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دفتر

خرسانة مسلحة1

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إعداد : لانا عدنان

CH 1 : Introduction. DATE. 5/2

8/3 → 10-11

4/4 → 9.30-10.30

*Advantages: (For Concrete).

1] Relativaly low cost material.

2] Fire resestance (1-3hrs) fire rating without special fire proofing.

3] Suitability of material for artchitectural & structural functions.

4] Rigidity.

5] low maintance.

6] Avaibility of materials.

*Disadvantages:

1] low ~~xx~~ tensile strength of conc.

2] Forms & shoring.

3] Relativaly low strength per unit weight or volume.

4] Time - Dependent volume charges:

* Drying Shrinkage (Shrinkage). * Creep (Strains under sustaind loads).

*Importance of steel:

1] Steel has a good bond with concrete.

2] Concrete & steel have nearly equal coefficents of thermal expansion.

3] Good dense concrete protects steel from corrosion/rusting.

*Sourses of uncertainty:

1] Actual load magnitude & distribution may differ from those assumed in the design.

2] Assumptions & simplifications in the analysis may result in different interval forces.

3] Actual behavior may be different.

4] Actual member dimenstions may differ from those specified in the design.

5] Reinforcement may not be in it's proper position.

6] Actual materials strength may be different from that specified in the design.

* Safty Philosophy :

Strength reduction factor < 1 → $\underbrace{\varnothing S_n}_{\text{nominal strength}} \geq \underbrace{\gamma Q_d}_{\text{Design load}}$ → Over load factor ≥ 1

(.65 - .9)

$\varnothing S_n$ → Reduce strength.

γQ_d → Ultimate load (factored load).

* Load factors & load combinations from ACI-code :

$U = 1.2\,DL + 1.6\,LL$

→ principale variable load. → 1 < دايما

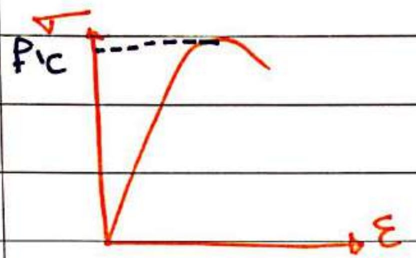
$U = .9\,DL + 1.6\,WL$

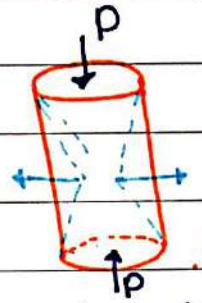
→ Companion action variable load. 1 > دايما

CH: Materials

DATE. 7/2/2018

* Mechanisim of failure in conc. loaded in compression.
* Stage of microcracking & failure in conc. subjected to uniaxial Compression:





دائماً الكسر بصير بسبب tension ،
يعني تأثير comp. بس بالفعل بيصير tension .

[1] Shrinkage of the paste during hydration (No-load bond cracks)
حواليه ال agg.

[2] At stresses up to 30 to 40% of the comp. strength → Bond cracks.

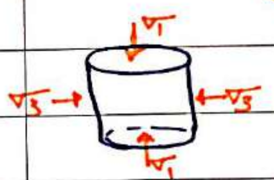
discontinuity limit → [3] At stresses up to 50 to 60% of the comp. strength → mortar cracks.
بتنشر بين ال agg particles.

Stable propagation | [4] At stresses up to 75% of the ultimate load.

critical stress → ↳ Number of mortar cracks increases.

unstable cracks propagation. → ↳ Fewer undamaged portions to carry the load.

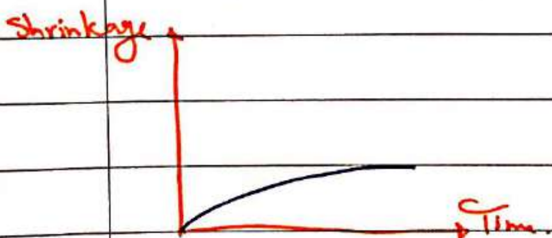
* Triaxial loading:



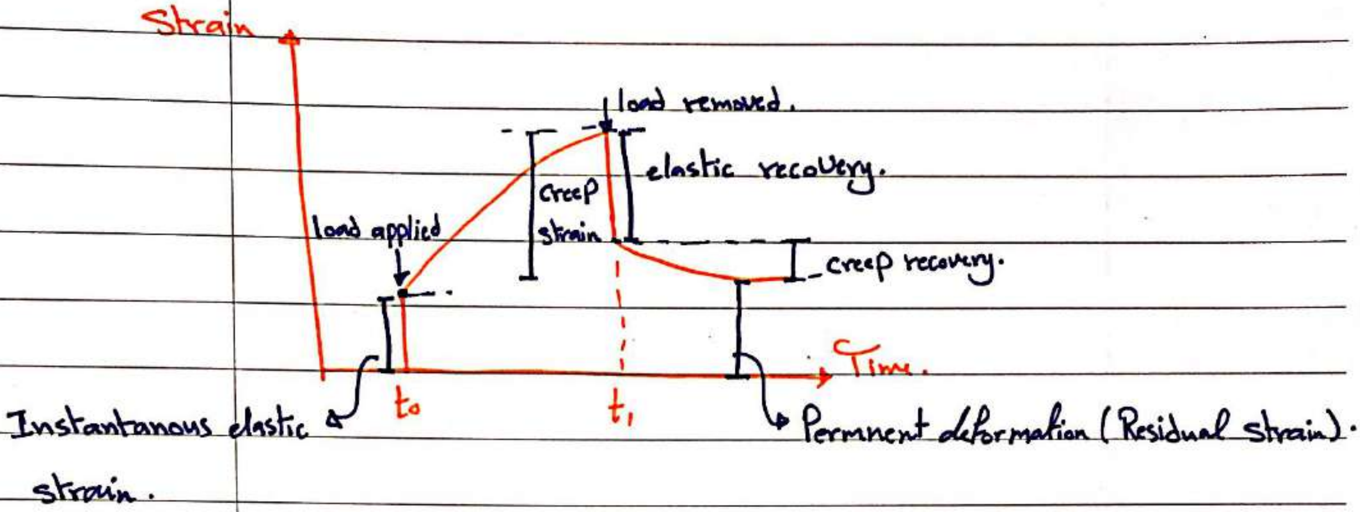
(Confining stress / pressure).

* Time-Dependent Volume Change:

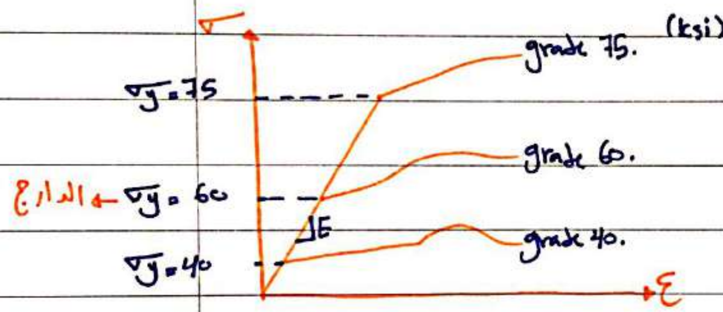
1. Drying Shrinkage (shrinkage).



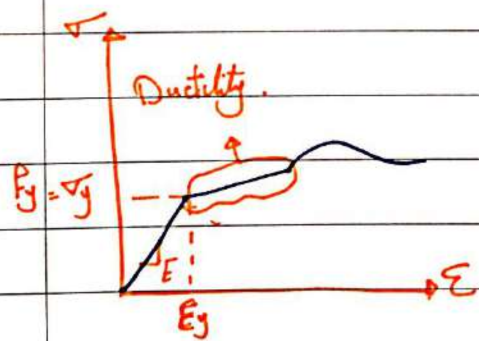
2- Creep: deformations / Strains under sustained load.



* Reinforcement:



E للكل ثابتة .



$\sigma = E\varepsilon$; $E \rightarrow 200\ GPa$.

CH 3:

* The Design Process: (objective of the design): (The structure should satisfy)

1] Appropriateness: Designed to serve it's intended use.

2] Economy: Over all cost doesn't exceed client's budget.

3] Structural Adequacy: structure must → Be strong enough to support all anticipated load.

Not deflect, tilt, vibrate or crack in away that affects it's usefullness.

4] Maintainability: minimum & simple maintance.

* The Design Process:

Phase I: Defining Client's needs & priorities.

Phase II: Development of project concepts

↳ Number of possible layouts.

↳ Preliminary cost estimates.

Phase III: Design of individual systems.

* Limit states & The Design of R.C:

When a structure or an element become unfit for it's intended use it is said to have reached a limit state.

* Groups of limit state:

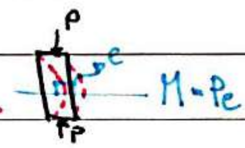
1] Ultimate limit state: Involve structural collapse of part or all the structure:

↳ loss of equilibrium.

↳ Rupture of critical parts of the structure.

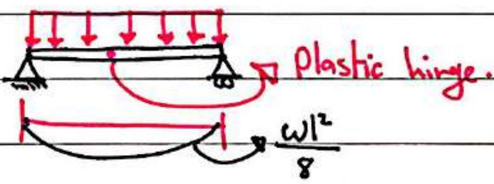
↳ Progressive Collapse.

↳ Instability due to deformations (Buckling).



↳ Fatigue.

↳ Formation of plastic mechanisim.



2] Serviceability limit state: Involves distruption of the functional use of the structure (but not collapse).

↳ Excessive deflections.

↳ Undesirable vibrations.

↳ Excessive crack widths.

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3] Special limit state. Involves damages or failure due to up normal Conditions.

↳ Extreme earthquacks.

↳ Fire → explosions.

↳ Vehicular collisions.

*Limit State Design Process:

- ~~Id~~ Identification of all potential mokes of failure.
- Determination of acceptable level of safty per ACI-code.

*Structural Safty:

- Sourses of uncertainty.
- Consequances of failure.

↳ Loss of life

↳ Cost of clearing debris.

↳ Cost of society in lost time.

*Design Procedures Specified in the ACI-code:

1] Strength design: $\phi S_n \geq \gamma Q_d$.

2] Working-stress-design: $\phi S_n \geq Q_d$ → service load / working load.

3] Plastic-Cabacity-limit Design: conceders redistributions of moments as successive cross sections ~~get~~ yield.

*Loading & Actions:

↳ Permenent loads.

↳ Accidental loads.

↳ Variable loads → Sustained loads (long-duration).

↳ Short-duration.

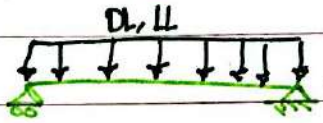
Flexure: Basic Concept: (Rectangular Beams).

$\phi S_n \geq \gamma Q_d$.

strength reduction (.65-.9) in analysis ϕ — $\phi M_n \geq M_u$ (Basic safty equ for flexure).

M_u ↳ Ultimate moment

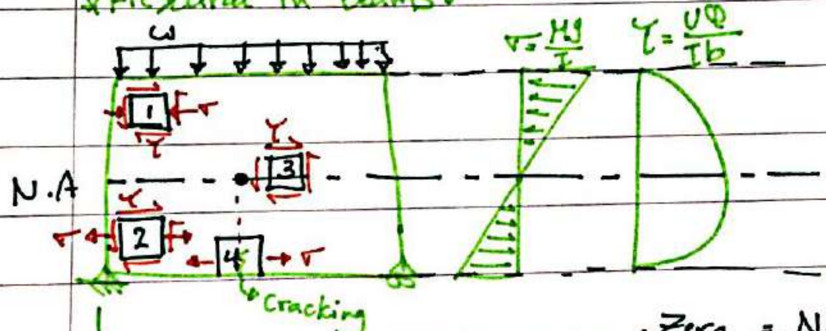
M_n ↳ nominal moment capacity.



$W_u = U = 1.2DL + 1.6LL$

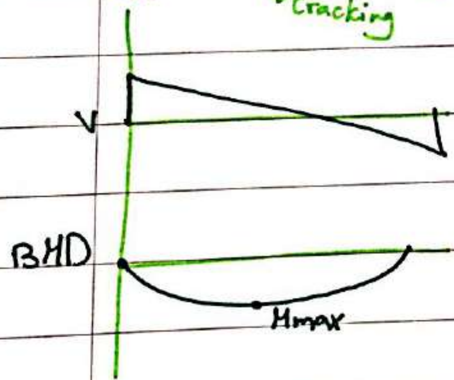
$M_{max} = \frac{wl^2}{8} = M_u$.

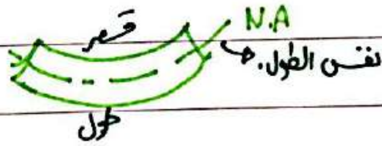
* Flexural in beams:



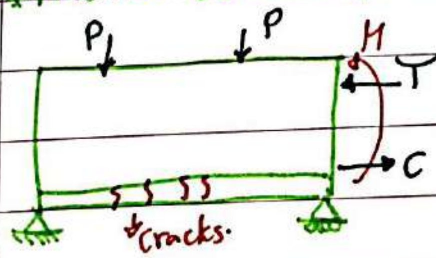
. Zero = N.A & Bending stress دالة الـ *

. Maximum = " " Shear stress = "





* Flexural behavior (Laboratory Testing).



* Stage A: Before Cracking.

* Stage B: Cracking.

* Stage C: After cracking before yielding of reinforcement.

Transfer of tensile force ~~from~~ from Conc. to steel.

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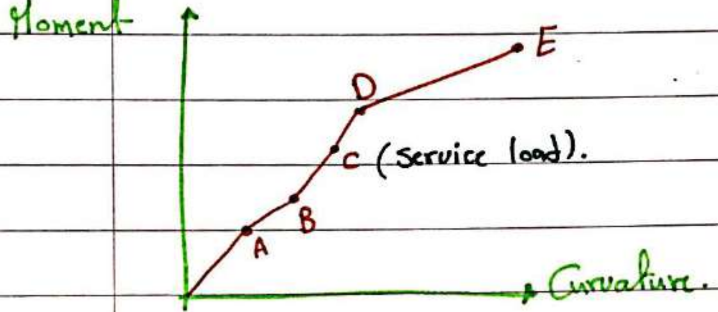
* Stage D: yielding of reinforcement.

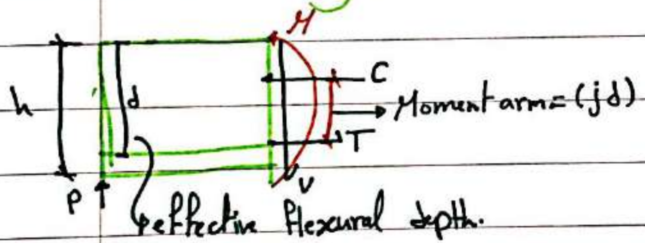
Curvature increases rapidly with very little increases in the moment.

* Stage E: Failure.

Beam failed as a result of crushing of the conc. on the top of the beam.



* Flexure Theory:

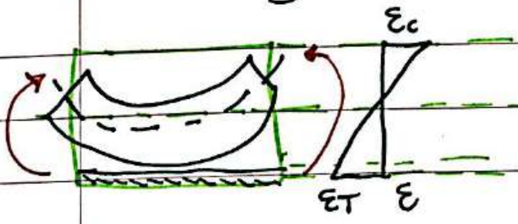


$C_c = T$ (couple forces).

$M_n = \frac{C_c}{T} (jd)$

* Basic Assumptions in Flexural Theory:

1] Sections perpendicular to the axis of bending that are plane before bending remain plane after bending.



2] The strain in the reinforcement is equal to the strain in the conc. at the same level (Perfect bond).

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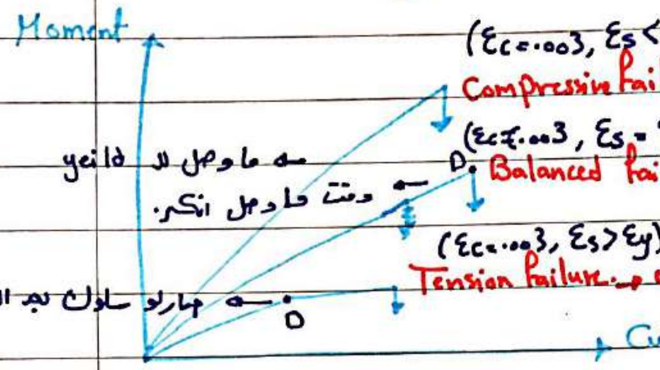
3] The tensile strength of concrete is neglected in flexural - strength calculations.

4] Concrete is assumed to fail when the maximum compressive ~~strength~~ strain reaches alimiling value
$\varepsilon_c = \varepsilon_{cu} = .003$.

(9)

* Flexure failure may accure in three diffirent ways:



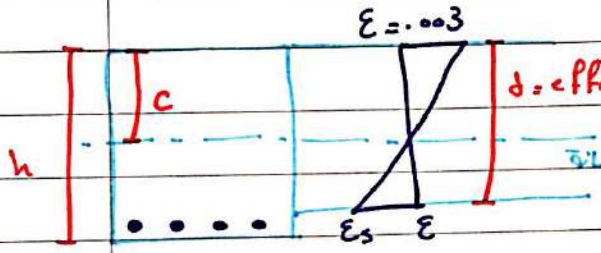
$(\varepsilon_c = .003,\ \varepsilon_s < \varepsilon_y)$
Compressive failure. → over reinforcement beam — Brittle مرفوضة بالنسبة لل ACI.

$(\varepsilon_c = .003,\ \varepsilon_s = \varepsilon_y)$
Balanced failure → balanced beam.

yeild ما وصل لل ←
وقت ما وصل اتكسر. ←

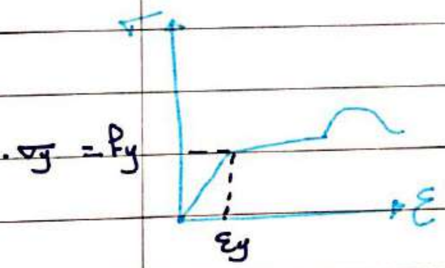
$(\varepsilon_c = .003,\ \varepsilon_s > \varepsilon_y)$
Tension failure. → under reinforce beam. → لل هاداللي بدنا

yeild. وصل لل بعد السلوك ما تكسر ←
ايا Ductile failure مقبول.



d = effective flexural depth.

* وقت حدوث ال failure ال comp. strength تكون بالـ ultimate يكون وصل ال
بدو الي بدنا الـ strain ال conc. بال comp. (ε_s)
ε_s → tensile in steel.



$\sigma = E\varepsilon$

$F_y = E\varepsilon_y\ ,\ \varepsilon_y = \dfrac{F_y}{E}$ ← جدول الطرح

grade 60 ← $F_y \cong 420$ MPa. , $\varepsilon_y = .0021$

$\varepsilon_s = .0022$ → مقبول (Tensile failure).

$\varepsilon_s = .0019$ → مرفوض (Compressive failure) → وهو الأول.

* Strain limits methodes for Analysis & Design:

– Per ACI-code → 4 types of failure.

مرفوض | مرفوض | مقبول | DATE.

Compression controlled failure.	Transition-controlled failure.	Tension-controlled failure.
Compression controlled beam.	Transition-controlled beam	Tension-controlled beam.

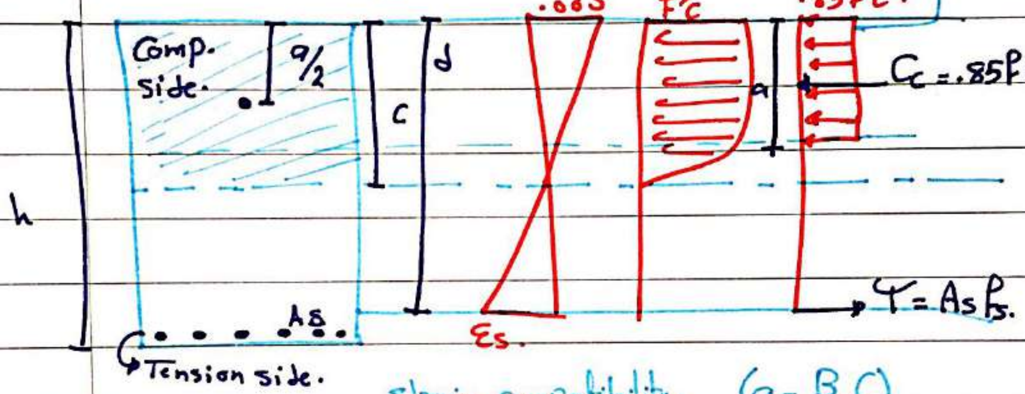
Balanced failure Beams. $\varepsilon_s = \varepsilon_y$ ... $\varepsilon_s = .005$ ... ε_s ... $\varepsilon_s \geq .005$

تقريباً ضعفين ε_y

$\varepsilon_c = .003$.

* Flexural Theory. (M_n). $\sigma = \frac{My}{I}$ (Normal)

Assumed equivalent Stress block.

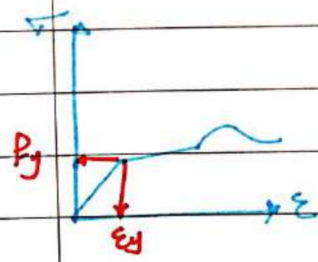


strain compatibility. ($a = \beta_1 C$)

$\frac{.003}{c} = \frac{\varepsilon_c}{d-c} \rightarrow \{\varepsilon_s = .003(\frac{d-c}{c})\}$.

$jd = (d - a/2)$.

For rectangular beams only, rectangular comp. Area.



if $\varepsilon_s < \varepsilon_y$ (Not yeild), $F_s = E\varepsilon_s$.

if $\varepsilon_s \geq \varepsilon_y$ (yeilded) $\rightarrow$ ($F_s = F_y$).

* Equlibrium:

$T = C_c$.

$A_s F_y = .85 f'c\ ab$ (Assume $\varepsilon_s \geq \varepsilon_y$, $f_s = F_y$).

$A_s F_y = .85 f'c\ ab$. $\rightarrow f_s = E\varepsilon$ (if not yeilded) * بنفرض غلط *

$a = \frac{A_s F_y}{.85 f'c\ b} \rightarrow$ Check Assumption

$M_n = \underbrace{T}_{C_c} (d - a/2)$.

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normal uniform stress. ← $T = C_c$

$A_s f_s = 0.85 f'_c\, a b.$ $f_s \rightarrow \varepsilon_s \geq \varepsilon_y,\ (f_s = f_y).$

$M_n = T_{C_c}\,(jd)$

$jd = (d - a/2) \rightarrow$ rectangular ~~comp~~ comp. areas

$\varepsilon_s = .003\left(\frac{d-c}{c}\right).$

لو طلعت مش متحقق، $\varepsilon_s < \varepsilon_y,\ f_s \neq f_y.$

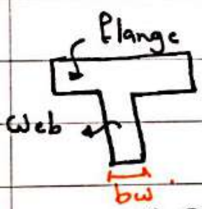
$f_s = E\varepsilon_s \rightarrow E\,(.003)\left(\frac{d-c}{c}\right).$

$(.85 f'_c\, b\,\beta_1)\,c^2 + (A_s E\, .003)\,C - A_s E\, .003\, d = 0$

نحلهم مع بعض. $A_s E\,(.003)\left(\frac{d-c}{c}\right) = .85 f'_c\, a b \rightarrow (\beta_1 c).$

* Minimum amount of Tension Reinforcement ($A_{s\,min}$) per ACI-code.

اقل مساحة علشان ما يتقطعش. بسبب الانشقاقات. ← $A_{s\,min}$ = larger of $\left\{ \frac{.25\sqrt{f'_c}}{f_y} b_w d,\ \frac{1.4}{f_y} b_w d. \right.$ for rectangular & T beams.



* Value of β_1:

$\beta_1 = .85 \quad (f'_c \leq 28\,MPa).$

$\beta_1 = .85 - .05\,\frac{f'_c - 28}{7} \quad (28 < f'_c \leq 56)$

$\beta_1 = .65 \quad (f'_c \geq 56\,MPa.).$

* الكلام هو فقط، بس لازم اعرف متى كل وحدة واستخدم الـ f'_c تبعها.

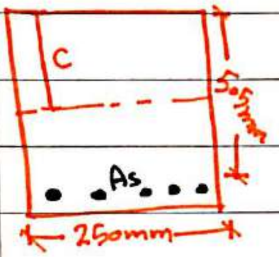
* Values of ϕ: ($\phi M_n \geq M_u$).

$\phi = .9$ (Tension-Controlled). ($\varepsilon_s \geq .005$)

$\phi = .65 + (\varepsilon_s - .002)\left(\frac{250}{3}\right)$ (Transition). ($\varepsilon_y < \varepsilon_s < .005$).

$\phi = .65$ (Balanced & Compression). ($\varepsilon_s \leq \varepsilon_y$).

*Analysis Example: (Design Moment Capacity ; ϕM_n).



*Compute a:

Assumed : ($\varepsilon_s \geq \varepsilon_y$, $f_s = f_y$).

$T = C_c$

$A_s f_y = .85 f'_c \, a b.$

$\underset{mm^2}{(1530)}\underset{MPa}{(420)} = .85\underset{MPa}{(20)}\, a \underset{mm}{(250)}$

$a = 151.2\ mm.$

$A_s = 1530\ mm^2.$

$f'_c = 20\ MPa.$

$f_y = 420\ MPa.$

*Check Assumption:

$c = a/\beta_1 = \dfrac{151.2}{.85} = 177.9\ mm.$

$*\ \varepsilon_s = .003\left(\dfrac{505-177.9}{177.9}\right) = .0055$

$*\ \varepsilon_y = \dfrac{420}{200\,000} = .0021$

$\varepsilon_s > \varepsilon_y \rightarrow$ Assumption is ok.

*Compute M_n & ϕM_n:

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

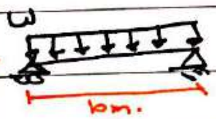
$= \underset{mm^2}{(1530)}\underset{MPa(N/mm^2)}{(420)}\left(\underset{mm}{505} - \dfrac{\underset{mm}{151.2}}{2}\right)$

$M_n = \cancel{270}\ 275.9\ kN \cdot m.$ → الرقم بيزبط بـ 10^{6} عشان يصير kN.m.

($\varepsilon_s \geq .005$) (Tension-controlled).

$\phi = .9$

$\phi M_n = (.9)(275.9) = 248.3\ kN \cdot m.$



فكرة تانية.. ←

$\dfrac{wL^2}{8} = 248.3$, د بطلع ال w..

*Check A_s min.

$A_s\ min \begin{cases} \dfrac{.25\sqrt{20}}{420} * 250 * 505 = 336\ mm^2. \\ \dfrac{1.4}{420} * 250 * 505 = 420\ mm^2. \end{cases}$

$A_s\ min = 420\ mm^2.$

$A_s = 1530 > A_s\ min \rightarrow$ ok.✓

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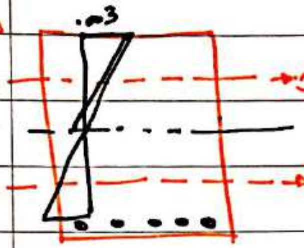
نفس السؤال الاضافي $A_s = 3060\ mm^2$.

بس ؟



بنفس طريقة حل السؤال الاضافي ✓..

*Compute a:

$3060 * 420 = .85(20)(a)(250)$

$a = 302.4\ mm.$

* check assumption:

$c = a/\beta_1 = \frac{302.4}{.85} = 355.8\ mm.$

$\varepsilon_s = .003\left(\frac{505-355.8}{355.8}\right) = .00126$

$\varepsilon_s < \varepsilon_y \rightarrow$ Not ok.✓.

$A_s(f_s = E\varepsilon_s) = .85 f'_c \beta_1 c\, b$

$3060 * 200*10^3 * .003\left(\frac{505-C}{c}\right) = .85 * 20 * .85 * C * 250$

$3612.5\ C^2 + 1836000C - 927180000 = 0$

$C = -254.12 \pm 566.78$

$C = 312.66\ mm.$

$\varepsilon_s = .003\left(\frac{505-312.66}{312.66}\right) = .00184 < \varepsilon_y \rightarrow$ Ok.✓

$f_s = (200*10^3)(.00184) = 368\ MPa.$

*Compute M_n & ϕM_n: $a = \beta_1 c = 265.76\ mm.$

$M_n = A_s f_s (d - a/2) = 3060(368)\left(505 - \frac{265.76}{2}\right) = 419\ kN \cdot m.$

$\varepsilon_s < \varepsilon_y$ (Comp. $\rightarrow .65 = \phi$).

$\phi M_n = (.65)(419) = 272.4\ kN \cdot m.$

* Check $A_{min} \rightarrow$ ok..✓

(Doubly reinforcement).

* Beams with Compression Reinforcement

DATE. 26/2/2018

First 8/3/2018

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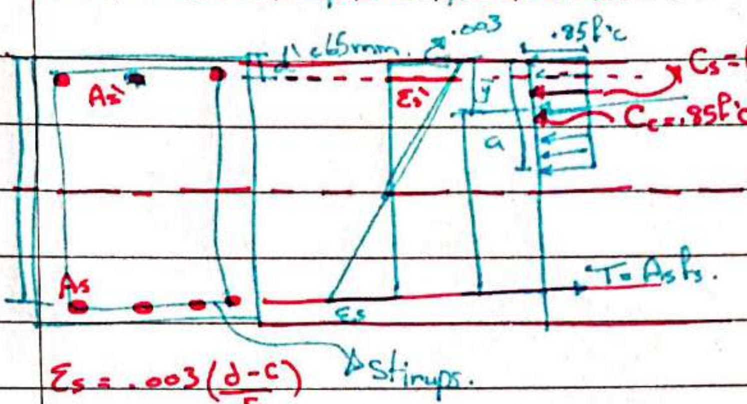


بالطالم $R = C_c + C_s$.

$\varepsilon_s = .003\left(\frac{d-c}{c}\right)$

$\varepsilon_s' = .003\left(\frac{c-d'}{c}\right)$.

$\{\bar{x}\, \Sigma A_i = \Sigma A_i\, x_i\}$.

* $T = C_c + C_s$.

$A_s f_s = .85 f'c\, ab + A_s' f_s'$.

* Assume $\varepsilon_s \ge \varepsilon_y$ $(f_s = f_y)$.

$\varepsilon_s' \ge \varepsilon_y$ $(f_s' = f_y)$.

$A_s f_y = .85 f'c\, ab + A_s' f_y$

$a \to c$. $\beta_1 c$

$f_s' = E\varepsilon_s' = E(.003)\left(\frac{c-d'}{c}\right)$.

بغير الفرضيه. 1] ε_s' check. (not ok, $\varepsilon_s' < \varepsilon_y$).

2] ε_s check. (not ok) $f_s = E\varepsilon_s = E(.003)\left(\frac{d-c}{c}\right) \to$ نجيب c جديدة.

~~نعوض~~ ونعيدها.

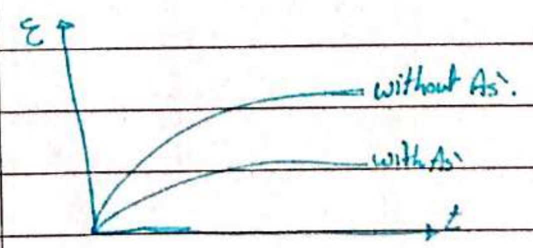
* $M_n = C_c\left(d - \frac{a}{2}\right) + C_s(d - d')$.

* ϕM_n

$\hookrightarrow A_s \to \varepsilon_s$.

* Reasons for Providing A_s'

1] Reduce sustained load deflections.



2] Increase Ductility (ε_s).

3] Change Mode of failure to Tension-controlled.

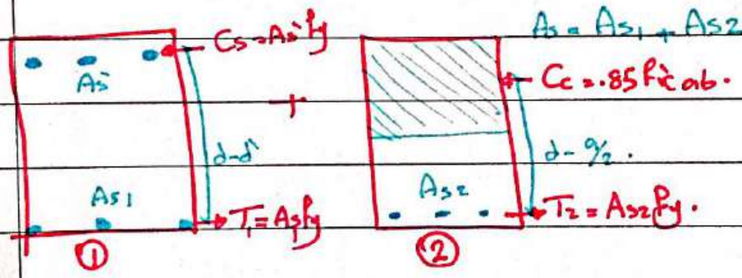
4] fabrication Ease: Providing small bars @ the corners of the stirrups to hold the ~~str~~ stirrups in place & help anchor the stirrups.
→ Ignore in strength calculations.

* Analysis of beams with A_s & A_s'.

$\varepsilon_s \geq \varepsilon_y$

* Case I: $\varepsilon_s' \geq \varepsilon_y$.

$A_s = A_{s1} + A_{s2}$. → $A_{s2} = A_s - A_{s1}$.



$T_1 = C_s$.	$T_2 = C_c$
$A_{s1} f_y = A_s' f_y$.	$A_{s2} f_y = .85 f'c\, ab$.
$A_{s1} = A_s'$.	$M_{n2} = C_c (d - a/2)$.
$M_{n1} = C_s (d - d')$.	$M_n = M_{n1} + M_{n2}$.

* Case II: $\varepsilon_s' < \varepsilon_y$.

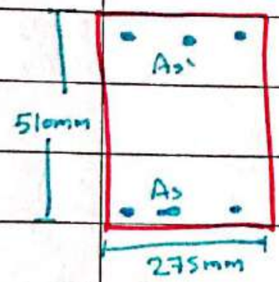
$T = C_c + C_s$.

$A_s f_y = .85 f'c\, ab + A_s' f_s'$
↳ $E \varepsilon_s' = E(.003)\left(\frac{c-d'}{c}\right)$.

من هون بطلع a. $(.85 f'c\, b)a^2 + (.003 E A_s' - A_s f_y)a - .003 E A_s' \beta_1 d' = 0.0$

* Analysis Example :

DATE.



$A_s' = 852\ mm^2$.

$A_s = 3060\ mm^2$.

$f'c = 20\ MPa$.

$f_y = 420\ MPa$.

* Compute a:

$T = C_c + C_s$.

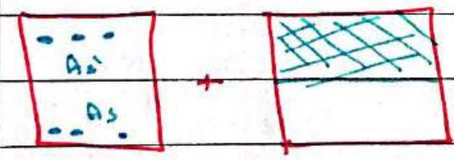
assume $\varepsilon_s \geq \varepsilon_y$ & $\varepsilon_s' \geq \varepsilon_y$.

$A_s f_y = .85 f'c\ ab + A_s' f_y$.

$(3060)(420) = (.85)(20)\ a\ (275) + (852)(420)$.

$a = 198\ mm$.

نزى الصفقة اعاضية.



$A_{s1} = 852\ mm^2$. $A_{s2} = 2208$

$a = 198\ mm$.

* Check Assumptions.

$c = \frac{198}{.85} = 233\ mm$.

$\varepsilon_s' = .003\left(\frac{233-65}{233}\right) = .0022 > \varepsilon_y = .0021$ ok ✓

$\varepsilon_s = .003\left(\frac{510-233}{233}\right) = .0036 > \varepsilon_y = .0021$ ok ✓

* M_n & ϕM_n.

$M_n = C_c(d - a/2) + C_s(d - d')$

$= (.85)(20)(198)(275)(510 - \frac{198}{2}) + 852(420)(510-65) = 540\ kN\cdot m$.

$\varepsilon_y < \varepsilon_s < .005$

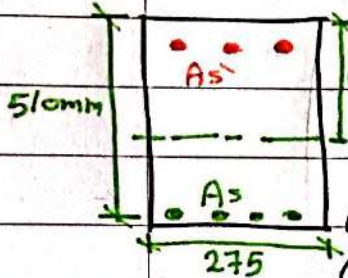
$\phi = .65 + (.0036 - .002)\left(\frac{250}{3}\right) = .783$.

$\phi M_n = (.783)(540) = 423\ kN\cdot m$.

* Check $A_{s\,min}$: (A_s).

28/12/2018

*Examples



*Compute a:

$T = C_c + C_s$.

Assume: ($\varepsilon_s \geq \varepsilon_y$, $f_s = f_y$) ($\varepsilon_s' \geq \varepsilon_y$).

$A_s f_y = .85 f'_c\, a b + A_s' f_y$.

$(3060)(420) = .85(20)\,a\,(275) + (1704)(420)$

$A_s' = 1704\,mm^2$ $a = 121.8\,mm$.

$A_s = 3060\,mm^2$. $C = \frac{a}{\beta_1} = 143.3\,mm$.

$f'_c = 20\,MPa$.

$f_y = 420\,MPa$.

*Check assumptions:

$\varepsilon_s' = .003\left(\frac{143.3 - 65}{143.3}\right) = .0016$

$\varepsilon_s' < \varepsilon_y \rightarrow$ Not ok..!

$A_s f_y = .85 f'_c \underbrace{a}_{\beta_1 c} b + A_s' f_s' \rightarrow [E \times .003(\frac{c - d'}{c})]$.

$3973.75 c^2 + 262800 c - 6645600 = 0$

$c = 33.07 \pm 133.48$

$C = 166.55\,mm$.

$a = \beta_1 C = 141.57\,mm$.

*Check ε_s:

$\varepsilon_s = .003\left(\frac{510 - 166.55}{166.55}\right) = .0062$, $\varepsilon_s > \varepsilon_y \rightarrow$ Ok.!

* Compute M_n & ϕM_n.

$M_n = C_c(d - a/2) + C_s(d - d')$.

$\varepsilon_s' = .003\left(\frac{166.55 - 65}{166.55}\right) = .0018 < \varepsilon_y$.

$f_s' = .0018 \times (200{,}000) = 360\,MPa$.

$M_n = .85(20)(141.57)(275)(510 - \frac{141.57}{2}) + (1704)(360)(510 - 65) = 563.7\,kN.m$.

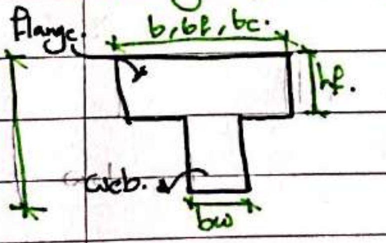
$\varepsilon_s > .005 \rightarrow$ Tension.. $\rightarrow \phi = .9$

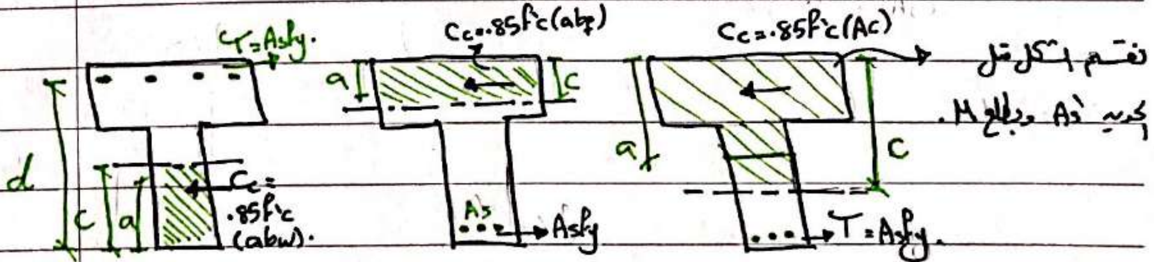
$\phi M_n = .9(563.7) = 507.3\,kN.m$.

* Check As min → Ok..!

[illegible] $\left\{ \begin{array}{l} As' \rightarrow \text{ بتأثر على } \phi M_n. \\ As \rightarrow \text{ بتأثر على } M_n \end{array} \right.$

* Analysis of T- Beams:



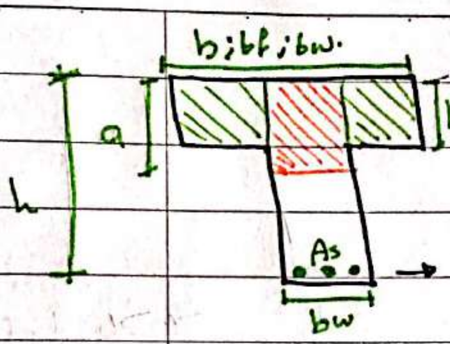


Case I (-ve M). Flange in Tension. $M_n = T(d - a/2)$, C_c

Case II: (+ve M). Flange in Comp. $M_n = T(d - a/2)$, C_c, $a < h_f$.

Case III: (+ve). Flange in Comp. $a > h_f$.

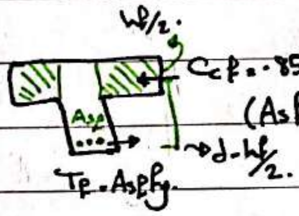
DATE.



C, L, I [illegible]
. beams

* Analysis of Nominal moment capacity for Flanged sections in positive Bending ($a > h_f$).

* Beam flange (beamf).



$C_{cf} = .85 f'_c (b-b_w) h_f$.

($A_{sf} + A_{sw} = A_s$).

$T_f = C_{cf}$.

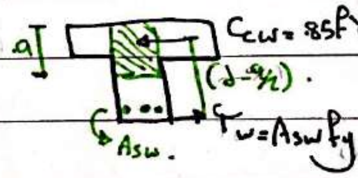
. [illegible] $A_{sf} f_y = .85 f'_c (b-b_w) h_f$.

$A_{sw} = A_s - A_{sf}$.

{$A_s = A_{sf} + A_{sw}$}.

$M_{nf} = \frac{C_{cf}}{T_f} (d - h_f/2)$.

* Beam web (Beam w).



$C_{cw} = .85 f'_c (a b_w)$.

$T_w = A_{sw} f_y$

$T_w = C_{cw}$ [illegible]

$A_{sw} f_y = .85 f'_c a b_w$.

$M_{nw} = \frac{T_w}{C_{cw}} (d - a/2)$

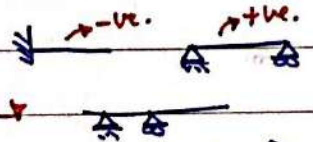
* $M_n = M_{nf} + M_{nw}$.

DATE. 5/3/2018.

13/3 → RC 1.
11/3 → foundation.
15/3 → ج.ع

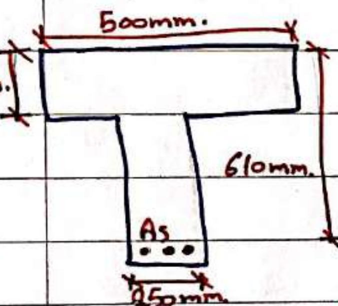
* Modification to $A_{s\,min}$:

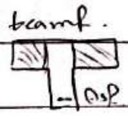
$$A_{s\,min} = \begin{cases} \frac{\sqrt{f'c}}{4 f_y} b_w d \\ \frac{1.4}{f_y} b_w d \end{cases}$$

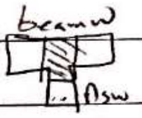


* For statically determinate beams when the flange portion is in tension ACI-Code recommends that b_w be replaced by smaller of $\begin{cases} 2b_w \\ b_f \end{cases}$.

Example.







The Moment capacity
$f'c = 20\,MPa$, $f_y = 420\,MPa$.
$A_s = 3060\,mm^2$.

* Compute a:

$T = C$ (assume $\varepsilon_s \geq \varepsilon_y$) (assume $a \leq h_f$).

$A_s f_y = .85 f'c\, a\, b_f$.

$(3060)(420) = .85(20)\,a\,(500)$.

$a = 151.2\,mm > h_f$ → T-beam action.

Beam f:

$T_f = C_{cf}$.

$A_{sf} f_y = .85 f'c (b - b_w)(h_f)$.

$(A_{sf})(420) = .85(20)(500-250)(125)$.

$A_{sf} = 1264.9\,mm^2$.

$A_{sw} = 3060 - 1264.9 = 1795.1\,mm^2$.

Beam w:

$T_w = C_{cw}$.

$A_{sw} f_y = .85 f'c\, a\, b_w$

$(1795.1)(420) = .85(20)\,a\,(250)$

$a = 177.4\,mm$. $C = 208.7\,mm$.

$\varepsilon_s = .003\left(\frac{610 - 208.7}{208.7}\right) = .0058 > \varepsilon_y$ → ok..!

DATE.

* M_n & ϕM_n:

$M_{nf} = A_{sf} f_y (d - \frac{h_f}{2})$.

$= 290.9$ kN.m.

$M_{nw} = A_{sw} f_y (d - \frac{a}{2}) = 393$ kN.m.

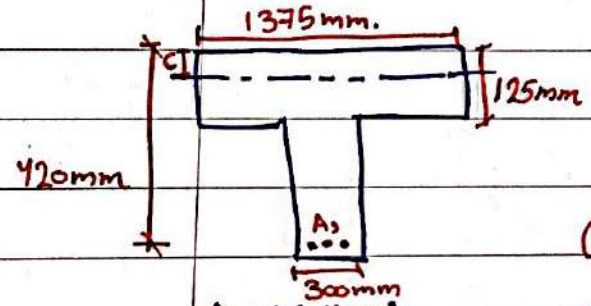
$M_n = 683.9$ kN.m.

$\phi M_n = .9(683.9) = 615.5$ kN.m.

$\phi = .9$ ($\varepsilon_s > .005 \rightarrow$ Tension).

* Check $A_{s\,min} \rightarrow$ ok ✓.

Example:



$A_s = 1704 mm^2$.

$f'_c = 20 MPa$.

$f_y = 300 MPa$.

* Compute a:

$T = C$ ($\varepsilon_s > \varepsilon_y$, $a \leq h_f$).

$A_s f_y = .85 f'_c a b$.

$(1704)(300) = .85(20)a(1375)$

$a = 21.9$ mm. $C = 25.8$ mm.

$\varepsilon_s = .003\left(\frac{420 - 25.8}{25.8}\right) = .046 > \varepsilon_y \rightarrow$ ok ✓

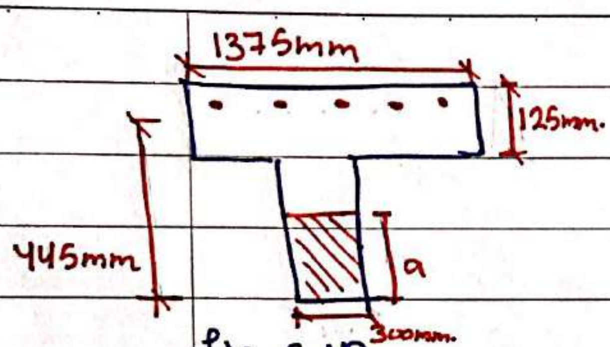
$M_n = A_s f_y (d - \frac{a}{2}) = 1704 \times 300 (420 - \frac{21.9}{2}) = \boxed{}$ kN.m 209.12

$\phi = .9 \rightarrow \varepsilon_s > \varepsilon_y \rightarrow$ Tension.

$\phi M_n = 188$ kN.m.

(22)

DATE.



$f'c = 20MPa.$

$fy = 300MPa.$

$\emptyset Mn \rightarrow$ Indeterminate beam.

$T = C \quad (\varepsilon_s \geq \varepsilon_y).$

$(2275)(300) = .85(20)a(300)$

$a = 133.6 mm.$

$\varepsilon_s = .0055$

$M_n = A_s f_y (d - a/2)$

$\emptyset = .9$

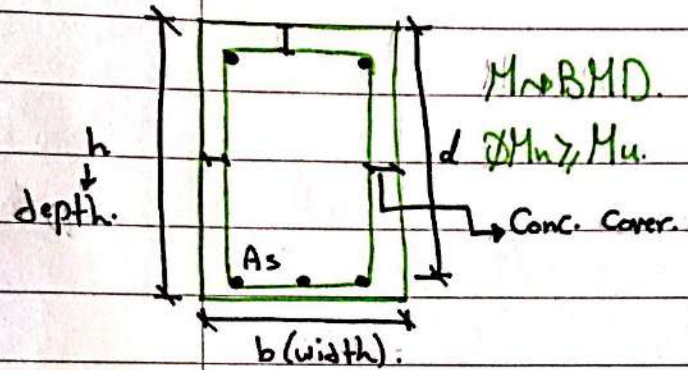
$\emptyset M_n = 232 \, KN.m.$

※ first.

(23)

DATE. 7/3/2018

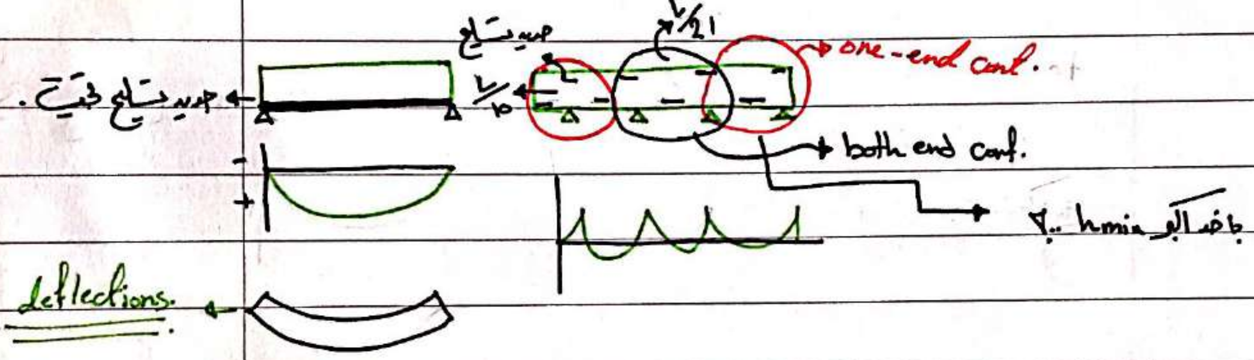
* Design of rectangulare beams:



$M_n \sim BMD.$

$\phi M_n \geq M_u.$

* Visualize expected deflected shape.



* Relation-Ship between Beam depth & deflections:

from ACI-code (h_{min} → To avoid deflection calculations) → (Table 9.5a).

Simply supported beam: $h_{min} = \frac{L}{16}$

Cantilever beam: $h_{min} = \frac{L}{8}$

* Concrete cover & Bar spacing (Per ACI-code).

min. cover ≡ 40mm (beams) (Normal exposure).

* Reasons for Cover.

1] Bond. 

2] Corrosion (rusting of steel).

3] strength loss → overheating (fire).

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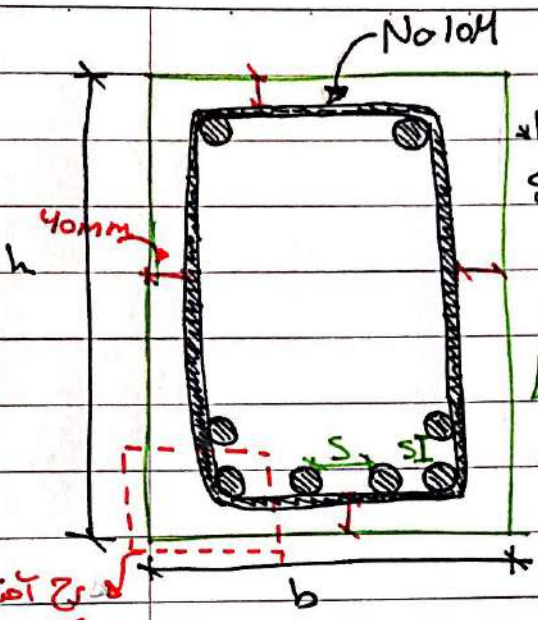


رح آخدها
والبرها..!

* Horizantal spacing:

S_{min} ≡ larger of:

↳ Bar diameter: d_B.

↳ 1.33 max. size of coarse aggregate.

↳ 25mm / diameter of vibrator.

* Virtical spacing:

S_{min} = larger of:

↳ 1.33 max. size of coarse aggregate.

↳ 25mm.

* $b_{min} = 2(40) + 2(10) + 6(20) + \underbrace{5(25)}_{\text{spacing}} = 345$ mm.

6No.20M → design

b = 350mm (Design) → ok!. (one layer).

b = 300mm (Per design) → Not ok ..! (Should be in 2 layers).

Extra distance = $(2d_s - .5d_b)$.

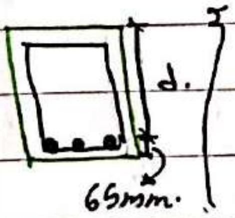
$\cancel{2(10) - .5(20)} = 10$ mm

$2(10) - .5(20)$

نزيد ع الـ b min.

$b_{min} = 345 + 20 = 365$ mm.

* Estimating $\underline{d}$ of a beam:



$d = h - 65^{mm}$ (one-layer).

$d = h - 90^{mm}$ (Two-layers).

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* Minimum beam width ($\underline{b}$) (Per ACI-code).

Shall not be less than 250^{mm}.

↳ Preferably = 300 mm.

* General Strength Design Requirment for Beams.

$\phi M_n \geq M_u$.

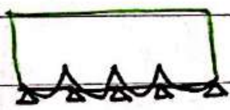
$W_u = 1.2DL + 1.6LL$ or $W_u = 1.4DL$.

$\phi M_n = \phi A_s f_y \underbrace{(d - a/2)}_{jd}$

$\phi M_n = \phi A_s f_y \, jd$.

↳ $\phi M_n = M_u$.

$M_u = \phi A_s f_y \, jd \rightarrow A_s = \dfrac{M_u}{\underset{0.9}{\phi} f_y \, jd}$] → when b & h are known..



↳ $j = (.87 - .91)$

↳ $j = \underline{\underline{0.9}}$

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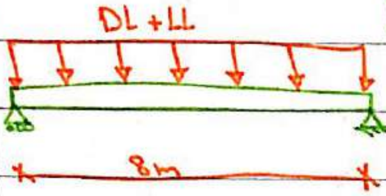
*Example: Design of Reinforcement when b & h are known.

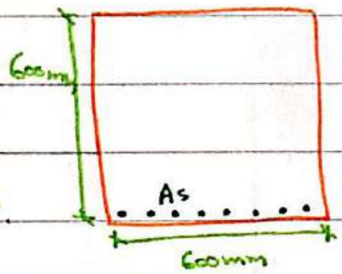


$f'_c = 20\,MPa.$

$f_y = 420\,MPa.$

$\gamma_{conc.} = 24\,kN/m^3.$

$LL = 35\,kN/m.$



DL = 14 kN/m (excluding self wt. of the beam).

$A_s = \dfrac{M_u}{\phi f_y\, jd} \rightarrow jd = d - a/2$

$.9d = d - a/2 \rightarrow a = .2d.$

1] Estimate M_u:

$M_u = \dfrac{W_u L^2}{8}$

Self wt. $= bh\gamma$

$= .6 * .6 * 24$

$= 8.64\,kN/m.$

$W_u = 1.2\,DL + 1.6\,LL$

$= 1.2(8.64 + 14) + 1.6(35)$

$= 83.2\,kN/m.$

$M_u = \dfrac{83.2 * 8^2}{8} = 665.6\,kN.m.$

2] Estimate d:

Assume 1 layer:

$d = h - 65^{mm} = 600 - 65 = 535\,mm.$

3] Check As min:

$\rightarrow 854\,mm^2.$

$\rightarrow 1070\,mm^2. = A_{s\,min}$

4] Compare As:

$A_s = \dfrac{665.2 * 10^6}{.9(420)(.9)(535)} = 3657\,mm^2. > A_{s\,min}$ (ok) → وبنعيد الدوري

Iterations

Iterations

$a = \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{3657 * 420}{.85(20)(600)} = 150.6\,mm.$

$A_s = \dfrac{665.2 * 10^6}{.9(420)(535 - \frac{150.6}{2})} = 3830.4\,mm^2.$

$a = \dfrac{3830.4 * 420}{.85(20)\,(600)} = 157.7\,mm.$

$A_s = \dfrac{665.2 * 10^6}{.9\,(420)(535 - \frac{157.7}{2})} = 3860.3\,mm^2.$

DATE.

[$a = 159\,mm$, $A_s = 3865.6\,mm^2$.] → Required.

5] Select Steel: لازم اختار العدد والحجم، لازم اختار

$8\,No.25M$; $A_s = 4080\,mm^2$. القيمة الأكبر والأقرب للـ required).

6] Check b_{min}:

$b_{min} = 2(40) + 2(10) + 8(25) + 7(25) + 2(2(10) - .5(25))$

$b_{min} = 490\,mm < b$ → One-layer (ok✓).

* انا بسوي الـ check وبشوف الـ 600، وبحسب الـ b_{min}، وبقارن القيمتين، بـ في حال كانت b_{min} أكبر من الـ b، فح يكون عنا 2-layers، ولازم ارجع احسب الـ As من جديد، وبكمل بنفس البروسيجر السابق.

7] Analysis ($\phi M_n \geq M_u$)($\phi = .9$) → فرض.

$$a = \frac{4080 * 420}{.85(20)(600)} = 168\,mm.$$

$$C = a/\beta = 197.6\,mm.$$

$$\varepsilon_s = .003\left(\frac{535 - 197.6}{197.6}\right) = .00512 > .005 \quad \text{(Tension Controlled, } \phi = .9).$$

(if it's not ok, we have to put A_s').

$$\phi M_n = .9(4080)(420)\left(535 - \frac{168}{2}\right) = 695.6\,kN.m > M_u \rightarrow ok.$$

8] Check required A_s based on computed a: ($jd = d - a/2$)
($.9d = d - a/2$)

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{665.6 * 10^6}{.9(420)(535 - \frac{168}{2})} = 3904\,mm^2.$$

< As provided (ok)! ← if it's not ok, we will repeat all the example

$$h_{min} = \frac{L}{16} = \frac{8000}{16} = 500\,mm < 600\,mm$$

ما بتغير شي، وحسبها عشان اعرف اذا صغير او deflection لا

(No deflection).

*Design of rectangulare beams. (when b, h & As are unknown).

$a = \frac{A_s f_y}{.85 f'_c b}$ $(\varepsilon_s \geq \varepsilon_y)$.

ρ (Percentage of reinforcement) "steel ratio"

$\rho = \frac{A_s}{bd} \rightarrow A_s = \rho bd$.

ω (mechanical steel ratio).

$a = \frac{\rho \cancel{b} d f_y}{.85 f'_c \cancel{b}} \rightarrow a = \rho \frac{f_y}{f'_c} \cdot \frac{d}{.85}$

$a = \omega \frac{d}{.85}$

بعوض بال

$\phi M_n = \phi \; .85 f'_c a b (d - a/2)$

$\phi M_n = \phi \left[b d^2 \underbrace{f'_c \omega (1 - .59\omega)}_{k_n, R_n} \right]$ ↳ flexural resistance factor.

$\downarrow$

M_u.

* $M_u = \phi b d^2 k_n$.

$\boxed{bd^2 = \frac{M_u}{\phi k_n}}$ ※. $\rightarrow A_s = \frac{M_u}{\phi f_y jd}$.

* Estimating self wt. of rectangulare beams.

[1] The weight of the rectangulare beams will roughly be 10 to 15% of the loads it must carry.

Self wt. = (10-15%) (DL+LL).

[2] $h \simeq \left(\frac{1}{18} \text{ to } \frac{1}{12}\right)$ of span length. → only to estimate self weight.

$b \simeq .5h$.

Self wt = $\gamma b h$.

DATE.

*Selection of a steel ratio:

- Economic consideration: ($\rho = .01$).

- Ductility Consideration:

$$0.35\rho_b \leq \rho \leq .40\rho_b.$$

Balanced steel ratio. $\rightarrow$ $\rho_b = \frac{A_{sb}}{bd}$ $\rightarrow$ balanced area of steel.

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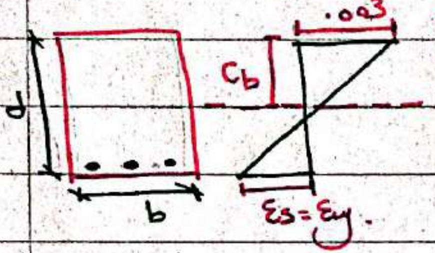
DATE. 19/3/2018

* Selection of ρ :

- Ductility Consideration:

$\rho = (.35 - .40)\, \rho_b$ ..



$\sum T = C$

$A_{sb} f_y = .85 f'_c\, ab.$

$\rho_b b d\, f_y = .85 f'_c\, \beta_1 C_b\, \cancel{b}$

$C_b = \dfrac{\rho_b d f_y}{.85 f'_c \beta_1}$

. balance ..

* Strain Compatibility

$\dfrac{.003}{C_b} = \dfrac{.003 + \varepsilon_y}{d}$ بدي اساويهم ببعض.

$C_b = \dfrac{.003}{.003 + \varepsilon_y} \times d.$

$\rho_b = \dfrac{.85 \beta_1 f'_c}{f_y} \cdot \left(\dfrac{.003}{.003 + \varepsilon_y} \right)$

$= \dfrac{.85 \beta_1 f'_c}{f_y} \left(\dfrac{600}{600 + f_y} \right)$ $= * \dfrac{E}{E}$ في حال ضربنا

* في حال ما اتذكر شي، بنستخدم (ρ) Economy.

* By placing Consideration:

It may be hard to place the reinforcement if ρ exceeds .015

$\rho = \dfrac{.24 \beta_1 f'_c}{f_y}$, $\rho = \dfrac{\beta_1 f'_c}{4 f_y}$.

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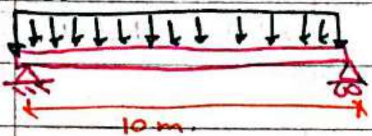
Example: Design of rectangulare beam: (b, h & A_s are not known).

$w = DL$ & LL.



$LL = 25.5$ kN/m.

$DL = 14.5$ kN/m (excluding self wt.)

$f'_c = 25$ MPa, $f_y = 420$ MPa.

$$*\ bd^2 = \frac{M_u}{\phi k_n} \ ; \ A_s = \frac{M_u}{\phi f_y jd}$$

1- Estimate self wt.

I] Self wt $= (10-15\%)(25.5+14.5)$

$= (4-6)$ kN/m.

OR

II] $h \simeq \left(\frac{1}{18} - \frac{1}{12}\right) * 10{,}000$

$\simeq (555 - 833)$ mm.

$b \simeq .5h \simeq (275 - 415)$ mm

Self wt. $= (3.66 - 8.3)$ kN/m.

Try self wt. $= 8$ kN/m.

2- Compute M_u:

$W_u = 1.2(14.5+8) + 1.6(25.5)$

$W_u = 67.8$ kN/m.

$$M_u = \frac{67.8 * 10^2}{8} = 848 \text{ kN.m.}$$

3- b & d.

$\rho = .01$ (economic consideration).

$$\omega = \rho \frac{f_y}{f'_c} = .01 * \frac{420}{25} = .168$$

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$\phi k_n = .9(25 \times .168 \times (1 - .59 \times .168))$

$\phi k_n = 3.41$ MPa.

$bd^2 = \dfrac{848 \times 10^6}{3.41} = 248.7 \times 10^6\ mm^3.$

possible choises $(d > b)$.

$b = 300mm;\ d = 910mm.$

$b = 400mm;\ d = 788\ mm$

$b = 450mm;\ d = 743mm.$ بعتمد على ال concept ؟

* Assume 2 layers:

$h = 788 + 90 = 878\ mm \approx 900mm.$

$b = 400mm,\ h = 900mm,\ d = 810mm.$

- Check self wt & revise M_u if required.

* Self wt. new = γbh.

$= 24(.4)(.9) = 8.64\ kN/m.$ → المشكلة بس تكون اكبر او تكون اصغر بكثير.

$W_{u\,new} = 1.2(142 + 8.64) + 1.6 \times 25.5$

$= 68.57\ kN/m.$

$M_{u\,new} = \dfrac{68.57 \times 10^2}{8} = 857\ kN.m$

ACT-code ← * If M_u is increased by 10% or more repeat the design.

بغيّر dimentions الـ.

$\dfrac{857 - 848}{848} \times 100\% = 1.1\% < 10\%.$ → Proceed using $M_u = 857\ kN.m$

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$b = 400mm$, $h = 900mm$, $d = 810\ mm$ (Two-layers).

$M_{u,new} = 857\ kN.m.$

- $A_s = \dfrac{M_u}{\varnothing f_y j d} = \dfrac{857\times10^6}{.9\times420\times.9\times810}$
  $= 3110\ mm^2$

- Iterations:

$a = \dfrac{3110\times420}{.85\times25\times400} = 153.7\ mm.$

$A_s = \dfrac{857\times10^6}{.9(420)(d-a/2)} = 3092\ mm^2$

$a = \dfrac{A_s\times420}{.85\times25\times400} = 152.8\ mm.$

$A_s = \dfrac{857\times10^6}{.85\times25(d-a/2)} = 3090\ mm^2.$

- Select steel.

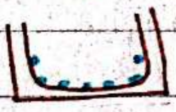
7 No.25M; $A_s = 3570\ mm^2$.

$b_{min} = 2\times40 + 2\times10 + 7(25) + 6(25) + 2(20 - .5\times25)$

$b_{min} = 440\ mm > 400mm.$

→ Assumption is ok ✓ (2-layers).

b_{min}(5No.25M) → ok ✓



- check A_s min.

$A_{s\,min} = \begin{cases} 964\ mm^2 \\ 1080\ mm^2. \end{cases}$

$A_{s\,min} = 1080\ mm^2.$

$A_s = 3570\ mm^2 > A_{s\,min}$ → ok ✓

- $\varnothing = .9$, $\varnothing M_n \geq M_u$.

$T = C_c$.

$a = \dfrac{3570\times420}{.85\times25\times400} = 176.4\ mm.$

$c = a/\beta_1 = 207.5\ mm.$

$\varepsilon_s = .003\left(\dfrac{810-207.5}{207.5}\right) = .0087 > .005$ ($\varnothing = .9$) (Tension controlled).

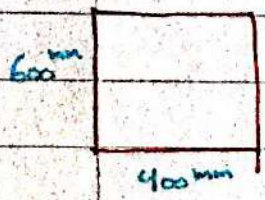
$M_n = 357\times420\times(810 - \dfrac{176.4}{2})$.

$M_n = 1082\ kN.m.$

$\varnothing M_n = 974\ kN.m > M_u = 857\ kN.m$ → ok ✓

$A_{s\,max} = 5229.3\ mm^2.$

* Design Example:

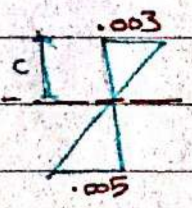


$M_u = 720$ kN.m.

$f'_c = 28$ MPa.

$f_y = 414$ MPa.

* Maximum Area of Tension Reinforcement.



$A_{s\,max} \rightarrow \varepsilon_s = .005 \rightarrow \rho_{max}.$

$A_{s\,max} = .319 \beta_1 \frac{f'_c}{f_y} bd.$

$A_s = \frac{M_u}{\phi f_y jd}.$

* assume 2-layers:

$d = 600 - 90 = 510$ mm.

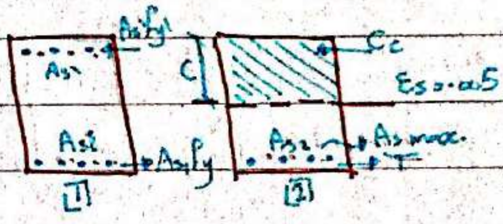
$A_s = \frac{720 \times 10^3}{.9 \times 414 \times .9 \times 510}$

$= 4210$ mm².

$A_{s\,max} = .319(.85)\frac{28}{414} \times 400 \times 510$

$= 3741$ mm².

* Doubly reinforcement Beam:



$A_{s2} = A_{s\,max} = 3741$ mm².

$T_2 = C_c \rightarrow \varepsilon_s = .005$

$A_s f_y = C_c + A'_s f'_s$

$+ A_{s1} f_y$ =

$A_s > A_{s\,max}$ (not tension controlled).

$\varepsilon_s < .005$ X

* Beam 2:

$T_2 = C_c$

$3741 \times 414 = .85 \times 28 \times a \times 400$

$a = 162.7$ mm., $c = 191.4$ mm.

As check ✓ ← $\varepsilon_s = .003\left(\frac{510 - 191.4}{191.4}\right) = .005$

$\phi M_n = \phi A_{s2} f_y (d - a/2)$

$= .9(3741)(414)\left(510 - \frac{162.7}{2}\right)$

$= 597.5$ kN.m.

$\phi M_{n1} = M_u - \phi M_{n2} = 720 - 597.5 = 122.5$ kN.m

$\phi M_{n1} = \phi A_{s1} f_y (d - d').$

$122.5 \times 10^6 = .9 A_{s1} (414)(510 - 65)$

$A_{s1} = 738$ mm².

$A_s = A_{s1} + A_{s2} = 738 + 3741 = 4479$ mm² (required)

Select steel for As:

use 9 No. 25M ; $A_s = 4590\ mm^2$

$b_{min} = 540^{mm} > 400 \rightarrow$ 2-layers.

b_{min} (6 No. 25) → $\underline{\underline{ok}}$!

$A_{s_1} = 4590 - 3741$

$= 849\ mm^2$.

* Beam 1:

$T_1 = C_{s_1}$

$A_{s_1} f_y = A_s' f_s'$

$\varepsilon_s' = .003\left(\frac{191.4 - 65}{191.4}\right)$

$= .00198 < \varepsilon_y$.

$(849)(414) = A_s' \times .00198 \times 200 \times 10^3$

$A_s' = 900\ mm^2$. (required)

* Select A_s'

(5 No. 16M, $A_s' = 995\ mm^2$). → ε_s [illegible]

$b_{min} =$

* Complete Analysis.

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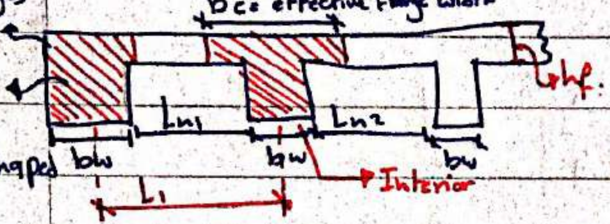
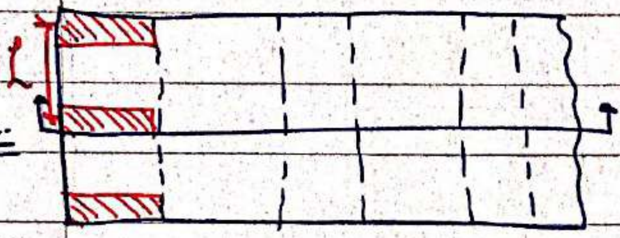
* Design of T-beam:

Exterior (edge).



Spandrel T-beam. (Inverted L-shaped beam).



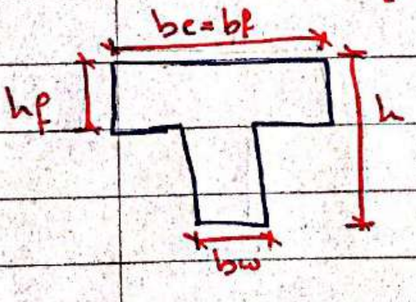
section.

* Spanderl T-beam:

b_e = smaller of $\begin{cases} b_w + \frac{L_{n1}}{2} \\ b_w + 6h_f. \\ b_w + \frac{L}{12} \end{cases}$

* Interior T-beam.

b_e = Smaller of $\begin{cases} b_w + \frac{L_{n1}}{2} + \frac{L_{n2}}{2} \\ b_w + 2(8h_f). \\ L/4. \end{cases}$

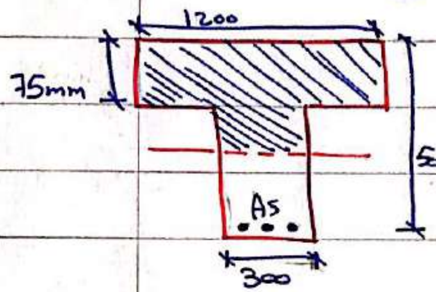


* In design:
  - Compression in web → rectangular beam ($j = .9$).
    - ↳ (Narrow Comp. Areas).
  - compression in flange → ($j = .95$) (wide comp. areas).

$A_s = \frac{M_u}{\phi f_y j d}$.

jd = moment arm.

* Example: Design of reinforcement.



$$A_s = \frac{740 \times 10^6}{(.9)(420)(.95)(500)} = 4121\ mm^2.$$

* Use 9No.25M; $A_s = \cancel{4950}\ 4590\ mm^2$.

- Check $A_{s\,min}$:

$Mu\,(+ve) = 740\,kN\cdot m.$ $\;\to \left\{ \begin{array}{l} 500\,mm^2 \\ 400\,mm^2 \end{array} \right. \to A_s > A_{s\,min}$ ok!

$f'_c = 21\,MPa.$

$f_y = 420\,MPa.$

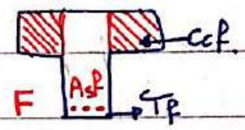
* $\emptyset M_n \geq M_u$ & $\emptyset = .9$

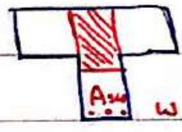
$a \leq h_f.$

$C = T$

$.85(21)a(1200) = 4590(420)$

$a = 90\,mm > h_f \to T$-beam





$T_f = C_f.$

$A_{sf} = 2869\,mm^2.$

$A_s = \cancel{4950}\ 4590 - 2869 = 1721\,mm^2.$

$T_w = C_w.$

$a = 135\,mm.$

$C = 158.8\,mm.$

$$\varepsilon_s = .003\left(\frac{500 - 158.8}{158.8}\right) = .0064 > .005 \to \emptyset = .9$$

$\emptyset M_n = \emptyset M_{nf} + \emptyset M_{nw}.$

$\emptyset M_{nf} = .9\,T_f\,(d - \frac{h_f}{2}) = 501.6\,kN\cdot m.$

provided. ← $\emptyset M_{nw} = .9\,T_w\,(d - \frac{a}{2}) = 281.4\,kN\cdot m.$

$\emptyset M_n = 783\,kN\cdot m > M_u = 740\,kN\cdot m.$

$\left\{ \right.$ As required < As provided.

* بعمل check ع الي بيعتمد على a الي هو Asw

* Check As required based on computed value of a:

$\emptyset M_n = \emptyset M_{nf} + \emptyset M_{nw}.$

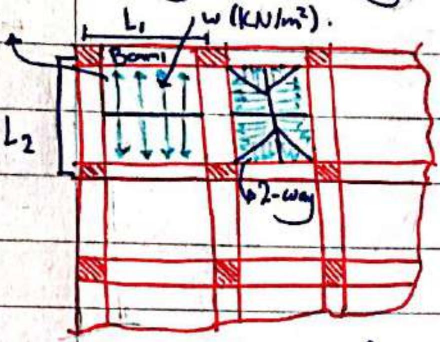
$501.6 + \emptyset M_{nw} = 740$

$\emptyset M_{nw} = 238.4\,kN\cdot m < 281.4\,kN\cdot m$ (ok!)

required. → $\dfrac{238.4 \times 10^6}{.9(420)(500 - \frac{135}{2})} = 1458\,mm^2. < 1721\,mm^2$ (ok!)

Tributary Area for beam 1

* Design of one-way solid slabs: (2D element).



$$\frac{\text{longer dimension}}{\text{shorter dimension}} \geq 2 \quad \text{(one-way)}.$$

$\frac{L_1}{L_2} > 2$

$w(kN/m^2) \times \frac{L_2}{2}$ → عشان احولو لل load دريفل موزع على L_1.

وعبئي بالاتجاه القصير ~~لأن~~ عشان الـ K (stiffness) بكون أعلى.

2-way → Non-uniform.

1-way → always uniform (always to short way).

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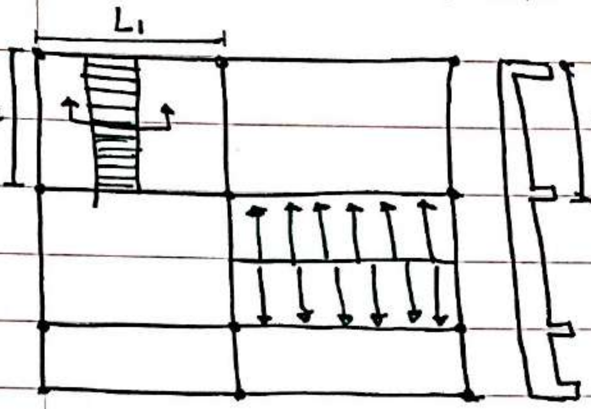
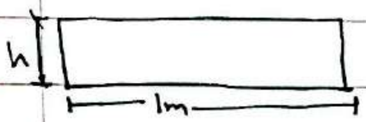


* الي بجد السلابة انها لازم تكون قائمة بـ 4 بيمات، وطول 12 بيم.

$h_{min} = \frac{L}{28}$ → الطول باتجاه القصير.

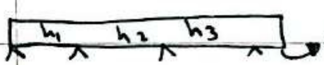
في حال طلعت اقل باخذ $\frac{L}{28}$.



$A_s = \frac{M_u}{\emptyset f_y\, jd}$ ($j \to .95$)

* Table 9.5A → h_{min}.

* Cover = 20 mm (Normal).



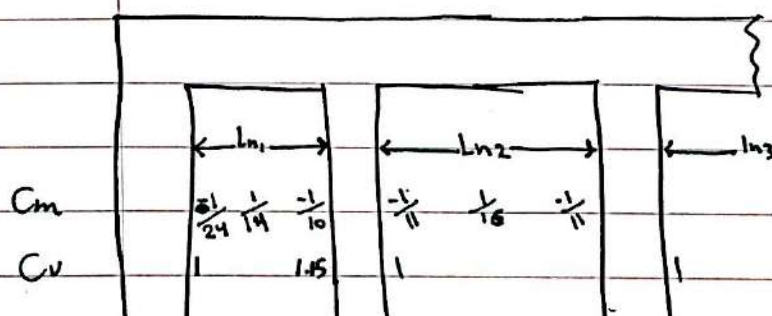
cantilever.

3 * يتم باخذ الـ h الاكبر.

* ACI- Moment & Shear coefficients for analysis & densign of non-prestressed one-way slaps & continous beams:

$M_u = C_m w_n l_n^2$.

$V_u = C_v \frac{w_u l_n}{2}$



* in one end continous

$C_{m} = $ Zero.

Girder، هو البيم الي بكون supported by other beams.

(يعني سلابة على بيم وهاد البيم على بيمز).

↳ girder

DATE.

*Use ~~#~~ only if:

1- Two spans or more.

2- Spans are are approximately equal Diff < 20%

3- loads are uniformly distributed.

4- $LL \leq 3DL$

5- member are prismatic.

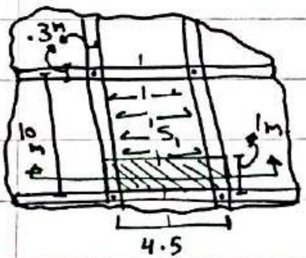
$A_{s\,min} = .0018\,bh$ (grade 60)

$A_{s\,min} = .002\,bh$ (grade 40 or 50).

$\rho_{min} \Rightarrow \rho = \frac{A_s}{bh}$.
$\downarrow$
$.01$

*Design Example:

« بنشتغل هلأ عالسلابة باتجاه القصير».



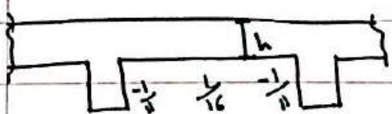
$f'_c = 28\,MPa$. DL $= 3\,kN/m^2$.

$f_y = 414\,MPa$. (excluding self wt).

LL $= 4\,kN/m^2$.

$\frac{10}{4.5} = 2.22 > 2 \rightarrow$ one-way.

1] Estimate the thickness (h).



from center to center:

$h_{min} = \frac{L}{28} = \frac{4500}{28} = 160\,mm$. , use $h = 180\,mm$. (both end cont.)
$\downarrow$ To avoid deflection.

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2) Estimate factored load :

Self wt = 24 kN/m² * .18 = 4.32 kN/m².

$W_u = 1.2(3+4.32) + 1.6*4$

$= 15.18\ kN/m^2$.   . b=1 لانه دهيم على شريحة

* Cover min = 20mm. (Normal).

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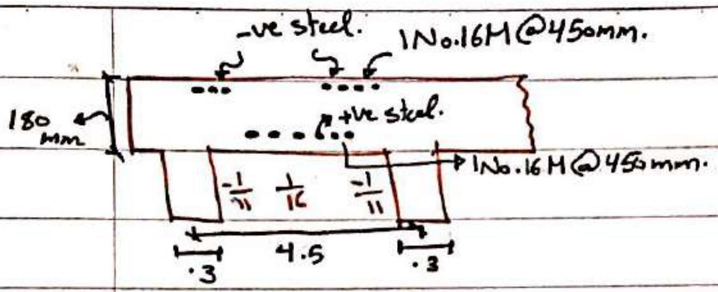
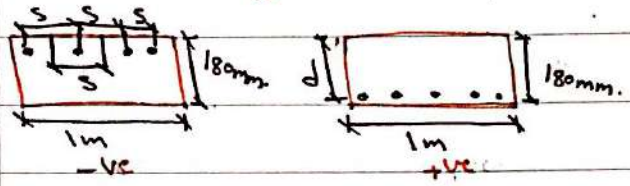


لما يطلع عدد الحديد،
بطلع الـ spacing .

$M_u(-ve) = \frac{15.18 * 4.2^2}{11} = 24.34 kN.m.$

$M_u(+ve) = \frac{15.18 * 4.2^2}{16} = 16.74 kN.m.$



$A_s = \frac{M_u}{\phi f_y jd}.$

بدنا نفرق الـ S.

Use No.16M. steel, $A_b = 199 mm^2$.

$d = 180 - 20 - \frac{16}{2} = 152^{mm}$.

*1) Design for $M_u(-ve)$.

$A_s = \frac{24.34 \times 10^6}{.9(414)(.95)(152)} = 452.4 mm^2/m$ (required).

$A_{s\,min} = .0018 \times 1000 \times 180 = 324 mm^2/m$.

$A_s > A_{s\,min}$.

$a = \frac{452.4 * 414}{.85 * 28 * 1000} = 7.87 mm.$

$A_s = \frac{24.34 \times 10^3}{.9 * 420 * (152 - \frac{7.87}{2})} = 441.2 mm^2/m.$

$a = 7.67 mm$, $A_s = 440.9 mm^2/m$.

$c = 9^{mm}$ $\varepsilon_s >> .005 \rightarrow \phi = .9$

$\varepsilon_s = .0477$.

$S = \frac{1000 A_b}{A_s}$, A_b = 1No.16M. (area of one bar).

$S \rightarrow A_b$

$1000 \rightarrow A_s$.

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$S = \frac{1000(199)}{440.9} = 451^{mm}$ → نقرب للأقل.

≈ 450 mm.

$S_{max} = \text{Smaller of} \begin{cases} 3h \\ 450\,mm. \end{cases}$

$3(180) = 540$ mm.

* $S_{max} = 450$ mm, use $S = 450$ mm.
  ↓ $S = S_{max}$ → ok.

* Design for $M_u(+ve) = 16.74$ kN.m /m.

$A_s = \frac{16.74 \times 10^6}{.9 \times 414 \times .95 \times 152} = 311\ mm^2.$

$A_{s\,min} = 324\ mm^2$, $A_s < A_{s\,min}$ → Use $A_{s\,min}$. → عادي استخدمت هنا $A_{s_{min}}$ لأنه
$A_s = 324\ mm^2$. (No iterations). هي Tension الحالة ما في داعي
اسوي check على ε_s !

* Spacing :

$S = \frac{1000 \times 199}{324} = 614$mm ≈ 600 mm.

$S_{max} = 450$mm > S → Use S_{max}.

$S = 450$ mm.

* Per ACI-code : Shrinkage & Temprature reinforcement ($A_{s\,min}$) is required per-pendicular to the span of the slab
  ↳ (long-direction)

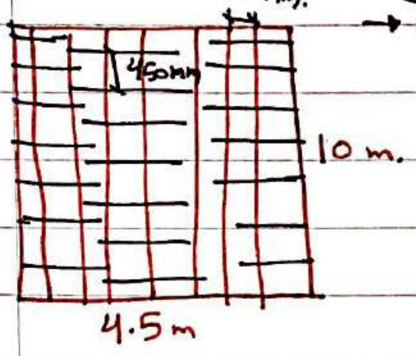
$A_{s\,min} = 324\ mm^2$. بنهتم بالحديد الطولي (بالاتجاه الطويل)، وبنهتم بالتشققات

$S = \frac{1000 \times 199}{324} = 614$mm. الموجودة في البلاطة، بسبب صغر قيمة h، نحط حديد طولي

عبار Asmin على بعد S .

Shrinkage & Temp. ← $S_{max} = \text{Smaller of} \begin{cases} 5h \\ 450\ mm. \end{cases}$

$5h = 5(180) = 900$ mm. / $S_{max} = 450^{mm} > S$, use $S = 450$ mm.



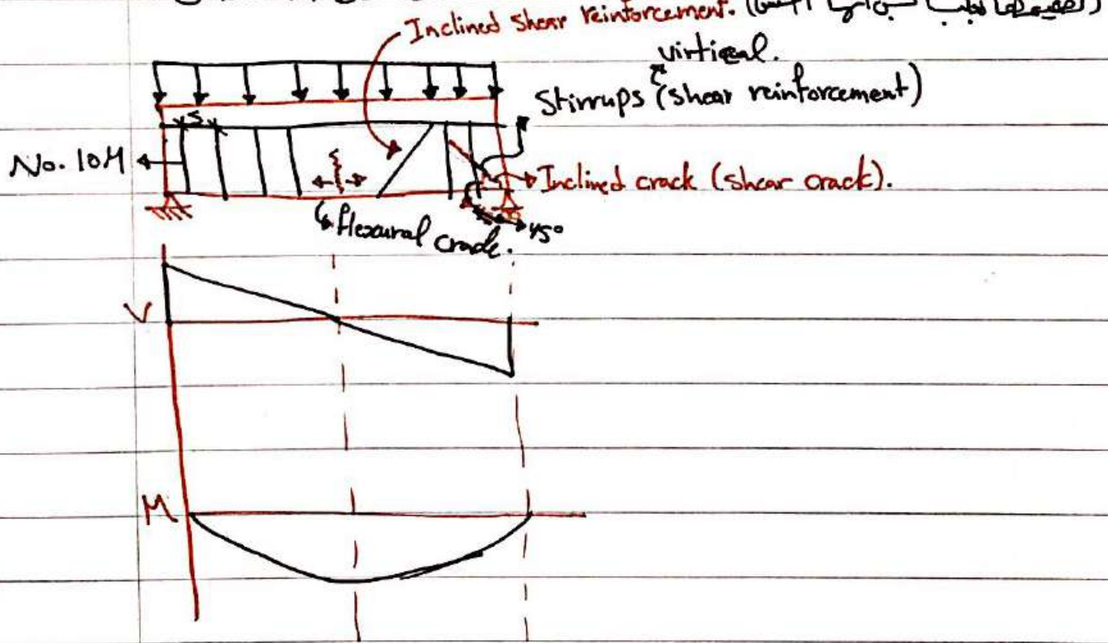
→ Top view of slab.

* عشان ما الكلف يستوي مصاري كثيرة لأني زودت كثير، فروح أصغر حجم الحديد.

No.13M ; $A_b = 129\ mm^2$.

$S = \frac{1000 * 129}{324} = 398\ mm.$

* Shear in Beams:



*Internal forces in a beam:

$V_n = V_c + V_s$ → Shear carried by stirrups.
↳ Shear carried by the conc.

→ dowel action of longituded steel

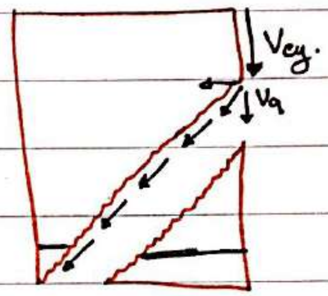
$V_c = V_{cy} + V_{ay} + V_d$

Shear in the uncracked conc. section. { aggregate intercarck on both force of the crack.

$V_c = \frac{\sqrt{f'c}}{6} b_w d.$

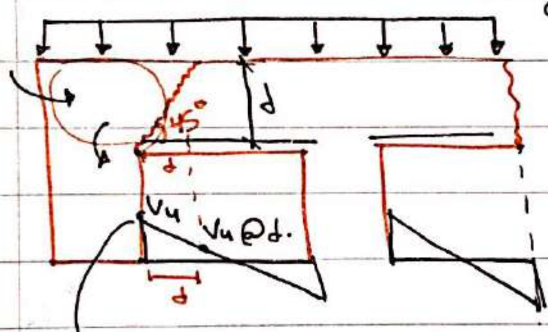
$\underline{\phi} V_n \geq V_u$

$\llcorner 0.75$



* Location of max. Shear for design of RC Beams (V_n) [Critical section].

Compression fan.



Vu @ face of support.

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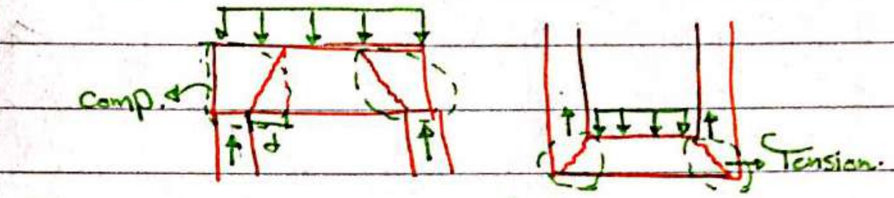
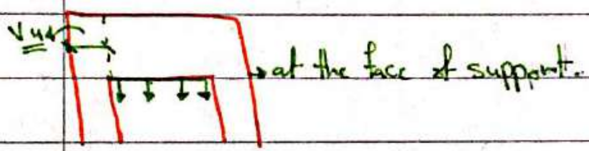
$\phi V_n \geq V_u$

- @ face.
- @ d from face of support: use only if:

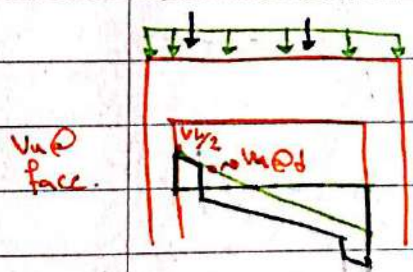
1] Support reaction introduces compression into the end region of the beam.



2] loads are applied on the top of the beam:



3] No concentrated force within a distance d from face of support.



* Analysis & Design of R.C. Beams for Shear.

$\phi V_n \geq V_u$.

$V_n = V_c + V_s$.

$V_c = \frac{1}{6}\sqrt{f'_c}\, b_w d$.

$\phi V_n = V_u$.

$\phi V_n = \phi(V_c + V_s) = V_u$.

$V_s = \frac{V_u}{\phi} - V_c$.

Case I: $V_u \leq \frac{\phi V_c}{2}$

No need for shear reinforcement.

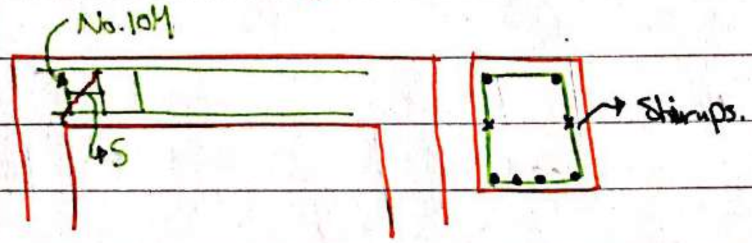
Case II: $\frac{\phi V_c}{2} < V_u \leq \phi V_c$.

Use minimum shear reinforcement.

S_{max} = smaller of $\begin{cases} \frac{16 A_v f_y}{\sqrt{f'_c}\, b_w} \\ A_v f_y / (0.33 b_w) \\ d/2 \\ 600\text{mm} \end{cases}$

$A_v f_y / (0.33 b_w)$ → موجود بنظام.

$d/2$ → عشان الكانة تآخد الـ main shear



$A_v = 2A_{v1}$

↳ Cross sectional area for one bar (stirrups).

Case III: $V_u > \phi V_c$; $V_s \leq \underbrace{\frac{2}{3}\sqrt{f'_c}\, b_w d}_{4V_c}$.

S_{max} = smaller of: $\begin{cases} \frac{A_v f_y d}{V_s} \\ \frac{16 A_v f_y}{\sqrt{f'_c}\, b_w} \\ \frac{A_v f_y}{0.33 b_w} \end{cases}$ → min →

هدول بيجيهم بالامتحان.

وهدول خاصين بـ case II

$V_s \leq \frac{1}{3}\sqrt{f'_c}\, b_w d$ · $\begin{cases} d/2 \\ 600\,\text{mm} \end{cases}$

$V_s > \underbrace{\frac{1}{3}\sqrt{f'_c}\, b_w d}_{2V_c}$ · $\begin{cases} d/4 \\ 300\,\text{mm} \end{cases}$

If $V_s > 4V_c$

↳ Enlarge the section.

$V_{s\,max} = 4V_c = \frac{2}{3}\sqrt{f'_c}\, b_w d$.

$M_u = C_m W_u l_n^2$. لما تكون L طويلة، المومنت رح يتحكم بالديزاين.

$V_u = C_v W_u \, l_n/2$. وإذا كانت قصيرة بالدوحات، الشير رح يحكم.

$V_{u\,max} = \phi (V_c + V_{s\,max})$.

$V_{u\,max} = \phi\, 5V_c$ → check بعمل بالبداية بحيث اكون الـ conc كافية أو لا ؟

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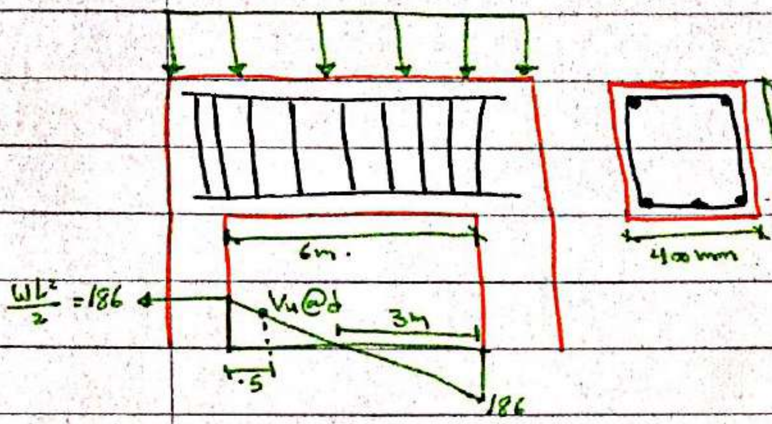
Example:

$LL = 20\,kN/m$, $DL = 25\,kN/m$.

$f'_c = 28\,MPa$.

$f_y = 300\,MPa$.



$W_u = 1.2\,(25) + (1.6)(20) = 62\,kN/m$.

$V_u @ d = 155\,kN$ ..شبه كذا..

$V_c = \frac{1}{6}\sqrt{28}\,\frac{(400)(500)}{1000} = 176.4\,kN$.

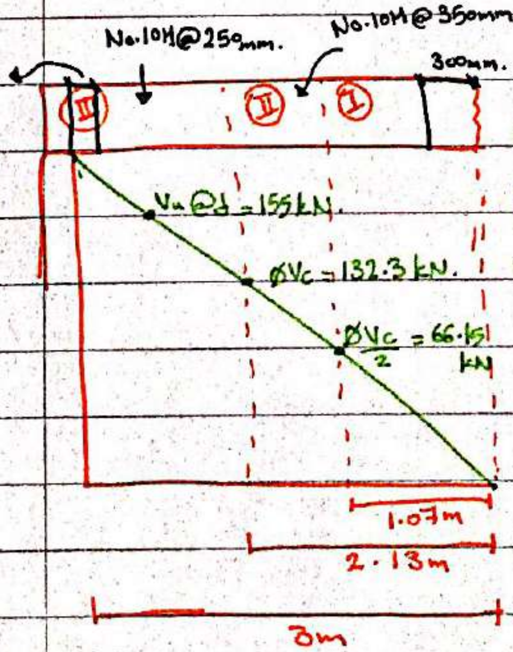
$V_{u\,max} = (.75)(5)(176.4) = 661.4\,kN$.

$155 < V_{u\,max}$ (Section is ok ✓).

10cm or $\frac{S}{2}$

وأي واحد فيهم

عادي !!.



$\phi V_c = .75(176.4) = 132.3\,kN$.

$\frac{\phi V_c}{2} = 66.15\,kN$.

$V_s = \frac{V_u @ d}{\phi} - V_c$

$= \frac{155}{.75} - 176.4 = 30.3\,kN$.

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Case III : $V_u > \phi V_c$ & $V_s < 4V_c$ // $V_s < 2V_c$.

S_{max} = Smaller of :

$$\frac{A_v f_y d}{V_s} = \frac{157 * 300 * 600}{30.3 * 10^3} = 777\,mm.$$

case II

$$\frac{16\, A_v f_y}{\sqrt{f'_c}\; b_w} = \frac{16(157)(300)}{\sqrt{28}\;(400)} = 356\,mm.$$

$$\frac{A_v f_y}{.33\, b_w} = \frac{157\,(300)}{(.33)\,(400)} = 356.8\,mm.$$

$$d/2 = \frac{600}{2} = 250\,mm. \checkmark$$

600 mm.

S = 250 mm.

A_v = 2 No. 10M = 157 mm².

Case II : $\frac{\phi V_c}{2} < V_u < \phi V_c$.

↳ Minimum shear reinforcement.

356 mm.

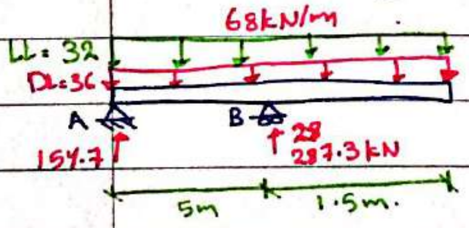
356.8 mm.

250 mm. ✓

600 mm.

S = 250 mm.

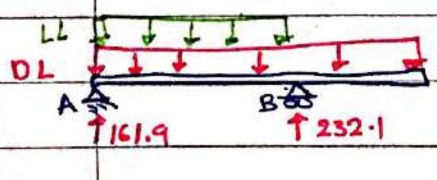
(مطلوب بالامتحان). * Cases of Loading :

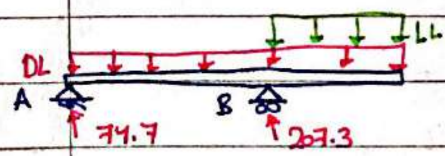


DL = 30 kN/m × 1.2 = 36 kN/m.

LL = 20 × 1.6 = 32 KN/m.

LL + DL = 68 kN/m.





* Columns: Combined Axial Load & Bending:

e = eccentricity.

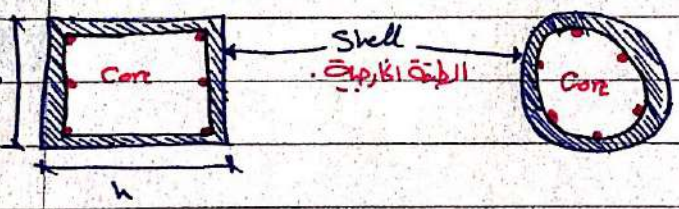
* Columns:

1] Short. 2] Slender.

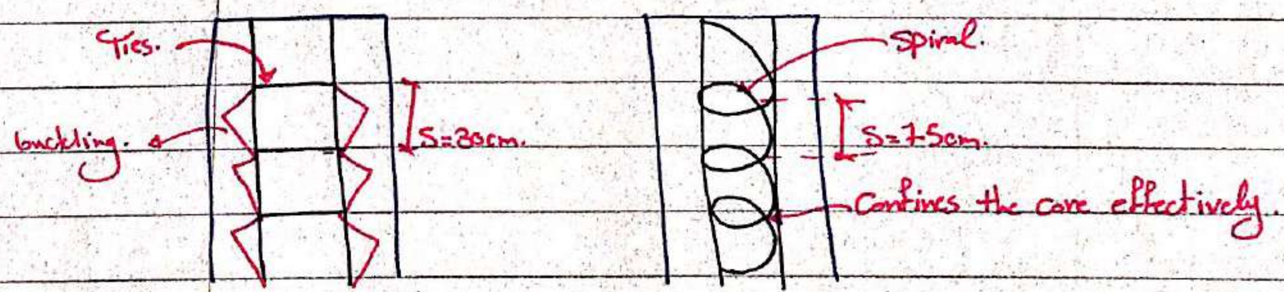
lateral deflection.
(Buckling).

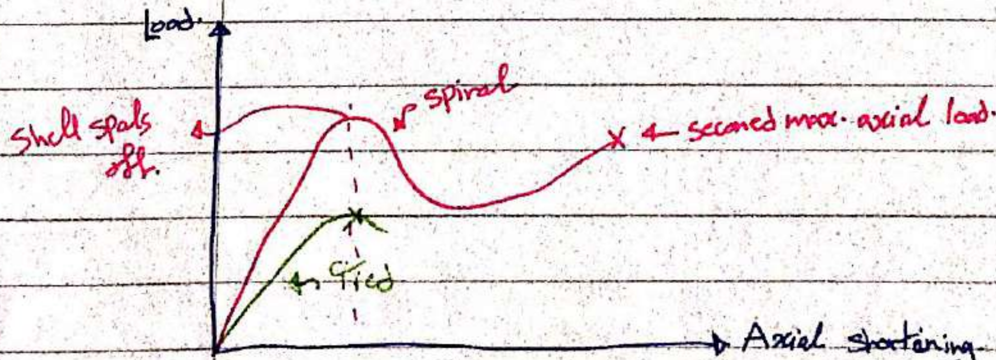
* Types of columns:

1] Tide Column. (Ø = .65) 2] Spiral Column. (Ø = .75).



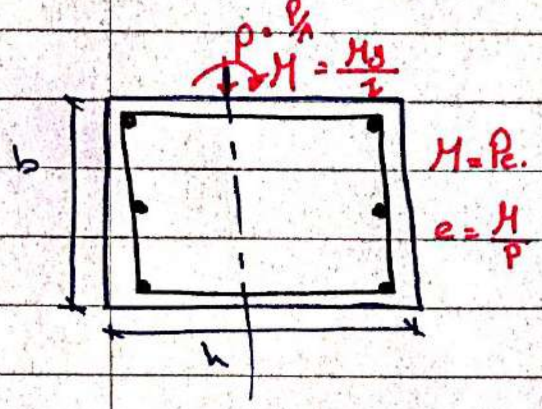
[95% of clums are tied].





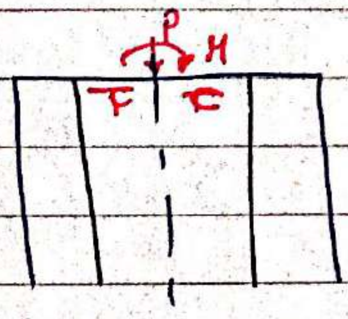
Ø → failure بحددها نوع ال

* Interaction Diagram: (I.D).



$M = Pe.$

$e = \frac{M}{P}$



$\frac{P}{A} + \frac{My}{I}$

$\text{comp.} = \frac{P}{A} + \frac{My}{I} = \underbrace{f_{cu}}$ → compression strength.

$\frac{P}{f_{cu} A} + \frac{My}{f_{cu} I} = 1.$

IF $M=0 \rightarrow \frac{P}{f_{cu} A} = 1 \rightarrow P = \overset{\frac{P_{max}}{A}}{f_{cu}} A = P_{max}.$

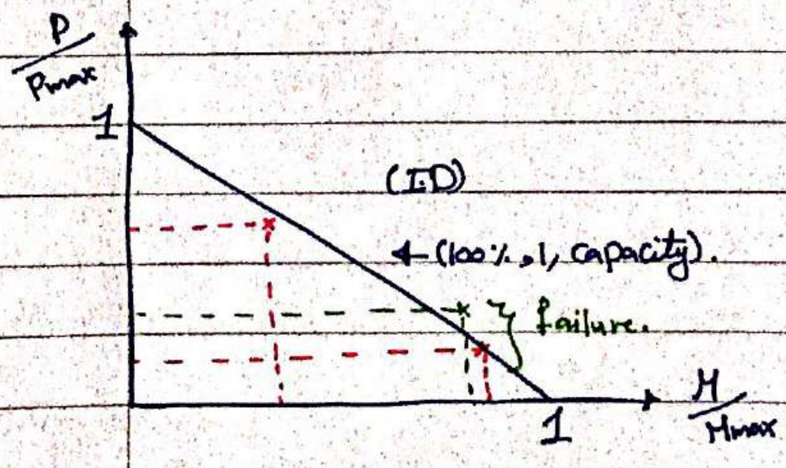
↳ Maximum axial load the moment can support.

IF $P=0 \rightarrow \frac{My}{I f_{cu}} = 1 \rightarrow M = \frac{\overset{\frac{M_{max} y}{I}}{f_{cu}} I}{y} = M_{max}.$

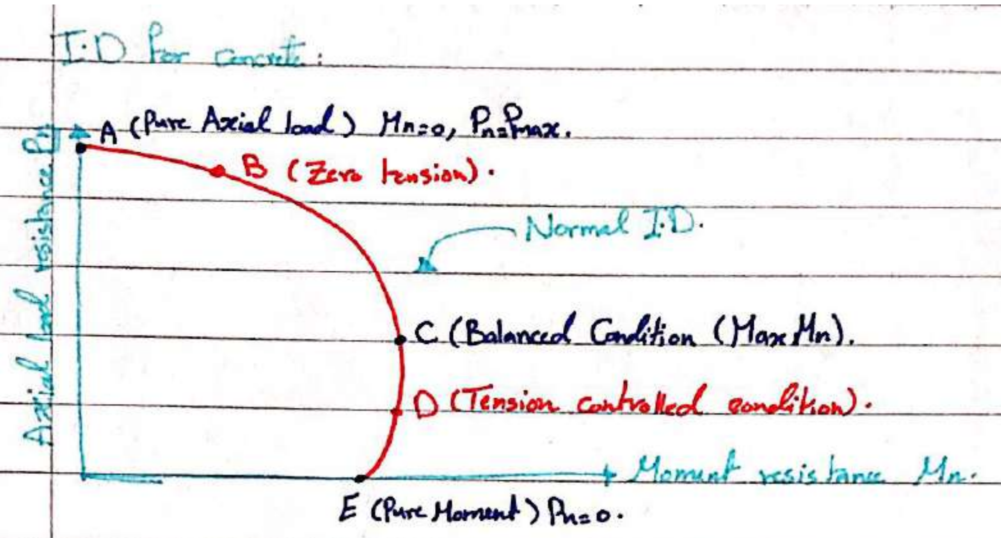
↳ Maximum moment the columns can support.

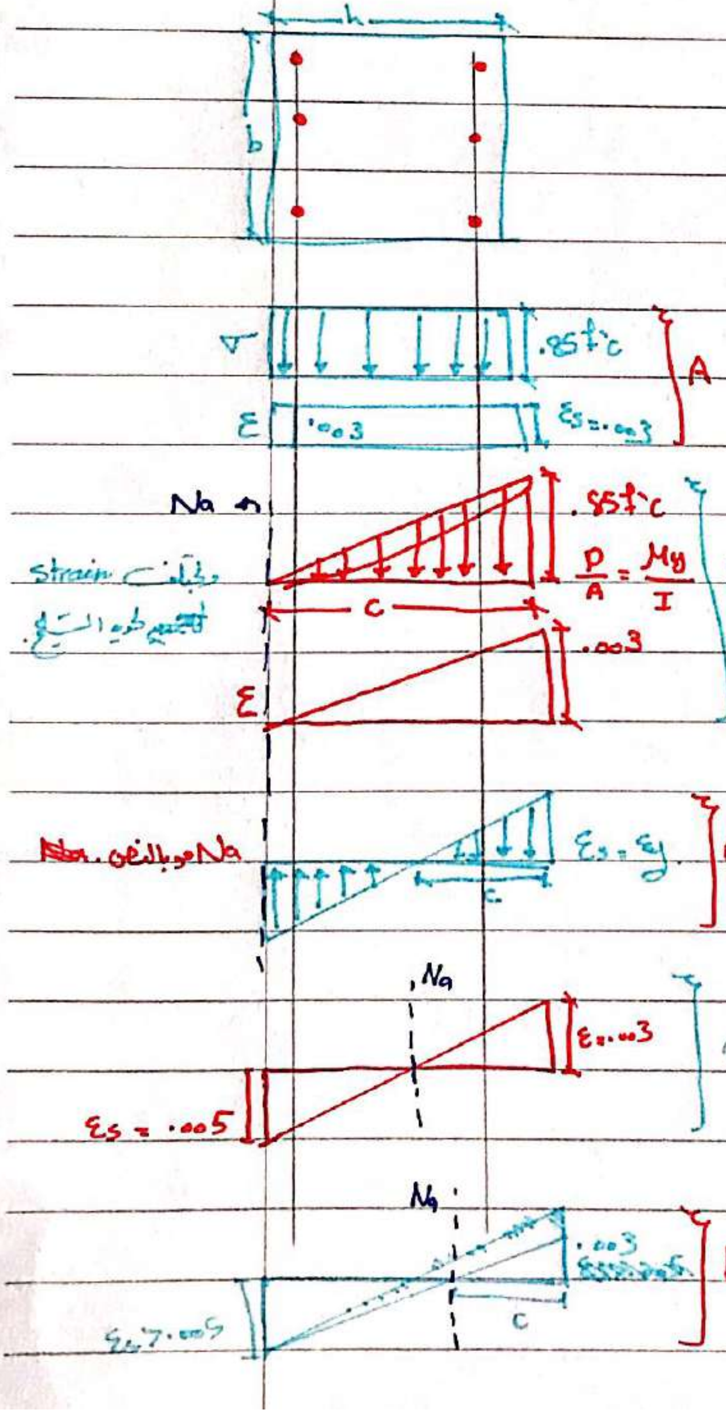
$\frac{P}{\frac{P_{max}}{A} \cdot A} + \frac{My}{\frac{M_{max} y}{I} \cdot I} = 1$

$\frac{P}{P_{max}} + \frac{M}{M_{max}} = 1$ (Interaction Equation).

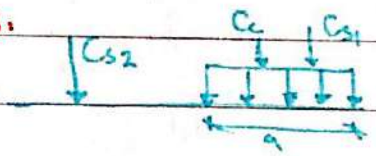


I.D for concrete:





at B.



Point A:

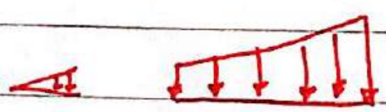
* بعد هاي النقطة بيدا (Zero tension)

$[P_n = .85f'_c(A_g - A_s) + A_s f_y]$

* A_g: Cross sectional area (b x h).

* اثناء اتجاهنا للنقطة B حيصير هالشكل:

. Zero حتى يصير



at Point B:

$c = h$; $P_n = C_c + C_{s1} + C_{s2}$.

at Point C:

$\varepsilon_{s1} = \varepsilon_y$; $P_n = C_c + C_{s2} - T_{s1}$

at Point D:

$\varepsilon_{s_1} = .005$

at Point E:

Like a beam.

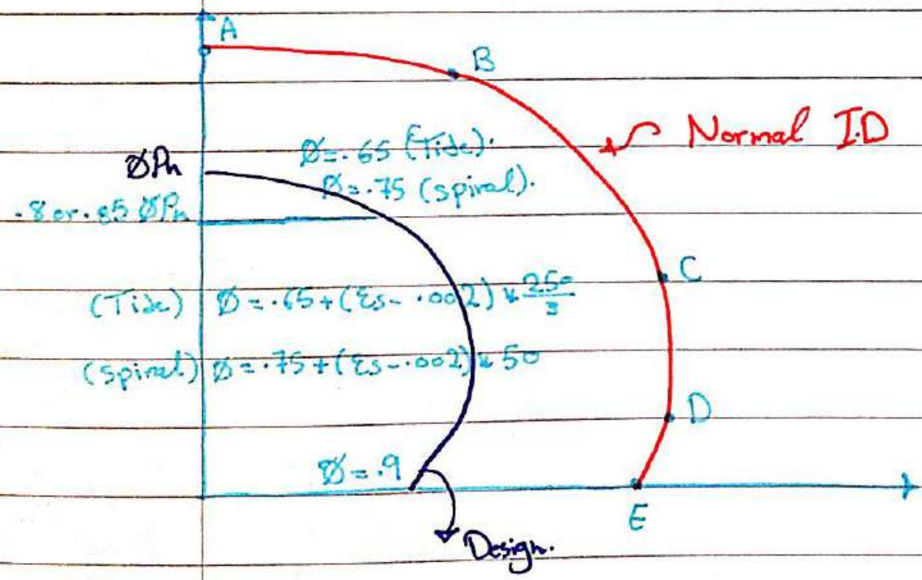
* Maximum Axial Load:

$\phi P_n = \phi P_{n\,max} = \phi\,[.85 f'_c\,(A_g - A_s) + A_s f_y]$.

* In Order to account the effect of accidental Moments:

The ACI specifies that the max load on:

- On a spiral column must not exceeds 85% of it's strength.
  $(\phi P_{n\,max} * .85)$
- On a tide column must not exceeds 80% of it's strength.
  $(.8\,\phi P_{n\,max})$.



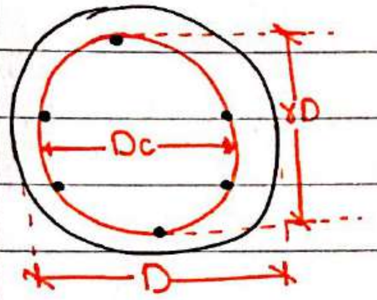
* الرسم غير دقيق،

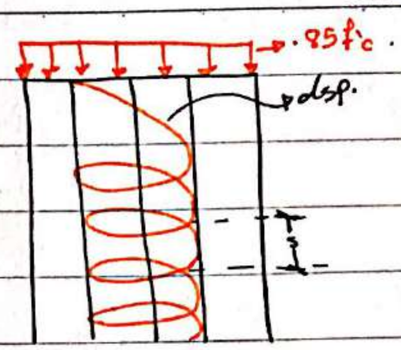
والمفروض عند النقطة E تكون المسافة قريبة بين الخطين لأن $\phi = .9$.

* والتصميم خليّنا بيها للـ Tide من $(0.8\,\phi P_n)$ والـ spiral $(.85\,\phi P_n)$، مثل ما هو موضح بالاشكال ..؟

« هنا ,إلي .. Sheet »

DATE. 23/4/2018





$S_{max} \leq \frac{\pi\, d_{sp}^2\, f_y}{.45\, D_c\, f'_c \left(\frac{A_g}{A_c} - 1\right)}$

$A_g \rightarrow \frac{\pi}{4} D^2$

$D_c \rightarrow$ Core diamiter (out to out of spiral).

$A_c \rightarrow \frac{\pi}{4} D_c^2$.

$S_{max} \leq 75$ mm.

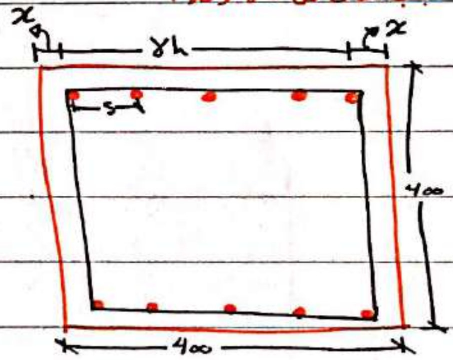
S_{min} = larger of:

$\rightarrow 25^{mm}$ $\rightarrow$ 1.33 max. size of CA.

* $S_{max} \rightarrow$ To confine the core effectively.
* $S_{min} \rightarrow$ To avoid problems in conc. placing.

* Design of tide column:

حسب ال S من γh و x.



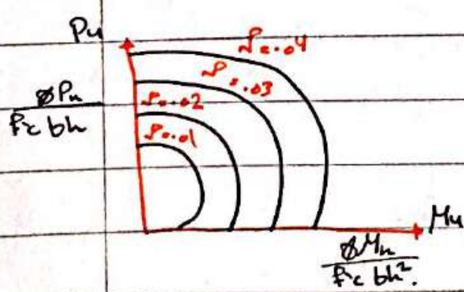
$P_u = 1550$ kN ; $M_u = 150$ kN.m.

$f'_c = 20$ MPa ; $f_y = 420$ MPa.

$\underset{b \times h}{A_g} \geq \frac{P_u}{.4(f'_c + \rho f_y)} = \frac{1550 * 10^3}{.4(20 + .015 * 420)}$

$A_g = 147338\ mm^2$ (Sequare col. $b=h$)

$b = h = 384$ mm; use 400×400 mm.



في رسمة بالتلخيص اوضح

ورسمنا بتطلع الرسم..

I.D: $\gamma = .6 \rightarrow \rho = .033$

I.D: $\gamma = .75 \rightarrow \rho = .028$

use No. 25M;

$x = 40 + 10 + \frac{25}{2} = 62.5$ mm.

$\gamma h + 2x = 400$

$\gamma = .69$

by iterations: $\rho = .03$

* If ρ exceeds 3-4% → large section should be chosen.

* $\rho_{min} = .01$

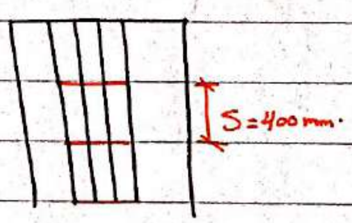
$\rho = .03 \rightarrow$ ok ✓ ; $\rho > \rho_{min} \rightarrow$ ok ✓

$A_s = \rho b h = .03 * 400 * 400 = 4800\, mm^2$ (required).

10 No. 25M; $A_s = 5100\, mm^2$

* Spacing for ties:

Smaller of: $\begin{cases} 16 d_p = 16 * 25 = 400 mm ✓ \\ 48 d_t = 48 * 10 = 480 mm \\ b = 400 mm \end{cases}$



S_{min} = larger of: $\begin{cases} 40 mm \\ 1.5 d_p = 1.5 * 25 = 37.5 mm \end{cases}$

$S_{min} \leq 150 mm$.

* ممكن يجيب مثال على ال spiral.

* Development, Anchorage & splicing of Reinforcement.

* Development length (L_d): Is the shortest length of bar in wich the bar stress can increase from zero to f_y.

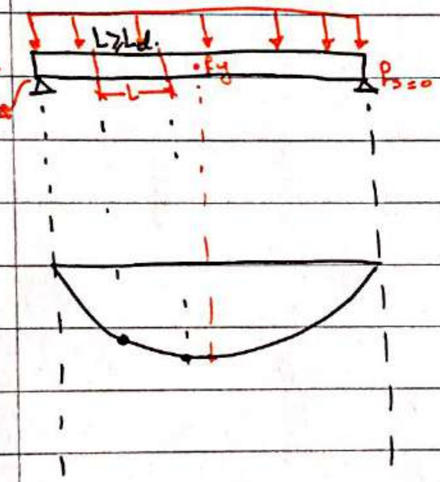
* If the distance from a point where the bar stress equals f_y to the end of the bar is less than (L_d), the bar will pull out of the concrete

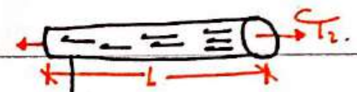
↳ Not enough bond stress to produce Equilibrium.

* لأنها نهاية قضيب الحديد وبدايته

$f_s = Zero.$

$f_y = M_{max}$ is





↳ μ_{avg} = Avg. bond stress.

$$\underset{\text{لأنها باتجاه } T_1}{T_1 +} \mu_{avg} \underbrace{\pi d_b L}_{\text{محيط بالطول}} = T_2.$$

$T_1 \neq T_2$ → لا يوجد مختلفة..

* الـ equ بجي من الاحتكاك بين الباطون والحديد.

* لازم يكون القضيب طول معين يحقق الـ bond عشان يصير عندي equilibrium.

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Scanned with CamScanner

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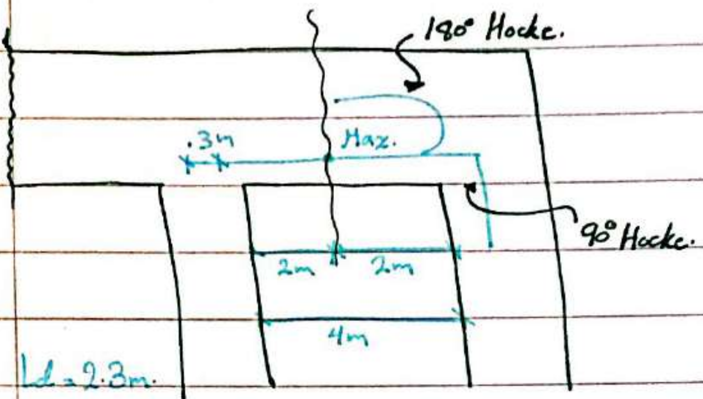
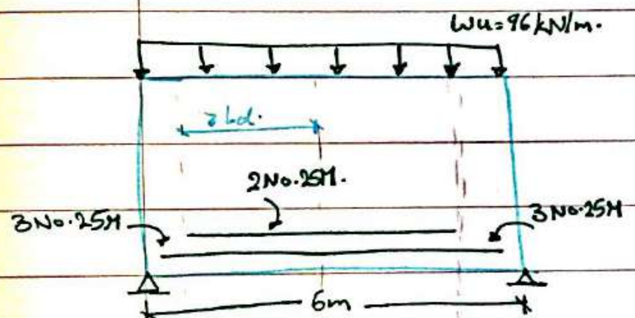
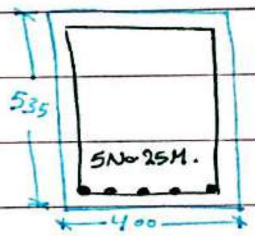
Hooks & Anchorage



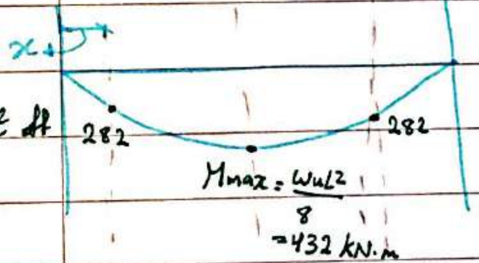
*Example : (Bar cut off).





$\phi M_n = 440\ kN.m.$

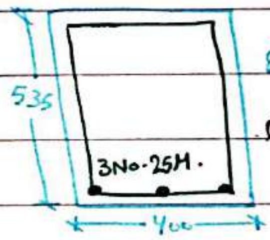
الكمرة حاليا طالع Mu



$Ld \leq 3m.$

theoritical cut off Point.

$M_{max} = \frac{W_u L^2}{8} = 432\ kN.m$

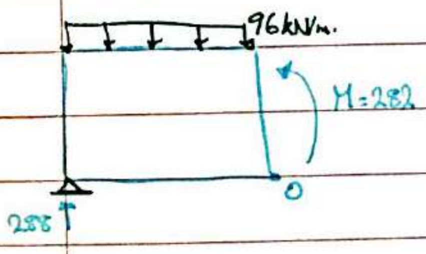


$\phi M_n = 282 kN.m.$

نفرض هاد الشكل ونطلع الـ ØMn.

وبعدها كل الي كتبته

*cut off two bars.



$\Sigma M_o = 0 \ ; \ x = 1.23m.$

use $x = 1.25m.$