Lecture nine

Equivalent Frame Method - General

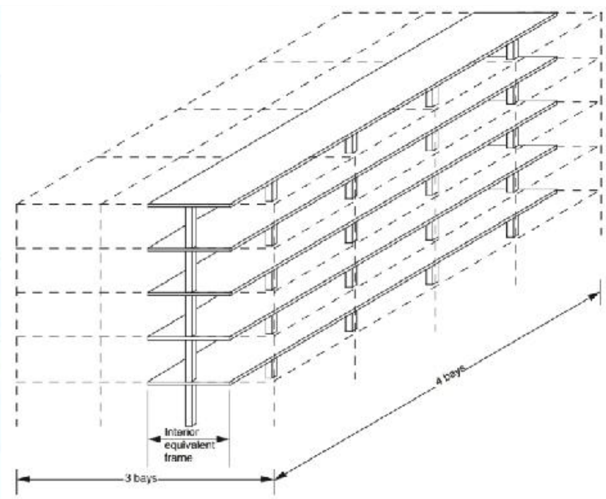
Equivalent Frame Method EFM is used to analyze and design structures not satisfy the limitations of DDM. The building is divided into equivalent frames in both directions, where each frame consists of slab-beam supported by columns.

8.11.2 *Equivalent frames*

8.11.2.1 The structure shall be modeled by equivalent frames on column lines taken longitudinally and transversely through the building.

8.11.2.2 Each equivalent frame shall consist of a row of columns or supports and slab-beam strips bounded laterally by the panel centerline on each side of the centerline of columns or supports.

8.11.2.3 Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of the adjacent panel.



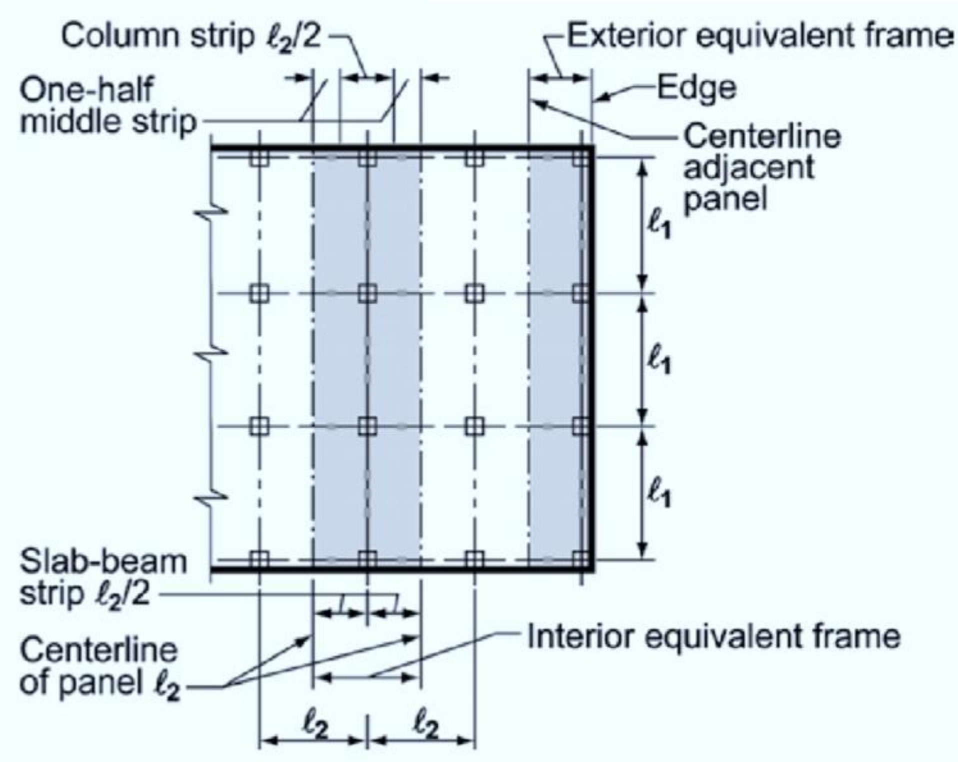


Fig. R8.11.2—Definitions of equivalent frame.

Torsional members attach the columns to slab-beam strips.

8.11.2.4 Columns or supports shall be assumed to be attached to slab-beam strips by torsional members transverse to the direction of the span for which moments are being calculated and extending to the panel centerlines on each side of a column.

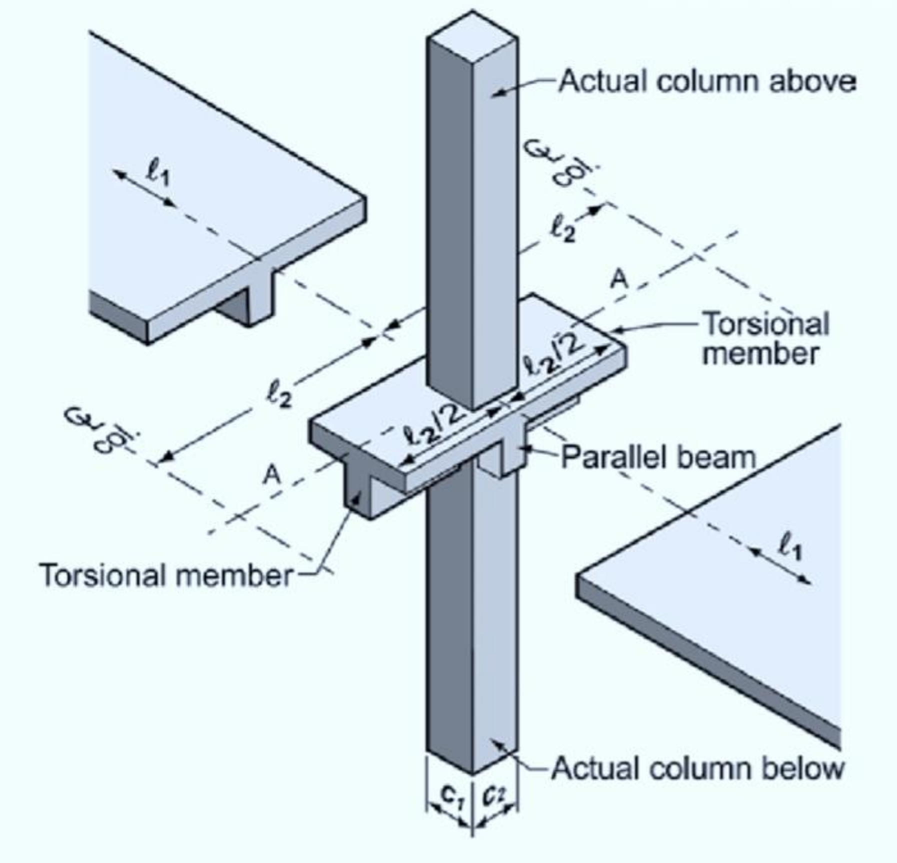
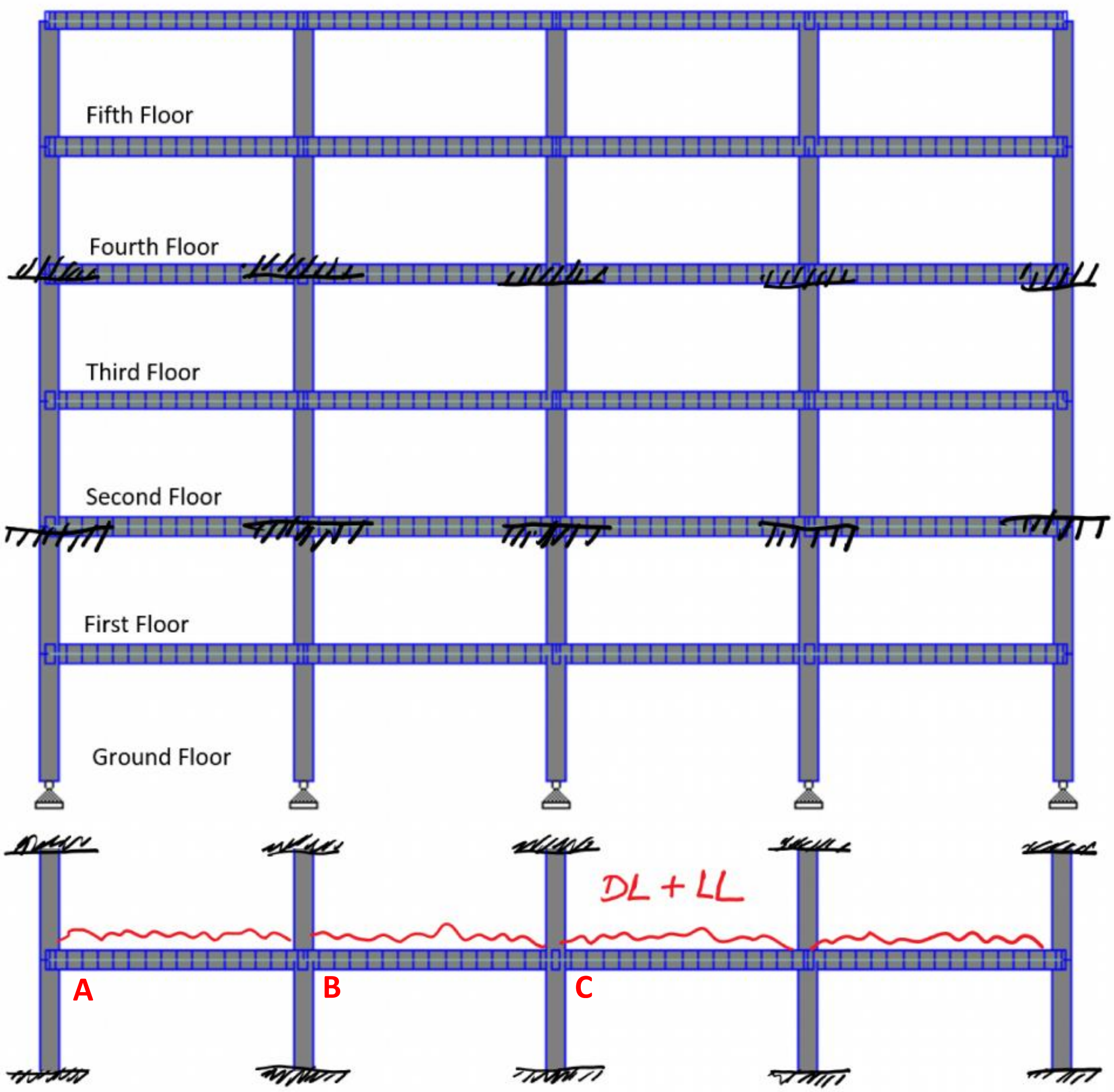


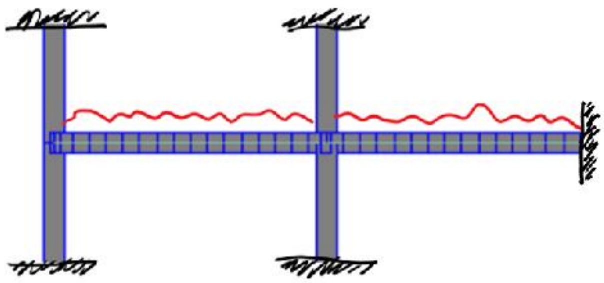
***Fig. R8.11.4**—Equivalent column (column plus torsional members).*

8.11.2.5 Analysis of each equivalent frame in its entirety shall be permitted. Alternatively, for gravity loading, a separate analysis of each floor or roof with the far ends of columns considered fixed is permitted.

8.11.2.6 If slab-beams are analyzed separately, it shall be permitted to calculate the moment at a given support by assuming that the slab-beam is fixed at supports two or more panels away, provided the slab continues beyond the assumed fixed supports.



To calculate the moment at support A, it is permitted to analyze part of the frame instead of the whole one.



What will be the frames used to find the moments at support B and C, respectively?

Live loads patterns

As the live loads not distributed equally on all spans by its nature, there are certain patterns of loading may give the largest positive and negative moments.

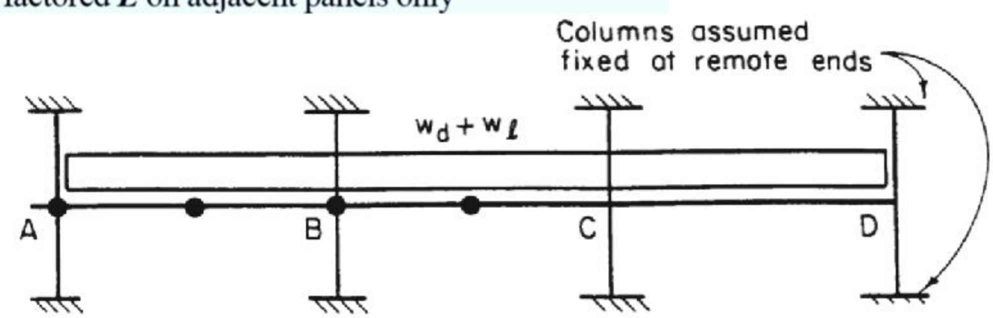
6.4.3 For two-way slab systems, factored moments shall be calculated in accordance with 6.4.3.1, 6.4.3.2, or 6.4.3.3, and shall be at least the moments resulting from factored L applied simultaneously to all panels.

6.4.3.1 If the arrangement of L is known, the slab system shall be analyzed for that arrangement.

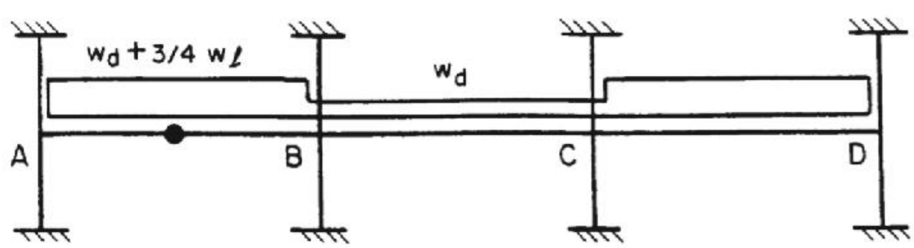
6.4.3.2 If L is variable and does not exceed $0.75D$, or the nature of L is such that all panels will be loaded simultaneously, it shall be permitted to assume that maximum M_u at all sections occurs with factored L applied simultaneously to all panels.

6.4.3.3 For loading conditions other than those defined in 6.4.3.1 or 6.4.3.2, it shall be permitted to assume (a) and (b):

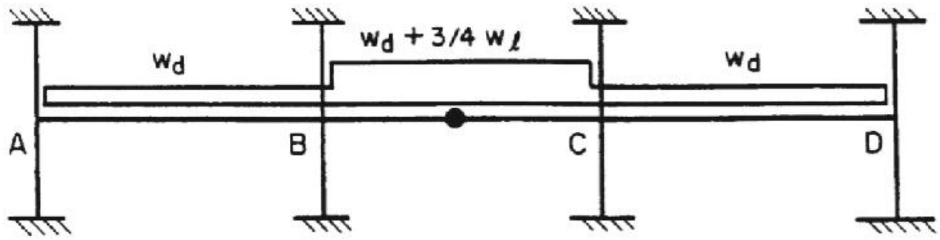
(a) Maximum positive M_u near midspan of panel occurs with 75 percent of factored L on the panel and alternate panels
(b) Maximum negative M_u at a support occurs with 75 percent of factored L on adjacent panels only



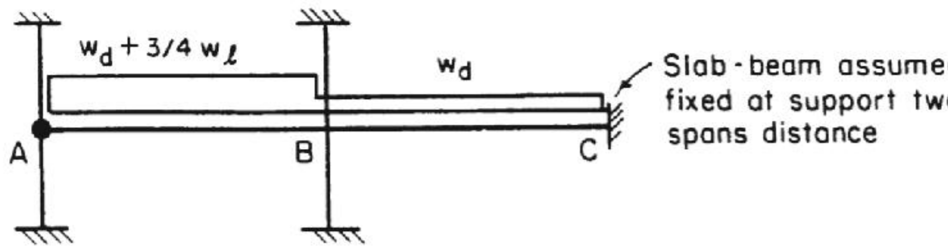
(1) Loading pattern for design moments in all spans with L ≤ 3/4 D



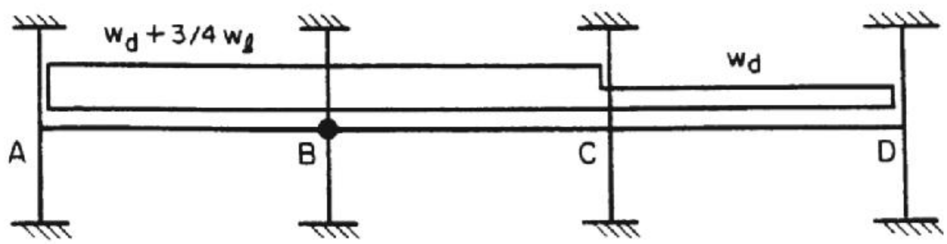
(2) Loading pattern for positive design moment in span AB*



(3) Loading pattern for positive design moment in span BC*



(4) Loading pattern for negative design moment at support A*



(5) Loading pattern for negative design moment at support B*

Flexural Stiffness of Slab-Beam Strip and Column

ACI 318-14 provisions calculate the flexural stiffness of the slab-beam strip and column, which are used in computer or hand calculations. The moment of distribution method is one of the common hand calculations method used to analyze the equivalent frame. The moment distribution may be summarized in the following steps:

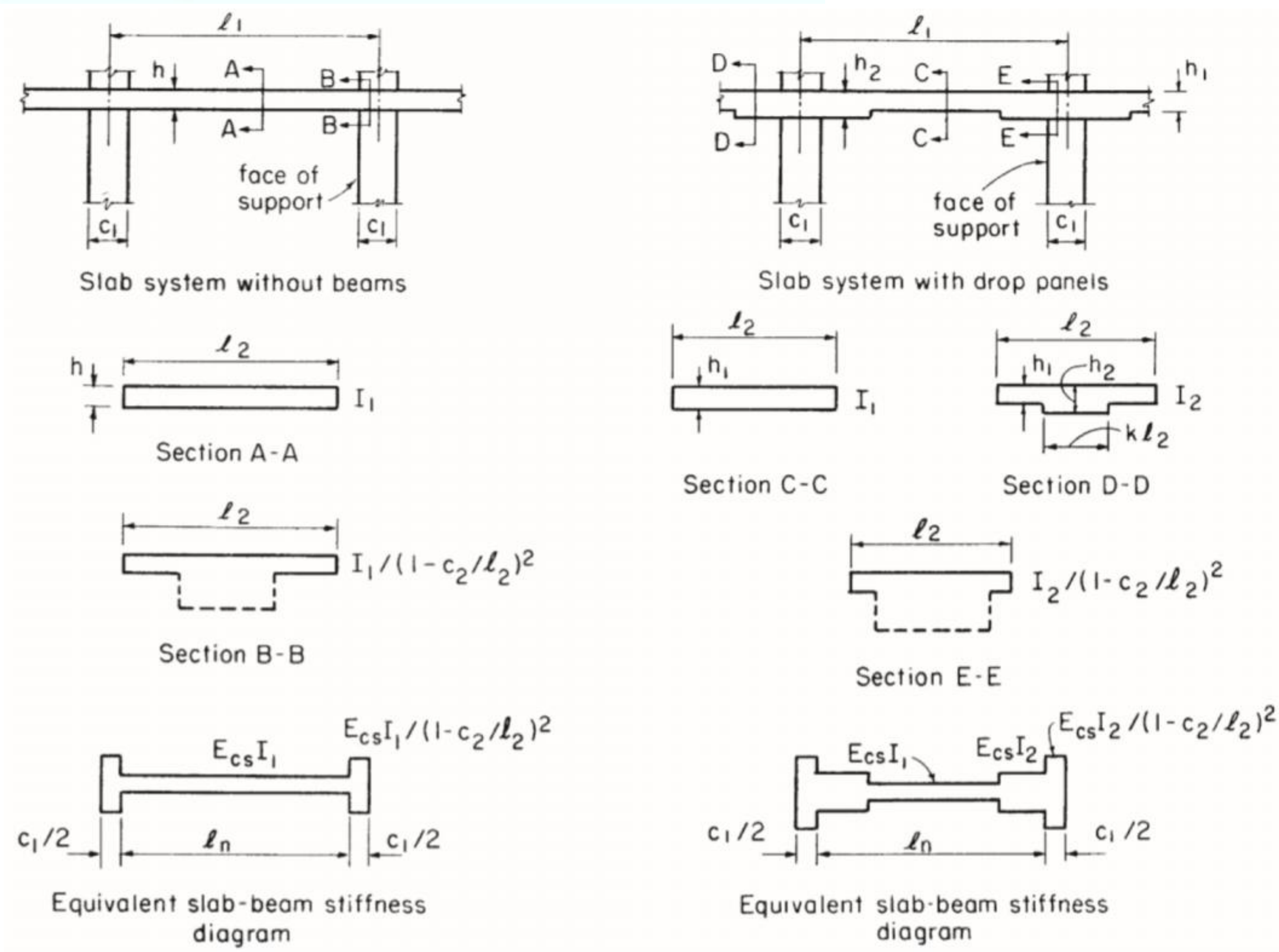
1- Fix all joints and calculate the fixed end moments at the ends of each span.
2- Release each joint and balance its moment.
3- Distribute the balance moment to the members linked to each joint according to their stiffness (i.e. using distribution factor).
4- At each member, part of the moment shall be carried over the span to the far end.

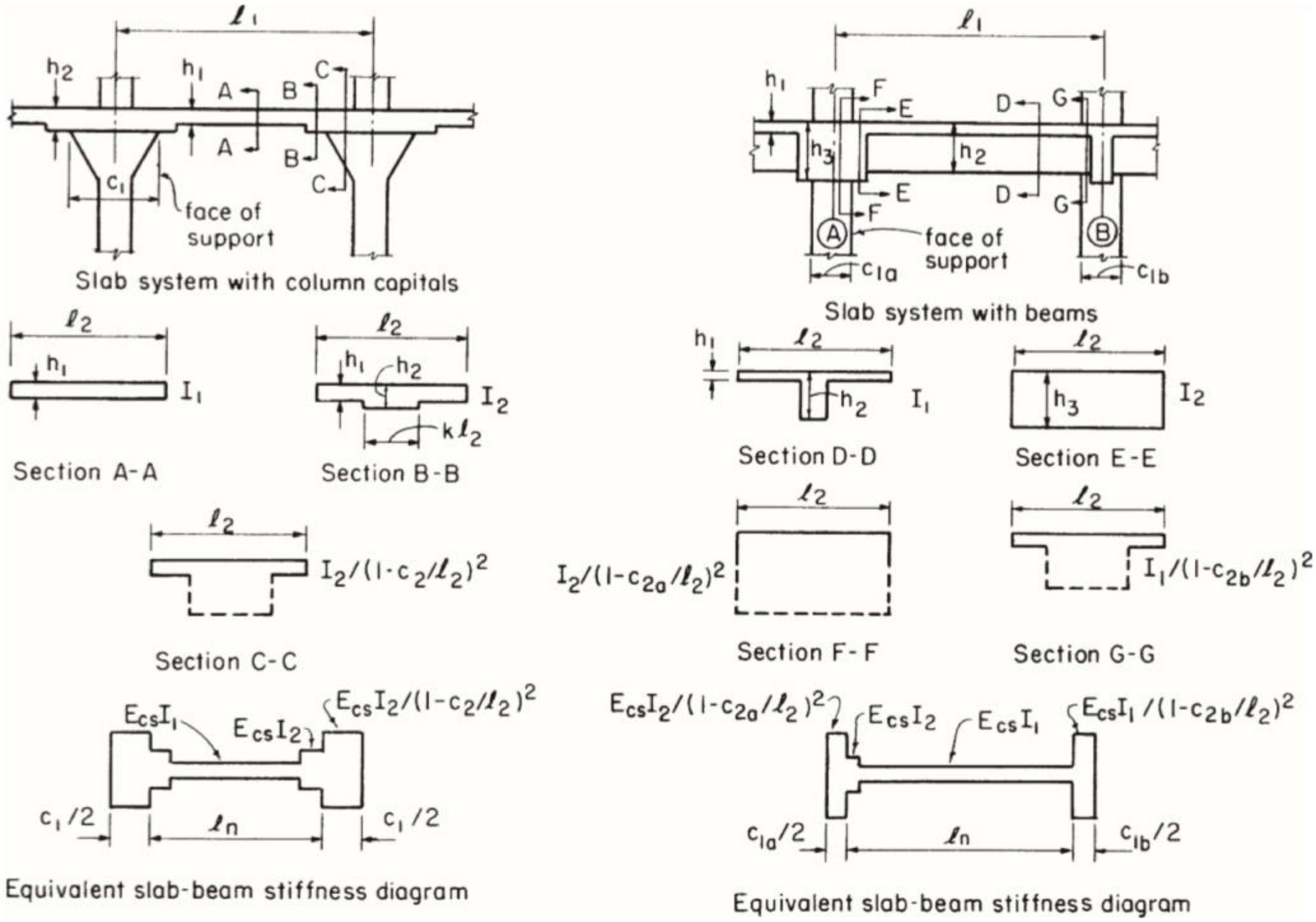
8.11.3 *Slab-beams*

8.11.3.1 The moment of inertia of slab-beams from the center of the column to the face of the column, bracket, or capital shall be assumed equal to the moment of inertia of the slab-beam at the face of the column, bracket, or capital divided by the quantity $(1 - c_2/\ell_2)^2$, where c_2 and ℓ_2 are measured transverse to the direction of the span for which moments are being determined.

8.11.3.2 Variation in moment of inertia along the axis of slab-beams shall be taken into account.

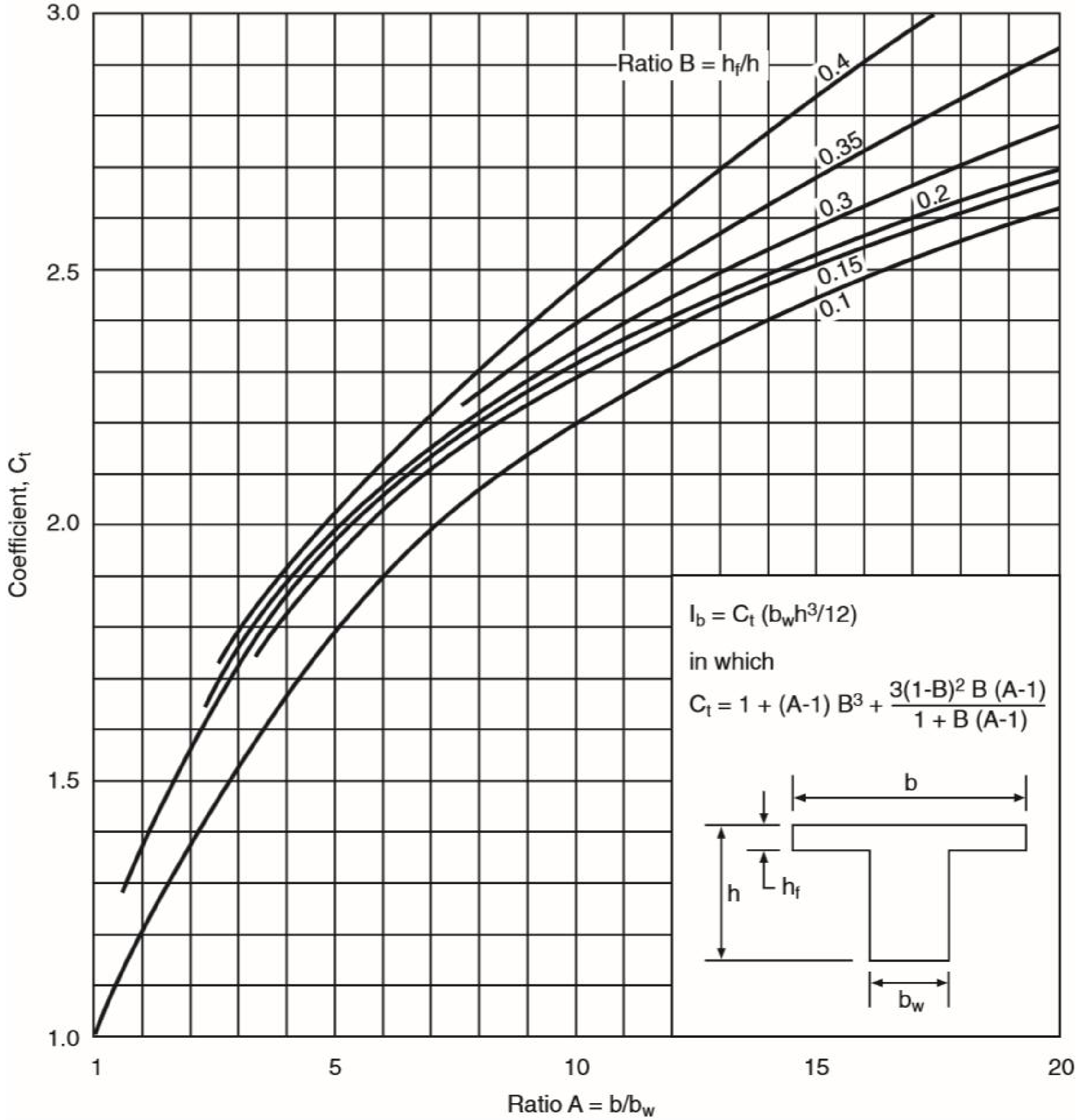
8.11.3.3 It shall be permitted to use the gross cross-sectional area of concrete to determine the moment of inertia of slab-beams at any cross section outside of joints or column capitals.




ℓ_1
h_2
h_1
A
B
C
c_1
face of support
Slab system with column capitals
ℓ_2
I_1
Section A-A
$k\ell_2$
I_2
Section B-B
$I_2/(1-c_2/\ell_2)^2$
Section C-C
$E_{cs}I_1$
$E_{cs}I_2$
$E_{cs}I_2/(1-c_2/\ell_2)^2$
$c_1/2$
ℓ_n
Equivalent slab-beam stiffness diagram
h_3
D
E
F
G
A
B
c_{1a}
c_{1b}
Slab system with beams
Section D-D
Section E-E
$I_2/(1-c_{2a}/\ell_2)^2$
Section F-F
$I_1/(1-c_{2b}/\ell_2)^2$
Section G-G
$E_{cs}I_2/(1-c_{2a}/\ell_2)^2$
$E_{cs}I_1/(1-c_{2b}/\ell_2)^2$
$c_{1a}/2$
$c_{1b}/2$
Equivalent slab-beam stiffness diagram

Example:

Moment of inertia of the slab-beam strip can be calculated from the following figure or equation:

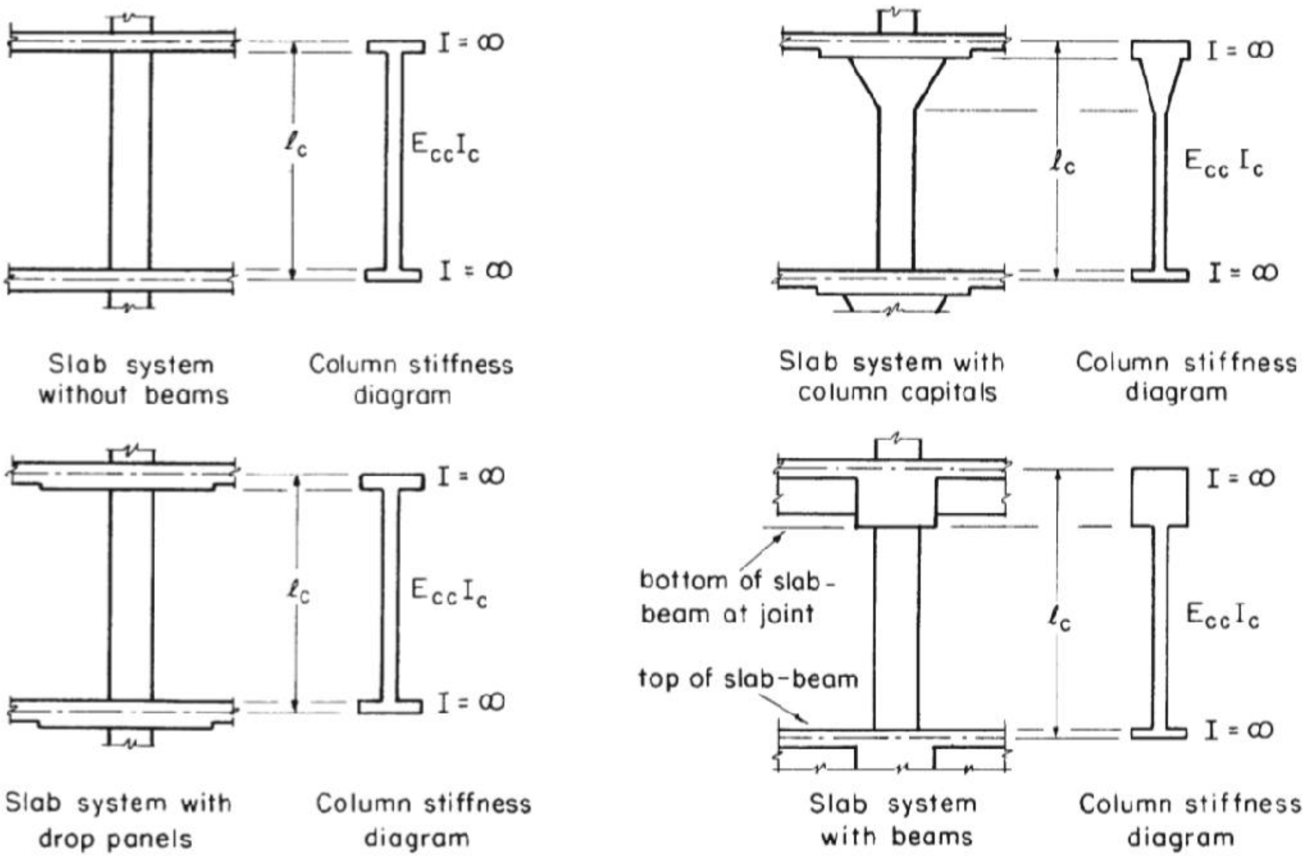


8.11.4 *Columns*

8.11.4.1 The moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed to be infinite.

8.11.4.2 Variation in moment of inertia along the axis of columns shall be taken into account.

8.11.4.3 It shall be permitted to use the gross cross-sectional area of concrete to determine the moment of inertia of columns at any cross section outside of joints or column capitals.



8.11.5 *Torsional members*

8.11.5.1 Torsional members shall be assumed to have a constant cross section throughout their length consisting of the greatest of (a) through (c):

(a) A portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined.
(b) For monolithic or fully composite construction, the portion of slab specified in (a) plus that part of the transverse beam above and below the slab.
(c) The transverse beam in accordance with 8.4.1.8.

8.11.5.2 Where beams frame into columns in the direction of the span for which moments are being calculated, the torsional stiffness shall be multiplied by the ratio of the moment of inertia of the slab with such a beam to the moment of inertia of the slab without such a beam.

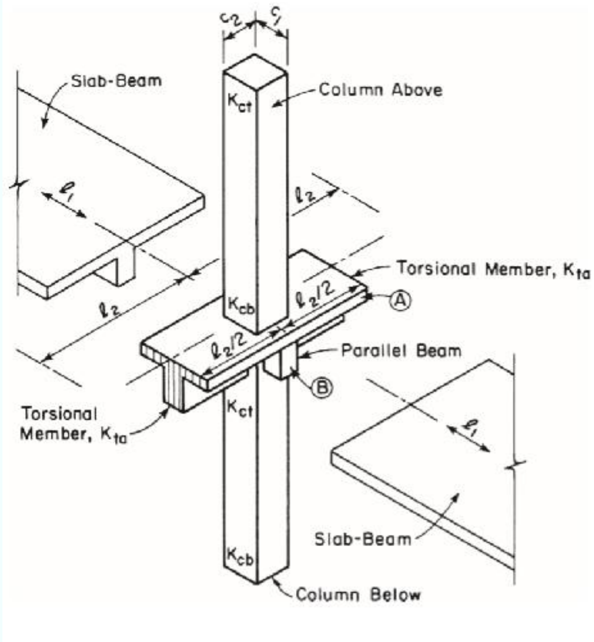


Figure 20-3 Equivalent Frame Members

$$K_t = \sum \frac{9E_{cs}C}{\ell_2\left(1-\frac{c_2}{\ell_2}\right)^3}$$

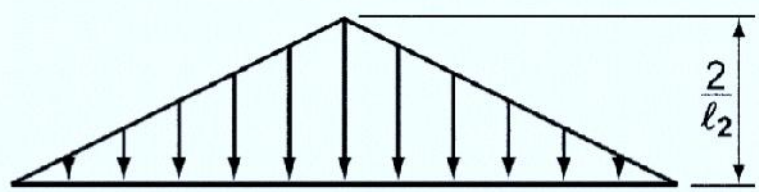


Fig. R8.11.5*—Distribution of unit twisting moment along column centerline AA shown in Fig. R8.11.4.*

$$K_{ta} = \frac{K_t I_{sb}}{I_s}$$

K_{ta} = increased torsional stiffness due to the parallel beam (note parallel beam shown in Fig. 20-3)

I_s = moment of inertia of a width of slab equal to the full width between panel centerlines, ℓ_2, excluding that portion of the beam stem extending above and below the slab (note part A in Fig. 20-3).

$$= \frac{\ell_2 h^3}{12}$$

I_{sb} = moment of inertia of the slab section specified for I_s including that portion of the beam stem extending above and below the slab (for the parallel beam illustrated in Fig. 20-3, I_{sb} is for the full tee section shown).

R8.11.4 *Columns*—Column stiffness is based on the length of the column from mid-depth of slab above to mid-depth of slab below. Column moment of inertia is calculated on the basis of its cross section, taking into account the increase in stiffness provided by the capital, if any.

If slab-beams are analyzed separately for gravity loads, the concept of an equivalent column, combining the stiffness of the slab-beam and torsional member into a composite element, is used. The column flexibility is modified to account for the torsional flexibility of the slab-to-column connection that reduces its efficiency for transmission of moments. The equivalent column consists of the actual columns above and below the slab-beam, plus attached torsional members on each side of the columns extending to the centerline of the adjacent panels, as shown in Fig. R8.11.4.

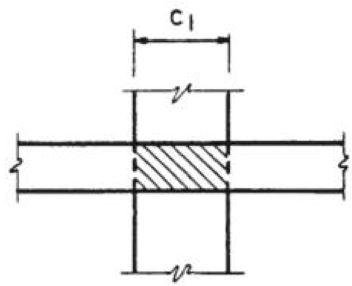
The equivalent column stiffness K_{ec} is found from the following equation, where the summation refers to the columns above and below joint and torsional members to the both sides of joint:

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_c} + \frac{1}{\Sigma K_t}$$

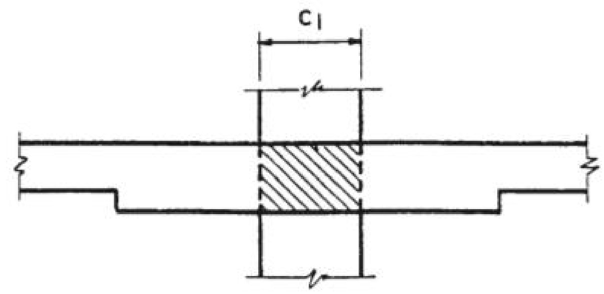
$$K_t = \Sigma \left[\frac{9E_{cs}C}{\ell_2 \left[1 - (c_2/\ell_2)\right]^3} \right]$$

C is the torsional constant found from the following equation:

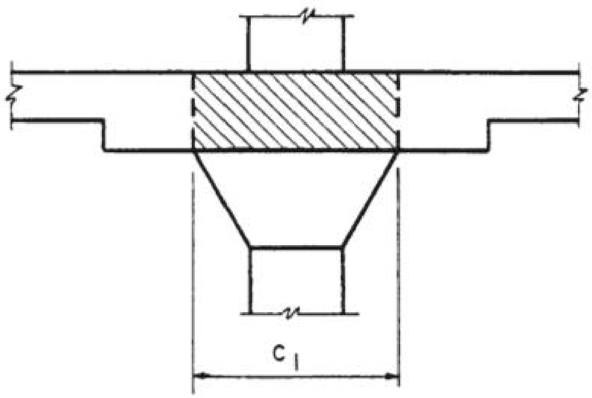
$$C = \Sigma \left[1 - 0.63 \left(\frac{x}{y} \right) \right] \frac{x^3 y}{3}$$



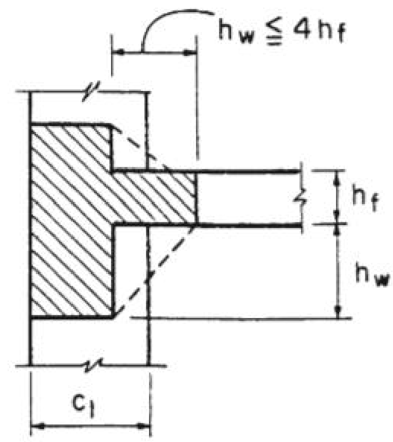
Condition (a)



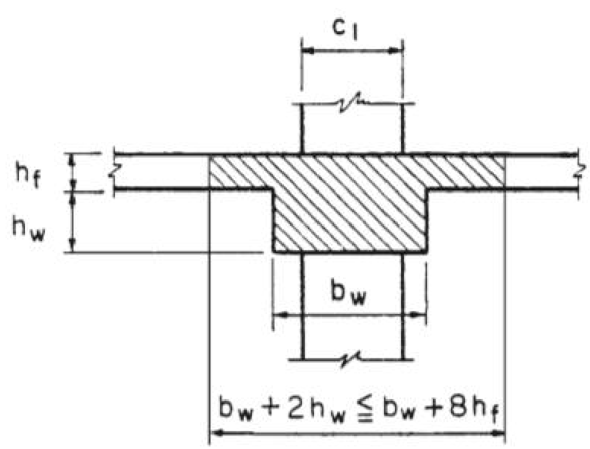
Condition (a)



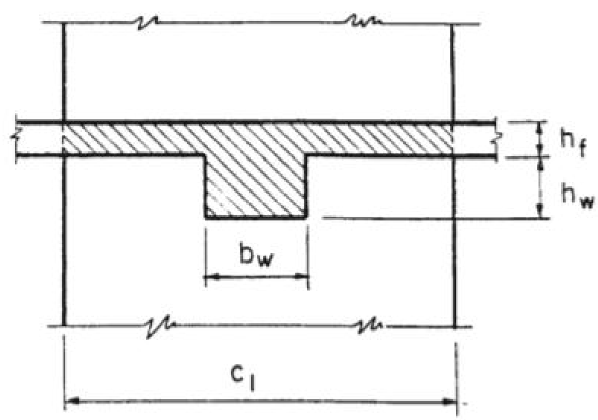
Condition (a)



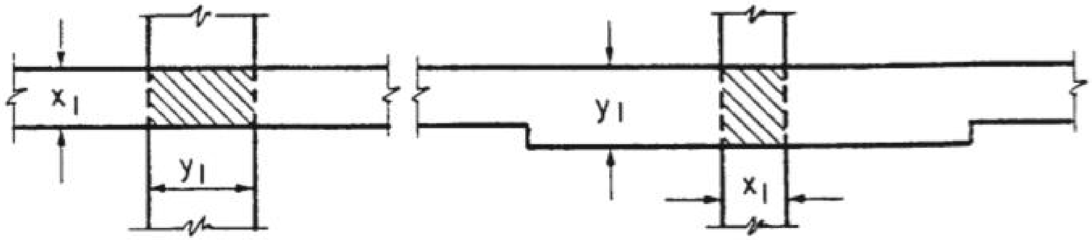
Condition (c)

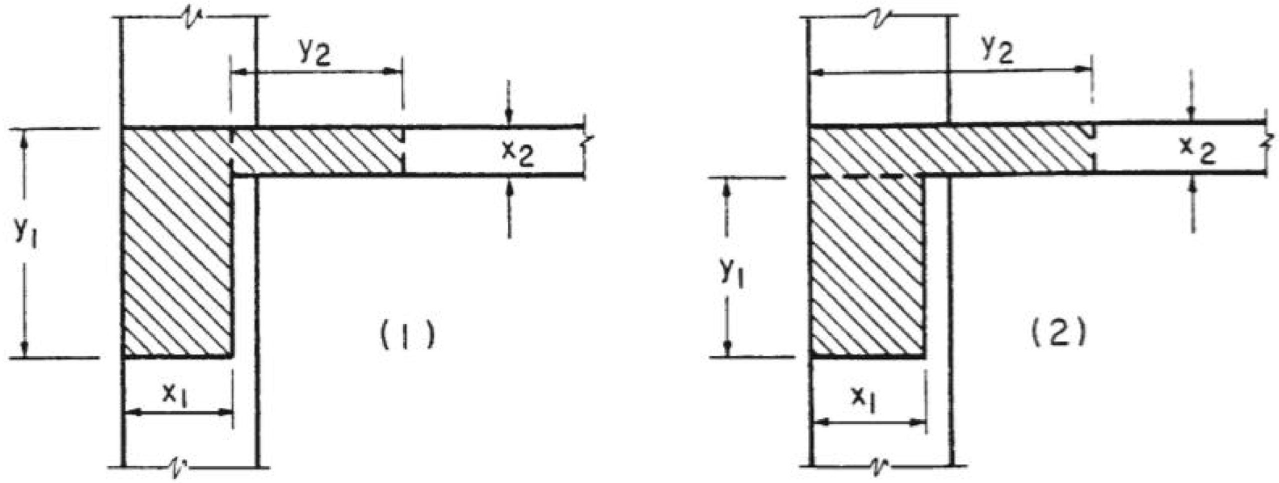


Condition (c)

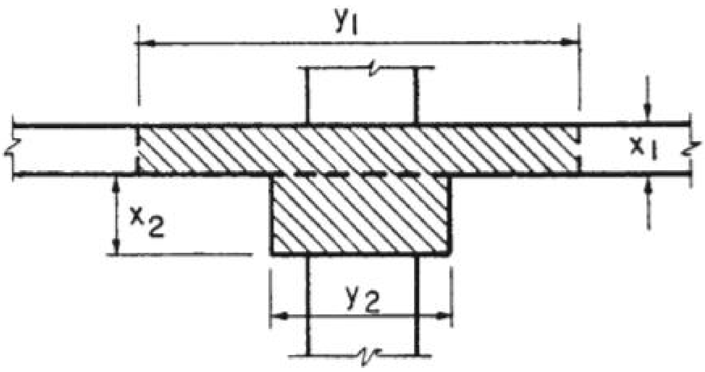


Condition (b)



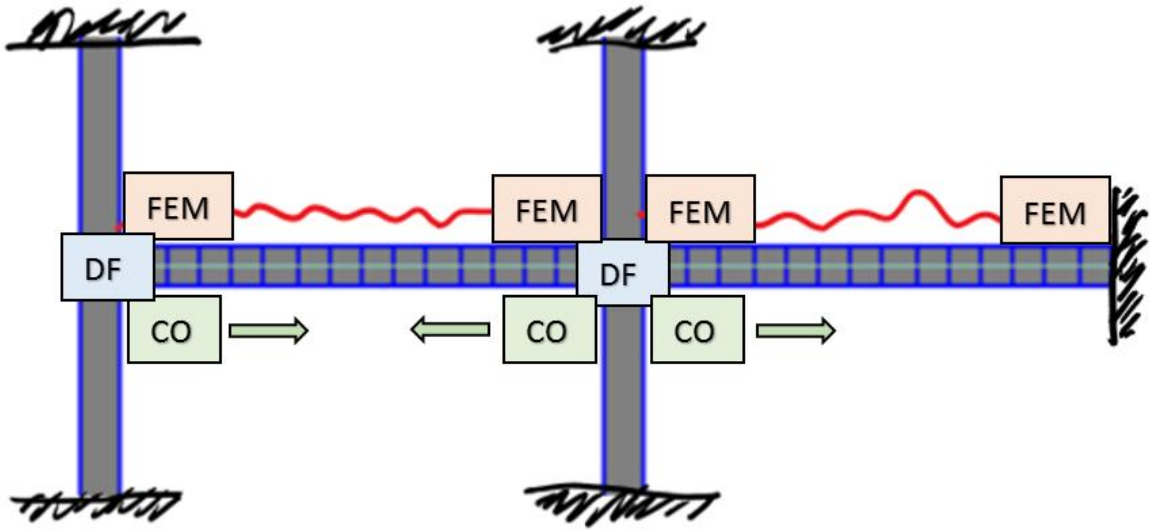


Use larger value of C computed from (1) or (2)



$$C = \sum\left[\left(1-0.63\frac{x_1}{y_1}\right)\frac{x_1^3 y_1}{3}\right]+\left[\left(1-0.63\frac{x_2}{y_2}\right)\frac{x_2^3 y_2}{3}\right]$$

Frame Analysis Using Moment Distribution Method



Fixed End Moment FEM = $W_u.L^2/12 = 0.0833\ W_u.L^2$

Carry Over factor CO = 0.5

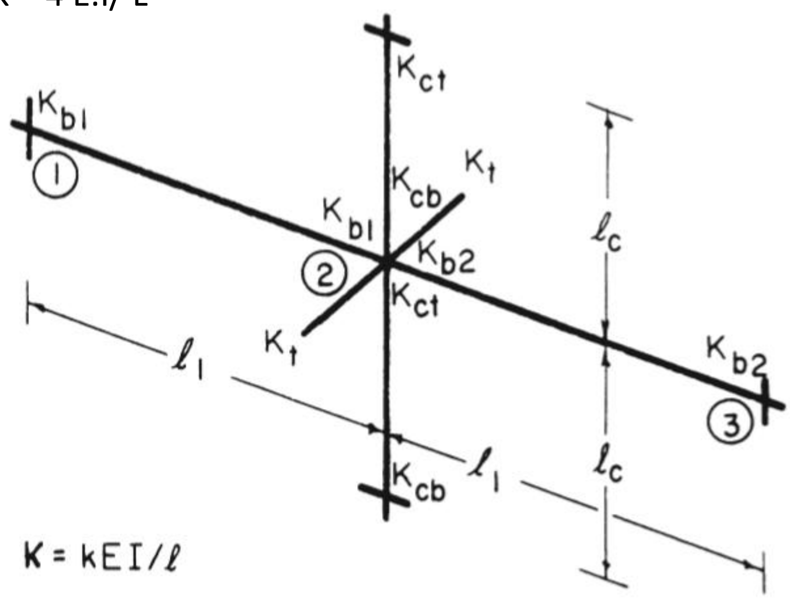
Distribution Factor DF = $K_i / \Sigma K$, where $K = 4\ E.I/ L$

$$\text{DF (span 2-1)} = \frac{K_{b1}}{K_{b1} + K_{b2} + K_{ec}}$$

$$\text{DF (span 2-3)} = \frac{K_{b2}}{K_{b1} + K_{b2} + K_{ec}}$$

Moreover, for equivalent column:

$$\text{DF} = \frac{K_{ec}}{K_{b1} + K_{b2} + K_{ec}}$$



The unbalanced moment determined for equivalent column shall be distributed to the actual columns above and below joint as follows:

$$\text{Portion of unbalanced moment to upper column} = \frac{K_{cb}}{(K_{cb} + K_{ct})}$$

$$\text{Portion of unbalanced moment to lower column} = \frac{K_{ct}}{(K_{cb} + K_{ct})}$$

The values mentioned of FEM, CO, and K are used for prismatic members, but the slab-beam and column are non-prismatic members. Hence, FEM, CO, and K will be found from the attached tables.

8.11.6 *Factored moments*

8.11.6.1 At interior supports, the critical section for negative M_u in both column and middle strips shall be taken at the face of rectilinear supports, but not farther away than $0.175\ell_1$ from the center of a column.

8.11.6.2 At exterior supports without brackets or capitals, the critical section for negative M_u in the span perpendicular to an edge shall be taken at the face of the supporting element.

8.11.6.3 At exterior supports with brackets or capitals, the critical section for negative M_u in the span perpendicular to an edge shall be taken at a distance from the face of the supporting element not exceeding one-half the projection of the bracket or capital beyond the face of the supporting element.

8.11.6.4 Circular or regular polygon-shaped supports shall be assumed to be square supports with the same area for location of critical section for negative design moment.

8.11.6.6 It shall be permitted to distribute moments at critical sections to column strips, beams, and middle strips in accordance with the direct design method in 8.10, provided that Eq. (8.10.2.7a) is satisfied.

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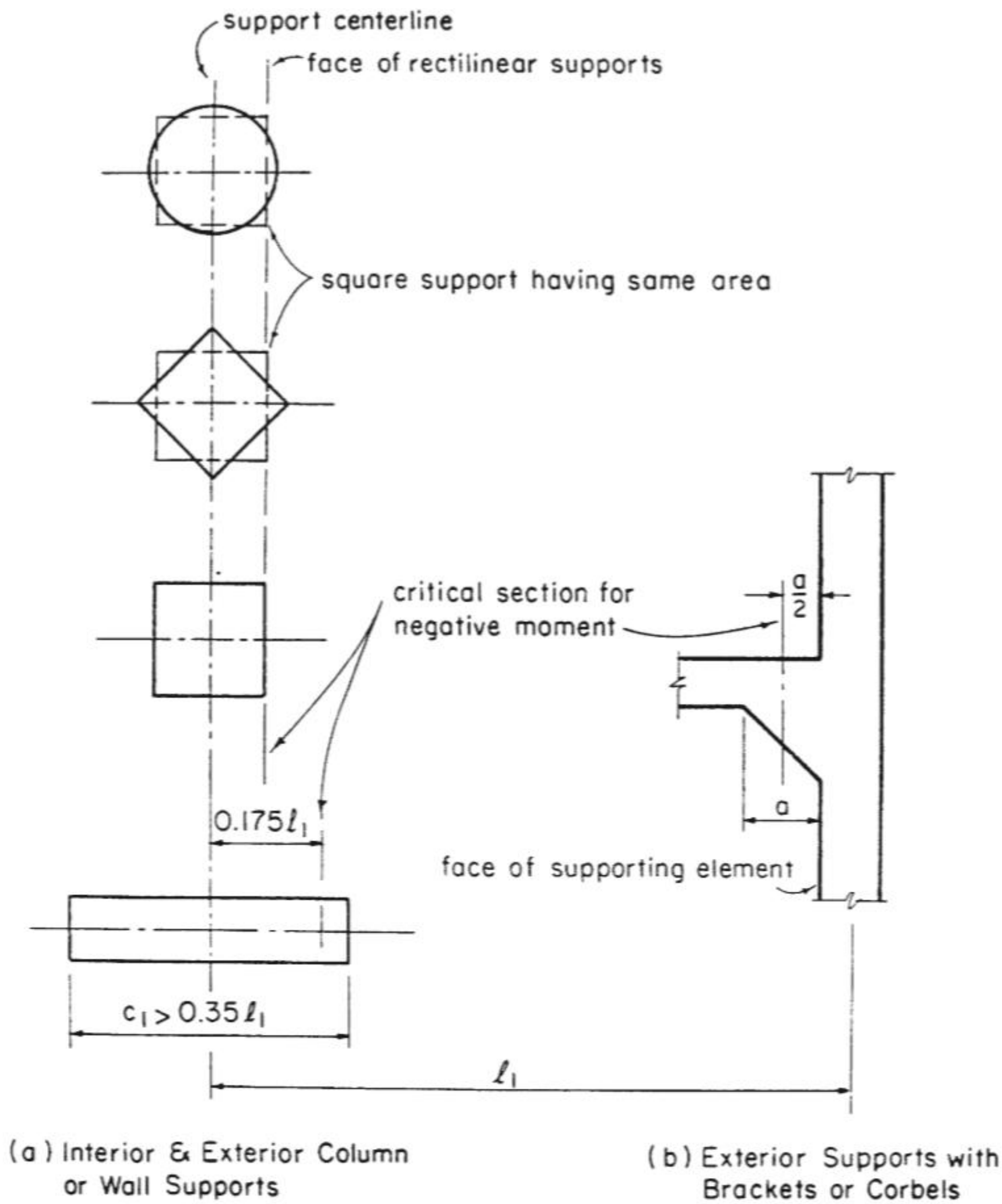
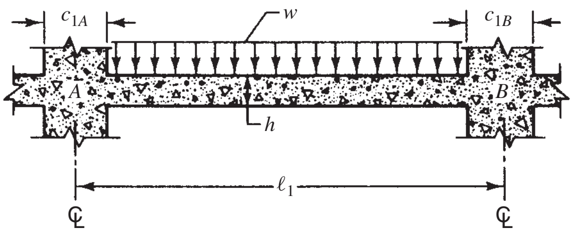


Example:

TABLE A.16 Moment Distribution Constants for Slabs Without Drop Panels[a]



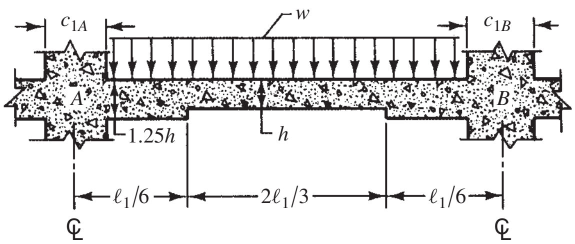
Column Dimension		Uniform Load FEM = Coef. $(w\ell_2\ell_1^2)$		Stiffness Factor†		Carryover Factor	
$\frac{c_{1A}}{\ell_1}$	$\frac{c_{1B}}{\ell_1}$	M_{AB}	M_{BA}	k_{AB}	k_{BA}	COF_{AB}	COF_{BA}
	0.00	0.083	0.083	4.00	4.00	0.500	0.500
	0.05	0.083	0.084	4.01	4.04	0.504	0.500
	0.10	0.082	0.086	4.03	4.15	0.513	0.499
	0.15	0.081	0.089	4.07	4.32	0.528	0.498
0.00	0.20	0.079	0.093	4.12	4.56	0.548	0.495
	0.25	0.077	0.097	4.18	4.88	0.573	0.491
	0.30	0.075	0.102	4.25	5.28	0.603	0.485
	0.35	0.073	0.107	4.33	5.78	0.638	0.478
	0.05	0.084	0.084	4.05	4.05	0.503	0.503
	0.10	0.083	0.086	4.07	4.15	0.513	0.503
	0.15	0.081	0.089	4.11	4.33	0.528	0.501
0.05	0.20	0.080	0.092	4.16	4.58	0.548	0.499
	0.25	0.078	0.096	4.22	4.89	0.573	0.494
	0.30	0.076	0.101	4.29	5.30	0.603	0.489
	0.35	0.074	0.107	4.37	5.80	0.638	0.481
	0.10	0.085	0.085	4.18	4.18	0.513	0.513
	0.15	0.083	0.088	4.22	4.36	0.528	0.511

TABLE A.16 (*Continued*)

Column Dimension		Uniform Load FEM = Coef. ($w\ell_2\ell_1^2$)		Stiffness Factor[†]		Carryover Factor	
$\frac{c_{1A}}{\ell_1}$	$\frac{c_{1B}}{\ell_1}$	M_{AB}	M_{BA}	k_{AB}	k_{BA}	COF_{AB}	COF_{BA}
0.10	0.20	0.082	0.091	4.27	4.61	0.548	0.508
	0.25	0.080	0.095	4.34	4.93	0.573	0.504
	0.30	0.078	0.100	4.41	5.34	0.602	0.498
	0.35	0.075	0.105	4.50	5.85	0.637	0.491
	0.15	0.086	0.086	4.40	4.40	0.526	0.526
	0.20	0.084	0.090	4.46	4.65	0.546	0.523
0.15	0.25	0.083	0.094	4.53	4.98	0.571	0.519
	0.30	0.080	0.099	4.61	5.40	0.601	0.513
	0.35	0.078	0.104	4.70	5.92	0.635	0.505
	0.20	0.088	0.088	4.72	4.72	0.543	0.543
0.20	0.25	0.086	0.092	4.79	5.05	0.568	0.539
	0.30	0.083	0.097	4.88	5.48	0.597	0.532
	0.35	0.081	0.102	4.99	6.01	0.632	0.524
	0.25	0.090	0.090	5.14	5.14	0.563	0.563
0.25	0.30	0.088	0.095	5.24	5.58	0.592	0.556
	0.35	0.085	0.100	5.36	6.12	0.626	0.548
0.30	0.30	0.092	0.092	5.69	5.69	0.585	0.585
	0.35	0.090	0.097	5.83	6.26	0.619	0.576
0.35	0.35	0.095	0.095	6.42	6.42	0.609	0.609

[a] Applicable when $c_1/\ell_1 = c_2/\ell_2$. For other relationships between these ratios, the constants will be slightly in error.
[†] Stiffness is $K_{AB} = k_{AB}E(\ell_2 h^3/12\ell_1)$ and $K_{BA} = k_{BA}E(\ell_2 h^3/12\ell_1)$

TABLE A.17 Moment Distribution Constants for Slabs with Drop Panels[a]



Column Dimension		Uniform Load FEM = Coef. ($w\ell_2\ell_1^2$)		Stiffness Factor[†]		Carryover Factor	
$\frac{c_{1A}}{\ell_1}$	$\frac{c_{1B}}{\ell_1}$	M_{AB}	M_{BA}	k_{AB}	k_{BA}	COF_{AB}	COF_{BA}
	0.00	0.088	0.088	4.78	4.78	0.541	0.541
	0.05	0.087	0.089	4.80	4.82	0.545	0.541
	0.10	0.087	0.090	4.83	4.94	0.553	0.541
0.00	0.15	0.085	0.093	4.87	5.12	0.567	0.540
	0.20	0.084	0.096	4.93	5.36	0.585	0.537
	0.25	0.082	0.100	5.00	5.68	0.606	0.534
	0.30	0.080	0.105	5.09	6.07	0.631	0.529
	0.05	0.088	0.088	4.84	4.84	0.545	0.545
	0.10	0.087	0.090	4.87	4.95	0.553	0.544
	0.15	0.085	0.093	4.91	5.13	0.567	0.543
0.05	0.20	0.084	0.096	4.97	5.38	0.584	0.541
	0.25	0.082	0.100	5.05	5.70	0.606	0.537
	0.30	0.080	0.104	5.13	6.09	0.632	0.532
	0.10	0.089	0.089	4.98	4.98	0.553	0.553
	0.15	0.088	0.092	5.03	5.16	0.566	0.551
0.10	0.20	0.086	0.094	5.09	5.42	0.584	0.549
	0.25	0.084	0.099	5.17	5.74	0.606	0.546
	0.30	0.082	0.103	5.26	6.13	0.631	0.541
	0.15	0.090	0.090	5.22	5.22	0.565	0.565
	0.20	0.089	0.094	5.28	5.47	0.583	0.563
0.15	0.25	0.087	0.097	5.37	5.80	0.604	0.559
	0.30	0.085	0.102	5.46	6.21	0.630	0.554
	0.20	0.092	0.092	5.55	5.55	0.580	0.580
0.20	0.25	0.090	0.096	5.64	5.88	0.602	0.577
	0.30	0.088	0.100	5.74	6.30	0.627	0.571
0.25	0.25	0.094	0.094	5.98	5.98	0.598	0.598
	0.30	0.091	0.098	6.10	6.41	0.622	0.593
0.30	0.30	0.095	0.095	6.54	6.54	0.617	0.617

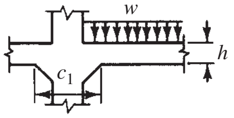
[a] Applicable when $c_1/\ell_1 = c_2/\ell_2$. For other relationships between these ratios, the constants will be slightly in error.
[†] Stiffness is $K_{AB} = k_{AB}E(\ell_2 h^3/12\ell_1)$ and $K_{BA} = k_{BA}E(\ell_2 h^3/12\ell_1)$

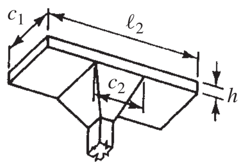
TABLE A.18 Moment Distribution Constants for Slab-Beam Members with Column Capitals

FEM (uniform load w) $= Mw\ell_2(\ell_1)^2$

K (stiffness) $= \dfrac{kE\ell_2 h^3}{12\ell_1}$

Carryover factor $= C$





$\frac{c_1}{\ell_1}$	$\frac{c_1}{\ell_2}$	M	k	C	$\frac{c_1}{\ell_1}$	$\frac{c_1}{\ell_2}$	M	k	C
	0.00	0.083	4.000	0.500	0.10	0.25	0.087	4.449	0.530
	0.05	0.083	4.000	0.500		0.30	0.087	4.535	0.535
	0.10	0.083	4.000	0.500		0.35	0.088	4.618	0.540
	0.15	0.083	4.000	0.500		0.40	0.088	4.698	0.545
	0.20	0.083	4.000	0.500		0.45	0.089	4.774	0.550
0.00	0.25	0.083	4.000	0.500		0.50	0.089	4.846	0.554
	0.30	0.083	4.000	0.500		0.00	0.083	4.000	0.500
	0.35	0.083	4.000	0.500		0.05	0.084	4.132	0.509
	0.40	0.083	4.000	0.500		0.10	0.085	4.267	0.517
	0.45	0.083	4.000	0.500		0.15	0.086	4.403	0.526
	0.50	0.083	4.000	0.500		0.20	0.087	4.541	0.534
	0.00	0.083	4.000	0.500	0.15	0.25	0.088	4.680	0.543
	0.05	0.084	4.047	0.503		0.30	0.089	4.818	0.550
	0.10	0.084	4.093	0.507		0.00	0.083	4.000	0.500
	0.15	0.084	4.138	0.510		0.05	0.085	4.170	0.511
	0.20	0.085	4.181	0.513		0.10	0.086	4.346	0.522
0.05	0.25	0.085	4.222	0.516		0.15	0.087	4.529	0.532
	0.30	0.085	4.261	0.518		0.20	0.088	4.717	0.543
	0.35	0.086	4.299	0.521	0.20	0.25	0.089	4.910	0.554
	0.40	0.086	4.334	0.523		0.30	0.090	5.108	0.564
	0.45	0.086	4.368	0.526		0.35	0.091	5.308	0.574
	0.50	0.086	4.398	0.528		0.40	0.092	5.509	0.584
	0.00	0.083	4.000	0.500		0.45	0.093	5.710	0.593
	0.05	0.084	4.091	0.506		0.50	0.094	5.908	0.602
	0.10	0.085	4.182	0.513		0.00	0.083	4.000	0.500
	0.15	0.085	4.272	0.519		0.05	0.085	4.204	0.512
	0.20	0.086	4.362	0.524		0.10	0.086	4.420	0.525

(*continues*)

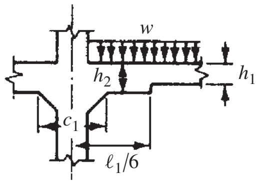
TABLE A.18 *(Continued)*

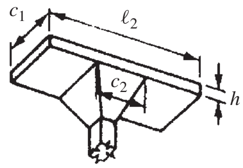
$\frac{c_1}{\ell_1}$	$\frac{c_1}{\ell_2}$	M	k	C	$\frac{c_1}{\ell_1}$	$\frac{c_1}{\ell_2}$	M	k	C
	0.15	0.087	4.648	0.538		0.20	0.090	5.348	0.563
	0.20	0.089	4.887	0.550	0.40	0.25	0.092	5.778	0.580
0.25	0.25	0.090	5.138	0.563		0.30	0.094	6.255	0.598
	0.30	0.091	5.401	0.576		0.35	0.095	6.782	0.617
	0.35	0.093	5.672	0.588		0.40	0.097	7.365	0.635
	0.40	0.094	5.952	0.600		0.45	0.099	8.007	0.654
	0.45	0.095	6.238	0.612		0.50	0.100	8.710	0.672
	0.50	0.096	6.527	0.623		0.00	0.083	4.000	0.500
	0.00	0.083	4.000	0.500		0.05	0.085	4.311	0.515
	0.05	0.085	4.235	0.514		0.10	0.087	4.658	0.530
	0.10	0.086	4.488	0.527		0.15	0.088	5.046	0.547
	0.15	0.088	4.760	0.542		0.20	0.090	5.480	0.564
	0.20	0.089	5.050	0.556	0.45	0.25	0.092	5.967	0.583
0.30	0.25	0.091	5.361	0.571		0.35	0.095	6.416	0.609
	0.30	0.092	5.692	0.585		0.40	0.096	6.888	0.626
	0.35	0.094	6.044	0.600		0.45	0.098	7.395	0.642
	0.40	0.095	6.414	0.614		0.50	0.099	7.935	0.658
	0.45	0.096	6.802	0.628		0.30	0.094	6.517	0.602
	0.50	0.098	7.205	0.642		0.35	0.096	7.136	0.621
	0.00	0.083	4.000	0.500		0.40	0.098	7.836	0.642
	0.05	0.085	4.264	0.514		0.45	0.100	8.625	0.662
	0.10	0.087	4.551	0.529		0.50	0.101	9.514	0.683
	0.15	0.088	4.864	0.545		0.00	0.083	4.000	0.500
	0.20	0.090	5.204	0.560		0.05	0.085	4.331	0.515
0.35	0.25	0.091	5.575	0.576		0.10	0.087	4.703	0.530
	0.30	0.093	5.979	0.593		0.15	0.088	5.123	0.547
	0.35	0.090	4.955	0.558		0.20	0.090	5.599	0.564
	0.40	0.090	5.090	0.565	0.50	0.25	0.092	6.141	0.583
	0.45	0.091	5.222	0.572		0.30	0.094	6.760	0.603
	0.50	0.092	5.349	0.579		0.35	0.096	7.470	0.624
	0.00	0.083	4.000	0.500		0.40	0.098	8.289	0.645
	0.05	0.085	4.289	0.515		0.45	0.100	9.234	0.667
	0.10	0.087	4.607	0.530		0.50	0.102	10.329	0.690
	0.15	0.088	4.959	0.546					

TABLE A.19 Moment Distribution Constants for Slab-Beam Members with Column Capitals and Drop Panels

FEM (uniform load w) $= Mw\ell(\ell_1^2)$

K (stiffness) $= \dfrac{kE\ell_2 h^3}{12\ell_1}$





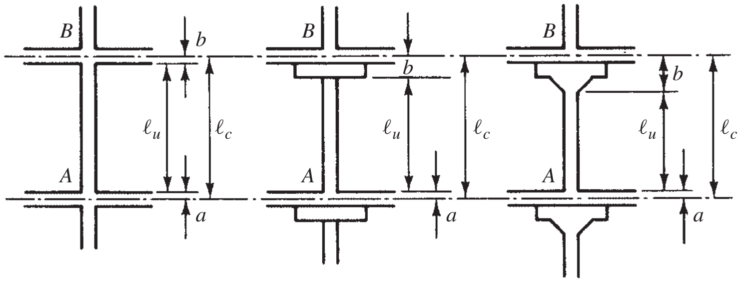
$\frac{c_1}{\ell_1}$	$\frac{c_1}{\ell_2}$	Constants for $h_2 = 1.25h_1$			Constants for $h_2 = 1.5h_2$		
		M	***k***	***C***	***M***	***k***	***C***
0.00	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.088	4.795	0.542	0.093	5.837	0.589
	0.10	0.088	4.795	0.542	0.093	5.837	0.589
	0.15	0.088	4.795	0.542	0.093	5.837	0.589
	0.20	0.088	4.795	0.542	0.093	5.837	0.589
	0.25	0.088	4.795	0.542	0.093	5.837	0.589
	0.30	0.088	4.797	0.542	0.093	5.837	0.589
0.05	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.088	4.846	0.545	0.093	5.890	0.591
	0.10	0.089	4.896	0.548	0.093	5.942	0.594
	0.15	0.089	4.944	0.551	0.093	5.993	0.596
	0.20	0.089	4.990	0.553	0.094	6.041	0.598
	0.25	0.089	5.035	0.556	0.094	6.087	0.600
	0.30	0.090	5.077	0.558	0.094	6.131	0.602
0.10	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.088	4.894	0.548	0.093	5.940	0.593
	0.10	0.089	4.992	0.553	0.094	6.042	0.598
	0.15	0.090	5.039	0.559	0.094	6.142	0.602
	0.20	0.090	5.184	0.564	0.094	6.240	0.607
	0.25	0.091	5.278	0.569	0.095	6.335	0.611
	0.30	0.091	5.368	0.573	0.095	6.427	0.615
	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.089	4.938	0.550	0.093	5.986	0.595
	0.10	0.090	5.082	0.558	0.094	6.135	0.602

(*continues*)

TABLE A.19 (*Continued*)

$\frac{c_1}{\ell_1}$	$\frac{c_1}{\ell_2}$	Constants for $h_2 = 1.25h_1$			Constants for $h_2 = 1.5h_2$		
		M	k	C	M	k	C
0.15	0.15	0.090	5.228	0.565	0.095	6.284	0.608
	0.20	0.091	5.374	0.573	0.095	6.432	0.614
	0.25	0.092	5.520	0.580	0.096	6.579	0.620
	0.30	0.092	5.665	0.587	0.096	6.723	0.626
	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.089	4.978	0.552	0.093	6.027	0.597
	0.10	0.090	5.167	0.562	0.094	6.221	0.605
0.20	0.15	0.091	5.361	0.571	0.095	6.418	0.613
	0.20	0.092	5.558	0.581	0.096	6.616	0.621
	0.25	0.093	5.760	0.590	0.096	6.816	0.628
	0.30	0.094	5.962	0.590	0.097	7.015	0.635
	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.089	5.015	0.553	0.094	6.065	0.598
	0.10	0.090	5.245	0.565	0.094	6.300	0.608
0.25	0.15	0.091	5.485	0.576	0.095	6.543	0.617
	0.20	0.092	5.735	0.587	0.096	6.790	0.626
	0.25	0.094	5.994	0.598	0.097	7.043	0.635
	0.30	0.095	6.261	0.600	0.098	7.298	0.644
	0.00	0.088	4.795	0.542	0.093	5.837	0.589
	0.05	0.089	5.048	0.554	0.094	6.099	0.599
	0.10	0.090	5.317	0.567	0.095	6.372	0.610
0.30	0.15	0.092	5.601	0.580	0.096	6.657	0.620
	0.20	0.093	5.902	0.593	0.097	6.953	0.631
	0.25	0.094	6.219	0.605	0.098	7.258	0.641
	0.30	0.095	6.550	0.618	0.099	7.571	0.651

TABLE A.20 Stiffness Factors and Carryover Factors for Columns



a/b	ℓ_u/ℓ_n	0.95	0.90	0.85	0.80	0.75
0.20	k_{AB}	4.32	4.70	5.33	5.65	6.27
	C_{AB}	0.57	0.64	0.71	0.80	0.89
0.40	k_{AB}	4.40	4.89	5.45	6.15	7.00
	C_{AB}	0.56	0.61	0.68	0.74	0.81
0.60	k_{AB}	4.46	5.02	5.70	6.54	7.58
	C_{AB}	0.55	0.60	0.65	0.70	0.76
0.80	k_{AB}	4.51	5.14	5.90	6.85	8.05
	C_{AB}	0.54	0.58	0.63	0.67	0.72
1.00	k_{AB}	4.55	5.23	6.06	7.11	8.44
	C_{AB}	0.54	0.57	0.61	0.65	0.68
1.20	k_{AB}	4.58	5.30	6.20	7.32	8.77
	C_{AB}	0.53	0.57	0.60	0.63	0.66
1.40	k_{AB}	4.61	5.36	6.31	7.51	9.05
	C_{AB}	0.53	0.56	0.59	0.61	0.64
1.60	k_{AB}	4.63	5.42	6.41	7.66	9.29
	C_{AB}	0.53	0.55	0.58	0.60	0.62
1.80	k_{AB}	4.65	5.46	6.49	7.80	9.50
	C_{AB}	0.53	0.55	0.57	0.59	0.60
2.00	k_{AB}	4.67	5.51	6.56	7.92	9.68
	C_{AB}	0.52	0.54	0.56	0.58	0.59

Notes:

1. Values computed by column analogy method.
2. $k_c = k_{AB}$ from table (EI_0/ℓ_n).