Civilittee
اللجنة الأكاديمية لقسم الهندسة المدنية
www.Civilittee-HU.com

دفتر

خرسانة مسلحة1

د. بلال ابو الفول

إعداد : هاني الجليلاتي

2/2/2016

- compression of concrete = 0.1 tension of concrete

Advantages of concrete Material: (1.4)

- Relatively low cost material
- fire resistance (1-3 hrs) fire rating without special fire proofing
- suitability of material for architectural & structural functions.
  ملاءمة المواد من أجل وظيفة المعمارية والميكانيكية
- Rigidity الصلابة
- low maintenance صيانة منخفضة
- Availability of material

Disadvantages:

- low tensile strength
- forms & shoring طوبار أو حركات (إنشاء طاقة)
- (??) Relatively low strength per unit weight or volume $(C_c = 10\, C_s)$ $(\rho_c = 30\, \rho_s)$
- Time dependent volume changes: Drying shrinkage
  Creep تزحّف

Importance of steel :-

- have nearly coefficient of thermal expansion ما يقرب من معامل التمدد الحراري
- has good bond with the concrete
- good dense concrete protects steel from rusting
  الخرسانة الكثيفة جيدة تحمي الحديد من الصدأ

يمنع انفصال الحديد عن الخرسانة

أحد أسباب عدم وصول الحديد لأنه أملس ويؤثر على هذه العملية (8)

مصادر عدم اليقين

Sources of uncretainty : (2-4)

- Actual load magnitude & direction may differ from those assumed in the design.

(يتم استخدام الأحمال codes الأردنية)

- Assumptions & simplifications in the analysis may result in different internal forces.
- Actual behavior may be different
- Actual member dimensions may differ from those specified in the design
- Reinforcement may not be in its proper position. (الموقع الصحيح)
- Actual material strength may be different from ~~those~~ that specified in the designer

Safety philosophy : (2-7)

$\phi S_n > \gamma Q_d$

S_n → Nominal strength; Q_d → design load

ϕ: Strength reduction factor < 1.0

γ: over load factor > 1.0 ??

(Load factor and Load combination from ACI-code and we found U).

load factor > load combinations per ACI-code.

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

- U: Ultimate load
- $U = \gamma Q_d$ → Factord
- DL: Principal varible load
- : Dead load
- LL: Life load

$U = 0.9\,DL + 1.6\,WL + 1.6\,HL$

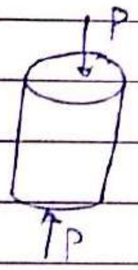
DL: companion action variable load
WL: wind load

CH.3: Materials

9/2/2015

(3-2)

There are four major stages in the Development of microcracking & failure in concrete subjected to uniaxial compressive loading:

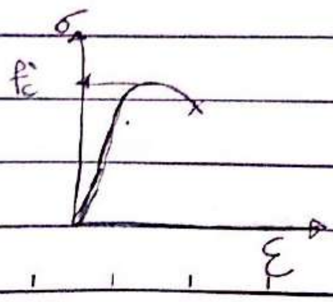


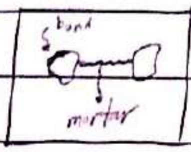
$\sigma_1 = \frac{P}{A}$

① Shrinkage of the paste during by hydration
→ No-load bond cracks.

② Stresses > 30 to 40 % of the compressive strength
→ bond cracks (between agg. and mortar)

③ → Discontinuity Limit
Strength > 50 to 60 % of the compressive strength
→ mortar cracks (between agg. and particle)





Hex's law in elastic region

stable Crack propagation

Critical stress

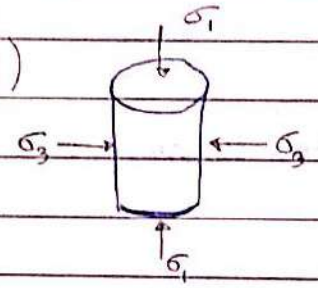
(4) at 75 to 80% of the ultimate load

→ Number of mortar cracks increasen

→ fewer undamaged partions to carry the load

unstable Crack propagation (الشرخ السريع)

triaxial loading: (3-4)

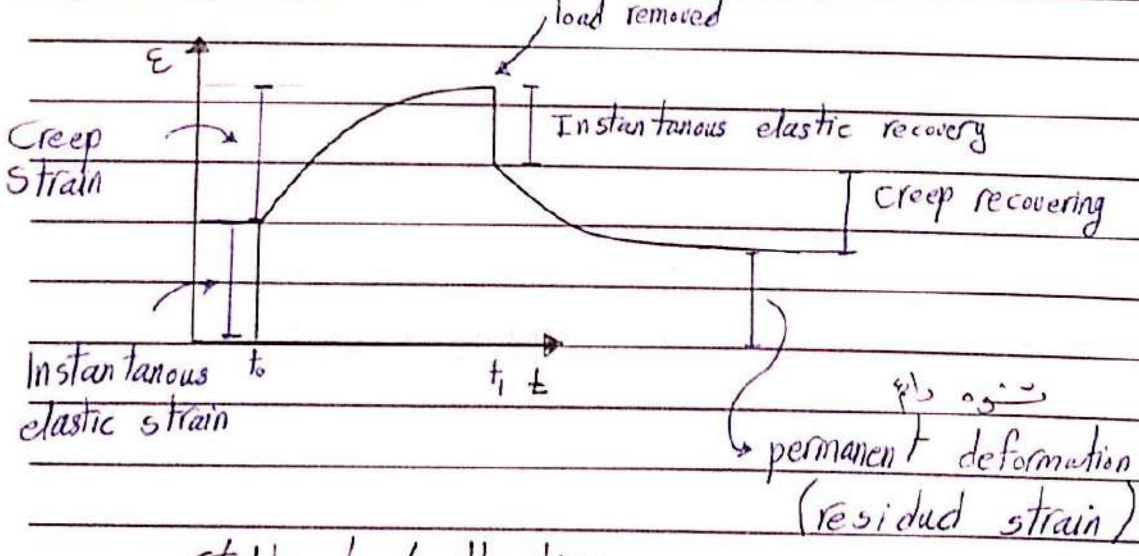


(confing pressure)

(confing pressure) زاد كلما

compressive زاد

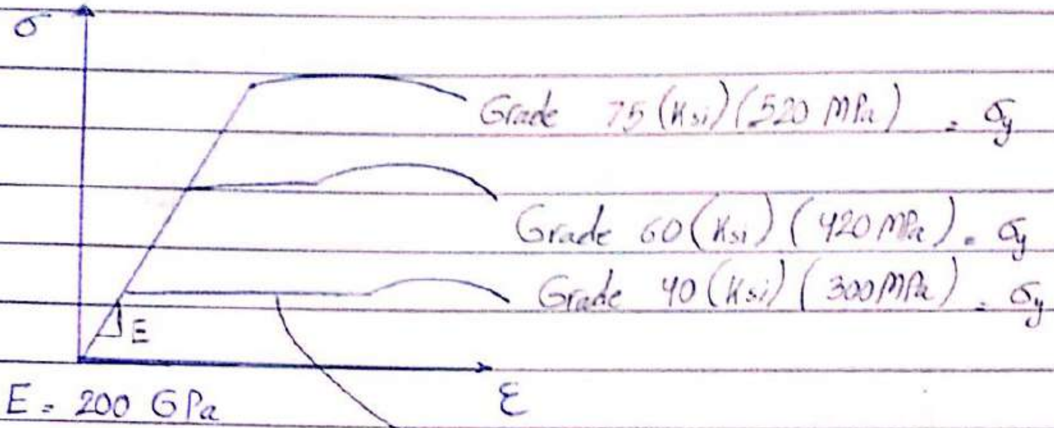
Creep test: (3-6)



→ stable load with time

يتم التوقف عن القراءة عندما يرجع (curve) ثابت.

Reinforcement: (3-14)



مفضل الحديد ومطاوعته في مناطق الزلازل

E = 200 GPa

الفرق مهم في سلوك كل دياجرام
حيث يعطي (Grade 40) فترة نعطي
ملاحظات قبل الانهيار اذا وصل الـ Load
(yield stress) مرحلة

G75 : it's Brittle material
same as Less Ductility

Cha. 2

7/2/2015

The Design Process:

* Objective of the design:- (2-1)
Structure should satisfy :-

1. ملائمة Appropriatness: designed to serve its intended use.
2. Economy : Over all cost < client's budget
3. Structural Adequacy :
   كفاية الإنشائية
   Structure must
   - be strong enough to support anticipated loads كان متوقع
   - Not deflect, tilt, vibrate
     ميلان
4. Maintainability : Minimum & simple
   الحفاظ الصيانة.

* The Design Process: (2-2)

phase I : Defining Clients needs and priorities

phase II : development of project concept تطوير مفهوم المشروع

→ No. of possible layouts

→ Preliminary cost estimation تقدير التكاليف الأولية

phase III : Design of Individual systems.
تصميم نظم فرعية

* Limit States of the Design of R.C : (2-3)

When a structure or an element becomes unfit for its intended use, it is said to have reached a Limit State. (عندما يصبح هيكل أو عنصر هيكلي غير صالح للاستخدام المقصود منها و يقال أنها وصلت الى حالة الـ)

– Groups of Limit states ;

(1) Ultimate Limit State : involves structural collapse of part or all the structure.

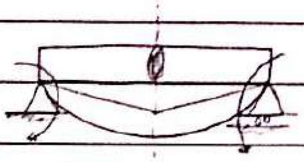
(a) loss of equilibrium

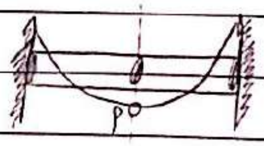
(b) Rupture تمزق

(c) Progressive collape انهيار تدريجي

(d) formation of a plastic mechanism تشكيل آلية بلاستيكية

(2) Plastic Mechansim → yielding of reinforcement → Plastic Hingeo





$\sigma = \frac{My}{I}$

$\Downarrow$

σ_y

عدم الاستقرار (e) Instability → buckling (التواء)

(f) fatigue تعب

② Serviceability Limit State : مرحلة حد الخدمة
Involves disruption of the functional use of the structure
(but Not collapse)

- excessive defliction انحراف مفرط
- excessive crack widths عرض الشق المفرط
- Undesirable vibrations اهتزاز غير مرغوب فيه

③ Special limit State (انهيار مفاجئ بشكل غير اعتيادي)
Involves damage or failure due to abnormal conditions.
- damage or collapse in extreme earthquakes . الضرر أو انهيار في الزلازل الشديدة
- structural effects of fire, explosions, or vehicular collision . الآثار الهيكلية لإطلاق نار والانفجارات أو اصطدام المركبات
- structural effects of corrosion or deterioration . الآثار الهيكلية للتآكل والتدهور
- long term physical or chemical instability . عدم الاستقرار الفيزيائية أو الكيميائية على المدى الطويل (عادة ليست مشكلة مع الهياكل الخرسانية)
(normally not a problem with concrete structures)

9/2/2016

Structural saftey: (2-4)

- Sources of uncertainty

- Consequences of failure عواقب الفشل
  - → Loss of Life
  - → cost of clearing debris
  - → cost to society in time lost
  - → the type of failure

- Design Procedures specified in the ACI-code (2-6)

(1) Strength Design :-

$$\phi S_n \geq \gamma \phi d$$

(2) Working-Stress-Design
working loads
(service loads)

$$\phi S_n \geq \phi d$$

(3) Plastic Design, Limit Design, capacity Design

- يتم تصميم البناء على أساس لو حمل (Actual load) على (member)
يتم توزيع هذا الحمل على باقي (members) المجاورة.

- Loading & Actions: (2-8)

Permanent	Accidental	Variable
المنشأ نفسه	نيران	أثاث
Dead load	براكين	بشر
من نفسه	زلازل	

CH 4

flexure : Basic concepts (Rectangular beams) (4-1)

Beams $\longrightarrow$ flexure + shear

flexure: Reduced nominal strength $\geq$ factored load effects

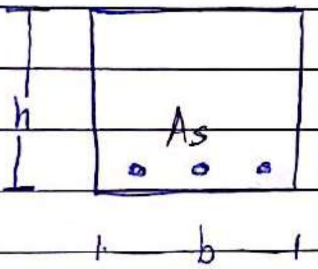
$$\phi M_n \geq M_u \longrightarrow \text{Basic saftey equ. for flexure}$$

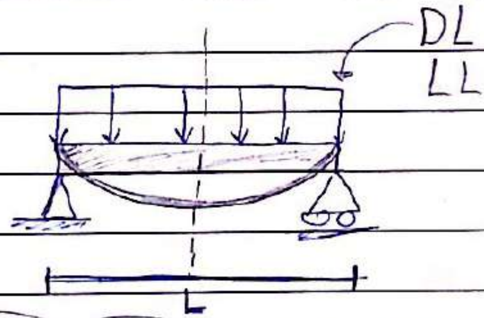
ϕ : Strength reduction factor

M_n : Nominal capacity (مقاومة M الفعلية)

M_u : Ultimate required moment (Factored loads)

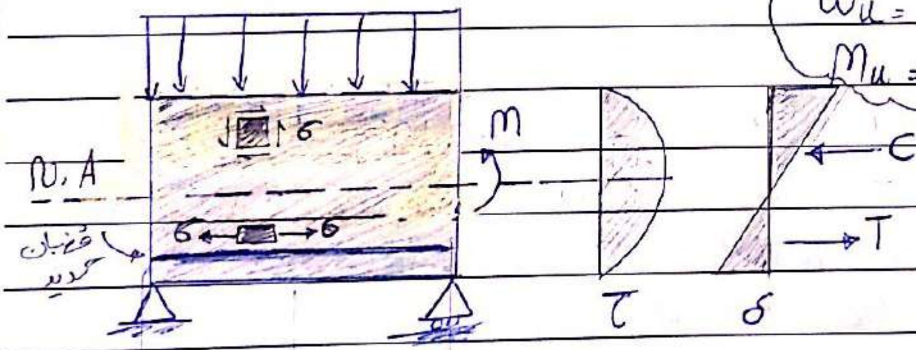
ϕM_n : Design moment factored moment resistance



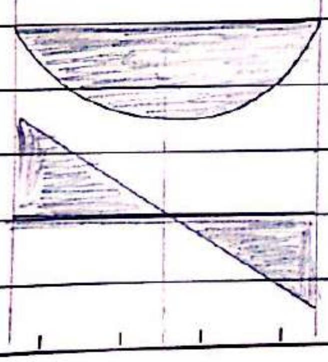


$$W_u = 1.2\,DL + 1.6\,LL$$

$$M_u = M_{max} = \frac{W_u L^2}{8}$$

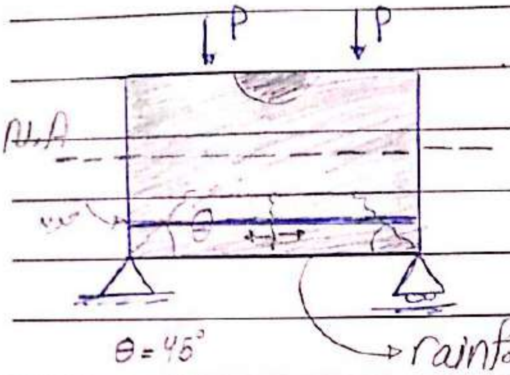


$$M \rightarrow \sigma = \frac{My}{I}$$



$$V \Rightarrow \tau = \frac{VQ}{Ib}$$

Flexural behavior (Lab testing) (4-2)



• الذي يحدد وقت حدوث
الكسر عندما C يختفي
يصبح $0 \neq \Sigma F_x$

Stage A: Before cracking

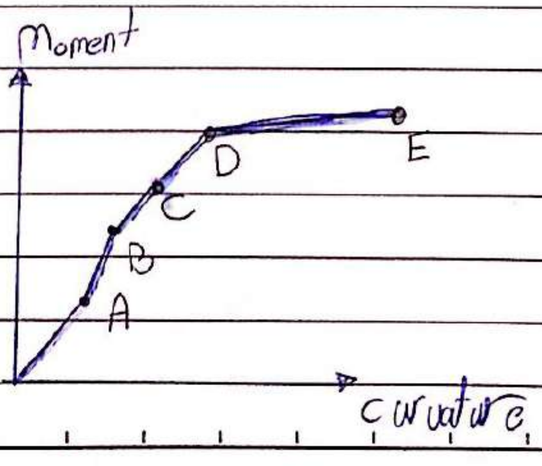
Stage B: Cracking

Stage C: After cracking,
before yielding of reinforcement

Stage D: yielding of reinforcement curvature increases
بسرعة rapidly with very little increasein the
moment.

Stage E: failure

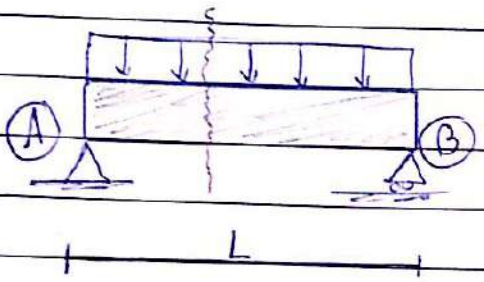
• Beam failed as a result of the crushing of the concrete at the top of the beam



11/2/2016

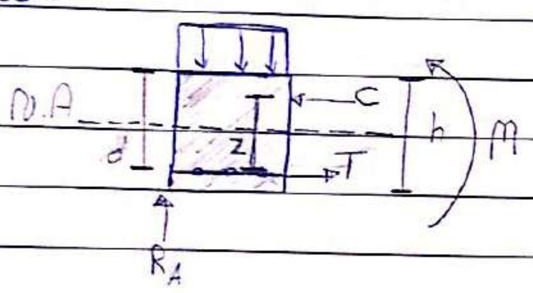
flexure theory:

?? $\phi M_n > M_u$



دائماً (Tensile force) على موضع الحديد .

sec:



M: internal resisting Moment

$- - - - \Rightarrow \Sigma M = 0.0$

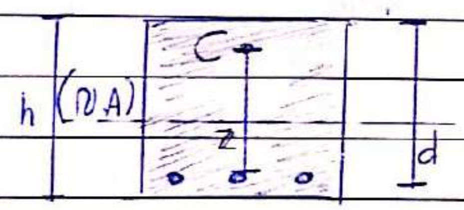
C, T : internal resisting forces

d : effective flexural depth

Z : moment arm (jd)

always ($j < 1.0$)

side view:



$C = T$

$M_n = C\,jd$

$= T\,jd$

. دائماً يتم التعامل مع (couple forces) ق
Comp. الخرسانة
Tensile الحديد

Basic assumptions in flexure theorey:

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

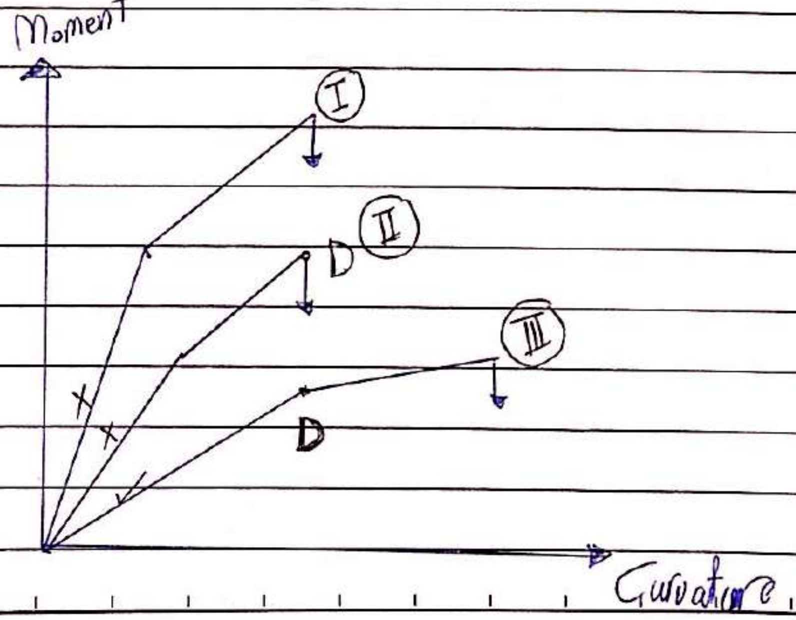
→ يجب أن يبقى الخط المستقيم كما هو

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

③ The tensile strength of concrete in neglected in flexural strength calculation.

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

flexural failure May occur in 3 different ways:



I: • overrainforced beam → compression failure

• sudden collapse

• ($\varepsilon_c = 0.003$, $\varepsilon_s < \varepsilon_y$)

• كأنه وضعنا حديد قوي أو أكثر من اللازم بالنسبة لـ (Concrete)

• نوع failure محدد من (Tensile strain)

II: balanced failure

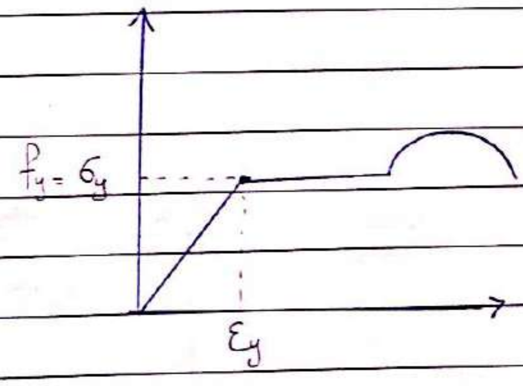
balanced beam

($\varepsilon_c = 0.003$, $\varepsilon_s = \varepsilon_y$)

III: ♡ يفضله المهندسون

Tension failure ($\varepsilon_c = 0.003$, $\varepsilon_s > \varepsilon_y$)

under rainforced beam



$f_y = 420$ MPa

$f_y = E\,\varepsilon_y$

$\varepsilon_y = \dfrac{420}{200\,000} = 0.0021$

when analysis ① → $\phi M_n \Rightarrow \phi = 0.65$

③ → $\phi M_n \Rightarrow \phi = 0.9$

14/2/2016

Strain Limits Method for Analysis & Design:

In ACI-code four types of beams depending on the anticipated mode of failure.

Compression controlled beam	Tranzition controlled beam	Tension controlled beam
compression controlled failure	Tranzition controlled failure	Tension controlled failure

$\rightarrow \varepsilon_y$

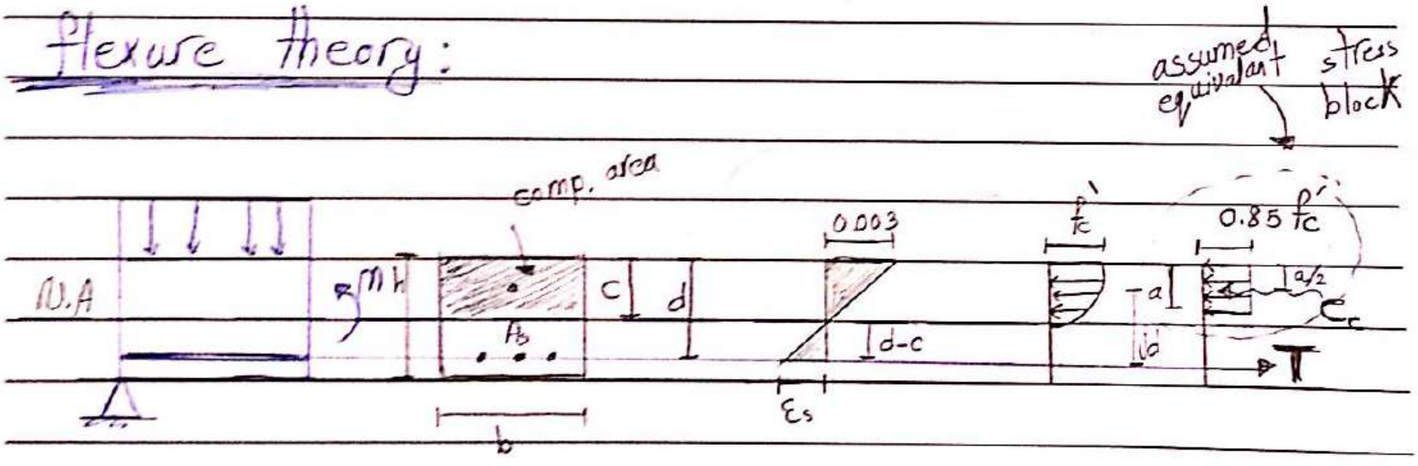
$\varepsilon_s = \varepsilon_y$

balanced beam
balanced failure

$\varepsilon_s = 0.005$

$\varepsilon_c = 0.003$

flexure theory:

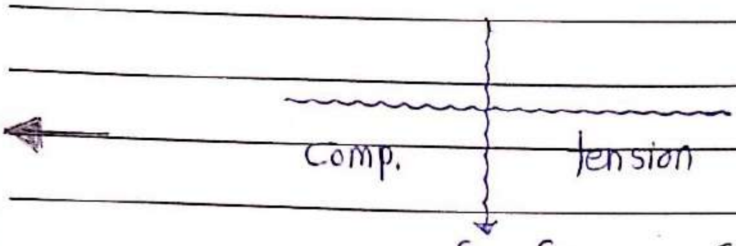


$$\frac{0.003}{c} = \frac{\varepsilon_s}{d-c}$$

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

$$a = \beta_1 c$$

$$\sigma = P/A$$

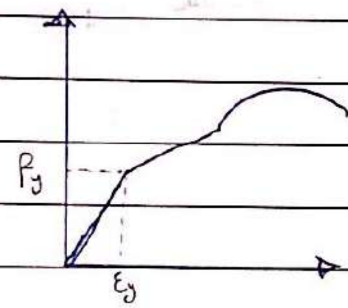


$\varepsilon_s = 0.0022$

balanced 0.0019

$\varepsilon_y = 0.0021$

- $C_c = 0.85\ f'_c\ ab$

- 

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

$\varepsilon_s < \varepsilon_y \Longrightarrow f_s = E\,\varepsilon_s$

- Assuming $\varepsilon_s \geq \varepsilon_y$ $(f_s = f_y)$

$T = A_s f_y \longrightarrow C_c = T$

$0.8\ f'_c\ ab = A_s f_y$

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

- $M_n = T(jd)$
  $= C(jd)$

- $jd = d - (a/2)$ → only for rectangular beams

$M_n = A_s f_y (d - a/2) = 0.85\ f'_c\ ab\,(d - a/2)$

16/2/2016

$\beta'_1 \leftarrow f'_c$ تعتمد

- $\beta_1 = 0.85 \rightarrow (f'_c \leq 28\,MPa)$

- $\beta_1 = \left(0.85 - 0.05\,\frac{f'_c - 28}{7}\right)$

$28\,MPa \leq f'_c \leq 56\,MPa$

- $f'_c > 56\,MPa$

$\beta_1 = 0.65$

Strength reduction factor ϕ :-

→ $\varepsilon_s \geq 0.005$ (Tension)

$\phi = 0.9$

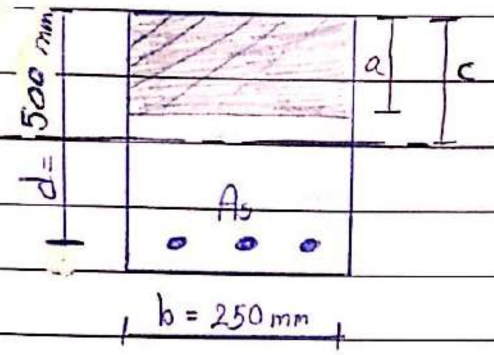
→ $\varepsilon_y < \varepsilon_s < 0.005$ (Tranzition)

$\phi = 0.65 + (\varepsilon_s - 0.002)(250/3)$

→ $\varepsilon_s \leq \varepsilon_y$ (compression balanced)

$\phi = 0.65 \rightarrow$ (تم الاعتبار إن 35% من المادة غير موجودة)

Analysis Example : (4-1M)



$A_s = 1530\ mm^2$

$f'_c = 20\ MPa$

$f_y = 420\ MPa$

Design Moment capacity : $\emptyset M_n$

(1) compute a (assuming the tension steel is yeilding)

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

$T = C_c$

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

$1530\ mm^2 * 420\ MPa = 0.85 * 20\ MPa * a * 250 mm$

$a = 151.2\ mm$

• $a = \beta_1 c$ $(\beta_1 = 0.85) \rightarrow f'_c < 20\ MPa$

$c = 151.2/0.85 = 177.9 mm$

(2) Check assumption (Check wether the tension steel is yielding)

$$\varepsilon_s = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{500-177.9}{177.9}\right)$$

$\varepsilon_s = 0.0054$

$\varepsilon_y = \frac{420}{200000} = 0.0021$

$\varepsilon_s > \varepsilon_y \Rightarrow$ Assumption $(f_s = f_y)$ ✓ ok.

③ Compute the nominal moment strength (M_n)

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

$$= 1530\,(mm^2) * 420\,(N/mm^2) * (500 - (151.2/2))\,(mm)$$

$$M_n = 273 * 10^6 \; N.mm$$

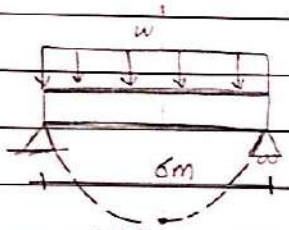
$$M_n = 273 \; KN.m$$

$\varepsilon_s = 0.0055 > 0.005 \longrightarrow$ tension $\phi = 0.9$

$\phi M_n = 0.9 * 273$

$= 245.7 \; KN.m$

المقطع بيتحمل (273 KN.m) قبل ان ينكسر.
ولكن يتم التصميم على (245 KN.m)



$$\phi M_n = \frac{wL^2}{8}$$

$\Longrightarrow$ Check ($A_{s\,min}$) ④ Confrim that the tension steel area exceeds

$$A_{s\,min} = \begin{cases} \dfrac{0.25\sqrt{f'_c}}{f_y}\, b\, d = \dfrac{0.25\sqrt{20}}{420} * 250 * 500 = 332.7 \; mm^2 \\[2ex] \dfrac{1.4}{f_y}\, b\, d = \dfrac{1.4}{420} * 250 * 500 = 416.67 \; mm^2 \end{cases}$$

$A_{s\,min} \simeq 417 \; mm^2$

$A_s = 1530 \; mm^2 > A_{s\,min}$

$\longrightarrow$ ok

لماذا يتم التأكد من A_s ؟؟

18/2/2016

Analysis Example:

$A_s = 3060\ mm^2$ $f'_c = 20\ MPa$ $f_y = 420\ MPa$

compute a $d = h = 500 mm$ $b = 250 mm$

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

$A_s f_y = 0.85 * f'_c * a * b$

$3060 * 420 = 0.85 * 20 * a * 250$

$a = 302.4\ mm \longrightarrow \times$

$C = a / \beta_1 = 355.8\ mm \longrightarrow \times$

Check Assumption:-

$\varepsilon_s = 0.003 \left(\frac{500 - 355.8}{355.8} \right) \longrightarrow \times$

$\varepsilon_s = 0.00121$

$\varepsilon_s < \varepsilon_y = 0.0021$

$f_s \neq f_y$

$f_s = E\,\varepsilon_s = E * 0.003 \left(\frac{d-c}{c} \right)$

$f_c = E\,\varepsilon$

$C = f_c A$

$A_s\ E * 0.003 \left(\frac{d-c}{c} \right) = 0.85\ f'_c\ \underbrace{a}_{\beta_1 C}\ b$

$(0.85\ f'_c\ b\ \beta_1) C^2 + (A_s E * 0.003) C - A_s E * 0.003\ d = 0.0$

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

$C = 312.66\ mm$

$a = \beta_1 C = 265.76\ mm$

$\varepsilon_s = 0.003 \left(\frac{500 - 312.66}{312.66} \right) = 0.00180$

$\varepsilon_s < \varepsilon_y \longrightarrow OK$

$f_s = E\,\varepsilon_s$

$= 200000 * 0.00184$

$\boxed{f_s = 368\ MPa}$

في هذا المثال
تم زيادة A_s عن المثال
السابق والنتيجة

$A_s \uparrow \quad a \uparrow$

$c \uparrow$

$\varepsilon_s \downarrow$

$M_n \uparrow$

— Compute M_n and $\emptyset M_n$

$M_n = A_s * f_s * (d - a/2)$

$= 2080 * 368 \left(500 - \dfrac{265.76}{2}\right)$

$M_n = 413\ KN.m$

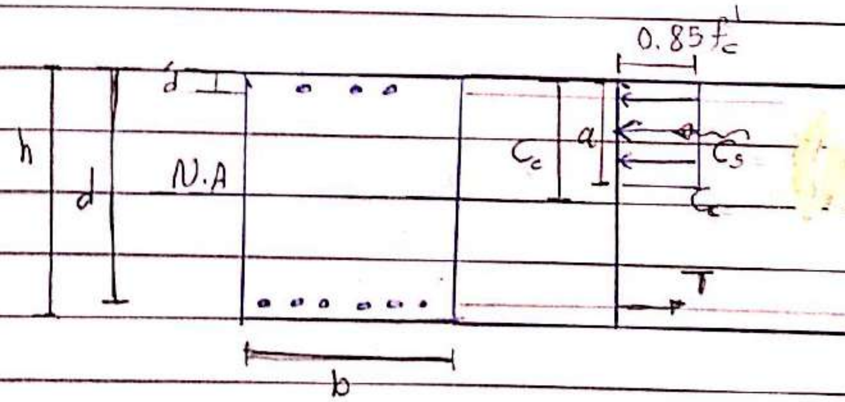
$\varepsilon_s = 0.00184 < \varepsilon_y \longrightarrow$ compression

$\emptyset M_n = 0.65 * 413 = 268.45\ KN.m$

— Check $A_{s\,min}$

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

Beams with compression & Tension reinforcement.



(1) Doubly reinforced beam

$C_s = A_s' \, f_y$

$C_c = 0.85\, f_c'\, ab$

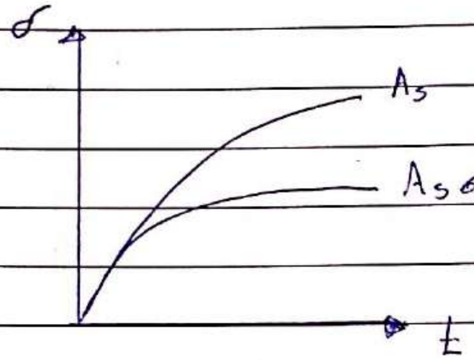
$T = A_s * f_y$

$\longrightarrow T = C_s + C_c$

Five Apple

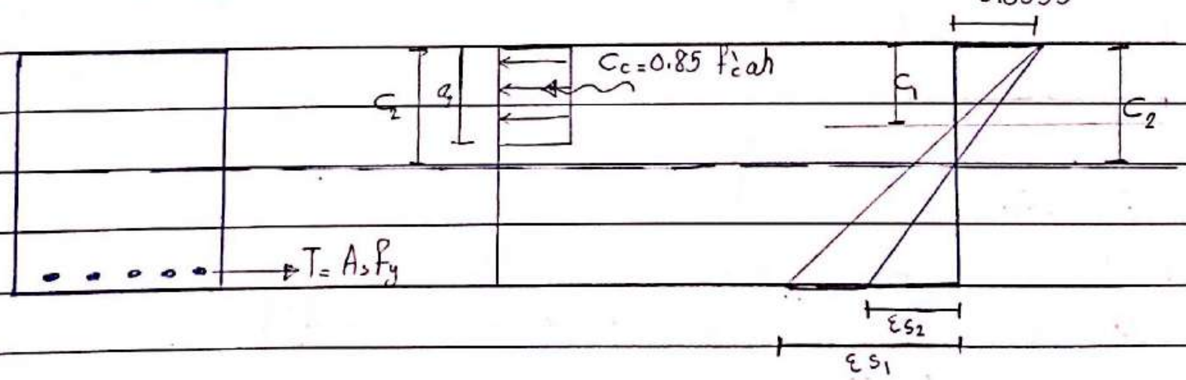
Reasons:

(1) Reduce sustained load deflections:



(2) Increase Ductility

(3) Change Mode of failure from (
to tension-controlled ($\varepsilon_s' = 0.0035$)



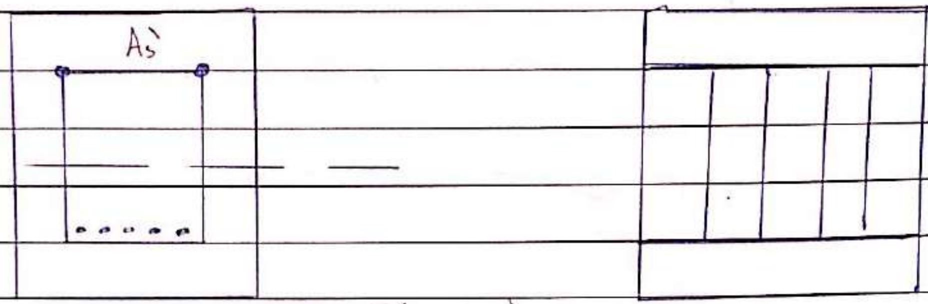
21/2/2016

سهولة التصنيع

④ fabrication Ease :

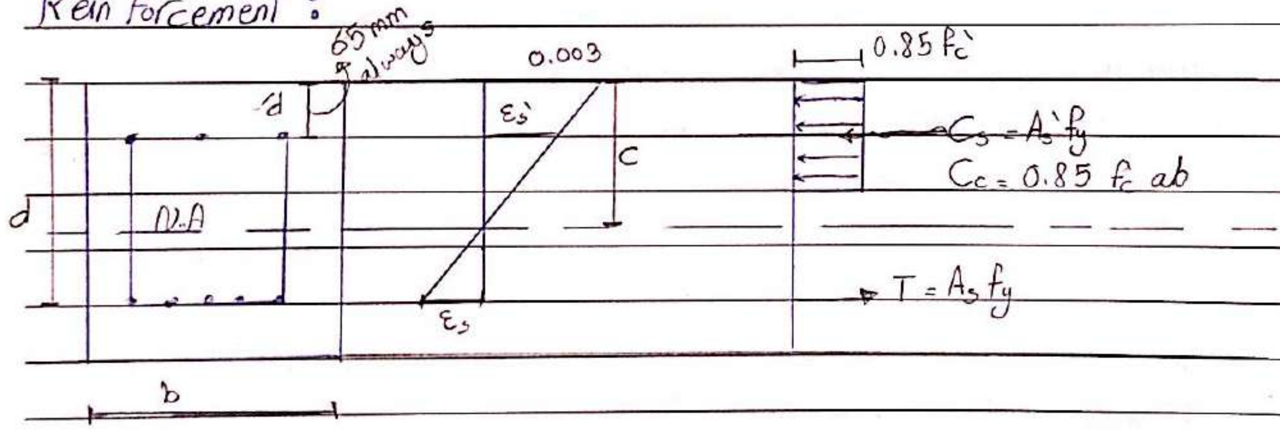
providing small bars in corners of stirrups to hold the stirrups in place.

نضع أسياخ صغيرة في زوايا الكانات لمسك الكانات في مكانها



σ_{yield} نزيد حدث (analysis) ق مصوج بعمل .

Analysis of Beam with tension & compression Reinforcement :



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

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

$T = C_c + C_s$

Assume $\varepsilon_s \geq \varepsilon_y$

$\varepsilon_s' \geq \varepsilon_y$

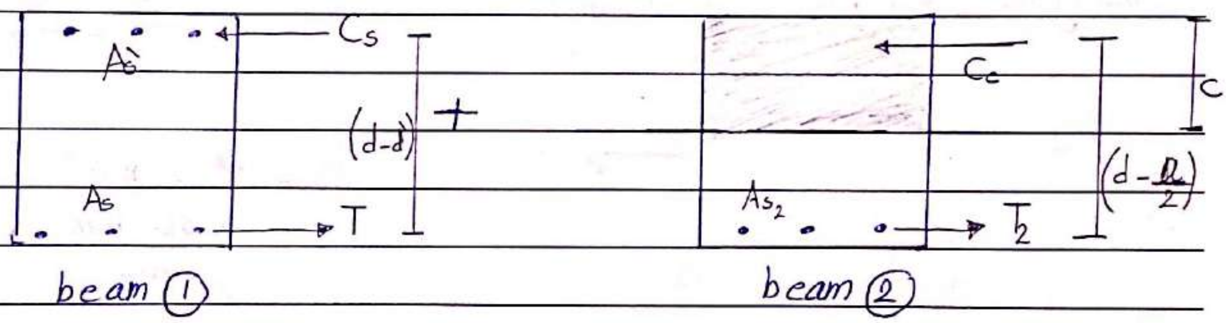
$A_s f_y = 0.85 f_c' ab + A_s' f_y$

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

$\varepsilon_s \geq \varepsilon_y$

case I :

$\varepsilon_s' \geq \varepsilon_y$



$C_s = T_1$

$A_s' f_y = A_{s_1} f_y$

$A_{s_1} = A_s'$

ما يكافئها لا يعني
نفس كمية الحديد

$a =$

$0.85 f' ab = A_{s_2} F_y$

$M_{n_1} = C_s (d - d')$

$M_{n_2} = C_c \left(d - \frac{a}{2}\right)$

$M_n = M_{n_1} + M_{n_2}$

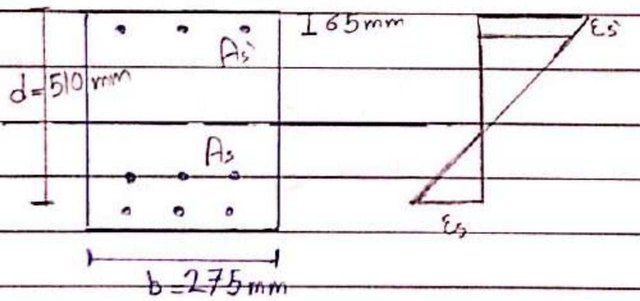
Case II : $\varepsilon_s' < \varepsilon_y$

$A_s f_y = 0.85 f_c' ab + A_s' f_s'$

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

$$(0.85 f_c' b)a^2 + (0.003 E A_s' - A_s f_y)a - 0.003(E A_s' \beta_1 d') = 0$$

Analysis example (ϕM_n) (23/2/2016)



$A_s' = 852 \ mm^2$
$A_s = 3060 \ mm^2$
$f_c' = 20$ MPa
$f_y = 420$ MPa
$\varepsilon_y = 0.0021$

(1) assume ε_s & $\varepsilon_s' \geq \varepsilon_y$

(2) $T = C_c + C_s$

$A_s f_y = 0.85 f_c' ab + A_s' f_y$

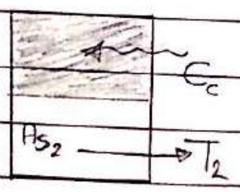
$(3060)(420) = 0.85(20)a(275) + (852)(420)$

$a = 198$ mm

OR (2 beam) ل (beam) تم تقسيم

$A_{s_1} = A_s' = 852 \ mm^2$

$A_{s_2} = 3060 - 852 = 2208 \ mm^2$



$A_{s_2} f_y = 0.85 \, f'_c \, ab$

$2208 * 420 = (0.85)(20)(a)(275) \rightsquigarrow a = 198 \text{ mm}$

$C = a/\beta_1 \Longrightarrow C = 233 \text{ mm}$

$\varepsilon'_s = 0.003 \left(\frac{233 - 65}{233} \right) = 0.0022$

$\varepsilon'_s > \varepsilon_y \Longrightarrow (f'_s = f_y)$ (oK)

$\varepsilon_s = 0.003 \left(\frac{510 - 233}{233} \right) = 0.0036$

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

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

$= 0.85 (20)(198)(272)\left(510 - \frac{198}{2}\right)$

$+ 852(420)(510 - 65)$

$M_n \simeq 540 \text{ KN.m}$

$C_c \neq T \longrightarrow T = C_c + C_s$

$\phi \xrightarrow{\text{ﺍﺧﻼ}} (\varepsilon_s = 0.0036)$

$\varepsilon_y < \varepsilon_s < 0.005 \Longrightarrow$ Tranzition

$\phi = 0.65 + (0.0036 - 0.002)(250/3)$

$= 0.783$

$\phi M_n = 0.783\,(540)$

$\simeq 422$ KN.m

→ Check $A_{s\,min}$ (A_s)

→ معادلتين ويتم اختيار الرقم الأكبر → $A_s = 3060 > A_{s\,min}$

في Analysis عادي لو كانت الناتج في أي نوع من أنواع failure
أما في Design يجب التصميم على أن نوع failure هو Tension

Ex $A_s' = 1704\ mm^2$
$A_s = 3060\ mm^2$
$f_c' = 20\ MPa$
$f_y = 420\ MPa$

هذا المثال هو شبيه بالمثال السابق لكن تم زيادة A_s' والنتيجة

$N.A \uparrow \rightarrow \varepsilon_s' \downarrow \rightarrow \varepsilon_s \uparrow$

(1) assume ε_s & $\varepsilon_s' \geq \varepsilon_y$

$$T = C_c + C_s$$

$$A_s f_y = 0.85 f_c' ab + A_s' f_y$$

$$3060(420) = 0.85(20)a(275) + 1704(420)$$

$$a = 121.8\ mm \Rightarrow C = 143.32\ mm$$

$$\varepsilon_s' = 0.003\left(\frac{143.32 - 65}{143.32}\right) = 0.0016$$

$$\varepsilon_s' < \varepsilon_y \Rightarrow f_s' \neq f_y$$

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

$$A_s f_y = 0.85 f_c' ab + A_s E\left(0.003\frac{(c-d')}{c}\right)$$

$$(3973.75)C^2 - (262800)C - 66456000 = 0.0$$

$$C = 33.07 \pm 133.48$$

$$C = 166.55\ mm \Rightarrow a = 141.57\ mm$$

$$\beta = 0.85$$

Five Apple

$$\varepsilon_s' = 0.003\left(\frac{166.55-65}{166.55}\right) = 0.0018$$

$$f_s' = E\,\varepsilon_s'$$

$$= 200000\,(0.0018) = 360\ MPa$$

$$\varepsilon_s = 0.003\left(\frac{510-166.55}{166.55}\right)$$

$$= 0.0062 > \varepsilon_y \Longrightarrow ok$$

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

$$= 0.85 * 20 * 141.57 * 275\left(510-\frac{141.57}{2}\right) + 1704 * 360\,(510-65)$$

$$M_n = 563.7\ KN.m$$

$$\varepsilon_s > 0.005 \Longrightarrow Tension \longrightarrow \phi = 0.9$$

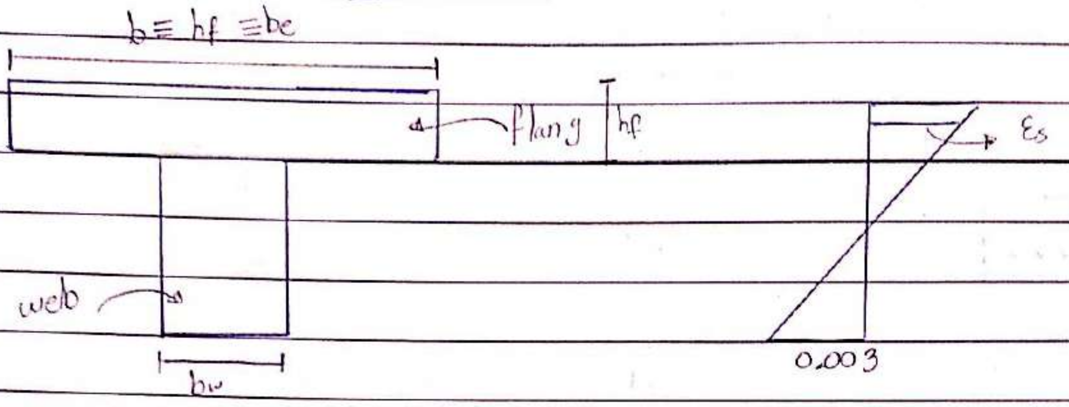
$$\phi M_n = 0.9 * 563.7 = 507.3\ KN.m$$

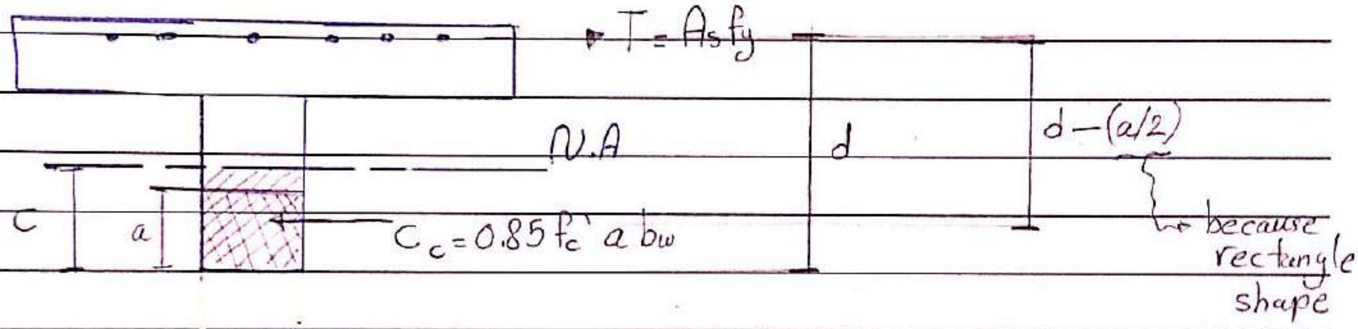
- Check $A_{s\,min}$

significant effect
عندما تتغير ϕ_2
لما أكون ف
Trazition
ونصبح ف
Tension

25/2/2016

flexure : T- Beams

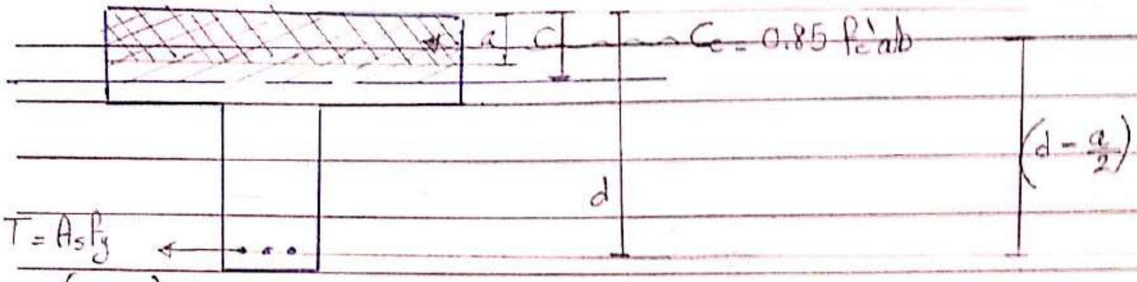




(−ve) M

$$A_s f_y = 0.85\, f_c`\, a\, b_w$$

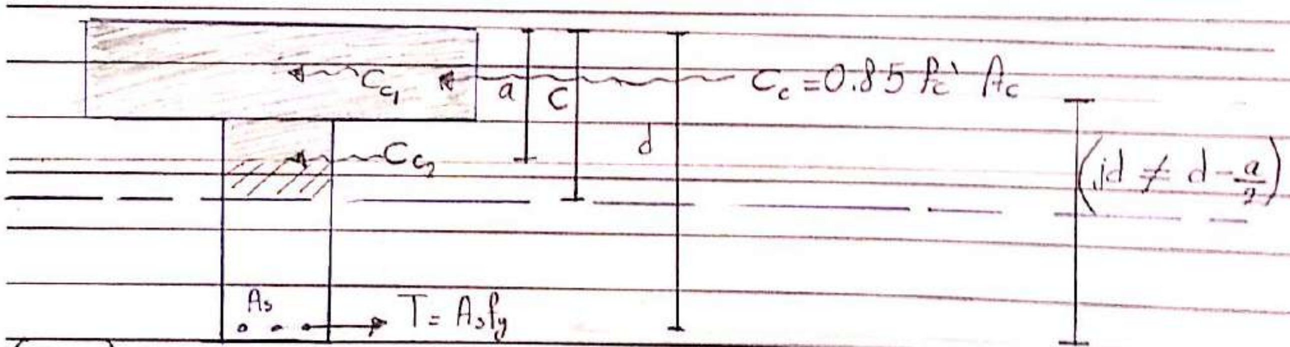
$$M_n = \frac{T}{C_c}\left(d - a/2\right)$$



$(+ve)M$

$$A_s f_y = 0.85 f_c' \, a b$$

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



$(+ve)M$

$$0.85 f_c' \left(b h_f + b_w \left(a - h_f\right)\right) = A_s f_y$$

$a > h_f$

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

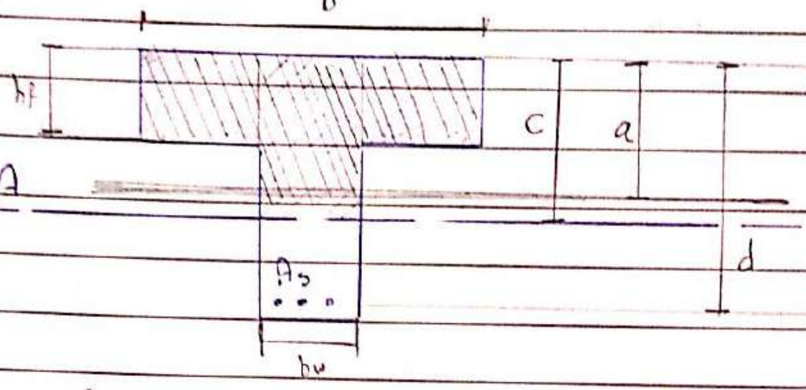
لإيجاد centroid لأي جسم

$$\bar{X} = \frac{\sum (A_i X_i)}{\sum A_i}$$

الطريقة الأتي

Analysis of Nominal Moment capacity for fanged in Positive Bending ($a > h_f$)

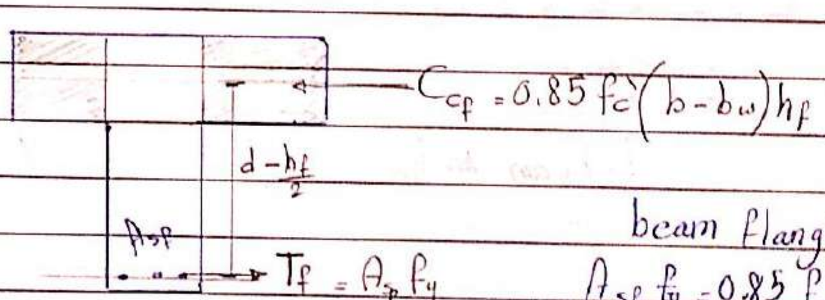
الطريقة الثانية



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

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

|| equal



beam flang(f)

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

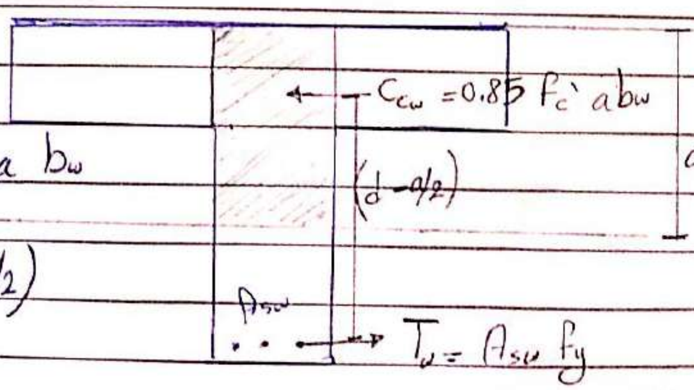
$M_{nf} = \frac{T_f}{C_{cf}}(d - h_f/2)$

beam flang(f)

$A_{sf} f_y = 0.85 f_c'(b - b_w)$

+

beamweb(w)

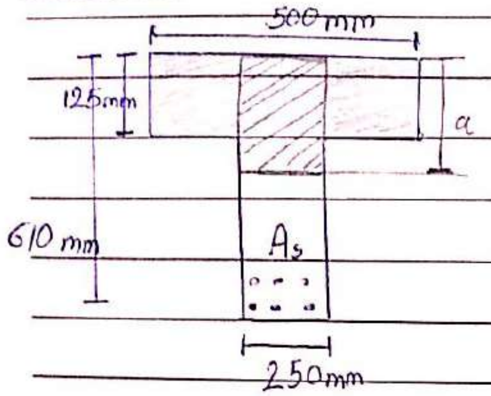


$A_{sw} f_y = 0.85 f_c' a b_w$

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

28/2/2016

Example



$f'_c = 20\ MPa$

$f_y = 420\ MPa$

$A_s = 3060\ mm^2$

Design +ve Moment capacity

- compute a

assume $\varepsilon_s \geq \varepsilon_y$

assume $a \leq h_f$

$C_c = T$

$0.85 f'_c\ ab = A_s f_y$

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

$a = 151.2\ mm > h_f = 125\ mm$

OR

لمعرفة إذا كان T-beam Action
OR Regtungular Action
من خلال حساب $ab = A_c$
ومقارنتها بـ A_f

T-beam Action

- Beam f

$C_{cf} = T_f$

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

$0.85 * 20 * 125 (500 - 250) = A_{sf} * 420$

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

- Beam W

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

$C_{cw} = T_w$

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

$0.85 (20)(a)(250) = 1795.1 (420)$

$a = 177.4\ mm$

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

$\varepsilon_s > \varepsilon_y = 0.0021 \Longrightarrow f_s = f_y$

- $M_{nf} = A_s f_y \left(d - \frac{h_f}{2} \right)$

$= 1264.9 * 420 \left(610 - 125/2 \right) = 290.9 \text{ kN.m}$

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

- $M_n = M_{nf} + M_{nw}$
$= 683.9 \text{ kN.m}$

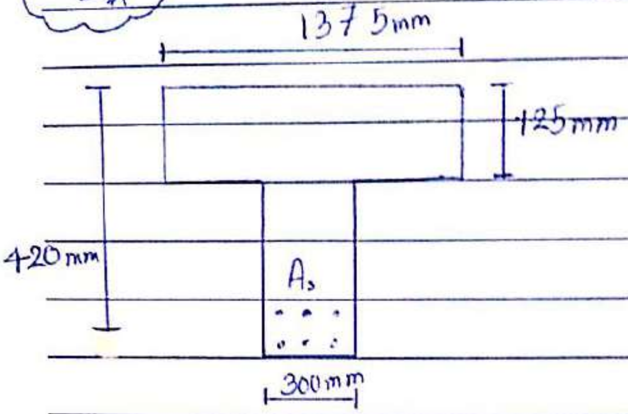
- $\varepsilon_s > 0.005 \Longrightarrow$ Tension $\Longrightarrow \phi = 0.9$

$\phi M_n = 615.5 \text{ kN.m}$

- $A_{s\,min} = \begin{cases} \frac{\sqrt{f'_c}}{4 f_y} b_w d = 406 \text{ mm}^2 \\ \frac{1.4}{f_y} b_w d = 509 \text{ mm}^2 \longrightarrow A_s > A_{s\,min} \end{cases}$

Per ACI-code:
for statically determinate beams where the flang partion is in tension, the ACI-code recommend, that (b_w) be replaced by smaller of $\begin{cases} 2b_w \\ b_e \end{cases}$

Ex



$f_c' = 20$ MPa
$f_y = 300$ MPa
$A_s = 1704\ mm^2$

Design the Moment capacity

. the same previous Ex. but

$A_s \downarrow$ $(C_c = T)$ $a \downarrow$ $b_f \uparrow$ $\Rightarrow$ $\varepsilon_s \uparrow$

. compute a

assume $\varepsilon_s \geq \varepsilon_y$

assume $a \leq h_f$

$C_c = T$

$0.85\ f_c'\ ab = A_s f_y$

$0.85 * 20 * a * 1375 = 1704 * 300$

$a = 21.9$ mm $< h_f \Rightarrow$ rect.

$c = 25.8$ mm

$\varepsilon_s = 0.046 > \varepsilon_y = \frac{300}{200000} = 0.0015$

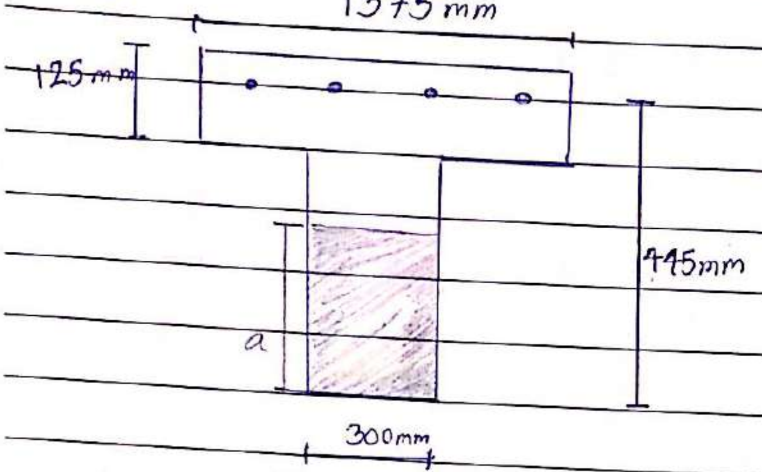
$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 209$ KN.m

$\phi = 0.9 \Rightarrow$

$\phi M_n = 188$ KN.m

1/3/2016

Ex



Indet. beam

$f'_c = 20$ MPa

$f_y = 300$ MPa

$A_s = 2272$ mm^2

assume $\varepsilon_s \geq \varepsilon_y$

$0.85 f'_c a \underbrace{b}_{300} = A_s f_y$

$a = 133.6$, $\varepsilon_s = 0.046$

$C = 157.2$ mm

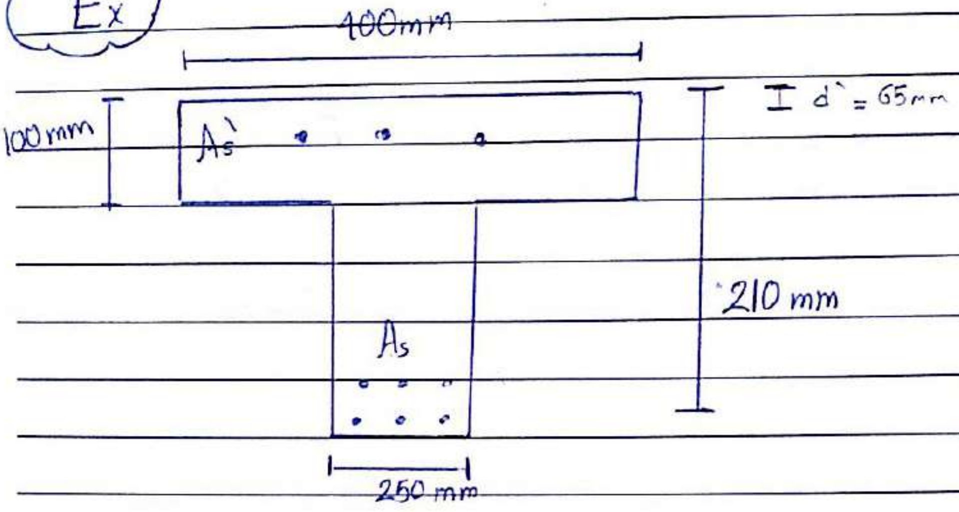
$\varepsilon_s = 0.0055 > \varepsilon_y \rightarrow$ ok , $\varepsilon_y = 0.0015$

$M_n = A_s f_y (d - a/2) = 258$ KN.m

$\phi M_n = 0.9 M_n = 232$ KN.m

check $A_{s,min} \rightarrow$ No modification for you

Ex



$A_s' = 852 mm^2$

$A_s = 3060 mm^2$

$f_c' = 20 MPa$, $f_y = 420 MPa$

$\phi M = ??$

- $\varepsilon_s' > \varepsilon_y$
  $\varepsilon_s > \varepsilon_y$ $\Big\}$ $\varepsilon_y = \frac{420}{200000} = 0.0021$

- compute a

assume $a \leq h_f$

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

$a = 136.4 mm < h_f \rightarrow$ Not ok

$\therefore$ (T shape)

$T = C_c + C_s$

$A_s f_y = 0.85 f_c' \left(b h_f + b_w (a - h_f) \right) + A_s' f_y$

$a = 158.20 mm$

$\beta = 0.85$

$C = 186.12 mm$

• check ε_s'

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

$$= 0.002 \ngtr \varepsilon_y \rightsquigarrow \text{Not ok}$$

• $T = C_c + C_s$

$$A_s f_y = 0.85 f_c'\left(b h_f + b_w(a - h_f)\right) + A_s' f_s'$$

$$A_s f_y = 0.85 f_c'\left(b h_f + b_w(\underset{\substack{\downarrow \\ 0.85c}}{a} - h_f)\right) + \left(A_s' * E * 0.003\left(\frac{c-d'}{c}\right)\right)$$

$c = 191.66\ mm \rightarrow a = 162.911\ mm \rightsquigarrow \varepsilon_s' < \varepsilon_y$

• Check ε_s , $\varepsilon_s = 0.0003 \ngtr \varepsilon_y \rightsquigarrow$ Not ok

$$T = C_c + C_s$$

$$A_s\left(E * 0.003 * \left(\frac{d-c}{c}\right)\right) = 0.85 f_c'\left(b h_f + b_w(0.85c - h_f)\right)$$

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

$$1836000\left(\frac{210-c}{c}\right) = 255000 + 212.5c + 511200\left(\frac{c-65}{c}\right)$$

$c = 158.87\ mm \quad \varepsilon_s < \varepsilon_y$

$a = 135.04\ mm \rightsquigarrow$

$M_T = M_f + M_w + M_{s'}$

$= 0.85 f_c' (b - b_w) h_f (d - h_f/2)$

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

$+ A_s' f_s' (d - d')$

$f_s' = 354.52$ MPa

$= 40800$
$+ 81772.12$
$+ 43797.40$

$= 166$ KN.m

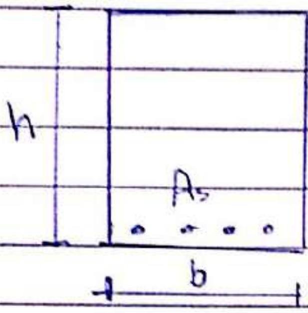
$\phi \rightarrow \varepsilon_s < \varepsilon_y \rightarrow \phi = 0.65$

$\phi M_n = 166 \text{ KN.m} * 0.65$
≈ 108 KN.m

check $A_{smin} = \begin{cases} \frac{0.25\sqrt{20}}{420} * 250 * 210 = 140 \text{ mm}^2 \\ \frac{1.4}{420} \; 250 \; 210 = 175 \text{ mm}^2 \end{cases}$

Design of rectangular Beams

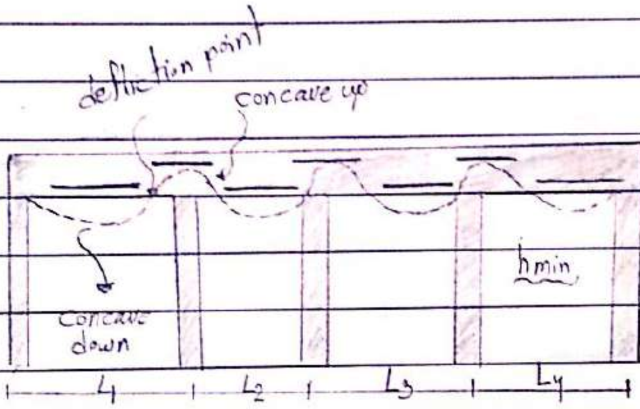
$$M_u \le \phi M_n$$



. المثالي نصل له مساواة إذاً $M_n = M_u$

ومن ثم نصل له $M_n > M_u$

. في rectangle لا يختلف في ما إذا كان (+ve) أو (-ve) (Moment) سوى أن موقع الحديد مختلف.



. كلما زاد عمق beam قل deflection

. concave up → C فوق   concave down → T فوق

T تحت   C تحت

. يتم عمل check على deflection لكل span من خلال hmin

ومن ثم أخذ hmin لأكبر span

. كل beam نفس الطول لذلك يتم تغيير As

* Relationship between beams $\underbrace{\text{depth}}_{h}$ & deflection:

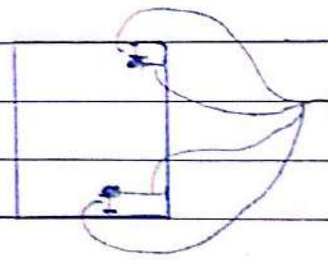
table (9.5 a) (To avoid deflection calc.)   لست مضطراً على check لـ deflection

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

$$h_{min}\ (Cant.) = \frac{L}{8}$$

انتبه في التعامل مع الجدول

* Concrete Cover & Bar spacing



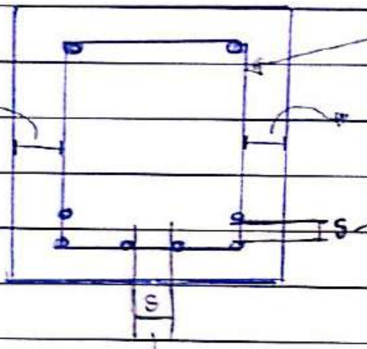
$\rightarrow h_{min} = 40\ mm = 4cm$

Resouns : { مطلوبة موجودة في الكتاب } ← ٤ أسباب

strumps

40mm

40mm (to Keep steel and bonding)

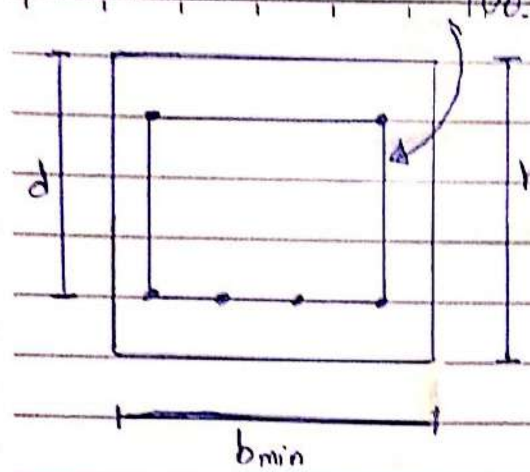


$\rightarrow S_{min}$ = larger of
- Bar diameter ; d_b
- 1.33 Max. C.A. size
- 25 mm
- Diameter of Vibrator

S_{min} = larger of
- 25 mm
- 1.33 Max. C.A. size

Five Apple

3/3/2016



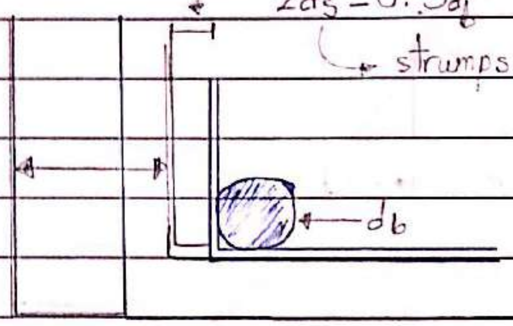
$$b_{min} = 2 * 40 \quad \text{cover}$$
$$+ 2 * 10 \quad \text{الكانات}$$
$$+ 4d_b + 3S_{min} \quad \text{تشبيك الحديد}$$
$$+ 2(2d_s - 0.5d_b)$$

الحديد
one or two layer

- Estimating the effective depth of a beam :

$d = h - 65mm$ (one layer)

$d = h - 90\,mm$ (two layer)



$b_{min} \geq$ → preferable (300 mm) أفضل

→ Absolute (250 mm) أساسي

should not be less than

- General Strength Design Requirments for Beams :

$$\phi M_n \geq M_u$$

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

$W_u = 1.4\,DL \rightsquigarrow DL > LL$ عندما تكون

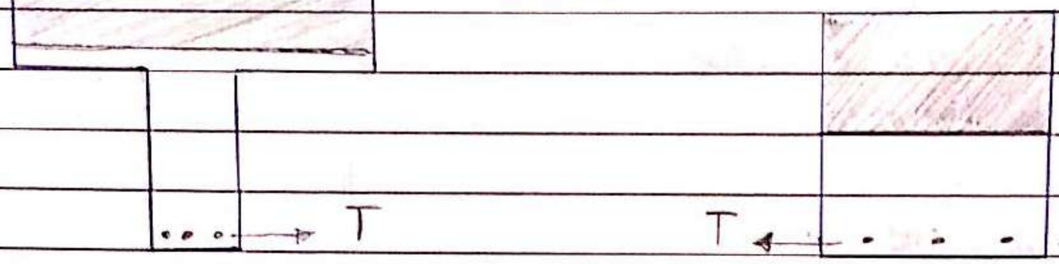
should equal 0.9 tension (ϕ)

$\phi M_n = M_u$

$\phi A_s f_y jd = M_u \rightsquigarrow A_s = \dfrac{M_u}{\phi f_y jd}$ $\quad j = (0.87 - 0.91)$

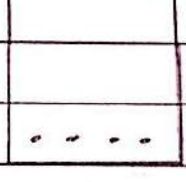
in exam. and ... $j = 0.9$

$j = 0.95$



Why $j = 0.95$ in T-beam
largest j in Regtangular beam ?

a in T-beam $<$ a in Regtangular beam

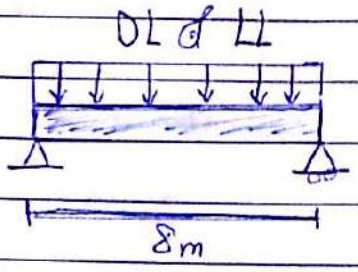


I 65 mm = Stirrump + cover + $\underline{\underline{r}}$ of steel

Five Apple

6/3/2016

* Design of reinforcement when b & h are Known :-

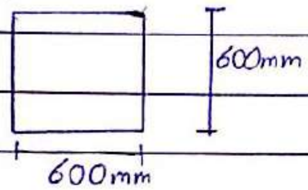


$LL = 35\ Kn/m$

$DL = 14\ KN/m$ (excluding self wt.)

$f'_c = 20\ MPa$, $f_y = 420\ MPa$

$\gamma_{con.} = 24\ KN/m^3$



→ $M_u = M_{max} = \dfrac{wL^2}{8}$

LL > DL

$W_u = 1.2\ DL + 1.6 LL$

self wt. $= 24 * 0.6 * 0.6 = 8.64\ KN/m$

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

$W_u = 83.2\ KN/m$ → دخل (safety factor) يعني (Design)

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

→ compute d

assume one-layer

$d = 600 - 65 = 535\ m$

→ $A_s = \dfrac{M_u}{\phi f_y\, jd}$

$= \dfrac{665.6 * 10^6\ (N.mm)}{0.9 * 420\ (N/mm^2) * 0.9 * (535\ mm)}$

$A_s = 3657\ mm^2$ (required)

6 bars + 2 bars

Provided → 8 No. 25 M; $A_s = 4080\ mm^2$ ✓
6 No. 29 M ; $A_s = 3870\ mm^2$ ✗

$b_{min} = 2*40 + 2*10 + 8*25 + 7*25 + 2(10*2 - 0.5*25)$

$b_{min} = 490\ mm < 600\ mm$
⇒ one-layer is ok ⇒ $d = 535\ mm$

should be
Provided > Required
$A_{s\,prov} > A_{s\,min}$

عندما ٨ نعطي
3) السؤال عدد bars
يتم ايجاد مساحة 1 bars
ونقسم على $A_{s\,req.}$

• check $A_{s\,min}$
$A_{s\,min} = 854\ mm^2$ ✗
$= 1070\ mm^2$ → ✓
$A_s = 4080 > A_{s\,min}$ ⇒ ok

• compute a
$A_s f_y = 0.85 f'_c\, a b$
$4080 * 420 = 0.85 (20)(a)(600)$
$a = 168\ mm$; $c = 197.6\ mm$

$\varepsilon_s = 0.003 \left(\frac{535 - 197.6}{197.6} \right)$

$\varepsilon_s = 0.0051 > 0.005$
⇒ Tension ⇒ $\phi = 0.9$

$\phi M_n = 0.9 * 4080 * 420 * \left(535 - \frac{168}{2}\right)$

$= 695.6\ kN.m > M_u$ ⇒ ok

→ Check $A_{s\,required}$ based on computed value of a.

$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{665.6 * 10^6}{0.9 (420)(535 - 168/2)}$

$A_s = 3904\ mm^2 < A_{s\,provided} = 4080\ mm^2$ ⇒ ok

8/3/2016

$\longrightarrow h_{min} = \frac{L}{16}$

$= \frac{8000\,mm}{16}$

$= 500\,mm \longrightarrow$

{ when
$h < h_{min}$
You are check
to deflection
& You are check to deflection in RC

Design of rectangular beams (b, h, A_s)

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

$\rho = \frac{A_s}{bd}$ (steel ratio, reinforcement ratio)

$A_s = \rho\, bd$

$a = \frac{\rho b d\, f_y}{0.85\, f'_c\, b} \Longrightarrow a = \frac{\rho\, f_y}{f'_c} \frac{d}{0.85}$ $\rightarrow$ w-mechanical steel ratio

$w = \frac{\rho\, f_y}{f'_c}$

$a = \frac{w\, d}{0.85}$

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

$\phi M_n = \phi \left(bd^2 \left(f'_c\, w\, (1 - 0.59w) \right) \right)$ $\rightarrow$ Flexural resistance factor (R_n, K_n)

$K_n = f'_c\, w\, (1 - 0.59w)$

$\phi M_n = \phi\, bd^2 K_n$

$M_u = \phi\, b d^2 K_n$

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

M_u (N.mm)
ω (Unless Unit)
ρ (Unless Unit)
K_n (N/mm^2)

→ To estimate self wt.

(1) The weight of a rectangular beam will be roughly (10 to 20%) of the loads it must carry
Unfactored

(2) OR $h \simeq$ (8–10%) of the span length.

$b \simeq (0.5)\,h$
self wt. $= \gamma b h$

لتحديد → ρ

الاعتبارات الاقتصادية
- Economic Consideration :-

$\rho = 0.01$

(Moment of inertia) (I) وذلك بسبب $b < d$ يجب أن تكون .
(بحسب الحفاظ عليها)

- By Placing considreation:
(It may be hard to place the reinforcement if ρ exceeds 0.015

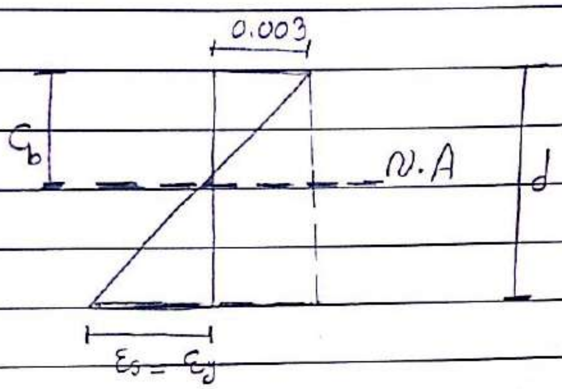
max. ←

Ductility consideration

(نستخدم في المناطق النشطة من ناحية الزلازل والبراكين)

$\rho \simeq (0.35 - 0.4)\,\rho_b$ → balanced steel ratio

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



$\rho_b = ??$

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

$$C_b = \frac{0.003}{0.003 + \varepsilon_y}\, d \quad \ldots (1)$$

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

$$A_{s_b} = \rho_b\, b\, d \quad , \quad C_b = a_b/\beta_1 = \frac{\rho_b\, d\, f_y}{0.85\, \beta_1\, f'_c} \quad \ldots (2)$$

equ. (1) & (2)

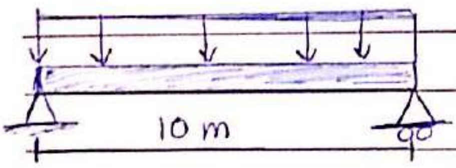
$$\rho_b = \frac{0.85\, \beta_1\, f'_c}{f_y}\left(\frac{0.003}{0.003 + \varepsilon_y}\right)$$

OR

$$\rho_b = \frac{0.85\, \beta_1\, f'_c}{f_y}\left(\frac{600}{600 + f_y}\right)$$

10/3/2016

Design Example



$LL = 25.5\ KN/m$

$DL = 14.5\ KN/m$ (excluding)

$f'_c = 25\ MPa$ $\quad \gamma_{conc} = 24\ KN/m^3$

$f_y = 420\ MPa$

- Estimate self. wt.

self wt. = $(10 - 20\%)(14.5 + 25.5)$

= $4 - 8\ KN/m$

OR $h = (8 - 10\%)\ 10m = (0.8m - 1m)$

$b \simeq (0.5h) = (0.4m - 0.5m)$

Self wt. $= 24 * 0.4 * 0.8 = 7.68\ Kn/m$

Try self wt. = $8\ KN/m$

- compute M_u

$W_u = 1.2(14.5 + 8) + (1.6 * 25.5) = 67.8\ Kn/m$

$M_u = \frac{67.8 * 10^2}{8} = 848\ KN.m \quad , \quad M_u = \frac{wL^2}{12}$

- compute b & d

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

$w = \rho \frac{f_y}{f'_c} \quad ; \quad \rho = 0.01$

$w = 0.01 * \frac{420}{25} = 0.168$

Five Apple

$\phi K_n = 0.9 * 25 * 0.168 (1 - 0.59 * 0.168)$

$\phi K_n = 3.41 \text{ MPa}$

$bd^2 = \frac{848 * 10^3}{3.41} = 248.7 * 10^6 \text{ mm}^3$

$b = 300$ mm , $d = 910$ mm ← assume b

$b = 400$ mm , $d = 788$ mm

$b = 450$ mm , $d = 743$ mm

$h = 788 + 90 = 878$ (Assumming 2 layer)

يجب الحفاظ على I
التي تؤثر على ϕM_n

Use $h = 900$ mm → $d = 900 - 90$
$b = 400$ mm → $d = 810$ mm

should be have kept $d > b$
. كلما زادت قيمة M_u
زادت قيمة العدد
وكلما زادت قيمة b
زاد فرض 1-layer

. check Self wt. & revise M_u

Self wt. $= 24 * 0.4 * 0.9 = 8.64$ Kn/m

$W_u = 1.2 (14.5 + 8.64) + (1.6 * 25.5)$
$= 68.57$ Kn/m

. $M_{u_{new}} = \frac{68.57 * 10^2}{8} = 857$ KN.m

(If $M_{u_{new}}$ is increased by 10% or more repeat the design.)

$\frac{857 - 848}{848} * 100\% = 1.1\% < 10\%$

Don't repeat → continue with $M_u = 857$ KN.m

. اذا كان $(\text{Self wt.})_{new}$ أقل من (Self wt.) المفروضة ليس من الذكي حساب M_u

Five Apple

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

$A_s = \dfrac{857 * 10^6}{0.9 * 420 * 0.9 * 810}$

$A_s = 3110\ mm^2$

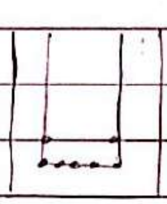
- Select steel

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

$b_{min} = 2*40 + 2*10 + 7*25 + 6*25 + 2(2*10 - 0.5*25)$

$= 440\ mm > 400 mm$

$\Rightarrow$ 2-layer is ok



layer 5 No. 25 M

$b_{min} = 340 < 400mm$

yes it's ok

- check $A_{s\,min}$

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

$A_s = 3570 > A_{s\,min} \Rightarrow$ ok

- check ε_s

$a = \dfrac{3570 * 420}{0.85 * 25 * 400} = 176.4\ mm$;

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

$\varepsilon_s = 0.003\left(\dfrac{810 - 207.5}{207.5}\right) = 0.0087 > 0.005$

$\Rightarrow$ Tension-controlled

$$\phi M_n = 0.9 * 3570 * 420 * \left(810 - \frac{176.4}{2}\right)$$

$$\phi M_n = 974 \text{ KN.m} > M_u \ (857 \text{ KN.m})$$

- Check $A_{s\,(required)}$ based on computed $\underline{a}$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{857 * 10^6}{0.9 * 420 * (810 - 176.4/2)}$$

$A_s = 3141 \text{ mm}^2 < A_s$ provided $\Rightarrow$ ok

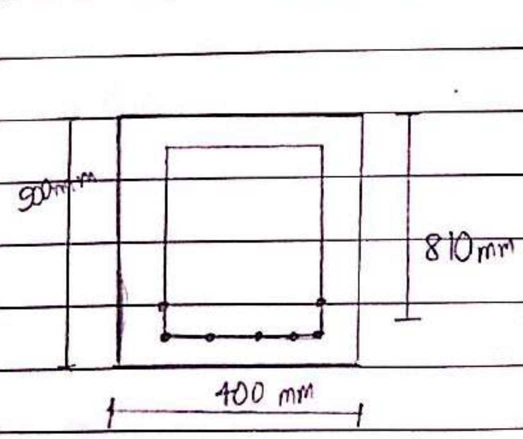
- Check h_{min} → هل نحتاج
لعمل check
على Deflection ؟؟

$$h_{min} = \frac{L}{16}$$

$$= \frac{10000}{16} = 625 \text{ mm} < 900 \text{ mm}$$

don't have any check for deflection

Finally :



7 No. 25M

$A_s = 3570 \text{ mm}^2$

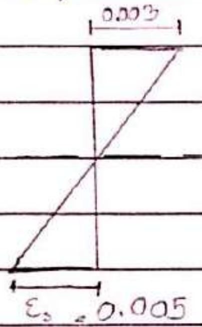
. Maximum Area of Steel

$A_{smax} = 0.75\, A_{sb}$ ⟶ كانت موجودة في code
سابقا وتم الغاؤها

$\varepsilon_s \geq 0.005$ (Tension)

واستبدالها
ب

. $A_{smax} \Longrightarrow \varepsilon_s = 0.005$



$T = C$

$$A_{smax} = \underbrace{0.319\, \beta_1 \frac{f'_c}{f_y}}_{\rho_{max}}\, b\, d$$

pre. Ex. : ⟶ $A_{sreq} = 3110\ mm^2$

⟹ $A_{smax} = 5230\ mm^2$

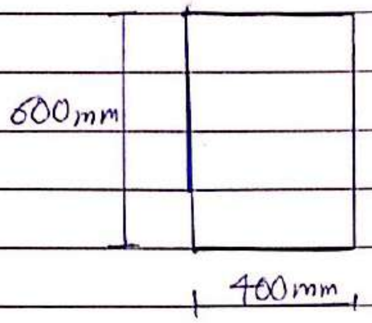
$A_{sprovided} = 3570\ mm^2$

$a, C, \varepsilon_s = 0.0087$

. $A_{smin} < A_{sreq.} < A_{sprov.}$

Five Apple

Design Example



$M_u = 720\ Kn.m$

$f'_c = 28\ MPa$

$f_y = 414\ MPa$

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

assume two-layer $d = 600 - 90$

$d = 510mm$

$$A_s = \frac{720 * 10^6}{0.9 * 414 * 0.9 * 510}$$

$A_s = 4210\ mm^2$ (required)

$$A_{s\,max} = 0.319 * 0.85 * \frac{28}{414} * 400 * 510$$

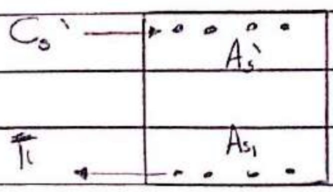
$A_{s\,max} = 3741\ mm^2$

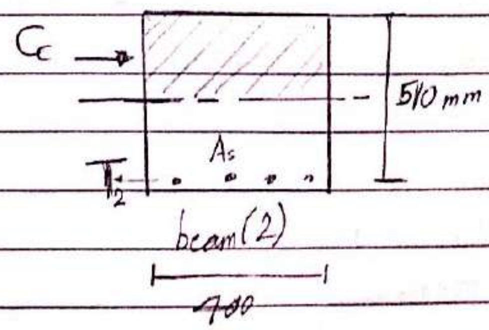
$A_s > A_{s\,max} \longrightarrow$ Not tension

$\varepsilon_s < 0.005$

$\Longrightarrow$ Doubly reinforced beam

$A_{s_2} = A_{s\,max} = 3741\ mm^2$





$a = \frac{A_{s\,max}\ f_y}{0.85\ f'_c\ b} = \frac{3741 * 414}{0.85 * 28 * 400} = 162.7\ mm$

$C = a/\beta_1 = 191.4\ mm$

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

$\phi M_{n_2} = \phi A_{s_2} f_y (d - a/2)$

$= 0.9 * 3741 * 414 \left(510 - \frac{162.7}{2}\right)$

$\phi M_{n_2} = 597.5\ KN.m$

$\phi M_n = \phi M_{n_1} + \phi M_{n_2} = M_u$

$\phi M_{n_1} + 597.5 = 720$

$\phi M_{n_1} = 122.5\ KN.m$

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

$= 0.003\left(\frac{191.4 - 65}{191.4}\right)$

$= 0.00198 < \varepsilon_y$

$\phi M_{n_1} = \phi A'_s f'_s (d - d')$

$122.5 * 10^3 = 0.9 * A'_s (396)(510 - 65)$

$A'_s = 772\ mm^2$

- $F'_s = 200000 * 0.00198$

$= 396\ MPa$

. Eqpm in Beam 1

$A_{s1} f_y = A'_s f'_s$

$A_{s1} * 414 = 772.4 * 369$

$A_{s1} = 738.8\ mm^2$

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

$= 738.8 + 3741$

$= 4479.8\ mm^2 \rightarrow A_s$ Required

Five Apple

*

15/3/2016

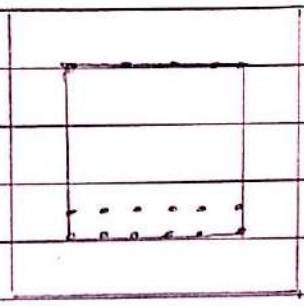
. Select Steel

$A_s \Longrightarrow$ 12 No. 22M; $A_s = 4640\ mm^2$

$b_{min} = 2*40 + 2*10 + 12*22 + 11*22 + 2(2*10 - 0.5*22)$

$= 624mm > 400\ mm$

$\longrightarrow$ 2 layer



bmin 5No. 16 MPa

12No. 22 MPa

b_{min} (6 bars) $\longrightarrow$ $b_{min} = 375mm < 400mm$

$4640 = A_{s1} + A_{s2}$

$A_{s1} = 4640 - 3741 = 899\ mm^2$

$A_s' * 396 = 899 * 414$

$A_s' = 940\ mm^2$

5 No. 16 MPa , $A_s' = 995\ mm^2$

$\varepsilon_s > 0.005 \Longrightarrow 0.0051$

$\phi M_n = 748\ KN.m$

new analysis

$a = 158.5\ mm$

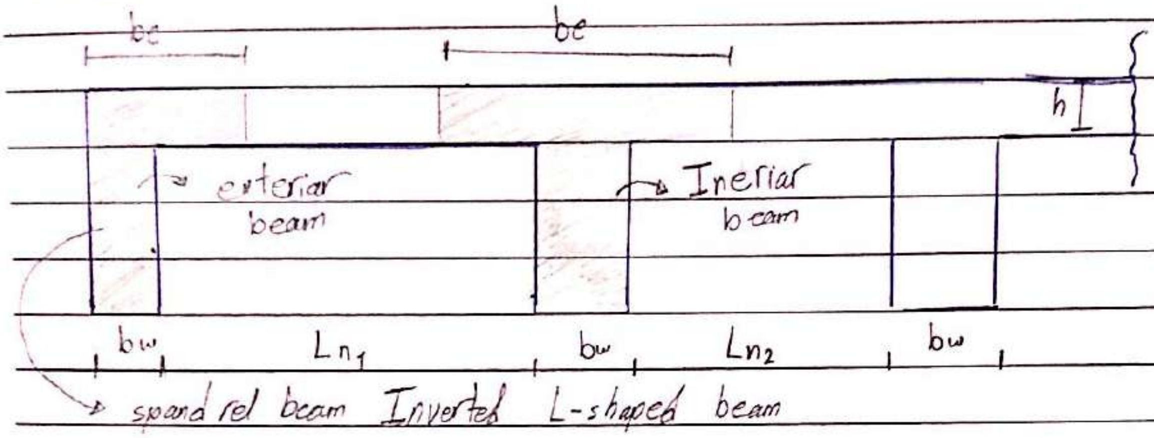
$c = 186.5\ mm$

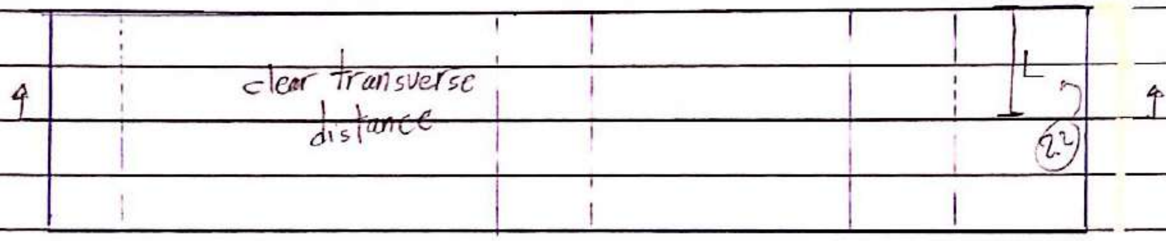
$M_n = 833\ KN.m$

$\varepsilon_s' = 0.002 = \varepsilon_y$

17/3/2016

Design of T-Beams :





Top view

L_n: span length

كل span
يتم العمل عليه لحال

Spandrel beam (Exteriar)

$$b_e \simeq \text{Smaller of} \begin{cases} b_w + (L_{n_1}/2) \\ b_w + 6h_f \\ b_w + (L/12) \end{cases}$$

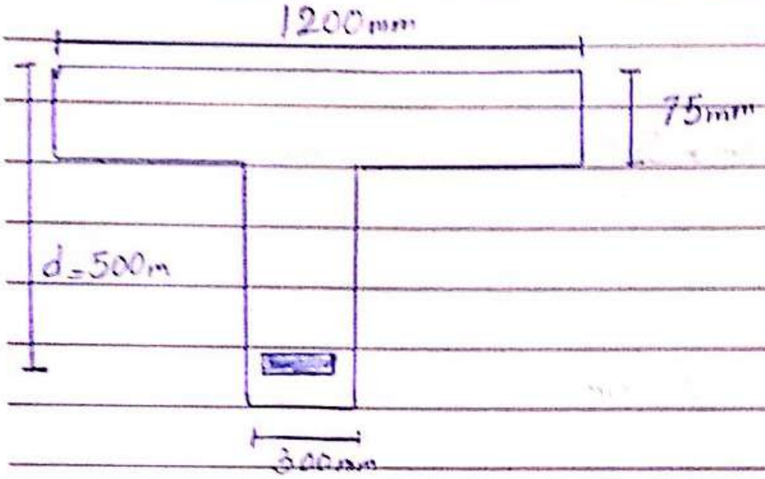
$\underline{L}$ span Length of the beam

Interior:

$$b_e \simeq \text{smaller of} \begin{cases} b_w + \frac{L_{n_1}}{2} + \frac{L_{n_2}}{2} \\ b_w + 2(8h_f) \\ L/4 \end{cases}$$

Five Apple

Design Example



$f'_c = 21$ MPa

$f_y = 420$ MPa

M_u (+ve) = 740 Kn.m

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

- wide compression areas $j = 0.95$ ← compressin in flange
- compression in the web $j = 0.9$ (0.87 – 0.91)

$A_s = \dfrac{740 * 10^6}{0.9 * 420 * 0.95 * 500}$

$A_s = 4121$ mm² → 9 No.25M ; $A_s = 4590$ mm²

- $b_{min} = 540$mm > 300mm →

تم عمل هذه الخطوة من أجل
معرفة كيفية وضع الحديد على طبقة أو طبقتين

- $A_{smin} = \begin{cases} 500 \text{ mm}^2 \\ 400 \text{ mm}^2 \end{cases}$, $A_s > A_{smin}$, oK ✓

- compute $\underline{a}$ & ε_s , $a \le h_f$

$$a = \frac{4590 * 420}{0.85 * 21 * 1200} = 90 \text{ mm} > h_f \Rightarrow \text{T-beam Action}$$

$$A_{sf} = \frac{0.85 * 21 * 75 * (1200 - 300)}{420} = 2869 \text{ mm}^2$$

$$A_{sw} = 4590 - 2869 = 1721 \text{ mm}^2$$

$$a = \frac{1721 * 420}{0.85 * 21 * 300} = 135 \text{ mm} \quad ; \; c = 158.8 \text{ mm}$$

$$\varepsilon_s = 0.003 \left(\frac{500 - 158.8}{158.8} \right) = 0.0064 > 0.005$$

$\Rightarrow$ Tension - Controlled

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

$$\phi M_{nf} = 0.9 * 2869 * 420 \left(500 - \frac{75}{2} \right) = 501.6 \text{ KN.m}$$

$$\phi M_{nw} = 0.9 * 1721 * 420 \left(500 - \frac{135}{2} \right)$$

$$= 281.4 \text{ Kn.m}$$

$$\phi M_n = 783 \text{ Kn.m} > M_u \Rightarrow \text{oK}$$

- Check A_s required based on computed $\underline{a}$.

$$\phi M_{nw} = \phi A_{sw} f_y \left(d - \frac{a}{2} \right)$$

$$M_u = \phi M_{nf} + \phi M_{nw}$$

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

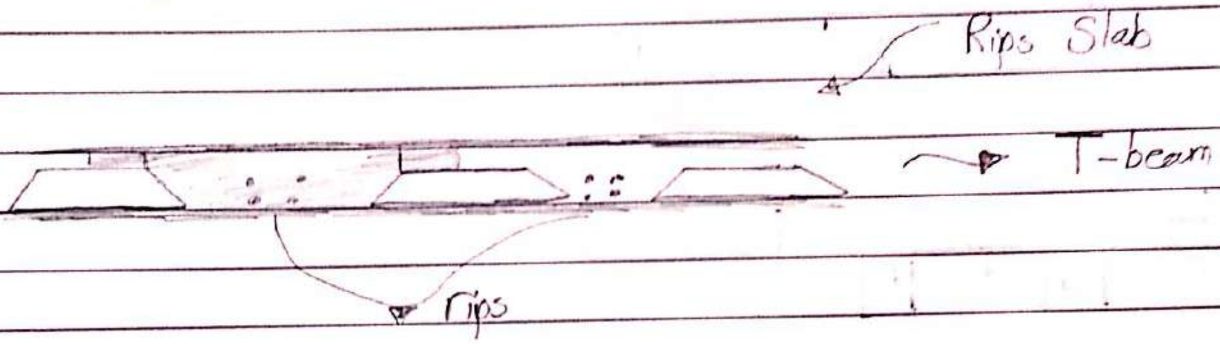
$$\phi M_{nw} = 238.4 \text{ Kn.m}$$

$$238.8 \times 10^3 = 0.9 \times A_{sw} \times 420 \left(500 - \frac{135}{2}\right)$$

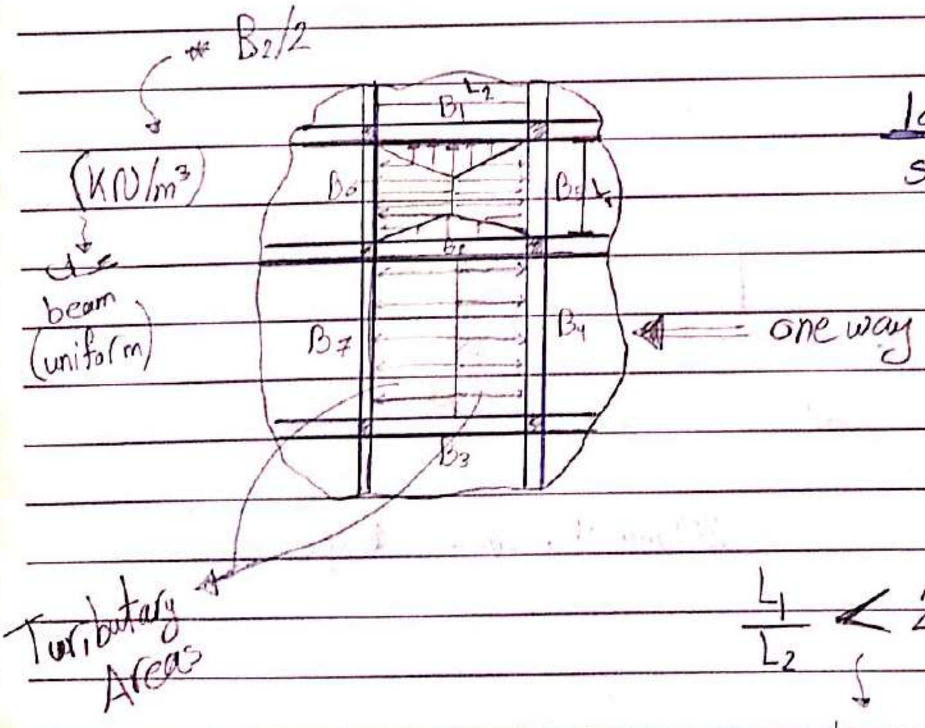
$A_{sw} = 1461\ mm^2 < 1721\ mm^2$

$\Rightarrow$ ok

One-way Solid Slab



الذي يقسم (slab) هو توزيع (beam).



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

< 2 (two way)

معامل الجساءة

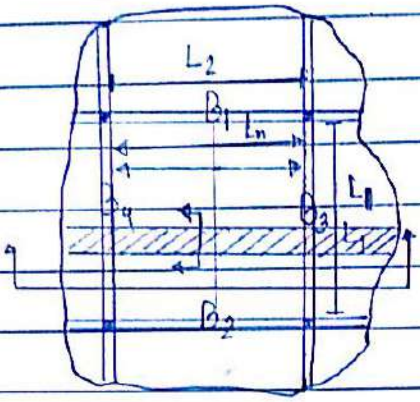
$\uparrow K = \frac{4EI}{L \downarrow}$

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

two way

22/3/2016

One way solid slab:

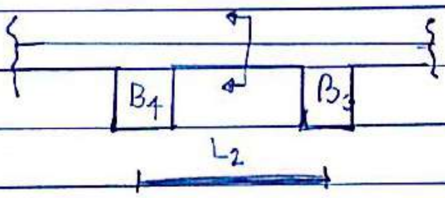


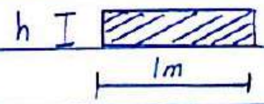
$\frac{L_1}{L_2} \geq 2 \rightarrow$ one way

يصمم كأنه
Rectangular

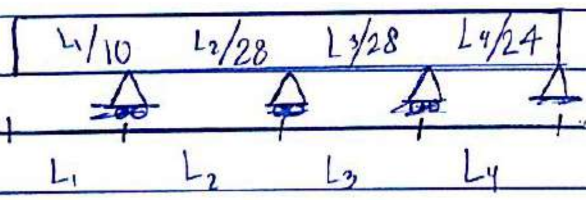
I 1.0m

ف الاتجاه القصير
1m وعرضه





h_{min} Table (9.5a)



Mininmum Cover = 20 mm (Normal exposure)

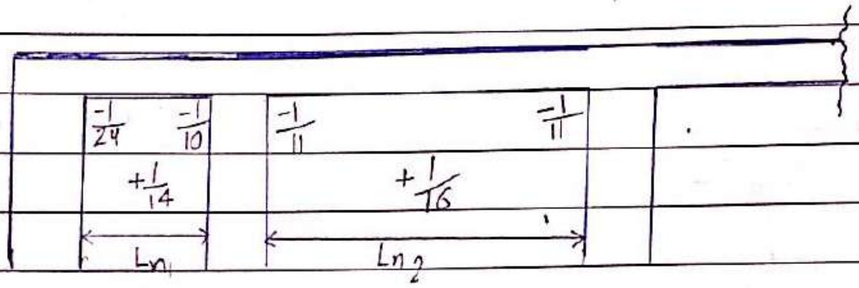
$A_s = \frac{M_u}{\phi f_y jd}$

j → 0.95 (compression flange)

ACI - Moment & shear Coefficients for Analysis & Design of Non-prestressed one-way Slabs & continuous beams.

$M_u = C_m (w_u L_n^2)$; $V_u = C_v (w_u L_n / 2)$

C_m



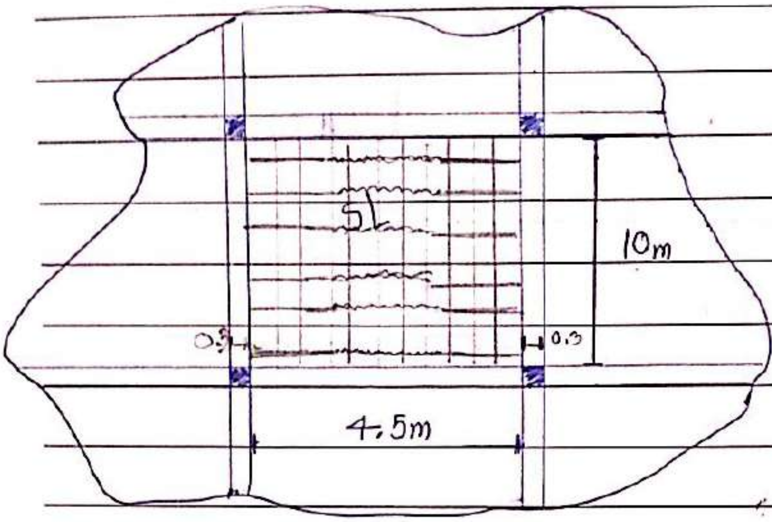
Use only if :

1. Two spans or more
2. Approximately Equal in length (Diff. ≤ 20%)
3. Uniformly distributed loading
4. $LL \le 3DL$
5. Prismatic Member

$A_{s\,min} =$
- ρ = 0.0018 bh ← Grade 60
- ρ = 0.002 bh ← Grade 40 or 50

$A_s = \rho b d$

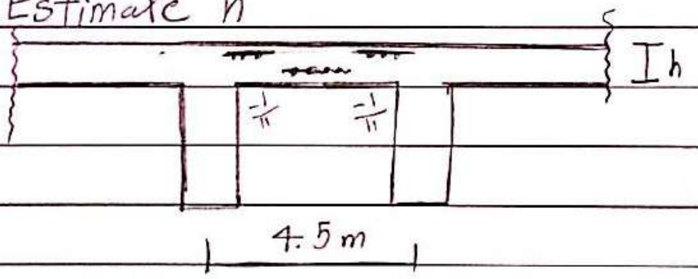
Example



$f'_c = 28$ MPa
$f_y = 414$ MPa
$W_{LL} = 4$ KN/m^2
$W_{DL} = 3$ KN/m^2
(excluding self wt.)
γ_c : if Not Given you will assume $= 24$ KN/m^3

- $\frac{10}{4} = 2.22 > 2 \longrightarrow$ one-way

- Estimate h



$h_{min} = L/28$
$= 4500/28$
$= 160$ mm
Try $h = 180$ mm ??

- compute factored Load W_u :

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

Self wt. $= 24 \frac{KN}{m^3} * 0.18$ ← (يتم ضربها بالمنطقة اللي لا رح نطبق Load عليها)

$= 4.32$ KN/m^2

$W_u = 1.2\,(3 + 4.32) + 1.6\,(4)$
$= 15.18$ KN/m^2

• compute M_u

$$M_u(-ve) = \frac{W_u L_n^2}{11}$$

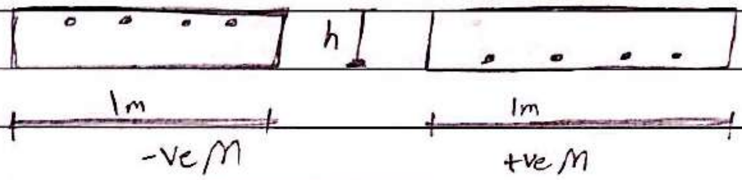
$\{ L_n = 4.5 - 2(0.15) = 4.2\,m$

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

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

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

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



Try No 16 M → $d = 180 - 20 - \frac{16}{2} = 152\ mm$ → (No. 16 M)

$M_u(-ve) = 24.34\ KN.m$

$$A_s = \frac{24.34 * 10^6}{0.9 * 414 * 0.95 * 152} = 452.4\ mm^2$$

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

$A_{s\,min} = 324\ mm^2$

$A_s > A_{s\,min}$ → ok ✓

في Slab
لا نحسب كم عدد
قضبان الحديد وإنما نحسب
عدد القضبان بمتر

$$a = \frac{452.4 \times 414}{0.85 \times 28 \times 1000} = 7.87\ mm$$

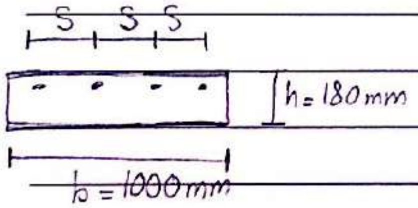
$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{a}{2}\right)} = \frac{24.34 \times 10^6}{0.9 \times 414 \left(152 - \frac{7.87}{2}\right)}$$

$A_s = 414.2\ mm^2$

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

$$A_s = \frac{24.34 \times 10^6}{0.9 \times 414 \times \left(152 - \frac{7.67}{2}\right)} = 440.9\ mm^2$$

$A_s = 440.9\ mm^2 \rightarrow a = 7.67\ mm \rightarrow \varepsilon_s = 0.0477 > 0.005$



$$S = \frac{1000\ A_b}{A_s}$$ → Area of 1 bar

1No. 16M, $A_b = 199 mm^2$

$S \rightarrow A_b$

$1000 \rightarrow A_s$

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

$\simeq 450\ mm$

27/3/2016

→ Per ACI code

S_{max} = Smaller of $\begin{cases} 3h = 540 mm \\ 450 mm \rightarrow ✓ \end{cases}$

- Use $S = 450 mm$

- $M_u (+ve) = 16.74$ kN.m

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

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

$A_s = 311 < A_{smin} \rightarrow$ Use A_{smin}

$A_s = A_{smin} = 324 mm^2$

لسنا بحاجة إلى عمل
check a
$A_s < A_{smax}$
$A_{smin} < A_{smax}$
∴ إذن مازلنا في
مرحلة
Tension

- $S = \frac{1000 * 199}{324}$

$= 614.2$ mm → يتم التقريب لرقم أقل
$\simeq 60$ cm لتسهيل التنفيذ
ولا أقربها للأكبر لأننا سنقلل
As ولكن لا استخدمها وذلك بسبب

→ $S_{max} = \begin{cases} 540 mm \\ 450 mm \rightarrow ✓ \end{cases}$

$S = 614.2 > S_{max} = 450 mm$

Use $S = 450 mm$

1 NO. 16 M @ 450 mm

Five Apple

Per ACI - Code:

Shrinkage & Temperature reinforcement is required perpendicular to the span of the Slab. (longer dimension)

$A_{s\,min}$ (m) ← كمية الحديد التي يجب وضعها للحماية من التشققات والتغير في درجات الحرارة

$A_{s\,min} = 0.0018\,bh = 324\,mm^2/m$

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

S_{max} = Smaller of $\begin{cases} 5h = 900\,mm \\ 450\,mm \rightarrow ✓ \end{cases}$

$S > S_{max} \Rightarrow$ Use $S = 450\,mm$

→ 1 No. 16 M @ 450 mm

• لتقليل تكلفة المشروع بالإمكان تغيير نوع الحديد ......

ولا يمكن وضع حديد 10

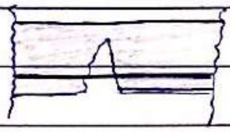
Five Apple

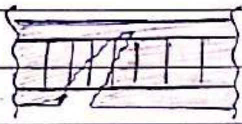
Shear in BeamS

29/3/2016

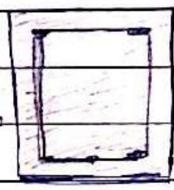
- bending Moment → تم تصميم As, h, b من خلاله

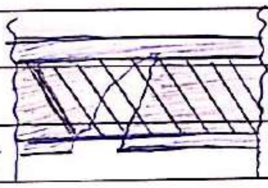
- Stirrumps
- shear reinforcement
- web reinforcement

Flexural crack  → Moment لوحده

Inclined vertical crak  → shear of and Moment

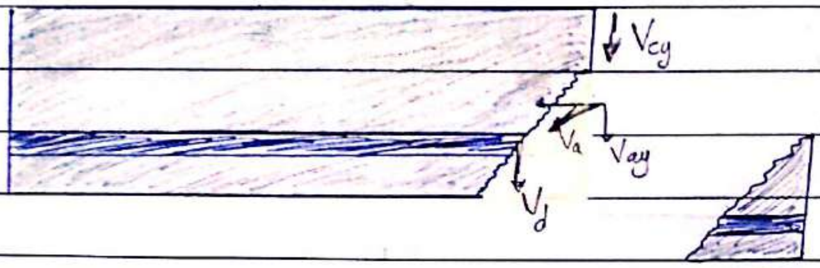
shear reinforcement

cross section  → أسهل من حيث التنفيذ لكن غير آمن كالنوع الثاني

Inclined shear reinforcement 

من الصعب وضع stirrumps على زاوية 45° على طول beam

Internal force in a Beam without Stirrups:



V_{cy} = Shear in the uncracked concrete section

V_a = Shear transfered across the crack by interlock of aggregate particles

V_d = Dowel action of longitudinal reinforcement

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

→ concret shear strength (Shear carried by the concrete)

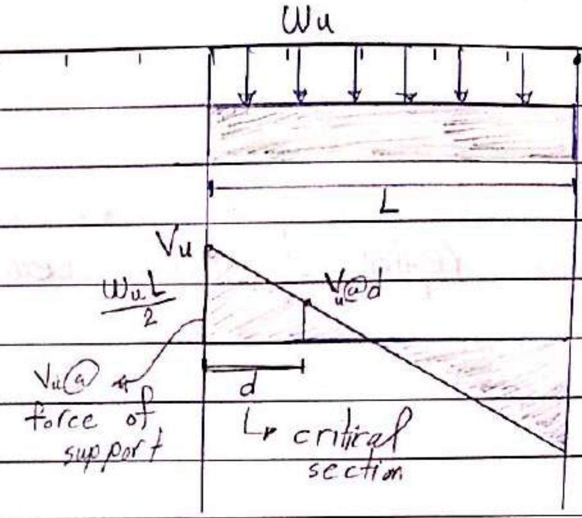
with Stirmps

V_s = Shear carried by the stirumps

$V_n = V_c + V_s$ → Nominal shear strength

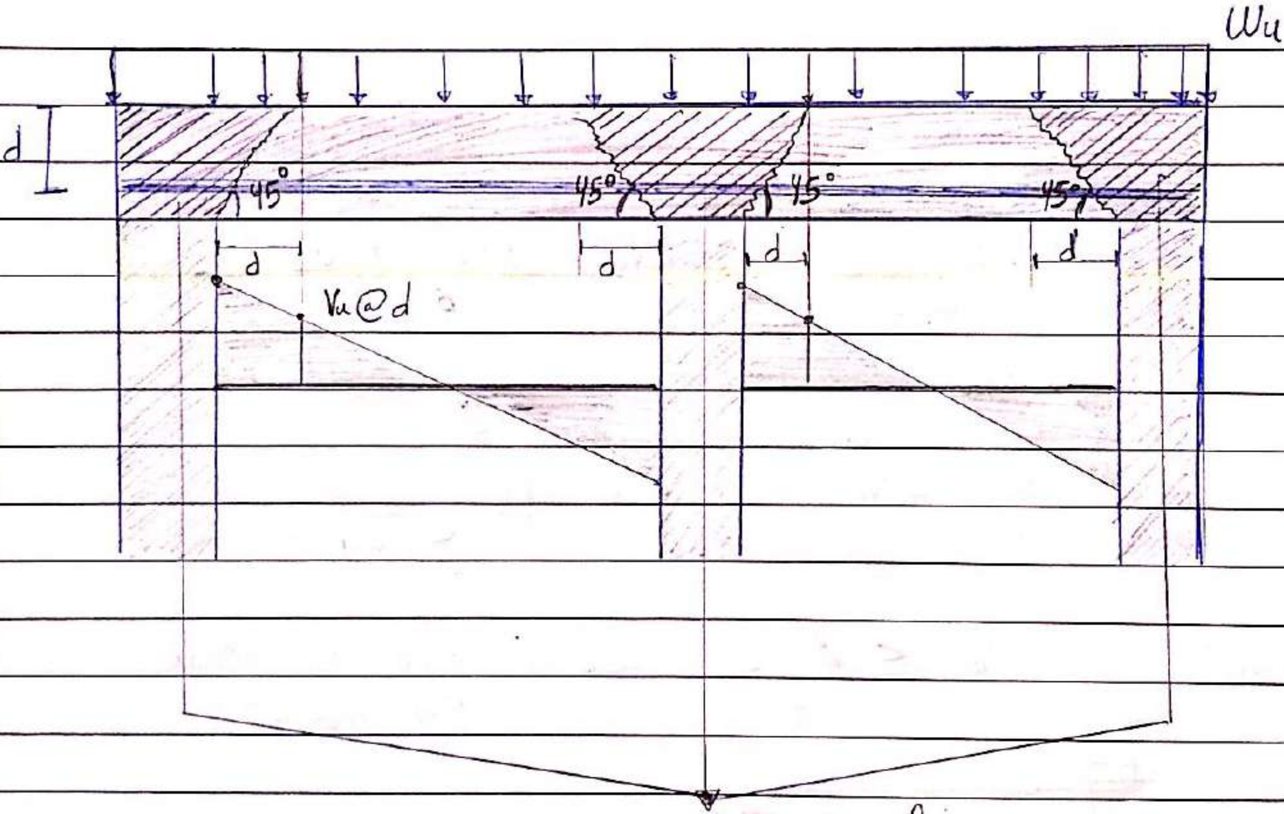
$\phi V_n \geq V_u$

ϕ = 0.75



- location of Max. Shear (V_u) for the design of Beams

Critical section



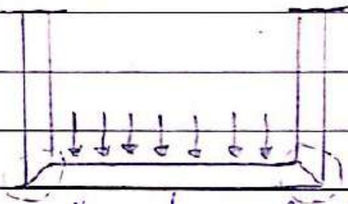
compression fan

تنتقل (Load) مباشرة
إلى (Support)

31/3/2016

- Use (V_u @ d) only if :

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



② loads are applied on the top of the beam

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

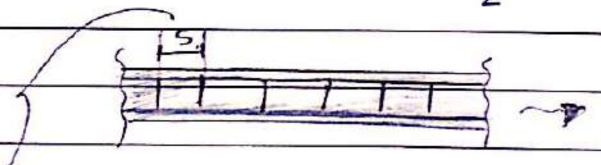
Analysis and design of R.C Beams for shear :

$\phi V_n \geq V_u$ → factored loads (@ d : V_u @ facesupport) (critical section)

$V_n = V_c + V_s$ → Strump

$\phi V_n = V_u$	$V_c = \dfrac{\sqrt{f_c'}}{6} b_w d$
$\phi (V_c + V_s) = V_u$	
$V_s = \dfrac{V_u}{\phi} - V_c$; 0.75 → ϕ	$V_s = \dfrac{A_v f_y d}{S}$; S → spaces

Case 1 : $V_u \leq \dfrac{\phi V_c}{2}$ → No need for shear reinforcement



→ يتم وضع (Strump)
لتثبيت الجزء العلوي من الحديد

يتم اختيارها حسب diameter الـ sted

(كلما زاد diameter الحديد زاد صعوبة ثني deflection زادت S)

Five Apple

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

$\Rightarrow$ Min. shear reinforcement

S_{max} = Smaller $\begin{cases} \dfrac{16 A_v f_y}{\sqrt{f_c'}\ b_w} & \text{--- (1)} \\ \dfrac{A_v f_y}{0.33\ b_w} & \text{--- (2)} \\ d/2 & \text{--- (3)} \\ 600\ mm & \text{--- (4)} \end{cases}$

$\boxed{\text{No. 10 M}}$

$A_v = 2A_{v_1}$

A_{v_1} = crosssectional area of stirups , $A_{v_1} = \frac{\pi}{4}(10)^2$

$\rightarrow \boxed{\text{1 No. 10 M}}$

Case III : $V_u > \phi V_c$ & $V_s \le \frac{2}{3}\sqrt{f_c'}\ b_w d \rightarrow 4V_c$

S_{max} = smaller of $\begin{cases} \dfrac{16 A_v f_y}{\sqrt{f_c'}\ b_w} \\ \dfrac{A_v f_y}{0.33\ b_w} \\ \dfrac{A_v f_y d}{V_s} \\ \left.\begin{matrix} d/2 \\ 600\ mm \end{matrix}\right\} \rightarrow V_s \le \frac{1}{3}\sqrt{f_c'}\ b_w d \quad 2V_c \\ \left.\begin{matrix} d/4 \\ 300\ mm \end{matrix}\right\} \rightarrow V_s > \frac{1}{3}\sqrt{f_c'}\ b_w d \quad 2V_c \end{cases}$

{ A_v : twice cross section from table

- $V_s > 4V_c \rightarrow$ Enlarge the section

$V_{s\,max} = 4V_c$

- $V_{u\,max} = \phi\,(V_c + V_{s\,max})$, where $V_{s\,max} \rightarrow 4V_c$

$$V_{u\,max} = \phi\, 5\, V_c$$

$V_u < V_{u\,max} \Longrightarrow$ ok ✓

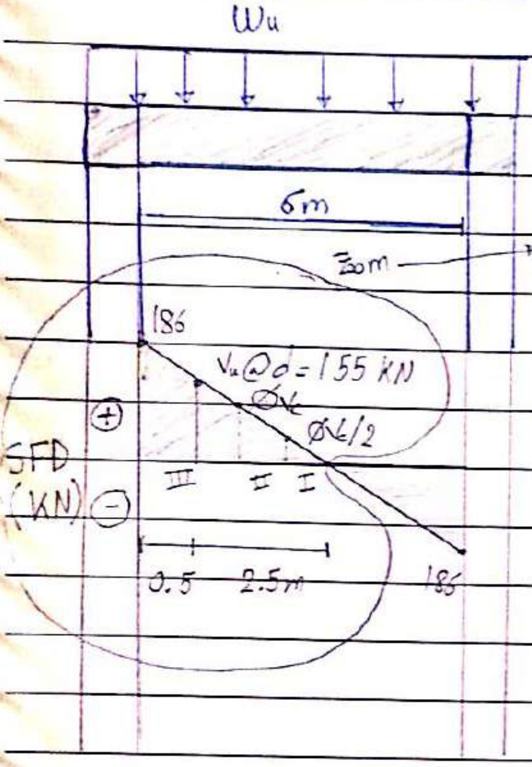
$V_u > V_{u\,max} \Longrightarrow \underbrace{\text{Enlarge the section}}_{\text{نزود section}}$

3/4/2016

Example Design for shear

$V_u = \phi V_n$

$V_u = \phi (V_c + V_s)$

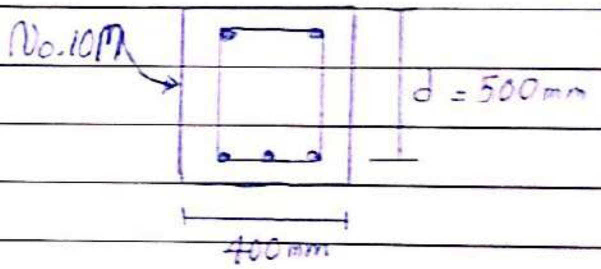


LL = 20 KN/m

DL = 25 KN/m (Includes)

$f_c' = 28$ MPa

$f_y = 300$ MPa



$W_u = 1.2 DL + 1.6 LL$

$= 1.2 * 25 + 1.6 * 20$

$= 62$ KN/m

$\frac{W_u L}{2} = \frac{62 * 6}{2} = 186$ KN

. $\frac{V_u @ d}{(2.5\text{m})} = \frac{186}{3} \rightarrow V_u @ d = 155$ KN

. $V_{u\,max} = \phi\, 5 V_c$

$V_c = \frac{\sqrt{28}}{6} * 400 * 500 = 176.4$ KN

$V_{u\,max} = 0.75 * 5 * 176.4 = 661.4$ KN

. $V_u @ d < V_{u\,max}$ → section is ok

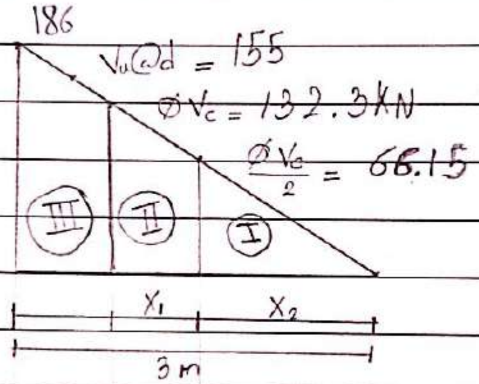
. كلما قل (shear) زادت المسافات بين (strump)

بالتالي قل (عددها)

$\phi V_c = 0.75 * 176.4$

$= 132.3\ KN$

$\dfrac{\phi V_c}{2} = \dfrac{132.3}{2} = 66.15\ KN$



$\dfrac{186}{3} = \dfrac{132.3}{X_1} = \dfrac{66.15}{X_2}$

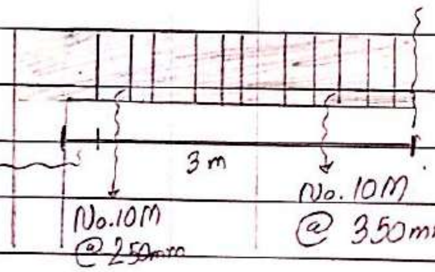
$X_1 = 2.13 m$

$X_2 = 1.067 m$

أول خانة

10 cm

OR S/2



فصّلنا الثلاث مناطق

وذلك للـ (Economic)

Case I : $V_u \le (\phi V_c/2)$

⟹ No need for shear reinforcement

$S = 350 mm$ (??)

Case II : $(\phi V_c/2) < V_u \le \phi V_c$

Min. shear reinforcement

$S = 250 mm$

Case III : $(V_u > \phi V_c)$ & $(V_s \le 4 V_c)$

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

$= \dfrac{155}{0.75} - 176.4 = 30.3\ KN. < 2 V_c$

S_{max} = Smaller of

$\frac{16 A_v f_y}{\sqrt{f_c'}\, b_w} = 356\ mm$, $A_v = 157\ mm$

$\frac{A_v f_y}{0.33 b_w} = 356.8\ mm$

$d/2 = 500/2 = 250\ mm$ ← use this

$600\ mm$

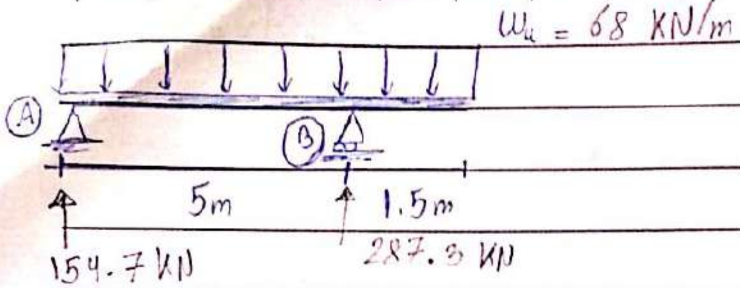
$\frac{A_v f_y d}{V_s} = 777.2\ mm$

$d/4$ X

$300\ mm$ X

5/4/2016

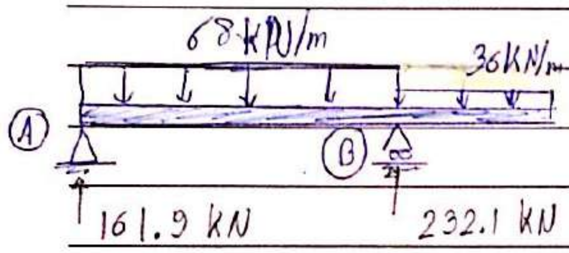
Cases of Loaded :

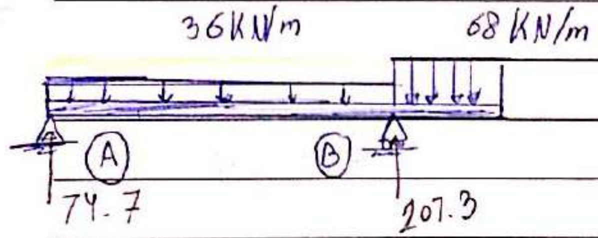


LL = 20 KN/m → 1.6 * 20 = 32 KN/m

DL = 30 KN/m → 1.2 * 30 = 36 KN/m

$\overline{68 \text{ KN/m}} = W_u$

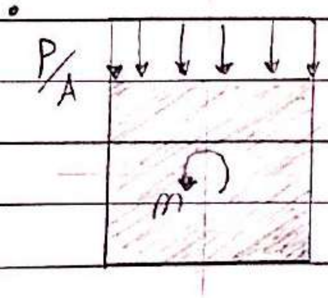




تم اختيار اعلى
reaction
من 3 cases
ولسنا بحاجة لل
check for
Shear
??

$R_A = 161.9$ KN

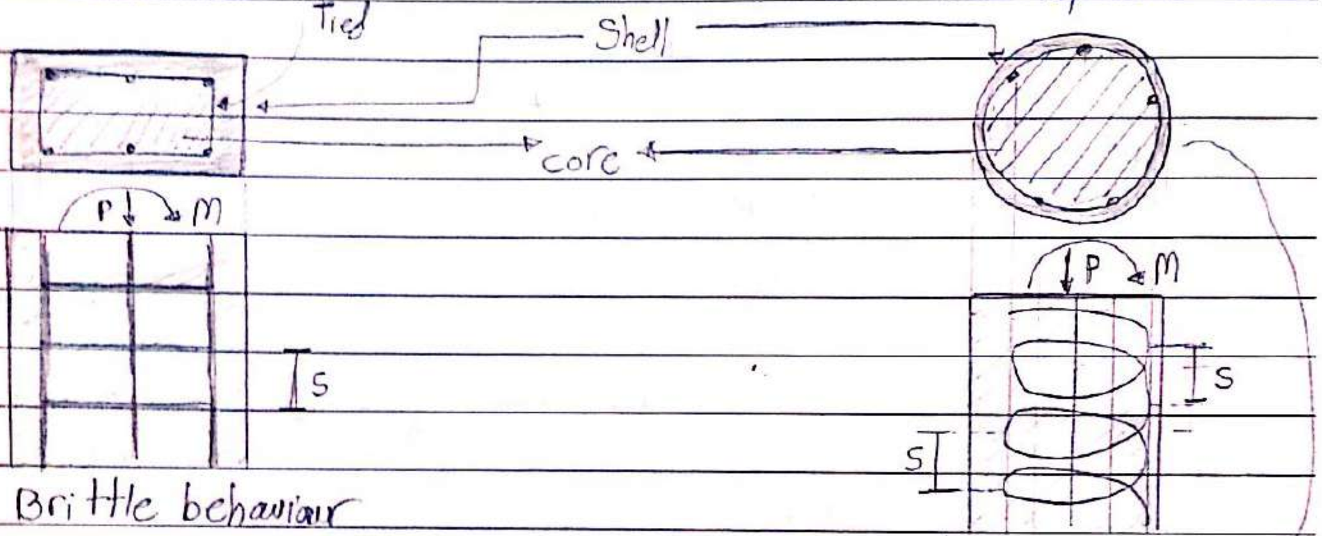
$R_B = 287.3$ KN



$$\sigma_{\substack{axial\\stress}} = \frac{P}{A} + \frac{My}{I} \text{ (bending)}$$

Columns : combined Axial Load & bending

Tied | Spiral



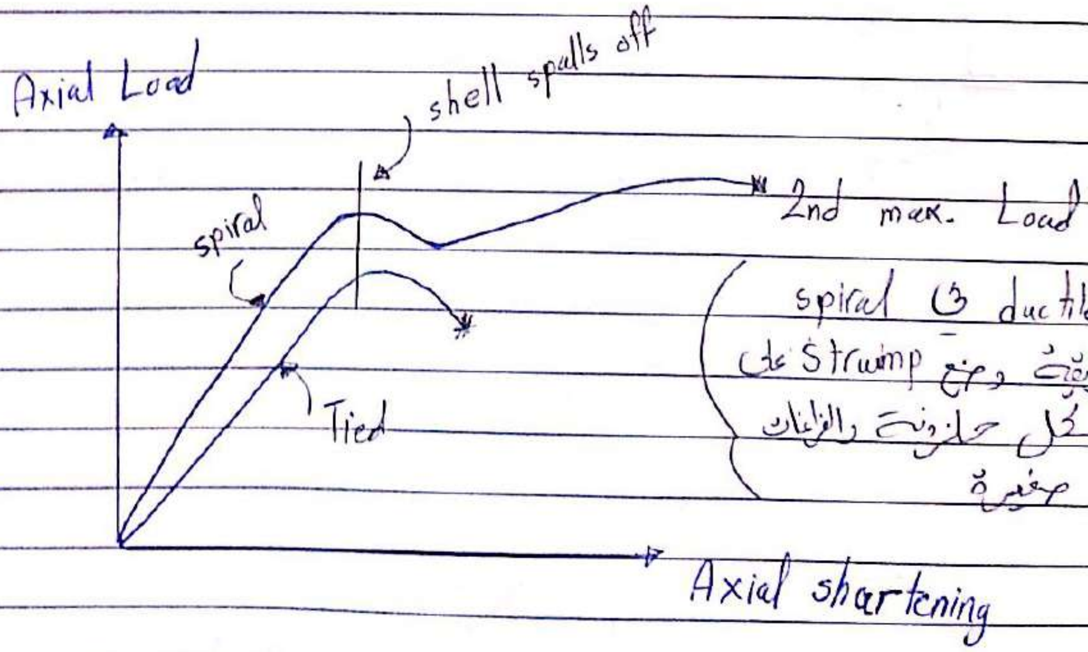
Brittle behaviour

Ductility behaviour

use In earthquecke area and in load high

More confinement for the core

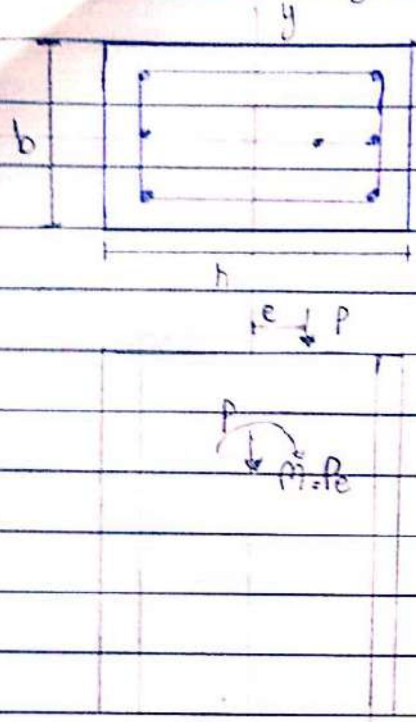
core : المنطقة الداخلية
shell : المنطقة الخارجية



سبب ان spiral ductile
هو طريقة وضع Struimp على
شكل حلزونية والفراغات
صغيرة

7/4/2016

Interaction Diagrams



X

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

f_{cu} → compressive strength

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

If $M = 0.0$, $\frac{P}{A f_{cu}} = 1.0$

$P_{max} = A f_{cu}$ ⟶ $f_{cu} = \frac{P_{max}}{A}$

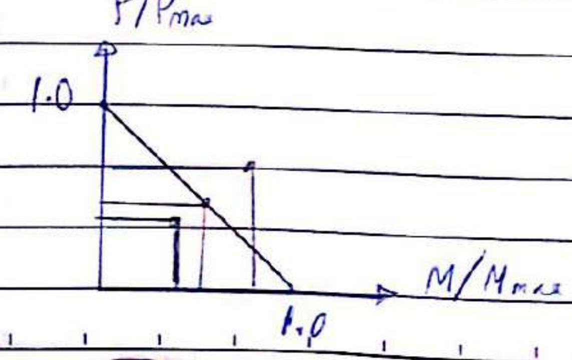
↳ The max. axial Load the column on support

. If $P = 0.0$, $\frac{My}{I f_{cu}} = 1.0$

$M_{max} = \frac{I f_{cu}}{y}$ ⟶ $f_{cu} = \frac{M_{max}\, y}{I}$

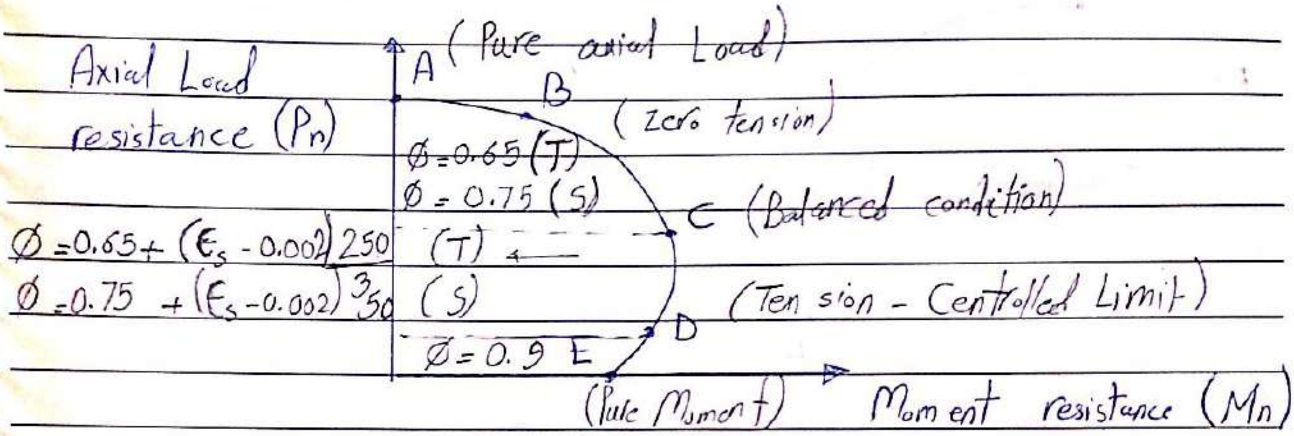
↳ The max. moment the column can support

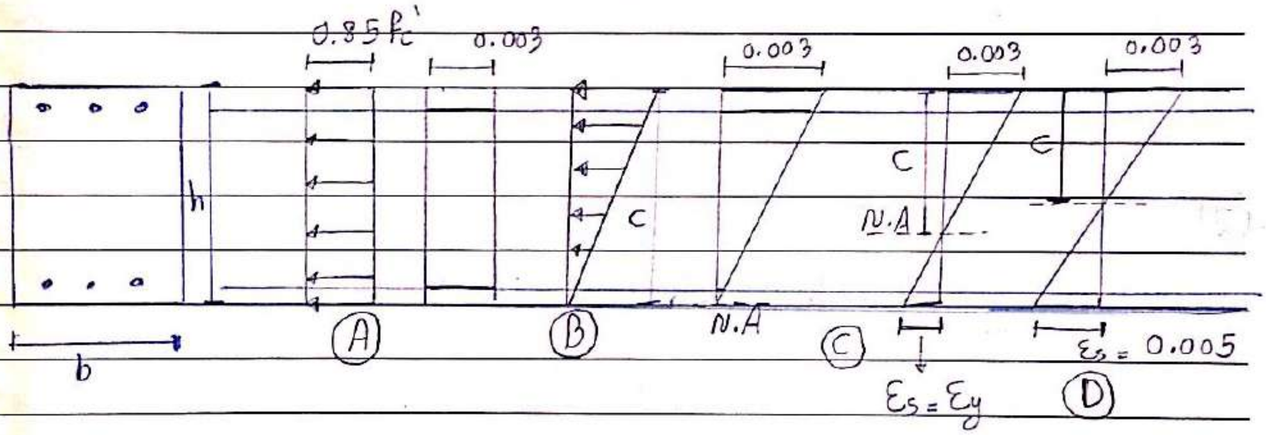
$$\frac{P}{P_{max}} + \frac{M}{M_{max}} = 1.0$$ Interaction Eqn.



78

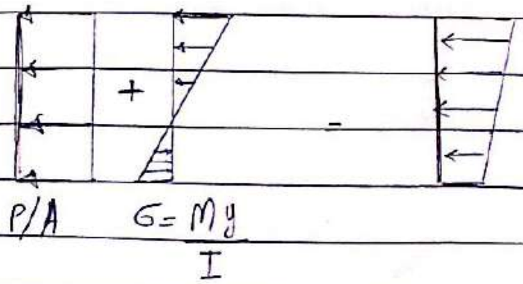
Interaction Diagram for concrete columns:





$$P_n = 0.85 f_c' * (A_g - A_s) + A_s F_y \rightarrow$$ طريقة point A

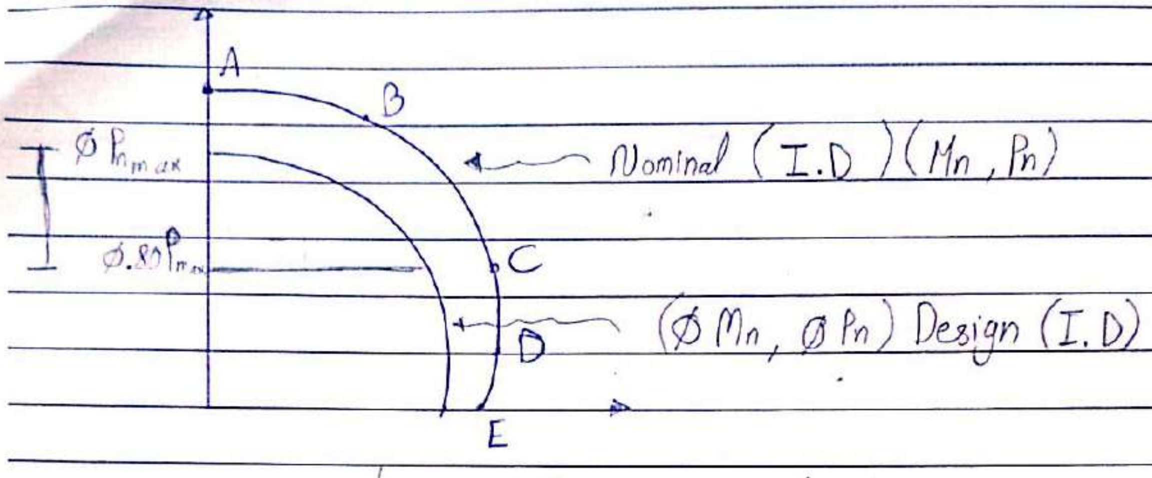
A_g → Gross cross sectional area ($b*h$)



• عند النقطة E
يرتفع N.A لفوق
$\varepsilon_s > 0.005$

Five Apple

10/4/2016



accept داخل الخط reject خارج الخط

D → 0.9

الحالات قبل

C → $\phi = 0.65$, $\phi = 0.75$

أو → حلى الحالات

Max. Axial Load

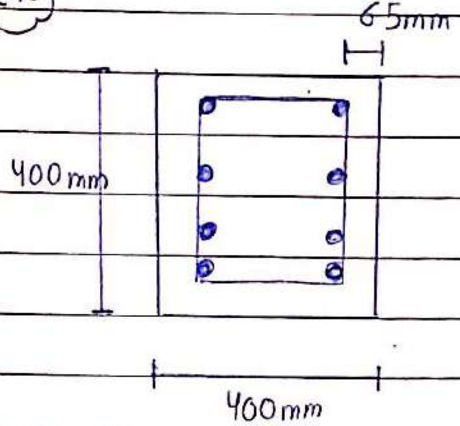
$\phi P_{n\,max} = \phi\,(0.85 f'_c\,(A_g - A_s) + A_s f_y)$

- To account for the effect of accidental moments, the ACI-code specifies that; the maximum Load
  - → on a spiral column must not exceed 85% of its strength ($\phi P_{n\,max}$) — $P_u \le 85\%\, P_{n\,max}$
  - → on a tied column must not exceed 80% of its strength ($\phi P_{n\,max}$) — $P_u \le 80\%\, \phi P_{n\,max}$

12/4/2016

Calculation of Interaction Diagram

Ex



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

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

Point A : Pure Axial Load

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

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

$P_n = 6773.69\ Kn$

$M_n = 0.0$

Point B : zero tension

$C = 400\ mm$, $a = \beta_1 C$, $\beta_1 = 1.09 - (0.008 * 35)$??

$\beta_1 = 0.81$

$a = 0.81 * 400$

$= 324\ mm$

$\frac{0.003}{400} = \frac{\varepsilon_{s1}}{65} = \frac{\varepsilon_{s2}}{400-65} \Rightarrow \varepsilon_{s1} = 4.875 * 10^{-4} < \varepsilon_y$

$f_{s1} \neq f_y$

$f_{s1} = 4.875 * 10^{-4} * 200000$

$= 97.5\ MPa$

$\varepsilon_{s2} = 0.00251 > \varepsilon_y$

$f_{s2} = f_y$

$C_c = 0.85 * 35 * 324 * 400 = 3855.6$ Kn

$\rightarrow C_c = 0.85 * 35 \, (324 - 400 - 2580)$

$C_{s1} = 2580 * 97.5 = 251.55$ kn

$C_{s2} = 2580 * (420 - 0.85 * 35) = 1006.8$ Kn

$P_n = C_c + C_{s1} + C_{s2} = 5114$ Kn

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

$= 3855.6 \left(200 - \frac{324}{2}\right) - 251.55 \, (200 - 65) + 1006.8 \, (200 - 65) = 248.5$ KN.m

Point C Balanced condition

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

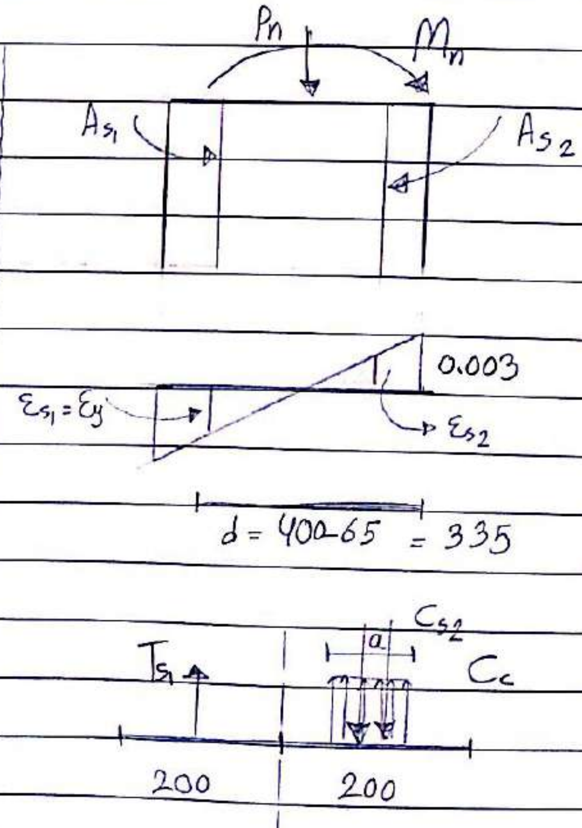
$\varepsilon_{s1} = 0.0021 = \varepsilon_y$

$c = 197$ mm

$a = 0.81 * 197 = 159.57$ mm

$\frac{\varepsilon_{s2}}{c - 65} = \frac{0.003}{c}$

$\varepsilon_{s2} = 0.00201$



Five Apple

$f_{s_2} = 0.00201 * 200000$
$= 402$ MPa

$C_c = 0.85 f_c' ab = 0.85 * 35 * 159.57 * 400$
$= 1899$ Kn

$C_{s_2} = 2580 (402 - 0.85 * 35) = 960.4$ Kn

$T_{s_1} = 2580 * 420 = 1083.6$ Kn

$P_n = C_c + C_{s_2} - T_{s_1} = 1775.8$ Kn

$M_n = \Sigma M_{\mathbb{C}}$

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

$+ 1083.6 (200 - 65) = 504$ Kn.m

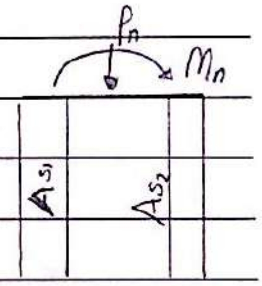
Point D   Tension Controlled Limit

$\frac{\varepsilon_{s1}}{335 - C} = \frac{0.003}{C}$, $\varepsilon_{s_1} = 0.005$

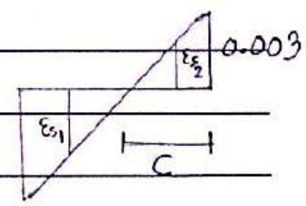
$C = 125.6$ mm

$a = 0.81 * 125.6 = 101.75$ mm

A_{s_2} (within a)

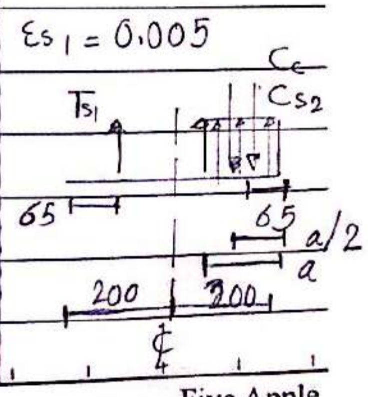


$\frac{\varepsilon_{s_2}}{C - 65} = \frac{0.003}{C} \rightarrow \varepsilon_{s_2} = 0.001447 < \varepsilon_y$



$f_{s_2} = 0.001447 * 200000$
$= 289.5$ MPa

$\varepsilon_{s_1} = 0.005$



$C_c = 0.85 * 35 * 101.75 * 400 = 1210.8$ Kn

$C_{s_2} = 2580 * (289.5 - 35 * 0.85) = 670.16$ Kn

$T_{s_1} = 2580 * 420 = 1083.6$ Kn

$P_n = C_c + C_{s_2} - T_{s_1} = 797.36$ Kn

$M_n = \sum M_{\text{¢}}$

$= 1210.8\left(200 - \frac{101.75}{2}\right) + 670.16\,(200 - 65)$

$+ 1083.6\,(200 - 65) = 417.32\ KN.m$

Point E Pure Bending (Beams)

Beam كأنه عادي

Ignore comp. reinforcement $(A_{s_2}) = 0.0$

$C = T$

$0.85 * 35 * a * 400 = 2580 * 420$

$a = 91.05\ mm$

$C = 91.05 / 0.81 = 112.41\ mm$

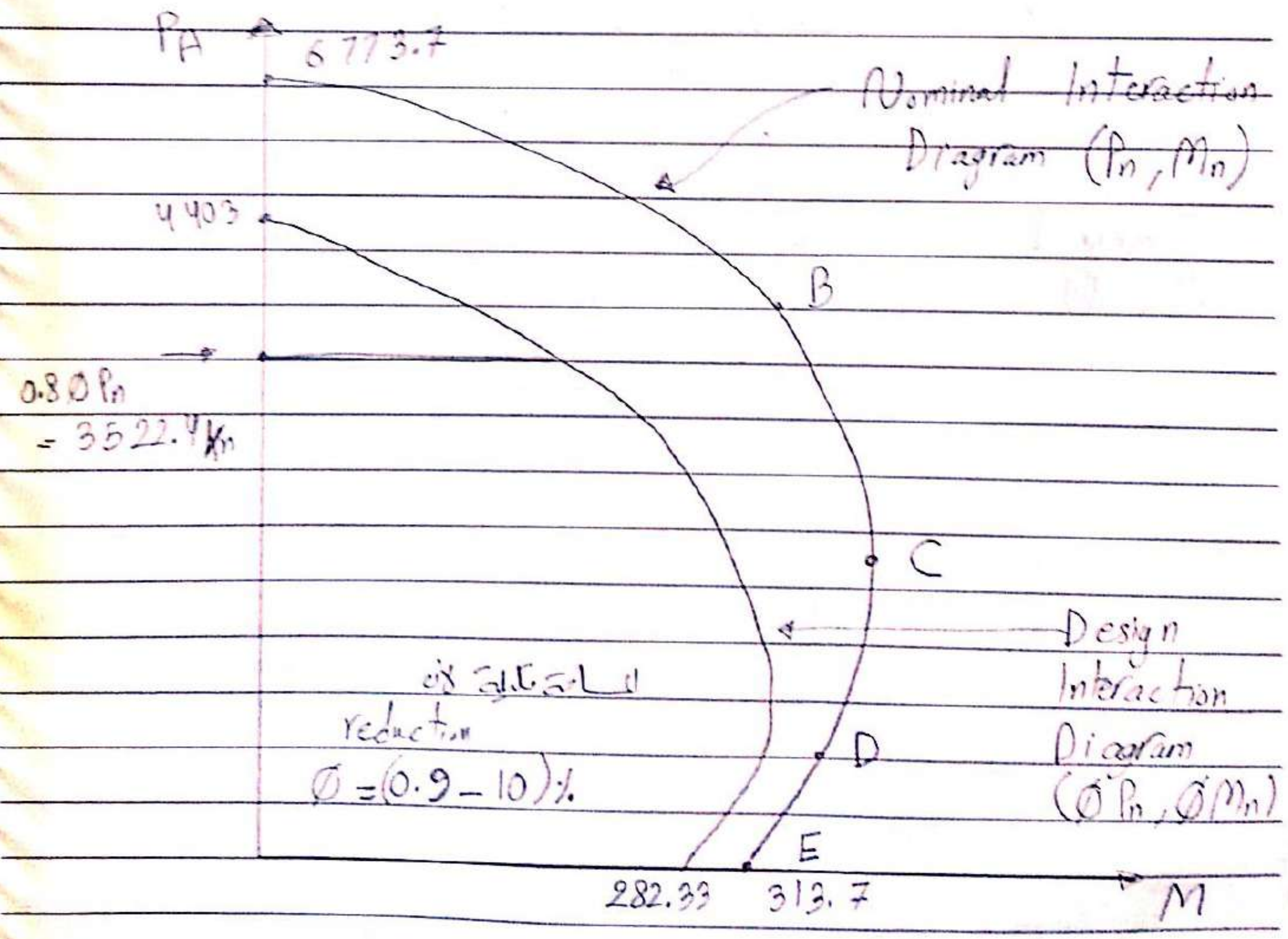
$\varepsilon_s = 0.003\left(\frac{335 - 112.41}{112.41}\right) = 0.00594 > \varepsilon_y$

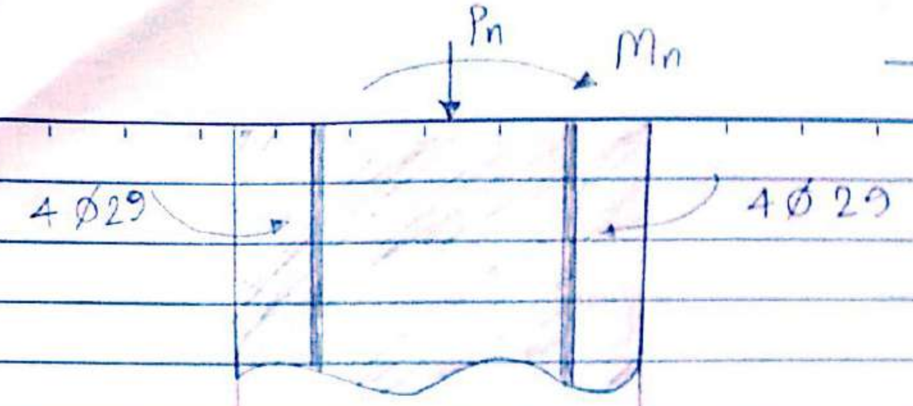
$M_n = A_{s_1} f_y \left(d - \frac{a}{2}\right)$

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

$P_n = 0.0$

Point	ε_t	ϕ	P_n (Kn)	M_n (Kn.m)	ϕP_n (Kn)	ϕM_n (Kn.m)
A	—	0.65	6773.7	0.0	4403	0.0
B	0.000485	0.65	5114	248.5	3324.1	161.5
C	0.0021	0.65	1775.8	504	1154.3	327.7
		C.C				
D	0.005	0.9	797.96	417.32	717.62	375.59
E	0.00594	0.9	0.0	313.67	0.0	282.3
		T.C				

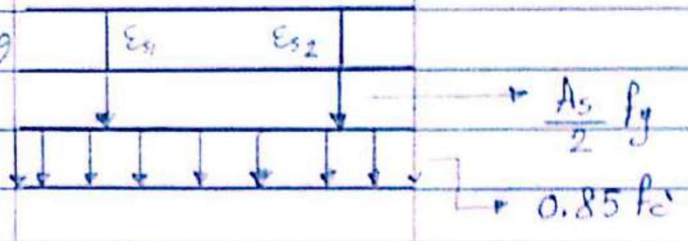




Point A

$P_n = 6773.69$

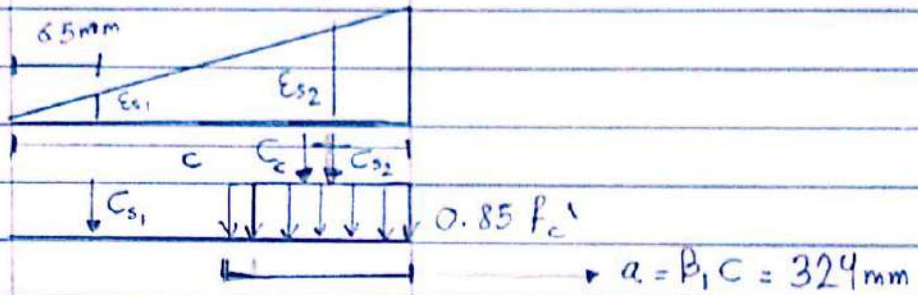
$M_n = 0.0$



Point B

$P_n = 5114$ Kn

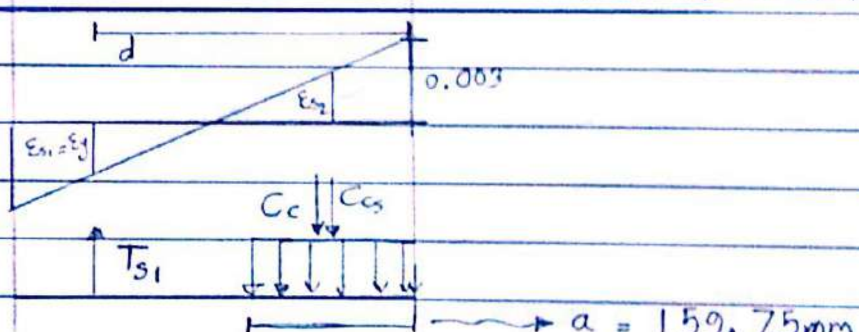
$M_n = 248.5$ Kn.m



Point C

$P_n = 1775.8$ Kn

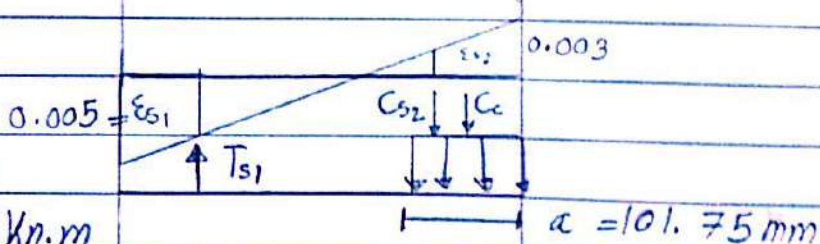
$M_n = 504$ Kn.m



Point D

$P_n = 797.96$ Kn

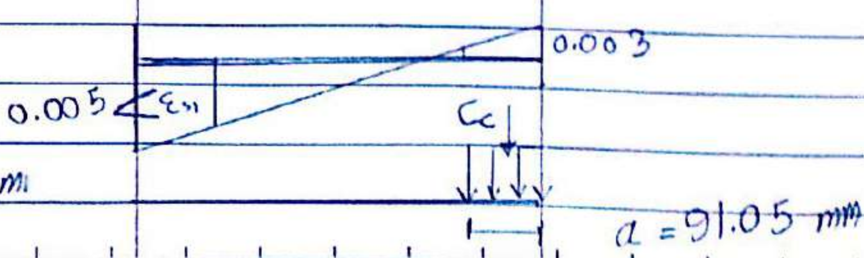
$M_n = 417.32$ Kn.m



Point E

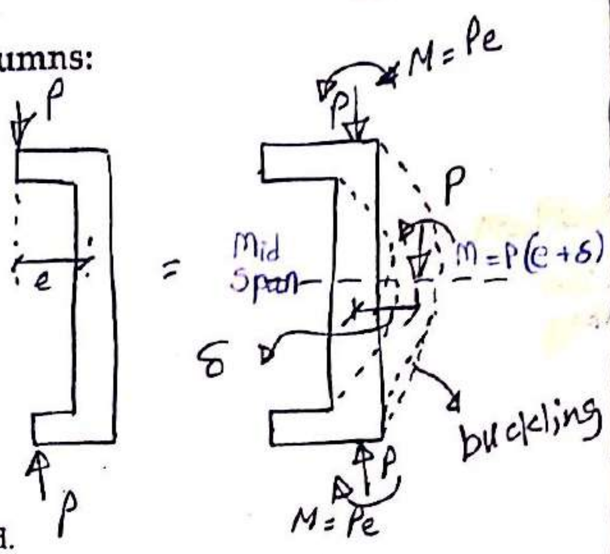
$P_n = 0.0$

$M_n = 313.67$ Kn.m



(14/4/2016)

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.

$\rho_s = \frac{A_s}{k r}$

-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

$$A_g(\text{trial}) \ge \frac{P_u}{0.4(f'_c + \rho f_y)}$$

b = h

-For spiral columns

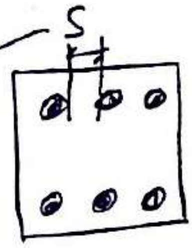
$$A_g(\text{trial}) \ge \frac{P_u}{0.5(f'_c + \rho f_y)}$$

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 $\phi P_n, \phi M_n$ 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 ≥

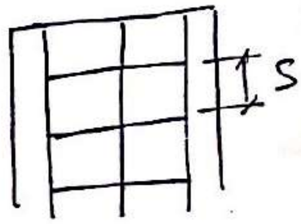


Larger of
- 1.5 db
- 1.5 in (40mm)
- 1⅓ Max. size of course Agg.

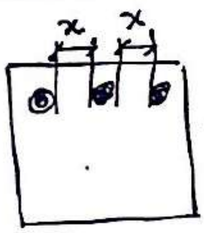
Clear distance limitation also apply to lap-spliced bars.

Spacing for Ties:

S_{max} shall not exceed (smaller of)
- 16 db
- 48 d
- 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.



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.

Spacing spiral

The max. spacing that will result in a ------ max. loud that equal or exceed the initial max. loud.

① $S_{max} \leq \dfrac{\pi d_{sp}^2 f_y}{0.45 D_c f_c' (A_g/A_c - 1)}$

d_{sp} = diam. of spiral
D_c = diam. of cone (out to out of the spiral)
A_{ch} = area of cone $= \frac{\pi}{4} D_c^2$

② ACI-code (max. spacing)
$S_{max} \leq 3in$ (75 mm)
to confine the cone ettenimetry

③ to avoid proplemy in concrete placing

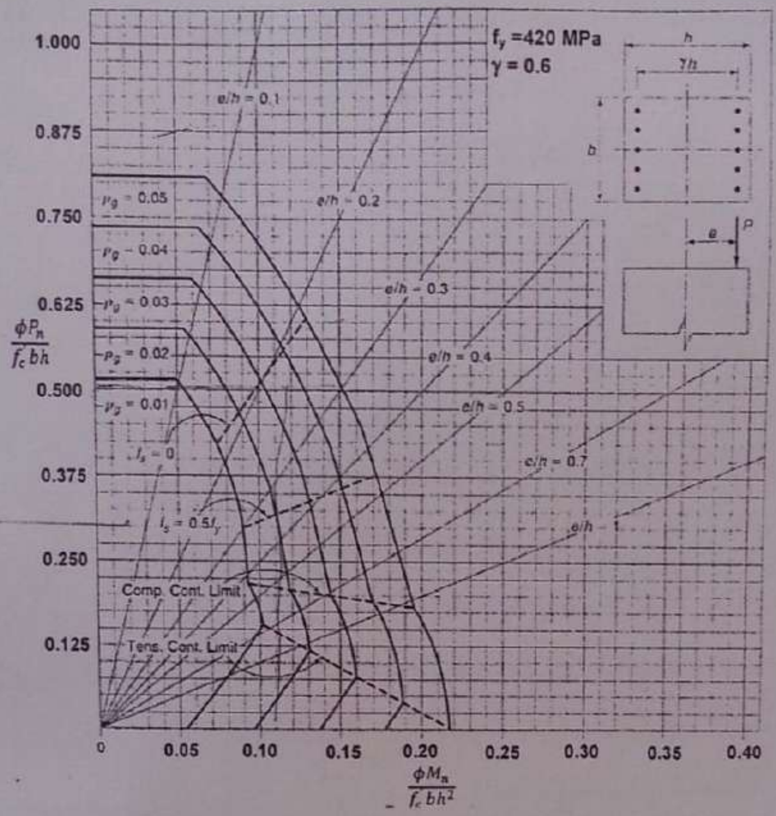
S_{min} = larger of $\begin{cases} 1 \text{ in } (25 \text{ mm}) \\ 1\frac{1}{3} \text{ C.A size} \end{cases}$

- For Analysis
  Given f_c', f_y, b, h, γ
  $A_s \rightarrow \rho_t$
  Find P_u, M_u ?

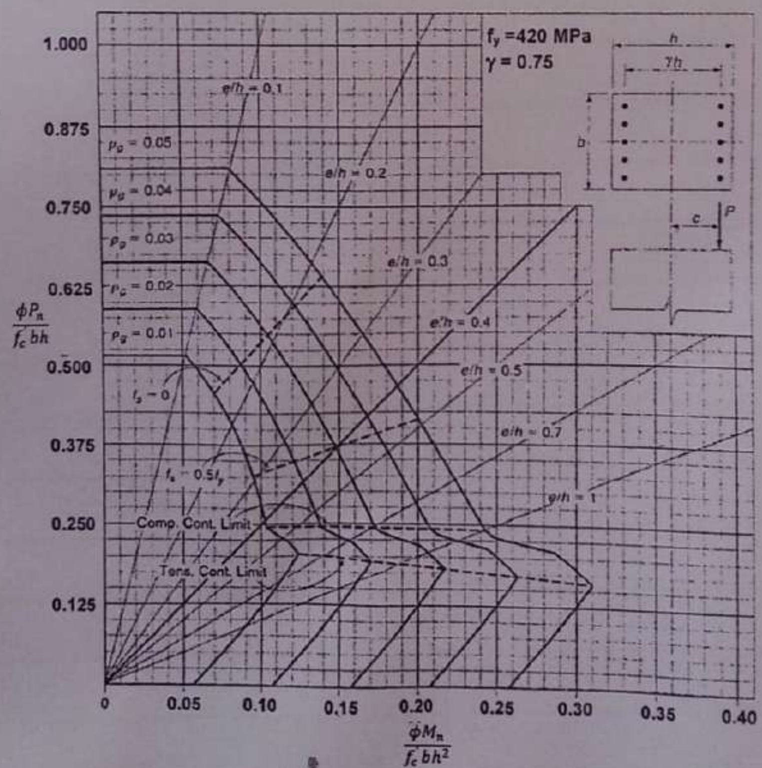
- e_{min} (min eccentricity) = 0.1 from largest dim
  = 0.1 h

$M_u = P.e = 0.1\, h\, P_u$

Rectangular Column Interaction Diagrams



النقطة ما حد الأكبر



17/4/2016

Design of a tied column

Ex

$P_u = 1550\ Kn$

$M_u = 150\ Kn.m$

$f_c' = 20\ MPa$

$f_y = 420\ MPa$

$$A_g \geq \frac{P_u}{0.4(f_c' + \rho f_y)} \quad ; \quad \rho = 0.015$$

$$= \frac{1550}{0.4(20 + 0.015 * 420)} = 147338\ mm^2$$

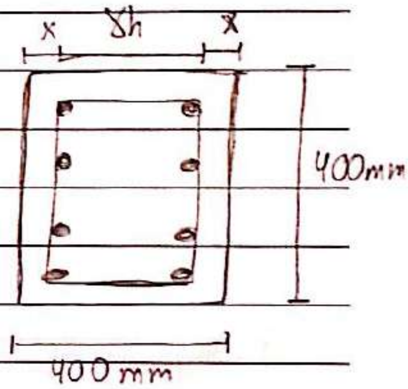
$A_g = bh \longrightarrow$ square col $b = h$

$b = h = 384\ mm$

use $400 * 400\ mm$

use nb. 25 M

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



$2(62.5) + \gamma(400) = 400$

$\gamma = 0.69$

$$\frac{\phi P_n}{f_c' bh} = \frac{1550 * 10^3}{20 * 400 * 400} = 0.484$$

$$\frac{\phi M_n}{f_c' bh^2} = \frac{150 * 10^6}{20 * 400 * 400^2} = 0.117$$

I.D, $\gamma = 0.6 \longrightarrow \rho = 0.0333$

I.D, $\gamma = 0.69 \longrightarrow \rho = 0.03$

I.D, $\gamma = 0.75 \longrightarrow \rho = 0.028$

ρ_{max}

- If $\rho_{computed}$ (I.D) exceeds (4%) → larger section should be chosen

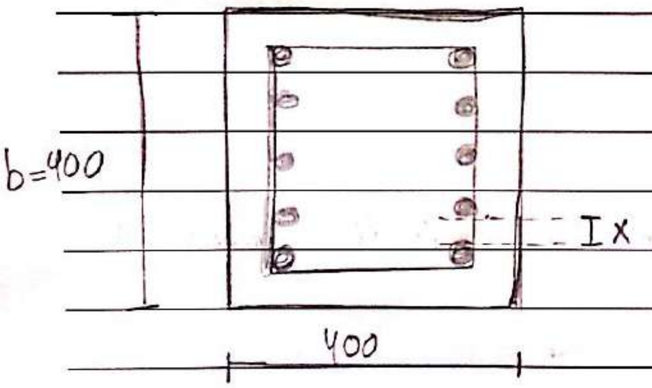
- If $\rho_{computed}$ (I.D) use then (0.01) → ρ_{min}

Use $\rho = 1\%$

$A_s = b h \rho = 0.03 * 400 * 400$
$= 4800\ mm^2$

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

• لازم يكون عندي أربع قضبان على أربع زوايا لذلك إذا (11) ⟵ خليهم (12 أو 10)



Check spacing between bars?

$S \geq 150 \rightarrow$ double tied
$S < 150\ mm \rightarrow$ single tied

??

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

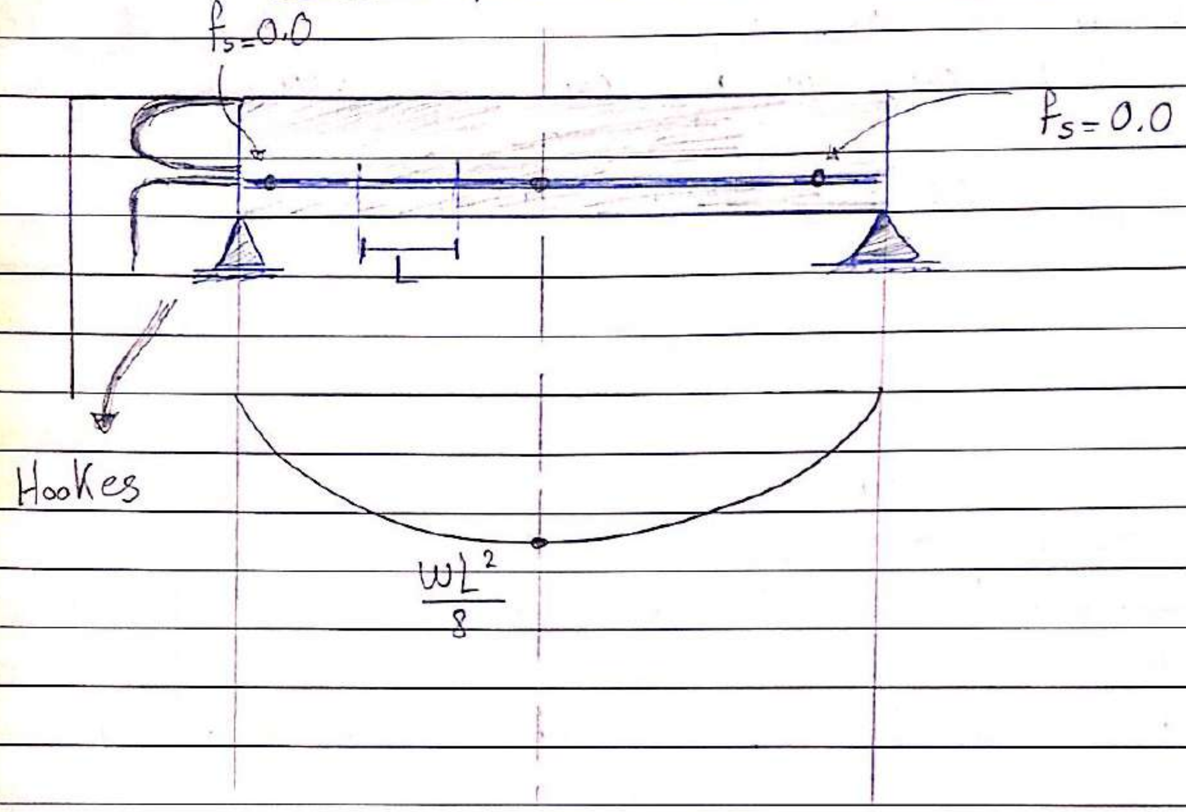
$= (2d_s - 0.5 d_b)$

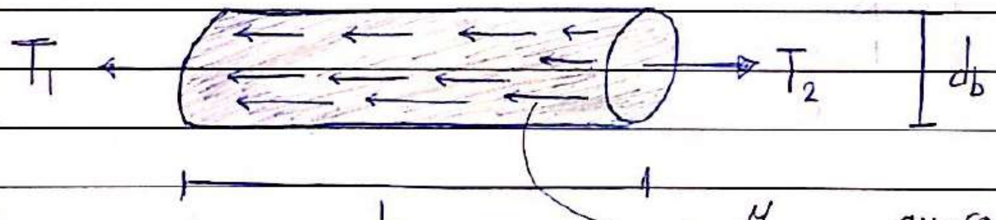
$S_{min} = 40\ mm$

• $X = \dfrac{400 - (2 * 40 + 2 * 10 + 5 * 25)}{4}$

$X = 40\ mm \geq S_{min} \Rightarrow$ OK.
$< 150\ mm$

Development of reinforcement





μ_{avg} = average bond stress

تتواجد هذه القوى لعدم تساوي T_1 و T_2 ، لكن μ_{avg}

$T_1 + \mu_{avg} (\pi d_b) L = T_2$; L: development Length (L_d)

الطول الكافي ليعطيني
قوة رابطة تحقق لي
المساواة لـ T_1 مع T_2

- The development length (L_d): is the shortest length of the barr in wich the bar stress can increase from zero to f_y.

19/4/2016

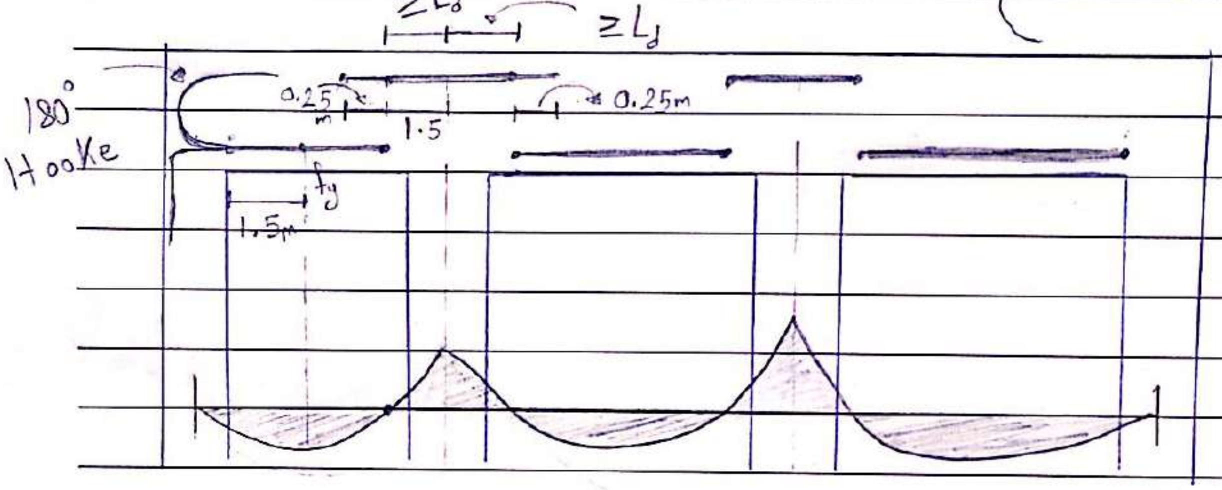
• If the distance from a point where the bars stress equals f_y to the end of the bar is Less than L_d ; the bar will pull out of the concrete.

→ Bond strese is not enough to produce Eqpm.

$f_y = 0.0$ → عند أطراف الحديد

$f_y\ max$ → عند M_{max}

تم حساب L_d كما هو معطاة في sheet 3



• L_d تحدد ما إذا كنت بحاجة لاستخدام (Standard Hookes)

• $L_d = 2m$

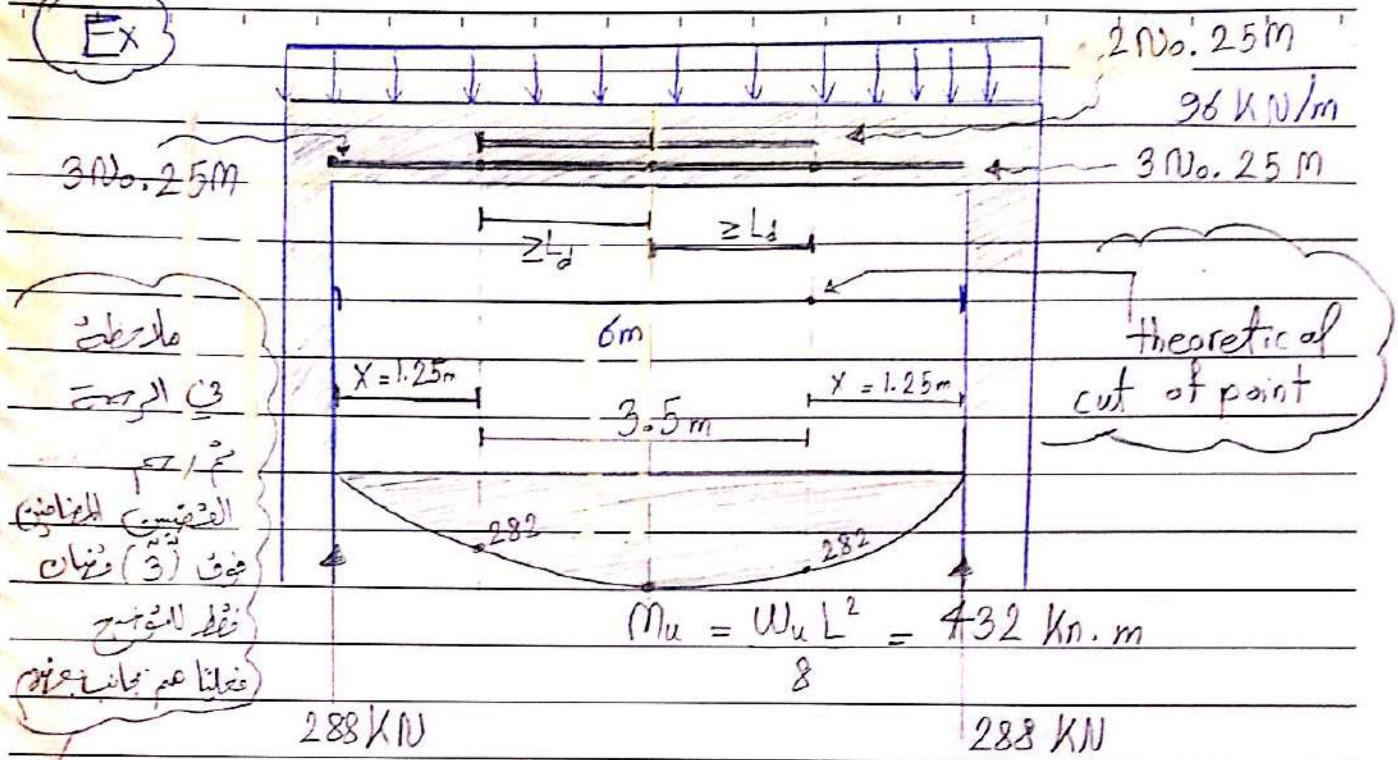
إذا كانت قيمة L_d تساوي (2m) وهي أكبر من نصف طول التسليح العلوي إذن نمد 0.5 للتسليح وذلك بامتداده (.25 m) من كلا الطرفين حتى تساوي $\left(\frac{L}{2} = L_d\right)$

$L_d = 2.5$

• إذا كانت قيمة L_d للتسليح السفلي تساوي 2.5 وهي أكبر من قيمة $\left(\frac{L}{2}\right)$ إذن نلجأ لـ (Hookes)

ويتم تحديد Hookes المراد حسب اختيار المهندس (90° or 180°) ومن جداول (sheet) 7.21

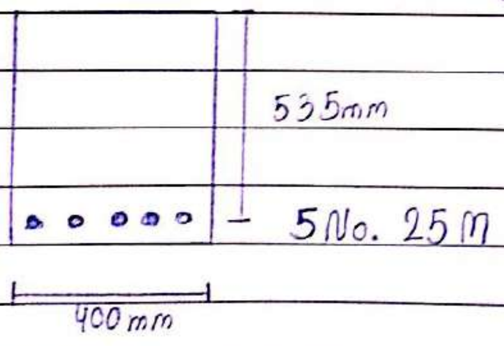
Ex



$$M_u = \frac{W_u L^2}{8} = 432 \ Kn.m$$

• ثم إيجاد (Reaction) حسب معادلات (Static)
وكانت النتيجة على الطرفين (288 Kn)

• ثم إيجاد design



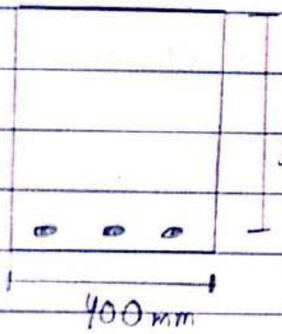
$\phi M_n = 440 \ Kn.m$

• بدلاً من أن يتم وضع (5) قضبان حديد على طول section
و للتقليل من تكلفة المشروع و ذلك لأنه عندما تم حساب
A_s تم حسابها على (M_u) ولكن على الأطراف (M)
أقل ونحتاج (A_s) أقل لذلك يتم اللجوء إلى (cut of bar)

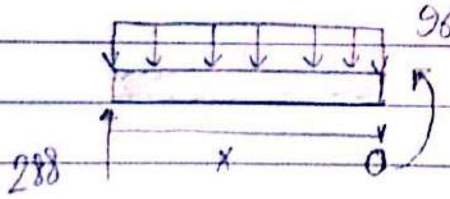
• يتم أخذ مقطع (x) من كلا الطرفين
ثم وضع (3) قضبان حديد بدلاً من (5) قضبان

وتم تحديد عدد القضبان التي سيتم وضعها في المسافة (x)
من قواعد (cut of bar)

ولحساب المسافة (x) يتم اللجوء للمعادلات (Static)



535mm ⟶ $\emptyset M_n = 282$ Kn.m
↓ تم حسابها
بتحديد As لـ (3) قضبان
حسب قواعد (cut of bar)



$\circlearrowleft \sum M_o = 0$

$$282 + 96\,x\left(\frac{x}{2}\right) - 288\,x = 0.0$$

$$x = 1.23\ m$$

• يتم وضع (3) قضبان حديد على طول (section)
و لكن عند مسافة (3.5m) يتم إضافة (2) قضيبين
حديد وذلك لمقاومة M_u

• يتم عمل check على L_d في الحالتين لذلك
(theoretical cut of point)

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	$\leq \phi 20$	$> \phi 20$
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 or Case 2: Clear spacing of bars being developed not less than $2d_b$ and clear cover not less than d_b	$l_d = \frac{f_y \alpha \beta \lambda}{2.1\sqrt{f_c'}} d_b$	$l_d = \frac{f_y \alpha \beta \lambda}{1.7\sqrt{f_c'}} d_b$
Other cases	$l_d = \frac{f_y \alpha \beta \lambda}{1.4\sqrt{f_c'}} d_b$	$l_d = \frac{f_y \alpha \beta \lambda}{1.1\sqrt{f_c'}} d_b$

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

L_d طلب

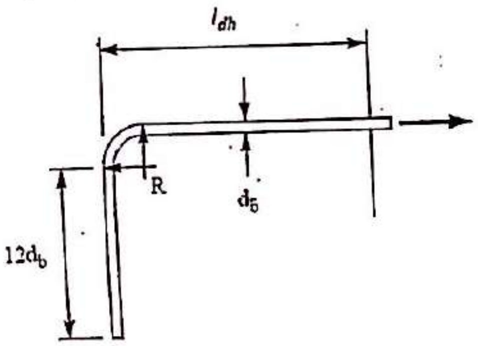
Table 1: Basic tension development-length ratio, l_d/d_b *(mm/mm)*

$$l_d = \frac{l_{dh}}{d_b} \times \beta\lambda \times d_b, \text{but not less than } 300\,mm$$

Bar size (mm)	$f_c = 21$ MPa Bottom bar	$f_c = 21$ MPa Top bar	$f_c = 25$ MPa Bottom bar	$f_c = 25$ MPa Top bar	$f_c = 28$ MPa Bottom bar	$f_c = 28$ MPa Top bar	$f_c = 30$ MPa Bottom bar	$f_c = 30$ MPa Top bar	$f_c = 35$ MPa Bottom bar	$f_c = 35$ MPa Top bar
	Case 1: Clear spacing of bars being developed not less than db, clear cover not less than db, and stirrups throughout ld not less than code minimum, or Case 2: Clear spacing of bars being developed not less than 2db and clear cover not less than db									
	$f_y = 420$ MPa, uncoated bars, normal weight concrete									
$\leq \phi 20$	43.6	56.7	40.0	52.0	37.8	49.1	36.5	47.5	33.8	43.9
$> \phi 20$	53.9	70.1	49.4	64.2	46.7	60.7	45.1	58.6	41.8	54.3
	$f_y = 300$ MPa, uncoated bars, normal weight concrete									
$\leq \phi 20$	31.2	40.5	28.6	37.1	27.0	35.1	26.1	33.9	24.1	31.4
	Other Cases:									
$\leq \phi 20$	64.5	83.9	59.1	76.9	55.9	72.7	54.0	70.2	50.0	65.0
$> \phi 20$	82.1	106.8	75.3	97.9	71.1	92.5	68.7	89.3	63.6	82.7
	$f_y = 300$ MPa, uncoated bars, normal weight concrete									
$\leq \phi 20$	46.8	60.8	42.9	55.7	40.5	52.6	39.1	50.9	36.2	47.1

- For top bars, more than 300 mm of fresh concrete is cast in the member (i.e. $\alpha = 1.3$)
- β is the coating factor, and λ is the lightweight concrete factor

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



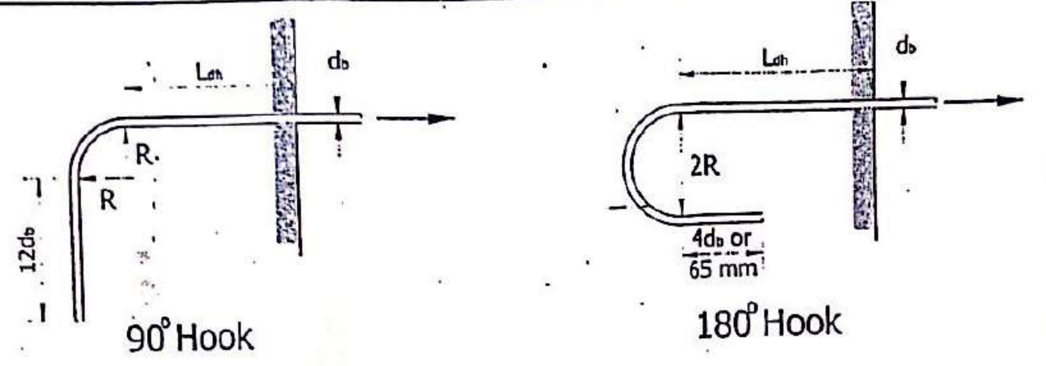
$\phi 10$ to $\phi 25$: $R = 3d_b$
$\phi 28$ to $\phi 32$: $R = 4d_b$
$\phi 50$: $R = 5d_b$

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:

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

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

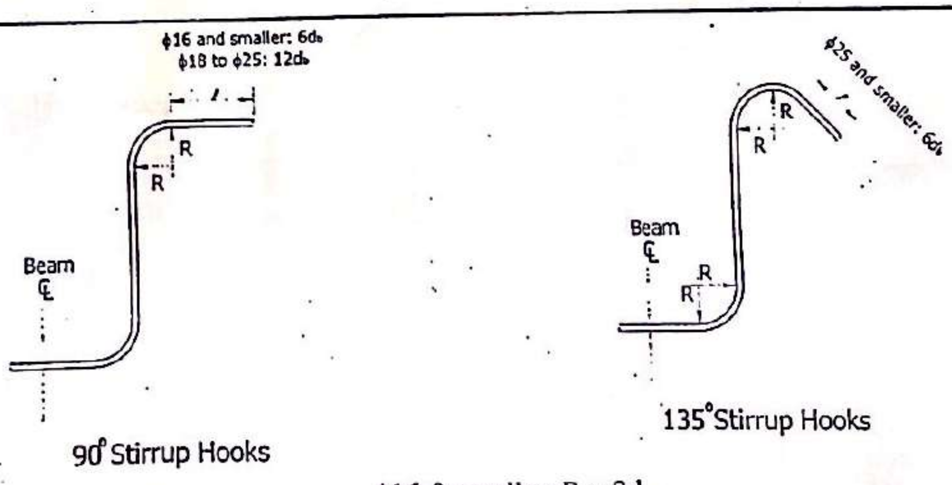
Standard Hooks – ACI sections 7.1 and 7.2.1



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

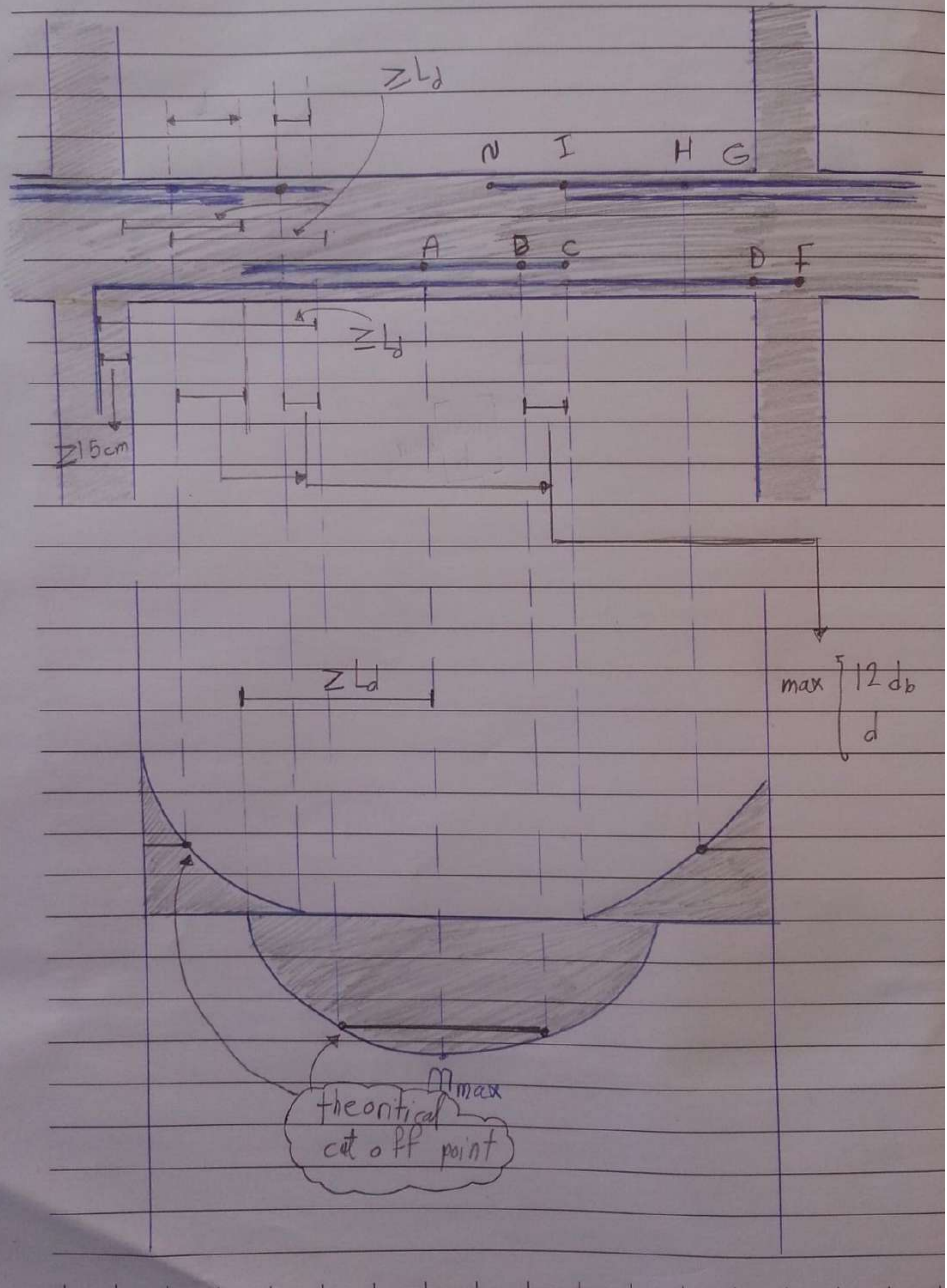
غير مطالب بهم

Stirrups and tie hooks – ACI section 7.1.3



$\phi16$ & smaller: $R = 2d_b$
$\phi18$ to $\phi25$: $R = 4d_b$

Bar cutoffs:



≥ Ld
N I H G
A B C D F
≥ Ld
≥ 15cm
≥ Ld
max { 12 db, d
Mmax
theoritical cut off point

وقت عل

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:

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.

RC1

بلال بولعول

٣٥

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

Bar No.	Number of Bars: 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

Bar No.	Number of Bars: 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]

Bar No.	Diameter (in.)	Number of bars in single layer: 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; $\frac{3}{4}$ 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 $(1.65 - 0.0003w_c)$ 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.