

## REFERENCES



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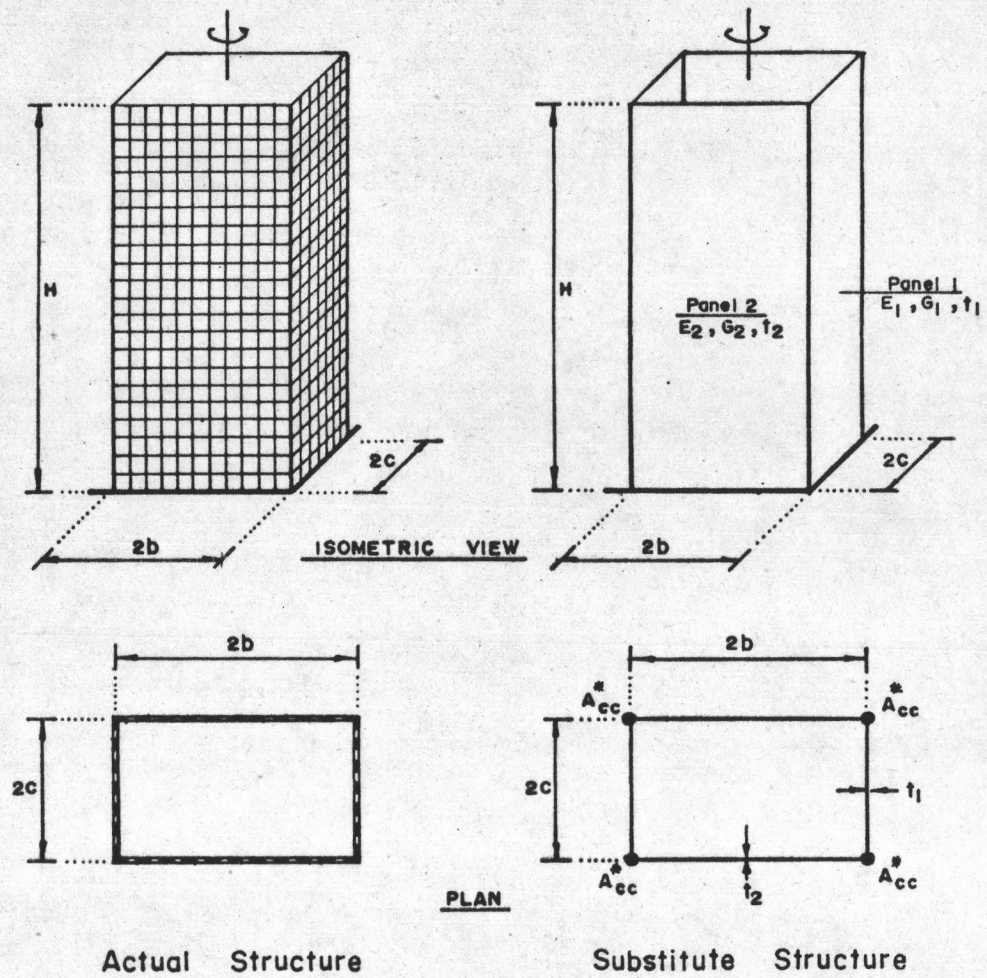


FIG. 1. Framed-Tube: Actual and Substitute Structures

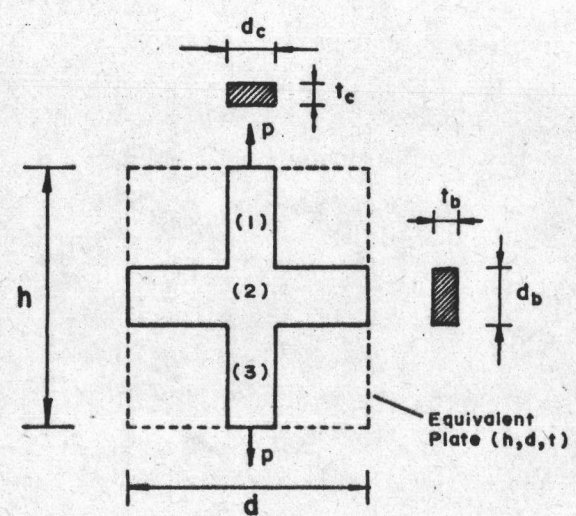


FIG. 2. Model for Determination of  $E_z$

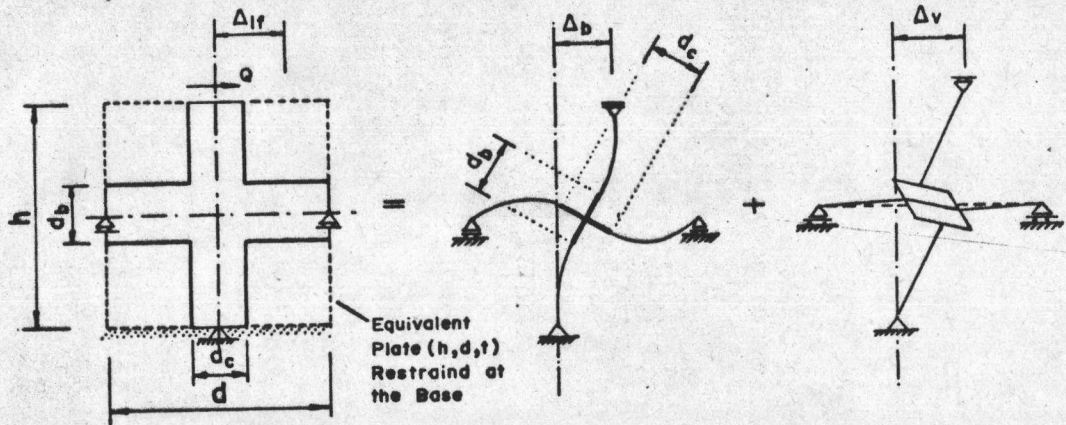


FIG. 3. Model for Determination of  $G_{zs}$

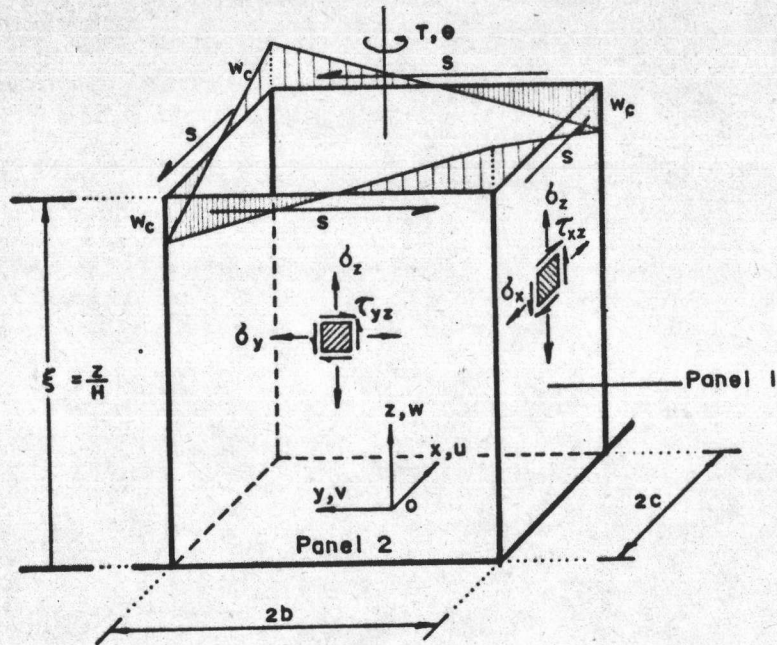


FIG. 4. Sign Conventions

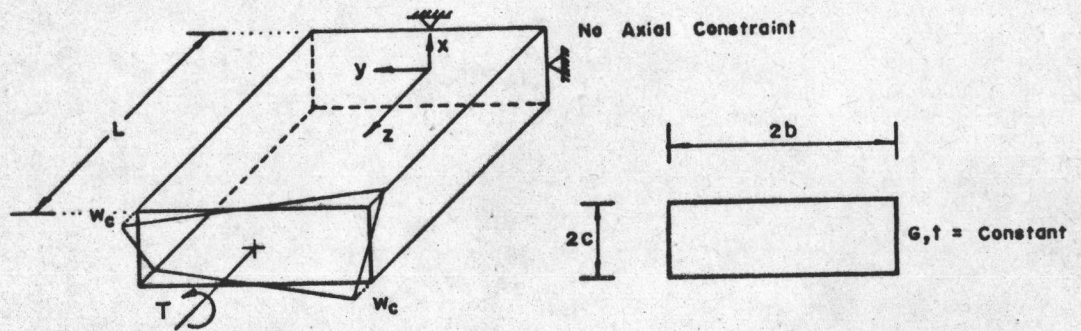


FIG. 5. Thin-Walled Cantiliver Box-Section



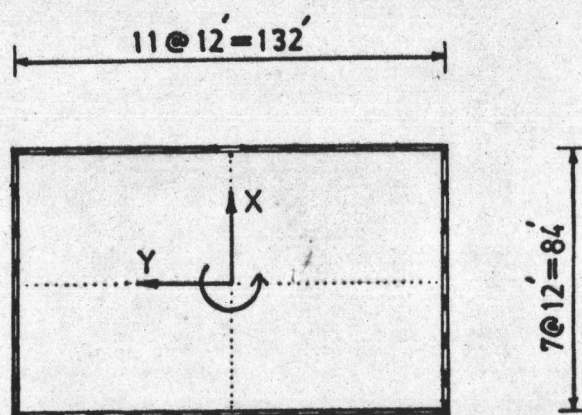


FIG. 6. Example 2: Typical Floor Plan

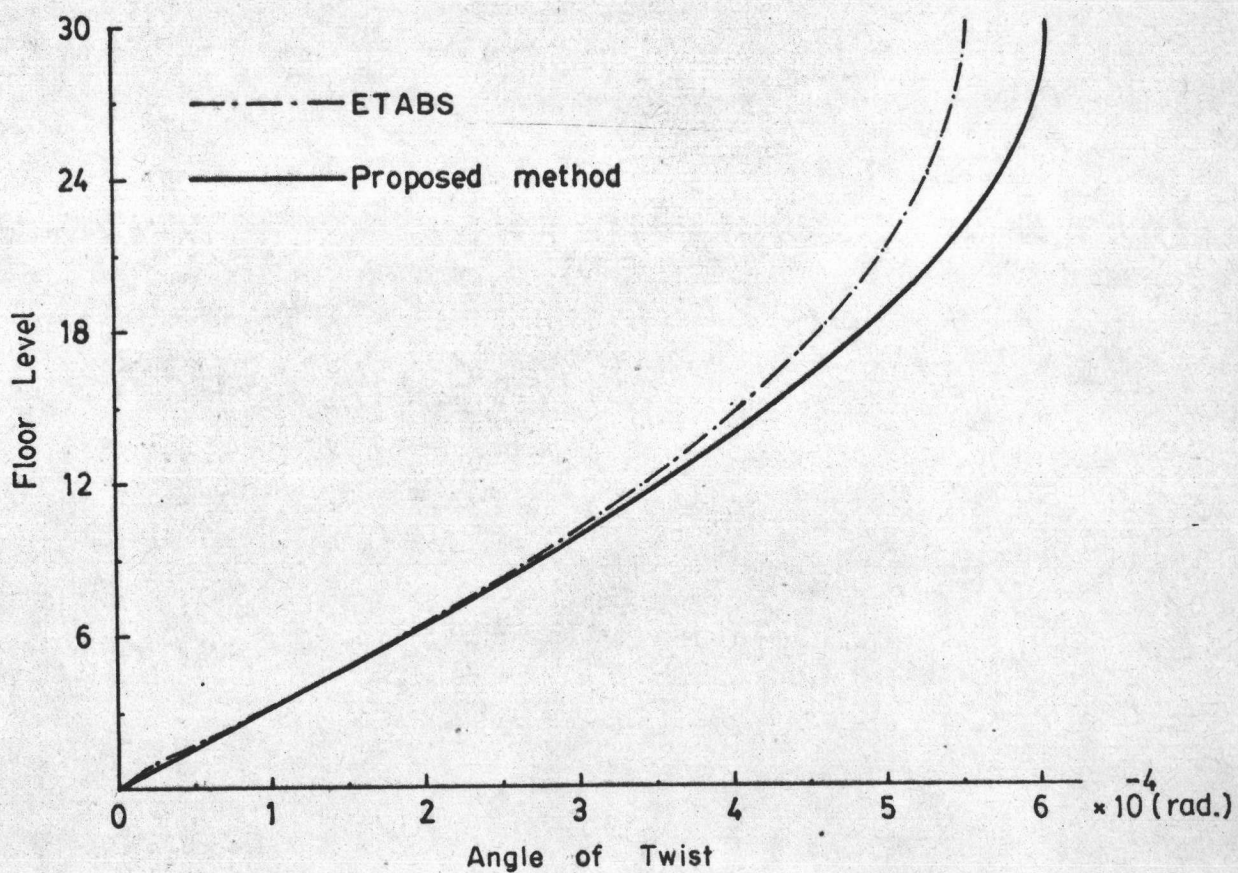


FIG. 7. Example 2: Twisting Angle

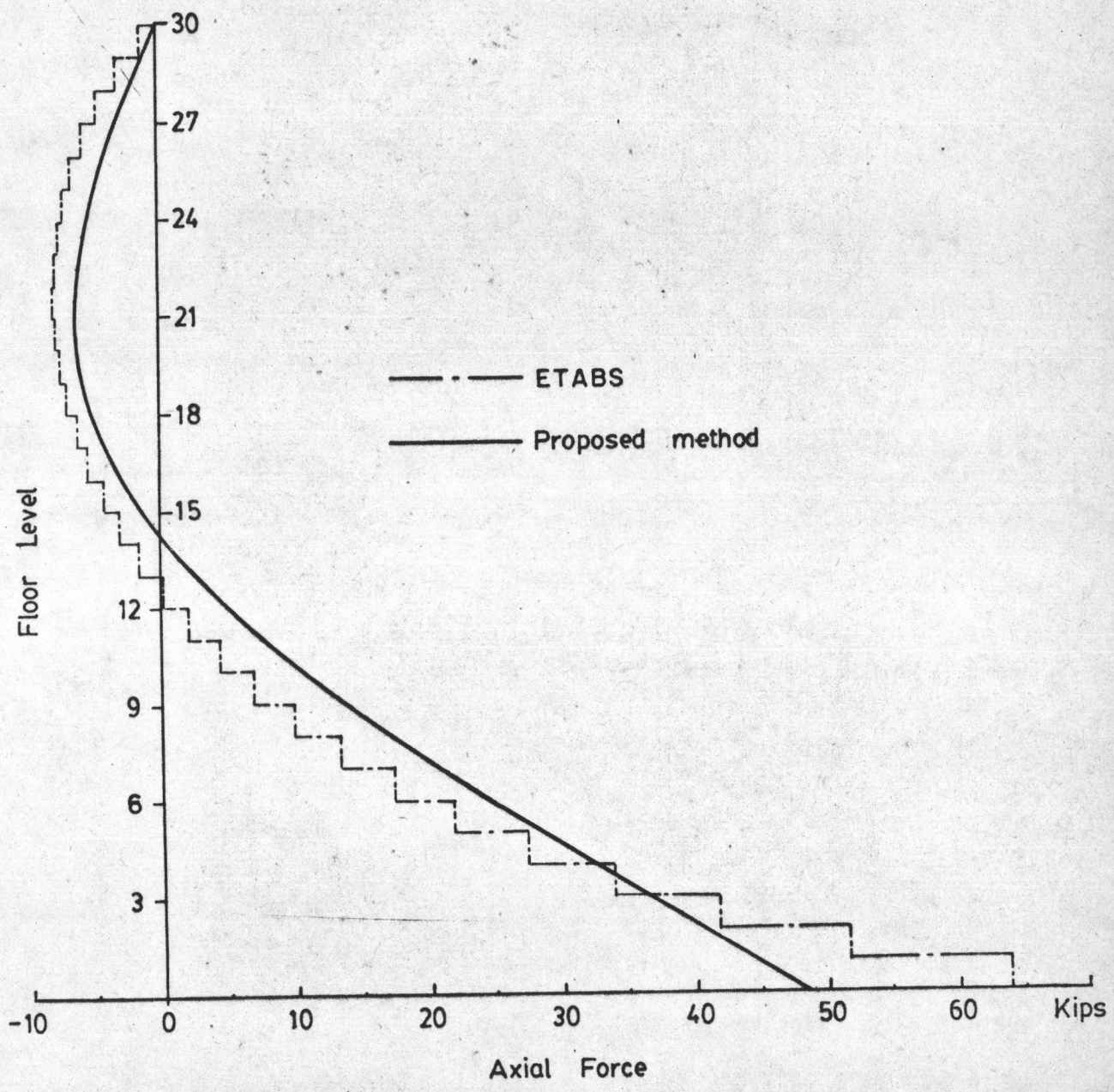
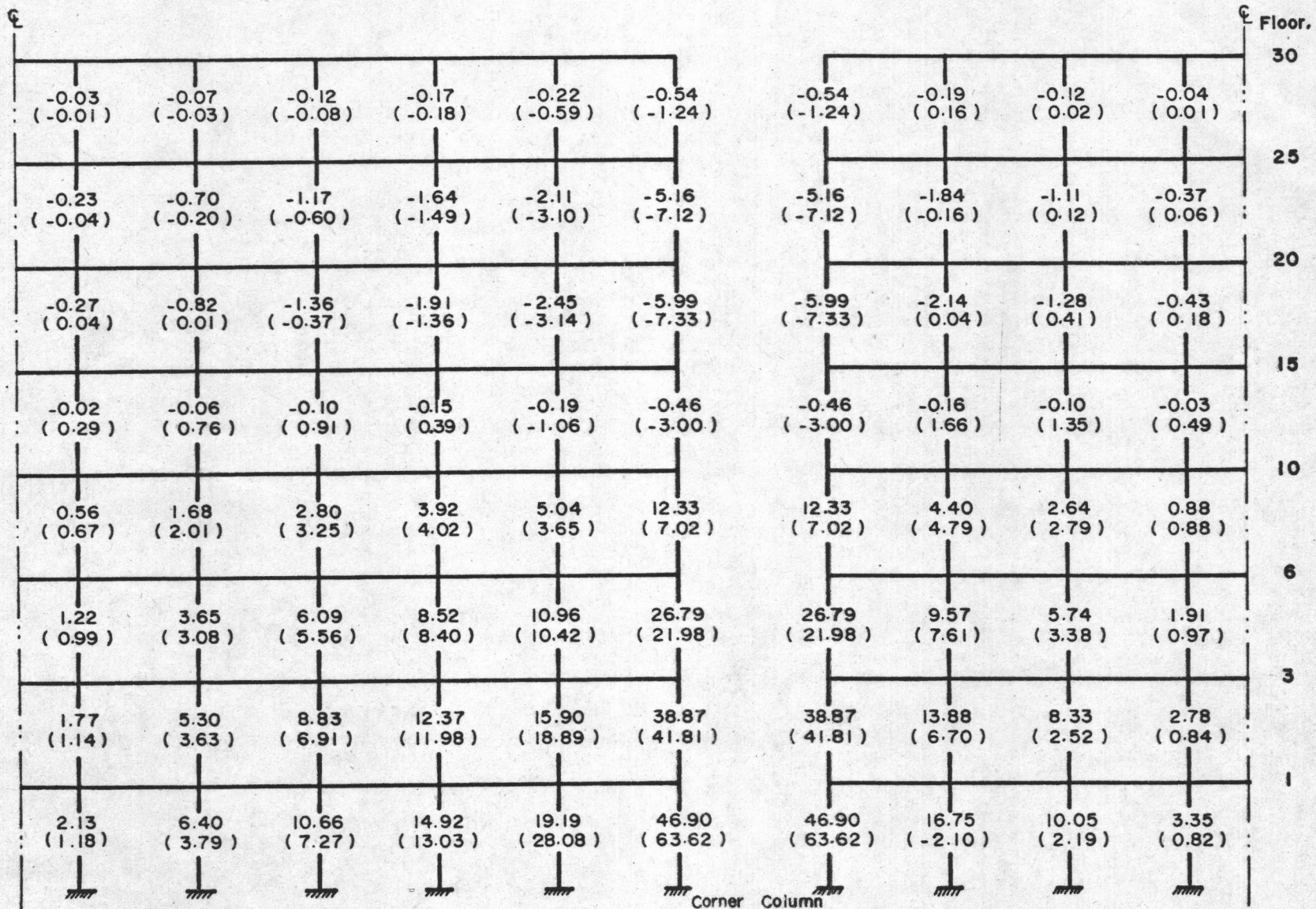


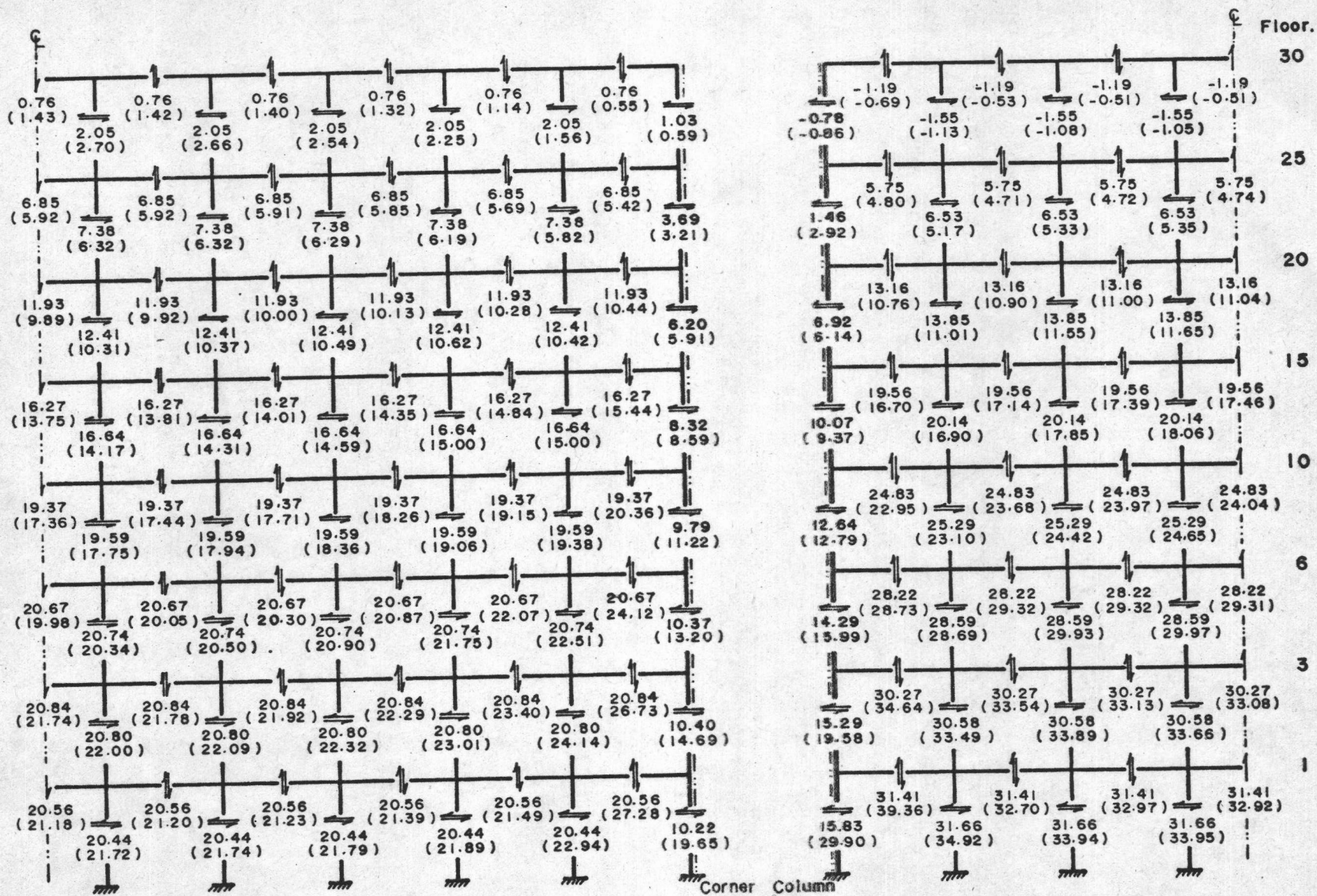
FIG. 8. Example 2: Variation of Axial Force in Corner Column





Note: Values in Parentheses are Results by ETABS

FIG. 9. Example 2: Axial Forces in Columns (Kips)



Note : Values in Parentheses are Results by ETABS

FIG. 10. Example 2: Shear Forces in Columns and Spandrel Beams (Kips)



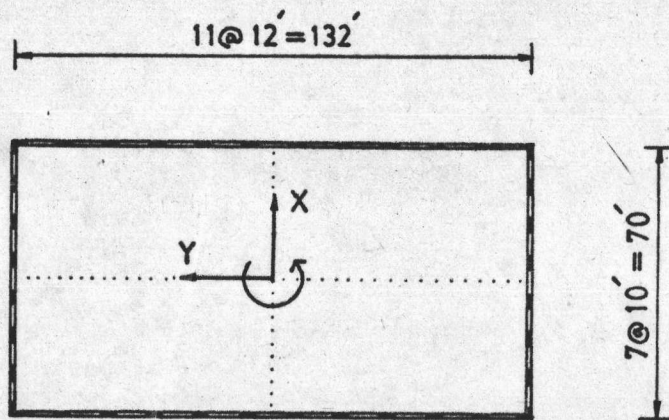


FIG.11. Example 3: Typical Floor Plan

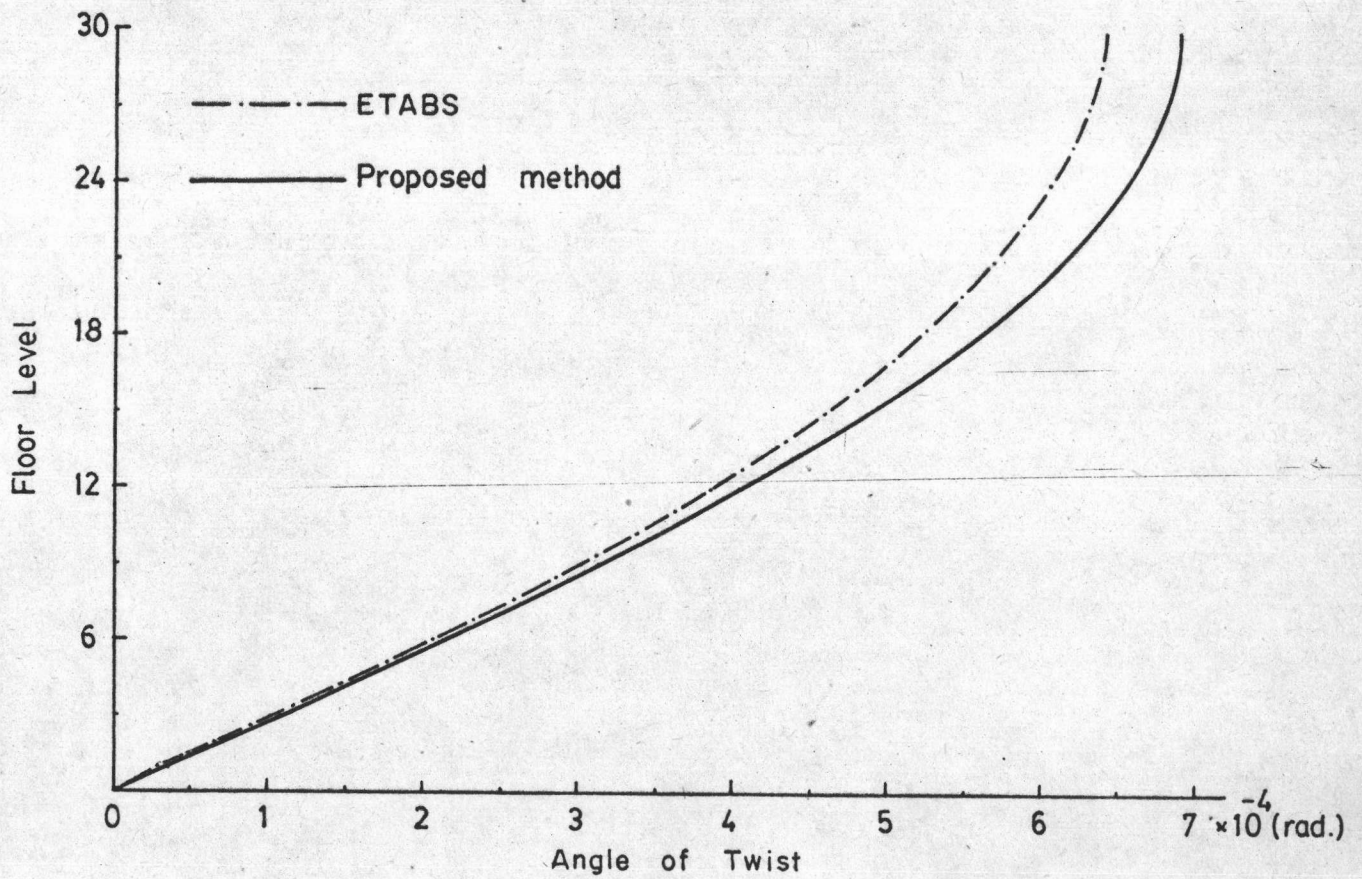


FIG.12. Example 3: Twisting Angle

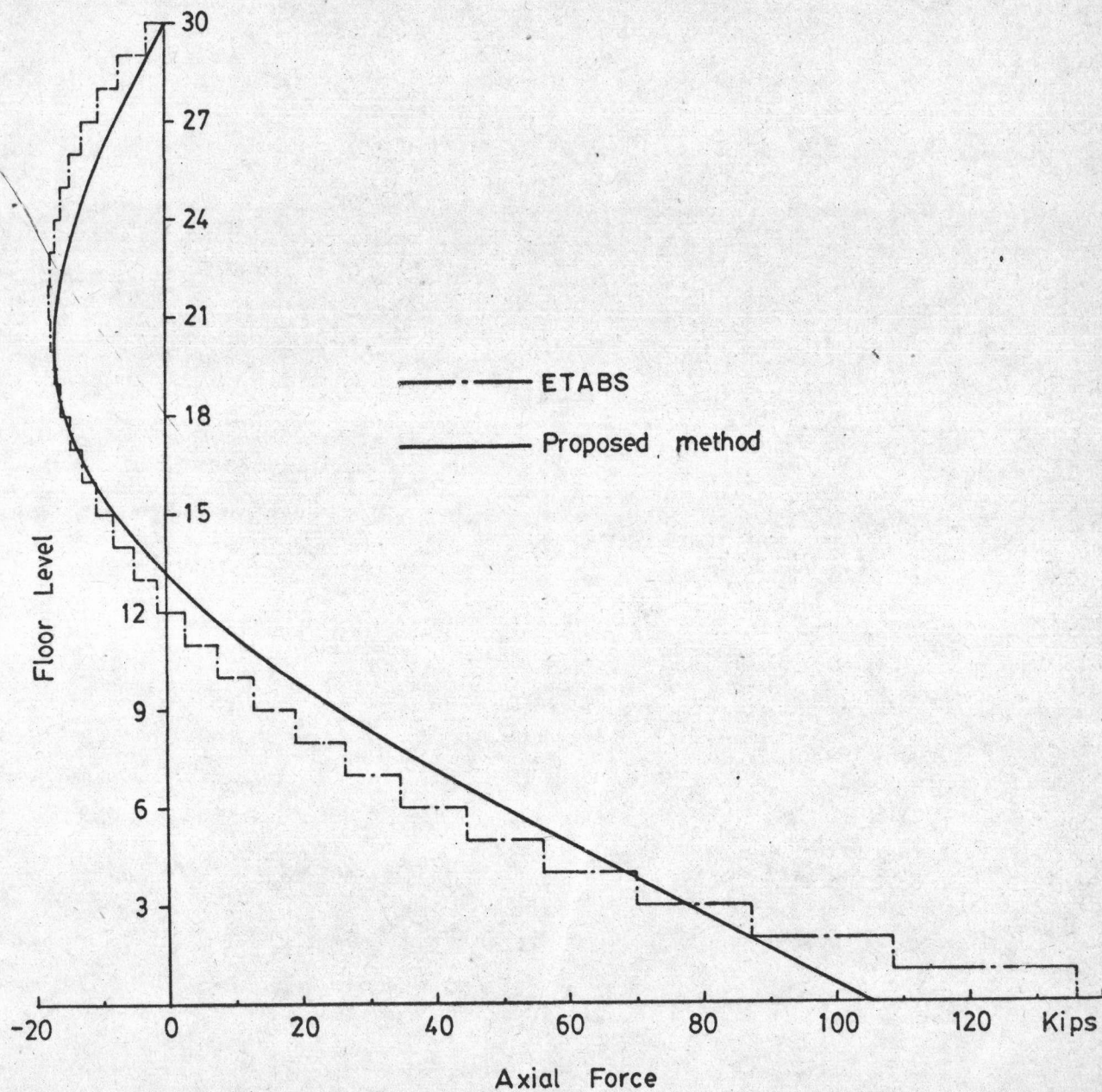


FIG.13. Example 3: Variation of Axial Force in Corner Column

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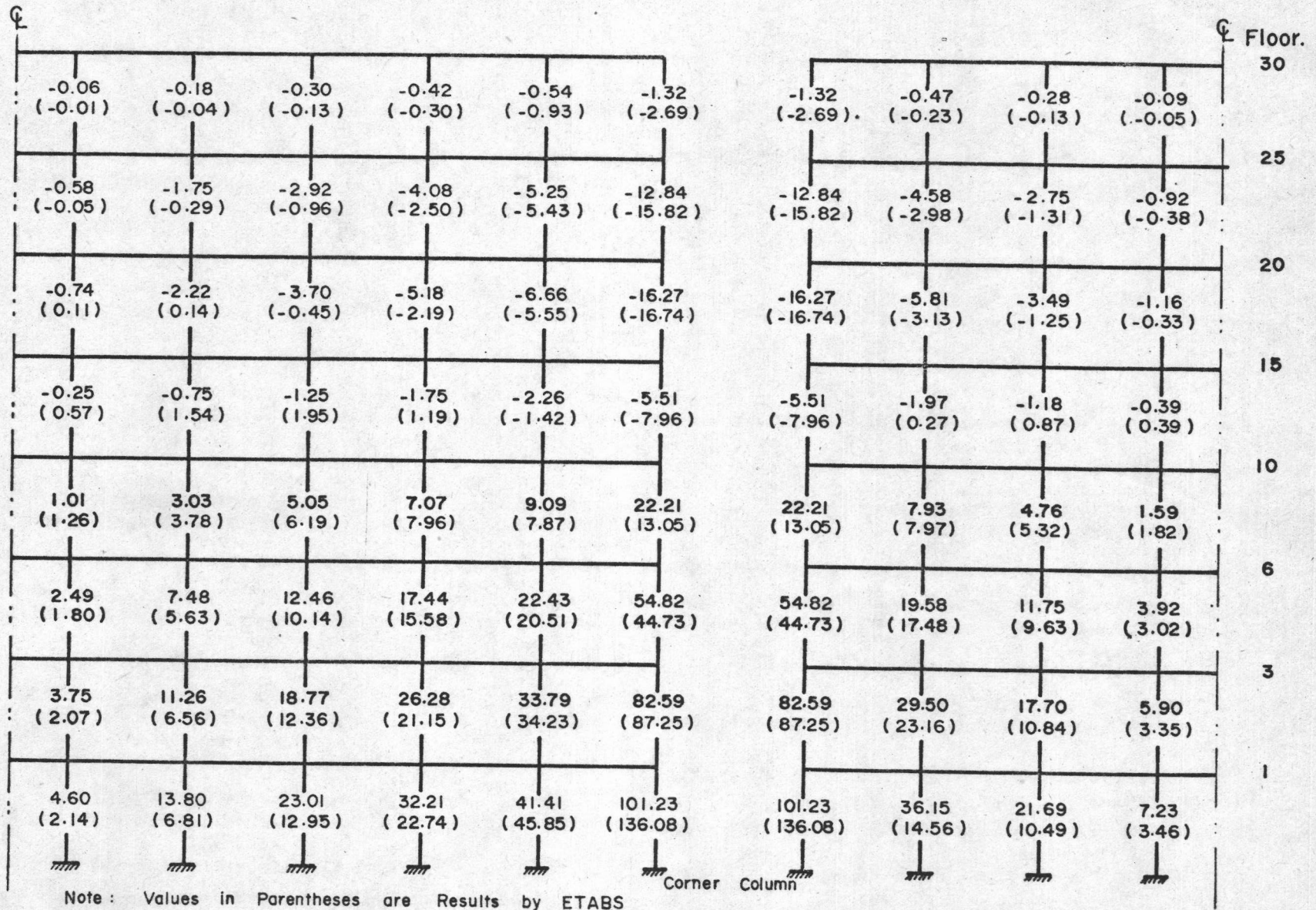
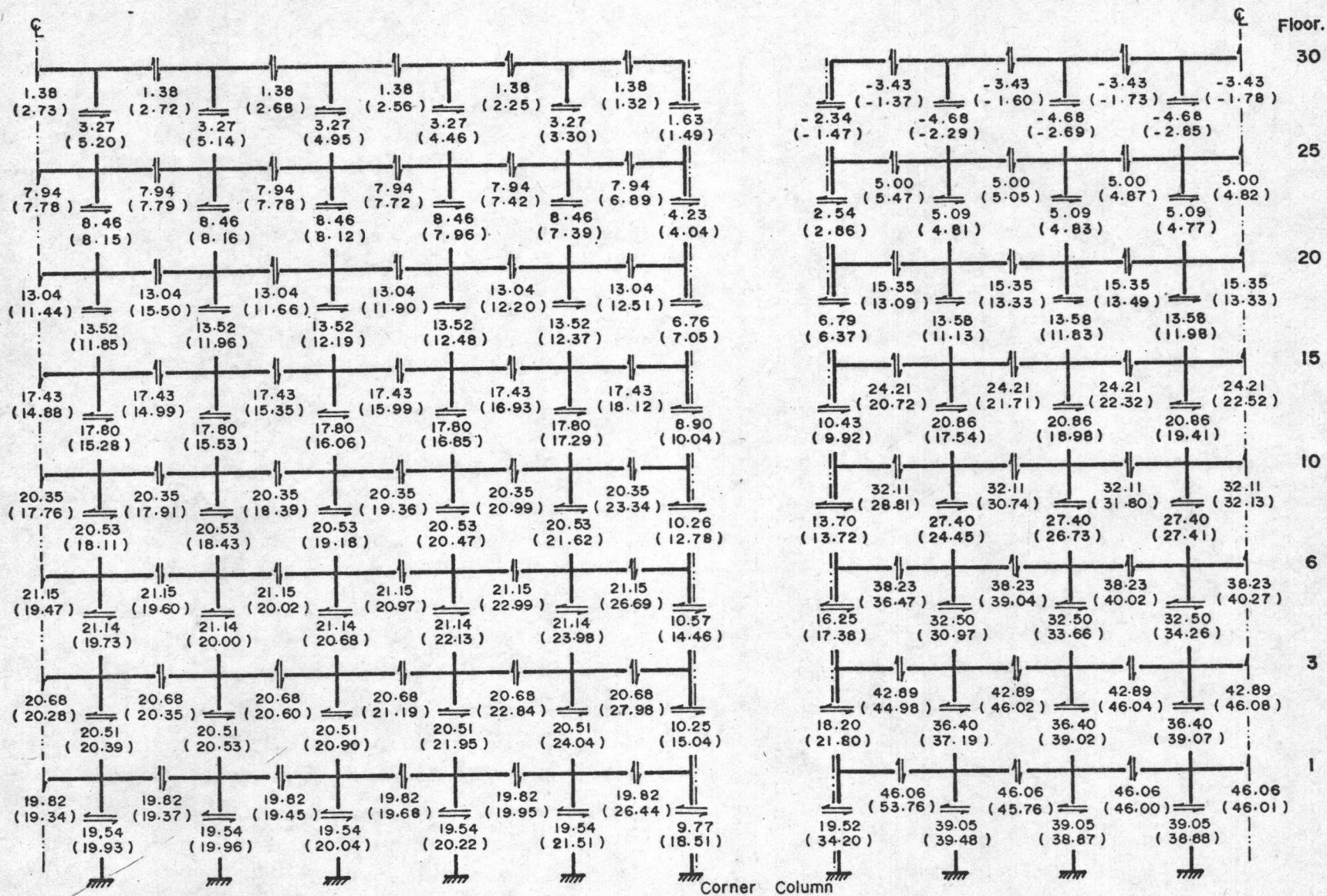


FIG.14. Example 3: Axial Forces in Columns (Kips)



Note: Values in Parentheses are Results by ETABS

FIG.15. Example 3: Shear Forces in Columns and Spandrel Beams (Kips)



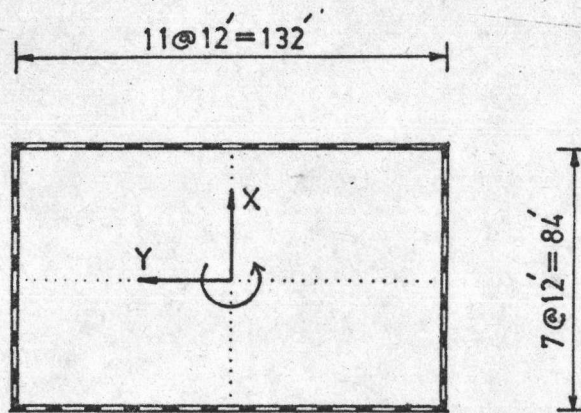


FIG.16. Example 4: Typical Floor Plan

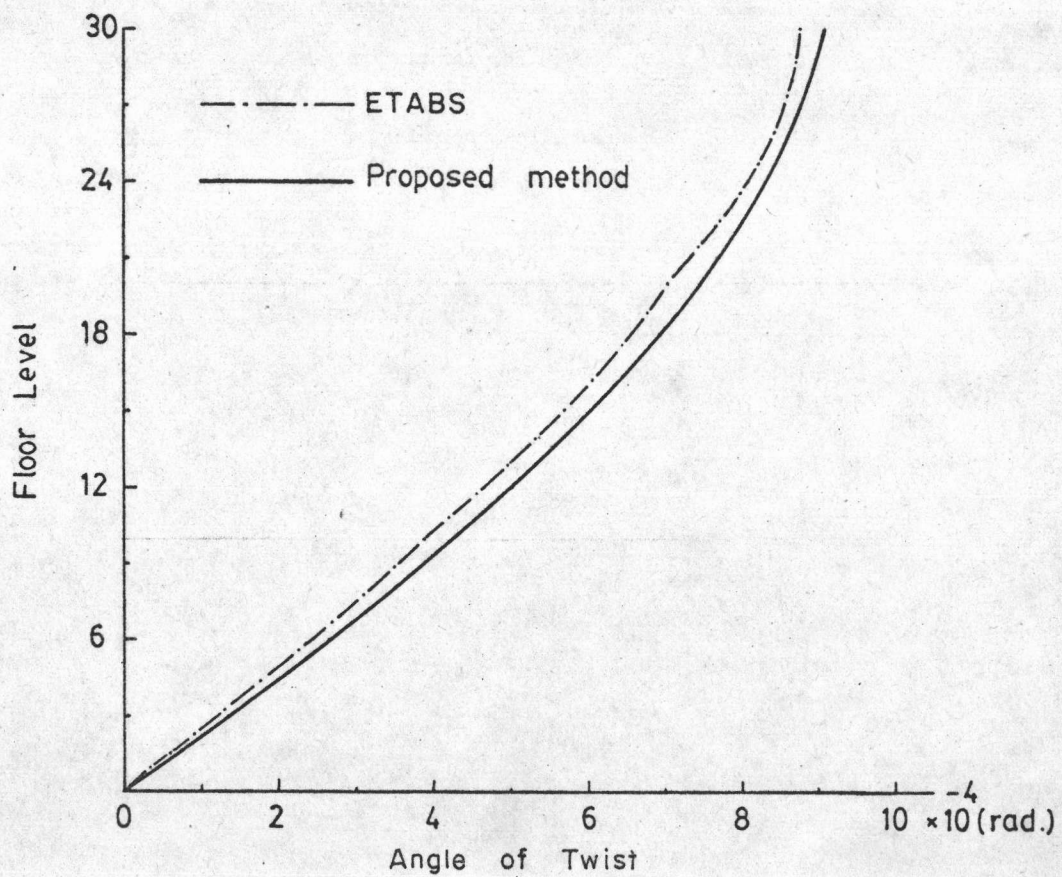


FIG.17. Example 4: Twisting Angle

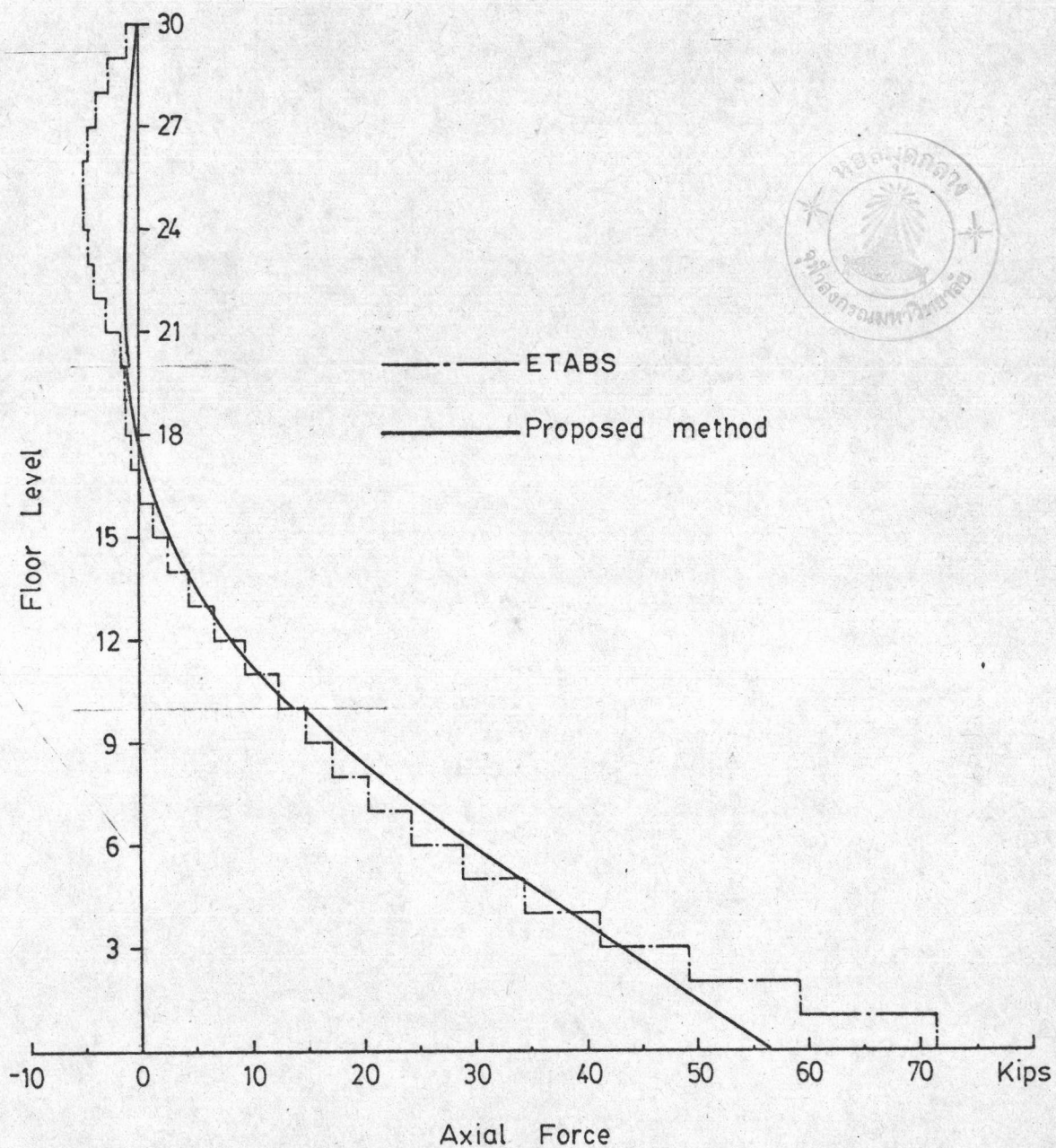


FIG. 18. Example 4: Variation of Axial Force in Corner Column



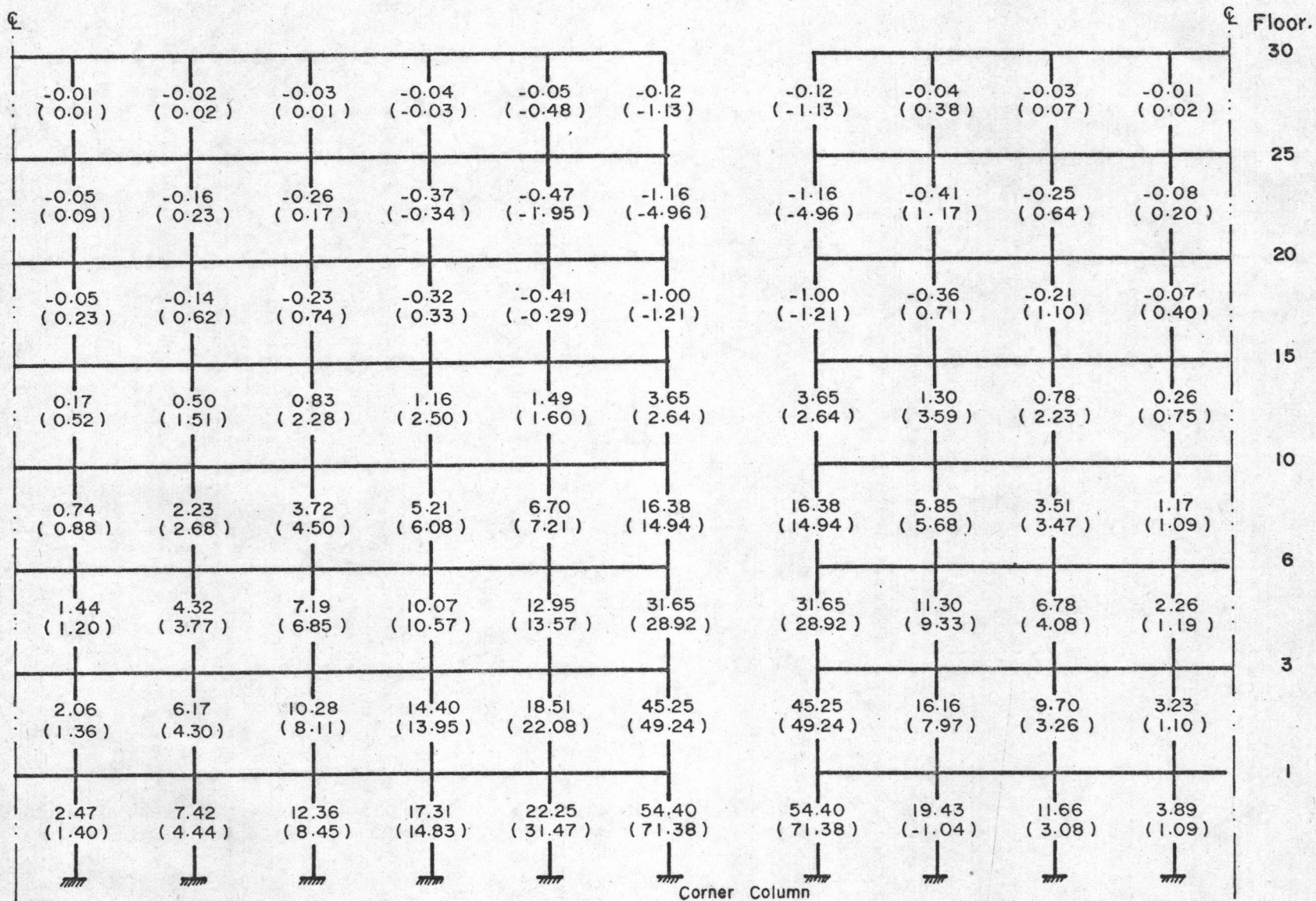
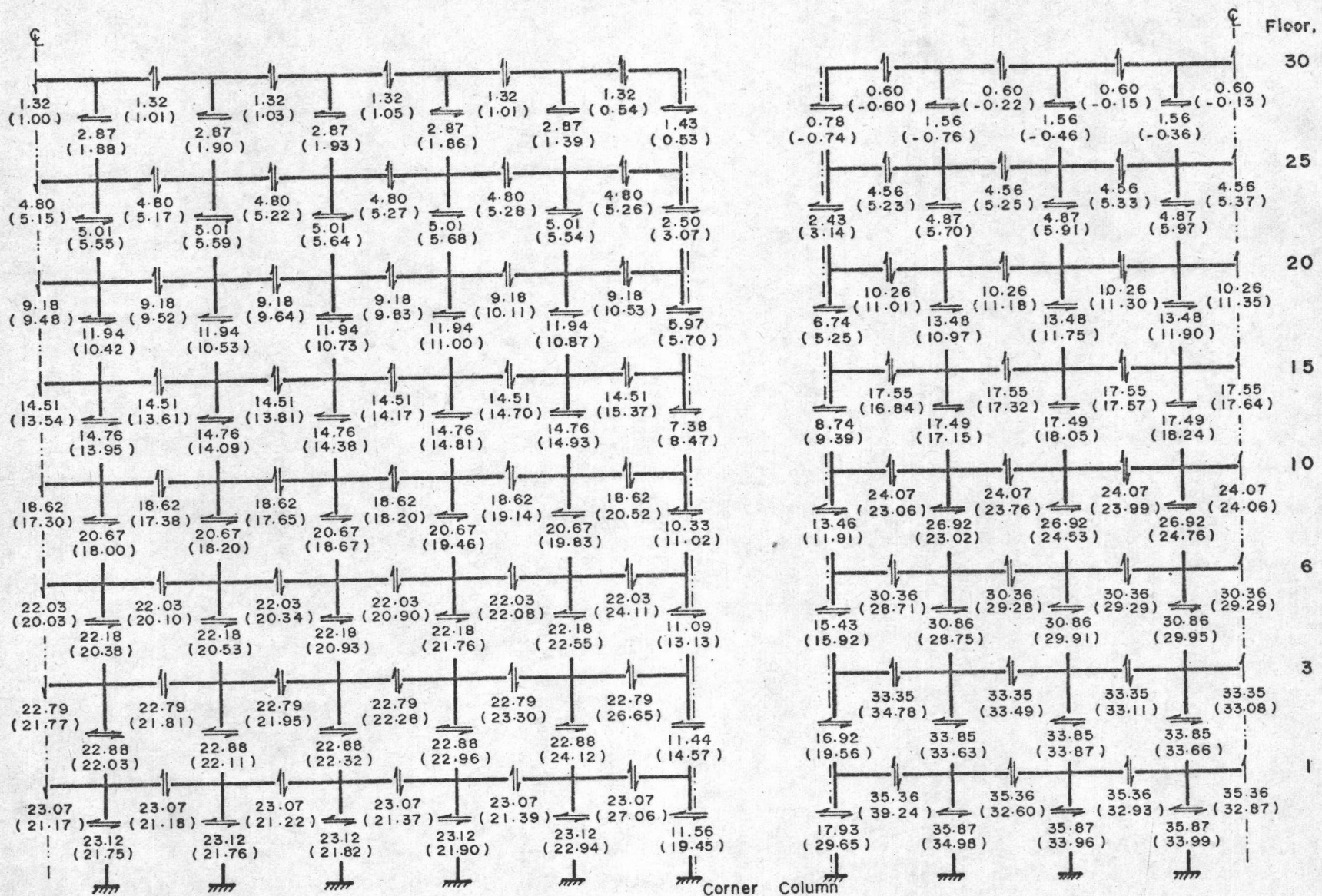


FIG.19. Example 4 : Axial Forces in Columns (Kips)



Note : Values in Parentheses are Results by ETABS

FIG. 20. Example 4: Shear Forces in Columns and Spandrel Beams (Kips)



Table 1. - Properties of Members

	Story	Interior Column				Corner Column			Spandrel Beam		
		Width (ft)	Area (ft <sup>2</sup> )	Shear Area (ft <sup>2</sup> )	Inertia (ft <sup>4</sup> )	Area (ft <sup>2</sup> )	Shear Area (ft <sup>2</sup> )	Inertia (ft <sup>4</sup> )	Depth (ft)	Shear Area (ft <sup>2</sup> )	Inertia (ft <sup>4</sup> )
Example 2,3	1-30	3.63	3.63	3.025	3.986	7.26	6.050	4.288	3.96	3.300	5.175
Example 4	1-10	3.40	3.40	2.833	3.275	6.80	5.667	3.558	3.60	3.888	3.000
	11-20	3.20	3.20	2.667	2.731	6.40	5.333	2.997	3.40	2.833	3.275
	21-30	2.80	2.80	2.333	1.829	5.60	4.667	2.063	2.80	2.333	1.829



Table.2.-Example 2 : Bending Moment in Corner Column of Framed Panel I

Story	Top moment		Bottom moment	
	Proposed	ETABS	Proposed	ETABS
30	-4.66	-3.02	4.66	3.91
25	13.12	13.17	-13.12	-10.31
20	27.83	26.11	-27.38	-23.24
15	40.47	39.07	-40.47	-36.25
10	50.83	52.74	-50.83	-50.07
6	57.46	65.45	-57.46	-63.11
3	61.46	78.78	-61.46	-78.68
1	63.64	80.58	-94.99	-159.80

Table.3.-Example 3: Bending Moment in Corner Column of Framed Panel I

Story	Top moment		Bottom moment	
	Proposed	ETABS	Proposed	ETABS
30	-9.40	-5.58	9.40	6.23
25	10.23	13.08	-10.23	-9.90
20	27.30	27.22	-27.30	-24.02
15	41.94	41.40	-41.94	-38.35
10	55.07	56.47	-55.07	-53.83
6	65.32	70.79	-65.32	-68.95
3	73.16	87.05	-73.16	-88.19
1	78.49	95.87	-117.16	-179.07



Table.4.— Properties of Framed Panels in Example 4

Story	Stiffness factor ( $12 I_D h / A_c d^3$ )	Shear lag parameter ( $G_{28} / E_z$ )
1 - 10	0.10	0.058
11 - 20	0.08	0.052
21 - 30	0.05	0.036

Table.5.— Example 4: Bending Moment in Corner Column of Framed Panel 1

Story	Top moment		Bottom moment	
	Proposed	ETABS	Proposed	ETABS
30	3.59	-2.82	-3.59	3.94
25	11.18	16.05	-11.18	-12.83
20	30.00	22.57	-30.00	-23.06
15	37.58	42.31	-37.58	-39.37
10	56.53	49.75	-56.53	-50.26
6	64.81	68.04	-64.81	-65.69
3	71.06	82.22	-71.06	-82.04
1	75.31	83.73	-107.58	-165.31

APPENDIX



Analysis of a thin-walled tube under torsion with warping unrestrained

By proposed method:

$$\text{Assume } \theta = K \cdot z \quad (i)$$

$$w_c = A \quad (ii)$$

where A, K = undetermined constants. Substituting Eqs. (i) and (ii) into the total strain energy equation, Eq. (28), leads to

$$\begin{aligned} 2 U_1 &= (2) \cdot \frac{1}{2} \int_0^H G \left( -\frac{A}{c} + b K \right)^2 t (2c) dz \\ &= 2 c H G t \left( -\frac{A}{c} + b K \right)^2 \end{aligned}$$

$$\begin{aligned} 2 U_2 &= (2) \cdot \frac{1}{2} \int_0^H G \left( \frac{A}{b} + c K \right)^2 t (2b) dz \\ &= 2 b H G t \left( \frac{A}{b} + c K \right)^2 \end{aligned}$$

$$U_c = 0$$

The total potential energy of external load is

$$V = - \int_0^H T \theta dz = - T H K$$

Then the total potential energy,  $\pi_p$ , is

$$\pi_p = 2 G t H \left[ c \left( -\frac{A}{c} + b K \right)^2 + b \left( \frac{A}{b} + c K \right)^2 \right] - T H K$$

Minimizing  $\pi_p$  with respect to A and K leads to

$$K = \frac{T(b+c)}{16 Gt b^2 c^2}$$

$$A = \frac{T(b-c)}{16 Gt bc}$$

Thus 
$$\theta = \frac{T(b+c)}{16 Gt b^2 c^2} \cdot z$$

$$w_c = \frac{T(b-c)}{16 Gt bc}$$

the shearing stresses are

$$\tau_{xz} = G \left[ -\frac{T(b-c)}{16 Gt bc^2} + b \frac{T(b+c)}{16 Gt b^2 c^2} \right] = \frac{T}{8 bct}$$

$$\tau_{yz} = G \left[ \frac{T(b-c)}{16 Gt b^2 c} + c \frac{T(b+c)}{16 Gt b^2 c^2} \right] = \frac{T}{8 bct}$$



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