V–3. A Simplified Method for Eccentricity Calculation

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ABSTRACT

A major problem in the design of structural masonry walls is the determination of the eccentricity at a floor/wall joint. This problem is usually resolved in design on an empirical basis although a few papers have been published suggesting methods for the calculation of such eccentricities. None of these, however, has the advantage of direct calculation of the eccentricity, some involve tedious trial and error procedures, and others involve solution of quadratic equations or the use of complex expressions.

This paper presents a simple method for calculation of the eccentricity at a floor/wall joint, which may be in any location in a single or multi-storey building, exterior or interior, and the floors may be of any type of R.C. construction, one way or two-way spanning.

Simple equations have been derived to calculate the eccentricity and calculated values are compared with results obtained from tests on a half scale and a full scale 2 storey single bay frames. These comparisons show good agreement between test and theory particularly when the precompression at the joint is equal to or greater than 0.3N/mm² (44 psi). For smaller precompression, the eccentricity calculated theoretically is over estimated and a modification for this condition is proposed.

1. INTRODUCTION

One of the major problems facing Engineers in the design of structural masonry walls is the determination of the eccentricity at the floor/wall joint. This matter has always been based on engineering judgement and experience. A few papers have been published suggesting methods for the calculation of such eccentricities, but none of these has the advantage of direct calculation of the joint eccentricity.

In this work, a suggested simple method for the calculation of the joint eccentricity is presented in any location in a single or multi-storey building, exterior or interior, and the floor slabs may be of any type of R.C. construction, one way or two way spanning, precast or cast in situ; the brick walls may be of solid or hollow construction.

To verify the suggested method, the equations developed have been used to calculate the eccentricities and the values are compared with test results obtained from half scale and full scale two storey single bay frames tested in the laboratory, these comparisons showed good agreement between tests and theory particularly when the precompression at the joint is equal or greater than 0.3 N/mm² (44 psi); for smaller precompressions, the eccentricity evaluated theoretically is over-estimated and a modification for this condition is proposed.

2. WALL ECCENTRICITIES:

a. Walls supporting one-way slab system:

The eccentricity of the wall gravity loads depends on the rigidity of the slab and the wall as well as the rigidity of the wall/floor joint. The load eccentricity may be calculated by considering the continuity condition at the joint[1] thus,

$$\theta_s - \theta_j = \theta_w \tag{1}$$

where θ_s = rotation of the slab, θ_w = rotation of the wall and θ_j represents the angle of deformation in the joint, which is a measure of the joint rigidity.

Where the floor slab forms a part of a structure of the type shown in Fig. 1 in which the wall/floor joints are rigid and the walls are bent in double or single curvature with equal end eccentricities the angle of rotation of the slab at the support is given by:

$$\theta_s = \frac{ML^3}{24(EI)_s} - \frac{ML}{2(EI)_s} \tag{2}$$

where:

$$M = P_L \cdot L_L + P_u \cdot L_u \tag{3}$$

$P_u = (n - 1)\dfrac{wL}{2} + (n - 1)G$

$P_L = P_u + wL/2$

w = uniformly distributed load of slab dead load and live load.

L = slab span.

$(EI)_s$ = flexural rigidity of slab.

G = dead load of wall per storey.

n = ordinal number of a floor slab reckoned from the topmost storey of a building.

Substituting eq. (3) into eq. (2) gives:

$$\theta = \frac{wL^3}{24(EI)_s} - \frac{(P_L \cdot e_L + P_u \cdot e_u)}{2(EI)_s} \tag{4}$$

where

e_u = eccentricity the compressive force immediately above the floor slab.

e_L = eccentricity of the compressive force immediately below the floor slab.

Assuming the joint moment rotation relation is linear[2], the value of θ_j may be given as

$$\theta_j = \frac{M}{\beta} \tag{5}$$

where β = joint stiffness

And the wall rotation θ_w may be expressed as,

$$\theta_w = \frac{P_L \cdot e_L \cdot H}{(EI)_w R} \tag{6}$$

in which H is the distance from wall/floor joint to point of inflection or the wall story height (which ever is smaller), R is a factor depending on the eccentricity, wall slenderness and type of wall curvature and $(EI)_w$ is the flexural rigidity of the wall. Thus substituting eq. (4), (5) and (6) into equation (1) and noting that, as previously assumed, $e = e_u = e_L$ we obtain,

$$\varepsilon = \frac{\overline{M}}{P_L \cdot t[(1 + \psi)(1 + \overline{\beta}) + \overline{K}/R]} \tag{7}$$

where

$$\varepsilon = e/t,$$
$$\overline{\beta} = \frac{2(EI)_s}{\beta L},$$
$$\overline{K} = \frac{2(EI)_s H}{(EI)_w L}$$
$$\overline{M} = \frac{wL^2}{12},$$
$$\psi = P_u/P_L$$

Assuming a rigid joint i.e. $\beta \rightarrow \infty$, it follows that $\overline{\beta} \rightarrow 0$ and eq. (7) reduces to the form:

$$\varepsilon = \frac{\overline{M}R}{P_L t[\overline{\psi}R + \overline{K}]} \tag{8}$$

where $\overline{\psi} = 1 + \psi$

The above equation is based on the assumption that the joint is fixed, so that equation (8) is valid in calculating the eccentricities provided that this condition applies and the fixed end moment, $\overline{M}$, could be reached if there was no rotation of the joint as a whole. Experiments show that fixity of the joint may never be fully achieved in actual structures (and hence the full fixed end moment, $\overline{M}$, may never be fully developed).

Introducing the relation[3],

$$M_o = \overline{M}\,\psi \tag{9}$$

where M_o = actual moment in the joint (i.e. slab restraining moment). Equation (9) is presented graphically in Fig. 2, it relates the actual moment at the joint, M_o, and the slab fixed end moment, $\overline{M}$, depending on the degree of the joint fixity as a measure of the total precompression at the joint, thus, when the precompression is equal to zero (i.e. $P_u = 0$), as the uppermost slab in a multi-storey building, the joint moment M_o is equal to zero, simulating a hinged support where the slab is free to rotate. As the precompression increases, the value of P_u/P_L increases, and hence the slab restraining moment increases, thus when the value of ψ approaches unity, the joint becomes fully fixed, and M_o is equal to the fixed end moment, $\overline{M}$.

To verify equation (9), theoretical values of M_o were compared with test results carried out on a full scale two storey frame[2] where the precompression varied from zero to 0.7 N/mm² (101 psi) and a half scale model[3] with precompression up to 1.64 N/mm² (237 psi). These comparisons show good agreement between eq. (9) and the test results for precompression greater than 0.3 N/mm² but gave over estimated values of M_o for smaller precompression, thus, a modification of eq. (9) is proposed which was compared with test results and gave close agreement. This proposed modification is given as:

$$M_o = \overline{M}\,(\psi/\overline{\psi}) \tag{10}$$

Thus, where the precompression on the joint is less than 0.3 N/mm² (44 psi), equation (10) should be used to calculate the moment at the joint, and equation (9) is used when the precompression is equal or greater than 0.3 N/mm². The limit of application of eq. (9) and (10) are suggested at 0.3 N/mm² precompression, it may be found that using the formulas at values slightly above or below this limit, two values for eccentricity could be found, in such cases, the average of the two eccentricities could be adopted. The theoretical results based on the previous conclusions and the test results are shown in Tables 1 and 2. Using equations (9) and (10), eq. (8) is rewritten to account for these conditions and we obtain,

$$\varepsilon = \frac{M_o R}{P_L \cdot t[\overline{\psi}R + \overline{K}]} \tag{11}$$

substituting eq. (9) and (10) into eq. (11) we finally obtain

$$\varepsilon = \frac{\overline{M}R\,(\psi/\overline{\psi})}{P_L \cdot t[\overline{\psi}R + \overline{K}]} \tag{12}$$

where the precompression $< 0.3\text{N/mm}^2$

and, $$\varepsilon = \frac{\overline{M}R\,\psi}{Rt[\overline{\psi}R + \overline{K}]} \tag{13}$$

where the precompression $\geqslant 0.3\text{N/mm}^2$

Equations (12) and (13) are valid irrespective of the joint location and will be used to calculate the eccentricities at any external joint in any system where the slab is supported in one way direction.

The factor R presented in eq. (12) and (13) was evaluated using a computer program which is programmed to calculate the equations developed by Colville[2] for moment-rotation equations relating the eccentricity at the joint, the buckling rotation of the wall and the load carrying capacity of the wall. From these results it was possible to relate the factor (R), the eccentricity at the joint and the ultimate capacity of the wall end rotation. The values of R to be used in eq. (12) and (13) are[3],

1. Walls bent in double curvature,
   R = 2.345 (uncracked walls, i.e. $\varepsilon < 1/6$)
   R = 1.275 (cracked walls, $\varepsilon \geqslant 1/6$)
2. Walls bent in single curvature;
   R = 1.85 for cracked and uncracked walls.

Finally, the eccentricity at the joint may be calculated for external walls supporting one way slab systems by using the appropriate value of the factor R and the related parameters by direct substitution of the values of these parameters involved as will be shown in the design example. It should be explained that, where the joint is an internal one, equations (12) and (13) can still be used provided that the fixed end moment $\overline{M}$ is taken as the difference between adjacent slab fixed end moments, i.e.

$$\overline{M} = \frac{w}{12}(L_1^2 - L_2^2) \quad (14)$$

where L_1 is the larger slab span and L_2 in the smaller slab span and the expressions of P_u and P_L are calculated based on the following,

$$P_u = (n - 1)\left[\frac{w}{2}(L_1 + L_2) + G\right] \quad (15)$$

$$P_L = P_u + \frac{w}{2}(L_1 + L_2) \quad (16)$$

To verify eq. (12) and (13) theoretical values are compared with experimental results carried out on the full scale frame[2] and the half scale model frame[3] and these results are tabulated in Tables 3 and 4, for walls bent in double curvature. This shows quite good agreement between the proposed equations and test results.

b. Walls supporting two way slab systems:

By a similar approach the following equations may be derived[3] assuming a uniformly distributed loading at the supporting walls[4]

1. External joints:

$$P_u = (n - 1)\frac{wL}{3} + (n - 1)G \quad (17)$$

$$P_L = P_u + \frac{wL}{3} \quad (18)$$

for walls supporting the short span of the slab.

$$P_u = (n - 1)\frac{wL}{3}\frac{(3 - m^2)}{2} + (n - 1)G \quad (19)$$

$$P_u = P_u + \frac{wL}{3}\frac{(3 - m^2)}{2} \quad (20)$$

for walls supporting the long span of the slab.

$$\varepsilon = \frac{\overline{M}R(\psi/\overline{\psi})(c_s/c_f)}{P_L \cdot t[\overline{\psi}R + \overline{K}]} \quad (21)$$

Where the precompression $< 0.3\text{N/mm}^2$

$$\varepsilon = \frac{\overline{M}R\psi(c_s/c_f)}{P_L \cdot t[\overline{\psi}R + \overline{K}]} \quad (22)$$

where the precompression $\geqslant 0.3\text{N/mm}^2$

where m = ratio of short span to long span, and C_s = factor depending on the slab configuration and type of support, and the values of C_s[4] are presented in Table 5. $C_f = 1/12$ since the basic equations were derived based on the full fixed end moment.

2. Internal joints:

The fixed end moment at the joint is taken as:

$$\overline{M} = \frac{w}{12}(L_1^2 - L_2^2) \quad (23)$$

and the expression for P_u and P_L are:

$$P_u = (n - 1)\frac{w}{3}(L_1 + L_2) + G \quad (24)$$

$$P_L = P_u + \frac{w}{3}(L_1 + L_2) \quad (25)$$

for walls supporting short spans, and

$$P_u = (n - 1)\left[\frac{wL_1}{3}\cdot\frac{(3 - m_1^2)}{2} + \frac{wL_2}{3}\cdot\frac{(3 - m_2^2)}{2} + G\right] \quad (26)$$

$$P_L = P_u + \left[\frac{wL_1}{3}\frac{(3 - m_1^2)}{2} + \frac{wL_2}{3}\frac{(3 - m_2^2)}{2}\right] \quad (27)$$

for walls supporting long spans.

Where: m_1 = ratio of short span to long span of the longer slab at one side of the joint, and m_2 = ratio of short to long span of the shorter slab at the other side of the joint.

It should be pointed out that in all cases when calculating the eccentricities at the joint using one of the above equations, these calculated eccentricities must not be greater than the failure eccentricity[2] which in the maximum limiting value of the joint eccentricity, $\varepsilon_{failure}$, defined as:

$$\varepsilon_f = \frac{1}{2}(1 - \eta) \quad (28)$$

where

ε_f = eccentricity corresponding to joint failure.
η = ratio of the axial load in the wall to the wall compressive strength.

Failure at the joint may also occur if the ultimate negative moment capacity of the slab is attained, thus, the values of $\overline{M}$ to be used in the eccentricity equations must be the smaller of the fixed end moment or the ultimate negative moment capacity of the slab.

3. Design example:

Consider a wall 8 floors below the roof of a single-bay multi-storey structure and assume the following data:

Floor:		Wall:	
(supported in one direction)			
Live load	= 5 kN/m^2	thickness	= 215 mm
dead load	= 4.0 kN/m^2	storey height	= 2.4 metres
thickness	= 200 mm	E	= 11.0 kN/mm^2
E	= 21 kN/mm^2	wall wt.	= 17.0 kN/m^3
L	= 6.0 metres	F_m	= 17.24 N/mm^2
I	= 6.66 $* 10^{-4}$m^4	E	= 11.0 kN/mm^2
		I	= 8.28 $* 10^{-4}$ m^4

with all floors loaded it may be assumed that the walls will be bent in double curvature with equal end eccentricities. Thus

$$P_u = 7 * \frac{9}{2} * 6.0 + 7 * 2.4 * 17.0 * 0.215$$

$$= 250.4 \text{ kN/m}$$

$$P_L = 250.4 + \frac{9 * 6.0}{2} = 277.4 \text{ kN/m}$$

$$\psi = \frac{250.4}{277.4} = 0.902, \qquad \overline{\psi} = 1.902$$

$$\overline{M} = \frac{wL^2}{12} = \frac{9 * (6)^2}{12} = 27.0 \text{ m.kn.}$$

$$\overline{K} = \frac{2(EI)_s\, H}{(EI)_w \cdot L}$$

$$= \frac{2 * 21 * 6.66 * 10^{-4} * 1.2}{11.0 * 8.28 * 10^{-4} * 6} = 0.614$$

$$\text{Precompression} = \frac{P_u}{t} = \frac{250.4 * 1000}{215 * 1000} = 1.164 \text{ N/mm}^2$$

assuming uncracked wall (i.e. $\varepsilon < 1/6$), $R = 2.345$

$$\therefore \varepsilon = \frac{27.0 * 2.345 * 0.902}{277.4 * 0.215\,(1.902 * 2.345 + 0.614)}$$

$$= 0.188 > 1/6$$

Thus the wall is cracked and $R = 1.275$;

$$\therefore \varepsilon = \frac{27.0 * 1.275 * 0.902}{277.4 * 0.215\,(1.902 * 1.275 + 0.614)}$$

$$= 0.171$$

check $$\varepsilon_f = \frac{1}{2}\left(1 - \frac{277.4}{17.24 * 215}\right)$$

$$= 0.462 > 0.171 \text{ O.K.}$$

REFERENCES

1. Sahlin, S. "Structural masonry", Prentice-Hall Inc. 1971.
2. Colville, J. "Analysis and Design of Brick Masonry Walls", Report submitted to the University of Edinburgh, 1977.
3. Awni, A. "The Compressive strength of brick masonry walls with reference to wall/floor slab interaction", Ph.D. Thesis, University of Edinburgh, 1980.
4. ACI Standard 318–63, "Building Code required for reinforced concrete", June, 1963.

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Notation:

C_f = coefficient (1/12)
C_s = moment coefficient for two way slabs, depending on slab configuration (from ACJ 318–63)
e = eccentricity of wall load
ε = e/t (relative eccentricity)
ε_f = eccentricity of joint at failure = ½ (1 − n)
E_s = Modulus of elasticity of concrete slab
E_w = modulus of elasticity of brick wall
f_m = wall compressive stress at failure
G = dead load of wall per storey
H = wall height (single curvature), or distance between point of inflection and wall end (double curvature)
I_w = second moment of area of brick wall
$\overline{K}$ = $\dfrac{(EI)_s \cdot H}{(EI)_w \cdot L}$
L = floor slab span
$L1$ = longer floor span on one side of joint
$L2$ = shorter floor span on the other side of joint
m = ratio of short span to long span of floor
m_1 = ratio of short span to long span of the larger slab floor on one side of the joint.
m_2 = ratio of the short span to long span of the shorter slab floor on the other side of the joint.
$\overline{M}$ = fixed end moment = $wL^2/12$
Mo = joint moment (i.e. actual slab restraining moment)
n = ordinal number of a floor slab reckoned from the topmost storey of a building.
P_u = compressive force acting on the wall immediately above a floor slab
P_L = compressive force acting on the wall immediately below a floor slab.
R = a factor depending on wall eccentricity, wall slenderness, and type of wall curvature.
t = actual wall thickness (solid walls), or, (2/3) the sum of the actual thickness of the two leaves (cavity walls).
w = total uniformly distributed load of slab dead load and live load.
ψ = Pu/p
$\overline{\psi}$ = $(1 + \psi)$
β = joint stiffness
$\overline{\beta}$ = $\dfrac{2(EI)_s}{L}$
η = ratio of axial load in the wall to the wall compressive strength.
θ_s = slab rotation
θ_w = wall rotation
θ_j = joint rotation (deformation).

TABLE 1—"Slab Restraining moments" half scale model

Slab U.D.L. (kN/m)	Precom. jacks (kN)	P_u (N/mm^2)	P_u (kN)	P_L (kN)	ψ	$\bar{\psi}$	$\bar{M}$* (m.kN)	$\bar{M}\psi$ (m.kN)	$\bar{M}(\psi/\bar{\psi})$ (m.kN)	Mo** (m.kN)	% diff.
1.75	0	0.073	3.72	7.65	.486	1.486	.625	—	0.204	.176	+15.9
1.75	10	0.268	13.72	17.65	.777	1.777	.625		0.273	.322	−17.8
1.75	20	0.463	23.72	27.65	.857	1.857	.625	.535		.448	+19.41
1.75	30	0.659	33.72	37.65	.895	1.895	.625	.559		.498	+12.24
1.75	40	0.855	43.72	47.65	.917	1.917	.625	.573		.527	+ 8.72
1.75	50	1.051	53.72	57.65	.931	1.931	.625	.581		.549	+ 5.82
1.75	60	1.247	63.72	67.65	.942	1.942	.625	.588		.565	+ 4.07
1.75	70	1.443	73.72	77.65	.949	1.949	.625	.593		.581	+ 2.06
1.75	80	1.639	83.72	87.65	.955	1.955	.625	.596		.590	+ 1.01
3.5	0	0.073	3.72	9.15	.406	1.406	1.25		0.361	.318	+13.5
3.5	10	0.268	13.72	19.15	.716	1.716	1.25		0.521	.607	−16.5
3.5	20	0.463	23.72	29.15	.813	1.813	1.25	1.016		.876	+15.98
3.5	30	0.659	33.72	39.15	.861	1.861	1.25	1.076		.995	+ 8.14
3.5	40	0.855	43.72	49.15	.889	1.889	1.25	1.111		1.082	+ 2.68
3.5	50	1.051	53.72	59.15	.908	1.908	1.25	1.135		1.112	+ 2.06
3.5	60	1.247	63.72	69.15	.921	1.921	1.25	1.151		1.140	+ 0.96
3.5	70	1.443	73.72	79.15	.931	1.931	1.25	1.163		1.132	+ 2.73
3.5	80	1.639	83.72	89.15	.939	1.939	1.25	1.173		1.124	+ 4.35

* $\bar{M}$ = Frame moments calculated by moment distribution method
** Mo = Joint moment (i.e. actual slab restraining moment) obtained by tests.

TABLE 2—"Slab restraining moments" Full scale model

Slab u.d.l. (kN/m)	precomp. (N/mm^2)	P_u (kN)	P_L (kN)	ψ	$\bar{M}$* (m.kN)	$\bar{M}\psi$ (m.kN)	Mo m.kN test	% diff.
2.41	0.3	54.4	58.5	.93	3.94	3.664	3.215	+13.96
2.41	0.5	93.7	97.8	.96	3.94	3.78	3.351	+12.80
2.41	0.7	130.7	134.8	.97	3.94	3.82	3.54	+ 7.90
3.98	0.3	54.4	61.2	.89	6.508	5.792	5.395	+ 7.35
3.98	0.5	93.7	100.5	.93	6.508	6.052	5.599	+ 8.09
3.98	0.7	130.7	137.5	.95	6.508	6.182	5.741	+ 7.68

* $\bar{M}$ based on moment distribution of the frame.

TABLE 4—"Joint eccentricities" full scale model

Slab U.D.L. (kN/m)	Precom. (N/mm^2)	P_u (kn)	P_L (kN)	ψ	ε_{Test}	ε_{Theory}	Test/theory % Diff.
2.41	0.3	54.4	58.5	.93	.134	0.131	+ 2.29
2.41	0.5	93.7	97.8	.96	.083	0.080	+ 3.75
2.41	0.7	130.7	134.8	.97	.063	0.0585	+ 7.69
3.98	0.3	54.4	61.2	.89	.220	0.180	+22.22
3.98	0.5	93.7	100.5	.93	.136	0.1265	+ 7.5
3.98	0.7	130.7	137.5	.95	.101	0.0936	+ 7.9

* based on $\bar{K} = \dfrac{2(EI)_s \cdot H}{(EI) \cdot L} = 0.736$

TABLE 3—"Joint eccentricities" half scale model test theory

Slab U.D.L. (kN/m)	Precom. (N/mm²)	P_u (kN)	P_L (kN)	ψ	$\bar{\psi}$	$\overline{M}$ (m.kN)	Mo (m.kN) test	ε_{test**}	$\varepsilon_{theory*}$	% Diff.
1.75	0.073	3.72	7.65	.486	1.486	.625	.176	0.138	0.134	+ 2.98
1.75	0.268	13.72	17.65	.777	1.777	.625	.322	0.0916	0.067	+36.1
1.75	0.463	23.72	27.65	.857	1.857	.625	.448	0.0778	0.064	+21.56
1.75	0.659	·33.72	37.65	.895	1.895	.625	.498	0.0623	0.060	+ 2.8
1.75	0.855	43.72	47.65	.917	1.917	.625	.527	0.0515	0.049	+ 5.1
1.75	1.051	53.72	57.65	.931	1.931	.625	.549	0.044	0.040	+ 9.45
1.75	1.247	63.72	67.65	.942	1.942	.625	.565	0.0384	0.034	+11.95
1.75	1.443	73.74	77.65	.949	1.949	.625	.581	0.0342	0.030	+11.76
1.75	1.639	83.72	87.65	.955	1.955	.625	.590	0.0307	0.027	+13.7
3.5	0.073	3.72	9.15	.406	1. 06	1.25	.318	0.220	0.183	+20.2
3.5	0.268	13.72	19.15	.716	1.716	1.25	.607	0.1648	0.121	+35.7
3.5	0.463	23.72	29.15	.813	1.813	1.25	.876	0.148	0.148	+ 0.0
3.5	0.659	33.72	39.15	.861	1.861	1.25	.995	0.122	0.114	+ 7.0
3.5	0.855	43.72	49.15	.889	1.889	1.25	1.082	0.104	0.092	+13.0
3.5	1.051	53.72	59.15	.908	1.908	1.25	1.112	0.088	0.078	+12.67
3.5	1.247	63.72	69.15	.921	1.921	1.25	1.140	0.0766	0.067	+13.64
3.5	1.443	73.72	79.15	.931	1.931	1.25	1.132	0.066	0.059	+11.86
3.5	1.639	83.72	89.15	.939	1.939	1.25	1.129	0.058	0.052	+11.53

* Based on $\overline{K}$ = 0.66

** test is obtained by dividing Mo (from tests) by the sum of the total force at the joint (i.e. $\varepsilon_{test} = Mo/(P_u + P_L)*t$)

TABLE 5—"Moment coefficients (C_s)*

Moments	Short span values of m** 1.0	0.9	0.8	0.7	0.6	0.5	Long span all values of m
CASE 1—Interior Panels: negative moment at:							
continuous edge	.033	.040	.048	.055	.063	.083	.033
discontinuous edge	—	—	—	—	—	—	—
CASE 2—One edge discontinuous negative moment at:							
continuous edge	.041	.048	.055	.062	.069	.085	.041
discontinuous edge	.021	.024	.027	.031	.035	.042	.021
CASE 3—Two edge discontinuous moment at:							
continuous edge	.049	.057	.064	.071	0.78	.090	.049
discontinuous edge	.025	.028	.032	.036	.039	.045	.025
CASE 4—Three edge discontinuous moment at:							
continuous edge	.058	.066	.074	.082	.090	.098	.058
discontinuous edge	.029	.033	.037	.041	.045	.049	.029
CASE 5—Four edge discontinuous moment at:							
continuous edge	—	—	—	—	—	—	—
discontinuous edge	.033	.038	.043	.047	.053	.055	.033

* From ACI Standard Building Code 318–63.

** Interpolate for intermediate values.

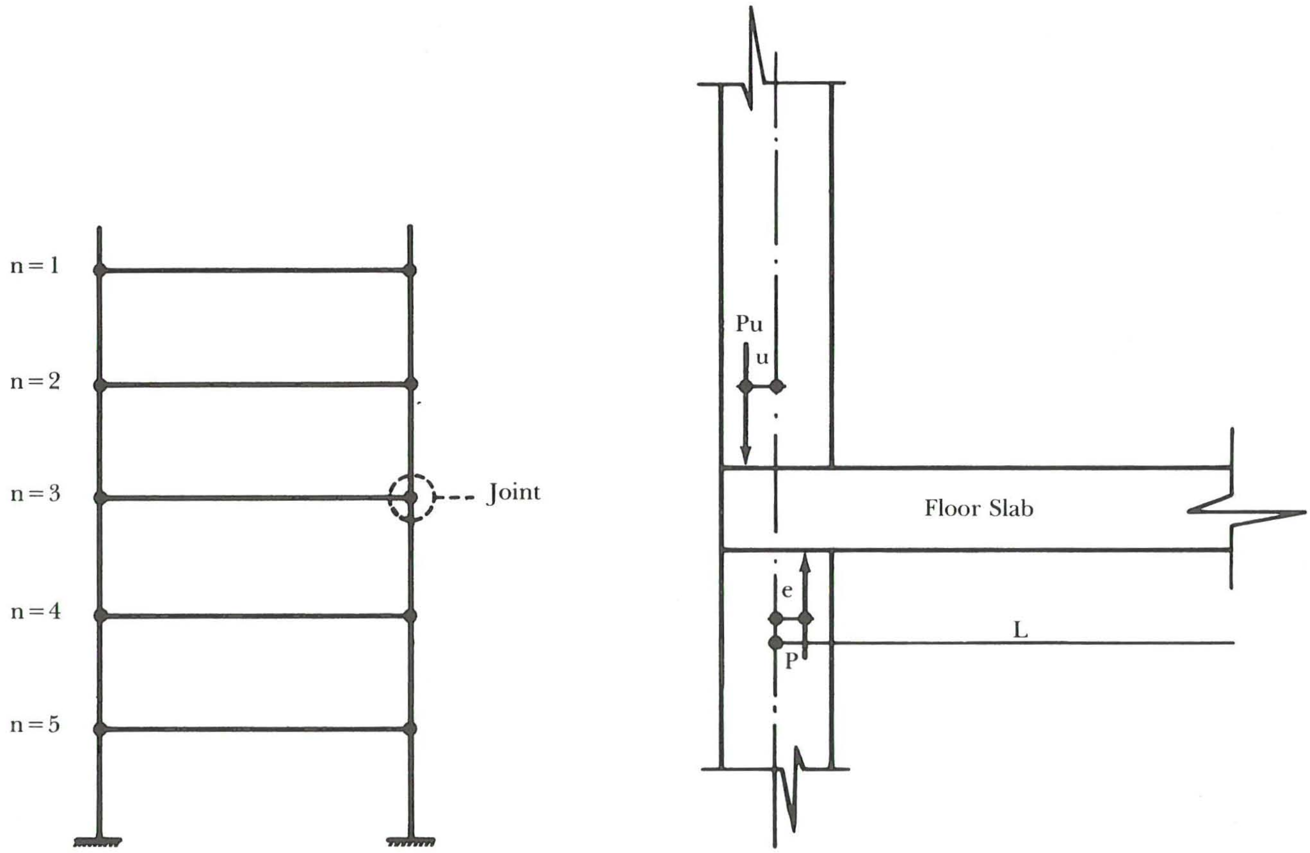

Figure 1.

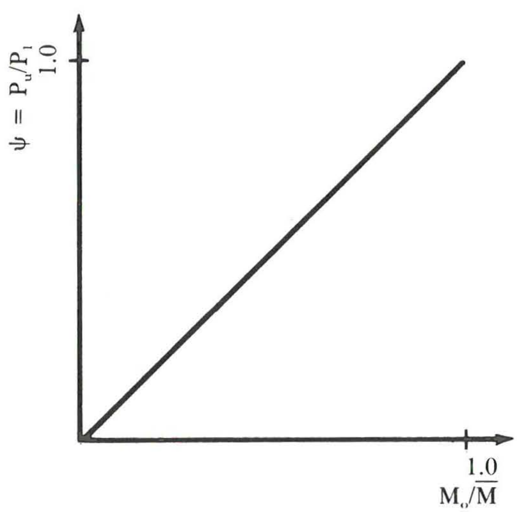

Figure 2.