REFRESHER COURSE and FIELD REFERENCE MANUAL for Site Engineers & Inspectors

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12th-SUN.DEC '10

REFRESHER COURSE

and

FIELD REFERENCE MANUAL

for

Site Engineers and Inspectors

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PREFACE TO THE FIRST EDITION

This book is targeted primarily to site engineers and inspectors who are engaged in actual supervision and inspection of building construction, and not particularly to designers in the design office. Obviously, in order to obtain high quality and efficiency of construction, site supervisors and inspectors need to have good knowledge about structural specifications, detailing requirements, concrete quality control, basics of structural analysis and foundation engineering and estimating of quantity and cost. Sometimes field engineers may also need to or want to make a quick check of the design of certain structural members which they are in doubt or even make simple designs on their own. For all these purposes they need, for quick reference and as an aid, a handy pocket manual or a compact resource book which include useful data, formulae, procedures and guidelines on topics normally encountered in actual construction. The author hopes that this small book will fulfil this need to some extent.

The author would like to express his sincere thanks to Daw Nan Kay Zar Wint, Structural Designer and Instructor at the PIONEER Structural Design Group for her valuable contribution and assistance in preparing this book in this unusual format. The author also appreciates the understanding shown by other members of the PIONEER Structural Design Group and the assistance and encouragement offered by friends and former students of RIT/YIT/YTU, far and near.

Nyi Hla Nge

Yangon

May, 2010

REFRESHER COURSE AND FIELD REFERENCE MANUAL
FOR
SITE ENGINEERS AND INSPECTORS

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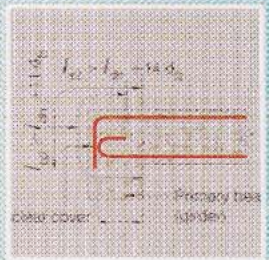


STRUCTURAL SPECIFICATIONS FOR R. C. CONSTRUCTION

A. STRUCTURAL SPECIFICATIONS FOR R. C. CONSTRUCTION

(1) Material Strengths

Concrete

* ACI Code uses cylinder strength $f_c^{'}$ as the specified compressive strength of concrete.
* In Myanmar cube strength test results are normally provided by concrete testing laboratories.
* Therefore, conversion is necessary if ACI Code is used in design.
* CQHP guidelines:

$$f_{c,cube}^{'} \cong f_c^{'} \div 0.78 \ (f_c^{'} \leq 3500 \text{ psi})$$

$$f_{c,cube}^{'} \cong f_c^{'} \div 0.80 \ (3500 < f_c^{'} \leq 5000 \text{ psi})$$

$$f_{c,cube}^{'} \cong f_c^{'} \div 0.81 \ (5000 < f_c^{'} \leq 6000 \text{ psi})$$

$$f_{c,cube}^{'} \cong f_c^{'} \div 0.83 \ (f_c^{'} > 6000 \text{ psi})$$

$f_c^{'}$ (psi)	$f_{c,cube}^{'}$ (psi)(CQHP)
2000	2564
2500	3205
3000	3846
3500	4487

* (REYNOLDS *et al*) According to EN Eurocode or EC 2 on Concrete Structures,

$$f_{c,cube}^{'} \cong f_c^{'} \div 0.80 \ (f_c^{'} \leq 6000 \text{ psi})$$

$$f_{c,cube}^{'} \cong f_c^{'} \div 0.82 \ (f_c^{'} \cong 6000 \text{ psi} - 10000 \text{ psi})$$

$$f_{c,cube}^{'} \cong f_c^{'} \div 0.86 \ (f_c^{'} \cong 13000 \text{ psi})$$

The ratio ($f_c^{'} / f_{c,cube}^{'}$) increases strongly with an increase in strength.

* See " PART D. CONCRETE QUALITY CONTROL " for acceptance criteria of concrete and for required (targeted) average compressive strength to be used in making mix design. Use ASTM C31 for making and curing specimens and ASTM C143 for slump test.

(1) Material Strengths *contd.*

Reinforcing Steel

* f_y (= yield strength) is the most useful property for designers.
* Tensile strength (f_{ult}) and percentage elongation are other important properties of steel.
* Plain bars — little bond; require end hooks for anchorage; seldom used today in construction
* Deformed bars — better bond; hooks can be eliminated in many cases
* Sizes available in inch (No. 3 through No. 11 , No. 14 , No. 18). For example,
  - No. 3 means 3/8 inch diameter bar
  - No. 6 means 6/8 (= 3/4) inch diameter bar
* Today, in Myanmar, most bars are available in millimeter.
* For steel on present-day Myanmar market,

  f_y < 40000 psi or ≅ 40000 psi (local)

  f_y = 50000 psi to 60000 psi (foreign)
* Test steel bars (2 specimens for each size and from every batch)

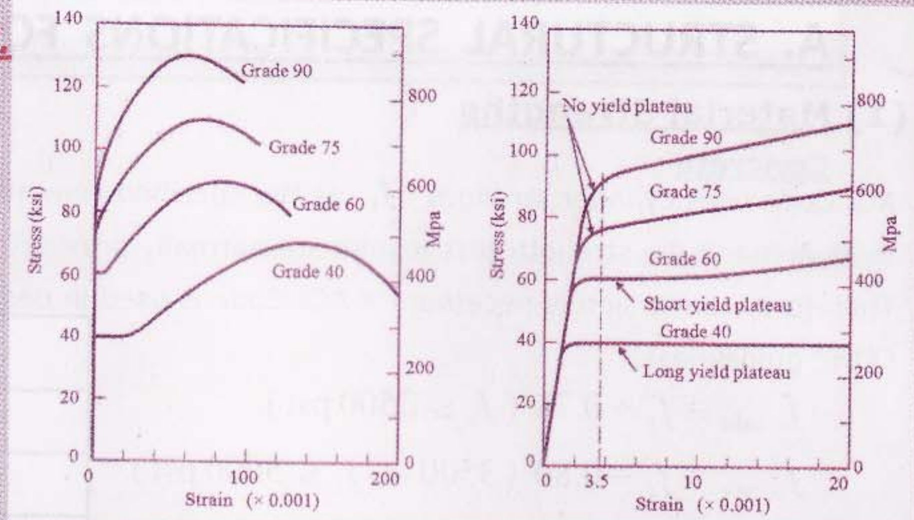


Fig. Typical stress-strain curves for reinforcing bars

Table - Summary of minimum ASTM strength requirements

ASTM Specification	Designation	Minimum Yield Strength, psi (MPa)	Minimum Tensile Strength, psi (MPa)
A 615	Grade 40	40000 (280)	60000 (420)
	Grade 60	60000 (420)	90000 (620)
	Grade 75	75000 (520)	100000 (690)
A 706	Grade 60	60000 (420) [78000 (540) max.]	80000 (550)[a]
A 996	Grade 40	40000 (280)	60000 (420)
	Grade 50	50000 (350)	80000 (550)
	Grade 60	60000 (420)	90000 (620)

[a] But not less than 1.25 times the actual yield strength

(2) Concrete Cover

Concrete Protection for Reinforcement (net concrete cover)

The following minimum thicknesses of concrete cover outside of the outermost steel are specified .

Particulars	Not exposed directly to weather or not in contact with ground	Exposed directly to weather or in contact with ground
Slabs and Walls	¾ in.	2 in. but 1 ½ in. for No. 5 (16mm) and smaller bars
Beams and columns	1½ in.	2 in.

If concrete is poured in direct contact with the ground without the use of forms, a cover of at least 3 in. shall be provided. However, it may be reduced, but not to be less than 2 in., if at least 3 in. thick (1:3:6) lean concrete is placed under the footing.

(3) Bar Spacing (For maximum size of aggregate 1 inch)

Type of member	Minimum clear spacing	Maximum c/c spacing
Beam	1⅓"	-
Column	1½ d_b (or) 1½"	-
Slab / Stair	3" (to save labour)	3h (or) 18" (one-way slab) 2h (or) 18" (two-way slab) Temp. & shr. steel: 5h (or) 18"

Note: clear distance between layers of beam reinforcement must not be less than 1 in. and the bars in the upper layer should be placed directly above those in the lower layer. Bars in beams must be placed symmetrically about the vertical centreline.

where d_b = diameter of the largest bar , h = thickness of slab

(9) Development Length of Standard Hooks , l_{dh}

l_{hb} = basic development length for standard hooks

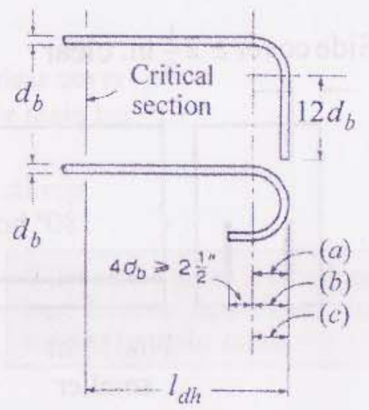
l_{dh} = development length for standard hooks

l_{dh} = l_{hb} × modification factor

$$\frac{l_{hb}}{d_b} = \frac{f_y}{50\sqrt{f_c'}}$$

Basic development length for standard hooks, l_{hb}

f_c' (ksi)	f_y (ksi)		
	40	50	60
2.5	16.0 d_b	20.0 d_b	24.0 d_b
3.0	14.6 d_b	18.3 d_b	21.9 d_b
3.5	13.5 d_b	16.9 d_b	20.3 d_b



(a) 4 d_b for Nos. 3 through 8 bars
(b) 5 d_b for Nos. 9 through 11 bars
(c) 6 d_b for Nos. 14 and 18 bars

Fig. Details of standard hooks

Modification factors (to multiply) :

(i) If side cover of main bars ≥ 2.5 in. clear, and for 90° hook if clear cover on bar extension also ≥ 2.0 in. 0.7

(ii) If hook is enclosed within closed stirrups at spacing ≤ 3 d_b along l_{dh} 0.8

(iii) Reinforcement in excess of that required .... $A_{s,\ required} / A_{s,\ provided}$

Note : Minimum l_{dh} = 8 d_b ≥ 6 in.

(4) Simplified Tension Development Length , l_d

$$\frac{l_d}{d_b} = \frac{f_y\,\alpha}{25\sqrt{f_c'}} \quad \text{for No. 6 (20 mm) and smaller bars}$$

$$\frac{l_d}{d_b} = \frac{f_y\,\alpha}{20\sqrt{f_c'}} \quad \text{for No. 7 (22 mm) and larger bars}$$

where α = 1.3 for top bars

(i.e., 12 inch or more of concrete is cast in a single concreting below the development length or splice in question)

= 1.0 for other bars

Modification factor = $A_{s,\,required} \,/\, A_{s,\,provided}$ (to multiply)

$l_d \geq$ 12 inch (in all cases)

Unmodified l_d for other bars

f_y (ksi)	f_c' (ksi)					
	2.5		3.0		3.5	
	≤20 mm	≥22 mm	≤20 mm	≥22 mm	≤20 mm	≥22 mm
40	32 d_b	40 d_b	29.2 d_b	36.5 d_b	27.0 d_b	33.8 d_b
50	40 d_b	50 d_b	36.5 d_b	45.6 d_b	33.8 d_b	42.3 d_b
60	48 d_b	60 d_b	43.8 d_b	54.8 d_b	40.6 d_b	50.8 d_b

For top bars multiply by 1.3.
Modify if applicable

For example, for 20 mm top bars, f_c' = 2500 psi concrete,

f_y = 40000 psi steel,

$A_{s,\,required}$ = 1.25 in^2 and $A_{s,\,provided}$ = 1.461 in^2,

$$l_d = \left(\frac{40000 \times 1.3}{25\sqrt{2500}}\right) \times \frac{20}{25.4} \times \frac{1.25}{1.461}$$

$$= 28 \text{ inch}$$

(4) Simplified Tension Development Length , l_d , contd.

Unmodified l_d (in.) for other bars

Bar size (mm)	$f'_c = 2.5$ (ksi)			$f'_c = 3.0$ (ksi)			$f'_c = 3.5$ (ksi)		
	f_y (ksi)			f_y (ksi)			f_y (ksi)		
	40	50	60	40	50	60	40	50	60
8	10.1	12.6	15.1	9.2	11.5	13.8	8.5	10.6	12.8
10	12.6	15.7	18.9	11.5	14.4	17.2	10.6	13.3	16.0
12	15.1	18.9	22.7	13.8	17.2	20.7	12.8	16.0	19.2
16	20.2	25.2	30.2	18.4	23.0	27.6	17.0	21.3	25.6
18	22.7	28.3	34.0	20.7	25.9	31.0	19.1	24.0	28.8
20	25.2	31.5	37.8	23.0	28.7	34.5	21.3	26.6	32.0
22	34.6	43.3	52.0	31.6	39.5	47.5	29.3	36.6	44.0
24	37.8	47.2	56.7	34.5	43.1	51.8	31.9	40.0	48.0
25	39.4	49.2	59.1	35.9	44.9	53.9	33.3	41.6	50.0

For top bars multiply by 1.3

(5) Compression Development Length , l_{dc}

$$\frac{l_{dc}}{d_b} = \frac{0.02\ f_y}{\sqrt{f_c'}} \geq 0.0003\ f_y$$

Modification factors (to multiply)

(i) $A_{s,\ required}\ /\ A_{s,\ provided}$

(ii) If confined, i.e., if No. 4 ties at not more than 4 in. spacing c/c is provided, may be modified with 0.75

(iii) $l_{dc}\ \geq$ 8 inch (in all cases)

Unmodified l_{dc}

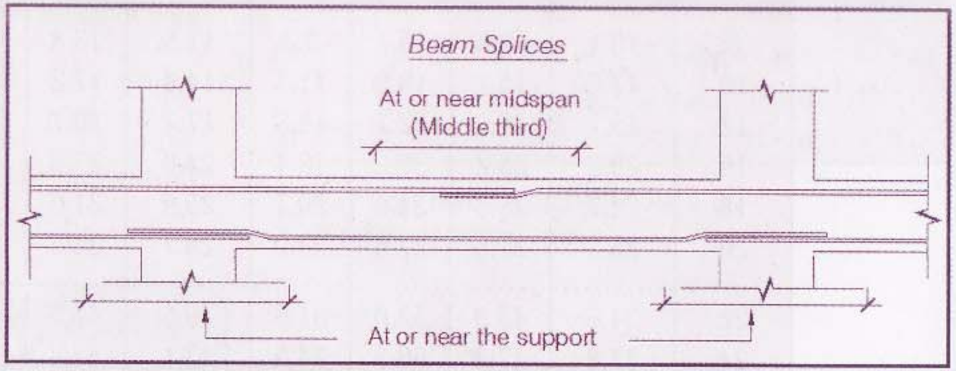
f_c'	f_y (ksi)		
(ksi)	40	50	60
2.5	16.0 d_b	20.0 d_b	24.0 d_b
3.0	14.6 d_b	18.3 d_b	21.9 d_b
3.5	13.5 d_b	16.9 d_b	20.3 d_b

Modify if applicable

(6) Lap Splice in Tension, $l_{splice,\ ten}$

Class A splice : $l_{splice,\ ten} = 1.0\ l_d$

Class B splice : $l_{splice,\ ten} = 1.3\ l_d$



Note : (i) l_d is obtained as in Section (4) but without applying the factor $A_{s,\ required}\ /\ A_{s,\ provided}$ for this purpose

(ii) Lap splices are generally Class B, except when $A_{s,\ provided} \geq 2 \times A_{s,\ required}$ over the length of the splice, and $\leq$ 50 % of the total reinforcement is spliced within the lap length

(iii) $l_{splice,\ ten}\ \geq$ 12 in. (in all cases)

(iv) Location of splice as shown (for gravity loads only)

(7) Lap Splice in Compression , $l_{splice, com}$

$$\frac{l_{splice,com}}{d_b} = 0.0005 f_y$$

Note : (i) For $f_c^{'} < 3000$ psi, increase by one-third

(ii) Minimum $l_{splice,com}$ is 12 inch

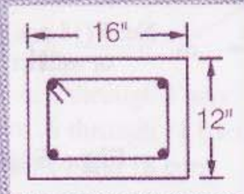
(iii) If lateral ties are used throughout the splice length having an area of ties of at least $0.0015\,h\,s$, where s is the tie spacing and h is the larger dimension of the member section, splice length may be multiplied by the factor

0.83 but ≥ 12 inch

$l_{splice,com}$ in d_b values

$f_c^{'}$ (ksi)	f_y (ksi)		
	40	50	60
2.5	26.6 d_b	33.3 d_b	39.9 d_b
≥3.0	20.0 d_b	25.0 d_b	30.0 d_b

For example



No. 3 ties are used for the column.

Tie spacing $s = 9$ in.

20 mm main bars with $f_y = 50000$ psi; $f_c^{'} = 2500$ psi.

To find the lap splice in compression,

check $0.0015\,h\,s = 0.0015 \times 16 \times 9 = 0.216$ in^2

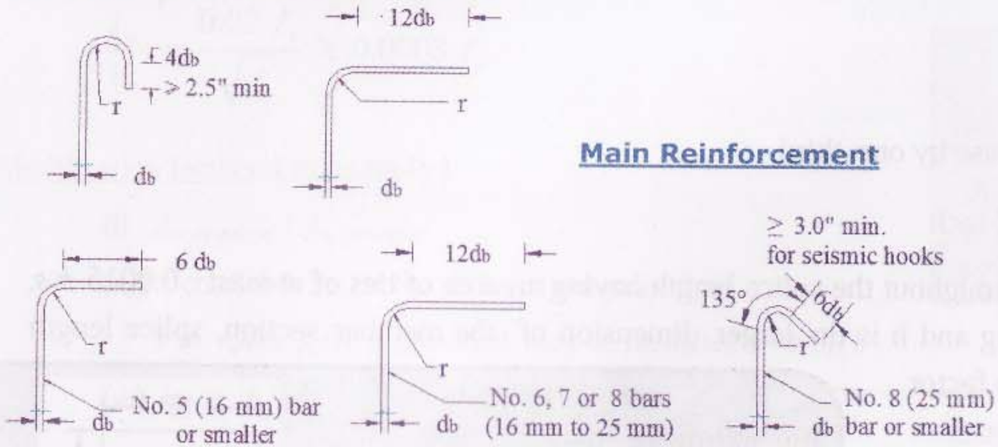
No. 3 ties, 2 legs,

Tie area $= 0.11 \times 2 = 0.22$ in^2 > 0.216 in^2

∴ Modification factor 0.83 can be used.

$$\therefore l_{splice,com} = 33.3 \times \frac{20}{25.4} \times 0.83 = 21.8 \text{ in., say 22 in.}$$

(8) Standard Bar Hooks



Stirrups and Ties

Fig. Standard bar hooks

Note: The table below gives readily-calculated values of the three common parameters for various bar sizes.

Minimum diameter of bend for standard hooks (main bars)

Bar size , d_b	Minimum diameter (inside)
No. 3 (10mm) through No. 8 (25mm)	$6d_b$

For stirrup and tie hooks, for bar sizes No. 5 (16mm) and smaller, the inside diameter of bend should not be less than $4d_b$ (ACI).

	Extension length (or) inside diameter (inch)		
Bar size d_b	$4d_b$	$6d_b$	$12d_b$
#6 (6 mm dia)	1	1 3/8	2 7/8
#6.5 (6.5 mm dia)	1	1 1/2	3
#8 (8 mm dia)	1 1/4	1 7/8	3 3/4
#10 (10 mm dia)	1 5/8	2 3/8	4 3/4
#12 (12 mm dia)	1 7/8	2 7/8	5 5/8
#16 (16 mm dia)	2 1/2	3 3/4	7 1/2
#18 (18 mm dia)	2 7/8	4 1/4	8 1/2
#20 (20 mm dia)	3 1/8	4 3/4	9 1/2
#22 (22 mm dia)	3 1/2	5 1/4	10 3/8
#24 (24 mm dia)	3 3/4	5 5/8	11 3/8
#25 (25 mm dia)	4	6	12

(9) Development Length of Standard Hooks , l_{dh} , contd.

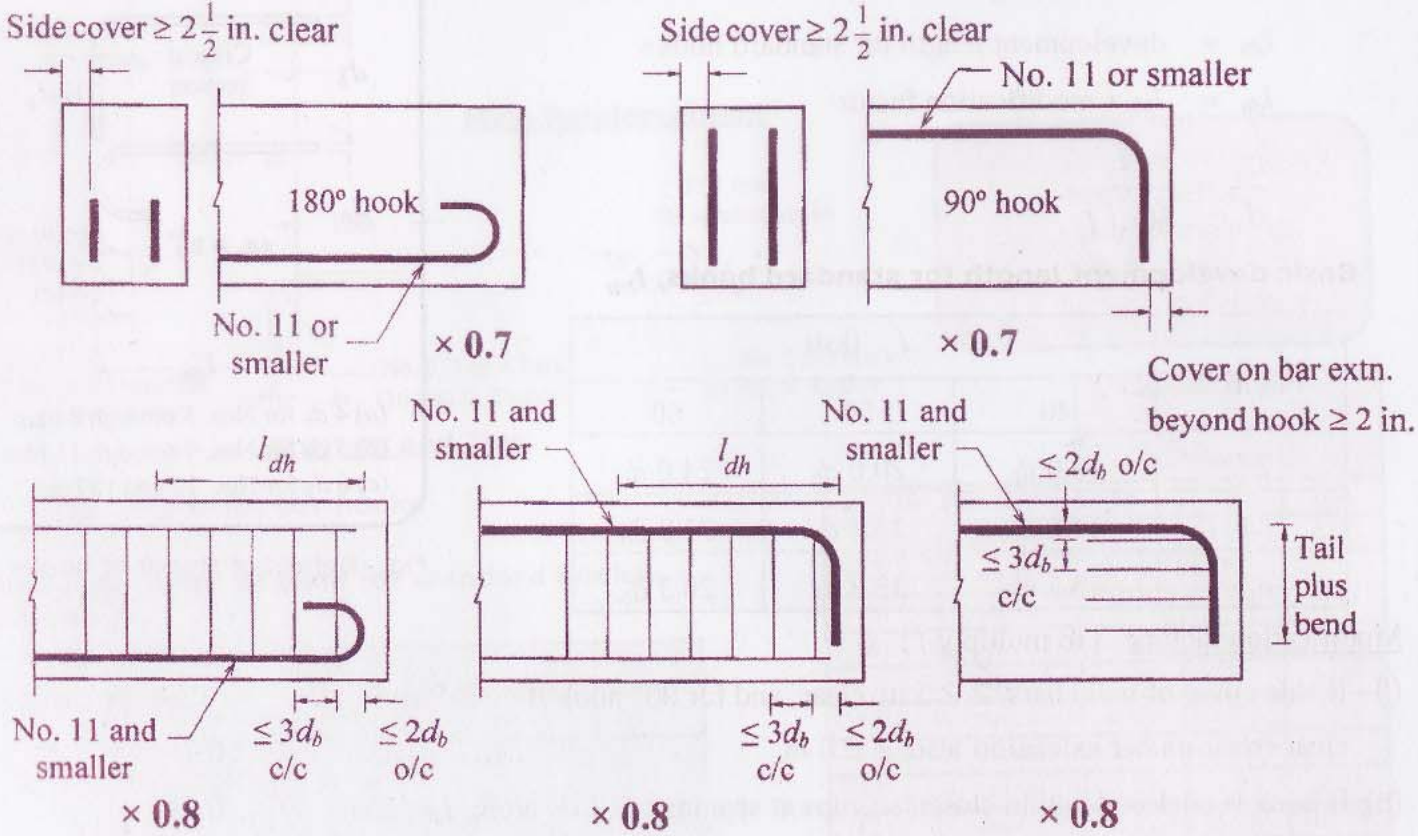
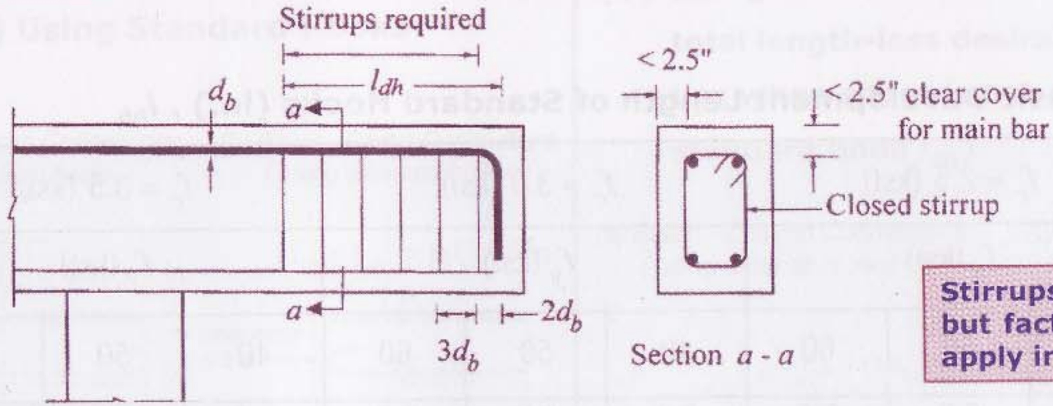
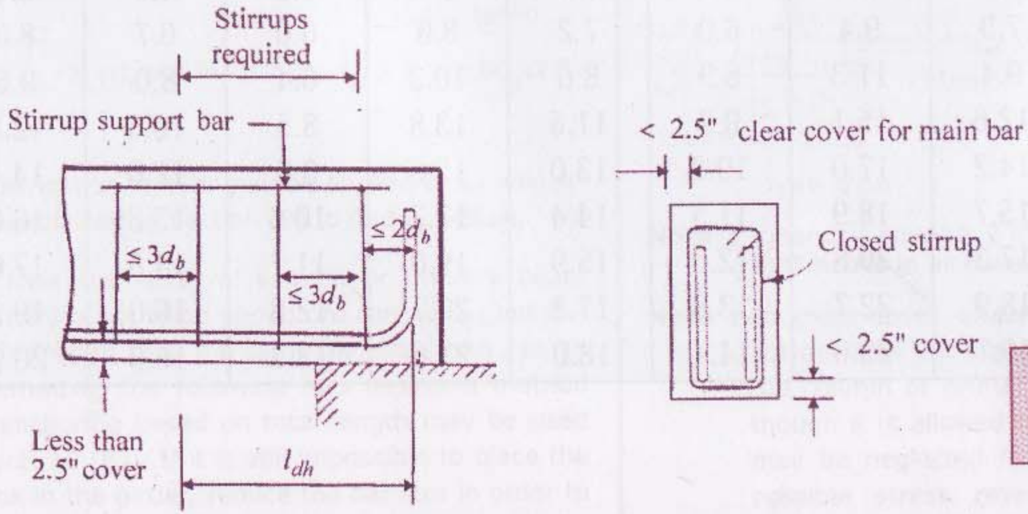


Fig. Modification factors

(9) Development Length of Standard Hooks , l_{dh} , contd.



Stirrups are required but factor 0.8 does not apply in this case



Stirrups are required but factor 0.8 does not apply in this case

Fig. Transverse reinforcement requirements at discontinuous ends with small cover distances

(9) Development Length of Standard Hooks , l_{dh} , contd.

Basic Development Length of Standard Hooks (in.) , l_{hb}

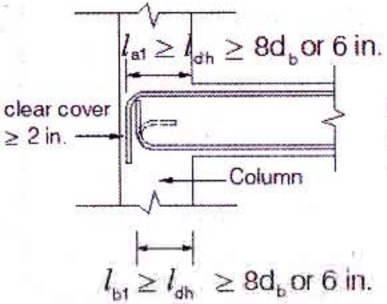
Bar size (mm)	$f_c' = 2.5$ (ksi)			$f_c' = 3.0$ (ksi)			$f_c' = 3.5$ (ksi)		
	f_y (ksi)			f_y (ksi)			f_y (ksi)		
	40	50	60	40	50	60	40	50	60
8	6.0	6.3	7.6	6.0	6.0	6.9	6.0	6.0	6.4
10	6.3	7.9	9.4	6.0	7.2	8.6	6.0	6.7	8.0
12	7.6	9.4	11.3	6.9	8.6	10.3	6.4	8.0	9.6
16	10.1	12.6	15.1	9.2	11.5	13.8	8.5	10.6	12.8
18	11.3	14.2	17.0	10.3	13.0	15.5	9.6	12.0	14.4
20	12.6	15.7	18.9	11.5	14.4	17.2	10.6	13.3	16.0
22	13.9	17.3	20.8	12.6	15.9	19.0	11.7	14.6	17.6
24	15.1	18.9	22.7	13.8	17.3	20.7	12.8	16.0	19.2
25	15.7	19.7	23.6	14.4	18.0	21.6	13.3	16.6	20.0

Modify if applicable

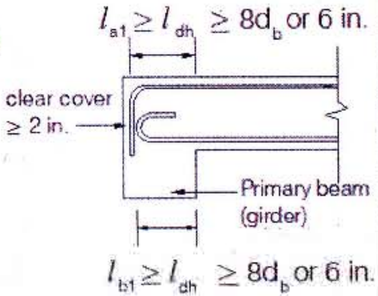
(10) End Anchorage Lengths

(A) Using Standard Hooks

(a) Beam - Column Connections
(using standard hooks)



(b) Beam - Girder Connections
(using standard hooks)

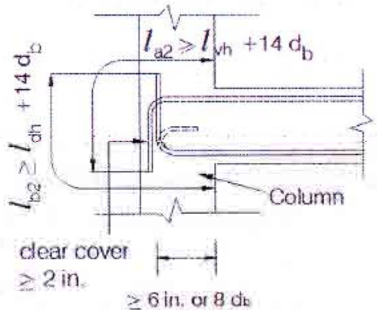


Note : Modification factors may be applied to l_{hb} values as explained in Section (9) to find l_{dh} values.

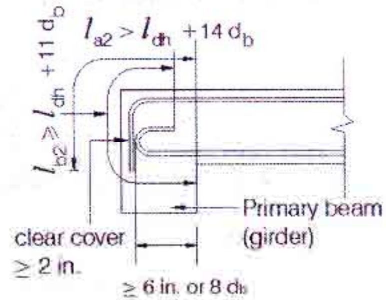
Note : In case the width of column or primary beam (girder) is not large enough to contain l_{dh} but if it is desired to keep the same bar size, then , as an alternative, the following less desirable method of anchoring based on total length may be used [Sect. 10 (B)]. If it is still impossible to place the hook in the girder, reduce the bar size in order to reduce the required anchorage length.

(B) Using Alternative Method (based on total length-less desirable and only for use when space is not adequate for standard hook l_{dh})

(a) Beam - Column Connections
(using alternative method)



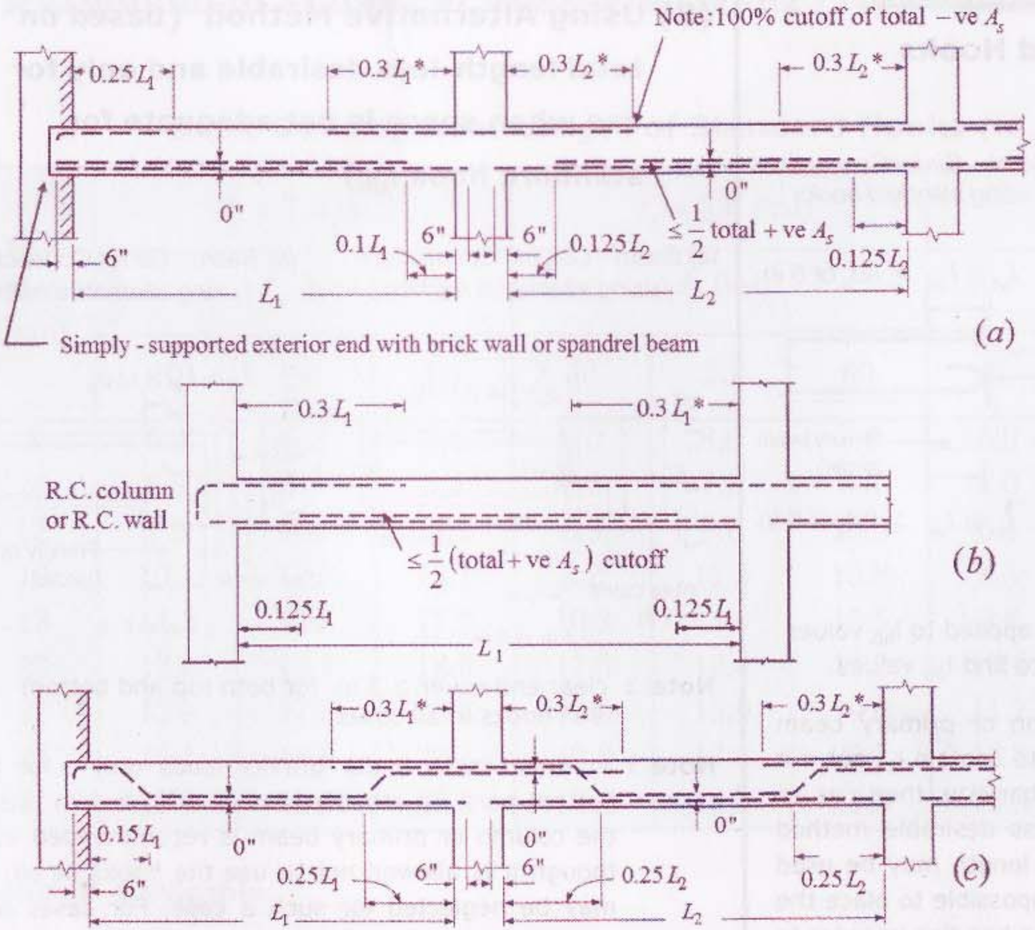
(b) Beam - Girder Connections
(using alternative method)



Note : clear end cover ≥ 2 in. for both top and bottom 90° hooks in all cases.

Note : in most cases under gravity loads only , for the bottom bars , a standard hook with ≥ 6 in. inside the column or primary beam is recommended even though it is allowed not to use the hooks at all; l_{b2} may be neglected for such a case. For cases with possible stress reversal, l_{b2} must be provided, however.

(11) Simplified Cutoff or Bend Points

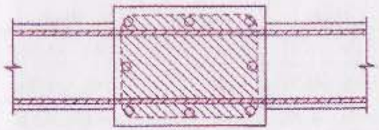


(Note that, from design experience, if not more than $\frac{1}{2}$(total −ve A_s) is cut off and the rest continued throughout, the cutoff points for negative bars may be considerably nearer to supports than those shown in the figure for nearly equal spans under u.d. loads)

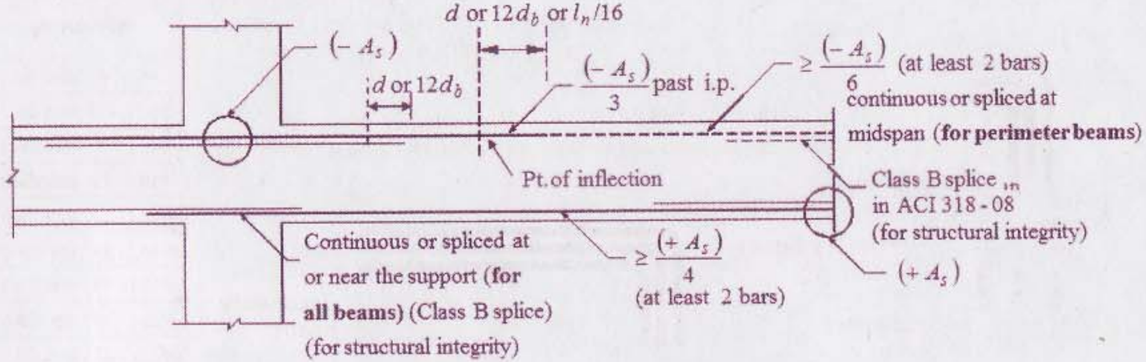
Fig. Cutoff or bend points for bars in beams and one-way slabs having approximately equal spans with uniformly distributed loads

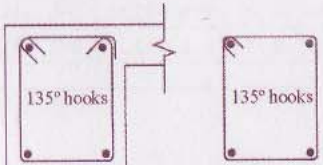
* *If adjacent spans are different in span lengths, use the larger of the two.*

(12) Structural Integrity Requirements



(*a*) Continuous beam bars passi through column core
(***for perimeter beams***)

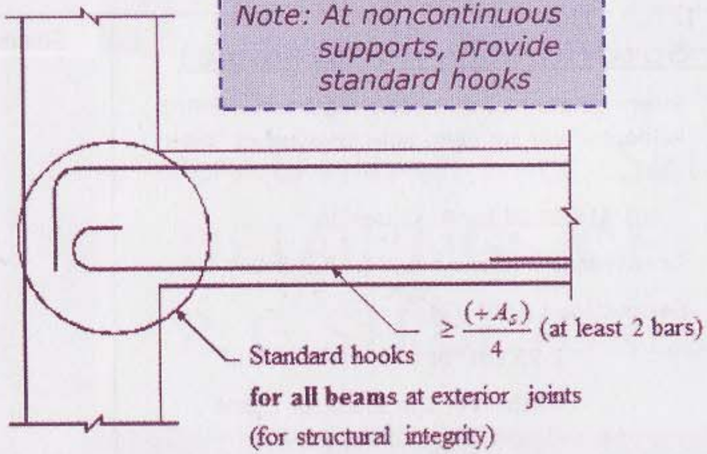




(*b*) Closed transverse reinforcement throughout the clear span at spacing ≤ d/2 c/c
(***for perimeter beams***)

To prevent progressive failure of building caused by failure of one member due to accidental or abnormal loading or deficient strength — basic concepts are to provide ties around the building and to create catenary action of beams between adjacent columns in case of failure of a column inbetween.

Note: At noncontinuous supports, provide standard hooks



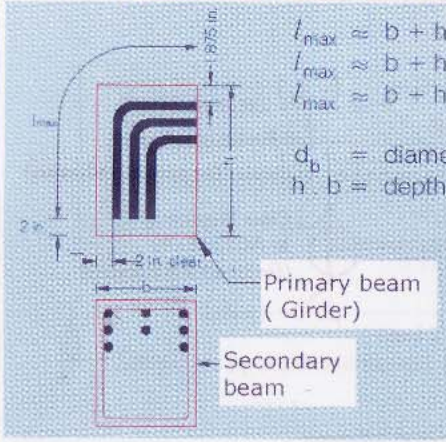
(*c*) Reinforcement in perimeter and other beams

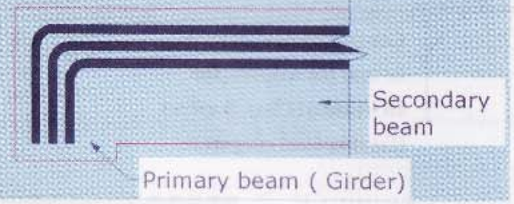
Fig. Reinforcement for structural integrity

(13) Anchorage Length Available in a Primary Beam (Girder)

$l_{max} \approx b + h - 5.875 - 2.5\, d_b$ (in.) (for the outermost layer)
$l_{max} \approx b + h - 5.875 - 6.5\, d_b$ (in.) (for the second layer)
$l_{max} \approx b + h - 5.875 - 10.5\, d_b$ (in.) (for the third layer)

d_b = diameter of main bar , in.
h , b = depth and width of primary beam , respectively , in.





Bar size d_b	l_{max} (inch) First layer	Second layer	Third layer
#6 (6 mm dia)	b+h -6.47	b+h -7.41	b+h -8.36
#6.5(6.5 mm dia)	b+h -6.51	b+h -7.54	b+h -8.56
#8 (8 mm dia)	b+h -6.66	b+h -7.92	b+h -9.18
#10 (10 mm dia)	b+h -6.86	b+h -8.43	b+h -10.01
#12 (12 mm dia)	b+h -7.06	b+h -8.95	b+h -10.84
#16 (16 mm dia)	b+h -7.45	b+h -9.97	b+h -12.49
#18 (18 mm dia)	b+h -7.65	b+h -10.48	b+h -13.32
#20 (20 mm dia)	b+h -7.84	b+h -10.99	b+h -14.14
#22 (22 mm dia)	b+h -8.04	b+h -11.50	b+h -14.97
#25 (25 mm dia)	b+h -8.34	b+h -12.27	b+h -16.21

(14) Other Specifications (for example)

Building Type : Intermediate moment-resisting frame (IMRF) without shear wall and not classified as "high-rise"

Code of Practice : ACI 318-2008 for R. C. design

Loading : Gravity load and wind load (ASCE 7-05)
Seismic load (UBC 97)

$q_{a,gross}$: 2.25 t/ft^2 at 8′ 6″ from N.G.L.

Software : ETABS Version 9.5.0 for frame analysis and preliminary design
SAFE PLUS Version 8.0.4 for foundation analysis and design

Detailing : IMRF detailing

Seismic Loading Criteria :

C_t = 0.03
Soil Profile Type = S_D
Zone Factor z = 0.15 (Yangon, Zone 2A – UBC)
Seismic Source Type = C
Distance to Source > 10 km
C_v = 0.32
C_a = 0.22
R = 5.5
I = 1

Wind Data : Basic wind Speed = 100 mph
Exposure Category B , Flat Ground
Assume " enclosed "
Occupancy Category II

ORDINARY & SEISMIC DETAILINGS OF R. C. STRUCTURES

B. ORDINARY & SEISMIC DETAILINGS OF R. C. STRUCTURES

(1) General

Seismic zones

According to UBC (Uniform Building Code),

Seismic Zones are 0, 1 (Low) ; 2A, 2B (Moderate) ; 3, 4 (High Seismic Risk).

Yangon is considered as in equivalent Zone 2A UBC (Myanmar Zone II/III)

Mandalay is considered as in equivalent Zone 4 UBC (Myanmar Zone V)

[***Note*** *: Myanmar Zones are : I, II, III, IV and V – see the Seismic Zone map*]

Moment-Resisting Frames and Detailing Requirements

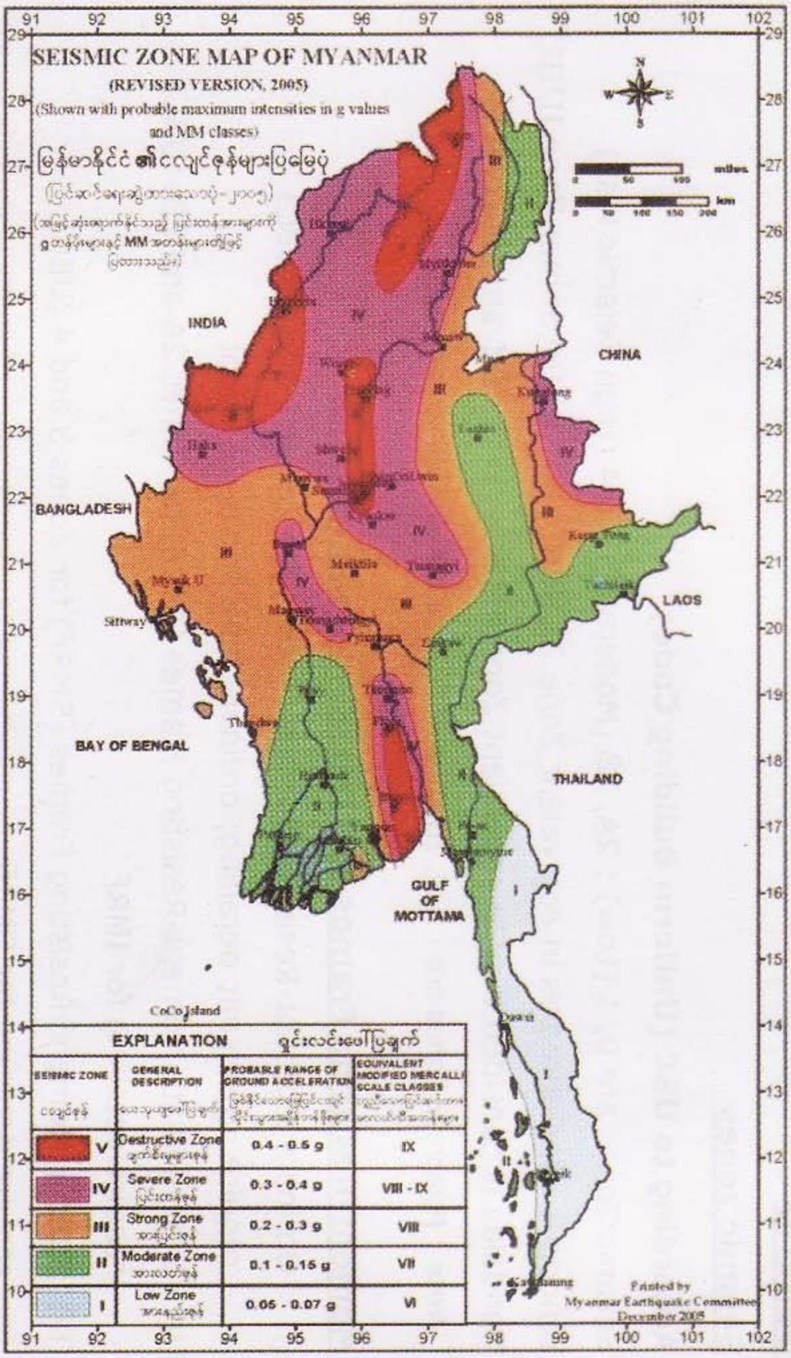
(i) Ordinary Moment-Resisting Frames (OMRF) for Zones 0 and 1 (UBC) — require no seismic detailing; ordinary detailing is sufficient

(ii) Intermediate Moment-Resisting Frames (IMRF) for Zones 2A and 2B (UBC) — require detailing for IMRF

(iii) Special Moment-Resisting Frames (SMRF) for Zones 3 and 4 (UBC) — require detailing for SMRF

Note : ***Dual system and other framing systems are not included in this discussion. Dual system means (shear wall / braced frame) + (OMRF / IMRF / SMRF) acting together.***

Seismic Zones and Earthquakes in Myanmar



Revised by Dr. Maung Thein, U Tint Lwin Swe and Dr. Sone Han (December 2005)

Fig. Seismic zone map of Myanmar

Seismic Zones and Earthquakes in Myanmar *contd.*

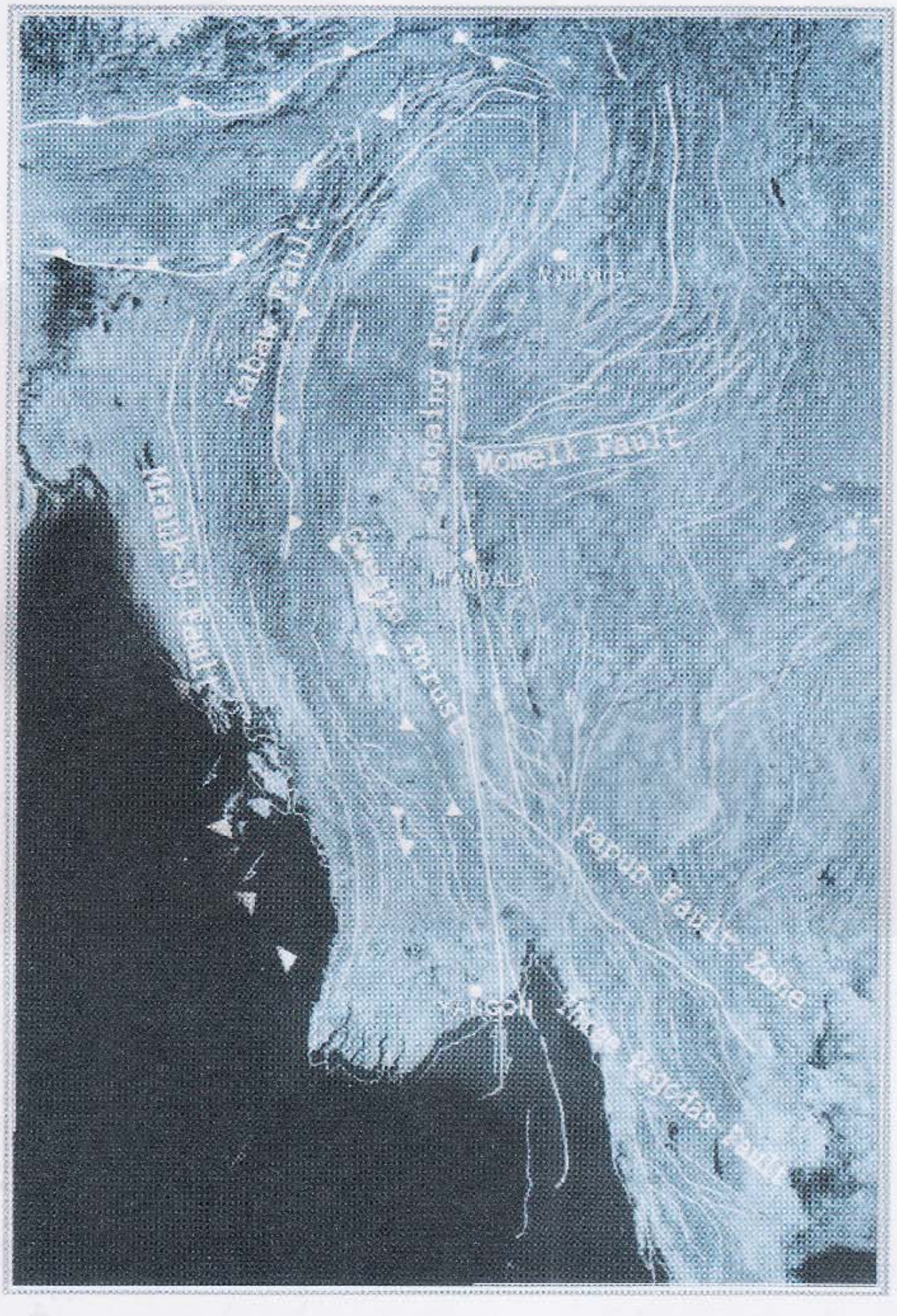
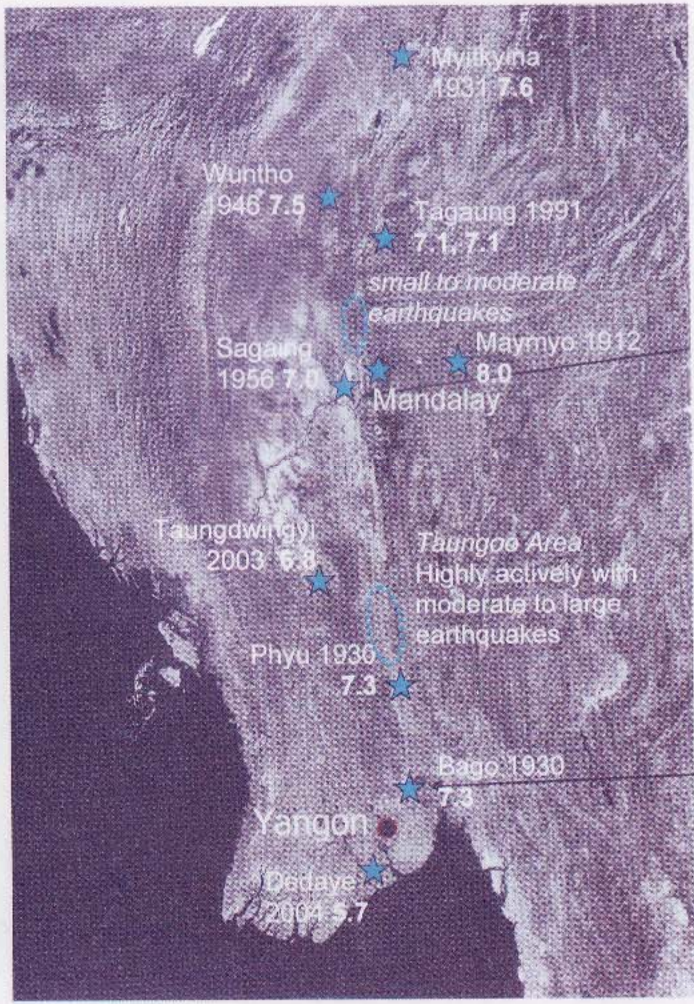


Fig. Active faults in Myanmar

Seismic Zones and Earthquakes in Myanmar *contd.*



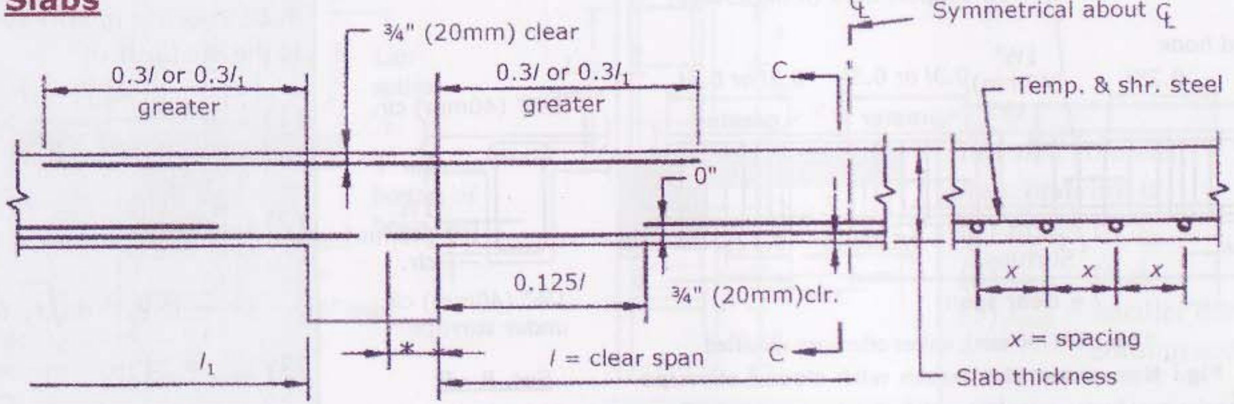
Historic earthquakes in AVA Era
1429, 1467, 1501, 1602, 1696, 1762, 1771, 1776, 1830, 1839

Historic Earthquakes in Bago
868, 875, 1564, 1567, 1582, 1588, 1590, 1757, 1768, 1830, 1888, 1913, 1917, 1920, 1930

Fig. Seismicity along the Sagaing fault

(2) Ordinary Moment-Resisting Frame Detailing

Slabs



* Min. 6" (150 mm), unless otherwise specified

Fig. Interior span, continuous one-way slab

For slabs,

Maximum spacing

= 3h (or) 18" (one-way slab – for main steel)

= 2h (or) 18" (two-way slab – for main steel)

= 5h (or) 18" (for temp. & shr. steel)

Note : $A_{s,min}$ for slab = 0.002 bh for fy = 40/50 ksi (= temp. & shr. steel)

= 0.0018 bh for f_y = 60 ksi (= temp. & shr. steel)

(2) Ordinary Moment-Resisting Frame Detailing *contd.*

Beams

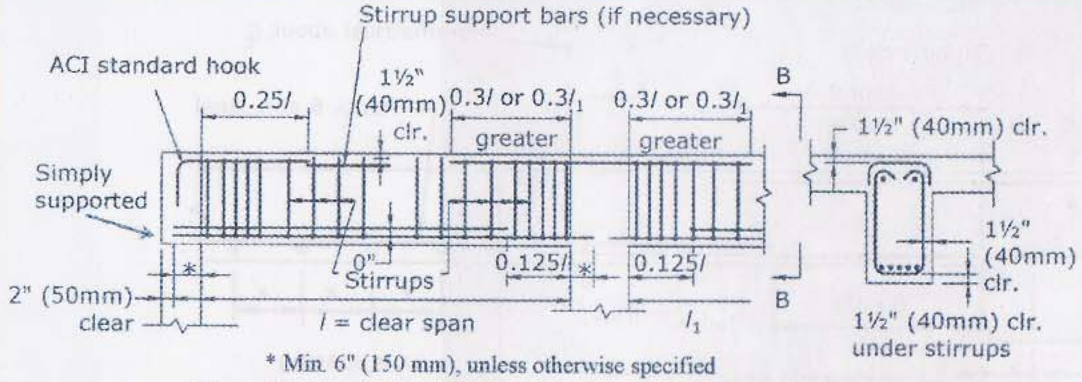


Fig. Non-perimeter beam with closed stirrups

Note : if the end of exterior-span beam is monolithic with R. C. column or shear wall, assume the same as interior-span beam.

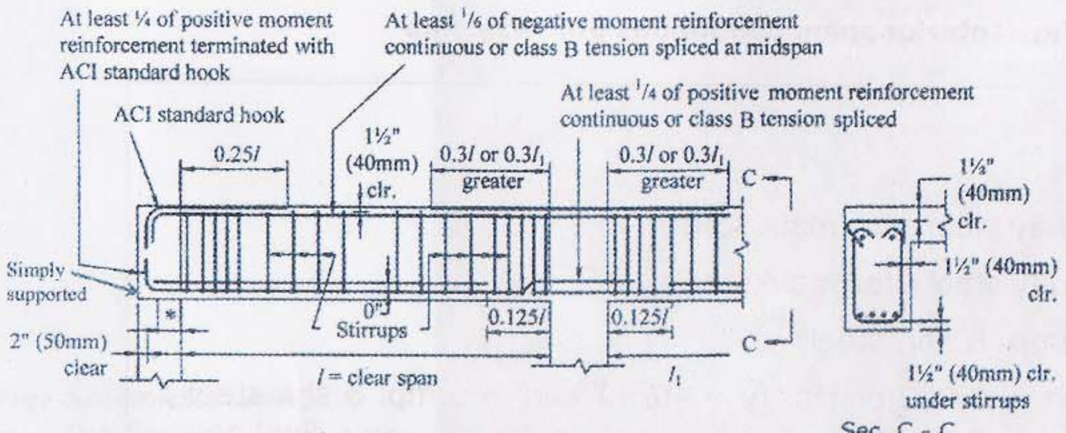


Fig. Perimeter beam

For *beams*,
max. spacing of stirrups
is the *smallest* of

(1) $s_{max} = \dfrac{A_v f_y}{0.75\sqrt{f_c'}\, b_w} \le \dfrac{A_v f_y}{50\, b_w}$

(2) $s_{max} = \dfrac{d}{2}$

$\left(= \dfrac{d}{4} \text{ if } V_s > 4\sqrt{f_c'}\, b_w d\right)$

(3) $s_{max} = 24$ in.

$(= 12 \text{ in. if } V_s > 4\sqrt{f_c'}\, b_w d)$

(4) $s_{max} = \dfrac{P_h}{8}$ or 12 in.

(closed stirrups for torsion case) where P_h is perimeter of centreline of stirrups.

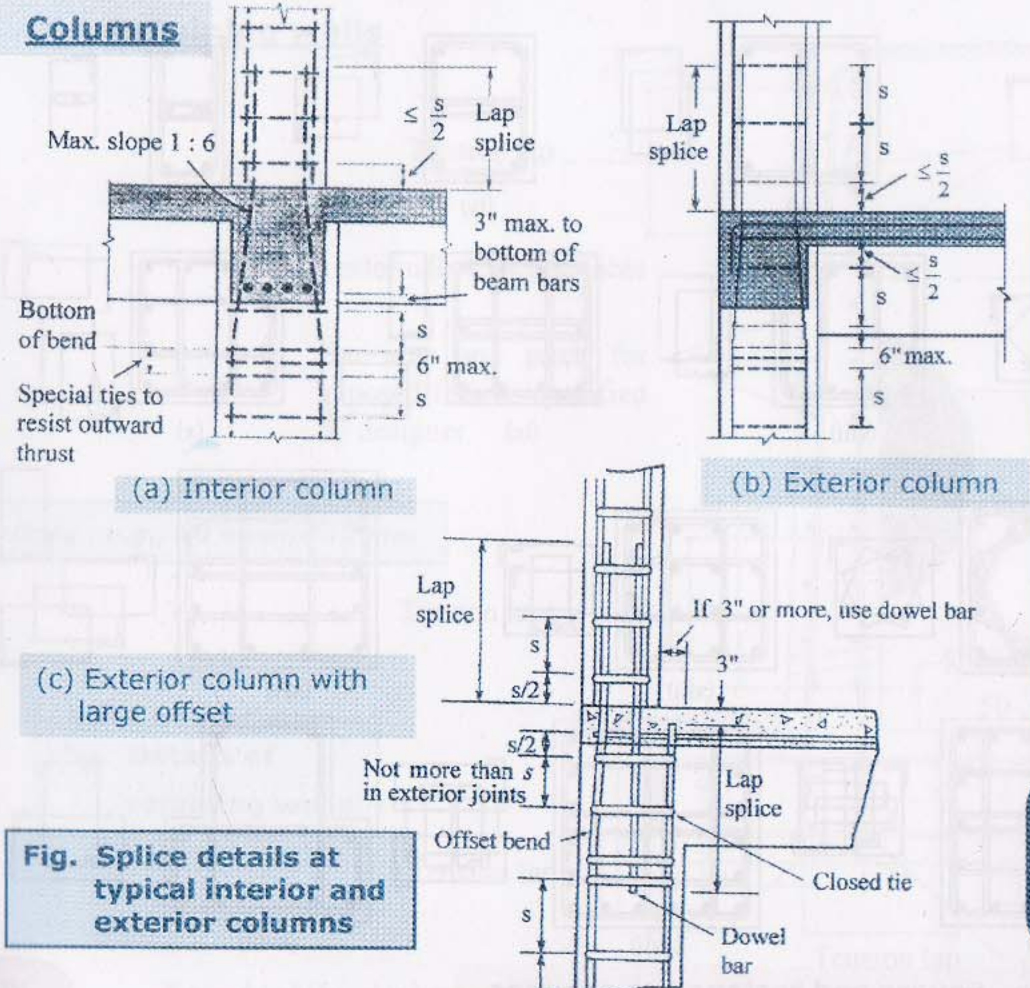
Note:

$$\rho_{min} = \frac{3\sqrt{f_c'}}{f_y} \ge \frac{200}{f_y}$$

for main steel

(2) Ordinary Moment-Resisting Frame Detailing *contd.*

Columns



(a) Interior column

(b) Exterior column

(c) Exterior column with large offset

Fig. Splice details at typical interior and exterior columns

For tied columns, max. spacing is

(1) $s_{max} = 16\ d_b$

(2) $s_{max} = 48\ d_t$

(3) s_{max} = smaller dimension of column section

Note: *ties at least No. 3 in size; no unsupported main bar shall be farther than 6 in. clear from a supported bar. Lateral support is to be provided by the corner of a tie having an included angle ≤ 135°*

Note: $\rho_{min} = 0.01$; $\rho_{max} = 0.08$

for longitudinal bars (non-seismic case)

(2) Ordinary Moment-Resisting Frame Detailing *contd.*

Columns *contd.*

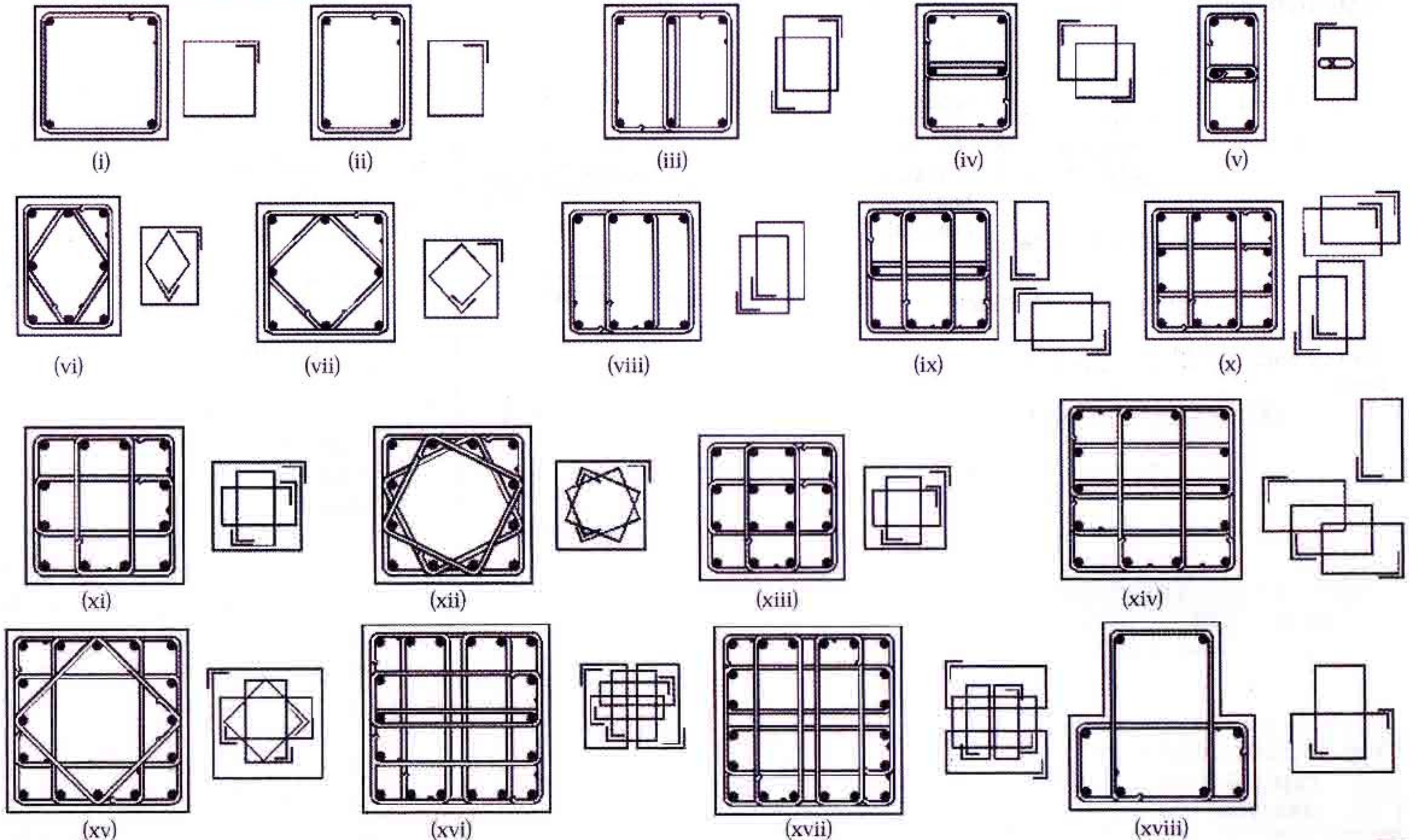
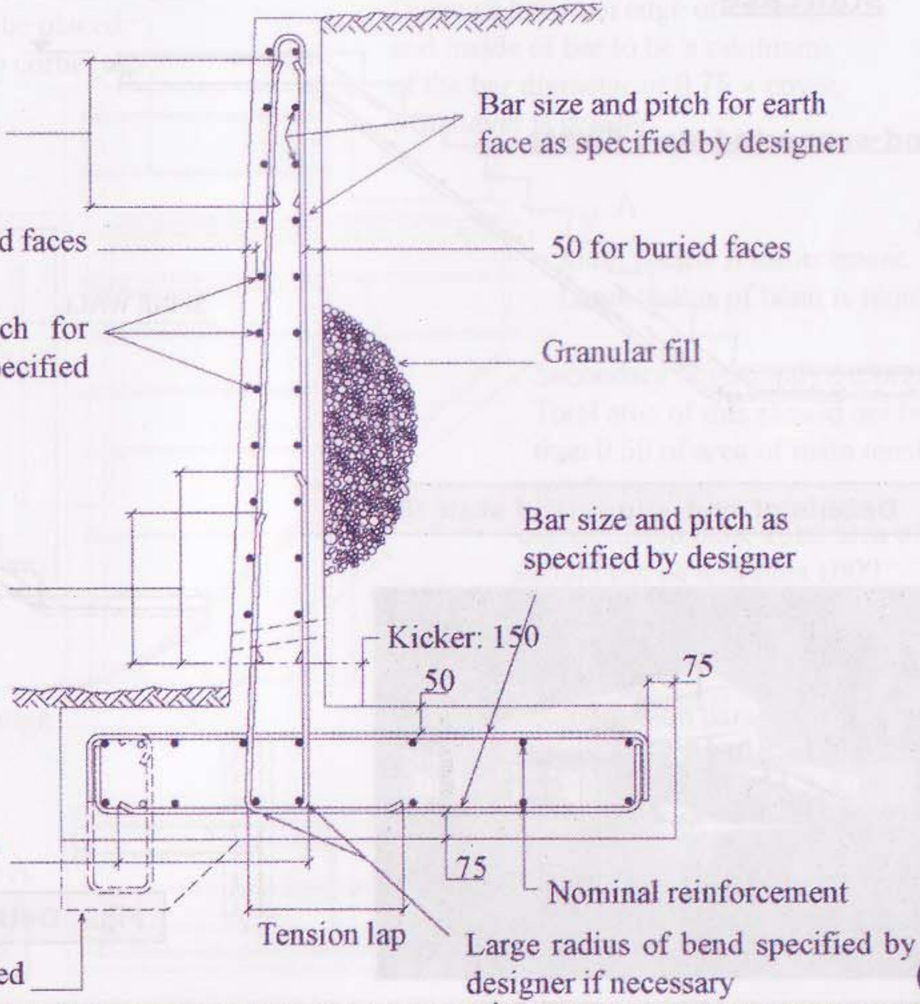


Fig. Square and rectangular columns

(3) Ordinary Detailing of Some Typical Structures

Retaining walls



Note : e.g., 40 means 40 mm

Fig. Details of retaining walls

(3) Ordinary Detailing of Some Typical Structures *contd.*

Staircase

End-supported stair flights

Fig. Details of end-supported stair flights

Cantilever stair flights

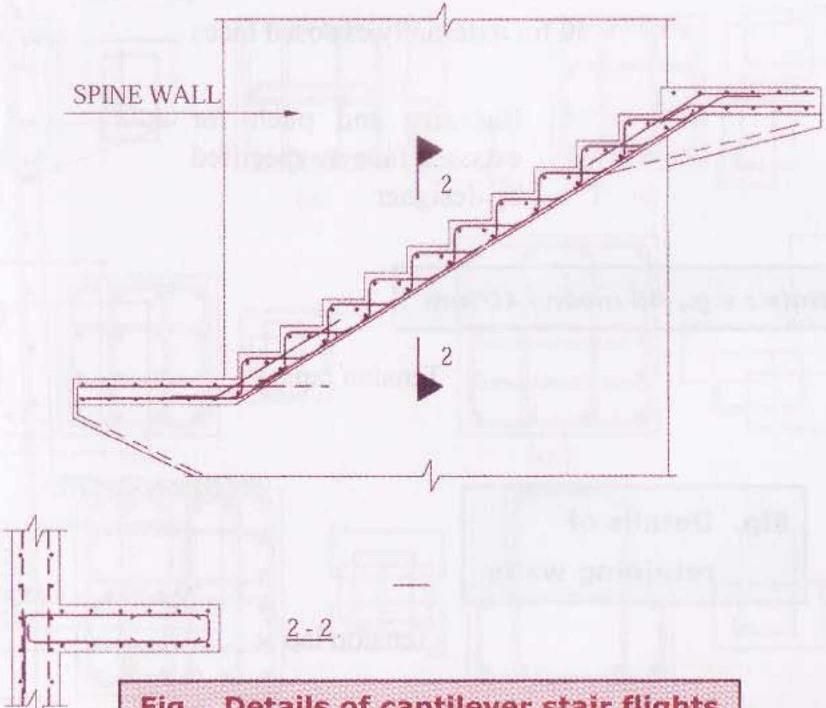


Fig. Details of cantilever stair flights

(3) Ordinary Detailing of Some Typical Structures *contd.*

Corbels

Corbels without weld

This detailing is suitable when using 16 mm bar size or smaller for the main tensile reinforcement.

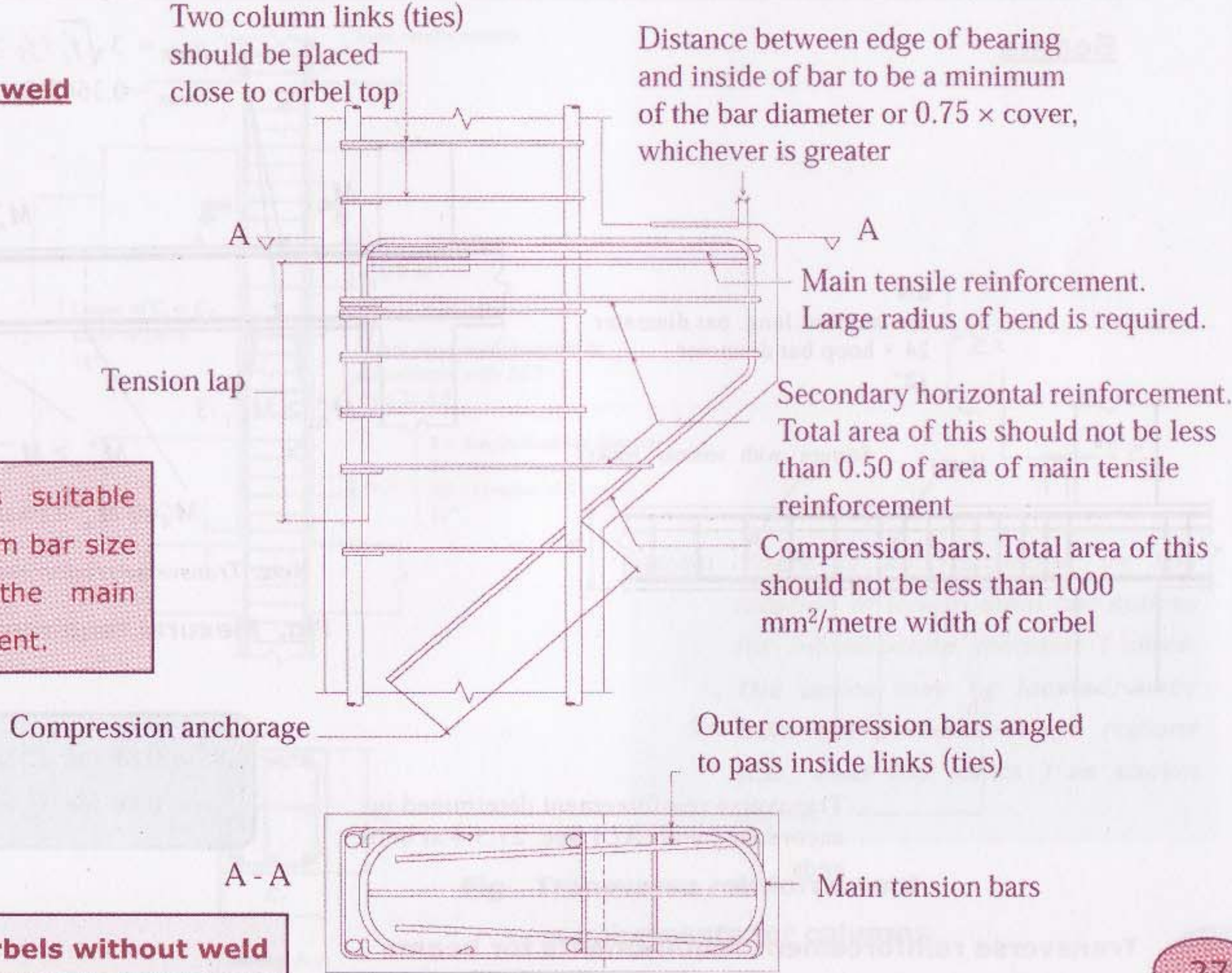


Fig. Details of corbels without weld

(4) Intermediate Moment-Resisting Frame Detailing

Beams

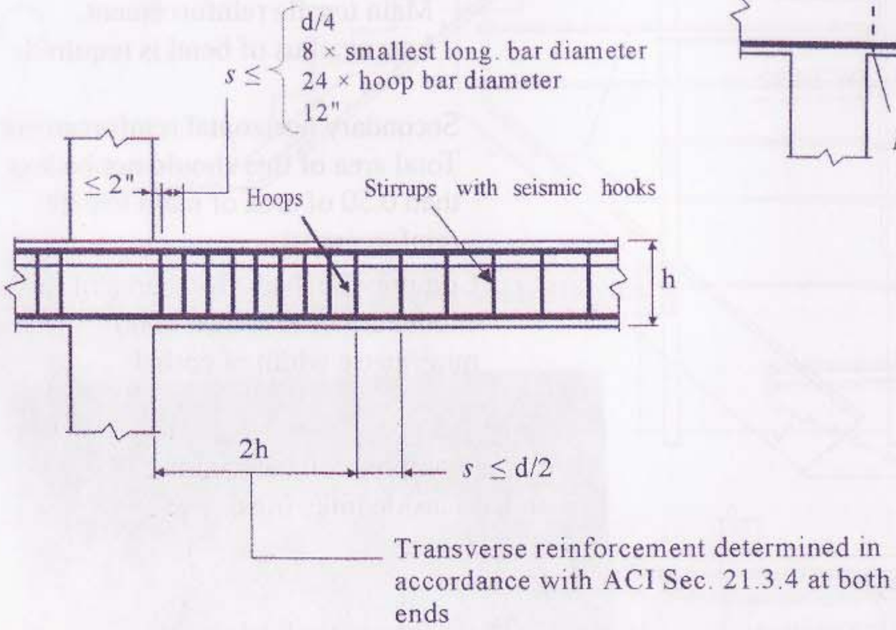
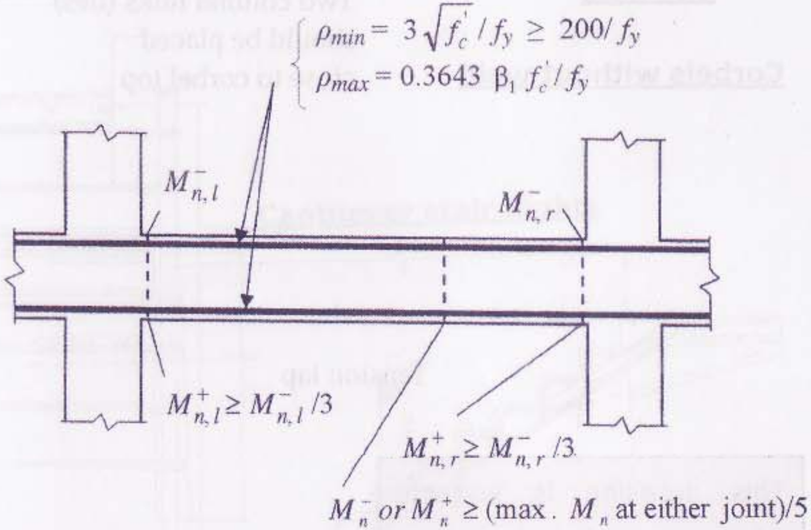


Fig. Transverse reinforcement requirements for beams



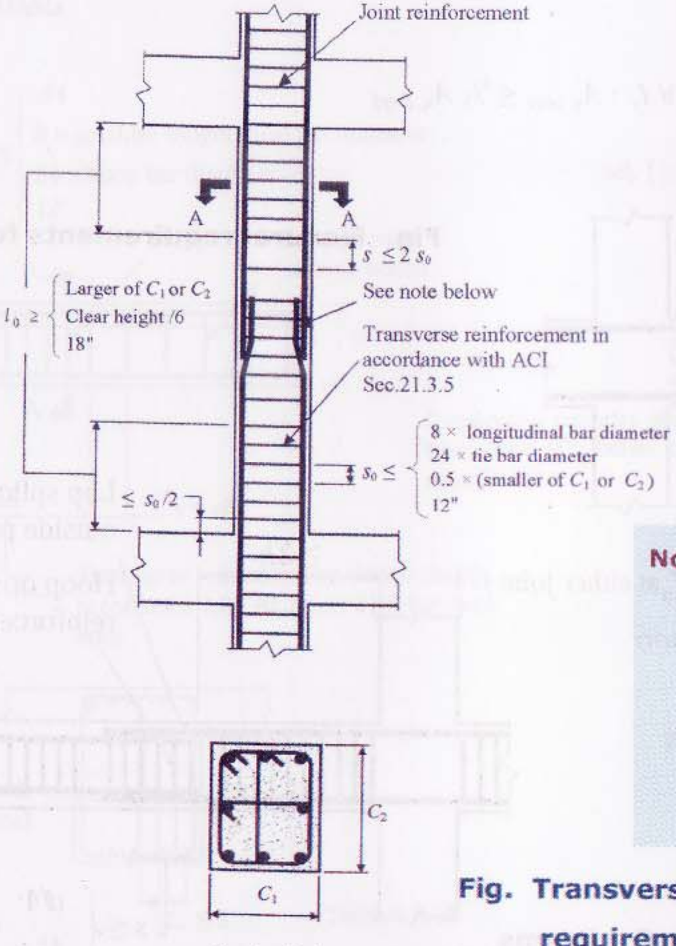
Note: *Transverse reinforcement not shown for clarity*

Fig. Flexural requirements for beams

$\beta_1 = 0.85$ for $f_c' \leq 4000$ psi
$= 0.80$ for $f_c' = 5000$ psi

(4) Intermediate Moment-Resisting Frame Detailing *contd.*

Columns



Note: ***There is no restriction on the location of longitudinal bar splices for intermediate moment frames. The splice may be located away from the potential hinge regions (i.e., near the joints) as shown above.***

Fig. Transverse reinforcement requirements for columns

(5) Special Moment-Resisting Frame Detailing

Beams

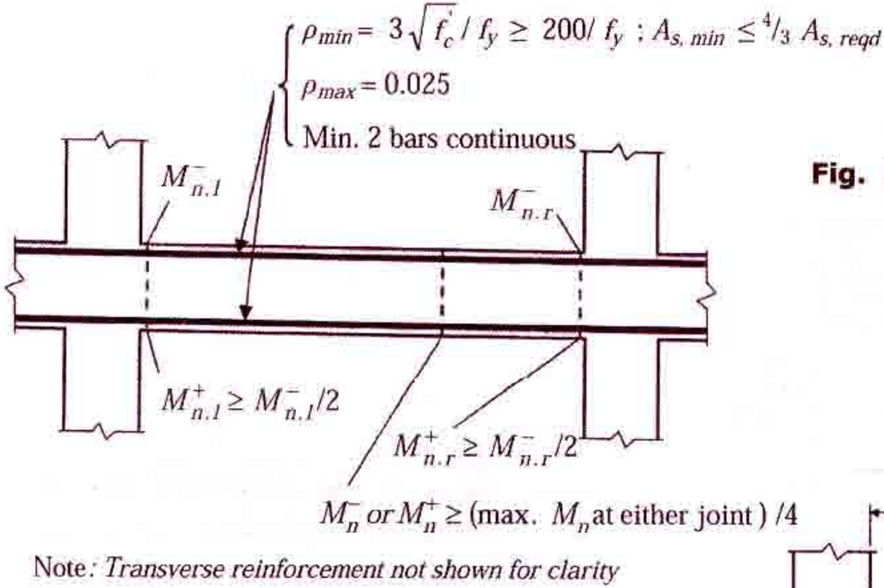


Fig. Flexural requirements for beams

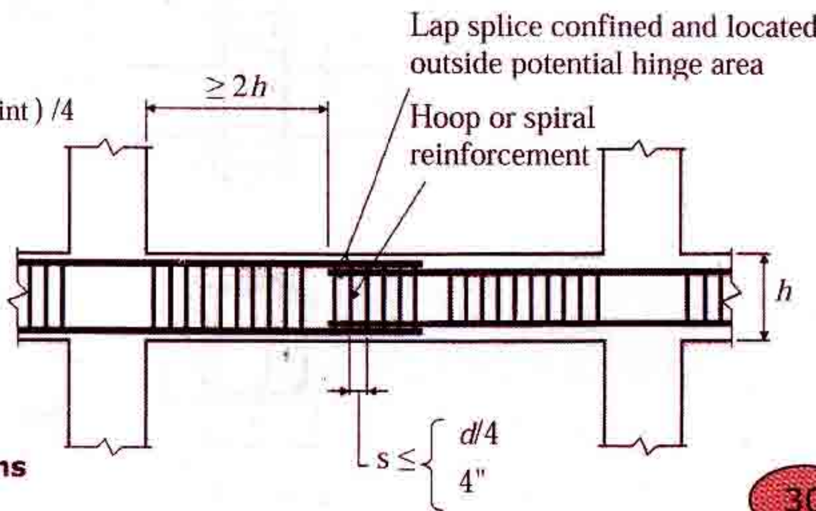


Fig. Lap splice requirements for beams

(5) Special Moment-Resisting Frame Detailing *contd.*

Beams *contd.*

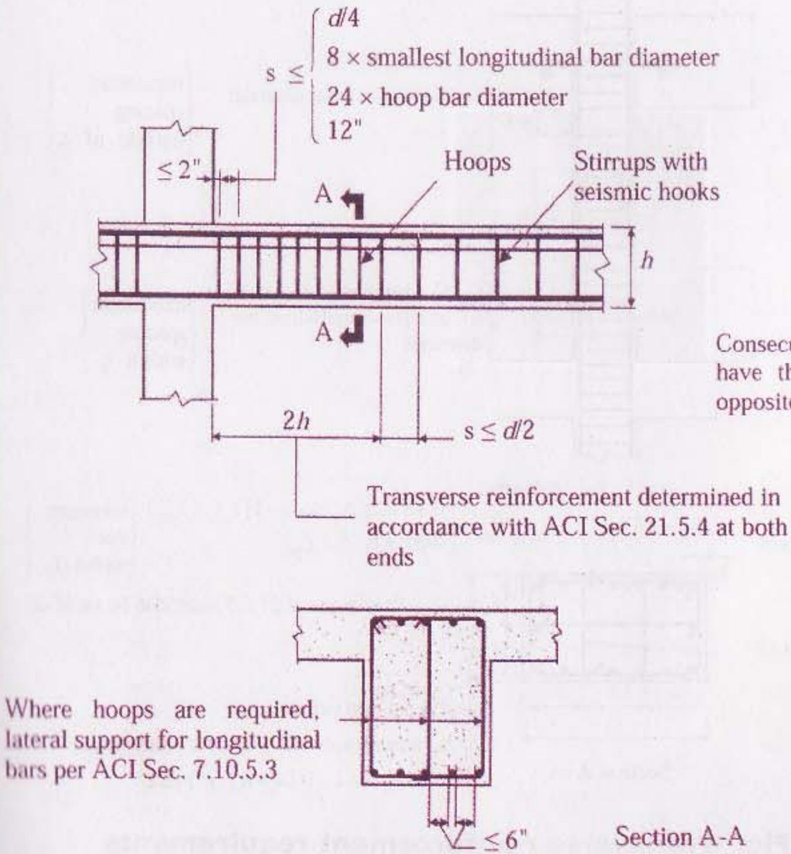


Fig. Transverse reinforcement requirements for beams

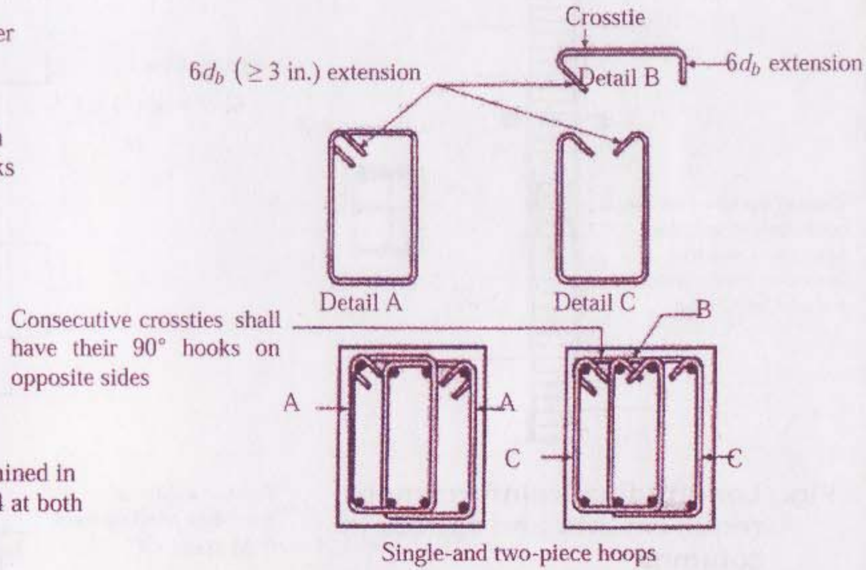


Fig. Hoop reinforcement for beams

(5) Special Moment-Resisting Frame Detailing *contd.*

Columns

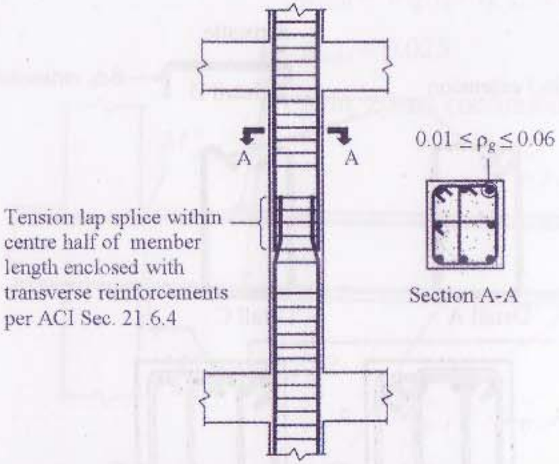


Fig. Longitudinal reinforcement requirements and splices in columns

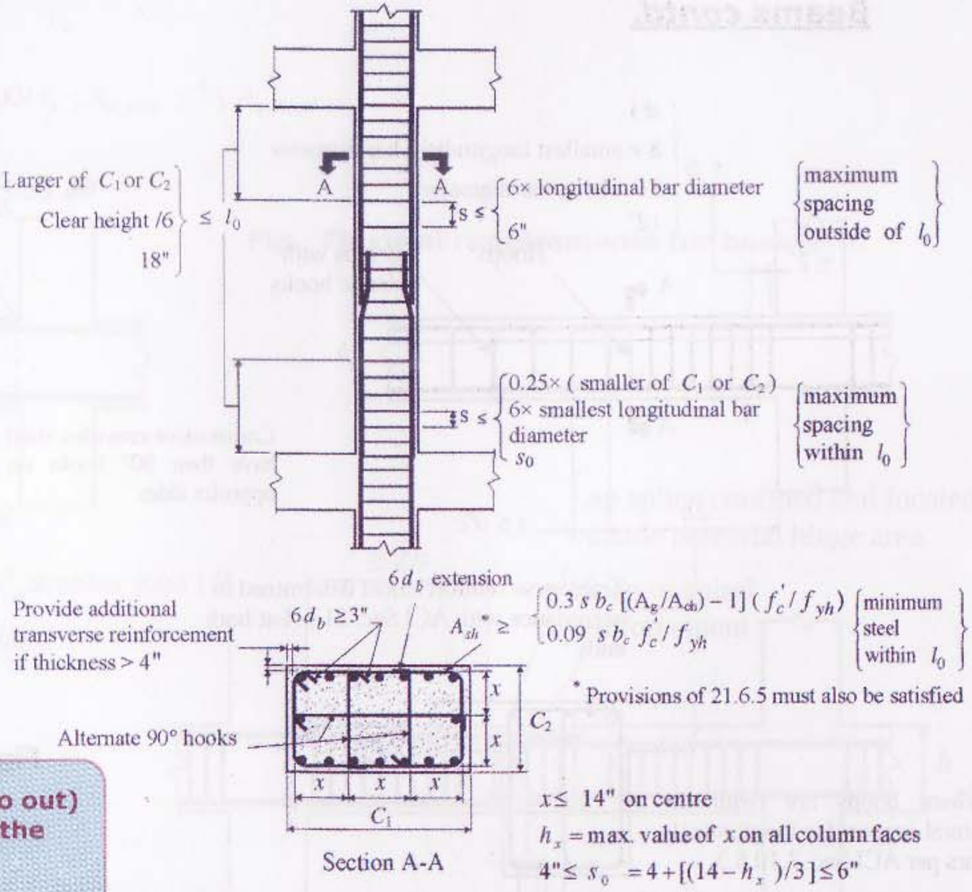


Fig. Transverse reinforcement requirements for columns (rectangular hoops)

b_c = dimension of column core (out to out) in the direction perpendicular to the legs that constitute A_{sh}

A_{ch} = cross sectional area of column core, measured out to out of stirrup steel

INSPECTION

C. INSPECTION

(1) Preliminary Inspection

- Check architectural and structural drawings and specifications
- Prepare/Check work schedule, origanization chart, record forms
- Check equipment, machinery, tools, power source, site office, construction materials, storage facilities, transport facilities, communication facilities, access roads, water, living quarters for staff, healthcare facilities, office facilities etc.
- Check/Recruit work force — engineers, technicians, foremen, skilled workers, unskilled workers, availability of temporary work force, availability of labour contractors, providers of ready-mix concrete, office staff
- Check building layout on ground — distances from control points, orientation, level with reference to a construction bench mark nearby
- Check irregularities or unusual features in terrain, soil conditions, subsurface structures and groundwater levels and report in time for possible changes in design or construction
- Talk and report to local authorities for future co-operation

(2) Materials Inspection

Cement

Types

Type I - normal portland cement for all uses of cement or concrete not subject to sulphate attack from soil or water or where the heat generated by hydration of cement will not cause an objectionable rise in temperature.

Type II - modified portland cement used in structures of considerable size such as large piers, heavy abutments, and heavy retaining walls to reduce temperature rise. It is also intended for places where added precaution against moderate sulphate attack is important as in drainage structures where sulphate concentrations in ground waters are high.

Type III - high-early-strength (or rapid-hardening) portland cement used when high strengths are desired at very early periods - from one to three days. It is used when it is desired to remove forms as soon as possible or to put the concrete into service quickly.

Type IV - low-heat portland cement used where the amount and rate of heat generated must be kept to a minimum. It is used in mass concrete such as large gravity dams where heat generated during hardening is a critical factor.

Type V - sulphate-resistant cement used only in construction exposed to severe sulphate action such as in soils or waters of high alkali content. It has slower rate of strength gain than normal portland cement.

(2) Materials Inspection *contd.*

Cement *contd.*

In U.S.A. each bag holds 1 cu. ft. of cement and weighs 94 lb. In Myanmar, a cement bag normally weighs 50 kg or 110 lb. The cement when used should be free-flowing and free of lumps. If cement contains lumps that cannot be easily broken up between the thumb and finger, it is advisable not to use it. Cement should be batched by weight and if batched by volume there can be considerable variation between batches. Unit weight of bulk cement should be taken into consideration in batching and measuring container should be adjusted accordingly.

Fine aggregate

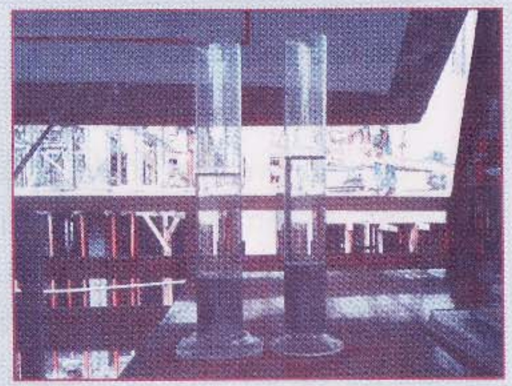
- It consists of particles of ASTM sieve no. 4 ($\cong$ 4.75 mm or 3/16 in. sieve opening) and less in size. Natural and manufactured sands are common. They have particles ranging from 3/16 in. down to sieve no. 200 ($\cong$ 0.075 mm or 0.0029 in. sieve opening).
- It must be clean and free from fine dust, loam, silt and clay because they prevent the cement paste from binding the aggregate particles, thereby reducing the strength of concrete.
- Sand should be washed with fresh water if silt content is high or if origin of sand is from under sea water. Coarse sand with larger fineness modulus is better in making concrete than the fine ones which are better in finishing the surface.

(2) Materials Inspection *contd.*

Fine aggregate *contd.*

Silt test

The silt test is used to detect the presence of extremely fine materials. Materials finer than No. 200 sieve is considered to be the approximate equivalent of the amount of silt. Fill the container to a depth of 2 inches with a representative sample of dry sand to be tested. Add water until the bottle is about three-fourths full. Shake vigorously for 1 minute. The last few shakes should be in a sidewise direction to level off the sand. Allow the jar to stand for an hour. During this time any silt present will be deposited in a layer above the sand. If the layer is more than 1/8 in. thick, the sand from which the sample is taken is not satisfactory for concrete work unless the excess silt is removed. This may be done by washing. Silt test should be made as a routine matter.



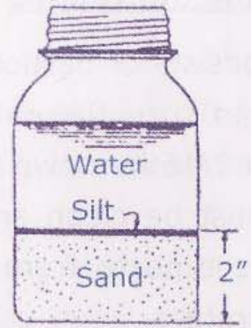
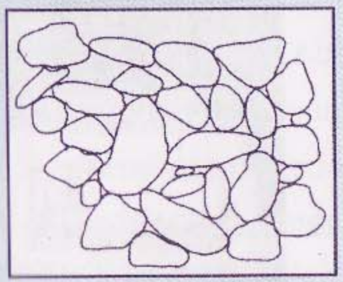


Fig. Silt test

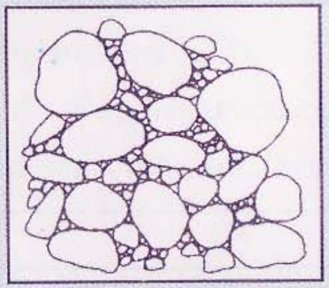
(2) Materials Inspection *contd.*

Coarse aggregate

- Usually gravel or crushed stone. Sizes range from 3/16 in. up to the maximum size permissible for the job.
- The natural mixture of fine and coarse aggregate usually does not make the most economical concrete unless it is first screened to separate the fine material from the coarse and then recombined in correct proportions. More cement paste is required to produce concrete of a given quality when there is high proportion of fine aggregate.
- Remove organic matter, mud and silt by washing; remove oversized aggregates by screening or at least by hand-picking. Crushed stones are generally cleaner than river shingles and give higher bond and tensile strengths of concrete. Compressive strength may be about the same or a little higher. Since clean aggregates are essential to quality concrete, washing is well worth the effort.



Poorly graded



Well graded

Fig. Poorly- and well-graded aggregates (FRENCH)

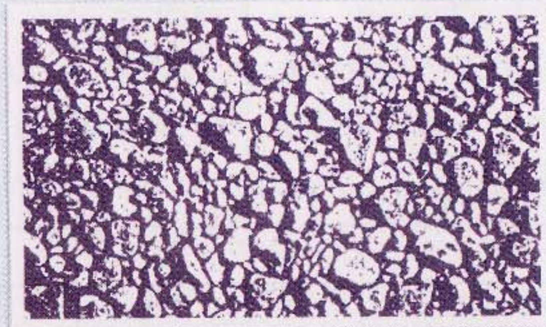


Fig. Well-graded coarse aggregate

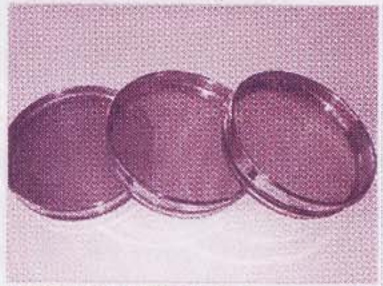
(2) Materials Inspection *contd.*

Sieve analysis for fineness modulus

Table - Typical computations of fineness modulus

Sr.	Sieve (US)	Percentage Coarser		
		Sand	Coarse Agg.	* Mixture, 40% Sand and 60% Coarse Agg.
1	3 in.	-	0	0
2	$1\frac{1}{2}$ in.	-	3	2
3	$\frac{3}{4}$ in.	-	49	29
4	$\frac{3}{8}$ in.	0	77	46
5	# 4 ($\frac{3}{16}$ in.)	4	96	59
6	# 8 ($\frac{3}{32}$ in.)	15	100	66
7	# 16 ($\frac{3}{64}$ in.)	37	100	75
8	# 30 ($\frac{3}{128}$ in.)	62	100	85
9	# 50 ($\frac{3}{256}$ in.)	85	100	94
10	# 100 ($\frac{3}{512}$ in.)	98	100	99
Total		301	725	555
Fineness Modulus		3.01	7.25	5.55

* 0.40 × % of sand plus 0.60 × % of coarse aggregate



Sieves used for gradation test.

A mechanical shaker used for sieve analysis.

(2) Materials Inspection *contd.*

Water

It means fresh water free from oil, sewage or excessive silt. Too much chloride content in water (e.g., sea water) encourages corrosion of reinforcement. Maximum chloride contents for prestressed, reinforced and nonreinforced concretes may be taken as 500, 1000 or 4500 ml/l , respectively (SIKA). Sea water or brackish water may be used for non-reinforced concrete but is not suitable for reinforced and especially prestressed concrete. Waste water should not be used as mixing water. Sea water leads to slightly higher early strength but lower long-term strength (up to 15%). A pH value of 6.0 to 8.0 is acceptable; a value of even 9.0 may be accepted. Turbidity limit is about 2000 ppm.

Admixture

Concrete admixtures are liquid for powder additives. They are added to the concrete mixed in small quantities to meet specific requirements. Superplasticizers are frequently used for better workability. Admixtures may also be used for durability, acceleration, retardation, waterproofing, colour, etc. Superplasticizer, air entrainer, set accelerator, hardening accelerator, set retarder, waterproofer, etc. are commonly used admixtures.

Reinforcing steel

For foreign products, check country of origin; for local products, check manufacturer, type, size; always check strengths (test results), surface condition.

See the table on page (2) for specifications of reinforcing steels

(3) Proportioning of Concrete

Objectives

- A properly designed concrete mix achieves three objectives;

  (1) required quality of hardened concrete

  (2) workability of fresh concrete

  (3) economy

- Important qualities of hardened concrete such as strength, watertightness and wear resistance depends mostly on water-cement ratio and curing.
- Workability is the property that determines the amount of work required to fully consolidate the concrete. Although workability is difficult to measure, a consistency test called slump test is used to estimate it.
- To achieve economy, the mix design is aimed at minimizing the amount of cement required without sacrificing concrete quality. Since quality is primarily dependent on water-cement ratio, water requirement should be minimized to reduce the cement requirements. Steps include the use of: (1) the stiffest practical mixture, (2) the largest practical size of well-graded aggregate, and (3) the optimum ratio of fine to coarse aggregates.

(3) Proportioning of Concrete *contd.*

Water-cement ratio

- If possible, tests should be made with the job materials to determine the relationship between water-cement ratio and strength. If data cannot be obtained due to time limitations, water-cement ratio may be estimated from data such as those given in the table.

Table Relationship between water-cement ratio and compressive strength of concrete

Compressive strength at 28 days, psi	Water-cement ratio by weight (Non-air-entrained concrete)
6000	0.41
5000	0.48
4000	0.57
3000	0.68
2000	0.82

Maximum Aggregate Size

- Generally, the maximum aggregate size should not exceed: (1) one-fifth the minimum dimension of the member, (2) three-fourths the clear space between reinforcing bars or between reinforcement and the forms. For unreinforced slabs on ground, the maximum size should not exceed one-third the slab thickness.
- Amount of mixing water required to produce a cubic yard of concrete of a given slump (a measure of fluidity) is dependent on the maximum size of aggregate -- the smaller the maximum size of aggregate, the greater the amount of water required. It is therefore advisable to use the well-graded and well-shaped aggregate of largest practicable maximum size to minimize water, and hence cement, content. For many aggregates, the optimum maximum size is ¾ in. from strength point of view.

(3) Proportioning of Concrete *contd.*

Maximum Aggregate Size *contd.*

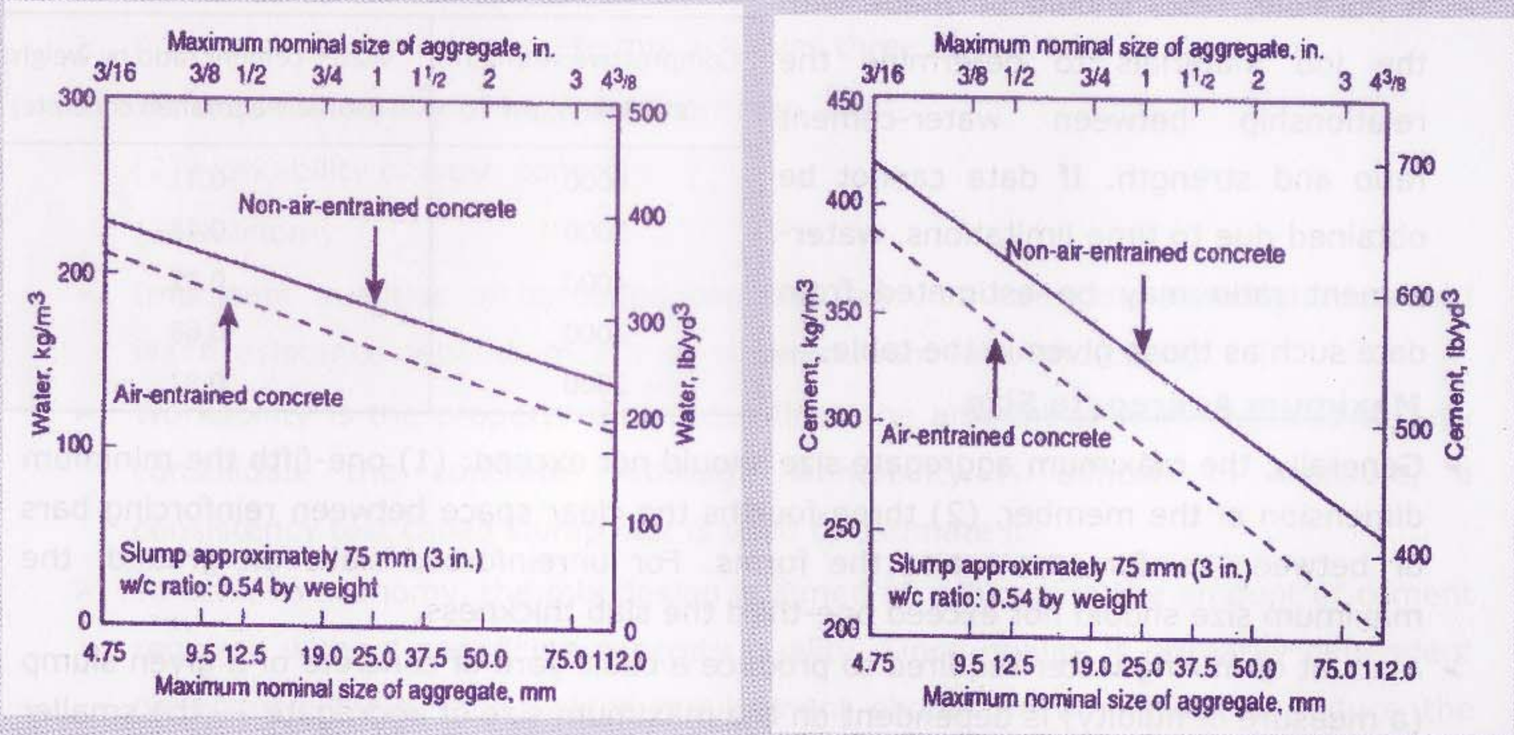


Fig. Cement and water contents in relation to maximum size of aggregate for air-entrained and non-air-entrained concretes. Less cement and water are required in mixtures having large, coarse aggregate (Portland Cement Association 2002)

(3) Proportioning of Concrete *contd.*

Slump

- The slump test is generally used as a measure of the fluidity of the concrete. Under conditions of uniform operation, changes in slump indicate changes in materials, mix proportions, or water content. To avoid mixes too stiff or too fluid, slumps within the limits given in the table are suggested. Slump tests are easy to carry out during concreting process and they should be done often so that timely corrections can be made in the proportions of the ingredient materials.

Table - Recommended slump for various types of construction

Types of construction	Slump, in.	
	Maximum*	Minimum
Reinforced foundation walls and footings	3	1
Plain footings, caissons, and substructure walls	3	1
Beams and reinforced walls	4	1
Building columns	4	1
Pavement and slabs	3	1
Mass concrete	2	1

** May be increased by 1 in. for methods of consolidation other than vibration*

(3) Proportioning of Concrete *contd.*

Trial mix

- The most direct method for determining optimum mix proportions is through the use of trial mixes. Such mixes may be (1) relatively small batches made with laboratory precision or (2) job-size batches made during the course of normal concrete production.
- In either method, water-cement ratio, maximum size of aggregate, air content, and range of slump must be selected first. By maintaining, in turn, all of these items constant except one as a variable, a series of mixes is made to determine the influence of that variable parameter. After making a sufficient number of series of trial mixes, based on considerations of workability, strength and economy, the most appropriate mix proportions are finally selected for use.

Job-size trial batches

- The first trial may be selected on the basis of experience or from established relationships such as the table given. The table is based on concrete having a slump of 3 to 4 in., with well-graded aggregates having a specific gravity of 2.65.

Table contd.

Water gal. per bag of cement	Max. size of agg. in.	Air content(entrapped air) %	Water gal. per yd^3 of conc.	Cement bags per yd^3 of conc.	With fine sand-fineness modulus = 2.50			With coarse sand-fineness modulus = 2.90		
					Fine agg. % of total agg.	Fine agg. lb per yd^3 of conc.	Coarse agg. lb per yd^3 of conc.	Fine agg. % of total agg.	Fine agg. lb per yd^3 of conc.	Fine agg. lb per yd^3 of conc.
5.5	3/8	3	46	8.4	55	1460	1190	59	1570	1080
5.5	1/2	2.5	44	8.0	47	1290	1460	51	1400	1350
5.5	3/4	2	41	7.5	39	1120	1760	43	1230	1650
5.5	1	1.5	39	7.1	35	1040	1940	39	1160	1820
5.5	1-1/2	1	36	6.6	31	960	2150	35	1080	2030
6	3/8	3	46	7.7	56	1520	1190	60	1630	1080
6	1/2	2.5	44	7.4	48	1340	1460	52	1450	1350
6	3/4	2	41	6.9	40	1170	1760	44	1280	1650
6	1	1.5	39	6.5	36	1090	1940	40	1210	1820
6	1-1/2	1	36	6.0	32	1000	2150	36	1120	2030
6.5	3/8	3	46	7.1	57	1550	1190	61	1660	1080
6.5	1/2	2.5	44	6.8	49	1380	1460	53	1500	1350
6.5	3/4	2	41	6.3	41	1220	1760	45	1330	1650
6.5	1	1.5	39	6.0	37	1130	1940	41	1250	1820
6.5	1-1/2	1	36	5.6	32	1030	2150	36	1160	2030

Table contd.

Water gal. per bag of cement	Max. size of agg. in.	Air content (entrapped air) %	Water gal. per yd^3 of conc.	Cement bags per yd^3 of conc.	With fine sand-fineness modulus = 2.50			With coarse sand-fineness modulus = 2.90		
					Fine agg. % of total agg.	Fine agg. lb per yd^3 of conc.	Coarse agg. lb per yd^3 of conc.	Fine agg. % of total agg.	Fine agg. lb per yd^3 of conc.	Fine agg. lb per yd^3 of conc.
7	3/8	3	46	6.6	58	1610	1190	61	1720	1080
7	1/2	2.5	44	6.3	49	1430	1460	53	1540	1350
7	3/4	2	41	5.9	42	1250	1760	45	1360	1650
7	1	1.5	39	5.6	37	1160	1940	41	1280	1820
7	1-1/2	1	36	5.2	33	1060	2150	37	1190	2030
7.5	3/8	3	46	6.2	58	1640	1190	62	1750	1080
7.5	1/2	2.5	44	5.9	50	1460	1460	54	1570	1350
7.5	3/4	2	41	5.5	42	1280	1760	46	1390	1650
7.5	1	1.5	39	5.2	38	1190	1940	42	1310	1820
7.5	1-1/2	1	36	4.8	34	1100	2150	38	1220	2030
8	3/8	3	46	5.8	58	1670	1190	62	1780	1080
8	1/2	2.5	44	5.5	51	1490	1460	54	1600	1350
8	3/4	2	41	5.2	42	1300	1760	46	1410	1650
8	1	1.5	39	4.9	38	1210	1940	42	1330	1820
8	1-1/2	1	36	4.5	34	1120	2150	38	1240	2030

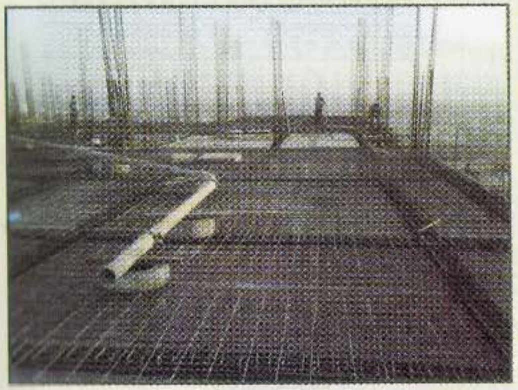
* *Increase or decrease water per cubic yard by 3 % for each increase or decrease of 1 in. in slump. For manufactured fine aggregate, increase percentage of fine aggregate by 3 and water by 2 gal per cubic yard of concrete. For less workable concrete, as in pavements, decrease percentage of fine aggregate by 3 and water by 1 gal per cubic yard of concrete. 1 US gal = 4.54 litre*

(4) Inspection before Concreting

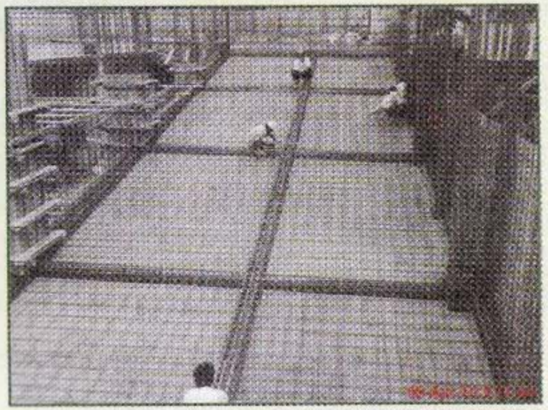
- Check location, alignment, level, plumb, dimensions and shape of members
- Check forms and scaffolding - provisions against settlement and buckling under weight (lateral bracing, supports, footings for supports and wedges), against bulging under concrete pressure (shores, ties); preparation of surfaces (plugging of holes, oiling); final clean-up; moistening of wood forms
- Check reinforcement in place - number of bars, size, length, concrete cover, splices, end anchorages, hooks, bends, spacing of main bars, stirrups and ties, stability (wiring, chairs, spacer blocks), cleanness (no loose rust, oil, paint)
- Check fixtures - location, orientation, cover, stability
- Check mixers (cleanness, working condition, blades, capacity), vibrators (type, size, number, spare units), power source, moulds (materials, geometrical correctness, watertightness, oiling)
- Check adequacy of concrete-making materials for continuous placement. Check measuring containers (size, number)
- Check provisions for curing (depends on which curing method is to be used)
- Check provisions for protection against sun, rain or wind and, if necessary, for concreting at night time
- Check leveling instruments for taking measurements before, during and after concreting
- Check adequacy of all necessary tools and men for the whole concreting operation
- Check expansion and contraction joints, if any
- Check safety provisions, such as safety net, safety helmet, etc,.

(4) Inspection before Concreting *contd.*

- For job mixing, the mixing site should be as close as practicable to the point where the concrete is to be placed; the aggregates should be placed so that they are convenient to the mixer operators. The cement must be stored in a dry location, raised above the earth and covered.



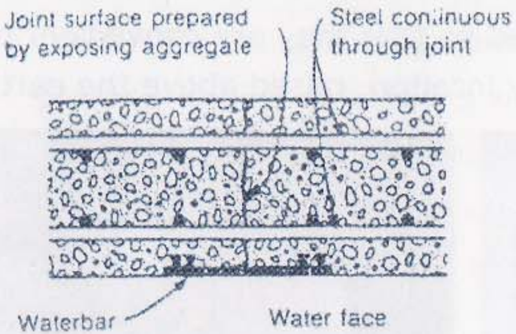
Ramp under inspection before concreting



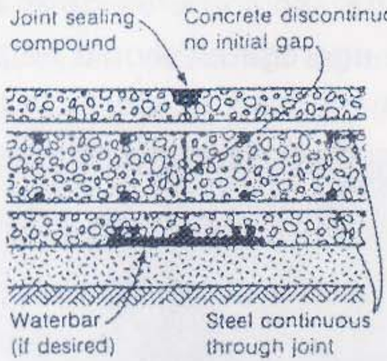
Use of safety net

(4) Inspection before Concreting *contd.*

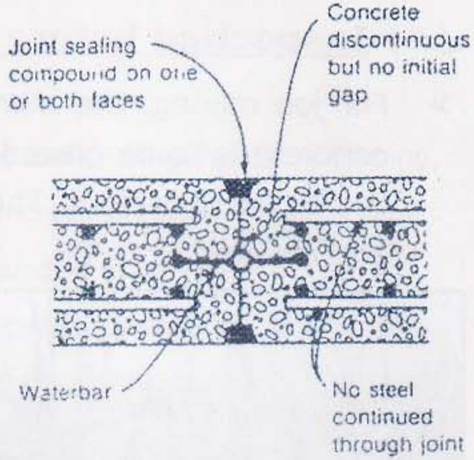
Expansion and contraction joints



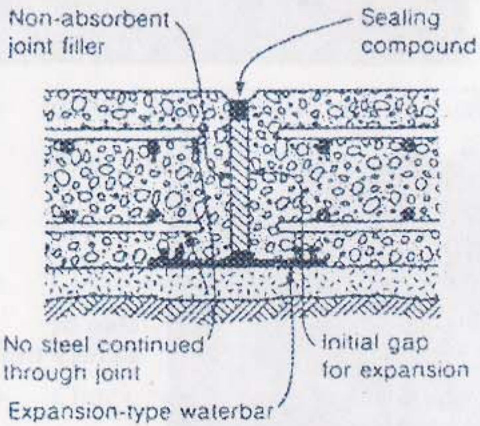
(a) Contraction joint in reservoir wall



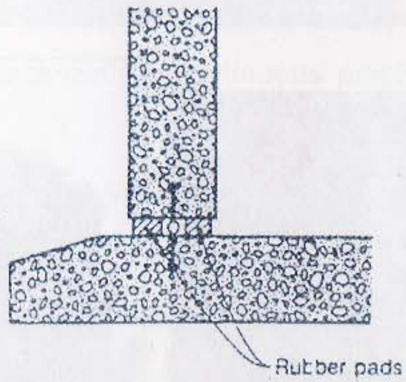
(b) Partial contraction joint in reservoir wall



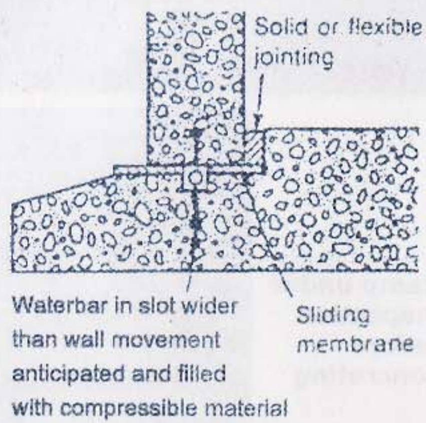
(c) Complete contraction joint in reservoir wall



(d) Expansion joint in reservoir wall

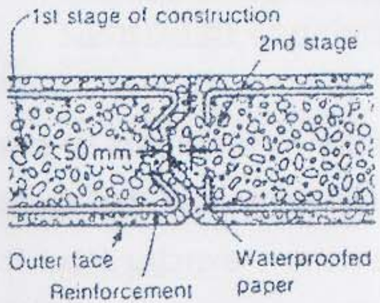


(e) With rubber pads



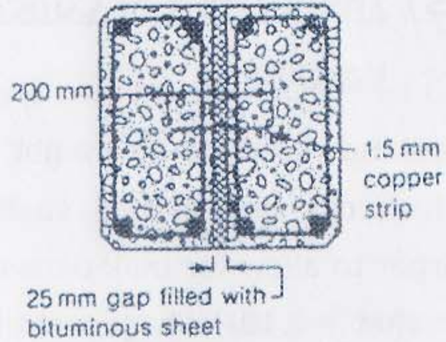
(f) With sliding membrane

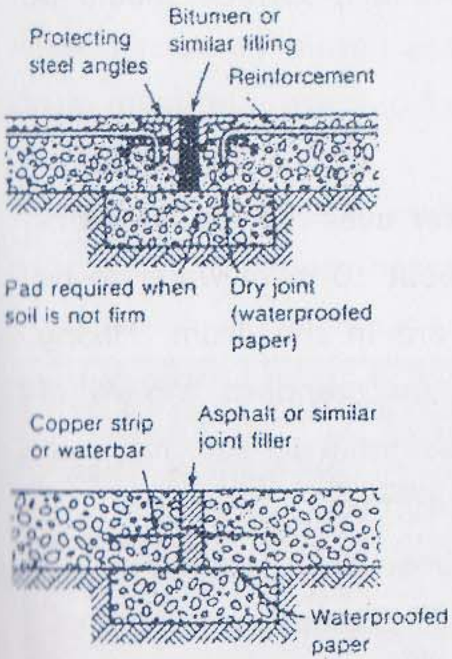
Expansion and contraction joints *contd.*



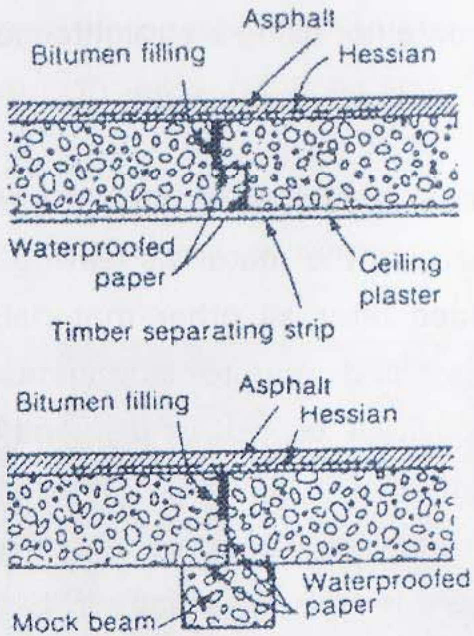
(g) Vertical joint in cantilevered retaining wall

(h) Expansion joint at column

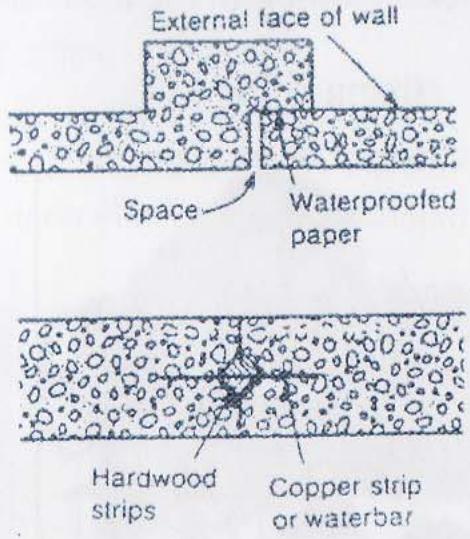




(j) Alternative designs for joints in floor laid on the ground



(k) Alternative designs for joints in roof slabs



(l) Alternative designs for joints in external walls of buildings

(5) Inspection of Concreting

Batching

This may be done by weight or by volume; if by volume, make measuring containers of size which can be handled easily by workers. Cement-measuring container should be made larger to allow for bulk density of cement. Or assume that 94 lb of cement is exactly 1 cu. ft. or that a 110 lb-bag contains 1.17 cu. ft. In measuring all materials, surface should be struck off level with the top of the container using a straightedge.

Mixing

Water should be fed into the mixer over the full period of charging the materials leaving about 10 % of water to be added after all other materials are in the drum. Mixing time of 1 minute is minimum for standard mixers of capacity 1 cu. yd. or less and additional 15 sec. for each additional ½ cu. yd. Preferred mixing time is three minutes after all materials have been placed in the mixer. There is little advantage if it is more than three minutes.

(5) Inspection of Concreting *contd.*

Control of consistency

The mix should be only as wet as absolutely necessary for proper placement. Usual test for consistency is slump test (ASTM C143 or BS 1881 : Part 102). It should be performed often throughout the concrete production period. See "Testing of concrete".

Cleaning

The mixer and other equipment or tools used in contact with concrete should be cleaned when necessary during use and thoroughly cleaned after use. Concrete build-up inside the drum must be prevented for top efficiency in mixing and discharging.

Conveying

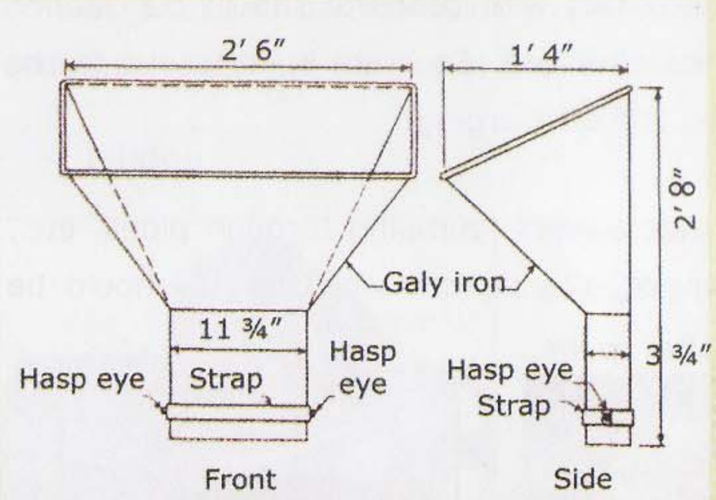
Buckets or pans, carts, wheel barrows, trucks, chutes, belts, pumping through pipes, etc., are used. Whenever concrete is dumped or dropped, the direction of the fall should be vertical to avoid segregation.



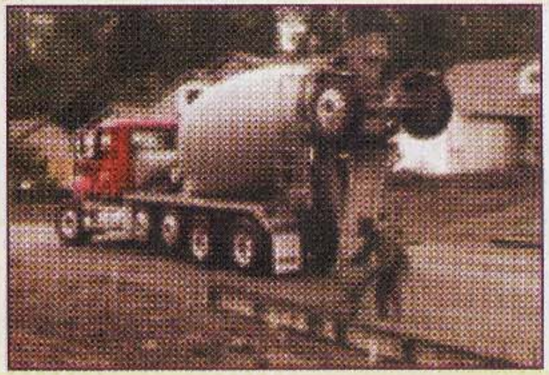
(5) Inspection of Concreting *contd.*

Placing

Direction of drop should be vertical. Rakes should not be used to spread concrete. The use of a chute is recommended when fresh concrete must be dropped more than 3 or 4 ft. In narrow wall forms, metal drop chutes are made rectangular to fit between reinforcing steel.

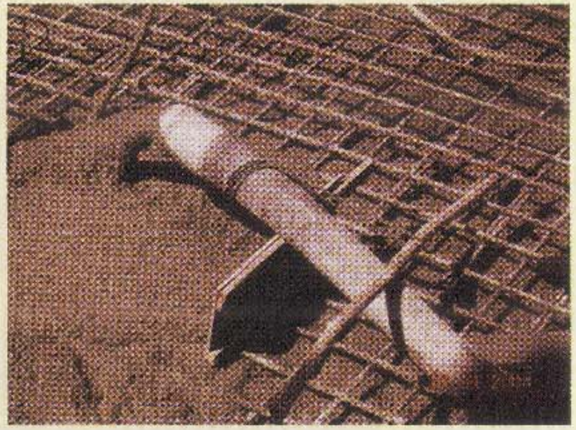


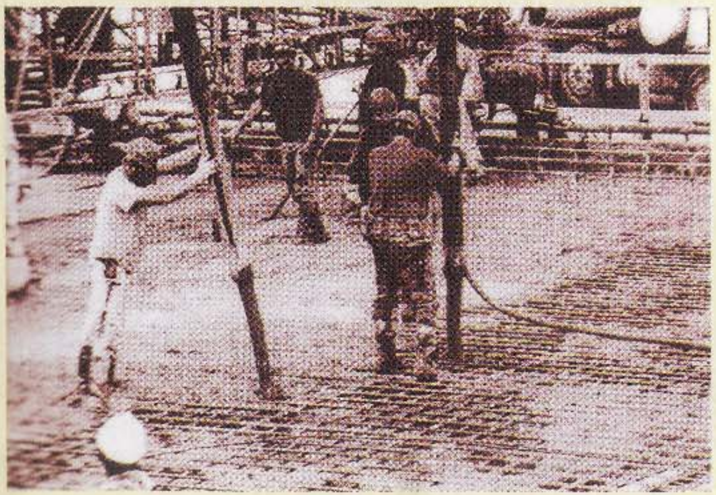
A rectangular drop chute with hopper at top for placing concrete in narrow walls.



(5) Inspection of Concreting *contd.*

Placing *contd.*

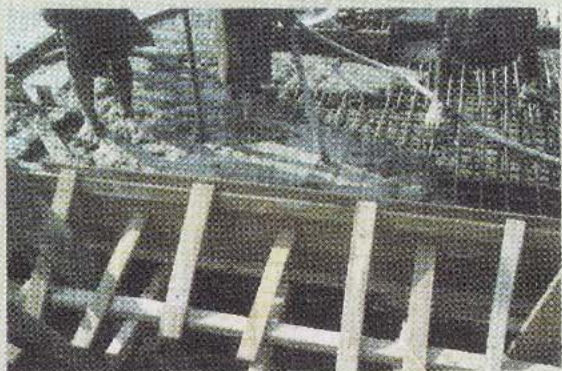




(5) Inspection of Concreting *contd.*

Consolidation

Concrete should be consolidated thoroughly and uniformly by means of hand tools or vibrators. Do not overvibrate as it tends to segregate and water and fine particles move upwards. Sufficient equipment (with spare) should be provided so that entire mixer output can be handled without delay. Vibration is from 5 to 15 sec at points of 18 in. to 30 in. Internal vibrators should be inserted vertically and not be dragged laterally through concrete. It should be withdrawn slowly while being vibrated. Form vibrators may be attached to the exterior of forms. They are especially useful for consolidating concrete in thin-walled members and where metal forms are used.



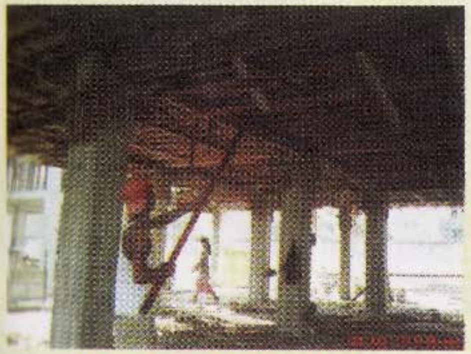
(6) Inspection after Concreting

Protection from damage

Protect damage to fresh concrete due to impact and marring of surface by rain, etc.

Removal of forms

Removal of beam form



Removal of slab form

Vertical forms can be removed in one day. Forms directly supporting the weight of concrete must be left in place for longer periods. Props for supporting slab forms may not be removed before 2 weeks and those supporting beams may not be removed before 3 weeks but it depends also on whether these slabs and beams are carrying loads from another floor above through another system of support for that storey. It is advisable not to remove all supports at the same time but leave some supports until the floor above the floor whose supporting props are being removed attains its own load-carrying capacity to an extent.

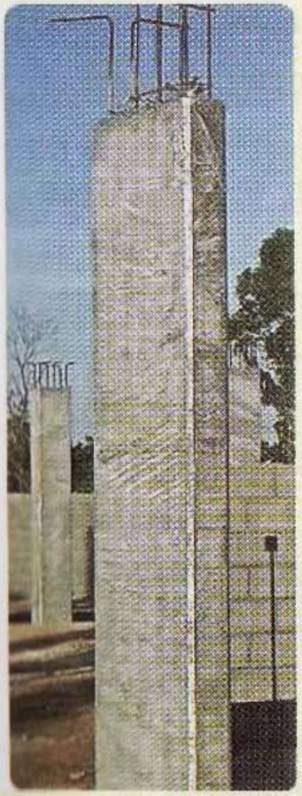
(6) Inspection after Concreting *contd.*

Curing

Exposed surfaces should be kept continuously moist for at least 7 days and many specifications require 14 days of curing. Since all the desirable properties of concrete improve by curing, the duration should be as long as practicable. Preferred methods of curing include continuous sprays, flowing or ponded water, continuous saturated coverings of sand, burlap or other absorbent materials. Materials used for water retention must be kept damp constantly during the curing period. Waterproof or plastic sheets are satisfactory curing agents; they are held close against concrete surface and the seams are tightly sealed. These materials should be applied as soon as the concrete has hardened sufficiently to prevent surface damage. Sealing compound can also be used to prevent evaporation from surfaces. Membrane curing compound should not be applied when there is free water on the surface. Nor the compound be applied after the concrete has dried out. The correct time to apply the membrane is when the water sheen disappears from the surface of the finished concrete. Chemical membranes are suitable not only for curing fresh concrete but also for further curing of concrete after removal of forms or after initial moist curing.

(6) Inspection after Concreting *contd.*

Curing *contd.*



Plastic wrapped around a concrete column

Concrete slab cured by ponding

Spraying curing compound

Wetting the gunny wrapped around the column

Spraying water to cure the slab

(7) Testing of Concrete

Sampling

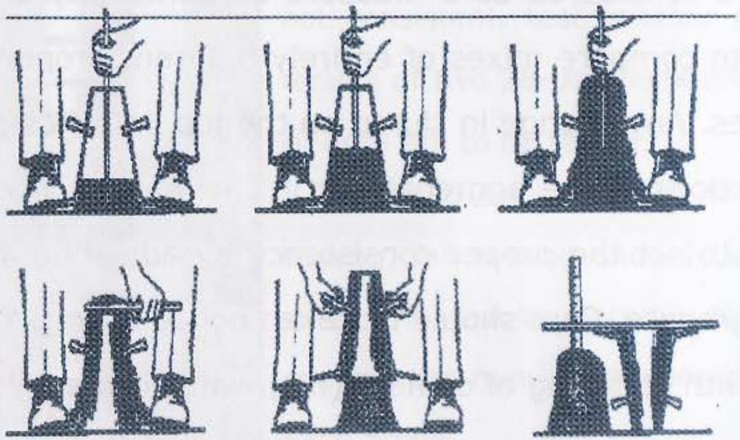
Use every precaution that will assist in obtaining samples that will be representative of the true nature and condition of the concrete sampled. ASTM C172 "Standard Method of Sampling Fresh Concrete" covers the sampling procedure. To take samples from stationary construction mixers, a receptacle should be passed through the discharge stream, at about the middle of the batch. From paving mixes, after the concrete is discharged on the subgrade, take portions from several points until the amount is greater than that required. When taking samples from revolving drum truck mixers or agitators, take three or more portions in regular increments throughout discharge of entire batch. Sampling is done by repeatedly passing a receptacle through the entire discharge stream or by diverting the stream completely so that it discharges into a container such as a wheelbarrow. The composite sample must be transported to the place where test specimens are to be moulded or where test is to be made and then remixed with a shovel the minimum amount to ensure uniformity, and used immediately for the specimens or tests. The sample must be protected from sunlight, rain and wind during the period between taking and using, which shall not exceed 15 minutes. The location in the structure where the concrete, from which the sample is taken, is cast must be noted down.

(7) Testing of Concrete *contd.*

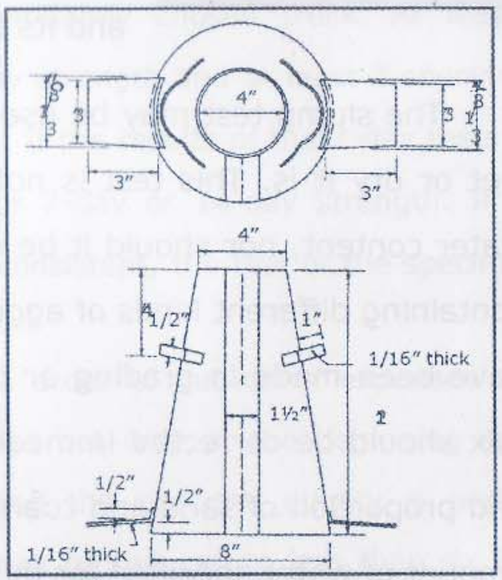
Consistency test

Slump test (ASTM C143 or BS 1881 Part 102)

ASTM C143-90a — the mould is dampened and placed on a flat surface such as smooth plank or slab of concrete. The mould is filled with concrete in *three layers*, each for one-third of the mould (one-third means 2 5/8 in., two-thirds means 6 1/8 in. and full means 12 in. from the bottom); each layer is rodded with *25 strokes* of tamping rod, a round steel rod 5/8 in. in diameter and 24 in. in length; the tamping end is rounded. The bottom layer is to be rodded throughout its depth, the second and top layers to be rodded throughout the depth of each layer. The entire test should be completed within 2 ½ min. Slump is measured as the distance of the displaced original centre of the top of the slumped pile.



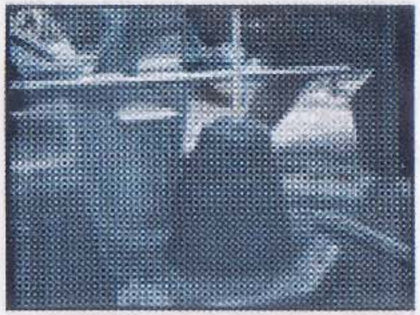
Slump test procedure



Truncated cone for slump test

Consistency test *contd.*

BS 1881 : Part 102:1983 — the mould is filled with concrete in *four layers*; each layer is

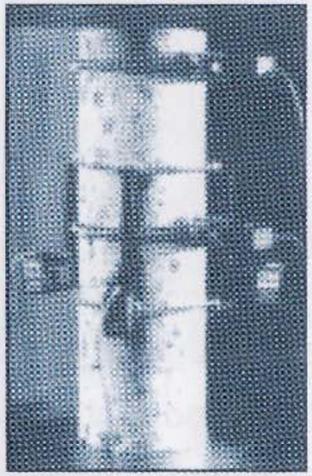


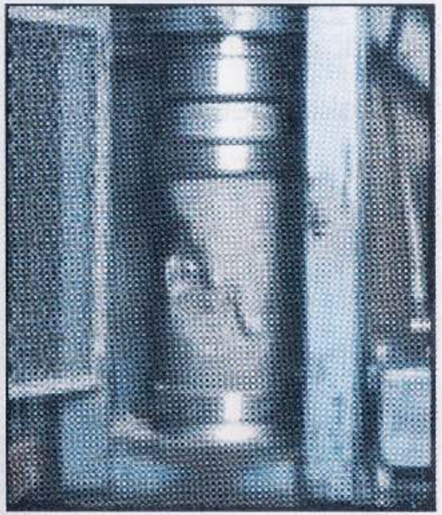
tamped *25 times* with a standard 5/8 in. diameter steel rod, rounded at the end. Immediately after filling, the top surface is struck with a trowel, the cone is slowly lifted vertically and the concrete will slump. The decrease in the height of the highest part is "slump" measured to the nearest ¼ in. The inside of the mould and its base should be moistened at the beginning of every test.

The slump test may be used as a rough measure of the consistency of concrete - too wet or dry it is. This test is not to be considered as a measure of workability or proper water content, nor should it be used to compare mixes of entirely different proportions or containing different kinds of aggregates. Any change in slump on the job indicates changes have been made in grading or proportions of the aggregate or in the water content. The mix should be corrected immediately to set the proper consistency by adjusting amounts and proportion of sand and coarse aggregate. Care should be taken not to change the total amount of water specified for mixing with each bag of cement (i.e., water-cement ratio).

(7) Testing of Concrete *contd.*

Strength test





According to ACI 318, minimum number of specimens or tests is specified as 1 test (at least 2 specimens from the same sample tested at the same age) for each 150 cu. yd. or for each 5000 sq. ft. of surface area actually placed, but not less than once in a concreting day.

According to CQHP, sampling frequency is once each day (or) once for each 50 m^3 of concrete placed. The number of specimens is 6 for each 50 m^3, 2 specimens from each randomly chosen truck. At least 2 specimens are to be tested for 7-day strength and at least 2 specimens are to be tested for 28-day strength. If the results of the 7-day tests are not consistent, test another pair for 7-day or 14-day strength. If the results of two 28-day tests are not consistent, the rest of the specimens (if any) are to be tested.

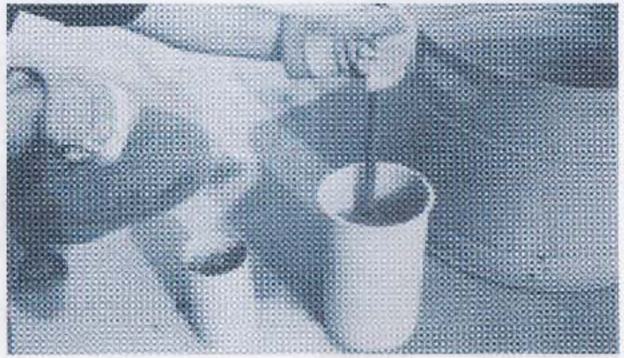
It should be suggested that , in order to cut down expenses on testing, full testing should be carried out only at the beginning and when the results become stable and the concrete quality is under control, the number of specimens may be reduced to less than six for each 50 m^3.

Strength test *contd.*

Making specimens (ASTM C31 and BS 1881: Part 108)

ASTM C31 — making and curing specimens in the field; cylinder size for maximum aggregate size of up to 2 in. is 6 in. × 12 in. Plastic moulds are common.

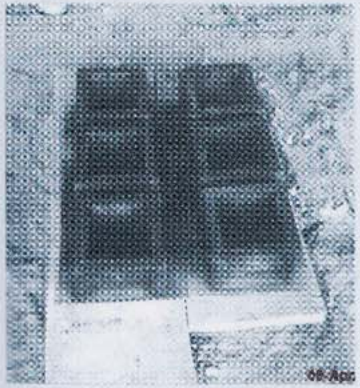
Making cylinders using plastic moulds



A sample of the concrete is taken at three or more regular intervals throughout the discharge of the entire batch. Use 3 layers of equal depth if rodding is applied and 2 layers of equal depth if vibration is applied. The selection of the method of consolidation is based on the slump. Concretes with slump1 in. or greater may be rodded or vibrated; and concretes with slump less than 1 in. shall be vibrated. The number of strokes per layer is 25 for 6 in. diameter cylinders and 50 for 8 in. diameter cylinders. The rod is of 5/8 in. round steel about 24 in. long with hemispherical tip. Reinforcing rods or other tools should not be used as the puddle rod. To vibrate cylinder moulds, two insertions of the vibrator are used for each layer at different points for a 6 in. × 12 in. cylinder. Avoid overvibration. The top is struck off with a wooden float to produce a flat, even and level surface, and the specimen is covered with a glass or metal plate or plastic lid to prevent evaporation.

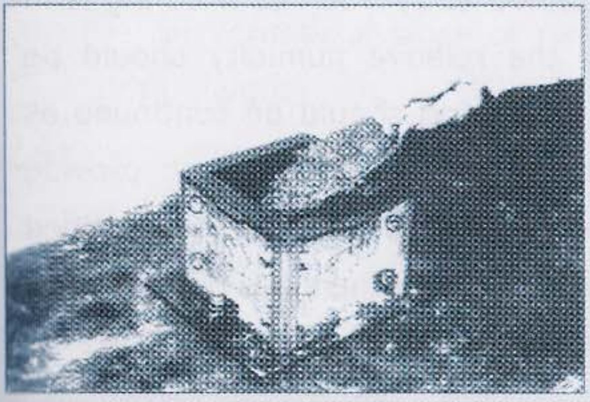
Making specimens (ASTM C31 and BS 1881) *contd.*

BS 1881: Part 108: 1983 or CS1: 1990 (Hong Kong Construction Standard) — the inside of the mould must be thinly coated with mould oil to prevent concrete from sticking to it. Each sample of concrete should consist of at least six increments when it is taken from a heap or lorry, and at least four increments when taken from a chute or conveyor. Whichever way the samples are taken, the parts must be thoroughly mixed together. Then, the 6 inch cube mould (for maximum size of aggregate 40 mm) is to be filled in 3 layers. Compaction can be by hand or by vibration. If compaction is by hand, then the number of strokes per layer 2 in. deep should be varied according to the type of concrete but should not be less than 35 strokes for 6 inch cube or 25 strokes for 4 inch cube.



Cube moulds

The compacting bar is 380 mm long steel bar, weighs 1.8 kg and has a 25 mm square end for ramming. When compaction is by vibration, the mould should be filled in equal layers as with compaction by hand. Vibration should cease as soon as the surface of the concrete becomes relatively smooth and air bubbles cease to appear. A trowel should be used to finish of the surface level with the top of the mould.



Filling the mould (for 150 mm cube 3 equal layers)

Curing specimens (ASTM C31 and BS 1881)

ASTM C31 — requires that during the first 24 hr, field-made specimens are to be kept in a storage box at air temperature between 60° F and 80° F. Protect from direct sunlight or extreme weather. Specimens should be removed from the moulds at the end of 24 hours and should then be stored (usually in the laboratory) in a "moist condition" at a temperature within 65° F to 75° F until the time of test. A "moist condition" is defined as that in which free water is maintained on the surfaces of the specimens at all times. In any event cylinders should not be moved or transported until at least 8 hours after final set.

Mist room in the laboratory

BS 1881 — in curing cubes, the curing temperature of the water in the curing tank should be maintained at 27°-30° C. If curing is in a mist room (or moist room), the relative humidity should be maintained at no less than 95 %. Curing should be continued as long as possible up to the time of testing. In order to provide adequate circulation of water, adequate space should be provided between cubes, and between the cubes and the sides of the curing tank. There should also be sufficient space between cubes if curing is done in a mist room.

(8) Inspection of Workmanship

Apart from inspection of concrete, inspection of workmanship is also part of the duty of the inspector, who should inspect:

(i) planeness of the area surfaces such as walls, floors, ceilings, underside of stairs

(ii) verticality of vertical elements such as columns, walls, door and window frames

(iii) horizontality of horizontal elements such as beams, floors, ceilings, door and window frames

(iv) angular correctness of the junctions such as walls, door and window frames

(v) level of each element such as unfinished levels and finished levels of floors, stair landings, ceilings, footings

(vi) the correctness of slope of ramps, steps of staircase and dimensions of rooms (length, width, height)

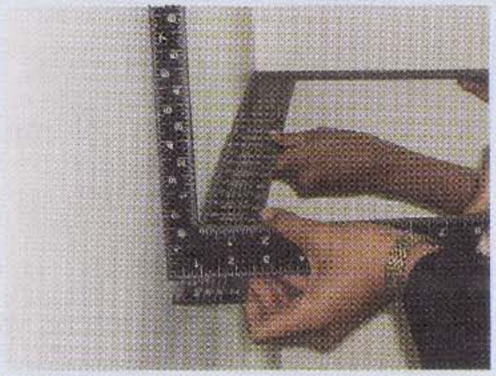
(vii) the surface texture of walls, floors and ceiling surfaces

(viii) the quality of putty, paints, polish, varnish and other coatings

Simple and inexpensive tools can be used to inspect the workmanship quality of many of the above-mentioned items.

(8) Inspection of Workmanship *contd.*

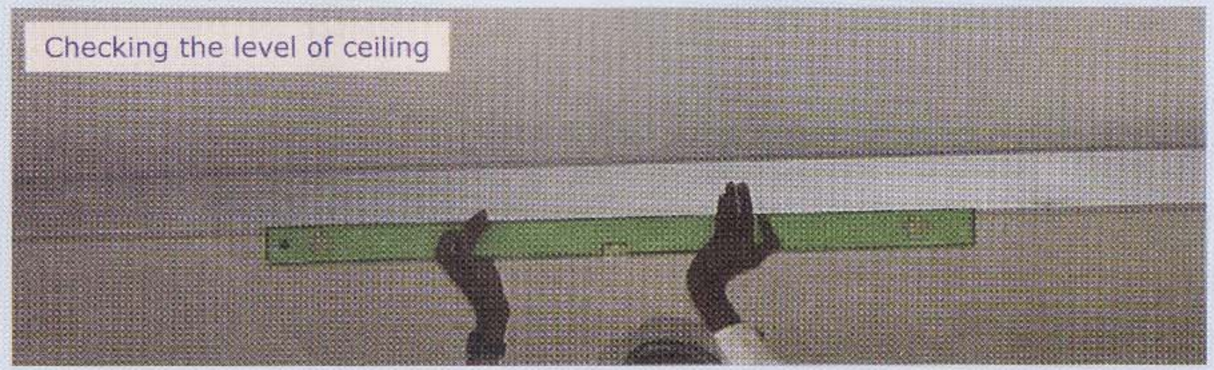
Simple inspection tools



Checking the interior corner angle

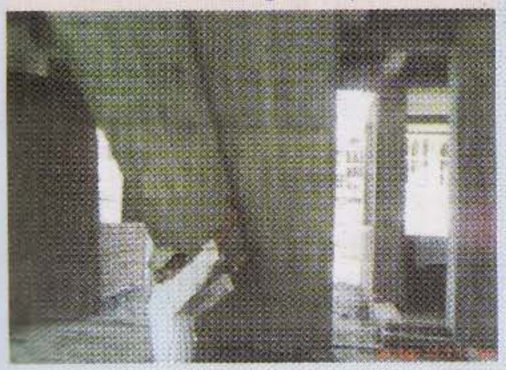
Checking whether the wall surface is plane

Checking the level of ceiling



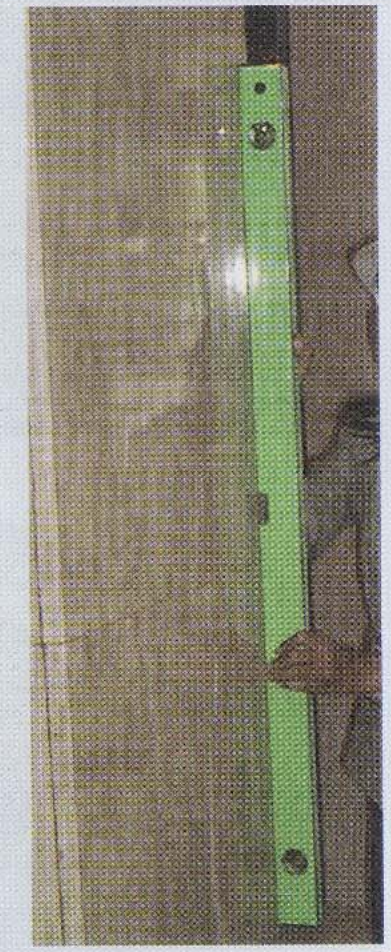
(8) Inspection of Workmanship *contd.*

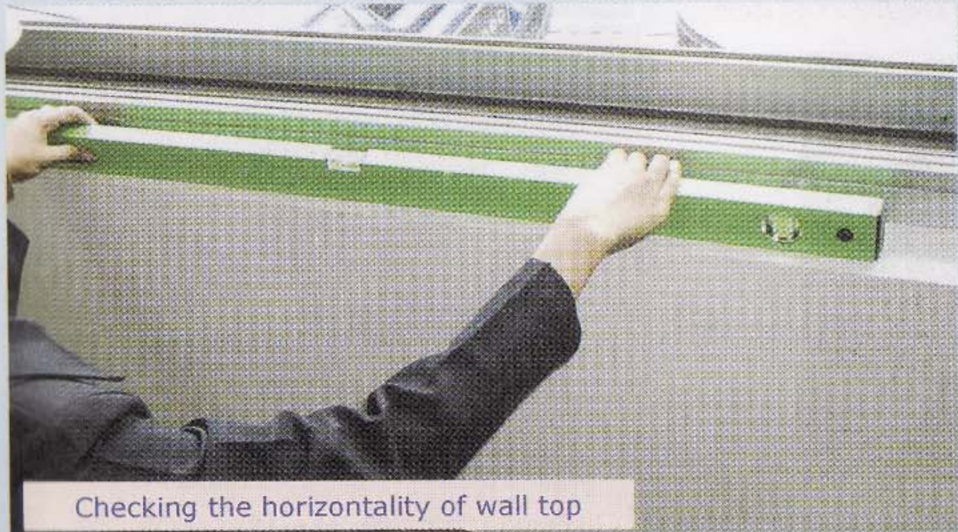
Checking whether the underside of the stair flight is plane



Checking the width of a room

Checking the verticality of tiled wall

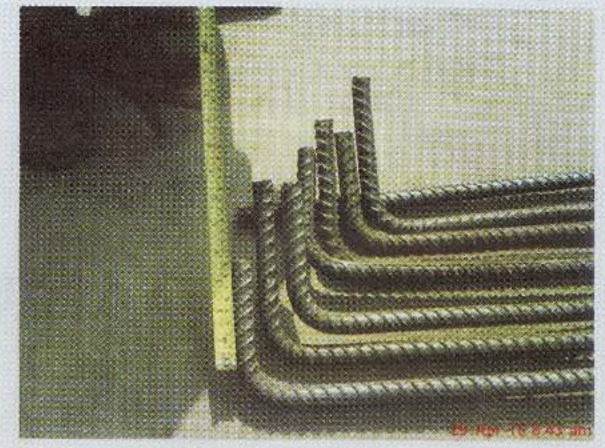




Checking the horizontality of wall top

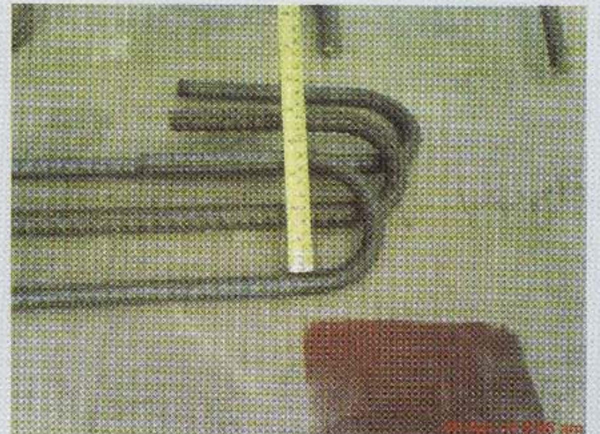
(8) Inspection of Workmanship *contd.*

Checking the spacing between two layers of steel



Checking the extension length of a 90° hook

Checking the radius of a 90° hook



Checking the radius of a 180° hook

CONCRETE QUALITY CONTROL

D. CONCRETE QUALITY CONTROL

(1) Standard Deviation

Standard deviation $s = \sqrt{\dfrac{\sum (X - \overline{X})^2}{n-1}}$

where X = individual strength test result, psi
$\overline{X}$ = average strength, psi
n = number of tests

For example, suppose six test results are 4000 psi, 2500 psi, 3000 psi, 4000 psi, 5000 psi and 2500 psi.

To fine the standard deviation, first find $\overline{X}$.

$$\overline{X} = \frac{\sum X}{n} = \frac{21000}{6} = 3500 \text{ psi.}$$

Deviation $(X - \overline{X})$	$(X - \overline{X})^2$
4000 − 3500 = + 500	+ 250,000
2500 − 3500 = −1000	+ 1,000,000
3000 − 3500 = − 500	+ 250,000
4000 − 3500 = + 500	+ 250,000
5000 − 3500 = +1500	+ 2,250,000
2500 − 3500 = −1000	+ 1,000,000
Total	+ 5,000,000

$$s = \sqrt{\frac{5,000,000}{5}} = 1,000 \text{ psi}$$

Standard deviations are normally established by at least 30 consecutive tests on representative materials.

If the number of tests is less than 30, but at least 15 tests are available, the standard deviation can be modified as follows:

Table - Modification factor for standard deviation when less than 30 tests are available (ACI Table 5.3.1.2)

No. of tests*	Modification factor for standard deviation †
Less than 15	Use Table at bottom of p.73
15	1.16
20	1.08
25	1.03
30 or more	1.00

* Interpolate for intermediate numbers of tests.

† Modified standard deviation to be used to determine required average strength f'_{cr} from the table at the top of p. 73

(2) Quality Control Level

For f_c' between 3000 psi and 4000 psi, the expected standard deviations representing different levels of quality control are as follows:

Standard Deviation	Representing
300 to 400 psi	Excellent Quality Control
400 to 500 psi	Good
500 to 600 psi	Fair
> 600 psi	Poor Quality Control

(3) Required Average Strength

If the concrete production facility has at least 30 consecutive strength test records representing materials and conditions similar to those expected, the required average strength used as the basis for mix design is given in the following table.

Table - Required average compressive strength when data are available to establish a sample standard deviation (ACI Table 5.3.2.1)

Specified compressive strength, f_c', psi	Required average compressive strength, f_{cr}', psi
$f_c' \leq 5000$	Use the larger value computed from Eq. (*a*) and Eq. (*b*) $f_{cr}' = f_c' + 1.34\ s$ (*a*) $f_{cr}' = f_c' + 2.33\ s - 500$ (*b*)
Over 5000	Use the larger value computed from Eq. (*c*) and Eq. (*d*) $f_{cr}' = f_c' + 1.34\ s$ (*c*) $f_{cr}' = 0.90\ f_c' + 2.33\ s$ (*d*)

If the standard deviation is unknown, the following guidelines shall be used.

Table - Required average compressive strength when data are not available to establish a sample standard deviation (ACI Table 5.3.2.2)

Specified compressive strength, f_c', psi	Required average compressive strength, f_{cr}', psi
Less than 3000	$f_c' + 1000$ (*a*)
3000 to 5000	$f_c' + 1200$ (*b*)
Over 5000	$1.10\ f_c' + 700$ (*c*)

(4) Frequency of Testing

Minimum number of strength tests (ACI) -- this number shall not be less than:

(1) once per day, nor less than,

(2) once for each 150 cu. yd. of concrete placed, nor less than,

(3) once for each 5000 sq. ft. of surface area of slabs or walls placed.

CQHP guidelines --

(1) once each day for each class of concrete placed

(2) once for each 50 m^3 of each class of concrete placed

(3) Six specimens for each 50 m^3, two from each randomly chosen truck, two specimens are to be tested for 7-day strength, two for 28-day strength, and if the 7-day results are not consistent, another two specimens are to be tested for 7-day or 14-day strength. If the 28-day results are not consistent, the rest of the specimens (if any) are to be tested.

Note : *in order to reduce expenses for testing, the rather conservative CQHP guideline no. (3) may be followed strictly only in the beginning of a project and lateron, when the results are stable, the number of specimens to be taken may be reduced.*

(5) Acceptance of Concrete

The strength level of an individual class of concrete is considered satisfactory if both of the following criteria are met:

(1) No single test strength (the average of the strengths of at least 2 cylinders from a batch) shall be more than 500 psi below the specified compressive strength f_c' (i.e., not less than 2500 psi for a specified 3000 psi concrete, for example)

(2) The average of any three consecutive test strengths must equal or exceed the specified compressive strength f_c' .

As an example, the following table lists strength test data from 5 batches of concrete. For each batch, two cylinders were cast and tested at 28 days. f_c' = 4000 psi. It is required to test the acceptability of this concrete.

(5) Acceptance of Concrete *contd.*

Test #	Cylinder #1	Cylinder #2	Test average	Average of 3 consecutive tests
1	3620	3550	3585	—
2	3970	4060	4015	—
3	4080	4000	4040	3880 *
4	4860	4700	4780	4278
5	3390	3110	3250 **	4023

* Average of 3 consecutive tests lower than $f_c^{'}$, not acceptable

** Test result with more than 500 psi below specified value $f_c^{'}$, not acceptable

It is important to let the concrete providers know about these acceptance criteria from the beginning to avoid misunderstanding between the users and the providers of concrete.
Note the difference in the meanings between $f_c^{'}$ (used by designers) and concrete Grade such as Grade C30 (used by concrete providers)

Table – Concrete strength class according to EC 2 (REYNOLDS *et al*)

Concrete strength class	Characteristic cylinder strength at 28 days $f_{ck}^{'}$, (Mpa)*	Characteristic cube strength at 28 days $f_{ck,cube}^{'}$, (Mpa)**
C 20/25	20	25
C 250/30	25	30
C 30/37	30	37
C 35/45	35	45
C 40/50	40	50
C 45/55	45	55
C 50/60	50	60
C 55/67	55	67
C 60/75	60	75
C 70/85	70	85
C 80/95	80	95
C 90/105	90	105

Note :

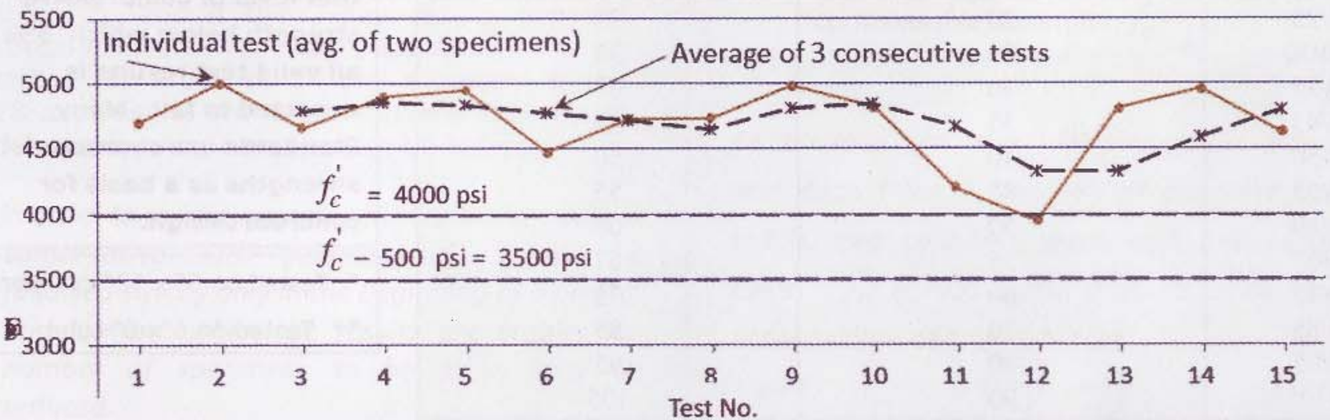
The characteristic strength is that level of compressive strength below which 5% of all valid test results is expected to fall . Many Standards use characteristic shrengths as a basis for concrete design.

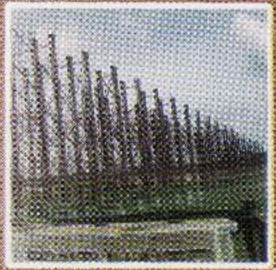
* Tested on 6" x 12" cylinders

** Tested on 6"x 6" cube

(6) Quality Control Charts (example)

Test #	Date of test	28-Day # 1	28-Day # 2	28-Day Average	28-Day Average (3-Consecutive)
1	05-March-10	4640	4770	4705	–
2	06-March-10	4910	5100	5005	–
3	10-March-10	4570	4760	4665	4792
4	12-March-10	4800	5000	4900	4857
5	13-March-10	5000	4900	4950	4838
6	17-March-10	4380	4570	4475	4775
7	19-March-10	4630	4820	4725	4717
8	21-March-10	4800	4670	4735	4645
9	25-March-10	5020	4940	4980	4813
10	28-March-10	4740	4900	4820	4845
11	30-March-10	4300	4110	4205	4668
12	02-April-10	4280	3620	3950	4325
13	05-April-10	4740	4880	4810	4322
14	08-April-10	4870	5040	4955	4592
15	09-April-10	4590	4670	4630	4798





STEEL CONSTRUCTION

E. STEEL CONSTRUCTION

(1) AISC Manual

Previously, Load and Resistance Factor Design (LRFD) was covered by 1999 AISC Specification and LRFD Manual of Steel Construction, 3rd edition. Allowable Stress Design (ASD) was covered by 1978 AISC Specification and Manual of Steel Construction, 9th edition. In 2005, the two approaches were unified in a single specification and a single manual, the 13th edition of Steel Construction Manual (AISC 325-05).

(2) Material Properties of Structural Steel

Table – Material properties

Steel Type	ASTM Designation or Grade of Structural Steel	f_y (ksi)	f_u (ksi)
Carbon Steel	A36	36	58-80
	A53 Grade B	35	60
	A500 Grade B	42 or 46	58
	A500 Grade C	46 or 50	62
High-Strength, Low-Alloy	A913	50-70	60-90
	A992	50-65	65
	A572 Grade 50	50	65
Corrosion-Resistant, High-Strength, Low-Alloy	A242	50	70
	A588	50	70

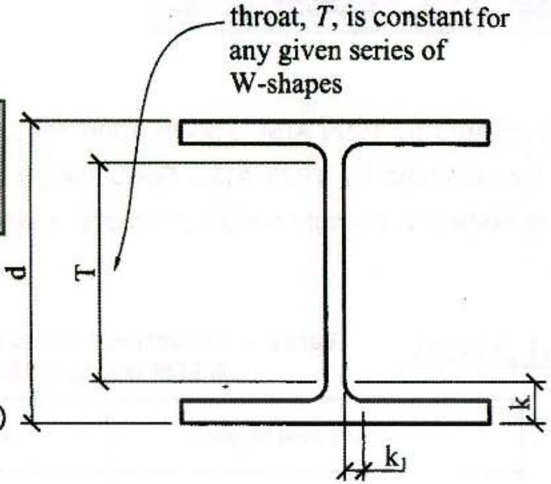
Table – Structural steel shapes and ASTM designations

Structural Steel Shapes	ASTM Designation
W-Shape	A913** A992*
M- and S- shapes	A36
Channels (C- and MC- Shapes)	A36* A572 Grade 50
Angles and plates	A36
Steel Pipe	A53 Grade B
Round HSS	A500 Grade B* A500 Grade C
Square and Rectangular HSS	A500 Grade B* A500 Grade C

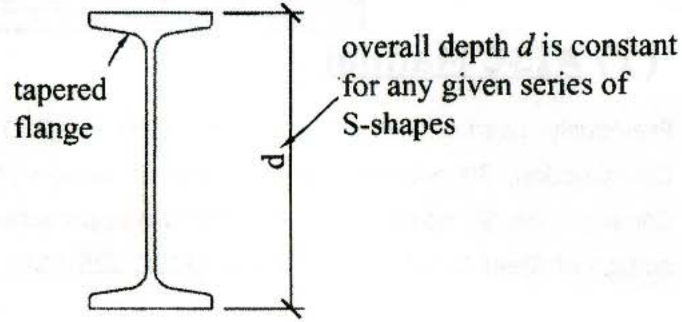
* Preferred material specification for the different shapes. ** A913 is a low-alloy, high-strength steel.

(3) Various Shapes of Sections

W14x90 means nominal depth 14" and own-weight 90 lb/ft

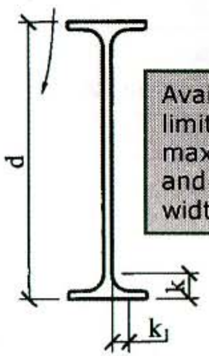


W-shape
(Wide-flanged shape)



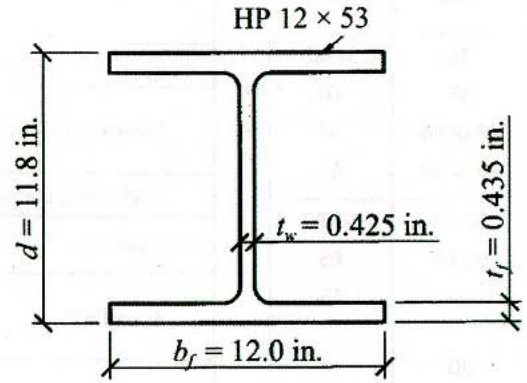
S-shape
(also known as American Standard Beam)

overall depth, d, is constant for any given series of M-shapes



Available sizes are limited;
max. depth = 12.5"
and max. flange width = 5"

M-shape

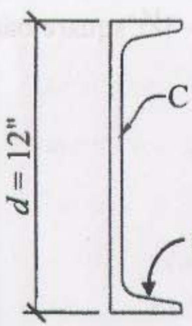


HP-shape (example)

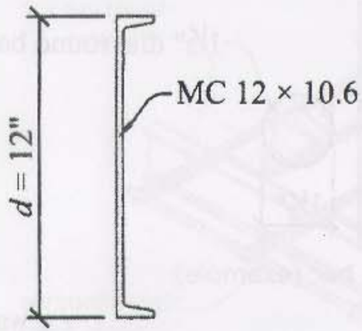
HP-shapes

* Commonly used as bearing piles
* Nominal depth approximately equal to flange width
* Flange and web thicknesses are approximately equal
* Web thicker than for comparable W-shape section
* Only a few sizes are available

(3) Various Shapes of Sections *contd.*



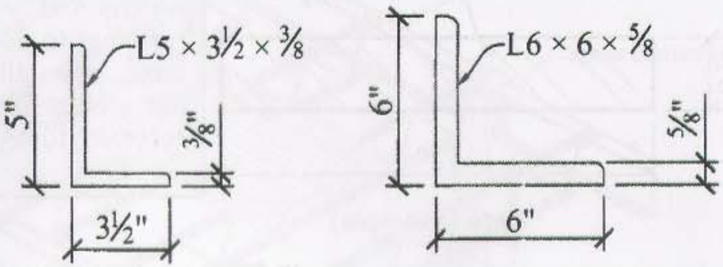
C-shape (example)
(American Standard Channel)



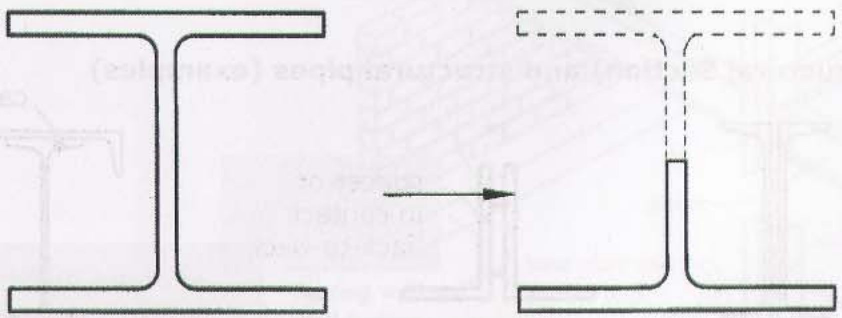
MC-shape (example)
(Miscellaneous Channel)

Unequal legs

Equal legs



Angle shapes (examples)



W 14x90

WT 7x45

WT-shape (example)

Structural tees are obtained by cutting W-shape, M-shape and S-shape in half to obtain WT-shape, MT-shape and ST-shape, respectively

Fig. Various shapes of structural steel

(3) Various Shapes of Sections *contd.*

Plates, Bars, Hollow and Built-up Sections

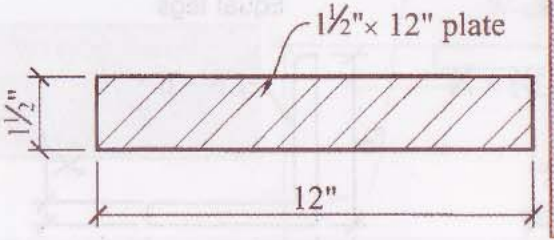
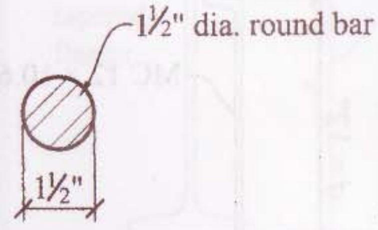
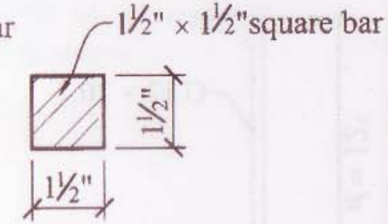


Plate (example)

Previously, widths ≤ 8" was referred to as *bars*; now, all flat stocks are referred to as *plates*



bar (example)



bar (example)

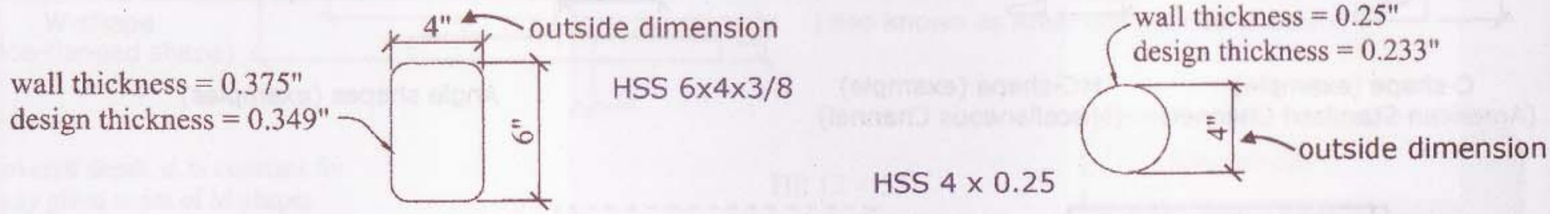


Fig. HSS(Hollow Structural Section) and structural pipes (examples)

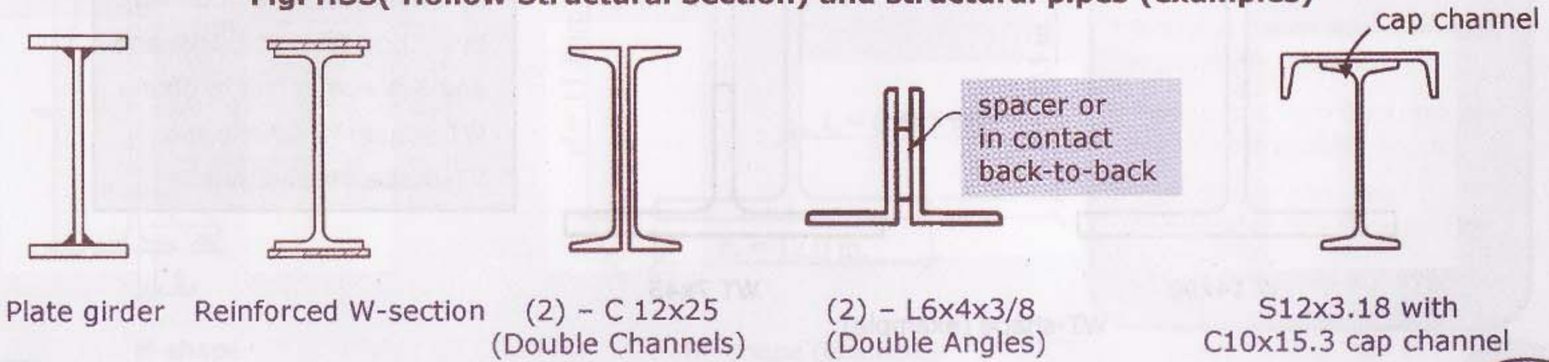


Fig. Built-up sections (examples)

(4) Basic Structural Steel Elements

Girders are typically connected to column flanges, while beams are usually connected to the web of the column.

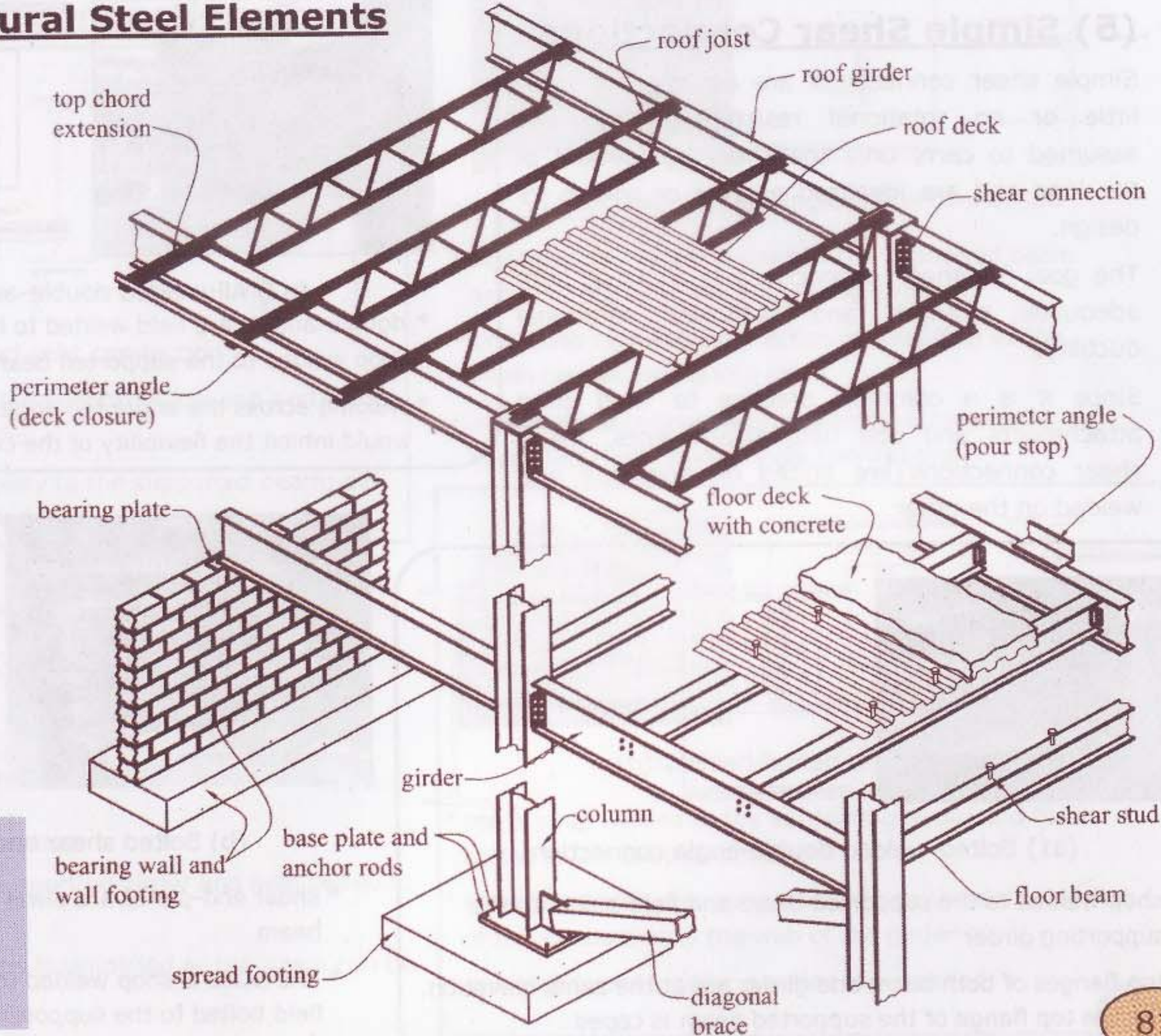


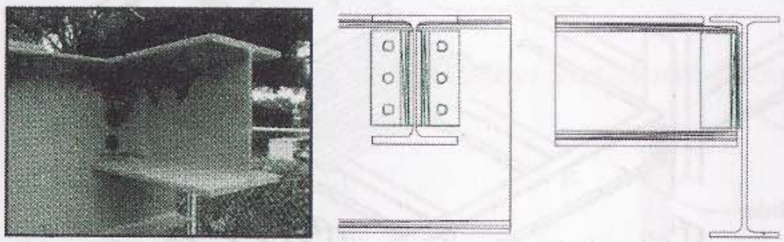
Fig. Typical steel building — basic structural elements (3-D)

(5) Simple Shear Connections

Simple shear connections are assumed to have little or no rotational resistance. They are assumed to carry only the shear component of the load and are idealized as pins or rollers for design.

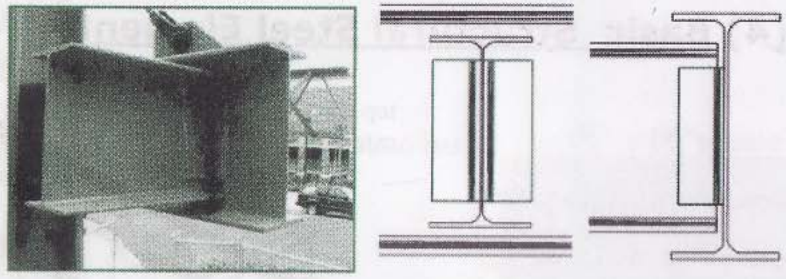
The goal for shear connections is to have both adequate strength and sufficient rotational ductility.

Since it is a common practice to weld shop attachments and bolt field attachments, many shear connections are bolted on one side and welded on the other.



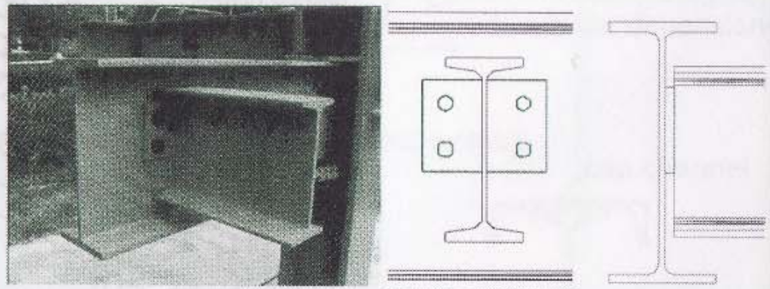
(a1) Bolted-welded double-angle connection

* shop welded to the supported beam and field bolted to the supporting girder
* top flanges of both beam and girder are at the same elevation. So, the top flange of the supported beam is coped



(a2) All-welded double-angle connection

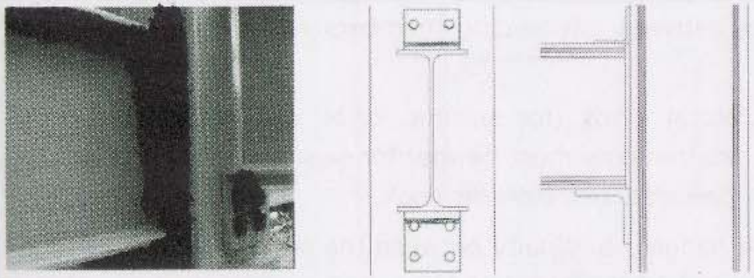
* double-angles are field welded to the supporting girder and shop welded to the supported beam
* welding across the entire top edge should be avoided since it would inhibit the flexibility of the connection



(b) Bolted shear end-plate connection

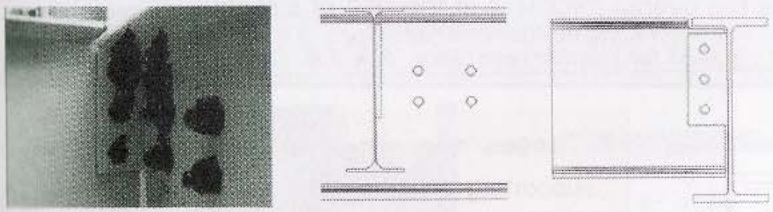
* shear end-plates are always welded to the supported beam
* end-plate is shop welded to the supported beam and field bolted to the supporting girder

(5) Simple Shear Connections *contd.*



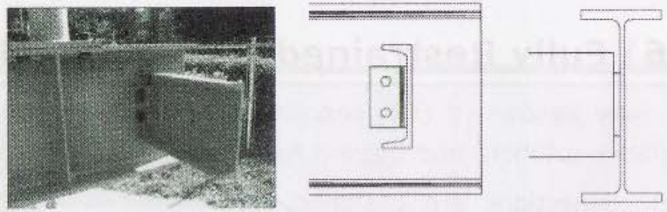
(c) Bolted unstiffened seat connection

* seat angle is field bolted to the supporting column and shop welded to the supported girder
* top angle only provides stability to the supported beam; all shear is assumed to be carried by the seat angle



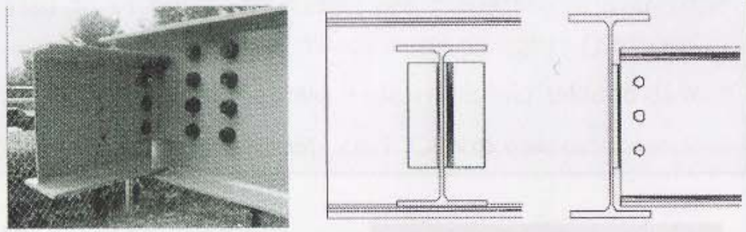
(d) Single-plate connection

* plate is shop welded to the supporting girder and field bolted to the supported beam
* one-sided connection; erection is simplified as the beam can be swung into place



(e) Bolted-welded single-angle connection

* single-angle is shop welded to the supported beam and field bolted to the supporting girder
* one sided connection; erection is simplified as the beam can be swung into place
* single-angle connections tend to have lower load capacities than double-angle connections



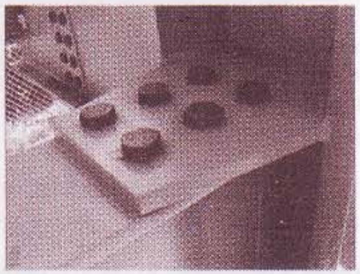
(h) Welded-bolted tee connection

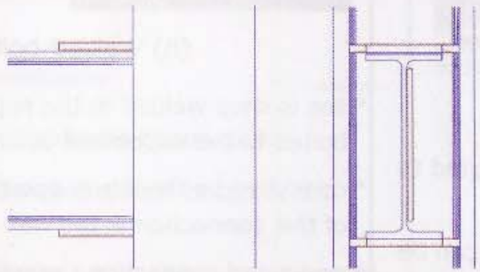
* tee is shop welded to the supporting girder and field bolted to the supported beam
* considered a flexible support condition since the support of this connection is the web of the girder
* one-sided connection ; erection is simplified

(6) Fully Restrained (FR) Moment Connections (Green et al.)

* A Fully Restrained (FR) connection assumes that the angles between intersecting members are maintained (i.e. no relative rotation) and there is full transfer of the moments.
* FR connections are designed to carry both gravity and lateral loads (for seismic loads with seismic response modification factor $R \leq 3.0$. For $R > 3.0$, additional design requirements must be met for seismic loads. In that case refer to AISC *Seismic Provisions for Structural Steel Buildings*. See also Sec. 8 of this book.
* To transfer the tension and compression forces carried by the flanges, continuity between the supported beam flanges and the supporting member must be realized. Hence, the flanges of the supported member are attached to either a connection element or directly to the supporting member.
* Moment connections also normally include a simple shear connection at the web of the supported member to carry the shear component of the beam reaction.
* Transverse stiffeners are plates fabricated to fit between the flanges of the column at the point(s) of concentrated loading (tension or compression).
* Web doubler plates are steel plates that are used to increase the overall thickness of the web of a section.

Note : according to ASCE 7-05 , for steel systems not specifically detailed for seismic resistance, R = 3.0.

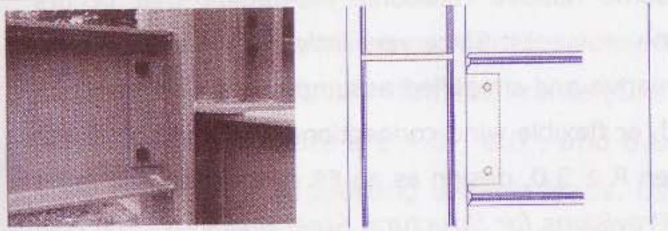




(a) Bolted flange-plates FR connection

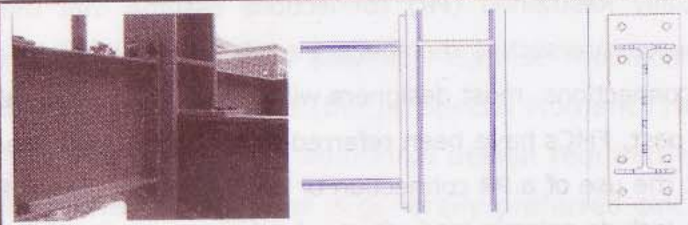
* flange-plates are shop welded to the supporting column and field bolted to the supported girder
* plates attached to the flanges of the girder are for transfer of the moment forces
* plate attached to the web of the girder is for transfer of the shear force

(6) Fully Restrained (FR) Moment Connections *contd.*



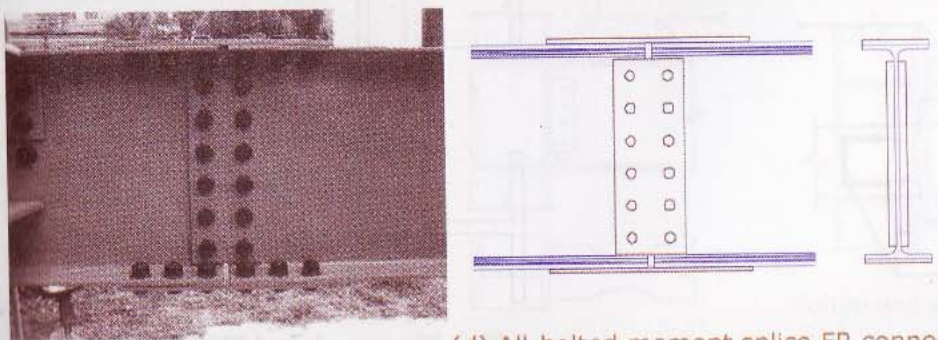
(b) Directly-welded-flanges FR connection

* a transverse stiffener is attached between the flanges of the support column. The plate is aligned to receive the concentrated force (tension or compression) from the girder flange. For all FR and PR Column Connections, column stiffening should be investigated to ensure that the connection flange forces do not exceed applicable limit states
* plate attached to the web of the girder is designed for shear
* effects of eccentricity in the shear connection are neglected



(c) Extended end-plate FR connection

* a transverse stiffener is attached between the flanges of the support column. The plate is aligned to receive the concentrated force (tension or compression) from the girder flange. For all FR and PR Column Connections, column stiffening must be investigated to ensure that the connection flange forces do not exceed applicable limit states



* plates attached to the flanges of the girders are designed for moment transfer
* plates attached to the webs of the girders are designed for shear transfer

(d) All-bolted moment splice FR connection

(7) Partially-Restrained (PR) and Flexible Moment Connections (FMC)

Partially Restrained (PR) connections assume that there will be some relative rotational movement that occurs between intersecting members, though there will still be transfer of the moments. Since very little data is available for PR connections, most designers will use an FMC, which allows conservative and simplified assumptions to be made. In the past, FMCs have been referred to as Type 2 with wind, semi-rigid, or flexible wind connections. It should be noted that the use of a PR connection or an FMC requires that R ≤ 3.0. When R ≥ 3.0, design as an FR connection and must also include seismic load effects. In that case refer to AISC *Seismic Provisions for Structural Steel Buildings*. See also Sec. 8 of this book.

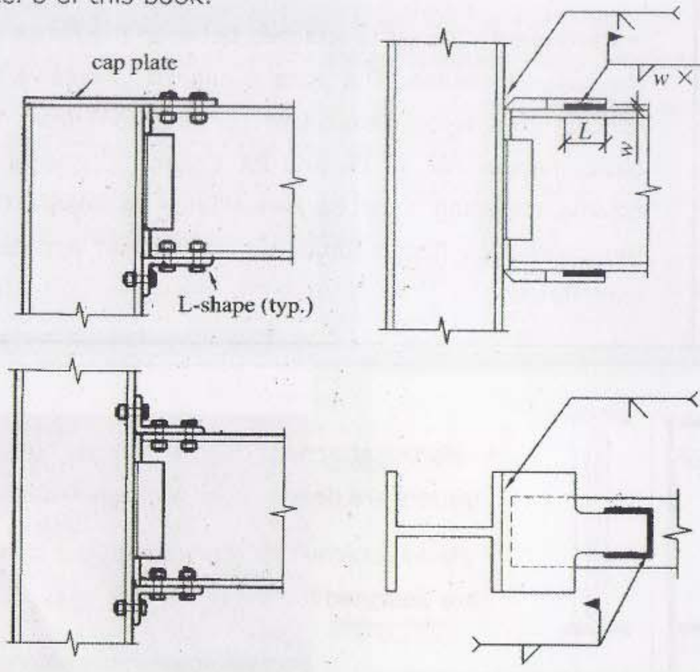


Fig. PR connections

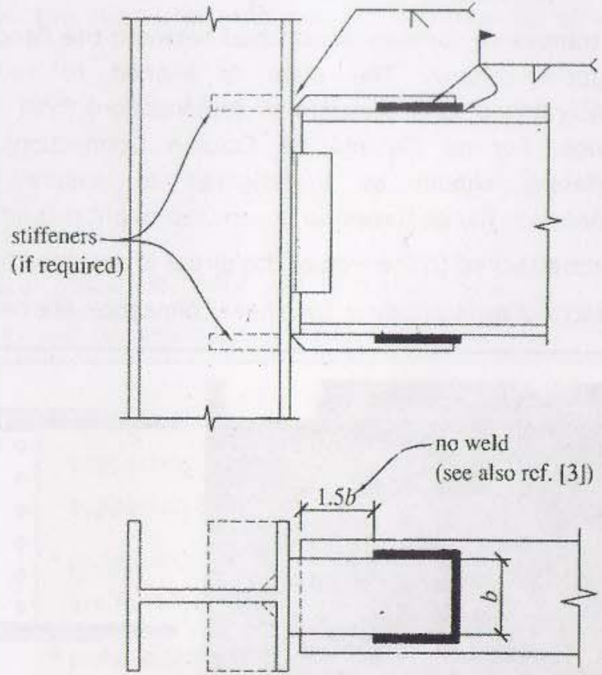


Fig. Flange-plated FMC

(8) Pre-Qualified Fully-Restrained Moment Connections for Seismic Loads

The AISC seismic provisions identify three basic moment frame types with seismic response modification factor R > 3.0; Ordinary Moment Frame (OMF) , Intermediate Moment Frame (IMF) , Special Moment Frame (SMF) . The R factors are 4.0 , 6.0 , and 8.0 , respectively. When R > 3.0 , additional design requirements must be met for supporting seismic loads. Using the pre-qualified connections is generally preferred since a rigorous analysis would be required for other connections that have not been tested.

> *Note*
> *Using a R factor ≤ 3.0 is highly desirable in that the analysis and design procedure is more simplified than a procedure that uses the OMF , IMF , or SMS requirements.*
> *In general, buildings with a Seismic Design Category (SDC)* of A , B , or C can usually be economically designed with R ≤ 3.0.*
> *This approach is recommended where possible.*
>
> **** SDC-C corresponds roughly to UBC Seismic Zone 2***

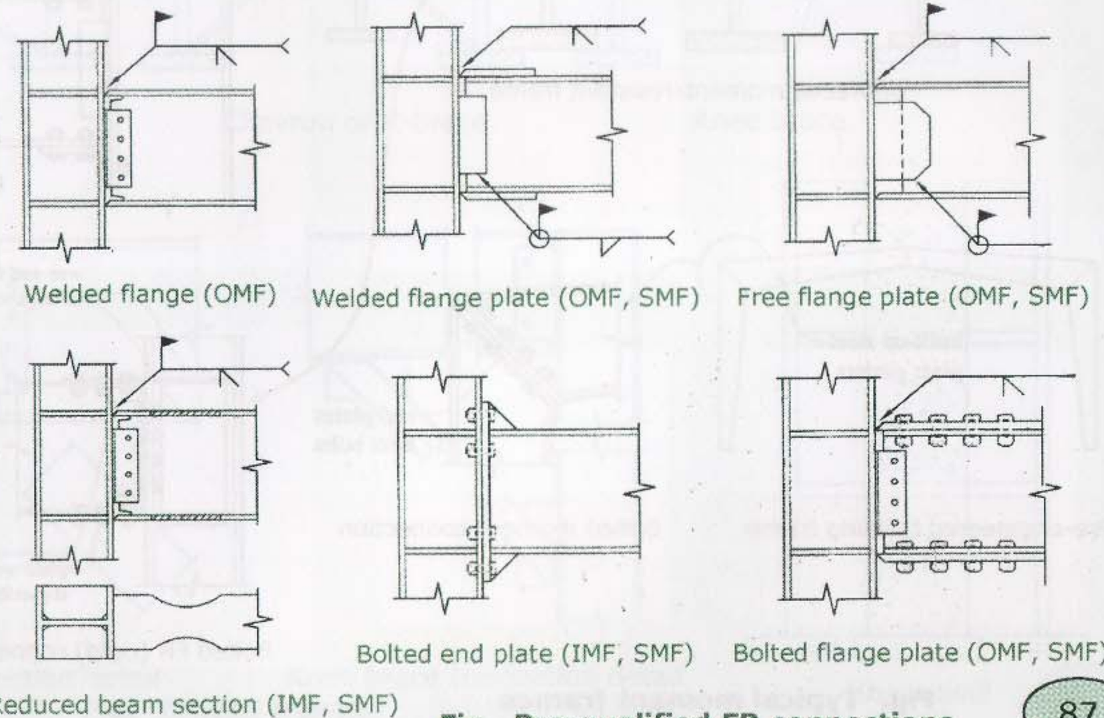


Fig. Pre-qualified FR connections

(9) Miscellaneous Connection Examples

Moment frames

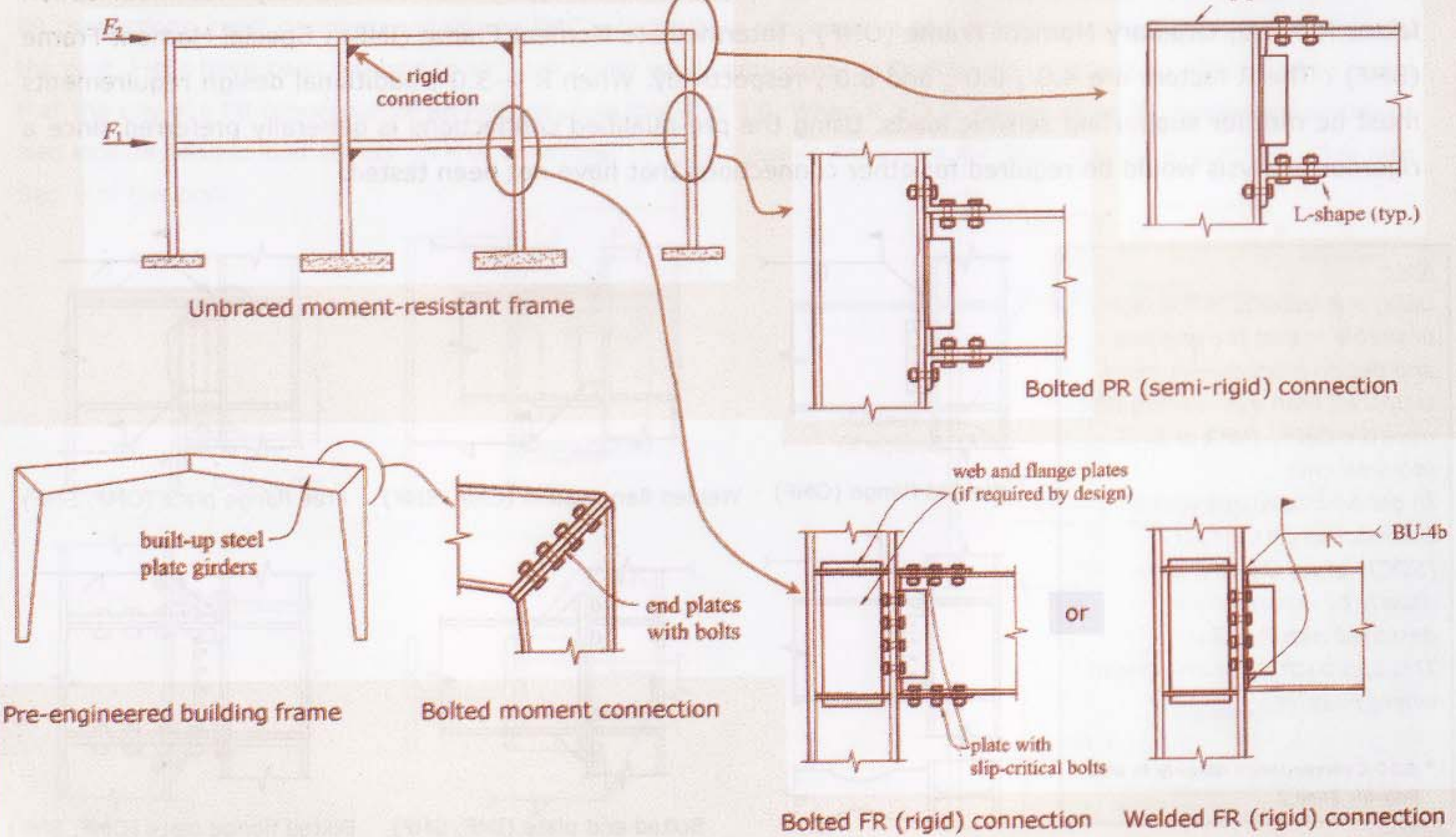
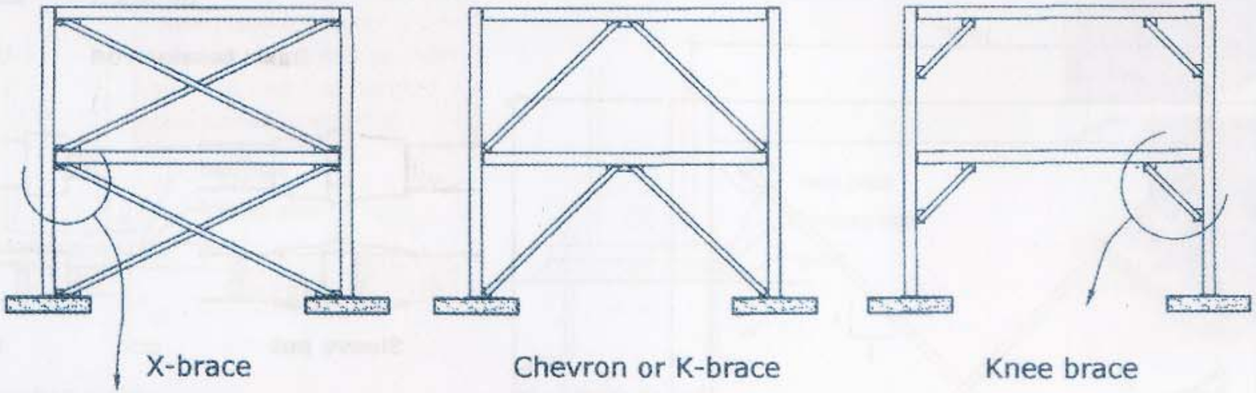


Fig. Typical moment frames

(9) Miscellaneous Connection Examples *contd.*

Braced frames



The beam-to-column connections of braced frames are simple shear connections

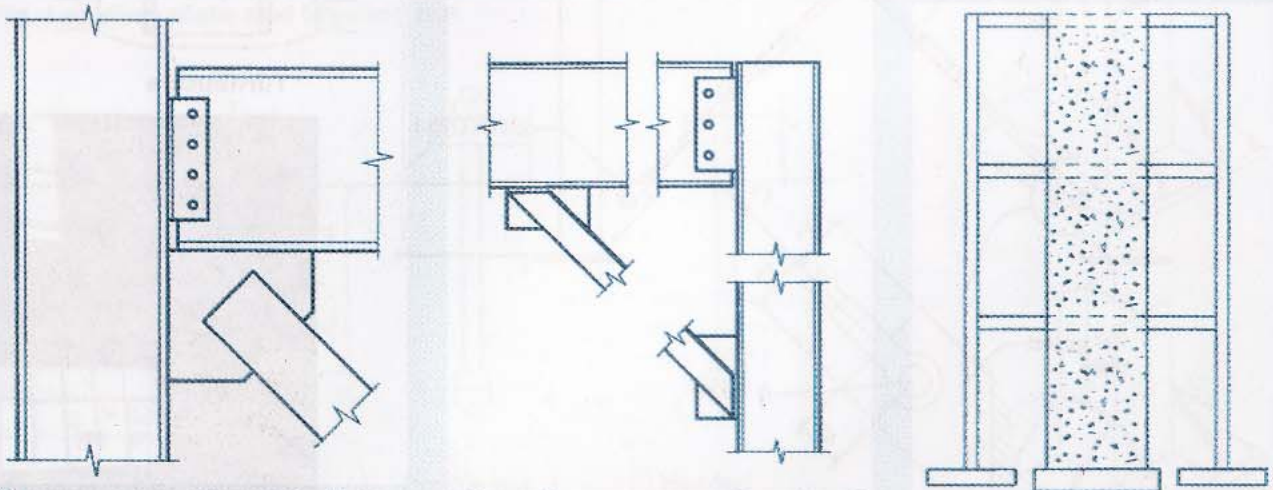
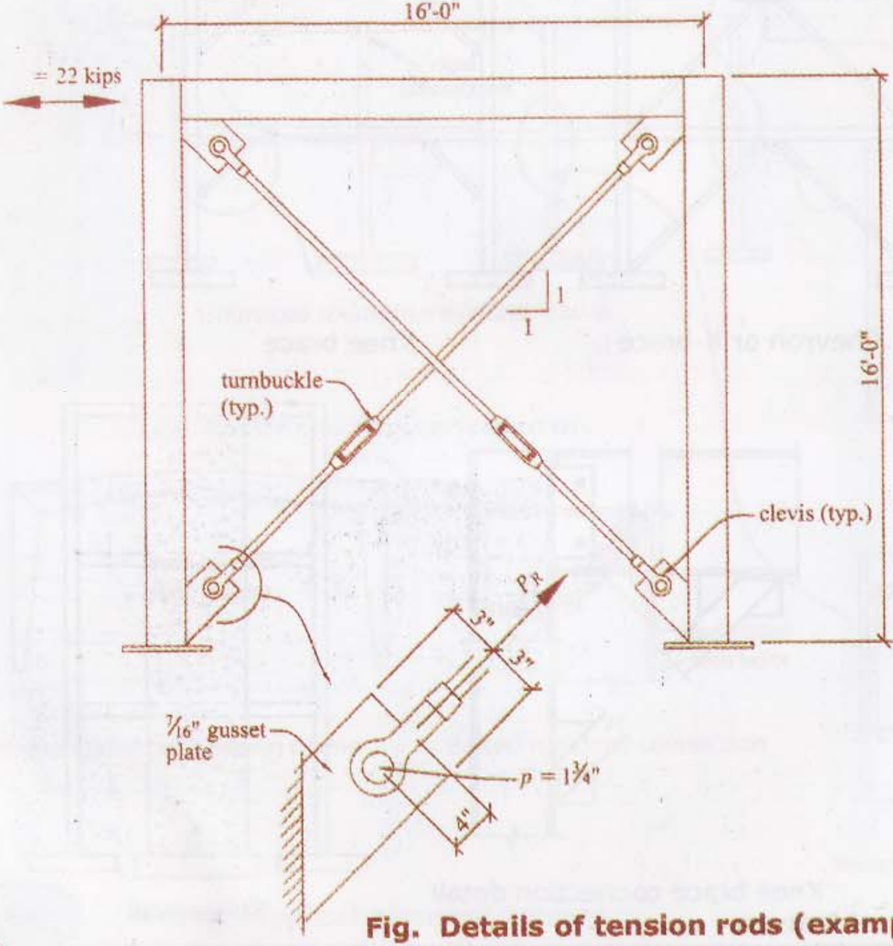
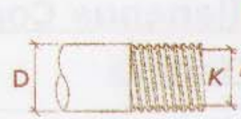


Fig. Braced frames

(9) Miscellaneous Connection Examples *contd.*

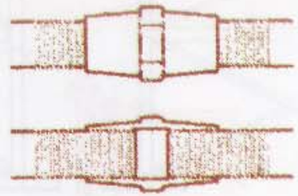
Tension Rods



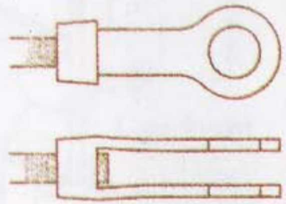


Basic tension rod

Upset end



Sleeve nut



Clevis

Turnbuckle

Fig. Details of tension rods (example)

(9) Miscellaneous Connection Examples *contd.*

Column base plates and anchor rods

Base plate is usually shop-welded to column on both sides of web and flanges. Use non-shrink grout.

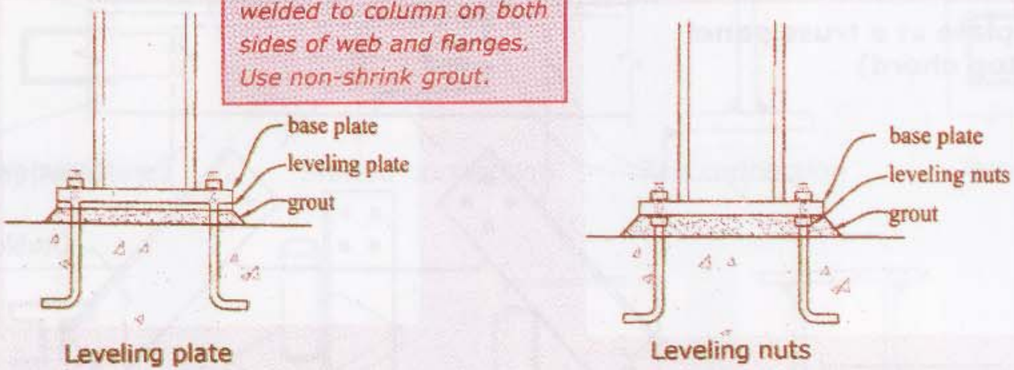


Fig. Leveling plate and leveling nut

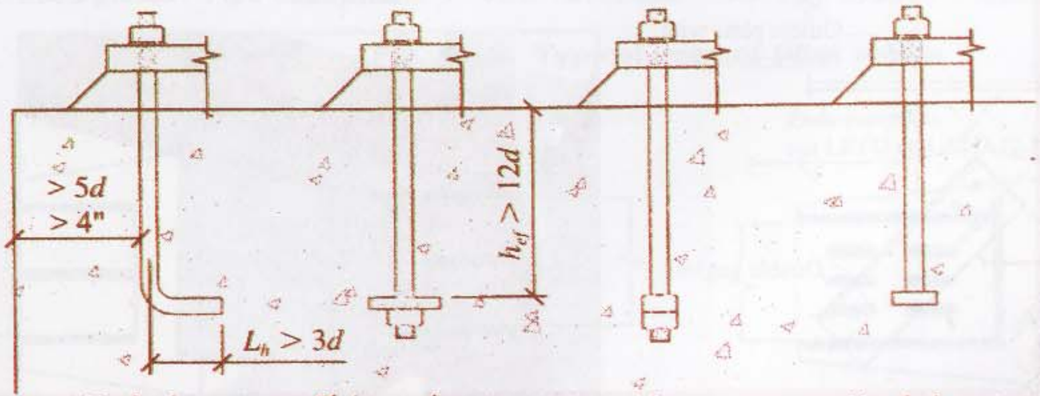
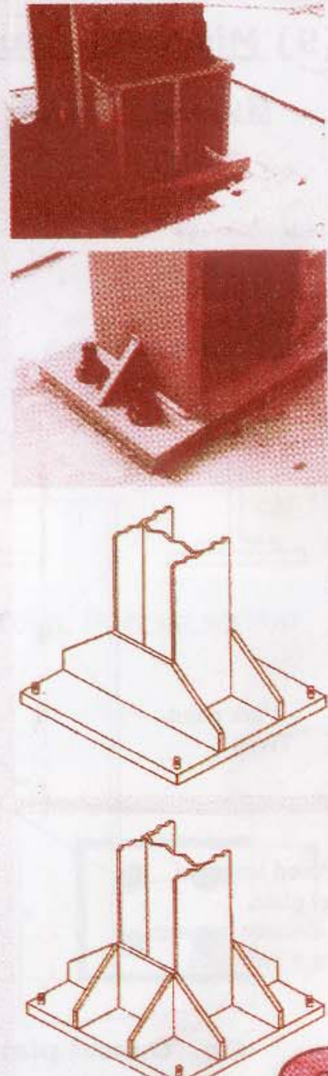


Fig. Types of anchor rods



(9) Miscellaneous Connection Examples *contd.*

Gusset Plates

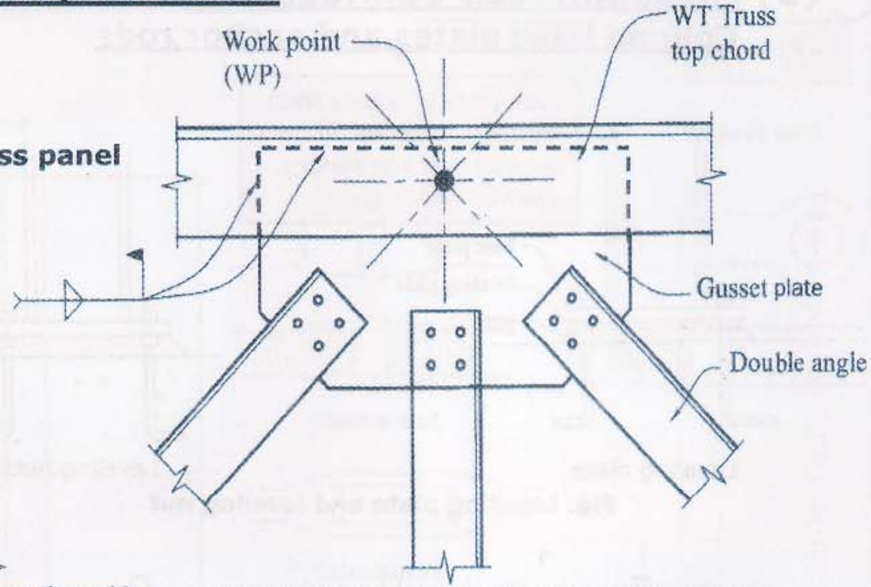


Fig. Gusset plate at a truss panel point (top chord)

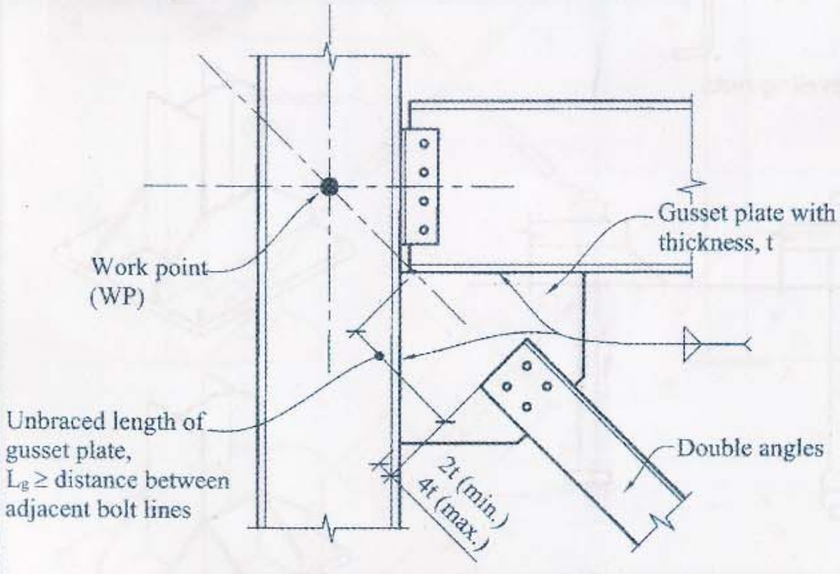
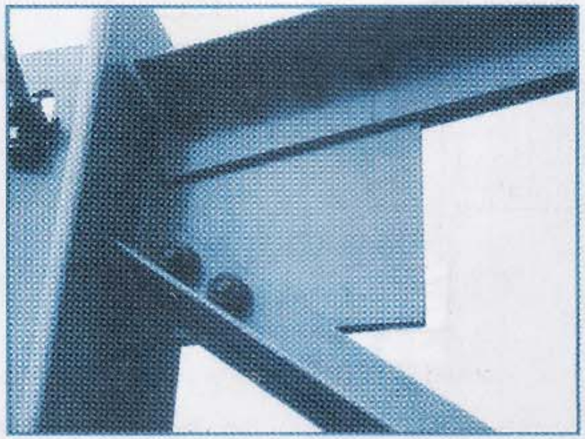
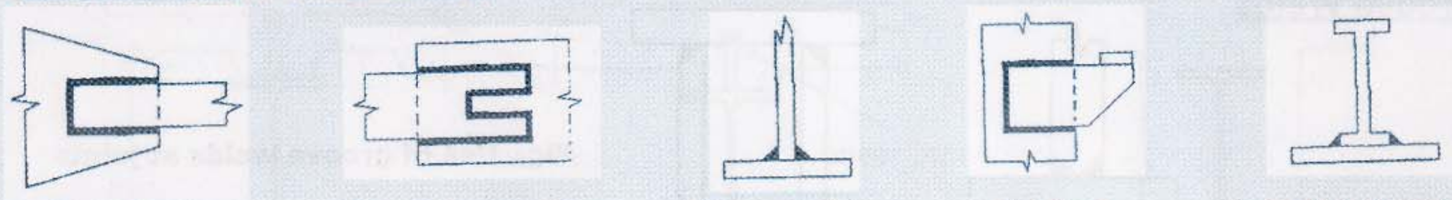


Fig. Gusset plate at a diagonal brace

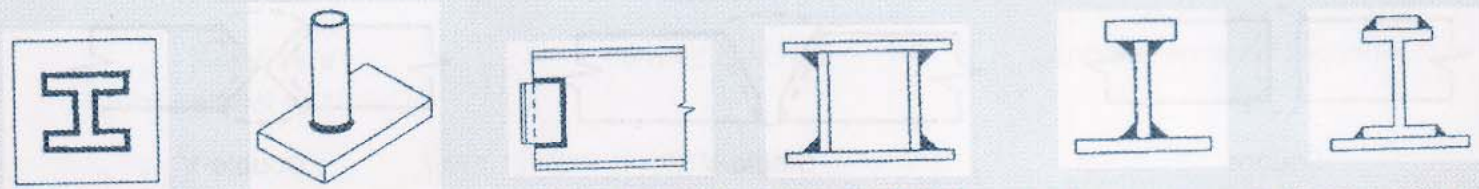


(10) Types of Welds

Fillet , Slot and Plug Welds



Lapped plates | Slotted connection | Tee connection | Brackets | Beam bearing plates



Column base plates | Pipe connection | Beam brackets | Built-up section | Built-up section | Built-up section

Fig. 5.5.4 Typical uses of fillet welds

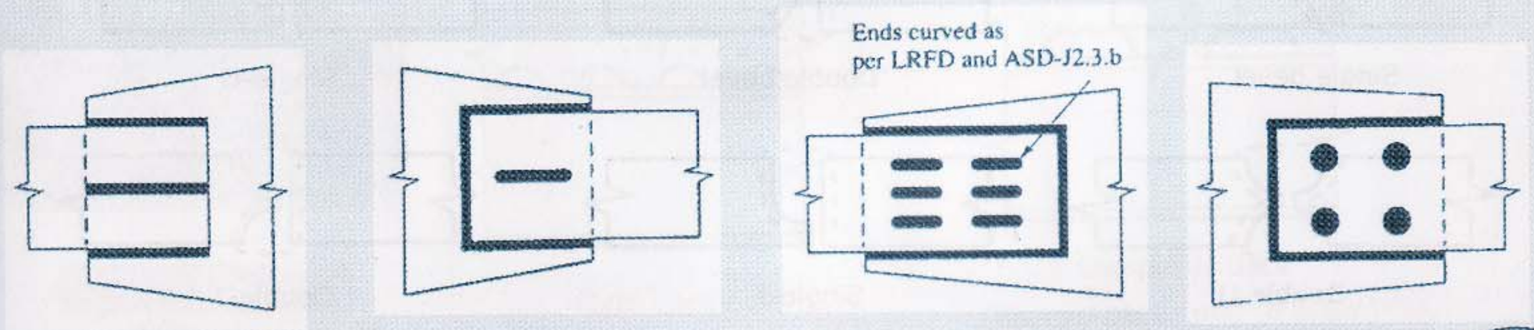


Fig. Slot and plug welds in combination with fillet welds

(10) Types of Welds *contd.*

Groove Welds

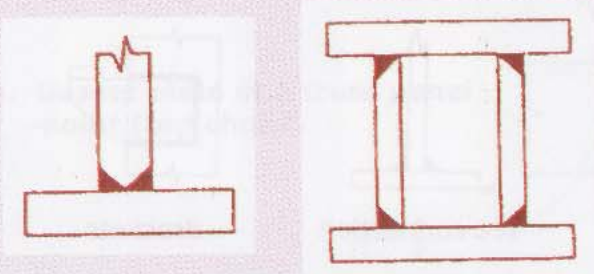


Fig. Use of groove welds at joints

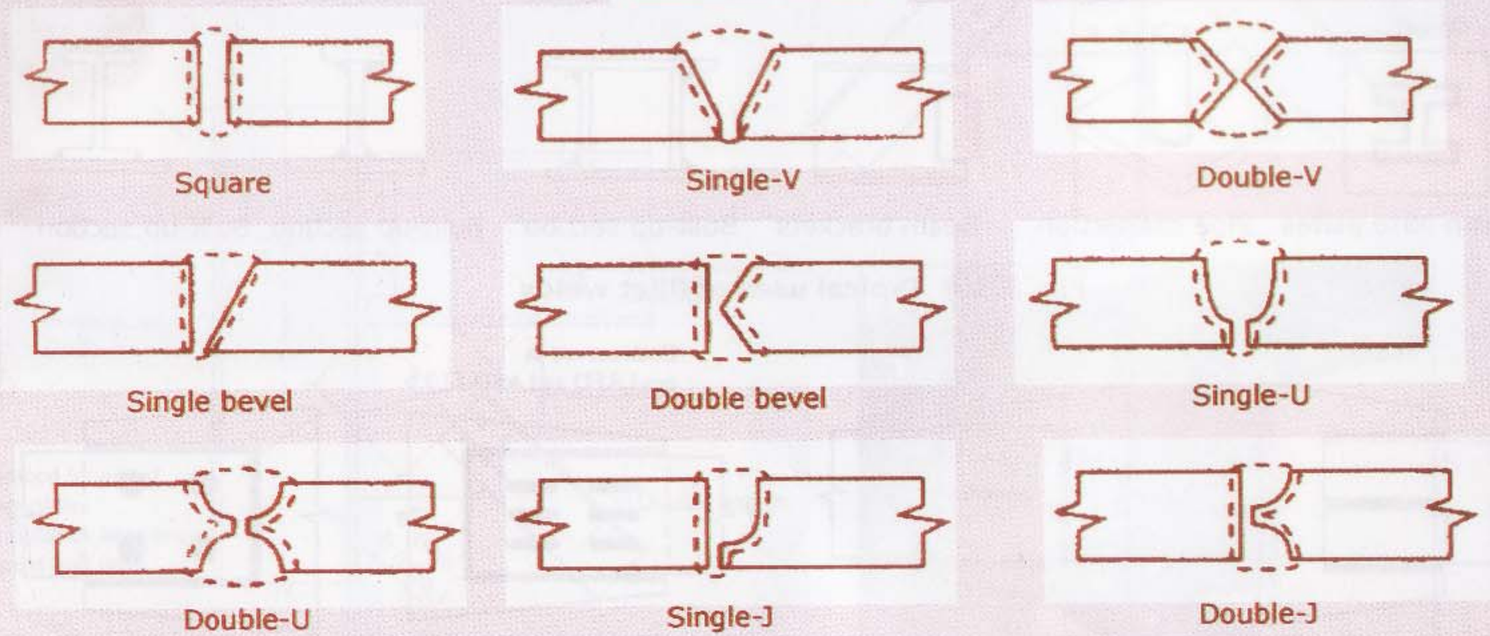
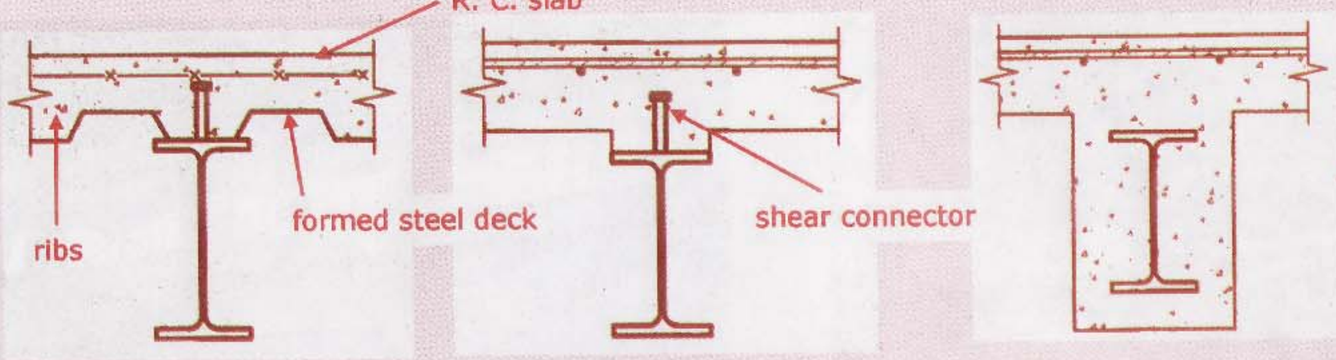


Fig. Types of groove welds

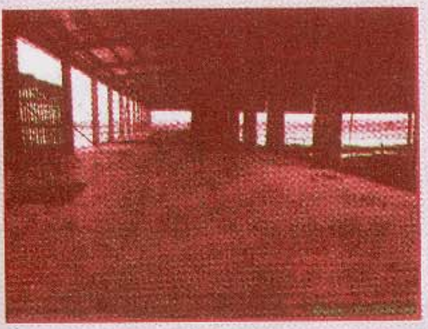
(11) Composite Beams



Metal deck
(Ribs parallel to girder)

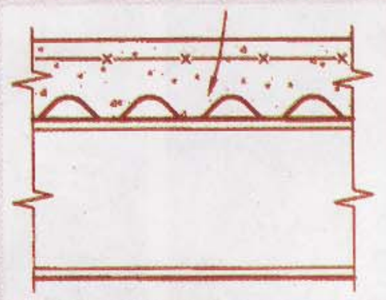
Formed concrete

Concrete-encased section



Metal deck flooring

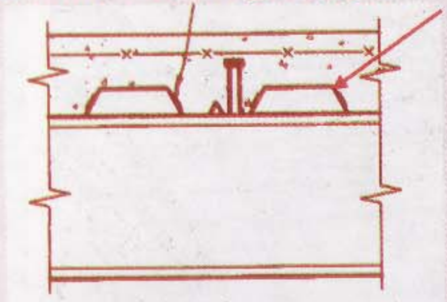
floor system is considered noncomposite for strength and composite for floor vibrations



Form deck

deck ribs are reinforced for additional bending strength

formed steel deck



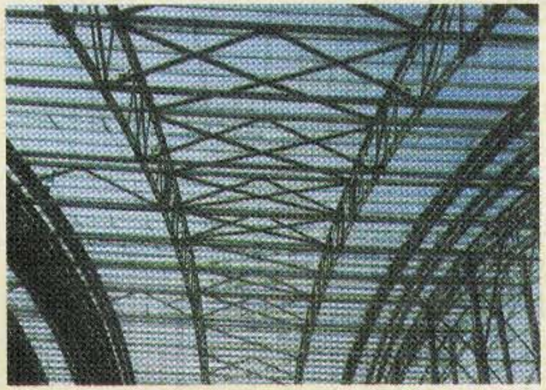
Composite deck
(Ribs perpendicular to beam)

Fig. Types of composite beams

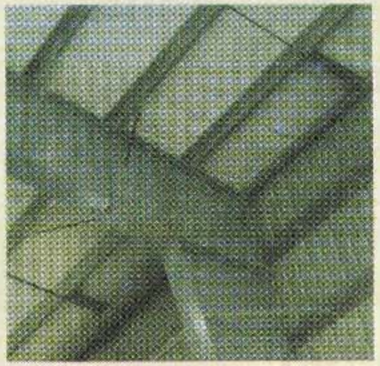
(12) Steel Construction Examples

Faulty column connection

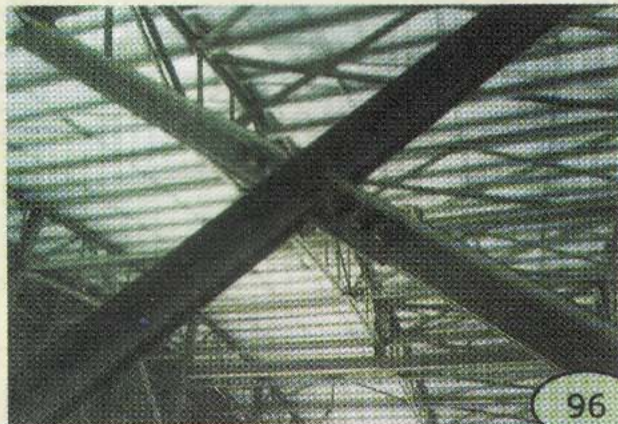
Column diagonal bracing



Roof diagonal bracing



Beam-column joint



SOILS AND FOUNDATIONS

F. SOILS AND FOUNDATIONS

(1) Soil Classification

General Definition of Soils

Coarse – grained soils : if more than 50% of dry weight is retained on No. 200 sieve (= 0.075 mm); they are called cohesionless or non-cohesive soils.

Fine – grained soils : if 50% or more of dry weight passes No. 200 sieve (= 0.075 mm); they are called cohesive soils.

Organic soils : those containing a high natural organic content

* **Coarse – grained fraction** includes gravels and sands with grains coarser than 0.075 mm or No. 200 sieve size.
* **Fine – grained fraction** includes silts and clays with soil grains finer than 0.075 mm.

Grained – size distribution of coarse - grained fraction is generally determined by means of **sieve analysis** and that of fine - grained fraction by means of **hydrometer analysis (ASTM D422).**

(1) Soil Classification *contd.*

Sieve analysis

US Standard sieve size and their sieve openings

US sieve size	Sieve opening (mm)	Sieve opening (in.)
3/4 in.	19.0	0.75
3/8 in.	9.50	0.375
# 4 (3/16 in.)	4.75	0.187
# 10	2.00	0.0787
# 20	0.850	0.0331
# 40	0.425	0.0165
# 60	0.250	0.0098
# 100	0.150	0.0059
# 200	0.075	0.0029

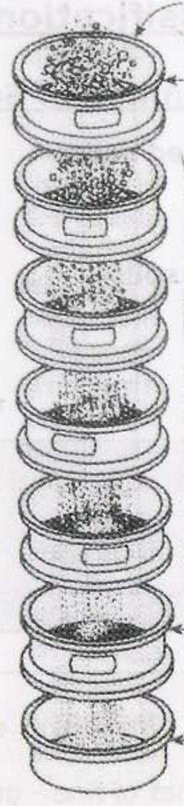


Fig. Sieve analysis (LIU & EVETT)

(1) Soil Classification *contd.*

Clay	< 0.002 mm
Silt	0.002 – 0.075 mm
Sand	0.075 – 2.00 mm
Gravel	2.00 – 75.0 mm
Cobble	75 .0 – 200 mm
Boulder	> 200 mm

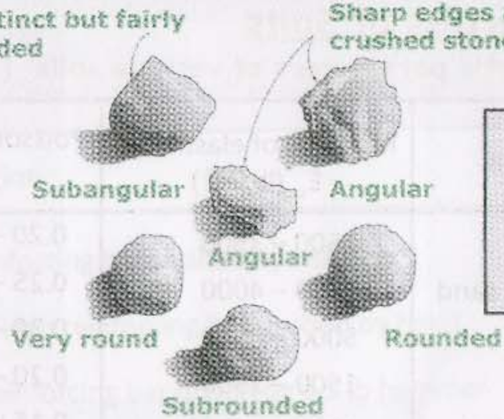


Fig. Shapes of granular soil particles (McCarthy)

Classification based on grain size (LIU & EVETT)

(All grain sizes are in millimeters)

Agency	Coarse-grained				Fine-grained	
	Gravel	Coarse sand	Medium sand	Fine sand	Silt	Clay
AASHTO*	75-2.00 (3 in. – No. 10)	2.00 – 0.425 (No. 10 – No. 40)		0.425 – 0.075 (No. 40 – No. 200)	0.075-0.002	< 0.002
USCS**	Coarse : 75 – 19.0 (3 in. – ¾ in.) Fine : 19.0 – 4.75 (3/4 in. – No. 4)	4.75 – 2.00 (No. 4 – No. 10)	2.00 – 0.425 (No. 10 – No. 40)	0.425 – 0.075 (No. 40 – No. 200)	Fines < 0.075 (Silt or clay)	

*** AASHTO = American Association of State Highway and Transportation Officials. This system is used mainly for highway, and not used in foundation construction.**

**** USCS = Unified Soil Classification System. This system is used in geotechnical work.**

(2) Properties of Soils

Table - Elastic parameters of various soils (DAS)

Type of soil	Modulus of elasticity, E_s (lb/in^2)	Poisson's ratio, μ_s
Loose sand	1500 – 3500	0.20 – 0.40
Medium dense sand	2500 – 4000	0.25 – 0.40
Dense sand	5000 – 8000	0.30 – 0.45
Silty sand	1500 – 2500	0.20 – 0.40
Sand and gravel	10000 – 25000	0.15 – 0.35
Soft clay	600 – 3000	–
Medium clay	3000 – 6000	0.20 – 0.50
Stiff clay	6000 – 14000	–

Table - Coefficients of subgrade reaction, k_1 (DAS)

Type of soil		Modulus of subgrade reaction, k_1 (lb/in^3)
Sand (dry or moist)	Loose	29 – 92
	Medium	91 – 460
	Dense	460 - 1380
Sand (saturated)	Loose	38 –55
	Medium	128 – 147
	Dense	478 - 552
Clay	Loose	44 – 92
	Medium	92 – 184
	Dense	> 184

Table - Approximate consistency classification of cohesive soils (LIU & EVETT)

Consistency	Field identification (CGS)	SPT - N	q_u (kN/m^2)
Very soft	Easily penetrated several centimeters by the fist	0 – 2	0 – 25
Soft	Easily penetrated several centimeters by the thumb	2 – 4	25 – 50
Medium	Can be penetrated several centimeters by the thumb with moderate effort	4 – 8	50 – 100
Stiff	Readily indented by the thumb but penetrated only with great effort	8 – 15	100 – 200
Very stiff	Readily indented by the thumbnail	15 – 30	200 – 400
Hard	Indented with difficulty by the thumbnail	> 30	> 400

CGS = Canadian Geotechnical Society ; q_u = Unconfined compressive strength (ultimate bearing capacity)

(2) Properties of Soils *contd.*

Table - Approximate textural classification of sands (FRENCH)

Textural	Field identification
Very loose	Easily penetrated by a ½ inch reinforcing bar pushed by hand
Loose	Penetrated with difficulty by a ½ inch reinforcing bar pushed by hand
Medium	Readily penetrated by a ½ inch reinforcing bar driven by a 5 lb hammer
Dense	Penetrated about 1 ft by a ½ inch reinforcing bar driven by a 5 lb hammer
Very dense	Penetrated about 3 inches by a ½ inch reinforcing bar driven by a 5 lb hammer

Table - Compactness condition of sands from SPT (CGS)

Compactness condition	SPT N-Index (blows per 0.3 m)
Very loose	0 – 4
Loose	4 – 10
Medium (Compact)	10 – 30
Dense	30 – 50
Very dense	> 50

Table - Relation between N values and angle of friction Ø for sands (DAS)

N	Ø
0 – 5	26 – 30
5 – 10	28 – 35
10 – 30	35 – 42
30 – 50	38 – 46

(3) Boreholes

Borehole Depth

According to Canadian practice, minimum borehole depth beneath the lowest part of the foundation generally should not be less than 6 m, unless bedrock or dense soil is encountered at a shallower depth (CGS).

According to CQHP guidelines, for shallow foundations, the depth of borehole should be at least 1.5 times the lesser dimension of the footing, but not less than 30 ft. For deep foundation, the equation $z = 15\ S^{0.7}$ (ft) should be used, where S is the number of storeys including the basement. However, boring shall not be terminated until minimum SPT value for the last three consecutive levels is 40.

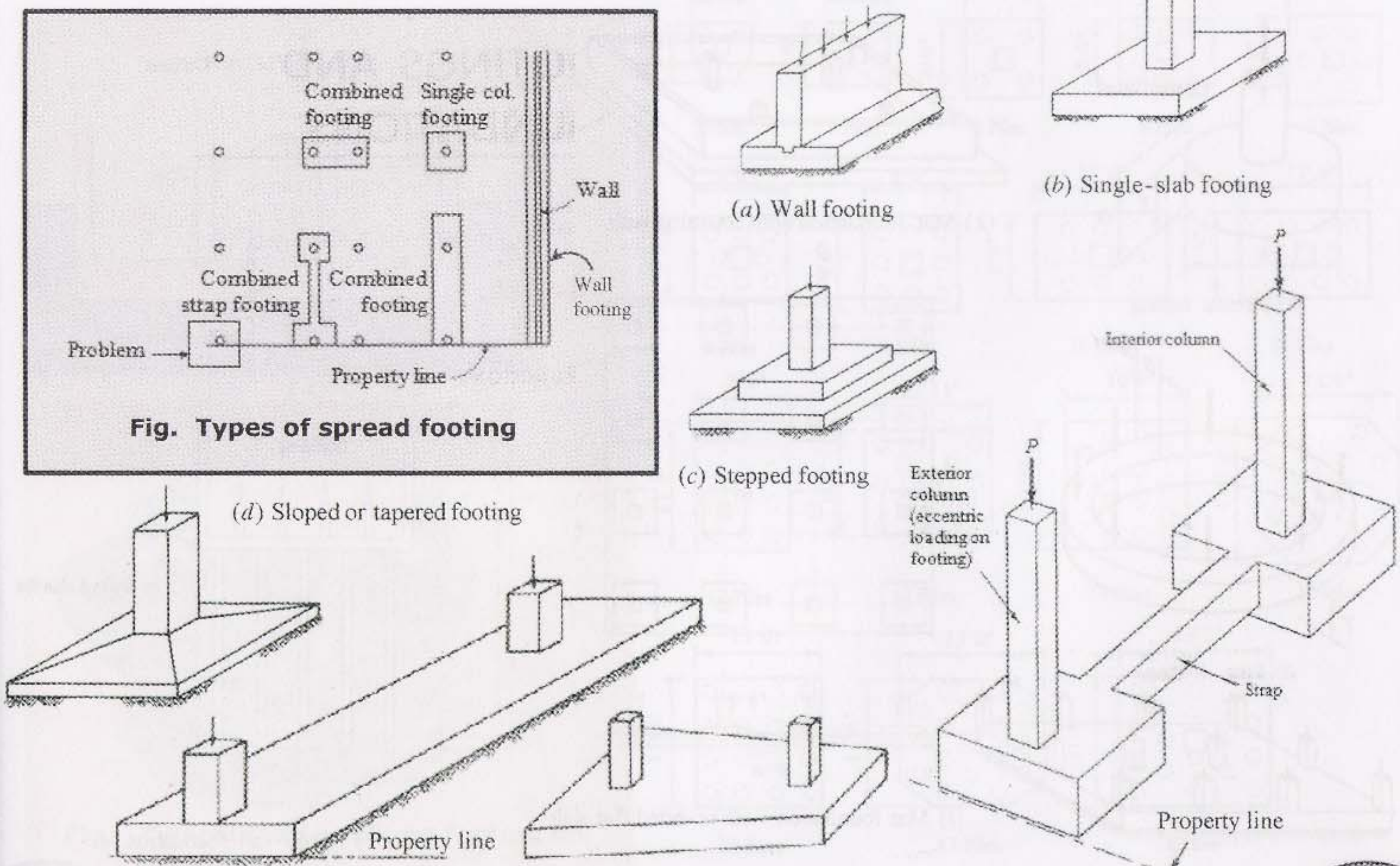
Sowers and Sowers use the following equations for hospitals and office buildings : $z = 10\ S^{0.7}$ for light steel or narrow concrete buildings and $z = 20\ S^{0.7}$ for heavy steel or wide concrete buildings, where z is in ft unit (DAS).

Number of Boreholes

For buildings smaller than about 1000 m^2 in plan area but larger than about 250 m^2, a minimum of four boreholes where the ground surface is level and the first two boreholes indicate regular stratification, may be adequate. Five boreholes are generally preferable (at building corners and centre), and especially if the site is not level. For buildings smaller than about 250 m^2, a minimum of three boreholes may be adequate. A single borehole may be sufficient for a concentrated foundation such as an industrial process tower base in a fixed location. Otherwise, the use of a single borehole for even a small project should be discouraged (CGS).

According to CQHP guidelines, for average soil and terrain conditions, (1) up to 10000 ft^2 projected building area, the number of boreholes shall be one per 2500 ft^2 of projected building area but not less than two holes; (2) for area more than 10000 ft^2, for the first 10000 ft^2, rule (1) is to be followed and for additional area, number of additional holes is one per 5000 ft^2 of building area.

(4) Different Types of Footings and Foundations



(a) Wall footing
(b) Single-slab footing
(c) Stepped footing
(d) Sloped or tapered footing
(e) Combined slab footing (rectangular)
(f) Combined slab footing (trapezoidal)
(g) Combined strap footing

(4) Different Types of Footings and Foundations *contd.*

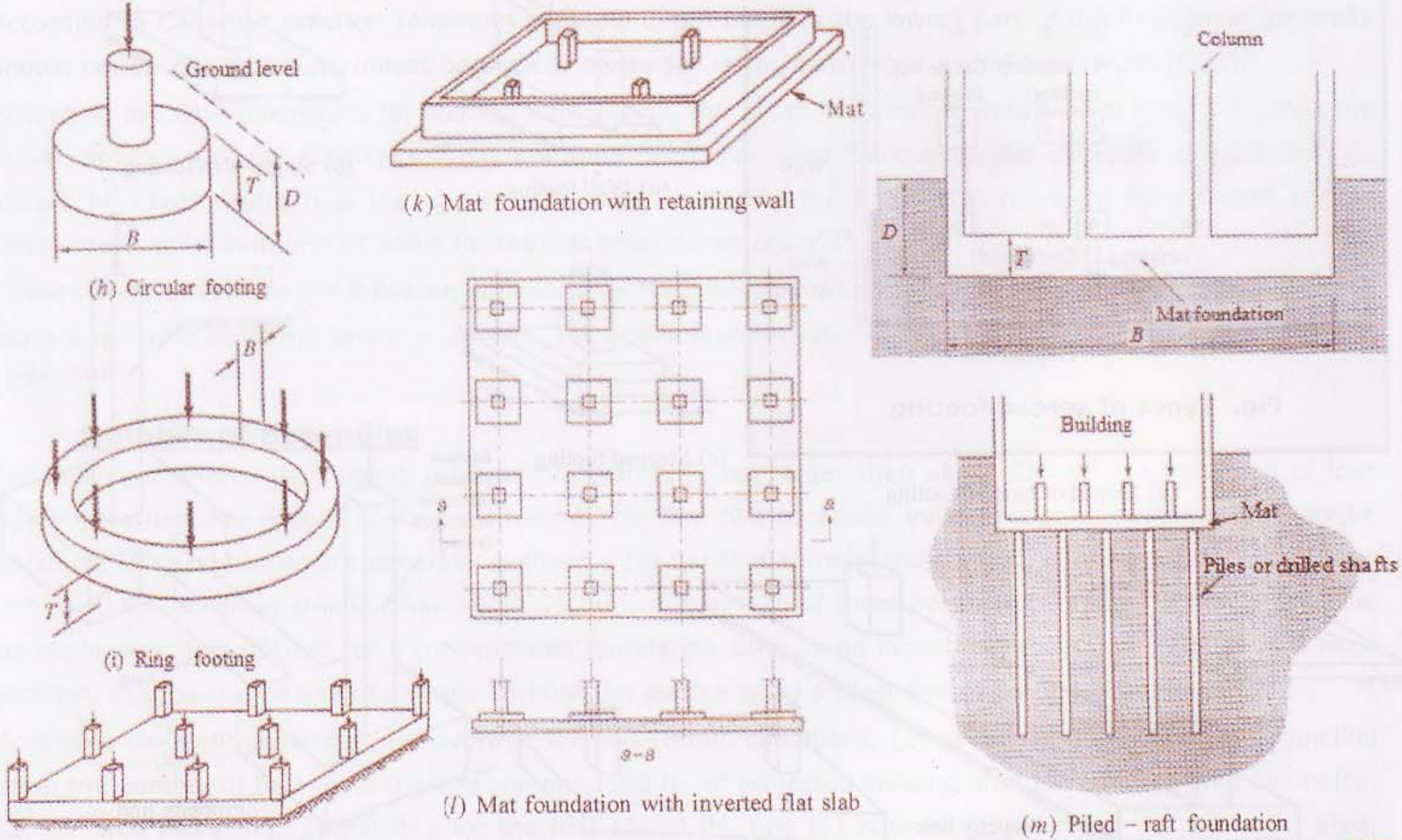


Fig. Different types of footings and foundations

(5) Pile Caps

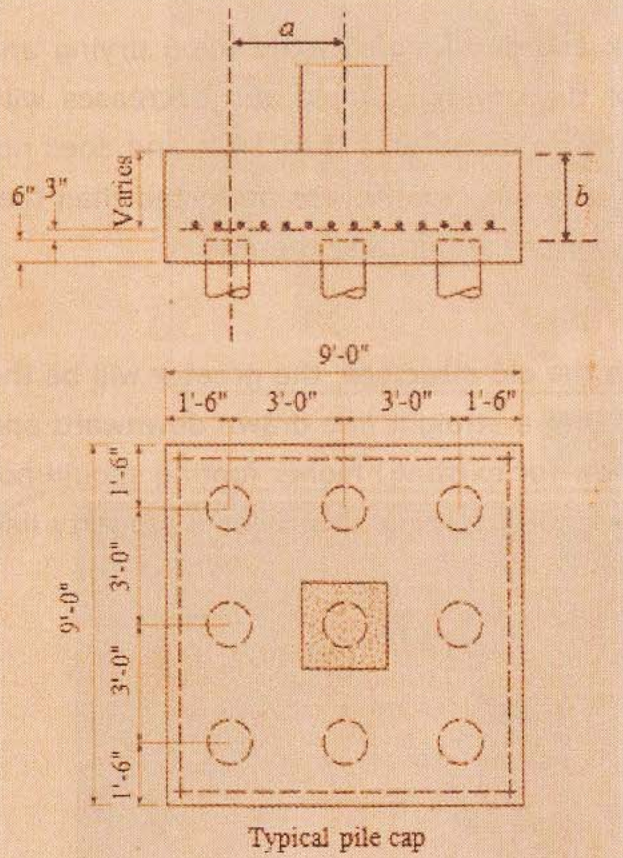


Fig. Typical single-column footing on piles (pile cap)

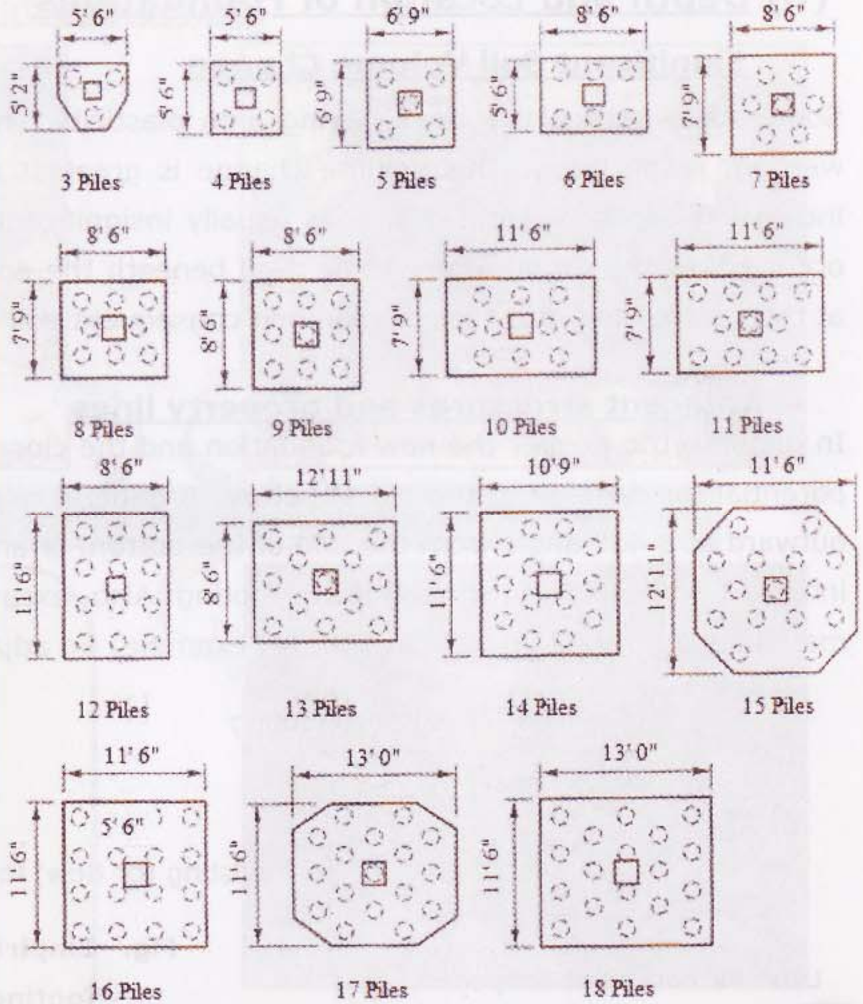


Fig. Typical configurations of pile caps

(6) Depth and Location of Foundations

Significant Soil Volume Change

Some soils, particularly clays having high plasticity, shrink and swell significantly upon drying and wetting, respectively. This volume change is greatest near the ground surface and decreases with increasing depth. Volume change is usually insignificant below a depth of 5 ft to 10 ft and does not occur below the groundwater table. Soil beneath the edges of a structure is less protected than that at the centre, and moisture change and consequent soil movement are greater there.

Adjacent structures and property lines

In general, the deeper the new foundation and the closer to the old structure, the greater will be the potential for damage to the old structure. A general rule is that a straight line drawn downward and outward at a 45° angle from the end of the bottom of any new (or existing) higher footing should not intersect any existing (or new) lower footing. Also, excavation for a footing at or near a property line may have a harmful effect (cave-in, for example) on adjacent land.

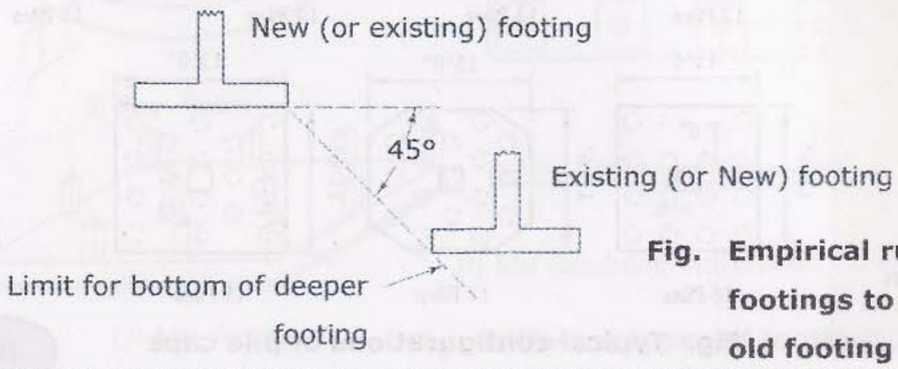
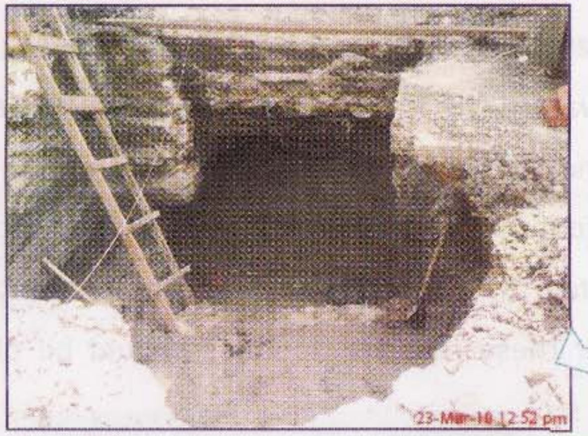


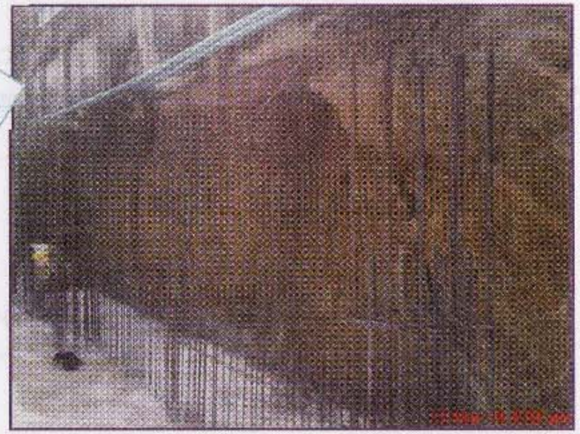
Fig. Empirical rule for the minimum spacing of footings to avoid interference between an old footing and a new footing

(6) Depth and Location of Foundations *contd.*



Caving-in of earth at basement excavations in poor soil

Undermined basement excavations in good soil remain intact for some time



(7) Ground Water (LIU & EVETT)

The presence of groundwater within soil immediately around a footing is undesirable for several reasons. First, footing constructions below groundwater level is difficult and expensive. Generally, the area must be drained prior to construction. Second, groundwater around a footing can reduce the strength of soils by reducing their ability to carry foundation pressures. Third, groundwater around a footing may cause hydrostatic uplift problem, and fourth, if groundwater reaches a structure's lowest floor, waterproofing problems are encountered. For these reasons, footings should be placed above the groundwater level whenever it is practical to do so.

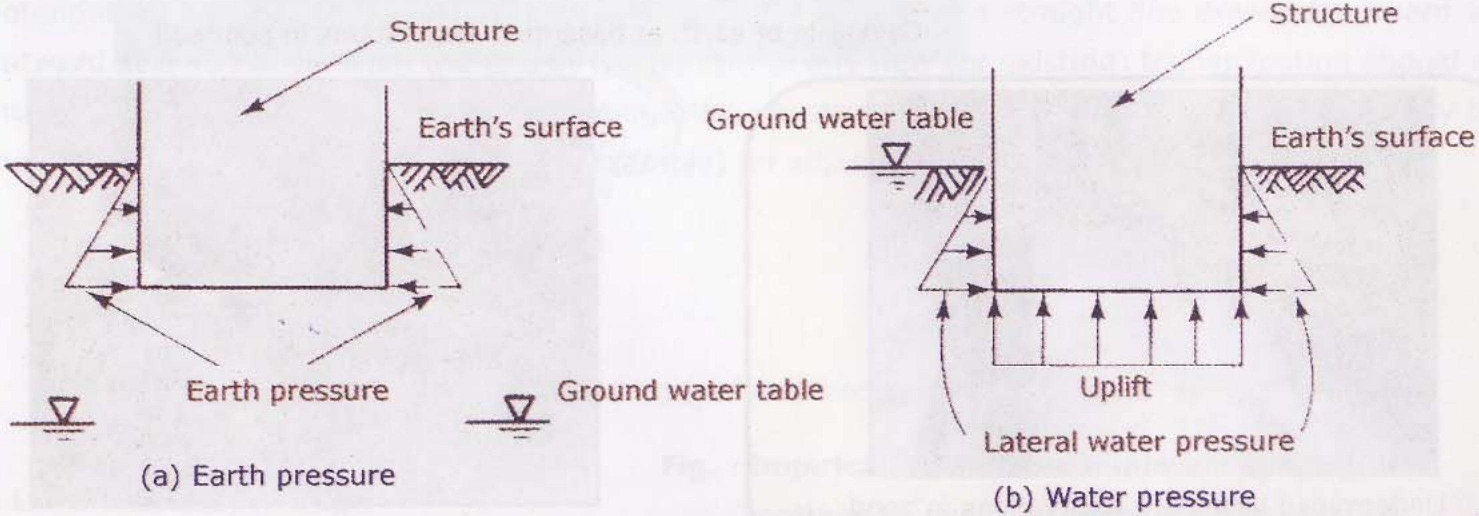


Fig. Earth pressure and uplift pressure due to ground water

(7) Ground Water *contd.*

Unlike clays, sands are severely affected by flooding or by the location of the water table. The strength of a sand is directly proportional to the unit weight. With the loss of roughly half the intergranular pressure when sand is submerged, there is a corresponding loss of half the strength. For design, it is commonly assumed that sand will lose half its strength when submerged. Clays are relatively unaffected by short-term submergence. Only sands are affected by submergence (FRENCH).

Fig. Ground water

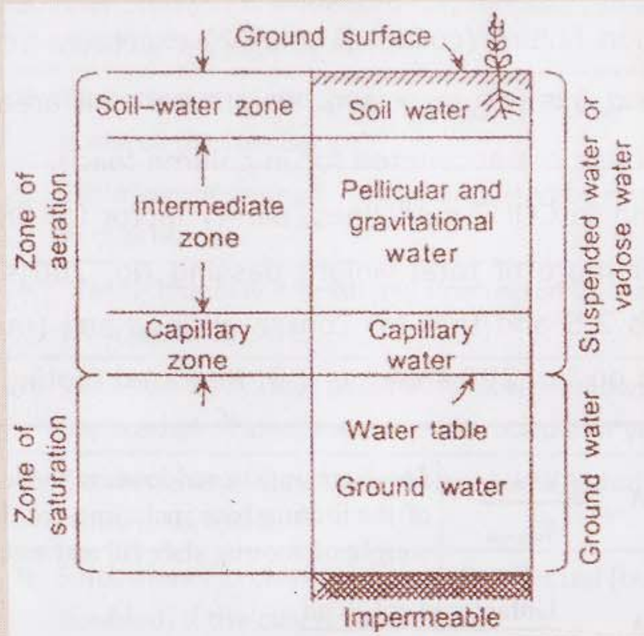


Fig. Zones of surface water (SOM & DAS)

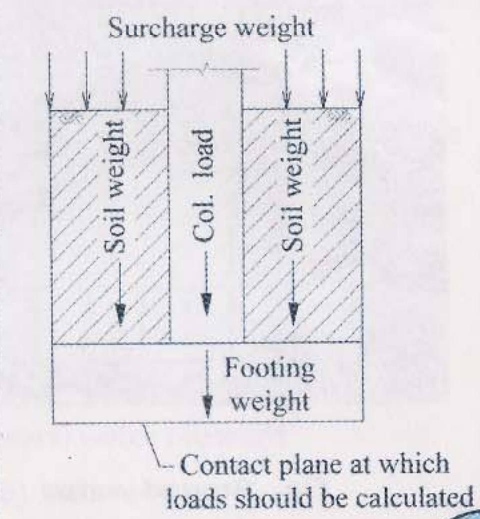
(8) Bearing Capacities and Safety Factor

The " ultimate bearing capacity " (q_{ult}) of a soil refers to the loading per unit area that will just cause shear failure in the soil. The " allowable bearing capacity " (q_a or $q_{a,\ gross}$) refers to the loading per unit area that the soil is able to support without unsafe movement. It is the "design" bearing capacity. The allowable bearing capacity is equal to the ultimate bearing capacity divided by the factor of safety, which is 2.5 to 3.0. Care must be taken to ensure that a footing design is safe with regard to (1) foundation failure (collapse) and (2) excessive settlement. Net or effective allowable bearing capacity $q_{a,\ net}$ or q_e) is ($q_{a,gross}$ - avg. weight per unit area due to concrete slab, earth fill and surcharge above footing base not accounted for in column load).

According to CQHP guidelines, safety factor for fine-grained soil (50% or more of total weight passing No. 200 sieve) is to be taken as 2.5 and that for course-grained soil (more than 50% retained on No. 200 sieve) is 3.0. Required footing area is :

$$A_{reqd} = \frac{D+L}{q_{a,gross}} \quad \left\{ \begin{array}{l} D+L \text{ are unfactored loads at the level} \\ \text{of the footing base including col. load,} \\ \text{weight of footing slab, fill and surcharge} \end{array} \right\}$$

$$\text{or} \quad A_{reqd} = \frac{\text{Unfactored col. load}}{q_{a,net} \ \text{ or } \ q_e}$$



(9) Comparison of Footings on Sands and Clays (FRENCH)

Footings on sands	Footings on clays
1. Sand strength increases with confining pressure, whether due to overburden or to footing loads.	1. Clay strength is relatively constant, regardless of the magnitude of any confining pressure.
2. Sand strength is due entirely to friction, measured by the angle of internal friction ϕ	2. Clay strength is due entirely to cohesion, or tensile strength
3. Sand strength is relatively insensitive to footing shape.	3. Clay strength is influenced considerably (up to 20%) by footing shape.
4. Sand strength increases markedly (doubled or tripled) with depth of burial of the footing.	4. Clay strength is relatively insensitive to depth of burial of the footing.
5. Loss of strength of sand is significant if the overburden (confining pressure) is removed or is eroded away.	5. Little loss of strength of clay is caused by removal of overburden.
6. Strength of a dry sand is cut in half when the sand is submerged in water.	6. Strength of clay is relatively unaffected by short-term submergence.
7. Settlements in sands occur soon after application of load, measured in weeks or a few months.	7. Settlements in clays occur very slowly following application of load, measured in months or years.
8. Settlements in sands can occur under relatively short-term loads.	8. Settlements in clays are relatively unaffected by short-term loads.
9. Settlements in a dry sand are essentially doubled if the sand is submerged (or if the water table rises).	9. Settlements in clays are markedly affected (but not doubled) if the clay is submerged.
10. Deposits of sand are best compacted by vibration and submergence, with some pressure	10. Deposits of clay are best compacted by long-term surcharge pressure

(10) Allowable Bearing Pressure in Sand Based on Settlement Consideration (DAS)

Empirical relations by Meyerhof (modified by Bowles)

$$q_{net\,(all)}\ (\text{k/ft}^2) = \frac{N_{60}}{25} F_d\, S_e \qquad (\text{for B} \le 4\ \text{ft})$$

$$q_{net\,(all)}\ (\text{k/ft}^2) = \frac{N_{60}}{4} \left(\frac{B+1}{B}\right) F_d\, S_e \qquad (\text{for B} > 4\ \text{ft})$$

where F_d = depth factor = $1 + 0.33\ (D_f / B) \le 1.33$; D_f = depth of footing , ft

S_e = tolerable settlement , in.

$$N_{60} = \frac{N\,\eta_H\,\eta_B\,\eta_S\,\eta_R}{60}$$

where N = measured penetration number

η_H = hammer efficiency (%) ;

In USA ; for rope and pulley (Safety hammer) (60%) , (Donut hammer) (45%)

In China ; for free fall (Donut hammer) (60%) , for rope and pulley (Donut hummer) (50%)

η_B = 1 (for diameter 60-120 mm) ; = 1.05 (for diameter 150 mm)

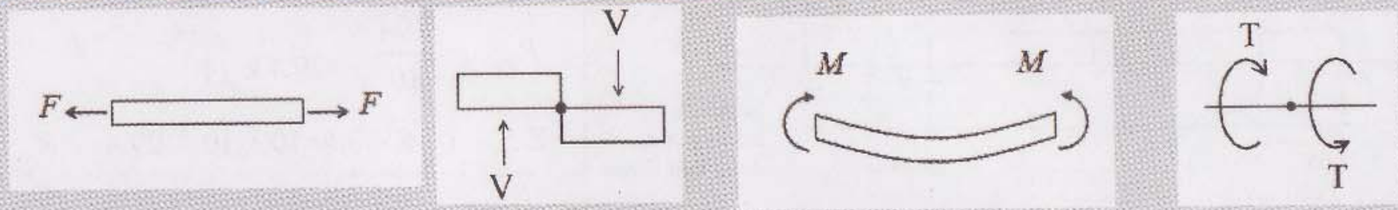
η_S = 1 (for standard sampler) ; = 0.8 (with liner for dense sand and clay)

η_R = 1 (for rod length > 10 m) ; = 0.95 (for length 6-10 m)

STRUCTURAL ANALYSIS FUNDAMENTALS

G. STRUCTURAL ANALYSIS FUNDAMENTALS

(1) Axial Force, Shear Force, Bending Moment and Torsion Diagrams



(+) Axial force (+) Shear force (+) Bending moment (+) Torsion

Fig. Sign conventions

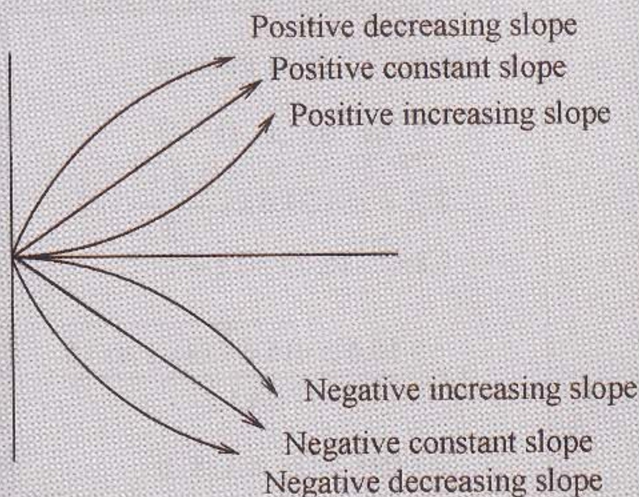
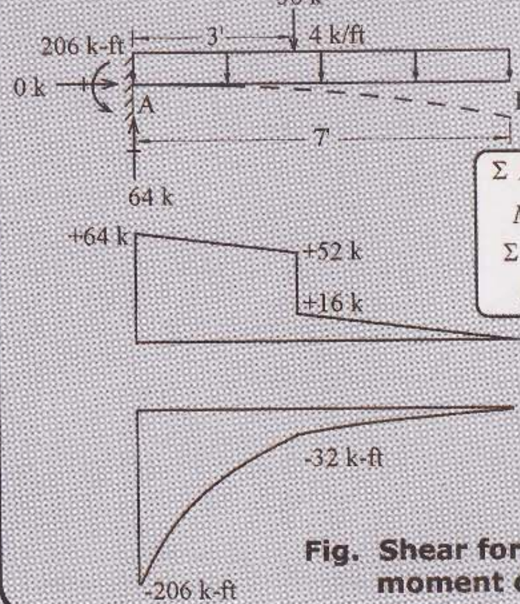


Fig. Various kinds of slope

Example



$\Sigma M_A = 0$; $36\times3 + 4\times7\times3.5 - M_A = 0$

$M_A = 206$ k-ft (↻)

$\Sigma F_y = 0$; $36 + 4\times7 - R_{Ay} = 0$

$R_{Ay} = 64$ k (↑)

Fig. Shear force and bending moment diagrams

(1) Axial Force, Shear Force, Bending Moment and Torsion Diagrams *contd.*

Example

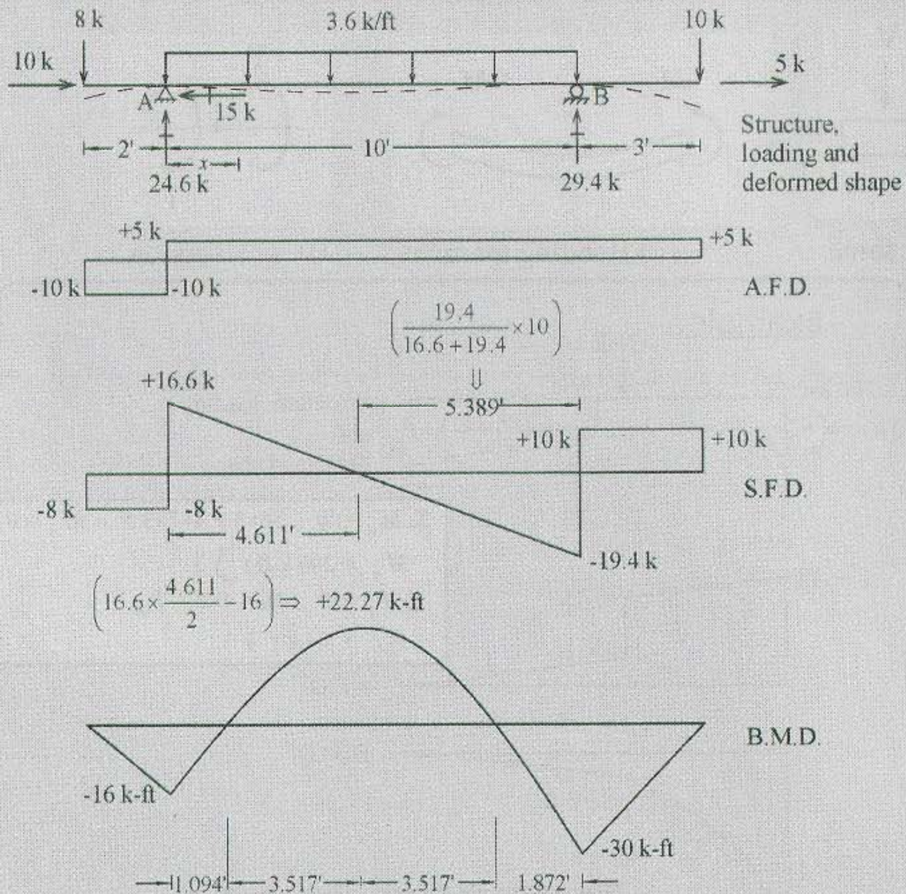


Fig. Axial force, shear force and bending moment diagrams

$$\Sigma M_A = 0\ ;\ 3.6\times10\times5+10\times13-8\times2-R_{By}\times10=0$$

$$R_{By} = \frac{294}{10} = 29.4\text{ k } (\uparrow)$$

$$\Sigma F_y = 0\ ;\ 8+3.6\times10+10-29.4-R_{Ay}=0$$

$$R_{Ay} = 24.6\text{ k } (\uparrow)$$

$$\Sigma F_x = 0\ ;\ 10+5-R_{Ax}=0$$

$$R_{Ax} = 15\text{ k } (\leftarrow)$$

Points of zero moment;

$$\sum M \text{ at } x \text{ ft} = 0$$

$$-8(2+x)+24.6x-\frac{3.6x^2}{2}=0$$

$$1.8x^2-16.6x+16=0$$

$$x=\frac{+16.6\pm\sqrt{16.6^2-4\times1.8\times16}}{2\times1.8}$$

$$=1.094 \text{ ft or } 8.128 \text{ ft}$$

(1) Axial Force, Shear Force, Bending Moment and Torsion Diagrams *contd.*

Example

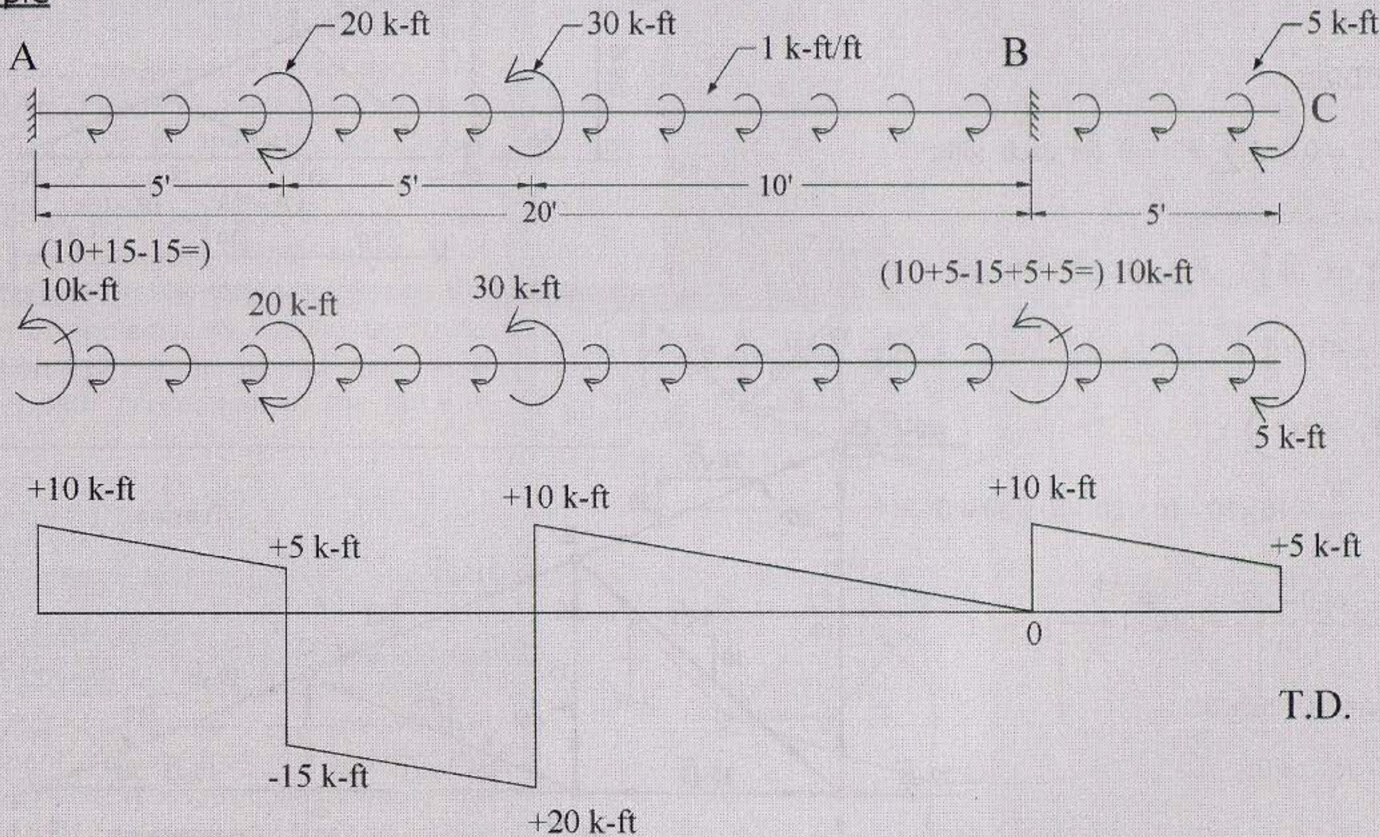
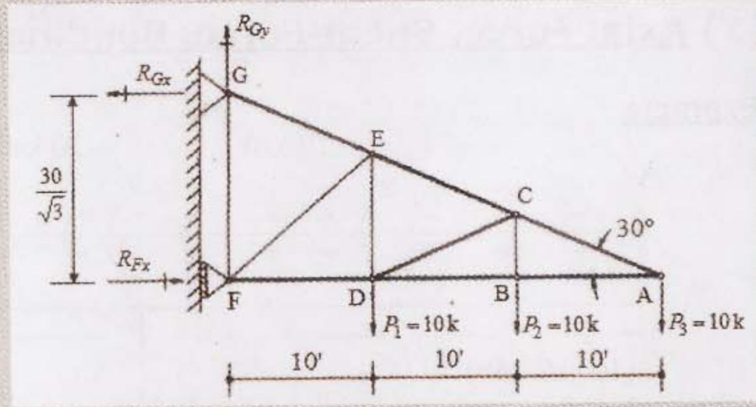


Fig. Torsion diagram

(2) Analysis of Truss

(i) Method of Joints



Example

$\sum F_x = 0$ and $\sum F_y = 0$ for each joint

To find the reactions ,

$\sum F_y = 0\ (+\uparrow);\ R_{Gy} - 10 - 10 - 10 = 0$;

$R_{Gy} = 30\text{ k}\ (\uparrow)$

$\sum M_F = 0\ (+\circlearrowleft)$;

$$R_{Gx} \times \frac{30}{\sqrt{3}} - 10\times10 - 10\times20 - 10\times30 = 0$$

$$R_{Gx} = \frac{600\times\sqrt{3}}{30} = 20\sqrt{3}\ (\leftarrow)$$

$\sum F_x = 0\ (+\rightarrow)$;

$R_{Fx} - R_{Gx} = 0$

$R_{Fx} = R_{Gx} = 20\sqrt{3}\ (\rightarrow)$

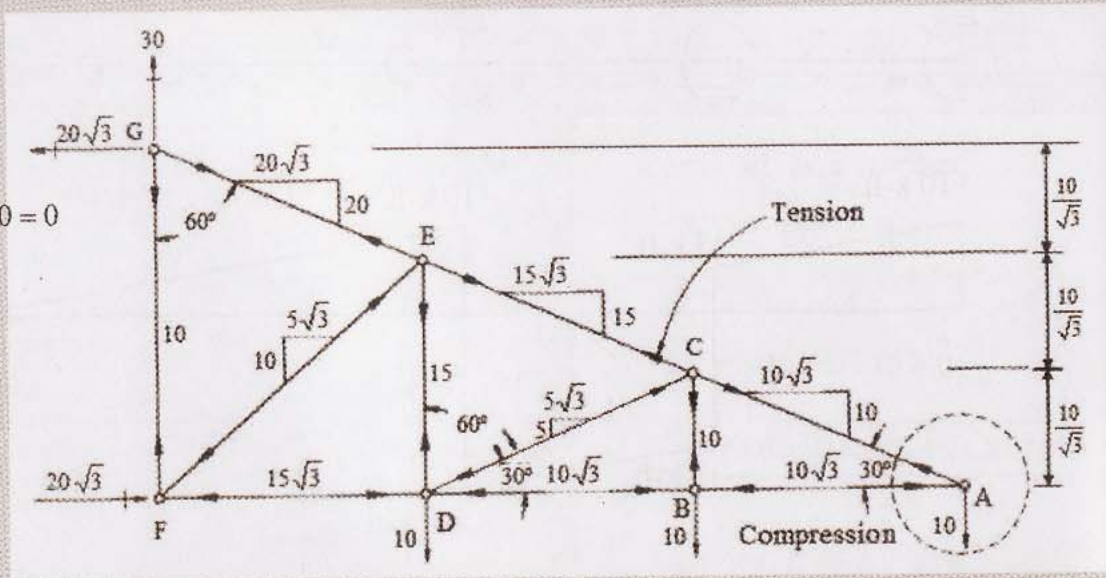


Fig. Truss analysis by method of joints

(2) Analysis of Truss contd.

(ii) Method of Sections

$\sum F_x = 0$, $\sum F_y = 0$ and $\sum M = 0$ for any cut body

After finding reactions, we may apply the method of joints at joint A, and then proceed to joint B. After that we cannot go to either joint C or b. We may start again at G and then go on to F, after which we cannot go on any more. It is therefore desirable to resort to the method of sections. By cutting the truss at section (1) - (1), we can easily find the horizontal component of the force in bar bd.

$\sum M_D = 0\ (+\circlearrowright)$;

$$50 \times 90 - 60 \times 30 - F_{bdh} \times 50 = 0$$

$$\therefore F_{bdh} = \frac{4500 - 1800}{50} = 54\text{ k (tension as assumed)}$$

Once F_{bdh} is found, F_{bdv}, the vertical component is found from the slope of the member. Now it is possible to apply the method of joints at b and then successively at each of the remaining joints.

Example

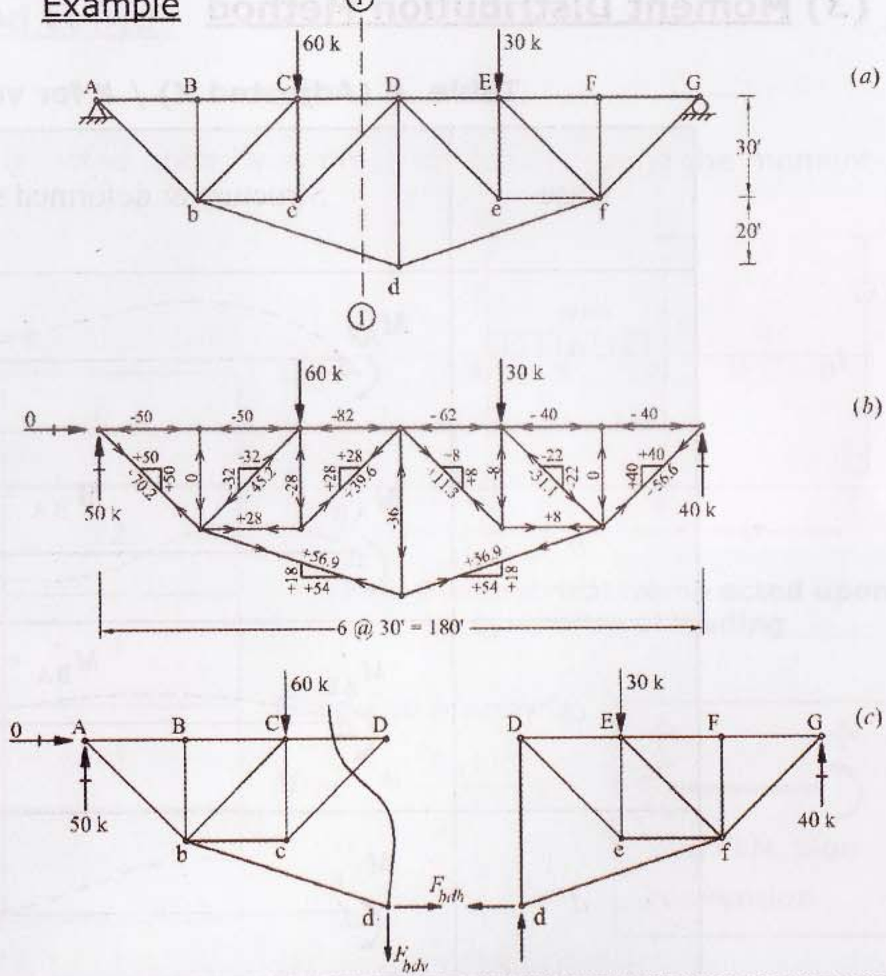
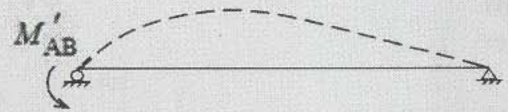
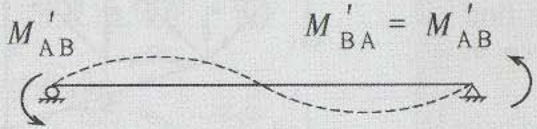
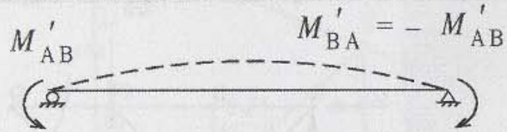
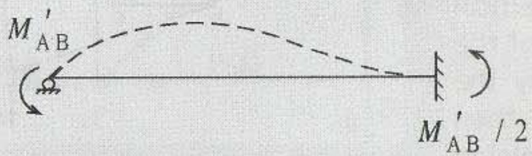


Fig. Compound truss analysis by method of joints and method of sections

(3) Moment Distribution Method

Table - (Adjusted *K*) / *K* for various conditions

Case	Structure & deformed shape	$\frac{\text{Adj.} K}{K}$
a		$\frac{3}{4}$
b		$\frac{3}{2}$
c		$\frac{1}{2}$
d		1

(3) Moment Distribution Method *contd.*

Example

Analyze the symmetrical frame shown in Fig. acted upon by symmetrical loading using the moment-distribution method.

Relative Stiffnesses

Table - Relative stiffnesses

Member	$\frac{I}{L}$	Adj. K_{rel}
AB , CD	$\frac{3I}{15}\times\frac{10}{I}$	2
BE , CF	$\frac{I}{10}\times\frac{10}{I}$	1
BG , CH	$\frac{1.33\,I}{10}\times\frac{10}{I}\times\frac{3}{4}$ *	1
BC	$\frac{3I}{15}\times\frac{10}{I}\times\frac{1}{2}$ **	1

* Case (a) adjustment (see Table for Adj. K / K)

** Case (c) adjustment (see Table for Adj. K / K)

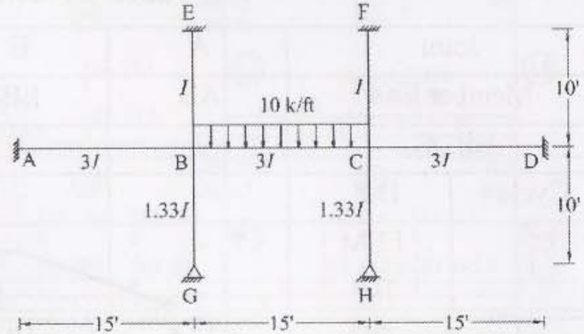


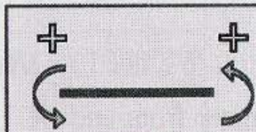
Fig. Symmetrical frame acted upon by symmetrical loading

Fixed-End Moments

$$M_{FBC} = \frac{10\times15^2}{12}$$

$$= +\ 187.5 \text{ k-ft}$$

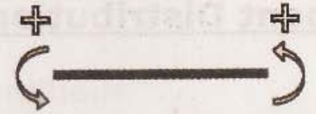
$$M_{FCB} = -\ 187.5 \text{ k-ft}$$



+ve FEM sign convention

(3) Moment Distribution Method *contd.*

Example contd.



+ve moment sign convention for Moment Distribution Method

Table - Moment distribution

Joint		A	E	G	B			
Member End		AB	EB	GB	BG	BE	BA	BC
Adj. K_{rel}		2	1	1	1	1	2	1
Cycle	D.F.	-	-	-	0.2	0.2	0.4	0.2
1	FEM	-	-	-	-	-	-	+ 187.5
	Bal	-	-	-	– 37.5	– 37.5	– 75.0	– 37.5
2	CO	–37.5	– 18.8	0	0	0	0	0
	Bal	-	-	-	-	-	-	-
Total		– 37.5	– 18.8	0	– 37.5	– 37.5	– 75.0	+ 150.0

We need to analyze only half of the structure. Note that by adjusting the stiffness of BC there is no carry-over moment from end BC to end CB. Similary, by adjusting the stiffness of BG, there is no carry-over moment from end BG and end GB. The results for the other half can be found by inspection. Thus,

$M_{DC} = + 37.5$ k-ft ; $M_{FC} = + 18.8$ k-ft ; $M_{HC} = 0$; $M_{CH} = + 37.5$ k-ft

$M_{CF} = + 37.5$ k-ft ; $M_{CD} = + 75.0$ k-ft ; $M_{CB} = - 150.0$ k-ft

(4) Concrete Building Frames by Moment Distribution
(Portland Cement Association Method)

Example

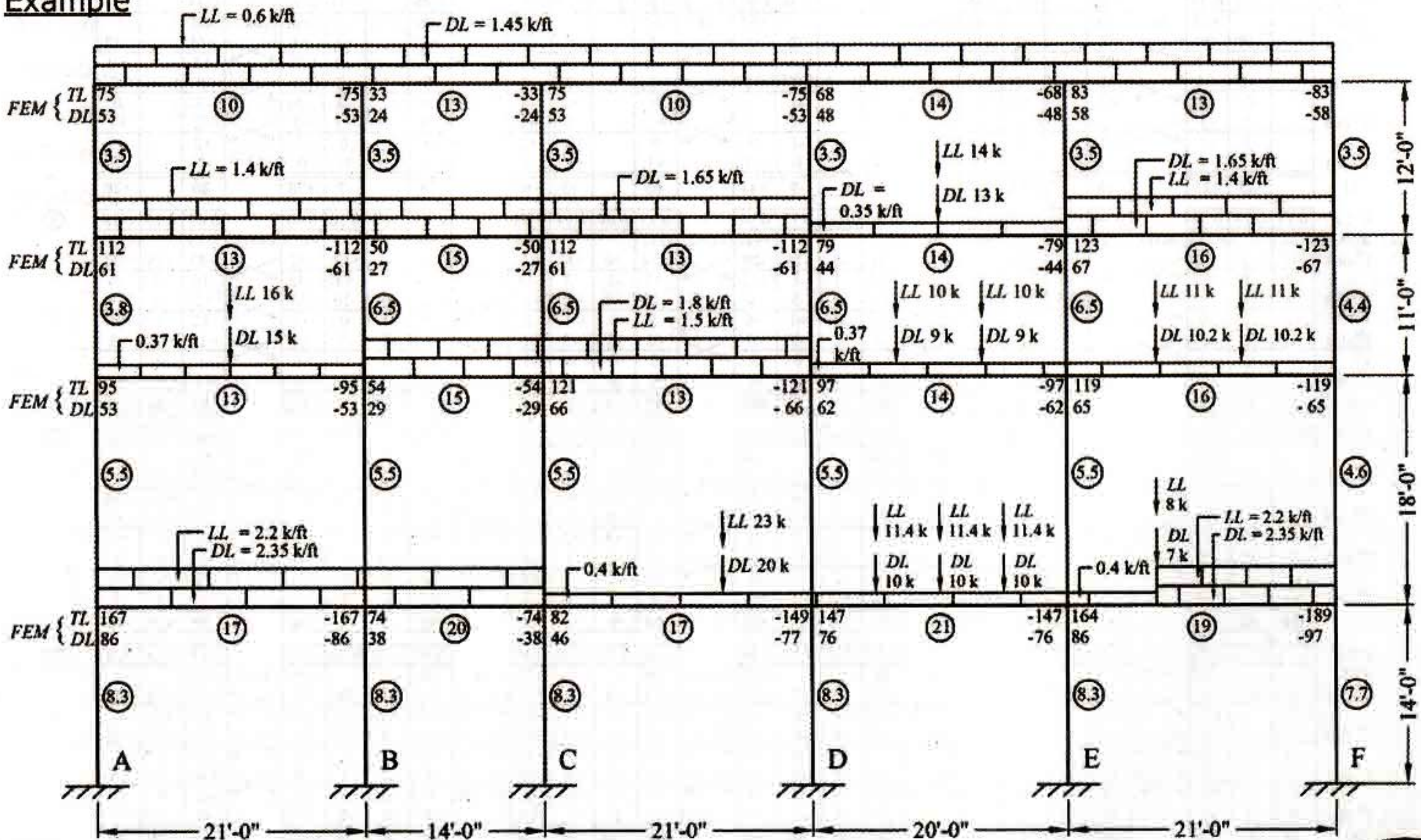


Fig. Given concrete building frame

Example *contd.*

Table - Moment distribution by P. C. A. Method

④

Ⓐ

	Col A	Col B	Bm
K/ΣK		0.26	0.74
FEM			+53
1st dist		-14	-39
		-10	+4
2nd dist		+2	+4
		0	+4
3rd dist		-1	-3
		-1	-1
4th dist		+1	+1
M		-23	+23

DL

Ⓑ

	Bm	Col A	Col B	Bm
K/ΣK	0.38		0.13	0.49
FEM	-53			+33
1st dist	+8		+2	+10
	-20		+4	-5
2nd dist	+8		+3	+10
	+2		+3	0
3rd dist	-2		-1	-2
	-2		0	-1
4th dist	+1		0	+2
M	-58		+11	+47

DL+LL

Ⓒ

	Bm	Col A	Col B	Bm
K/ΣK	0.49		0.13	0.38
FEM	-33			+53
1st dist	-10		-2	-8
	+5		-4	-3
2nd dist	+1		0	+1
	+5		-1	0
3rd dist	-2		-1	-1
	-1		-1	0
4th dist	+1		0	+1
M	-34		-9	+43

DL

Ⓓ

	Bm	Col A	Col B	Bm
K/ΣK	0.36		0.13	0.51
FEM	-53			+68
1st dist	-5		-2	-8
	-4		+3	+3
2nd dist	-1		0	-1
	0		+2	-4
3rd dist	+1		0	+1
	0		0	+2
4th dist	+1		0	-1
M	-63		+3	+60

DL-LL

Ⓔ

	Bm	Col A	Col B	Bm
K/ΣK	0.46		0.11	0.43
FEM	-68			+58
1st dist	+5		+1	-4
	-4		-4	+23
2nd dist	-7		-2	-6
	0		-3	-5
3rd dist	+4		+1	-3
	0		0	-1
4th dist	-1		0	0
M	-71		-7	+78

DL

Ⓕ

	Bm	Col A	Col B
K/ΣK	0.74		0.21
FEM	-58		
1st dist	+46		+12
	+2		+9
2nd dist	-9		-2
	-3		0
3rd dist	+2		+1
	+2		+1
4th dist	-2		-1
M	-20		+20

③

Ⓐ

K/ΣK	0.17	0.19	0.64
FEM			+112
1st dist	-19	-21	-72
	-7	-5	+15
2nd dist	0	-1	-2
	+1	+2	+9
3rd dist	-2	-2	-8
	0	0	11
4th dist	0	0	-1
M	-27	-27	+54

DL+LL

Ⓑ

K/ΣK	0.34	0.09	0.17	0.40
FEM	-112			+27
1st dist	+29	+8	+14	+34
	-36	+1	0	-17
2nd dist	+17	+5	+9	+21
	-1	+2	-2	-6
3rd dist	+2	0	1	+2
	-4	0	0	-4
4th dist	+3	+1	+2	+4
M	-102	+17	+24	+61

DL

Ⓒ

K/ΣK	0.40	0.09	0.17	0.34
FEM	-27			+112
1st dist	-34	-8	-14	-29
	-17	-1	-1	+12
2nd dist	-11	-2	-5	-9
	-11	0	+1	+6
3rd dist	-7	-2	-3	-6
	11	0	0	3
4th dist	-2	0	-1	-1
M	-52	-13	-23	+88

DL+LL

Ⓓ

K/ΣK	0.35	0.09	0.18	0.38
FEM	-112			+44
1st dist	+24	+6	+12	+26
	-15	-1	-3	-14
2nd dist	+12	+3	+6	+12
	-5	0	-1	-10
3rd dist	+6	+1	+3	+6
	-3	0	0	-1
4th dist	+1	0	+1	+2
M	-92	+9	+18	+65

DL

Ⓔ

K/ΣK	0.35	0.09	0.16	0.40
FEM	-44			+123
1st dist	-28	-7	-12	-32
	+13	0	+3	+42
2nd dist	-20	-5	-10	-23
	+6	-1	0	-1
3rd dist	-2	-1	-1	-2
	13	0	12	-6
4th dist	-4	-1	-2	-4
M	-76	-15	-20	+111

DL+LL

Ⓕ

K/ΣK	0.67	0.15	0.18
FEM	-123		
1st dist	+83	+18	+22
	-16	+6	+6
2nd dist	+2	+1	+1
	-12	-1	-3
3rd dist	+11	+2	+3
	-1	0	0
4th dist	+1	0	0
M	-55	+26	+29

②

Ⓐ

K/ΣK	0.17	0.25	0.58
FEM			+53
1st dist	-9	-13	-31
	-11	-15	0
2nd dist	+4	+7	+15
	0	-2	0
3rd dist	0	+1	+1
	-1	-1	-3
4th dist	+1	+1	+3
M	-16	-22	+38

DL

Ⓑ

K/ΣK	0.33	0.16	0.14	0.37
FEM	-53			+54
1st dist	0	0	0	-1
	-16	+7	+7	-2
2nd dist	+1	+1	+1	+1
	+8	+5	+3	+3
3rd dist	-6	-3	-3	-7
	0	0	0	+2
4th dist	-1	0	0	-1
M	-67	+10	+8	+49

DL+LL

Ⓒ

K/ΣK	0.37	0.16	0.14	0.33
FEM	-54			+66
1st dist	-4	-2	-2	-4
	0	-7	-3	-5
2nd dist	+6	+2	+2	+5
	0	-3	-2	-3
3rd dist	+3	+1	+1	+3
	-4	-2	-1	-2
4th dist	+3	+2	+1	+3
M	-50	-9	-4	+63

DL

Ⓓ

K/ΣK	0.33	0.17	0.14	0.36
FEM	-66			+97
1st dist	-10	-6	-4	-11
	-2	+6	+4	+6
2nd dist	-5	-2	-2	-5
	+3	+3	+2	0
3rd dist	-3	-1	-1	-3
	+2	-2	+1	+4
4th dist	-3	-2	-1	-3
M	-84	0	-1	+85

DL-LL

Ⓔ

K/ΣK	0.33	0.16	0.13	0.38
FEM	-97			+65
1st dist	+11	+5	+4	+12
	-6	-6	-5	+21
2nd dist	-1	-1	-1	-1
	-3	-5	-4	-10
3rd dist	+7	+4	+3	-8
	-2	0	0	0
4th dist	+1	0	0	-1
M	-90	-3	-3	+96

DL

Ⓕ

K/ΣK	0.64	0.18	0.18
FEM	-65		
1st dist	+41	+12	+12
	+6	+11	+14
2nd dist	-19	-6	-6
	0	0	+1
3rd dist	-1	0	0
	+4	+2	+1
4th dist	-5	-1	-1
M	-39	+18	+21

①

Ⓐ

K/ΣK	2.18	0.27	0.55
FEM			+167
1st dist	-30	-45	-92
	-7		+22
2nd dist	-3	-4	-8
	+4		+10
3rd dist	-2	-4	-8
	0		+2
4th dist	0	-1	-1
M	-38	-54	+92

DL+LL

Ⓑ

K/ΣK	0.34	0.11	0.16	0.39
FEM	-167			+38
1st dist	+44	+14	+21	+50
	-46	0	0	-9
2nd dist	+19	6	+9	+21
	-4	0	0	-7
3rd dist	+4	+1	+2	+4
	-4	-2	0	-4
4th dist	+3	+1	+2	+4
M	-151	+20	+34	+97

DL

Ⓒ

K/ΣK	0.39	0.11	0.16	0.34
FEM	-38			+82
1st dist	-17	-5	-7	-15
	-25	-1	0	+12
2nd dist	-14	-4	-6	-12
	-11	+1	0	+5
3rd dist	-7	-2	-2	-6
	+2	0	0	+4
4th dist	-2	-1	-1	-2
M	-401	-12	-16	+68

DL+LL

Ⓓ

K/ΣK	0.33	0.11	0.16	0.40
FEM	-149			+76
1st dist	+24	+8	+12	+29
	-8	-2	0	-17
2nd dist	+9	+3	+4	+11
	-6	-1	0	-15
3rd dist	+7	+2	+4	+9
	-3	0	0	-2
4th dist	+2	0	+1	+2
M	-124	+10	+21	+93

DL

Ⓔ

K/ΣK	0.39	0.10	0.16	0.35
FEM	-76			+164
1st dist	-34	-9	-14	-31
	+15	+2	0	+58
2nd dist	-29	-8	-12	-26
	+6	0	0	-3
3rd dist	-4	-1	-1	-3
	+	+2	0	-5
4th dist	-5	-1	-2	-4
M	-122	-15	-29	+166

DL+LL

Ⓕ

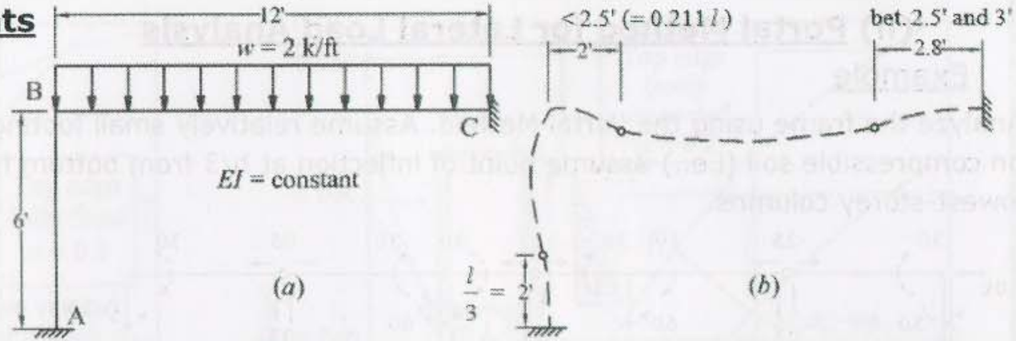
K/ΣK	0.61	0.15	0.24
FEM	-189		
1st dist	+116	+28	+45
	-16	+6	0
2nd dist	+6	+2	+2
	-13	-3	0
3rd dist	+10	+2	+4
	-2	0	0
4th dist	+1	0	+1
M	-87	+35	+52

(5) Approximate Methods

(i) By Estimating Inflection Points

Example

Analyze the frame by estimating locations of points of inflection. Known cases shown in Fig. may be referred to in estimating the locations.



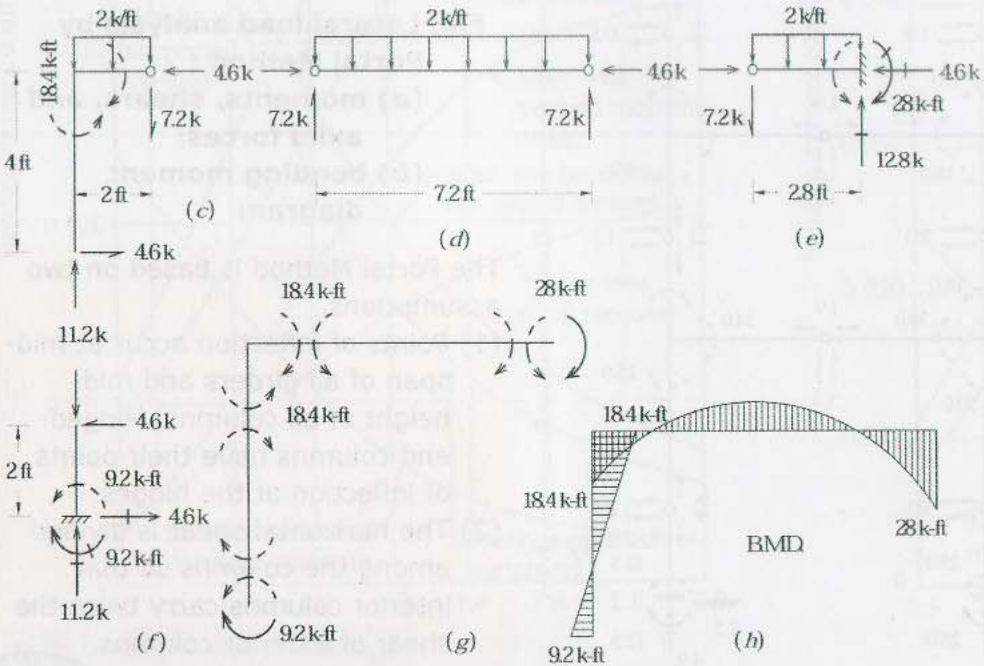


Fig. Approximate analysis of the rigid frame

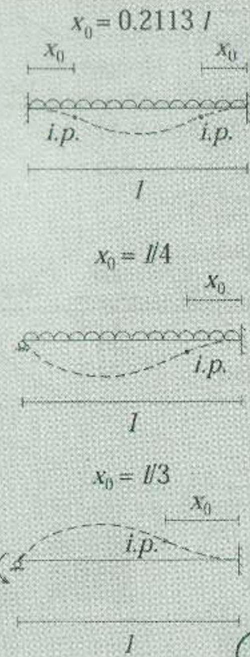


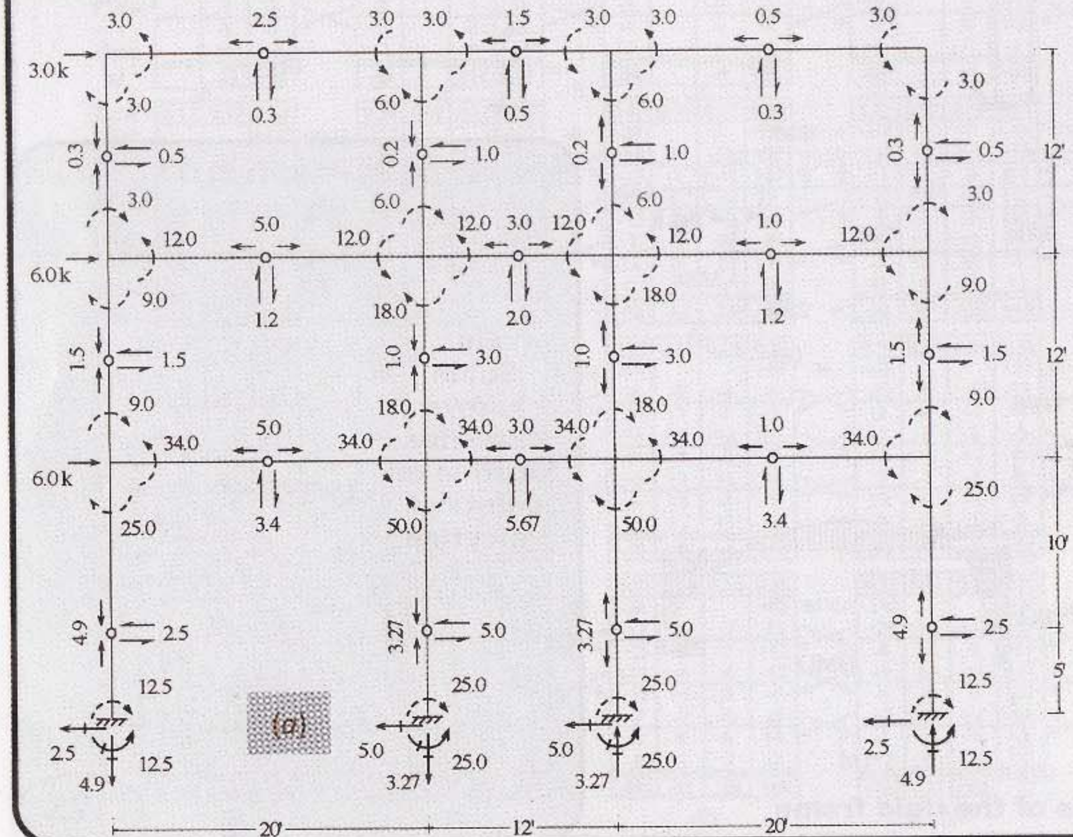
Fig. Some known cases for location of points of inflection

(5) Approximate Methods *contd.*

(ii) Portal Method for Lateral Load Analysis

Example

Analyze the frame using the Portal Method. Assume relatively small footings on compressible soil (i.e.,) assume point of inflection at h/3 from bottom for lowest-storey columns.



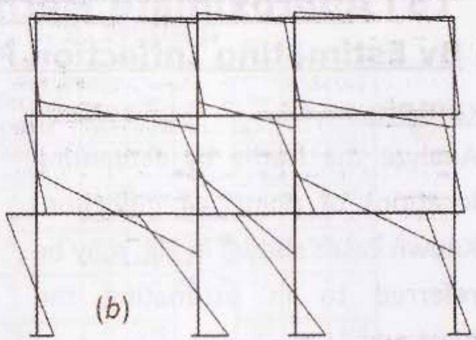


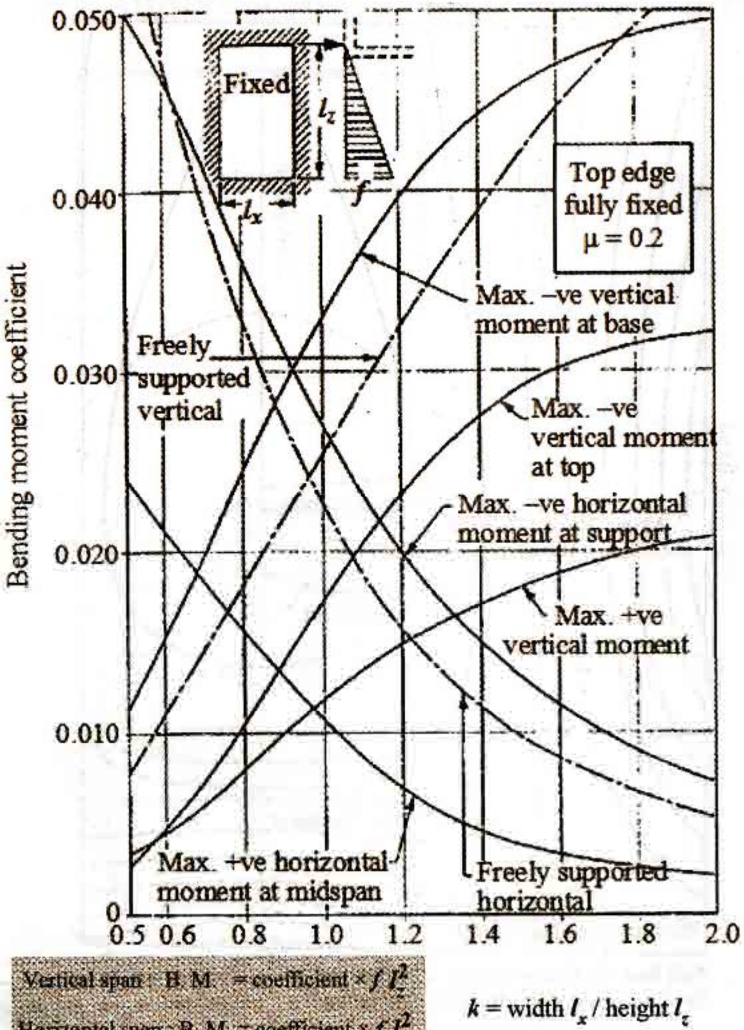
Fig. Lateral load analysis by Portal Method :
(*a*) moments, shears, and axial forces;
(*b*) bending moment diagram

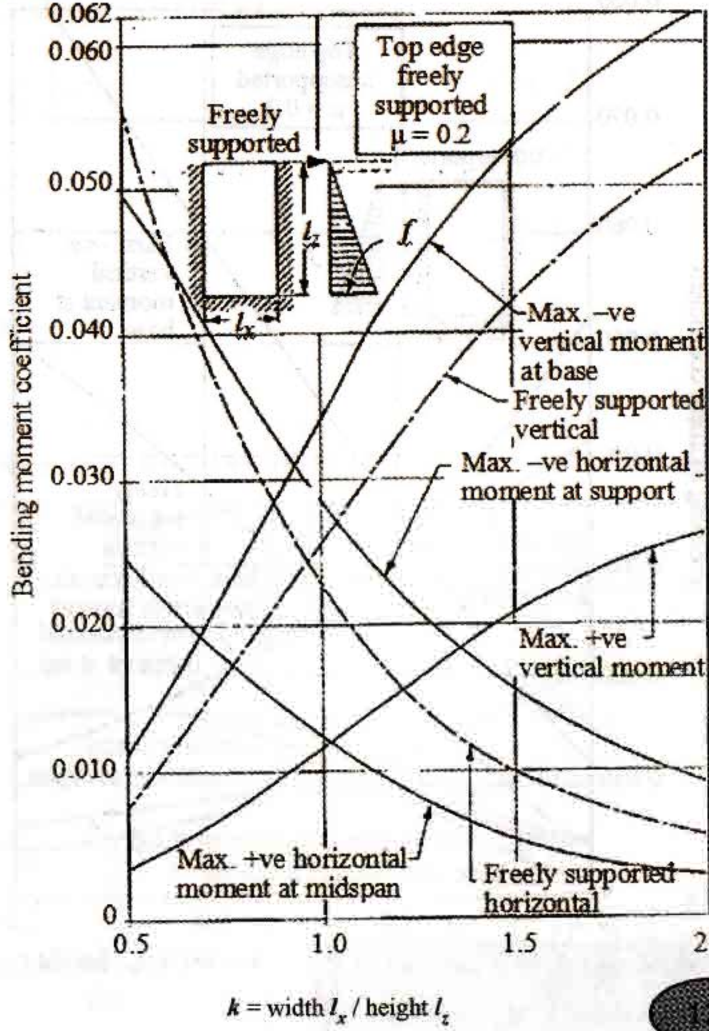
The Portal Method is based on two assumptions.

(1) Points of inflection occur at mid-span of all girders and mid-height of all columns. Hinged-end columns have their points of inflection at the hinges.

(2) The horizontal shear is divided among the columns so that interior columns carry twice the shear of exterior columns.

Two-Way Slabs - Triangularly Distributed Loads

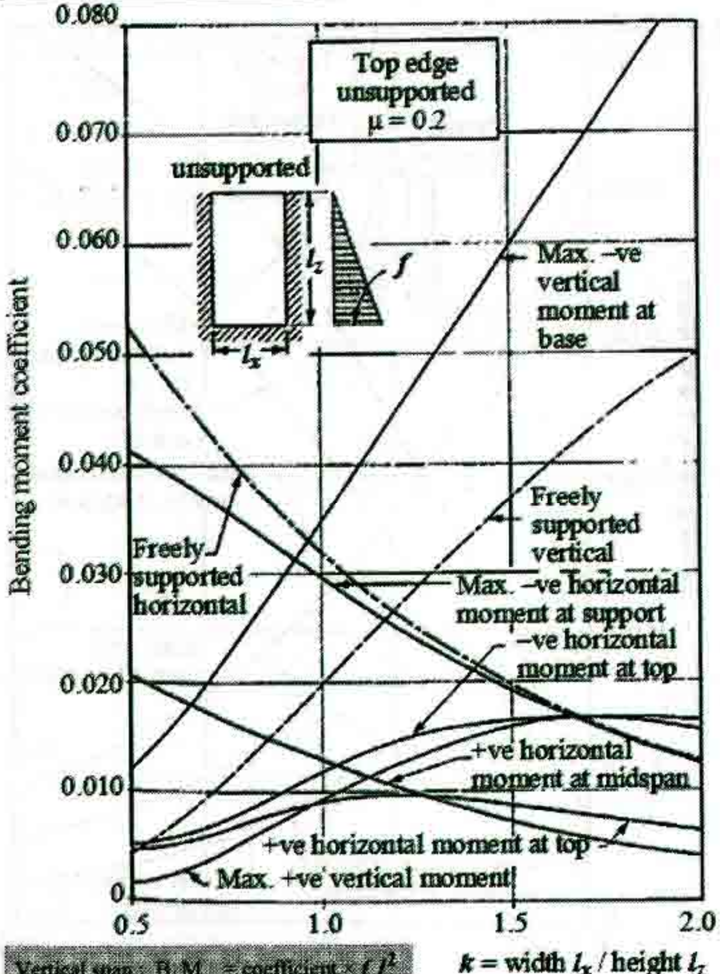




Vertical span : B. M. = coefficient × $f\,l_z^2$

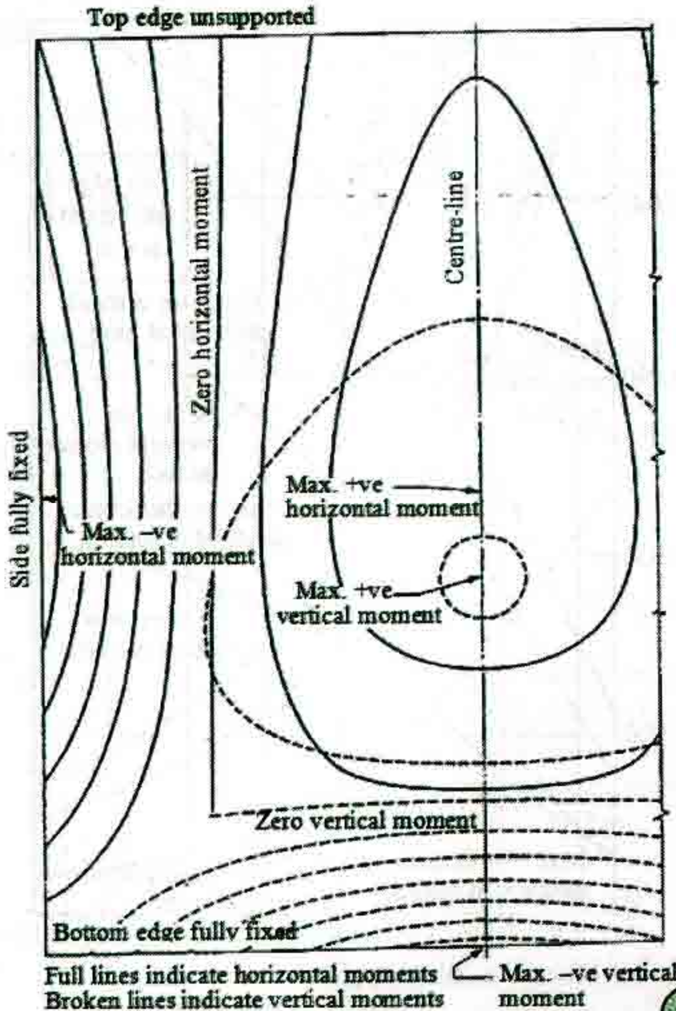
Horizontal span : B. M. = coefficient × $f\,l_x^2$

(6) Analysis Aids Two-Way Slabs - Triangularly Distributed Loads *contd.*



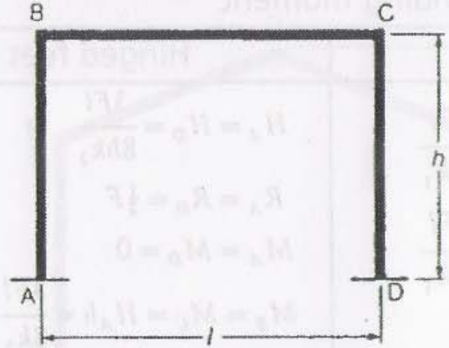
Vertical span : B.M. = coefficient × $f\,l_z^2$

Horizontal span : B.M. = coefficient × $f\,l_x^2$



Full lines indicate horizontal moments
Broken lines indicate vertical moments

(6) Analysis Aids — Portal Frames (Special Cases)



$$F = \text{total load}$$

$$I_{AB} = I_{CD}$$

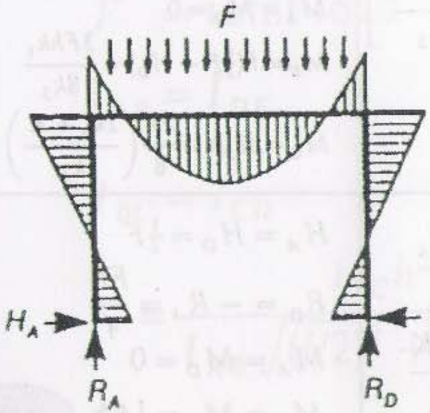
$$K = \frac{I_{BC}h}{I_{AB}l}$$

$$k_1 = K + 2$$

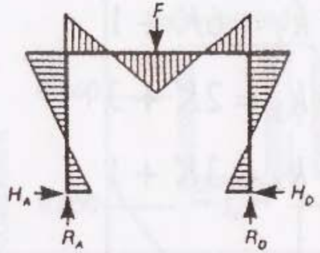
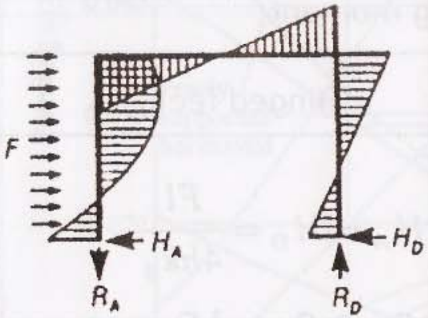
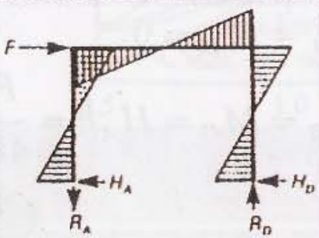
$$k_2 = 6K + 1$$

$$k_3 = 2K + 3$$

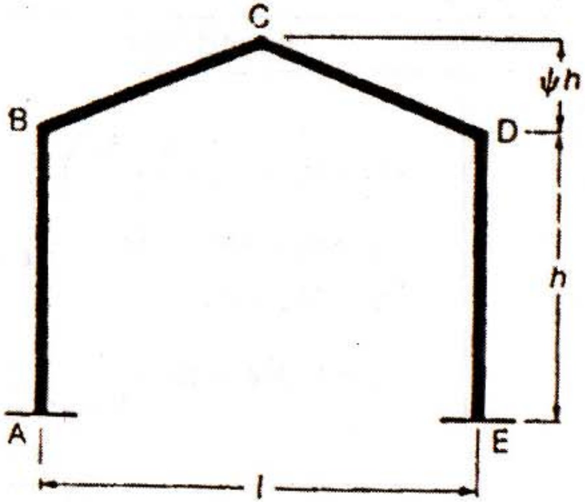
$$k_4 = 3K + 1$$

Loading	Reaction and bending moment	
	Fixed feet	Hinged feet
 	$H_A = H_D = \frac{Fl}{4hk_1}$ $R_A = R_D = \frac{1}{2}F$ $M_A = M_D = \frac{Fl}{12k_1}$ $M_B = M_C = \frac{Fl}{6k_1}$	$H_A = H_D = \frac{Fl}{4hk_3}$ $R_A = R_D = \frac{1}{2}F$ $M_A = M_D = 0$ $M_B = M_C = H_A h = \frac{Fl}{4k_3}$

(6) Analysis Aids — Portal Frames (Special Cases) contd.

Loading	Reaction and bending moment	
	Fixed feet	Hinged feet
	$H_A = H_D = \frac{3Fl}{8hk_1}$ $R_A = R_D = \frac{1}{2}F$ $M_A = M_D = \frac{Fl}{8k_1}$ $M_B = M_C = \frac{Fl}{4k_1}$	$H_A = H_D = \frac{3Fl}{8hk_3}$ $R_A = R_D = \frac{1}{2}F$ $M_A = M_D = 0$ $M_B = M_C = H_A h = \frac{3Fl}{8k_3}$
	$H_A = F - H_D$ $H_D = \frac{Fk_3}{8k_1}$ $R_A = -\frac{FhK}{lk_2} = -R_D$ $M_A = \frac{Fh}{4}\left[\frac{K+3}{6k_1} + \frac{4K+1}{k_2}\right]$ $M_B = h(H_A - \frac{1}{2}F) - M_A$ $M_C = H_D h - M_D$ $M_D = \frac{Fh}{4}\left[\frac{K+3}{6k_1} - \frac{4K+1}{k_2}\right]$	$H_A = \frac{F}{8}\left(\frac{6k_3 - K}{k_3}\right)$ $H_D = F - H_A$ $R_D = -R_A = \frac{Fh}{2l}$ $M_A = M_D = 0$ $M_B = h(\frac{1}{2}F - H_D) = \frac{3Fhk_1}{8k_3}$ $M_C = H_D h = \frac{Fh}{8}\left(\frac{2k_3 + K}{k_3}\right)$
	$H_A = H_D = \frac{1}{2}F$ $R_A = -R_D = -\frac{3FhK}{lk_2}$ $M_A = M_D = \frac{Fhk_4}{2k_2}$ $M_B = M_C = \frac{3FhK}{2k_2}$	$H_A = H_D = \frac{1}{2}F$ $R_D = -R_A = \frac{Fh}{l}$ $M_A = M_D = 0$ $M_B = M_C = \frac{1}{2}Fh$

(6) **Analysis Aids** **Gable Frames (Special Cases)**



F = total load

$I_{AB} = I_{DE}$

$I_{BC} = I_{CD}$

$$K = \frac{I_{BC}h}{I_{AB}\sqrt{(l/2)^2 + (\psi h)^2}}$$

$$k_1 = K + 2$$

$$k_4 = 3K + 1$$

$$k_7 = K + 3 + \psi(3 + \psi)$$

$$k_9 = 2K + 3(2 + \psi)$$

$$k_{10} = 3K + \psi(4K + 1)$$

$$k_{12} = K(K + 4 + 6\psi) + \psi^2(4K + 1)$$

$$k_{13} = K + \psi(2K + 1)$$

$$k_{15} = K(K + 4 + 3\psi)$$

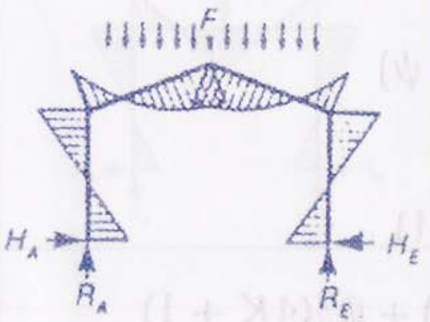
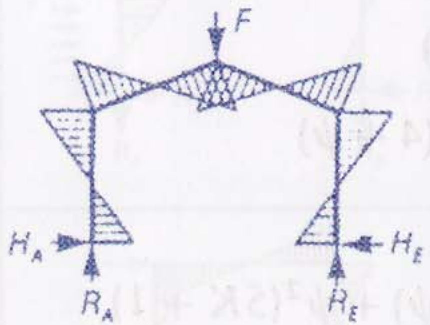
$$k_{19} = 8(K + 3) + 5\psi(4 + \psi)$$

$$k_{20} = K + 3 + 2\psi$$

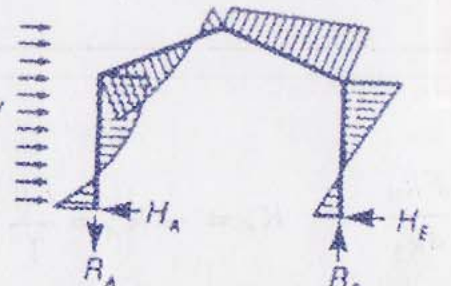
$$k_{21} = 2K(K + 4 + 5\psi) + \psi^2(5K + 1)$$

$$k_{22} = 2(10K + 3) + \psi(9K + 6) + \psi^2$$

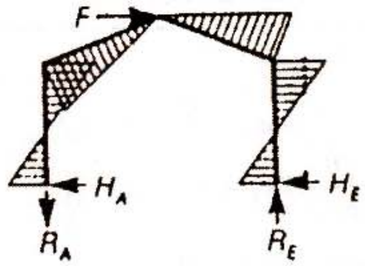
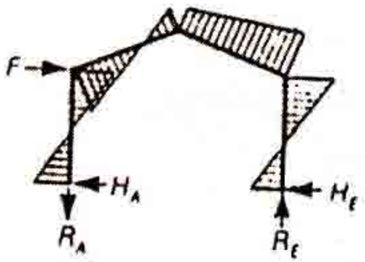
(6) Analysis Aids — Gable Frames (Special Cases) *contd.*

Loading	Reactions and bending moments	
	Fixed feet	Hinged feet
	$H_A = H_E = \frac{Fl}{8h}\left[\frac{4K + \psi(5K+1)}{k_{12}}\right]$ $R_A = R_E = \frac{1}{2}F$ $M_A = M_E = \frac{Fl}{48}\left[\frac{K(8+15\psi) + \psi(6-\psi)}{k_{12}}\right]$ $M_B = M_D = H_A h - M_A$ $M_C = H_A h(1+\psi) - \frac{1}{8}Fl - M_A$	$H_A = H_E = \frac{Fl}{32h}\left(\frac{8+5\psi}{k_7}\right)$ $R_A = R_E = \frac{1}{2}F \quad M_A = M_E = 0$ $M_B = M_D = H_A h$ $M_C = H_A h(\psi+1) - \frac{Fl}{8}$
	$H_A = H_E = \frac{Fl}{4h}\left(\frac{k_{10}}{k_{12}}\right)$ $R_A = R_E = \frac{1}{2}F$ $M_A = M_E = \frac{Fl}{4}\left(\frac{k_{13}}{k_{12}}\right)$ $M_B = M_D = H_A h - M_A$ $M_C = H_A h(1+\psi) - \frac{1}{4}Fl - M_A$	$H_A = H_E = \frac{Fl}{8h}\left(\frac{3+2\psi}{k_7}\right)$ $R_A = R_E = \frac{1}{2}F \quad M_A = M_E = 0$ $M_B = M_D = H_A h$ $M_C = H_A h(1+\psi) - \frac{1}{4}Fl$

(6) Analysis Aids   Gable Frames (Special Cases) *contd.*

Loading	Reactions and bending moments	
	Fixed feet	Hinged feet
$F(\text{total}) = wh(1+\psi)$  	$H_E = \frac{wh}{4k_{12}}(Kk_{20} + \psi k_{21})$ $H_A = F - H_E$ $R_E = -R_A$ $= \frac{wh^2}{8lk_4}[4K(1+3\psi) + \psi^2(12K+5)]$ $M_A = \frac{wh^2}{24}\left\{\left[\frac{K(K+6+15\psi)+\psi^2 k_{22}}{k_{12}}\right] + \frac{12k_5 + \psi[12(3K+2)+3\psi]}{2k_4}\right\}$ $M_B = H_A h - M_A - \frac{1}{2}wh^2$ $M_C = H_E h(1+\psi) - \frac{1}{2}lR_E - M_E$ $M_D = H_E h - M_E$ $M_E = \frac{wh^2}{24}\left\{\left[\frac{K(K+6+15\psi)+\psi^2 k_{22}}{k_{12}}\right] - \frac{12k_5 + \psi[12(3K+2)+3\psi]}{2k_4}\right\}$	$H_E = \frac{wh}{16k_7}(2k_9 + K + \psi k_{19})$ $H_A = wh(1+\psi) - H_E$ $R_E = -R_A = \frac{wh^2}{2l}[1 + \psi(2+\psi)]$ $M_A = M_E = 0$ $M_B = H_A h - \frac{1}{2}wh^2$ $M_C = H_E h(1+\psi) - \frac{1}{2}R_E l$ $M_D = H_E h$

(6) Analysis Aids   Gable Frames (Special Cases) *contd.*

Loading	Reactions and bending moments	
	Fixed feet	Hinged feet
	$H_A = H_E = \frac{1}{2}F$ $R_E = -R_A = \frac{1}{l}[Fh(1+\psi) - 2M_E]$ $M_A = M_E = \frac{Fh}{4}\left(\frac{3K+2}{2K+1}\right)$ $M_B = M_D = H_A h - M_A$ $M_C = 0$	$H_A = H_E = \frac{1}{2}F$ $R_E = -R_A = \frac{1}{l}[Fh(1+\psi)]$ $M_A = M_E = 0$ $M_B = M_D = \frac{1}{2}Fh$ $M_C = 0$
	$H_E = \frac{FK}{2}\left(\frac{k_{15}}{k_{12}}\right)$ $H_A = F - H_E$ $R_E = -R_A = \frac{3Fh}{2l}\left(\frac{K}{3K+1}\right)$ $M_A = \frac{Fh}{2}\left(\frac{\psi k_{13}}{k_{12}} + \frac{k_4+1}{2k_4}\right)$ $M_B = H_E h - M_A$ $M_C = H_E h(1+\psi) - \frac{1}{2}lR_E - M_E$ $M_D = H_E h - M_E$ $M_E = \frac{Fh}{2}\left(\frac{\psi k_{13}}{k_{12}} - \frac{k_4+1}{2k_4}\right)$	$H_E = \frac{Fk_9}{4k_7}$ $R_E = -R_A = \frac{Fh}{l}$ $H_A = F - H_E$ $M_A = M_E = 0$ $M_B = H_A h$ $M_D = H_E h$ $M_C = H_E h(1+\psi) - \frac{1}{2}Fh$

ANALYSIS OF RATES

H. ANALYSIS OF RATES

(1) Earthwork

1. Earthwork Excavation for foundation (for 100 Cft)

(to a depth of 5 ft & removing within 100 ft)

Workers (*ordinary soil*)	1 ½	Man-day
Workers (*medium soil*)	2	Man-day
Workers (*hard soil*)	3	Man-day

2. Earthwork Excavation for digging (for 100 Cft)

(in sand or clay or laterite up to 10 ft initial depth)

Workers	3	Man-day

3. Earthwork Excavation for digging drain (for 100 Rft)

(1' 6" at top, 9" at bottom & average depth 12")

Workers (*ordinary soil*)	2	Man-day
Workers (*hard soil*)	3	Man-day

(2) Mortar

1. Cement mortar (for 100 Cft)

		1 : 2	**1 : 3**	**1 : 4**	**1 : 6**
Cement	(Lbs)	4140	2970	2250	1500
	(Cft)	(46)	(33)	(25)	(16 ⅔)
Sand	(Cft)	92	100	100	100
Workers	(Man-day)	4	4	4	4

2. Damp proof cement motor 1:2 (for 100 Cft)

(with 5% Pudlo by weight of cement)

Cement	(48 Cft)	4320	Lbs
Pudlo		216	Lbs
Sand		96	Cft
Workers		4	Man-day

3. Composite mortar for plaster

(for 100 Cft)		**1 : 1 : 6**	**1 : 2 : 6**	**1 : ½ : 4**
Cement	(Lbs)	1500	1440	2178
	(Cft)	(16 $^2/_3$)	(16)	(24.2)
Lime	(Cft)	16 $^2/_3$	32	12.1
Sand	(Cft)	100	100	96.8
Workers	(Man-day)	4	4	4

(3) Concrete (Hand-Mixed)

1. Cement concrete (for 100 Cft)

Proportion	Cement (Lbs)	Chipping/River shingle (Cft)	Sand (Cft)	Mason (Man-day)	Workers (Man-day)
1 : 1 ½ : 3	2790 (31 Cft)	92 (¼" to ¾")	46	1	10
1 : 2 : 4	2070 (23 Cft)	92 (¼" to ¾")	46	1	10
1 : 2 ½ : 5	1710 (19 Cft)	94 (¾")	47	1	10
1 : 3 : 6	1440 (16 Cft)	96 (1 ½")	48	1	8
1 : 4 : 8	1170 (13 Cft)	104 (¾" to 1 ½")	52	1	10

2. Cement concrete for damp proof course (DPC) with 5% pudlo to the weight of cement (for 100 Cft)

Proportion	Cement (Lbs)	Pudlo (Lbs)	Chipping/River shingle (Cft)	Sand (Cft)	Mason (Man-day)	Worker (Man-day)
1 : 2 : 4 (1 ½" tk.)	248 (2¾ Cft)	12	12 (¼" to ¾")	6	1	1
1 : 2 : 4 (1" tk.)	173 (2 Cft)	8 ¾	8 (¼" to ¾")	4	¾	¾

(4) Reinforced Concrete (Hand-Mixed)

1. Reinforced concrete (for 100 Cft)

Proportion	Cement (Lbs)	Chipping/River shingle (Cft)	Sand (Cft)	Mason (Man-day)	Workers (Man-day)
1 : 2 : 4	2070 (23 Cft)	92 (¼" to ¾")	46	2	15

(5) Concrete (By Machine)

1. Cement concrete (for 100 Cft) (Mixing & placing)

Proportion	Cement (Lbs)	Chipping/River shingle (Cft)	Sand (Cft)	Fuel (Gals)	Mason (Man-day)	Workers (Man-day)	Machine driver (Man-day)
1 : 1½ : 3	2790 (31 Cft)	92 (¼" to ¾")	46	2	1	8	½
1 : 2 : 4	2070 (23 Cft)	92 (½" to ¾")	46	2	1	* 3	½
1 : 3 : 6	1440 (16 Cft)	96 (1 ½")	48	2	1	6	½
1 : 4 : 8	1170 (13 Cft)	104 (¾" to 1 ½")	52	2	1	8	½

** Mixing only without placing*

(6) Brickwork

1. First class in cement mortar (for 100 Cft)

Proprotion	Cement (Lbs)	Brick (No)	Sand (Cft)	Masons (Man-day)	Workers (Man-day)
1 : 2	1035 (11.5 Cft)	1350	23	4	6
1 : 3	780 (8 2/3 Cft)	1350	26	4	6
1 : 4	630 (7 Cft)	1350	28	4	6

2. First class in cement mortar for cornice (6" deep) (for 100 Rft)

Proprotion	Cement (Lbs)	Brick (No)	Sand (Cft)	Masons (Man-day)	Workers (Man-day)
1 : 3	150	275	5	4	3

3. First class in composite mortar (for 100 Cft)

Proprotion	Cement (Lbs)	Brick (No)	Lime (Cft)	Sand (Cft)	Masons (Man-day)	Workers (Man-day)
1 : 1 : 6	372	1350	4 1/8	24 3/4	4	6

(7) Stonework

1. 1 : 3 Cement mortar (for 100 Cft)

Type	Rubble stone (Cft)	Cement (lbs)	Sand (Cft)	Mason (Man-day)	Workers (Man-day)
Coursed rubble stone	125 (selected)	900	30	9	6
Random rubble stone	150 (rough dressed)	1180	40	4	5

(8) Plastering and Pointing

1. **Plastering with cement mortar (for 100 Sft)**

Proprotion	Cement (lbs)	Sand (Cft)	Mason (Man-day)	Workers (Man-day)
1 : 2 (½" tk.)	225	5	1	2
1 : 3 (½" tk.)	150	5	1	2
1 : 4 (½" tk.)	112.5	5	1	2
1 : 3 (¾" tk.)	225	7.5	1 ½	3

(9) Ceiling

1. Asbestos cement ceiling , joists at 2' centres (for 256 Sft)

Type	4' × 4' asbestos cement sheets (Sft)	Joists (Cft)	1 ½" × ½" Beadings (Cft)	1 ½" Wood screws (No)	Nails (Lbs)	Carpenters (Man-day)
4" × 2" joists	282	8.8	0.96	216	1 ½	7 ½
3" × 2" joists	282	6.6	0.96	216	1 ½	6 ½

2. Asbestos sheet & A. C. plain sheet ceiling , joists at 4' centres and 2' centres cross joists (for 256 Sft)

Type	4' × 4' sheets (Sft)	Joists (Cft)		1 ½" × ½" Beadings (Cft)	1 ½" Wood screws (No)	Nails (Lbs)	Carpenters (Man-day)
Asbestos sheet	282	4.89 (4"x2")	4.6 (2"x2")	0.96	216	1 ½	7 ½
A. C. plain sheet	282	3.67 (3"x2")	4.6 (2"x2")	1.28	216	1 ½	7

3. A. C. Plain sheet ceiling without ceiling joists (for 256 Sft)

Type	4' × 4' sheets (Sft)	1 ½" × ½" Beadings (Cft)	1 ½" Wood screws (No)	Nails (Lbs)	Carpenters (Man-day)
A. C. plain sheet	282	0.96	216	½	3 ½

(9) Ceiling contd.

4. Plywood ceiling , joists at 2' centres (for 256 Sft)

Type	Plywood (3-ply) (Sft)	Scantlings (Cft)	2" × ½" Beatings (Cft)	Wood screws (No)	Nails (Lbs)	Carpenters (Man-day)
with 3" × 2" joists	295	6.6	-	288	2	6 ½
with 3" × 1½" joists	295	10.35	2.3	288	3	7 ½

(10) Roofing

1. Danyingon (mangalore pattern) clay tile roofing with 2"x1" battens (for 100 Sft)

Type	Tiles (No)	Battens (Cft) (close)	Battens (Cft) (ord.)	4"x2" rafter , @ 2' c/c (Cft)	Wire nails & spikes (Lbs)	Carpenters (Man-day)	Workers (Man-day)
with 4"x2" common rafter	156	9.58 (2"x1")	1.32 (1½"x1")	3.42	5	3	3
without rafter	156	-	1.76 (2"x1") , @ 13" c/c	-	1 ½ + ½ (binding wire)	1 ½	2

(10) Roofing contd.

2. 32 G. C. G. I. sheets (for 100 Sft)

7 ft C. G. I. sheet (No)	G. I. roofing nails (Lbs)	Carpenters (Man-day)	Workers (Man-day)
9	1 ½	1 ½	1

3. Corrugated A. C. sheet roofing with ¼" dia. hook bolts and washers (for 100 Sft)

Trafford or corrugated A. C. sheet 7' long (No)	Hook bolts and washers (Lbs)	Carpenters (Man-day)	Workers (Man-day)
5	5	2	1

4. Ridging of G. I. plain sheet 24" girth with 9" end laps fixed complete (for 100 Rft)

G. I. plain sheet (Rft)	G. I. roofing nails with washers (Lbs)	Carpenters (Man-day)
112	1 ½	3 1/3

5. G. I. valley guttering 3' wide without planks (for 22.5 Rft)

G. I. plain sheet (Sft)	Nails (Lbs)	Carpenters (Man-day)
72	¼	1

(10) Roofing contd.

6. Valley gutter of 32. G. I. plain sheet 24" girth with 9" end laps supported on ½" thk. valley boards fixed complete with 2" x 1" fillets (for 100 Rft)

G. I. plain sheet (Rft)	Planks 6"x½" (Cft)	2"x1" fillets (Cft)	Nails (Lbs)	Earth oil (Gal)	Carpenters (Man-day)	Workers (Man-day)
112	9.58	3.19	2 ½	1 ½	5	1 ½

(11) Steel work and Formwork

Steel work

Mild steel bar, bent and fixed (for 1 Cwt)

Particulars	Floor, roof & beam		Column, brace & wall
	1/2" Ø	5/8 – 1" Ø	1/2" – 1"Ø
Steel bar (Lbs)	117.6	117.6	117.6
Binding wire (Lbs)	1	1	1
Steel fixer (Man-day)	1	3/4	1
Worker (Man-day)	1	3/4	1

Formwork

Timber shuttering (for 100 Sft)

Timber scantling	15	Cft
Timber planks 1"	110	Sft
Nails & spikes	3	Lbs
M. S. bolts and washers if required		
Carpenters	4	Man-day
Workers	2	Man-day

* Add 1 more carpenter for beams, lintels and walls; 2 more carpenters for stairs and columns; 2 more carpenters for T & G timberwork; 1 more carpenter for each additional storey height. Shuttering can be used a minimum of 3 times. Add timber plank 10 Sft for T & G work.

(12) Painting & Washing

1. **White washing three coats (for 100 Sft)**

Strained lime	1	Cft
Rice*	3/8	Lb
Workers	3/8	Man-day
Sundries including brushes		

* Use liquid of glue instead of rice.

2. **Painting three coats with white zinc (for 1000 Sft)**

Priming coat red lead	30	Lbs
White zinc paint for 2nd & 3rd coats	45	Lbs
Putty	4	Lbs
Painters	10	Man-day
Workers	10	Man-day
Sundries including brushes		

3. **Painting three coats (new work) with ready mixed paint of any approved colour (for 1000 Sft)**

Ready mixed paint	75	Lbs
Putty	4	Lbs
Painters	10	Man-day
Workers	10	Man-day
Sundries including brushes		

4. **Painting three coats to wood work in posts, chowkets, facia boards, eave boards and stringers (for 1000 Sft)**

Ready mixed paint	75	Lbs
Putty	4	Lbs
Painters	12½	Man-day
Workers	10	Man-day

(13) Miscellaneous Notes

Allowing wastage of building materials

The quantities of material given below allow for breakages, wastage, carriage, etc.

(A)	Scantlings (all timbers)		10 %
(B)	Small timber 2"x 2" cross section and below		15 %
(C)	Doors and windows		15 %
(D)	X. P. M. wire netting		10 %
(E)	Glass		50 %
(F)	Roofing tiles		
	(i)	Mangalore pattern	20 %
	(ii)	Cement roof tiles	10 %
	(iii)	Asbestos cement sheets	10 %
(G)	Floor tiles	5 %	
	Marble slabs	7 %	
(H)	Plywood	15 %	
(I)	Steel rods	5 %	

(13) Miscellaneous Notes *contd.*

Quantity of timber of various sizes per ton

1 ton = 50 ft^3	3" x 1" = 2400'	4" x 4" = 450'	6" x 1" = 1200'	10" x 1" = 720'
1" x ½" = 14400'	3" x 2" = 1200'	5" x 1" = 1440'	6" x 2" = 600'	10" x 2" = 360'
1½"x ½" = 9600'	3" x 3" = 800'	5" x 2" = 720'	6" x 6" = 200'	10"x10" = 72'
2" x ½" = 7200'	4" x 1" = 1800'	5" x 3" = 480'	8" x 1" = 900'	12" x 1" = 600'
2" x 1" = 3600'	4" x 2" = 900'	5" x 4" = 360'	8" x 2" = 450'	12" x 2" = 300'
2" x 2" = 1800'	4" x 3" = 600'	5" x 5" = 288'	8" x 8" = 112.5'	12"x12" = 50'

Properties of various kinds of Myanmar timber

Type	Specific gravity (lb/ft^3)	Moisture content (%)	Bending strength (lb/in^2)	Modulus of elasticity (1000 lb/in^2)	Compression parallel(lb/in^2)	Hardness (Radial)(lb)	Impact max. drop (in.)
In	56 (Green) 53 (Dry)	50.3	11595	1754	5640	1420	40
Ingyin	53-58 (Dry)	54.3	12830	2021	6835	1510	40
Kanyin	70 (Green) 49 (Dry)	73.4	9410	1478	4530	910	29
Teak	55 (Green) 40 (Dry)	49.4	11460	1640	5710	980	36
Padauk	67 (Green) 54 (Dry)	43.8	15975	1897	8200	2010	52
Pyingadoe	72 (Green) 56 (Dry)	48.6	15555	2265	8015	1925	43
Thityar	78 (Green) 64 (Dry)	46.3	14305	2339	8220	1865	46

(13) Miscellaneous Notes *contd.*

Weight of Angles

Size of angles	Lbs per rft	Size of angles	Lbs per rft	Size of angles	Lbs per rft
1 1/4" x 1 1/4" x 1/4"	1.91	3 ½" x 3 ½" x 1/4"	5.74	3" x 2 ½" x 3/8"	6.54
1 1/4" x 1 1/4" x 1/8"	1.01	4" x 4" x 3/4"	18.49	3" x 2½" x 1/4"	4.47
1 ½" x 1 ½" x 1/4"	2.34	4" x 4" x 5/8"	15.68	3 ½" x 2 ½" x 3/8"	7.17
1 ½" x 1 ½" x 3/16"	1.79	4" x 4" x 1/2"	12.75	3 ½" x 2 ½" x 1/4"	4.89
2" x 2" x 3/8"	4.62	4" x 4" x 3/8"	9.73	3 ½" x 3" x 1/2"	10.20
2" x 2" x 1/4"	3.19	6" x 6" x 7/8"	31.10	3 ½" x 3" x 3/8"	7.81
2" x 2" x 3/16"	2.43	6" x 6" x 3/4"	28.69	3 ½" x 3" x 1/4"	5.32
2 ½" x 2 ½" x 1/2"	7.65	6" x 6" x 5/8"	24.17	4" x 2 ½" x 3/8"	7.81
2 ½" x 2 ½" x 3/8"	5.90	6" x 6" x 1/2"	19.55	4" x 2 ½" x 1/4"	5.32
2 ½" x 2½" x 1/4"	4.04	6" x 6" x 3/8"	14.82	4" x 3" x 1/2"	11.05
3" x 3" x 1/2"	9.35	2" x 1 ½" x 1/4"	2.76	4" x 3" x 3/8"	8.45
3" x 3" x 3/8"	7.17	2 ½" x 1 ½" x 1/4"	3.19	4" x 3 ½" x 5/8"	14.61
3" x 3" x 1/4"	4.89	2 ½" x 2" x 3/8"	5.26	4" x 3 ½" x 1/2"	11.91
3 ½" x 3 ½" x 5/8"	13.55	2 ½" x 2" x 1/4"	3.61	4" x 3 ½" x 3/8"	9.09
3 ½" x 3 ½" x 1/2"	11.05	3" x 2" x 3/8"	5.90	5" x 3" x 1/2"	12.75
3 ½" x 3 ½" x 3/8"	8.45	3" x 2" x 1/4"	4.04	5" x 3" x 3/8"	9.73

(12) Miscellaneous Notes *contd.*

Approximate Number of Galvanized Corrugated Sheets per Ton

Thickness	Corrugation	6	6 ½	7	7 ½	8	8 ½	9	9 ½	10
16 B. G.	8/3	58	54	50	47	44	41	39	37	35
16 B. G.	10/3	49	43	42	39	37	35	33	31	29
18 B. G.	8/3	74	68	64	59	56	52	49	46	44
18 B. G.	10/3	62	56	53	50	46	43	41	39	37
20 B. G.	8/3	95	88	81	76	71	67	63	60	57
20 B. G.	10/3	79	73	68	64	59	56	53	50	47
22 B. G.	8/3	116	107	99	93	87	82	77	73	69
22 B. G.	10/3	97	90	83	78	73	68	65	61	58
24 B. G.	8/3	140	130	120	112	105	98	93	88	84
24 B. G.	10/3	117	108	100	94	88	83	78	74	70
26 B. G.	8/3	185	172	159	149	139	131	124	117	111
26 B. G.	10/3	155	143	133	124	116	109	103	98	93
28 B. G.	8/3	200	185	172	161	150	141	133	126	120
28 B. G.	10/3	167	154	143	133	125	118	111	105	100
30 B. G.	8/3	240	222	206	192	180	170	160	151	144

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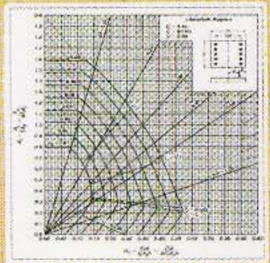


APPENDICES

A. Reinforced Concrete Design Aids
B. Reinforced Concrete Design Data and Equations
C. Analysis Formulae
D. Mensuration
E. Length of Bent Bars
F. Conversion of Units

APPENDIX (A) : Reinforced Concrete Design Aids

TABLE A.1

Size , diameter , area and weight of standard bars

Bar Size No. (or) mm		Diameter in.	Cross - Sectional Area , in²	Nominal Weight , lb/ft
6 mm		0.236	0.044	0.149
	No. 2	0.250	0.049	0.167
6.5 mm		0.256	0.051	0.175
8 mm		0.315	0.078	0.265
	No. 3	0.375	0.110	0.376
† 10 mm	† 9.5 mm	0.374	0.110	0.374
	10 mm	0.394	0.122	0.414
12 mm		0.472	0.175	0.595
† 13 mm	† 12.7 mm	0.500	0.196	0.668
	No. 4	0.500	0.196	0.668
No. 5		0.625	0.307	1.044
† 16 mm	† 15.9 mm	0.626	0.308	1.047
	16 mm	0.630	0.312	1.060
18 mm		0.709	0.395	1.342
	No. 6	0.750	0.442	1.503
† 19 mm	† 19.1 mm	0.752	0.444	1.511
20 mm		0.787	0.487	1.656
† 22 mm	22 mm	0.866	0.589	2.004
	† 22.2 mm	0.874	0.600	2.041
	No. 7	0.875	0.602	2.045
24 mm		0.945	0.702	2.385
† 25 mm	25 mm	0.984	0.761	2.588
	† 25.4 mm	1.000	0.786	2.671
	No. 8	1.000	0.786	2.671
	No. 9	1.128	1.000	3.399
	No. 10	1.270	1.267	4.309
	No. 11	1.410	1.562	5.311

** Area = $0.7857\, d^2$ (in^2)*
** Weight = $3.400 \times$ area (lb/ft)*
** The nominal diameter of a deformed bar is the equivalent diameter of a round plain bar having the same weight per foot as the deformed bar.*
† Soft metric bars

TABLE A.2

Area of groups of standard bars, in²

Bar Size No.(or) mm		Number of Bars 1	2	3	4	5	6	7	8	9	10	11	12
6 mm		0.044	0.088	0.132	0.175	0.219	0.263	0.307	0.351	0.395	0.438	0.482	0.526
	No. 2	0.049	0.098	0.147	0.196	0.246	0.295	0.344	0.393	0.442	0.491	0.540	0.589
6.5 mm		0.051	0.103	0.154	0.206	0.257	0.309	0.360	0.412	0.463	0.515	0.566	0.617
8 mm		0.078	0.156	0.234	0.312	0.390	0.468	0.546	0.624	0.701	0.779	0.857	0.935
	No. 3	0.110	0.221	0.331	0.442	0.552	0.663	0.773	0.884	0.994	1.105	1.215	1.326
10 mm	9.5 mm	0.110	0.220	0.330	0.440	0.550	0.659	0.769	0.879	0.989	1.099	1.209	1.319
	10.0 mm	0.122	0.244	0.365	0.487	0.609	0.731	0.852	0.974	1.096	1.218	1.340	1.461
12 mm		0.175	0.351	0.526	0.701	0.877	1.052	1.228	1.403	1.578	1.754	1.929	2.104
13 mm	12.7 mm	0.196	0.393	0.589	0.786	0.982	1.179	1.375	1.571	1.768	1.964	2.161	2.357
	No. 4	0.196	0.393	0.589	0.786	0.982	1.179	1.375	1.571	1.768	1.964	2.161	2.357
	No. 5	0.307	0.614	0.921	1.228	1.535	1.841	2.148	2.455	2.762	3.069	3.376	3.683
16 mm	15.9 mm	0.308	0.616	0.924	1.232	1.539	1.847	2.155	2.463	2.771	3.079	3.387	3.695
	16 mm	0.312	0.624	0.935	1.247	1.559	1.871	2.182	2.494	2.806	3.118	3.429	3.741
18 mm		0.395	0.789	1.184	1.578	1.973	2.367	2.762	3.157	3.551	3.946	4.340	4.735
	No. 6	0.442	0.884	1.326	1.768	2.210	2.652	3.094	3.536	3.978	4.420	4.862	5.303
19 mm		0.444	0.889	1.333	1.777	2.221	2.666	3.110	3.554	3.999	4.443	4.887	5.331
20 mm		0.487	0.974	1.461	1.949	2.436	2.923	3.410	3.897	4.384	4.871	5.358	5.846
22 mm	22 mm	0.589	1.179	1.768	2.358	2.947	3.537	4.126	4.715	5.305	5.894	6.484	7.073
	22.2 mm	0.600	1.200	1.801	2.401	3.001	3.601	4.201	4.802	5.402	6.002	6.602	7.202
	No. 7	0.602	1.203	1.805	2.406	3.008	3.609	4.211	4.812	5.414	6.016	6.617	7.219
24 mm		0.701	1.403	2.104	2.806	3.507	4.209	4.910	5.612	6.313	7.015	7.716	8.418
25 mm	25 mm	0.761	1.522	2.283	3.045	3.806	4.567	5.328	6.089	6.850	7.611	8.373	9.134
	25.4 mm	0.786	1.571	2.357	3.143	3.929	4.714	5.500	6.286	7.071	7.857	8.643	9.428
	No. 8	0.786	1.571	2.357	3.143	3.929	4.714	5.500	6.286	7.071	7.857	8.643	9.428
	No. 9	1.000	1.999	2.999	3.999	4.999	5.998	6.998	7.998	8.997	9.997	10.997	11.997
	No. 10	1.267	2.535	3.802	5.069	6.336	7.604	8.871	10.138	11.405	12.673	13.940	15.207
	No. 11	1.562	3.124	4.686	6.248	7.810	9.372	10.934	12.496	14.058	15.621	17.183	18.745

TABLE A.3

Area of bars in slabs, in^2/ft

Bar Size	Area of one	Spacing of Bars, in													
No (or) mm	bar, A_{bar}	3	3 1/2	4	4 1/2	5	5 1/2	6	6 1/2	7	7 1/2	8	9	10	12
6 mm	0.044	0.175	0.150	0.131	0.117	0.105	0.096	0.088	0.081	0.075	0.070	0.066	0.058	0.053	0.044
No. 2	0.049	0.196	0.168	0.147	0.131	0.118	0.107	0.098	0.091	0.084	0.079	0.074	0.065	0.059	0.049
6.5 mm	0.051	0.206	0.176	0.154	0.137	0.123	0.112	0.103	0.095	0.088	0.082	0.077	0.069	0.052	0.051
8 mm	0.078	0.312	0.267	0.234	0.208	0.187	0.170	0.156	0.144	0.134	0.125	0.117	0.104	0.093	0.078
No. 3	0.110	0.442	0.379	0.331	0.295	0.265	0.241	0.221	0.204	0.189	0.177	0.166	0.147	0.133	0.110
10 mm	0.122	0.487	0.417	0.365	0.325	0.292	0.266	0.243	0.225	0.209	0.195	0.183	0.162	0.146	0.122
12 mm	0.175	0.701	0.601	0.526	0.467	0.421	0.382	0.351	0.324	0.301	0.280	0.263	0.234	0.210	0.175
No. 4	0.196	0.785	0.673	0.589	0.524	0.471	0.428	0.393	0.362	0.337	0.314	0.295	0.262	0.236	0.196
No. 5	0.307	1.227	1.052	0.920	0.818	0.736	0.669	0.614	0.566	0.526	0.491	0.460	0.409	0.358	0.307
16 mm	0.312	1.247	1.069	0.935	0.831	0.748	0.680	0.623	0.575	0.534	0.499	0.467	0.416	0.374	0.312
18 mm	0.394	1.578	1.352	1.183	1.052	0.947	0.861	0.789	0.728	0.676	0.631	0.592	0.526	0.473	0.394
No. 6	0.442	1.767	1.515	1.325	1.178	1.060	0.964	0.884	0.816	0.757	0.707	0.663	0.589	0.530	0.442
20 mm	0.487	1.948	1.670	1.461	1.200	1.160	1.062	0.974	0.899	0.835	0.779	0.730	0.649	0.584	0.487
22 mm	0.589	2.357	2.020	1.768	1.571	1.414	1.286	1.178	1.088	1.010	0.943	0.884	0.786	0.707	0.589
No. 7	0.601	2.405	2.062	1.804	1.604	1.443	1.312	1.203	1.110	1.031	0.962	0.902	0.802	0.722	0.601
24 mm	0.701	2.805	2.404	2.104	1.870	1.683	1.530	1.402	1.295	1.202	1.122	1.052	0.935	0.841	0.701
25 mm	0.761	3.043	2.609	2.283	2.029	1.826	1.660	1.522	1.405	1.304	1.217	1.141	1.014	0.913	0.761
No. 8	0.785	3.142	2.693	2.356	2.094	1.885	1.714	1.571	1.450	1.346	1.257	1.178	1.047	0.942	0.785

* $\text{Area } (in^2/ft) = \dfrac{12}{\text{spacing (in)}} \times A_{bar}$

TABLE A.4

Limiting steel reinforcement ratios for tension-controlled members

f_y, psi	f'_c, psi	β_1	ρ^a $\epsilon_t = 0.005$	$\rho_{max}{}^a$ $\epsilon_t = 0.004$	$\rho_{min} = \frac{200}{f_y}$	$\rho_{min} = \frac{3\sqrt{f'_c}}{f_y}$
40,000	3000	0.85	0.0203	0.0232	0.0050	0.0041
	4000	0.85	0.0271	0.0310	0.0050	0.0047
	5000	0.80	0.0319	0.0364	0.0050	0.0053
	6000	0.75	0.0359	0.0410	0.0050	0.0058
	7000	0.70	0.0390	0.0446	0.0050	0.0063
	8000	0.65	0.0414	0.0474	0.0050	0.0067
	9000	0.65	0.0466	0.0533	0.0050	0.0071
50,000	3000	0.85	0.0163	0.0186	0.0040	0.0033
	4000	0.85	0.0217	0.0248	0.0040	0.0038
	5000	0.80	0.0255	0.0291	0.0040	0.0042
	6000	0.75	0.0287	0.0328	0.0040	0.0046
	7000	0.70	0.0312	0.0357	0.0040	0.0050
	8000	0.65	0.0332	0.0379	0.0040	0.0054
	9000	0.65	0.0373	0.0426	0.0040	0.0057
60,000	3000	0.85	0.0135	0.0155	0.0033	0.0027
	4000	0.85	0.0181	0.0206	0.0033	0.0032
	5000	0.80	0.0213	0.0243	0.0033	0.0035
	6000	0.75	0.0239	0.0273	0.0033	0.0039
	7000	0.70	0.0260	0.0298	0.0033	0.0042
	8000	0.65	0.0276	0.0316	0.0033	0.0045
	9000	0.65	0.0311	0.0355	0.0033	0.0047
75,000	3000	0.85	0.0108	0.0124	0.0027	0.0022
	4000	0.85	0.0145	0.0165	0.0027	0.0025
	5000	0.80	0.0170	0.0194	0.0027	0.0028
	6000	0.75	0.0191	0.0219	0.0027	0.0031
	7000	0.70	0.0208	0.0238	0.0027	0.0033
	8000	0.65	0.0221	0.0253	0.0027	0.0036
	9000	0.65	0.0249	0.0284	0.0027	0.0038

[a] $\rho = 0.85\beta_1 \frac{f'_c}{f_y} \frac{0.003}{0.003 + \epsilon_t}$

TABLE A.5a

Flexural resistance factor: $R = \rho f_y \left(1 - 0.588 \frac{\rho f_y}{f'_c}\right)$ psi

ρ	f_y = 40,000 psi				f_y = 60,000 psi			
	f'_c, psi				f'_c, psi			
	3000	4000	5000	6000	3000	4000	5000	6000
0.0005	20	20	20	20	30	30	30	30
0.0010	40	40	40	40	59	59	60	60
0.0015	59	59	60	60	88	89	89	89
0.0020	79	79	79	79	117	118	118	119
0.0025	98	99	99	99	146	147	147	148
0.0030	117	118	118	119	174	175	176	177
0.0035	136	137	138	138	201	204	205	206
0.0040	155	156	157	157	229	232	233	234
0.0045	174	175	176	177	256	259	261	263
0.0050	192	194	195	196	282	287	289	291
0.0055	211	213	214	215	309	314	317	319
0.0060	229	232	233	234	335	341	345	347
0.0065	247	250	252	253	360	368	372	375
0.0070	265	268	271	272	385	394	399	403
0.0075	282	287	289	291	410	420	426	430
0.0080	300	305	308	310	435	446	453	457
0.0085	317	323	326	329	459	472	479	485
0.0090	335	341	345	347	483	497	506	511
0.0095	352	359	363	366	506	522	532	538
0.0100	369	376	381	384	529	547	558	565
0.0105	385	394	399	403	552	572	583	591
0.0110	402	412	417	421	575	596	609	617
0.0115	419	429	435	439	597	620	634	643
0.0120	435	446	453	457	618	644	659	669
0.0125	451	463	471	476	640	667	684	695
0.0130	467	480	488	494	661	691	708	720
0.0135	483	497	506	511	681	714	733	746
0.0140	499	514	523	529	702	736	757	771
0.0145	514	531	540	547	722	759	781	796
0.0150	529	547	558	565	741	781	805	821
0.0155	545	563	575	582	760	803	828	845
0.0160	560	580	592	600		825	852	870
0.0165	575	596	609	617		846	875	894
0.0170	589	612	626	635		867	898	918
0.0175	604	628	642	652		888	920	942
0.0180	618	644	659	669		909	943	966
0.0185	633	660	676	686		929	965	989
0.0190	647	675	692	703		949	987	1013
0.0195	661	691	708	720		969	1009	1036
0.0200	675	706	725	737		988	1031	1059

TABLE A.5b

Flexural resistance factor : $R = \rho f_y \left(1 - 0.588 \frac{\rho f_y}{f'_c}\right)$ **psi**

	f_y = 40,000 psi				f_y = 60,000 psi			
	f'_c, psi				f'_c, psi			
ρ	3000	4000	5000	6000	3000	4000	5000	6000
0.003	117	118	118	119	174	175	176	177
0.004	155	156	157	157	229	232	233	234
0.005	192	194	195	196	282	287	289	291
0.006	229	232	233	234	335	341	345	347
0.007	265	268	271	272	385	394	399	403
0.008	300	305	308	310	435	446	453	457
0.009	335	341	345	347	483	497	506	511
0.010	369	376	381	384	529	547	558	565
0.011	402	412	417	421	575	596	609	617
0.012	435	446	453	457	618	644	659	669
0.013	467	480	488	494	661	691	708	720
0.014	499	514	523	529	702	736	757	771
0.015	529	547	558	565	741	781	805	821
0.016	560	580	592	600	779	825	852	870
0.017	589	612	626	635		867	898	918
0.018	618	644	659	669		909	943	966
0.019	647	675	692	703		949	987	1013
0.020	675	706	725	737		988	1031	1059
0.021	702	736	757	771			1073	1104
0.022	728	766	789	804			1115	1149
0.023	754	796	820	837			1156	1193
0.024		825	852	870			1196	1237
0.025		853	882	902				1280
0.026		881	913	934				1322
0.027		909	943	966				1363
0.028		936	972	997				
0.029		962	1002	1028				
0.030		988	1031	1059				
0.031		1014	1059	1089				
0.032			1087	1119				
0.033			1115	1149				
0.034			1142	1179				
0.035			1170	1208				
0.036			1196	1237				
0.037				1265				
0.038				1294				
0.039				1322				
0.040				1349				
0.041				1376				

Table A.6a

Maximum number of bars as a single layer in beam stems

$\frac{3}{4}$ in. Maximum Size Aggregate, No. 4 (# 13) Stirrups[a]

Bar No.		Beam Width b_w, in.											
Inch-Pound	SI	8	10	12	14	16	18	20	22	24	26	28	30
5	16	2	4	5	6	7	8	10	11	12	13	15	16
6	19	2	3	4	6	7	8	9	10	11	12	14	15
7	22	2	3	4	5	6	7	8	9	10	11	12	13
8	25	2	3	4	5	6	7	8	9	10	11	12	13
9	29	1	2	3	4	5	6	7	8	9	9	10	11
10	32	1	2	3	4	5	6	6	7	8	9	10	10
11	36	1	2	3	3	4	5	5	6	7	8	8	9
14	43	1	2	2	3	3	4	5	5	6	6	7	8
18	57	1	1	2	2	3	3	4	4	4	5	5	6

1 in. Maximum Size Aggregate, No. 4 (# 13) Stirrups[a]

Bar No.		Beam Width b_w, in.											
Inch-Pound	SI	8	10	12	14	16	18	20	22	24	26	28	30
5	16	2	3	4	5	6	7	8	9	0	11	12	13
6	19	2	3	4	5	6	7	8	9	9	10	11	12
7	22	1	2	3	4	5	6	7	8	9	10	10	11
8	25	1	2	3	4	5	6	7	7	8	9	10	11
9	29	1	2	3	4	5	6	7	7	8	9	9	10
10	32	1	2	3	4	5	6	6	7	7	8	9	10

[a]Minimum concrete cover assumed to be $1\frac{1}{2}$ in. to the No. 4 (No. 13) stirrup.

TABLE A.6b
Maximum number of bars as a single layer in beam stems

(For max. size of aggregate = 1 in. : stirrups ; 8 mm if $b_w \le 12$ in ; 10 mm if $b_w > 12$ in.)

Criterion : min. clear spacing bet. bars ≥ 4/3 max. size of agg.(= 1.33 in.) or one bar dia.; clear cover over stirrups = 1.5 in.

Bar Size	Beam Stem Width, b_w , in.									
No. (or) mm	8	9	10	12	14	16	18	20	22	24
6 mm	3	4	4	6	7	8	9	11	12	13
No. 2	3	4	4	6	7	8	9	11	12	13
6.5 mm	3	4	4	6	7	8	9	11	12	13
8 mm	3	4	4	5	7	8	9	10	11	13
No. 3	3	3	4	5	6	7	9	10	11	12
10 mm	3	3	4	5	6	7	9	10	11	12
12 mm	3	3	4	5	6	7	8	9	10	11
No. 4	3	3	4	5	6	7	8	9	10	11
No. 5	2	3	3	4	5	6	7	8	9	11
16 mm	2	3	3	4	5	6	7	8	9	10
18 mm	2	3	3	4	5	6	7	8	9	10
No. 6	2	3	3	4	5	6	7	8	9	10
20 mm	2	3	3	4	5	6	7	8	9	10
22 mm	2	3	3	4	5	6	7	7	8	9
No. 7	2	3	3	4	5	6	7	7	8	9
24 mm	2	2	3	4	5	5	6	7	8	9
25 mm	2	2	3	4	4	5	6	7	8	9
No. 8	2	2	3	4	4	5	6	7	8	9

Table A.7

Design strength $\emptyset M_n$ for slab sections 12 in. wide, kips-ft ; $f_y = 60$ ksi;

$$\phi M_n = \phi \rho f_y b d^2 (1 - 0.59 \rho f_y / f'_c)$$

f'_c, psi	ρ	Effective Depth d, in. 3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	8.0	9.0	10.0	12.0
3000	0.002	0.9	1.3	1.7	2.1	2.6	3.2	3.8	4.5	5.2	6.7	8.5	10.5	15.2
	0.003	1.4	1.9	2.5	3.2	3.9	4.7	5.6	6.6	7.7	10.0	12.7	15.6	22.5
	0.004	1.9	2.5	3.3	4.2	5.1	6.2	7.4	8.7	10.1	13.2	16.7	20.6	29.6
	0.005	2.3	3.1	4.1	5.1	6.4	7.7	9.1	10.7	12.4	16.3	20.6	25.4	36.6
	0.006	2.7	3.7	4.8	6.1	7.5	9.1	10.8	12.7	14.8	19.3	24.4	30.1	43.4
	0.007	3.1	4.2	5.5	7.0	8.7	10.5	12.5	14.7	17.0	22.2	28.1	34.7	49.9
	0.008	3.5	4.8	6.3	7.9	9.8	11.8	14.1	16.5	19.2	25.0	31.7	39.1	56.3
	0.009	3.9	5.3	7.0	8.8	10.9	13.1	15.6	18.4	21.3	27.8	35.2	43.4	62.6
	0.010	4.3	5.8	7.6	9.6	11.9	14.4	17.1	20.1	23.3	30.5	38.6	47.6	68.6
	0.011	4.7	6.3	8.3	10.5	12.9	15.6	18.6	21.8	25.3	33.1	41.9	51.7	74.4
4000	0.002	1.0	1.3	1.7	2.1	2.7	3.2	3.8	4.5	5.2	6.8	8.6	10.6	15.3
	0.003	1.4	1.9	2.5	3.2	3.9	4.8	5.7	6.7	7.7	10.1	12.8	15.8	22.7
	0.004	1.9	2.6	3.3	4.2	5.2	6.3	7.5	8.8	10.2	13.3	16.9	20.8	30.0
	0.005	2.3	3.2	4.1	5.2	6.5	7.8	9.3	10.9	12.6	16.5	20.9	25.8	37.2
	0.006	2.8	3.8	4.9	6.2	7.7	9.3	11.0	13.0	15.0	19.6	24.9	30.7	44.2
	0.007	3.2	4.3	5.7	7.2	8.9	10.7	12.8	15.0	17.4	22.7	28.7	35.5	51.1
	0.008	3.6	4.9	6.4	8.1	10.0	12.1	14.5	17.0	19.7	25.7	32.5	40.1	57.8
	0.009	4.0	5.5	7.2	9.1	11.2	13.5	16.1	18.9	21.9	28.6	36.2	44.7	64.4
	0.010	4.4	6.0	7.9	10.0	12.3	14.9	17.7	20.8	24.1	31.5	39.9	49.2	70.9
	0.011	4.8	6.6	8.6	10.9	13.4	16.2	19.3	22.7	26.3	34.3	43.4	53.6	77.2
	0.012	5.2	7.1	9.3	11.7	14.5	17.5	20.9	24.5	28.4	37.1	46.9	57.9	83.4
	0.013	5.6	7.6	9.9	12.6	15.5	18.8	22.4	26.2	30.4	39.8	50.3	62.1	89.5
	0.014	6.0	8.1	10.6	13.4	16.6	20.0	23.8	28.0	32.5	42.4	53.6	66.2	95.4
	0.015	6.3	8.6	11.2	14.2	17.6	21.2	25.3	29.7	34.4	45.0	56.9	70.2	101.2
5000	0.002	1.0	1.3	1.7	2.2	2.7	3.2	3.8	4.5	5.2	6.8	8.6	10.6	15.3
	0.003	1.4	1.9	2.5	3.2	4.0	4.8	5.7	6.7	7.8	10.1	12.8	15.9	22.8
	0.004	1.9	2.6	3.4	4.3	5.2	6.3	7.6	8.9	10.3	13.4	17.0	21.0	30.2
	0.005	2.3	3.2	4.2	5.3	6.5	7.9	9.4	11.0	12.8	16.7	21.1	26.0	37.5
	0.006	2.8	3.8	5.0	6.3	7.8	9.4	11.2	13.1	15.2	19.9	25.1	31.0	44.7
	0.007	3.2	4.4	5.7	7.3	9.0	10.9	12.9	15.2	17.6	23.0	29.1	35.9	51.7
	0.008	3.7	5.0	6.5	8.3	10.2	12.3	14.7	17.2	20.0	26.1	33.0	40.8	58.7
	0.009	4.1	5.6	7.3	9.2	11.4	13.8	16.4	19.2	22.3	29.1	36.9	45.5	65.5
	0.010	4.5	6.1	8.0	10.2	12.5	15.2	18.1	21.2	24.6	32.1	40.6	50.2	72.3
	0.011	4.9	6.7	8.8	11.1	13.7	16.6	19.7	23.1	26.8	35.1	44.4	54.8	78.9
	0.012	5.3	7.3	9.5	12.0	14.8	17.9	21.3	25.1	29.1	37.9	48.0	59.3	85.4
	0.013	5.7	7.8	10.2	12.9	15.9	19.3	22.9	26.9	31.2	40.8	51.6	63.7	91.8
	0.014	6.1	8.3	10.9	13.8	17.0	20.6	24.5	28.8	33.4	43.6	55.2	68.1	98.1
	0.015	6.5	8.9	11.6	14.7	18.1	21.9	26.1	30.6	35.5	46.3	58.6	72.4	104.3
	0.016	6.9	9.4	12.3	15.5	19.2	23.2	27.6	32.4	37.5	49.0	62.1	76.6	110.3
	0.017	7.3	9.9	12.9	16.4	20.2	24.4	29.1	34.1	39.6	51.7	65.4	80.8	116.3

Table A.8

Allowable maximum number of bars on one side of column section

(For 1 " maximum aggegrate size ; clear spacing ≥ 1.5 in)

Side width (in)	Allowable max. no. of bars on one side						
	# 12	# 16	# 18	# 20	# 22	# 24	# 25
6	4	4	-	-	-	-	-
7	4	4	-	-	-	-	-
8	8	4	-	-	-	-	-
9	3	2	2	2	-	-	-
10	3	3	3	3	-	-	-
12	4	4	4	4	3	3	3
14	5	5	5	4	4	4	4
16	6	6	6	5	5	5	5
18	7	7	6	6	6	6	6
20	-	8	7	7	7	7	7
22	-	9	8	8	8	7	7
24	-	9	9	9	9	8	8
26	-	-	10	10	9	9	9
28	-	-	11	11	10	10	10
30	-	-	12	11	11	11	11
32	-	-	-	12	12	12	11
34	-	-	-	13	13	12	12
36	-	-	-	14	14	13	13

Note : cover to bar centre is 2.5 in. for all sizes except 2 in. for 6" x 6" , 7" x 7" and 8" x 8" props.

TABLE A. 9
Coefficient for negative moments in slabs[a]

$M_{a,neg} = C_{a,neg}\, w l_a^2$
$M_{b,neg} = C_{b,neg}\, w l_b^2$

where w = total uniform dead plus live load

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,neg}$		0.045		0.050	0.075	0.071		0.033	0.061
	$C_{b,neg}$		0.045	0.076	0.050			0.071	0.061	0.033
0.95	$C_{a,neg}$		0.050		0.055	0.079	0.075		0.038	0.065
	$C_{b,neg}$		0.041	0.072	0.045			0.067	0.056	0.029
0.90	$C_{a,neg}$		0.055		0.060	0.080	0.079		0.043	0.068
	$C_{b,neg}$		0.037	0.070	0.040			0.062	0.052	0.025
0.85	$C_{a,neg}$		0.060		0.066	0.082	0.083		0.049	0.072
	$C_{b,neg}$		0.031	0.065	0.034			0.057	0.046	0.021
0.80	$C_{a,neg}$		0.065		0.071	0.083	0.086		0.055	0.075
	$C_{b,neg}$		0.027	0.061	0.029			0.051	0.041	0.017
0.75	$C_{a,neg}$		0.069		0.076	0.085	0.088		0.061	0.078
	$C_{b,neg}$		0.022	0.056	0.024			0.044	0.036	0.014
0.70	$C_{a,neg}$		0.074		0.081	0.086	0.091		0.068	0.081
	$C_{b,neg}$		0.017	0.050	0.019			0.038	0.029	0.011
0.65	$C_{a,neg}$		0.077		0.085	0.087	0.093		0.074	0.083
	$C_{b,neg}$		0.014	0.043	0.015			0.031	0.024	0.008
0.60	$C_{a,neg}$		0.081		0.089	0.088	0.095		0.080	0.085
	$C_{b,neg}$		0.010	0.035	0.011			0.024	0.018	0.006
0.55	$C_{a,neg}$		0.084		0.092	0.089	0.096		0.085	0.086
	$C_{b,neg}$		0.007	0.028	0.008			0.019	0.014	0.005
0.50	$C_{a,neg}$		0.086		0.094	0.090	0.097		0.089	0.088
	$C_{b,neg}$		0.006	0.022	0.006			0.014	0.010	0.003

[a] A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

TABLE A. 10
Coefficient for dead load positive moments in slabs[a]

$M_{a,pos,dl} = C_{a,dl}\, w l_a^2$
$M_{b,pos,dl} = C_{b,dl}\, w l_b^2$

where w = total uniform dead load

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,dl}$	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
	$C_{b,dl}$	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020
0.95	$C_{a,dl}$	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
	$C_{b,dl}$	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
0.90	$C_{a,dl}$	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
	$C_{b,dl}$	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.85	$C_{a,dl}$	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
	$C_{b,dl}$	0.026	0.012	0.022	0.019	0.011	0.017	0.025	0.017	0.013
0.80	$C_{a,dl}$	0.056	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
	$C_{b,dl}$	0.023	0.011	0.020	0.016	0.009	0.015	0.022	0.015	0.010
0.75	$C_{a,dl}$	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
	$C_{b,dl}$	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
0.70	$C_{a,dl}$	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
	$C_{b,dl}$	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006
0.65	$C_{a,dl}$	0.074	0.032	0.054	0.050	0.036	0.054	0.065	0.044	0.034
	$C_{b,dl}$	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.60	$C_{a,dl}$	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
	$C_{b,dl}$	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.55	$C_{a,dl}$	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
	$C_{b,dl}$	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.50	$C_{a,dl}$	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
	$C_{b,dl}$	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

[a] A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

TABLE A. 11

Coefficient for live load positive moments in slabs[a]

$M_{a,pos,ll} = C_{a,ll}\, w l_a^2$

$M_{b,pos,ll} = C_{b,ll}\, w l_b^2$

where w = total uniform live load

Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,ll}$	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
	$C_{b,ll}$	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
0.95	$C_{a,ll}$	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
	$C_{b,ll}$	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
0.90	$C_{a,ll}$	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
	$C_{b,ll}$	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
0.85	$C_{a,ll}$	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
	$C_{b,ll}$	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
0.80	$C_{a,ll}$	0.056	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
	$C_{b,ll}$	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
0.75	$C_{a,ll}$	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
	$C_{b,ll}$	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.013
0.70	$C_{a,ll}$	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
	$C_{b,ll}$	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.011
0.65	$C_{a,ll}$	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
	$C_{b,ll}$	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
0.60	$C_{a,ll}$	0.081	0.058	0.071	0.067	0.059	0.068	0.077	0.065	0.059
	$C_{b,ll}$	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
0.55	$C_{a,ll}$	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063
	$C_{b,ll}$	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
0.50	$C_{a,ll}$	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
	$C_{b,ll}$	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004

[a] A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

TABLE A. 12

Ratio of load W in l_a and l_b directions for shear in slab and load on support

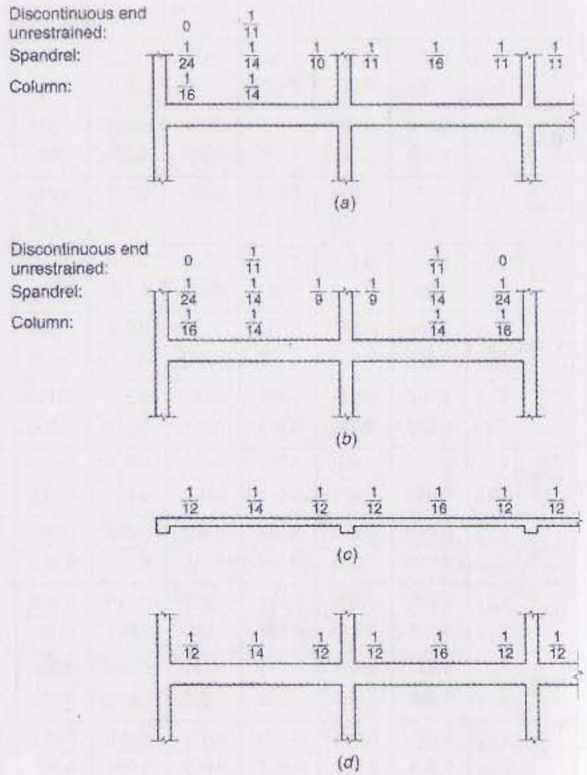
Ratio $m = \frac{l_a}{l_b}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	W_a	0.50	0.50	0.17	0.50	0.83	0.71	0.29	0.33	0.67
	W_b	0.50	0.50	0.83	0.50	.017	0.29	0.71	0.67	0.33
0.95	W_a	0.55	0.55	0.20	0.55	0.86	0.75	0.33	0.38	0.71
	W_b	0.45	0.45	0.80	0.45	0.14	0.25	0.67	0.62	0.29
0.90	W_a	0.60	0.60	0.23	0.60	0.88	0.79	0.38	0.43	0.75
	W_b	0.40	0.40	0.77	0.40	0.12	0.21	0.62	0.57	0.25
0.85	W_a	0.66	0.66	0.28	0.66	0.90	0.83	0.43	0.49	0.79
	W_b	0.34	0.34	0.72	0.34	0.10	0.17	0.57	0.51	0.21
0.80	W_a	0.71	0.71	0.33	0.71	0.92	0.86	0.49	0.55	0.83
	W_b	0.29	0.29	0.67	0.29	0.08	0.14	0.51	0.45	0.17
0.75	W_a	0.76	0.76	0.39	0.76	0.94	0.88	0.56	0.61	0.86
	W_b	0.24	0.24	0.61	0.24	0.06	0.12	0.44	0.39	0.14
0.70	W_a	0.81	0.81	0.45	0.81	0.95	0.91	0.62	0.68	0.89
	W_b	0.19	0.19	0.55	0.19	0.05	0.09	0.38	0.32	0.11
0.65	W_a	0.85	0.85	0.53	0.85	0.96	0.93	0.69	0.74	0.92
	W_b	0.15	0.15	0.47	0.15	0.04	0.07	0.31	0.26	0.08
0.60	W_a	0.89	0.89	0.61	0.89	0.97	0.95	0.76	0.80	0.94
	W_b	0.11	0.11	0.39	0.11	0.03	0.05	0.24	0.20	0.06
0.55	W_a	0.92	0.92	0.69	0.92	0.98	0.96	0.81	0.85	0.95
	W_b	0.08	0.08	0.31	0.08	0.02	0.04	0.19	0.15	0.05
0.50	W_a	0.94	0.94	0.76	0.94	0.99	0.97	0.86	0.89	0.97
	W_b	0.06	0.06	0.24	0.06	0.01	0.03	0.14	0.11	0.03

[a] A crosshatched edge indicates that the slab continues across, or is fixed at, the support; an unmarked edge indicates a support at which torsional resistance is negligible.

TABLE A. 13
Moment and shear values using ACI coefficients†

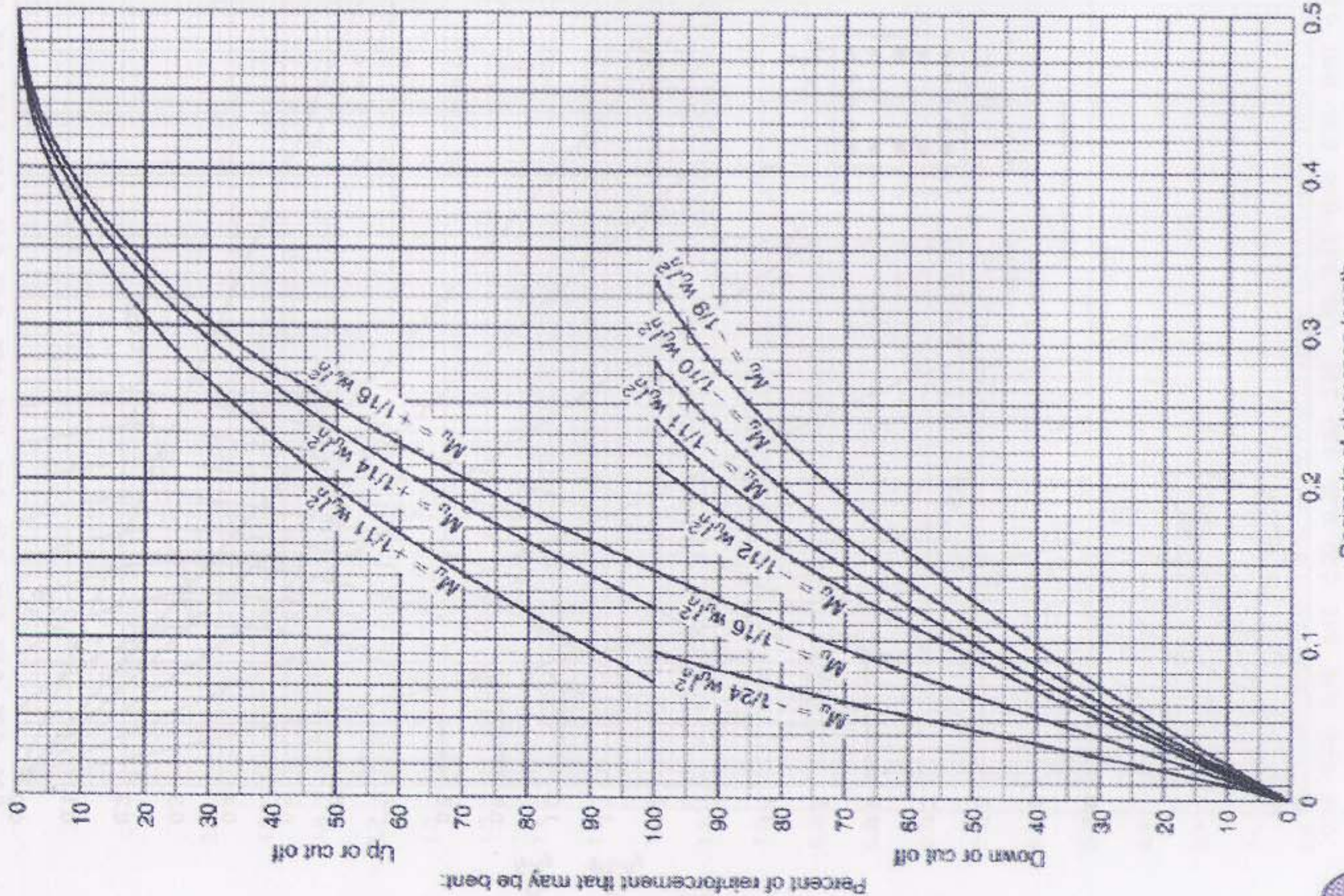
Positive moment	
End spans	
If discontinuous end is unrestrained	$\frac{1}{11} w_u l_n^2$
If discontinuous end is integral with the support	$\frac{1}{14} w_u l_n^2$
Interior spans	$\frac{1}{16} w_u l_n^2$
Negative moment at exterior face of first interior support	
Two spans	$\frac{1}{9} w_u l_n^2$
More than two spans	$\frac{1}{10} w_u l_n^2$
Negative moment at other faces of interior supports	$\frac{1}{11} w_u l_n^2$
Negative moment at face of all supports for (1) slabs with spans not exceeding 10 ft and (2) beams and girders where ratio of sum of column stiffness to beam stiffness exceeds 8 at each end of the span	$\frac{1}{12} w_u l_n^2$
Negative moment at interior faces of exterior supports for members built integrally with their supports	
Where the support is a spandrel beam or girder	$\frac{1}{24} w_u l_n^2$
Where the support is a column	$\frac{1}{16} w_u l_n^2$
Shear in end members at first interior support	$1.15 \frac{w_u l_n}{2}$
Shear at all other supports	$\frac{w_u l_n}{2}$

† w_u = total factored load per unit length of beam or per unit area of slab.
l_n = clear span for positive moment and shear and the average of the two adjacent clear spans for negative moment.



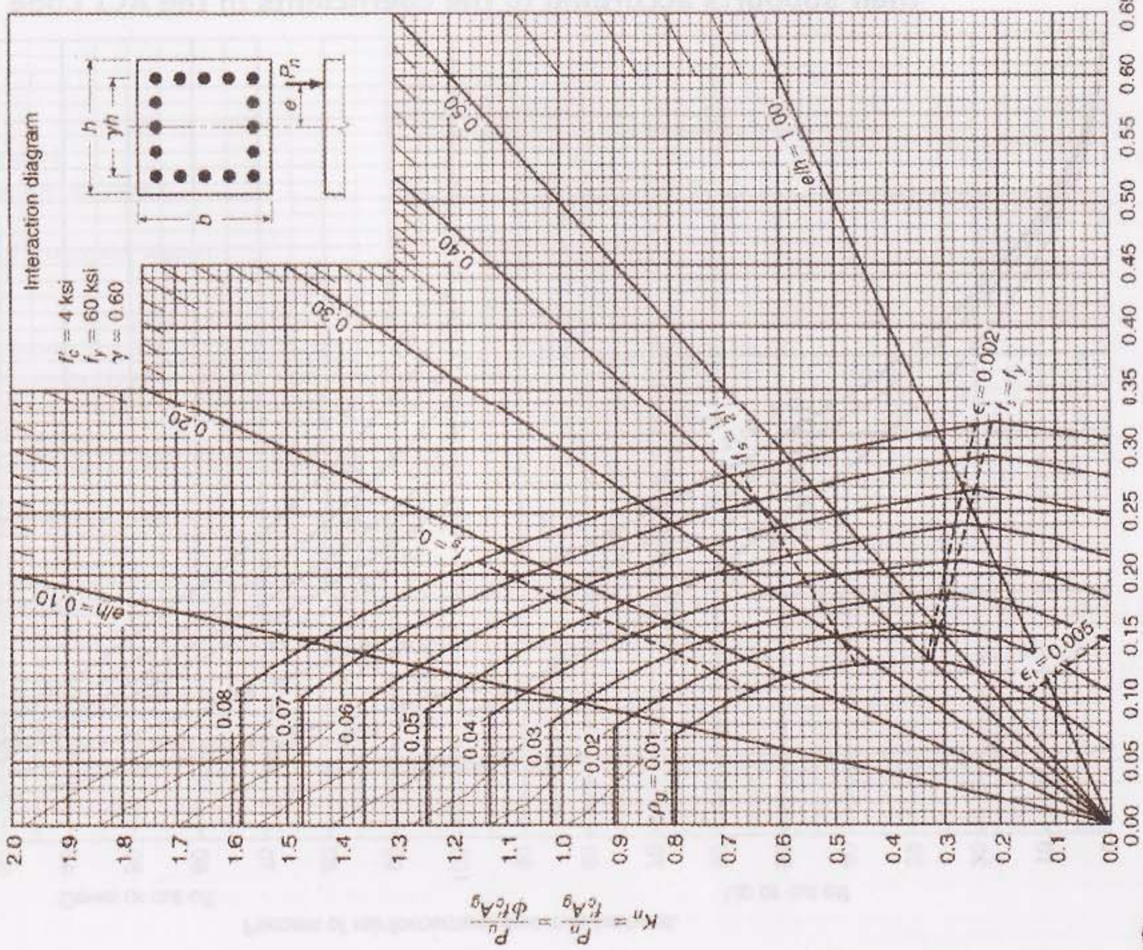
GRAPH A.1

Approximate location of theoretical points where bars can be bent up or down or cut off for continuous beams, uniformly loaded and built integrally with their supports according to the coefficients in the ACI Code



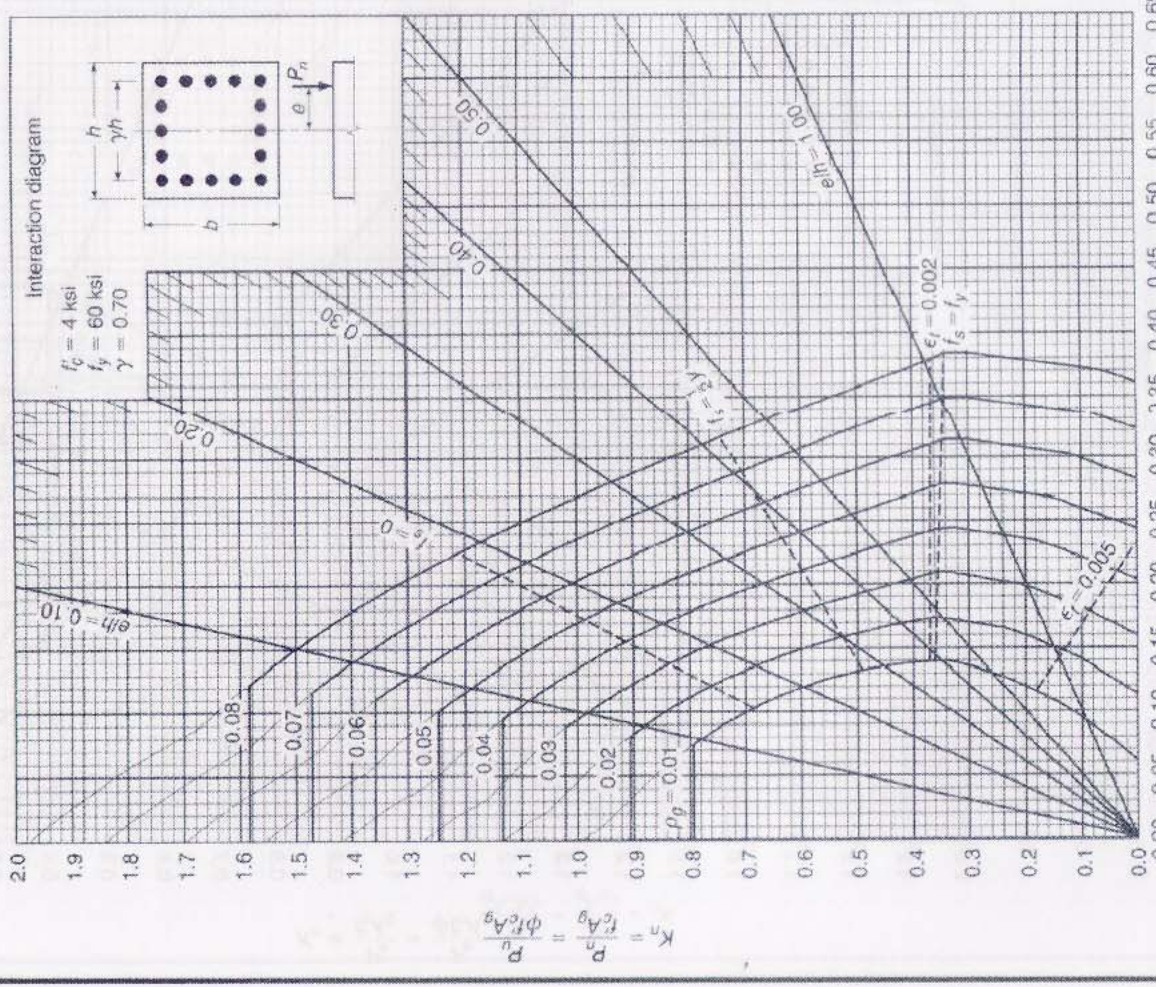
GRAPH A.2

Column strength interaction diagram for rectangular section with bars on four faces and $\gamma = 0.60$



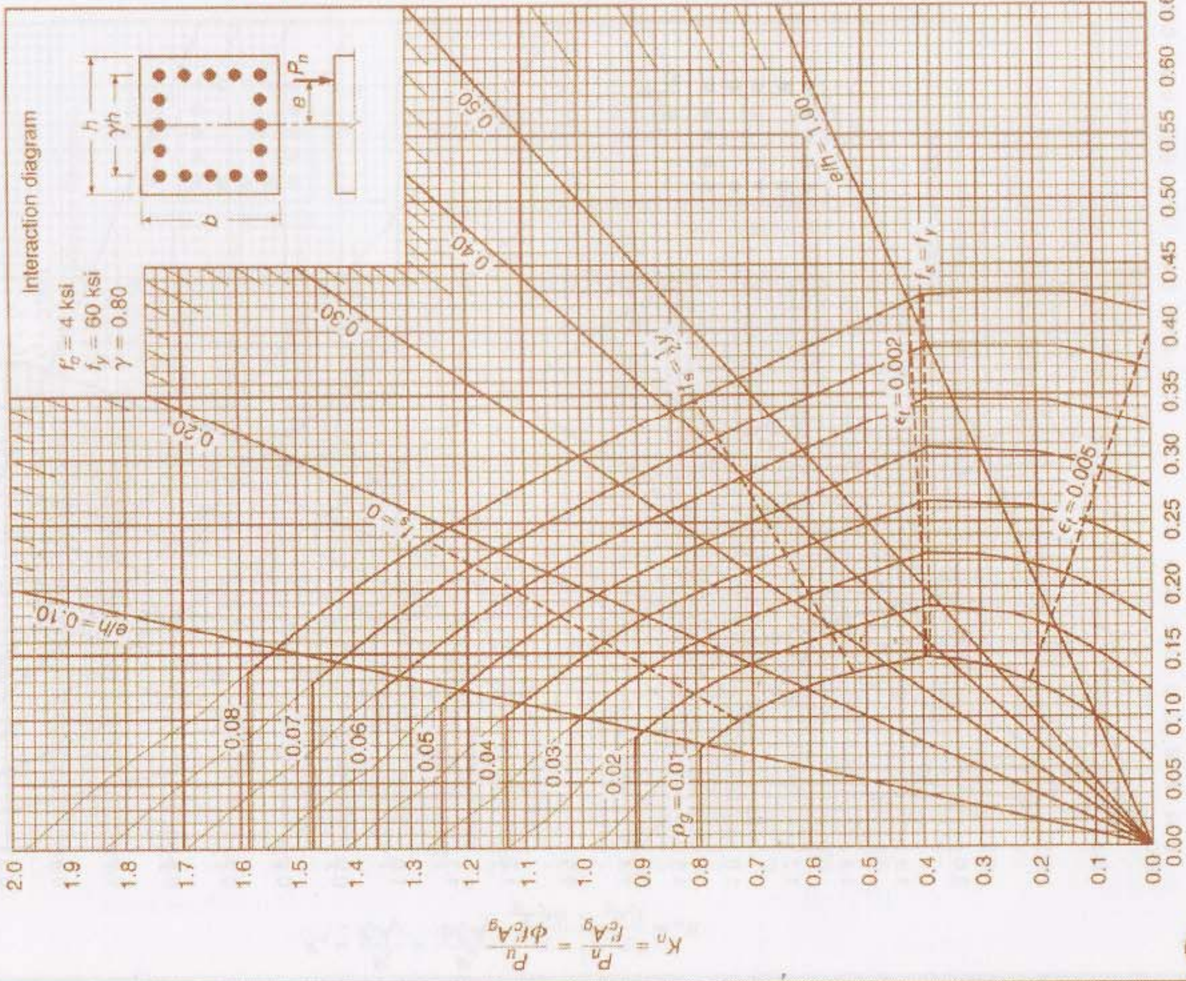
GRAPH A.3

Column strength interaction diagram for rectangular section with bars on four faces and $\gamma = 0.70$



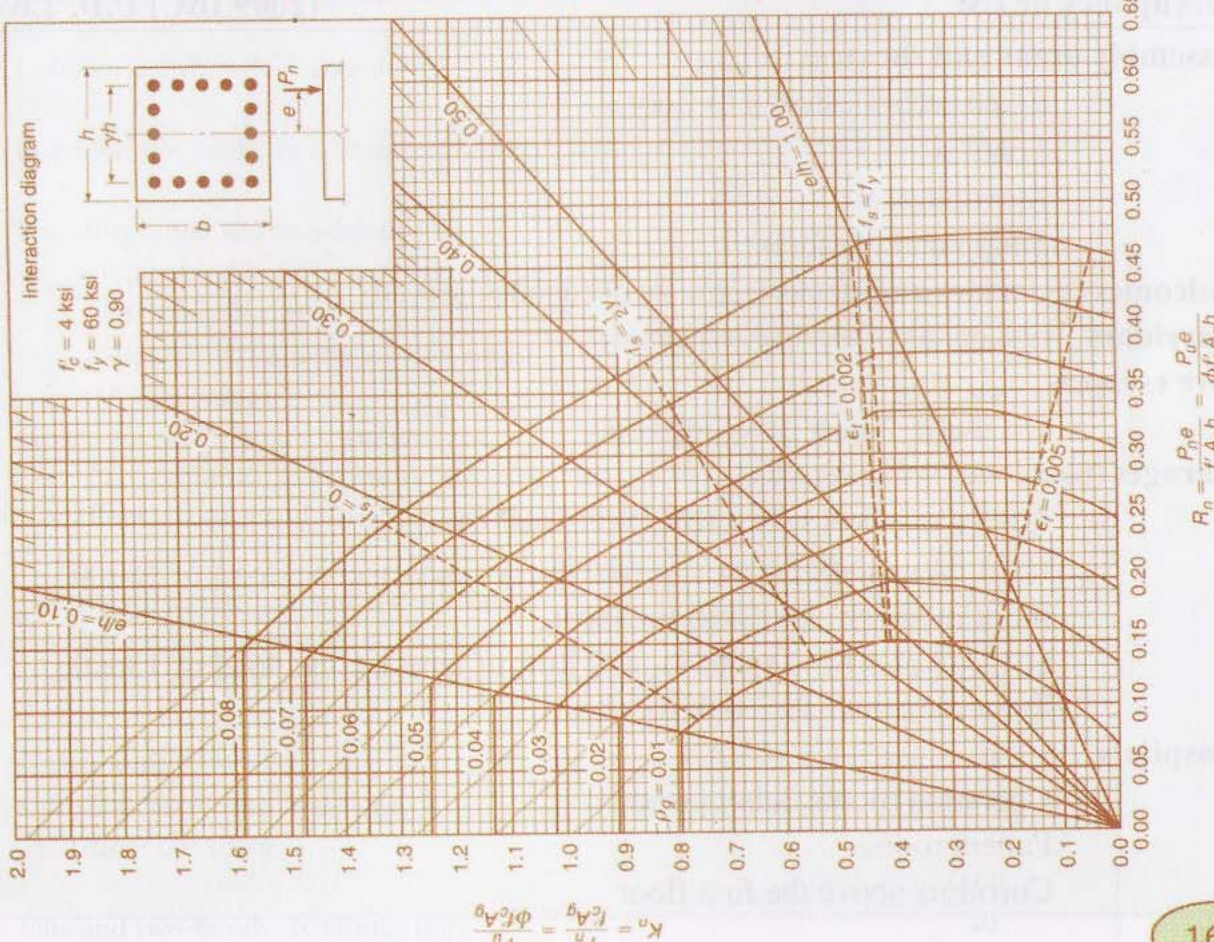
GRAPH A.4

Column strength interaction diagram for rectangular section with bars on four faces and $\gamma = 0.80$



GRAPH A.5

Column strength interaction diagram for rectangular section with bars on four faces and $\gamma = 0.90$



APPENDIX (B) : Reinforced Concrete Design Data and Equations

Loads

Occupancy or Use	(2009 IBC) U.D. Live Load, psf
Assembly areas and theatres	
Fixed seats (fastened to floor)	**60**
Lobbies	100
Movable seats	**100**
Stages and platforms	125
Balconies (exterior) and decks (same as occupancy served)	–
Corridors (except as otherwise indicated)	100
Fire escapes	**100**
On single-family dwellings only	**40**
Garages (passenger vehicles only)	40
Trucks and buses (See IBC Sec. 1607.6)	
H 20-44 and HS 20-44 — 640 lb/ft of lane + conc. load of 18000 lb for moment / 26000 lb for shear	
H 15-44 and HS 15-44 — 480 lb/ft of lane + conc. load of 13500 lb for moment / 19500 lb for shear	
Hospitals	
Operating rooms, laboratories	60
Patient rooms	40
Corridors above the first floor	80

Loads *contd.*

Occupancy or Use	(2009 IBC) U.D. Live Load, psf
Office Buildings	
Lobbies and first-floor corridors	100
Offices	50
Corridors above the first floor	80
Residential	
Dwellings (one and two-family)	
Uninhabitable attics without storage	10
Uninhabitable attics with limited storage	20
Habitable attics and sleeping areas	30
All other areas	40
Hotels and multifamily dwellings	
Private rooms and corridors serving them	40
Public rooms and corridors serving them	100
Roofs	
Ordinary flat, pitched, and curved roofs	20
Roofs used for promenade purposes	60
Roofs used for roof gardens or assembly purposes	100
Schools	
Classrooms	40
Corridors above the first floor	80
First-floor corridors	100
Stairs and exits	100
One and two-family dwellings only	40

Load Factors (ACI 318-2008)

Condition	Factored Load or Load Effect U	
Basic*	$U = 1.2\,D + 1.6\,L$	(a)
Dead plus Fluid	$U = 1.4\,(D + F)$	(b)
Rain, Temperature, Soil, Fluid and Wind in addition to Dead and / or Live	$U = 1.2\,(D{+}F{+}T) + 1.6\,(L{+}H) + 0.5\,(L_r \text{ or } S \text{ or } R)$	(c)
	$U = 1.2D + 1.6\,(L_r \text{ or } S \text{ or } R) + (1.0\,L \text{ or } 0.8\,W)$	(d)
	$U = 1.2D + 1.6\,W + 1.0\,L + 0.5\,(L_r \text{ or } S \text{ or } R)$	(e)
	$U = 0.9D + 1.6\,W + 1.6\,H$	(f)
Earthquake, Soil and Snow in addition to Dead and /or Live	$U = 1.2\,D + 1.0\,E + 1.0\,L + 0.2\,S$	(g)
	$U = 0.9\,D + 1.0\,E + 1.6\,H$	(h)

Note:
D = dead load;
L = live load;
L_r = roof live load;
S = snow load;
R = rain load;
F = fluid pressure load;
H = weight or lateral pressure from soil and water in soil;
T = self-straining force, i.e., cumulative effect of temperature, creep, shrinkage and differential settlement;
W = wind load;
E = earthquake load

Exceptions:

(i) the load factor for the live load L in Eqs. (d), (e) and (g) shall be permitted, according to the ACI code, to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 100 psf.

(ii) where wind load W has not been reduced by a directionality factor, it shall be permitted to use $1.3W$ in place of $1.6W$ in Eqs. (e) and (f).

(iii) where E, the load effects of earthquakes, is based on service-load seismic forces (from editions earlier than ASCE 7-93, UBC 97), $1.4E$ shall be used in place of $1.0E$ in Eqs. (g and h).

Strength Reduction Factors (ACI 318-2008)

Strength Condition	Strength Reduction Factor ϕ *ACI 318-2008 (1999)*
Tension-controlled section	0.90 *(0.90)*
Compression-controlled section	
Members with spiral reinforcement (e.g., spiral columns)	0.75 *(0.75)*
Other reinforced members (e.g., tied columns)	0.65 *(0.70)*
Shear and torsion	0.75 *(0.85)*
Bearing on concrete	0.65 *(0.70)*
Post-tensioned anchoring zone	0.85
Strut-and-tie models	0.75

Design Requirements

Design strength ≥ required strength (under factored loads)

or, $\phi S_n \geq U$

i.e., $\phi M_n \geq M_u$; $\phi V_n \geq V_u$

$\phi T_n \geq T_u$; $\phi P_n \geq P_u$

where the subscripts ***n*** denote the **nominal strengths** in flexure, shear, torsion and axial load, the subscripts ***u*** denote the **factored load** moment, shear, torsion and axial force.

Flexural Design

Singly-reinforced rectangular beams

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

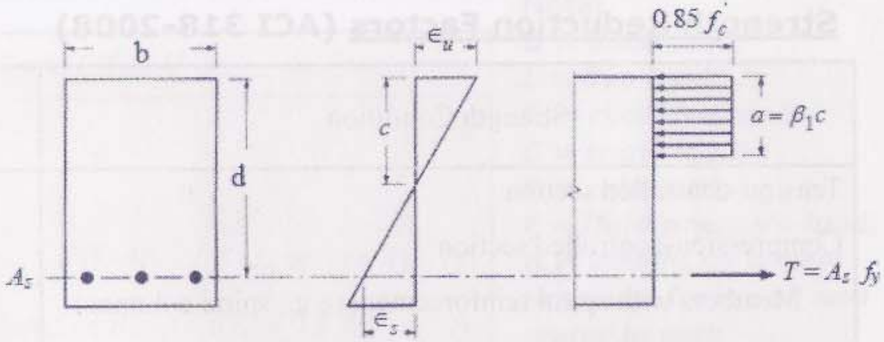


Fig. Singly-reinforced rectangular beam

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{\rho b d f_y}{0.85 f_c' b} = \frac{\rho f_y d}{0.85 f_c'}$$

$$\rho_{max} = 0.85 \beta_1 \frac{f_c'}{f_y} \frac{\in_u}{\in_u + 0.004}$$

Note : $\in_t = 0.004$ to find ρ_{max}

$$\phi M_n = \phi \rho f_y b d^2 \left(1 - 0.59 \frac{\rho f_y}{f_c'} \right)$$

where, $\in_u = 0.003$; $\beta_1 = 0.85$ for $f_c' \leq 4000$ psi

$$\phi M_n = \phi R b d^2$$

$$\rho_{min} = \frac{3\sqrt{f_c'}}{f_y} \geq \frac{200}{f_y} \quad \left\{ \begin{array}{l} \text{if } f_c' \leq 4000 \text{ psi,} \\ \frac{200}{f_y} \text{ controls} \end{array} \right\}$$

$$R = \rho f_y \left(1 - 0.59 \frac{\rho f_y}{f_c'} \right)$$

If $\in_t \geq 0.005$ or $\frac{c}{d_t} \leq 0.375$, use $\phi = 0.90$

If $\in_t = 0.004$ or $\frac{c}{d_t} \leq 0.429$, use $\phi = 0.816$ (corres. to ρ_{max})

If $0.004 < \in_t < 0.005$, use $\phi = 0.65 + (\in_t - 0.002) \frac{250}{3}$

$\in_t$ = steel strain in outermost layer

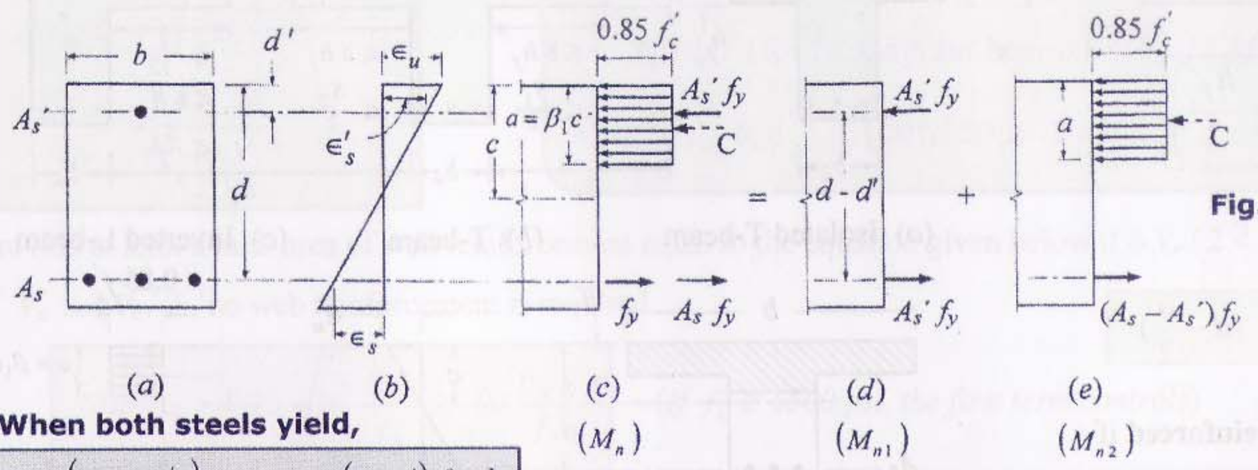
$\in_t = \in_s$ if there is only one layer of steel

Flexural Design *contd.*

Doubly-reinforced rectangular beams

In practical design, if ρ_{act} (tensile) $\leq \rho_{max}$ (singly-reinforced) $\Rightarrow$ disregard the compression bars

if ρ_{act} (tensile) $> \rho_{max}$ (singly-reinforced) $\Rightarrow$ also consider the compression bars



$$\rho = \frac{A_s}{bd} \qquad \rho' = \frac{A_s'}{bd}$$

Fig. Doubly-reinforced rectangular beam

$$\bar{\rho}_b = \rho_b + \rho'$$

$$\bar{\rho}_{max} = \rho_{max} + \rho'$$

When both steels yield,

$$a = \frac{(A_s - A_s') f_y}{0.85 f_c' b} = \frac{(\rho - \rho') f_y d}{0.85 f_c'}$$

$$\bar{\rho}\ (\text{for } \epsilon_t \geq 0.005) = \rho\ (\text{for } \epsilon_t \geq 0.005) + \rho'$$

$$M_n = M_{n1} + M_{n2} = A_s' f_y (d - d') + (A_s - A_s') f_y (d - \frac{a}{2})$$

If $\rho_{act} \leq \bar{\rho}$ (for $\epsilon_t \geq 0.005$), use $\phi = 0.90$.

If $\bar{\rho}$ (for $\epsilon_t \geq 0.005$) $< \rho_{act} < \bar{\rho}_{max}$,

ϕ must be found by interpolation.

Min. limiting steel ratio,
$$\bar{\rho}_{cy} = 0.85\,\beta_1 \frac{f_c'}{f_y} \frac{d'}{d} \frac{87{,}000}{87{,}000 - f_y} + \rho'$$

$\therefore$ If $\rho_{act} < \bar{\rho}_{cy}$, compression steel does not yield, i.e., $\epsilon_s' < \epsilon_y$ and $f_s' < f_y$.

Flexural Design *contd.*

Flanged Sections

If $a = \dfrac{A_s f_y}{0.85 f_c' b} > h_f$, analyze as T-beam

$$A_{sf} = \frac{0.85\, f_c' \left(b - b_w\right) h_f}{f_y}$$

$$M_{n1} = A_{sf}\, f_y \left(d - \frac{h_f}{2}\right)$$

$$a = \frac{\left(A_s - A_{sf}\right) f_y}{0.85\, f_c'\, b_w}$$

$$M_{n2} = (A_s - A_{sf})\, f_y\, (d - \frac{a}{2})$$

$M_n = M_{n1} + M_{n2}$

A **T-beam is underreinforced** if

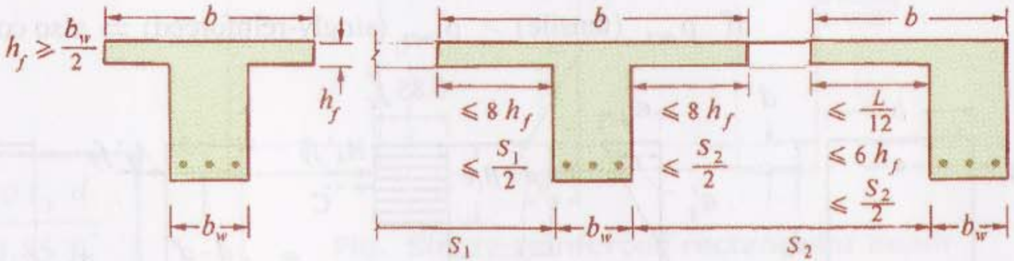
$$\text{Act. } \rho_w \left(= A_s / b_w d\right) \le \rho_{w,max} \left(= \rho_{max} + \rho_f\right)$$

Fig. Effective flange width of flanged beams

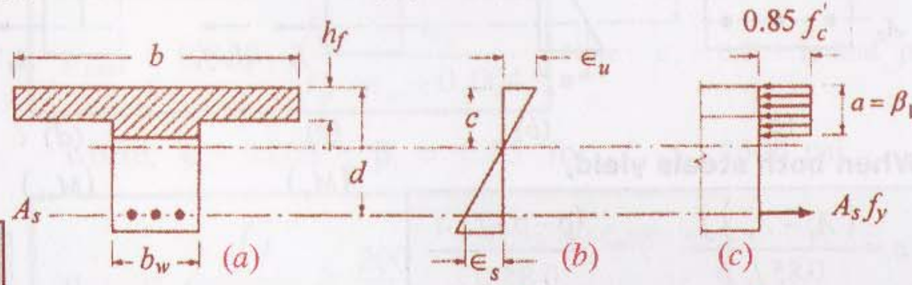
(a) $b \le 4\, b_w$

(b) 1/ $b \le b_w + \frac{S_1}{2} + \frac{S_2}{2}$ 2/ $b \le b_w + 16\, h_f$ 3/ $b \le \frac{L}{4}$

(c) 1/ $b \le b_w + \frac{S_2}{2}$ 2/ $b \le b_w + 6\, h_f$ 3/ $b \le b_w + \frac{L}{12}$

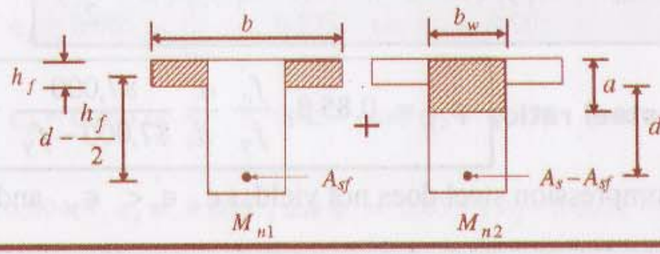


(a) Isolated T-beam (b) T-beam (c) Inverted L-beam



ρ_{max} is the same as for singly-reinforced rectangular section ; $\epsilon_t = 0.004$; $\rho_f = \dfrac{A_{sf}}{b_w\, d}$ and $\rho_f = \dfrac{A_{sf}}{b_w\, d}$

$\rho_{min} = 3\sqrt{f_c'}\,/ f_y$ *and* $200 / f_y$ applies to T-beams, too. For T-beams, the ratio ρ should be computed based on the web width b_w.



Shear Design

$$V_u \le \phi V_n$$

$$\phi = 0.75 \text{ for shear}$$

$$V_u \le \phi V_c + \frac{\phi A_v f_y d}{s}$$

$$s \le \frac{\phi A_v f_y d}{V_u - \phi V_c}$$

$$V_c = \left(1.9\sqrt{f_c'} + 2500\,\frac{\rho_w V_u d}{M_u}\right) b_w d \le 3.5\sqrt{f_c'}\, b_w d$$

where $\rho_w = A_s / b_w d$ (for T-beams having web width b_w)

or A_s / bd (for rectangular beams). $\boldsymbol{V_u d / M_u \le 1.0}$

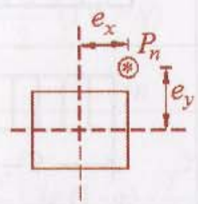
$$V_c = 2\sqrt{f_c'}\, b_w d \qquad \text{(conservative)}$$

Provide at least a min. area of web reinforcement equal to the equation given below if $\phi V_c / 2 < V_u \le \phi V_c$.

If $V_u \le \phi V_c / 2$, no web reinforcement is required.

$$\text{Min. } A_v = 0.75\sqrt{f_c'}\,\frac{b_w s}{f_y} \ge 50\,\frac{b_w s}{f_y}$$

(If $f_c' \ge 4500$ psi, the first term controls)



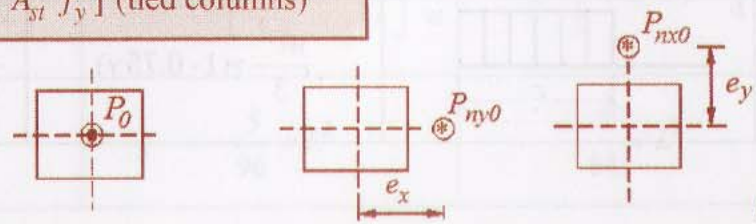
Column Design

For concentrically-loaded tied columns,

Max. design strength $= 0.52\,[\,0.85\, f_c'\,(A_g - A_{st}) + A_{st}\, f_y\,]$ (tied columns)

For biaxial bending, Bresler's equation is

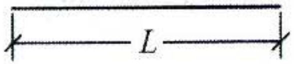
$$\frac{1}{P_n} = \frac{1}{P_{nx0}} + \frac{1}{P_{ny0}} - \frac{1}{P_0}$$

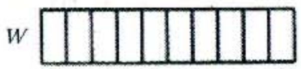
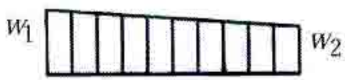
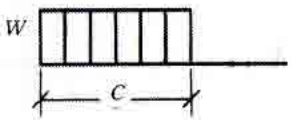
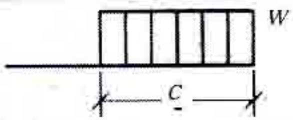


APPENDIX (C) : Analysis Formulae

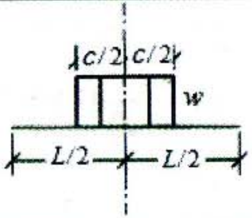
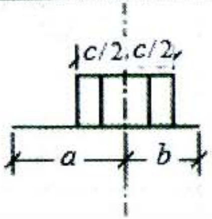
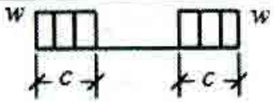
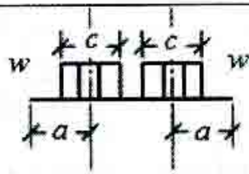
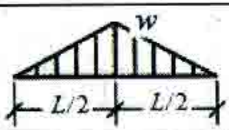
Fixed-End Moment Formulae

$$\alpha = \frac{a}{L} \; ; \; \beta = \frac{b}{L} \; ; \; \gamma = \frac{c}{L}$$

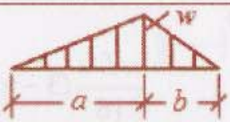
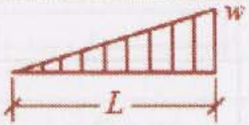
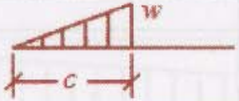
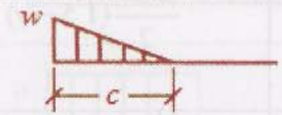
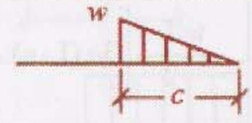
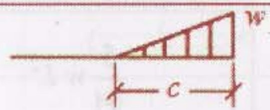


No	Loading case	M_{FAB}	M_{FBA}	M_{FAB}
1		$\frac{wL^2}{12}$	$-\frac{wL^2}{12}$	$\frac{wL^2}{8}$
2		$\frac{L^2}{60}(3w_1+2w_2)$	$-\frac{L^2}{60}(2w_1+3w_2)$	$\frac{L^2}{120}(8w_1+7w_2)$
3		$\frac{wc^2}{3}(1.5-2\gamma+0.75\gamma^2)$	$-\frac{wc^2}{3}\gamma(1-0.75\gamma)$	$\frac{wc^2}{8}(2-\gamma)^2$
4		$\frac{wc^2}{3}\gamma(1-0.75\gamma)$	$-\frac{wc^2}{3}(1.5-2\gamma+0.75\gamma^2)$	$\frac{wc^2}{8}(2-\gamma^2)$

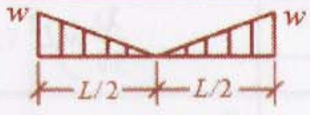
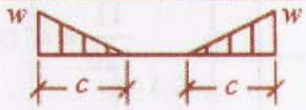
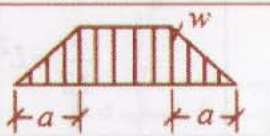
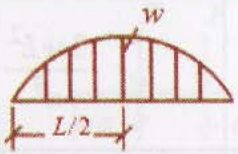
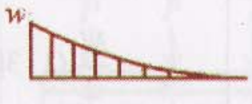
Fixed-End Moment Formulae *contd.*

No	Loading case	M_{FAB}	M_{FBA}	M_{FAB}
5		$\frac{wLc}{24}(3-\gamma^2)$	$-\frac{wLc}{24}(3-\gamma^2)$	$\frac{wLc}{16}(3-\gamma^2)$
6		$wc\left[a\beta^2+\frac{\gamma^2}{12}(L-3b)\right]$	$-wc\left[b\alpha^2+\frac{\gamma^2}{12}(L-3a)\right]$	$\frac{\omega bc}{2}(1-\beta^2-0.25\gamma^2)$
7		$\frac{wc^2}{3}(1.5-\gamma)$	$-\frac{wc^2}{3}(1.5-\gamma)$	$\frac{wc^2}{2}(1.5-\gamma)$
8		$wLc\left[\alpha(1-\alpha)-\frac{\gamma^2}{12}\right]$	$-wLc\left[\alpha(1-\alpha)-\frac{\gamma^2}{12}\right]$	$\frac{wLc}{2}\left[3\alpha(1-\alpha)-\frac{\gamma^2}{4}\right]$
9		$\frac{5}{96}wL^2$	$-\frac{5}{96}wL^2$	$\frac{5}{64}wL^2$

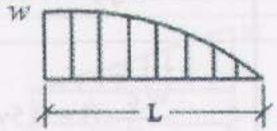
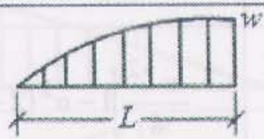
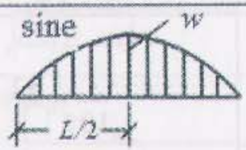
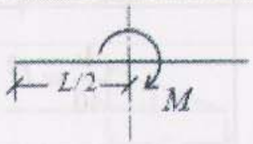
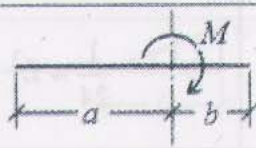
Fixed-End Moment Formulae *contd.*

No	Loading case	M_{FAB}	M_{FBA}	M_{FAB}
10		$\frac{wL^2}{30}\left[1+\beta+\beta^2-1.5\beta^3\right]$	$-\frac{wL^2}{30}\left[1+\alpha+\alpha^2-1.5\alpha^3\right]$	$\frac{wL^2}{120}(1+\beta)\left(7-3\beta^2\right)$
11		$\frac{1}{30}wL^2$	$-\frac{1}{20}wL^2$	$\frac{7}{120}wL^2$
12		$\frac{1}{20}wL^2$	$-\frac{1}{30}wL^2$	$\frac{1}{15}wL^2$
13		$\frac{wc^2}{3}\left(1-1.5\gamma+0.6\gamma^2\right)$	$-\frac{wc^2}{4}\gamma(1-0.8\gamma)$	$\frac{wc^2}{6}\left(2-2.5\gamma+0.6\gamma^2\right)$
14		$\frac{wc^2}{6}\left(1-\gamma+0.3\gamma^2\right)$	$-\frac{wc^2}{12}\gamma(1-0.6\gamma)$	$\frac{wc^2}{6}\left(1-0.75\gamma+0.15\gamma^2\right)$
15		$\frac{wc^2}{4}\gamma(1-0.8\gamma)$	$-\frac{wc^2}{3}\left(1-1.5\gamma+0.6\gamma^2\right)$	$\frac{wc^2}{6}\left(1-0.6\gamma^2\right)$
16		$\frac{wc^2}{12}\gamma(1-0.6\gamma)$	$-\frac{wc^2}{6}\left(1-\gamma+0.3\gamma^2\right)$	$\frac{wc^2}{12}\left(1-0.3\gamma^2\right)$

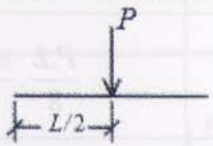
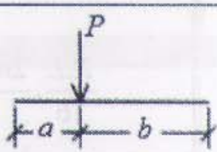
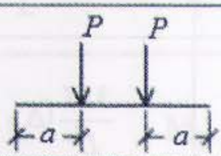
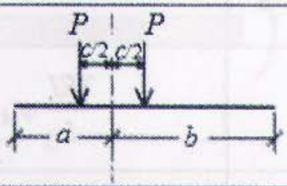
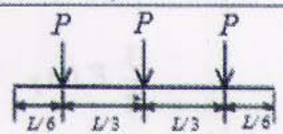
Fixed-End Moment Formulae *contd.*

No	Loading case	M_{FAB}	M_{FBA}	M_{FAB}'
17		$\frac{1}{32}\, wL^2$	$-\frac{1}{32}\, wL^2$	$\frac{3}{64}\, wL^2$
18		$\frac{w\,c^2}{6}(1-0.5\gamma)$	$-\frac{w\,c^2}{6}(1-0.5\gamma)$	$\frac{w\,c^2}{4}(1-0.5\gamma)$
19		$\frac{wL^2}{12}\left[1-\alpha^2(2-\alpha)\right]$	$-\frac{wL^2}{12}\left[1-\alpha^2(2-\alpha)\right]$	$\frac{wL^2}{8}\left[1-\alpha^2(2-\alpha)\right]$
20		$\frac{1}{15}\, wL^2$	$-\frac{1}{15}\, wL^2$	$\frac{1}{10}\, wL^2$
21		$\frac{1}{60}\, wL^2$	$-\frac{1}{60}\, wL^2$	$\frac{1}{40}\, wL^2$
22		$\frac{1}{30}\, wL^2$	$-\frac{1}{60}\, wL^2$	$\frac{1}{24}\, wL^2$

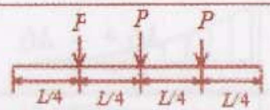
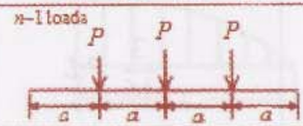
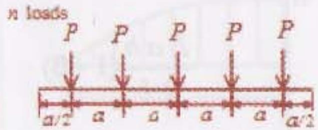
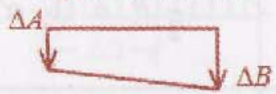
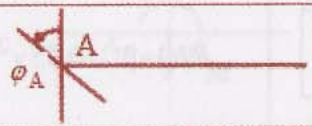
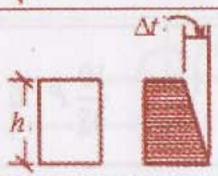
Fixed-End Moment Formulae *contd.*

No	Loading case	M_{FAB}	M_{FBA}	M_{FAB}
23		$\frac{1}{60} w L^2$	$-\frac{1}{30} w L^2$	$\frac{1}{30} w L^2$
24		$\frac{1}{15} w L^2$	$-\frac{1}{20} w L^2$	$\frac{11}{120} w L^2$
25		$\frac{1}{20} w L^2$	$-\frac{1}{15} w L^2$	$\frac{1}{12} w L^2$
26		$\frac{2 w L^2}{\pi^3}$	$-\frac{2 w L^2}{\pi^3}$	$\frac{3 w L^2}{\pi^3}$
27		$-\frac{M}{4}$	$-\frac{M}{4}$	$-\frac{M}{8}$
28		$-M\beta(3\alpha - 1)$	$-M\alpha(3\beta - 1)$	$-\frac{M}{2}(1 - 3\beta^2)$

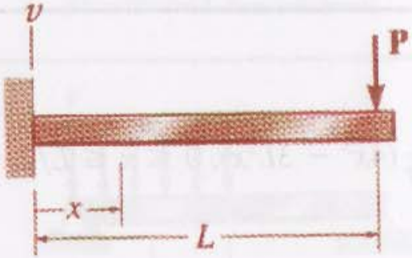
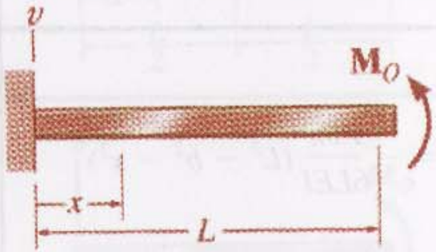
Fixed-End Moment Formulae *contd.*

No	Loading case	M_{FAB}	M_{FBA}	M_{FAB}
29		M_1	$-M_2$	$M_1 + \frac{1}{2} M_2$
30		$\frac{PL}{8}$	$-\frac{PL}{8}$	$\frac{3}{16} PL$
31		$P.a.\beta^2$	$-P.b.\alpha^2$	$\frac{Pab}{2L}(1+\beta)$
32		$P.a\ (1-\alpha)$	$-P.a\ (1-\alpha)$	$\frac{3}{2} P.a\ (1-\alpha)$
33		$P\left[2a\beta^2 + \frac{a\gamma^2}{2} - b\gamma^2\right]$	$-P\left[2b\alpha^2 + \frac{b\gamma^2}{2} - a\gamma^2\right]$	$Pb\left\|1-\beta^2-0.75\gamma^2\right\|$
34		$\frac{19}{72}P.L$	$-\frac{19}{72}P.L$	$\frac{19}{48}P.L$

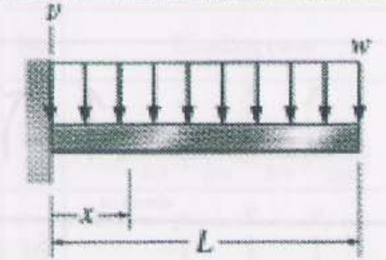
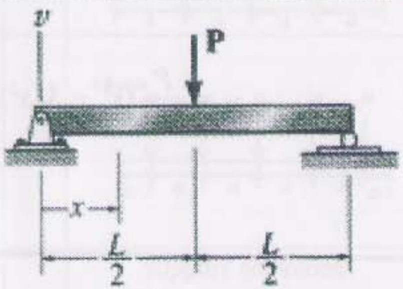
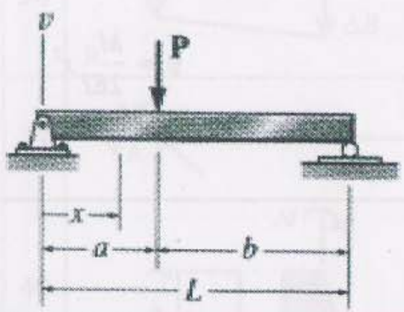
Fixed-End Moment Formulae *contd.*

No	Loading case	M_{FAB}	M_{FBA}	M_{FAB}
35		$\frac{5}{16} P.L$	$-\frac{5}{16} P.L$	$\frac{15}{32} P.L$
36		$\frac{P.L}{12}.\frac{n^2-1}{n}$	$-\frac{P.L}{12}.\frac{n^2-1}{n}$	$\frac{P.L}{8}.\frac{n^2-1}{n}$
37		$\frac{P.L}{24}.\frac{2n^2+1}{n}$	$-\frac{P.L}{24}.\frac{2n^2+1}{n}$	$\frac{P.L}{16}.\frac{2n^2+1}{n}$
38	support settlement 	$\frac{6EI}{L^2}(\Delta_B-\Delta_A)$	$\frac{6EI}{L^2}(\Delta_B-\Delta_A)$	$\frac{3EI}{L^2}(\Delta_B-\Delta_A)$
39		$\frac{4EI}{L}\varphi_A$	$\frac{2EI}{L}\varphi_A$	$\frac{3EI}{L}\varphi_A$
40		$EI\alpha_T.\frac{\Delta_t}{h}$	$-EI\alpha_T.\frac{\Delta_t}{h}$	$\frac{3}{2}EI\alpha_T.\frac{\Delta_t}{h}$

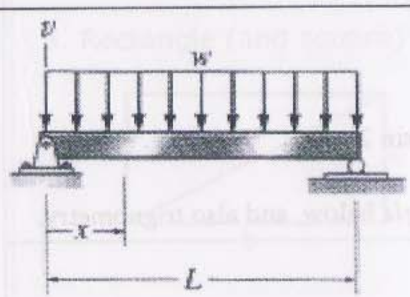
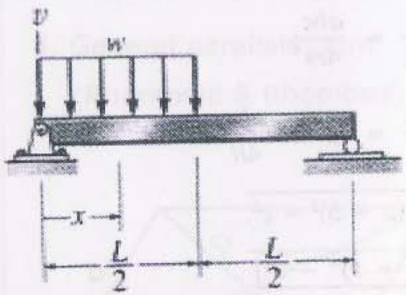
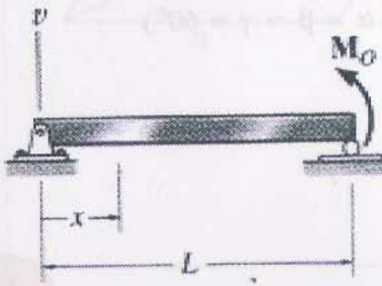
Beam Deflections and Slopes

Loading	$v + \uparrow$	$\theta + \curvearrowleft$	Equation $+ \uparrow \; + \curvearrowleft$
	$v_{max} = -\frac{PL^3}{3EI}$ at $x = L$	$\theta_{max} = -\frac{PL^2}{2EI}$ at $x = L$	$v = \frac{P}{6EI}(x^3 - 3Lx^2)$
	$v_{max} = \frac{M_O L^2}{2EI}$ at $x = L$	$\theta_{max} = \frac{M_O L}{EI}$ at $x = L$	$v = \frac{M_O}{2EI}x^2$

Beam Deflections and Slopes *contd.*

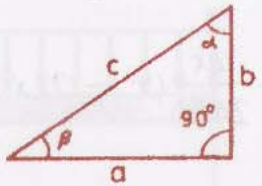
	$v_{max} = -\frac{wL^4}{8EI}$ at $x = L$	$\theta_{max} = -\frac{wL^3}{6EI}$ at $x = L$	$v = -\frac{w}{24EI}(x^4 - 4Lx^3 + 6L^2x^2)$
	$v_{max} = -\frac{PL^3}{48EI}$ at $x = L/2$	$\theta_{max} = \pm\frac{PL^2}{16EI}$ at $x = 0$ or $x = L$	$v = -\frac{P}{48EI}(4x^3 - 3L^2x),\ 0 \le x \le L/2$
		$\theta_L = -\frac{Pab(L+b)}{6LEI}$ $\theta = \frac{Pab(L+a)}{6LEI}$	$v = -\frac{Pbx}{6LEI}(L^2 - b^2 - x^2)$ $0 \le x \le a$

Beam Deflections and Slopes *contd.*

	$v_{max} = -\dfrac{5wL^4}{384EI}$ at $x = \dfrac{L}{2}$	$\theta_{max} = \pm \dfrac{wL^3}{24EI}$	$v = -\dfrac{wx}{24EI}(x^3 - 2Lx^2 + L^3)$
		$\theta_L = -\dfrac{3wL^3}{128EI}$ $\theta = \dfrac{7wL^3}{384EI}$	$v = -\dfrac{wx}{384EI}(9L^3 - 24Lx^2 + 16x^3)$ $0 \le x \le L/2$ $v = -\dfrac{wL}{384EI}(8x^3 - 24Lx^2 + 17L^2x - L^3)$ $L/2 \le x \le L$
	$v_{max} = -\dfrac{M_O L^2}{9\sqrt{3}EI}$	$\theta_L = -\dfrac{M_O L}{6EI}$ $\theta = \dfrac{M_O L}{3EI}$	$v = -\dfrac{M_O x}{6EIL}(x^2 - 3Lx + 2L^2)$

APPENDIX (D) : Mensuration

1. Right triangle



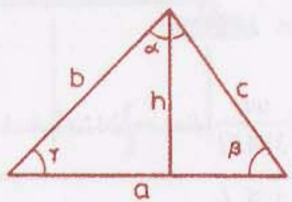
(One angle 90°)

$p = a + b + c; \quad c^2 = a^2 + b^2;$

$A = \dfrac{ab}{2} = \dfrac{a^2}{2}\tan\beta = \dfrac{c^2}{4}\sin 2\beta = \dfrac{c^2}{4}\sin 2\alpha.$

For additional formulas, see *General Triangle* below, and also trignometry.

2. General triangle (and equilateral)



For General Triangle :

Triangle) $\quad p = a + b + c.$ Let $s = {}^1/_2\,(a + b + c).$

$r = \dfrac{\sqrt{s(s-a)(s-b)(s-c)}}{s}; \quad R = \dfrac{a}{2\sin\alpha} = \dfrac{abc}{4rs};$

$A = \dfrac{ab}{2} = \dfrac{ab}{2}\sin\gamma = \dfrac{b^2 \sin\gamma \sin\alpha}{\sin\beta} = rs = \dfrac{abc}{4R}.$

Length of median to side $c = \tfrac{1}{2}\sqrt{2\,(a+b)^2 - c^2}.$

Length of bisector of angle $\gamma = \dfrac{\sqrt{ab\,[(a+b)^2 - c^2]}}{a+b}.$

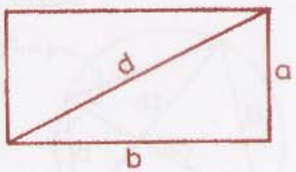
For Equilateral Triangle ($a = b = c$ and $\alpha = \beta = \gamma = 60^\circ$)
(Equal sides and equal angles)

$p = 3s; \quad r = \dfrac{s}{2\sqrt{3}}; \quad R = \dfrac{s}{\sqrt{3}} = 2r;$

$h = \dfrac{s\sqrt{3}}{2}; \quad s = \dfrac{2h}{\sqrt{3}}; \quad A = \dfrac{s^2\sqrt{3}}{4}.$

Mensuration *contd.*

3. Rectangle (and square)



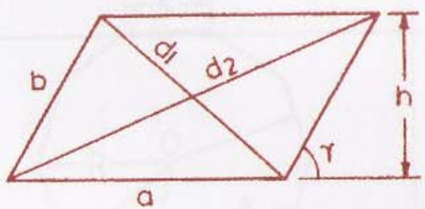
For Rectangle :

$$p = 2(a + b); \quad d = \sqrt{a^2 + b^2}; A = ab.$$

For Square $(a = b = s)$

$$p = 4s; d = s\sqrt{2}; \quad s = \frac{d}{\sqrt{2}}; \ A = s^2 = \frac{d^2}{2}.$$

4. General parallelogram
(Rhomboid & Rhombus)



For General Parallelogram (Rhomboid) :

(Opposite sides parallel)

$$p = 2(a + b); \quad d_1 = \sqrt{a^2 + b^2 - 2ab\cos\gamma};$$

$$d_2 = \sqrt{a^2 + b^2 + 2ab\cos\gamma}; \quad {d_1}^2 + {d_2}^2 = 2(a^2 + b^2);$$

$$A = ah = ab\sin\gamma.$$

For Rhombus $(a = b = s)$:

(Opposite sides parallel and all sides equal)

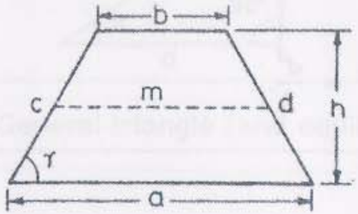
$$p = 4s; \quad d_1 = 2s\sin\frac{\gamma}{2}; \quad d_2 = 2s\cos\frac{\gamma}{2}; \quad {d_1}^2 + {d_2}^2 = 4s^2;$$

$$d_1 \cdot d_2 = 2s^2\sin\gamma; \quad A = sh = s^2\sin\gamma = \frac{d_1 d_2}{2}$$

Mensuration *contd.*

5. General Trapezoid

(and Isosceles Trapezoid)



Let mid-line bisecting non-paralledl sides = m. Then $m = \dfrac{a+b}{2}$

For General Trapezoid :

(Only one pair of opposite sides parallel)

$$p = a + b + c + d\,;\ A = \frac{(a+b)\,h}{2} = mh.$$

For Isosceles Trapezoid $(d = c)$:

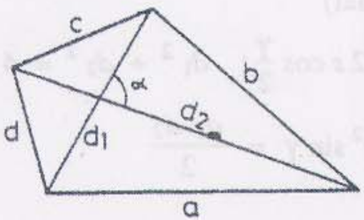
(Non-parallel sides equal)

$$A = \frac{(a+b)h}{2} = mh = \frac{(a+b)\,c\sin\gamma}{2}$$

$$= (a - c\cos\gamma)\,c\sin\gamma = (b + c\cos\gamma)\,c\sin\gamma$$

6. General quardrilateral

(Trapezium)



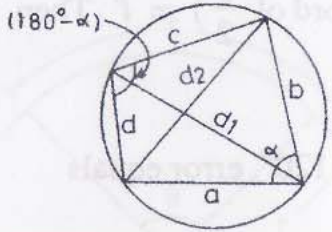
(No sides parallel)

$$P = a + b + c + d$$

$A = ½\, d_1 d_2 \sin\alpha$ = sum of areas of the two triangles formed by either diagonal and the four sides.

Mensuration *contd.*

7. Quardrilateral inscribed in Circle



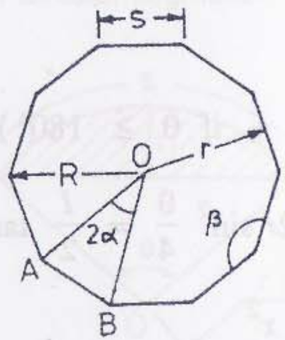
(Sum of opposite angles = 180°)

$$ac + bd = d_1 d_2.$$

Let $s = \frac{1}{2}(a + b + c + d) = \frac{p}{2}$ and α = angle between sides a and b.

$$A = \sqrt{(s - a)(s - b)(s - c)(s - d)} = \tfrac{1}{2}(ab + cd)\sin\alpha.$$

8. Regular polygon (and general polygon)



For Regular Polygon : (Equal sides and equal angles)

Let n = number of sides.

Central angle $= 2\alpha = \frac{2\pi}{n}$ radians.

Vertex angle $= \beta = \frac{(n-2)}{n}\pi$ radians.

$p = ns$; $s = 2r\tan\alpha = 2R\sin\alpha$;

$r = \frac{s}{2}\cot\alpha$; $R = \frac{s}{2}\operatorname{cosec}\alpha$;

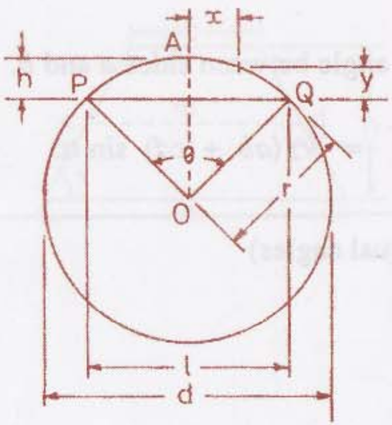
$A = \frac{nsr}{2} = nr^2\tan\alpha = \frac{nR^2}{2}\sin 2\alpha = \frac{ns^2}{4}\cot\alpha$ = sum of areas of the n equal triangles such as OAB.

For General Polygon :

A = sum of areas of constituent triangles into which it can be divided.

Mensuration *contd.*

9. Circle (and circular arc)



For Circle :

$d = 2r$;

$c = 2\pi r = \pi d$;

$A = \pi r^2 = \dfrac{\pi d^2}{4} = \dfrac{c^2}{4\pi}$

For Circle Arc:

Let arc $PAQ = s$; and chord PA = (chord of $\frac{s}{2}$) = l'. Then,

$$s = r\,\theta = \frac{d\theta}{2};\ s = \frac{8l' - l}{3}$$

For θ small, error is very small ; for $\theta = 120^\circ$, error equals about 1 part in 400 ,

$$l = 2\,r \sin\frac{\theta}{2};\ l = 2\sqrt{2\,hr - h^2} \quad \text{(approximate formaula)}$$

$$r = \frac{s}{\theta} = \frac{l}{2\sin\frac{\theta}{2}};\ r = \frac{4\,h^2 + l^2}{8\,h} \quad \text{(approxiamte formula)}$$

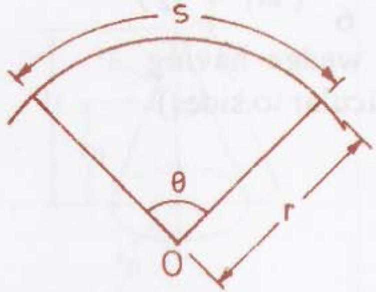
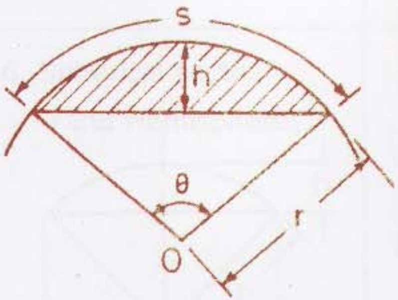
$$h = 4 \mp \sqrt{r^2 - \frac{l^2}{4}} \quad (-\text{ if } \theta \le 180^\circ;\ +\text{ if } \theta \ge 180^\circ)$$

$$= r\,(1 - \cos\frac{\theta}{2}) = r \text{ versin}\frac{\theta}{2} = 2r\sin^2\frac{\theta}{4} = \frac{l}{2}\tan\frac{\theta}{4}$$

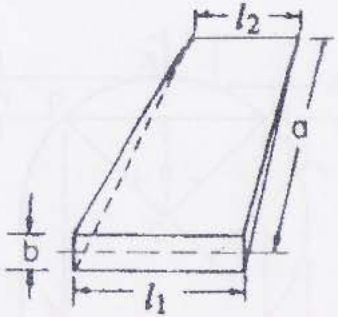
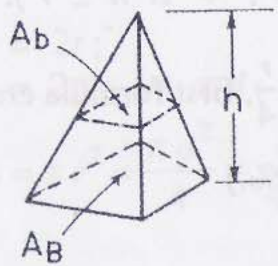
$$= r + y - \sqrt{r^2 - x^2}$$

Side ordinate $y = h - r + \sqrt{r^2 - x^2}$

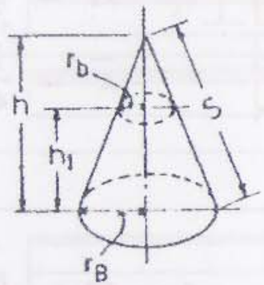
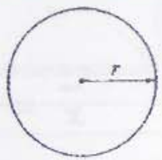
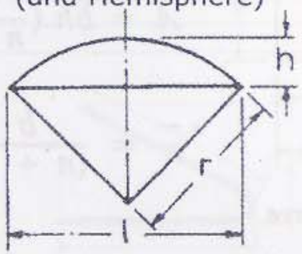
Mensuration *contd.*

10. Circular sector (and semicircle) 	For Circular Sector : $A = \frac{\theta r^2}{2} = \frac{s r}{2}$ *For Semicircle* : $A = \frac{\pi r^2}{2}$
11. Circular segment 	$A = \frac{r^2}{2}(\theta - \sin\theta)$ $= \tfrac{1}{2}[sr \mp l(r - h)]$ $(-$ if $h \le r;$ $+$ if $h \ge r)$. $A = \frac{2lh}{3}$ *or* $\frac{h}{15}(8l' + 6l)$. For $h = \frac{r}{4}$, first formula error is about 3.5% and second less than 1.0%.)

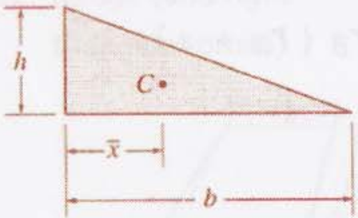
Mensuration *contd.*

12. Wedge (and right triangular prism) 	*For Wedge :* (Narrow-side rectangular) ; $V = \frac{ab}{6}(2l_1 + l_2)$ *For Right Triangular Prism* (or wedge having parallel triangular bases perpendicular to sides) $l_2 = l_1 = l$; $V = \frac{abl}{2}$
13. General pyramid (and frustum of pyramid) 	*For General Pyramid :* $V = \frac{h A_B}{3}$; $A_l = \frac{s p_B}{2}$ *For Frustum of general pyramid :* $A_l = \frac{s}{2}(p_B + p_b)$ $V = \frac{h}{3}(A_B + A_b + \sqrt{A_B A_b})$

Mensuration *contd.*

14. Right circular cone (and frustum of right circular cone) 	*For Right Circular Cone :* $A_l = \pi r_B s = \pi r_B \sqrt{r_B^2 + h^2}$; $A_t = \pi r_B (r_B + s)$; $V = \dfrac{\pi r_B^{2} h}{3}$ *For Frustum of Right Circular Cone :* $s = \sqrt{h_1^2 + (r_B - r_b)^2}$; $A_l = \pi s (r_B + r_b)$; $V = \dfrac{\pi h_1}{3}(r_B^{2} + r_b^{2} + r_B r_b)$
15. Sphere 	Let diameter $= d$; $A_t = 4\pi r^2 = \pi d^2$; $V = \dfrac{4\pi r^2}{3} = \dfrac{\pi d^3}{6}$
16. Spherical sector (and Hemisphere) 	*For Spherical Sector :* $A_t = \dfrac{\pi r}{2}(4h + l)$; $V = \dfrac{2\pi r^2 h}{3}$ *For Hemisphere* (letting $h = \dfrac{l}{2} = r$) $A_t = 3\pi r^2$; $V = \dfrac{2\pi r^3}{3}$

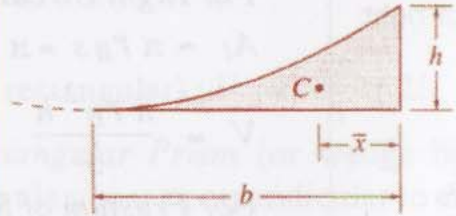
Area and Centroid Location of Some Geometric Shapes



Triangle

$$A = \frac{1}{2} bh$$

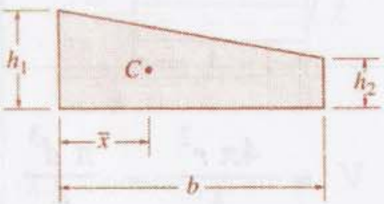
$$\bar{x} = \frac{1}{3} b$$



Parabolic spandrel

$$A = \frac{1}{3} bh$$

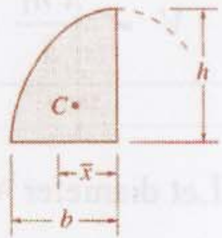
$$\bar{x} = \frac{1}{4} b$$



Trapezoid

$$A = \frac{1}{2} b \, (h_1 + h_2)$$

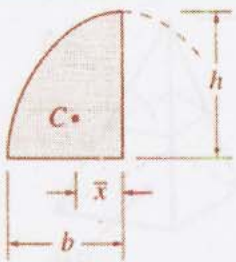
$$\bar{x} = \frac{b \, (2h_2 + h_1)}{3 \, (h_1 + h_2)}$$



Semi-segment of *n*th degree curve

$$A = bh \, (\frac{n}{n + 1})$$

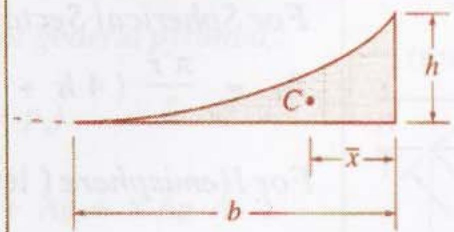
$$\bar{x} = \frac{b \, (n + 1)}{2 \, (n + 2)}$$



Semi Parabola

$$A = \frac{2}{3} bh$$

$$\bar{x} = \frac{3}{8} b$$

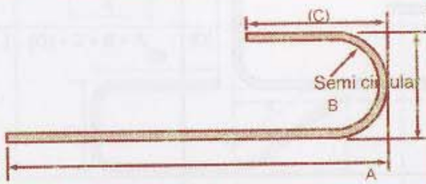
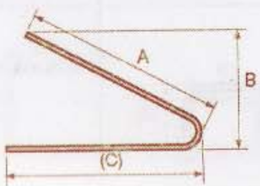
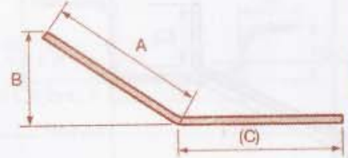
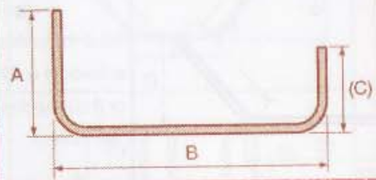
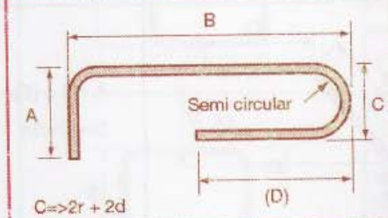
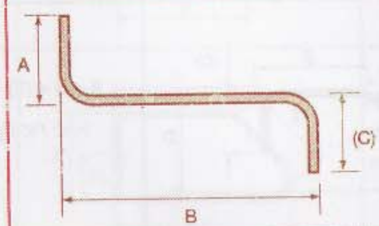


Spandrel of *n*th degree curve

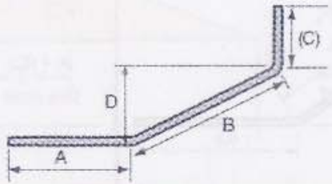
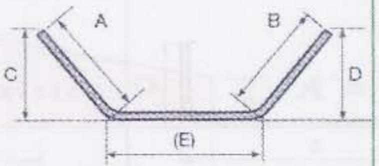
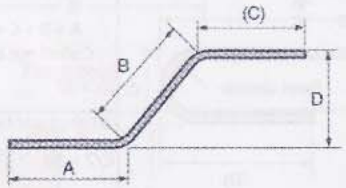
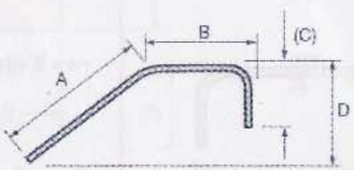
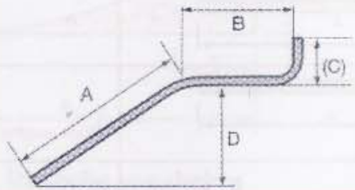
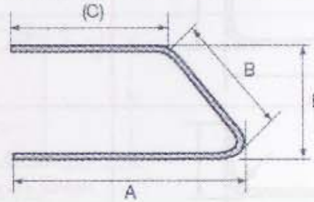
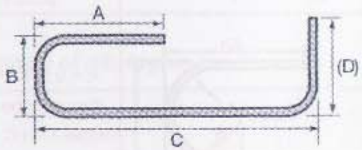
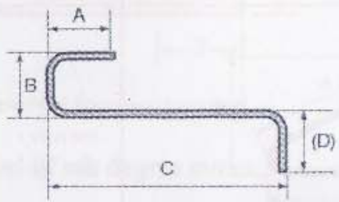
$$A = bh \, (\frac{1}{n + 1})$$

$$\bar{x} = \frac{b}{(n + 2)}$$

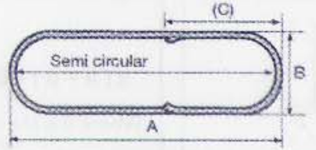
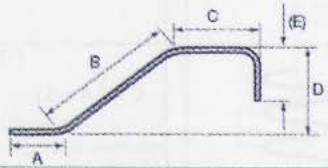
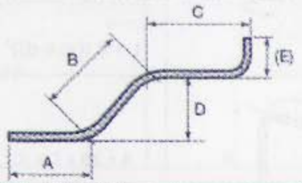
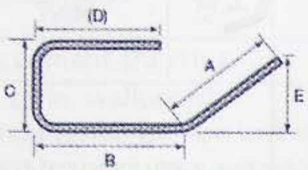
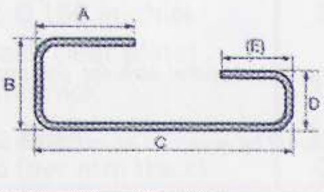
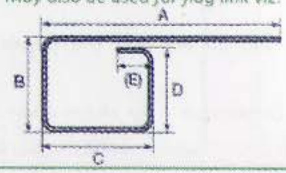
APPENDIX (E) : Length of Bent Bars

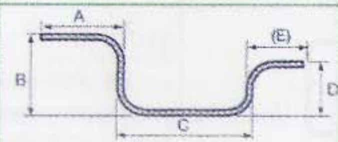
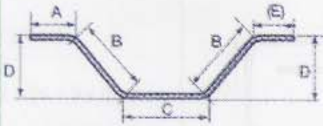
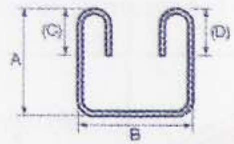
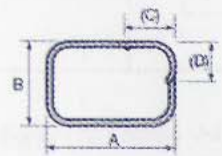
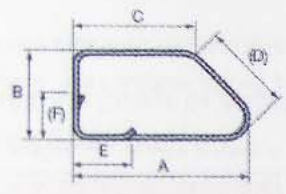
Shape code	Shape	Total length of bar (*L*) measured along centre line
00		A
01		A Stock lengths . *See Note 4*
11		A + (B) − 0.5 r − d
12		A + (B) − 0.43R − 1.2d *Neither A nor B shall be less than (R + 6d)*
13		A + 0.57B + (C) − 1.6d *B shall not be less than 2(r + d). Neither A nor C shall be less than (½B + 5d). See note 3*
14		A + (C) − 4d *See note 1*
15		A + (C) *See note 1*
21		A + B + (C) − r − 2d
22		A + B + C + (D) - 1.5r - 3d *C shall not be less than 2(r + d).* *(D) shall not be less than C/2 + 5d*
23		A + B + (C) − r − 2d

Length of Bent Bars *contd.*

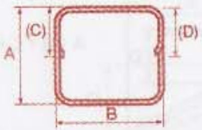
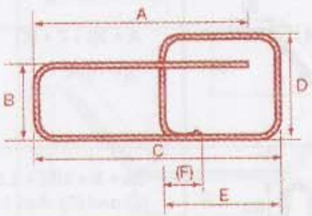
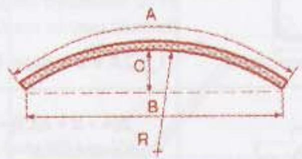
Shape code	Shape	Total length of bar (*L*) measured along centre line
24		A + B + (C) *A and (C) are at 90° to one another*
25		A + B + (E) *If E is the critical dimension, schedule a 99 and specify A or B as the free dimension.* *See note 1.*
26		A + B + (C) *See note 1.*
27		A + B + (C) − 0.5 r − d *See note 1.*
28		A + B + (C) − 0.5 r − d *See note 1.*
29		A + B + (C) − r − 2d *See note 1.*
31		A + B + C + (D) - 1.5r - 3d
32		A + B + (D) − 1.5 r − 3d

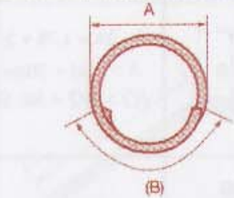
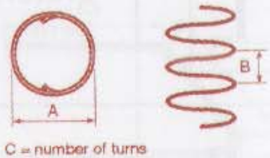
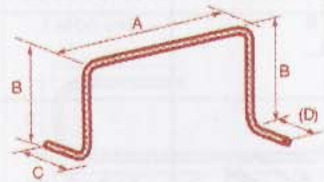
Length of Bent Bars *contd.*

Shape code	Shape	Total length of bar (L) measured along centre line
33		2A + 1.7B + 2(C) − 1d *A > 12d + 30mm. B > 2(r + d). (C) > B/2 + 5d. See note 3*
34		A + B + C + (E) − 0.5 r − d *See note 1.*
35		A + B + C + (E) − 0.5 r − d *See note 1.*
36		A + B + C + (D) − r − 2d *See note 1.*
41		A + B + C + D + (E) − 2 r − 4d *May also be used for flag link viz:* 

Shape code	Shape	Total length of bar (L) measured along centre line
44		A + B + C + D + (E) − 2 r − 4d
46		A + 2B + C + (E) *See note 1.*
47		2A + B + 2(C) + 1.5 r − 3d *(C) and (D) shall be equal and not more than A. Where (C) and (D) are to be minimized the following formula may be used:* L = 2A + B + max. (21d, 240)
51		2(A + B + 2(C)) − 2.5 r − 5d *(C) and (D) shall be equal and not more than A or B. Where (C) and (D) are to be minimized the following formula may be used:* L = 2A + 2B + max. (16d, 160)
56		A + B + C + (D) + 2(E) − 2.5 r − 5d *(E) and (F) shall be equal and not more than B or C. See notes 1 and 2.*

Length of Bent Bars *contd.*

Shape code	Shape	Total length of bar (L) measured along centre line
63		2A + 3B + 2(C) − 3r − 6d *(C) and (D) shall be equal and not more than A or B. Where (C) and (D) are to be minimized the following formula may be used:* L = 2A + 3B + max(14d, 150)
64		A + B + C + 2D + E + (F) − 3 r − 6d See note 2.
67		A

Shape code	Shape	Total length of bar (L) measured along centre line
75		π (A − d) + B *Where B is the lap*
77		C π (A − d) *Where B is greater than A/5 this equation no longer applied, in which case the following formula may be used:* L = C ((π(A-d))² + B2)^1/2
98		A + 2B + C + (D) − 2r − 4d Isometric sketch.

Notes

1 The length equations for shapes 14, 15, 25, 26, 27, 28, 29, 34, 35, 36 and 46 are approximate and where the bend angle is greater than 45º, the length should be calculated more accurately allowing for the difference betweenthe specified overall dimensions and the true length measured along the central axis of the bar.

2 5 bends or more may be impractical within permitted tolerances.

3 For shapes with straight and curved lengths (e.g. shape codes 12, 13, 22, 33 and 47) the largest practical mandrel size for the production of a continuous curve is 400 mm.

4 Stock lengths are available in a limited number of lengths (e.g. 6m, 12m). Dimension A for shape code 01 should be regarded as indicative and used for the purpose of calculating total length. Actual delivery lengths should be by agreement with the supplier.

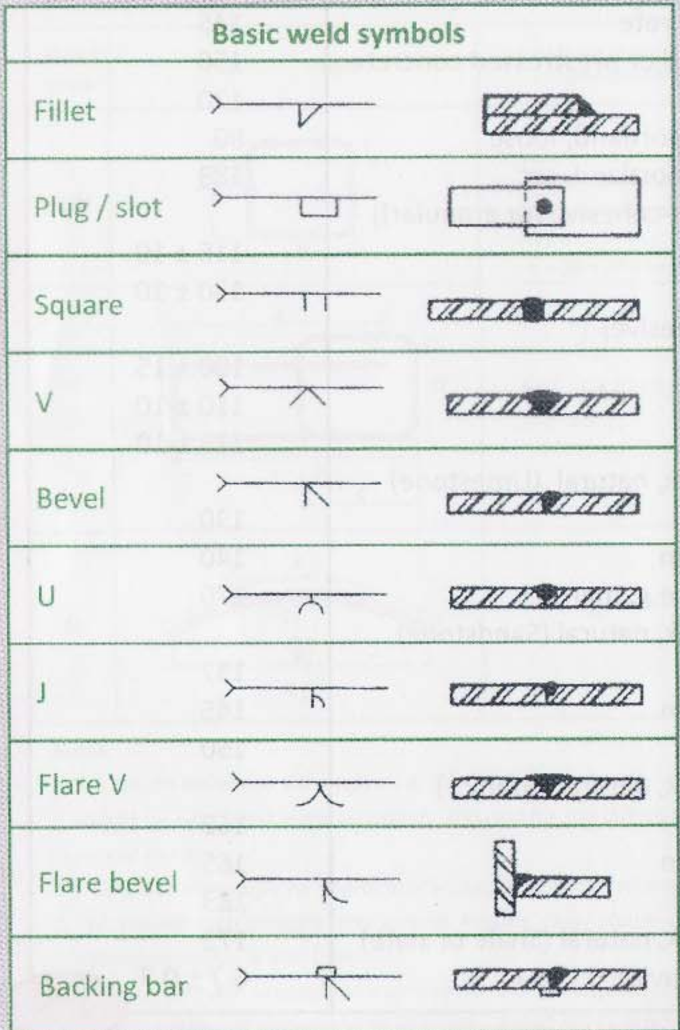
APPENDIX (F) : Unit Weights

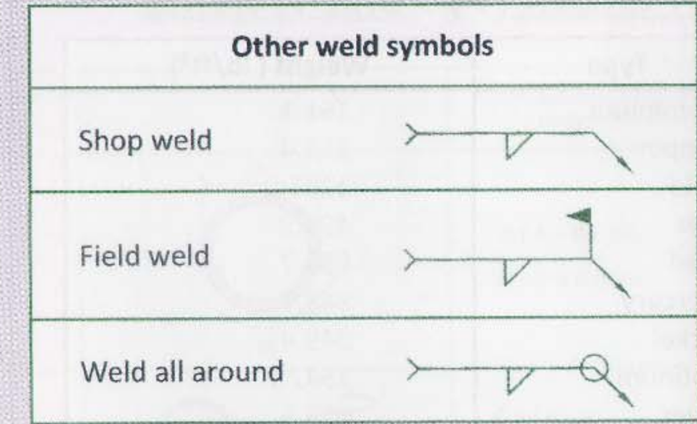
Type	Weight (lb/ft³)
Aluminium	161.1
Copper	555.4
Gold	1205.0
Iron	491.3
Lead	686.7
Mercury	848.7
Nickel	549.4
Platinum	1342.2
Silver	655.5
Tin	448.2
Uranium	1167
Zinc	448.6
Sea Water	64.0
Water	62.4
Ice	57.2

Type	Weight (lb/ft²)
Asbestos cement sheeting,	
Flat 0.25 in. wallboard	1.4
Fully-compressed	2.5
Glass (Sheet)	
32 oz, 0.156 in. thick	2
Glass (Cast, clear plate)	
0.25 in. thick	3.3
0.5 in. thick	6.5
Plywood (per mm thick)	0.125 ± 0.025

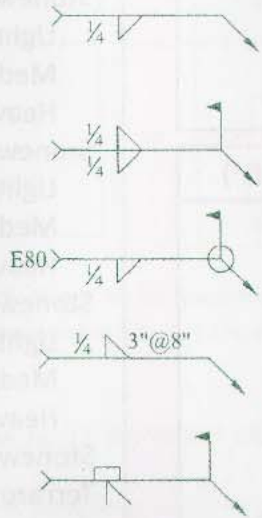
Type	Weight (lb/ft³)
Plain concrete	145
Reinforced or prestressed concrete	150
Bricks	120
Cement, portland, loose	90
Cement, portland, set	183
Soils (Non-cohesive (or granular))	
Loose	115 ± 10
Dense	130 ± 10
Soils (Cohesive)	
Soft	100 ± 15
Firm	110 ± 10
Stiff	125 ± 10
Stonework, natural (Limestone)	
Light	130
Medium	140
Heavy, e.g. marble	170
Stonework, natural (Sandstone)	
Light	137
Medium	145
Heavy	150
Stonework, natural (Granite)	
Light	162
Medium	165
Heavy	183
Stonework, natural (Shale or slate)	175
Terrazo (Paving 0.625 in.)	6.7 ± 0.7

APPENDIX (G) : Weld Symbols





Weld symbol examples



a. *¼" fillet weld, shop welded on near side*

b. *¼" fillet weld, field welded on both sides*

c. *¼" fillet weld, field weld all around, use E80 electrodes*

d. *¼" fillet weld, shop welded on far side, 3"-long welds at 8" on center*

e. *single bevel weld, field welded with backing bar*

APPENDIX (H) : Steel Design Glossary

Amplification Factor A multiplier used to increase the computed moment or deflection in a member to account for the eccentricity of the load.

Annealing A process in which steel is heated to an intermediate temperature range, held at that temperature for several hours, and then allowed to slowly cool off to room temperature. The resulting steel has less hardness and brittleness, but more ductility.

Beam-Column A column that is subjected to axial compression loads as well as bending moments.

Bearing Wall Construction Building construction where all the loads are transferred to the walls and thence down to the foundations.

Braced Frame A frame that has resistance to lateral loads supplied by some type of auxiliary bracing.

Buckling Load The load at which a straight compression member assumes a deflected position.

Built-Up Member A member made up of two or more steel elements bolted or welded together to form a single member.

Camber The construction of a member bent or arched in one direction so that it won't look so bad when the service loads bend it in the opposite direction.

Cast Iron An iron with a very low carbon content.

Cladding The exterior covering of the structural parts of a building.

Cold-Formed Light-Gage Steel Shapes Shapes made by cold bending thin sheets of carbon or low-alloy steels into desired cross sections.

Column A structural member whose primary function is to support compressive loads.

Compact Section A section that has a sufficiently stocky profile so that it is capable of developing a fully plastic stress distribution before buckling.

Composite Beam A steel beam made composite with a concrete slab by providing shear transfer between the two

Composite Column A column constructed with rolled or built up steel shapes, encased in concrete or with concrete placed inside steel pipes or tubes

Coping The cutting back of the flanges of a beam to facilitate its connection to another beam

Drift Lateral deflection of a building.

Drift Index The ratio of lateral deflection of a building to its height.

Effective Length The distance between points of zero moment in a column; that is, the distance between its inflection points.

Euler Load The compression load at which a long and slender member will buckle elastically.

Eyebar A pin-connected tension member whose ends are enlarged with respect to the rest of the member so as to make the strength of the ends approximately equal to the strength of the rest of the member.

Fillet Weld A weld placed in the corner formed by two overlapping parts in contact with each other

First-Order Analysis Analysis of a structure in which equilibrium equations are written based on an assumed nondeformed structure.

Gage Transverse spacing of bolts measured perpendicular to the long direction of the member

Girder A rather loosely used term usually indicating a large beam and perhaps one into which smaller beams are framed.

Groove Welds Welds made in grooves between members that are being joined. They may extend for the full thickness of the parts (complete-penetration groove welds) or they may extend for only a part of the member thickness (partial-penetration groove welds)

Steel Design Glossary *contd.*

Instability A situation occurring in a member where increased deformation of that member causes a reduction in its load-carrying ability.

Ironworker A person performing steel erection (it's a name carried over from the days when iron structural members were used).

Joists The closely spaced beams supporting the floors and roofs of buildings.

Local Buckling The buckling of the part of a larger member that precipitates failure of the whole member.

Mild Steel A ductile low-carbon steel.

Net Area Gross cross-sectional area of a member minus any holes, notches, or other indentations.

Nominal Loads The magnitudes of loads specified by a particular code.

Nominal Strength The theoretical ultimate strength of a member or connection.

Noncompact Section A section that cannot be stressed to a fully plastic situation before buckling occurs. The yield stress can be reached in some but not all of the compression elements before buckling occurs.

P-Delta Effect Changes in column moments and deflections due to lateral deflections.

Pitch The longitudinal spacing of bolts measured parallel to the long direction of a member

Plate Girder A built-up steel beam

Ponding A situation on a flat roof where water accumulates faster than it runs off.

Sag Rods Steel rods that are used to provide lateral support for roof purlins. They also may be used for the same purpose for girts on the sides of buildings

Second-Order Analysis Analysis of a structure for which equilibrium equations are written that include the effect of the deformations of the structure.

Section Modulus The ratio of the moment of inertia taken about a particular axis of a section divided by the distance to the extreme fiber of the section measured perpendicular to the axis in question.

Service Loads The loads that are assumed to be applied to a structure when it is in service (also called *working loads*).

Shear Center The point in the cross section of a beam through which the resultant of the transverse loads must pass so that no torsion will occur.

Shear Wall A wall in a structure that is specially designed to resist shears caused by lateral forces such as wind or earthquake in the plane of the wall.

Sidesway The lateral movement of a structure caused by unsymmetrical loads or by an unsymmetrical arrangement of building members.

Slenderness Ratio The ratio of the effective length of a column to its radius of gyration, both pertaining to the same axis of bending.

Stiffener A plate or an angle usually connected to the web of a beam or girder to prevent failure of the web

Story Drift The difference in horizontal deflection at the top and bottom of a particular story.

Strain-Hardening Range beyond plastic strain in which additional stress is necessary to produce additional strain.

Unbraced Frame A frame whose resistance to lateral forces is provided by its members and their connections.

Unbraced Length The distance in a member between points that are braced.

Upset Rods Rods whose ends are made larger than the regular bodies of the rods. Threads are cut into the upset ends, but the area at the root of the thread in each rod is larger than that of the regular part of the bar

Web Buckling The buckling of the web of a member

Web Crippling The failure of the web of a member near a concentrated force

Wrought Iron An iron with a very high carbon content.

APPENDIX (I) : Conversion of Units

SI Conversion Factors Pound-Inch Units to SI Units

Overall Geometry	
Spans	1 ft = 0.3048 m
Displacements	1 in. = 25.4 mm
Surface area	1 ft^2 = 0.0929 m^2
Volume	1 ft^3 = 0.0283 m^3
	1 yd^3 = 0.765 m^3
Structural Properties	
Cross-sectional dimensions	1 in. = 25.4 mm
Area	1 in^2 = 645.2 mm^2
Section modulus	1 in^3 = 16.39 × 10^3 mm^3
Moment of inertia	1 in^4 = 0.4162 × 10^6 mm^4
Material Properties	
Density	1 lb/ft^3 = 16.03 kg/m^3
Modulus and stress	1 lb/in^2 = 0.006895 MPa
	1 kip/in^2 = 6.895 MPa

SI Conversion Factors Pound-Inch Units to SI Units *contd.*

Loadings	
Concentrated loads	1 lb = 4.448 N
	1 kip = 4.448 kN
Density	1 lb/ft^3 = 0.1571 kN/m^3
Linear loads	1 kip/ft = 14.59 kN/m
Surface loads	1 lb/ft^2 = 0.0479 kN/m^2
	1 kip/ft^2 = 47.9 kN/m^2
Stress and Moments	
Stress	1 lb/in^2 = 0.006895 MPa
	1 kip/in^2 = 6.895 MPa
Moment or torque	1 lb-ft = 1.356 N-m
	1 kip-ft = 1.356 kN-m

Metric Equivalents of Length* , Mass* & Area*

Length (Metric to English)

Metric		English	Unit
1 mm.	=	0.03937	in.
	=	0.003281	ft.
	=	0.001094	yd.
1 cm.	=	0.3937	in.
	=	0.03281	ft.
	=	0.01094	yd.
1 metre	=	39.37	in.
	=	3.2808	ft.
	=	1.0936	yd.
	=	0.1988	rd.
	=	0.04971	chain
	=	$0.0_3 6214$	mi.
1 kilometre	=	3280.833	ft.
	=	1093.611	yd.
	=	198.838	rode
	=	49.7095	chains
	=	0.6214	mi.

Mass (Metric to English)

Metric		English	Unit
1 milligram	=	0.01543	grain
	=	$0.0_4 3215$	oz. Troy
	=	$0.0_4 3527$	oz. avoir.
1 gram	=	15.4324	grains
	=	0.03215	oz. Troy
	=	0.03527	oz. avoir.
	=	$0.0_2 2679$	lb. Troy
	=	$0.0_2 2205$	lb. avoir.
1 kilogram	=	32.1508	oz. Troy
	=	35.2740	oz. avoir.
	=	2.6792	lb. Troy
	=	2.2046	lb. avoir.
	=	$0.0_2 1102$	short ton
	=	$0.0_3 9842$	long ton
1 metric ton	=	2204.62	lb. avoir.
	=	1.1023	short tons
	=	0.9842	long ton

Area (Metric to English)

Metric		English	Unit
1 sq. mm.	=	1973.55	cir. mils
	=	0.001550	sq. in.
	=	$0.0_4 10764$	sq. ft.
	=	$0.0_5 1196$	sq. yd.
1 sq. m.	=	1,549.9969	sq. in.
	=	10.7639	sq. ft.
	=	1.1960	sq. yd.
	=	0.002471	sq. chain
	=	$0.0_3 2471$	acre
	=	$0.0_6 3861$	sq. mi.
1 hectare	=	107,638.7	sq. ft.
	=	11,959.85	sq. yd.
	=	24.710	sq. chain
	=	2.4710	acres
	=	0.003861	sq. mi.
1 sq. km.	=	10,763,867.36	sq. ft.
	=	1,195,985.26	sq. yd.
	=	2,471.050	sq. chains
	=	247.1045	acres
	=	0.3861	sq. mi.

* A subscript after a figure indicates the number of times it is repeated. Thus $0.0_3 8 = 0.0008$

Metric Equivalents of Density* , Velocity* & Volume*

Density (Metric to English)	
1 gm/cm³	= 0.03613 lb/in³
	= 62.430 lb/ft³
	= 8.3454 lb/U. S. gal
1 kg/m³	= $0.0_4 3613$* lb/in³
	= 0.062430 lb/ft³
	= 1.6856 lb/yd³
	= $0.0_2 8345$* lb/U.S. gal
1 metric ton/m³	
	= 62.4286 lb/ft³
	= 1685.487 lb/yd³
	= 0.8458 short ton/yd³
	= 0.7525 long ton/yd³

Velocity (Metric to English)	
1 cm/sec	= 0.3937 in./sec
	= 0.03281 ft/sec
	= 1.9685 ft/min
	= 0.2237 mi/hr
	= 0.01943 knot
1 m/sec	= 39.37 in./sec
	= 3.2808 ft/sec
	= 196.85 ft/min
	= 2.2369 mi/hr
	= 1.9426 knots
1 m/min	= 0.6562 in./sec
	= 0.05468 ft/sec
	= 3.2808 ft/min
	= 0.03728 mi/hr
	= 0.03238 knot
1 km/hr	= 0.9113 ft/sec
	= 54.6806 ft/min
	= 0.62138 mi/hr
	= 0.5396 knot

Volume (Metric to English)			
1 cu. mm.	=	$0.0_4 6102$	cu. in.
	=	$0.0_3 2705$	fluid dr.
	=	$0.0_4 3381$	fluid oz.
1 cu. cm.	=	0.06102	cu. in.
	=	$0.0_4 3531$	cu. ft.
	=	$0.0_5 1308$	cu. yd.
	=	$0.0_4 2838$	bushel
	=	0.2705	fluid dr.
	=	0.03381	fluid oz.
	=	0.001057	quart
	=	$0.0_3 2642$	gallon
1 litre	=	61.02398	cu. in.
	=	0.035313	cu. ft.
	=	0.0013079	cu. yd.
	=	0.028377	bushel
	=	1.0567	quart
	=	0.2642	gallon
1 hectolitre	=	6,102.398	cu. in.
	=	3.5313	cu. ft.
	=	0.13079	cu. yd.
	=	2.8377	bushels
1 cu. metre	=	61,023.38	cu. in.
	=	35.3133	cu. ft.
	=	1.3079	cu. yd.
	=	28.3773	bushels
	=	1,056.682	quarts
	=	264.170	gallons

* A subscript after a figure indicates the number of times it is repeated. Thus $0.0_3 8 = 0.0008$

List of Structural Engineering Books

by

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FORMER PROFESSOR OF CIVIL ENGINEERING
YANGON INSTITUTE OF TECHNOLOGY

1. "Fundamentals of Structural Analysis" ; 1st edition, 2008, 487 pages
2. "Essentials of Concrete Inspection, Mix Designs, Quality Control" ; 1st edition, 2008, 337 pages
3. "Reinforced Concrete Design" ; 1st edition, 2010, 1200 pages
4. "Refresher Course and Field Reference Manual for Site Engineers and Inspectors" ; 1st edition, 2010, 210 pages