الإمبراطور

في

الخرسانة المسلحة 1

عز الدين عاشور

بسم الله الرحمن الرحيم

الدوسية الاتية دوسية الخرسانة المسلحة واحد
عبارة عن المحاضرات الصفية بالترتيب للدكتور بلال أبو الفول ...
بالإضافة إلى شرح لشرح الدكتور وتلخيص لأساليب الحل
وبعض النصائح إضافة لبعض أسئلة السنوات المضافة لمادة الفيرست أما مادة سنوات مادة السكند والفاينل سيتم تزويدها بمرفق خارجي لاحقا ...

إضافة للتعديلات الطفيفة على المادة التي حصلت في اخر فصل ...

الرجاء التنبه وعدم الاتباع العمياني للدوسية فكل بني ادم خطاء ...

كل تمنياتي لكم بالتوفيق والسداد
والله يرزقكم الA+ كل الحب لكم

عز الدين عاشور

للاستفسار عن أي شي غير واضح بالدوسية تواصل معي عبر الفيس
https://www.facebook.com/ezrawille

Advantages of concrete

1) Relatively low cost material
2) Fire resistance 1 to 3 hours without special fire proofing
3) Suitability of material for structural and architectural functions
4) Rigidity
5) availability of material

disadvantages of concrete

1) Low tensile strength
2) Forms and shoring ⟶ يتطلب extra time and work
3) Relatively low strength per unite weight or unite vloume
4) Time dependent Volume changes ⟶ Drying Shrinkage due to Hydration
   ⟶ Creep (Deformation under sustained load)

Sources of Uncertainty

كلما زادت الـUncertainty
زاد الـSafety Factor

1) Actual load magnitude and distribution may differ from assumed in the design
2) Asumption and simplification in the analysis may result in different internal forces
3) Aactual behavior may be different
4) Actualmember Dimension may differ from those in the desing
5) Reinforcment may not be in its proper position
6) Actual material strength may be different from specified in the design

Safety Philosophy

$$\Phi\, S_n >= \gamma\, Q_d$$

Q : strength reductin factor < 1(always)

γ : Over load factor > 1 but it can be < 1 (not always more than 1) مهم

Sn: nominal strength

Qd : design load or actual load or unfactoral load

Load factor and combaination per ACI code

$U = (1.2)DL + (1.6)LL$ → Principle Variable load

$U = (0.9)DL + (1.6)WL$ → Combaination action variable load

DL : Dead load مثل وزن المبنى

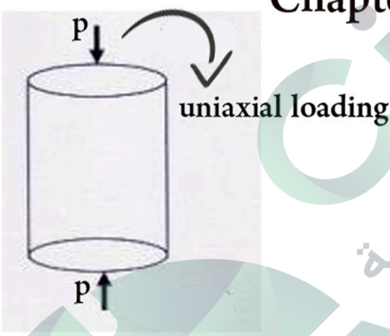
U : ultimate load

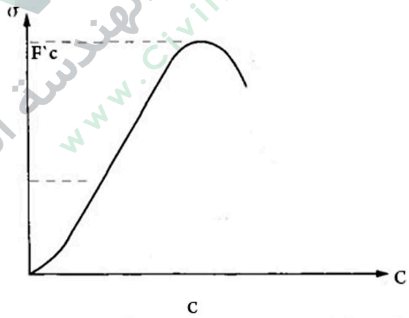
LL : Live load

في المعادلة الثانية أصبح الـsafety للـDL 0.9
(ليش قللنا لما صار مع الـwind)
لأن تأثير DL عمودي بينما WL أفقي فالـcombaination أقل

لماذا الـ safety للـ DL أقل من الـ LL ؟
لأن DL ثابت تقريباً أما الـLL ممكن يتغير

Chapter 2 :(Materials)





Stages of micro loading and failure in concrete subject to uniaxial loading :

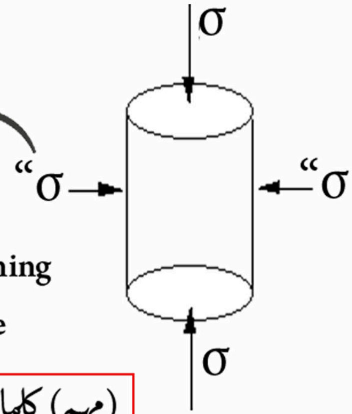
1-Shrinkage of the paste durning hydration -No load bond cracks بين الـpaste والـagg

2-At stresses up to 30% to 40% of compressive strength → Bond Cracks
حولين جزيئات الـAgg

3-Discontinuity Limit at stresses up to 60% of compressive strength (Mortar Cracks)
بين جزيئات الـagg
بعد ذلك يبدأ → stable crack progation

4-Critical stress At 75% of the Ultimate load عندها No. of Mortar Cracks increase
→ Unstable crack progation → Fewer undamged portions to carry the load
حتى لو أوقفت القوة المؤثرة لن تتوقف التشققات بعد هذه المرحلة

Triaxial loading



Confining Stress

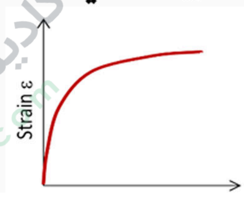
Gives more stress than Uniaxial because confining resist indirect stress that try tp spill the sample

Compressive stress ازداد الـ Confining stress كلما زاد (مهم)

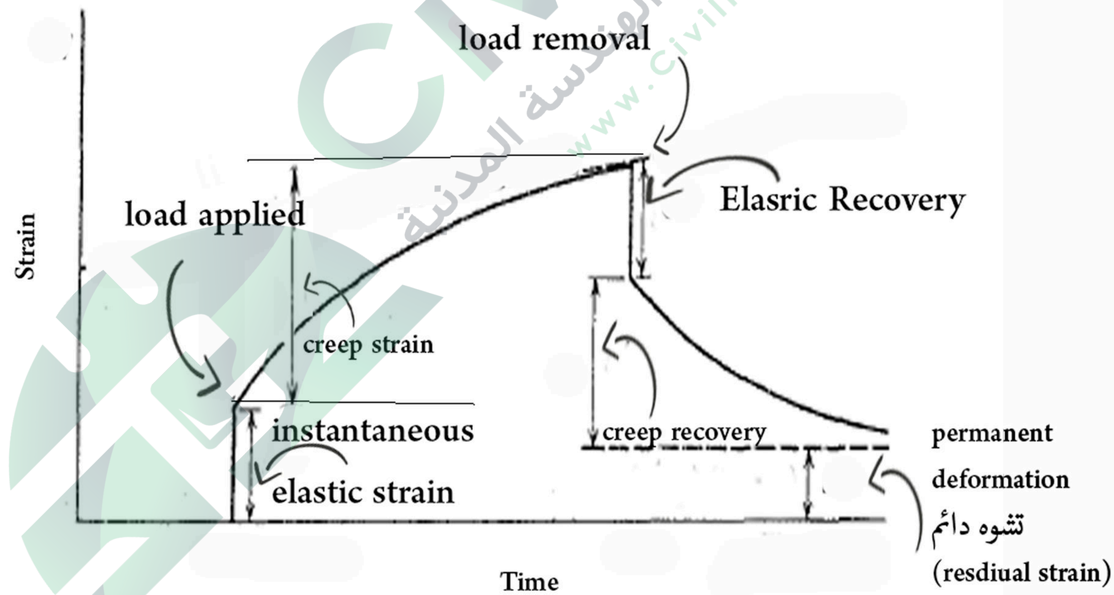
الـConfining stress يقاوم الTension الناتج من الـComp stress

Time dependent Volume Changes → Shrinkage

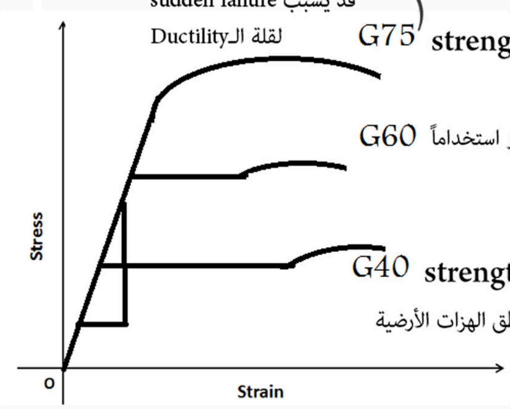
→ Creep (Strain with time لما يكون عندي)



Creep Test



Reinforcment



G75 strength وأعلى ductility أقل

G60 الأكثر استخداماً

G40 strength وأقل ductility أعلى

يستخدم في مناطق الهزات الأرضية

كلهم الهم نفس الـmodulas of elasticity

يطبق Hooks law

$\sigma = \varepsilon \times E$

فقط في منطقة الـelastic region

Chapter Three

The design process

Objectives Of Design -Structure Should Satisfy

1-Appropriateness : Designed to serve it intended use

2-Economy : overall Cost should not exceedes client's budget

متوقع

3-Structure Adquency (كفاءه) :be strong enough to support al anticipated loads and it should not diflect , tilit ميلان ,vibrate or crack in a way that affects its use-fullness

4-maintainability قابلية الصيانة :Minimum and simple maintence procdure

The Design process

Phase 1 :Defining client's needs and priorites

Phase 2 :Development of project Concept → Preliminary cost estimates
→ No. of possible Layouts

Phase 3 :Design of indivdiual systems

Structural Saftey : sources of uncertainity تضاف للـsources السابقة

consequences of failure : -loss of life -cost of clearing depris

- cost to society in lost time

Limit State and the Design of Reinforced concrete

limit state : When a structure or an element become unfit for it intended use it said it reached the limit state

Groups of limit state

1-Ultimiate limit state : structure collapse of part or all the structure

Examples : 1-loss of equilibrium 2-Rupture of critical parts of the structure

3-Progressive collapse انهيار تدريجي 4-instablity due to deformation (buckling)

5-fatigue سببه لوود متكرر 6-formation of plastic mechanisim

2-servicability limit state :Involves distruption of the functional use of the strucuter

Examples : 1-excessive deflection 2-undesirable vibration 3-excessive crack width

مثل تعطل سكة الحديد

3-Special limit state : invovles damage or failure due to abnormal conditions

1-extreme earthquicks 2-fire 3-explosions 4-vehicular collisions

انتبه الزلازل المتوقع حدوثه وليست ال extreme تقع ضمن Ultimate limit state وليس special

limit state desing process

- identifiction of all anticipated modes of failure
- Determination of all acceptable level of safety per ACI code

Design process specified in ACI code

1-strength design $\Phi Sn \geq \gamma Qd$

2-Working stress design $\Phi Sn \geq Qd$ ← unfactoral load

working load or structural load

3-platic design method ,limit design ,capacity design

Loading and actions

- Permanent (weight of the structure)
- accidentail 1-explosions 2-collisions 3-vehicualr
- Variable
  - Short time load (people)
  - sustained load (long durtion)

Chapter Four

Flexural basic concept (Rectangular beam)

$$\Phi\, Mn \geq Mu$$

ملاحظة :

Beam --يتعرض---->flexural and shear

coloumns --يتعرض--->axial and bending

Φ :strength reduction factor (0.6 يتراوح بين 0.9)

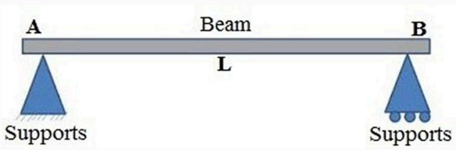
Mn : nominal moment capacity

Mu : Required ultimiate moment

Φ Mn : Design moment capacity

مهم جدا تكون عارف هاي المعلومات

simply supported beam

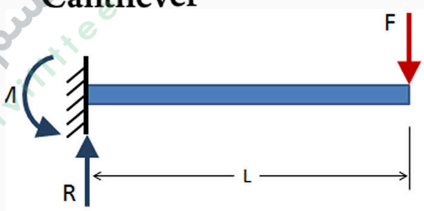


$Mu = \frac{wL^2}{8}$ ماكس مومنت

$V_{MAX} = \frac{wL}{2}$ ماكس شير

مومنت موجب والتسليح سفلي

Cantilever



$Mu = \frac{wL^2}{2}$ ماكس مومنت

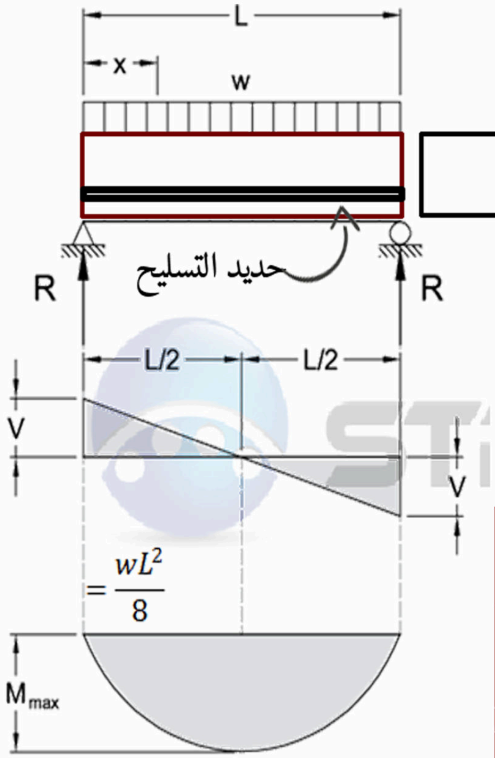
$V = wL$ ماكس شير

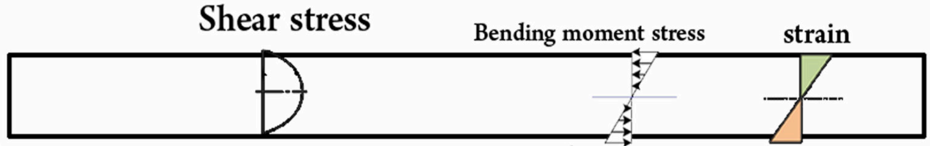
مومنت سالب والتسليح علوي

FBD

SFD

BMD





صفر عالاطراف
وماكس علـN.A

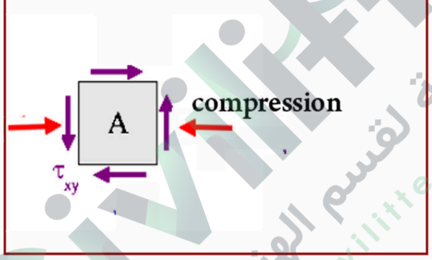
ماكس عالأطراف
وصفر علـN.A

$$\tau = \frac{VQ}{It}$$ مش حفظ

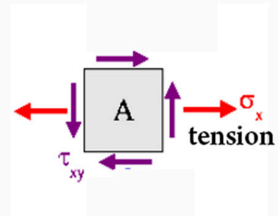
N.A : Neutral axis

بكون بلستروید اذا كان سمترك

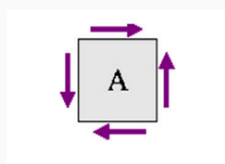
لو أخذنا element فوق الـN.A



لو أخذنا element تحت الN.A

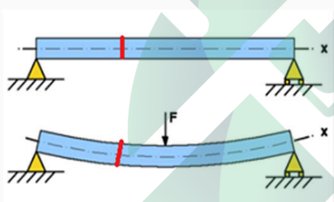


لو أخذنا element على الـN.A



Basic assumption in flexural theory

1-Sections perpendicular to the axis of bending that are planes before bending remains planes after bending



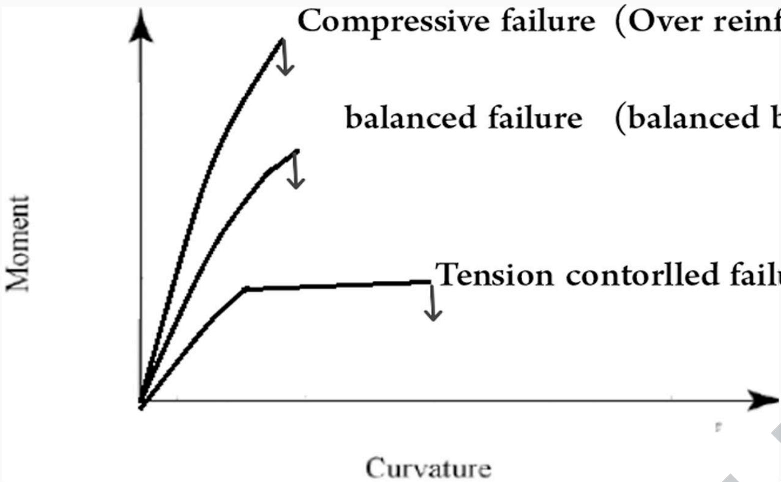
2-strain reinforcment = Strain of the concrete at the same level (perfect bonding)

3-tensile strength of concrete is neglected in flexural calculations

4-Concrete is assumed to fail when the maximum compressive strain reaches a limiting value

$$\varepsilon_{cu} = 0.003$$

Flexural failure may occur in three ways



Compressive failure (Over reinforced concrete) $\varepsilon_s < \varepsilon_y$

balanced failure (balanced beam) $\varepsilon_s = \varepsilon_y$

Tension contorlled failure (under reinforcment) $\varepsilon_s > \varepsilon_y$

سترين حديد التسليح في منطقة التنشن ε_s

$\varepsilon_c = \varepsilon_{cu} = 0.003$ (الذي يحدد أن البيم وصل للـfailure)

إياك تخربط بين ε_c و ε_y أوعى تحكيلي $\varepsilon_y = 0.003$

انتبه سؤال TوF

Over reinforced beam means that the beam has more reinfocment than required

False

Strain limit method for analysis and design

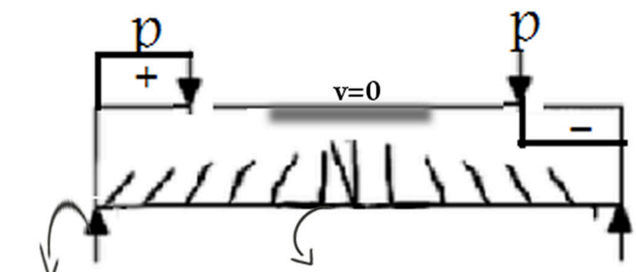
there are four types ofbeams depending on anticipated modes of failure

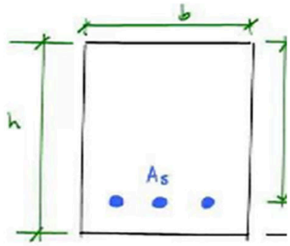
compression controlled beam compression controlled failure	Tranzition controlled beam Tranzition controlled failure	Tension controlled-beam Tention controlled failure

$\varepsilon_s = \varepsilon_y$
balanced beam

$\varepsilon_s = 0.005$

Flexural behavior





under pure shear

الكراك بزاوية 45 هنا

max moment

من هنا يبدأ الكراك

beams failed as a result of the crushing of concrete at the top of the beam

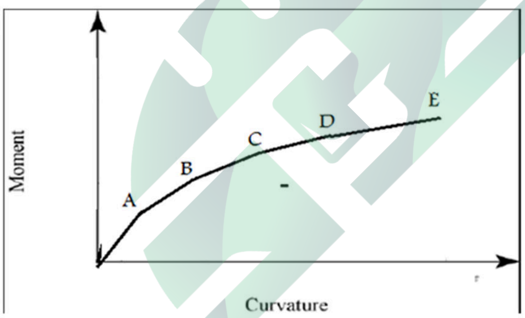
Stage A : before cracking

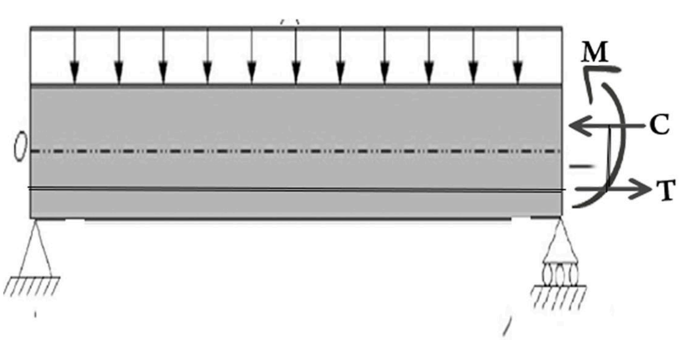
Stage B : Cracking (Start from the max moment)

Stage C : After cracking before yielding of reinfocment when stress transmit from concrete to steel there will be micro cracking

Stage D : yielding of reinforcment curvture increase rapidly with very little increase in moment من أسباب تسليح المنطقة العلوية

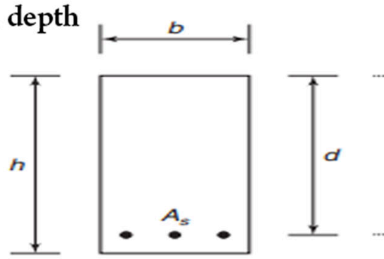
Stage E : Failure





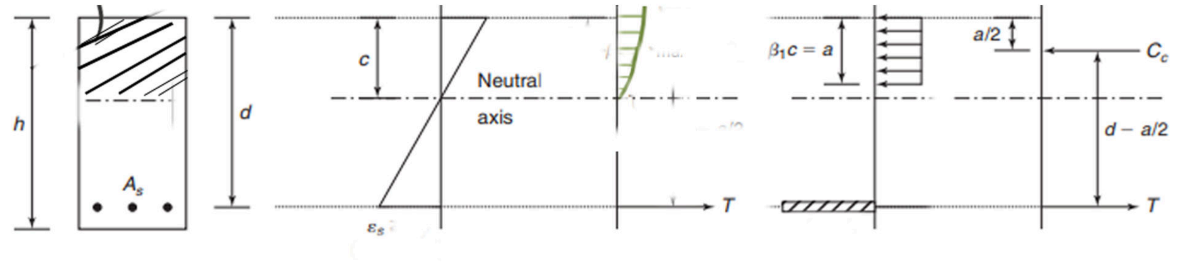
المسافة بين C وT اسمها moment arm

h : over all depth



d: effective depth

من منتصف حديد التسليح لاخر البيم فوق



(a) Singly reinforced section. (b) Strain distribution. (c) Stress distribu (d) Internal forces.

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

$$C_c = 0.85\, f'_c\, ba$$

Cc , 0.85 f'c assumed to be easy with calculations and it calls equivalent stress block

مهم بيجي سؤال دائره

$$T = A_s f_s$$

If $\varepsilon s < \varepsilon y$ then $fs = E\,\varepsilon_s$

If $\varepsilon s > \varepsilon y$ then $fs = fy$

$$a = \beta\, c$$

قيمة Φ(حسب قيمة εs)

$\Phi = 0.9$ if $\varepsilon s >= 0.005$
(tension controlled)

$\Phi = 0.65 + (\varepsilon s - 0.002)250/3$
if $\varepsilon_y < \varepsilon_s < 0.005$
(Tranziton)

$\Phi = 0.65$ if $\varepsilon_s <= \varepsilon_y$
(Compresion and balanced)

قيمة β(حسب f'c)

$\beta = 0.85$ if f'c <= 28Mpa

المعادلة مش حفظ

$\beta = 0.85 - 0.05(f'c - 28/7)$
if 28Mpa < f'c <= 56Mpa

$\beta = 0.65$ if f'c > 56Mpa

Minimum amount of Tension renfocement

مصطلحات بتفيدك :

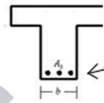
لما يكون عندي min باخذ الـ larger of

ولما يكون عندك max باخذ الـ smaller of

As min (Larger of) $= \dfrac{0.25\sqrt{f'_c}}{f_y} b_w d,$

$= \dfrac{1.4 b_w d}{f_y}$

في الـ rectangular beams b w = b

أما بالـT بيمز 

مهم : وضعت هذه الكميات As min لإنه عند انتقال اللود من الـConcrete للـsteel بكون مفاجيء Sudden لذلك وضعت من أجل

To guarantee safe transfer of tension focre (tension مليون خط تحت) from Concrete to Steel

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خطوات حل سؤال الـanalysis (إفهم ولاتحفظ)

نعوض كل من الاتي بالوحد الاتية :(لا تحول)

f'c بالـMpa

b بالـ mm

As بالـmm2 والـfy بالـ Mpa

1) نفرض أن εs > εy وبالتالي fs =fy

2)نساوي (T=Cc) ------> (0.85f'c ab = AsFy)

دخلها باالالة الحاسبة كمعادلة بمجهول x لإيجاد a رح تطلع a بوحدة mm

3) نجد C من المعادلة فوق وانتبه لقيمة بيتا حسب F'c ثم نجد εs

4) نتحقق من الفرض تاعنا هل (εs >εy) ---> نعم ----> خطوة 5

-----> لا ----> ارجع للخطوة 1 وضع بدل Fy -->Fs حيث (fs = E εs)

ثم جد و a وشييك لـεs للتأكد لازم تطلع εs <εy

ملاحظة لما تستخدم fs رح يكون قيمة c مجهولة فيها بالتالي بصير معك معادلة بمجهول C على الطرفين

استخدم الالة الحاسبة لحل معادلة الهذي بدون ما تنقل شي ولا تحفظ المعادلة التربيعية الي رح يكتبها الدكتور

عوض قيمة $E = 200*10^3$ بهذا الشكل بالمعادلة حتى تطلع c بالـmm

(As * E * 0.003 (d-c)/c = 0.85 f'c b * c * β)

5)نجد Mn حيث (Mn = إحدى القوتين في المسافة بينهما)

في حالة Rectangular beam

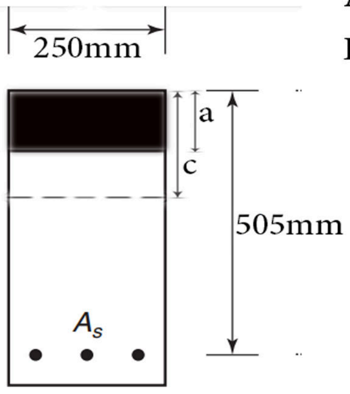
$$M_n = T\left(d - \frac{a}{2}\right) = C_c\left(d - \frac{a}{2}\right)$$

إضرب الناتج بـ10^{-6} حتى يعطيك بالـKn.m

6) جد قيمة فاي حسب قيمة εs ثم ΦMn

7) Chech As min

Example 1



As = 1530 mm2   f'c=20 Mpa   fy=420 Mpa

E=200Gpa   find the design moment capacity

Sol

assume $\varepsilon s >= \varepsilon y$ then fs =fy

T=Cc-----> 0.85 f'c a b = As fy

a*250*20*0.85 = 420 * 1530

a= 151.2 mm   c=a/β which β = 0.85

c=177.88 mm

now check εs ------> εs = 0.003 *(d-c)/c   εs=0.0055

chech εy=fy/E = 420/(200*10^3) =0.0021

εs > εy (yielded) assumption ok

Mn = T(d-a/2) = Asfy*(d-(a/2)) =1530 *420 * (505-0.5*151.2)=275.9 Kn.m

حسب قيمة εs كما سبق Φ εs > 0.005 Φ=0.9 ΦMn=248.3 Kn.m

Now check As min = 336 mm2 or 420 mm2 (the larger) and 420< 1530 then Ok

ملاحظة ΦMn=Mu

لو طلب W

$\Phi M_n = \frac{W l^2}{8}$

نفس السؤال السابق ولكن غيرنا الـ As=3060 mm2

Cc=T 0.85 f'c a b = As fy   a =302.4mm   c=355.76mm

check εs =0.003 *(d-c)/c = 0.001512 < εy=0.0021

assumption not ok

الان نرجع للمعادلة T=Cc ونضع بدل Fy نضع Fs حيث Fs= E * εs حيث يكون المجهول بالمعاجلة هو c

0.85 f'c * c *β *b = As fs

$c*0.85*250*20*0.85 = \frac{(505-c)}{c}*0.003*1000*200*3060$ c=312.65mm

a=c*0.85 =265.76mm

دخلها بمجهول x بدل c بالالة الحاسبة

εs = 0.00184 <εy then ok

Mn=Asfs(d-(a/2))=or 0.85 f'c a*b*(d-(a/2))=
420 kn.m

تابع

$\varepsilon s < \varepsilon y$ then $\Phi = 0.65$ $\Phi Mn = 273$ Kn.m

Check As min then ok

ملاحظات هامة جداً :

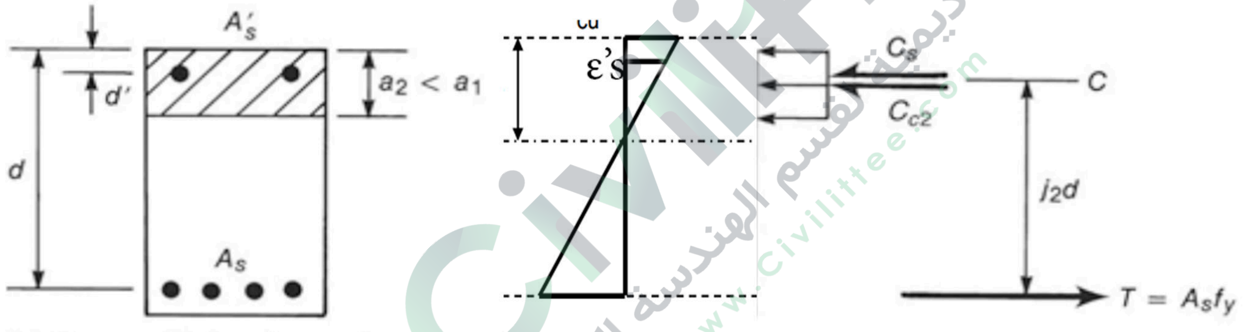
زيادة كمية الحديد في منقطة التنشن لها (significant effect) على (Mn) nominal moment capacity

ولها تأثير قليل على design moment capacity

زيادة الحديد تقلل من εs

(ΦMn)

زيادة الحديد في منطقة الـcompression كما سترى لاحقاً لها تأثير كبير على الـDesign moment capacity

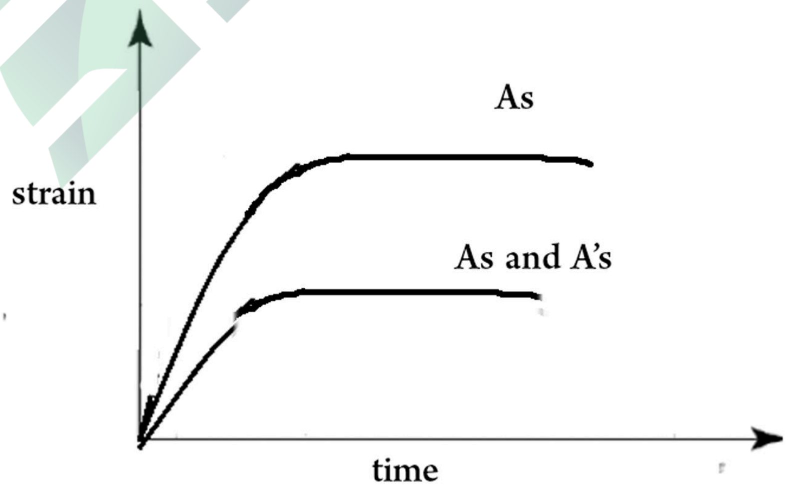


(b) Beam with tension and compression steel.

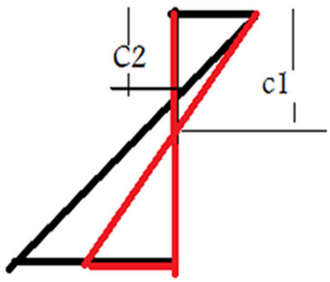
Reasons of doubly rein forcment

1-Reduce sustained load deflection (the creep).

يساعد الخرسانة في منطقة الـcomp على تحمل الضغط بالتالي سيكون الانحناء أقل



2-Increase Ductility



C2 : بعد النويترال اكسس في حالة الـdoubly reinfocrment

C1 : بعد النويترال اكسس بحالة الـsingle reinforcment

حيث أن استخدام الحديد في منطقة الكومبرشن يؤدي لارتفاع الـN.A تجاه منطقة الكومبشرن

3-change mode of failure

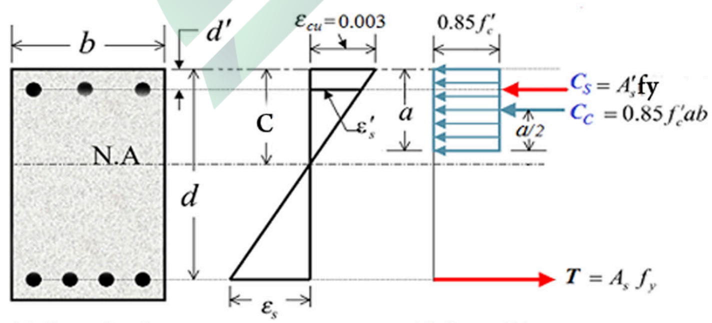
from compresion / balance / tranzition to tension controlled due to increase of ƐS

4-Fabrication Ease

providing small bars in concrete of the stirrups to hold the steel in place

إذا سلحت منطقة الكومبريشن لأجل هذا السبب فإن قيمة As’ تهمل ويعامل البيم كـsingle reinforcment

Analysis of beams with tension and compression reinfocment



(a) Cross Section (b) Strain Diagram (c) Stress Diagram

$$\varepsilon'_s = 0.003\left(\frac{c-d'}{c}\right)$$

طرق حل الـDoubly reinforcment

الطريقة الاأولى :

1) نفرض أن $\varepsilon s >= \varepsilon y \rightarrow fs = fy$ and $\varepsilon's >= \varepsilon y \rightarrow fs' = fy$

2) $T = Cs + Cc$ حيث $As\ fy = 0.85\ f'c\ a\ b + As'\ fy$ ثم نجد a و c

3) نشيك كالتالي (ركز منيح وافهم فهم تلاقيه سهل)

check $\varepsilon's$

- o.k
  - Check εs
    - o.k
    - not ok
      - ارجع وحط بدل fy ضع fs (في حد Cc فقط) انتبه !!!
      - حيث $fs = E\ \varepsilon s$
      - ثم جد c ثم a ثم شيك مرة أخرى ولكن انتبه هون انه تشييك $\varepsilon s < \varepsilon y$ هو ok
      - $As\ fs = 0.85\ f'c\ a\ b + As'\ fy$
- not ok
  - ارجع وحط بدل fy ضع f's حيث $f's = E\ \varepsilon's$
  - ثم جد c ثم a ثم شيك مرة أخرى
  - $As\ fy = 0.85\ f'c\ a\ b + As'\ f's$
  - Check εs
    - ok
    - not ok
      - ارجع وحط بدل fy ضع fs
      - حيث $fs = E\ \varepsilon s$
      - ثم جد c ثم a ثم شيك مرة أخرى
      - $As\ fs = 0.85\ f'c\ a\ b + As'\ fs$

4) بعد ما شيكنا وكل شي OK نجد Mn $Mn = Cc(d-(a/2)) + Cs\ (d-65)$

$$Mn = 0.85\ f'c\ ab * (d-(a/2)) + As\ (f's \text{ or } fy) * (d-65)$$

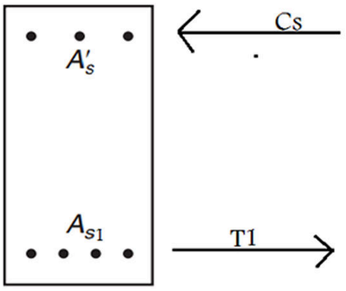
5) نجد Φ (انتبه $\varepsilon's$ مالها دخل بنوع الـfailure) الذي يحدد نوع الـfailure هو εs

نجد Φ حسب εs ثم ΦMn ثم شييك As min فقط حيث A's مالها Min

الطريقة الثانية (مهة بالتصميم)

نقسم البيم لقسمين

قسم نهمل فيه الكونكريت



$Cs = T1$

في حالة الاثنين yielded

$As1\ Fy = As'\ Fy$

then $As1 = As'$

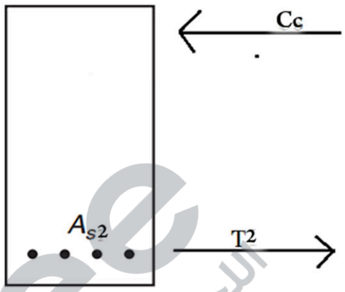
حيث

$As = As1 + As2$

نرجع هون

ونجد As2

قسم نهمل فيه الحديد الـcomp



$Cc = T2$

$AsFy = f'c\ a\ b\ 0.85$

جد c و a ثم شييك $\varepsilon's$ و εs

بعد هيك

$\Phi Mn1 = Cs\ (d-65)$

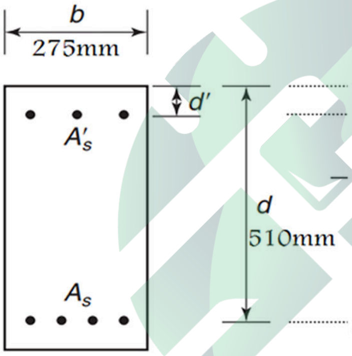
$\Phi Mn2 = 0.85\ f'c\ a\ b * (d-(a/2))$

$\Phi Mn = \Phi Mn1 + \Phi Mn2$

ثم Check As min

Example

اذا ما أعطاك d' لازم تعرف انها 65 mm



As'=852 mm

As=3060mm

f'c=20Mpa

fy=420Mpa

find desing moment capacity?

Sol : Assume $\varepsilon's >= \varepsilon y$ so that $f's = fy$

$\varepsilon s >= \varepsilon y$ so that $fs = fy$

$T = Cs + Cc \quad As\ fy = A's\ fy + 0.85\ f'c\ a\ b$ a=198mm

c=233mm

check $\varepsilon's = 0.003(c-d')/c = 0.0022 > \varepsilon y$ ok

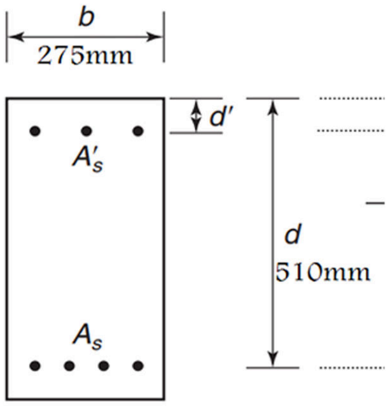
check $\varepsilon s = 0.003(d-c)/c = 0.0036 > \varepsilon y$ ok

$Mn = Cc(d-0.5*a) + Cs\ (d-d') = 0.85\ f'c\ a\ b *(d-0.5*a) + As'\ Fy\ (d-65) = 540$ kn.m

$\varepsilon s = 0.0036$ so Φ حسب العلاقة $\Phi = 0.65+(0.0036-0.002)250/3 = 0.783$

$\Phi Mn = 422$kn.m now check As min take the largest = 467.5mm2

As <Asmin then ok



same example with As'=1700mm

Assume $\varepsilon s >= \varepsilon y$ and $\varepsilon' s >= \varepsilon y$

T=Cc+Cs then a=122mm and c=143.5

now check $\varepsilon' s = 0.0016 > \varepsilon y$ not ok

$f's = \varepsilon' s * E = 200*1000*0.003*(c-d')/c$

$As\ fy = 0.85\ f'c\ \beta\ c + A's *200*1000*0.003*(c-d')/c$

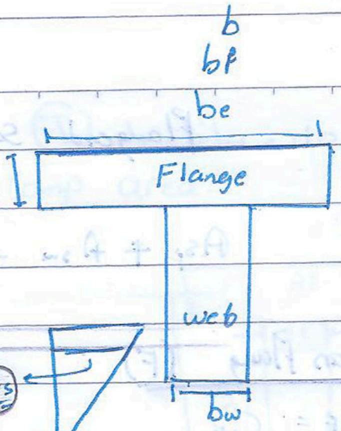
c =166.55mm a=141.57mm

$\varepsilon' s = 0.0018 < \varepsilon y$ then ok $\varepsilon s = 0.0062 > \varepsilon y$ then ok

$Mn = 0.85\ f'c\ a\ b\ (d-0.5*a) + A's\ f's\ *(d-d') = 563.7$ Kn.m

$\Phi Mn = 507.3$ kn.m

كما تلاحظ زيادة الحديد في منطقة الكومبريشن لها تأثير كبير على الـdesign moment capacity

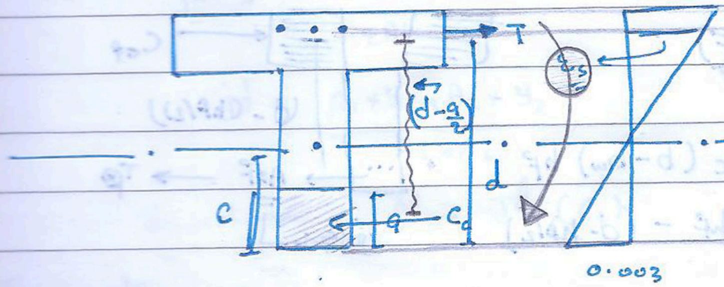


Flexural in T-Beam

(Flange in Tension)

دائماً تحت N.A الفلانج

Case I : • −ve Moment

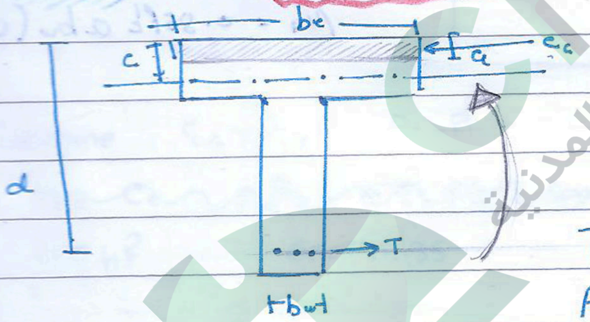


$T = C_c \longrightarrow A_s f_y = 0.85 f'_c \, a b_w$

$M_n = C_c \left(d - \frac{a}{2}\right)$ مثل ال Rectangular beams

Case II : +ve Moment $a < h_f$

بما أنه ال منطقة ال Comp كمان تعامل Rectan... مثل ال Rectangular beams

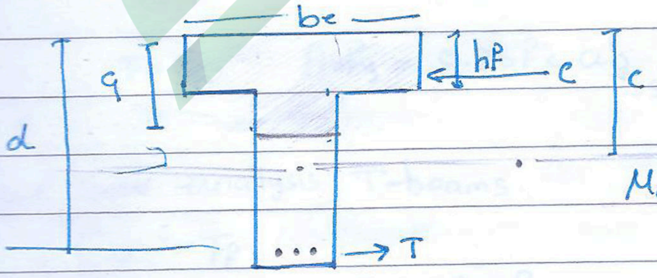


$T = C_c$

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

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

Case III : • +ve moment $a > h_f$



$T = C_c$

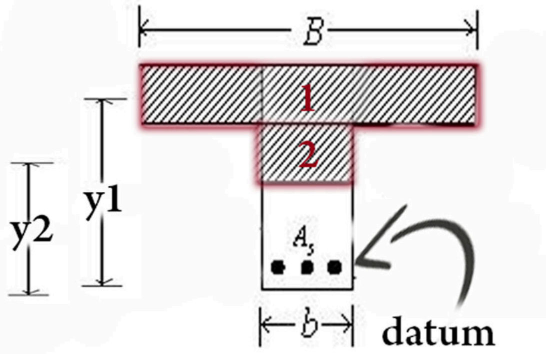
$A_s f_y = 0.85 f'_c \left(b_e h_f + b_w (a - h_f)\right)$

$M_n = C_c \, (jd) \longrightarrow$ Not $\left(d - \frac{a}{2}\right)$

$M_n = 0.85 f'_c (b - b_w) h_f \, (d - h_f/2) + 0.85 f'_c \, a b_w \left(d - \frac{a}{2}\right)$

طرق حل أسئلة الـT Beams

إيجاد السنترويد Jd : الطرق الثانية أفضل



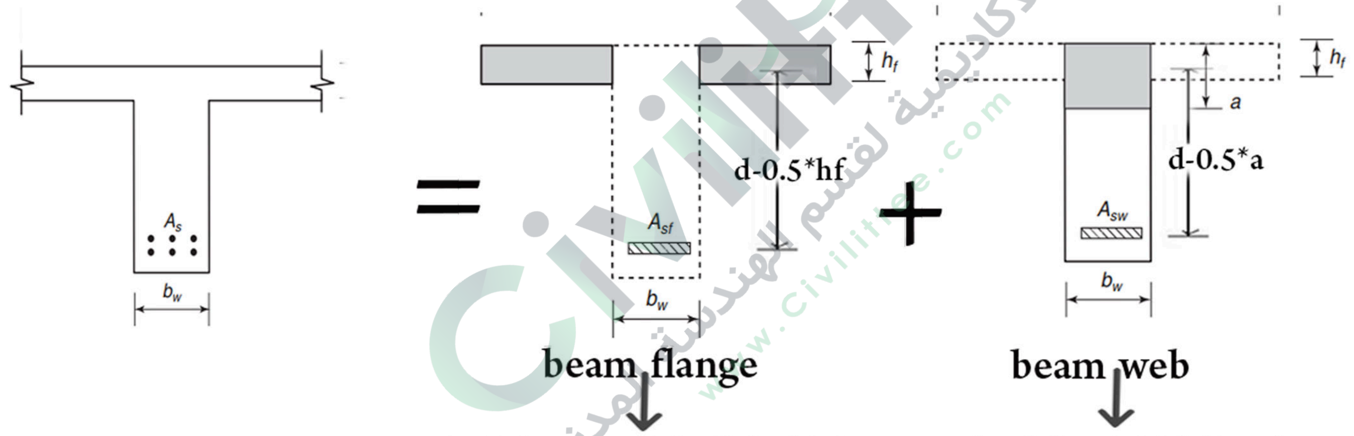
Y1:المسافة من الdatum لمنتصف المساحة 1

Y2:المسافة من الdatum لمنتصف المساحة 2

$$jd = \Sigma y_i A_i / (\Sigma A_i)$$

$$jd = (A1\ y1 + A2\ y2)/(A1+A2)$$ then $Mn = T\ jd$

الطريقة الثانية



beam flange

$Asf * fy = 0.85 * f'c\ (b-bw)\ hf$

$Mnf = Tf\ (d-(hf/2))$

or $Mn_f = 0.85 * f'c\ (b-bw)\ hf * (d-(hf/2))$

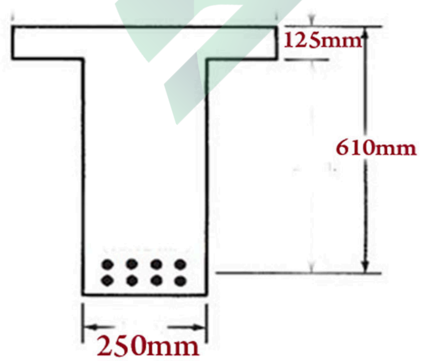
beam web

$Asw * fy = 0.85 * f'c\ a\ bw$

$Mnw = 0.85f'c\ a\ bw * (d-(a/2))$

Then $Mn = Mnf + Mnw$

Example $f'c = 20Mpa$ $fy=420\ Mpa$ $As = 3060mm2$

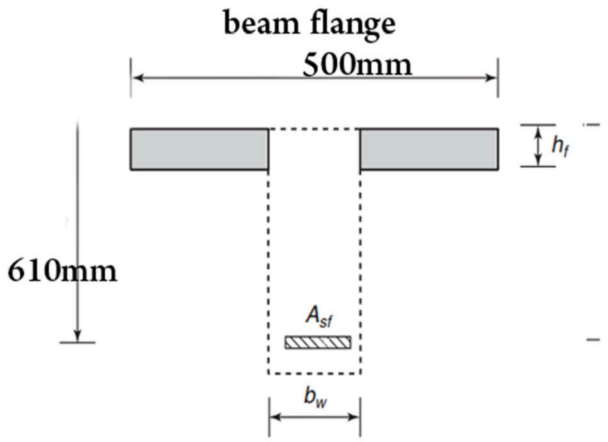


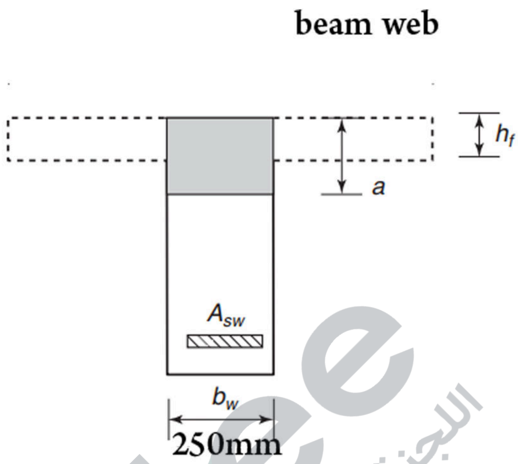
بما أنه حديد التنشن بالأسفل فهو ve+ مومنت والفلانج تتعرض لـcomp لكن نفرض أول شي أنه a < hf ونحل على مثل حل الـrecntagular beam هكذا :

Assume $\varepsilon s > \varepsilon y$ and $a < hf$

T=Cc..... $As\ fy = 0.85\ f'c\ ab$ $a=151.2mm > hf$ not ok

إذا الان نحل على أساس انه T بيم ونفصل





Ccf = Tf   0.85 f'c (b-bw)*hf=Asf fy

Asf = 1264.9 mm2

بعد ما أوجدنا Asf من بيم فلانج

Asw =As -Asf = 3060-1264.9 = 1795.1mm2

Ccw = Tw

0.85 f'c a bw = Asw Fy

a = 177.4mm   and c =208.7 mm

check Ɛs =0.0058 > Ɛy   ok

Mnf =1264.9 * 420 (610- 125/2) =290.9 kn.m

Mnw =1795.1 *420 * (610 -177.4/2) =393 kn.m

Mn =290.9 +393 = 683.9 kn.m

Ɛs = 0.0058 >0.005 so Φ = 0.9   ΦMn = 615.5 Kn.m

Check As min   406mm2 or 509mm2 take the larger and As>Asmin then ok

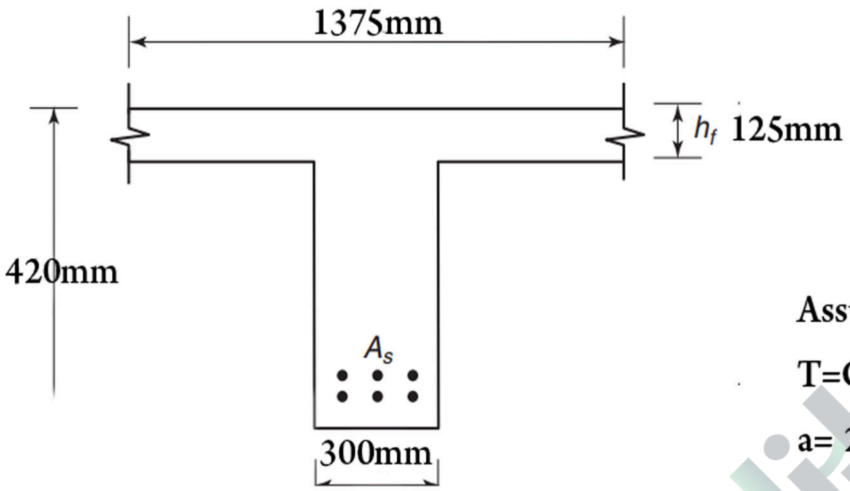
ملاحظة : لما يكون عندي الفلانج بوضع الـtension يعني حديد الـtension فوق

نعوض بدل bw الأصغر من الاتي   2bw or be (flange width)

ولكن بشرطين !!

1-flange in tension

2-beam must be statically determined(Cantiliever or Simply supported)



$f'c$ = 20Mpa

Fy= 420 Mpa

As=1704 mm2

Assume $\varepsilon s >= \varepsilon y$ $a<=hf$

T=Cc As fy =0.85 f'c a b

a= 21.9 mm<hf ok

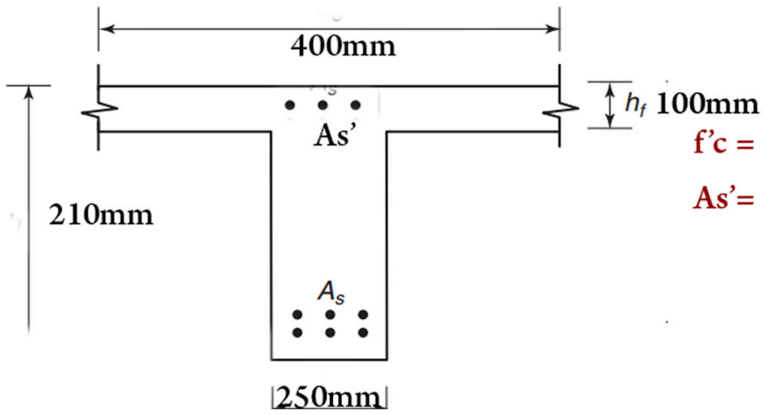
c=25.8 mm

εs = 0.003*(420-25.8)/25.8=0.046>εy ok

Mn= 1704*420*(420- 21.9/2) = 292.74 kn.m

Φ=0.9 ΦMn=263.47

سؤال شامل



f'c = 20 Mpa **fy=420Mpa** **As=3060mm2**

As'= 852mm2

$\varepsilon'_s \geq \varepsilon_y$ then $f'_s = f_y$

$\varepsilon_s \geq \varepsilon_y$ then $f_s = f_y$ $a \leq h_f$

$\varepsilon_y = 0.0021$ $0.002 = \varepsilon_y$ مش دائماً تعتبر

$f_y = 420$ إلا إذا كان

$A_s f_y = 0.85 f'_c + A'_s f_y$ $a = 136.4\,mm < h_f$ not o.k

$T = C_c + C_s$

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

$a = 158.20\,mm$

$c = 186.12\,mm$

$\varepsilon'_s = 0.002 < \varepsilon_y$ not o.k

$A_s f_y = 0.85 f'_c \,(b h_f + b_w (a - h_f)) + (A_s' * E * 0.003 (\frac{c-d'}{c}))$
($a \rightarrow 0.85c$)

$c = 191.66\,mm$ $a = 162.911\,mm$ $\varepsilon'_s < \varepsilon_y$

check ε_s, $\varepsilon_s = 0.0003 > \varepsilon_y$ not o.k

$T = C_c + C_s$ $A_s = (E * 0.003 (\frac{d-c}{c}))$

$= 0.85 f'_c \,(b h_f + b_w (0.85c - h_f))$

$+ A_s' E \,(0.003 (\frac{c-d'}{c}))$

$c = 158.87\,mm$ $a = 135.04\,mm$

$M_n = 0.85 f'_c (b - b_w) h_f (d - h_f/2)$

$+ 0.85 f'_c (b_w a)(d - a/2)$

$+ A_s' f'_s (d - d') = 166\,kN.m$

$\phi = 0.65$ $\phi M_n = 108\,kN.m$

check $A_{s\,min}$ $175\,mm^2$

True and False أسئلة

1-increasing the amount of tension reinfocment in any reincfoced concrete beam will increase the nominal strength and reduce the ductility True

2-the nominal moment capacity of any reinforced concrete beam can be increase significantly by providing reinfocrment in the compression side of the section False

in the tension side التصحيح

3-The ACI requires a minimum amount of tension reinforcment to be placed in the beamto guarantee safe transfer of the compression force from concrete to steel after cracking of the concrete before yielding of the reinforcment False

of tension force التصحيح

4-In the design safety philosophy for reinforced concrete beams the actual strength is reduced and the design load is increased True

5-when concrete is subjected to uniaxial compression the critical stress occurs at 50% to 60% of the ultimate load False the correction (75 to 80%)

6-when increasing the confining pressure on concrete subjected to tri-axial loading the compressive strength is expected to decrease False Correction (to increase)

7-reinforcment with the highest grade has the highest ductility

False correction (the lowest ductility)

8-economy is considered as one of the design objectives in the design process Ture

9-if the structure collapse due to fatigue it is said to have reached the ultimate limit state Ture

10-the serviceability limit state involves damage or failure of the part or all the structure- due to abnormal coditions False Correction (special limit state)

11-Steel reinfrocment of grade 40 shows the highest ductility among the other grades True

12-Increasing in the confining pressure on a concrete specimen increase the compressive strength True

13-In the uniaxial compressive test on concrete the unstable crack progation starts at the discountinouty limit False

14-In saftey design philosphu the strength reduction factor is always less than one and the overload factor is always more then one False

15-At the ultimate condition if the tensile strain in reinforcement has not reached the yield strain the beam strain then the beam is called an under - reinforcment beam
False

16-in beam analysis problems more reduction to the nominal momnet capacity is used if the expected mode of failure is tension-controlled

False لأني بضرب ب90% بالتنشن كونترولد .. يعني الريدكشن قليل(less reduction)

17-rigidity may be considered as one of the advantage of the concrete material
True

16-Reinforcment grade 60 has more ductility and less strength than grade 75
True

18-If the structure faild due to progressive collapse due to an explosion it is said to have reached the ultimate limit state
false

19-Increasing the compressive reinfrocment in a doubly reinforced concrete is not expected to have a significant effect on the nominal moment capacity but it might increase the design moment capacity True

20-in a creep test for a concrete speicmen the elastic recovery occurs at the moment of load application False at load removeal

21-Increasing the tenssion reinforcment in a doubly reinforced concrete is expected to have significant effect on the nominal moment capacity but it might decrease the design moment capacity Ture

22-the discontinouty limit ouccurs at stresses up to 30 to 40% of the compressive strength false

23-An over reinforced beam has more reinforcment than reqiuerd Flase

24-In a beam analysis problems if the tensile strain in the reinfrocment at the ultimate condition is less than the yield strain compressive reinforcment should be used to increase ductility False

25-the working stress design procedure based on the unfactored loads True

26-The ACI code requires min area of steel in beam to ensure tension-cotrolled failure
False (to ensure safe transmit of tension force from conc to steel)s

1-which of the following is NOT a reason why the equavilent rectangular compressive stress block for concrete is used in the analysis of flexural strength

A)the actual compressive stress on the concrete section is of a parabolic shape which is difficult to deal with

b)it gives good approximation to the magnitude and location of the resultant compressive force on the concrete section

c) It is an exact representaion of the compressive stress for concrete

Answer is C

2-which of the following statement is in correct

a)when a structure becomes unfit for it intended use it is said to have reach the limit state

b)Rupture of critical parts of the structure could lead the structure to reach its ultimate limit state

c)servicability limit state involves a structural collapse of part or all the structure

Answer is C

3-which of the following statment is incorrect

A)the actual strength of the structural members will almost always differ from the values calculation by the design

B)For safe design strength reduction factor is always less than one and the overload factor s are always more than one

Answer is b

which of the following is Not a factor affecting the choice of concrete for a structure

A)Economy , fire resistance and rigidity

B)suitability of material for architectural and structurak functions

C)Low tensile strength and time dependet volume changes

D)Availability of materials and low maintenance

Answer is : C

Which of the following statments correctly describes the mortar cracks that develop in concrete due to uniaxial compressive loading

A)developed before the concrete is loaded due to shrinkage of the paste that occurs during hydration

B)Develop when concrete is subjected to stresses greater than 30% to 40% of its compressive strength

c)develops when the load is increased beyonde 50 to 60% of the ultimate load

Answer is C

T or F

1-working -stress design metholodgy considers the redistribution of the moments as successive cross sections yield

False

2-the formation of plastic mechanism invovles yielding of concrete in the location of maximum bending stress

False

3-when concrete is loaded by compression instantaneous elasic strain develops . If the load remains on the concrete ,creep strain develop with time

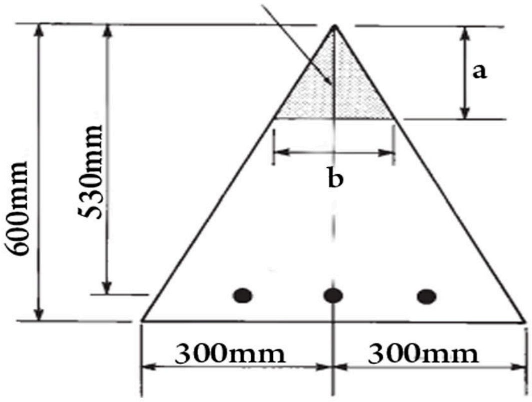
True

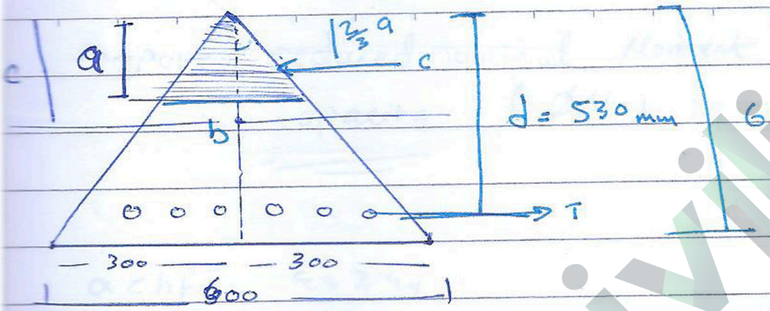
4-the addition of compressive reinforcment to a concrete beam section reduces the compressive force on the concrete and therefore
reducing the creep strain
True

5-In flexural theory it is assumed that concrete is assumed to fail when maximum tensile strain reaches a limiting value of 0.003
(false (compressive strain

6-The ACI code requiers a minimum amount of tension reinforcment in a beam section to gurantee safe transfer of the tensile force from concrete to steel
True

f'c = 28 Mpa   fy = 420 Mpa   As=3032mm2





$f_y = 420$ MPa

$f'_c = 28$ MPa

$A_s = 3032$ MPa

assume $\varepsilon_s > \varepsilon_y$ $f_s = f_y$

$T = C_c \rightarrow A_s f_y = 0.85 f_c (\frac{1}{2} b a)$

$\frac{300}{\frac{1}{2} b} = \frac{600}{a}$

$A_s f_y = 0.85 f_c (\frac{1}{2} a^2)$ $a = 327$ mm

$C = 384.7$ mm

$b = \frac{300 a}{0.5(600)}$

$a = b$

$\varepsilon_s = 0.003 (\frac{d-c}{c}) = 0.0011$ $\varepsilon_y = 0.0021$

$\varepsilon_s < \varepsilon_y$ not ok

$A_s * E * \varepsilon_s = 0.85 f_c \frac{1}{2} a^2$ $a = 318.5$ mm

$C = 374.7$

$\varepsilon_s = 0.003 \cdot \frac{d-c}{c} = 0.0012 < \varepsilon_y$ ∴ ok

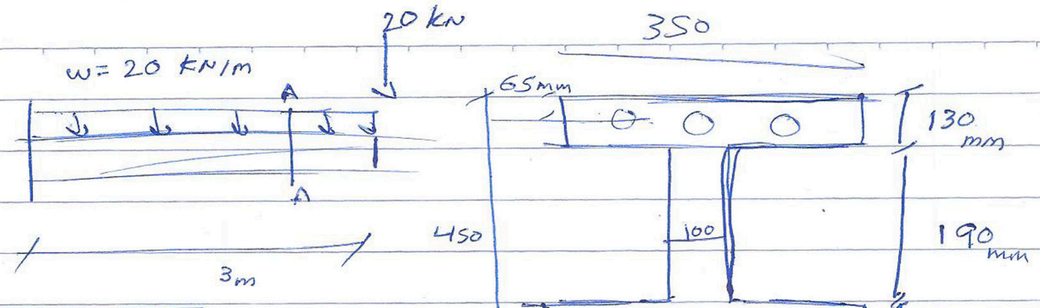
$M_n = T (d - \frac{2}{3} a)$

$= 0.85 f_c * \frac{1}{2} a^2 * (530 - \frac{2}{3} 318.5) *$

$= 460.17$ kN.m

$\phi = 0.65$ ~~$\phi M_n = 1179$ kN.m~~

$\phi M_n = 299.1$ kN.m



Find ϕM_n

$\varepsilon_s \geq \varepsilon_y$ and $a \leq h_f$ $\qquad A_s = 2413$

$f_y = 420$

$A_s f_y = 0.85 f'_c * a * 350 \qquad f'_c = 25$

$a = 136.3$ mm $> h_f$ not o.k

$A_s f_y = 0.85 f'_c * (A_c) \qquad A_c =$ [sketch] $+$ [sketch]

$A_c \; (350-100)130 + a(100)$

then $a = 157.9$ mm

$c = 178.73 \qquad d = 385$

check $\varepsilon_s = 0.00346 > \varepsilon_y$ o.k

~~$M_n = 0.85 f'_c (\ldots) *$~~

$M_n = 0.85 * 25 ((350-100)130) * (d - h_f/2)$
$+ 0.85 \times 25 \; (a\,100)(d - a/2)$

$M_n = 320.76$ kN.m

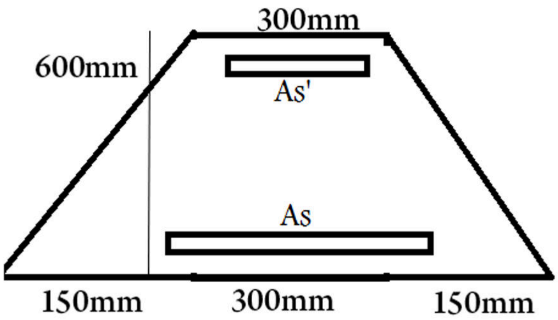
من الطارقة, $\phi = 0.7716$

$\phi M_n = 247.5$ kN.m

show which beam cross section is adequate to carry the given load

هذا الكلام يعني أنه لازم تجد Mu ومقارنتها مع phi Mn إذا طلعت (phi Mn) أكبر أو تساويها فالجواب yes وإلا No ما قدرت أحله لأن بسؤال السنوات في لود موجود اخر البيم مش مبين قيمته ... لو فرضنا قيمته 20Kn

Mu= 20*3+60*1.5=150 Kn.m



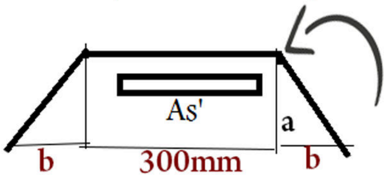
determein the depth og the neutral axis

fy =420Mpa f'c =20 Mpa As=4020mm2

As' = 852 mm2 d= 535mm d'=65mm

depth of neutral axis is c

Assume Ɛ's >=Ɛy and Ɛs>Ɛy   T=Cc + Cs   Asfy=0.85 f'c (Area) + As' Fy



الـArea عبارة عن مساحة مستطيل + مساحة المثلثين
لكن قاعدة نخليها بدلالة a حتى تصير المجهول الوحيد
بالمعادلة نجيبها من تشابه المثلثات

$$\frac{a}{b} = \frac{600mm}{150mm} \qquad b = a\ 0.25$$

Area = 2 * (0.5 *b*a)+ 300*a = 2*(0.5*0.25*a*a) +300 a

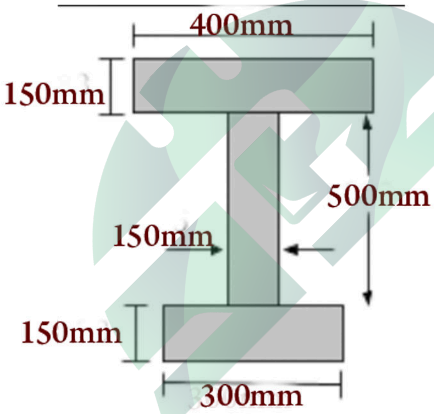
As fy = 0.85 f'c *Area + As' Fy معادلة بمجهول a

من الالة الحاسبة

a = 220.4mm c= a /0.85 = 259.3mm

Check Ɛ's =0.0024 >Ɛy then ok

check Ɛs = 0.0032>Ɛy then ok



the beam section below is reinfoced for negative moment with 1900mm2 of tension reinforcment and 750 mm2 of compression reinformcent assumin both tension and compression reinforcment are yielded determine the depth of the assumed equavlent stress block f'c=25 Mpa fy=420Mpa

المطلوب إيجاد a مومنت سالب يعني التنشن فوق

d=150+500+150-65
735mm

assume a<hf Ɛ's >=Ɛy و Ɛs>=Ɛy

T=Cc+Cs   Asfy=0.85 f'c (300*a) +As' Fy

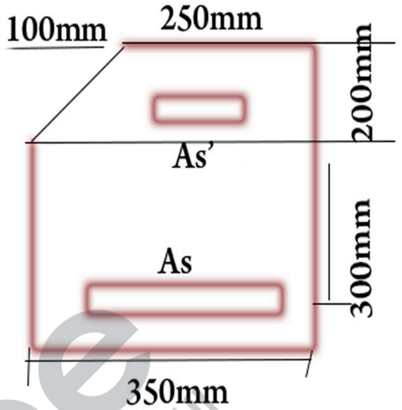
a=75.76 mm<hf so ok   c= 89.13mm

Ɛ's=0.0008 <Ɛy not ok   Asfy=0.85f'c *c*0.85* b + As' E 0.003*(c-65)/c   c=112.3 mm

a=95.45mm

check Ɛ's=0.00126<Ɛy ok   Ɛs=0.0166>Ɛy ok determine the depth of the compression area fy=420 Mpa f'c =30Mpa As=4000mm2

As' = 500mm d'=65



المطلوب c

Assume $\varepsilon'_s \geq \varepsilon_y$ and $\varepsilon_s \geq \varepsilon_y$ and a<200

T= Cc+Cs

As Fy= 0.85 f'c (Area) + As' Fy

مساحة المثلث + المستطيل =Area

نفرض قاعدة المثلث b وارتفاعه a

a/b = 200/100   b= 0.5a

Area = 0.5*0.5*a*a + 250*a   As Fy = 0.85 f'c *(0.5*0.5*a*a + 250*a) + As' fy

a=193.24 < 200 ok

الان انتبه انه f'c فوق 28 لذلك نجيب بيتا من القانون

β = 0.8357   c=231.23 mm

d=500

check ε'_s =0.00215>0.0021 so ok

check ε_s=0.00348>ε_y so ok

Rectangular beam 300mm width and 550mm effective depth determine the maximum area of tension reinforcmentto be place in the section for it to be a balanced section

Fy=420Mpa f'c=35Mpa

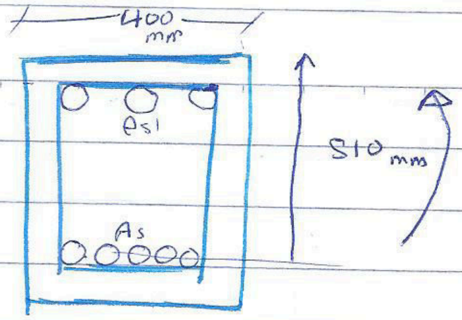
balanced section means ε_s=0.0021=0.003(550-c)/c   c=323.53mm

as f'c >28 and <56 we get β from the equation β=0.8   a=258.82mm

As fy = 0.85 f'c a b   As=5499.9mm2

No. ______

Date ______



$f'_c = 28$ MPa $f_y = 420$ MPa

A. ϕM_n? in a single reinfor.. beam

$A_s' = 0$ $A_s = 4210\ mm^2$

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

$T = C_c \longrightarrow A_s f_y = 0.85 f'_c * a * b$

$4210 * 420 = 0.85 * 28 * a * 400$

$a = 185.73$ mm

$c = 218.5$ mm

$\varepsilon_s = 0.003 \times \left(\frac{d-c}{c}\right) = 0.004 > 0.002 \therefore$ o.k

$M_n = 0.85 * 28 * 185.73 * 400 * (510 - 185.73/2)$

$= 737.5$ kN.m

$\phi =$ حسب درة $= 0.82$ $\phi M_n = 604.79$ kN.m

b. compute C ? (Doubly reinforcment) $A_s' = 995\ mm^2$ $A_s = 4640\ mm^2$

$\varepsilon_s' \geq \varepsilon_y$ $\varepsilon_s \geq \varepsilon_y$

$T = C_c + C_s \longrightarrow A_s f_y = 0.85 f'_c * a * b + A_s' f_y$

$a = 160.8$ mm

$c = 189.1$ mm

$\varepsilon_s' = 0.003 \times \left(\frac{d-d'}{c}\right) = 0.0019 < \varepsilon_y$ not o.k

$A_s F_y = 0.85 f'_c * a * b + A_s' * E * \varepsilon_s'$

$c = 192$ mm

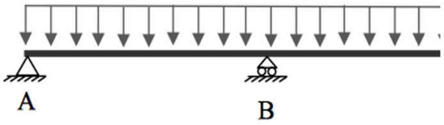
$a = 163.22$ mm

$\varepsilon_s' = 0.003 \times \left(\frac{c-d'}{c}\right) = 0.0019 < \varepsilon_y$ o.k

$\varepsilon_s = 0.003 \times \left(\frac{d-c}{c}\right) = 0.00497\ \varepsilon_y$ o.k

أفكار أسئلة الفصل الماضي :

السؤال الأول كان معطيني continous beam بهذا الشكل



وطلب هل السوبورت b بتحمل اللود

لازم تطلع phi Mn بالطريقة العادية لكن لكن العبرة من السؤال تعرف وين مكان حديد التسليح لأنه كان قال إنه T beam خوذ سكشن وشوف اتجاه المومنت أو ارسم شير مومنت دايقرام عالسريع أو بالنظر لأن الجزء B ولليمن عبارة عن كانتلفر فالحديد علوي .

هنا تنتهي مادة الفيرست مع بضع سنوات ... للتقوية والمزيد من الأسئلة حل أسئلة الطالب أنس دواس ...

بالتوفيق للـ30/30

Second (The Design)

Design of rectangular beam : (the relationship between beam depth and deflection)

(موجود أخر الدوسية) Use Table 9.5 hmin to avoid deflection

ملاحظة : ازدياد h يعني زيادة فيمة I (second moment of inertia) بالتالي زيادة مقاومة البيم للانحناء أي بيم سيحتوي على انحناء ولكنه يجب أن يكون ضمن الـACL

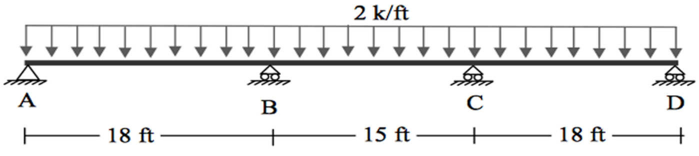
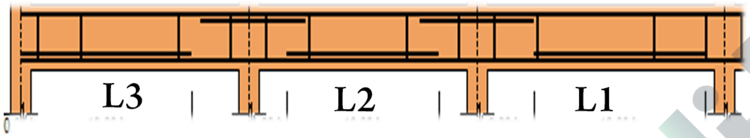


Figure 1

مثال هذا continouse beam



هذا البيم اله 3 h min بين كل عمود بنروح الجدول عند beames or) (ribbed on way slab

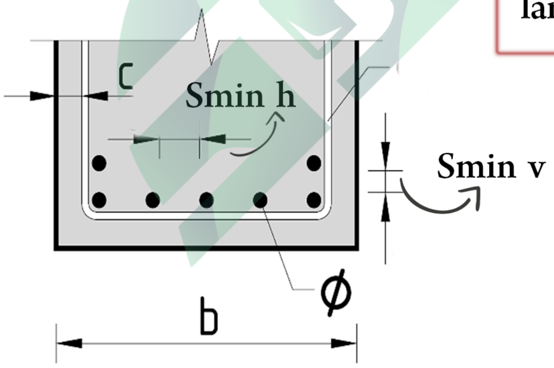
بالنسبة للجزء رقم 1 بما أنه مش متصل بشي من اليمين ومتصل من الشمال فقط فهو One end continous لذلك hmin = L1/18.5 (L from center to centor)

بالنسبة للجزء 2 فهو متصل من اليمين واليسار لذلك هو both end continous بالتالي hmin=L/21, والجزء 3 نفس الجزء 1 ... هيك بصير عندك ثلاث قيم للـhmin نختار أكبر وحدة فيهم لو ما فهمت بشكل كامل مش مشكلة للأمام رح تفهم بمثال

Concrete Cover and Bar Spacing

cover for beams is = 40 mm

Smin v	→ 25mm
larger of	→ 1.33 max coarse aggregate size

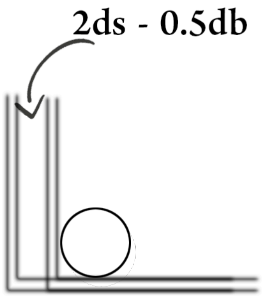


Smin H	→ Bar diameter
	→ 25mm
	→ diameter of vibrator
	→ 1.33 max c.a size

Resons of cover : 1-Protect steel from fire

2-Protect steel from abrasion

3-to bond concrete and steel so both act togther



حديد التسليح المفروض يجي على الزاوية القائمة لكن أحياناً مابيجي
لذلك اعتبرنا مسافة (2ds-0.5db) ضمن حساب bar spacing

حساب الـb min

bmin : the minimum width of the beam to put the bars in one layer

b min = 2 * cover + 2* sirrups diameter + n (No. of bars)* diameter of bar + (n-1) * Smin +2(2ds-0.5db)

bmin = 2* 40 +2*10+ n*db + (n-1) * Smin + 2(2ds-0.5db)

if b>bmin bars in (one layer)

if b<bmin bars in (two layers)

Estimating the effective depth of the beam (d)

d = h-65mm if one layer	d= h-90mm if two layer

bmin should not be less than : preferably (300mm)
absolutly (250mm)

العبرة من عملية تصميم البيم إني أجد كمية الحديد As و أبعاد البيم

Mu اضربها بـ10^6
وعوض الباقي بـmm والـMpa

معلومات وقوانين لازم تعرفها قبل ما ندخل التصميم

$$As = \frac{M_u}{\phi f_y (jd)}$$

نعتبر Φ هنا = 0.9 دائماً

Ultimate load or U or W الأكبر
U= 1.2DL + 1.6 LL or U=1.4DL

قيمة j :

j=0.9 for narrow comp area (للبيم المستطيل أو T بيم والكومبرشن علـweb)

j=0.95 for wide comp area (T بيم كومبرشن ع الفلانج)

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

سؤال التصميم بكون على شكلين
الأول أبعاد البيم معطاه
الثاني أبعاد البيم غير معطاه أو أحدهم معطاه

طريقة حل النوع أول (أبعاد البيم معطاه)

1) إيجاد الـ Mu وذلك عن طريقة ← إيجاد U من U=1.2DL+1.6LL

← في حال قلي DL(including Self weight)s عوض مباشرة

أما DL(excluding self weight)s بدك تحسب self weight ثم تضيفه لل

DL ثم تعوض بـU Self weight = b *h *gamma

← $Mu = WL^2/8$ for simply supp $Mu= WL^2/2$ for cantiliever

2)نفرض One layer or two layer ثم نجد d

3) نجد As = Mu/Φfyjd ثم بنسوي iteration هكذا

As = Mu/Φfyjd	As = Mu/Φfy(d-0.5a)	As = Mu/Φfy(d-0.5a)
a= Asfy/f'c 0.85 b	a= Asfy/f'c 0.85 b	As= requierd

4) نجد As provided من الجدول بتشوف قيمة الـrequired الي طلعت واختار أكبر منها بشوي أو حسب تقديرك المهم أكبر بالامتحان بقيدك بدياميتر معين ...

5) check one leayer or two layer للـ one layer (b>bmin)s وللـtwo layer (b<bmin)s

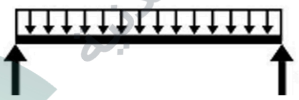
إذا ماكانو ok ارجع للخطوة 2 واحسب d جديدة وارجع عيد (لا تقلق ما رح يوخذ وقت اذا ممارس)

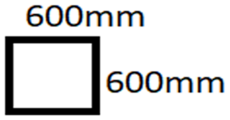
6) check Asmin (حالياً) وAs max(لاحقاً بس توخذه) إذا طلع أقل من الـmim استخدم الـMin

7) الان نرجع للـanalysis مثل الفيرست ولازم يطلع ΦMn>=Mu

Example

f'c=20Mpa





fy=420Mpa

LL=35 Kn/m DL=14Kn/m (excluding self wt) gamma=24Kn/m

sol ::: self wt = gamma b h $=24*600*600*10^{-6}$ =8.64Kn/m

DL=8.64+14=22.64 kn/m U=1.2Dl+1.6LL=83.2Kn/m

Assume One layer d=h-65=535mm

As requierd = My/Φfy jd =3657mm2 now itreations

a=Asfy/f'c0.85b =150.6mm As=Mu/Φfy(d-0.5*a) = 3830.4mm2

a=157.7 mm As=3860.2mm2 a==159mm As=3865.6mm that is As required

As provided from table =4080mm2   8No25M

بدور على ديامتر وعدد بارز تحقق الـrequired أو أكثر

لو قيدني بديامتر معين وبدي أعرف كم سيخ بروح للجدول عند الديامتر الي اعطاني ياه بجيب المساحة لـ1 bar عندئذ

= As req /As of one bar = عدد الاسياخ   العشري يقرب للأكبر

check one layer bmin=40*2+2*10+8*25+7*25+2(2*10-0.5*25)=490mm <600   ok

لو طلع b أقل من الb min ارجع افرض two layers وعيد حساب d ثم As لا تقلق الشغلة مش صعبة مثل ما تتوقع أو يمكن تكبير الديامتر لكن إن ما فرض عليك ارجع عيد الحسابات أفضلك

check As min = 854mm2 or 1070 mm2 larger is 1070mm2 and it is ok

الان نرجع للأanalysis للتأكد من الـtension controlled

As fy = 0.85 f'c ab   a=168mm   c =197.6mm   Ɛs=0.0051 then it is tension con-
<trolled Mn=772.8Kn.m   Phi Mn=695.9Kn.m   PhiMn>Mu then ok

الطريقة الثانية : b and h and As are Unknowns شوية قوانين مش حفظ

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

steel ratio
reinforcment ratio
percentage of reinfocrment

$$a = \rho \frac{f_y}{f'_c} \times \frac{d}{0.85}$$

$$\omega = \rho \frac{f_y}{f'_c}$$

Mechanical steel ratio

$$a = \frac{\omega d}{0.85}$$

$$Kn \text{ or } R = \omega f'_c(1 - 0.59\omega)$$

flexural resistance factor

$$Mu = \Phi Mn = \Phi(bd^2\, \omega f'_c(1 - 0.59\omega))$$

$$bd^2 = \frac{M_u}{\phi Kn}$$

طريقة حل هذا النوع (ملاحظة الكلام كله هذا شارحه الدكتور ضمن مثال رح تشوفه للأمام انا حبيت أبين الاسلوب قبل ما تدخل ع المثال)

1-estimation of self weight : the weight og rectangular beam will be roughly (10%-15%) . of the unfactored loads it must carry

1-self weight =(10%-15%)*(DL+LL)

2- h=(1/18 to 1/12) of L

L : span length in mm

then b=0.5 h then self wt = gamma*hb

قيم b و h هذي بس عشان فرض الـself wt لا تيجي وتقلي b و h معلومات

2- قيمة ρ :

ρ for economic considrations= 0.01 (أو إذا ما حكى كم قيمتها اعتبرها هيك)

ρ Ductility considrations= (0.35-0.4) ρb ρb=balanced steel ratio

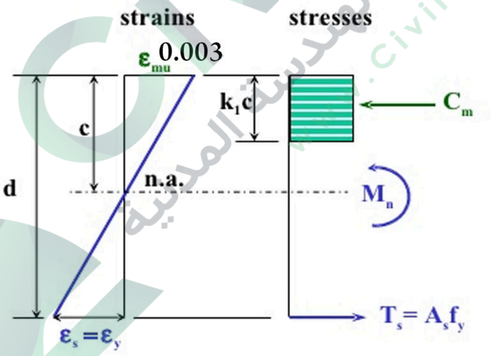
$$\rho_b = \frac{A_{sb}}{bd}$$

Asb: كمية الحديد في حالة الـbalanced condition εs=εy

by placing considreations it may be hard to place the reinforcment

if ρ exceeds =0.015 that means ρmax = 0.015 ما لازم تتعداه

strain compactability



من تشابه المثلثات

strain compatibility

$$\frac{0.003}{c} = \frac{0.003 + \varepsilon_y}{d}$$

$$c = \frac{0.003}{0.003 + \varepsilon_y} d \quad \text{....1}$$

$$a = \frac{A_{sb} f_y}{0.85 f'_c b} = \frac{\rho bd f_y}{0.85 f'_c b}$$

$$c = \frac{\rho bd f_y}{0.85 f'_c \beta} \quad \text{....2}$$

مش حفظ
لكن افهم فهم
لأنه ممكن يقلك
بدي عن سترين معين
مش εy
نفس الخطوات بس بتغير
بدل εy حط القيمة

from 1 and 2 $\rho_b = \frac{0.85\beta_1 f'_c}{f_y}\left(\frac{0.003}{0.003+\varepsilon_y}\right) x \frac{E}{E} = \frac{0.85\beta_1 f'_c}{f_y}\left(\frac{600}{600+\varepsilon_y}\right)$

3- نجد Mu و Kn (حسب قوانينهم سابقا)

4- $bd^2 = \frac{M_u}{\phi Kn}$

هون بدنا نفرض 3 قيم تقريباً لـb وبنوجد 3قيم ل d
نختار القيم الي فيها تقريباً 0.5b=d لكن غالبا بالامتحان بحددلك قيمة b وإذا بدك تقرب قيمة d قربها لأكبر ثم نجد h حسب فرض الـlayers

one leayer h=d+65 two layers h=d+90

5-check h min   hmin لازم تكون أكبر من h قيمة h الي طلعناها
إذا ماكانت خوذ الـhmin وارجع احسب الـd

6-check self wt and recvise Mu

Self weightبما أنه صرنا عارفين قيمة h و b الان بدنا نحسب القيمة الفعلية لـ

self wt new = gamma *bh   w new =1.2 DL +1.6 LL   لا تنسى تضيف الـself wt للـDL

Mu new $=WL^2/8$ if simply supp or $WL^2/2$ if cantiliever

ثم حساب النسبة $\frac{M_u^{\text{جديد}} - M_u^{\text{قديم}}}{M_u^{\text{قديم}}} \times 100\%$

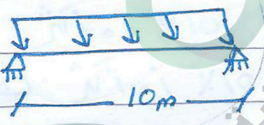
يجب أن لا تكون الزيادة 10% أو أكثر

إذا كانت الزيادة في 10% أو أكثرنعيد الـdesign عندئذ بنحط bd^2= Mu new /Φkn

7-find As and do the iterations then As privided then check Asmin and max

8-check b min   9-analysis

Example 8



LL = 25.5 kN/m   DL = 14.5 kN/m (excluding self wt)

f'_c = 25 MPa f_y = 420 MPa

1 Estimate Self wt

$h = (\frac{1}{18} - \frac{1}{12}) * 10{,}000 = (555 - 833)$

$b \simeq 0.5h = (277 - 416)$

Self wt = γbh = (3.68 − 8.3) = 8 kN/m

2 نجد b و d

$bd^2 = \frac{Mu}{\phi kn}$ → $Mu = \frac{wL^2}{8}$ w = 1.2(14.5+8)+1.6(25.5)

Mu = 848 kN/m

ρ = 0.01 ناخذ الـρ اذا ما اعطى

$\omega = \frac{0.01 * 420}{25} = 0.168$ → $\phi kn = 0.9[25(0.168)(1 - 0.59(0.168))$
= 3.41

$bd^2 = \frac{848}{3.41} = 248.7 * 10^6$ mm³

نفرض قيمة لـ b   b = 300   d = 910

0.5b = d تقريبا   نختار هذه لأن ← b = 400 mm   d = 788 mm

No.
Date

$h = d + 90 = 878\ mm$ (assume 2-layers)

نجعلها $= 900_{mm}$ للتسهيل

$d = 900 - 90 = 810\ mm$

- check h_{min}

$h_{min} = \frac{L}{16} = \frac{10,000}{16} = 625\ mm < 900 \therefore o.k$

- check self wt and revise Mu

$self\ wt_{new} = 24 * 0.9 * 0.4 = 8.64$

$\omega_{new} = (8.64 + 14.5)1.2 + 1.6(25.5) = 68.57\ kN$

$Mu = \frac{68.57(10)^2}{12} = 857\ kN.m$

* يجب أن لا تكون الزيادة 10% أو أكثر

* اذا كانت الزيادة في Mu 10% أو أكثر نعيد ال Design

* عندئذ سنستخدم القيمة الجديدة لحساب As أو $bd^2 = \frac{Mu_{جديد}}{\phi k_{cn}}$ بدل ما نغير الابعاد

لأنه الحديد بكفي .

* حساب النسبة

$$\frac{Mu_{جديد} - Mu_{قديم}}{Mu_{قديم}} \times 100\%$$

$= 1.1\% < 10\%\ O.k$ ✓

- $As = \frac{Mu}{\phi f_y jd} = \frac{857 \times 10^6}{0.9(420)0.9(810)} = 3110\ mm^2$

itterations

	①	②
$As = 3110\ mm^2$	$As = 3092\ mm^2$	$As = 3090.5\ mm^2$
$a = 153.7\ mm$	$a = 152.8\ mm^2$	(requiered)

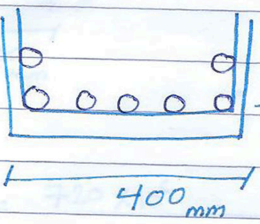
نلجأ للجدول use 7 No25 M ; $As = 3570\ mm^2$

- check As_{min}
- 964 mm^2
- 1080 $mm^2 < As = 3570\ mm^2$ O.k ✓

check b_{min} = 2*40 + 2*10 + 7*25 + 6*25 + 2(20 − 0.5x25) = 440 mm^2

440 mm > 400 mm ✓ o.k

two layers



→ حطينا 5 تحت عشان حاطط كانه فوق

b_{min} (5 No 25 M) o.k

— Analysis $a = \frac{357.5*420}{0.85 x \ldots} = 176.4 mm$

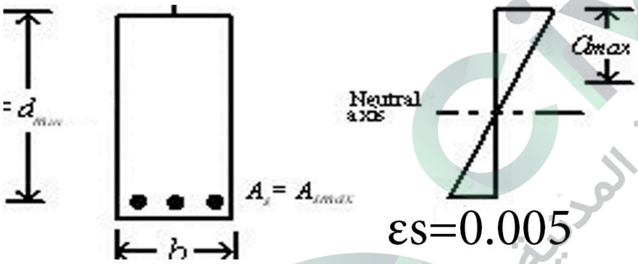
C = 207.5 mm

$\varepsilon_s = 0.003 \left(\quad \right) = 0.0087$ tension controlled

$\phi = 0.9$ $\phi M_n = 974$ kN.m > $M_u = 857$ kN.m

o.k

Maximum Area of tension reinforcment



εs=0.005

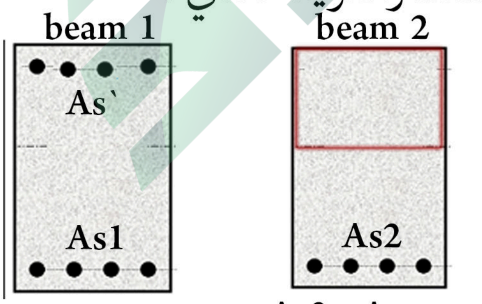
$$As\ max = 0.319\ \beta\ \frac{f'_c}{f_y} bd$$

→ ρ max

بالامتحان بجيب ρ max أو As max لذلك اعرف إنه

Asmax = ρmax bd

في حال As>As max بصير السؤال doubly reinforcment والطريقة كالتالي :



As2 =As max

1)نقسم البيم لقسمين

2)بيم 2 already yielded بالتالي εs=0.005

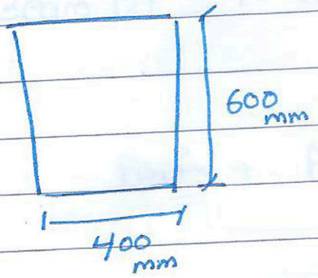
بيم 2 (T=Cc) وجيب a و c (قيم هوa لكل البيم)

3) احسب ΦMn2=ΦAs2 fy (d-0.5*)

4)نحسب ΦMn1 من ΦMn1=Mu-ΦMn2 ثم من ΦMn1 بنحسب As1 ثم مجموعهم بعطيني As ثم جيب الـprovided بعد ما جبت provided احسب قيمة As1 الجديدة حيث As2 ثابتة وهيAs max بعدها من بيم 1 T=Cs (As1 fy= As` fy or fs)s حتى نعرف Fs or fy شييك على ε's ثم As` provided (زيادة As` تزيد εs) analysis (5

Date

Example $f'_c = 28$ MPa $f_y = 414$ MPa $M_u = 720$ MPa



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

• بما أنه طلعت عالي ,2 يعطيني حديد كثير
لذلك على الأغلب هيكون (2 - layers)

- assume (2-layers)

$d = 600 - 90 = 510$ mm

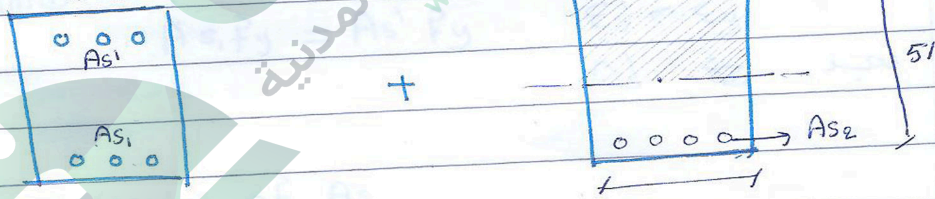
- $A_s = \frac{720 \times 10^6}{0.9 \times 414 \times 0.9 \times 510} = 4210\ mm^2$

- $A_{s\,max} = 0.319\ (0.85)\ \frac{28}{414} \times 400 \times 510 = 3741\ mm^2$

$A_s > A_{s\,max} \longrightarrow$ (Not tension Controlled)

- Doubly reinforced beam :

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



$A_{s2} = A_{s\,max} = 3741\ mm^2$

$\varepsilon_s = 0.005$ لأنه already yielded

Beam ② $T = C_c$ $3741 \times 414 = 0.85 \times 28 \times a \times 400$

$a = 162.7$ mm $c = 191.4$ mm

Check ε_s عشان نتأكد أنه a و c مضبوطات
لازم $\varepsilon_s = 0.005$

- $\varepsilon_s = 0.003\left(\frac{510 - 191.4}{191.4}\right) = 0.005$ ✓

$\phi M_{n2} = \phi A_{s2} f_y (d - a/2) = 597.5$ kN.m
($\phi \to 0.9$)

لأنه بهمني بس من المتبقي من Mu

$\phi M_{n1} = 720 - 597.5 = 122.5$ kN/m

• ما بصير نستخدم ($A_s = A_{s1} + A_{s2}$) لأنه رح يعطيني أقل من Tension Controlled

Beam ② $T = C_c$ $A_{s2} f_y = 0.85 f'_c\, a b$

منها $a = \ldots$ $C = 191.4$ (mm)

$\varepsilon_s = 0.005$

$A_{s1} f_y + A_{s2} * f_y = 0.85 f'_c\, a b + C_s$

نفسها ← القوية

لذلك ما أثرنا على المعادلة

يعني

$T_1 = C_s = A = A_{s1} f_y$

$\phi M_{n1} = \phi (A_{s1} f_y)(d - d')$ → لو بدك نستخدم $A_s' * f$

$\phi \to 0.9$

بدل تشيك لع بالأول عشان تعرف أو نستخدم f_s أو f_y

$A_{s1} = 728\ mm^2$

$\cdot A_s = A_{s1} + A_{s2} = 728 + 3741 = 4469\ mm^2$ (requiered)

beam ① $T = C_s$ $\varepsilon_s' < \varepsilon_y$

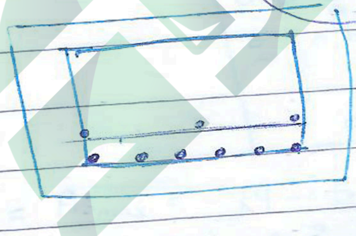
$A_{s1} f_y = A_s' f_y$

نجد A_s'

- Selection of As

As provided 9 No 25 M ; $A_s = 4590\ mm^2$

$b_{min} = \ldots > 400$ ∴ two layers o.k



beam ① نغير A_{s1} *

لما اخترنا Provided

$A_{s1} + A_{s2} = 728 + 3741$

$A_{s1} = 4590 - 3741 = 849\ mm^2$

لو بدي أحسب A_s' بدي أعمل check على ε_s'

$\varepsilon_s' = 0.00198$ not yielded $f_s = f_s' = 396\ MPa$

$A_{s1} f_y = A_s' f_s'$ $849 * 414 = A_s' \times 396$ $A_s' = 900\ mm^2$

A_s' provided $= 995\ mm^2$ ما في داعي ارجع احسب A_{s1} لانه زيادة تزيده A_s'

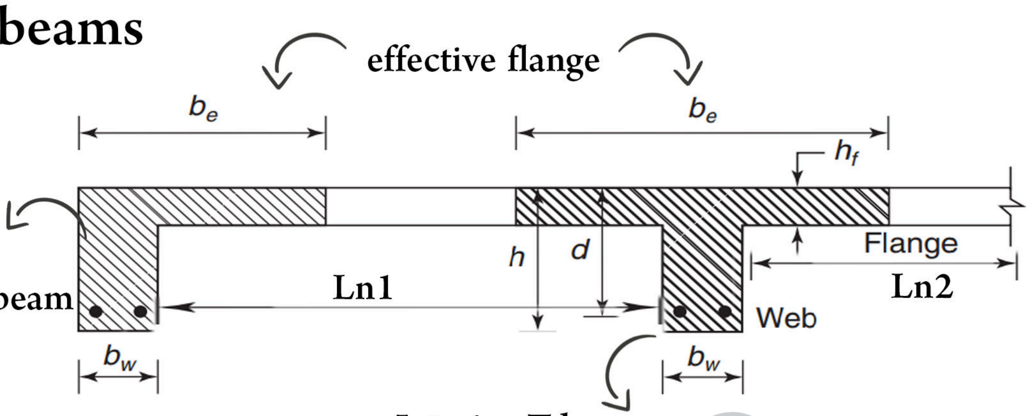
- Aanalysis ε_s yield ε_s not yield $f_s = f_s'$ ε b

Design of T-beams

spandrel beam

inverted L shaped beam

edge beam



Interior T-beam

Ln : clear distance

hf : edge of the flange

or edge of the slab

يتشكل الـTبيم لأن الصبة للبيم والسلاب مع بعض

for spandrel beam حفظ

$$b_e = \text{smaller of} \begin{cases} b_w + Ln/2 \\ b_w + 6\,h_f \\ b_w + \ell/12 \end{cases}$$

for T Beam Interior حفظ

$$b_e = \text{smaller of} \begin{cases} b_w + Ln1/2 + Ln2/2 \\ b_w + 2\,(8\,h_f) \\ \ell/4 \end{cases}$$

مهم جداً Ln المسافة بين بيمين Face to face

L بتكون على نفس البيم من العمود الأول للعمود الثاني center to center

هالكلام مهم لسؤال السلاب لقدام ولو مافهمت احفظهم هيك رح تفهم بعدين

أسلوب حل الـT beam ..(أسهل سؤال)

1) بعطيني المومنت أو بعطيك W غالبا .. المهم بس يكون عندك المومنت أول شي بتجيبه As=Mu/Φfy jd

2) في تصميم الـT beam ما بسوي itrations

ما بشيرك على الـAs max لما يكون flange in Comp

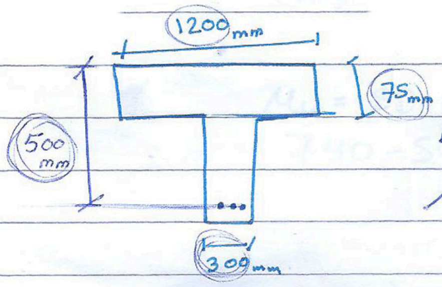
ما بحسب b min

الان بنجيب As provided ونشييك As min

3) analysis

4) Check As required based on computed a (ويجب أن يكون Provided > Req)

رح تفهم كل شي بالمثال الان



$f'_c = 21$ MPa

$f_y = 420$ MPa

M_u (+ve) = 740 kN.m

$$A_s = \frac{M_u}{\phi f_y j d} = \frac{740 \times 10^6}{0.9 * 420 * 0.95 * 500} = 4121.41 \; mm^2$$

required

* ما بستخدم كالعادة $A_{s\,max}$ عند Flange in Comp

* ما بستخدم iteration كالعادة عند Flang in Comp لانه $jd = (d - a/2)$

$A_{s\,min} = \begin{cases} 500 \; mm^2 & O.K \\ 400 \; mm^2 \end{cases}$ A_s provide = 4590 mm^2 9 No28M

ما بفحص b_{min}

- analysis

$a \le h_f$

$T = C$ $a = 90_{mm} > 75$ ∴ not O.K

① beam (F) ② beam (W)

$A_{s\,w} = 4590 - 2868 = 1721 \; mm^2$

$T = C$ $a = 134.98$ mm

$c = 158.8$ mm

$A_{s1}\, 420 = 0.85 * f'_c * 75 * (900)$

$A_{s1} = 2868$ mm

$\varepsilon_s = 0.006$ Yielded

Provided فعلية $\phi M_w = 281.3$ kN.m $\phi M_{nF} = 501.5$ kN.m

$\phi M_n = 783$ kN.m > $M_u = 740$ O.K

• Check As required based on Computed a.

ما نحسب w

$$A_{s\,w} = \frac{M_u \text{ (للـ w)}}{(\phi f_y (d - a/2))}$$

$\phi M_n = \phi M_{nw} + \phi M_{nF}$ ثابتة صحيحا

$M_u = 501.6 + \phi M_{nw}$ a تقريبا

$740 - 501.6 = M_{u\,w} = 238.4$ kN.m

مطلوبة req

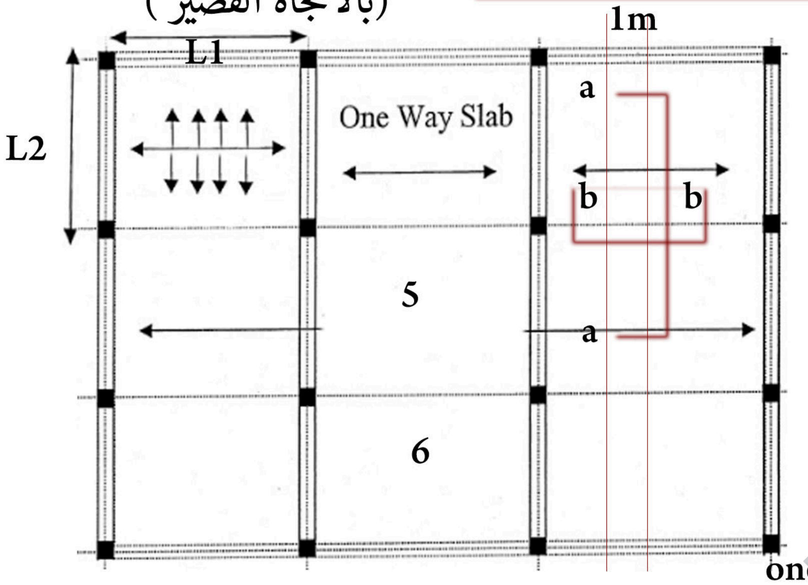
$$A_{s\,w\,req} = \frac{238.4 \times 10^6}{0.9 \times 420 (500 - \frac{135}{2})} = 1460 < 1721$$

O.k

req < Provided لازم

One way Solid Slab

طريقة انتقال اللود
من منتصف السلاب
(بالاتجاه القصير)



لماذا ينتقل اللود بالاتجاه القصير ؟
لأن stiffness (K=4EI/L)
علاقة عكسية مع الطول

$$\frac{\text{Larger dimension}}{\text{shorter dimesion}} = L1/L2$$

>=2 one way solid slab

<2 two way slab

تصميم السلاب بكون للمتر المربع لذلك w بتكون Kn/m2

مهم جداً حتى تعرف إنه both end ولا one end دائما اطلع بالاتجاه القصير

يعني لو اعتبرنا بالرسمة فوق L2 هو الاتجاه القصير فإن سلاب 5 both end و سلاب 6 one end

توزيع اللود بالـtwo way solid بهذا الشكل

Section a-a

section bb

دائماً الـwidth بالسلاب = 1000mm

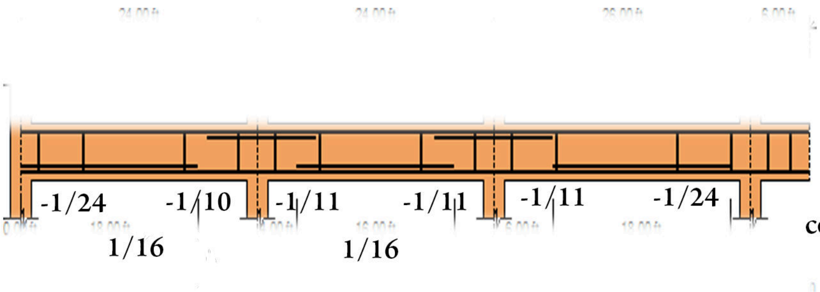
h min حساب الـ slab thickness
تذكر دائماً الـ L المستخدمة بالاتجاه القصير
لازم تحدد بالأول one end او two end ثم روح للجدول

cover of slab = 20mm

As= Mu/Φ fy jd
j=0.95 in one way solid slab

ACI code moment and shear cofficient for analysis and design of non-prestressed one way slabs and continous beams

$Mu= Cm(WuLn^2)$ $Vu = Cv (Wu Ln /2)$



تصميم السلاب بكون تصميم على مومنتين موجب وسالب .. الارقام بالرسمة هي الـcofficicnets ولما نصمم بستخدم أكبر مومنت

As min = 0.0018 bh for G60 steel

As min =0.002 bh for G40,50 steel مابنستخدمه

خطوات تصميم السلاب :

0) تشييك على one way or to two way slab أكيد لازم يظلع معك الطول كبير تقسيم الطول القصير أكبر من 2

1)estimate h min بدك تحدد both end or one end وتستخدم الجدول وتجد hmin والقيمة الي بتطلع معك زيدها 10 أو 20mm أو ستخدمها نفسها

2) Compute factoral load نحسب الـ (self wt = gamma * h) بطلع معي بوحدة kn/m2 زيدها للـDL واحسب U

3)التصميم للمونت السالب بتوجد أكبر مومنت سالب بعدها نحسب d حيث d= h -cover (20mm)-(db/2) db بحدده بالامتحان

-نجد As =Mu /Φfy jd رح يطلع معك كمية الحديد للمتر الواحد بس

- شييك As min ثم إذا طلع أقل من الـmin استخدم الـmin إذا طلع اكبر سوي itreations وبطلع معك req

- لاتجيب الـprovided بدل ذلك نحسب الـspacing

$$spacing = \frac{1000\ Ab}{As\ req}$$

Ab: area of one bar

-بعد ما جبت الـspacing بدك تشييك عالـS max

Smax(smaller of) = 3h or 450 mm لو طلع أكبر من Smax بستخدم Smax

4) تصميم المومنت الموجب نفس اسلوب خطوة 3 لكن للمومنت الموجب

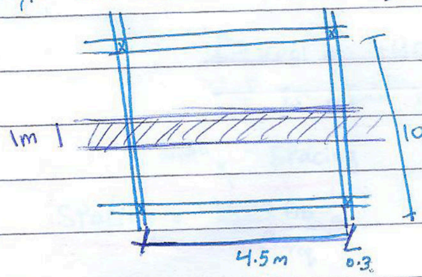
Per ACI code : shrinkage and temp reinfocment is required perpendeicular to spans of the slabs

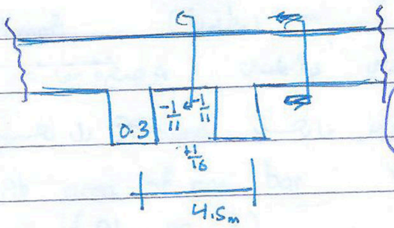
للصتميم من أجل shrinkage والـtemp هون بستخدم الـAs min ثم بجيب الـspacing

وبشييك على Smax هذي : Smax (Smaller of) =5h or 450 mm

• Design Example : $f'_c = 28$ Mpa $f_y = 414$ Mpa $w_{LL} = 4$ kn/m²

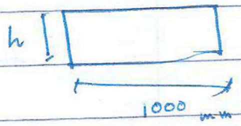
$w_{DL} + w_{LL}$ تجيب $U = 1.2DL + 1.6LL$ استخدم بدي ياها لحالعفين ← unfactored load $w_{DL} = 3$ kn/m²





$\gamma = 24 \frac{kn}{m^3}$ اسنة

$L_n = 4.5 - (2)(0.15)$ ln اول شي احسب
$= 4.2$ m



sol : ① Check one-way or two-way

$\frac{10,000}{4.5 \times 10^3} = 2.2272$ ∴ One way Center to Center

② estimate h_{min} both end con $h_{min} = \frac{l}{28} = \frac{4.5 \times 10^3}{28}$ (الفرق العنصر ↑)

استخدم زيادة 20 mm او min عند $= 160$ mm

$h = 180$ mm

3- Compute Factoral load

Self wt $= 24 \frac{kn}{m^3} * h_{(m)} = 24 \times 0.18 = 4.32$ kn/m²

$U = (4.32 + 3) \times 1.2 + 1.6(4) = 15.18$ kn/m²

* Mu — موجبة سالب Supp فوق ال*

$Mu(-ve) = \frac{W_u l_n^2}{11} = \frac{15.18 (4.2)^2}{11} = 24.34$ kn.m (+) موجبة

بحسب البنين وبحضرالأكبر (السالب) بفرقش نفس الاشي both end cont
ال (+) أحسب وحدة

معطى بالوتحي try use No 16

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

$As = \frac{Mu}{\phi f_y j d} = \frac{24.34 \times 10^6}{0.9 \times 414 \times 0.95 \times 152} = 452.4$ mm² للمتر الواحد

$As_{min} = [324$ mm² $< As$ o.k

itrrations $a = \frac{As f_y}{0.85 f'_c \times b} = 7.87$ mm

$As = 441$ mm² $a = 7.67$ mm $As = 440.9$ mm²

$As_{(req)} = 440.9$ mm² (مطلوبة)

. ماضي داي check ع a صغيرة مقارنة بـ h شكلة ع كبيرة

بالسلابز ماجسب Provided بل بجسب Spacing (width)

Spacing $= \frac{1000 \, Ab}{As_{req}}$ Ab, area of one bar 1 No16
($Ab = 199$ mm²)

$S = \frac{1000 \times 199}{440.9} = 451$ mm use 450 mm

. Per ACI $S_{max} = \{ 3h$, 450 mm $\}$ 3h = 450 mm

S_{max} بستخدم S_{max} لو طلع أكبر منه use 450 mm

$M_u(+ve) = \frac{w_u l_n^2}{16} = 16.74$ kn.m

$A_s = 311 mm^2$ check $A_{s\,min} = 0.0018(1000)(180) = 324 mm^2$

$A_s < A_{s\,min}$

use $A_{s\,min}$

~~ملاحظة~~ لما صمم لما يطلع عندي $A_{s\,min} > A_s$ لا اسوي etration

لانه بقلل etration A_s

$S = \frac{1000 \times 199}{324} = 614.2$ mm $S_{max} =$ → $3(189) = 614.2$ mm → 450 mm

use 450 mm

* Per ACI-code:

- Shrinkage and Temp reinforcement is required Perpendicular to spans of the slabs

للسقف (longer dimention) → $A_{s\,min}$

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

$S = \frac{1000 \times 199}{424} = 614.2 mm$

S_{max} smaller of = $5h = 900$; 450 mm ✓

use $S = 450$ mm

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

No. ______

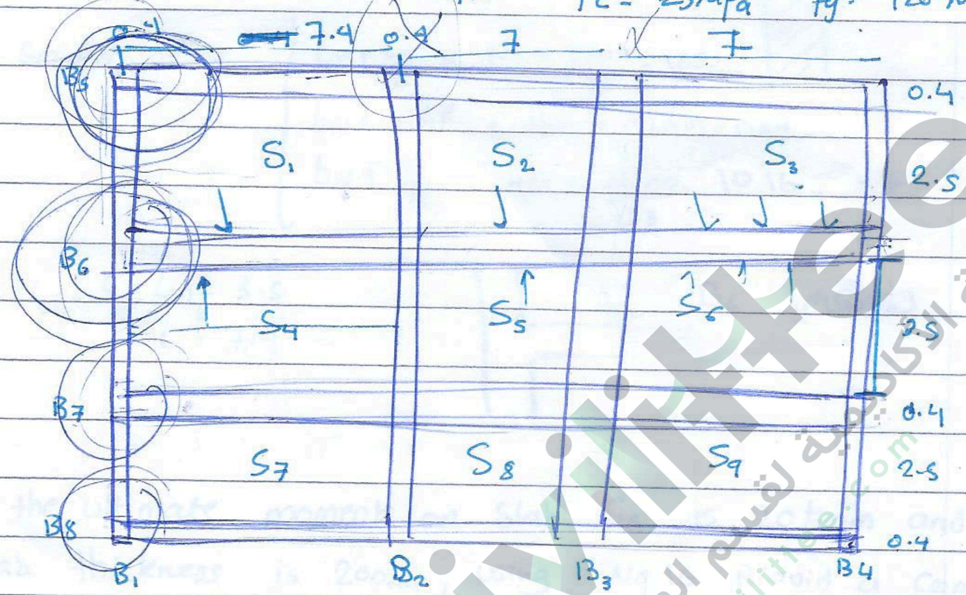
سنوات

Date ______

· all dimension in m (Face-to face)

DL (inculding Self wt) = 3 kN/m²

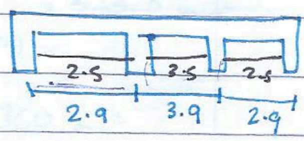
LL = 2.5 kN/m² f'_c = 25MPa f_y = 420 MPa



1- the min thickness in mm of S_4 to avoid deflection Calculation is most nearly:

two way on $h_{min} = \frac{L}{28} = \frac{3.5+2(0.2)}{28} \times 1000 = 139.28$ mm

2- the min thickness in mm of beam B_3 to avoid deflection Calculation is



✓ $h_{min} = \frac{L}{21} = 185.71$

$h_{min} = \frac{L}{18.5} = 156.75$

$h_{min} = 135.13$

نقشار الأثر

3- the Ultimate load in kN transfered from slabs to beam B_6 is most nearly

$U = 1.2 \times 3 + 1.6(2.5) = 7.6$ kN/m² → الي بحمله السلاب بالمتر المربع الواحد

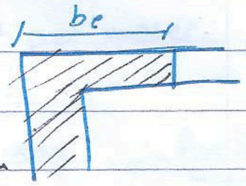
W in B_6 = $7.6 \times \frac{3.9}{2} + 7.6 \times \frac{2.9}{2} = 25.84$ kN/m.

من السلابة الي تحت   من السلابة الفوق

No. ______

Date ______

4. the effective Flange width in mm of beam B_5 is most nearly? assume ($h_f = 200$ mm)

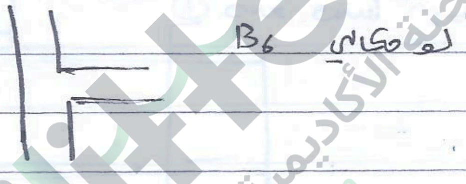


be smaller of =

$$\begin{cases} b_w + \frac{L_n}{2} = 400 + \frac{2500}{2} = 1650 \text{ mm} \\ b_w + 6h_f = 400 + 6(200) = 1204 \text{ mm} \\ b_w + \frac{L}{12} = 400 + \frac{7400}{12} = 1016 \text{ mm}^2 \end{cases}$$

$L_{n_1} = 2.5$ $L_{n_2} = 3.5$

$L = 7.4$



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

$h = 200$ mm $U = 7.6$ kN/m² $M_u = 20$ k.m

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

$$A_s = \frac{M_u}{\phi f_y jd} = \frac{20 \times 10^6}{0.95 * 420 * 0.9 (172)} = 323.8 \text{ mm}^2$$

$A_{s\,min} = 0.0018 bh = (0.0018)(1000)(200) = 360$ mm²

$A_s < A_{s\,min}$ we take $A_{s\,min} = 360$ mm²

$$\text{Spacing} = \frac{1000 A_b}{A_s} = 552.77 \text{ mm}$$

Spacing max = → 3(200) mm → 450 mm we use S = 450 mm

* long dimetion $A_{s\,min}$ نستخدم

$A_{s\,min} = 360$ mm²

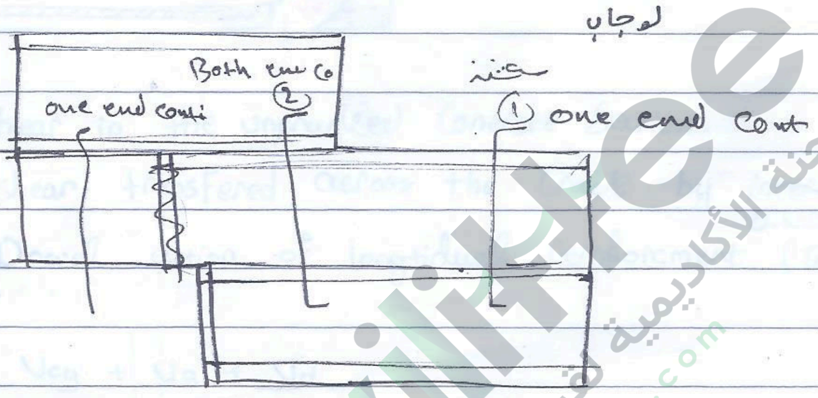
S = 552.7 mm we use $S_{max} = 450$ mm

No. ______

Date ______

⑥ 1ϕ slab (S5) is reinforcement with 1N014 spaced at 300 mm the provided area of steel in mm^2 per meter width of the slab is most nearly:

$$Spacing = \frac{1000 A_b}{A_s} = 300 \qquad A_s = 513.1\ mm^2$$



هنا تنتهي مادة السكند ...

سأقوم بإرفاق ملف ملحق بأسئلة السنوات محلولة قبيل الامتحان بإذن الله

وفي حال لاحظت أي خطأ أو عندك استفسار بالدوسية أو لطلب الملحق الكترونياً

تواصل معي على الفيس:

https://www.facebook.com/ezrawille

Final

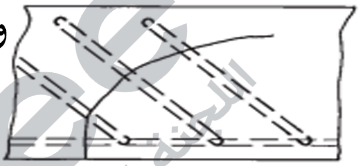
shear in beams

Sittups : shear reinfocment web reinforcment

الNormal stress بعمل crack بزاوية 90 لذلك الحديد يكون عمودي على الكراك

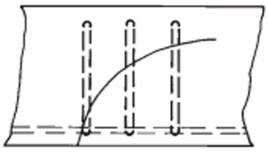
الShear stress بعمل crack بزاوية 45

قلنا أن الحديد يجب أن يكون عمودي على تأثير الشير أو الكراك

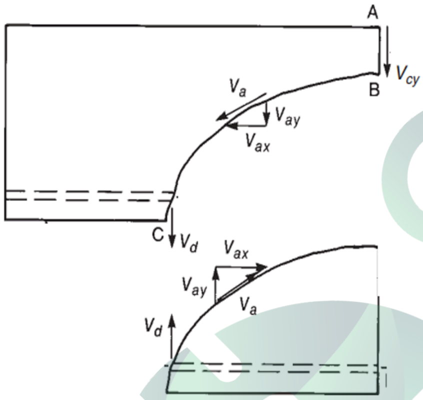


(c) Inclined shear reinforcement.

ولكن هذا الشيك صعب التطبيق عمليا لذلك نستخدم الـstirrups



يكون معظم الـsirrups بالقرب من الـsupp حيث الـShear stress Max



Internal forces in beam without stirrups

Vcy : Shear in the uncracked concrete section

Va :Shear transferd across the crack by interlock

Vd :Dowel action of longtidunal reinforcment

$Vc = Vcy + Va + Vd$

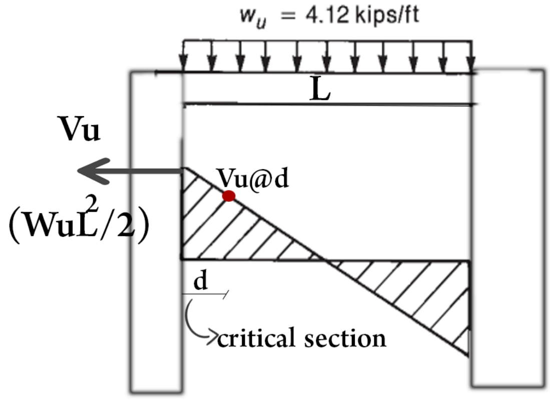
Vc : concrete resistance strength

(Shear carried by the Conctere)

With stirrups : Vs : shear carried by the stirrups

$Vn = Vc + Vs$ Vn:nominal shear strength

$\Phi Vn >= Vu$ Φ in shear is always = 0.75

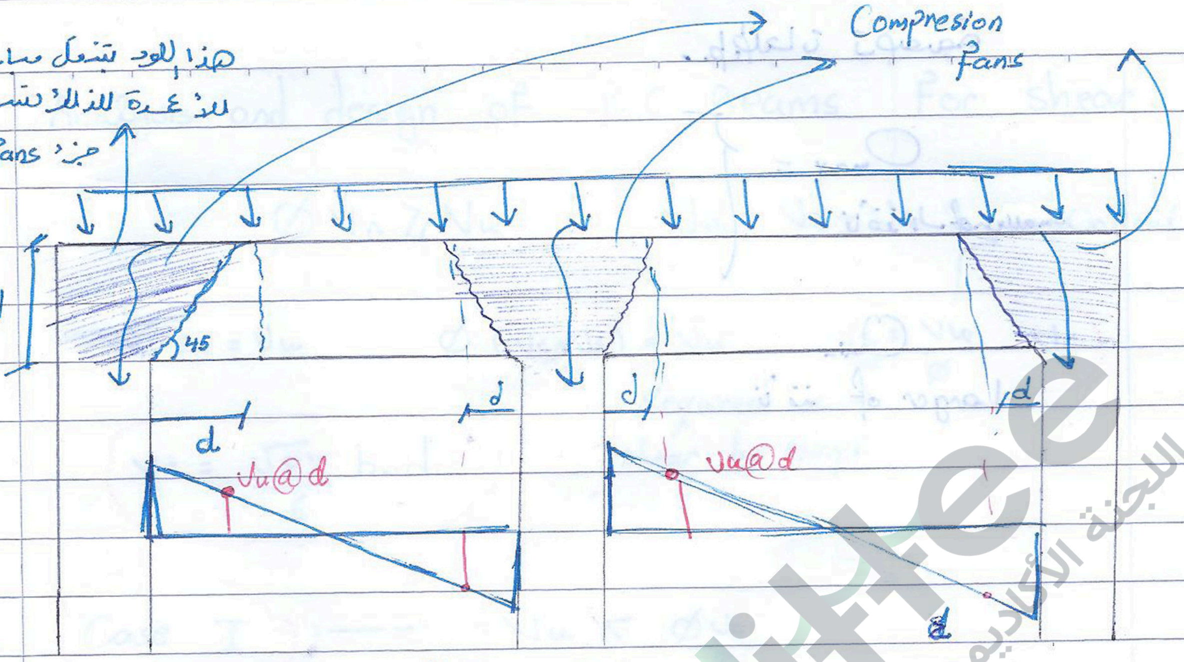


d : effective depth

Vu : at face of the support

لكن نحن بنتعامل بحساباتنا مع Vu@d

No.
Date



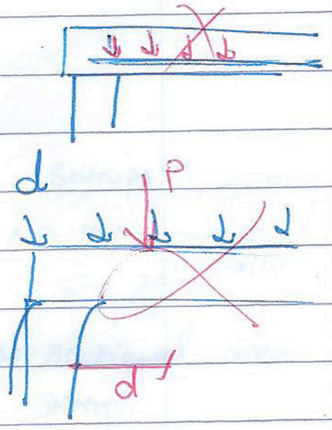
d في كل مكانة تنعمل بالمسافة لأن الزاوية تنرسلها 45°

Use Vu@d only if 8—

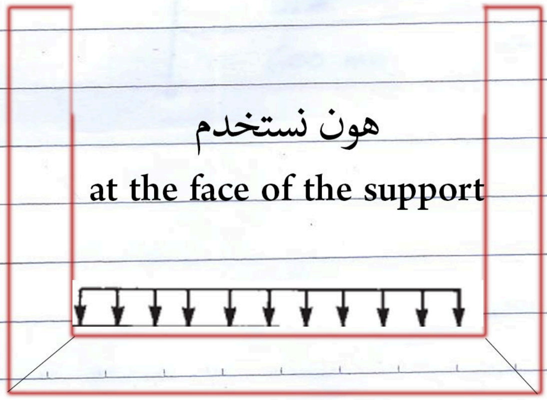
1- Support reaction ~~reduces~~ introduces Comprison into the end region of the beam.

2- loads are applied into the top of the beams

3- No Concentrated force within a distance d from the face of the Support.



~~Analysis~~



No. ______

Date ______

Analysis and design of R.C Beams for shear.

$\phi V_n \geq V_u \qquad V_n = V_c + V_s$ → at Critical Section

$\phi V_n = V_u \qquad \phi (V_c + V_s) = V_u \qquad V_s = \frac{V_u}{\phi} - V_c$

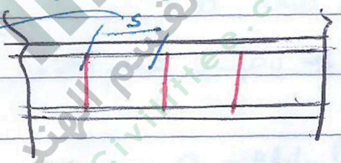
Required shear by stirrps

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

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

• No need for shear reinforcement. (الباطون وحده يقاوم الشير) (ما بحتاج stirrups)

متروكة للمهندس ← ممكن يستخدم الـ stirrups حتى أمسك حديد المسابح فقط



* الفائدة من spacing • منع تقوس الحديد.
* كلما قوة الحديد أعلى spacing أكبر وذلك صحيح.

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

• min reinforcement

S_{max} smaller of (مش الأقل) =

$$\frac{16 A_v f_y}{\sqrt{f'_c}\, b_w}$$

$$\frac{A_v f_y}{0.33\, b_w}$$

$$d/2$$

$$600 \text{ mm}$$

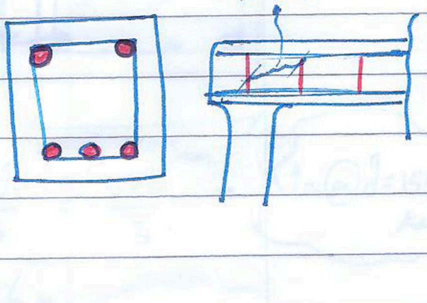
$A_v = 2 A_{v1} = 2 \frac{\pi}{4} d_s^2$ → 10

(2) ← لأن التشقق ليؤثر على حبتين من Stirrups

حديد الـ Stirrups هو حديد NO 10M

A_{v1} : Cross sectional area of Stirrups

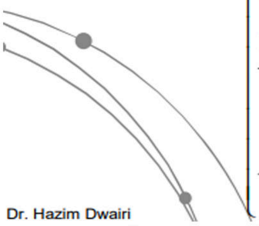
شعر كراك



No. ______
Date ______

Case III :- $V_u > \phi V_c$
and
$V_s \le \frac{2}{3}\sqrt{f'_c}\, b_w d \rightarrow 4 V_c$

$$S = smallest\ of \begin{cases} 600\ mm \text{ or } \dfrac{d}{2};\ if\ V_s \le 2V_c \\ 300\ mm \text{ or } \dfrac{d}{4};\ if\ V_s > 2V_c \\ \dfrac{A_v f_y}{0.33 b_w} \\ \dfrac{16 A_v f_y}{b_w \sqrt{f'_c}} \\ \dfrac{A_v f_y d}{V_s} \end{cases}$$



Dr. Hazim Dwairi

* $V_{s_{max}} = 4 V_c$ max shear carried by stirrups

if Vs>Vmax ehtn enlarge the section

$V_{s_{max}} = \frac{2}{3}\sqrt{f'_c}\, b_w d$ (b and h)

* يجب عدم تجاوز $4V_c$ وإلا بزيد h و b .

$V_{u_{max}} = \phi (V_c + V_{s_{max}}) = 5\phi V_c$

الماكس بعد الشير

IF $V_u < V_{umax}$ O.K

$V_u > V_{max}$ enlarge the Section

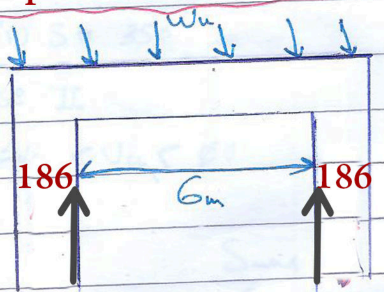
بالتالي لو طلب الماكس شير كاامل فهو :

$V_{u\,max} = \Phi\,(V_c + V_{s\,max}) = 5\,\Phi V_c$

if Vu<Vmax then ok

if Vu>Vmax then elarge the section

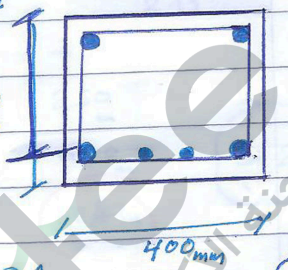
Example



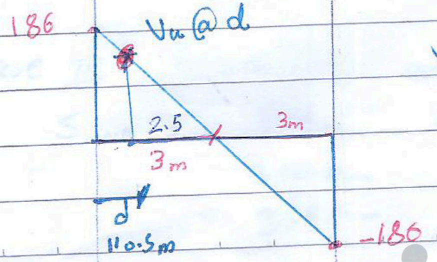
LL = 20 kN/m   DL = 25 kN/m (Including)

f'_c = 28 MPa

f_y = 300 MPa



$W_u = 1.6 * 20 + 1.2 * 25 = 62$ kN/m



Vu@d / Vu@d / 2.5 = 186 / 3   Vu@d = 155 kN

Vu max = 5 Vc ϕ

$V_c = \frac{\sqrt{f'_c}}{6} b_w d$ = 176.4 kN

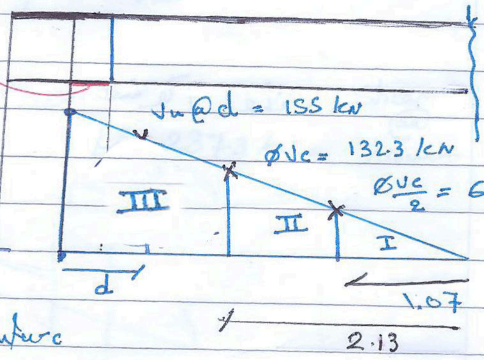
Vu max = 661.4 kN

Vu@d < Vumax O.K

$\frac{\phi V_c}{2}$ = 66.15   ϕVc = 132.3   ? Case   الآن نحدد أي

$V_s = \frac{V_u@d}{\phi} - V_c$ = 30.3 kN

Vu@d > ϕVc   Vs < 4Vc   1.07



صمون بحط أول كانة 10 cm or S/2

Case I

No need to shear reinforcement

(أوجب اعتبار الحد) S = 350 mm

Case II

$\frac{\phi V_c}{2} < V_u < \phi V_c$ min reinforc

S_{max} Smaller of = 777 mm, 356 mm, 356.8 mm   250 mm, 600 mm, 250 mm   S = 250 mm

Case III   Vu > ϕVc and Vs ≤ 4Vc

S_{max} = 777 mm   d/2 = 250

356.8 mm   600 mm

356 mm

Vs = 30.3 kN < 2Vc

S = 250 mm

No. ____

Date ____

مطالعة ذاتية

Cases of loading

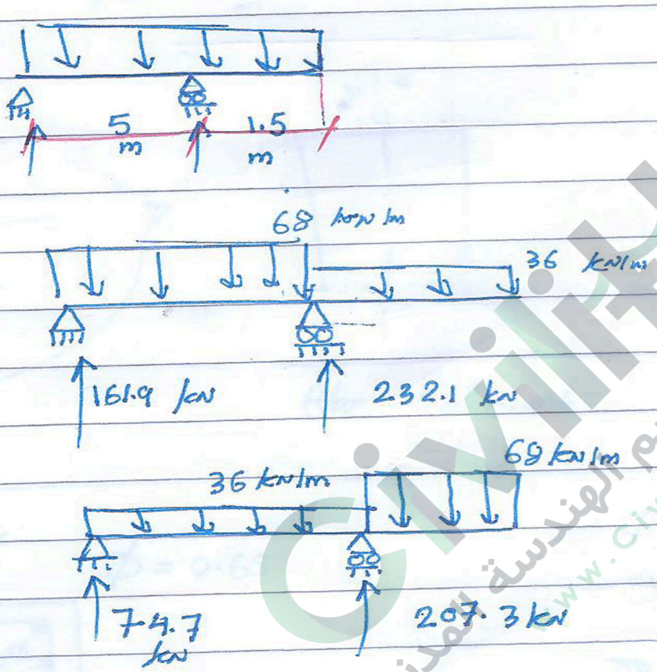
LL = 20 kN/m

DL = 30 kN/m

1.6 x 20 = 32

1.2 x 30 = 36

W_u = 68 kN/m

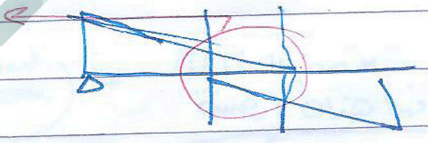


نأخذ أكبر قيمة لل reaction من الحالات الثلاث

R_A = 161.9 kN R_B = 287.3 kN

* لعمل Shear and Moment dia

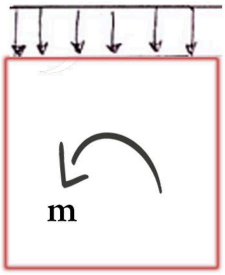
رغم اختلاف الشكل هذه الحالة لا تعني Case I لأنها

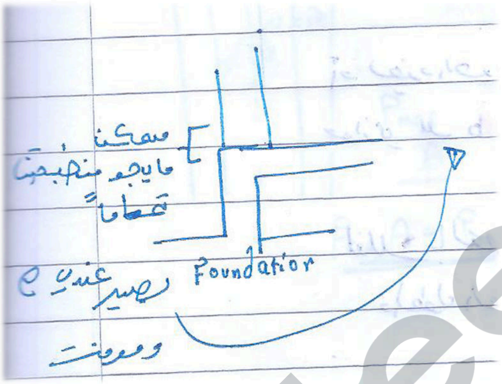


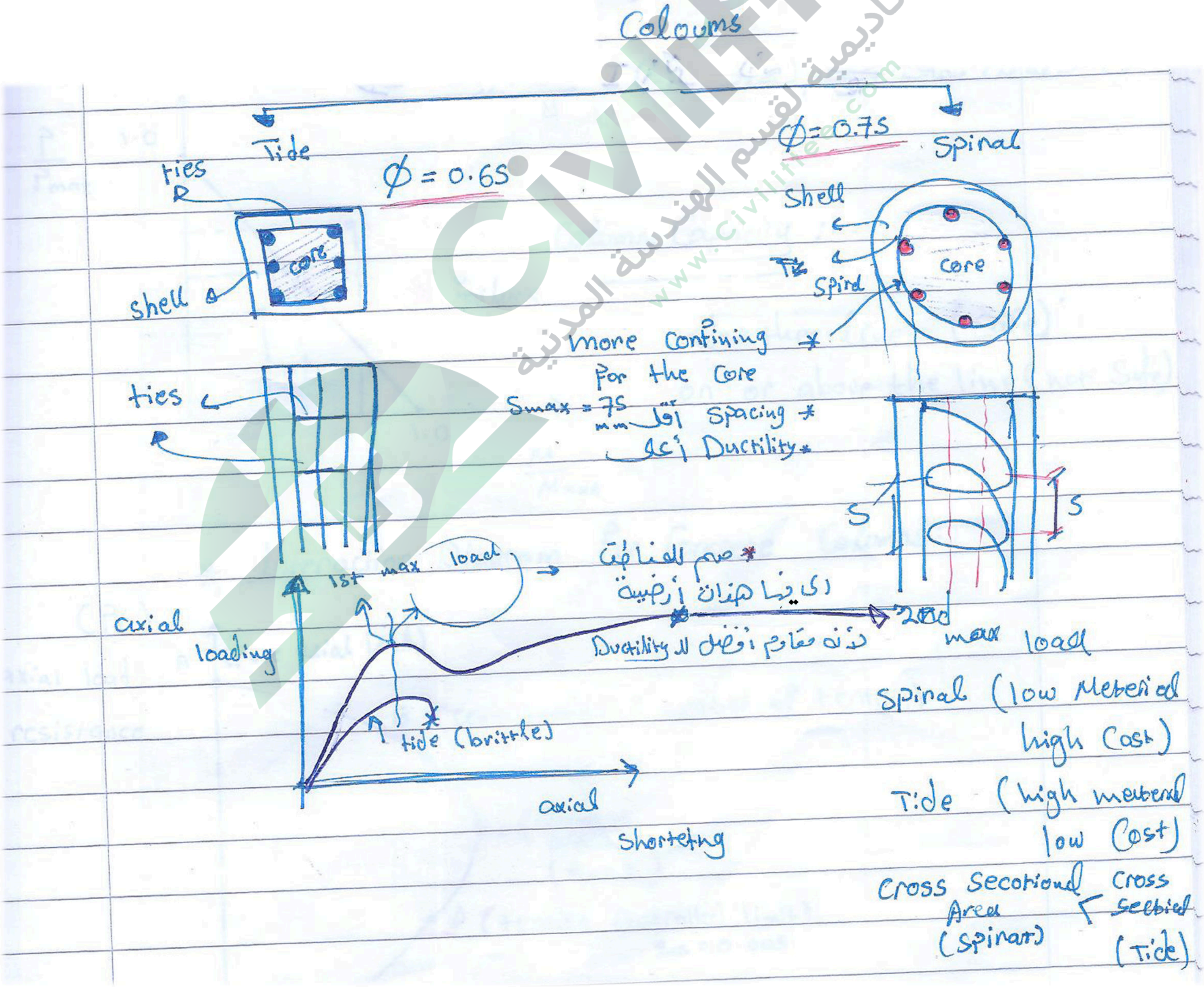
Design of Coloumns

Combined axial loads and bending

Normal stress σ=p/a **bending σ=My/I**



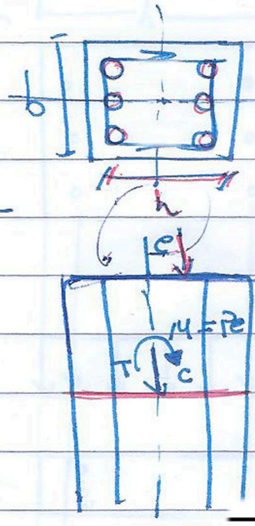




No. ______

Date ______

* Interaction Diagram



$\frac{P}{A} + \frac{My}{I} = f_{cu}$ (compresive Strength)

تركيب العمود جهز لشكل (ما تحت)

لأنه $I = \frac{bh^3}{12}$ تعطيني قوة أكبر

Interaction eqatuion

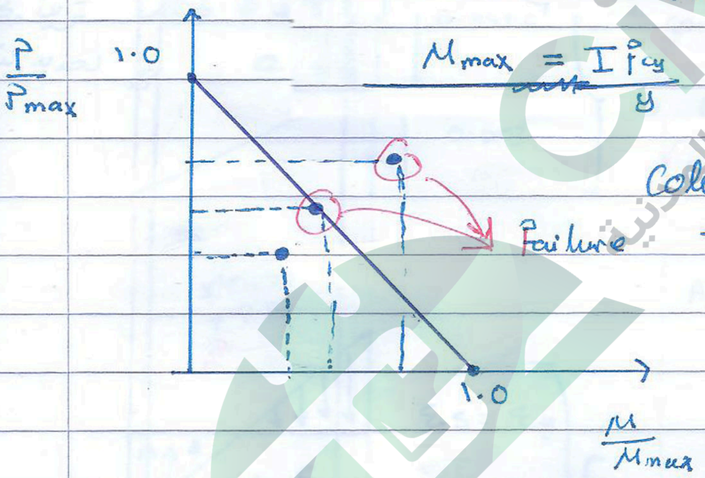
$$\frac{P}{f_{cu} A} + \frac{My}{f_{cu} I} = 1.0$$

$\frac{P_{max}}{A}$ ← f_{cu} → $f_{cu} = \frac{M_{max}\, y}{I}$

$$\frac{P}{P_{max}} + \frac{M}{M_{max}} = 1.0$$

$f_{cu} A = P_{max}$ (Max axial load of the column Can Support)

$M_{max} = \frac{I f_{cu}}{y}$ (أكبر موومنت ممكن يتحمله العمود)

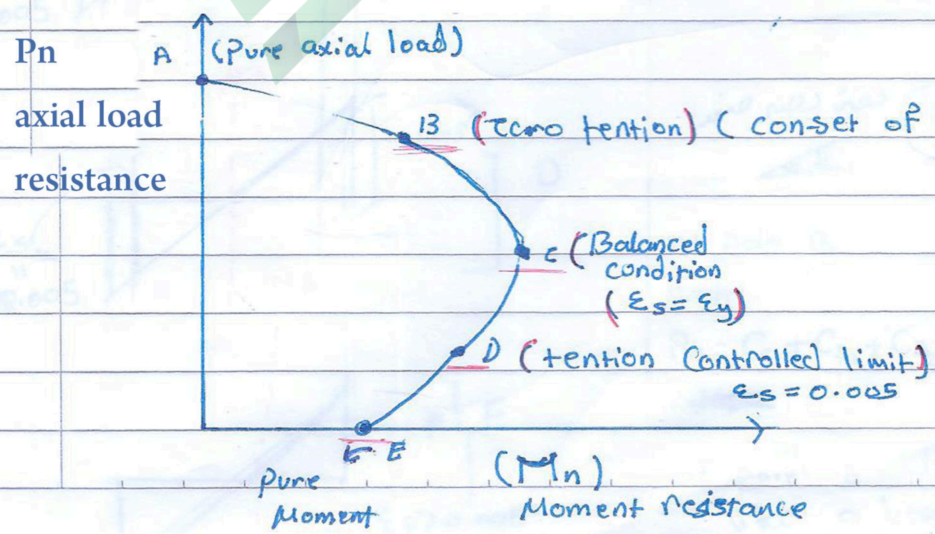


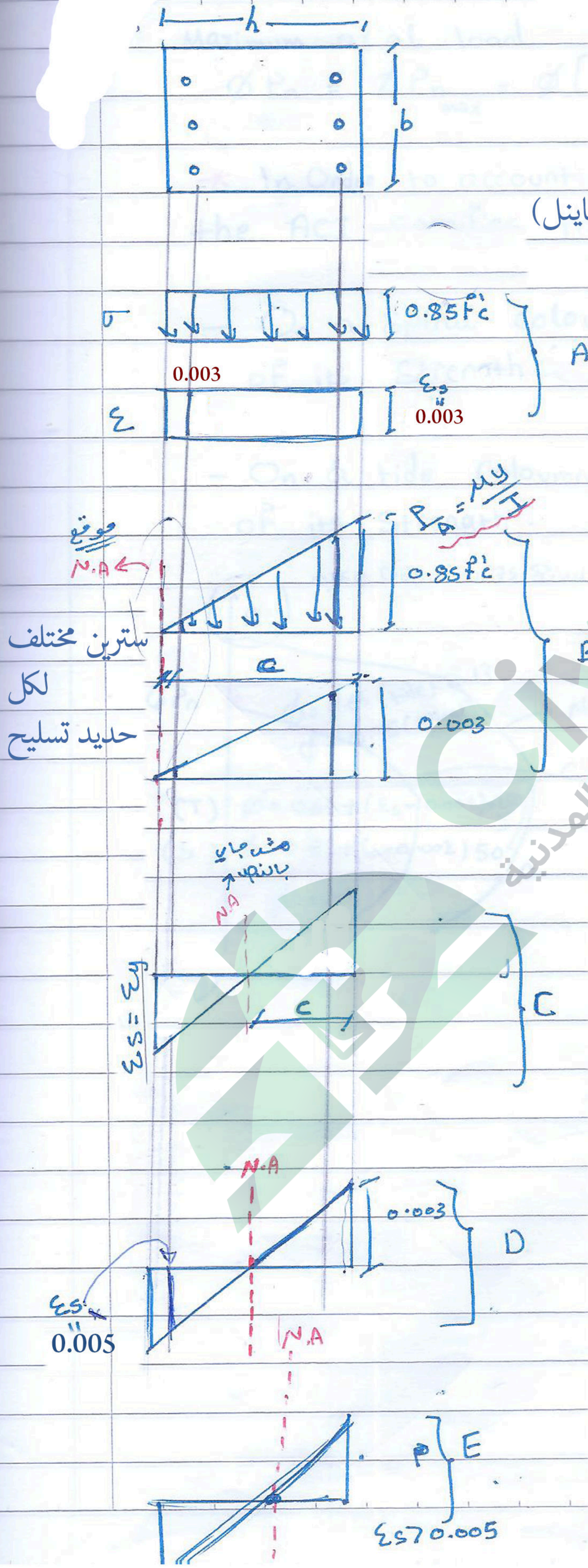
Column Capacity :

under ther line (Safe)

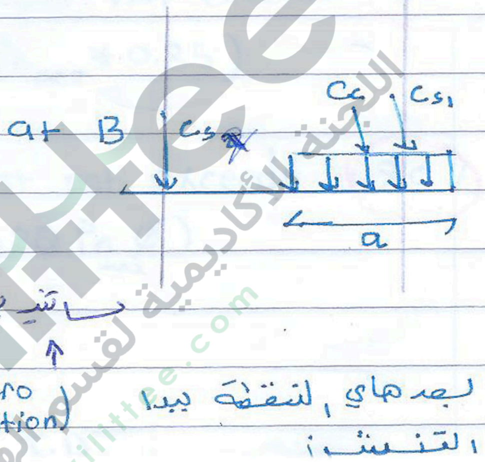
on or above the line (not Safe)

* Interaction Diagram for Concrete Columns:





مقدار السترين بالحديد في حالة الـpure load هو : (أجا فاينل) هو 0.003



توضيح كل نقطة من النقاط في الصفحة الجاي

معلومة جدا هامة ..
كيف تميز بين h ,و b للعمود :
h : عمودي على الـaxis of bending
أو يفرد الحديد دائماً باتجاه b

No. ______

Date ______

Calculation of Interaction Diagram *

Point A (Pure axial load)

Ag :cross sectional area (bh)

$$P_n = 0.85 f'_c (A_g - A_s) + A_s f_y \qquad M_n = 0.0$$

Point B

① $C = h$ ② $a = C\beta$

③ نجد ε_{s_1} و ε_{s_2} منه تشابه المثلثات

④ نشيك على ε_y

$\varepsilon_{s_2} > \varepsilon_y$ → $f_s = f_y$

⑤ $C_c = 0.85 f'_c * a * b$

$C_{s_1} = F_s * \frac{A_s}{2}$ — تعوض F_s اذا $\varepsilon_{s_2} < \varepsilon_y$

(Comp block الواقع داخل) $C_{s_2} = \frac{A_s}{2} * ($fs or fy $- 0.85 f'_c)$

طرحنا لأنه بدي السترس تاع الحديد دون الكونكريت
عشانه داخل بالبلوك بكونو مع بعض

ε_y

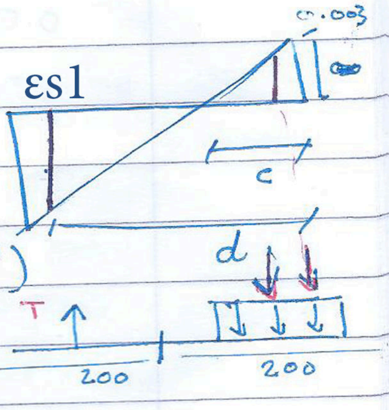
$$* \; P_n = C_c + C_{s_1} + C_{s_2}$$

$$* \; M_n = C_c\left(\frac{h}{2} - \frac{a}{2}\right) + C_{s_1}\left(\frac{h}{2} - d'\right) + C_{s_2}\left(\frac{h}{2} - d'\right)$$

Point C (Balanced Condition)

$$\frac{\varepsilon_{s_1}}{d-c} = \frac{0.003}{c} \quad \text{find } c \quad \{\varepsilon_{s_1} = \varepsilon_y\}$$

$$\frac{\varepsilon_{s_2}}{c - 65} = \frac{0.003}{c} \quad \text{find } \varepsilon 2$$



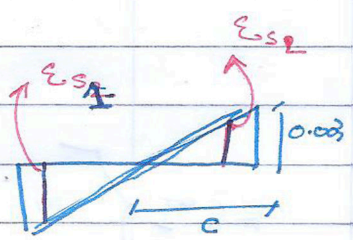
$$P_n = C_c + C_{s_2} - T_{s_1}$$

$$M_n = C_c\left(\frac{h}{2} - \frac{a}{2}\right) + C_{s_1}\left(\frac{h}{2} - d'\right) + T_{s1}\left(\frac{h}{2} - d'\right)$$

No.

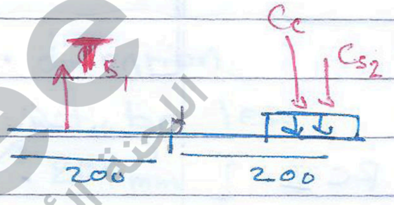
Date

Point D   Tension Controlled limit :



$\varepsilon_{s_1} = 0.005$

$\frac{\varepsilon_{s_1}}{d-c} = \frac{0.003}{c}$ (c نجيب)



$\frac{\varepsilon_{s_2}}{c-65} = \frac{0.003}{c}$

$P_n = C_c + C_{s_2} - T_{s_1}$

$M_n = \Sigma M_\phi = C_c\,(200 - \frac{a}{2}) + C_{s_2}(200-65) + T_{s_1}(200-65)$

Point E   Pure Bending (Ignore Comp reinforcement)

① $T = C$

$0.85 f'_c * a * b = \left(\frac{A_s}{2}\right) F_y$ ← داخل

(A_{s_1} أو) (ع الـ تنشن) . $a =$ $c =$

② $\varepsilon_s = ? > \varepsilon_y$

③ $M_n = A_{s_1} F_y (d - a/2)$ $P_n = 0.0$

$\left(\frac{A_s}{2}\right)$

أنواع الأعمدة (تسميات)

Pure axial eccentricity تتعرض للود بدون مومنت مفي

uni axial y وإما X إما eccentricity حول محور واحد يتعرض لمومنت نتيجة

bei axial Y و X يتعرض لمومنت حول محورين

Point E pure bending

ignore compression reinforcment

T=C $0.85f'c\, ab = \frac{As\ total}{2} fy$ دائماً هون fy

$\frac{As\ total}{2}$ → As2 أو تاع التنشن

then find a= and c=

2-check $\varepsilon s >= \varepsilon y$?

3-Mn=As2 fy (d-0.5*a) Pn=0.0

(As2 = As total/2)

Maximum axial load :

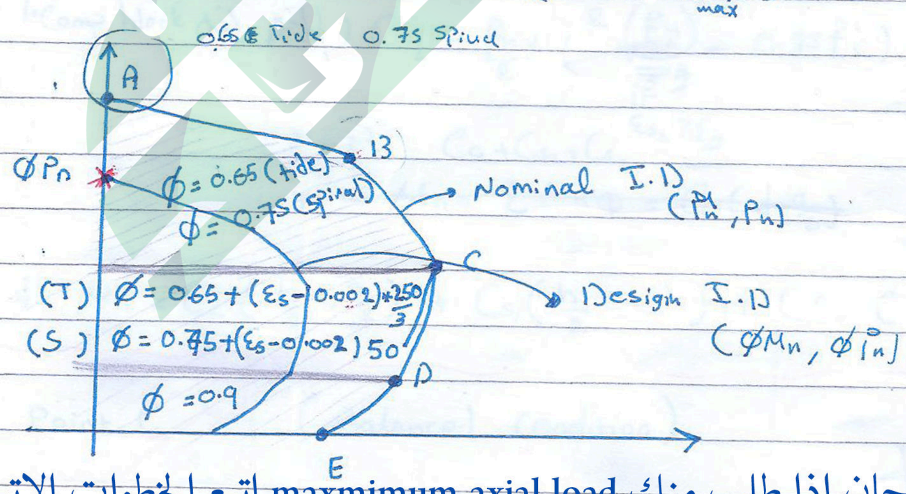
$\phi P_n = \phi P_{n\,max} = \phi [0.85 f'c (A_g - A_s) + A_s f_y]$

Ag :cross sectional area (bh)

* In Order to account the effect of accidential Moments : the ACI specifies that the max load on :

- On a spiral coloumn must not exceeds 85 % of its strength. $(\phi P_{n\,max} * 0.85)$

- On a tide Coloumn must not exceeds 80 % of its strength. $(0.8\ P_{n\,max}\ \phi)$



مهم جدا جدا بالامتحان إذا طلب منك maxmimum axial load اتبع الخطوات الاتية

1) جد ΦPn من المعادلة فوق

2) حدد هو tide ولا spiral

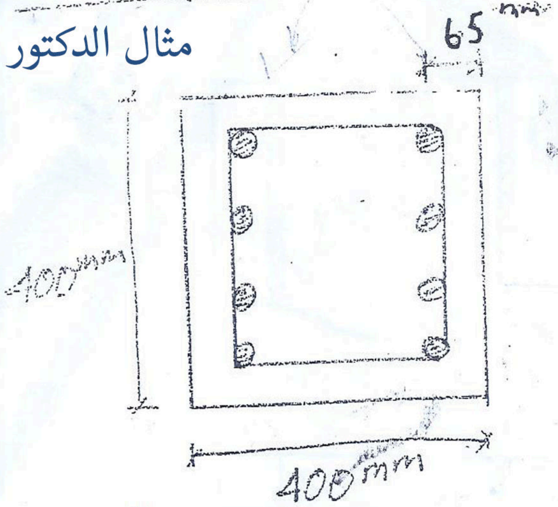
3) قله but it must not exceedes 0.8ΦPn or 0.85 ΦPn

واحسبله 0.85 ΦPn أو 0.8ΦPn حسب نوع العمود

د. بلال المصول   خرسانة مسلحة (1)   ٤/١٢

Example :- Calculation of Interaction Diagram.

مثال الدكتور



$f'_c = 35\ MPa$, $f_y = 420\ MPa$

$A_s = 8\ No.29\ M = 5160\ mm^2$

العمودي على axis of bending

Point A: Pure Axial load.

$P_n = 0.85\ f'_c (A_g - A_s) + A_s f_y$

$P_n = 0.85 \times 35 \times (400 \times 400 - 5160) + 5160 \times 420$

$P_n = 6773.69\ kn.$
$M_n = 0.0$

Point B:- zero Tension

$c = 400\ mm$; $a = \beta_1 c$; $\beta_1 = 1.09 - (0.008 \times 35)$
$\beta_1 = 0.81$

$\Rightarrow a = 0.81 \times 400$

$a = 324\ mm$

* $\frac{0.003}{400} = \frac{\varepsilon_{s1}}{65} = \frac{\varepsilon_{s2}}{400-65}$ $\Rightarrow \varepsilon_{s1} = 4.875 \times 10^{-4} < \varepsilon_y$ $(f_{s1} \neq f_y)$

$\varepsilon_{s2} = 0.00251 > \varepsilon_y$ $(f_{s2} = f_y)$

$f_{s1} = 4.875 \times 10^{-4} \times 200,000$
$= 97.5\ MPa$

$C_c = 0.85 \times 35 \times (324 \times 400) = 3855.6\ kn$

$C_{s1} = 2580 \times 97.5 =$ 251.55kn

com stress — هنا تأثير الكونكريت لأنه C_2 لأنها شد وضغط

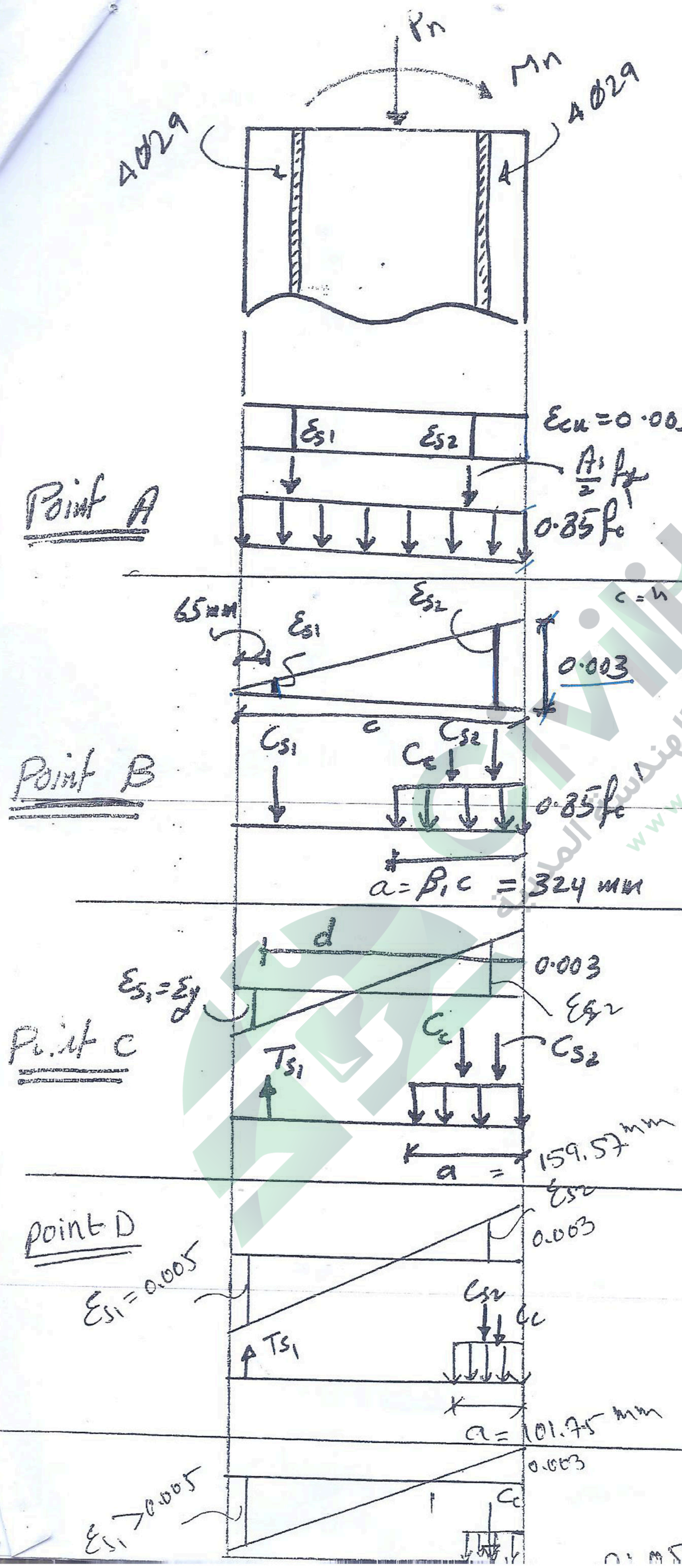
$C_{s2} = 2580\ (420 - 0.85 \times 35) = 1006.8\ kn.$

* $P_n = C_c + C_{s1} + C_{s2} = 5114\ kn.$

$M_n = \sum M_{⊈} = C_c(200 - \frac{a}{2}) - C_{s1}(200 - d') + C_{s2}(200 - d')$

$= 3855.6 \times (200 - \frac{324}{2}) - 251.55(200-65) + 1006.8(200-65)$

$= \ldots 48.5\ kn.m$



Point A:
$P_n = 6773.69$
$M_n = 0.0$

Point B:
$P_n = 5114$ kN
$M_n = 248.5$ kN.m

Point C:
$P_n = 1775.8$ kN
$M_n = 504$ kN.m

Point D:
$P_n = 797.36$ kN
$M_n = 417.32$ kN.m

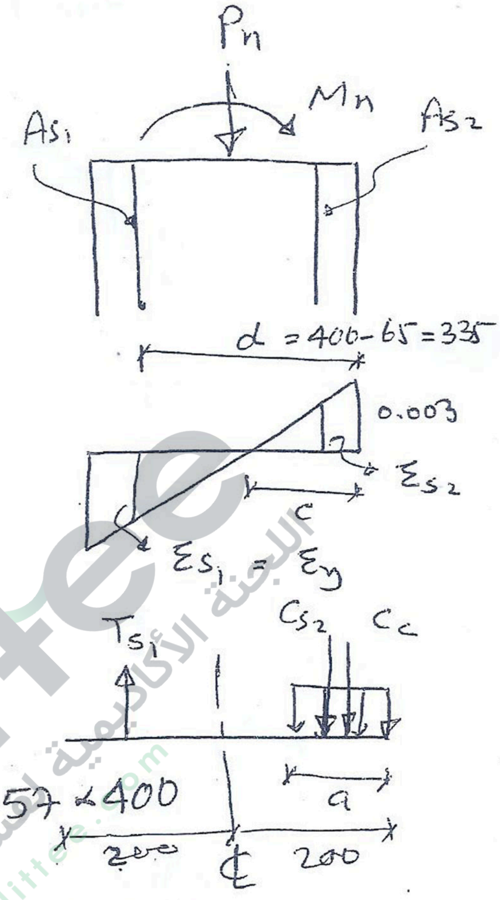
$P_n = 0.0$
$M_n = 313.67$ kN.m

Point C :- Balanced Condition

$$\frac{\varepsilon_{s1}}{d-c} = \frac{0.003}{c} \qquad \varepsilon_{s1} = 0.0021$$

$$\Rightarrow c = 197 \text{ mm}$$

$$a = 0.81 \times 197 = 159.57 \text{ mm}$$



$$\frac{\varepsilon_{s2}}{c-65} = \frac{0.003}{c} \Rightarrow \varepsilon_{s2} = 0.00201 < \varepsilon_y$$

$$\Rightarrow f_{s2} = 0.00201 \times 200{,}000 = 402 \text{ MPa}$$

$$C_c = 0.85 f'_c a b = 0.85 \times 35 \times 159.57 \times 400 = 1899 \text{ kN}$$

$$C_{s2} = 2580 (402 - 0.85 \times 35) = 960.4 \text{ kN}$$

$$T_{s1} = 2580 \times 420 = 1083.6 \text{ kN}$$

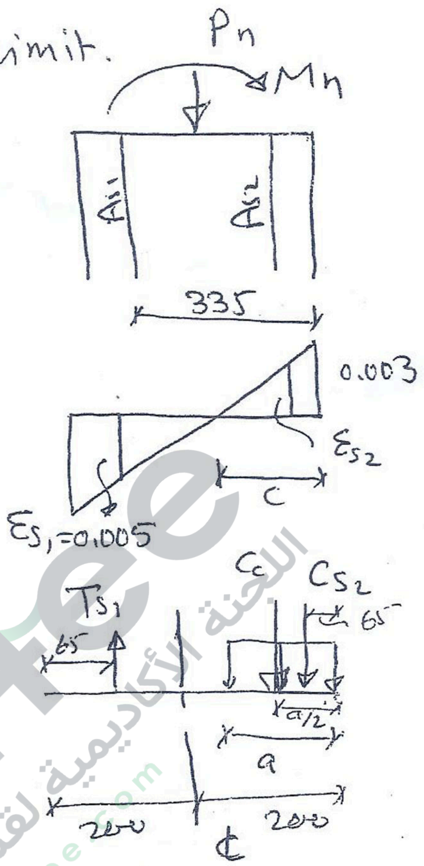
$$P_n = C_c + C_{s2} - T_{s1} = \boxed{1775.8 \text{ kN}}$$

$$M_n = \Sigma M_{℄}$$

$$= 1899 \times \left(200 - \frac{159.57}{2}\right) + 960.4 \times (200 - 65)$$

$$+ 1083.6 \times (200 - 65) = \boxed{504 \text{ kN.m}}$$

Point D: Tension Controlled - limit.



$$\frac{\varepsilon_{s1}}{335-c} = \frac{0.003}{c} \quad ; \quad \varepsilon_{s1} = 0.005$$

$\Rightarrow c = 125.6^{mm}$

$a = 0.81 * 125.6 = 101.75 \text{ mm}$

A_{s2} (within a)

$$\frac{\varepsilon_{s2}}{c-65} = \frac{0.003}{c} \Rightarrow \varepsilon_{s2} = 0.001447 < \varepsilon_y$$

$\Rightarrow f_{s2} = 0.001447 * 200{,}000$
$= 289.5 \text{ Mpa.}$

$C_c = 0.85 * 35 * 101.75 * 400 = 1210.8 \text{ kn.}$

$C_{s2} = 2580 * (289.5 - 35*0.85) = 670.16 \text{ kn.}$

$T_{s1} = 2580 * 420 = 1083.6 \text{ kn.}$

$P_n = C_c + C_{s2} - T_{s1} =$ ~~2964.5~~ $\boxed{797.36 \text{ kn.}}$

$M_n = \Sigma M_{℄}$

$= 1210.8 * (200 - \frac{101.75}{2}) + 670.16 * (200 - 65)$
$+ 1083.6 * (200 - 65)$

$\boxed{= 417.32 \text{ kn.m}}$

Point E: Pure Bending (Beams)

Ignore Comp. reinforcement (A_{s2})

$C = T$

$0.85 \times 35 \times a \times 400 = 2580 \times 420$

$a = 91.05$ mm

$c = \frac{91.05}{0.81} = 112.41$ mm

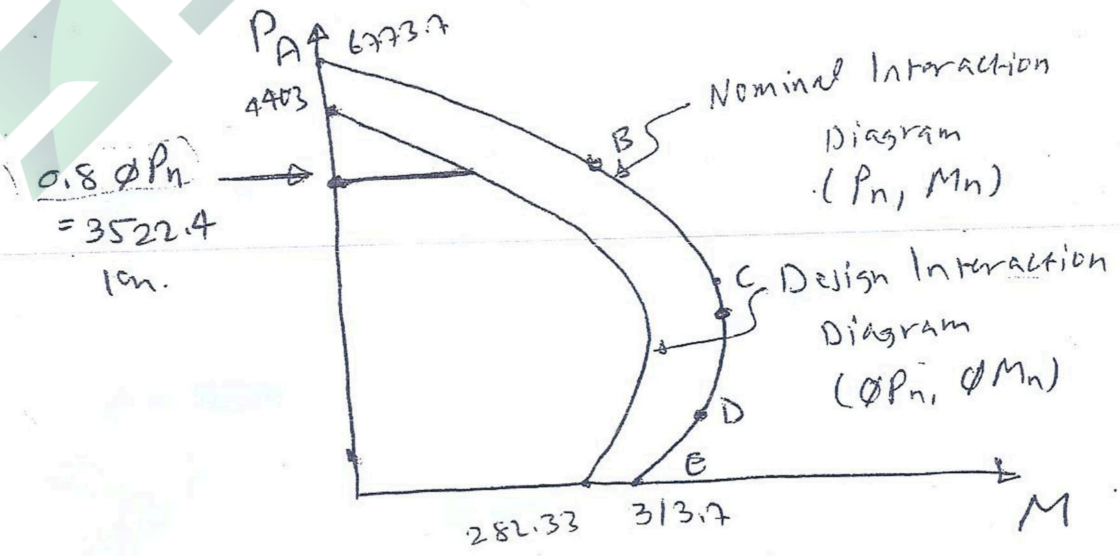
$\varepsilon_s = 0.003 \left(\frac{335 - 112.41}{112.41} \right) = 0.00594 > \varepsilon_y$

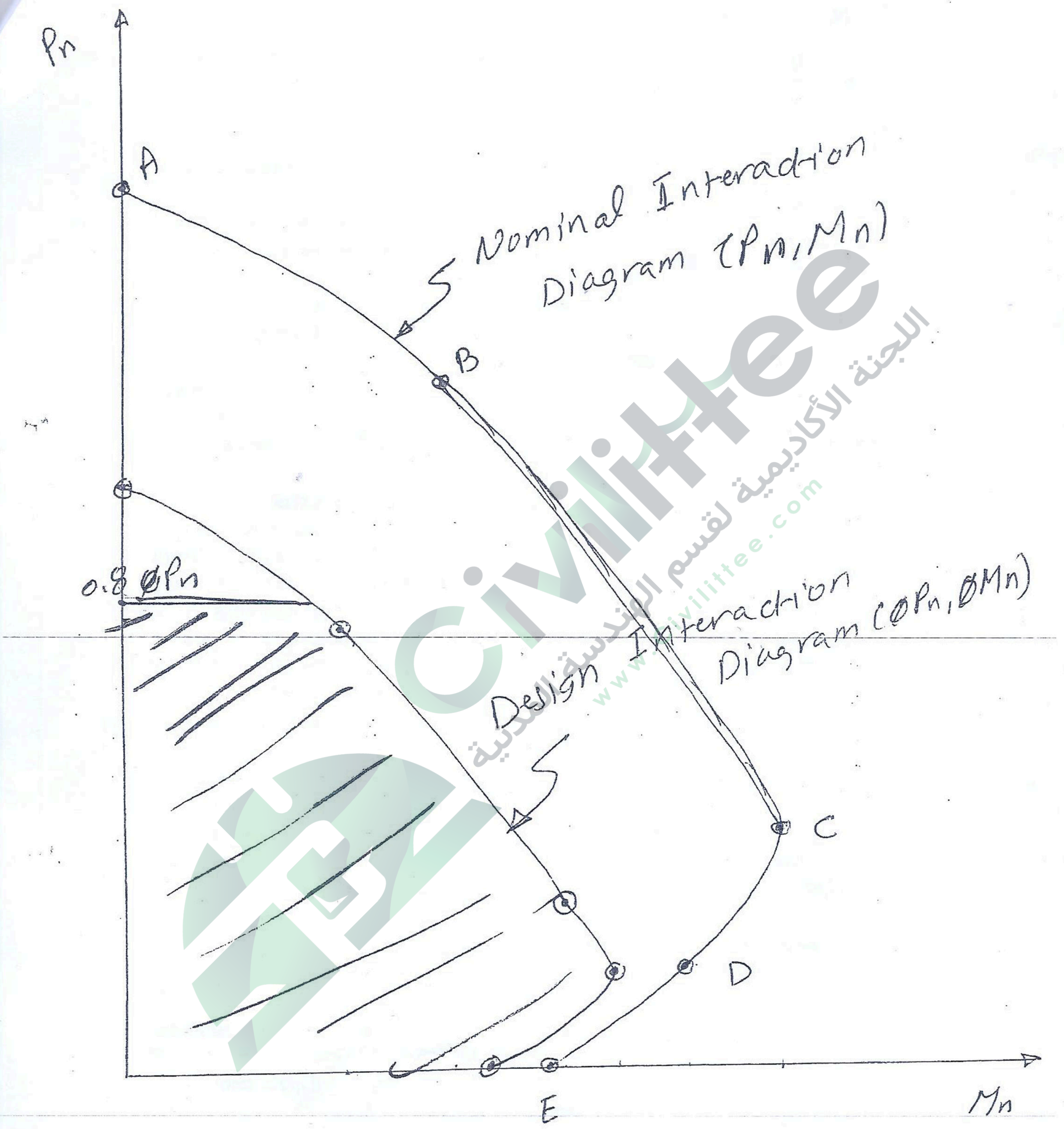
$M_n = A_{s1} f_y \left(d - \frac{a}{2}\right)$

$= 2580 \times 420 \left(335 - \frac{91.05}{2}\right) = 313.67$ kN.m

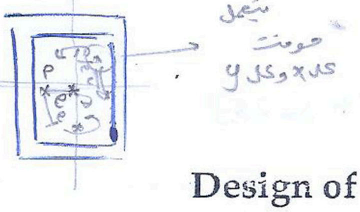
$P_n = 0.0$

Point	ε_t	ϕ	P_n (kN)	M_n (kN.m)	ϕP_n (kN)	ϕM_n (kN.m)
A	—	0.65 (C.C.)	6773.7	0.0	4403	0.0
B	0.0004875	0.65 (C.C.)	5114	248.5	3324.1	161.5
C	0.0021	0.65 (C.C.)	1775.8	504	1154.3	327.7
D	0.005	0.9 (T.C.)	797.36	417.32	717.62	375.59
E	0.00594	0.9 (T.C.)	0.0	313.67	0.0	282.3



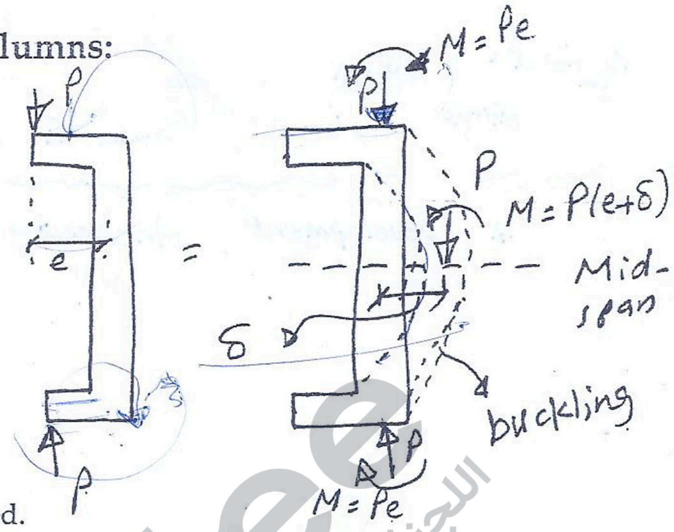


Pn
A
Nominal Interaction Diagram (Pn, Mn)
B
0.8 ØPn
Design Interaction Diagram (ØPn, ØMn)
C
D
E
Mn



Design of Short Columns:

-Slender or Long Columns and Short Columns



M @ends = Pe
M @ midheight = P (e + δ)

-The deflection increases the moment for which the column must be designed.

- Because of the increase in the maximum moment due to deflections, the axial-load capacity is reduced. This reduction in the axial-load capacity results from what are referred to as slenderness effects.

A slender column is defined as a column that has a significant reduction (>5%) in its axial-load capacity due to moments resulting from lateral deflections of the column.

Choice of materials properties and reinforcement ratios :

-In small buildings, f'c in columns ≅ f'c in floors ≅ 28 – 31 MPa.
-In tall buildings, f'c in columns > f'c in floors, to reduce the column size.
-Per ACI code section $0.01 \le \rho_{st} \le 0.08$

Although the code allows $\rho_{max} = 0.08$ it is generally very difficult to place this amount of steel in a column, particularly if lapped splices are used.

Example: 400mm X 400mm column, Ast = 0.08 X400X400 = 12,800 mm²

If No. 25M used ⟹ 25 bars are required.

It will be difficult to place 25 bars in a 400X400 mm column.

- Tables A-10 and A-11 give ρ_{max} for various column sizes for square and circular columns. (3-5 or 6%).
- Most economical tied- column sections ⟹ ρ_{st} (1-2)%.
  For Spiral columns ρ_{st} (2.5-5)%, because they resist higher axial loads.
- Per ACI code: Min. No. of bars in a tied column is 4. and Min. No. of bars in a spiral column is 6.
- Almost universally: an even No. of bars is used in a rectangular column maintain symmetry about the axis of bending. All bars are the same size.
-

Estimating the column size:

For very small values of moment, the column size is governed by the maximum axial load capacity

-For tied columns $$A_g(trial) \ge \frac{P_u}{0.4(f'_c + \rho f_y)}$$

-For spiral columns $$A_g(trial) \ge \frac{P_u}{0.5(f'_c + \rho f_y)}$$

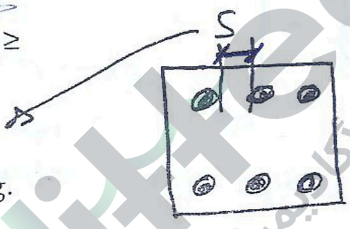
Both of these equations will tend to under estimate the column size if there are moments present, because they correspond roughly to the horizontal line portion of the $\varnothing P_n, \varnothing M_n$ interaction diagram.

Although the ACI – Code does not specify a minimum column size, the min. dimension of a cast –in- place tied column should not be less than 8 in (200mm) and preferably not less than 10 in (250mm). The diameter of spiral column should not be less than 12 in (300mm).

Bar spacing requirements and cover requirements:

- Per ACI code, the clear cover ≥ 1.5 in (40 mm)
- Min clear distance between longitudinal bars ≥

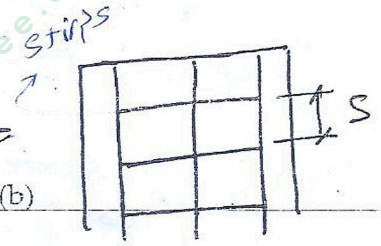
Larger of
- 1.5 db
- 1.5 in (40mm)
- 1⅓ Max. size of course Agg.



Clear distance limitation also apply to lap-spliced bars.

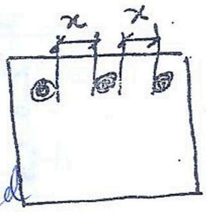
Spacing for Ties:

S_{max} shall not exceed (smaller of)
- 16 db
- 48 d_t (dia of tie)
- Least dimension of the colulmn (b)



A bar is adequately supported against lateral movements if it is located at a corner of a tie and if the dimension X is 6 in (150 mm) or less.

X ≤ 15 cm — adequitly supported



Choice of Column Type:

e/h = Eccentricity Ratio

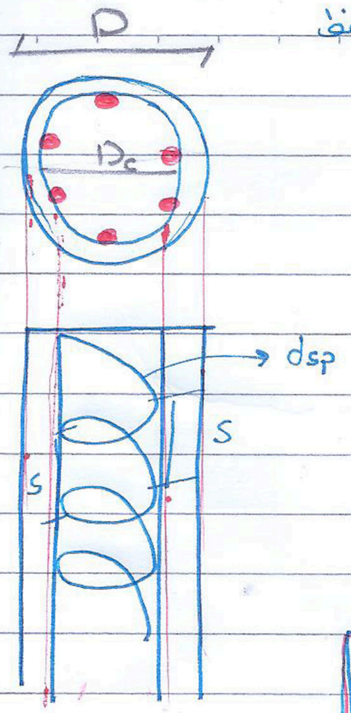
$\frac{M}{P} = e$

e/h < about 0.10 (small eccentricity, small moment)	Spiral column is more efficient; in terms of load capacity, Ø=0.70 (S) and 0.65 (T). In terms of maximum axial load capacity, 0.85ØPn (S) and 0.8ØPn (T)
e/h > 0.20 (large eccentricity, large moment)	A tied column with bars in the faces farthest from the axis of bending is most efficient. Even more efficiency can be obtained by using a rectangular column to increase the depth perpendicular to the axis of bending.
e/h< 0.20 *and M exists about both axes*	A Tied column with bars in four faces are used.

No. ______

Date ______

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



مش دغف

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

$$S_{max} = \frac{\pi\, d_{sp}^2\, f_y}{0.45\, D_c\, f'_c \left(\frac{A_g}{A_c} - 1\right)} \qquad A_c = \frac{\pi}{4} D_c^2$$

Dc : diameter of Core

لو عشري لتقريب للأقل

$S_{max} = 75\ mm$

S_{min} = langer of { 25 mm ; 1.33 Max Corse Agg }

we use it min مهم سبب استخدام الـ

to avoid Problems in Concrete placing

طلع أقل منه min
use min

طلع أكبر منه max
use max

بعد ما تطلع Sمن المعادلة فوق

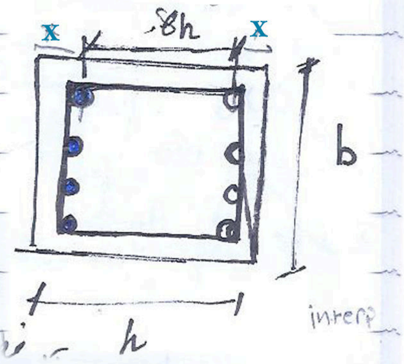
شييك على الماكس 75mm و الـmin

وإنتبه بالامتحان إن أعطاك الديامتر كامل اطرح منه cover*2 عشان يعطيك Dc

خطوات تصميم tide colomun

1)جد Ag ثم إما بعطيك h أو b وإما بقلك square عندها h=b والان نجيب الابعاد من Ag=bh

2)تستطيع التقريب للأكبر بالابعاد ...بحدد لك بالامتحان قطر الحديد والمطلوب الان جد gamma كالتالي



h= gamma *h +2 X

حيث X (X=cover + ds + db/2)

يا إما تطلع معك gamma عدد موجود بالجدول أو بينهم

بينهم لازم تستخدم الرسمتين نسوي interpolation

interp

3) استخدام الرسمات أول شي جد $\frac{\phi P_n}{f'_c bh}$ و $\frac{\phi M_n}{f'_c bh^2}$ مهم جدا تعرف إنه

$\Phi M_n = M_u$

$\Phi P_n = P_u$

بتوجد ρ أو 2 من ρ وبتعمل interpolation إذا اكثر من غاما

تابع الصفحة الجاي

4) تشييك على ρ لازم $0.01 \le \rho_{st} \le 0.08$ لكن برضه ما لازم تطلع عن 0.04 إذا أكثر استخدم 0.04

5) الان نجد As حيث As= ρ bh ثم جيب الـprovided من الجدول (اختر عدد قضبان حديد زوجي)

6) الان تحديد spacing of ties ولازم تحفظ الاتي

7) الان تحديد spacing of bars

احفظ برضه

Larger of {
1.5 db
1.5 in (40mm)
1⅓ Max. size of course Agg.

Spacing for Ties:

S_{max} shall not exceed (smaller of) maximum {
16 db
48 d_t → dia of tide
Least dimension of the colulmn (b

8) check Smin value of X أوجد X من قانون ال bmin حيث x نفسه Smin

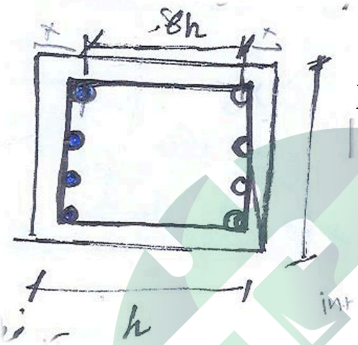
ويجب أن يكون Smin <= X <= 15 cm

Desing Tide colomun example Design the coloumn which Pu=1550Kn square colomun

Mu=150Kn f'c=20Mpa fy=420Mpa

Ag = Pu/0.4(f'c + ρfy) Ag=1550*1000/0.4(20+0.015 *420) Ag=147338mm2

b=h b*b=h*h=147338 b=h 384 mm تقريب b=h=400mm



X = 40+10+25/2=62.5mm

h=gamma*h+2x gamma=0.69

القيمة بين الرسمتين لذلك بدنا تستخدم الرسمتين ونسوي انتربوليش

$\frac{\phi P_n}{f'_c bh} = \frac{1550 \times 10^3}{20 \times 400 \times 400} = 0.48$ $\frac{\phi M_n}{f'_c bh^2} = \frac{150 \times 10^6}{20 \times 400 \times 400^2} = 0.12$

gamma = 0.6 ρ=0.033

gamma=0.65 ρ=?? ρ=0.03

gamma=0.75 ρ=0.28

ρ: 0.3 < ρ_{max} O.k | $\rho_{min} = 0.01$, ρ= 0.03 > ρ_{min} o.k

لازم تختار عدد قضبان الحديد زوجي حتى يكون سمترك

As = ρbh = 4800mm use 10No25M As=5100mm

حسب القانون فوق

now we get Spacing of ties (400mm or 488mm or 400mm) take 400mm

now find Spacing of bars (Smin) then check X Smin=40mm

X *4 +2*(2ds-0.5db) + 25*5 + 2*10 + 40*2 =400

Smin <=X<=150mm

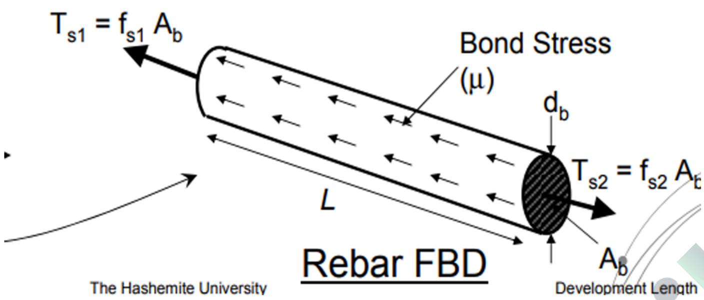
then ok

مهم جدا أوع تضربلي ب10 بارز لأن نحن هون بنستخدم قضبان الي بمنطقة التنشن فقط والي هي نص القضبان

Development length

Development length : is the shortest length in the bar in which bar stress can increase from zero to fy

if the distance from a point where the bar stress equal fy to the end of the bar is less than (Ld) then the bar will pullout of the concrete because there is no enough bond stress to produce equilibrium

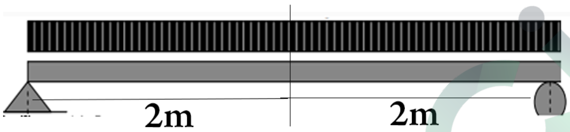


$\therefore \mu_{avg}$ = avg bond stress

force = π db μavg * L

T1 + μavg * πdb * Ld = T2

ينتقل ال bond stress باتجاه T الأقل



لازم أقارن Ld مع L حيث
L من نقطة الماكس المومنت لنهاية الحديد حيث
السترس صفر

هون مثلا : لو طلعت Ld =1.5 فهو ok أما لو طلعت Ld =2.5m إذا كان contious بمد الحديد
وإن لم يكن نستخدم الـhooks

Dr. Hazim Dwairi

Reinforced Concrete Design I

Development Length of Straight Bars and Standard Hooks

For deformed bars, ACI318-05 Section 12.2.2 defines the development length l_d given in the table below. Note that l_d shall not be less than 300 mm.

Case	$\le \phi 20$	$> \phi 20$
Case 1: Clear spacing of bars being developed not less than d_b, clear cover not less than d_b, and stirrups throughout l_d not less than code minimum حدد الحالة بالأول بعدها اختر القانون المناسب or **Case 2:** Clear spacing of bars being developed not less than $2d_b$ and clear cover not less than d_b	$l_d = \dfrac{f_y \alpha \beta \lambda}{2.1\sqrt{f_c'}} d_b$	$l_d = \dfrac{f_y \alpha \beta \lambda}{1.7\sqrt{f_c'}} d_b$
Other cases	$l_d = \dfrac{f_y \alpha \beta \lambda}{1.4\sqrt{f_c'}} d_b$	$l_d = \dfrac{f_y \alpha \beta \lambda}{1.1\sqrt{f_c'}} d_b$

The terms in the foregoing equations are as follows:

α = **reinforcement location factor**

Horizontal reinforcement so placed that more than 300 mm of fresh concrete is cast in the member *below* the development length 1.3

Other reinforcement 1.0

β = **coating factor**

Epoxy-coated bars with cover less than $3d_b$, or clear spacing less than $6d_b$ 1.5

All other epoxy-coated bars 1.2

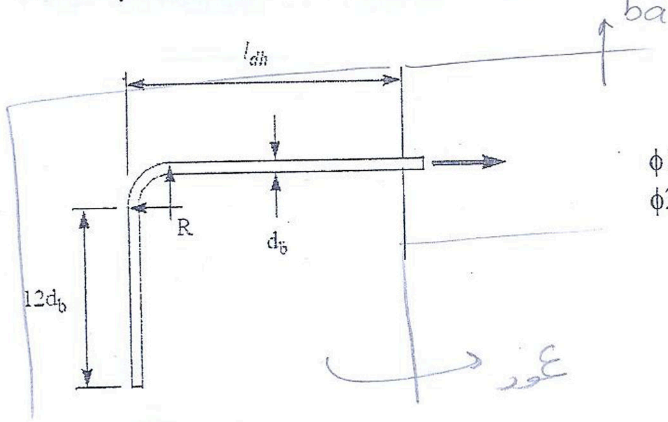
Uncoated reinforcement 1.0

λ = **lightweight aggregate concrete factor**

When lightweight concrete is used 1.3

Normal weight concrete is used 1.0

When there is insufficient length available to develop a straight bar, standard hooks are used. The standard 90 degree hook is shown below:



$\phi10$ to $\phi25$: $R = 3d_b$
$\phi28$ to $\phi32$: $R = 4d_b$
$\phi50$: $R = 5d_b$

The development length of a hook, *ldh*, is given by the following equation. Note that the development length shall not be less than 8db nor less than 150mm:

$$l_{dh} = \frac{0.24 f_y \beta \lambda}{\sqrt{f_c'}} d_b \geq \text{larger of} \begin{cases} 8d_b \\ 150mm \end{cases}$$

where β = the coating factor = 1.2 for epoxy coated bars and 1.0 for uncoated reinforcement, and λ is the lightweight aggregate factor = 1.3 for lightweight aggregate concrete. For other cases β and λ, shall be taken as 1.0

Standard Hooks ¬ ACI sections 7.1 and 7.2.1

$\phi10$ to $\phi25$: $R = 3d_b$
$\phi28$ and $\phi32$: $R = 4d_b$
$\phi50$: $R = 5d_b$

Bar Cutoffs:

Bars can be cutoff where they are no longer needed to resist tensile forces or where the remaining bars are adequate to do so. The following rules apply per the ACI code:

ممنوع يوقف الحديد قبلها

1. Every bar must continue a distance (**d or 12d_b**) beyond the theoretical cutoff point. (B-C) & (H-I).

2. Full development length l_d must be provided beyond the critical sections (A-C) & (B-F). Critical sections include points of maximum positive and negative moments and points where reinforcing bars adjacent to the bar under consideration are cutoff or bent.

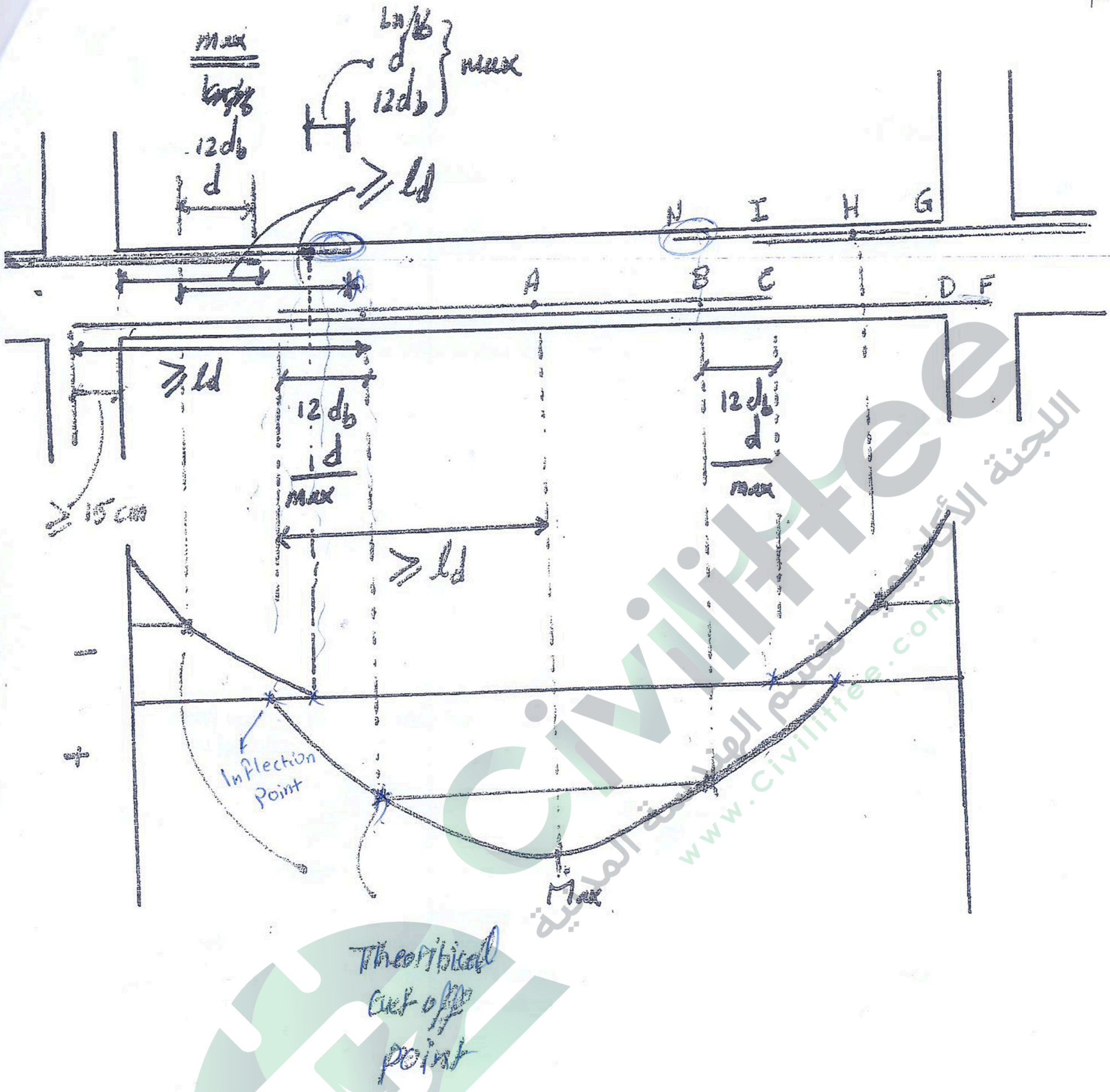
3. At least $As^+/3$ (one – third of the positive moment reinforcement) but not less than two bars in simple spans or $As^+/4$ in continuous spans must be continued at least 15cm into the supports. (D-F).

4. At least $As^-/3$ must be extended beyond the point of zero moment a distance not less than **d or 12d_b** or ***ln/16***.

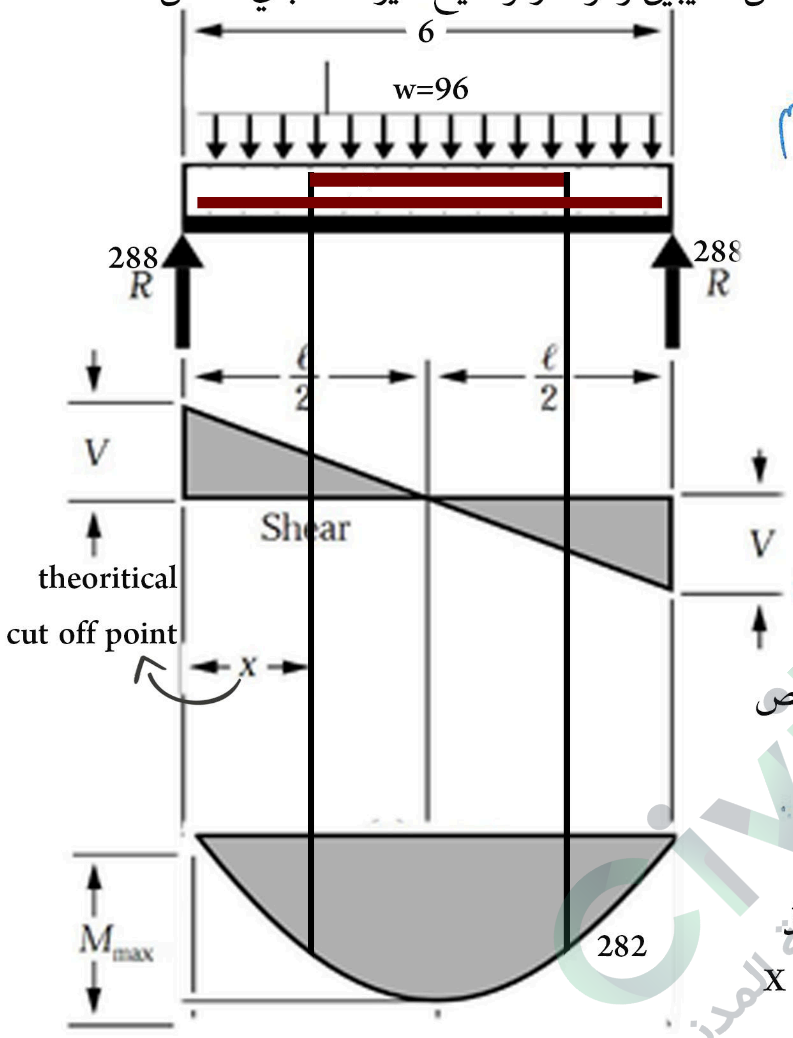
5. Cutoff 50% of the positive moment steel and extend the other 50% into the support.

6. At least $As^+/4$ at mid span, not less than two bars, shall be spliced at or near the mid span.

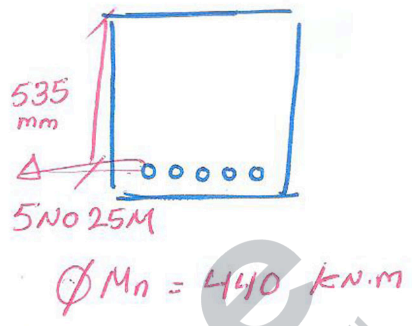
7. At least $As^-/6$ shall be spliced at or near mid span.


max
Ln/16
d
12db
max
12db
d
≥ ld
N
I
H
G
A
B
C
D
F
≥ ld
≥ 15 cm
12db
d
max
12db
d
max
≥ ld
−
+
Inflection Point
Max
Theoritical cut-off point

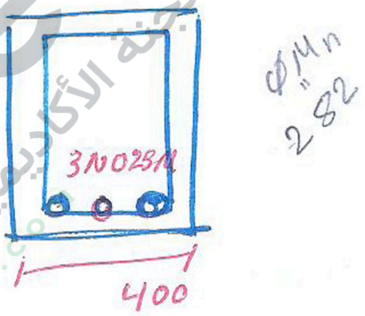
بيجي السؤال مثلا جد ال theoritical cut off point بعد قص قضيبين وهون ركز منيح كثير لان الباقي 3 مش تحط 2



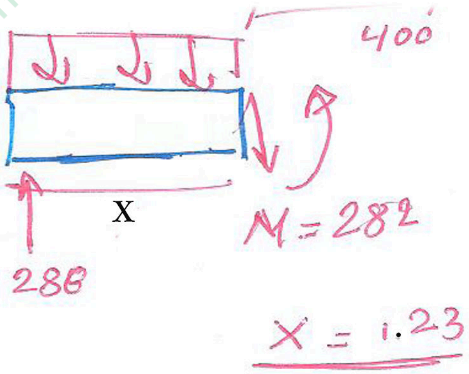
الحديد الكلي 5 قضبان



نحسب ØMn للقضبان بعد القص



نحسب X بأخذ سكشن بطول X



2 No 25 L = 1.75 m

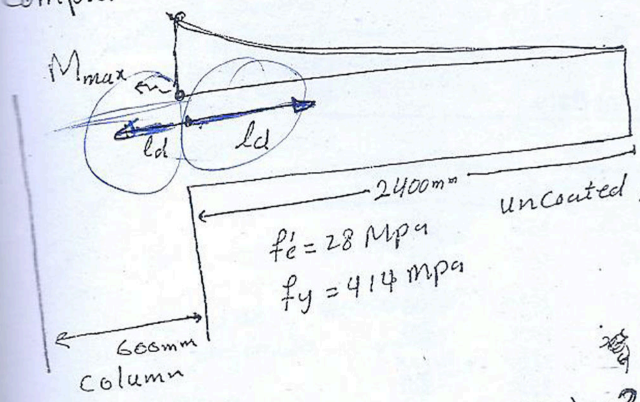
3 No 25 M. L = 3 m

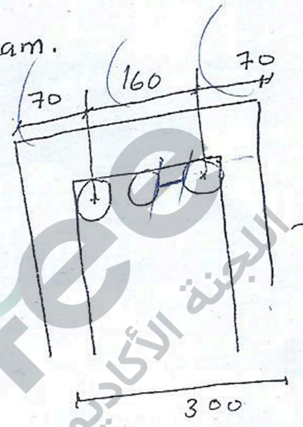
Ld both are ok

مثال مهم من دوسية الطالب لطف الصباري

Example

Comput l_d for the 3 Φ 25 in the cantilever beam.





sol:-

clear spacing bet. bars = $\frac{300-2(70)-2(25)}{2} = 55\ mm$ — Case 2 — table

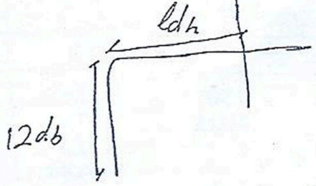
$d_b = 25\ mm$; $2d_b = 50\ mm$ → $55 > 2d_b$

$l_d = \frac{f_y \alpha \beta \lambda}{1.7\sqrt{f'_c}} \cdot d_b = \frac{414 \times 1.3 \times 1 \times 1}{1.7\sqrt{28}} \times 25 = 1495.74\ mm < (2400-40)mm$ → ok

طول البيم من جهة اليمين المتاحة.

but $l_d = 1495.74 > 600\ mm$ → المسافه المتاحه داخل العمود هي على يسار M_{max}

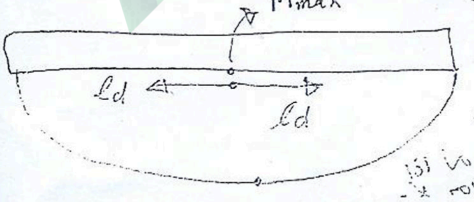
So, provide 90° hook



check $l_{dh} = \frac{0.24 \times 414 \times 1 \times 1}{\sqrt{28}} \times 25 = 469mm < 600mm$

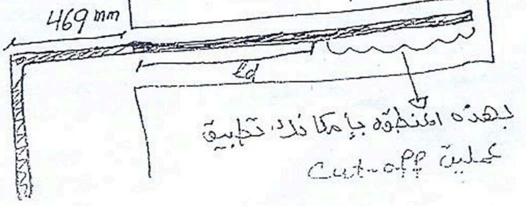
OK

لو كان S.S



نوجد ld وننقلها مع دفنها طول البيم للتأكد ما اذا كنا نحتاج for hook T.

التسليح النهائي للبيم



هنا تنتهي مادة RC one

سأقوم بإرفاق ملف ملحق بأسئلة السنوات الفاينل محلولة قبيل الامتحان بإذن الله

وفي حال لاحظت أي خطأ أو عندك استفسار بالدوسية أو لطلب الملحق الكترونياً

تواصل معي على الفيس:

https://www.facebook.com/ezrawille

بالتوفيق للجميع أتمنى أني لم أقصر في هذا الملخص
سائلاً منكم الدعاء بالتوفيق ...
ودعائي لكم بالموفقية والنجاح الدائم

عز الدين عاشور

RCL
بلال ابو لغول
٣٥

TABLE A-4 Total Area (in.^{2}) of Multiple U.S. Reinforcement Bars

	Number of Bars					
Bar No.	1	2	3	4	5	6
3	0.11	0.22	0.33	0.44	0.55	0.66
4	0.20	0.40	0.60	0.80	1.00	1.20
5	0.31	0.62	0.93	1.24	1.55	1.86
6	0.44	0.88	1.32	1.76	2.20	2.64
7	0.60	1.20	1.80	2.40	3.00	3.60
8	0.79	1.58	2.37	3.16	3.95	4.74
9	1.00	2.00	3.00	4.00	5.00	6.00
10	1.27	2.54	3.81	5.08	6.35	7.62
11	1.56	3.12	4.68	6.24	7.80	9.36

TABLE A-4M Total Area (mm^2) of Multiple SI Reinforcement Bars

	Number of Bars					
Bar No.	1	2	3	4	5	6
10	71	142	213	284	355	426
13	129	258	387	516	645	774
16	199	398	597	796	995	1190
19	284	568	852	1140	1420	1700
22	387	774	1160	1550	1930	2320
25	510	1020	1530	2040	2550	3060
29	645	1290	1930	2580	3220	3870
32	819	1640	2460	3280	4090	4910
36	1010	2010	3020	4020	5030	6040

TABLE A-5 Minimum Beam Width (in.) for Multiple U.S. Bars per Layer; Interior Exposure[a]

	Diameter	Number of bars in single layer				
Bar No.	(in.)	2	3	4	5	6
4	0.50	7.0	8.5	10.0	11.5	13.0
5	0.625	7.0	8.5	10.5	12.0	13.5
6	0.75	7.0	9.0	11.0	12.5	14.0
7	0.875	7.5	9.0	11.0	13.0	15.0
8	1.00	7.5	9.5	11.5	13.5	15.5
9	1.128	8.0	10.0	12.5	14.5	17.0
10	1.27	8.0	10.5	13.0	15.5	18.0
11	1.41	8.5	11.0	14.0	17.0	19.5

[a]Clear cover of 1.5 in.; No. 3 double-leg stirrup; ¾ in. maximum-size aggregate.

7.7 — Concrete protection for reinforcement

7.7.1 — Cast-in-place concrete (nonprestressed)

The following minimum concrete cover shall be provided for reinforcement, but shall not be less than required by 7.7.5 and 7.7.7:

	Minimum cover, mm
(a) Concrete cast against and permanently exposed to earth	75
(b) Concrete exposed to earth or weather:	
No. 19 through No. 57 bars	50
No. 16 bar, MW200 or MD200 wire, and smaller	40
(c) Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists:	
No. 43 and No. 57 bars	40
No. 36 bar and smaller	20
Beams, columns:	
Primary reinforcement, ties, stirrups, spirals	40
Shells, folded plate members:	
No. 19 bar and larger	20
No. 16 bar, MW200 or MD200 wire, and smaller	15

9.5 — Control of deflections

9.5.1 — Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that adversely affect strength or serviceability of a structure.

9.5.2 — One-way construction (nonprestressed)

9.5.2.1 — Minimum thickness stipulated in Table 9.5(a) shall apply for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness can be used without adverse effects.

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

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

Notes:
1) Span length ℓ is in mm.
2) Values given shall be used directly for members with normalweight concrete (w_c = 2300 kg/m^3) and Grade 60 reinforcement. For other conditions, the values shall be modified as follows:
a) For structural lightweight concrete having unit weight in the range 1500-2000 kg/m^3, the values shall be multiplied by $(1.65 - 0.0003 w_c)$ but not less than 1.09, where w_c is the unit weight in kg/m^3.
b) For f_y other than 420 MPa, the values shall be multiplied by $(0.4 + f_y/700)$.

TABLE 9.5(b) — MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\ell/180$*
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\ell/360$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\ell/480$‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\ell/240$§

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

† Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§ Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.