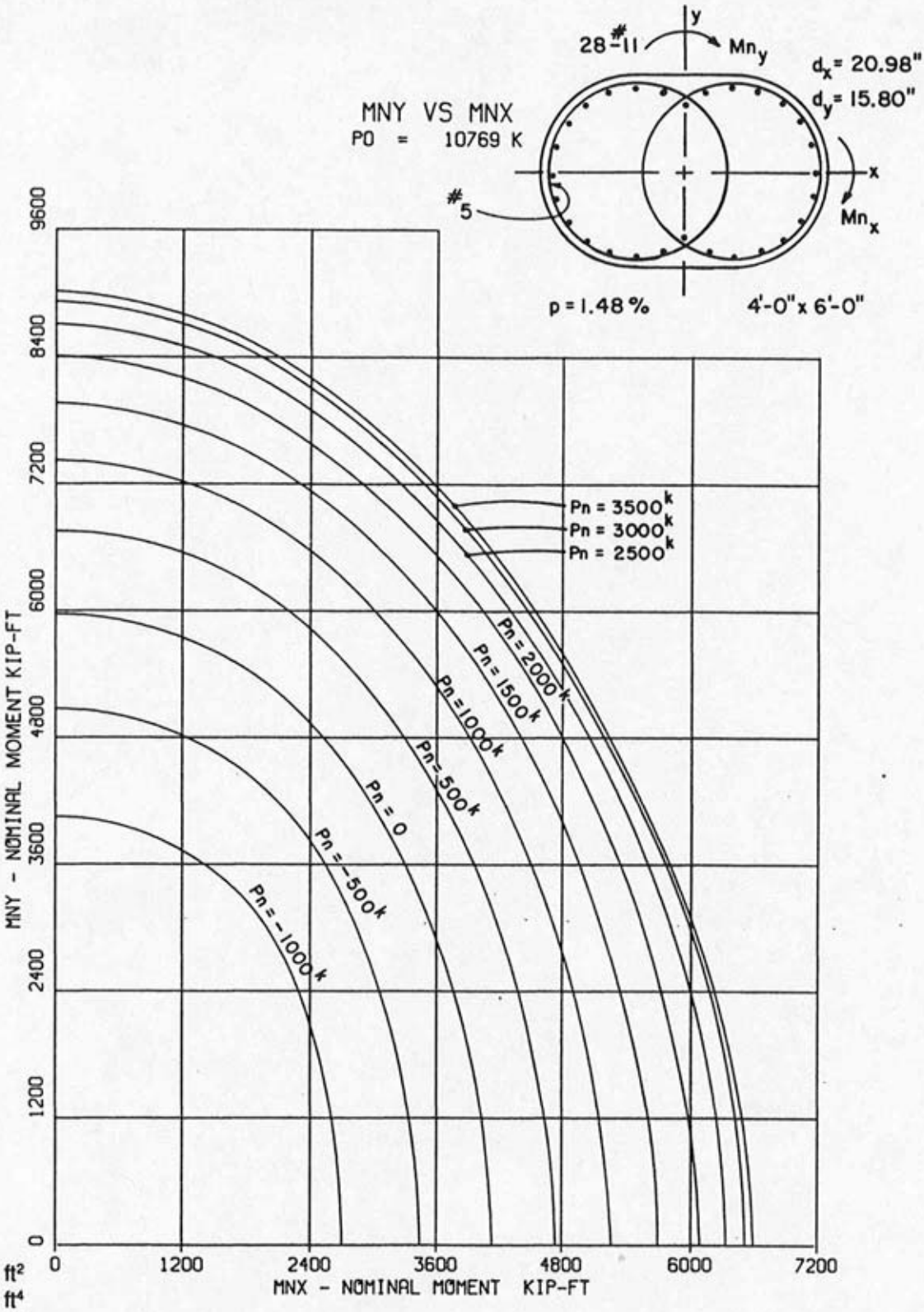
A = 20.6 ft²
I_x = 23.2 ft⁴
I_y = 49.1 ft⁴
I_z = 72.4 ft⁴

MNY VS MNX
PO = 10434 K

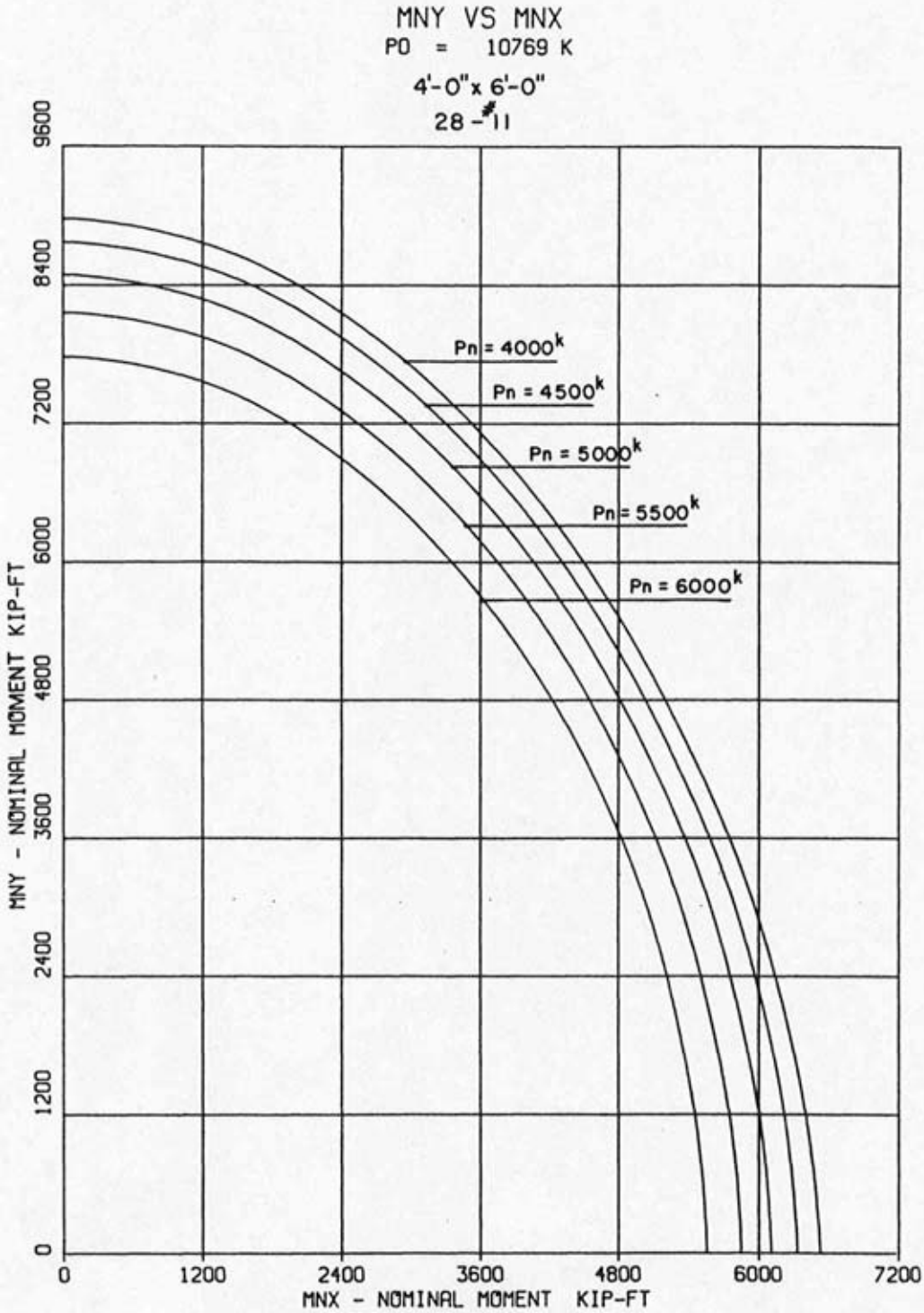
4'-0" x 6'-0"

30-#10

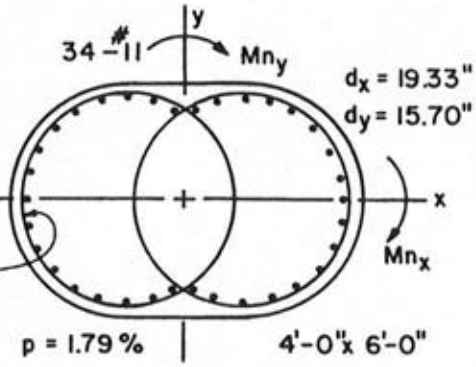


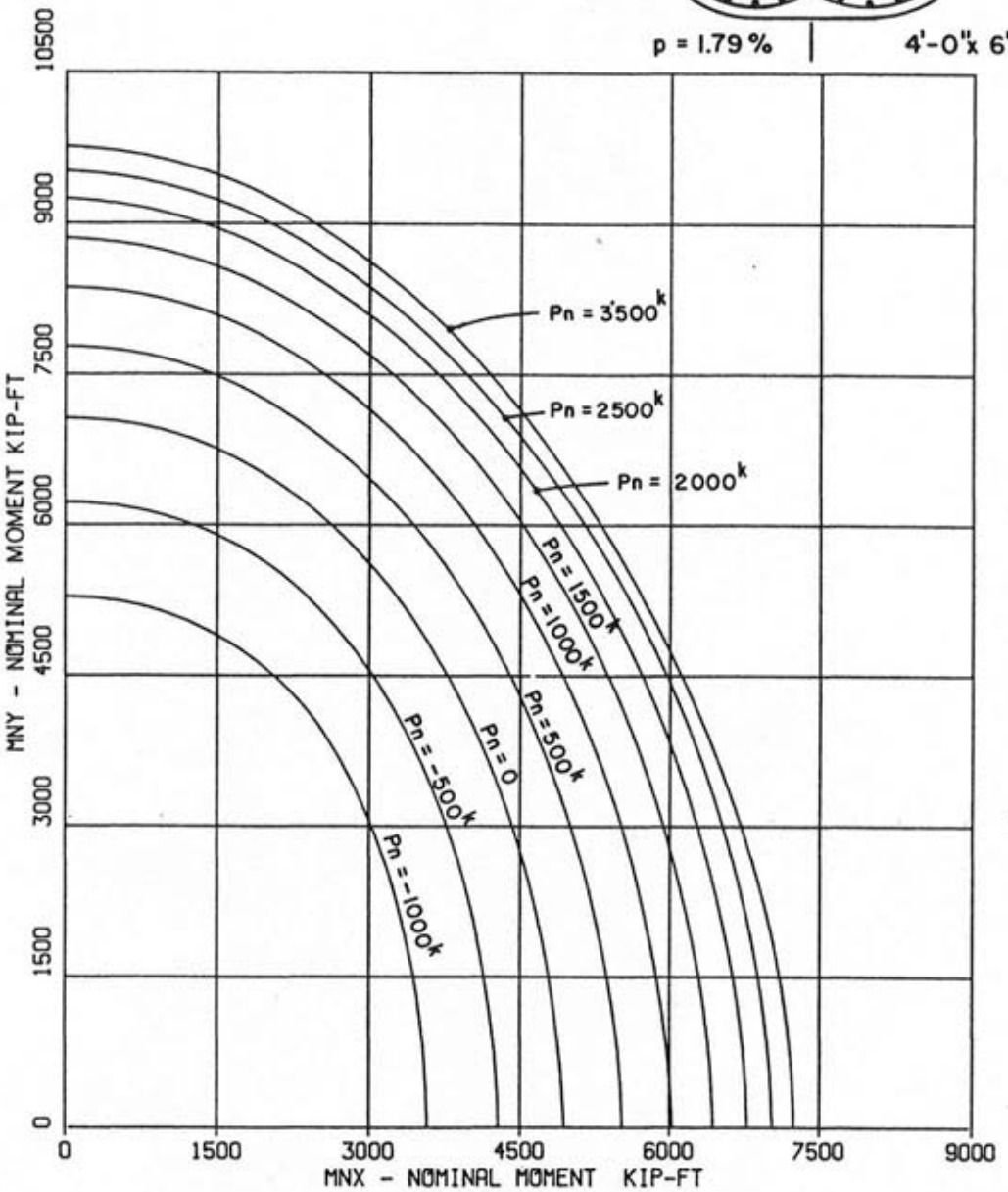


A = 20.6 ft²
I_x = 23.2 ft⁴
I_y = 49.1 ft⁴
I_z = 72.4 ft⁴


MNY VS MNX
PO = 10769 K
4'-0"x 6'-0"
28 - #11
Pn = 4000k
Pn = 4500k
Pn = 5000k
Pn = 5500k
Pn = 6000k
MNY - NOMINAL MOMENT KIP-FT
MNX - NOMINAL MOMENT KIP-FT
0
1200
2400
3600
4800
6000
7200
8400
9600

MNY VS MNX
PO = 11331 K

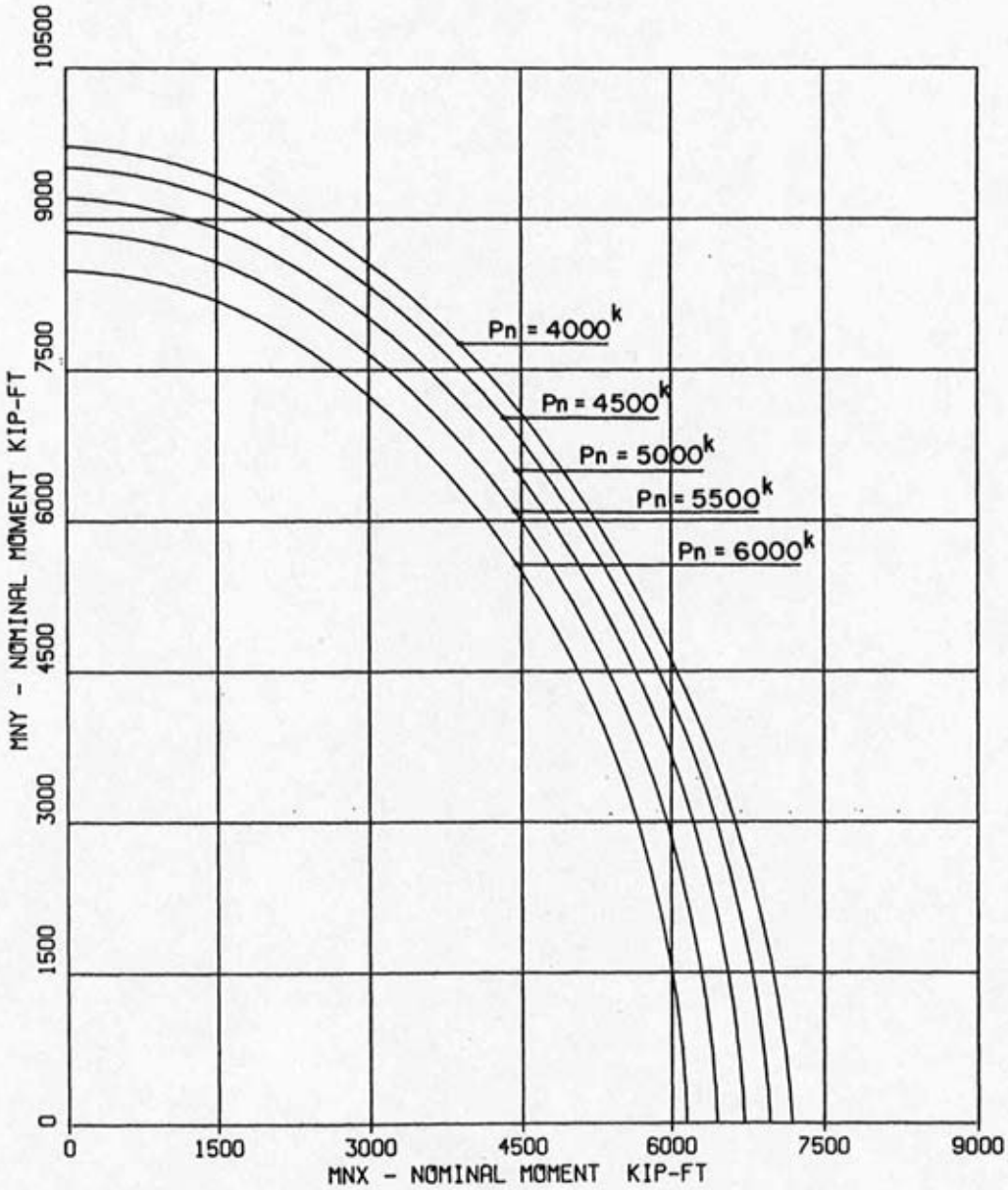


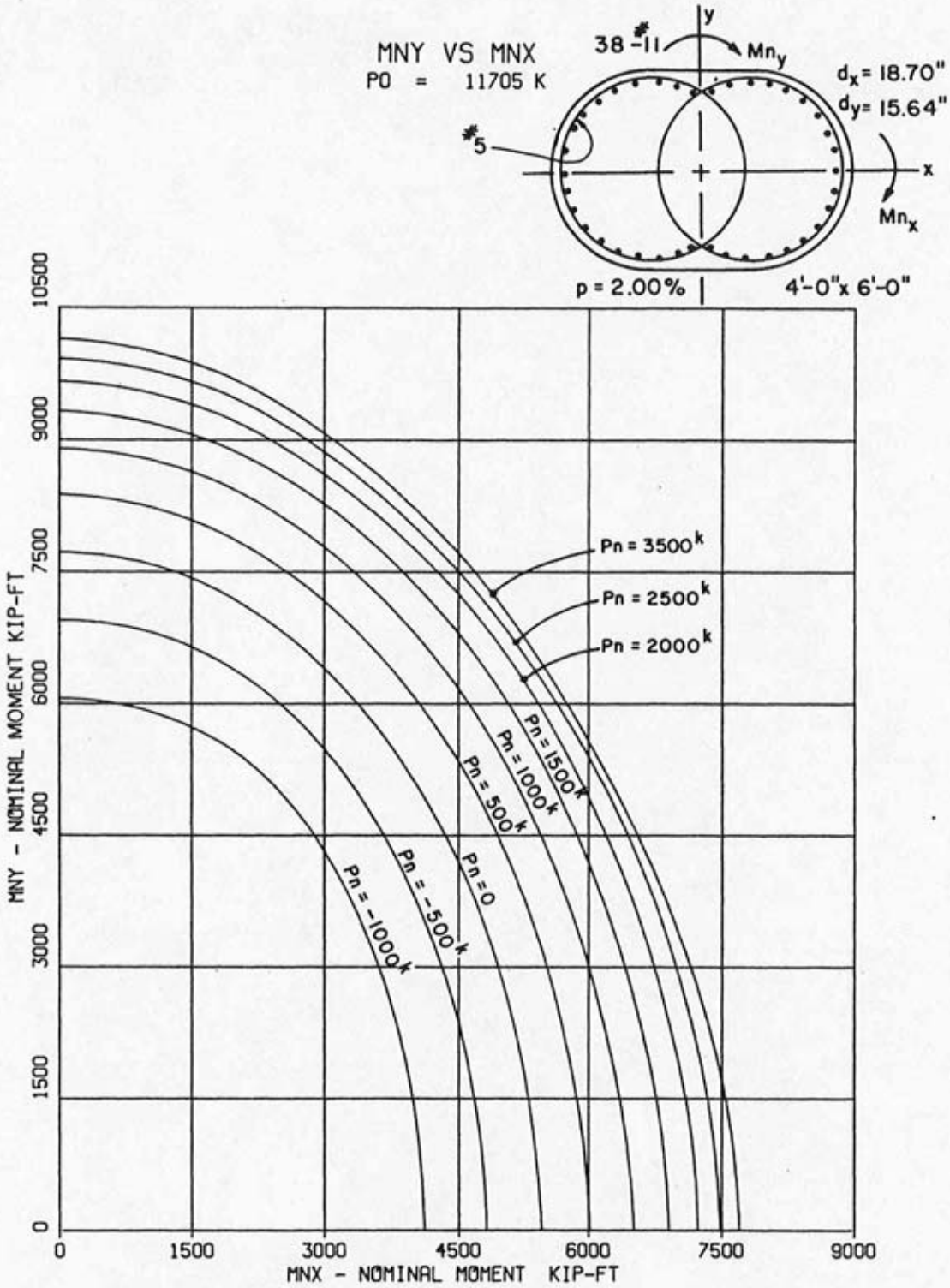


$A = 20.6\ ft^2$
$I_x = 23.2\ ft^4$
$I_y = 49.1\ ft^4$
$I_z = 72.4\ ft^4$

MNY VS MNX
PO = 11331 K

4'-0"x 6'-0"
34 - #11

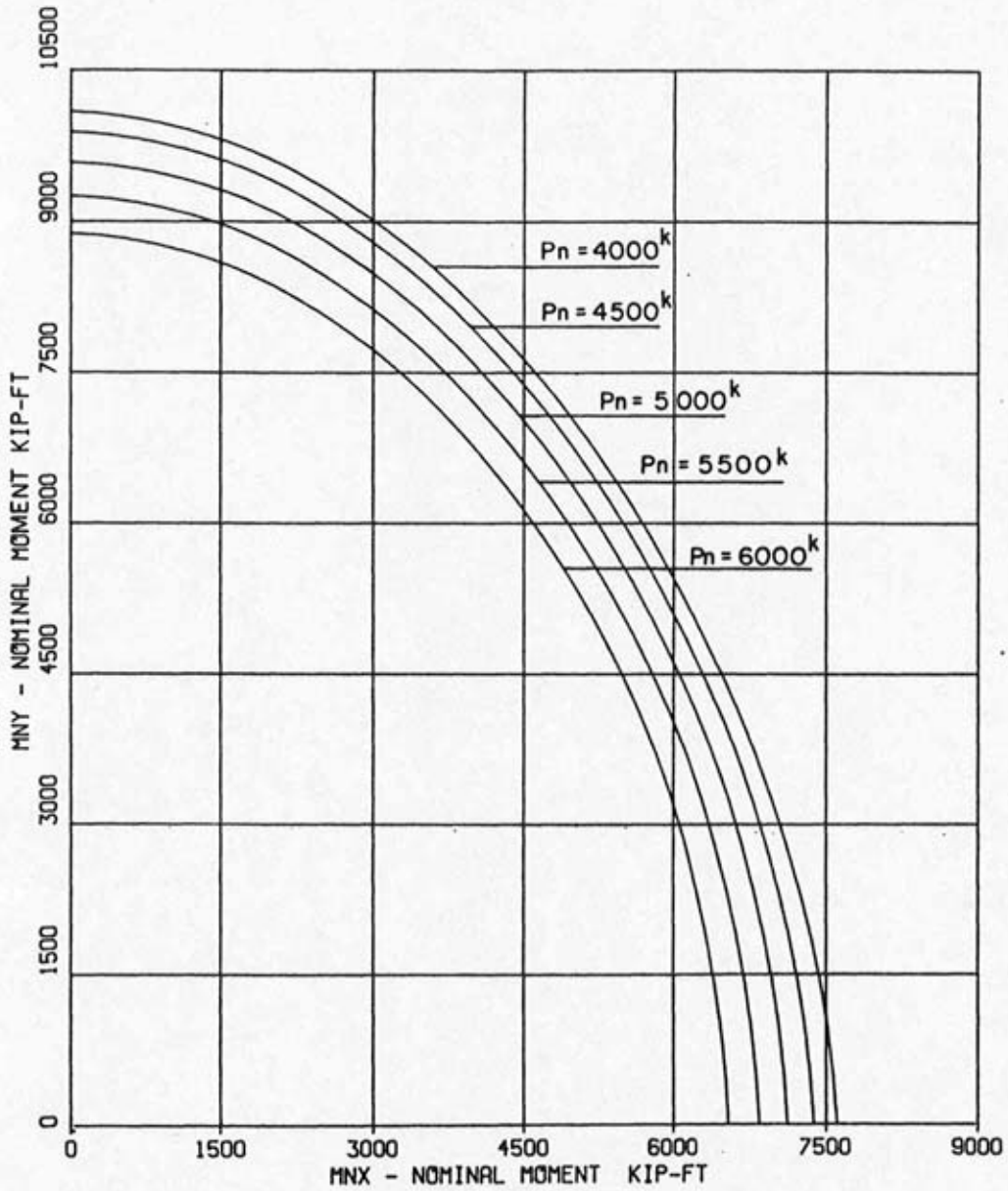


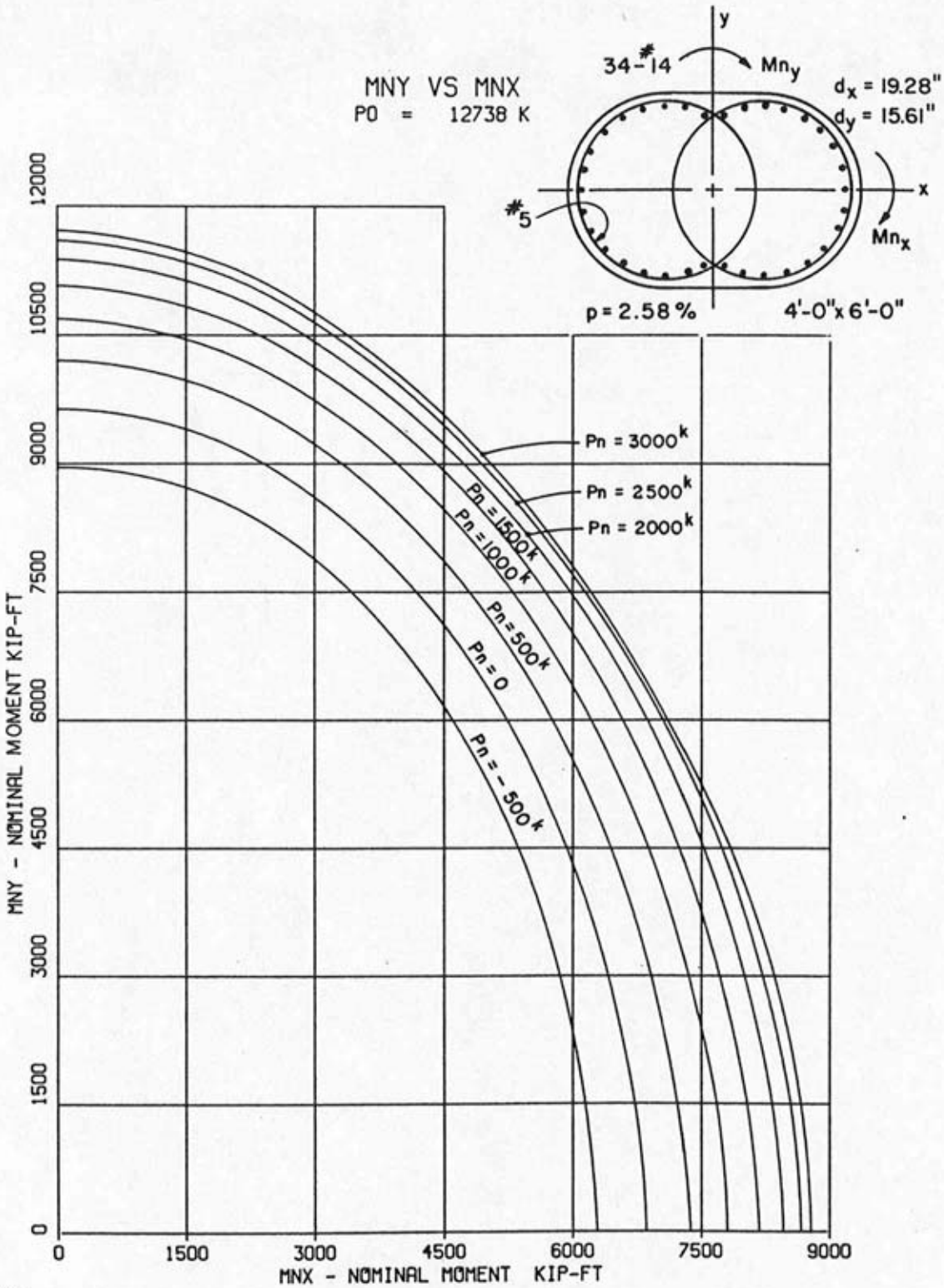


$A = 20.6\ ft^2$
$I_x = 23.2\ ft^4$
$I_y = 49.1\ ft^4$
$I_z = 72.4\ ft^4$

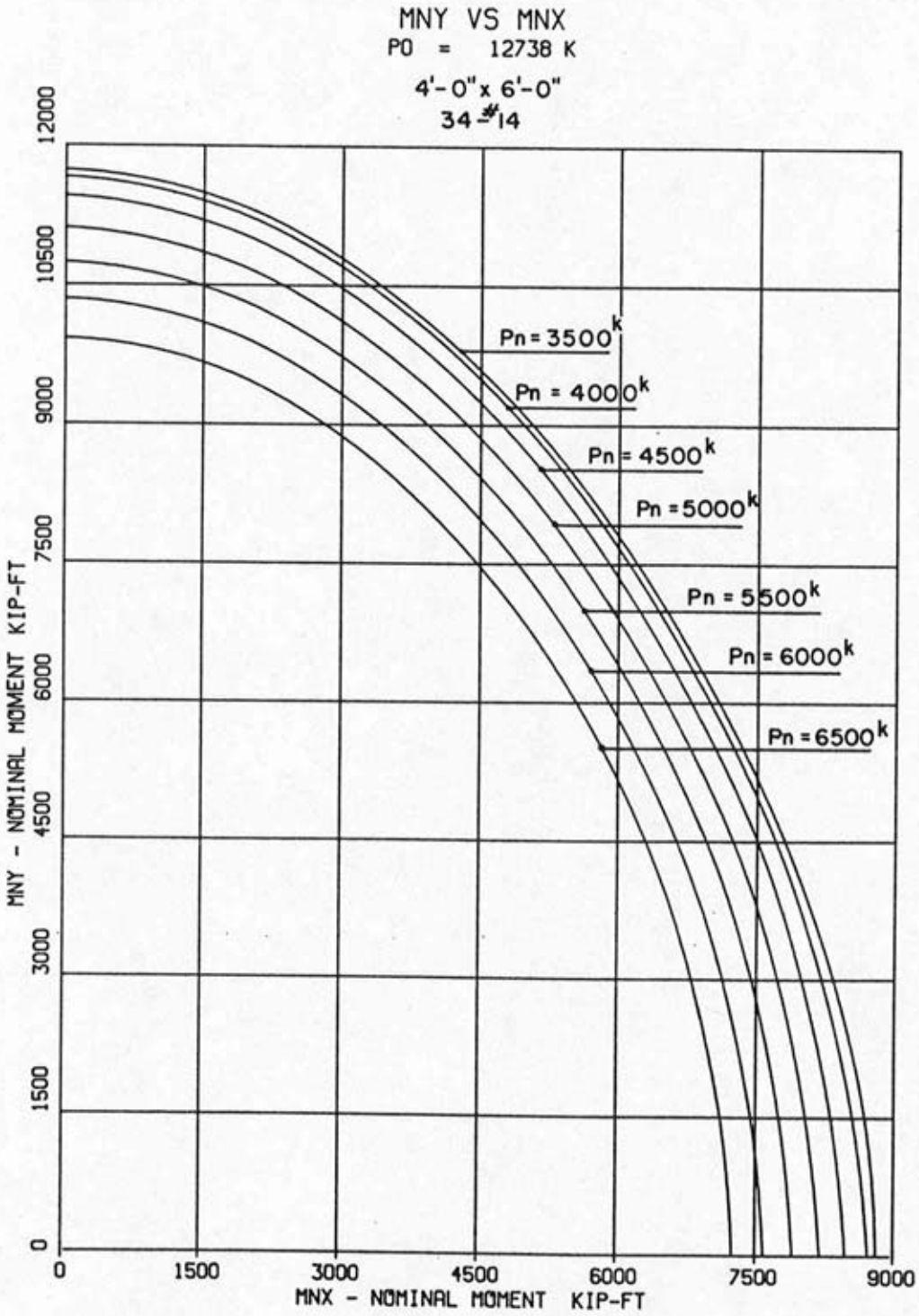
MNY VS MNX
PO = 11705 K

4'-0" x 6'-0"
38-#11

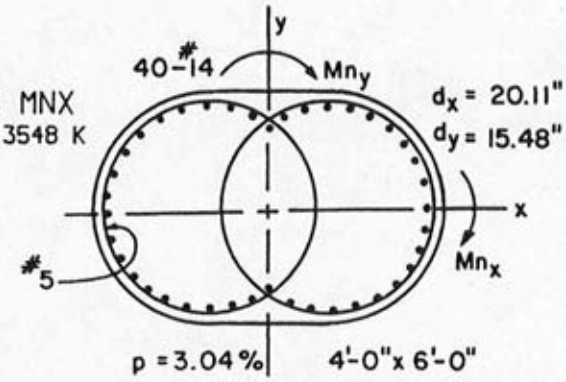


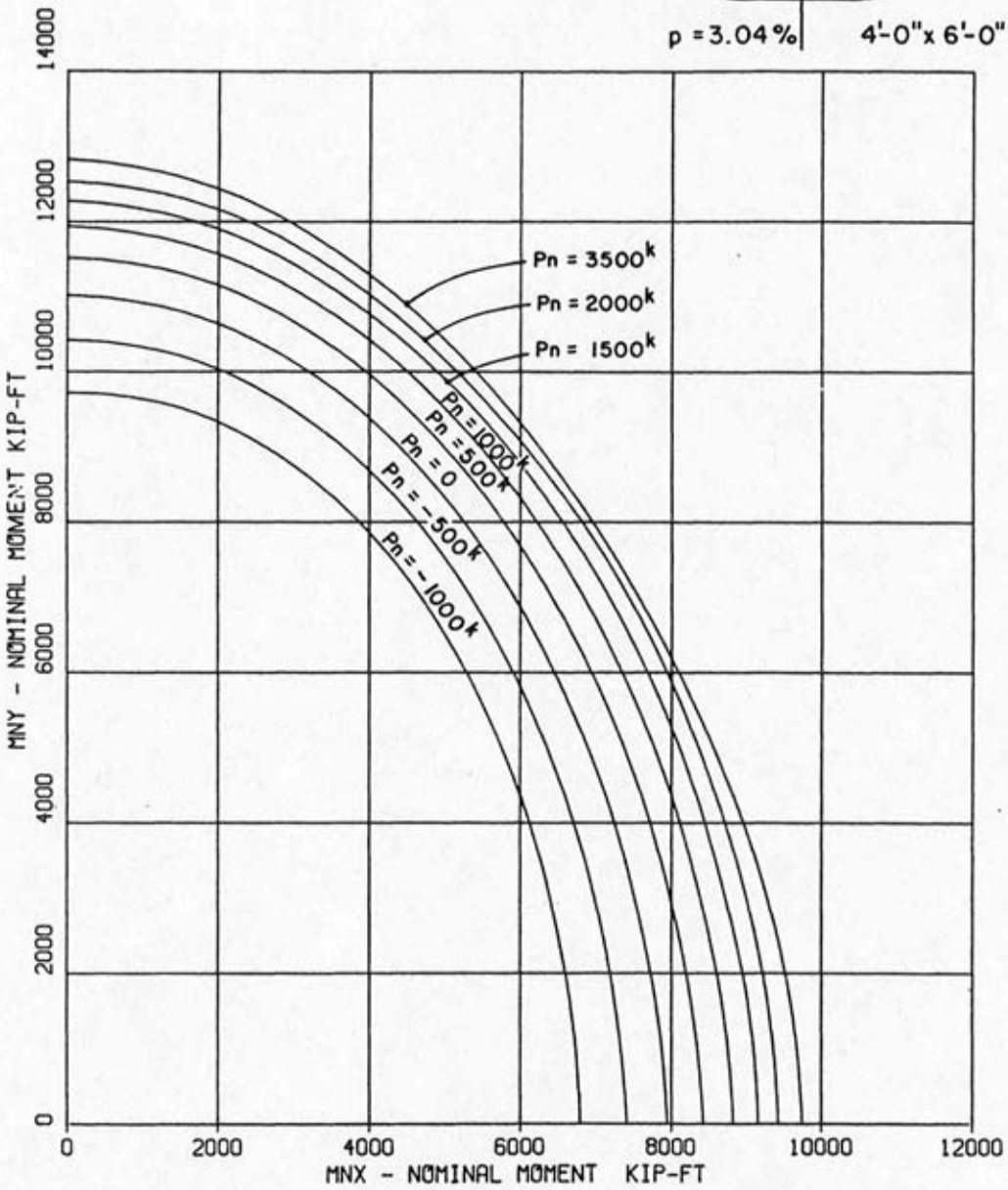


A = 20.6 ft²
I_x = 23.2 ft⁴
I_y = 49.1 ft⁴
I_z = 72.4 ft⁴


MNY VS MNX
PO = 12738 K
4'-0" x 6'-0"
34-#14
Pn = 3500k
Pn = 4000k
Pn = 4500k
Pn = 5000k
Pn = 5500k
Pn = 6000k
Pn = 6500k
MNY - NOMINAL MOMENT KIP-FT
0
1500
3000
4500
6000
7500
9000
10500
12000
MNX - NOMINAL MOMENT KIP-FT
0
1500
3000
4500
6000
7500
9000

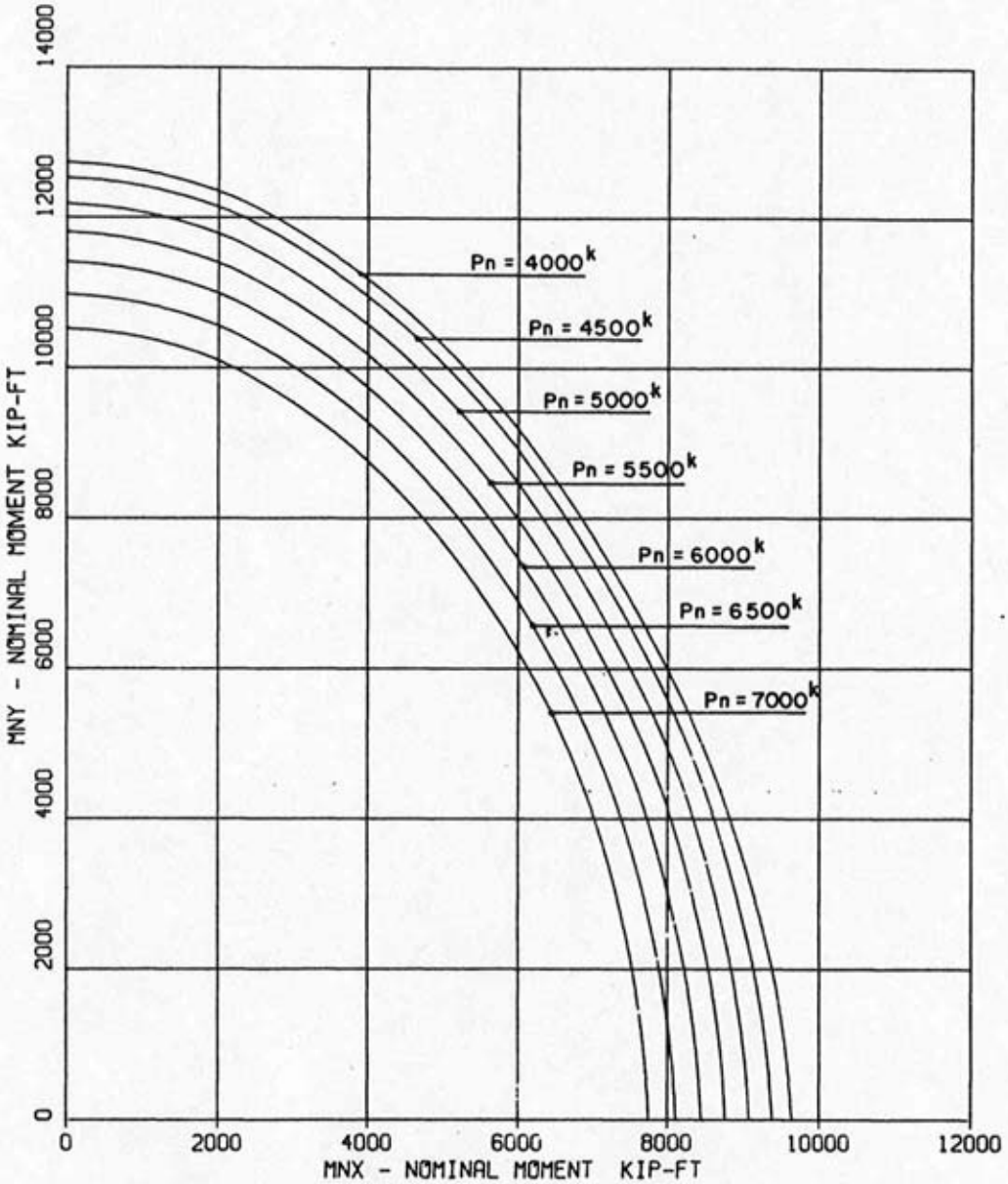
MNY VS MNX
PO = 13548 K

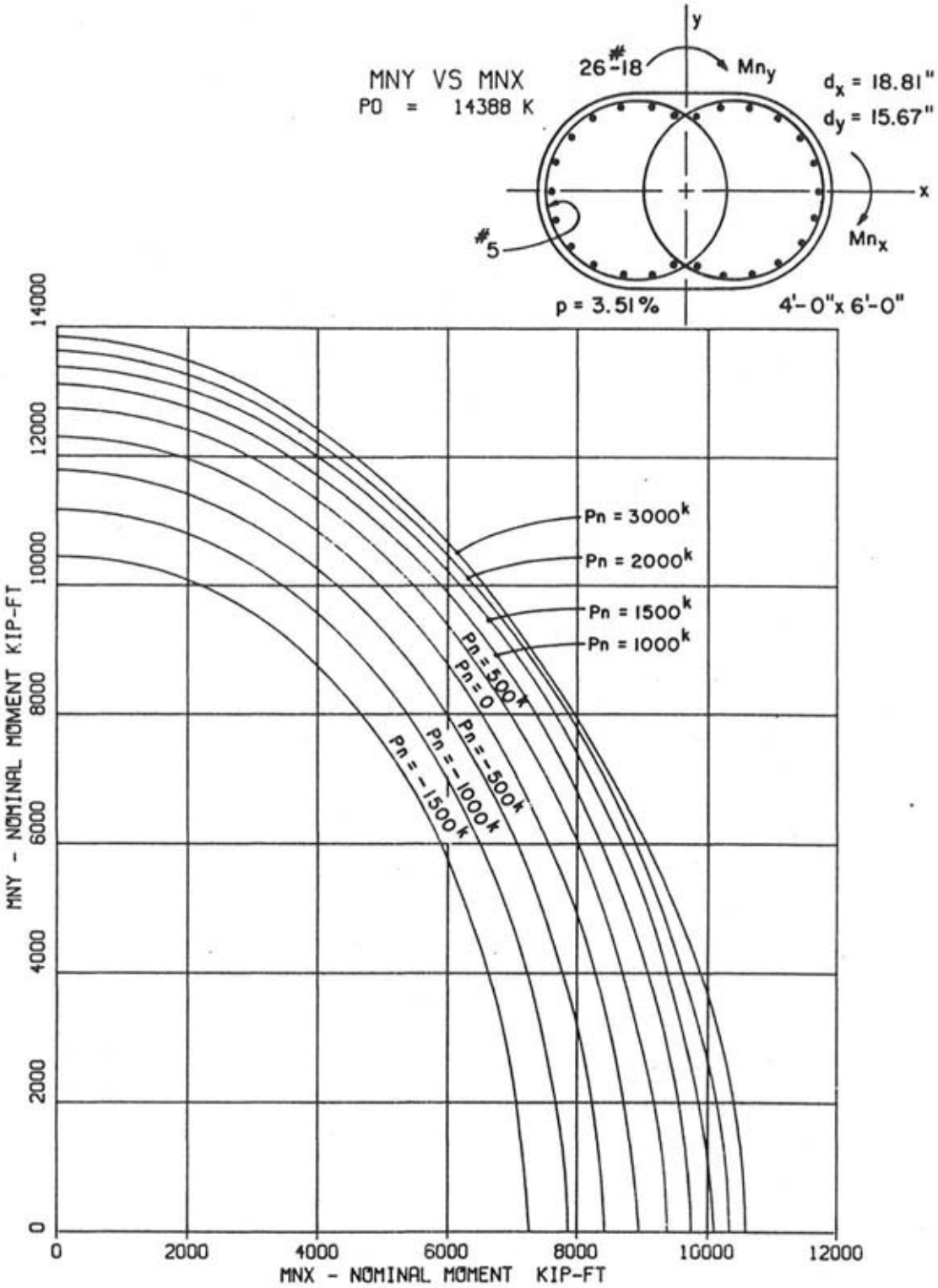




$A = 20.6\ ft^2$
$I_x = 23.2\ ft^4$
$I_y = 49.1\ ft^4$
$I_z = 72.4\ ft^4$

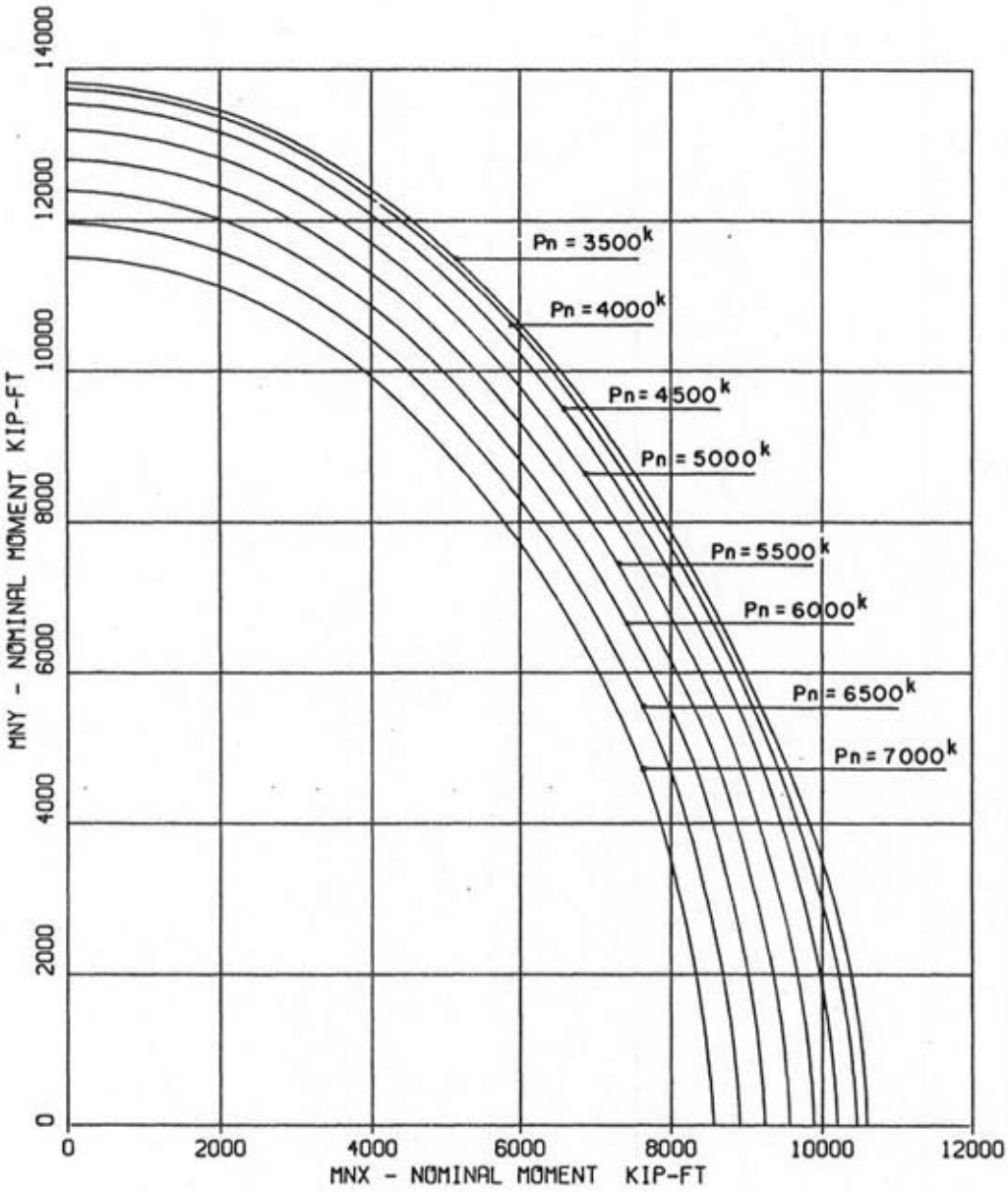
MNY VS MNX
PO = 13548 K
4'-0" x 6'-0"
40-#14



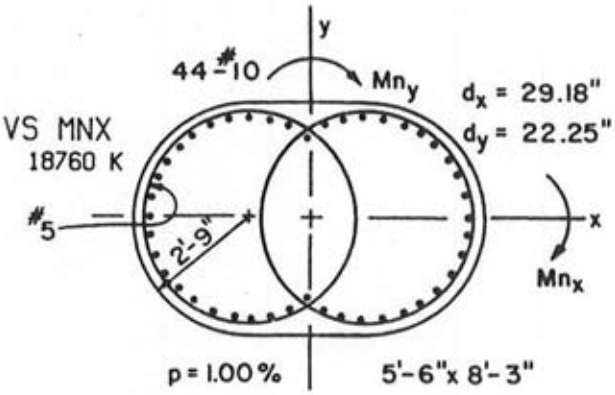


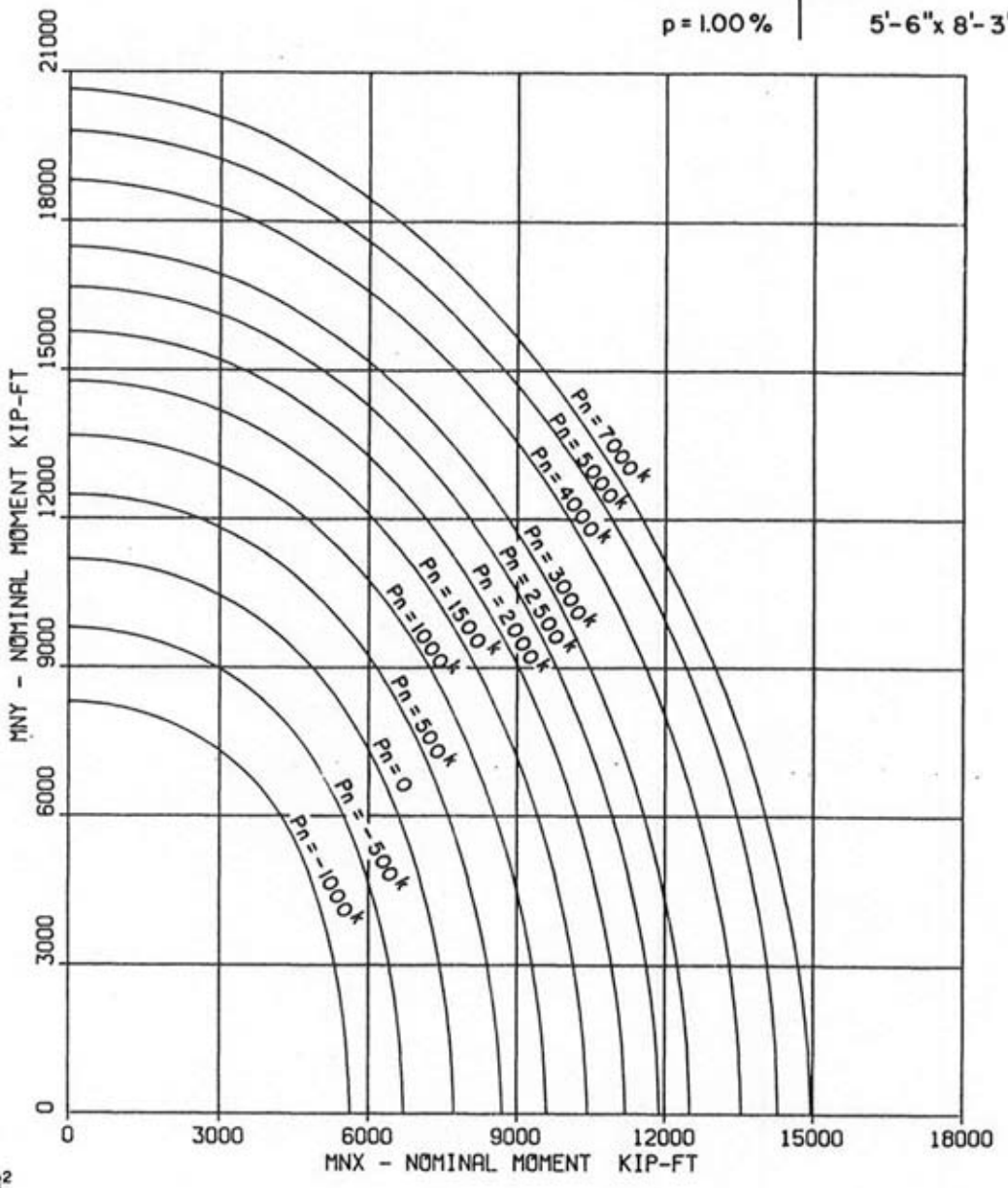
A = 20.6 ft²
I_x = 23.2 ft⁴
I_y = 49.1 ft⁴
I_z = 72.4 ft⁴

MNY VS MNX
PO = 14388 K
4'-0" x 6'-0"
26-#18



MNY VS MNX
PO = 18760 K



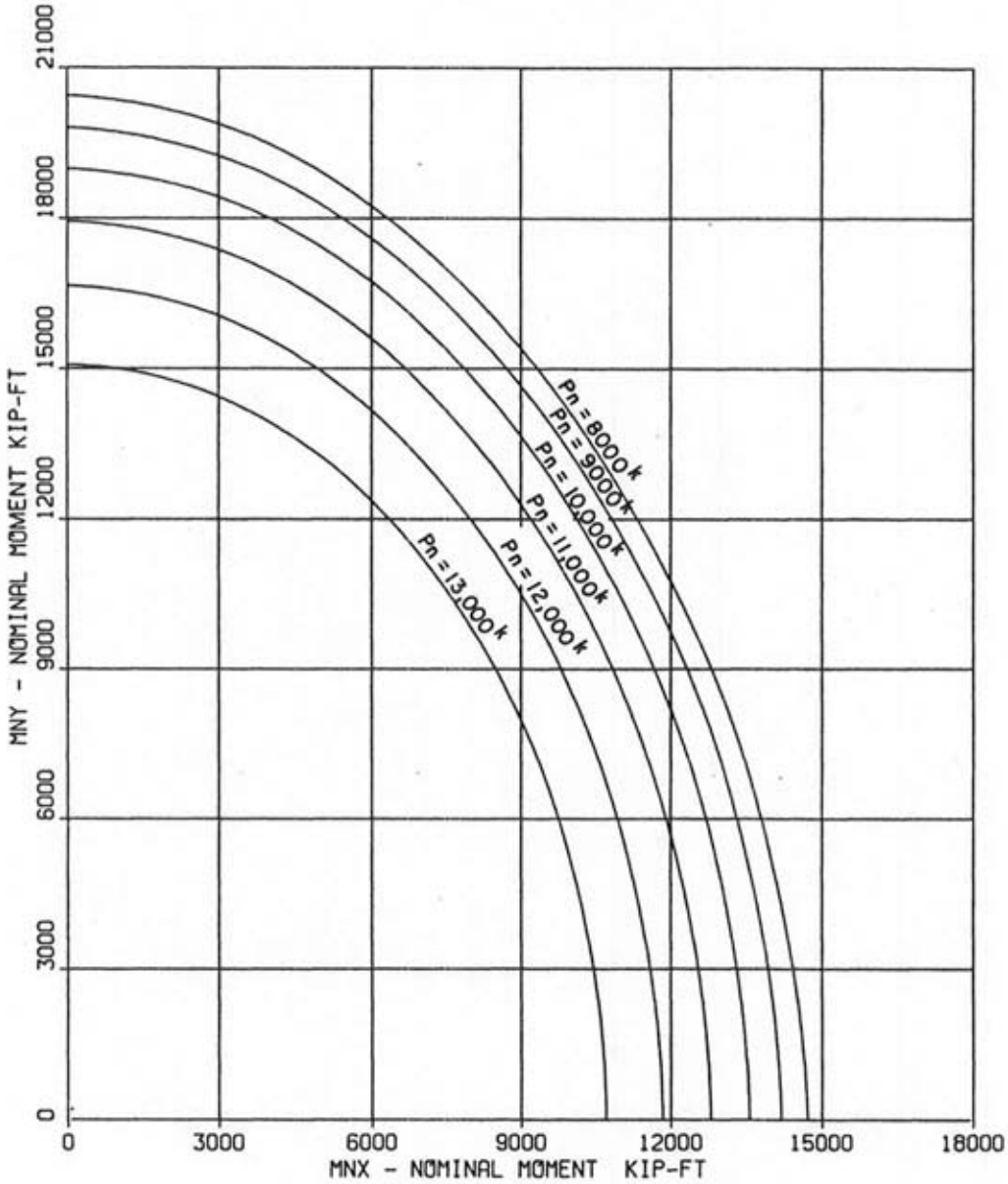


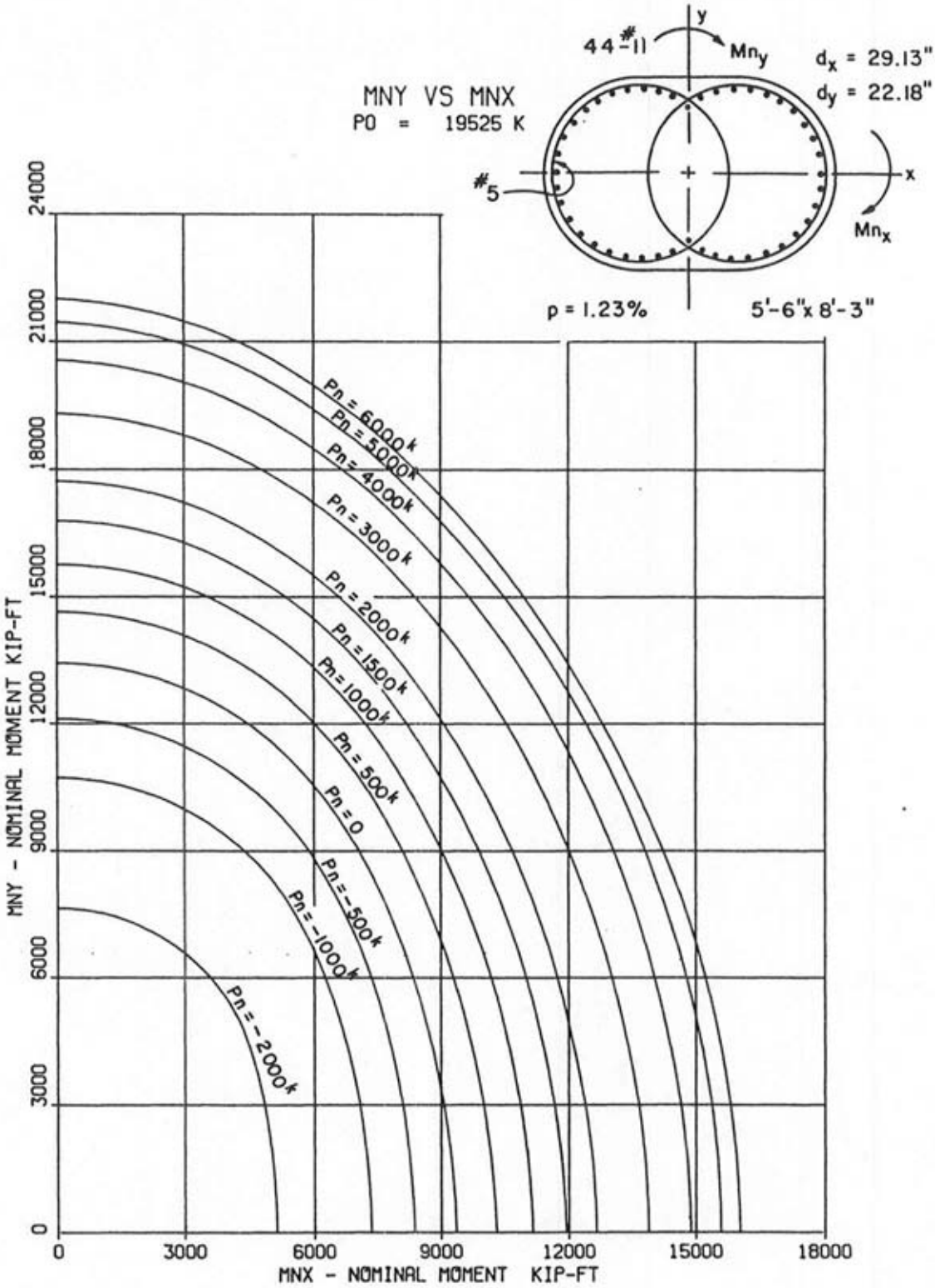
$A = 38.9\ ft^2$
$I_x = 83.1\ ft^4$
$I_y = 175.6\ ft^4$
$I_z = 258.7\ ft^4$

MNY VS MNX
PO = 18760 K

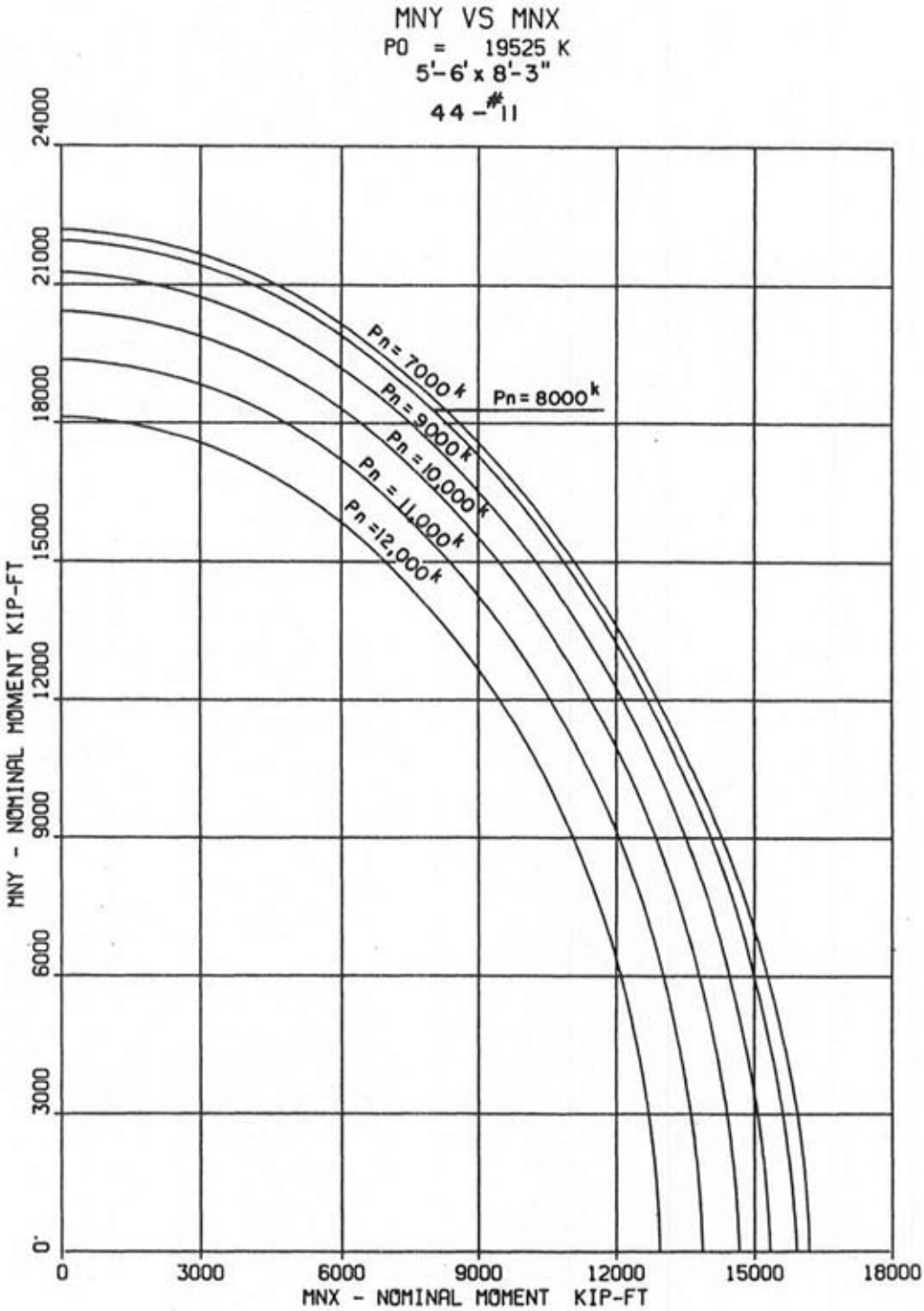
5'-6" x 8'-3"

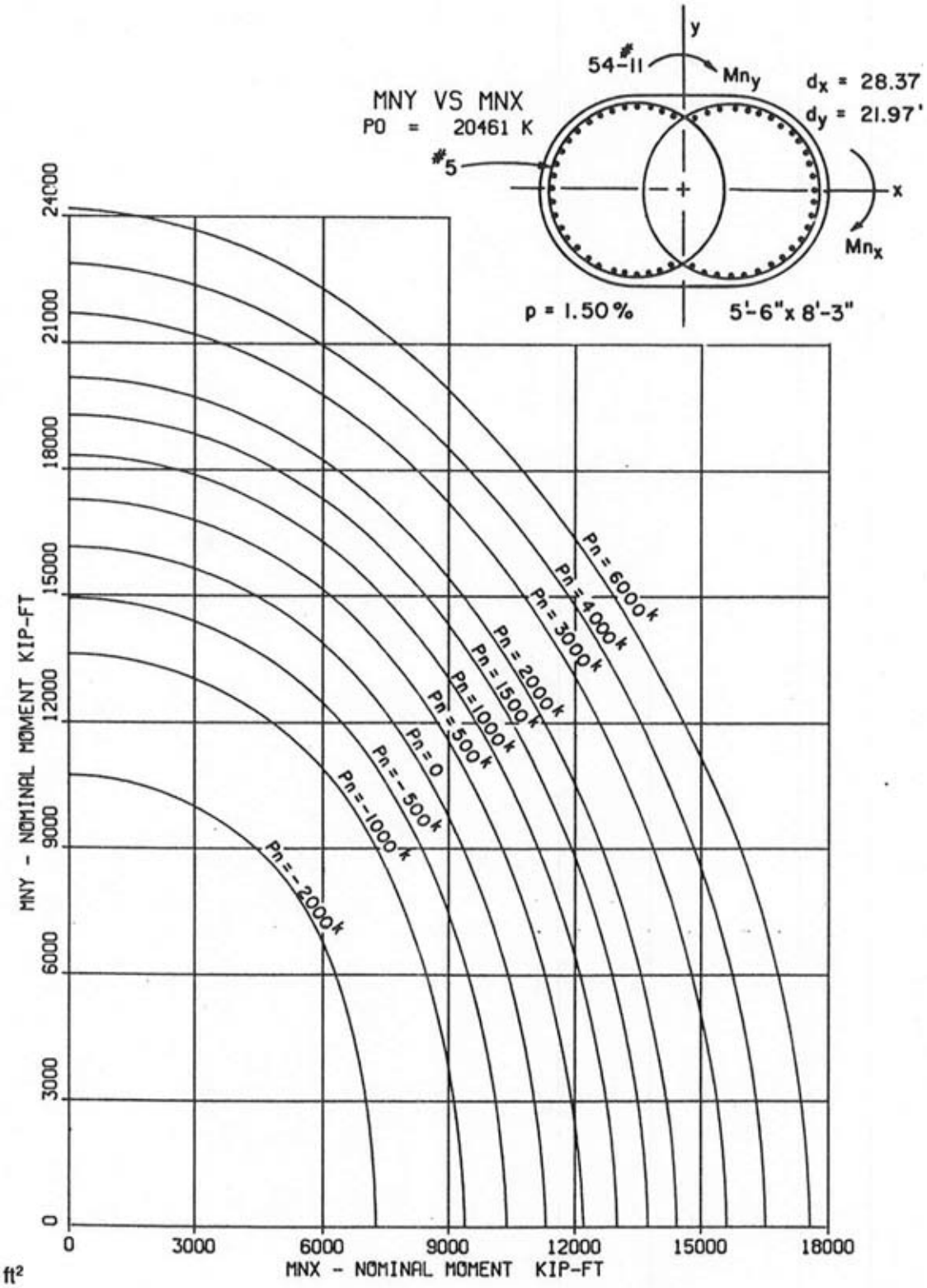
44-#10



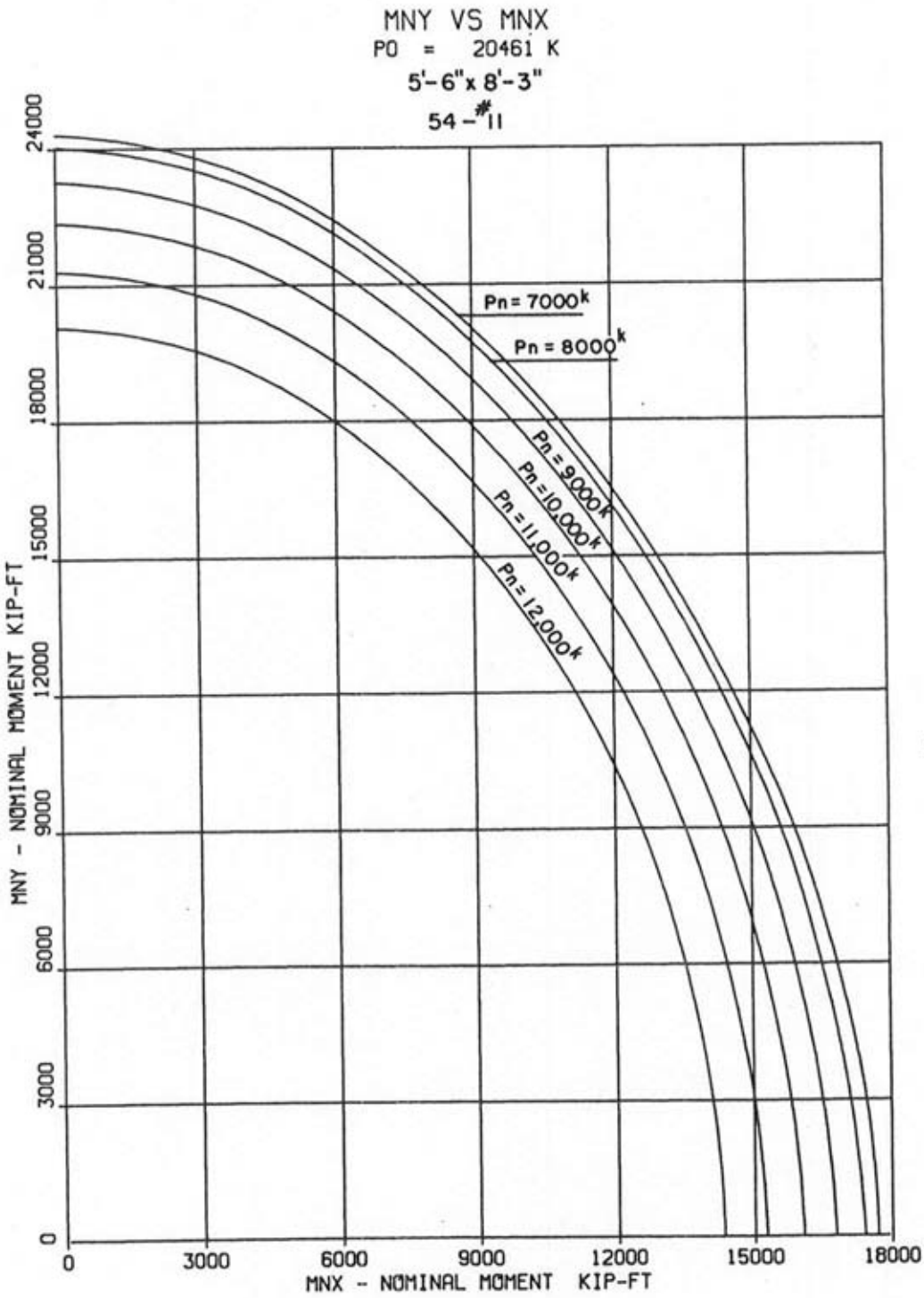


A = 38.9 ft²
I_x = 83.1 ft⁴
I_y = 175.6 ft⁴
I_z = 258.7 ft⁴

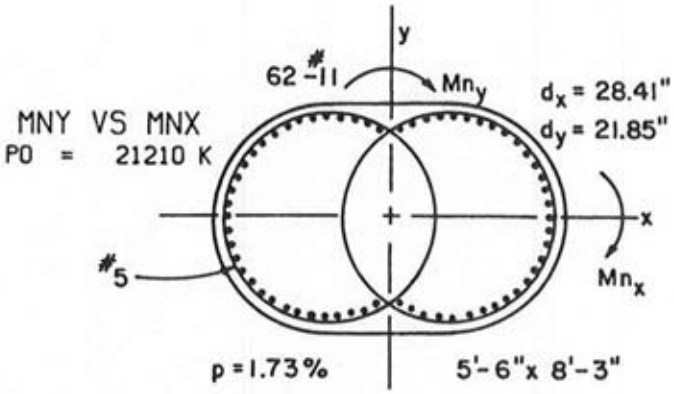

MNY VS MNX
PO = 19525 K
5'-6' x 8'-3"
44 -#11
Pn = 7000 k
Pn = 8000 k
Pn = 9000 k
Pn = 10,000 k
Pn = 11,000 k
Pn = 12,000 k
MNY - NOMINAL MOMENT KIP-FT
MNX - NOMINAL MOMENT KIP-FT
0
3000
6000
9000
12000
15000
18000
21000
24000

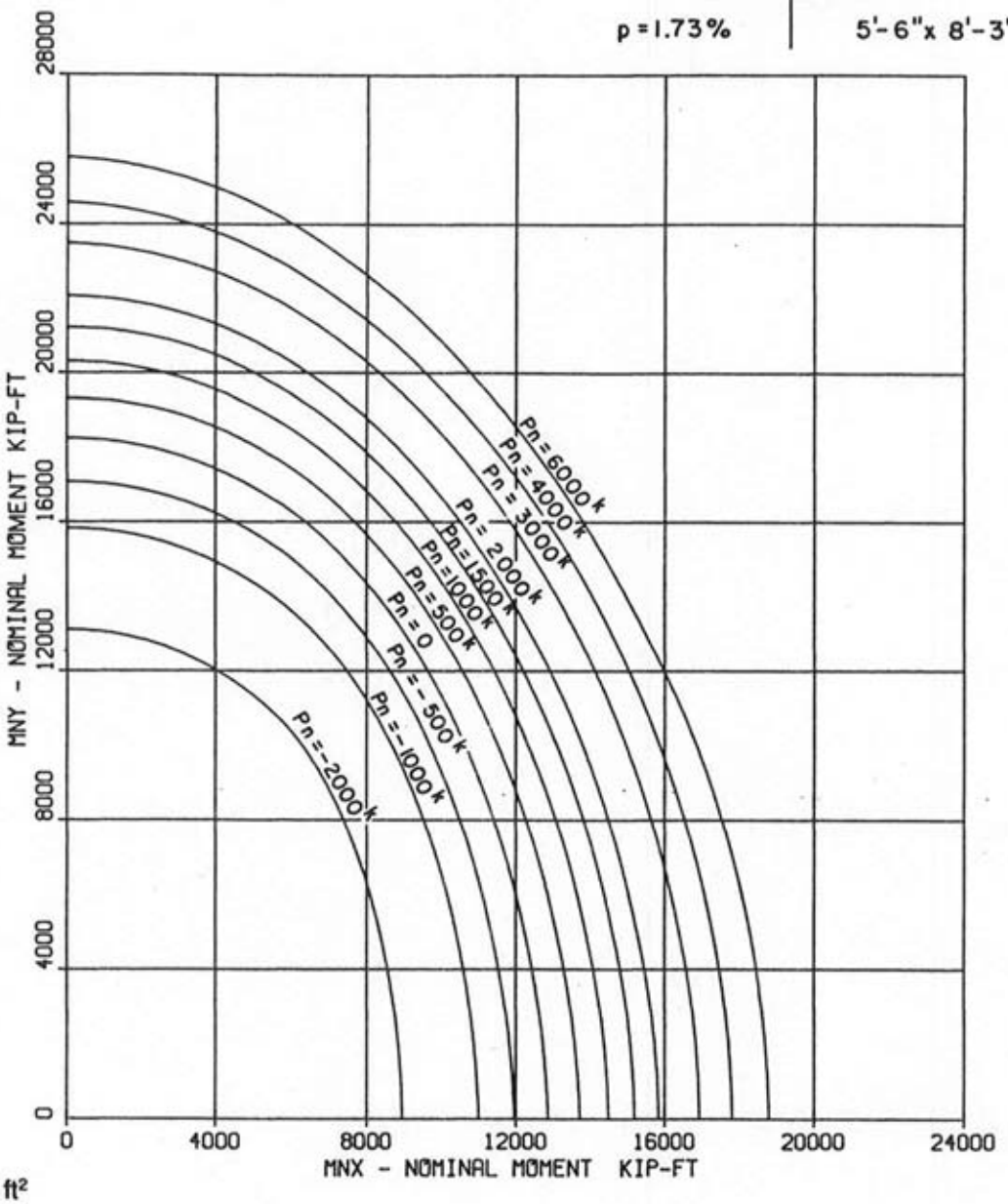


A = 38.9 ft²
I_x = 83.1 ft⁴
I_y = 175.6 ft⁴
I_z = 258.7 ft⁴


MNY VS MNX
PO = 20461 K
5'-6"x 8'-3"
54 - #11
Pn = 7000k
Pn = 8000k
Pn = 9000k
Pn = 10,000k
Pn = 11,000k
Pn = 12,000k
MNY - NOMINAL MOMENT KIP-FT
MNX - NOMINAL MOMENT KIP-FT
0
3000
6000
9000
12000
15000
18000
21000
24000

MNY VS MNX
PO = 21210 K



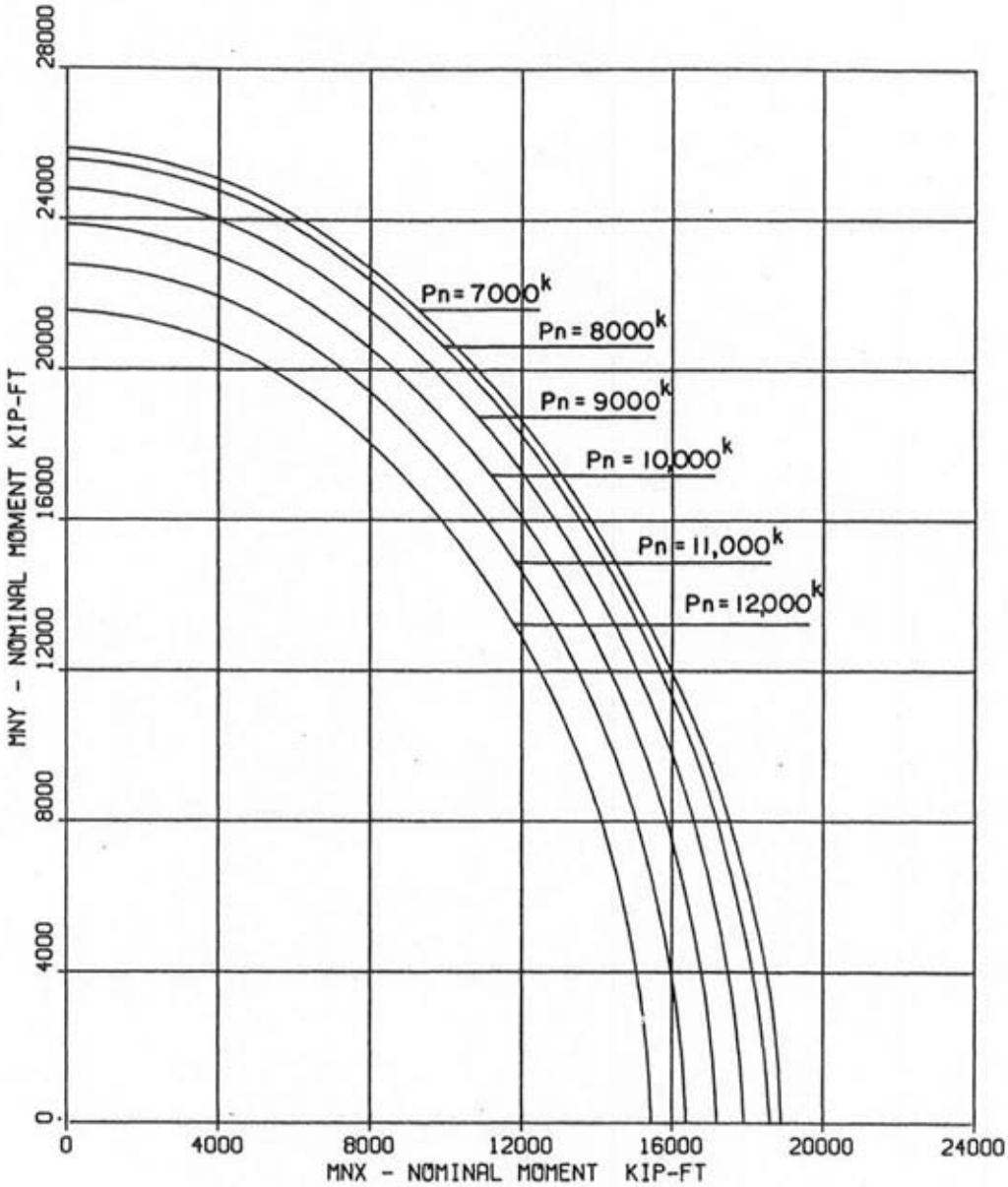


A = 38.9 ft²
I_x = 83.1 ft⁴
I_y = 175.6 ft⁴
I_z = 258.7 ft⁴

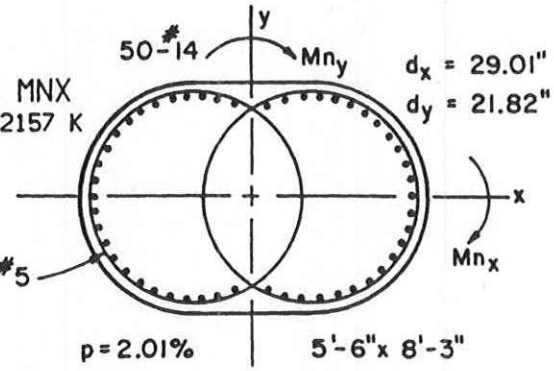
MNY VS MNX
PO = 21210 K

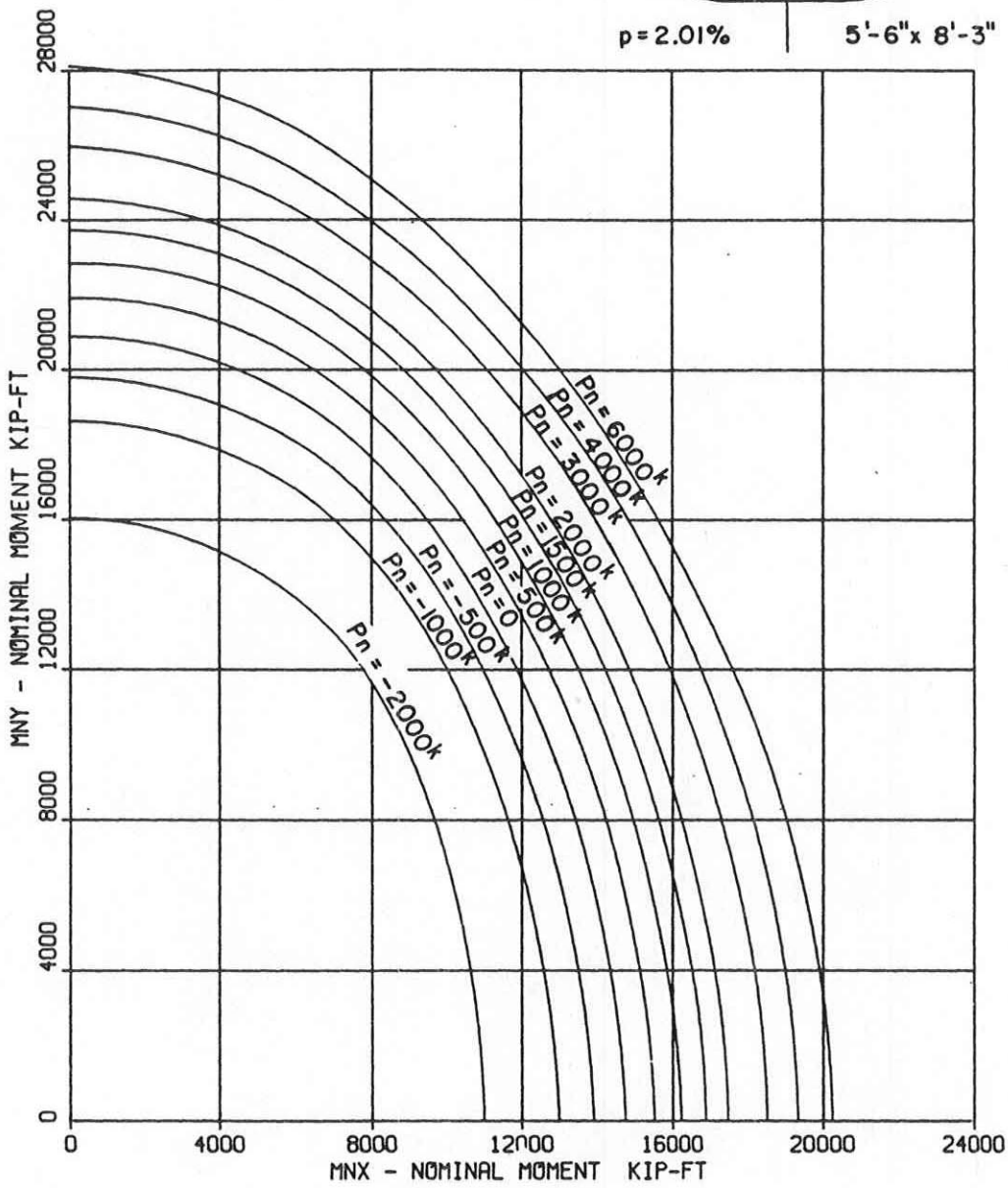
5'-6"x 8'-3"

62 - #11



MNY VS MNX
PO = 22157 K



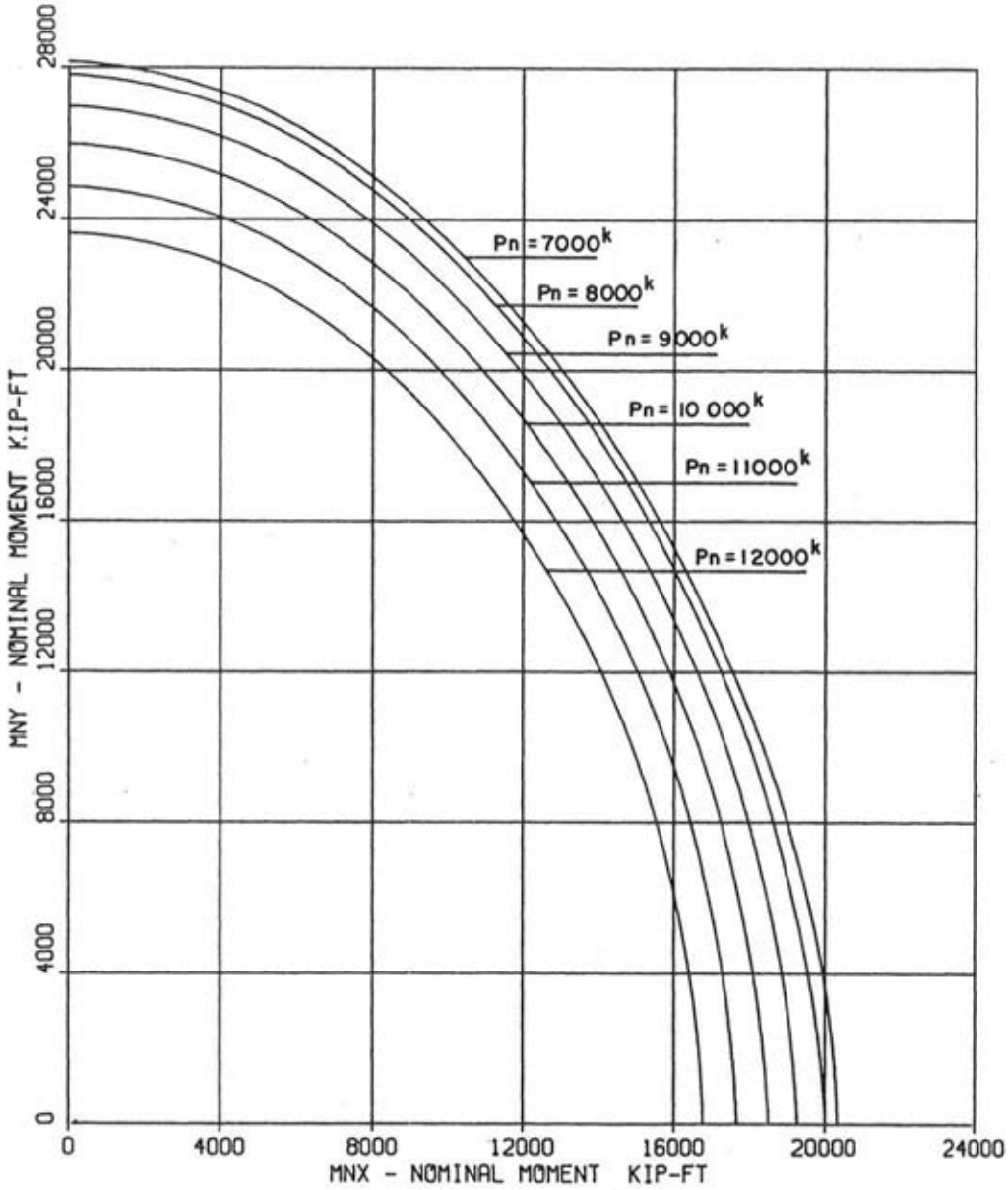


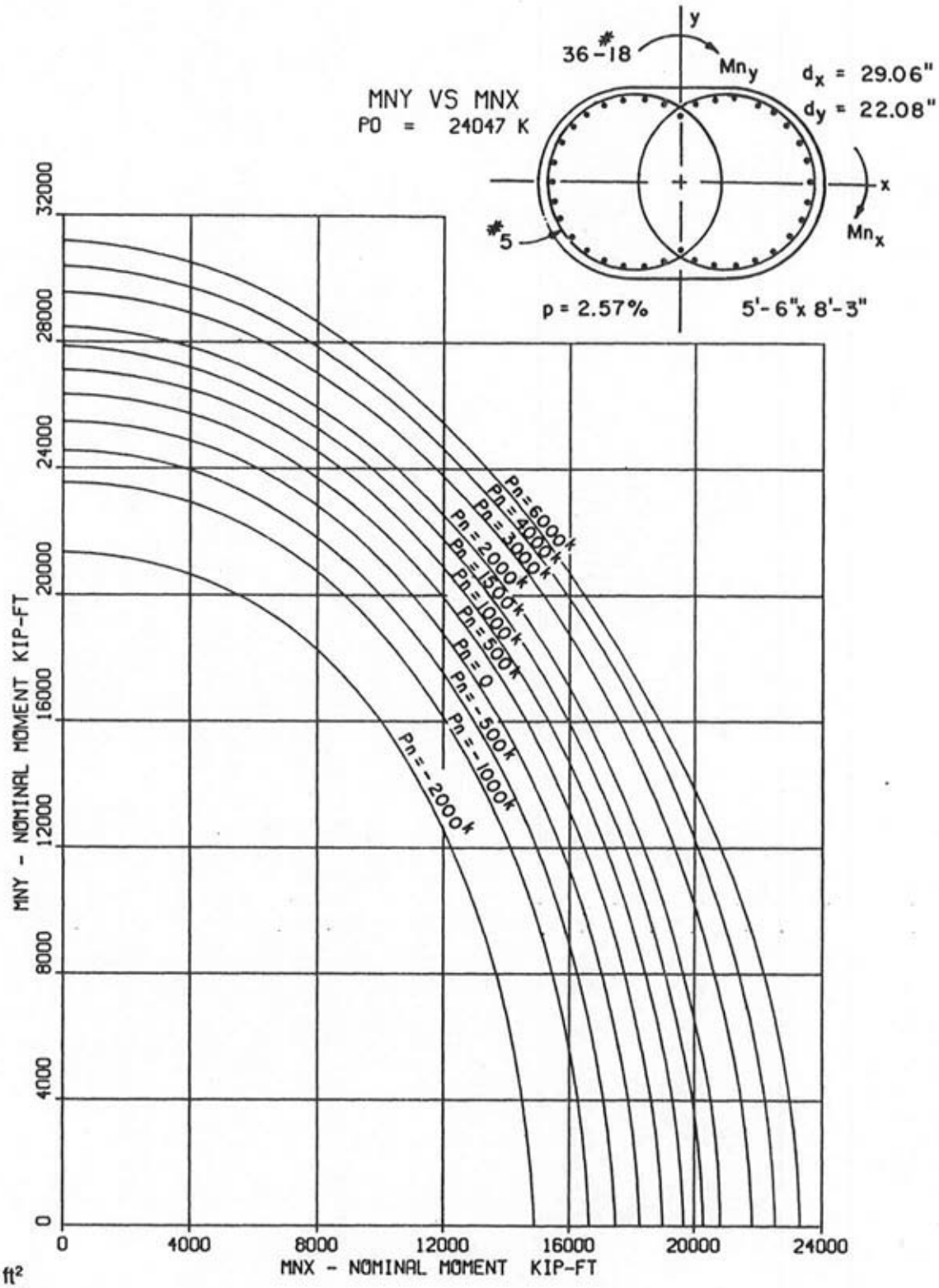
A = 38.9 ft²
I_x = 83.1 ft⁴
I_y = 175.6 ft⁴
I_z = 258.7 ft⁴

MNY VS MNX
PO = 22157 K

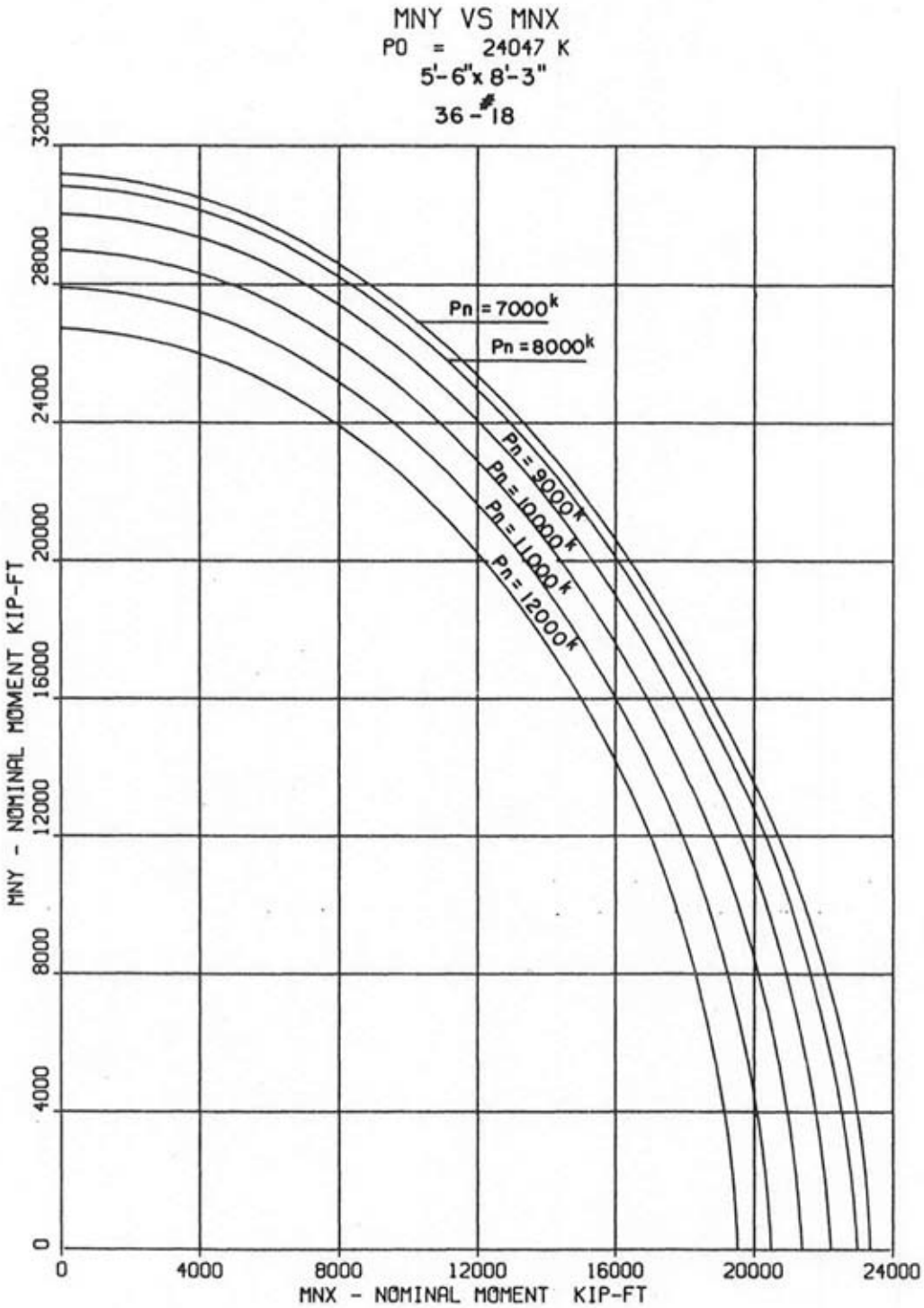
5'-6" x 8'-3"

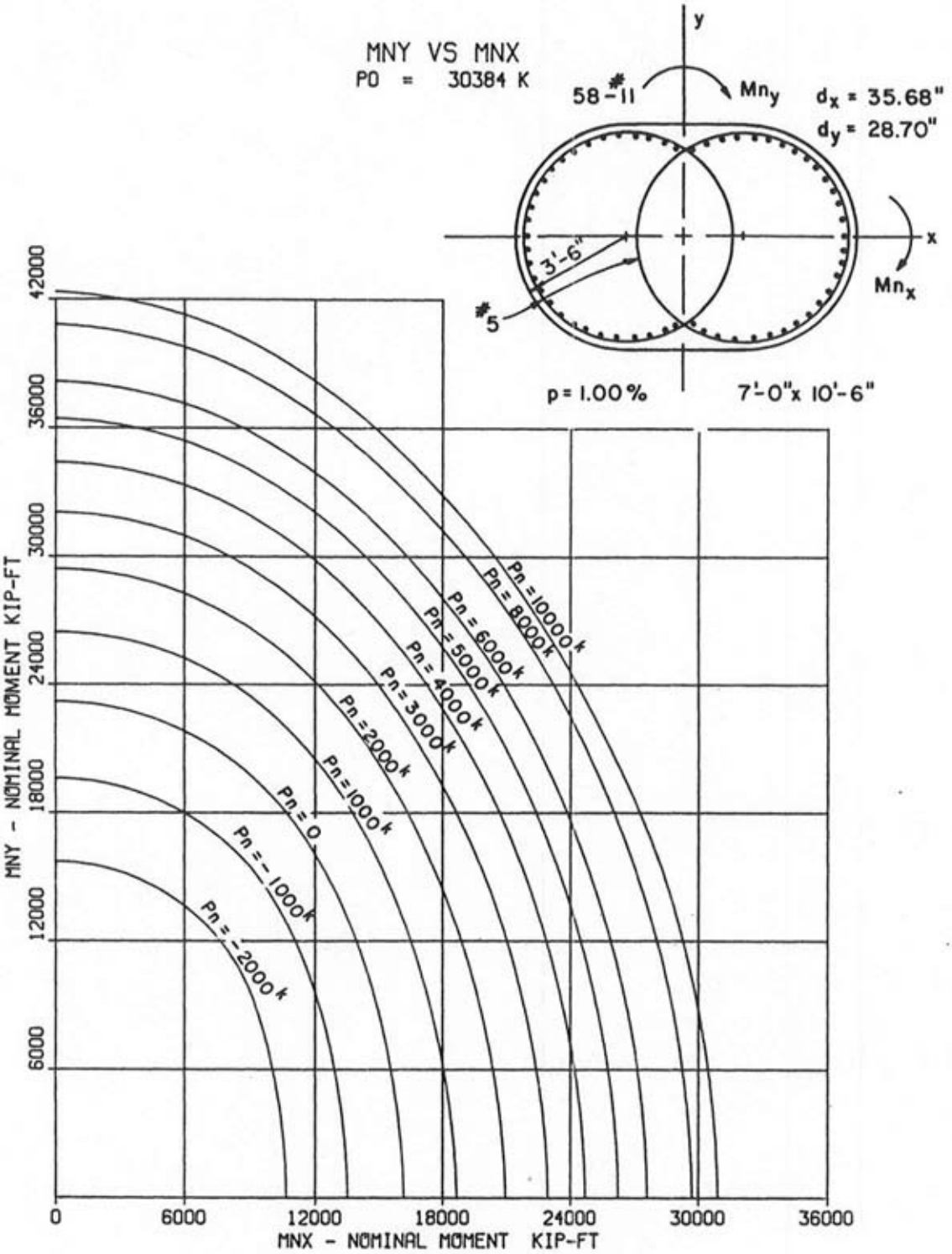
50-#14





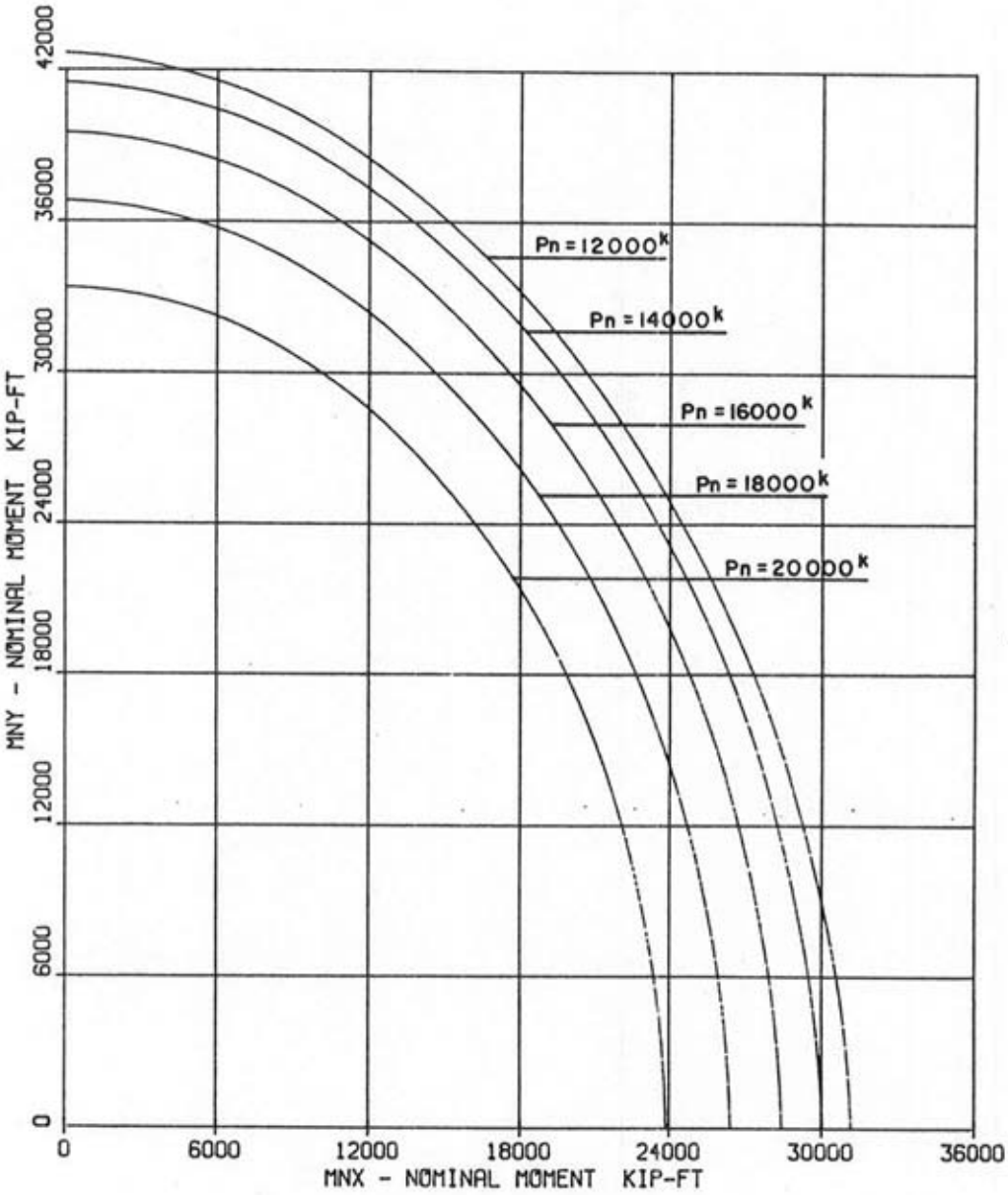
A = 38.9 ft²
I_x = 83.1 ft⁴
I_y = 175.6 ft⁴
I_z = 258.7 ft⁴

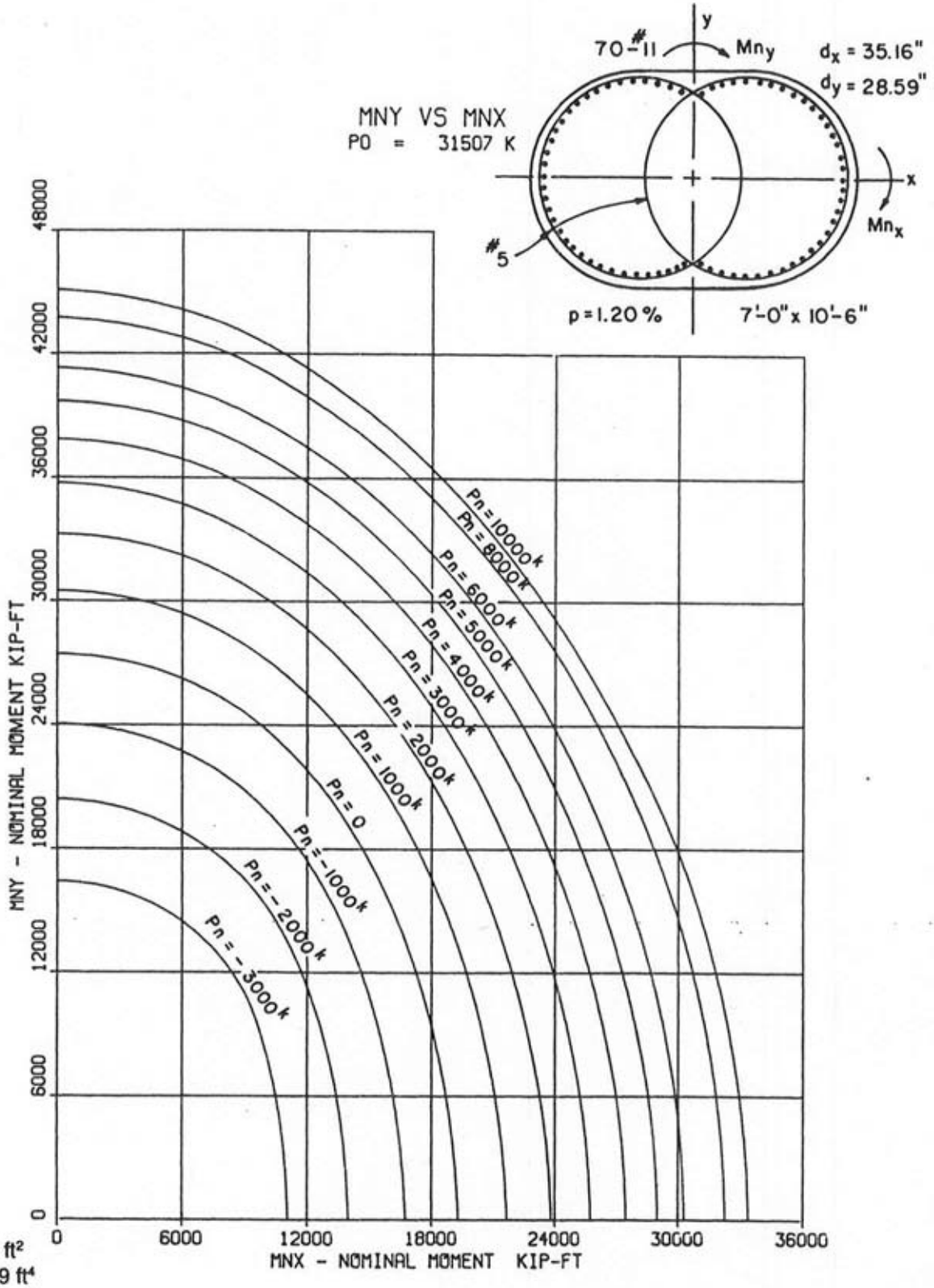

MNY VS MNX
PO = 24047 K
5'-6"x 8'-3"
36 - #18
Pn = 7000k
Pn = 8000k
Pn = 9000k
Pn = 10000k
Pn = 11000k
Pn = 12000k
MNY - NOMINAL MOMENT KIP-FT
MNX - NOMINAL MOMENT KIP-FT
0
4000
8000
12000
16000
20000
24000
28000
32000



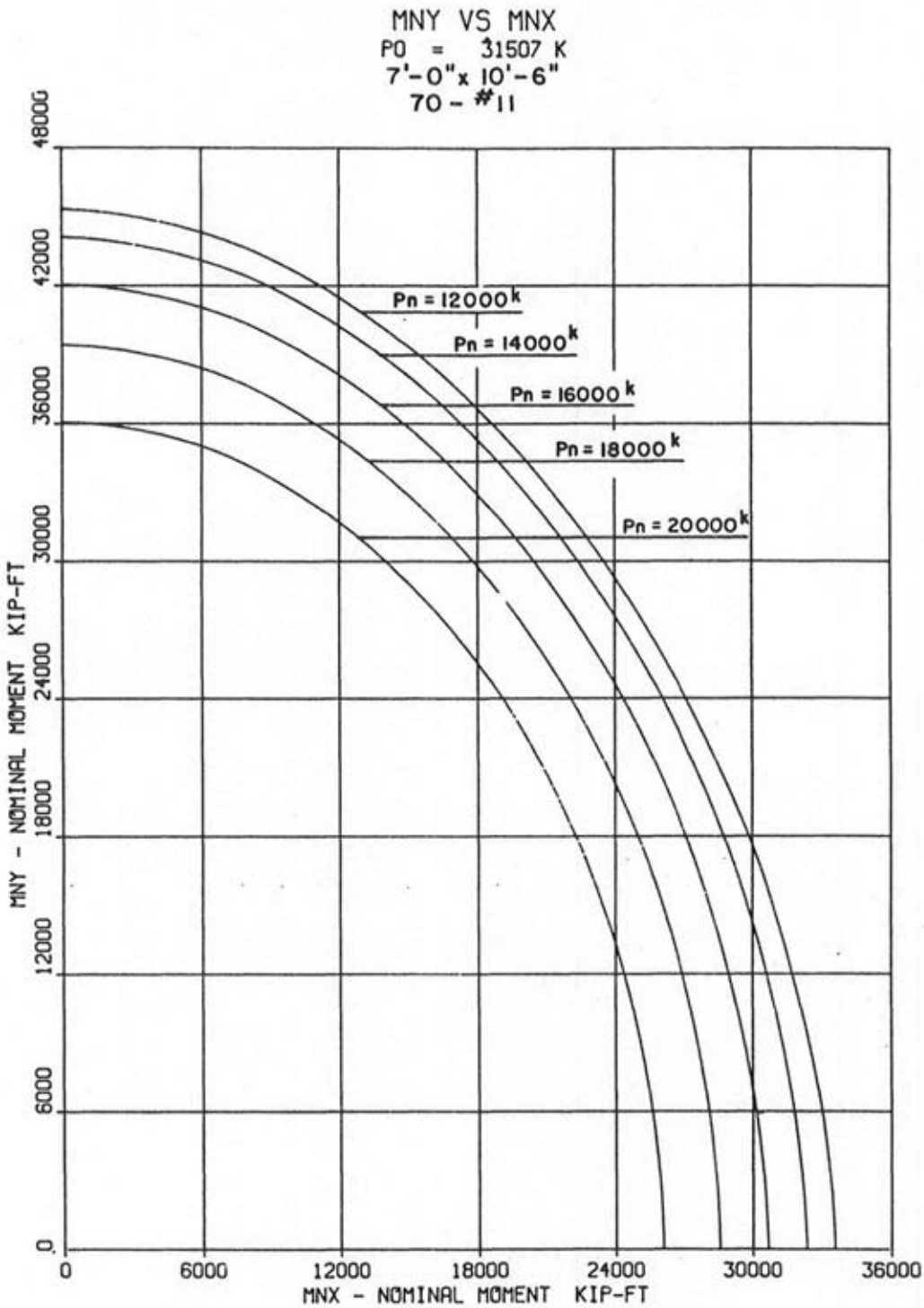
A = 63.0 ft²
I_x = 217.9 ft⁴
I_y = 460.8 ft⁴
I_z = 678.7 ft⁴

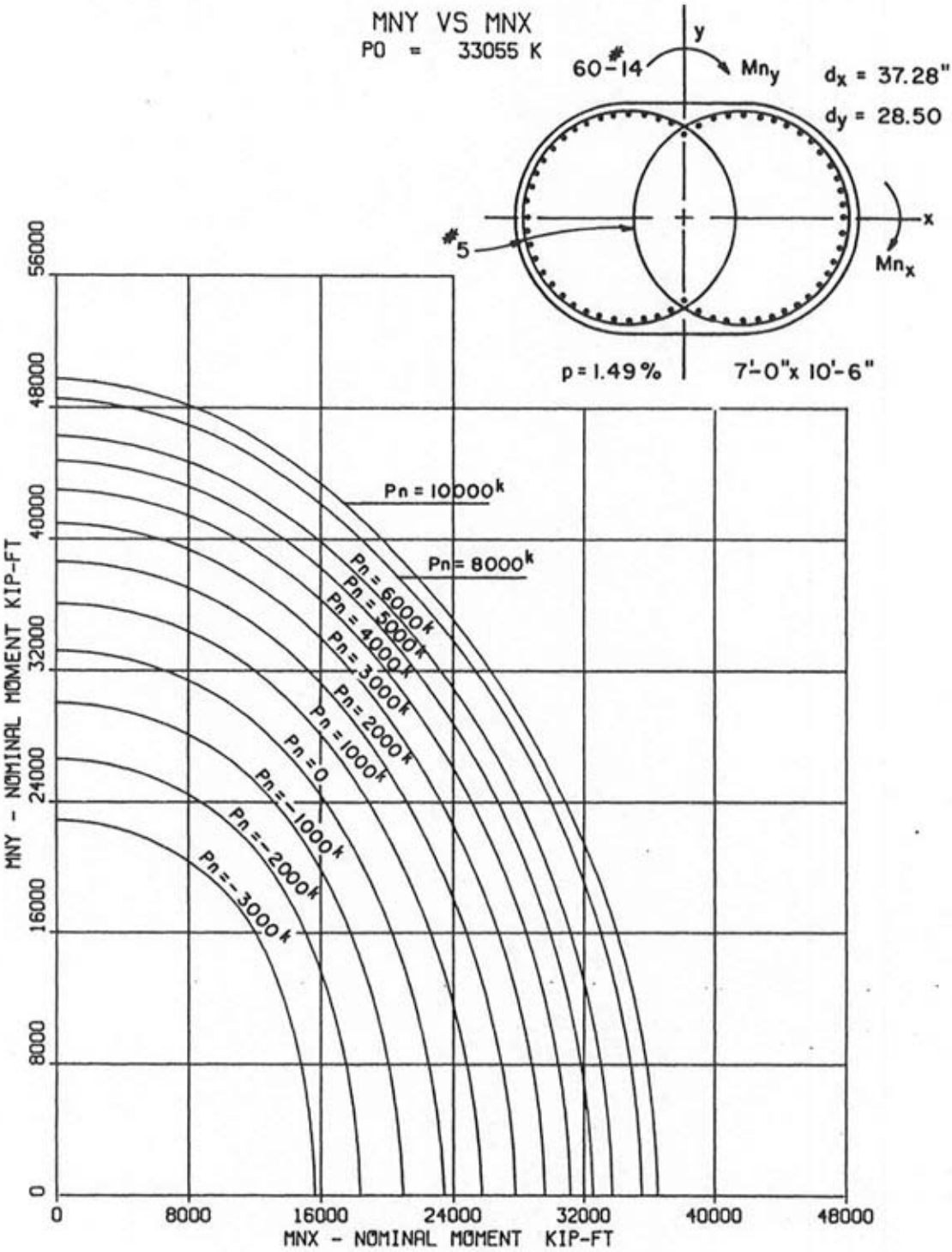
MNY VS MNX
PO = 30384 K
7'-0" x 10'-6"
58 - #11





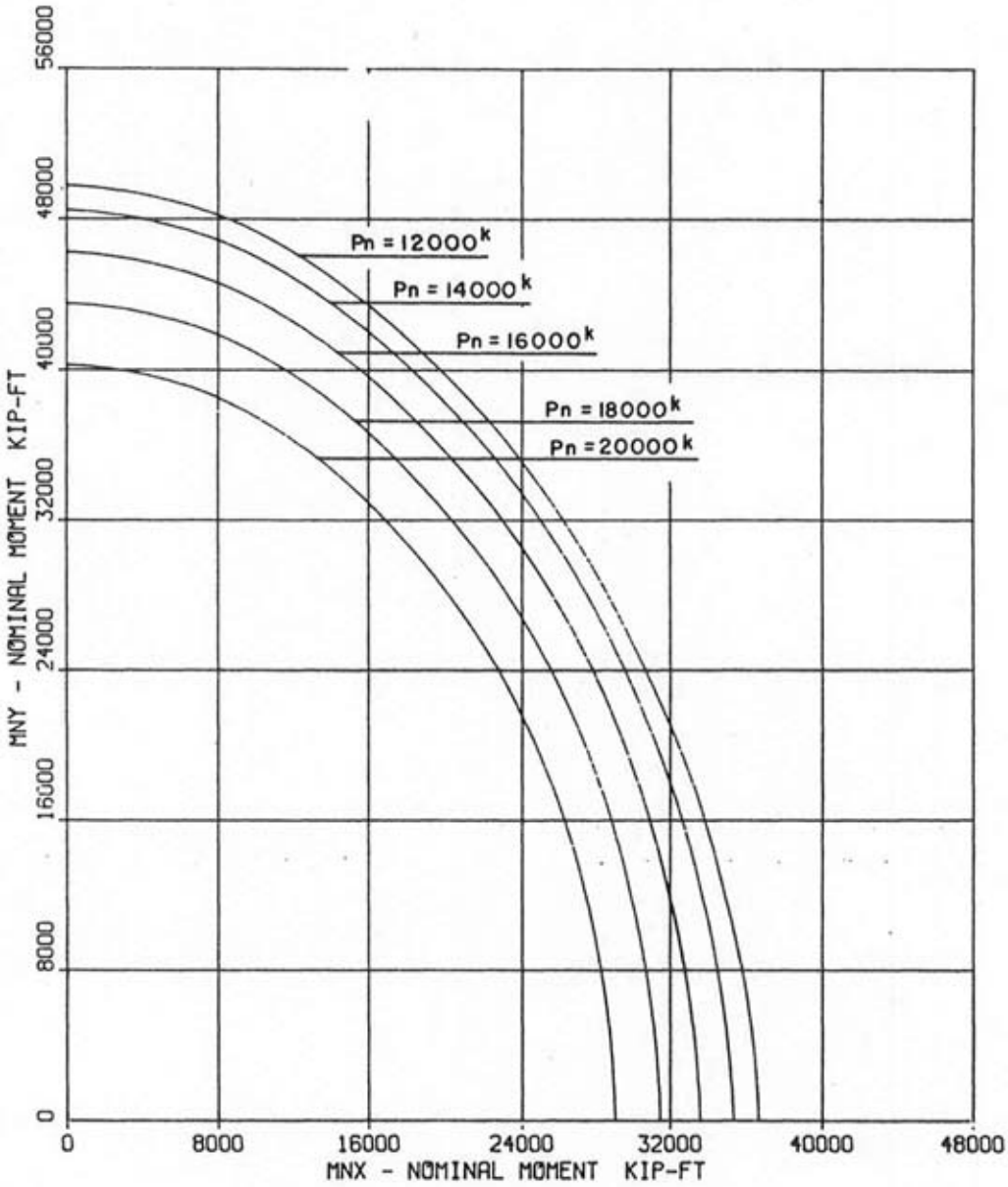
A = 63.0 ft²
I_x = 217.9 ft⁴
I_y = 460.8 ft⁴
I_z = 678.7 ft⁴

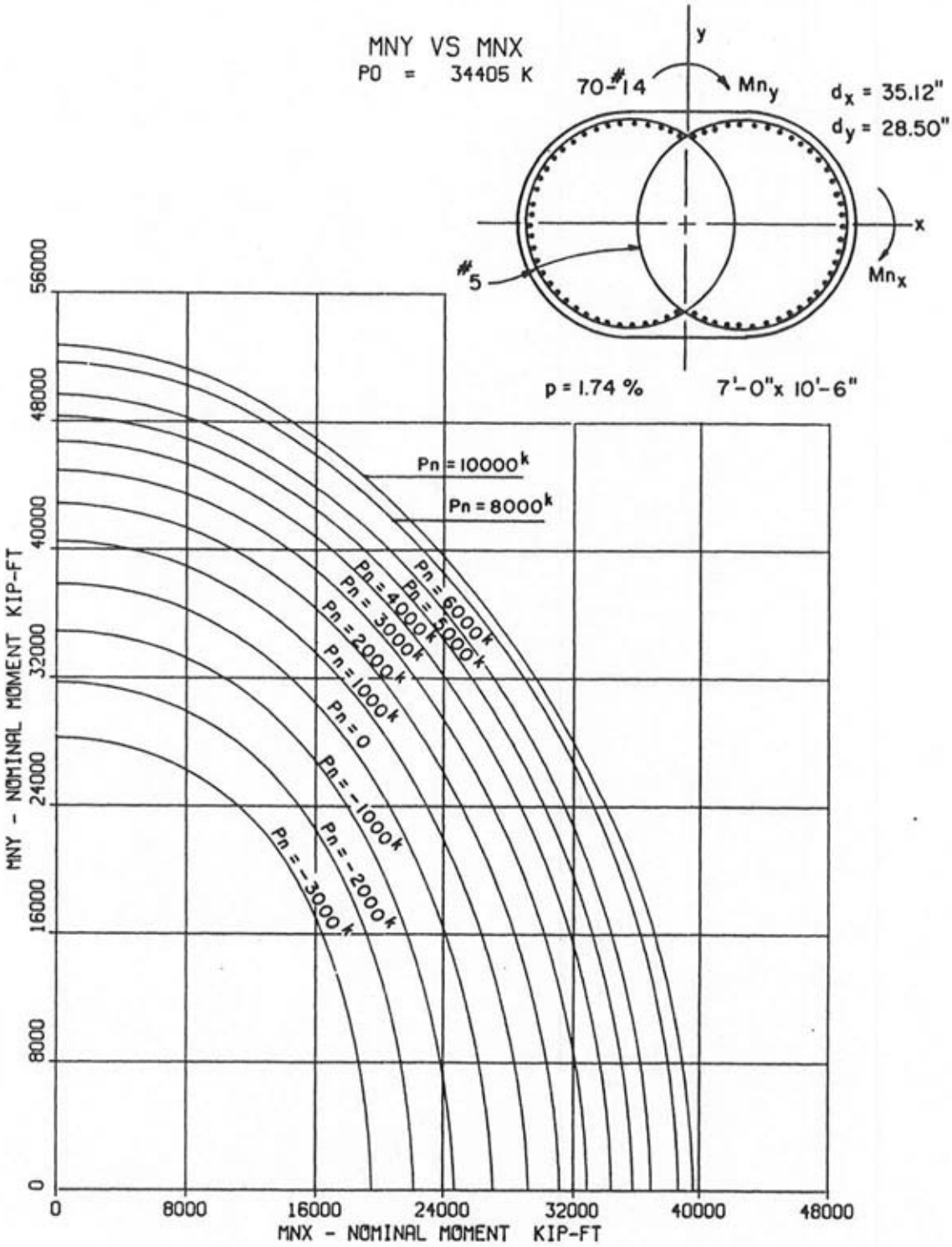

MNY VS MNX
PO = 31507 K
7'-0" x 10'-6"
70 - #11
Pn = 12000k
Pn = 14000k
Pn = 16000k
Pn = 18000k
Pn = 20000k
MNY - NOMINAL MOMENT KIP-FT
MNX - NOMINAL MOMENT KIP-FT
0
6000
12000
18000
24000
30000
36000
42000
48000



A = 63.0 ft²
I_x = 217.9 ft⁴
I_y = 460.8 ft⁴
I_z = 678.7 ft⁴

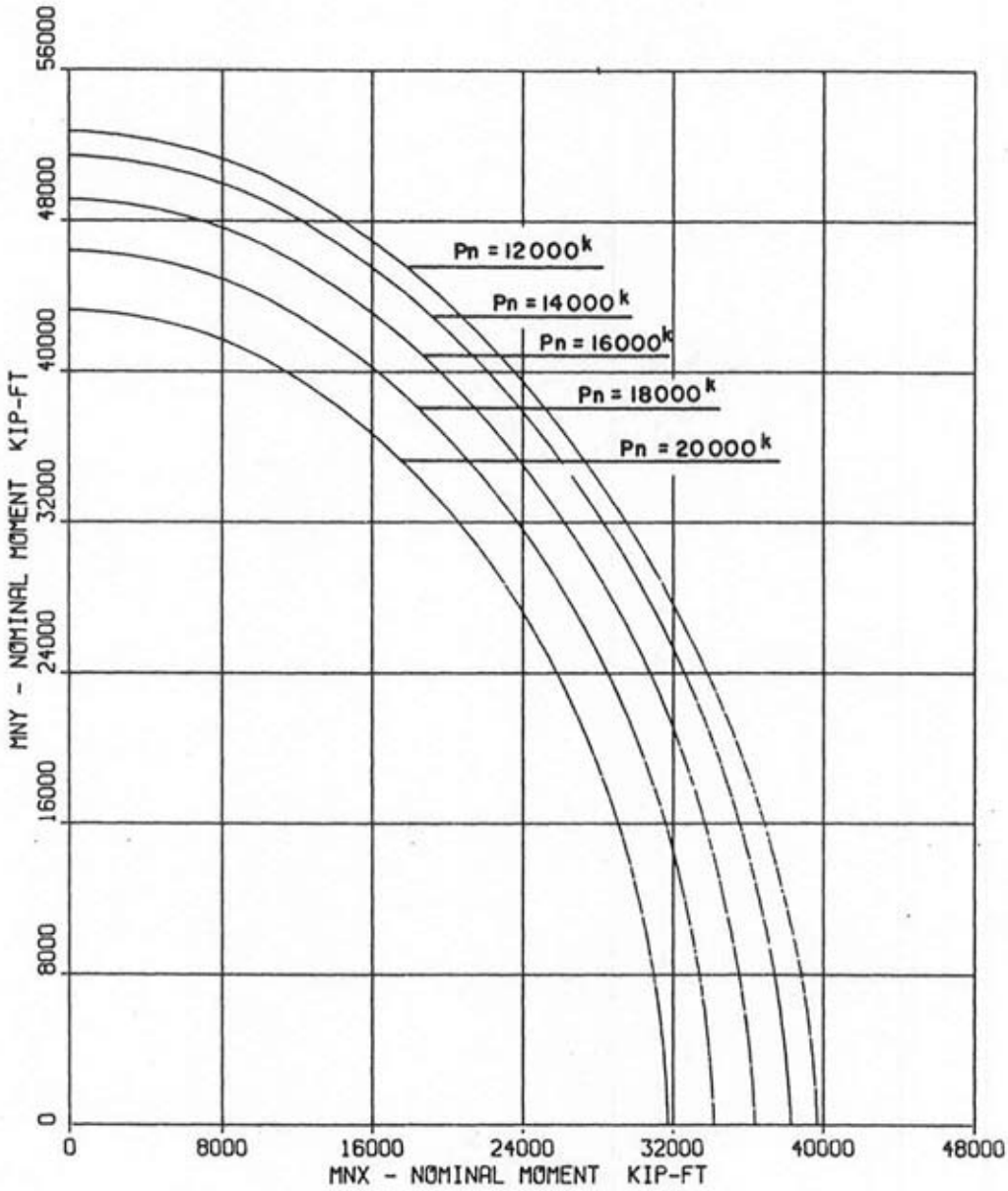
MNY VS MNX
PO = 33055 K
7'-0" x 10'-6"
60 - # 14

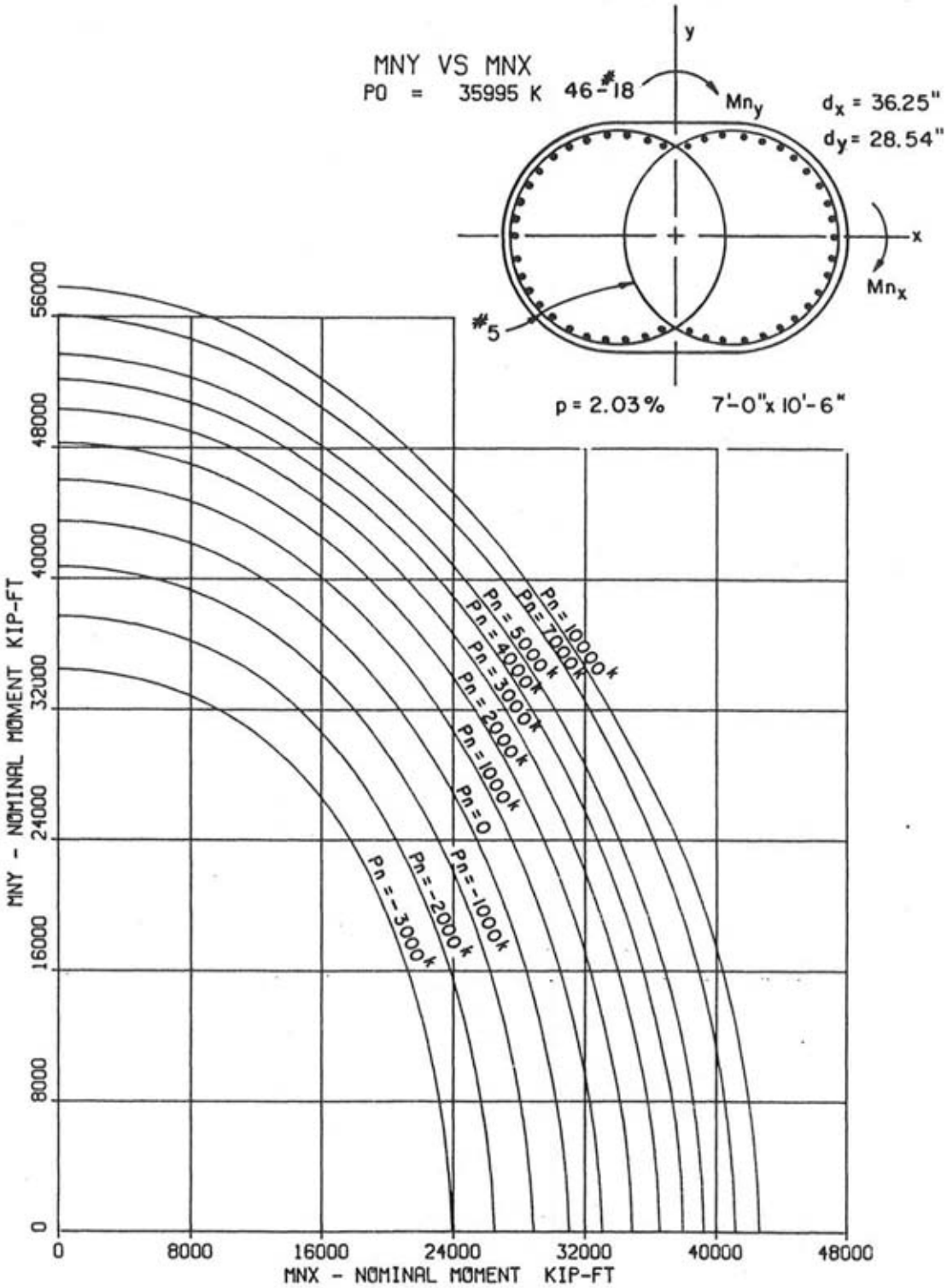




A = 63.0 ft²
I_x = 217.9 ft⁴
I_y = 460.8 ft⁴
I_z = 678.7 ft⁴

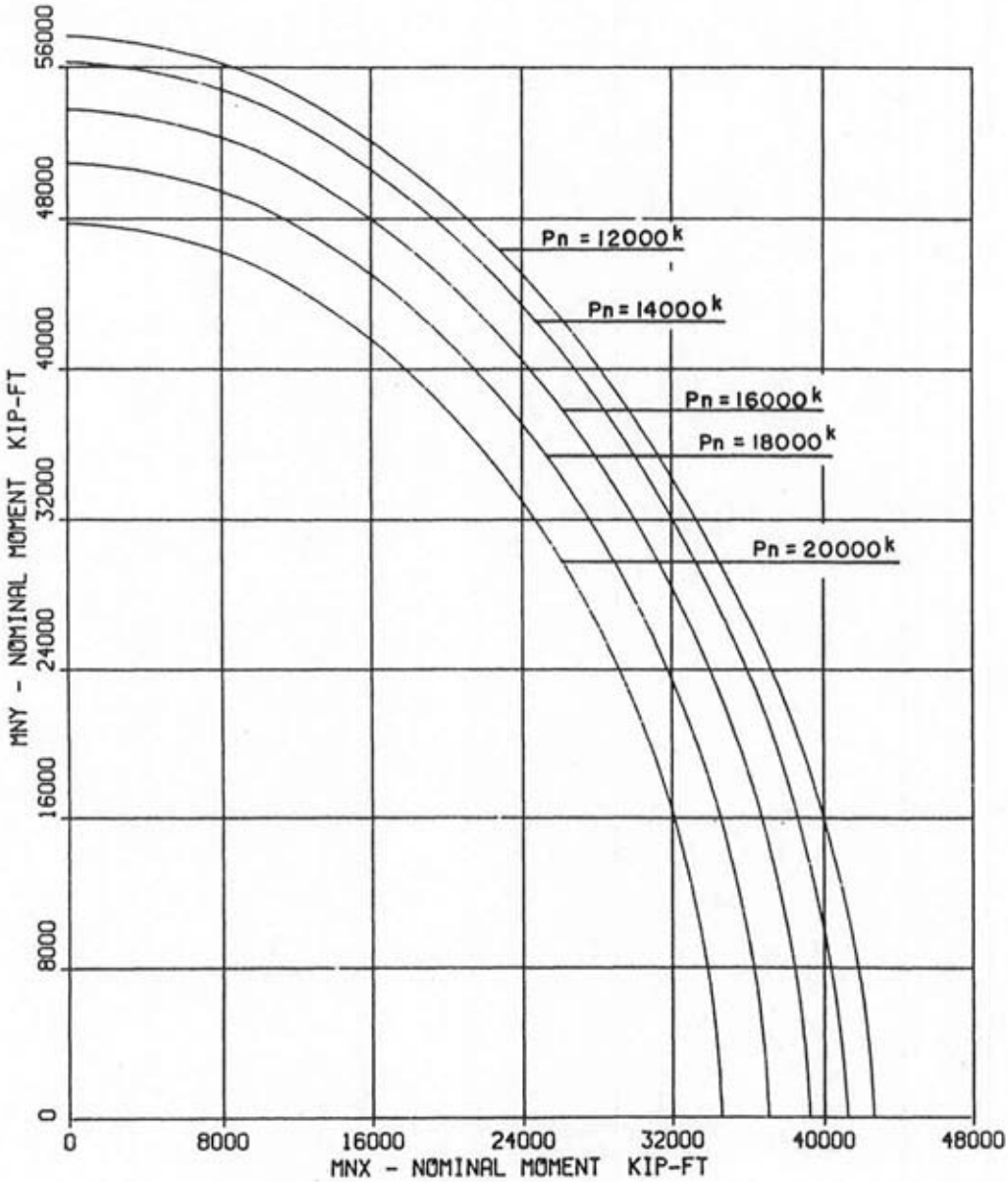
MNY VS MNX
PO = 34405 K
7'-0" x 10'-6"
70 - #14

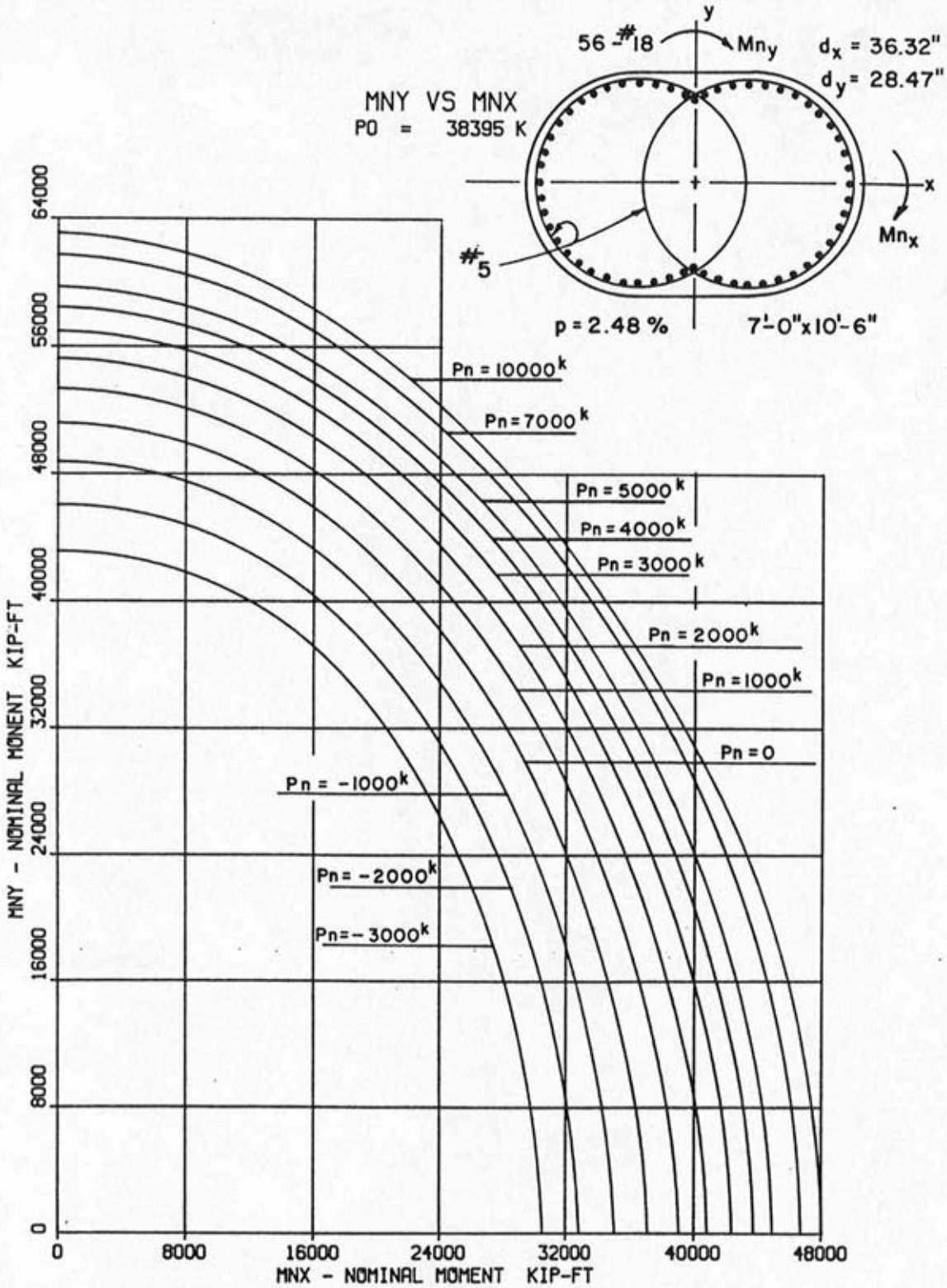




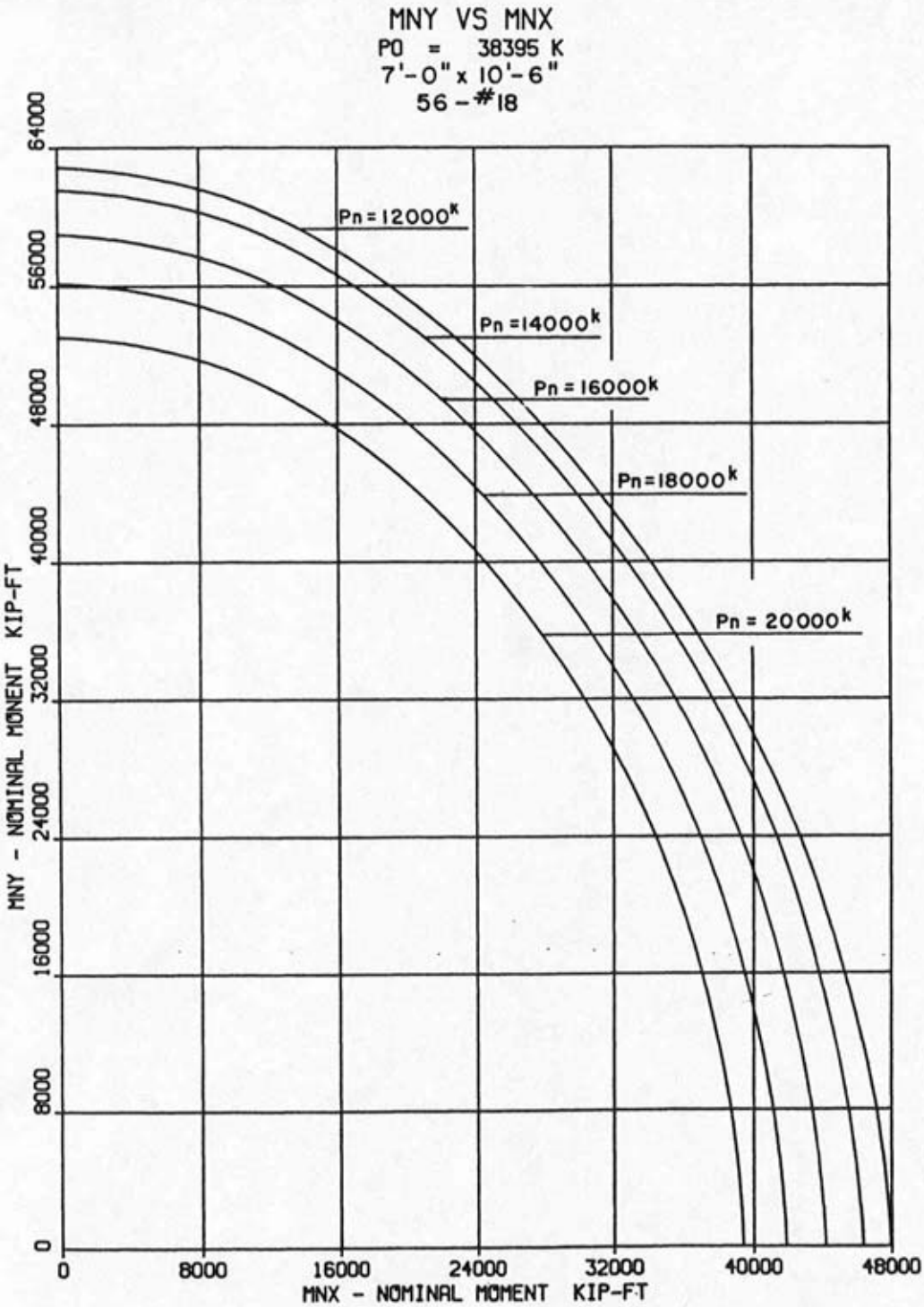
A = 63.0 ft²
I_x = 217.9 ft⁴
I_y = 460.8 ft⁴
I_z = 678.7 ft⁴

MNY VS MNX
PO = 35995 K
7'-0" x 10'-6"
46 – #18





$A = 63.0\ ft^2$
$I_x = 217.9\ ft^4$
$I_y = 460.8\ ft^4$
$I_z = 678.7\ ft^4$


MNY VS MNX
PO = 38395 K
7'-0" x 10'-6"
56 - #18
Pn = 12000k
Pn = 14000k
Pn = 16000k
Pn = 18000k
Pn = 20000k
MNY - NOMINAL MOMENT KIP-FT
MNX - NOMINAL MOMENT KIP-FT
0
8000
16000
24000
32000
40000
48000
56000
64000