COMPARISON BETWEEN Codes

ECP (EGYPTIAN CODE OF PRACTICE)

ACI (AMERICAN CONCRETE INSTITUTE)

BSI (BRITISH STANDARD INSTITUTE)

PREPARD BY

STRUCTURAL DEPARTMENT TEAM

CHECKED BY

STRUCTURAL DEPARTMENT SENIOR

ENG. WAEL MOHMED ABDUL MAQSOOD

INTRODUCTION

This comparison between codes provides minimum requirements for design and construction of structural concrete members of any structure in different codes, That doesn't mean that the structural design safe in code and unsafe at the other one But this difference due to factor of safety used, the quality control of materials used in the construction …etc. as a result we have to study carefully those differences between codes in order to we can do a smart design.

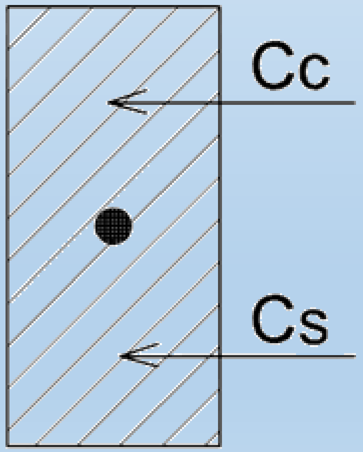
- COMPARISON BETWEEN ECP & ACI & BSI

Columns Design

Egyptian code (2007)

AXIAL LOADED COLUMN:

- عند تصميم العمود يتم حساب اقصى حمل يتحمله العمود و منه يتم حساب مقاومه الضغط الخرسانى للعمود و مقاومه الضغط الناتج من تسليح العمود كما هو موضح بالمعادله التالية :



$P_{ult.} = Cc + Cs$

- $Pu = C_C + C_S$
- $Pu = \frac{0.67 * Fcu * Ac}{\gamma c} + \frac{Asc * Fy}{\gamma s}$

Egyptian code (2007)

- Pu : مقاومة العمود
- Cc : مقاومة القطاع الخرساني
- Cs : مقاومة تسليح العمود
- γ_c , γ_s: Factor of safety for material .

Where : $\gamma_c = 1.75$, $\gamma_s = 1.34$

$$\gamma_C = 1.5[(\frac{7}{6}) - (\frac{e \backslash t}{3})] \quad , \quad \gamma_S = 1.15[(\frac{7}{6}) - (\frac{e \backslash t}{3})]$$

Egyptian code (2007)

$$Pu = C_C + C_S$$

$$Pu = \frac{0.67 * Fcu * Ac}{\gamma c} + \frac{Asc * Fy}{\gamma s}$$

$$Pu = \frac{0.67 * Fcu * Ac}{1.75} + \frac{Asc * Fy}{1.34}$$

- The Existence of moment leads to a reduction in axial load capacity. Thus the code impose a further reduction on the column strength by reducing the capacity by 10% So, the equation:

$$Pu = \frac{0.90 * 0.67 * Fcu * Ac}{1.75} + \frac{0.90 * Asc * Fy}{1.34}$$

Egyptian code (2007)

$$Pu = 0.35*Fcu*Ac+0.67*Asc*Fy$$

Where :

Ac: Area of concrete.

Asc : Total Area of steel.

Fcu : Concrete strength.

Fy : yield strength for steel.

ACI CODE

- و فى الكود الامريكى يتم حساب الحمل الذي يتحمله العمود ومنه يتم عمل تصميم للعمود

- $Po = 0.85*Fc` (Ag - Ast) + Fy*Ast$

Where :

Ag : The gross area of concrete.

Ast : Total area of the longitudinal reinforcement.

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Conditions for column (not shear wall)	$t \leq 5\ b$	$t \leq 4\ b$	**$t \leq 4\ b$**
Effect of shape of column in slenderness ratio.	The shape of cross section of column is taken into consideration.	The shape of cross section of column is taken into consideration	**The shape of cross section of column isn't taken into consideration**

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
As_{min}	$\frac{0.6}{100}\ A_g$	$\frac{1}{100}\ A_g$	$\frac{0.4}{100}\ A_g$
As_{max}	4% A_g for Interior col. 5% A_g for Edge col. 6% A_g for Corner col.	8% A_g	**6% A_g**
S_{max}	250 mm	6 inch=150 mm	**155 mm**
S_{min}	70mm	25mm	**30mm**

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr><td>Classification of structure according to buckling design.</td><td>Braced & Unbraced</td><td>Sway & non- Sway</td><td>Braced & Un Braced</td></tr>
<tr><td>Maximum slenderness ratio.</td><td>

	Rec.	Circular
Braced	30	25
Unbraced	23	18

</td><td>not braced against side sway
$\frac{K\ Lu}{r} \leq 22$
braced against side sway
$\frac{K\ Lu}{r} \leq 34 - 12(M1/M2) \leq 40$</td><td>Slenderness Limits For Columns $L_0 < 60b$
Slenderness of unbraced Columns
$L_0 = \frac{100b^2}{h} \leq 60b$</td></tr>
<tr><td>Neglect slenderness effect if :</td><td>Braced $\frac{KH}{b1\ or\ t1} < 15$
Unbraced $\frac{KH}{b1\ or\ t1} < 10$</td><td>Non-sway
$\frac{K\,Lu}{r} \leq 34 - 12\ (\frac{M_1}{M_2}) \leq 40$
Sway
$\frac{K\ Lu}{r} \leq 22$</td><td>$L_e = \beta L_0$
Values of β for braced columns
<table>
<tr><td rowspan="2">End Condition at top</td><td colspan="3">End Condition at bottom</td></tr>
<tr><td>1</td><td>2</td><td>3</td></tr>
<tr><td>1</td><td>0.75</td><td>0.80</td><td>0.90</td></tr>
<tr><td>2</td><td>0.80</td><td>0.85</td><td>0.95</td></tr>
<tr><td>3</td><td>0.90</td><td>0.95</td><td>1.00</td></tr>
</table>
</td></tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>Neglect slenderness effect if :</td>
<td>فى حالة الأعمدة فى المبانى المقيدة جانبيا يؤخذ طول الانبعاج He مساويا للأصغر من
$H_e = H_0\left(0.7 + 0.05(\alpha_1 + \alpha_2)\right) \leq H_0$
$H_e = H_0\left(0.85 + 0.05(\alpha_{min})\right) \leq H_0$
وفى حالة الأعمدة فى المبانى الغير مقيدة جانبيا يؤخذ طول الانبعاج He مساويا للأصغر من
$H_e = H_0\left(1 + 0.15(\alpha_1 + \alpha_2)\right) \geq H_0$
$H_e = H_0\left(2 + 0.3(\alpha_{min})\right) \geq H_0$

$\alpha = \Sigma \frac{\Sigma \frac{E_cI_c}{H_0}}{\frac{E_cI_b}{L_b}}$
حيث أن H_0 هو ارتفاع العمود الخالص
وα_{min} هى القيمة الأصغر من α_1 عند الطرف السفلى و α_2 عند الطرف العلوى</td>
<td>K: effective length factor (for braced (non sway) frame ≤ 1

Lu: unsupported height of column from top of floor to the bottom of the beams or slab in the slab above

R: radius of gyration, equal to 0.3 and 0.25 times the overall depth of rectangular and circular columns , respectively

M1/M2: ratio of the moments at the two ends of the column</td>
<td>Values of β for unbraced columns
<table>
<tr><th rowspan="2">End Condition at top</th><th colspan="3">End Condition at bottom</th></tr>
<tr><th>1</th><th>2</th><th>3</th></tr>
<tr><td>1</td><td>1.2</td><td>1.3</td><td>1.6</td></tr>
<tr><td>2</td><td>1.3</td><td>1.5</td><td>1.8</td></tr>
<tr><td>3</td><td>1.6</td><td>1.8</td><td>-</td></tr>
<tr><td>4</td><td>2.2</td><td>-</td><td>-</td></tr>
</table>
Where:
L_0: clear height between end restraints
b: width of column
h: depth of cross section
L_e:effective height of column</td>
</tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>Cover</td>
<td>
<table>
<tr><td>Fire Resistance</td><td>Columns</td></tr>
<tr><td>Unite (hour)</td><td>unite (mm)</td></tr>
<tr><td>0.5</td><td>20</td></tr>
<tr><td>1</td><td>20</td></tr>
<tr><td>1.5</td><td>20</td></tr>
<tr><td>2</td><td>25</td></tr>
<tr><td>3</td><td>25</td></tr>
<tr><td>4</td><td>25</td></tr>
</table>
</td>
<td>-Concrete not exposed to weather or in contact with ground:
Primary reinforcement,
Ties, stirrups, spirals............40</td>
<td>
<table>
<tr><td>Fire Resistance</td><td>Columns</td></tr>
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</table>
</td>
</tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

- **<u>Notes:</u>**
- Column according to (ECP) is unsafe due to large factor of safety although it is safe according to (BSI & ACI).

- As (BSI) < As (ACI)

- COMPARISON BETWEEN ECP & ACI & BSI

Beam Design

- COMPARISON BETWEEN ECP & ACI & BSI

Shear Design

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Critical Section	$At\,\frac{\boldsymbol{d}}{\boldsymbol{2}}\,from\ the$ $face\ of\ support$	$At\,\frac{\boldsymbol{d}}{\boldsymbol{2}}\,from\ the$ $face\ of\ support$	$At\,\frac{\boldsymbol{d}}{\boldsymbol{2}}\,from\ the$ $face\ of\ support$
Max. strength (stress/force)	$*\ q_{max} = 0.7\sqrt{\frac{f_{cu}}{\boldsymbol{\gamma}_C}}\ \mathrm{N}/mm^2$ $or\ 4\ N/mm^2\ least\ of\ them$ $-\ stress$	$*\ V_{max} = \emptyset 8\sqrt{f'_c}\,b_w\,d$ $-\ Force.\quad (English\ unit)$	$*\ V_{max} = 0.8\sqrt{f_{cu}}$ $or\ 5\ N/mm^2\ least\ of\ them$ $-\ stress$

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Nominal shear strength of concrete (stress/force)	$* \quad q_{cu} = 0.24 \sqrt{\frac{f_{cu}}{\gamma_C}}\ \mathrm{N/mm^2}$ • *stress* • *depends on* f_{cu}	$* \quad \emptyset V_C = \emptyset 2 \sqrt{f'_c}\, bd$ • *Force ... ... (English unit)* • *depends on* f'_c , b & d.	$* \quad V_c = \mathbf{0.79} \left[\frac{100 A_S}{b_v d}\right]^{\frac{1}{3}} \left(\frac{400}{d}\right)^{\frac{1}{4}} / \gamma_m$ • *depends on* f_{cu} , d & A_S
Design value of shear (stress/force)	$* \; q_u = \frac{Q_u}{b.d}$	$* \quad V_U \rightarrow$ *shear force.*	$* \quad V_U \rightarrow \frac{Q_U}{bd}$

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>R. F. T</td>
<td>$- IF\ q_u < q_{cu}$
$\rightarrow Use\ min.\ shear\ RFT.$
$5\emptyset 8/m$
$- IF\ q_{cu} < q_u \leq q_{max}$
$\rightarrow Use\ shear\ RFT.$

$* \left(q_u - \frac{q_{cu}}{2} \right) = \frac{A_{st}}{b.s} * \left(\frac{f_y}{\gamma_s} \right)$

$- IF\ q_u > q_{max}$
$\rightarrow Increse\ dimension.$</td>
<td>
<table>
<tr><th colspan="2"></th><th>$V_u \le \phi V_c/2$</th><th>$\phi V_c/2 < V_u \le \phi V_c$</th><th>$\phi V_c < V_u$</th></tr>
<tr><td colspan="2">Required area of stirrups, A_V</td><td>none</td><td>The larger of
$0.75\sqrt{f'_c}\frac{b_w s}{f_{yt}}$ and $\frac{50 b_w s}{f_{yt}}$</td><td>The largest of
$\frac{(V_u - \phi V_c)s}{\phi f_{yt} d}$, $0.75\sqrt{f'_c}\frac{b_w s}{f_{yt}}$ and $\frac{50 b_w s}{f_{yt}}$</td></tr>
<tr><td rowspan="3">Stirrup spacing, s</td><td>Required</td><td>–</td><td>The smaller of
$\frac{A_v f_{yt}}{0.75\sqrt{f'_c}\, b_w}$ and $\frac{A_v f_{yt}}{50 b_w}$</td><td>The smallest of
$\frac{\phi A_v f_{yt} d}{V_u - \phi V_c}$, $\frac{A_v f_{yt}}{0.75\sqrt{f'_c}\, b_w}$ and $\frac{A_v f_{yt}}{50 b_w}$</td></tr>
<tr><td rowspan="2">Maximum</td><td rowspan="2">–</td><td rowspan="2">The smaller of
$d/2$ and 24 in.</td><td>For
$(V_u - \phi V_c) \le \phi 4\sqrt{f'_c} b_w d$, s is the smaller of $\frac{d}{2}$ and 24 in.</td></tr>
<tr><td>For
$\phi 4\sqrt{f'_c} b_w d < (V_u - \phi V_c) \le \phi 8\sqrt{f'_c} b_w d$,
s is the smaller of $\frac{d}{4}$ and 12 in.</td></tr>
</table>
</td>
<td>$- IF\ V_u < \frac{V_C}{2}$
$\rightarrow No\ shear\ RFT\ is\ used$

$- IF\ V_u < V_c$
$\rightarrow Min\ shear\ RFT\ is\ used$

$\left(\frac{A_{sv}}{S_v}\right) = \left(\frac{0.4 b_v}{0.95 F_{yv}}\right)$ **if V≤V_c+0.4b_vd**

$\left(\frac{A_{sv}}{S_v}\right) = \left(\frac{V - V_c}{0.95 F_{yv} b_t}\right)$ **if V≤V_c+0.4b_vd**</td>
</tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Max spacing between stirrups	$* \frac{d}{2} or\ 200mm$ $the\ least\ of\ them$	$*\ d/2$	**0.75*d*t or four times the web thickness for flanged members When *V* exceeds 1.8*V*c, the maximum spacing should be reduced to 0.5*d*t .**
Min spacing between stirrups	$100mm$	Minimum practical spacing ≈ 3 in. or 4 in.	
Stirrups indention	$45°$	$30°\ or\ more$	$45°\ or\ more$

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Max. grade of stirrups (f_y)	$400\ N/mm^2$	$60.000\ Psi \cong 400\ N/mm^2$	$460\ N/mm^2$
Slabs & footings	$-$ *No shear RFT is used shear is only resisted by concrete.*	$-$ *Shear RFT is used.*	$-$ *Shear RFT is used.*

- COMPARISON BETWEEN ECP & ACI & BSI

Flexural design

Egyptian code (2007)

يتم أولا التأكد من القطاع الخرساني انه آمن باستخدام المعادلة الاتية :

$$d = C1\sqrt{\frac{Mu * 10^5}{b * Fcu}}$$

Where:

d : depth of SEC.

Mu : Ultimate Moment Affecting on SEC.

b : Width of SEC.

Fcu : Characteristic **Concrete** Strength

C : Factor where Cmin=2.65 & Cmax=4.85

Egyptian code (2007)

- بمعلومية القطاع الخرساني ال b*d وكذلك قيمة ال Mult نستطيع التعويض فى المعادلة السابقة وايجاد قيمة ال C .

- فاذا كانت قيمة ال C أقل من ال Cmin =2.65 يكون القطاع غير امن فنزيد أبعاد قطاع الخرسانة .

- واذا كانت قيمة ال C أكبر من ال Cmax فيكون القطاع more safe فلابد من ان تتراوح قيمة ال C بين ال Cmin و Cmax .

- بعد التاكد من أن القطاع امن يتم التعويض فى المعادلة الاتية لايجاد قيمة الحديد في القطاع As .

$$As = \frac{Mu}{Fy * d * J}$$

Egyptian code (2007)

$$As = \frac{Mu}{Fy * d * J}$$

Where

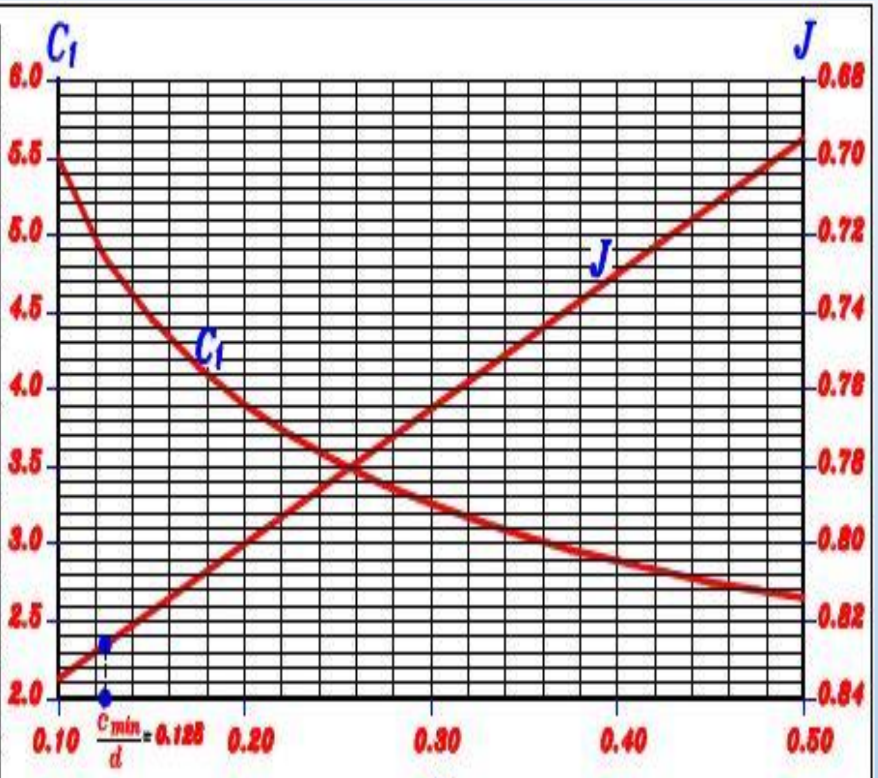
Fy : Yield Stress

d : depth of SEC

Mu: Ultimate Moment Affecting on Section

J: Factor Get It's Value From Table According to Value Of C1

$\frac{c}{d}$	C_1	J
0.1250	4.85	0.826
0.1375	4.64	0.821
0.1500	4.46	0.817
0.1625	4.29	0.813
0.1750	4.15	0.808
0.1875	4.02	0.804
0.2000	3.90	0.800
0.2125	3.79	0.795
0.2250	3.70	0.791
0.2375	3.61	0.786
0.2500	3.53	0.782
0.2625	3.45	0.778
0.2750	3.38	0.773
0.2875	3.32	0.769
0.3000	3.26	0.765
0.3125	3.20	0.760
0.3250	3.15	0.756
0.3375	3.10	0.752
0.3500	3.05	0.747
0.3625	3.00	0.743
0.3750	2.96	0.739
0.3875	2.92	0.734
0.4000	2.89	0.730
0.4125	2.85	0.726
0.4250	2.82	0.721
0.4375	2.78	0.717
0.4500	2.75	0.713
0.4625	2.72	0.708
0.4750	2.70	0.704
0.4875	2.67	0.700
0.5000	2.65	0.695



$$d = C_1 \sqrt{\frac{M_{U.L.}}{f_{cu}\, b}}$$

$$A_s = \frac{M_{U.L.}}{J\, f_y\, d}$$

ACI CODE

يتم أولا التأكد من قطاع الخرساني انه امن باستخدام المعادلة الاتية

لايجاد قيمة ال Rn :

$$\emptyset\, Rn = \frac{Mu}{bd^2}$$ ………… *We can get Rn=√√√*

Where

d: depth of Section.

Mu: Ultimate Moment Affecting on Section.

b: Width of Section.

$\emptyset = 0.9$

ACI CODE

Rn From Table (1-C) depend on Fc' & B1

$$\boldsymbol{Rn = \rho\left(1 - \frac{0.5\rho Fy}{0.85\, Fc'}\right)} \qquad \boldsymbol{\rho\, min \leq \rho \leq \rho max}$$

$$\boldsymbol{\rho\, min = \frac{3\sqrt{fc'}}{Fy}} \qquad \boldsymbol{\rho\, max = \frac{0.319\, \beta 1 fc'}{Fy}}$$

Where R_n :STANDS FOR NOMINAL RESISTANCE (STRENGTH)

ρ: ratio of ***As*** to ***bd***

β1 = 0.85 … *for fc'* ≤ 4000 psi

= 0.85 - 0.05 ($\frac{fc'-4000}{1000}$) …. *for* 4000 psi < *fc'* < 8000 psi

= 0.65 … *for fc'* ≤ 8000 psi

BSI CODE

- بالتعويض فى المعادلة التالية بأبعاد العمود ال d , b .
- وكذلك قيمة ال Mult .
- فنحصل على قيمة K والتى لابد ان تقل عن 0.156 .

$$\mathbf{K}= Mu\frac{10^5}{b*d^2*Fcu} = \;.. \qquad < (K' = 0.156)$$

- بعد التعويض فى المعادلة السابقة والتأكد من أن قطاع الخرسانه امن تحت تأثير قيمة Mult .
- وتحقيق شرط أن K تقل عن (K`=0.156) عدا ذلك يتم زيادة التسليح في منطقة الضغط.

BSI CODE

- يتم بعد ذلك حساب قيمة تسليح الخرساني باستخدام المعادلة التالية :

$$\boldsymbol{As} = \mathrm{Mu} * \frac{10^5}{0.87 * Fy * (d * 0.95)}$$

$where$:

Fy = characteristic concrete strength.

Fcu = characteristic yield strength.

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
The Strain Of Concrete ζcu	0.003	0.003	0.0035
B1 section	$\frac{kL}{10} + b \quad or \;\; 6ts + b$ (L section) $\frac{kL}{5} + b \;\; or \;\; 16ts + b$ ((T section)	$0.12L \;\; Or \;\; 6ts$ *0.25 L Or 8ts*	$b + \frac{L}{10}$ $b + \frac{L}{5}$

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Minimum Area Steel Asmin	$\frac{1.1}{Fy}bd$ *The least* $1.3Asreq$ *the biggest* $\frac{0.15}{100}bd(H.T.S)$	* $3\sqrt{fc'}bw\frac{d}{fy}$ *Where:* $3\sqrt{fc'}bw\frac{d}{fy} \geq 200\text{bw}\frac{\text{d}}{\text{fy}}$	Table (1-2)
Depth of compression block	0.8 c	0.85 c	0.9 c

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Conditions For using Empirical Methods	*L.L* $\ngtr$ *1.5D.L* *Spans difference not exceed 20%*	*L.L* $\ngtr$ *3 D.L*	*L.L* $\ngtr$ *D.L*
Min spacing	H.agg. + 5mm Bigger $\emptyset\ max$ $\emptyset\ max$ 2.5cm	$\emptyset \geq 1\ inch$	Hagg + 5mm Bigger 2.5cm
Moment & Normal Sections	We can neglect Normal Force if : $N \leq 0.04\ fcu\ bt$	We Can neglect Normal if *e=M/N≤(*1 in. or 0.05*h)*	We Can neglect Normal Force if: $N \leq 0.1\ fcu\ bt$

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
One Way slab (+ve moment) Intermediate span Empirical	$\frac{WL^2}{12}$ $=0.083\,WL^2$ $= 0.083\ \ F\,L$	$\frac{WL^2}{16}$ $=0.0625\,WL^2$ $=0.0625\ F\,L$	***0.09 FL***
Concrete Strength	$0.67\frac{Fcu}{\gamma c}$	0.85 fc'	$0.67\frac{Fcu}{\gamma c}$
Concrete Young's Modulus	$4400\sqrt{Fcu}\ \ N/mm^2$	$= 57000\sqrt{fc'}$ Ps $= 4200\sqrt{fcu}$ N/mm^2 $= 4700\sqrt{fc'}$ N/mm^2	$= 5.5\sqrt{\frac{fcu}{\gamma m}}$ KN/mm^2 $= 4490\sqrt{fcu}$ N/mm^2

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Yield Stress in Steel Fy	Deformed Steel Fy $\leq 400\ N/mm^2$ Mild Steel Fy $\leq 280\ N/mm^2$ Cold drown smooth welded mesh Fy $\leq 300\ N/mm^2$	Fy > 80.000 psi Fy $= 550\ N/mm^2$	Hot rolled mild steel Fy $= 250\ \mathrm{N/mm^2}$ Height Yield (hot rolled or cold worked) Fy $= 460\ \mathrm{N/mm^2}$

- COMPARISON BETWEEN ECP & ACI & BSI

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<td>Cover</td>
<td>
<table>
<tr><td>Fire Resistance</td><td colspan="2">Beams unite (mm)</td></tr>
<tr><td>Unite (hour)</td><td>Simply Supported</td><td>Continouse</td></tr>
<tr><td>0.5</td><td>20</td><td>20</td></tr>
<tr><td>1</td><td>20</td><td>20</td></tr>
<tr><td>1.5</td><td>30</td><td>25</td></tr>
<tr><td>2</td><td>45</td><td>40</td></tr>
<tr><td>3</td><td>60</td><td>50</td></tr>
<tr><td>4</td><td>70</td><td>60</td></tr>
</table>
</td>
<td>-Concrete not exposed to weather or in contact with ground:

Primary reinforcement, ties, stirrups, spirals............ 40</td>
<td>
<table>
<tr><td>Fire Resistance</td><td colspan="2">Beams unite (mm)</td></tr>
<tr><td>Unite (hour)</td><td>Simply Supported</td><td>Continouse</td></tr>
<tr><td>0.5</td><td>20</td><td>20</td></tr>
<tr><td>1</td><td>20</td><td>20</td></tr>
<tr><td>1.5</td><td>20</td><td>20</td></tr>
<tr><td>2</td><td>40</td><td>30</td></tr>
<tr><td>3</td><td>60</td><td>40</td></tr>
<tr><td>4</td><td>70</td><td>50</td></tr>
</table>
</td>
</tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

Situation	Definition of percentage	Minimum percentage	
		$f_y = 250\ N/mm^2$ %	$f_y = 460\ N/mm^2$ %
Tension reinforcement			
Sections subjected mainly to pure tension	$100A_s/A_c$	0.8	0.45
Sections subjected to flexure:			
a) flanged beams, web in tension:			
1) $b_w/b < 0.4$	$100A_s/b_wh$	0.32	0.18
2) $b_w/b \geq 0.4$	$100A_s/b_wh$	0.24	0.13
b) flanged beams, flange in tension:			
1) T-beam	$100A_s/b_wh$	0.48	0.26
2) L-beam	$100A_s/b_wh$	0.36	0.20
c) rectangular section (in solid slabs this minimum should be provided in both directions)	$100A_s/A_c$	0.24	0.13
Compression reinforcement (where such reinforcement is required for the ultimate limit state)			
General rule	$100A_{sc}/A_{cc}$	0.4	0.4
Simplified rules for particular cases:			
a) rectangular column or wall	$100A_{sc}/A_c$	0.4	0.4
b) flanged beam:			
1) flange in compression	$100A_{sc}/bh_f$	0.4	0.4
2) web in compression	$100A_{sc}/b_wh$	0.2	0.2
c) rectangular beam	$100A_{sc}/A_c$	0.2	0.2
Transverse reinforcement in flanges or flanged beams (provided over full effective flange width near top surface to resist horizontal shear)	$100A_{st}/h_fl$	0.15	0.15

Table (1-2)

- COMPARISON BETWEEN ECP & ACI & BSI

Slabs Design

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Types of Slabs	• solid slab • Flat slab • Hollow block slab	• **One way slab** • **Two way slab** a)- Two way beam supported slab b)- Flat plate c)- Flat slab d)- Waffle slab	• **solid slab** • **Flat slab** • **Hollow block slab**

Solid Slab Design

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>Thickness of slabs</td>
<td>

- **<u>One way slab</u>**

-simply supported slab
$t_{min} = \frac{l}{30}$
-one end continuous
$t_{min} = \frac{l}{35}$
-both ends continuous
$t_{min} = \frac{l}{40}$
-cantilever
$t_{min} = \frac{l}{10}$
Where l is the direction of the load
</td>
<td>

- **<u>One way slab</u>**

-simply supported slab
$t_{min} = \frac{l}{20}$
-one end continuous
$t_{min} = \frac{l}{24}$
-both ends continuous
$t_{min} = \frac{l}{28}$
-cantilever
$t_{min} = \frac{l}{10}$
Where l is the direction of the load
</td>
<td>
Table 3.9 — Basic span/effective depth ratio for rectangular or flanged beams
<table>
<tr><th>Support conditions</th><th>Rectangular section</th><th>Flanged beams with $\frac{b_w}{b} \leq 0.3$</th></tr>
<tr><td>Cantilever</td><td>7</td><td>5.6</td></tr>
<tr><td>Simply supported</td><td>20</td><td>16.0</td></tr>
<tr><td>Continuous</td><td>26</td><td>20.8</td></tr>
</table>
</td>
</tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>Thickness of slabs</td>
<td>

- **<u>Two way slab</u>**

-simply supported slab
$t_{min} = \frac{l}{35}$
-one end continuous
$t_{min} = \frac{l}{40}$
-both ends continuous
$t_{min} = \frac{l}{45}$

<u>* Where</u>
l :is short span length measured from center to center
</td>
<td>

- **<u>Two way slab</u>**

-fy = 280 (N/mm^2)
$t_{min} = \frac{l}{36}$

-fy = 420 (N/mm^2)
$t_{min} = \frac{l}{33}$

-fy = 520 (N/mm^2)
$t_{min} = \frac{l}{28}$

* <u>Where</u>
l : is long span length measured face to face from support
</td>
<td>

Table 3.9 — Basic span/effective depth ratio for rectangular or flanged beams

<table>
<tr><th>Support conditions</th><th>Rectangular section</th><th>Flanged beams with $\frac{b_w}{b} \leq 0.3$</th></tr>
<tr><td>Cantilever</td><td>7</td><td>5.6</td></tr>
<tr><td>Simply supported</td><td>20</td><td>16.0</td></tr>
<tr><td>Continuous</td><td>26</td><td>20.8</td></tr>
</table>
</td>
</tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

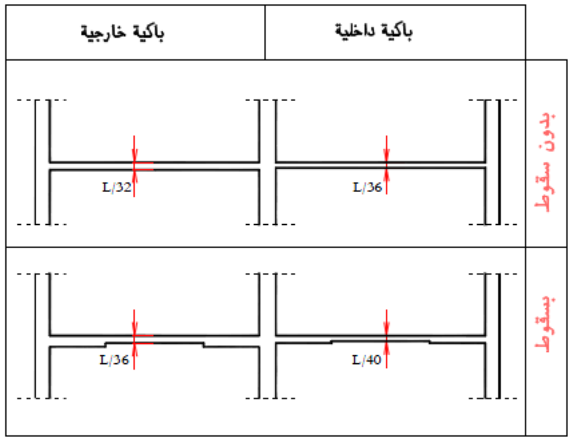
<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>Cover</td>
<td>
<table>
<tr><td>Fire Resistance</td><td colspan="2">Floors unite (mm)</td></tr>
<tr><td>Unite (hour)</td><td>Simply Supported</td><td>Continouse</td></tr>
<tr><td>0.5</td><td>15</td><td>15</td></tr>
<tr><td>1</td><td>20</td><td>20</td></tr>
<tr><td>1.5</td><td>25</td><td>20</td></tr>
<tr><td>2</td><td>35</td><td>25</td></tr>
<tr><td>3</td><td>45</td><td>35</td></tr>
<tr><td>4</td><td>55</td><td>45</td></tr>
</table>
</td>
<td>-Shells, folded plate members:
No. 19 bar and larger..........20
No. 16 bar, MW200 or MD200 wire, and smaller…………… 13</td>
<td>
<table>
<tr><td>Fire Resistance</td><td colspan="2">Floors unite (mm)</td></tr>
<tr><td>Unite (hour)</td><td>Simply Supported</td><td>Continouse</td></tr>
<tr><td>0.5</td><td>20</td><td>20</td></tr>
<tr><td>1</td><td>20</td><td>20</td></tr>
<tr><td>1.5</td><td>25</td><td>20</td></tr>
<tr><td>2</td><td>35</td><td>25</td></tr>
<tr><td>3</td><td>45</td><td>35</td></tr>
<tr><td>4</td><td>55</td><td>45</td></tr>
</table>
</td>
</tr>
</table>

Flat Slab Design

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Thickness of slabs	see below	see below	see below

ECP



<u>* Where</u>

l :is long span length measured from center to center

ACI

TABLE 9.5(c)—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS*

	Without drop panels‡			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
f_y, MPa†	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
280	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
420	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
520	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

*For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.

†For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.

‡Drop panels as defined in 13.2.5.

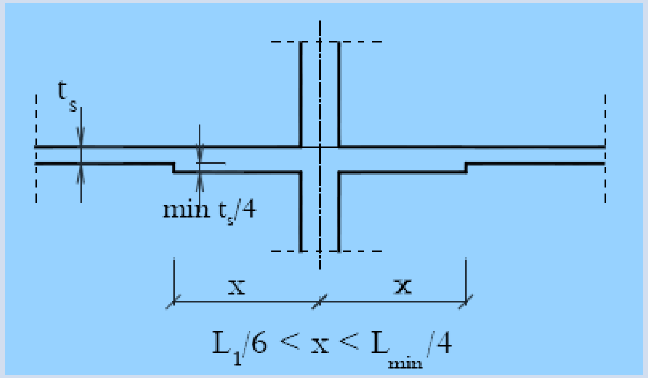
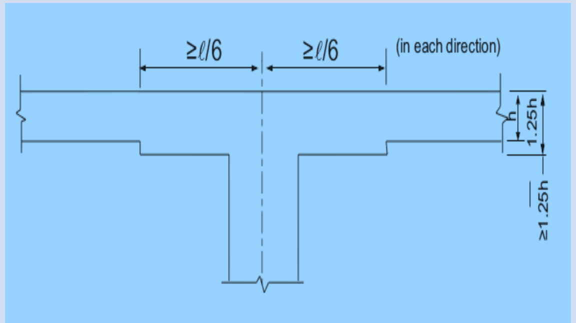
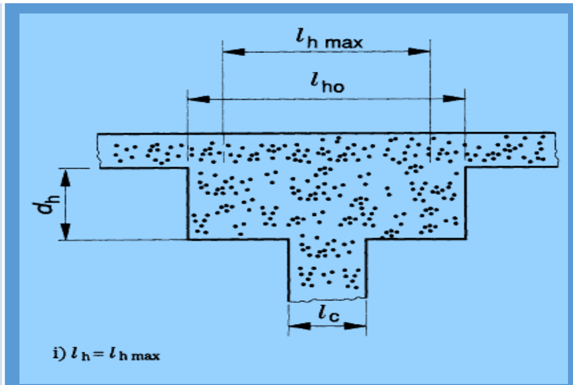
§Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

BSI

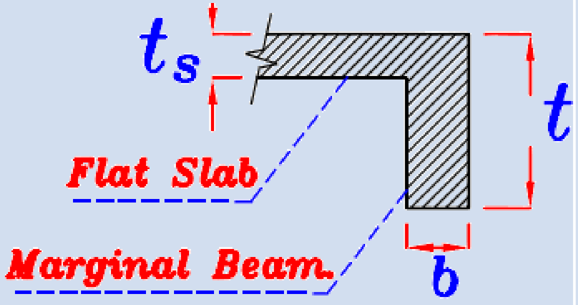
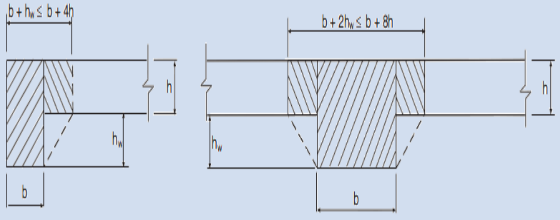
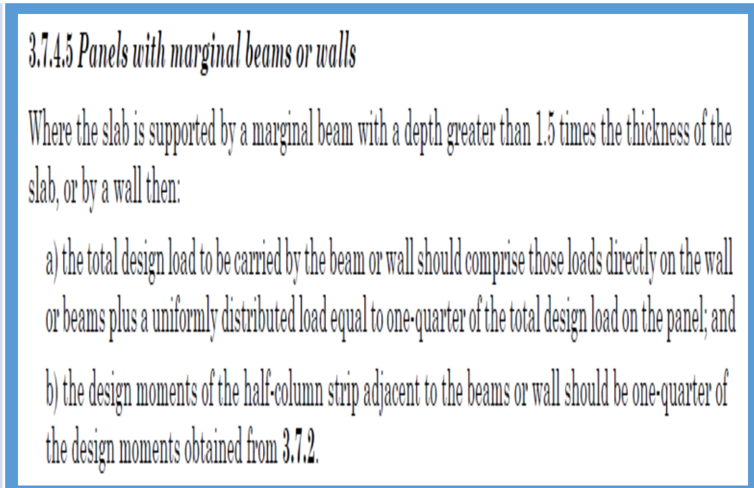
Table 3.9 — Basic span/effective depth ratio for rectangular or flanged beams

Support conditions	Rectangular section	Flanged beams with $\frac{b_w}{b} \leq 0.3$
Cantilever	7	5.6
Simply supported	20	16.0
Continuous	26	20.8

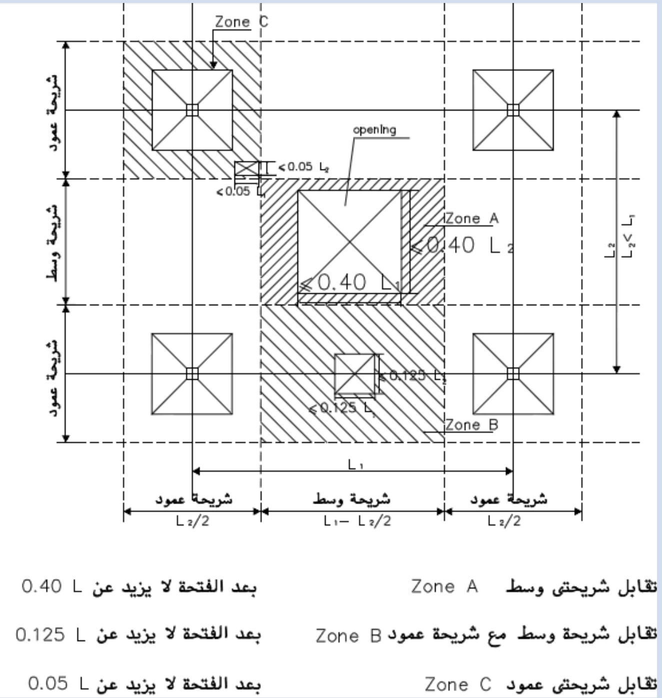
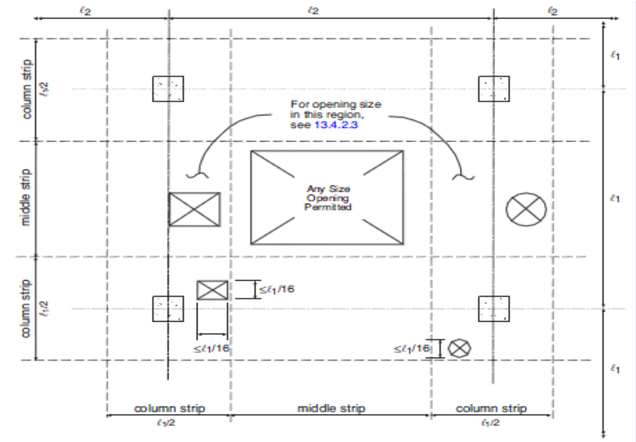
- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr><td>Drop panel</td><td>
$t_{drop\ min.} = 1.25\ t_{slab}$
$\frac{l_{short}}{3} < L_{drop} < \frac{l_{min}}{2}$</td><td>
Figure 18-2 Drop Panel Details (13.2.5)
$t_{drop\ min.} = 1.25\ t_{slab}$
$\frac{l}{3} \le L_{drop}$</td><td>
$\frac{l}{3} \le L_{drop}$</td></tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Marginal beam	 $t_{Beam} \geq 3\ t_{slab}$	 Figure 18-5 Effective Beam Section (13.2.4) <u>Effective beam section :</u> • Edge beam $b + h_w \leq b + 4\ t_{slab}$ • Interior beam $b + 2\ h_w \leq b + 8\ t_{slab}$ $t_{Beam} \geq 2.5\ t_{slab}$	 *3.7.4.5 Panels with marginal beams or walls* Where the slab is supported by a marginal beam with a depth greater than 1.5 times the thickness of the slab, or by a wall then: a) the total design load to be carried by the beam or wall should comprise those loads directly on the wall or beams plus a uniformly distributed load equal to one-quarter of the total design load on the panel; and b) the design moments of the half-column strip adjacent to the beams or wall should be one-quarter of the design moments obtained from 3.7.2. $t_{Beam} \geq 1.5\ t_{slab}$

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Opening in the slab		*Figure 18-8 Permitted Openings in Slab Systems without Beams for $\ell_2 > \ell_1$* 18-7 In case of col. Strip intersecting with col. Strip: $L_{void} \le l_1 / 16$ In case of col. Strip intersecting with middle strip : $L_{void} \le l_1 / 8$ In case of middle Strip intersecting with middle strip :	***1-Holes in areas bounded by column strips:*** that their greatest dimension in a direction parallel to a centre-line of the panel does not exceed 0.4 l . ***2-Holes in areas common to two column strips:*** that in aggregate their length or width does not exceed one-tenth of the width of the column strip ***3-Holes in areas common to a column strip and middle strip:*** that in aggregate their length or width does not exceed one- quarter of the width of the column strip

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Check punching	$q_{pu} \leq q_{cpu}$ ____ safe punching • *depends on concrete* only	$V_u \leq \emptyset\, V_n$ ____ safe punching Where : $V_n = V_c + V_s$ $V_s = \frac{A_v\, d\, f_{yt}}{s}$ • *depends on concrete* And steel	**If $\upsilon \leq 1.6\, \upsilon_c$ use :** $\Sigma A_{sv} \sin\alpha \geq \frac{(\upsilon - \upsilon_c)ud}{0.95 f_{yv}}$ • *depends on concrete* *And steel*

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr><td>Check punching</td><td>

<u>Actual Punching Stress</u>

$$q_{pu} = \frac{\beta\, Q_{pu}}{b_o d}$$

<u>Allowable Punching Stress</u>

Min. of

$$q_{cpu} = 0.316(0.5 + \frac{a}{b})\sqrt{\frac{f_{cu}}{\gamma_c}}$$

$$q_{cpu} = 0.316\left(0.2 + \frac{\alpha d}{bo}\right)\sqrt{\frac{f_{cu}}{\gamma_c}}$$

And not exceed that

$$q_{cpu} = 0.316\sqrt{\frac{f_{cu}}{\gamma_c}}$$

</td><td>

<u>Check for Wide-beam action Shear</u> :

$$Vu \le \emptyset Vn$$

$$Vu \le \emptyset(2\lambda\sqrt{fc'}\, bwd$$

<u>Check for Two way Shear</u>

$$V_u \le \; minimum\ of:$$

1- $\left(2 + \frac{4}{\beta}\right)\lambda\sqrt{fc'}\;\; b_o d$

2- $\left(\frac{\alpha_s\, d}{b_o} + 2\right)\lambda\sqrt{fc'}\;\; b_o d$

3- $4\lambda\sqrt{fc'}\;\; b_o d$

</td><td>

<u>**Actual Punching Stress**</u>

$$\upsilon = \frac{\beta\, V}{u\, d}$$

<u>**Allowable Shear Stress**</u> (υ_c)

refer to table (2.1)

<u>**Max. Shear Stress**</u>

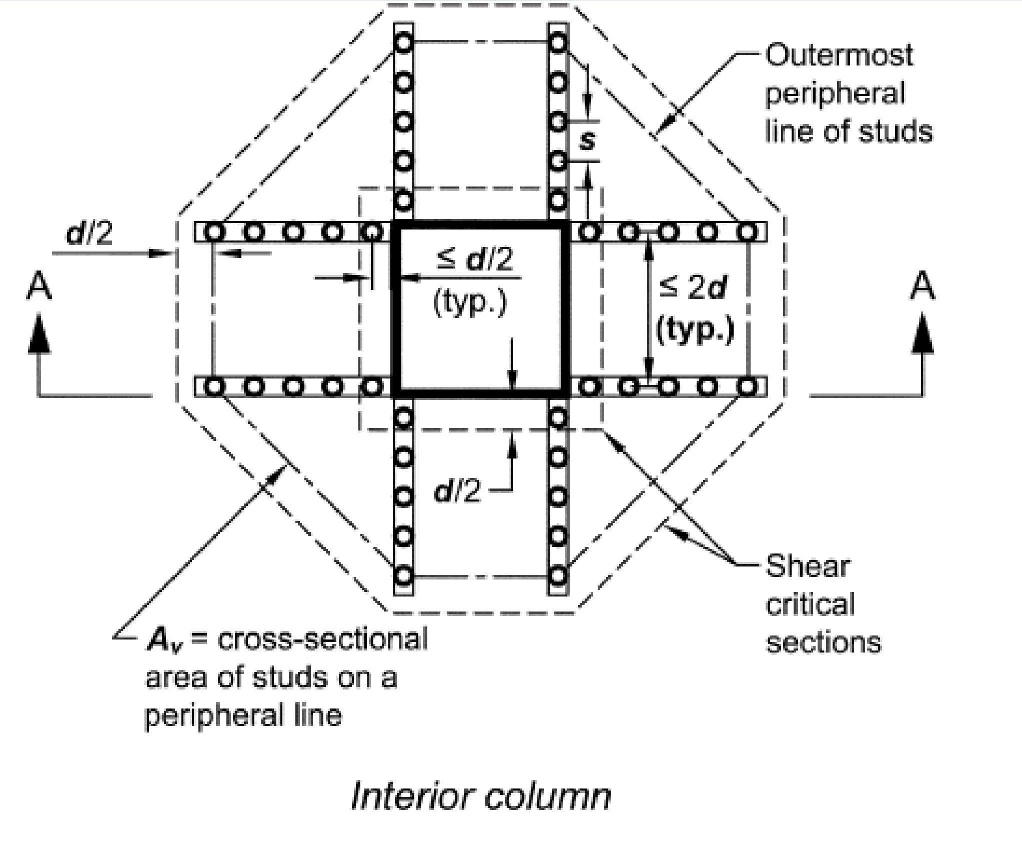
$$\upsilon_{max} = 0.8\sqrt{f_{cu}}$$

</td></tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Check punching	if un safe punching • Increase slab thickness • Use drop panel	$A_v = \frac{(V_u - \emptyset\, V_c)\, S}{\emptyset\, d\, f_{yt}}$ if un safe punching • Increase fc' • increase slab thickness by Using drop panel • Providing shear reinforcement (bars, wires , steel I shape Or headed shear studs)	***If* $\upsilon \le 1.6\, \upsilon_c$ use :** $\Sigma A_{sv} \sin\alpha \ge \frac{(\upsilon - \upsilon_c) u d}{0.95 f_{yv}}$ ***If* $1.6\, \upsilon_c < v \le 2\, \upsilon_c$ use :** $\Sigma A_{sv} \sin\alpha \ge \frac{5(0.7\upsilon - \upsilon_c) u d}{0.95 f_{yv}}$ ***If* $v > 2\, \upsilon_c$ use :** • Increase fc' • increase slab thickness by Using drop panel • Providing shear reinforcement

- COMPARISON BETWEEN ECP & ACI & BSI



headed shear studs

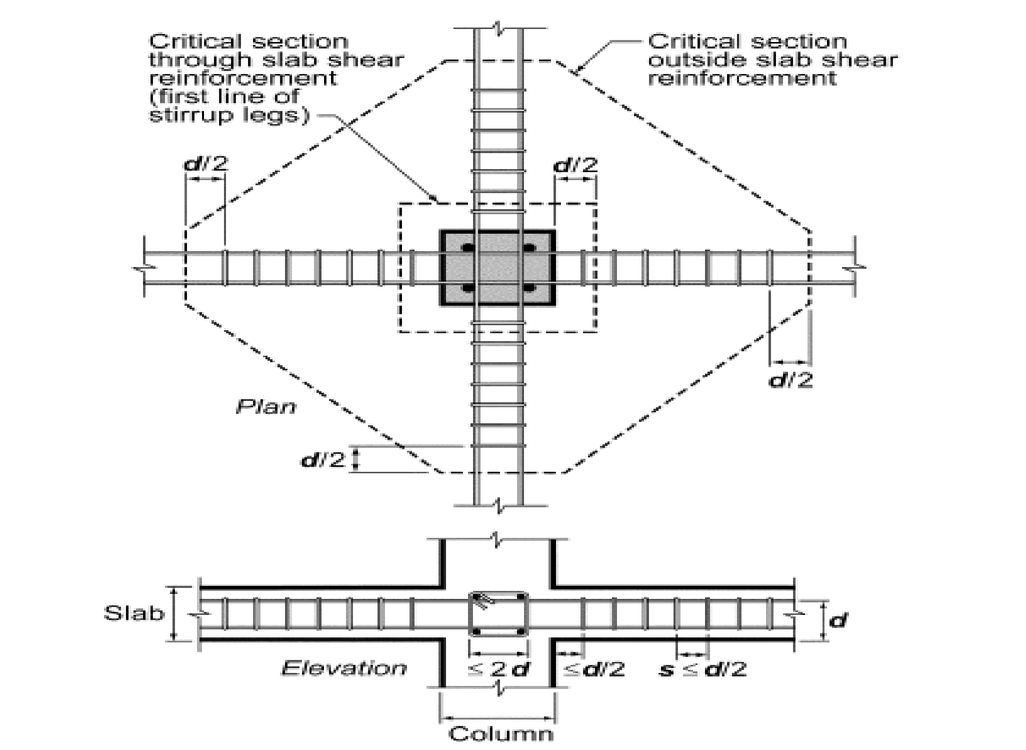


Fig. R11.11.3(d)—Arrangement of stirrup shear reinforcement, interior column.

shear reinforcement stirrups

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Check punching	Where : Q_{pu} : Punching Force b_o : Length of the Perimeter of Critical Sec d : Effective depth of slab β: 1.15 for Interior columns 1.30 for Edge columns 1.50 for Corner columns	Where : V : Punching Force β: ratio of (long side/ short side) of the column concentrated load or reaction area α_s: 40 for interior columns 30 for edge columns 20 for corner columns b_o : perimeter of critical section	**Where :** υc : design concrete shear stress V :design ultimate value of the concentrated load. uo : effective length of the perimeter which touches a loaded area. υ : design shear stress. u : effective length of the outer perimeter of the zone. β: 1.15 for Interior columns 1.25 for Edge columns 1.40 for Corner columns ΣAsv :is the area of shear reinforcement f_{yv} :is the characteristic strength of shear reinforcement (in N/mm^2) α : is the angle between the shear reinforcement and the plane of the slab.

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Critical sec.	- take critical sec. at distance (d/2)	<u>for Wide-beam action Shear</u> - Take critical sec. at distance (d) from support. <u>for Two way Shear</u> - Take critical sec. at distance (d/2) from support.	**<u>1st take critical sec. at distance (1.5d)</u>** **If $\upsilon \leq \upsilon_c$ use then no further checks are needed.** **<u>If not</u>** **- take the critical sec. at distance d/2 and get As for shear**

- COMPARISON BETWEEN ECP & ACI & BSI

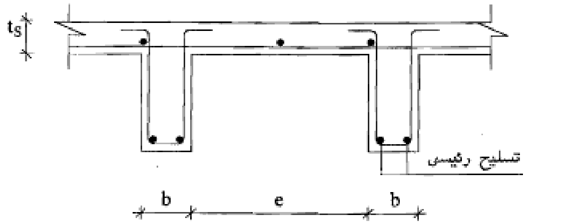
Table 3.8 — Values of v_c design concrete shear stress

$\frac{100A_s}{b_v d}$	Effective depth mm							
	125	150	175	200	225	250	300	$\geq$ 400
	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
$\leq$ 0.15	0.45	0.43	0.41	0.40	0.39	0.38	0.36	0.34
0.25	0.53	0.51	0.49	0.47	0.46	0.45	0.43	0.40
0.50	0.67	0.64	0.62	0.60	0.58	0.56	0.54	0.50
0.75	0.77	0.73	0.71	0.68	0.66	0.65	0.62	0.57
1.00	0.84	0.81	0.78	0.75	0.73	0.71	0.68	0.63
1.50	0.97	0.92	0.89	0.86	0.83	0.81	0.78	0.72
2.00	1.06	1.02	0.98	0.95	0.92	0.89	0.86	0.80
$\geq$ 3.00	1.22	1.16	1.12	1.08	1.05	1.02	0.98	0.91

Table (2.1)

Hollow block design

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
PANS FORM	٦-٢-٢ البلاطات ذات الأعصاب والقوالب المفرغة **Hollow Block Slabs** ٦-٢-٢-١ عام – عند حساب البلاطات ذات القوالب المفرغة لا تعتبر هذه القوالب فعالة استاتيكياً. – يجب استيفاء الاشتراطات التالية الخاصة بالأبعاد (شكل ٦-٤): ١ – لا تزيد المسافة الخالصة بين الأعصاب e على ٧٠٠ مم. ٢ – لا يقل عرض الأعصاب b عن ١٠٠ مم أو ثلث العمق t أيهما اكبر. ٣ – لا يقل سمك بلاطة الضغط t_s عن ٥٠ مم أو عُشر المسافة e أيهما اكبر. – يجب أن تتحمل البلاطة بين الأعصاب بأمان الأحمال المركزة التي قد تؤثر مباشرة عليها .  e بحد أقصى ٧٠٠ مم b ١٠٠ مم أو t/3 أيهما أكبر t_s ٥٠ مم أو e/10 أيهما أكبر شكل(٦-٤) قطاع وأبعاد البلاطات ذات الأعصاب و القوالب المفرغة	-PAN WIDE **<u>(500-750)mm</u>**	**-In situ ribs should be spaced at centers not exceeding <u>1.5 m</u> & should not exceed <u>four times their width</u>**

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>RIBS</td>
<td>

٦-٢-٢ البلاطات ذات الأعصاب والقوالب المفرغة

Hollow Block Slabs

٦-٢-٢-١ علم

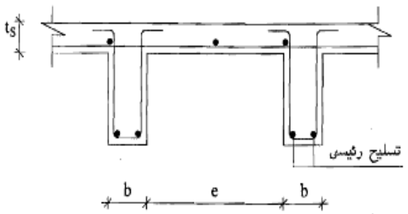
- عند حساب البلاطات ذات القوالب المفرغة لا تعتبر هذه القوالب فعالة استاتيكياً.
- يجب استيفاء الاشتراطات التالية الخاصة بالأبعاد (شكل ٦-٤):

١ – لا تزيد المسافة الخالصة بين الأعصاب e على ٧٠٠ مم.

٢ – لا يقل عرض الأعصاب b عن ١٠٠ مم أو ثلث العمق t أيهما اكبر.

٣ – لا يقل سمك بلاطة الضغط t_s عن ٥٠ مم أو عُشر المسافة e أيهما اكبر.

- يجب أن تتحمل البلاطة بين الأعصاب بأمان الأحمال المركزة التي قد تؤثر مباشرة عليها .



e بحد أقصى ٧٠٠ مم

b ١٠٠ مم أو t/3 أيهما أكبر

t_s ٥٠ مم أو e/10 أيهما أكبر

شكل(٦-٤) قطاع وأبعاد البلاطات ذات الأعصاب و القوالب المفرغة

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<td>

TABLE 9.5(a)—MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED

	Minimum thickness, h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Standard depth :
6,8,10,12,14,16&20
We can use tapered rib with dimension

</td>
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Table 3.9 – Basic span/effective depth ratio for rectangular or flanged beams

Support conditions	Rectangular section	Flanged beams with $\frac{b_w}{b} \le 0.3$
Cantilever	7	5.6
Simply supported	20	16.0
Continuous	26	20.8

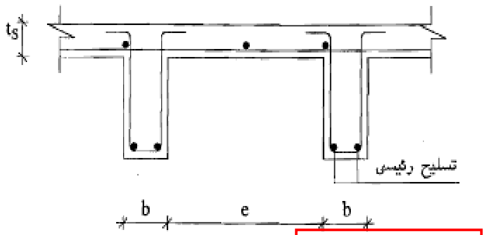
The minimum width of rib will be determined by considerations of cover , bar spacing and fire.

</td>
</tr>
</table>

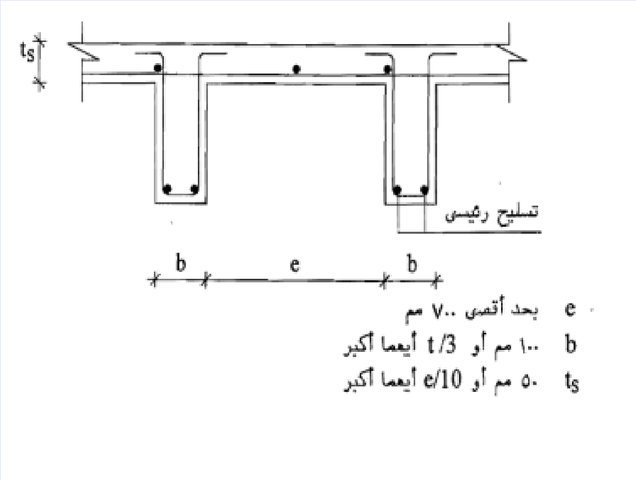
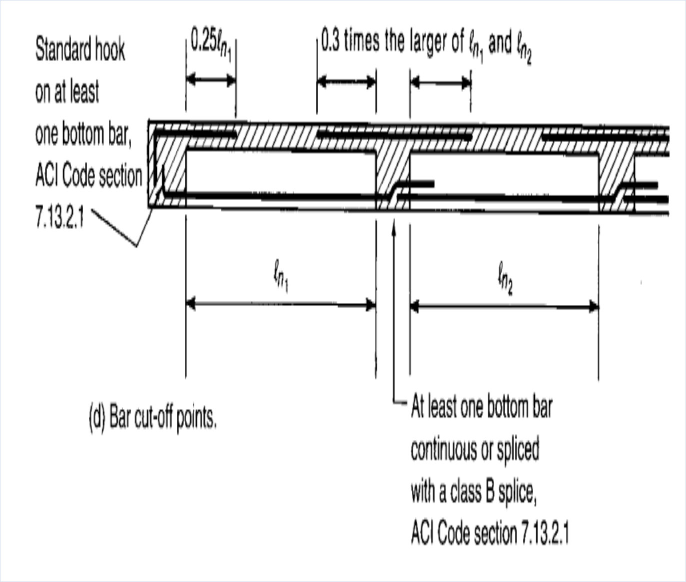
- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Cross rib	- إذا كان الحمل الحي أصغر من أو يساوي ٣ كيلونيوتن/م٢ وكانت البحور أطول من ٥,٠ م ، يجب أن تزود البلاطة بعصب عرضي واحد على الأقل عند منتصف البحر. ويجب ألا يقل القطاع والتسليح السفلي لهذا العصب العرضي عنه في الأعصاب الرئيسية ، ويكون تسليحه العلوي لنصف تسليحه السفلي على الأقل. - وإذا زاد الحمل الحي على ٣ كيلونيوتن/م٢ وكانت البحور تتراوح بين ٤,٠ م ، ٧,٠ م تزود البلاطة بعصب عرضي واحد ، أما إذا زادت البحور على ٧,٠ م تزود البلاطة بثلاثة أعصاب عرضية وتكون هذه الأعصاب العرضية بنفس الأبعاد والتسليح المذكورة فيما سبق.	One at mid for span (6-9)m & 2 ribs at third points for spans over (9) m.	

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Cover Slab	٦-٢-٢ البلاطات ذات الأعصاب والقوالب المفرغة **Hollow Block Slabs** ٦-٢-٢-١ علم – عند حساب البلاطات ذات القوالب المفرغة لا تعتبر هذه القوالب فعالة استاتيكياً. – يجب استيفاء الاشتراطات التالية الخاصة بالأبعاد (شكل ٦-٤): ١ – لا تزيد المسافة الخالصة بين الأعصاب e على ٧٠٠ مم. ٢ – لا يقل عرض الأعصاب b عن ١٠٠ مم أو ثلث العمق t أيهما اكبر. ٣ – لا يقل سمك بلاطة الضغط t_s عن ٥٠ مم أو عُشر المسافة e أيهما اكبر. – يجب أن تتحمل البلاطة بين الأعصاب بأمان الأحمال المركزة التي قد تؤثر مباشرة عليها .  e بحد أقصى ٧٠٠ مم b ١٠٠ مم أو t/3 أيهما أكبر t_s ٥٠ مم أو e/10 أيهما أكبر **شكل(٦-٤) قطاع وأبعاد البلاطات ذات الأعصاب و القوالب المفرغة**	Slab thickness over permanent fillers shall be not less than one-twelfth the clear distance between ribs ,nor less than 40 mm .	**-not** less than 20mm or **1/10 clear distance between ribs**

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Reinforcement	- لا تقل مساحة مقطع أسياخ التوزيع العمودية على الأعصاب في المتر عن القيم المعطاة في البند (٦-٣-١-١٠) ، وتكون أقل كمية لأسياخ التوزيع في البلاطة (موازية للأعصاب) هي ٣ ϕ ٦ مم/ متر، على أن يوضع سيخ قطر ٦ مم بين كل عصبين وسيخ عند كل عصب كما هو موضح بشكل (٦-٤).  شكل(٦-٤) قطاع وأبعاد البلاطات ذات الأعصاب و القوالب المفرغة	 	**Cover slab Rein.** **Consideration should be given to providing a single layer of welded steel fabric, having a cross-sectional area of not less than 0.12 % of the topping, in each direction; the spacing between wires should not be greater than half the center-to-center distance between ribs.**

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>Fire resisting</td>
<td>

جدول (٢-١٤-أ) القيم الدنيا للأبعاد بالمليمتر لمقاومة الخرسانة للحريق

<table>
<tr><th colspan="3">مدة الحريق (ساعة)</th><th>٠,٥</th><th>١,٠</th><th>١,٥</th><th>٢,٠</th><th>٣,٠</th><th>٤,٠</th></tr>
<tr><td rowspan="8">الحـد الأدنى للأبعاد الخرسانية (مم)</td><td colspan="2">البعد الأصغر للعمود</td><td>٢٠٠</td><td>٢٠٠</td><td>٢٥٠</td><td>٣٠٠</td><td>٤٠٠</td><td>٤٥٠</td></tr>
<tr><td colspan="2">عرض الكمرة بسيطة الارتكاز</td><td>١٢٠</td><td>١٢٠</td><td>١٥٠</td><td>٢٠٠</td><td>٢٤٠</td><td>٢٨٠</td></tr>
<tr><td colspan="2">عرض الكمرة المستمرة</td><td>١٢٠</td><td>١٢٠</td><td>١٢٠</td><td>١٥٠</td><td>٢٠٠</td><td>٢٤٠</td></tr>
<tr><td colspan="2">سمك البلاطة (بسيطة أو مستمرة)</td><td>٨٠</td><td>١٠٠</td><td>١١٠</td><td>١٣٠</td><td>١٥٠</td><td>١٧٠</td></tr>
<tr><td rowspan="3">سمك الحائط الخرسانى</td><td>μ < % ٠,٤*</td><td>١٥٠</td><td>١٥٠</td><td>١٨٠</td><td>---</td><td>---</td><td>---</td></tr>
<tr><td>%٠,٤<μ<%١</td><td>١٢٠</td><td>١٢٠</td><td>١٤٠</td><td>١٦٠</td><td>٢٠٠</td><td>٢٤٠</td></tr>
<tr><td>μ> %١</td><td>١٢٠</td><td>١٢٠</td><td>١٢٠</td><td>١٢٠</td><td>١٥٠</td><td>١٨٠</td></tr>
<tr><td rowspan="7">سمك الغطاء الخرسانى خارج الكانات (مم)</td><td colspan="2">غطاء تسليح العمود</td><td>**٢٠</td><td>٢٠</td><td>٢٠</td><td>٢٥</td><td>٢٥</td><td>٢٥</td></tr>
<tr><td colspan="2">غطاء تسليح الكمرة البسيطة</td><td>**٢٠</td><td>٢٠</td><td>٣٠</td><td>٤٥</td><td>٦٠</td><td>٧٠</td></tr>
<tr><td colspan="2">غطاء تسليح الكمرة المستمرة</td><td>**٢٠</td><td>**٢٠</td><td>٢٥</td><td>٤٠</td><td>٥٠</td><td>٦٠</td></tr>
<tr><td colspan="2">غطاء تسليح البلاطة البسيطة</td><td>١٥</td><td>٢٠</td><td>٢٥</td><td>٣٥</td><td>٤٥</td><td>٥٥</td></tr>
<tr><td colspan="2">غطاء تسليح البلاطة المستمرة</td><td>١٥</td><td>٢٠</td><td>٢٠</td><td>٢٥</td><td>٣٥</td><td>٤٥</td></tr>
<tr><td rowspan="2">غطاء تسليح الحائط</td><td>%٠,٤ < μ < %١</td><td>٢٥</td><td>٢٥</td><td>٢٥</td><td>٢٥</td><td>٢٥</td><td>٢٥</td></tr>
<tr><td>μ > %١</td><td>١٥</td><td>١٥</td><td>٢٥</td><td>٢٥</td><td>٢٥</td><td>٢٥</td></tr>
</table>

* μ هى النسبة المئوية للتسليح الطولى فى الحائط.

** يمكن تقليل سمك الغطاء الخرسانى إلى ١٥ مم إذا كان المقاس الاعتبارى الأكبر للركام الكبير المستخدم لا يتجاوز ١٥ مم.

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<u>1 hour:</u>
(20)mm for slab
<u>2 hour:</u>
(25)mm for slab

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<table>
<tr><th rowspan="2">Fire resistance</th><th rowspan="2">Minimum beam width (b)</th><th rowspan="2">Rib width (b)</th><th rowspan="2">Minimum thickness of floors (h)</th><th colspan="3">Column width (b)</th><th colspan="3">Minimum wall thickness</th></tr>
<tr><th>Fully exposed</th><th>50 % exposed</th><th>One face exposed</th><th>p < 0.4 %</th><th>0.4 % < p < 1 %</th><th>p > 1 %</th></tr>
<tr><td>h</td><td>mm</td><td>mm</td><td>mm</td><td>mm</td><td>mm</td><td>mm</td><td>mm</td><td>mm</td><td>mm</td></tr>
<tr><td>0.5</td><td>200</td><td>125</td><td>75</td><td>150</td><td>125</td><td>100</td><td>150</td><td>100</td><td>75</td></tr>
<tr><td>1</td><td>200</td><td>125</td><td>95</td><td>200</td><td>160</td><td>120</td><td>150</td><td>120</td><td>75</td></tr>
<tr><td>1.5</td><td>200</td><td>125</td><td>110</td><td>250</td><td>200</td><td>140</td><td>175</td><td>140</td><td>100</td></tr>
<tr><td>2</td><td>200</td><td>125</td><td>125</td><td>300</td><td>200</td><td>160</td><td>—</td><td>160</td><td>100</td></tr>
<tr><td>3</td><td>240</td><td>150</td><td>150</td><td>400</td><td>300</td><td>200</td><td>—</td><td>200</td><td>150</td></tr>
<tr><td>4</td><td>280</td><td>175</td><td>170</td><td>450</td><td>350</td><td>240</td><td>—</td><td>240</td><td>180</td></tr>
<tr><td colspan="10">NOTE 1 These minimum dimensions relate specifically to the covers given in Table 3.4 and Table 4.9.
NOTE 2 p is the area of steel relative to that of concrete.</td></tr>
</table>

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</tr>
</table>

- COMPARISON BETWEEN ECP & ACI & BSI

<table>
<tr><th>Point of comparison</th><th>ECP</th><th>ACI</th><th>BSI</th></tr>
<tr>
<td>Cover</td>
<td>
<table>
<tr><th>Fire Resistance</th><th colspan="2">Floors unite (mm)</th></tr>
<tr><td>Unite (hour)</td><td>Simply Supported</td><td>Continouse</td></tr>
<tr><td>0.5</td><td>15</td><td>15</td></tr>
<tr><td>1</td><td>20</td><td>20</td></tr>
<tr><td>1.5</td><td>25</td><td>20</td></tr>
<tr><td>2</td><td>35</td><td>25</td></tr>
<tr><td>3</td><td>45</td><td>35</td></tr>
<tr><td>4</td><td>55</td><td>45</td></tr>
</table>
</td>
<td>Shells, folded plate members:
No. 19 bar and larger 20
No. 16 bar, MW200 or MD200 wire, and smaller…………… 13</td>
<td>
<table>
<tr><th>Fire Resistance</th><th colspan="2">Ribs unite (mm)</th></tr>
<tr><td>Unite (hour)</td><td>Simply Supported</td><td>Continouse</td></tr>
<tr><td>0.5</td><td>20</td><td>20</td></tr>
<tr><td>1</td><td>20</td><td>20</td></tr>
<tr><td>1.5</td><td>35</td><td>20</td></tr>
<tr><td>2</td><td>45</td><td>35</td></tr>
<tr><td>3</td><td>55</td><td>45</td></tr>
<tr><td>4</td><td>65</td><td>55</td></tr>
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Footing Design

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Thickness	$\nless$ 300 mm	Assume Overall Footing Thickness=33 in Check Shear	**Assume Thickness and Check Shear**
Min Cover	mm50 المغمورة والمعرضة للهواء mm70 المعرضة للبلل والجفاف	75mm	**70mm**

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Check punching Check shear	Check shear Actual Shear Stress $q_u = \frac{Qu}{bd}$ Allowable Shear Stress $q_{su} = 0.15\sqrt{\frac{Fcu}{\gamma c}}$ Check punching Actual Punching Stress $q_{pu} = \frac{\beta\, Q_{pu}}{b_o d}$ Allowable Punching Stress As slabs	Check for Wide-beam action Shear : $Vu \leq \emptyset Vn$ $Vu \leq \emptyset(2\lambda\sqrt{fc'}\, bwd$ Check for Two way Shear $V_u \leq \ minimum\ of:$ 1- $\left(2+\frac{4}{\beta}\right)\lambda\sqrt{fc'}\ \ b_o d$ 2- $\left(\frac{\alpha_s\, d}{b_o}+2\right)\lambda\sqrt{fc'}\ \ b_o d$ 3- $4\lambda\sqrt{fc'}\ \ b_o d$	**Actual Punching Stress** $\upsilon = \frac{\beta\, V}{u\, d}$ **Allowable Shear Stress (υ_c)** refer to table (2.1) **Max. Shear Stress** $\upsilon_{max} = 0.8\sqrt{f_{cu}}$

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Check punching	if un safe punching • Increase footing thickness • Use drop panel	$A_v = \frac{(V_u - \emptyset V_c)\, S}{\emptyset\, d\, f_{yt}}$ if un safe punching • Increase fc' • increase footing thickness or Using drop panel • Providing shear reinforcement stirrups	**If $\upsilon \leq 1.6\ \upsilon_c$ use :** $\Sigma A_{sv} \sin\alpha \geq \frac{(v - v_c)ud}{0.95 f_{yv}}$ **If $1.6\ \upsilon_c < v \leq 2\ \upsilon_c$ use :** $\Sigma A_{sv} \sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95 f_{yv}}$ ***If $v > 2\ \upsilon_c$* use :** • Increase fc' • increase footing thickness or Using drop panel • Providing shear reinforcement

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Check punching	Where : Q_{pu} : Punching Force b_o : Length of the Perimeter of Critical Sec d : Effective depth of slab β: 1.15 for Interior columns 1.30 for Edge columns 1.50 for Corner columns	Where : V : Punching Force β: ratio of (long side/ short side) of the column concentrated load or reaction area α_s: 40 for interior columns 30 for edge columns 20 for corner columns b_o : perimeter of critical section	**Where :** υc : design concrete shear stress V :design ultimate value of the concentrated load. uo : effective length of the perimeter which touches a loaded area. υ : design shear stress. u : effective length of the outer perimeter of the zone. β: 1.15 for Interior columns 1.25 for Edge columns 1.40 for Corner columns ΣAsv :is the area of shear reinforcement f_{yv} :is the characteristic strength of shear reinforcement (in N/mm^2) α : is the angle between the shear reinforcement and the plane of the slab.

- COMPARISON BETWEEN ECP & ACI & BSI

Point of comparison	ECP	ACI	BSI
Check punching	$q_{pu} \le q_{cpu}$ ___ safe punching • *depends on concrete* only	$V_u \le Ø V_n$ ___ safe punching Where : $V_n = V_c + V_s$ $V_s = \frac{A_v\ d\ f_{yt}}{s}$ • *depends on concrete* And steel	**If $\upsilon \le 1.6\ \upsilon_c$ use :** $\Sigma A_{sv} \sin\alpha \ge \frac{(\upsilon - \upsilon_c)ud}{0.95 f_{yv}}$ • *depends on concrete* *And steel*

THANK YOU