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دفتر

خرسانة مسلحة1

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RC1

لطفي المصباركي

Ch1

هذا الملخص يشمل دفتر الدكتور بلال ابو الفول .

* Advantages of concrete:-

1. Relatively low cost materials.
2. Fire resistance (1-3 hrs) without special fire proofing.
3. Suitablity of materials for ~~arch~~ architectural and structral functi.

له مقارنة مع الحديد فإن الحديد يحتاج لطلب خاص إذا احتجت لشكل معين .

4. Rigidity
5. low maintenance. صيانة اقل
6. Availablity of materials.

* Dis advantages :-

1. low tensile strength. قوة الشد
2. form and shoring. (تدعيم الطوبار وفكه يحتاج لوقت)
3. Relatively low strength per unit length or volume.
4. Time dependent volume changes,

Creep (take along time)

Drying shrinkage (non-load)

* Important Of Steel:-

معامل التمدد الحراري

* Concrete and steel have nearly or equal coeficient of thermal expansi
* Steel has good bond with concrete (almost perfect)
* good dense concrete protect steel from rusting (الصدأ).

له إذا حدث الصدأ سيعمل على تقليل الرابطه بين الحديد و الخرسانه .

* We mean by reinforcement of concrete:- a combination of concrete and steel that provide the tensile strength lacking in the concrete.

(الزياده او تعويض ضعف الخرسانه بقوة الشد) بزيادة

* Sources of uncertainty:- معادلات التصميم SF مصادر جعلتنا نفرض

1. actual load magnitude and distribution may differ from those assumed in the design.
2. assumptions and simplifications in the analysis may result in diffrent internal f
3. Actual behavior maybe diffrent.
4. Actual member dimensions may differ from those specified in the design.
5. Reinforcement may not be in its proper position.
6. Actual material strength may be diffrent from the specified in the design.

* Safety philosophy:-

$$\underline{\emptyset S_n} \geqslant \underline{\gamma Q_d}$$

$\emptyset S_n$ ↓ reduction nominal Strength

γQ_d ↓ ultimate load (fatorial load)

$\emptyset$:- Strength reduction factor ≤ 1

S_n :- nominal Strength

γ :- over load factor, most of time $\gamma > 1.0$ but it can be < 1.0 in combination action variable loads.

Q_d :- Design load (actual load or unfaltorial load)

⊛ load factors and combinations per ACI-code:- → هتاجهم في التصميم (مادة السكند)

* $\underset{\text{ultimate load}}{U} = 1.2DL + 1.6LL$ → principle variable load.

or

* $U = \underbrace{0.9}_{\text{compination action variable load}} DL + 1.6WL + 1.6HL$ *

DL :- Dead load like Structural weight .....

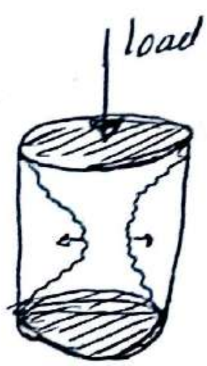
LL :- live load , WL : wind load HL: lateral load.

⊛ ch2 :- Materials

* Development of microcracking and failure in concrete subjected to Unaxlal compressive loading:

مراحل حدوث الكسر
عند تعريض اسطوانة الخرسانة لقوة عمودية :-

$\sigma = \frac{P}{A}$

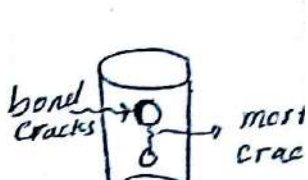


* Stages:-

1- shrinkage of paste during hydration ⇒ bond cracks or non-load bond cracks) ⇒

2- Stress 30%-40% of compressiv strength
⟹ bond cracks (about aggregate surface)
⟶ Discontinuity limit.

3- Stress 50%-60% of compressive Strength
⟹ mortar cracks. (between aggregates)



Stable

⟶ critical Stress.

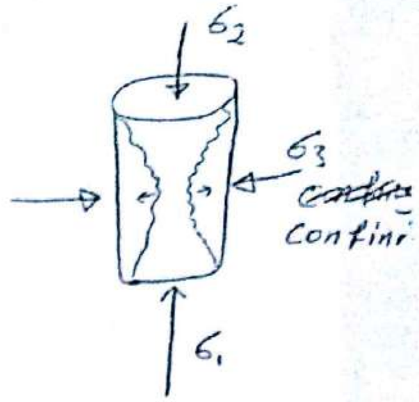
④ At (75%-80%) of ultimate load → no. of mortar cracks increase
⇒ fewer undamaged portions to carry the load.

Unstable Crack propagation

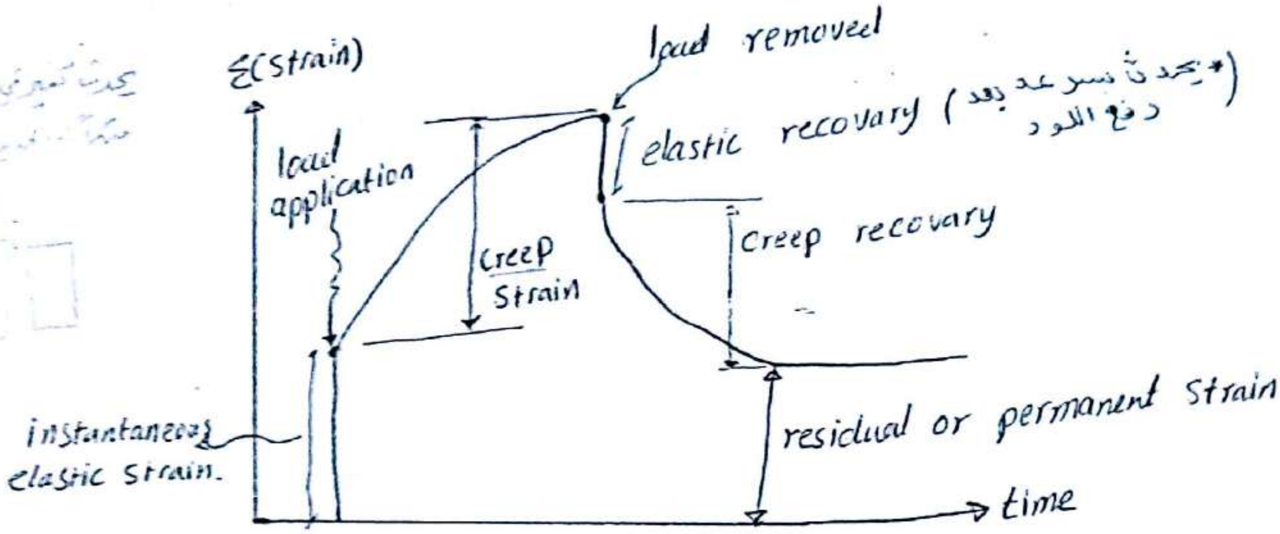
له حتى لو اوقفت القوه المؤثره بعد هذه المرحله لن تتوقف التشققات.

• ⊗ triaxial load "confining stress"

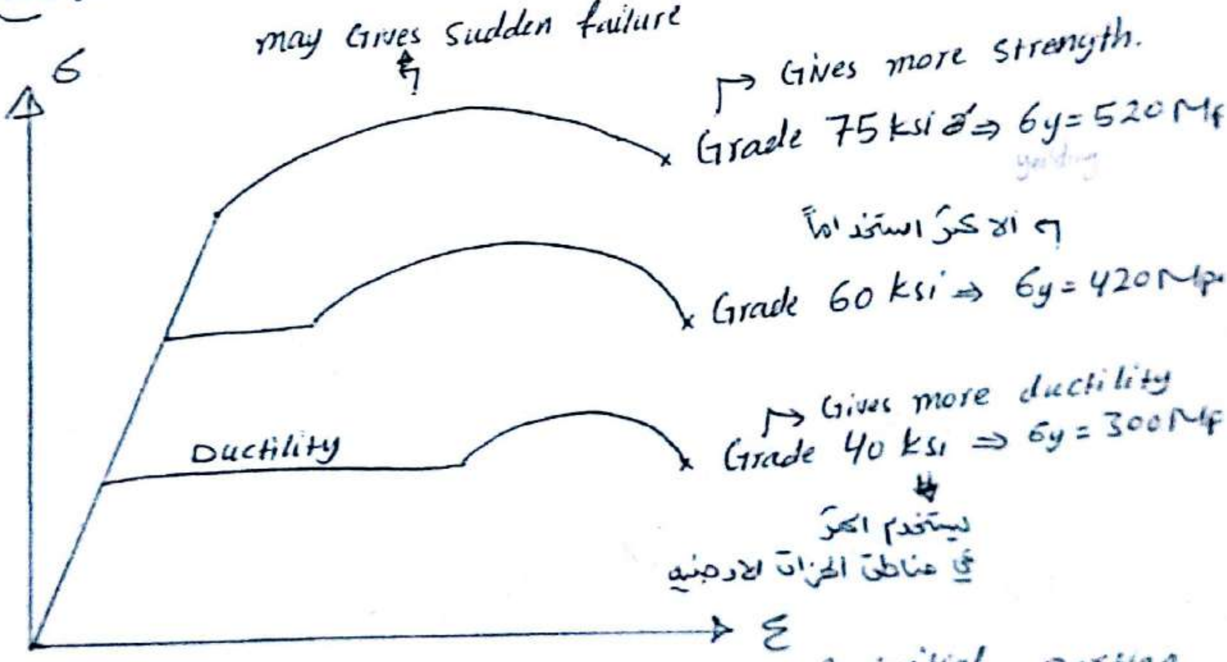
• Gives more stress than unaxial because confining resist indirect stress that try to ~~to~~ spill the sample.



⊗ Creep test:- (depend in time)



* Reinforcement:-



* All graded have the same slope of initial portion of 6-ε diagram above. (Elastic)

CH3 The Design process:-

* Objectives of design:- Structure should Satisfy:-

① appropriateness:- "designed to serve its intend use"
ملائم

② Economy:- Overall cost $\leq$ client's budget.

كفاءه
③ Structure adequacy:- Structure must be strong enough to s anticipated load and not diflect, vibrate or tilt...
المتوقعه ميلان

④ maintainablity:- صيانه
minimum & simple.

* The Design process:-

① phase 1:- define client's need and priorities.

② phase 2:- ~~Def~~ Development of project concepts:-
- ↳ No. of possible layouts. (2D projects)
- ↳ prelimentary cost estimate. عرض تكلفة المشروع.

③ phase 3:- Design of individual systyems.

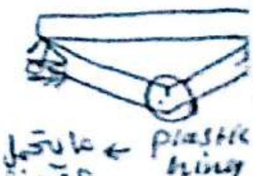
* limit States and design of RC:-

When astructure or an element becomes unfit for its intend use it said to have reached a limit state. غير قادر على استيفاء الغرض.

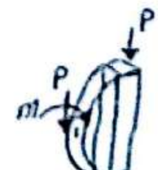
* Groups of limit States:-

① Ultimate limit State:- involves structure Collapse of part or all the Structure

Examples:-
- loss of equilibrium
- rapture of critical parts.
- prograssive callapse. ← الانهيار المتتالي كانهيار طابق فوق طابق اخر اسفل منه.
- formation of plastic mechanism



- lateral deformation (Buckling)



- Fatigue (repeated loads)

② **Serviceablity limit State:-**

involves disruption (إضطراب) of the functional use of the Structure. (but not collapse)

Ex:- - opening cracks
- undesirable vibration.

مثل سكة الحديد لو يحدث له انحناء بيبطل القطار يمشي عليها ولكنها لم تنهار.

③ **Special limit State:-**

involve damage or failure due to abnormal conditions

Ex:- - fir or explosion
- extreme earthquicks

ـه وهذا لايعني الزلازل المتوقع حدوثها.

⊛ **limit States design process:-**

⊛ Structural Safety:-

① Sources of uncertainty

② consequences of failure:-
- loss of life
- cost of clearing debris.
- cost in society in lost time

*** Design procedures specified in the ACI-code**

① Stregth design: $\phi S_n \geq \gamma Q_d$

② Working Stress design

Working loads (Structure loads) $\phi S_n \geq Q_d$ ← unfactorial load.

③ plastic Design method, limit design, capacity design.

*** loading And Actions**

- Parmenant
  - Ex:- the weight of the Structure
- Accidental
  - Ex:- - explosions
  - - Collisions
  - - Vehicular
- Variable
  - Short time
    - Ex:- people
  - long tim
    - Ex:- Sustained load
    - مثل الاثاث

* CH4:- Flexure Basic concept (Rectangular Beams).

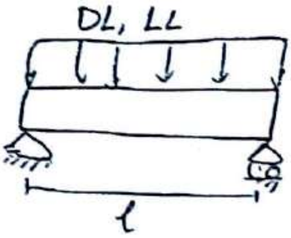
لم يتعرض عال Bending moment.

Beams ⇒ flexure & shear

*** Basic Safty equ.:-**

$\phi M_n \geq M_u$

- ϕ → Strength reduction fator
- M_n → nominal moment capacity
- ϕM_n → Design moment capacity
- M_u → Required. ultimate moment (faktorial loads)



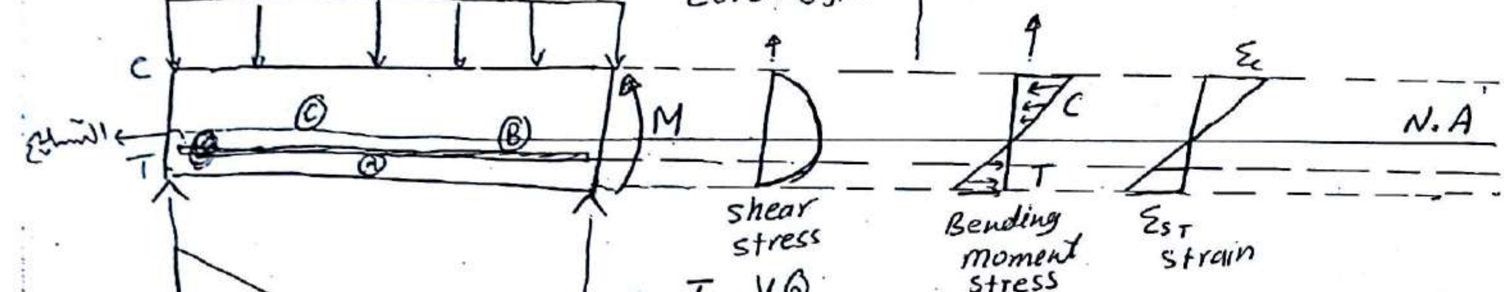
$W_u = 1.2\,DL + 1.6\,LL$

$M_u = \dfrac{W_u l^2}{8}$ → Simply supported beam.

*** flexure in Beams:-**

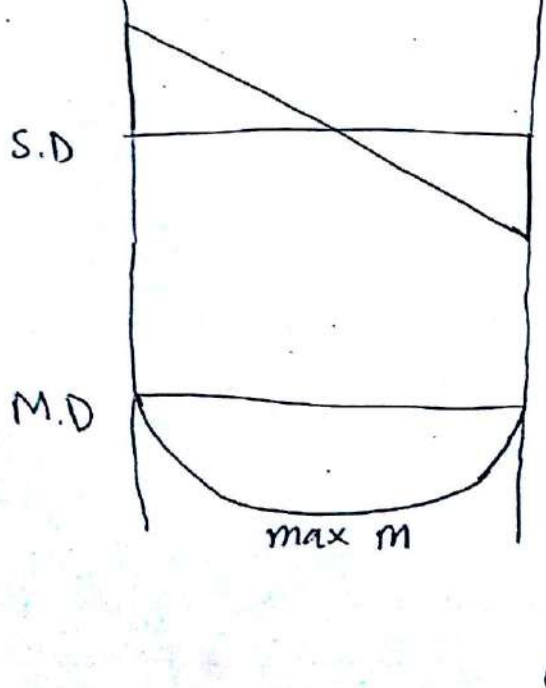
max: (N.A) عل | max بالأطراف

Zero: عالأطراف | Zero → N.A عال



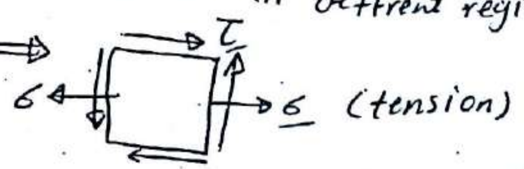
$V \rightarrow \tau = \dfrac{VQ}{Ib}$

$M \rightarrow \sigma = \dfrac{My}{I}$

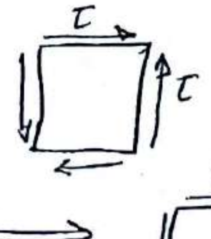


Element of the Beam in deffrent region

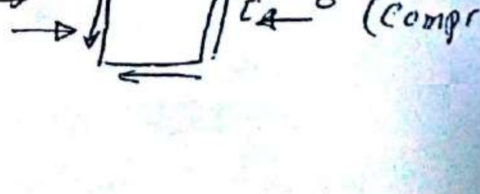
Ⓐ under N.A ⇒



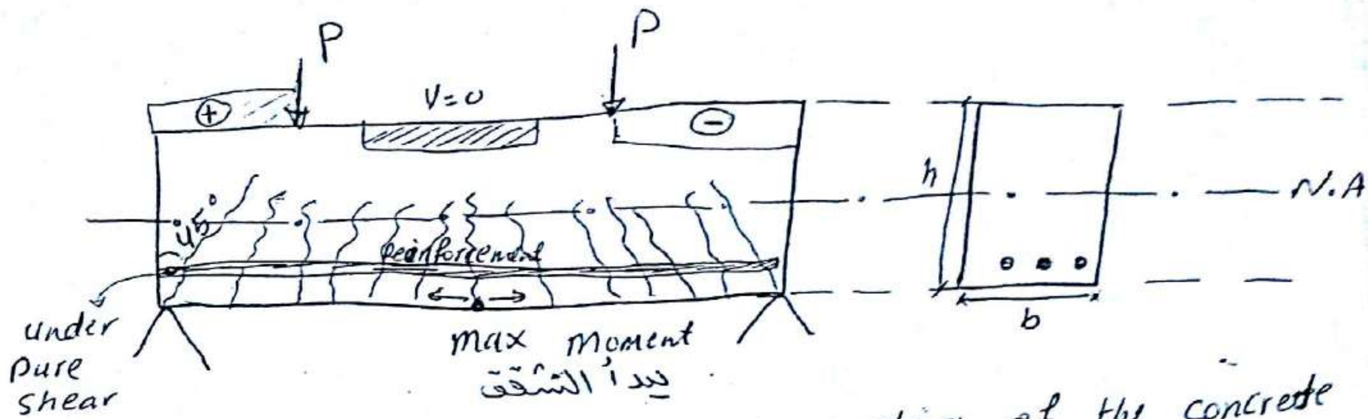
Ⓑ in N.A ⇒



Ⓒ Above N.A ⟶

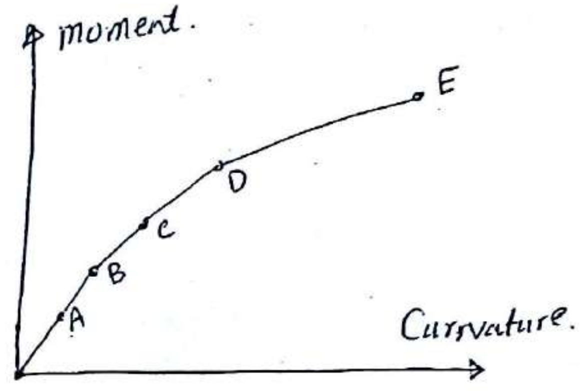


⊕ Flexural Behavior (laborotary testing)

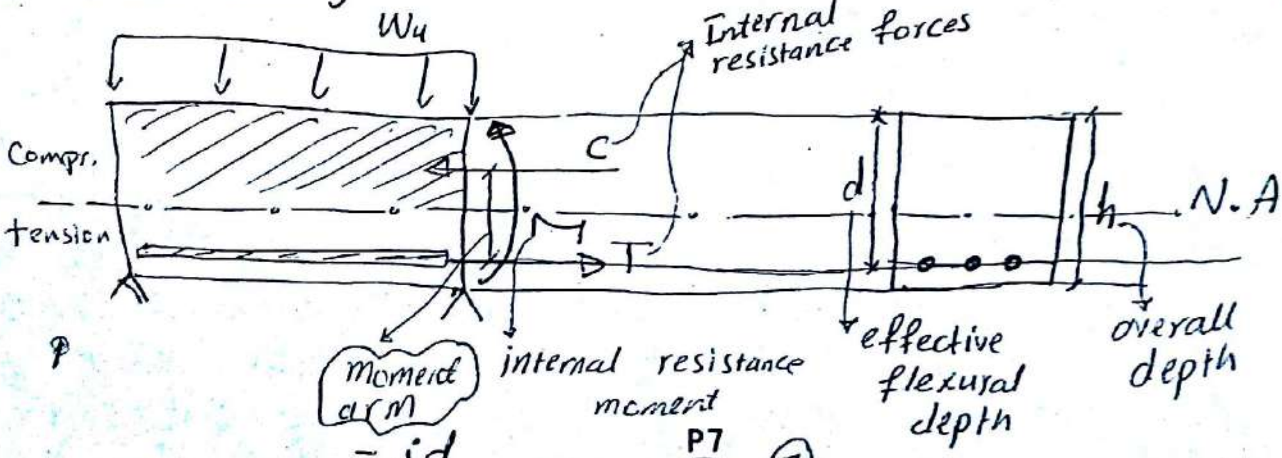


Beams failed as a result of the crushing of the concrete at the top of the beam.

* Stage A: Before cracking
* Stage B: Cracking (Start from the max. moment)
* Stage C :- After cracking and before yeilding of reinforcement. When stress transmit from Concrete to Steel there will be micro cracking
* Stage D :- yeilding of reinforcement
  ⇒ Curveture increases rapidly with very little increase in mome
  * وهذا السبب الذي يجعلنا نسلح المنطقة العليا ايضاً.
* Stage E :- failure

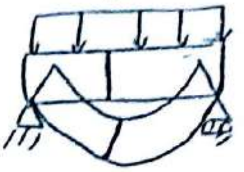


* Flexural theory



* Basic assumptions in flexural theory:-

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

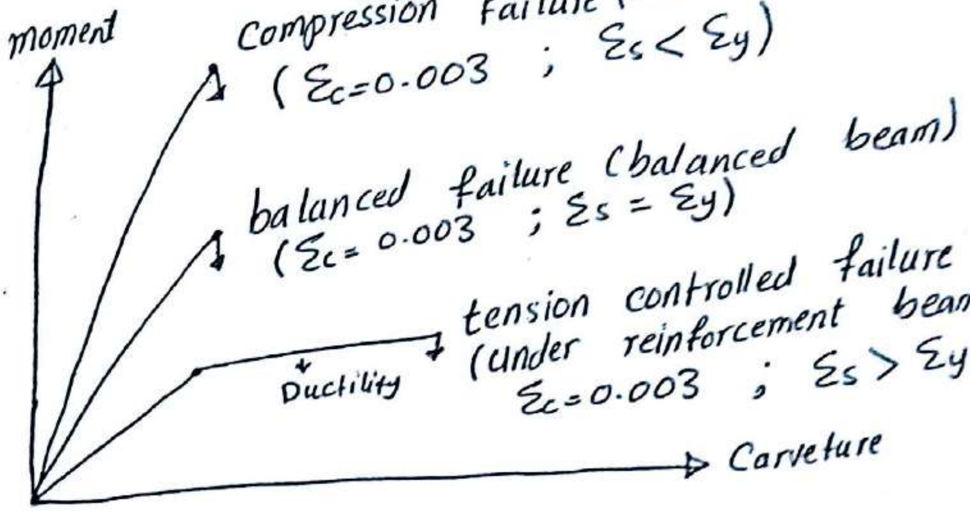


② the strain of the reinforcement is equal to the strain of the Concrete at the same level. (perfect bonding)

③ the tensile strength of the concrete is neglected in flexural calculations. اهمل قوه تحمل الخرسانه للشد في منطقة (T)

④ Concrete is assumed to fail when the maximum compressive strain reaches a limiting value. $\boxed{\varepsilon_{cu} = 0.003}$

* flexural failure may occure in three diffrent ways:-



$\varepsilon_c = \varepsilon_{cu} = 0.003 \longrightarrow$ هذا هو الذي يحدد ان ال Beam وصل الى الانهيار (failure)

ε_s when $\varepsilon_c = 0.003$ ~~sts~~ satisfy the kind of failure.

Note: Over Reinforcement beam means the beam that has less reinforcement than requi[red]

* Strain limits method for analysis and design:-

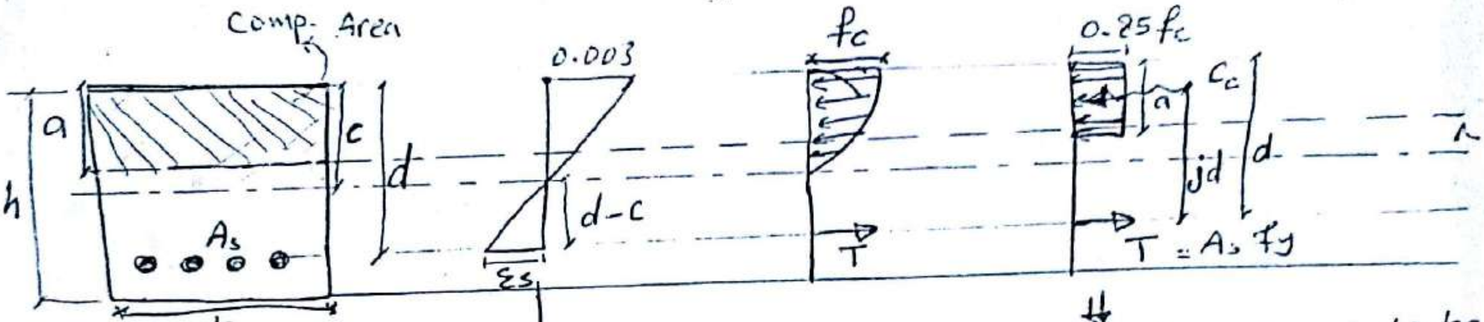
In the ACI-code four types of beams depending on anticipated mo[de] of failure:-

* Compression Controlled beam	* Tranzition controlled beam	* Tension-Controlled beam
compression controlled failure	tranzition controlled failure	tension- Controlled failure

$\varepsilon_s = \varepsilon_y$ $\varepsilon_s = 0.005$

balanced beam

$$F = \frac{P}{A}$$



to find ε_s

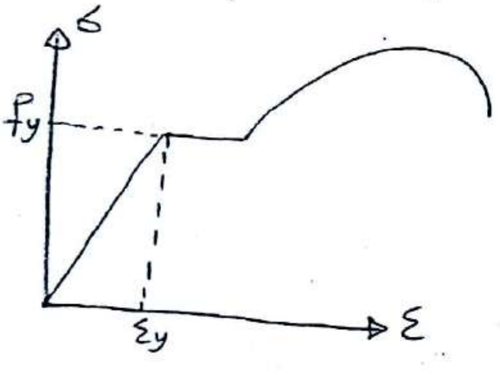
$$\frac{0.003}{c} = \frac{\varepsilon_s}{d-c}$$

$$\Rightarrow \boxed{\varepsilon_s = \frac{0.003(d-c)}{c}}$$

$C_c = 0.85 f_c$ assumed to be Uniform to be easy with Calculations and it Called (equivelant stress block)

Where $\boxed{a = \beta c}$

* $C_c = 0.85 f_c \underbrace{ab}_{\text{comp. area.}}$



from σ-ε diagram we note that:-

1- in the first portion, the steel is not yeitded and we can apply Hox's low

$\Rightarrow \varepsilon_s < \varepsilon_y \Rightarrow \boxed{f_s = E\varepsilon_s}$

2- when $\varepsilon_s \geq \varepsilon_y \Rightarrow \boxed{f_s = f_y}$

* Analysis process:-

Assume $f_s = f_y$ $(\varepsilon_s \geq \varepsilon_y)$

$C_c = T$

$0.85 f_c ab = A_s f_y$

$$\Rightarrow a = \frac{A_s f_y}{0.85 f_c ab}$$

find $c = \frac{a}{\beta}$ then check ε_s

$M_n = T(jd)$ or C_c

$jd = (d - \frac{a}{2}) \rightarrow$ only for rectangular compression area

$M_n = 0.85 f_c ab(d - \frac{a}{2})$

or $= A_s f_y (d - \frac{a}{2})$

* $a = \beta c$

where

$\boxed{\beta = 0.85}$ when $f'_c \leq 28$ Mpa

$\boxed{\beta = 0.85 - 0.05\left(\frac{f'_c - 28}{7}\right)}$; $28 < f'_c \leq 56$

$\boxed{\beta = 0.65}$ when $f'_c > 56$ Mpa

ϕM_n ϕ: strength reduction factor.

① when $\varepsilon_s \geq 0.005$ (tension Controlled)

$\Rightarrow \boxed{\phi = 0.9}$ less reduction

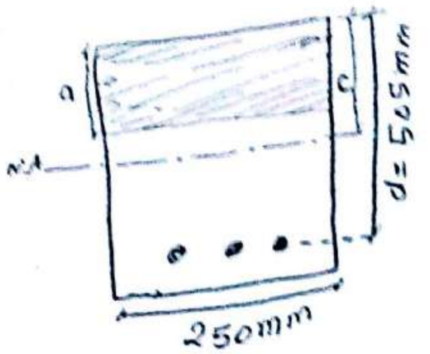
② $\varepsilon_y < \varepsilon_s < 0.005$ (tranzition)

$\Rightarrow \boxed{\phi = 0.65 + (\varepsilon_s - 0.002)\frac{250}{3}}$ ليتحفظ

③ $\varepsilon_s \leq \varepsilon_y$ (Compression and balanced)

$\boxed{\phi = 0.65}$ more reduction

* *Analysis Example (1)*



$A_s = 1530\, mm^2$

$f'_c = 20\, Mpa \quad f_y = 420\, Mpa \quad E = 200\, Gpa$ (for steel)

Des: find design Moment capacity ϕM_n

Sol:-

* comput a

assume $\varepsilon_s \geq \varepsilon_y \Rightarrow f_s = f_y$

$T = C_c$

$A_s f_y = 0.85 f'_c\, a b$

$1530 \times 420 = 0.85(21)\, a \times 250$

$\Rightarrow \boxed{a = 151.2\, mm}$

$\because f'_c \leq 28\, Mpa \Rightarrow \beta = 0.85$

$c = \frac{151.2}{0.85} = \boxed{177.9\, mm}$

* check ε_s

$\varepsilon_s = 0.003 \frac{(d - c)}{c}$

$= 0.003 \frac{(505 - 177.9)}{177.9}$

$\Rightarrow \boxed{\varepsilon_s = 0.0055}$ where $\varepsilon_y = \frac{f_y}{E} = \frac{420}{200{,}000} = \boxed{0.0021}$

$0.0055 > 0.0021$

then

$\varepsilon_s > \varepsilon_y$

assumption is ok

* Compute M_n, ϕM_n

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

$= 1530 \times 420 (505 - \frac{151.2}{2})$

$= \boxed{275.9\, kN.m}$

Nominal moment capacity

$\phi = 0.9$ since $\varepsilon_s = 0.0055 > 0.005$

$\Rightarrow \phi M_n = 0.9 \times 275.9 = \boxed{248.3\, kN.m}$

Design moment capacity.

* Check $A_{s\,min}$

اقل قيد لكمية الحديد ممكن تكون في التسليح.

$A_{s\,min} = \begin{cases} \frac{0.25\sqrt{f'_c}}{f_y} bd = 336\, m \\ \frac{1.4}{f_y} bd \quad \ldots = \boxed{420\, mm} \end{cases}$

معطاه بالامتحان ليست للحفظ

$\Rightarrow A_{s\,min} = 420\, mm^2$ (the larges one)

* ACI-code apply minimum value of A_s:-

to guarantee safe transfer of beam tension force from concrete to steel

تم وضعها بسبب الانتقال المفاجئ لقوة الشد من الخرسانة الى الحديد فيمكن ان يحدث كسر.

EX2

the same previous one (EX1) but

$A_s = 2 * 1530 = \boxed{3060\ mm^2}$

When we double A_s.

Sol:-

assume $\varepsilon_s \le \varepsilon_y \Rightarrow f_s = f_y$

$A_s f_y = 0.85 f_c' a b$

$\Rightarrow a = \dfrac{3060 * 420}{0.85 * 20 * 250} = \boxed{302.4\ mm}$

$\boxed{C = 355.76\ mm}$

*check ε_s

$\varepsilon_s = 0.003 \dfrac{(505 - 355.76)}{355.76} = 0.001258$

$\Rightarrow \varepsilon_s < \varepsilon_y = 0.0021$ <u>Not Ok</u>

$\Rightarrow f_s = E \varepsilon_s$

$= 200{,}000 * \left(\dfrac{505 - c}{c}\right)$

$\because A_s f_s = 0.85 f_c' a b$

$3060 * 200 \times 10^3 \left(\dfrac{505 - c}{c}\right) = 0.85 * 20 * \overset{0.85}{\beta} C * 250$

$\Rightarrow \boxed{C = 312.66\ mm}$

لازم تكون قريبه من قيمه C السابقة من اجل المحافظة على الحد أ (Comp. contr.)

$a = \beta C = 0.85 * 312.66 = \boxed{265.076\ mm}$

$\varepsilon_s = 0.003 \dfrac{(505 - 312.66)}{312.66} = 0.00184 < 0.0021$

<u>ok</u>

لازم تطلع - not yeilded

$f_s = E * \varepsilon_s = 200 \times 10^3 \times 0.00184 = \boxed{368\ MPa}$

القيمه تتضرب ÷ (10^6)

* comput M_n, ϕM_n

$M_n = 3060 * 368 (505 - \dfrac{265.76}{2})$

$= \boxed{419\ kN.m}$

$\phi = 0.65$ Since $\varepsilon_s = 0.00184 < 0.002$ Comp. Controlled

$\phi M_n = 0.65 * 419 = \boxed{272.4\ kN.m}$

* check A_s min

as previous one <u>O.k</u>

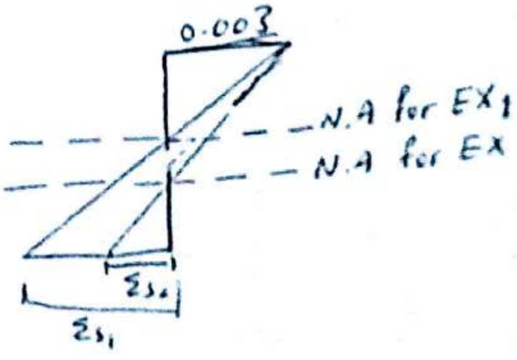
<u>Notes</u>

كمقارنه بين المثال الاول والثاني :-

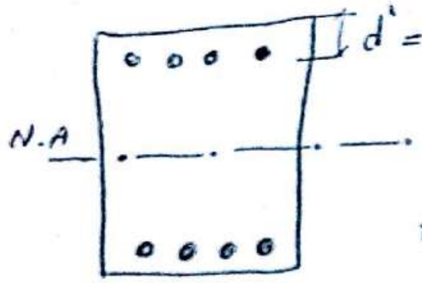
عندما زدنا كمية الحديد بكمية كبيرة كان له تأثير سلبي حيث احرج البيم من مرحله (tension) الى مرحلة (Compression)

- ولكن قد يكون له تأثير ايجابي حيثما يحتفظ بمرحلة (tension)

* increase amount of reinforcement in the tension has significant effect in <u>the nominal moment capacity</u> (Increase M_n)



Beams with tension and compression Reinforcements:-

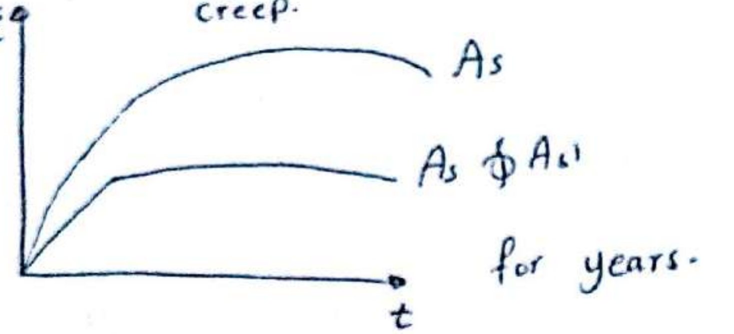


إذا لم يعطيك قيمتها بالسؤال ← تكون هذه القيمة.

the Beam is called (doubly Reinforcement Beam)

* Reasons:-

① Reduce Sustained load deflections: (creep)



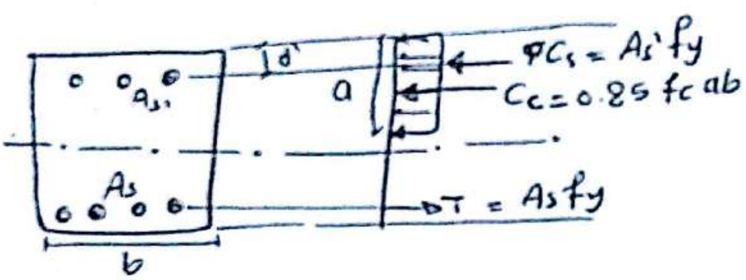
الحديد في منطقة Comp ساعد الخرسانة في تحمل جزء من الضغط عليه ولذلك سيكون الانحناء أقل مما لو كانت مسلحة من منطقة واحدة فقط

ملاحظة:-

في (Design) لا يمكن الانقاص من كمية الحديد كحل لمشكلة التواجد في مرحلة (Comp) لأن الذي يحكمني في قيمة A_s هي $\underline{M}$.

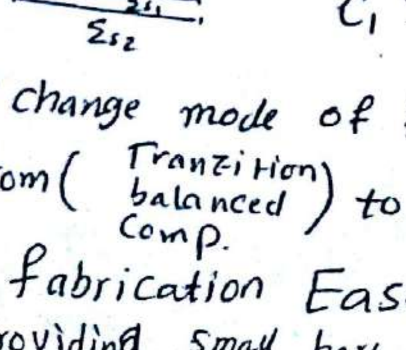
لذلك نلجأ لتسليح منطقة (Comp) كحل.

(انتبه!) هذا التعديل والاضافة فقط في التصميم لكن في التحليل لا يمكنني تعديل اي شئ على البيم.



$T = C_c + C_s$

② Increase <u>Ductility</u>!



$C_1 > C_2$ which will move the N.A up to Comp. Area and gives an increase in ductility.

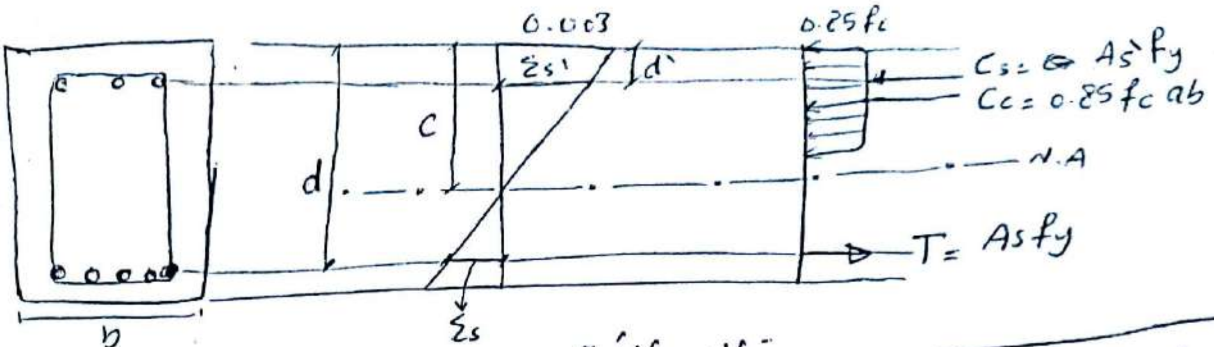
③ change mode of failure.

from (Tranzition / balanced / Comp.) to tension due to the increase of ε_s.

④ fabrication Ease (سهولة التصنيع)

providing small bars in the corners of the stirrups to hold the st. in place. إذا سلحت منطقة Comp. من اجل هذا السبب فإن قيمة A_s' تهمل وتعامل (Single)S

* Analysis of Beams with tension and compression Reinforcement.



تشابه مثلثات

$$\boxed{\varepsilon_s = 0.003\left(\frac{d-c}{c}\right)} \quad ; \quad \frac{\varepsilon_{s'}}{(c-d')} = \frac{0.003}{c} \Rightarrow \boxed{\varepsilon_{s'} = \frac{0.003(c-d')}{c}}$$

* there are two ways to solve analysis example :-

(I) assume $\varepsilon_{s'} \geq \varepsilon_y$ & $\varepsilon_s \geq \varepsilon_y$

$f_{s'} = f_y$ $\quad f_s = f_y$

$T = C_c + C_s \Rightarrow A_s f_y = 0.85 f_c ab + A_s' f_y$

check $\varepsilon_{s'}$ → Ok ⇒ go and check ε_s → Ok ✓ / → Not ok ⇒ $f_s = E\varepsilon_s$ -

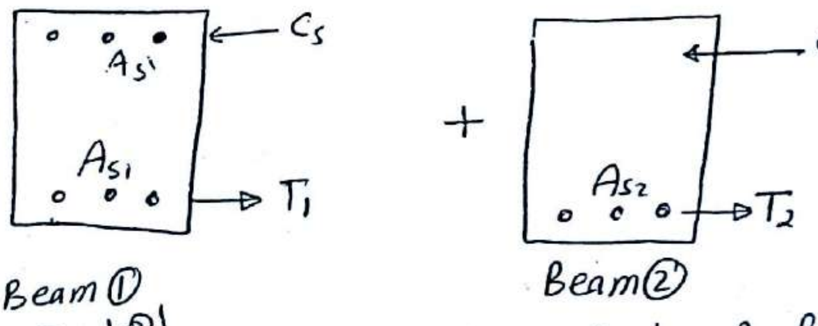
→ Not ok ⇒ $f_{s'} = E\varepsilon_{s'}$ → back to the equ.

then

$M_n = C_c(d - \frac{a}{2}) + C_s(d-d')$ & ϕM_n

(II) when $\varepsilon_s \geq \varepsilon_y$ → هذه الطريقة معتمدة في التصميم

Cas I



; where $A_s = A_{s1} + A_{s2}$

Beam (1)

اهمل وجود الخرسانة

$C_s = A_s' f_y$

$T_1 = A_{s1} f_y$

$A_{s1} f_y = A_{s'} f_y$

$\Rightarrow \boxed{A_{s1} = A_{s'}}$

له هذا في حاله الاثنين (A_s', A_s) yeilded

Beam (2)

$0.85 f_c ab = A_{s2} f_y$

then;

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

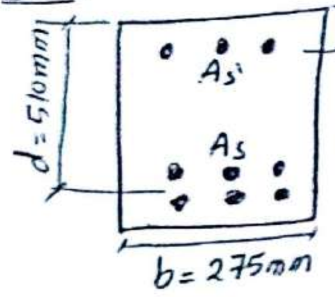
ϕM_n ✓ Cas II when $\varepsilon_{s'} < \varepsilon_y$

$\Rightarrow f_{s'} = E\varepsilon_{s'}$

$\Rightarrow$ back to equ.

* Analysis example:-

Ex₁



$A_s' = 852\ mm^2$ find design moment capacity?

$A_s = 3060\ mm^2$

$f_c' = 20\ Mpa$

$f_y = 420\ Mpa$

Sol:-

* comput a:- assume $\varepsilon_s' \geq \varepsilon_y$; $\varepsilon_s \geq \varepsilon_y$

$f_s' = f_y$ $\quad f_s = f_y$

$T = C_c + C_s$

$A_s f_y = 0.85 f_c' a b + A_s' f_y$

$3060 \times 420 = 0.85(20) a \times 275 + 852 \times 420$

$\Rightarrow \boxed{a = 198\,mm}$; $C = \frac{198}{0.85} = \boxed{233\,mm}$

* Check ε_s'

$\varepsilon_s' = 0.003\left(\frac{c-d'}{c}\right) = \frac{0.003(233-65)}{233}$

$= 0.0022 > \varepsilon_y = 0.0021$ Ok

* Check ε_s

$\varepsilon_s = \frac{0.003(d-c)}{c} = \frac{0.003(510-233)}{233}$

$= 0.0036 > \underset{\varepsilon_y}{0.0021}$ Ok

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

$= 0.85 \times 20 \times 198 \times 275\left(510 - \frac{198}{2}\right)$

$+ 852 \times 420(510 - 65) = \boxed{540\ kN.m}$

$\because \varepsilon_s = 0.0036 \Rightarrow \underset{\text{tranzition}}{\varepsilon_y < \varepsilon_s < 0.005}$

$\phi = 0.65 + (0.0036 - 0.002)\frac{250}{3} = \boxed{0.783}$

$\phi M_n = 0.783 \times 540 = \boxed{422\ kN.m}$

Check $A_{s\,min}$ — note: A_s' doesn't have minimum.

$A_s = \begin{cases} \frac{0.25\sqrt{f_c'}\,bd}{f_y} \\ \frac{1.4}{f_y}bd \end{cases}$ take the largest.

Ex₂ the same ex₁ but we doubled $A_s' = 2 \times 852 = 170$

Sol:-

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

$A_s f_y = 0.85 f_c a b + A_s' f_y$

$3060 \times 420 = 0.85 \times 20 \times a \times 275 + 1704.$

$\Rightarrow \boxed{a = 121.8\,mm}$; $\boxed{c = 143.3}$

check $\varepsilon_s' = 0.0016 < \varepsilon_y = 0.00$

$\Rightarrow f_s' = E \times 0.003\left(\frac{c-d'}{c}\right)$

↳ back to equ.

$A_s f_y = 0.85 f_c \underset{BC}{a} b + A_s' E \times 0.003($

you will find

$\boxed{C = 166.55\,mm}$ $a = 141.57$

$\Rightarrow \varepsilon_s' = 0.0018 < \varepsilon_y$ Ok

check ε_s

$\varepsilon_s = 0.0062 > \varepsilon_y$ Ok

$M_n = 0.85 \times 20 \times 141.57 \times 275(510-$

$+ 1704 \times \underbrace{200 \times 10^3 \times 0.0018}_{f_s'}(510-$

$= \boxed{563.7\ kN.m}$

$\because \varepsilon_s = 0.0062 > 0.005 \Rightarrow \phi = 0.$

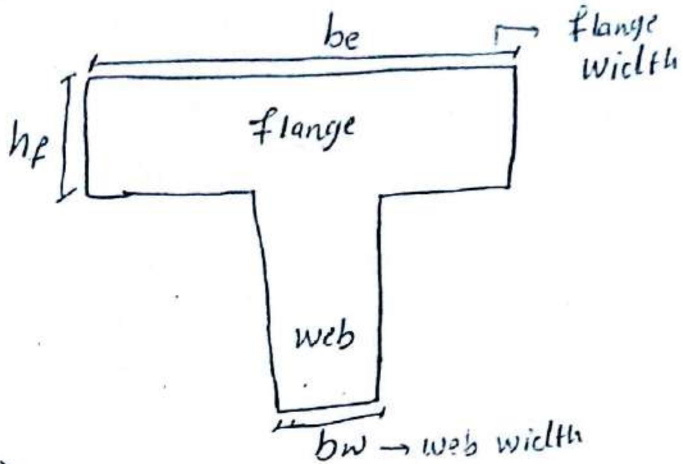
$\phi M_n = \boxed{507.3}\ kN.m$

!حط مقارنة مع المثال الاول وانا:-

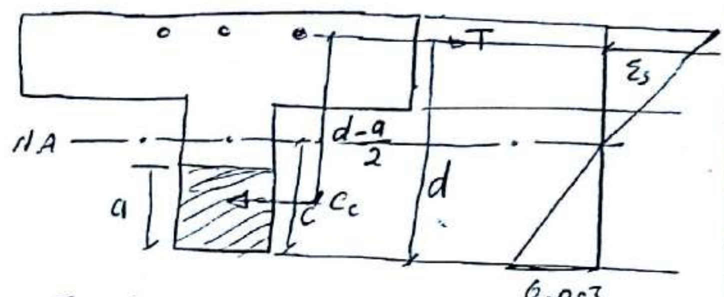
* Increase the comp. Reinforce: has significant effect in des moment capacity.

Not in nominal moment ca

* Flexure in T-Beam



(I)

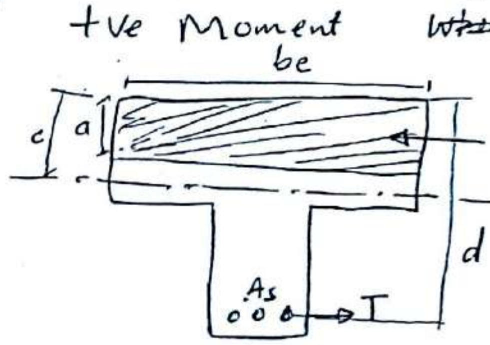


$T = C_c$ — -ve Moment

$A_s f_y = 0.85 f_c a b_w$

$M_n = \underset{C_c}{T}(d - \frac{a}{2})$ as Rectangular beam

(II) +ve Moment



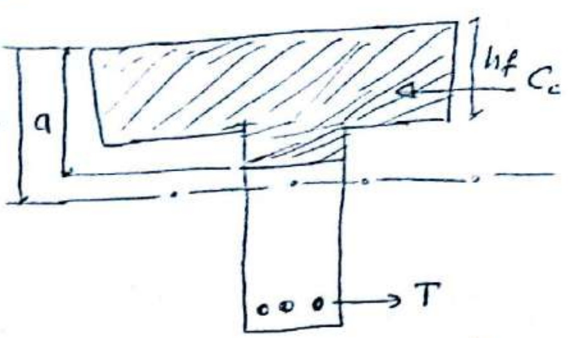
In this case also as rectangular beam since compression area is rectangular $(a < h_f)$

$T = C_c$

$A_s f_y = 0.85 f_c a \underline{b_e}$ ↱ width of compression area

$M_n = \overset{T}{C_c}(d - \frac{a}{2})$

(III) +ve Moment. $(a > h_f)$



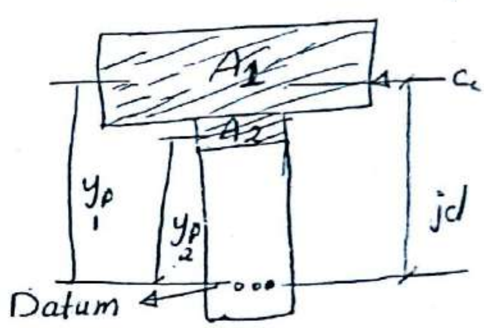
$0.85 f'_c (b h_f + b_w(a - h_f)) = A_s f_y$

$M_n = \overset{C_c}{T}(\underline{jd})$

Not $(d - \frac{a}{2})$

* tow ways to solve :-

(I) find the centroid of comp. a

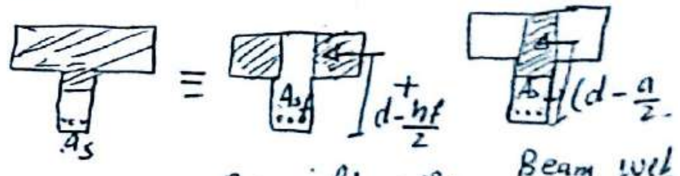


$jd = \frac{\sum A_i y_i}{\sum A_i}$ — y_i : distance from the center of shape to dat.

$jd = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2}$

then $M_n = T\, jd$ OK

Or



$A_{sf} f_y = 0.85 f'_c (b - b_w) h_f$ للتحصيل الزميد

$\Rightarrow A_{sf}$ ✓

$A_{sw} = A_s - A_{sf}$ ✓

$A_{sw} f_y = 0.85 \ldots a$

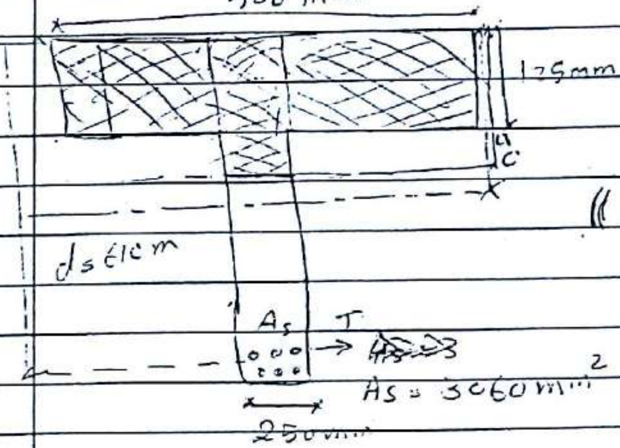
$M_{nf} = T_f (d - \frac{h_f}{2})$

$M_{nw} = T_w (d - \ldots$

$\Rightarrow M_n = M_{nf} + M_{nw}$

No.

Example :-



$f'_c = 20$ Mpa

$f_y = 420$ Mpa

((positive moment capacity))

* Solution

* لنبدأ الحل افترض انه Rectangular Beam

assume $\varepsilon_s > \varepsilon_y$

assume $a < h_f$

* حل آخر للمرين :

$T = C_c$

$A_s f_y = 0.85 f'_c \, a b$

$3060 * 420 = 0.85 * 20 * a * 500$

$\Rightarrow a = 151.2 \; mm > h_f \Rightarrow$ Analysis as T-Beam

$A_s f_y = 0.85 f'_c (A_c)$

لـ او جدها وبعد ذلك اقارنها

مع (Area of flange)

اذا كانت اقل ← o.k يعني

براجع حل على اساس انه مستطيل .

T-Beam Action

Beam f :-

$C_{cf} = T_f$

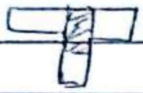
$0.85 f'_c (b - b_w) h_f = A_{sf} f_y$

$0.85 * 20 (500 - 250) * 175 = A_{sf} * 420$

$\Rightarrow A_{sf} = 1264.9 \; mm^2$

No.

* Beam w :-



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

$C_{cw} = T_w$

$0.85 f'_c \, a b_w = A_{sw} f_y$

$0.85 * 20 * a * 250 = 1795.1 * 420$

$\Rightarrow a = 177.4 \, mm$ * a اكبر من a الواردة قبل.

$c = 208.7 \, mm$

$\varepsilon_s = 0.003 \left(\frac{610 - 208.7}{208.7} \right) = 0.0058$

انتبه المقارنة لازم تكون نفس الفرض $\varepsilon_s > \varepsilon_y$ O.k where $\varepsilon_y = 0.0021$

$M_{nf} = 1264.9 * 420 \left(610 - \frac{125}{2} \right) = \cancel{290.9} \; 290.9 \, kn$

$M_{nw} = 1795.1 * 420 \left(610 - \frac{177.4}{2} \right) = 393 \, kn.m$

$M_n = M_{nf} + M_{nw} = 683.9 \, kn.m$

$\varepsilon_s = 0.0058 > 0.005 \Rightarrow$ Tension Contr

$\Rightarrow \phi = 0.9$

$\phi M_n = 0.9 (683.9) = 615.5 \, kn.m$

* هذا الحد اللي بنتعامل معه

No.

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

$$A_{s\,min} = \begin{cases} \dfrac{\sqrt{f_c'}}{4 f_y} b_w d = 406\ mm^2 \\ \dfrac{1.4}{f_y} b_w d = 509\ mm^2 \checkmark \end{cases}$$

O.k

per ACI-Code → For $A_{s\,min}$ Only.

الشرطين لازم يتحققوا الاثنين مع بعض.

For statically determinate beams where the flange portion is in tension, ACI (recommend) that (b_w) be replaced by the smaller of

$$\begin{cases} 2 b_w \\ b_e \rightarrow \text{the width of the flange} \end{cases}$$

بس ال bw الي نعوضها في $A_{s\,min}$

لازم شرط

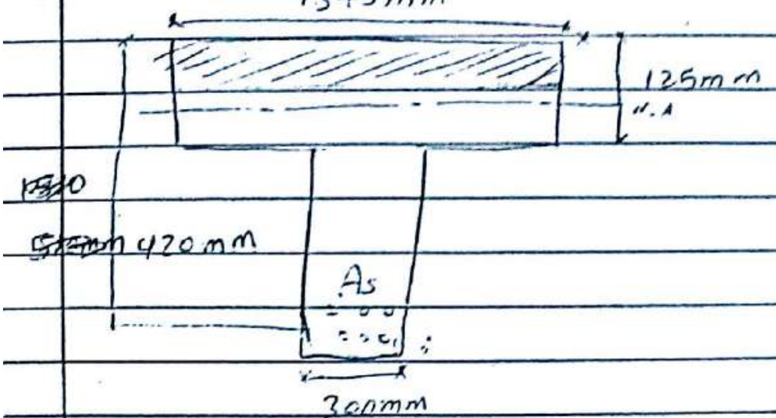
السبب لانه لا نخرج الا في منطقة ال comp تكون المساحة بين ... الحديد صفر.

types of statically determinant Beams :-

1- Cantilever Beam → ~~the~~ always up reinforce
2- simply supported Beam
⋮

(18)

Example:-



$f'_c = 20$ Mpa

$f_y = 420$ Mpa

$A_s = 1704$ mm²

(*) Compute a

Assume $\varepsilon_s \geq \varepsilon_y \rightarrow 0.0021$

assume $a \leq h_f$

$1704 * 420 = 0.85 * 20 * a * 1375$

$a = 21.9$ mm $< h_f$ O.k

$c = 25.8$ mm

$\varepsilon_s = 0.003 \left(\frac{420 - 25.8}{25.8} \right)$

$\Rightarrow \varepsilon_s = 0.046 > \varepsilon_y$ O.k

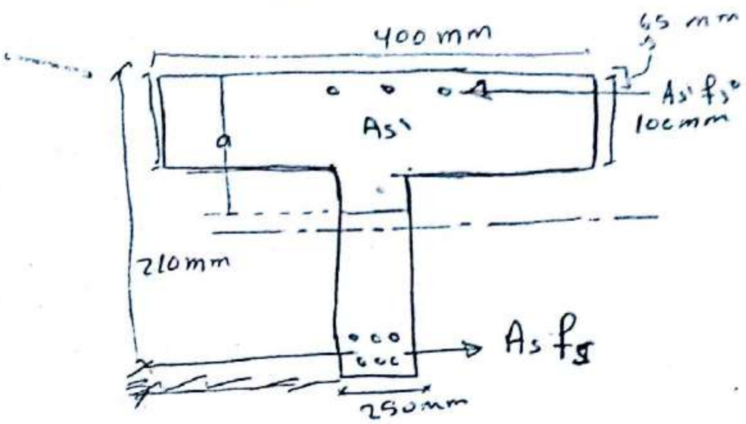
$M_n = 1704 * 420 \left(420 - \frac{21.9}{2} \right) = 292.74$ Mpa

$\because \varepsilon_s = 0.046 > 0.005 \Rightarrow$ tension $\Rightarrow \phi = 0.9$

$\phi M_n = 0.9(292.74) = 263.476$ Mpa

You can check $A_{s\,min}$

Ex:-



$A_{s'} = 852\ mm^2$

$A_s = 3060\ mm^2$

$f'_c = 20\ Mpa$

$f_y = 420\ Mpa$

Sol:-

assume ①: $\varepsilon_s \geq \varepsilon_y \Rightarrow f_s = f_y$

assume ②:- $\varepsilon_{s'} \geq \varepsilon_y \Rightarrow f_{s'} = f_y$

assume ③:- $a \leq (h_f = 100mm)$

$\because T = C_c + C_s$

$A_s f_y = 0.85 f'_c ab + f_y A_{s'}$

$\Rightarrow 3060 * 420 = 0.85 * 20 * a * 400 + 420 * 852$ assumption ① Not O.k ✗

$\Rightarrow a = 136.37 mm > h_f$

analysis as T-Beam.

$0.85 f'_c (\overline{b h_f + b_w (a - h_f)}) + f_s A_{s'} = A_s f_s$

$0.85 * 20 (400(100) + 250(a-100)) + 420 * 852 = 3060 * 420$ ---①

$\Rightarrow \boxed{a = 158.2 mm}$, $\boxed{C = 186.11 mm}$

Check

$\varepsilon_{s'} = 0.003 \left(\frac{186.11 - 65}{186.11} \right) = 0.00195 < 0.0021$ not ok

Return to equ①

$\Rightarrow 0.85 * 20 (400(100) + 250 (\overset{0.85C}{a} - 100)) + 852 * 200 * 10^3 * 0.003 \left(\frac{C - 65}{C} \right) = 3060 * 420$

$\boxed{a = 162.9 mm}$ $\boxed{C = 191.64 mm}$

$\varepsilon_{s'} = 0.003 \left(\frac{191.64 - 65}{191.64} \right) = 0.00198 < \varepsilon_y$ ✓

$\varepsilon_s = 0.003 (210 - 191.64) = 0.00028 < \varepsilon_y$ not Ok

$\Rightarrow$

$$0.85*20(400(100)+250(0.85c-100))+852*200\times10^3*0.003\left(\frac{c-65}{c}\right)$$
$$=3060*0.003\left(\frac{210-c}{c}\right)*200\times10^3$$

$\Rightarrow$ $\boxed{c=135.46\,mm}$ $\boxed{a=115.14\,mm}$

$$\varepsilon_s'=0.003\left(\frac{135.46-65}{135.46}\right)=0.00156<\varepsilon_y \checkmark$$
$$\varepsilon_s=0.003\left(\frac{210-135.46}{115.46}\right)=0.00165<\varepsilon_y \checkmark$$

<u>$\emptyset M_n$</u>

$$M_n=\left(0.85*20*40000*(210-\frac{100}{2})\right)+\left(0.85*20*250*15.14*(110-\frac{15.14}{2})\right)$$
$$+\left(852*200\times10^3*0.003\left(\frac{135.46-65}{135.46}\right)(210-65)\right)$$
$$=153946720.2\times10^{-6}=153.94\ kN.m$$

$\because$ $\varepsilon_y=0.00165<0.005$ compression

$\Rightarrow \emptyset=0.65$

$\therefore \emptyset M_n=0.65*153.94=100\ kN.m$

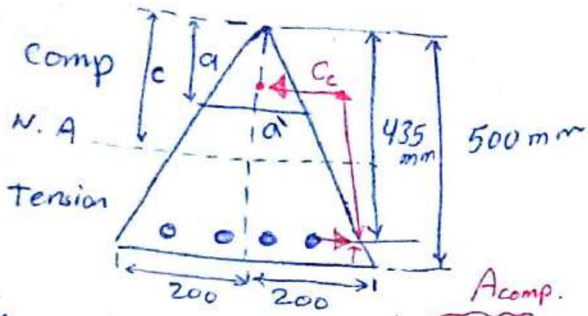
⊛ الى هنا مادة الفيرست

⊛ لاتنسى! حل الاسئلة الموجودة على دوسية المهندس / انس دواس

وبالتوفيق لكم جميعاً

" دعواتكم " "لطف الحباري."

II



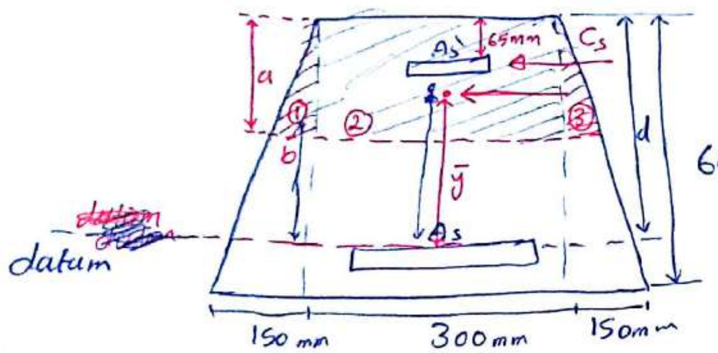
$\because \frac{a'}{a} = \frac{400}{500}$

$\Rightarrow a' = \frac{4}{5} a$

★ $A_s f_s = 0.85 f'_c (\overbrace{\tfrac{1}{2} a' a}^{A_{comp.}})$

$= 0.85 f'_c (\frac{1}{2} * \frac{4}{5} (a)^2)$

★ $M_n = \underset{C_c}{T} * (d - \frac{2}{3} a)$



600 mm ; d=535mm

★ $A_s f_y = 0.85 f'_c (A_1 + A_2 + A_3)$; where $A_1 = A_3 \Rightarrow \frac{a}{b} = \frac{600}{150} \Rightarrow b = \frac{1}{4} a$

$+ C_s A_s'$

$= 0.85 f'_c (2(\frac{1}{2} * \frac{1}{4} a^2) + 300 * a) + C_s A_s'$

To find the centroid of comp. area

$\Rightarrow \bar{y} = \frac{\Sigma A y_i}{\Sigma A}$ where y_i :- is the distance from the centroid of each area to the datum.

$= \frac{2(\frac{1}{2} * \frac{1}{4} a^2 * (d - \frac{2}{3} a)) + (300 * a (d - \frac{a}{2}))}{\frac{1}{8} a^2 + 300(a)} = ----$

$M_n = C_c (\bar{y}) + C_s (d - 65)$

RC1 - Second

لطف الصباري

※Civilittee

*Design of Rectangular Beams

*Relationship Between beam depth and deflection:-

Use table 9.5 → h_{min} (to avoid deflection calculations)

for example:-

$h_{min}(S.S) = \frac{\ell}{16}$

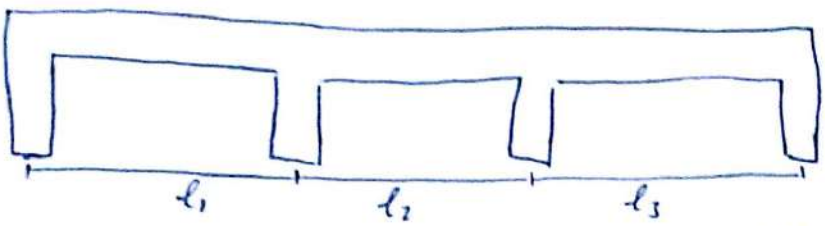
↓ Simply Supported

Or for Cantiliver

$h_{min} = \frac{\ell}{8}$

* كلما زادت h، كلما زادت مقاومة البيم للانحناء نتيجة لزيادة قيمة (I)

* في اي بيم يوجد هناك انحناء و لكن علينا ان نصممه لكي يكون هذا الانحناء ضمن المواصفات (ACI) وان لا يحدث له انهيار.



⊛ كيف احسب h_{min} لهذا البيم؟ ... هو ان اقوم بحساب قيمة h_{min} عند كل قيمة ℓ واختار الاكبر.

Ex:-

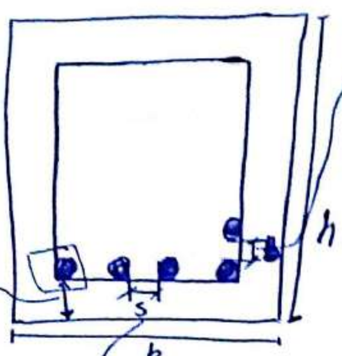
$h_{min} = \frac{\ell_1}{16}$ ...

$h_{min} = \frac{\ell_2}{16}$ ...

$h_{min} = \frac{\ell_3}{16}$ ...

} choose the largest.

② Concrete cover and Bar spacing:-

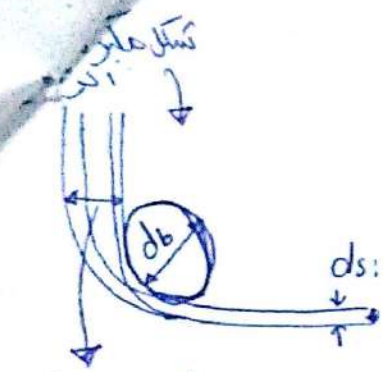


S_{min} = larger of
- → 25 mm
- → 1.33 max. C.A.Size (Coarse aggregate Size.)

* for ACI min = 4cm for normal exposure
if abnormal go to ACI-code

S_{min} = larger of
- → Bar diameter
- → 1.33 max C.A.Size
- → 25 mm
- → diameter of vibrater (not in ACI-co

①



for ACI-code this space $= (2d_s - 0.5d_b$

Reasons of cover:-

- protect steel from fire
- protect steel from abrasion
- to bond concrete and steel so that both elements act togather (perfect bond)
- Addition cover in the slaps of garage not to reduce the allowable cover below due to traffic.

b_{min}:- the min. width of the beam to put the bars in ~~one layer~~ one layer.

So;

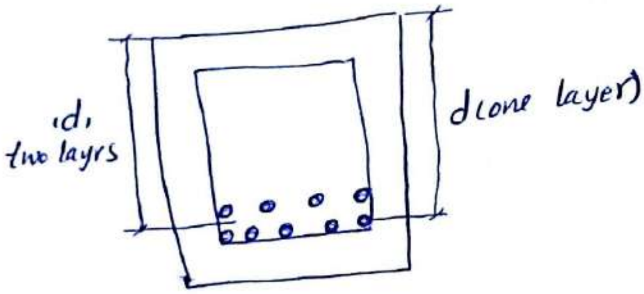
cover, d_s, عدد الفراغات بين البارس

$$b_{min} = 2*40 + 2*10 + n\, d_b + (n-1) S_{min} + 2[2d_s - 0.5d_b]$$

if $b < b_{min} \Longrightarrow$ 2 layers.

⊛ Estimating the effective depth of beam:-

from ACI code
- $d \simeq h - 65$ (one layer)
- $d \simeq h - 90$ (two layers)



b_{min}
- → preferable (300mm)
- → absolute (250 mm)

الفرق بينها والسابقة :-

* b_{min} السابقة كانت لكي اتأكد من عدد الصفوف اللي اقدر احط فيه التسليح

* b_{min} هذه هي اللي يعتمد عليها التصميم و هي محكومة على اساس «M».

* المقصود بتصميم البيم هو ايجاد كمية حديد التسليح (A_s) بالاضافة الى ابعاد البيم بناء على الاثقال المتوقع حملها منه واعتماداً على المواصفات (b,h)

* General Strength Design Requirments for Beams :-

* $\phi M_n \geq M_u$; $W_u = 1.2\,DL + 1.6\,LL$; $W_u = 1.4\,DL$

لم نبدأ بالتساوي

نختار الأكبر

استخدمها لما تكون DL اكبر بكثير من LL

$\phi M_n = M_u$

$\phi A_s f_y j d = M_u$

$$\Rightarrow \boxed{A_s = \frac{M_u}{\phi f_y j d}} \longrightarrow \boxed{\phi = 0.9} \text{ always because it's a design.}$$

* there are tow kind of problems :-

① if b, h are given → في هذه الحالة إذا كان المهندس المعماري هو من حدد الابعاد ولكن بشرط ان لا يقل عن b_{min}

* $j = (0.87 - 0.9)$ in RC1 $j = 0.9$ in narrow compression areas.
* $j = 0.95$ → for wid compression area (like T-beams with flange comp.)

* Design process :- → See the example from notebook

ملاحظة:- ممكن بالسؤال ما يطلب منك تصميم كامل للبيم ويطلب اجزاء من السؤال فلازم تجاوب على قدر السؤال.

* estimate M_u based on $W_u = 1.2DL + 1.6LL$

↓

estimate one layer and find $d = h - 65$

↓

Find $A_s = \frac{M_u}{\phi f_y j d}$ (Required) → لأننا استخدمنا التساوي

↓

Go to table (A4M) to find A_s provided

بروح عالجدول وادور على ثاني اقرب رقم او اول اقرب رقم لكن الثاني افضل لأحفظ اكثر امان.

$A_s = \boxed{\quad}\ mm^2$

n NO. M

↓

Check b_{min} → if $> b_o$ → Go back and estimate two layers

or choose A_s provided from the table with large diameter to reduce no. of spacing.

↓

Check $A_{s\,min}$ → عادةً تكون اللود كبير ⇒ if $A_s < A_{s\,min}$ ⇒ choose $A_{s\,min}$.

↓

Compute a (like analysis)

↓

Check $A_{s\,new}$ required based on computed a $\quad A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})}$

if $A_{s\,provided} > A_{s\,new}$ required → ok

" < " " → Go back and choose A_s from table according to $A_{s\,new}$ required.

2nd kind of problem when (b, h, A_s) are Unknowns:-

$\because T = C \Rightarrow A_s f_y = 0.85 f'_c a b$

$\Rightarrow a = \dfrac{A_s f_y}{0.85 f'_c b}$

$\because \rho = \dfrac{A_s}{bd}$ (Steel ratio ; reinforcemnt ratio)

$\boxed{A_s = \rho b d}$; $a = \left(\rho \dfrac{f_y}{f'_c}\right)\left(\dfrac{d}{0.85}\right)$

ω (mechanical steel ratio)

$\phi M_n = \phi\, 0.85 f'_c a b (d - \frac{a}{2})$

$= \phi \left[b d^2 \boxed{\omega f'_c (1 - 0.59\omega)} \right]$

flexural resistance factor (k_n , R_n)

$\Rightarrow \phi M_n = \phi b d^2 k_n$

$\Rightarrow \boxed{b d^2 = \dfrac{\phi M_n}{\phi k_n} = \dfrac{M_u}{\phi k_n}}$

$\boxed{A_s = \dfrac{M_u}{\phi f_y j d}}$

@ the weight of a rectangular beam will be roughly (10 - 20%) of the loads it must cary ; self wet = (10 - 20%) (DL + LL)

Or @ assum avalue for $h = (8 - 10\%)\, l$; l :- span length of the beam

then $b = 0.5h$

ليست القيم النهائية للتصميم والآن فقط لغرض self wet

$\Rightarrow \boxed{\text{self wet} = \gamma b h}$

ρ → economic consideration $\boxed{\rho = 0.01}$ → هذه هي التي عادةً بنصمم عليها.

by placing consideration it may be hard to place the reinforcement if ρ exceed 0.015.

ρ → Ductility Consideration $\rho = (0.35 - 0.4)\rho_b$

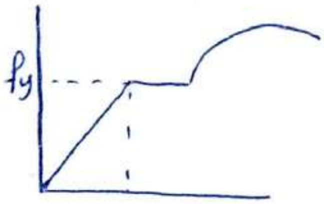
where ρ_b : balanced steel ratio $= \dfrac{A_{sb}}{bh}$ → balanced A_s له كمية الحديد التي تجعلني في مرحلة balanced.

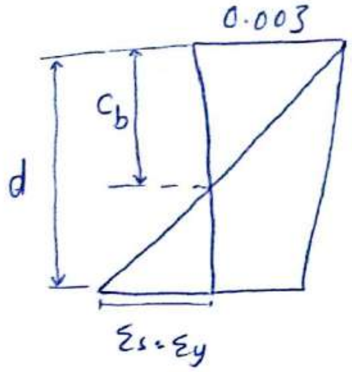
...ility consideration

$\rho_b = \dfrac{A_{sb}}{bd}$

$\rho = (0.35 - 0.4)\rho_b$

Strain لزيادة اقلل من كمية الحديد لذلك تقريباً نأخذ كلنا A_{s_b}





$$\Rightarrow \frac{0.003}{c_b} = \frac{0.003+\varepsilon_y}{d} \Rightarrow c_b = \frac{0.003\, d}{0.003+\varepsilon_y}$$

$$\because a_b = \frac{A_s f_y}{0.85 f'_c b} = \frac{\rho_b\, b d\, f_y}{0.85 f'_c b}$$

$$\Rightarrow c_b = \frac{a_b}{\beta} = \frac{\rho_b\, \cancel{d}\, f_y}{0.85 f'_c \beta_1} = \frac{0.003\, \cancel{d}}{0.003+\varepsilon_y}$$

$$\Rightarrow \boxed{\rho_b = \frac{0.85 f'_c \beta_1}{f_y}\left(\frac{0.003}{0.003+\varepsilon_y}\right)} \rightarrow \text{ليس للحفظ}$$

ولكن عليك فهم طريقة الاشتقاق لانه ربما يجيب سؤال

→ find the reinf. When $\varepsilon_s = 0.0035$??

Design process (see design example in note book)

- estimate self wet (self wet = (10 - 20%)(DL + LL) or $h = (8-10\%)L$, $b = 0.5h$] → S.W. = γb...

↓

Compute M_u for example $M_u = \dfrac{w_u \ell^2}{8}$ if (S.S.B)

or $M_u = \dfrac{w_u \ell^2}{2}$ if cantilever beam.

⋮

↓

Comput b & d

$bd^2 = \dfrac{M_u}{\phi k_n}$; assume any value of b Started with $b_{min} = 300$ then find h

↓

assume two layers then; $h = \cancel{b}\, d + 90 = \boxed{}$ → هذه القيمة يمكنك تقريبها للاكبر والتقريب يكون لل 5 . مثلاً اذا كانت

780 $\xleftarrow{\text{تقريباً}}$ 800 وهكذا ...

وبعدها اوجد قيمة d الجديدة $\boxed{d = h - 90}$

↓

check self wet and revise M_u ; self wet. = $\gamma b \overset{\text{الجديدة}}{h}$ → $w_{u\,new}$

then; $M_{u\,new} = \cancel{\frac{w_u \ell^2}{8}}$ ~~make check~~ if $M_{u\,new} < M_{u\,old}$ Ok → continue design.

{ make a check when $M_{u\,new} > M_{u\,old}$ } $= \dfrac{M_{u\,new} - M_{u\,old}}{M_{u\,new}} \times 100\%$ → if < 10% Ok ; → if > 10% repeat the design and use $M_{u\,new}$.

↓

Comute $A_{s\,required} = \frac{M_u}{\phi f_y j d}$; you can check $A_{s\,min}$ if A_s is small.

check $A_{s\,max}$ where $\boxed{A_{s\,max} = 0.0319 \beta \frac{f'_c}{f_y} bd}$ → ليست للحفظ معطاه بالامتحان*

if $A_s < A_{s\,max}$

Ok

select steel find $A_{s\,provided}$ from the table

find b_{min}.

check ε_s and complete a then find M_n, ϕM_n

check $A_{s\,required}$ based on computed a

هلا حظات :

* في هذا المثال لو طلعت مثلاً 2-layers وعندما فحصت ε_s كانت قيمتها (0.0048) او شي قريب من 0.005 وكنت مستخدم d على اساس انه 2-layers.

في هذه الحالة بقدر استخدم d الخاصه بـ (One layer) حيث انه سيتم رفع قيمه ε_s الى 0.005 او اكثر ويكون تصميمي صحيح.

if $A_s > A_{s\,max}$

⇒ Double reinforcement → increase ductility.

assume $A_{s2} = A_{s\,max}$

then $a = \frac{A_{s\,max} f_y}{0.85 f'_c b}$ → $\varepsilon_s = 0.005$ لا لازم تطلع هذه القيمه لأني مستخدم $A_{s\,max}$.

then $c = \frac{a}{\beta}$

find $\phi M_{n2} = \phi A_{s\,max} f_y (d - \frac{a}{2})$

$\because M_u = \phi M_{n2} + \phi M_{n1}$

$\Rightarrow \phi M_{n1} = \phi M_u - \phi M_{n2}$

check $\varepsilon_{s'} = 0.003 \left(\frac{c - d'}{c} \right)$

if $\varepsilon_{s'}$ → yeilded $f_{s'} = f_y$

→ Not yeilded $f_{s'} = E \varepsilon_{s'}$

then ⇒ $A_{s'} = \frac{\phi M_{n1}}{\phi f_{s'}(d - d')}$

$\because A_{s1} f_y = A_{s'} f_{s'}$ ⇒ you can got A_{s1}

$\because A_s = A_{s1} + A_{s2}$ Required

select steel and check b_{min}.

modify $A_{s'}$ (A_s provided)

$A_{s1} = A_{s\,provided} - A_{s\,max}$

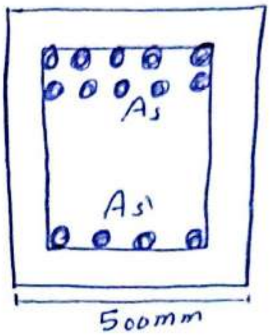
then you got $A_{s'}$ from $A_{s'} f_{s'} = A_{s1} f_y$ and select $A_{s'}$

continue as design (ε_s , ϕM_n)

gm prob. → past exam

2.5 m long Cantilever beam with a rectangular across section having a width of 500 mm. Based on Site limitations the overall depth should not exceed 700 mm. provide the most economic beam design to resist an ultimate negative moment of 1300 kN.m (Use No. 25M steel assume two layers of tension reinf.) perform only the following checks: (the two layers assumption and tension controlled assumption) $f'_c = 25$ Mpa, $f_y = 420$ Mpa

Sol:- $M_u = 1300$ kN.m



$\because bd^2 = \dfrac{M_u}{\phi K_n}$; $K_n = w f'_c (1 - 0.59 w)$

$w = \rho \dfrac{f_y}{f'_c}$ economic

$= 0.01 \times \dfrac{420}{25} = 0.168$

$K_n = 0.168 \times 25 (1 - 0.59 (0.168)) = 3.74$ Mpa

$\Rightarrow 500 \times d^2 = \dfrac{1300 \times 10^6}{0.9 \times 3.74} \Rightarrow d = 878.88$ mm

$\Rightarrow h = 968.88$ mm > 700 mm $\Rightarrow$ use $\boxed{h = 700}$ عن السؤال الزعلان انه h لايتجاوز (700 mm)

$\Rightarrow d = 700 - 90 = \boxed{610 \text{ mm}}$

$A_s = \dfrac{M_u}{\phi f_y j d} = \dfrac{1300 \times 10^6}{0.9 (420) \times 0.9 \times 610} = \boxed{6264.39 \text{ mm}^2}$

check $A_{smax} = 0.319\, b d \beta_1 \dfrac{f'_c}{f_y} = 0.319 \times 500 \times 610 \times 0.85 \times \dfrac{25}{420} = \boxed{4922.6 \text{ mm}^2}$

$\because A_s > A_{smax} \Longrightarrow$ Not tension cont. So (doubly reinforcement)

let $A_{s2} = A_{smax} = 4922.66$ mm^2

$\Rightarrow a = \dfrac{A_{s2} f_y}{0.85 f'_c b} = \dfrac{4922.66 \times 420}{0.85 \times 25 \times 500} = \boxed{194.6 \text{mm}} \Rightarrow \boxed{C = 228.9 \text{ mm}}$

$\phi M_{n2} = \phi A_{s2} f_y (d - \frac{a}{2}) = 0.9 \times 420 \times (4922.6)(610 - \frac{194.6}{2})$

$\Rightarrow \boxed{\phi M_{n2} = 954 \text{ kN.m}} \Rightarrow \phi M_{n1} = 1300 - 954 = \boxed{346 \text{ kN.m}}$

Check if $\varepsilon_s' = 0.003 \left(\dfrac{228.9 - 65}{228.9} \right) = 0.002148 > \varepsilon_y = 0.0021$ ∴ yeilded

$\Rightarrow f_{s'} = f_y = 420$ Mpa

$$\Rightarrow A_{s'} = \frac{\phi M_{n1}}{\phi f_{s'}(d-65)} = \frac{346 * 10^6}{0.9 * 420(610-65)} = \boxed{1679.53\ mm^2}$$

$$\because A_{s1} f_y = A_{s'} f_y \Rightarrow \boxed{A_{s1} = A_{s'} = 1679.53\ mm^2}$$

$$\Rightarrow A_s = A_{s1} + A_{s2} = 1679.53 + 4922.66 = \boxed{6602.19\ mm^2} \text{ required}$$

Go to <u>table A-4M</u> and select Steel (you have to choose No 25 from question)

$\because$ 1 No.25 = 510 mm² $\Rightarrow$ try 14 No.25 $= 14 * 510 = \boxed{7140\ mm^2}$

<u>Check</u> $b_{min} = 2(40) + 2(10) + 14(25) + 13(25) + 2(20 - 12.5)$
$= \boxed{790\ mm} > 500$ two layers.

② modify $A_{s'}$ $\quad \because A_{s1} = A_{s\,provided} - A_{s\,max} = 7140 - 4922.66 =$

$\Rightarrow A_{s'} = A_{s1} = \boxed{2217.34}$ required. $A_{s'}$

select $A_{s'}$ from A-4M table $\Rightarrow$ Use 5No.25

$\boxed{A_{s'\,provided} = 2550\ mm^2}$

- To check tension Controlled

$\because T = C_c + C_s \Rightarrow 420 * 7140 = 0.85 * 25 * 500 * a + 2550(420)$

$\Rightarrow \boxed{a = 181.44\ mm} \Rightarrow \boxed{c = 213.45\ mm}$

$\varepsilon_{s'} = 0.003\left(\frac{213.45 - 65}{213.45}\right) = 0.002086 < 0.0021$ not yeilded,

So! $420 * 7140 = 0.85(25) * 500 * \overset{0.85}{\beta} c + 2550 * 200 \times 10^3 * 0.003\left(\frac{c - 65}{c}\right)$

$\Rightarrow \boxed{c = 214.074\ mm} \Rightarrow \boxed{a = 181.96\ mm}$

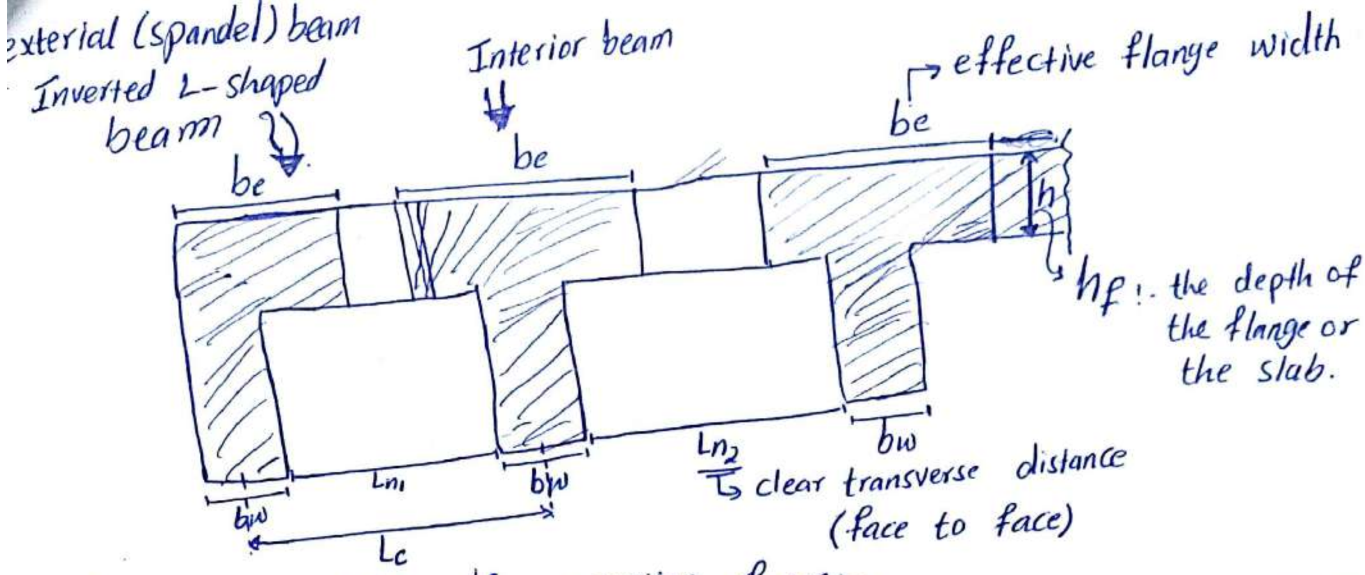
حتى لو ما عملت هذه الخطوة صحيح لأن الفرق قليل.

$\varepsilon_{s'} = 0.003\left(\frac{214.074 - 65}{214.074}\right) = 0.002089 < 0.0021$ <u>Ok</u>

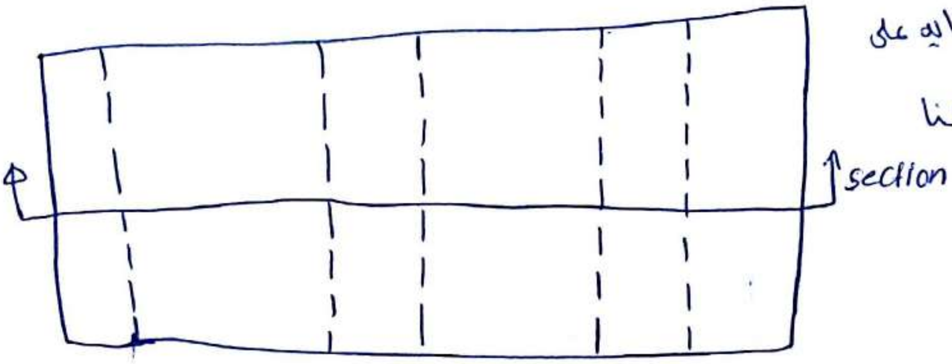
$\varepsilon_s = 0.003\left(\frac{610 - 214.074}{214.074}\right) = 0.005548 > 0.005$

then tension controlled <u>Ok</u>

Design of T-Beam.



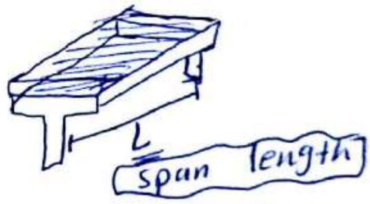
the above section is a section from:



⊛ انواع T-Beams :-
١- ممكن اصممه من البدايه على اساس انها T-Beam.
٢- ممكن ينتج معنا وقت تنفيذ صب الاسطح (Slap).

⊛ Spanderal Beam :-

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



Where L: is span length of the beam which is the length goes inside the paper in the above section.

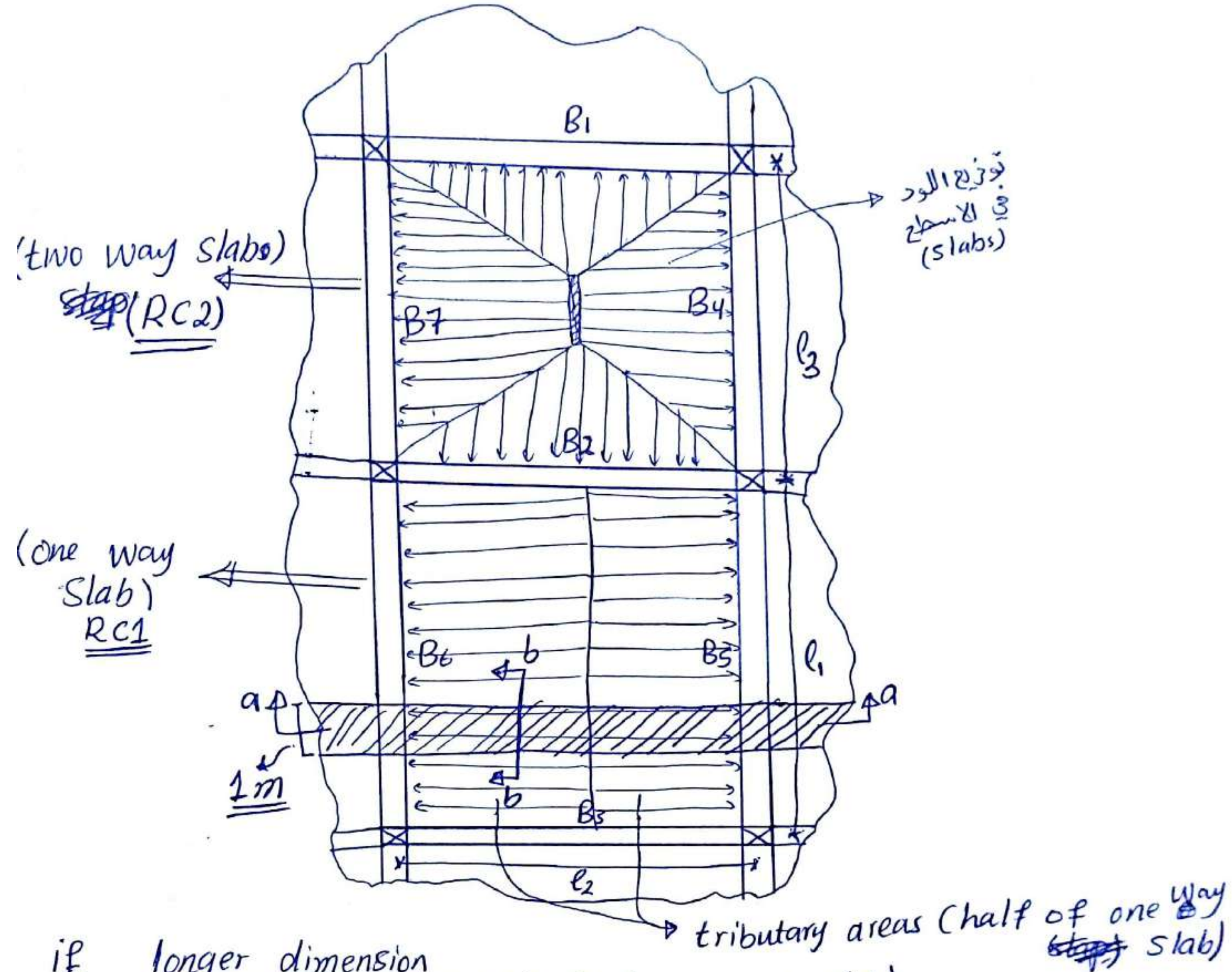
⊛ T-Beam (Interior) :-

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

* See example in the note book!

⊛ قوانين مهمة للحفظ !

One-Way Solid Slabs :- one way → يقصد به اتجاه انتقال اللــود .



if $\frac{\text{longer dimension}}{\text{shorter dimension}}$ → ≥ 2 (one-way slab)
→ < 2 (two-way slab)

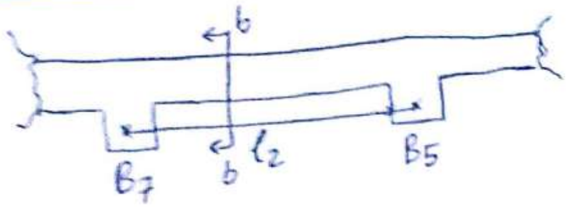
$\boxed{\frac{kn}{m^2} \rightarrow * \frac{B_2}{2} \Rightarrow kn/m}$ → توزيع W_u على البيم .

ملاحظات :-

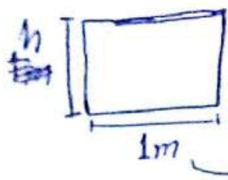
بما ان :- $\left(\text{Stiffness in a beam} \; k = \frac{4EI}{l} \right)$ ⇐ الاثقال تروح عالبيم اللي فيه k اكبر و L اقصر .

لذلك دائماً اللـود يتوزع بالاتجاه القصير .

Section a-a



Section b-b

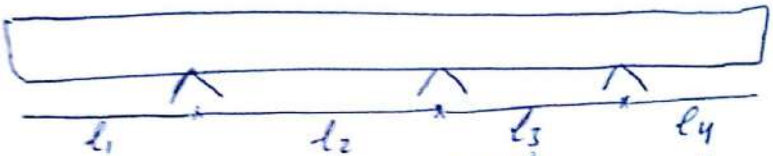


لأن تصميم (slab) راح يكون تصميم بالنسبة لمتر واحد وباقي الـ(slab) ينطبق عليه.

* h_{min} → (table 9.5 a):-

Go to the section of the slabs in the table:-

for example h_{min} for this slab:



l_1 ⇓ Cantilever	l_2, l_3 Simply supported	l_4 ⇓ Cantilever
So, $h_{min} = \frac{l_1}{10}$	$l_{min} = \frac{l_2}{28}$; $l_{min} = \frac{l_3}{28}$	$h_{min} = \frac{l_4}{10}$

بعد ما نوجدهم ← كلهم نحتاج الأكبر وتكون هي قيمة h_{min}

* Concrete Cover:-

$\boxed{Cover = 20\,mm}$ for normal exposure.

$$A_s = \frac{M_u}{\phi f_y j d}$$ where $j = 0.95$ always for slabs (wide comp. area)

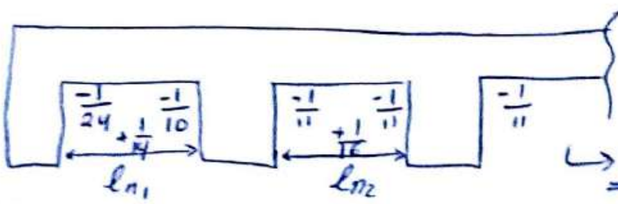
* ACI-moment and shear coeficient for analysis and design of non-prestressed one way slabs and contineous beams:-

$$M_u = C_m (w_u l_n^2) \quad ; \quad V_u = C_v \left(\frac{w_u l_n}{2} \right)$$

where C_m & C_v can be gotten from the fig. from ACI code

Ex:-

C_m



معطى بالامتحان.

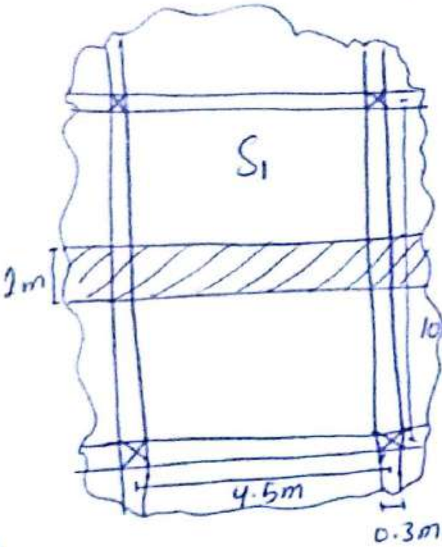
→ if Cantilever use $\frac{W_u l^2}{2}$

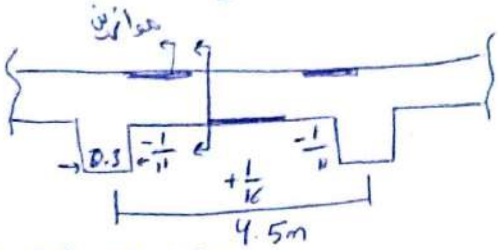
But this can be used only if:-

1- there are two spans or more.
2- spans are approximately in length < 20% diffrence.
3- Uniformaly distributed load.
4- $LL \leq 3DL$
5- member are prismatic (the same dimensions)

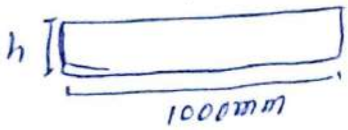
example:- $f'_c = 28\,MPa$; $f_y = 414\,MPa$ $W_{LL} = 4\,kN/m^2$

$W_{DL} = 3\,kN/m^2$ excluding self wet.





$l_n = 4.5 - 2(0.15) = 4.2\,m$



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

sol:-

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

* estimate h_{min}:- both end cont. $\Rightarrow h_{min} = \frac{l}{28} = \frac{4500}{28} = 160\,mm$

you can $\boxed{h = 180\,mm}$

⊖ compute factorial load.

$W_u = 1.2DL + 1.6LL$; self wet. $= 24\,\frac{kN}{m^3} \times \underbrace{0.18}_{h} = 4.32\,kN/m^2$

$W_u = 1.2(3 + 4.32) + 1.6(4) = \boxed{15.18\,kN/m^2}$

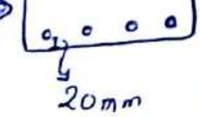
⊛ M_u

$M_u(-ve) = \frac{W_u l_n^2}{11} = \frac{15.18(4.2)^2}{11}$

$= 24.34\,kN.m$

try use No. 16

بالسؤال لازم يحدد لك نوع الحديد المستخدم



$\Rightarrow d = 180 - 20 - \frac{16}{2} = 152\,mm$

$A_s = \frac{M_u}{\phi f_y j d} = \frac{24.34 \times 10^6}{0.9 \times 414 \times 0.95 \times 152}$

$\Rightarrow A_s = 452.4\,mm^2$

Check

$A_{s\,min} = 0.0018\,bh = 0.0018 \times 1000 \times 180$

$= 324\,mm^2$

$A_s > A_{s\,min} \Rightarrow$ ok

$\Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{452.4 \times 414}{0.85 \times 28 \times 1000}$

$= 7.87\,mm$

do etrations bet. a and A_s to reach more convergance in values.

$\Rightarrow A_{s\,new} = \frac{24.34 \times 10^6}{0.9 \times 414 \times (152 - \frac{7.87}{2})} = 441.2\,mm^2$

$a = \frac{414.2 \times 414}{0.85 \times 28 \times 1000} = 7.67$

$A_{s\,new} = \frac{24.34 \times 10^6}{0.9 \times 414 (152 - \frac{7.67}{2})} = 440.9\,mm^2$

if you find a $a = 7.67\,mm$

then $\boxed{A_{s\,required} = 440.9\,mm^2}$

هنا مافي داعي اعمل check للـ ε_s / لان قيمة a صغيرة مقارنة بـ h لذلك ستكون كبيرة (ε_s)

$$\boxed{\text{Spacing } (s) = \frac{1000\,A_b}{A_s}}$$

where : A_b: area of one bar

1 No. 16 $\quad A_b = 199\ mm^2$

↳ from the table

$$S = \frac{1000 * 199}{440.9} = 451\,mm$$

Use $\boxed{S = 450\,mm}$ → نقرب للأصغر لأنو كلما صغر ال spacing بعطينا A_s اكبر.

* per ACI-code:-

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

Use $\boxed{S = 450\,mm}$ لو طلعت S أكبر من S_{max} استخدم S_{max}

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

$$A_s = \frac{M_u}{\phi f_y jd} = \frac{16.74 * 10^6}{0.9 * 414 * 0.95 * 152} = \boxed{311\ mm^2}$$

Check $A_{s\,min}$.

$$A_{smin} = 0.0018(1000)(180) = 324\ mm^2$$

$A_s < A_{smin}$; So use A_{smin}

* ما في داعي اعمل الـ iteration طالما وانا استخدم A_{smin}. و كذلك لا داعي لعمل (ϵ_s check) لانها بالتأكيد رح تعطي (tension controlled)

$$S = \frac{1000 * 199}{324} = 614.2\,mm$$

$$S_{max} = \begin{cases} 3(180) = 540\,mm \\ 450\,mm \checkmark \end{cases}$$

$\because\ >$ $\quad$ Use $\boxed{S = 450\,mm}$

* per the ACI-code:-

* Shrinkage and temprature reinforcement is required perpendicular to spans of the slabs.

(longer dimension) → A_{smin}

So

$A_{smin} = 0.0018bh = 324\,mm^2$ per each meter

$$S = \frac{1000 * 199}{324} = 614.2\ mm$$

$$S_{max} \text{ smaller of} = \begin{cases} 5h = 900 \\ 450\,mm \checkmark \end{cases}$$

Use $\boxed{S = 450\,mm}$

Since $S > S_{max}$

you can use another No. of reinforcement for longer dim.

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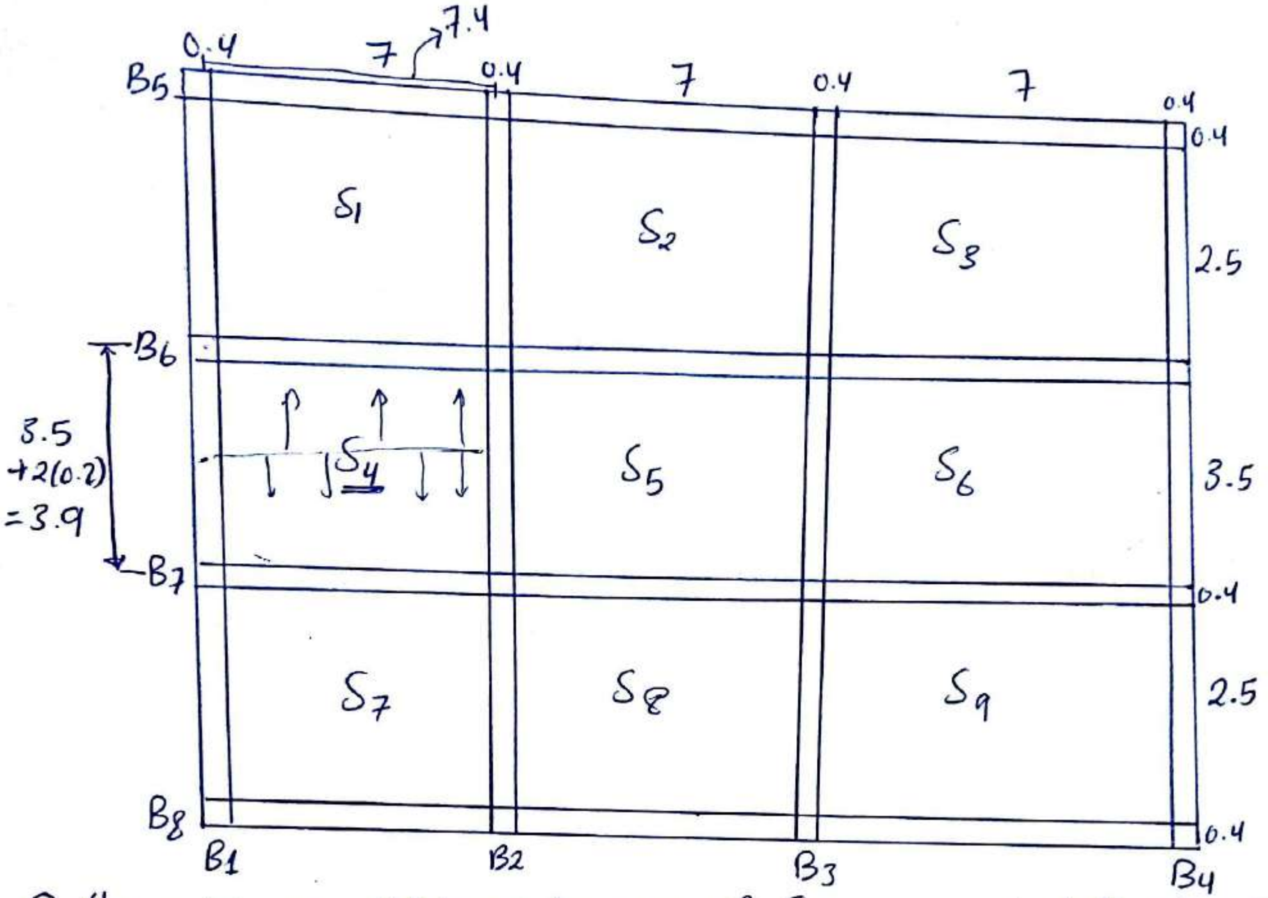
* A_{smin}

$A_{smin} = 0.0018bh \rightarrow$ Grade 60

$A_{smin} = 0.002bh \rightarrow$ Grade 40 prob → past exam

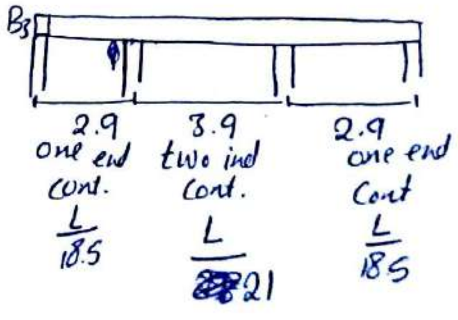
the drawing below shows a system of a solid one way slabs, beams and columns. all dimension are in m (face to face) the loading in the slabs including DL (including self wet) = 3 kN/m², LL = 2.5 kN/m². Use $f'_c = 25$ Mpa, $f_y = 420$ Mpa.



① the minimum thickness in mm of S_4 to avoid deflection calculation is most nearly:-

∵ two way cont. So $h_{min} = \frac{L}{28} = \frac{3.9 * 1000}{28} = \boxed{139.28 mm}$

② the minimum thickness in mm of beam B_3 to avoid deflection calculations is:-



h_{min} (one end cont.) $= \frac{2.9 * 1000}{18.5} = 156.75$ mm

h_{min} (two end cont) $= \frac{3.9 * 1000}{21} = 185.7$ mm

⟹ $\boxed{h_{min} = 185.7 mm}$ the Largest

Scanned by CamScanner

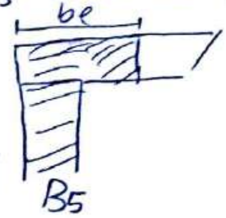
the ultimate load in kN transfered from slabs to beam B_6 is most nearly:-

$\because W_u = 1.2\,DL + 1.6\,LL = 1.2(3) + 1.6(2.5) = 7.6\ kN/m^2$

W_u in B_6 = $W_u * \frac{3.9}{2} + W_u * \frac{2.9}{2}$

$= 7.6 * \frac{3.9}{2} + 7.6 * \frac{2.9}{2} = \boxed{25.84\ kN/m}$

(4) the effective flange width in mm of Beam B5 is most nearly assum ($h_f = 200mm$)



$$b_e = \text{smaller of} \begin{cases} b_w + \frac{l_n}{2} = 400 + \frac{2500}{2} = 1650mm \\ b_w + \frac{L}{12} = 400 + \frac{7400}{2} = 1016.66mm \\ b_w + 6h_f = 400 + 6(200) = 1204\ mm \end{cases}$$

then $\boxed{b_e = 1016.6mm}$

(5) If the ultimate moment on slab S_5 is 20kN.m and the slab thickness is 200mm, using No. 16 provid a complete design of the slab (in the short and long dimension)

$M_u = 20\ kN.m$

$h = 200mm$ → لو ما جابهالك ترجع من الجدول تجيبها

$d = 200 - \frac{20}{2} - \frac{16}{2} = \boxed{172mm}$

$A_s = \frac{M_u}{\phi f_y jd} = \frac{20*10^6}{0.9*420*0.95*172}$

$\Rightarrow \boxed{A_s = 323.8\ mm^2}$

$A_{s_{min}} = 0.0018\,b\,h$

$= 0.0018(1000)(200) = 360\ mm^2$

$\because A_s < A_{smin}$ then us $\boxed{A_{s_{min}} = 360mm^2}$

$S = \frac{1000 * A_b}{A_s}$; $A_b = 199\ mm^2$ from table

$= \frac{1000 * 199}{360} = 552.77mm$

$$S_{max} = \text{smaller of} \begin{cases} 3(200) = 600mm \\ 450\ mm \end{cases}$$

take $\boxed{S = 450\ mm}$

*longitudal dimension

use $\boxed{A_{s_{min}} = 360\ mm^2}$

$S = \frac{1000 * 199}{360} = 552.77\ mm$

$$S_{max} \begin{cases} 5(200) = 1000 \\ 450 \end{cases}$$

Use $\boxed{S = 450mm}$

(6) If slab S_5 is reinforcement with No.14 spaced at 300mm the provided areas of steel in mm^2 per meter width of the slab is most nearly:-

$\because A_{b(No.14)} = \frac{\pi}{4}(14)^2 = 153.93mm^2$

$\because S = \frac{1000 A_b}{A_s} \Rightarrow 300 = \frac{1000(153.93)}{A_s}$

$\Rightarrow \boxed{A_s = 513.12\ mm^2}$