New 2018

Civilittee

اللجنة الأكاديمية للهندسة المدنية

تلخيص

منشآت فولاذية

وائل اسعيد

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STEEL CONSTRUCTIONS

منشآت فولاذية

وائل غسان اسعيد

WAEL . G . IS'EED

تحتوي هذه الدوسية على حل جميع امثلة السلايدات بالتفصيل بالاضافة

الى جميع ملاحظات الدكتور حسن كتخدا والدكتور بلال ابو الفول .

في نهاية كل من الفيرست و السكند والفاينل يوجد شيت مقترح

للامتحان بالاضافة الى بعض من اهم اسئلة السنوات السابقة مع حلها .

((الدوسية لا تغني عن سلايدات المادة او شرح الدكتور))

always: $P_u = 1.2 D_L + 1.6 L_L$

$W_u = 1.2 W_D + 1.6 W_D$

* Strength (capacity) $\geq$ Ultimate load.

* Only LRFD Philosophy will be used.

مهم ⇐ 1 ft = 12 in ⇐ مهم

Chapter 3

Analysis & Design of tension member.

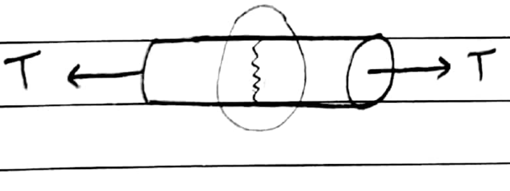
*Tension member: a member with an axial load at the center.

Stress $f = \frac{P}{A}$ f: Ksi , A: in^2
P: Kips

⇒ Three types of failure in tension member:

*في أي سؤال Tension يجب أنه نتحقق ونشيك على ٣ أشياء :-

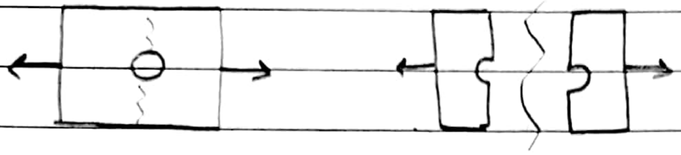
1) Yielding: at the gross cross-section of member



*الـ fail يكون في الـ member
نفسه مش عالأطراف

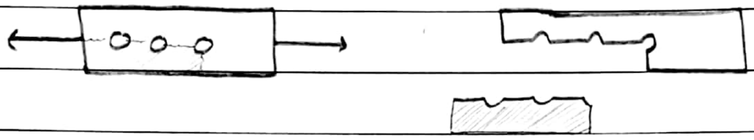
2) Fracture: at the effective area (Ae) (at connection)

كسر



هذا الـ fail يكونه
في منطقة الـ Connection
عبر ثقوب البراغي.

3) Block shear: * الـ fail يكونه في الـ Connection
على هذا الشكل ←



① yielding : $\phi_t P_n = \phi f_y A_g$. $\phi = 0.9$

P_n ↓ nominal strength ; f_y ↓ yield stress ; A_g ↓ gross area.

$(\boxed{50} - 65)$ ↑ (نأخذ الرقم الأصغر)

* قيم f_y , f_u : من المانيوال حسب نوع الحديد المستخدم. (نأخذ الرقم الأصغر)

* قيمة A_g من part 1 من المانيوال حسب اسم السكشن

② Fracture : $\phi_t P_n = \phi f_u A_e$ $\phi = 0.75$

f_u ↓ ultimate stress ← لانه يحدث كسر نستخدم

$A_e = U A_n$

A_e : effective net area.

A_n : net area.

U : reduction coeffecient.

$A_n = A_g - A_h$

A_h : area of holes.

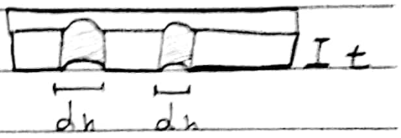
* في التصميم نحسب ϕP_n من الـ ٣ حالات ونأخذ الأصغر. ←

دائماً يجب أنه تكونه ⇒ $\boxed{\phi P_n \geq P_n}$

Net Area : $A_n = A_g - A_h$

A_h = عدد البراغي $* t * d_h$

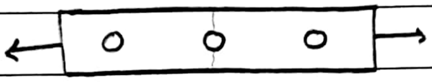
d of bolts



$d_h = d_b + 1/16 + 1/16$

Example: PL 1 x 16

1 → thickness
16 → Width



find the net area , $d_B = 7/8$ in.

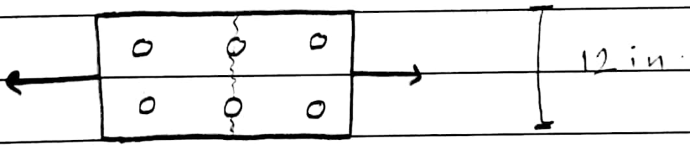
$A_n = A_g - A_h = 1 * 16 - 1 * (1)\left(7/8 + \frac{1}{16} + \frac{1}{16}\right) = 15 \text{ in}^2$

1 → عدد البراغي في اتجاه الكسر

(1) → t

* ملاحظة: إتجاه الكسر عامودي على اتجاه ال load

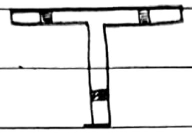
Example: PL 7/8 X 12 $d_B = 3/4$ in.

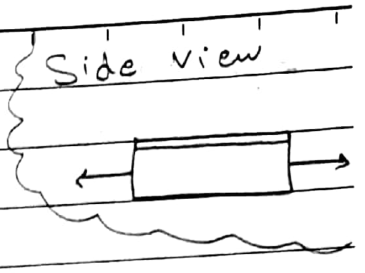


Sol: $A_n = A_g - A_h$

$A_n = \frac{7}{8} * 12 - 2 * (7/8)(3/4 + 1/8) = 8.96875 \text{ in}^2$

Example: ⊙ ← load اتجاه ال





WT 10.5 X 61

$d_b = 7/8$

Sol: $A_n = 17.9 - 2(0.96)(7/8 + 1/8) - 1(0.6)(7/8 + 1/8)$

A_g ، برغيين في ال (flange) ، t_f ، برغي في ال web ، t_w

$A_n = 15.38 \ in^2$

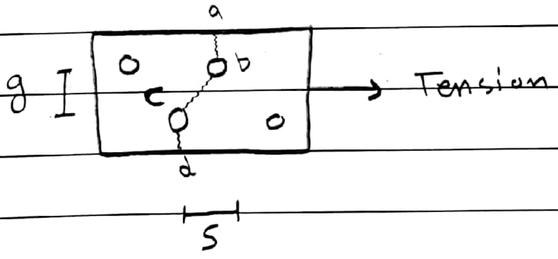
* A_g, t_w, t_f من المانيوال Part 1 فيه كل تفاصيل السكشن

⇒ Effect of staggered holes.

* تغيير ترتيب البراغي بشكل في حال حدوث كسر يكونه بشكل مائل.

* عملياً تقوي ال Connection لإنها تزيد من An .

S: Tension باتجاه ال



g: Tension عامودي عال

* عند حساب An لكل خط مائل نجمع $t * \frac{s^2}{4g}$

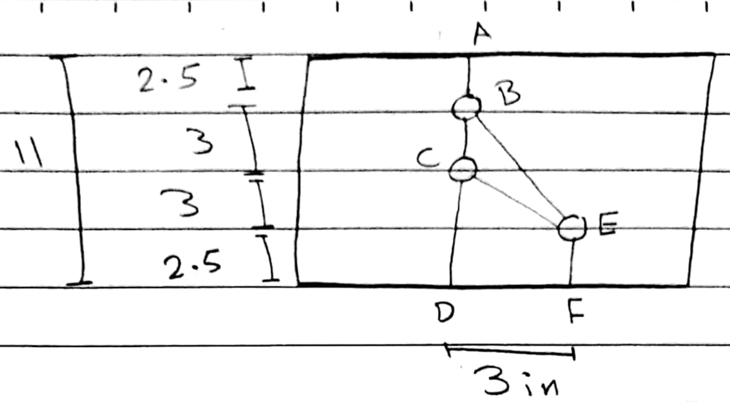
$$\Rightarrow A_n = A_g - A_{holes} + t * \frac{s^2}{4g}$$

* من الممكن أنه يكونه هناك اكثر من مسار للكسر ، نحسب An لكل منهم ونختار الاقل

Example:

$t_{Plate} = 0.5$ in

$d_B = 3/4$ in



* في هذا السؤال يوجد أكثر من Path للكسر ، انت بتحددهم وبتحسب An لكل واحد

Sol: ① Path ABCD (No Staggered)

$$A_n = 11 * 0.5 - 2(0.5)(3/4 + 1/8) = 4.625 \text{ in}^2$$

② Path ABCEF

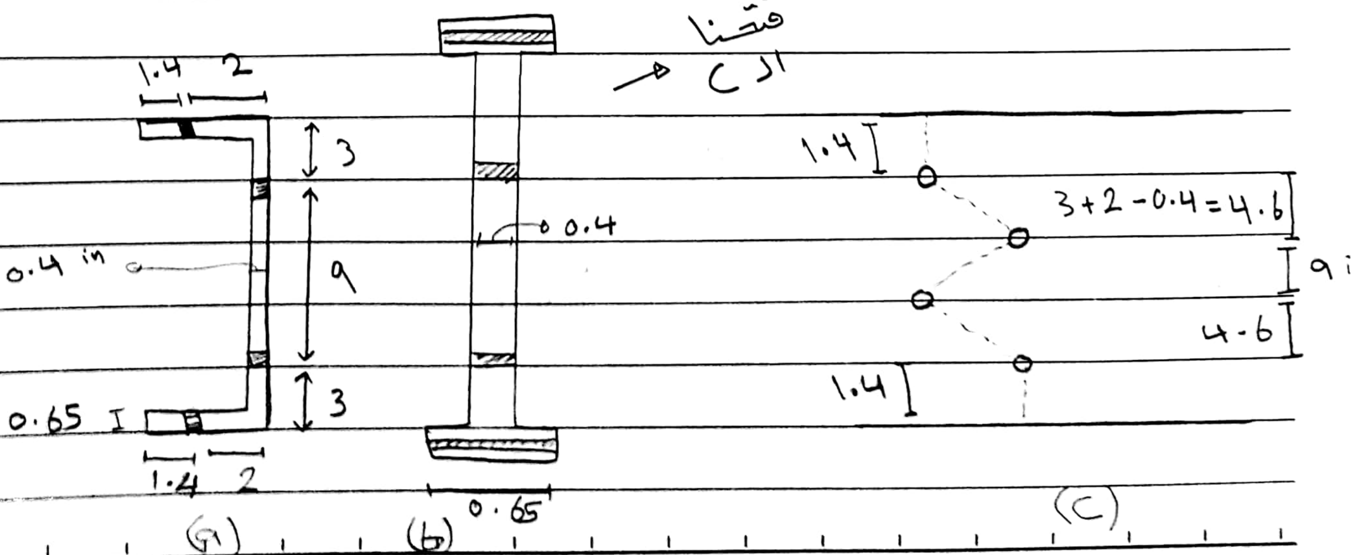
$$A_n = 11 * 0.5 - 3(0.5)(3/4 + 1/8) + 0.5 * \frac{(3)^2}{4(3)} = 4.5625 \text{ in}^2$$

③ Path ABEF

$$A_n = 11 * 0.5 - 2(0.5)(3/4 + 1/8) + 0.5 * \frac{(3)^2}{4(6)} = 4.8125 \text{ in}^2$$

$\Rightarrow A_n (control) = \boxed{4.5625 \text{ in}^2} \Leftarrow$ * الأصغر (الأضعف) يحدث عندها الكسر

Example 2: C 15 x 33.9 $d_B = 3/4$ in



* في هذا السؤال ممكن ما يعطيك الشكل ومفصل عليه كل شي، تمام مثل
هيك .. بيكونه معطيك رقم السكشن فـ بتروح عالمانيوال
وبتجيب كل اشي ناقص.

انت لازم تحسبه $4.6 = 3 + 2 - 0.4 = 4.6$

نطرح منهم الـ (t)

الأقل لل ($\underline{\underline{f}}$ or $\underline{\underline{w}}$)

مثل ← أحسبه

* الرسمة ع خاوة لازم معطيك إياها ومعطيك قيمة الـ g والباقي انت

Path ABCDEF (مطلوب في السؤال).

* الخط المائل BC و DE يمر في الـ (w & f) .. لذا نأخذ الأُفرج (t)
في staggered.

من المانيوال ← $S = 3$ ، $g = 4.6$ ، $t = \frac{0.4 + 0.65}{2}$

$$A_n = 10 - 2(0.65)(3/4 + 1/8) - 2(0.4)(3/4 + 1/8)$$

له برغيين في الـ f وبرغيين في الـ w

$$+ 2 * \left(\frac{0.4+0.65}{2}\right)\left(\frac{(3)^2}{4(4.6)}\right) + 0.4\left(\frac{(3)^2}{4(9)}\right)$$

له خطين BC و DE

للخط CD يمر فقط في الـ web

$$A_n = 8.7761 \text{ in}^2$$

Example 3: غير مطلوب

* Effective area & U-factor

for welded → $A_e = A_g U$ استخدمنا A_g لانه ما في براغي

〃 bolted → $A_e = A_n U$

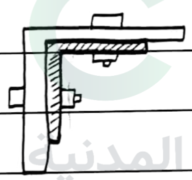
↳ Shear Lag

* حساب قيمة U يكون من خلال أكثر من case حسب شكل السكشن وتشبيكه وإذا كان في أكثر من قيمة نختار الأكبر. (for economy)

Table D3.1 shear lag factors في المانيوال

أول صفحة بعد السكاشن (كل هذول ل موجودين)

$\boxed{1}$ case 1 $(U = 1)$ لما يكون مربط من كل الجهات



$\boxed{2}$ case 2 * وهي الحالة العامة لكل السكاشن (مش مربط من كل الاتجاهات)

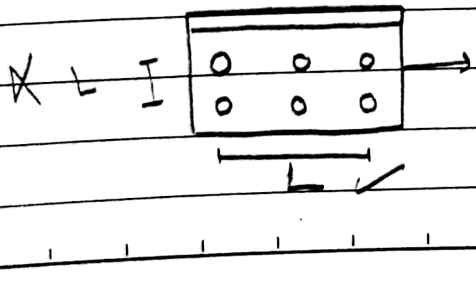
→ $U = 1 - \bar{X}/L$

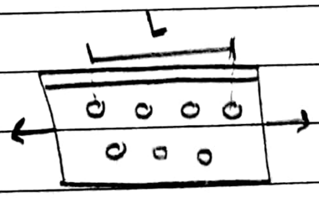
* ما عدا ال Plates ⇐ $U = 1$

(من المانيوال وممكن تأخذ $\bar{y}$ حسب الحالة)

$\bar{X}$: هي البعد بين السطح المثبت الى Centroid الشكل.

L : أطول مسافة بين البراغي (center to center)

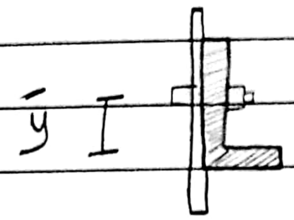


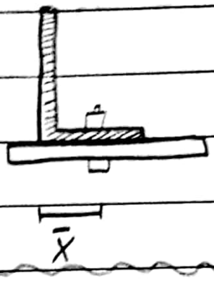


* اتجاه L موازي لإتجاه ال load.

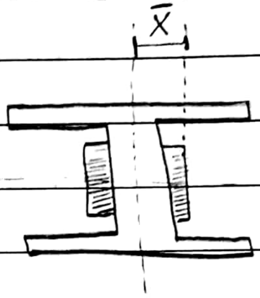
* اذا كانه مثبت بالـ x نأخذ $\bar{x}$

* اذا مثبت بالـ y نأخذ $\bar{y}$





حالة خاصة
W-shape



$\bar{x}$: تكون هي المسافة من center الشكل الى نهاية الـ plate المثبت بها.

$\boxed{3}$ case 7 : for $\underline{\underline{W}}$, M, S, $\underline{\underline{T}}$ sections.

التربيط في الـ flange $\rightarrow$ $b_f \geq \frac{2}{3} d \rightarrow U = 0.9$

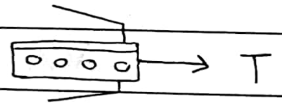
$\rightarrow$ $b_f < \frac{2}{3} d \rightarrow U = 0.85$

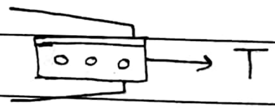
التربيط في الـ web $\rightarrow$ $U = 0.7$

$\boxed{4}$ Case 8 : Single Angles

في اتجاه الـ load { 4 or more fastners $U = 0.8$

2 or 3 " $U = 0.6$



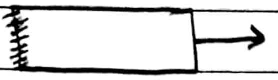


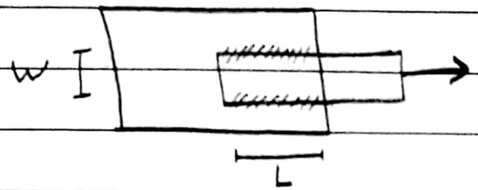
* اذا كانت قيمة $U \neq 1$ ... نستخدم دائماً $\underline{\underline{\text{case } 2}}$ مع case 8 أو case 7

ونختار القيمة الأكبر.

* عند عدم وجود قياسات للسكشن (يعني ديزاين) نستخدم 7 أو 8 فقط

Case 3 & 4 : for welded connections





اذا كان اللحام عامودي على اتجاه ال load دغري

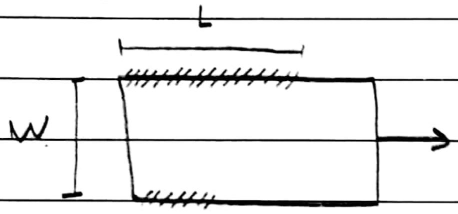
تكون $U = 1$

* Longtudinal welds

⊛ $L > 2w$ $U = 1$

⊛ $1.5w < L < 2w$ $U = 0.87$

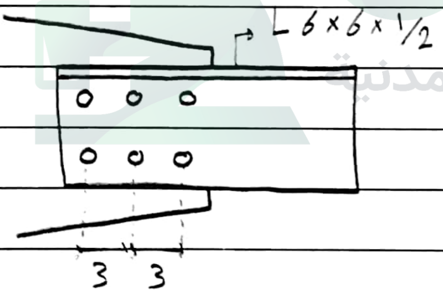
⊛ $w < L < 1.5w$ $U = 0.75$

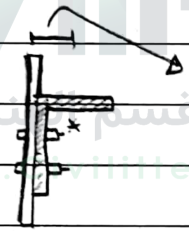


L : هي طول اللحام الأكبر

Example 1 : find the effective net area

$d_B = 5/8$





1.67 in ← انت بدك تجيبها

Sol: $A_e = A_n U$

* $A_n = 5.77 - 2(0.5)(5/8 + 1/8) = 5.02 \text{ in}^2$

$U \rightarrow$ from case 8 → 3 fastners → $U = 0.6$

$\hookrightarrow$ from case 2 → $1 - \bar{x}/L = 1 - \frac{1.67}{6}$ → $U = 0.722$

use larger

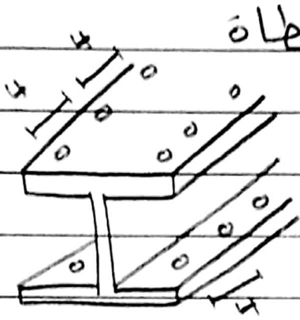
$A_e = 0.722(5.02) = 3.623 \text{ in}^2$

Example 2: W 10x45, two lines of 3/4 in bolts in flange, A572 grade 50

* at least 3 bolts in each line 4 in on center. (No staggerd)

+ في الامتحان بيحددلك بالزبط عدد البراغي

* في هذا السؤال الرسمة تكون معطاة



* Determine the Tensile design strength ΦP_n.

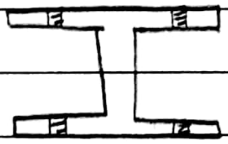
Sol: * يجب أن نشيك على ٣ شغلات .. block shear, fracture, yielding

← بس هاي مشروحة بالأخير

① Yielding : $\Phi P_n = 0.9 f_y A_g = 0.9(50)(\underbrace{13.3}_{\text{من المانيوال}}) = 598.5\ K$

② Fracture : $\Phi P_n = 0.75 f_u A_e$

Cross section



$\Rightarrow A_n = 13.3 - 4(3/4 + 1/8)(0.62)$ → t_f

لأنه ٤ البراغي في flange

في هذه الحالة الكسر يكون في ٤ براغي

$A_n = 11.13$ in

U → from case 7 : $b_f > 2/3\, d$ → $U = 0.9$ (control) ✓

→ case 2 : $U = 1 - \bar{x}/L = 1 - 0.907/8 = 0.8866$ ✗

لـ نحضرها من سكشن WT 5X22.5

لانه يمثل نصف السكشن الأصلي.

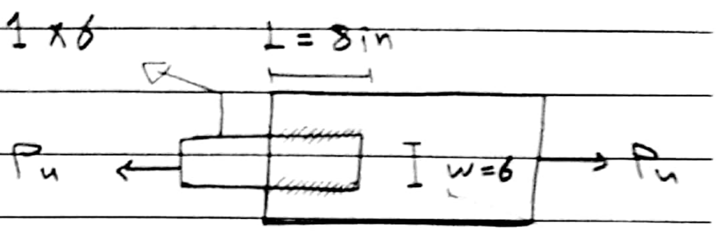
$\Rightarrow A_e = 0.9(11.13) = 10.017\ in^2$

$\Phi P_n = 0.75(65)(10.017) = 488.33$ Kips.

$\Rightarrow \boxed{\Phi P_n = 488.33\ Kips}$ ⇐ الأصغر

Example:

$F_y = 50$, $F_u = 65$



* find Tensile design strength ϕP_n .   (واضح من إسم ال plate) ممكن تكون مش معطاه

Sol: ① Yielding = $\phi P_n = \overset{\phi}{0.9}\,\overset{F_y}{(50)}\,\overset{A_g}{(6*1)}$ = 270 Kips.

② fracture : $\phi P_n = 0.75\,f_u\,A_e$

$A_e = U A_g$, case 4 → $w < L < 1.5w$

لانه لحام

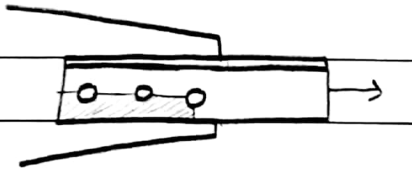
$\Rightarrow U = 0.75$.

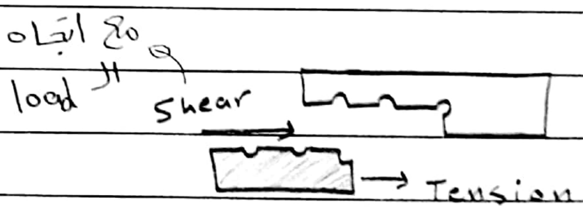
$\phi P_n = 0.75 (65) (0.75 * 6 * 1) = 219.4$ Kips.

$\Rightarrow$ use smaller $\Rightarrow \phi P_n = 219.4$ Kips

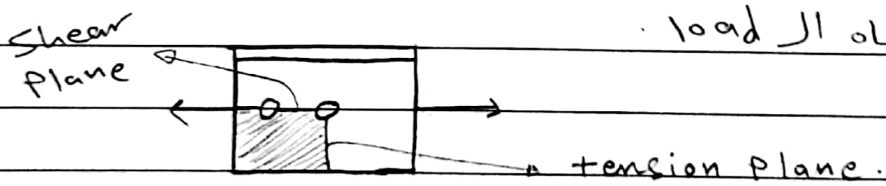
* Block Shear:

bolted ⇒

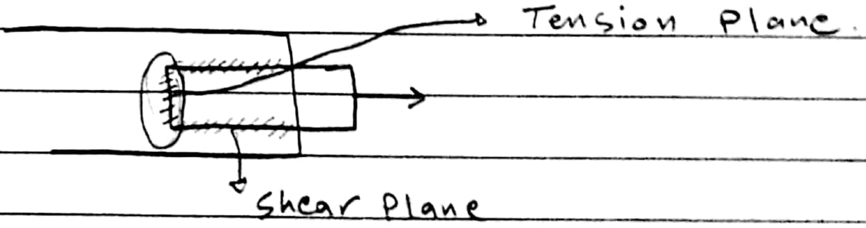




(لـ عامودي على اتجاه ال load.



Welded :



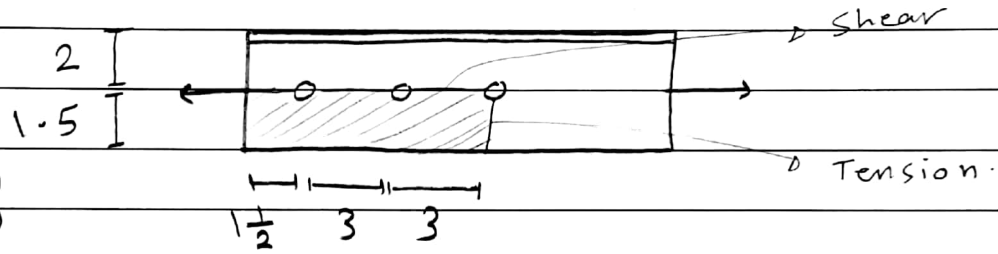
* موضوع الـ block shear عبارة عن معادلة فقط وتعويض أرقام فيها

$$\phi P_n = 0.75\left[0.6 F_u A_{nv} + F_u A_{nt} \le \underbrace{0.6 F_y A_{gv} + F_u A_{nt}}_{\text{upper limit}}\right]$$

* A_{nv} : net area of shear plane
A_{nt} : " " " tension plane
A_{gv} : gross area of shear plane.

Example : L $3\frac{1}{2}$ X $3\frac{1}{2}$ X 3/8

A36
$f_y = 36$ Ksi
$f_u = 58$ Ksi
$d_B = 7/8$ in



Sol: $\phi P_n = 0.75[0.6 f_u A_{nv} + F_u A_{nt} \le 0.6 F_y A_g + f_u A_{nt}]$

$$A_{nv} = A_{gv} - A_{holes} = \overset{(3+3+1.5)}{(7.5)}\overset{(t)}{\left(\frac{3}{8}\right)} - \underset{\text{عدد البراغي}}{2.5}\left(\frac{7}{8}+\frac{1}{8}\right)\left(\frac{3}{8}\right)$$

$$\boxed{A_{nv} = 1.875 \text{ in}^2}$$

$$A_{nt} = A_{gt} - A_{holes} = 1.5 * \frac{3}{8} - \underset{\text{عدد البراغي}}{0.5}\left(\frac{7}{8}+\frac{1}{8}\right)\left(\frac{3}{8}\right)$$

$$\boxed{A_{nt} = 0.375 \text{ in}^2}$$

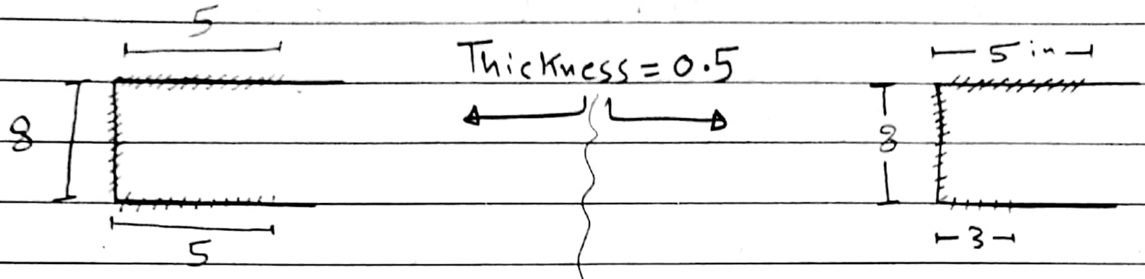
* $A_{gv} = 2.8125 \text{ in}^2$.

بالتعويض في معادلة الـ block shear $\Rightarrow$ $\phi P_n = 0.75[87 < 82.5]$ take upper limit.

$$\boxed{\phi P_n = 0.75 * 82.5 = 61.875 \text{ Kips}}$$

* Block Shear for welding : $A_{nt} = A_{gt}$ لانه لا يوجد براغي

$A_{nv} = A_{gv}$



$A_{gv} = A_{nv} = 2(5)(0.5) = 5\ in^2$

$A_{gt} = A_{nt} = 8(0.5) = 4\ in^2$

$A_{nv} = A_{gv} = 5*0.5 + 3*0.5 = 4\ in^2$

$A_{nt} = A_{gt} = 8*0.5 = 4\ in^2$

* الـ block shear أقل أهمية منال Yielding, fracture في الديزاين

Design of tension members

⊛ Design procedure : get P_u from loads on the structure

↳ $1.2D + 1.6L$

1) Yielding : $A_g \geq \frac{P_u}{0.9 f_y}$

2) fracture $A_e \geq \frac{P_u}{0.75 f_u}$

3) Slenderness ratio : $r \geq L/300$ → Length of the tension member in inches.

↓ radius of gyration = $\sqrt{I/A}$

4) block shear : as a check only if there is a block shear or no !

↳ بالأخير

Example 1 : L of tension member = 5 ft + 9 in = 5(12) + 9 = 69

$D_L = 18$, $L_L = 52$, $A_{36} \rightarrow f_y = 36$, $f_u = 58$.

⇒ Sellect a rectangular cross section member (Plate) with $d_B = 7/8$ in

⟹ (one line of bolts) .

Sol: $P_u = 1.2(18) + 1.6(52) = 104.8$ Kips .

① Yielding : $A_g \geq \frac{104.8}{0.9 * 36} = 3.23 \text{ in}^2$

② fracture : $A_e \geq \frac{104.8}{0.75 * 58} = 2.409 \text{ in}^2$

Try $A_g = 3.5$ & assume $t = 1$ & $w = 3.5$ in.

$A_n = A_e = A_g - A_{hole} = 3.5 * 1 - 1 * 1 * (7/8 + 1/8)$

(Plate, U=1)

$A_e = 2.5 \text{ in}^2 > 2.409$ ok

لو ما زبط
ارجع وكبر
Ag

③ Slenderness : $r \geq L/300$.

$$r = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{\frac{3.5(1)^3}{12}}{3.5 * 1}} = 2.887 > \frac{69}{300} = 0.23$$

ok ✓.

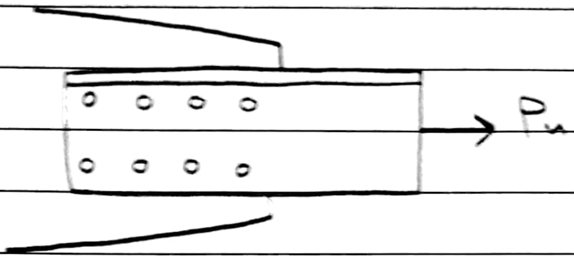
⇒ use PL 1x3.5

* لازم أشيك على الـ block shear كمان بس فش رسمة فما بقدر .

٢٠٠ 3×3
6×6
5×5

Example 2: select an unequal Leg angle tension member with L = 15 ft to resist $D_L = 35$
$L_L = 35$
use A36
$(f_y = 36, f_u = 58)$

$d_B = \frac{3}{4}$ in



Sol: Page 1-46

* افتح على الصفحة قبل الأخيرة بالـ Angles في المانيوال وانظر الـ table اللي تحت.

[illegible]

* من عند الـ 5 نقط معطى يكونه عندك two lines من البراغي مثل ما السؤال موضحلك بالرسمة .. يعني مصدرك تختار

وما يكونه equal leg
L 5×0×0
L 6×0×0
L 7×0×0

$\Rightarrow P_u = 1.2D_L + 1.6LL = 154$ KiPs.

① yielding $\Rightarrow Ag_{required} \geq \frac{P_u}{0.9f_y} \Rightarrow \boxed{Ag \geq 4.75 \text{ in}^2}$

② fracture $\Rightarrow Ae_{req} \geq \frac{P_u}{0.75f_u} \Rightarrow \boxed{Ae = 3.54 \text{ in}^2}$

③ Slenderness $\Rightarrow r_{min} \geq \frac{L}{300} = \frac{15 \times 12}{300} \Rightarrow \boxed{r_{min} \geq 0.6}$

* روح عالمانيوال وبلش من L5×0×0 بحيث $Ag > 4.75$ $r_{min} > 0.6$ واعمل تشيك على Ae (لانه بنحسبها حساب).

* طبعاً لازم تبلش من L5 لانه كما بدك يكونه economy مش تروح تختار اكبر واحد وخلصنا كذا

Five Apple

*Start with $L5 \times 3\frac{1}{2} \times \frac{5}{8}$ في السلايدات مبلش من سكشن اكبر بس الصح إنك تبلش من الصغير

$A_g = 4.92 > 4.75$, $\underset{(r_z)}{r_{min}} = 0.746 > 0.6$

ما في غير path واحد →

$$A_n = 4.92 - \underset{\text{عدد البراغي}}{2}\ (\underset{\text{سماكة}}{5/8})(3/4 + 1/8) = 3.826\ in^2$$

$$U \begin{cases} \rightarrow \text{from case 2} : 1 - \bar{x}/L \rightarrow \text{No } L \\ \rightarrow \text{case 8} : U = 0.8 \end{cases}$$

(Length of the connection not the member)

$\Rightarrow A_e = 3.061 < 3.54$ (Neglect)

Second
→ Try $L5 \times 3\frac{1}{2} \times \frac{3}{4}$ كمانه مش رح يزبط

تبع السلايدات *$L6 \times \frac{1}{4} \times \frac{1}{2}$, $A_g = 4.75$ ✓ $r_{min} = 0.864 > 0.6$ ✓.

$A_n = 4.75 - 2(\frac{1}{2})(3/4 + 1/8) = 3.875\ in^2$

$U = 0.8 \longrightarrow A_e = 3.1\ in^2$ ✗ (neglect)

وهكذا حتى نوصل لواحد مناسب.

* طبعاً في الامتحان بيجيبلك واحد بطلع معك بسرعة مش هيك.

→ Try $L8 \times 4 \times \frac{1}{2}$ $\Rightarrow A_g = 5.75$, $r_{min} = 0.863$

$A_e = 0.8\left[5.75 - 2(\frac{1}{2})(3/4 + 1/8)\right] = 3.9 > 3.54$ ok ✓.

$\Rightarrow$ use $L8 \times 4 \times \frac{1}{2}$

* لازم تعمل check على الـ block shear بس ما في معطيات فخلص إنساه.

* في طريقة أخرى للـ design عن طريق الجداول من Part 5 من المانيوال.

* قيمة موحدة لـ $\frac{f_y}{f_u}$ وقيمة تقريبية لـ Ae ← 0.75Ag

* لنفس المثال السابق أوجد section بحيث أنه

$$P_u < \begin{array}{l} \phi P_n \text{ yielding} \\ \phi P_n \text{ fracture} \end{array}$$

⇒ Page 5-15 start with $L5 \times 3\frac{1}{2} \times \frac{5}{8}$

وهو نفسه أول واحد بلشنا فيه قبل.. يعني نفس الإشي.

yielding = 159 > 154

fracture =

* بعدين بنرجع على Part 1 من المانيوال وبنجيب باقي معلومات السكشن وبنعمله Analysis مثل قبل.

$\downarrow$

A_g, r_{min}

* هذه الطريقة مش مهمة.. المهم إنه تعرف تحل مثل أول لانه في الـ Tension design ما بيصدلك طريقة تستخدمها.

Design of threaded rods & cables

⇒ get $P_u = 1.2D + 1.6L$

① yielding : $A_g \geq \frac{P_u}{0.9 f_y}$ ⇒ $A_g = \frac{\pi}{4} d^2$ → get $\underline{\underline{d}}$.

② fracture : $A_g \geq \frac{P_u}{0.75 * f_u * 0.75}$ ⇒ $A_g = \frac{\pi}{4} d^2$ → get $\underline{\underline{d}}$.

تساوي Ae لإنه ما في براغي.

بسبب المسننات

⇒ select Larger $\underline{\underline{d}}$. (تأكد من الدكتور #)

Example: A threaded rods is to be used as a bracing member that must resist a service tensile load of 2 Kips dead load & 6 Kips live load, What size rod is required if A36 steel is used ?

Sol: $P_u = 1.2(2) + 1.6(6) = 12$ Kips

① Yielding , $A_g \geqq \frac{P_u}{0.9 f_y} \Rightarrow A_g = \frac{12}{0.9(36)} = 0.37 \text{ in}^2$

$A_g = \pi/4 \, d^2 \longrightarrow d = 0.687$ in.

② fracture , $A_g \geq \frac{P_u}{0.75 * f_u * 0.75} \Rightarrow A_g = 0.3678 \text{ in}^2$

$d = 0.684$

⇒ Larger $d = 0.687$ in

Chapter 4

Analysis & Design of compression members.

* Compression member : Columns : elements that are subjected to axial compressive force at the center.

* $\boxed{f = P/A}$

⇒ modes of failure in compression members :-

1) Flexural (Euler) buckling. (على ال member كامل)

2) Local buckling. (يحدث على اجزاء في ال member)

3) Flexure torsional buckling.

* الإشتقاقات مش مطلوبة *

$$\boxed{P_{cr} = \frac{\pi^2 E A}{(KL/r)^2}} \quad , \quad \boxed{F_e = \frac{\pi^2 E}{(KL/r)^2}}$$

P_{cr} → Euler buckling load

F_e → Euler Buckling stress

مهم
نعوض L بال inches

P_{cr} : وهي القيمة ال load التي يحدث عندها ال buckling

* as $\frac{KL}{r} \uparrow$ ... $P_{cr} \downarrow$ ⇒ يعني العامود أضعف

$\frac{KL}{r} \downarrow$ ... $P_{cr} \uparrow$ ⇒ أقوى " "

* قيمة K اذا كان التثبيت Pin - Pin تكون 1.

* قيمة E حفظ دائماً 29,000

$E = 29,000$
احفظ دائماً.

Example: a W12 x 50 column, used to support an axial compressive load of 145 Kips, The Length is 20 ft, & it's pin-pin, is it stable or not!

Sol: * في الـ W-shapes .. الـ x هو الـ strong والـ y هو الـ weak
وذلك لأنه قيمة I_x دائماً أكبر من I_y .. ($r_x > r_y$)
بس ممكن أنه يكون هناك bracing بالـ y تقلب الأمور زي ما رح نشوف لقدام شوي.

$r_x = 5.18$ * إفتح عالمانيوال ⇐
$r_y = 1.96$ (controls) لأنه الأضعف عالسكشن.

نحسب P_{cr} في الأضعف فقط.
ولكن سنحسب هنا الاثنين لنرى الفرق.

$$* \; P_{cr(y)} = \frac{\pi^2 E A}{(KL/r_y)^2} = \frac{\pi^2 (29,000)(14.6)}{\left(\frac{1 * 20 * 12}{1.96}\right)^2}$$
منيوال

$P_{cr(y)} = 278.7$ Kips * عند هذه القيمة سيحدث buckling في الـ y-axis.

$$* \; P_{cr(x)} = \frac{\pi^2 (29,000)(14.6)}{\left(\frac{1 * 20 * 12}{5.18}\right)^2} = 1946.6 \text{ Kips}$$
في الـ x

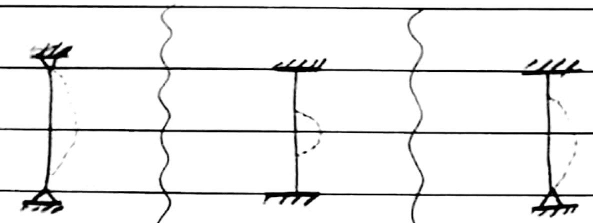
* كما تلاحظ أنه قيمة x مثل ما هو متوقع أكبر من قيمة y .. عشانه هيك لما بنحدد مين من الـ (x أو y) هو الـ control بنحسب اله فقط.

بالأخير ⇒ $P_{cr} = 278.7 > 145$ ok ✓
stable.
أكبر قيمة ممكن يتحملها قبل ما يصير عنده buckling.

* Effective Length ↓   * قيمة K تعتمد على التثبيت.

* وهو عبارة عن الطول اللي رح يتأثر بال buckling

موجودين في المانيوال الحالات كلهم بعد ٣ أو ٤ صفحات من جداول U .



	P-P	F-F	F-P
Theo →	K=1	K=0.5	K=0.7
design →	$\boxed{K=1}$	$\boxed{K=0.65}$	$\boxed{K=0.8}$

* الحالات اللي فيها Sides-way تكون قيمة K اكبر من 1 .. كل الحالات موجودين في المانيوال.

* ناخذ قيم ال design ↑

* Column types:

1) Short column: خيالي .. مش موجود بالواقع

No buckling. نظرياً * failure stress = yielding stress

2) Long columns: (elastic)

buckling will occur

قبل ما يوصل ال elastic بيصير فيه buckling

3) Intermediate columns: (Inelastic) (buckling will occur)

* بناخذ load اكبر ... جزء منه في ال elastic وجزء في ال inelastic .

* In compression

$$\phi_c P_n \geq P_u$$

$$\phi_c P_n = \phi\, F_{cr}\, A_g \qquad \phi = 0.9$$

⇒ to get Fcr :-

KL/r must be < 200

① if $KL/r \leq 4.71\sqrt{E/f_y}$ ⇒ $F_{cr} = \left[0.658^{f_y/f_e}\right] f_y$

inelastic (intermediate column)

② if $KL/r > 4.71\sqrt{E/f_y}$ ⇒ $F_{cr} = 0.877\, f_e$

elastic (Long column).

$$* \; f_e = \frac{\pi^2 E}{(KL/r)^2}$$

Example 1: W14 x 74 , A992 steel , L = 20 ft
Pinned ends (K = 1) , compute design compressive strength ($\phi_c P_n$).

Sol: check which axis controls. (هون واضح انه y بس
لقدام ممكن يتغير الوضع)

$$KL/r_x = \frac{1 * 20 * 12}{6.04} = 39.73$$

$$KL/r_y = \frac{1 * 20 * 12}{2.48} = 96.77$$ (controls) لأكبر يعني الأضعف

$96.77 < 200$ OK ✓.

$4.71\sqrt{E/f_y} = 113$

$\rightarrow 96.77 < 113 \rightarrow$ inelastic.

$\Rightarrow F_{cr} = \left[0.658^{f_y/f_e}\right] f_y \quad (f_y = 50)$

$f_e = \frac{\pi^2 (29,000)}{(96.77)^2} = 30.564$ Ksi

$\rightsquigarrow F_{cr} = 25.21$

$\phi_c P_n = \phi f_{cr} A_g = 0.9(25.21)(21.8) = 494.62$ Kips.

عندها رح يصير buckling حول الـ y .

ص (4-22) طريقة أخرى للحل عن طريق المانيول * ②

(مهما كانت المقطع)

فقط عن طريق Fy و KL/r فقط نحصل على ΦFcr

من غير ما أعرف إنه كانت elastic أو inelastic .. بتاخذ ΦFcr جاهزة .

لمثالنا ⇝ $f_y = 50, \quad KL/r_y = 96.77$

	KL/r	Φfcr
⇒	96 ⟶	22.9
	97 ⟶	22.6

by interpolation $\Rightarrow \phi f_{cr} = 22.669$

$\Rightarrow \phi_c P_n = \phi_c f_{cr} * A_g = 494.18$ Kips ✓

نفس الجواب

③ * طريقة أخرى للحل عن طريق المانيوال (4-1)

فقط للمقاطع الأمريكية (W-sections only) و $f_y = 50$

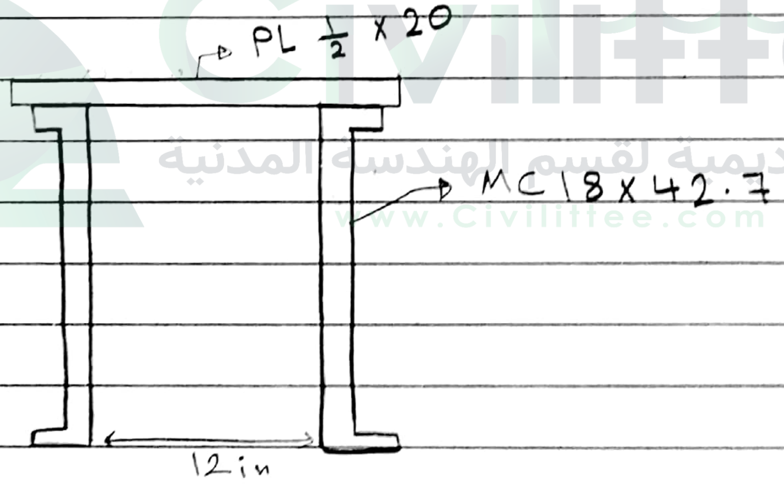
و KL_y هي ال control ..... اذا كانت X نحسب equivalent زي
مارح نشوف قدام

لمثالنا ⇒ $KL_y = 20\ ft$, $f_y = 50$, W14X74

جاهزة ⇒ $\Phi_c P_n = 494$ Kips

Example 2: $F_y = 50$ Ksi , length of the column = 23.75 ft
* fixed-Pinned ($K = 0.8$)
determine $\Phi_c P_n$.

مهم جداً



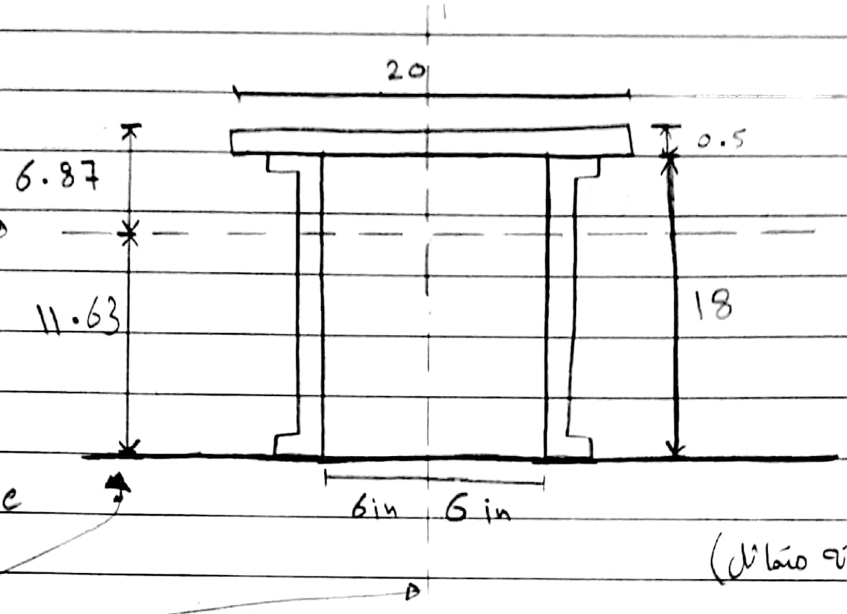
Sol: * في اسئلة ال built up اذا كان الشكل غير متماثل في ال X أو ال Y أو الاثنين معاً فيجب أنه سنترويد للشكل كامل.

* بالإضافة لحساب x ، I_x ، I_y للشكل كامل لأننا نحتاجها لإيجاد قيمة r_x ، r_y للشكل.

* في حل هذا السؤال سنقوم بتفصيل كل شيء ولكن في الإمتحان يجب أن تكون سريع.

٥ أهم خطوة .

* من المانيوال جيب كل معلومات السكشن وفرغها على الرسمة .



MC

$A = 12.6$

$I_x = 554$

$I_y = 14.3$

$\bar{x} = 0.877$

$\bar{y} = 9$ (واضح لانه متماثل)

* الشكل متماثل في الـ y-axis لذا فإنه موقع السنترويد في المنتصف ولكن غير متماثل في الـ x فسنحسب ارتفاع السنترويد $\bar{y}$

* يجب أن تحدد (reference) والأفضل أن يكون أوطى نقطة في الشكل

من سنترويد المستطيل الى ref — عدد 2 هم — من سنترويد الـ MC الى ref

$$* \bar{y} = \frac{\sum A\bar{y}}{\sum A} = \frac{(20*0.5)(18+0.25)+2(12.6)(9)}{(20*0.5)+2(12.6)}$$

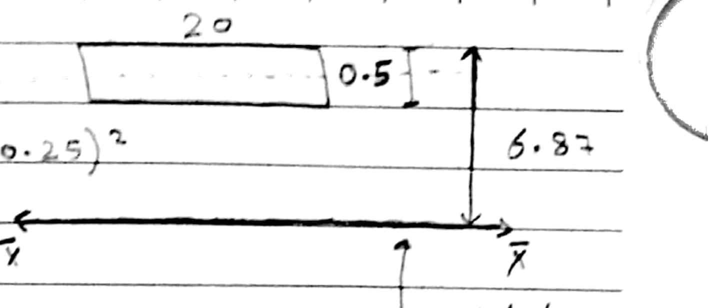
$$\boxed{\bar{y} = 11.63 \text{ in}}$$ from ref line

* متى نحدد أي axis هو الـ control نقوم بحساب $\frac{KL_y}{r_y}$ و $\frac{KL_x}{r_x}$ وقيمة r تساوي $\sqrt{\frac{I}{A}}$ لذا فسنحسب قيمة I_x و I_y للشكل كامل.

* اذا كان سنترويد القطعة لا يقع على سنترويد الشكل كامل فإننا نجمع Ad^2 لقيمة I .. حيث أن d تساوي المسافة بين سنترويد القطعة والشكل كامل.

ملاحظة : $I = \frac{bh^3}{12}$ h : هي اللي بيكونه المحور المطلوب عامودي عليها

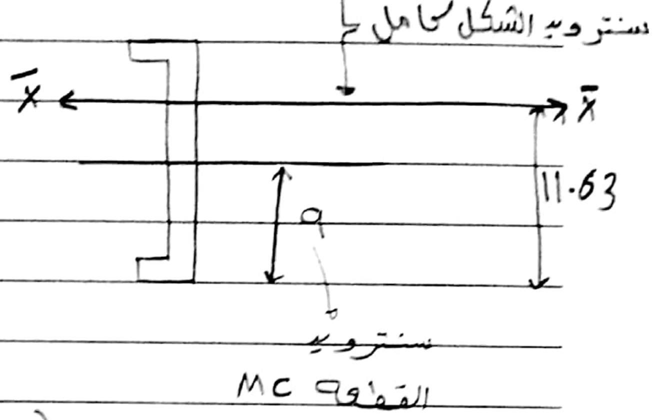
* $\underline{\underline{I_x}}$ for the Plate



$$I_x = \frac{(20)(0.5)^3}{12} + (20)(0.5)(6.87-0.25)^2$$

$$I_x = 438.45 \text{ in}^4$$

* I_x for the MC



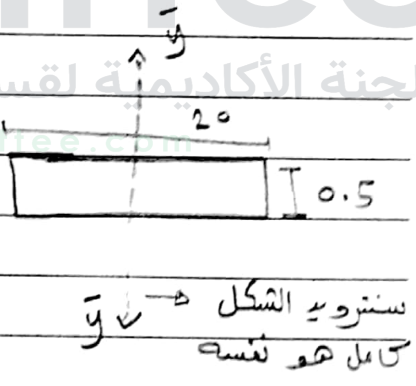
$$I_x = 554 + (12.6)(11.63-9)^2$$

$$I_x = 641.153$$

$$\text{total } I_x = 438.45 + \underset{\substack{\uparrow \\ \text{لأنه عدد الـ MC} \\ \text{يساوي 2}}}{2}(641.153)$$

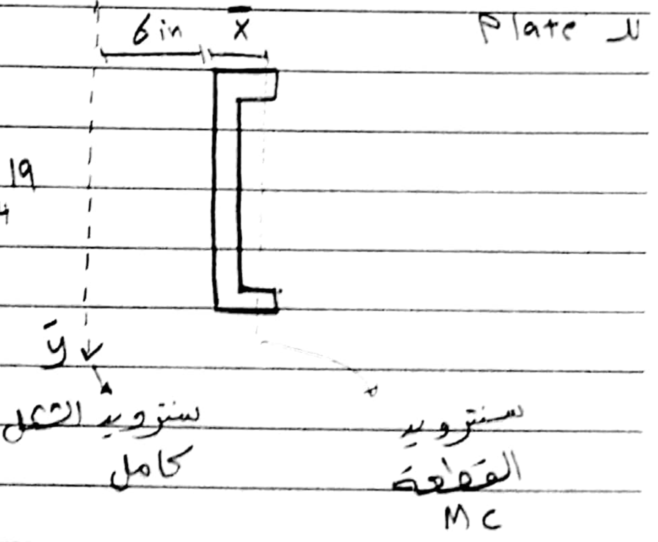
$$\boxed{I_x = 1721 \text{ in}^4}$$

* $\underline{\underline{I_y}}$, I_y for the plate



$$I_y = \frac{(0.5)(20)^3}{12} = 333.33 \text{ in}^4$$

I_y for MC



$$I_y = 14.3 + (12.6)(6+0.877)^2 = 610.19 \text{ in}^4$$

total

$$I_y = 333.33 + \underset{\substack{\uparrow \\ \text{عدد الـ MC}}}{2}(610.19)$$

$$\boxed{I_y = 1554 \text{ in}^4}$$

* Check which axis controls :-

$KL/r_x = \dfrac{0.8(23.75)*12}{\sqrt{\dfrac{1721}{(20*0.5)+2(12.6)}}} = 32.61$

$\downarrow$

$\sqrt{I_x/A_{tot}}$

$\dfrac{KL}{r_y} = 34.31$ (control)

$34.31 \leqslant 4.71\sqrt{E/f_y} \Rightarrow 34.31 \leqslant 113.43$ ✓

$\Rightarrow F_{cr} = \left[0.658^{f_y/f_e}\right] f_y$

$* f_e = \dfrac{\pi^2 (29,000)}{(34.31)^2} = 243.13$

$\Rightarrow F_{cr} = 45.876$ ksi

$\Rightarrow \phi_c P_n = 0.9 F_{cr} A_g$

$= 0.9(45.876)(20*0.5 + 2(12.6))$

$$\boxed{\phi_c P_n = 1453.4 \text{ Kips}} \quad \#$$

أو معطى * $\Rightarrow KL = 0.8(23.75) = 19 \quad , \quad f_y = 50$

من 4-22

في المانيوال

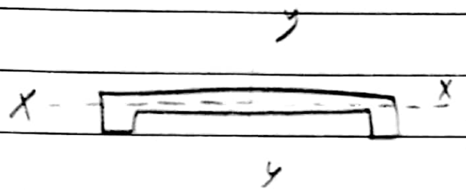
$\Rightarrow \phi f_{cr} = 43.8$

A_g

$\Rightarrow \phi_c P_n = 43.8(10 + 2(12.6))$

والحلين صح %100 $\boxed{\phi_c P_n = 1541.76 \text{ Kips}}$ #

*Note: in W-sections: x is the major axis
y is the minor axis



* في السؤال السابق.. لو كانه الـ C مبطوح

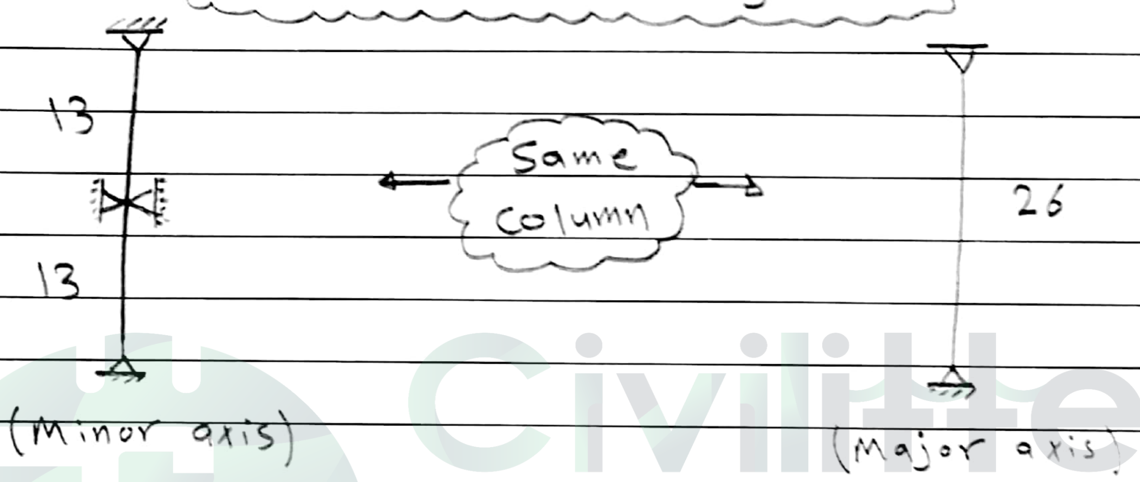
فإنه القيم المأخوذة من المانيوال ستنقلب $\begin{pmatrix} x \to y \\ y \to x \end{pmatrix}$

$I_x \to I_y$

$I_y \to I_x$

* * *

More on effective length (KL)

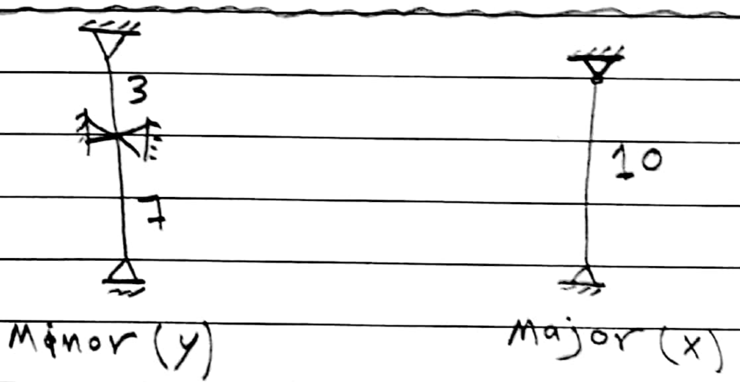


$\Downarrow$ $KL_y = 13$

$\Downarrow$ $KL_x = 26$

* * *

مثال للتوضيح :



$KL_y = 3$

$KL_y = 7$ (control).

$KL_x = 10$

$\Rightarrow KL_y = 7$

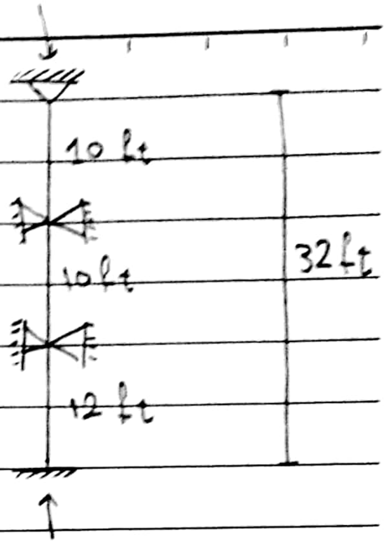
* انتبه لنوع التثبيت لأنه يغير قيمة K

Example: W14x90

$F_y = 50$ ksi

No bracing on (X-X)

only on y-y ⇒



determine $\phi_c P_n$.

F-P

$\uparrow$

Sol: X : $\frac{KL_x}{r_x} = \frac{0.8\,(32)*(12)}{6.14} = 50.03$

r_x : من المانيوال

$Y: 1)\ KL_y = 0.8(12) = 9.6$

$2)\ KL_y = 1\ (10) = 10$

$3)\ KL_y = 1\ (10) = 10 \longrightarrow$ (control in y).

$\frac{KL_y}{r_y} = \frac{10*12}{3.7} = 32.43 < 50.03$

r_y: من المانيوال

$\Rightarrow \frac{KL_x}{r_x} = 50.03$ controls. $< 4.71\sqrt{E/f_y}$

$\Rightarrow f_{cr} = [0.658^{f_y/f_e}]f_y$, $f_e = \frac{\pi^2\,(29{,}000)}{(50.03)^2}$

$f_{cr} = 41.64$ ksi

$\phi_c P_n = 0.9\,(41.64)\,(\overset{A_g}{26.5}) = \boxed{993.06 \text{ Kips}}$ #

* اذا كانه محددلك بالسؤال انه تستخدم tables 4-1 ([illegible])

ف لازم تكونه KL_y هي الـ control ولكن في

هذا السؤال الـ KL_x هي الـ control ولا نستطيع الدخول الى المانيوال م

equivalent نسميها
↑

بهذه القيمة... فنقوم بحساب قيمة مكافئة لها بال y لنستخدمها في المانيوال بدلاً من KLx.

← طبعاً يجب مقارنتها مع KLy الأصلية .. اذا كانت أقل نأخذ القيمة الأكبر ونكمل الحل

$\Rightarrow KL_x = 0.8\,(32) = 25.6$, KL_y (الأصلية) $= 10$

$$\text{equivalent} = \frac{KL_x}{(r_x/r_y)} = \frac{25.6}{1.66} = 15.42 > 10$$

← هاي القيمة موجودة جاهزة في المانيوال 4-1 تحت السكشن المطلوب

15.42 ↓ control ↓ استخدمها في المانيوال

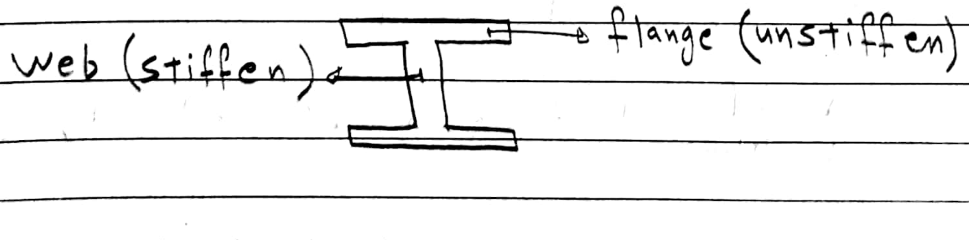
KL	$\Phi_c P_n$
15 →	1000
15.42 →	??
16 →	978

by interpolation $\Rightarrow \Phi_c P_n = 990.76$ Kips.

*Local buckling.

*وهو أنه يحدث buckling في جزء من ال member وليس العامود كامل.

☒ هنا الموضوع داخل فقط لـ (check) اذا كان في (local buckling) أو لا.



* in W-shape → local buckling in flange
أو → local buckling in web

* if the flange or the web is slender then the local buckling will happen.

* في الديزاين يفضل عدم اختيار سكاشن فيها مشكلة Local buckling

ويمكن تميزها من المانيوال من حرف c فوق إسم السكشن

* نعمل check عن طريق قيمة λ للـ flange و λ للـ web ومقارنتها مع قيمة λr حيث أنها يجب أن تكون أقل منها ليكون السكشن ما فيه مشكلة local buckling .

if * $\lambda_f < \lambda_{r(f)}$ (ok) → No local buckling.
$\lambda_w < \lambda_{r(w)}$ (ok)

* قيمة $\lambda_f = \frac{bf/2}{t_f}$ وقيمة $\lambda_w = \frac{h_w}{t_w}$ نأخذهم جاهزين من المانيوال (part 1) .

$$\Rightarrow \boxed{\begin{array}{l} \lambda_f < \lambda_{r_f} = 0.56\sqrt{E/f_y} \\ \lambda_w < \lambda_{r_w} = 1.49\sqrt{E/f_y} \end{array}}$$

* اذا تحقق الشرطين بيكون ما في مشكلة local buckling

* اكتبهم عالشيت اسهل من استخدام المانيوال لانه بيخربط .

* اذا اختل شرط واحد منهم تكون مشكلة الـ local buckling في ذلك الجزء

<u>Example:</u> Investigate the local buckling for W14X74
$F_y = 50$ ksi

<u>Sol:</u> * اذا كتبلك في السؤال show your calculations يجب انه تحسب λr_f و λr_w ومقارنتها مع λf و λw وغير ذلك يكونه الجواب من خلال حرف الـ c فوق اسم الـ section

* W14 X 74 have No $\underline{\underline{C}}$ letter then ⇒ No local buckling.

بس الحل الصح

$$\Rightarrow \lambda f = 6.41 \Longrightarrow$$ من المانيوال

$$\lambda w = 25.4$$

$$\Rightarrow \lambda r_f = 0.56\sqrt{\frac{29{,}000}{50}} = 13.49 > 6.41 \quad \text{ok} \checkmark$$

$$\lambda r_w = 1.49\sqrt{\frac{29{,}000}{50}} = 35.88 > 25.4 \quad \text{ok} \checkmark$$

⇒ No local buckling

* لو كانه السكشن فيه هاي المشكلة لازم نقلل الـ strength عن طريق حساب Q-factors وادخالهم على معادلات Fcr بس مش مطلوب غير نعمل check وخلص.

* لهون مادة الفيرست

* شيت مقترحة (الشيت اللي انا استخدمتها) وحل سنوات مهمة

يتبع ↓

Sheet 1.

tension

$$\underset{\downarrow}{A_n} = A_g - A_{holes} + t * \frac{s^2}{4g} \quad \text{(for each path)}$$

take the smallest.

U: from manual (shear lag) → use Larger L (between bolts or welding)

* block shear design strength:

$$\downarrow$$

$$\phi P_n = 0.75 \left[0.6 F_u A_{nv} + F_u A_{nt} \leqslant 0.6 F_y A_{gv} + F_u A_{nt} \right]$$

* Design of tension member

* $P_u = 1.2D + 1.6L$

① yielding : $P_u \leqslant 0.9 f_y A_g \longrightarrow A_g \geqslant \frac{P_u}{0.9 f_y}$

② fracture : $P_u \leqslant 0.75 f_u A_e \longrightarrow A_e \geqslant \frac{P_u}{0.75 f_u}$

③ Slenderness : $r \geqslant \frac{L}{300}$ → inches (L of the member)

④ block shear : check only using equation above ↑.

* Design of threaded rods & cables.

① yielding ⇒ $A_g \geqslant \frac{P_u}{0.9 f_y}$ → get $\underline{\underline{d}}$.

② fracture ⇒ $A_g \geqslant \frac{P_u}{0.75 * 0.75 * f_u}$ → get $\underline{\underline{d}}$.

⇒ use larger $\underline{\underline{d}}$.

Sheet 2.

Compression

L/r : Slenderness ratio , $r = \sqrt{I/A}$

Euler buckling load } $P_{cr} = \frac{\pi^2 E A}{(KL/r)^2}$, Euler buckling stress } $f_e = \frac{\pi^2 E}{(KL/r)^2}$

$P_u \leq \phi P_n$, $\phi_c P_n = 0.9 f_{cr} A_g$.

$\frac{KL}{r}$ → in inches

must be < 200.

① if $KL/r \leq 4.71\sqrt{E/f_y} \Rightarrow f_{cr} = \left[0.658^{f_y/f_e}\right] f_y$ inelastic. intermediate-column

② if $KL/r > 4.71\sqrt{E/f_y} \Rightarrow f_{cr} = 0.877 f_e$ elastic Long-column

*or use 4-22 & get ϕf_{cr} for any section.

* or use 4-1 & get $\phi_c P_n$ for american W shapes only.

* to check Local buckling.

check flange : $\lambda_f = \frac{bf/2}{tf} < \lambda_{rf} = 0.56\sqrt{E/f_y}$ ok.

check web : $\lambda_w = \frac{h_w}{t_w} < \lambda_{rw} = 1.49\sqrt{E/f_y}$ ok.

in built up sections

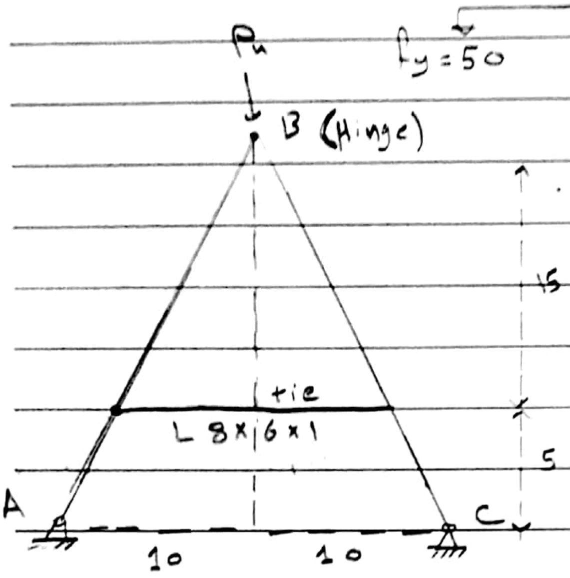
* calc $\bar{y}$ & $\bar{x}$ if not symmetric ... $\bar{y} = \Sigma A y / \Sigma A$

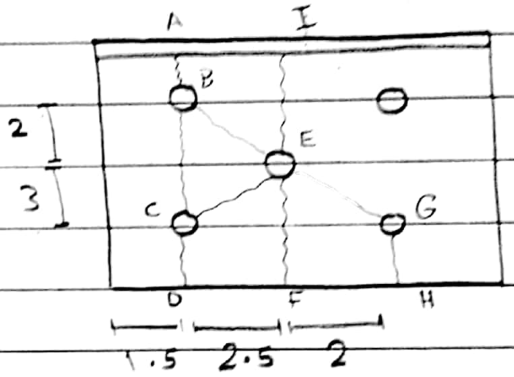
* calc I_x & I_y for the whole section.. $I_x = I_x + \boxed{Ad^2}$ → مش دايماً.

سنوات فيرست

Q1: (13 Points): for the figure below, the (tension) tie member section is L 8x6x1 & made of A992 steel, bolt diameter = 7/8 in

$f_y = 50$ $\quad$ $f_u = 65$





a) Calc tensile design strength (ϕP_n) for yielding. (1 Point)

sol $\Rightarrow \phi_t P_n = 0.9 f_y A_g = 0.9(50)(13) = 585$ kips

b) determine the controlling A_n for the section (5 Points)

sol: Path ABCD $= A_n = 13 - 2(1)(7/8 + 1/8) = 11$ in² (control).

Path ABECD $= A_n = 13 - 3(1)(7/8 + 1/8) + 1 * \frac{(2.5)^2}{4(2)} + 1 * \frac{(2.5)^2}{4(3)} = 11.3$ in²

Path ABEFG $= A_n = 13 - 3(1)(7/8 + 1/8) + 1 * \frac{(2.5)^2}{4(2)} + 1 * \frac{(2)^2}{4(3)} = 11.11$ in²

Path ABEN $= A_n = 13 - 2(1)(7/8 + 1/8) + \frac{1(2.5)^2}{4(2)} = 11.44$ in²

Path IEGH $= A_n = 13 - 2(1)(7/8 + 1/8) + 1 * \frac{(2)^2}{4(3)} = 11.33$ in²

c) Calc effective net area (A_e) (3 Points)

U → from case 8 = 0.6

↳ from case 2 $= 1 - \frac{1.65}{4.5} = 0.63$ (control)

$\Rightarrow A_e = 0.63\,(11) = 6.93 \ \text{in}^2$

d) based on a → c , determine the tensile design strength. (

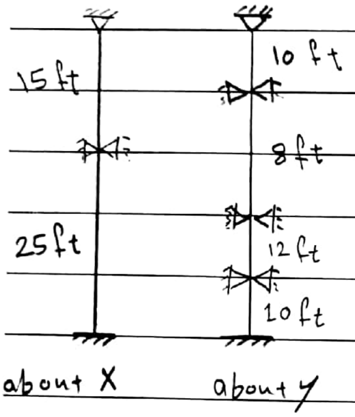
→ yielding = 585 Kips.

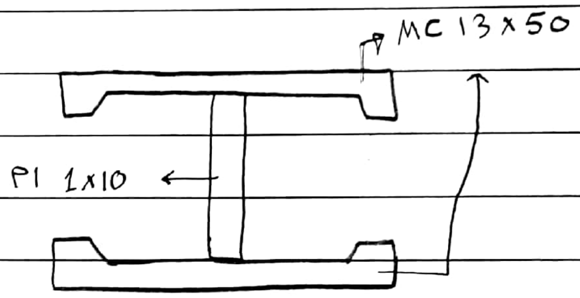
→ fracture = $0.75\,(65)\,(6.93) = 337.84$ Kips

$\Rightarrow \boxed{\phi_t P_n = 337.84 \text{ Kips}}$

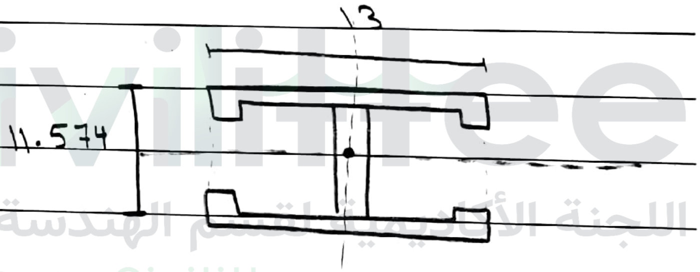
e) based on d , find the maximum value of P_n . (2 P

Q2: (15 Pt): for the figure below, the compression member is made of 2MC13X50 & Plate PL 1X10, The Column is made of steel $f_y = 48$, $f_u = 60$ (Neglect the local buckling).





a) determine radii of gyration (r_x & r_y). (6 Pts)



* الشكل symmetric لذا

السنترويد في المنتصف $\bar{y} = 5.787$

أخذنا قيمة I_y لأنه مبطوح

$$I_x = \frac{1(10)^3}{12} + 2(16.4) + 14.7\left(\frac{11.574}{2} - 0.974\right) = 186.88 \text{ in}^4$$

I_x

$$I_y = \frac{10(1)^3}{12} + 2(314) = 628.8 \text{ in}^3$$

$$\Rightarrow r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{186.88}{(1)(10) + 2(14.7)}} = 2.18$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{628.8}{10 + 2(14.7)}} = 3.995$$

b) according to above, which axis controls : (show your calculations) (3 Pts)

X : 1) $\frac{KL_x}{r_x} = \frac{1(15)*12}{2.18} = 82.57$

2) $\frac{KL_x}{r_x} = \frac{0.8(25)(12)}{2.18} = 110.1$ (control in x)

y 1) $\frac{KL_y}{r_y} = \frac{1(10)(12)}{3.99} = 30.08$

$\frac{KL_y}{r_y} = \frac{1(8)(12)}{3.99} = 19.2$

$KL/r_y = \frac{1(12)(12)}{3.99} = 36.1$ (control in y)

$KL/r_y = \frac{0.8(10)(12)}{3.99} = 24.06$

$\Rightarrow$ X-axis in the control with $KL_x/r_x = 110.1$

c) if the control $\frac{KL}{r} = 55$, find $\Phi_c P_n$. (5 Pts)

$\Rightarrow 55 < 4.71\sqrt{E/f_y} = 115.8$ ok

$\rightarrow F_{cr} = \left[0.658^{f_y/f_e}\right] f_y$, $f_e = \frac{\pi^2(29,000)}{(55)^2} = 94.62$ ksi

$F_{cr} = 38.82$ ksi

$\Rightarrow \Phi_c P_n = 0.9(38.82)(1*10+2(14.7)) = 1376.56$ Kips

d) based on c ↑ .. if $P_u = 1000$, is the member adequate? (1 Pt)

$\Phi_c P_n \geq P_u$

$1376.56 > 1000$ $\Rightarrow$ OK ✓ adequate.

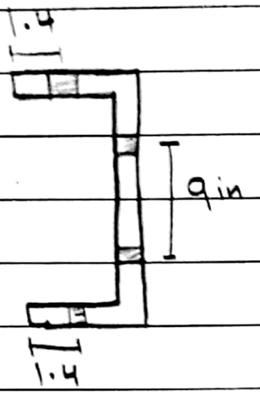
سنوات فيرست

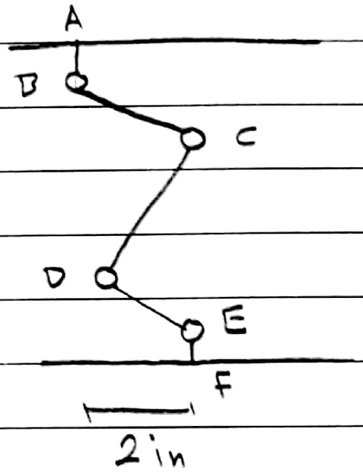
Q3: (10Pts): The tension member section is made of C15×50 with A36 steel & $d_B = \frac{1}{2}$ in.

$\downarrow$

$f_y = 36$

$f_u = 58$



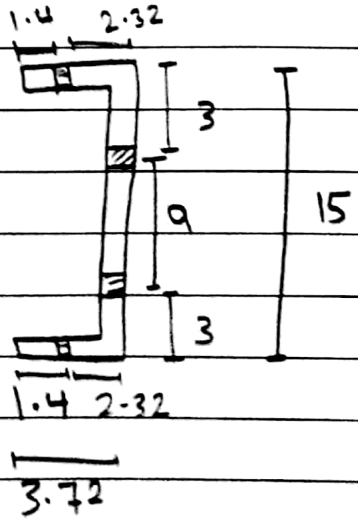


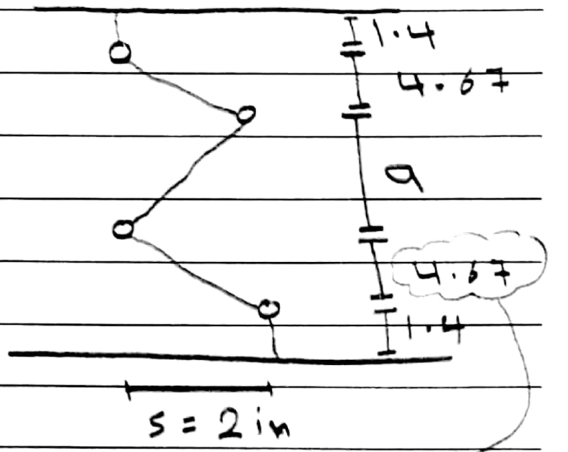
a) Calc the tensile design strength for yielding (1 Pt)

sol: $\phi_t P_n = 0.9 f_y A_g = 0.9(36)(14.7) = 476.28$ kips

b) determine the controlling net area An. (5 Pt)

* عبي هاي المعلومات عالرسمة من المانيول دغري





$3 + 2.32 - 0.65 = 4.67$

$\downarrow$

(t_f) السماكة الأقل

$$A_n = 14.7 - \overset{\text{٢ في ال } f}{2(0.65)}\left(\frac{1}{2}+\frac{1}{8}\right) - \overset{\text{٢ في ال } w}{2(0.716)}\left(\frac{1}{2}+\frac{1}{8}\right)$$

$$+ \underbrace{(0.716)\left(\frac{2^2}{4(9)}\right)}_{\text{الخط المائل CD}} + \underbrace{2\left(\frac{0.716+0.65}{2}\right)\left(\frac{2^2}{4(4.67)}\right)}_{\text{الخطين BC و DE}}$$

$A_n = 13.36 \text{ in}^2$

c) Calc effective net Area (2 Pts)

$U = 1$ لانه مثبت من ال f وال w معاً.

$\boxed{A_e = 13.36 \text{ in}^2}$

d) based on a → c, find controlling tensile design strength. (2 Pts).

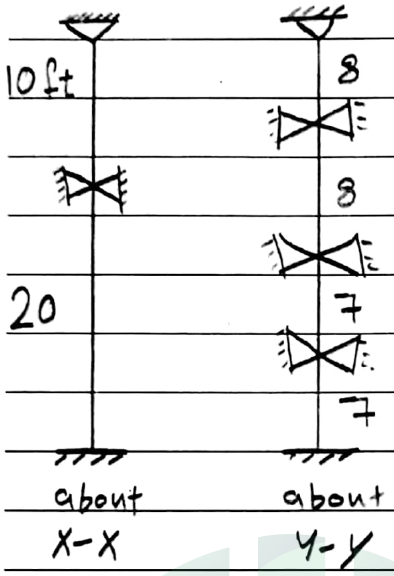
Yielding = 476.28 Kips

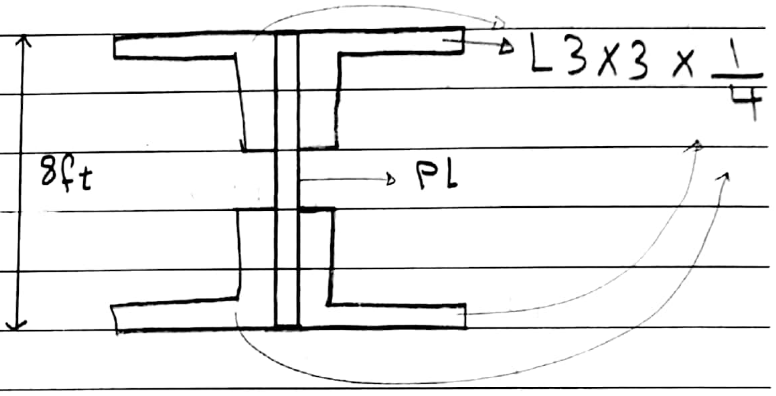
fracture: $\phi_t P_n = 0.75 f_u A_e$

$= 0.75(58)(13.36) = 581.16$ Kips

Controlling $\phi_t P_n = 476.28$ Kips

Q4: (15 Pts): a compression member has a built up section as shown in figure below, The built up section is made of four angles L 3x3x1/4 and Plate that has width of 8 in, The steel grade used is ($f_y = 55$ Ksi, $f_u = 70$ Ksi)





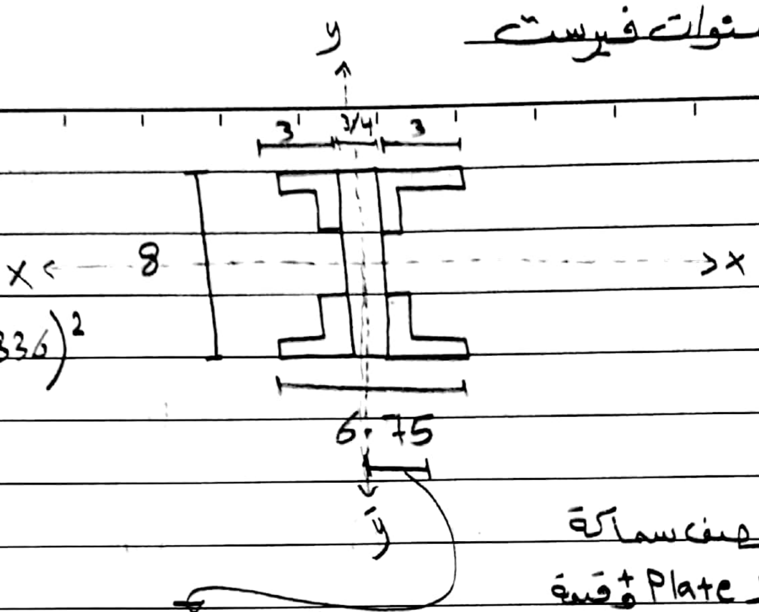
a) if the ultimate compression force on the column is equal to 500 Kips, & assume $\Phi_c F_{cr} = 42.52$ Ksi, find the thickness of the plate (PL). (3 Pts)

sol: $P_u = \Phi_c P_n \Rightarrow \Phi_c P_n = \Phi_c F_{cr} A_g$

$500 = 42.52 * \underset{\substack{\downarrow \\ \text{4 angles.}}}{4(1.44)} (8 * t) \Rightarrow t = 0.255$ in

b) if the used Plate is PL 3/4 X 8, Determine the radii of gyration with respect to the centroidal axis x & y (r_x & r_y).

sol: symmetric $\Rightarrow \bar{y} = 4$ (at the center)



$$I_x = \frac{0.75\,(8)^3}{12}$$

$$+ 4(1.23) + 4(1.44)(4-0.836)^2$$

$$I_x = 94.58 \text{ in}^4$$

نصف سماكة الـ Plate + قيمة $\bar{X}$ هي المسافة بين سنترويد الـ Angle وسنترويد الشكل كامل في الـ y.

$$I_y = \frac{8\,(0.75)^3}{12} + 4(1.23)\left(\frac{0.75}{2} + 0.836\right)^2$$

$$I_y = 7.497 \text{ in}^4$$

$$\Rightarrow r_x = \sqrt{I_x/A} = \sqrt{\frac{94.58}{4(1.44) + 8 * 0.75}} = 2.836$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{7.497}{4(1.44) + 8 \times 0.75}} = 0.798$$

c) if the used plate is 3/4 X 8, which axis is control, show your calcs, (3 Pts).

x: 1) $KL/r_x = 1(10)(12)/2.836 = 42.3$

2) $KL/r_x = 0.8(20)(12)/2.836 = 67.7$ (control in x)

y: 1) $KL/r_y = 1(8)(12)/0.798 = 120.3$ (control in y)

2) $KL/r_y = 0.8(7)(12)/0.798 = 84.2$

$\Rightarrow$ y-axis is the control with $KL/r_y = 120.3$

d) if the used plate is PL 3/4 x 8 & the control $\frac{KL}{r} = 90$ & the steel is ($F_y = 55$, $F_u = 70$) find the design compressive strength of the member. (4 Pts).

Sol: $KL/r = 90 < 4.71\sqrt{E/f_y} = 108.15$ (inelastic)

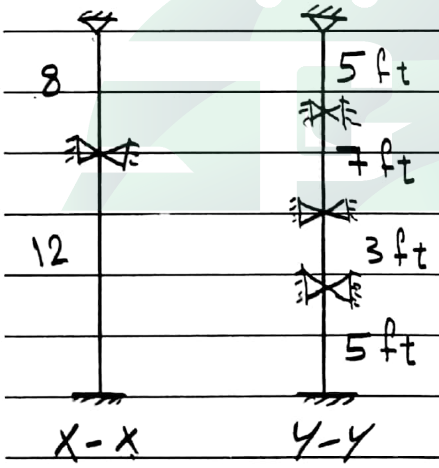
$\Rightarrow F_{cr} = \left[0.658^{f_y/f_e}\right] f_y$, $f_e = \frac{\pi^2 (29{,}000)}{(90)^2} = 35.34$ Ksi

$\Rightarrow F_{cr} = 28.67$ Ksi

$\Rightarrow \Phi_c P_n = 0.9 (F_{cr}) A_g = 0.9 (35.34)(4(1.44) + 8 * 0.75) = 374$ Ksi.

Q5: W14 x 145 section, Steel ($f_y = 50$ Ksi)

Determine $\Phi_c P_n$ using table 4-1 only. (5 Pts)



Solution: check which axis is control:

$x: KL/r_x = 1(8)(12)/6.33 = 15.41$

$: KL/r_x = 0.8(12)(12)/6.23 = 18.49$ ✓

$y: KL/r_y = 1 * 7 * 12 / 6.23 = 13.48$

$\Rightarrow$ X-axis is the control axis

$\Rightarrow$ equivalent in $y = \frac{KL_x}{r_x/r_y} = \frac{0.8(12)}{1.59} = 6.04$ ft.

KL		$\Phi_c P_n$
6	$\longrightarrow$	1870
7	$\longrightarrow$	1860

by interpolation at KL = 6.04 ft $\rightarrow \Phi_c P_n = 1869.6$ kips

Second

Design of compression members.

*Design Procedure :

get P_u (from structure) , $\Phi_c P_n = \Phi_c f_{cr} A_g$.

1) assume $f_{cr} = [2/3 - 1] f_y$ or $\frac{KL}{r} = f_y$ (& get $\Phi_c f_{cr}$ from table 4-22)

في الإمتحان هو بعطيك
القيمة اللي تفرضها .

2) $A_g \geq \frac{P_u}{\Phi_c f_{cr}}$ → get a section (Manual Part 1)

لا تختار واحد عليه حرف c

3) check $KL/r < 200$ ok

→ $KL/r \leq 4.71\sqrt{E/f_y}$ → $f_{cr} = \left[0.658^{f_y/f_e}\right] f_y$

→ $KL/r > 4.71\sqrt{E/f_y}$ → $f_{cr} = 0.877 f_e$

where $f_e = \pi^2 E / (KL/r)^2$

4) get $\Phi_c P_n$, compare with P_u, if not ok

sellect a larger section & repeat ↑ ..

5) check local buckling : $\lambda_w < \lambda_r$ ok ✓.
$\lambda_f < \lambda_r$

*طريقة أخرى للديزاين : (مهم جداً لإنه هو بيحددلك الطريقة)

* using table 4-1 (للمقاطع الأمريكية فقط)

1) assume KL_y controls → get a section

2) check which axis is the control, if y-control, ✓

* if x-control: → equivalent = $\frac{KL_x}{(r_x/r_y)}$

* if equivalent > control → get a new section using equivalent value.

Example: Select a W18 shape of A992 steel ($f_y = 50$, $f_u = 65$) to resist service dead load = 100 kips & service live load = 300 kips, & the effective Length KL = 26 ft.

Sol: $P_u = 1.2(100) + 1.6(300) = 600$ kips

⇒ assume $F_{cr} = 2/3\,(50) = 33$ ksi

بالامتحان هو بيعطيك القيمة

$A_g \geq \frac{P_u}{\phi_c F_{cr}} = \frac{600}{33(0.9)} \rightarrow A_g \geq 20.2$

*نختار أصغر W18 له Ag أكبر من 20.2 ولا يوجد عليه c .

⇒ Try W18×71 , $A_g = 20.8$

get $KL/r_y = \frac{26*12}{1.7} = 183.5 < 200$

هي الـ control
لانه مافي bracing

$KL/r_y = 183.5 > 4.71\sqrt{E/f_y} = 113.4$

$\Rightarrow F_{cr} = 0.877 F_e \quad , \quad F_e = \frac{\pi^2 (29{,}000)}{(183.5)^2} = 8.5$

$F_{cr} = 7.45$ ksi

$\Rightarrow \phi_c P_n = 0.9(7.45)(20.8) = 139.6 < 600$ (very bad)

* في هذه الحالة نكبر السكشن ونختار اللي فوقه وكمانه نحاول لنوصل لواحد OK . في الإمتحان من مرة وحدة بطلع معك لانه هو بعطيك قيمة الفرض Fcr .

في السلايدات غير قيمة Fcr لقيمة اقل $\Rightarrow$ $F_{cr} = 20$ ksi

$\hookrightarrow A_g \geq 33.3\ in^2$

Try W18x119 $\quad A_g = 35.1$

$* \ KL/r_y = \frac{26*12}{2.69} = 116 > \boxed{113} \rightarrow 4.71\ KL/r_y$

$\Rightarrow F_{cr} = 0.877\left(\frac{\pi^2(29{,}000)}{\left(\frac{26*12}{2.69}\right)^2}\right) = 18.66$ ksi

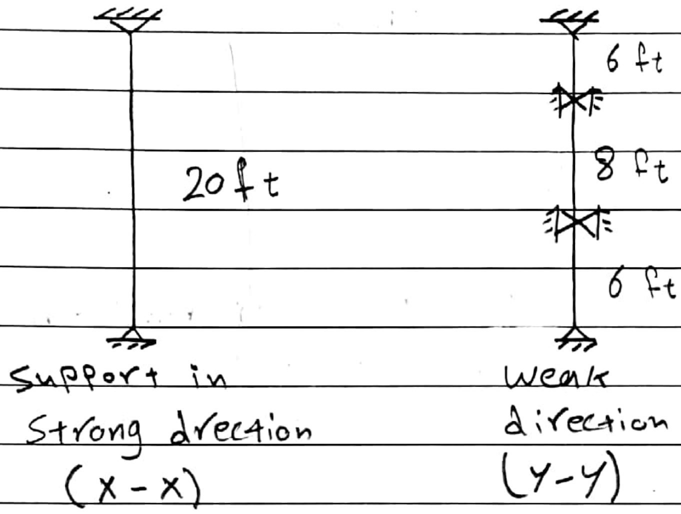
$\Rightarrow \phi_c P_n = 0.9(18.66)(35.1) = 589.6 < 600$ (Neglect)

اللي اكبر منه دغري اكيد رح يزبط

$\Rightarrow$ Try W18x130 , $A_g = 38.2$

$\phi_c P_n = 646 > 600$ OK ✓ use W18x130

Example 2: The column shown is subjected to an ultimate load of 840 Kips, use A992 sellect a W-Shape. ($f_y = 50$, $f_u = 65$)



Design → using table 4-1

Sol: assume y-controls : $KL_y = 1(8) = 8$ (control).

$KL_y = 1(6) = 6$

* Select a section with $KL = 8$ & $\phi_c P_n > 840$.

في الإمتحان يجب أنه يحددلك W14 أو W12 أو W10 أو ...

So: W14 x 74 $\phi_c P_n = 878$
W12 x $\boxed{72}$ $\phi_c P_n = 884$
W10 x 77 $\phi_c P_n = 922$
→ كلهم safe ولكن نختار الأخف

→ W12 x 72 , $\phi_c P_n = 884$ Kips

Check which axis controls: $\frac{KL_y}{r_y} = \frac{8 * 12}{3.04} = 31.57$

$\frac{KL_x}{r_x} = \frac{20 * 12}{5.31} = 45.2$ (control)

⇒ X- is the control axis :

$$\Rightarrow \text{equivalent} = \frac{KL_x}{r_x/r_y} = \frac{20}{\left(\frac{5.31}{3.04}\right)} = 11.45 > 8$$

اللي اشتغلنا عليها قبل

⇒ select a new section : (اختار اللي اكبر من اللي كنا مختارينه قبل)

Try W12x79

KL	$\phi_c P_n$
11 →	910
12 →	887

at 11.45 ⇒ by interpolation, $\phi_c P_n = 899.65$

$\approx 900 > 840$

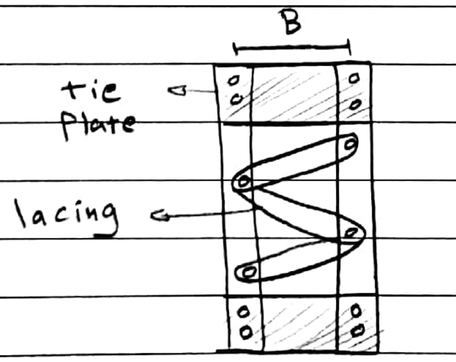
ok.

use W12x79

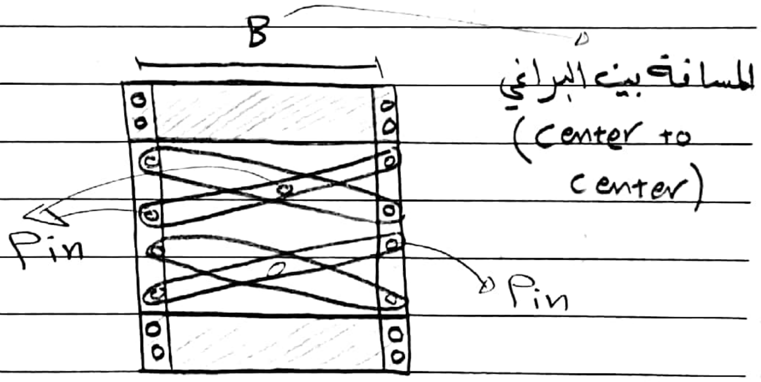
* Design of lacings and ties

*تستخدم لتثبيت اجزاء ال (built up section) مع بعضها.

two types
- single lacing
- double lacing

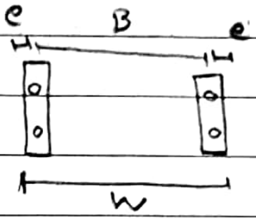


* single lacing



* double lacing

* Procedure of Design :

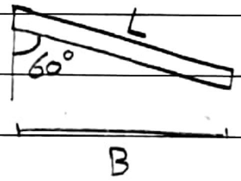


if B. not given ⟿ B = W − 2e

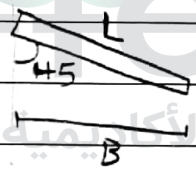
Where (e) is the edge distance, get e_{min} from manual (table J3.4) بعد جداول الـ U والـ shear lag بكم صفحة

↳ depends on bolt diameter (d_B) : ضع قيمة e الأكبر من الجدول

* if B < 15 → Single lacing → $\beta = 60°$ → get (L) Length of the lacing



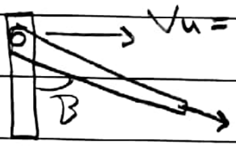
* if B > 15 → double lacing → $\beta = 45°$ → get (L) Length of lacing.



* get ultimate compression force Horizantally on the column

$\{V_u = 0.02\phi_c P_n\} \xrightarrow{\div 2}$ on each side (حسب على كم جهة رح نحط منهم)

$V_u \longrightarrow$ ⇒



$\to V_u = F\sin\beta$

get $\underline{\underline{F}}$

F : force on lacing.

* calc I , A (كرموز) → get $r = \sqrt{I/A}$ بدلالة t

$\frac{KL}{r}$ = maximum 140 (if single) → get (t) وخلي قيمتها مقبوله في الحياة العملية

200 (if double)

⇒ get new $\left(\frac{KL}{r}\right)$ check if $\overset{<}{>}$ $4.71\sqrt{E/f_y}$

* get Fcr ⇒ $A_g = \frac{F}{\phi F_{cr}}$ → force on the lacing

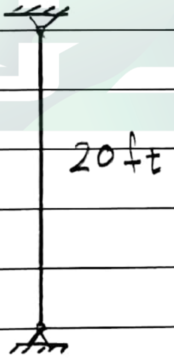
⇒ from Ag get t & b of the lacing #

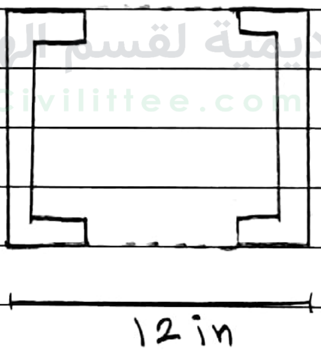
* Design of the tie plate (مباشر)

→ $t = B/50$
→ $L = B$ ⇒ use $t \times L \times W$
→ $W = B + 2e$

Example: *) Design the lightest C12
*) Design lacing & Plates

* $P_{live} = 300$, $P_{Dead} = 100$, $B = 8.5$ (given)
$F_y = 50$ ksi , $d_B = 3/4$ in





Sol: $P_u = 1.2(100) + 1.6(300) = 600$ kips.

assume $KL/r = 50$

(زي ما حطينا قبل
هذا الفرض
بيكون معطى)

⇒ from Manual (4-22) → $\phi F_{cr} = 37.5$

$A_g \geq \frac{P_u}{\phi_c f_{cr}} \geq \frac{600}{37.5}$, A_g (total) = 16 in²

* A_g for 1 C ⇒ $A_g \geq 8\ in^2$

⇒ Try C12×30 , $A_g = 8.81$

to get KL/r → calc I_x & I_y (its a built up section)

$I_x = 2(162) = 324\ in^4$

$I_y = 2(5.12) + 2(8.81)(6 - 0.674)^2 = 510.1\ in^4$

$$\Rightarrow KL/r_x = \frac{20*12}{\sqrt{\frac{324}{2(8.81)}}} = 55.97 \quad \text{(control)}$$

$$KL/r_y = \frac{20*12}{\sqrt{\frac{510.1}{2(8.81)}}} = 44.6$$

$$\Rightarrow 55.97 < \overset{113}{4.71\sqrt{E/f_y}} \rightarrow F_{cr} = \left[0.658^{f_y/f_e}\right] f_y$$

$$f_e = \frac{\pi^2(29{,}000)}{(55.97)^2} = 91.37\ ksi \Rightarrow F_{cr} = 39.76\ ksi$$

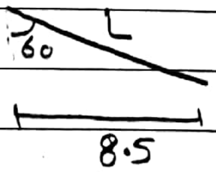
$\Rightarrow \Phi_c P_n = \phi F_{cr} A_g = 630.6$ kips > 600 ok ✓

⇒ use 2 C12×30

(*) Design of lacing :

B = 8.5 < 15 → single lacing → use B = 60°

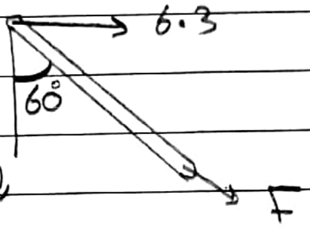
* Length of the lacing ⇒



$\sin 60 = \frac{8.5}{L}$ $\boxed{L = 9.81\ in}$

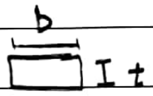
$V_u = 0.02\,(630.6) = 12.6$ Kips (total)

on each side → $V_u = 6.3$ Kips



$\Rightarrow 6.3 = F \sin 60 \Rightarrow F = 7.27$ Kips

Cross section of the lacing:



$I = \dfrac{bt^3}{12}$, $A = bt$

$\Rightarrow r = \sqrt{I/A} = \sqrt{\dfrac{\frac{bt^3}{12}}{bt}} = \sqrt{\dfrac{t^2}{12}} = \dfrac{t}{\sqrt{12}}$

assume $KL/r = 140$ (max value).

$\dfrac{1\,(9.81)}{(t/\sqrt{12})} = 140 \longrightarrow t = 0.2427$ in

$\Rightarrow$ use $\boxed{t = 0.25 \text{ in}} \longrightarrow$ New $\dfrac{KL}{r} = 135.9 > 4.71\sqrt{\dfrac{E}{F_y}}$

$\Rightarrow F_e = \dfrac{\pi^2 (29{,}000)}{(135.9)^2} = 15.5 \longrightarrow F_{cr} = 12.96$ ksi

of lacing $\Rightarrow A_g = \dfrac{F}{\phi F_{cr}} = \dfrac{7.27}{0.9\,(12.96)} = 0.623 \text{ in}^2 \rightarrow t = 0.25$

$b = 2.493$

من المانيوال

$*$ Min Length $= L + 2e = 9.81 + 2(1.25) = 12.3$ in

استخدمها 14 لغايات جمالية وعملية

→ use $0.25 \times 2.5 \times 14$

$t \times b \times L$

*Design of tie plate:

$t = 8.5/50 = 0.17$
$L = 8.5$
$W = 8.5 + 2(1.25) = 11$

use tie $0.17 \times 8.5 \times 11$

في السلايدات ⇒ $3/16 \times 8.5 \times 12$

عشان تكون عالقدر بالزبط (لغايات جمالية فقط).

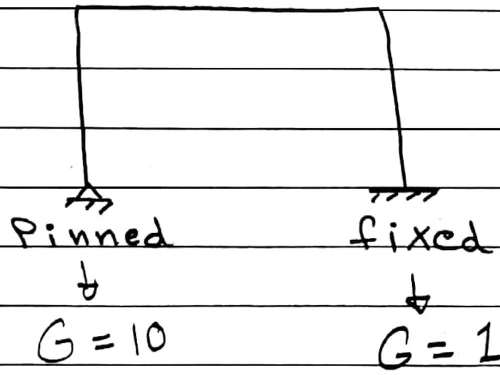
Effective Length using alignment charts.

→ to calculate the accurate $\underline{\underline{K}}$ exactly: (more economy)

*نقوم بحساب G لكل عامود عند الـ top (a) والـ bottom (b) ثم من خلال هذه القيم الناتجة نحصل على قيمة K_x من الـ charts في المانيوال حسب اذا كان فيه sidesway أو لا .. (السؤال بيحدد لك)

$$* \quad G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)}$$

c : column
g : girder
I : $\underline{\underline{I_x}}$ نأخذ



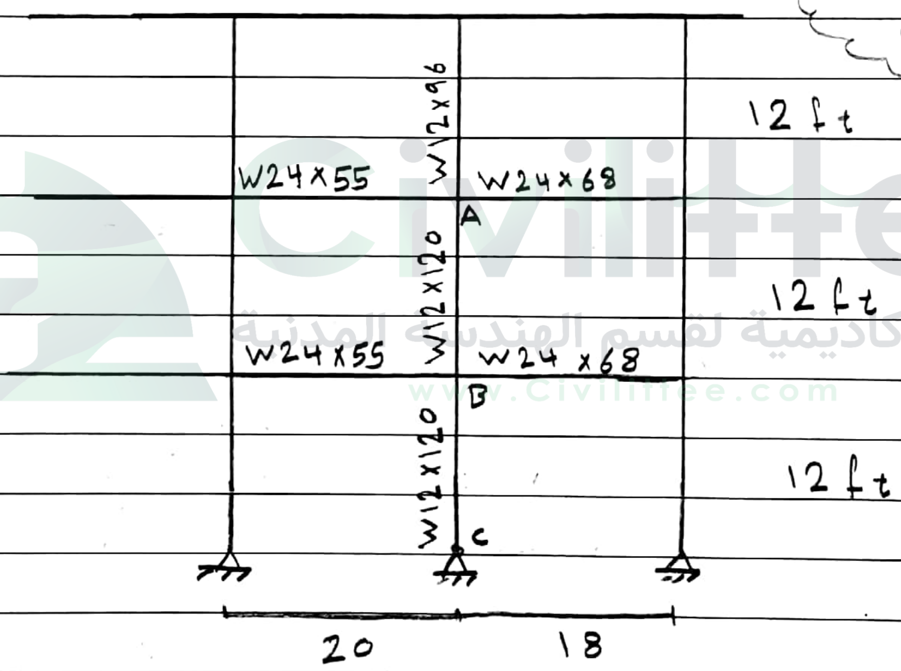
* ال charts موجودين في المانيوال بعد جداول ال U و K بأكم صفحة

* اذا كانة ال frame ← braced يعني ما فيه Sides way
تكون اكبر قيمة لـ K هي 1 واقل قيمة 0.5

* واذا كانة العكس اكبر قيمة لـ K هي 20 واقل قيمة 1

Example : unbraced frame
find Kx for colums AB, BC

* يعني فيه
Sides way
K > 1



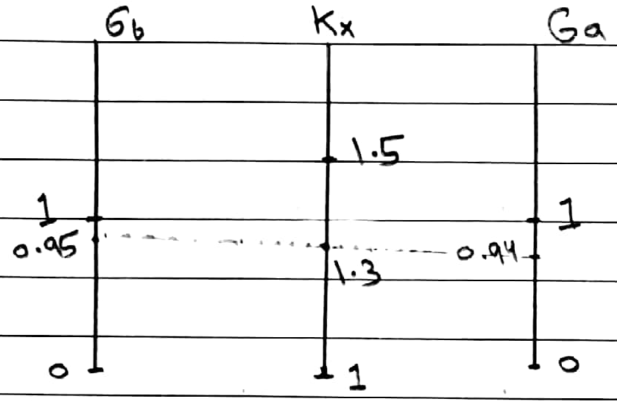
Sol: for column AB.

$$G \text{ at } A \text{ (top)} = G_a = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)}$$

$$G_a = \frac{\frac{833}{12} + \frac{1070}{12}}{\frac{1350}{20} + \frac{1830}{18}} = 0.94$$

G at B (bottom) $G_b = \dfrac{\dfrac{1070}{12} + \dfrac{1070}{15}}{\dfrac{1350}{20} + \dfrac{1830}{18}} = 0.95$

*from the chart:



⇒ $\boxed{K_x = 1.3}$

ولو قيمة ثانية قريبة عادي.

* for column BC

G_a top = 0.95 (↑↑) at point B.

G_b (at C) = 10 (Pin)

⇒ from the chart: $\boxed{K_x = 1.85}$

* Inelastic

⇒ You should check $\dfrac{K_x L}{r_x}$ if $\overset{\text{أقل}}{<}$ $4.71\sqrt{E/f_y}$

(inelastic) ⟶ get τ_a (table 4-21 من المانيوال)

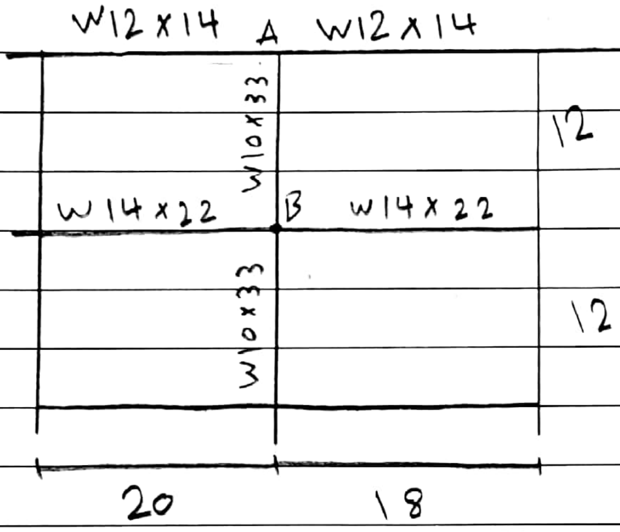
* صفحة وحدة قبل table 4-22 بالزبط

New $G_a = \tau_a G_a$

$G_b = \tau_a G_b$

Example: Unbraced frame → K > 1

* Determine Kx for member AB
* DL = 35.5 kips , LL = 142 kips
* A992 steel ($f_y = 50$, $f_u = 65$)



frame جزء من ال *

Sol:

AB

* at A: $G_a = \dfrac{\frac{171}{12}}{\frac{88.6}{20} + \frac{88.6}{18}} = 1.52$

$G_b = \dfrac{\frac{171}{12} + \frac{171}{12}}{\frac{199}{20} + \frac{199}{18}} = 1.36$

→ $K_x = 1.45$ (for elastic)

* K_xL_x/r_x (for AB) $= \dfrac{1.45(12)\overset{\text{→ to inches}}{(12)}}{4.19} = 49.8 < 4.71\sqrt{E/F_y}$

* from table 4-21 : $\dfrac{P_u}{A_g} = \dfrac{1.2(35.5) + 1.6(142)}{9.71} = 27.79$

P_u/A_g	τ_a
27	0.835
28	0.804

$F_y = 50$.

⇒ by interpolation ... $\tau_a = 0.81051$

New: $G_a = 0.81051(1.52) = 1.23$

$G_b = 0.81051(1.36) = 1.1$

→ $K_x = 1.35$

inelastic

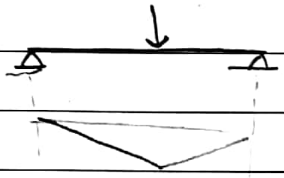
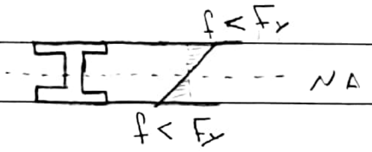
Chapter 5

* Analysis & Design of Beams
→ bending

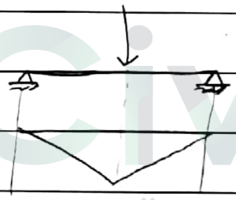
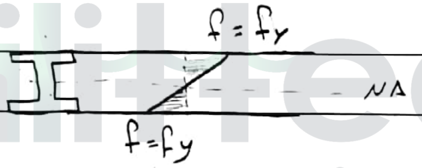
Beam : member only with lateral (transverse) loads only

→ Pure bending : stress = $\frac{My}{I}$ only.

* زيادة ال load تدريجياً :

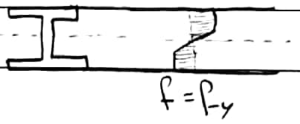
①  

(No fail)

② Increase load : (at yielding)
 

yielding
عالأطراف
(No fail)

* $\boxed{M_y = F_y S_x}$ → from manual Part 1. $\left(S_x = \frac{I_x}{c}\right)$

③ Increase load : (>yielding)
 

(No fail)

* جزء منه plastic وجزء elastic

④ Plastic moment (at P_u) : 
full plasticity (كله صار elastic)

(fail)

* $\boxed{M_p = f_y Z_x}$ → from manual Part1

أعلى قيمة ممكن يوصلها

* في المرحلة ④ في منطقة الـ maximum moment عند الـ full plasticty بتكون عندي ما يسمى plastic hinge .

عندها يحدث الـ failure

→ **Plastic** neutral axis

يكون في السنترويد فقط اذا كانه symmetric .

* Location of plastic neutral axis : at $\boxed{\underset{\text{compression}}{A_c} = \underset{\text{tension}}{A_t}}$

* $M_p = F_y Z_x$

اذا كانه السكشن مش أمريكي

نحسبها حساب .. ونحسب بالأول $\bar{y}$ وهي مكانه PNA

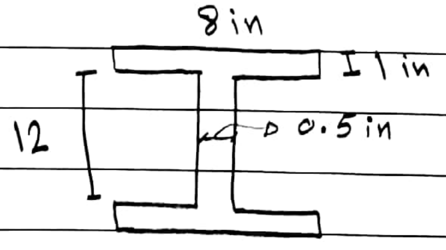
* Steps : 1) get $\bar{y}$ from $A_c = A_t$

2) get $Z_x = \sum A_i \bar{y}_i$ ⟵ بعد كل area عن موقع الـ PNA

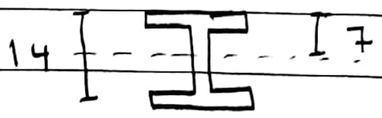
Example: (Not american shape) , steel G572 ($f_y = 50$)

find 1) S_x , M_y

2) Z_x , M_p



Sol: 1) $S_x = \frac{I_x}{c}$,



$c = 7$

$$I_x = \left(\frac{8(1)^3}{12} + (8*1)(6.5)^2 \right) * \underset{\downarrow}{2} + \frac{0.5(12)^3}{12} = 749.4 \text{ in}^4$$

two flanges

$$\boxed{S_x = 107 \text{ in}^3}$$

$* M_y = f_y S_x = 50(107) = 5350$ kip-in

$5350/12 = \boxed{446 \text{ kip-ft}} \rightarrow$ وحدة المومنت (مهم)

2) $A_c = A_t \rightarrow \underline{y} = 7$ (becouse symmetry)

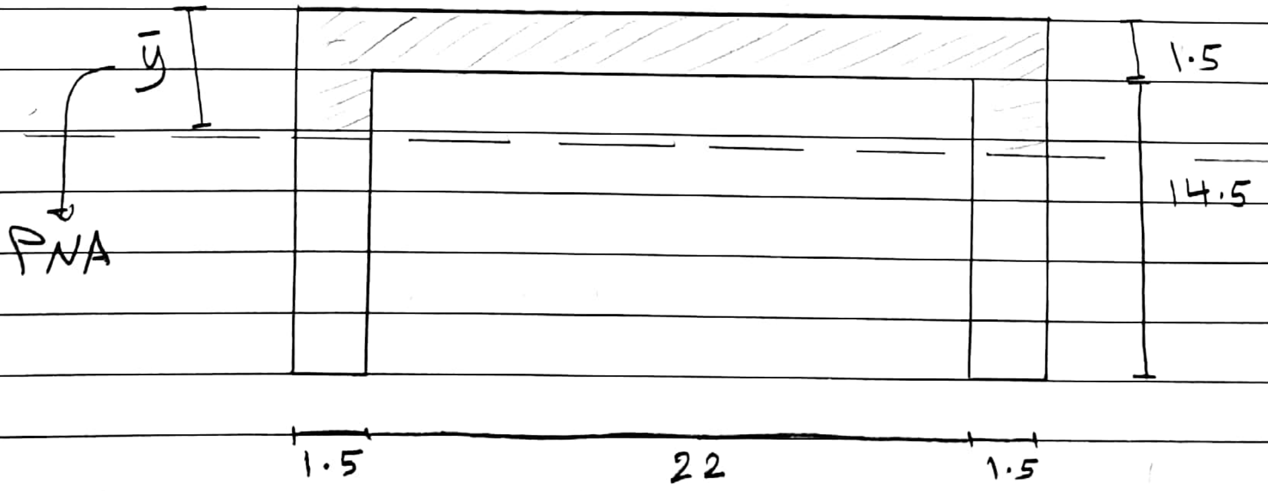
$Z_x = \Sigma A_i y_i$ (for comp & tension areas)

$= 2(8*1*6.5) + 2(6*0.5*3) = 122$ in^3

$M_p = F_y Z_x = 50(122) = 6100$ kip-in

$= 508.3$ kip-ft.

Example: find Z_x ... not symmetry.



Sol: to get $\bar{y}$ ... $A_c = A_t$

$$22*1.5 + 2*(1.5*y) = (16-y)(1.5)(2)$$

$$\bar{y} = 2.5 \text{ in}$$

* $Z_x = \sum A_i y_i = 22*1.5(2.5-0.75) + 2.5(1.5)(1.25)*2$

$+ 1.5*13.5*2*\left(\frac{13.5}{2}\right)$

كل area مضروبة ببعد السنترويد تبعها عن ال PNA

$$Z_x = 340.5 \text{ in}^3 .$$

* اذا كان الشكل built up و symmetric وكان جزء منه مقطع أمريكي نأخذ قيمة Z_x لهذا الجزء من المانيوال.

* Lateral torsional buckling.

* تأثيرها في الجزء العلوي (compression) من السكشن وهذا الجزء هو الذي يتم تدعيمه في (cantiliver و simply supported)

* اذا دفنت هذا الجزء في ال concrete يعني إنك انتهيت من LTB

* وممكن تعمل تثبيت براغي والمسافة بينهم L_b على مسافات معينة.

* $M_n = M_p$ (if embeded in concrete or L_b is very short)

M_n ↓ strength (capacity)

embeded ← $L_b = 0$

very short ↓ احسن كل ما كانت أقصر

Five Apple

L_b : unbraced Length of comp flange (between bracings)

⟹ Classifications of shapes (in terms of compactness)

[1] Compact section : *ما عندي مشاكل بالسكشن (أحسن اشي)

$\lambda_f \leqslant \lambda_{p_f}$, $\lambda_w \leqslant \lambda_{p_w}$

[2] Non-compact sections : (مقبول وأوكي بس مش مثل ١)

$\lambda_p < \lambda_f < \lambda_r$, $\lambda_p < \lambda_w < \lambda_r$ اذا واحد منهم تحقق يعني non-compact.

[3] Slender : تعيس جداً (الأسوأ)

*لحساب λ_p و λ_r لا ترجع للمانيوال + اكتبهم عالشيت في الامتحان

	λ	λ_p	λ_r
Flange	$b_f/2t_f$	$0.38\sqrt{\frac{E}{f_y}}$	$\sqrt{E/f_y}$
Web	h/t_w	$3.76\sqrt{\frac{E}{f_y}}$	$5.7\sqrt{\frac{E}{f_y}}$

compact | non compact | Slender

λ_p λ_r

*اذا كان مكتوب فوق اسم السكشن حرف f في المانيوال يعني non compact.

*دائماً تكون المشكلة في الـ flange لذا نحسب λ للـ flange فقط.

* اذا كانت السكشن compact نحسب ϕMn من خلال LTB

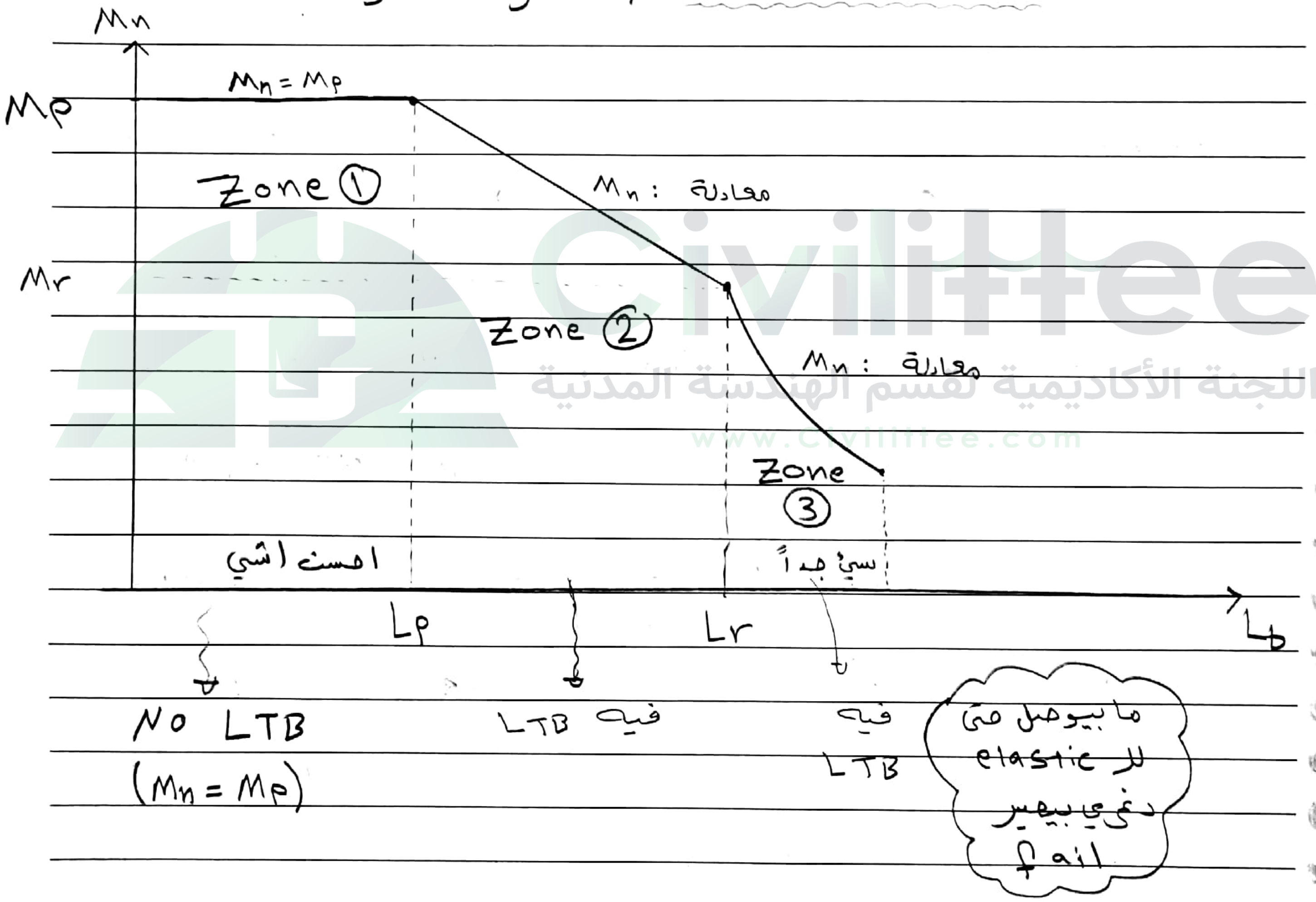
زي ما رح نشوف قدام شوي . Zone 1, 2, 3

* اذا كانت non-compact نحسب ϕMn من خلال LTB و FLB كمانه

FLB ← معادلة وحدة .

LTB ← Zone 1, 2, 3

*Bending Strength of Compact Shapes:



☒ Zone ①: $\phi M_n = \phi M_p$, $\phi M_p = \phi f_y Z_x$ $(\phi = 0.9)$

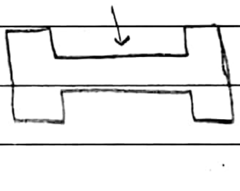
$\hookrightarrow L_b < L_p$

$\rightsquigarrow$ from manual Part (3-2) جداول الـ Z_x

قانونه *

$$\boxed{L_p = 1.76\, r_y \sqrt{\frac{E}{f_y}}}$$ #

*ملاحظة :



مستحيل إنه تطلع السكشن مثل هيك
لأنه الـ ϕM_n ستصبح قليلة جداً.

Example 1: W16X31 , A992 steel $(f_y = 50,\ f_u = 65)$

*Continuous lateral support $(L_b = 0$ (Zone 1))

is it adequate in flexure or not!

$W_D = 450$ lb/ft (exclude self weight)

$W_L = 550$ lb/ft

Sol: $W_u = 1.2D + 1.6L$

$= 1.2(450 + \underset{s.w}{31}) + 1.6(550) = 1457.2$ lb/ft

$= 1.4572$ kip/ft

$M_u = WL^2/8 = (1.4572)(30)^2/8 = 163.9$ kip-ft

*Check compactness: $\lambda_f = 6.28$, $\lambda_{p_f} = 0.38\sqrt{E/f_y}$

$= 9.15$

$\Rightarrow \lambda_f < \lambda_{p_f}$ (compact)

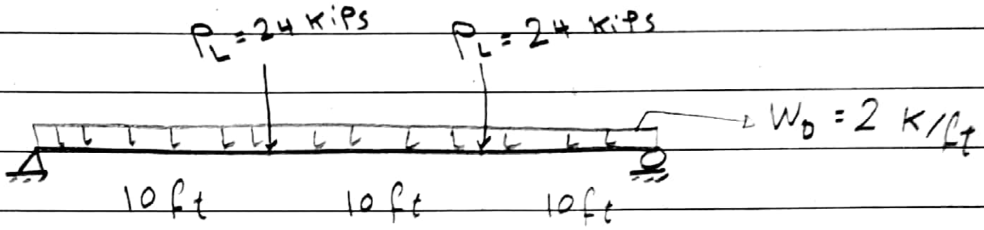
*Continuous lateral support $\Rightarrow L_b = 0$ (Zone 1)

Then: $\phi M_n = \phi M_p = 0.9\underset{f_y}{(50)}\underset{Z_x}{(54)}/12 = 202.5$ kip-ft

$\phi M_n > M_u \longrightarrow$ OK ✓ adequate

Example2: $F_y = 50$, continuous lateral support ($L_b = 0$ Zone 1)

Design



*Sol: get M_u لايجاد قيمة M_{max} نرجع للمانيوال tables (3-23)

ونبحث عن حالة ال load مثل الموجودة في السؤال ونعوض في قانونه M_{max}.

(أو أنه نقوم برسم شير ومومنت وحساب قيمة M_n)

*using case 1 + case 9

$$M_u = \frac{WL^2}{8} + Pa = \frac{2\overset{\text{dead لانها}}{(1.2)}(30)^2}{8} + 24\overset{\text{live load}}{(1.6)}(10)$$

$M_u = 654$ kip-ft

والآن نختار سكشن من جداول Z_x (table 3-2) حيث نبحث عن سكشن له $\phi_b M_{px}$ اكبر أو تساوي M_u ونختار أول سكشن في المجموعة ال (bold) لانه يكون هو الأخف وهيك طلب السؤال design the lightest section.

→ use W24x68 ($\Phi M_n = 664$ kip-ft)

*Check M_u adding self weight

$$M_u = \frac{1.2(2+0.068)(30)^2}{8} + 1.6(24)(10) = 663.18 \text{ Kip-ft}$$

Still $M_u < \phi M_n$ → use W24x68 ✓

$663 < 664$

* ملاحظة

لو لو ما زبط نختار الـ bold اللي اكبر منه وهو W 24x76

$\boxed{*}$ Zone 2: $L_p < L_b < L_r$

$$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7 f_y S_x h_o}{EJC}\right)^2}}$$

L_r ↓ اكتبه على الشيت ↓ أو من جداول Z_x (3-2)

* كل اشي بتجيبه من المانيوال Part 1

* قيمة c دائماً تساوي 1 .

$$* \; M_n = C_b\left[M_p - (M_p - 0.7 F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p$$

(بشكل عام)

use ↓ this

$$\phi M_n = C_b\left[\phi M_p - BF(L_b - L_p)\right] \leq \phi M_p$$

مقطع أمريكي

* من جداول الـ Z_x نأخذ قيمة L_p و L_r و BF و ϕM_p

C_b : modification factor (for each L_b) (for non-uniform moment)

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_a + 4M_b + 3M_c} \leq 3$$

M_a : M at 1/4 L_b

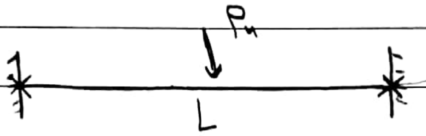
M_b : M at 1/2 L_b

M_c : M at 3/4 L_b

* في شيت لبعض حالات الـ loads محسوبة قيمة C_b جاهزة

لازم تكون معك في الامتحان ↓

Example: bracing at ends :



→ lateral bracing

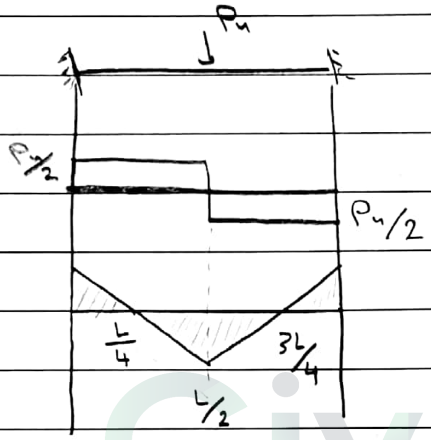
(لاتؤثر على رسمة الـ V والـ M)

(قيمة $L_b = L$ لأنه

مثبت عالأطراف فقط)

Calculate C_b :

Sol: من شيت الحالات جاهزة وقيمتها ١.92 بس هيك طريقة حسابها



$* \; M_u = \frac{PL}{8}$ (from structure or Manual 3-23 Case 16)

$M_A = 0$, $M_b = PL/8$, $M_c = 0$.

$$\Rightarrow C_b = \frac{12.5(PL/8)}{2.5(PL/8) + 3(0) + 4(PL/8) + 3(0)} = 1.923$$

قيمة M_a, M_b, M_c نحسبهم من معادلة M_x بتعويض قيمة X =

$\left.\begin{matrix} \frac{1}{4} L_b \\ \frac{1}{2} L_b \\ \frac{3}{4} L_b \end{matrix}\right\} = X$

Example: W12x30 , A992 steel ($F_y = 50$, $F_u = 65$)
$L_b = 10$, $C_b = 1$

Compute Flexural design strength:

__Solution:__ *Check compactness

$\lambda_f = 7.41$, $\lambda_{p_f} = 0.38\sqrt{E/f_y} = 9.15 \Rightarrow$ __Compact__ ($\lambda < \lambda_p$)
(No F)

$Z_x = 43.1$

* من جداول Z_x دور على السكشن حسب قيمة Z_x وطلع قيمة L_p, L_r وقيمة BF و ϕM_p

$\Rightarrow L_b = 10$, $L_p = 5.37$, $L_r = 15.6$, $BF = 5.89$
$\phi M_p = 162$

$L_p < L_b < L_r \Rightarrow$ Zone $\underline{2}$.

$\phi M_n = C_b\left[\phi M_p - BF(L_b - L_p)\right] \leq \phi M_p$ (لانه المقطع أمريكي)

$= 1\left[162 - 5.89(10 - 5.37)\right] < 162$

$134.7 < 162$

$\Rightarrow \phi M_n = 134.7$ Kip-ft.

* في الإمتحان اذا ما كانت $F_y = 50$ ... قيمة $\frac{L_p}{L_r}$ لازم تحسبهم عن طريق المعادلات الطويلة.

*Design by Z_x tables Procedure:

1) assume Zone 1 ($\phi M_n = \phi M_p$)

2) get a section where ($\phi M_p > M_u$)

3) you have a section → check compactness

4) Check which Zone → Calc ϕM_n.

5) if $\phi M_n > M_u$ (ok✓) → Calc New M_u adding self weight & check again.

Example 2: (Design): $M_u = 290$ kip-ft

$F_y = 50$ ksi

$L_b = 10$ ft

$C_b = 1$

Sol: assume Zone 1, $\phi M_p = \phi M_n$

From tables of Z_x → select section ($\phi M_p > 290$) (lightest)

Try W18 x 40 ($\phi M_p = 294$)

$L_p = 4.49$, $L_r = 13.1$, BF = 13.3

$L_p < L_b < L_r$ → Zone $\underline{\underline{2}}$

$\phi M_n = 1 [294 - 13.3 (10 - 4.49)] \leq 294$

$\phi M_n = 220.7 < \underset{\substack{\downarrow \\ 290}}{M_u}$ (Neglect)

Try the next bold section , W21x44

$\phi M_p = 358$, $L_p = 4.45$, $L_r = 13$, $BF = 16.8$

also Zone 2 ($L_p < L_b < L_r$)

→ $\phi M_n = 1[358 - 16.8(10 - 4.45)] = 264.8 < 290$

✗ Neglect

→ Try W21x48^f ⟶ . (Non-comp إنتبه السكشن

$\phi M_p = (398)$, $L_p = 6.09$, $L_r = 16.6$, $BF = 14.7$

(Zone 2) : $\phi M_n = 1[398 - 14.7(10 - 6.09)]$

$= 340.52 > 290$ ok ✓

use it

* بما أنه السكشن Non compact لازم نحسب كمان ϕM_n من FLB

بس هاي مشروحة بالأخير.

☒ $\boxed{3}$ Zone 3 : $L_b > L_r$ أسوأ حالة.

$$\phi M_n = \phi F_{cr} S_x \leq \phi M_p$$

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2}\sqrt{1 + 0.078 \frac{J c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$

نعوضها بال in

in

مهم جداً

* Design by charts Procedure :

بعد جداول Z_x table 3-10

*ركز مع الدكتور لما يشرح ويوضح الـ Charts

1) get M_u , L_b , C_b

2) use charts with $(L_b$ & $\frac{M_u}{C_b})$ → get a section $(\Phi M_p, \Phi M_n)$

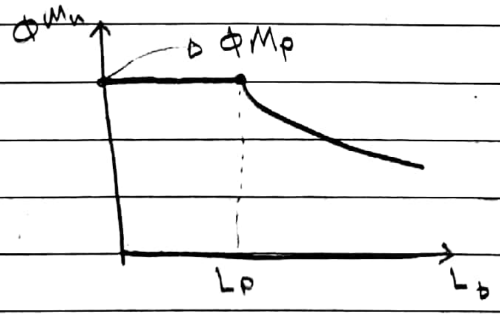
3) Check if $C_b \Phi M_n > \Phi M_p$ (use smaller as ΦM_n)

4) $\Phi M_n \geq M_u$ OK ✓

5) Calc new M_u adding self weight & check $\Phi M_n > M_u$

*عند استخدام الـ Charts : نطلع من قيمة L_b بخط مستقيم للأعلى ونقاطعه مع خط أفقي من قيمة $(\frac{M_u}{C_b})$ وعند نقطة التقاطع بنطلع لفوق لحد ما نقطع أول خط (Solid) لانه بيكون هو الـ lightest وعند هاي النقطة ناخذ قيمة (ΦM_n) ونقوم باللحاق بخط السكشن لأكثر من صفحة ممكن (مش ضروري يضل bold رح يصير متقطع بصفحات ثانية) لحد ما نوصل لقيمة ΦM_p ونأخذها .

هيك بيكونه شكل الـ Curve

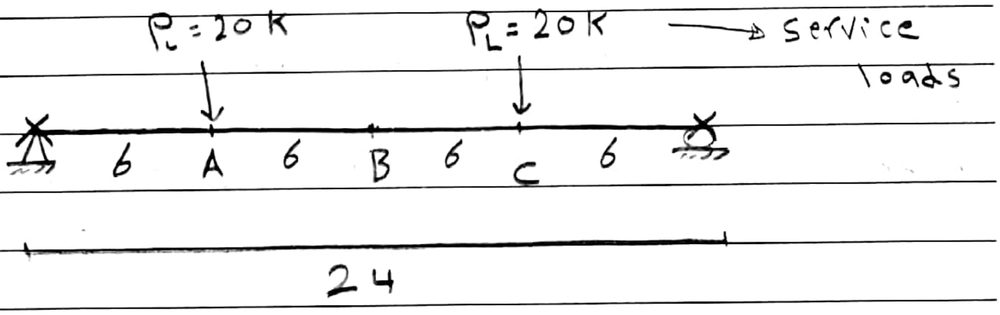


*ملاحظة: قيمة C_b لكل السكاشن في الـ Charts موحدة.. $C_b = 1$

Example 1: (Design by charts)

steel A572 grade 50 ($F_y = 50$)

*bracing at the ends (يعني Lb تساوي الطول كامل)

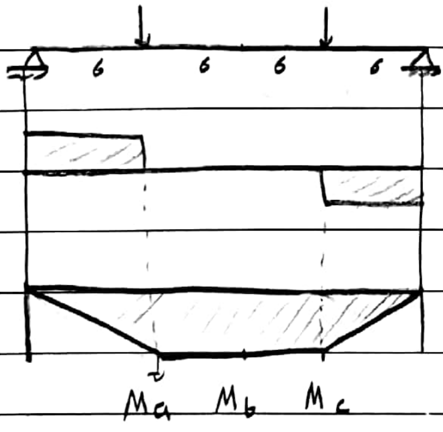


Sol: $L_b = 24$ ft

$\Rightarrow M_u \rightarrow$ (Table 3-23) case 9 : $M_u = Pa$

$$M_u = \overset{\text{Live load}}{1.6} * 20 * 6 = 192 \text{ Kip-ft}$$

$$* C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_a + 4M_b + 3M_c}$$



$\Rightarrow M_a = M_b = M_c = M_{max}$

$\boxed{C_b = 1}$

بما انها 1 .. نأخذ السكشن الصح من المانيوال من أول ضربة .

using charts with $L_b = 24$ $\Rightarrow$ Page 3-126

$\frac{M_u}{C_b} = 192$

Try W12 x 53

أول خط solid فوق نقطة التقاطع

$\Rightarrow \Phi M_n = 208.5$ Kip-ft > 192 ✓

$\Phi M_p = 292$

لانه عبارة عن
distributed load
على طول السكشن .

$$* \text{Calc New } M_u \text{ With S.W} \longrightarrow \frac{WL^2}{8}$$

$$\Rightarrow M_u = 192 + \frac{1.2(0.053)(24)^2}{8} = 196.58 \text{ kip-ft}$$

$$M_u < \phi M_n\,(208.5) \quad \underline{\underline{ok}} \longrightarrow \text{use } W12\times53$$

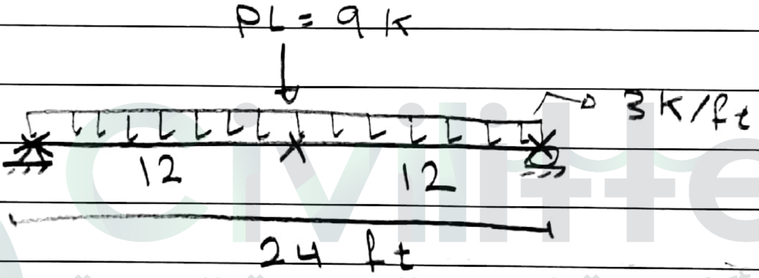
* لو ما زبط نختار أول سكشن solid فوق أول خط استخدمناه .

Example 2: A992 steel $(F_y = 50,\ f_u = 65)$

Lateral bracing at ends & mid span

* 30% of the distributed load is DL & 70% is LL.

* Design by Charts.



Sol: * $L_b = 12$

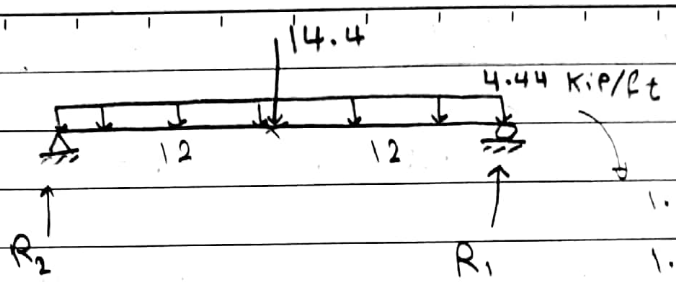
$$* \quad M_u = \frac{PL}{4} + \frac{WL^2}{8} = \frac{1.6*9*24}{4} + \frac{1.2*\overset{30\%\,(DL)}{0.3}*3*(24)^2}{8}$$

$$+ \frac{1.6*\overset{70\%\,(LL)}{0.7}*3\,(24)^2}{8}$$

$$\boxed{M_u = 406.1 \text{ kip-ft}}$$

$$* \quad C_b = \frac{12.5 M_{max}}{2.5M_{max} + 3M_a + 4M_b + 3M_c}$$

يجب أن نحسب Cb مرتين يعني لكل Lb مرة بس لانه متماثل اح نحسب مرة وحدة.

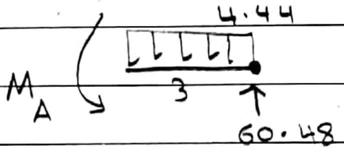


reactions

$1.2*0.3*3 + 1.6*0.7*3 = 4.44$

$* \; R_1 = R_2 = 60.48$ kips.

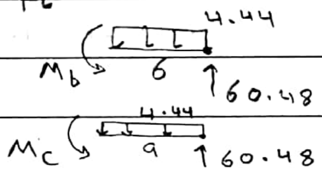
$\Rightarrow M_A \; (at \; \frac{1}{4}*12) \Rightarrow$ نأخذ Section :



$M_A = 60.48(3) - 4.44(3)(\frac{3}{2}) = 161.5$ Kip-ft.

$M_B = 283$ Kip-ft. $(at \; \frac{1}{2}*12)$

$M_c = 364.5$ Kip-ft. $(at \; \frac{3}{4}*12)$



* أو انك تطلع معادلة المومنت عند أي نقطة (M_x) مثل ما تعلمنا بمادة المقاومة... بس إنت خذ سكاشن مثل ما سوينا فوق أسهل.

عوض في المعادلة $\Rightarrow \boxed{C_b = 1.36}$

* ادخل الى الـ Charts بهذه القيم: $* \; L_b = 12 , \; \frac{M_u}{C_b} = 299$

نقطة التقاطع رح تكون بصفحة 3-123 فوق ... بس أول خط Solid رح تقطعه هو بصفحة 3-121

$\Rightarrow$ Try W21X48

$\phi M_n = 311 \quad , \; \phi M_p = 398 \quad (for \; C_b = 1)$

*Check $1.36 * 311 = 423 > \phi M_p$

$\Rightarrow$ use $\phi M_n = 398$ Kip-ft.

$M_u = 406.1 > \phi M_n$ X Neglect.

Try W 18x55 *أول واحد Solid فوق القديم

↳ $\phi M_n = 335$ kip-ft , $\phi M_p = 420$ kip-ft

check : $1.36 * 335 = 455.6 > 420$

→ use $\phi M_n = 420$ kip-ft.

⇒ $\phi M_n > M_u$ ok

→ calc new M_u (add s.w as a dead load).

$$M_u = 406.1 + \frac{1.2(0.055)(24)^2}{8}$$

$M_u = 410.9$ kip-ft $< \phi M_n$ OK ✓

use W18x55.

* Non-compact shapes :

*if non-compact you should calc ϕM_n from FLB not only LTB

$$\boxed{\lambda_p < \lambda < \lambda_r}$$

$$\boxed{\phi M_n = \phi\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right]}$$

مش مهم * Slender : $\lambda > \lambda_r$

$$M_n = \frac{0.9EK_cS_x}{\lambda^2} \quad , \quad K_c = \frac{4}{\sqrt{h/t_w}}$$

* ملاحظة: اذا كان السكشن non-compact نقوم بحساب قيمة M_p عن طريق العادلة $(F_y Z_x)$ ونحسب منها ϕM_n للـ LTB (Zone 1, 2, 3) أما قيمة ϕM_n للـ FLB نأخذها من المانيوال (ϕM_p) من جداول Z_x من غير ما نعوض بمعادلتها الطويلة.

مهم جداً

Example: a simply supported beam with a span length of 45 feet is laterally supported at it's ends & subjected to the service loads:

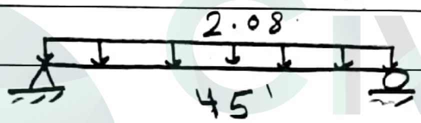
DL = 400 lb/ft (include self weight)

LL = 1000 lb/ft

if $F_y = 50$, is W14x90 adequate?

Sol: $W_u = 1.2(400) + 1.6(1000) = 2080$ lb/ft

$= 2.08$ kip/ft.



$$\Rightarrow M_u = \frac{wL^2}{8} = \frac{2.08(45)^2}{8} = 526.5 \text{ kip-ft}$$

* Check compactness:

$$\lambda_f = 10.2, \quad \lambda_p = 0.38\sqrt{\frac{29{,}000}{50}} = 9.15, \quad \lambda_r = \sqrt{E/F_y} = 24.1$$

$$\lambda_p < \lambda_f < \lambda_r \quad \text{(non-compact)}$$

يعني بدك تحسب FLB و LTB

FLB

$$\Rightarrow \phi M_p = \phi F_y Z_x = 0.9(50)(157)/12 = 588.75 \text{ kip-ft}.$$

$$\phi M_n = \phi\left[M_p - (M_p - 0.7F_y S_x)\left(\frac{\lambda - \lambda_p}{\lambda_p - \lambda_r}\right)\right] = 573 \text{ kip-ft}.$$

نأخذها من المانيوال
جاهزة من جداول Z_x

LTB ⇒ $L_b = 45$, $L_p = 15.2$, $L_r = 42.6$

⇒ $L_b > L_r$ (Zone 3)

$\phi M_n = \phi f_{cr} S_x$

$$f_{cr} = \frac{1.14\,\pi^2\,(29{,}000)}{(45*12/4.11)^2}\sqrt{1+0.078*\frac{4.06*1}{143*13.3}\left(\frac{45*12}{4.11}\right)^2}$$

→ C_b من الشيت جاهزة (لانه الحالة موجودة)

→ inches

→ inches

$f_{cr} = 37.2$ ksi

⇒ $\phi M_n = 0.9(37.2)(143)/12 = 398.97$ kip-ft

نأخذ القيمة الأصغر * $\phi M_n = 398.97 < M_u = 526.5$

X not adequate

* ملاحظة: عندما نقوم بالتصميم باستخدام الـ Charts لـ(member)
فيه اكثر من segment يعني اكثر من Lb فإننا نقوم
بالتصميم للـ segment الذي عليه الماكس مومنت فقط
وباقي الـ segments نقوم بعمل Analysis فقط للتأكد
كفاءة الـ section اللي افترناه لانه اجباري يكونه
نفس السكشن على طول الـ member .

مهم جداً

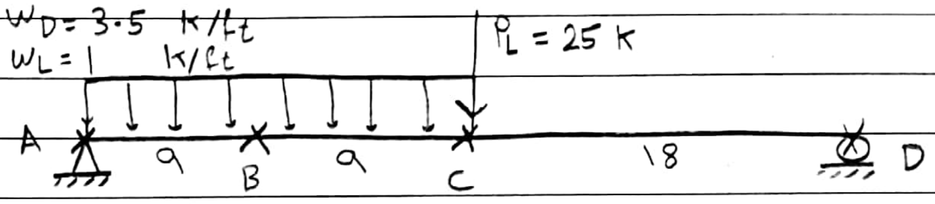
مثل Q4 في السنوات

Example: lateral bracings at A, B, C & D.

سؤال شامل .

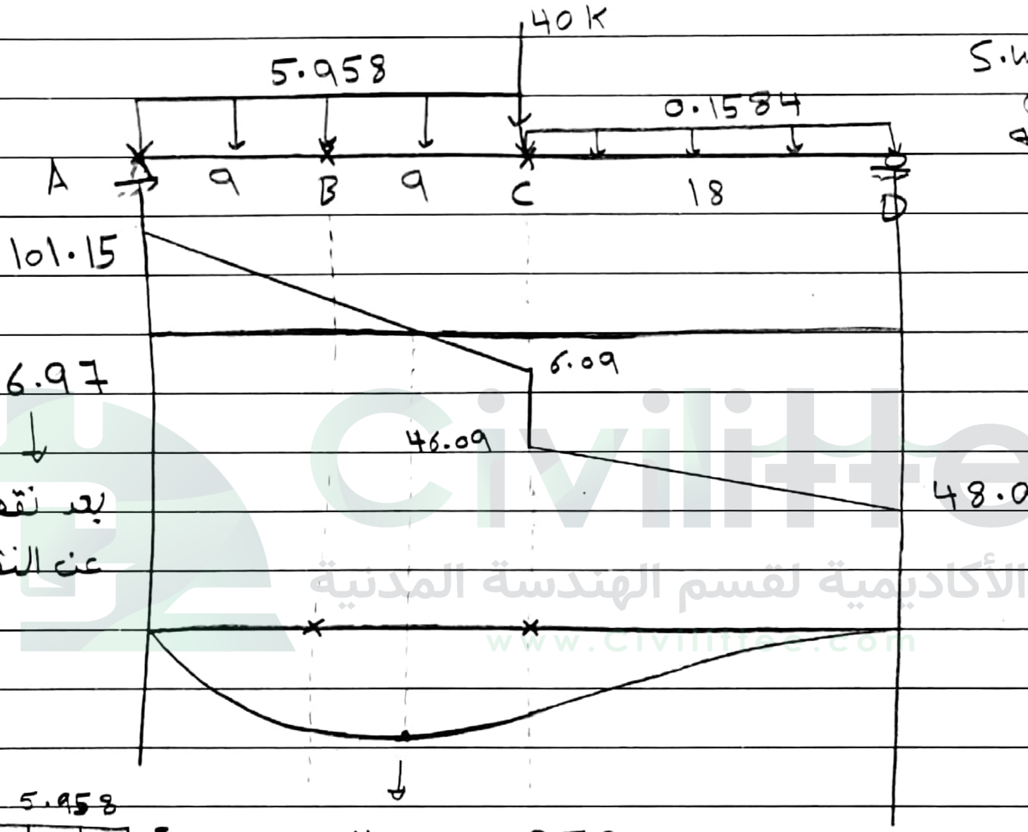
$F_y = 50$

is W14x132 adequate in flexure ?



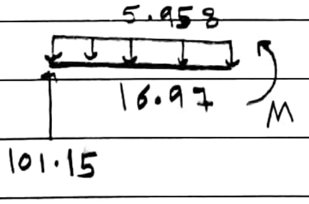
*Sol:

* دخلنا ال S.W
كـ



$\frac{101.2}{5.958} = 16.97$

بعد نقطة الـ Mmax
عن النقطة A .



$\Rightarrow M_{max} = 858$

* انت بدك تطلع ال reactions وتحدد مكانه الـ Mmax وتحسب قيمتها
أو هو يعطيك رسمة للومنت جاهزة .

* ملاحظة : دائماً بلش بال Segment الي عليه Mmax (متى في الديزاين أول خطوة)

* Check compactness : $\lambda_f = 7.15 < 9.15$

compact . (only LTB)

*Segment BC & AB لانهم نفس L_b

$L_b = 9$, $L_p = 13.3$ ⟹ (Zone 1) No need to calc C_b

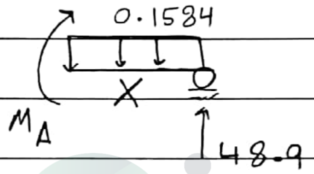
$\Rightarrow \phi M_n = \phi M_p = 0.9\,\overset{f_y}{(50)}\,\overset{Z_x}{(234)}/12 = 877.5 > M_u$ ok.

*Segment CD

$*L_b = 18$, $L_p = 13.3$, $L_r = 56$ } $\phi M_p = 878$, $BF = 7.7$

→ (Zone 2)

⟹ to calc C_b →



M_a (at 4.5 ft) = 218.45

M_b (at 9 ft) = 433.68

⟸ M_c (at 13.5) = 645.72

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_a + 4M_b + 3M_c}$$

M_{max} (at 18 ft) = 854.54

↓ in segment CD

$C_b = 1.653$

$\Rightarrow \phi M_n = C_b [\phi M_p - BF(L_b - L_p)]$

$= 1.653[878 - 7.7(18 - 13.3)] = 1391.51 > \phi M_p$

⟿ $\phi M_n = 878$ Kip-ft.

⟹ The control $\phi M_n = 877.5 > M_u$

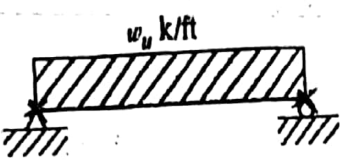
ok ✓ adequate.

*لهون مادة السكشن

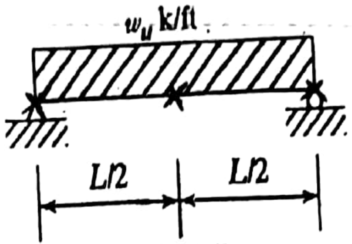
د.حسن كتوا

Steel

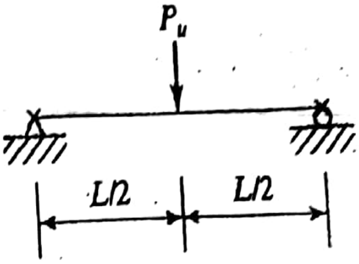
Simply supported



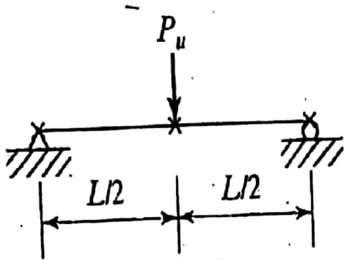
$C_b = 1.14\ R_m$



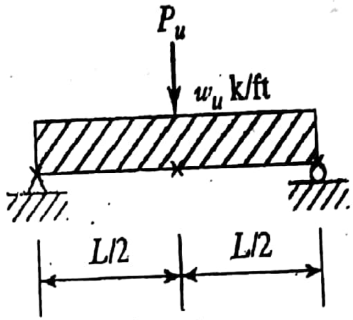
$C_b = 1.30\ R_m$



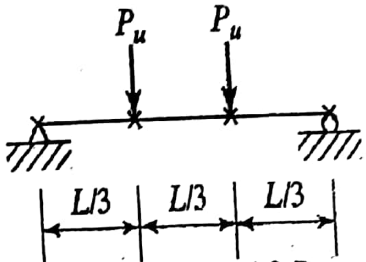
$C_b = 1.32\ R_m$



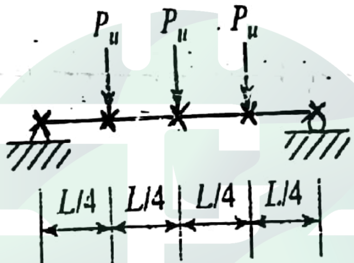
$C_b = 1.67\ R_m$



C_b varies



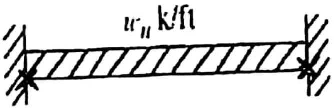
Midsection $C_b = 1.0\ R_m$
End sections $C_b = 1.67\ R_m$



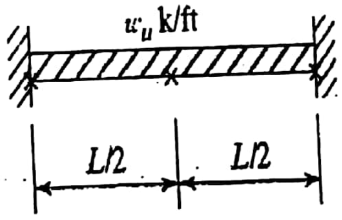
$C_b = 1.11\ R_m$ for two center sections and $1.67\ R_m$ for end ones

$C_b = 1.0\ R_m$

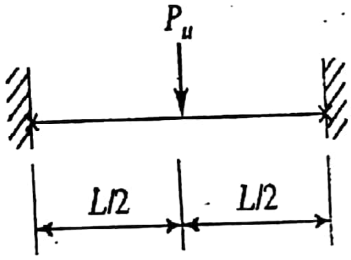
$C_b = 2.27\ R_m$



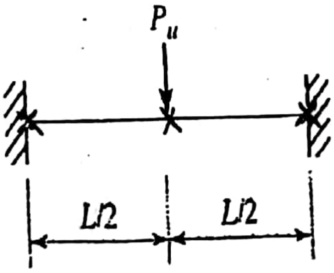
$C_b = 2.38\ R_m$



$C_b = 2.38\ R_m$



$C_b = 1.92\ R_m$



$C_b = 2.27\ R_m$

$R_m = 1$ always

Design of compression

شيت سكند II

$P_u \leq \phi_c P_n$ $\qquad \phi_c P_n = \phi_c f_{cr} A_g$

$\phi = 0.9$

assume $f_{cr} = \frac{2}{3} f_y$ or $\frac{KL}{r} = f_y$ → get $\phi_c f_{cr}$ (table 4-22)

$A_g = \frac{P_u}{\phi_c f_{cr}}$ → get a section. (Part 1)

* check : $KL/r < 200$ ok.

1) $\frac{KL}{r} \leq 4.71\sqrt{E/f_y}$ → $F_{cr} = \left[0.658^{f_y/f_e}\right] f_y$

where $f_e = \frac{\pi^2 E}{(KL/r)^2}$

2) $\frac{KL}{r} \geq 4.71\sqrt{E/f_y}$ → $f_{cr} = 0.877 f_e$

if not ok ∝ ... sellect larger section & repeat.

*Check local buckling.

$\lambda_w < \lambda_r$
$\lambda_f < \lambda_r$ ok.

Shear lag manual

rfo

or use 4-1 ... assume KL_y control → get a section → check whichone controls.

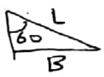
equivalent $= \frac{K_x L_x}{(r_x/r_y)}$ → get new section if eq > control.

---

* in built up sections : calc I_x, I_y to use in $r_x, r_y = \sqrt{\frac{I}{A}}$

→ KL/r_x , KL/r_y

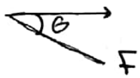
Design for lacings & ties.



*get $\underline{\underline{B}}$ from getting $\underline{\underline{e}}$, if $B < 15$ single → $B = 60°$ ⇝ get $\underline{\underline{L}}$ of lacing
$B > 15$ double → $B = 45°$

* $V_u = 0.02 * \phi_c P_n \xrightarrow{\div 2}$ each side → (get force in lacing).

$V_u = F\cos\theta$



→ get F

* get I, A (كروس) → get $r = \sqrt{I/A}$ (بلاتة t) → $\frac{KL}{r} = \overset{max}{140}$ ↳ 200 ⇝ get $t \approx 0$

→ get new $\frac{KL}{r} \gtrless 4.71\sqrt{\frac{E}{f_y}}$ → get f_{cr} → $A_g = \frac{F}{\phi f_{cr}}$ → get t & b of lacing.

* Min L = L + 2e

for tie plates :
$t = B/50$
$L = B$ ⇒ use t × L × W
$W = B + 2e$

effective length

شيت سكند $\boxed{2}$

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} \rightarrow \begin{cases} G_a \text{ top} \\ G_b \text{ bot} \end{cases} \rightarrow \text{from chart get } \underline{\underline{K_x}}.$$

*check $K_xL/r_x \leq 4.71\sqrt{\frac{E}{f_y}}$ (إذا أقل) → (inelastic) → get τ_a (4-21) ص 214

new $G_a = \tau_a G_a$

new $G_b = \tau_a G_b$ → New K_x.

Beams

First check compactness.

$M_y = f_y S_x$

$M_p = f_y Z_x$

$S_x = I_x/c$

$Z_x = \Sigma A_i y_i$

get $\bar{y}$ $A_c = A_t$

	λ	λ_p	λ_r
Flange	$b_f/2t_f$	$0.38\sqrt{\frac{E}{f_y}}$	$\sqrt{\frac{E}{f_y}}$
Web	h/t_w	$3.76\sqrt{\frac{E}{f_y}}$	$5.7\sqrt{\frac{E}{f_y}}$

Compact | λ_p | Non comp | λ_r | Slender

*L_b ⇒ Zone 1 | L_p | Zone 2 | L_r | Zone 3

* $L_p = 1.76\, r_y \sqrt{\frac{E}{f_y}}$

* $L_r = 1.95\, r_{ts} \frac{E}{0.7 f_y} \sqrt{\frac{J\,c}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7 f_y S_x h_o}{E J c}\right)^2}}$ $\boxed{\text{Part 1}}$

$\phi M_n \geq M_u$

① Zone ① $L_b < L_p$ $\quad \underset{0.9}{\phi} M_n = \phi M_p = \phi f_y Z_x$ $\quad$ Kip-ft وحدة المومنت

No C_b.

Zone ② $L_p < L_b < L_r$

$$M_n = C_b\left[M_p - (M_p - 0.75 f_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p.$$

use

$$\phi_b M_n = C_b\left[\phi_b M_p - BF(L_b - L_p)\right] \leq \phi_b M_p.$$

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_a + 4M_b + 3M_c}$$ (for each L_b)

from cases or table 3-23 $\quad M(x)$

↳ to get M_a, M_b, M_c.

Zone ③ $L_b > L_r$.

$$\phi M_n = \phi f_{cr} S_x / 12$$

$$f_{cr} = \frac{C_b \pi^2 E}{(L_b / r_{ts})^2} \sqrt{1 + 0.078 \frac{J c}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2}$$

Part 1

Design by charts

get L_b, M_u, C_b

→ get in the Chart with L_b & $\frac{M_u}{C_b}$ → get Section ϕM_n, ϕM_p.

Check .. if $C_b (\phi M_n) > \phi M_p$ → use ϕM_p as ϕM_n.

→ $\phi M_n \geq M_u$ ok. ✓

→ Calc new M_u adding selfweight & check $M_u < \phi M_n$.

Design by Z_x tables

assume Zone 1 $(\phi M_n = \phi M_p)$ → get Section $(\phi M_p > M_u)$.

⇒ Section ⇒ Check compactness ⇒ Check which Zone.

→ Calc ϕM_n & Check $\phi M_n > M_u$ ... ok → add selfweight & calc New M_u.

Non-Compact Check FLB $(\lambda_p < \lambda_b < \lambda_r)$.

$$\phi M_n = \phi \left[M_p - (M_p - 0.7 F_y S_x) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right]$$

اذا ما بدك تحسبها حساب

FLB

* Calc $\phi M_p = \phi f_y Z_x / 12$ → from tables Z_x get ϕM_p ... it will be

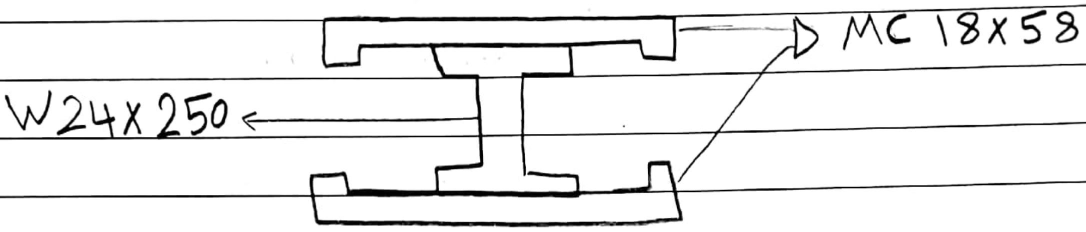
Slender : $\lambda > \lambda_r$

$$M_n = \frac{0.9 E K_c S_x}{\lambda^2}$$

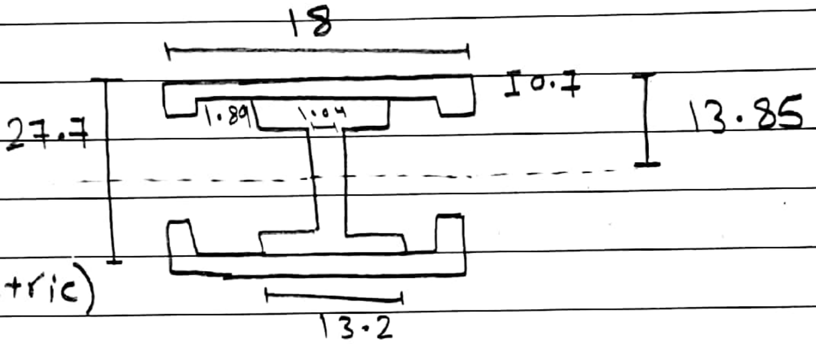
$$K_c = \frac{4}{\sqrt{h / t_w}}$$

سنوات سكند

Q1 (5 Points): Calculate Z_x for the built up section shown:



Sol: بالتفصيل :



$\bar{y} = 13.85$ (symmetric)

$* \; Z_x = \Sigma A\hat{y} = 2(17.1)(13.85 - \underset{\downarrow \, \bar{x}}{0.862})$ → C-sections

$+ 2(1.89 * 13.2)(13.85 - 0.7 - \frac{1.89}{2})$ → flanges

$+ 2(1.04 * (13.15 - 1.89)) * (\frac{13.15 - 1.89}{2})$ → web

$Z_x = 1185 \text{ in}^3$

→ Z_x of W24x250

طريقة أخرى * $Z_x = 2(17.1)(13.85 - 0.862) + 744$

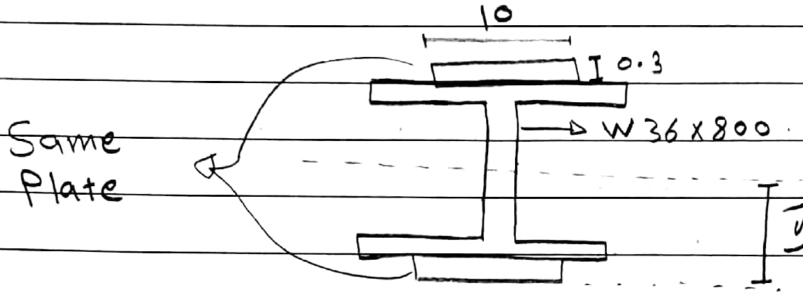
$Z_x = 1188 \text{ in}^3$

* عندما يكون الشكل symmetric وفي المنتصف سكشن امريكي ومركزه منطبق على مركز الشكل كامل نأخذ قيمة Z_x له جاهزة من المانيوال.

*(حل على الطريقة الثانية ، الأولى للتوضيح)

سنوات سابقة

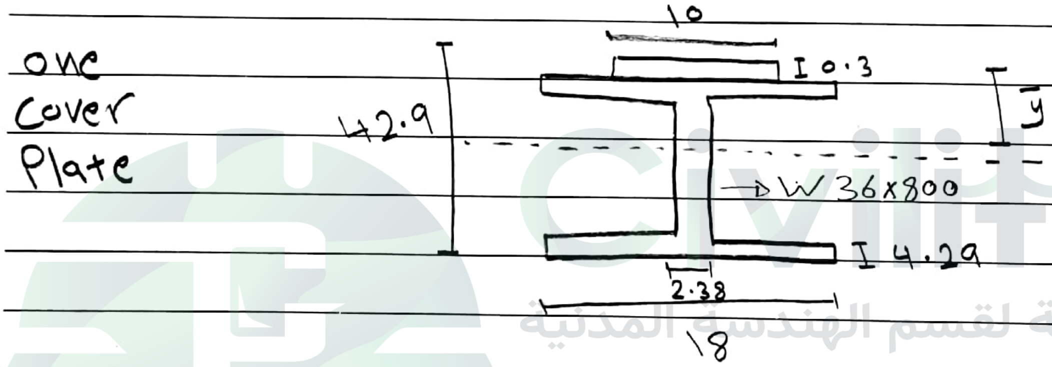
Q2: (4 Pts) * Calculate Z_x for the whole section.



Sol: Symmetric $\Rightarrow \bar{y} = (21.6)$ in.

$$Z_x = \Sigma A y = 2(10*0.3)\left(21.6-\frac{0.3}{2}\right) + \underset{Z \text{ of } W36x800}{3650} = 3778.7 \, in^3$$

Q3: (6 Points) * Calculate Z_x for the whole section.



Sol: Not Symmetric: $A_c = A_t$

هنا يجب أنه نحل بالتفصيل

$$(10*0.3) + (18*4.29) + 2.38(\bar{y} - 0.3 - 4.29)$$

$$= (18*4.29) + 2.38(42.9 - \bar{y} - 4.29)$$

$$\Rightarrow \bar{y} = 20.97 \text{ in}$$

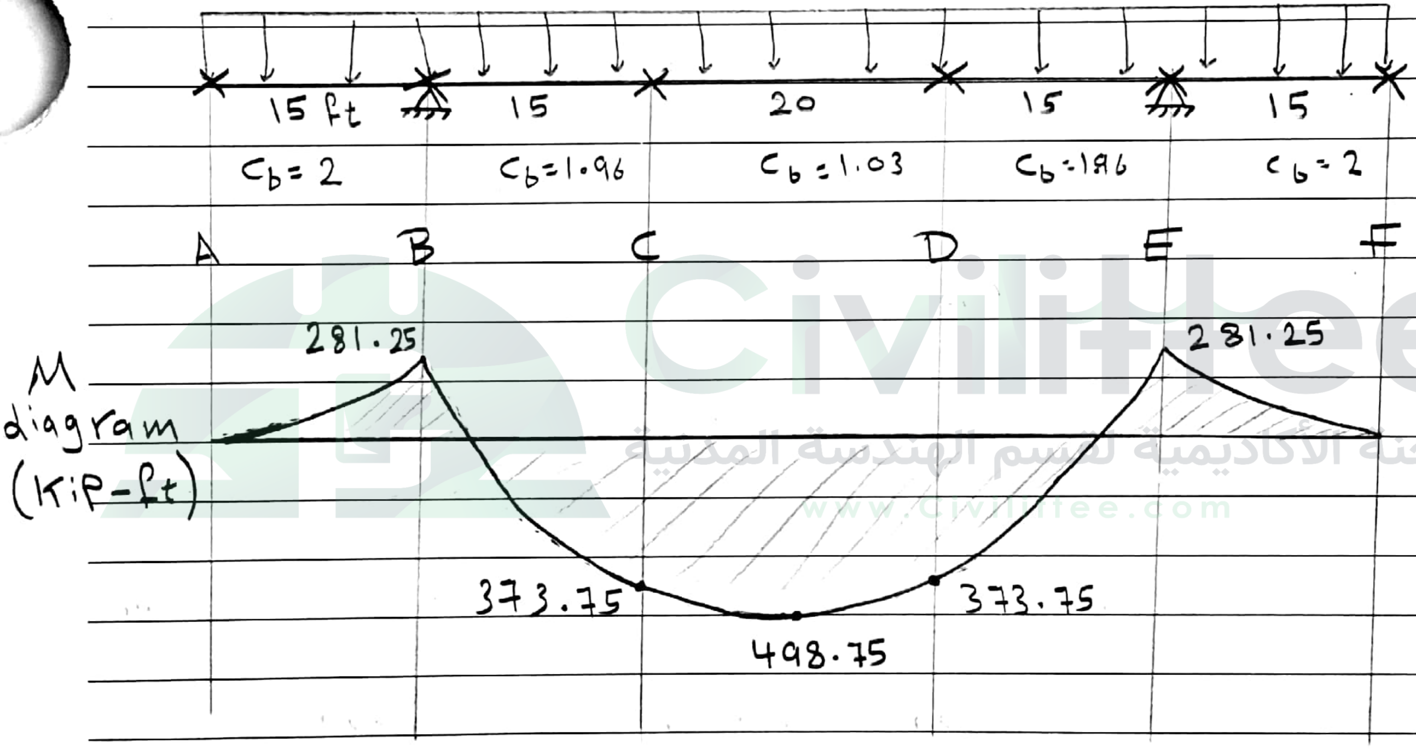
$$Z_x = \Sigma A \hat{y} = \underset{\text{plate}}{(10*0.3)\left(20.97-\frac{0.3}{2}\right)} + \underset{\text{upper web}}{(2.38)(20.97-0.3-4.29)/2}$$

$$(18*4.29)(20.97-0.3-4.29/2) \rightarrow \text{upper flange}$$

$$+(18*4.29)(21.93-4.29/2) \rightarrow \text{Lower flange}$$

Lower web $\leftarrow$ $+2.38(21.93-4.29)/2 = \boxed{Z_x = 3061.24 \, in3}$

Q4: (10 Points): Select the lightest W-Section of A992 Steel ($F_y = 50$), for the beam Shown in the figure below that can support the uniformly distributed ultimate load Shown, the lateral supports at A, B, C, D, E, F, the values of C_b calculated, use The Charts in the design. (Neglect the self weight of the beam)



Solution: Start with segment CD (Max moment)

$\Rightarrow$ $M_u = 498.75$ Kip-ft, $L_b = 20$, $C_b = 1.03$

enter the chart with: $L_b = 20$

$M_u / C_b = 484.2$ $\Rightarrow$ Page 3-119

Use W24×84, $\phi M_n = 521$, $\phi M_p = 840$

Check : $1.03(521) = 536.63 < 840$ ✓

$\Rightarrow \phi M_n = 537$ kip-ft $> M_u$ ok ✓

*Segments BC & DC (same L_b & C_b & Moment).

* Analysis for W24x84.

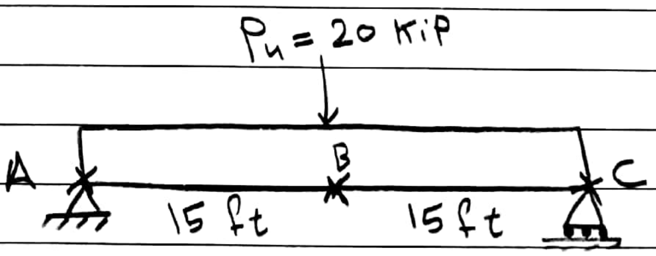
$M_{max} = 373.75 < \phi M_n$ $\xrightarrow{} 537$. ok

*Segments AB & EF, (Analysis for W24x84).

$M_{max} = 281.25 < \phi M_n$ ok.

$\Rightarrow$ Use W24x84

Q5: (8 Points): The beam shown in the figure is laterally supported at A, B & C, The beam is made of W14x26 & carrying a concentrated load $P_u = 20$ kips, The steel has $F_y = 50$ ksi, Neglect the self weight, is the beam adequate in flexure?



سنوات سكنر

Sol : $M_u = \frac{PL}{4} = \frac{20(30)}{4} = 150$ kip-ft

* from the C_b Sheet $\Rightarrow$ $C_b = 1.67$

* Check compactness.

$\lambda_f = 5.98$, $\lambda_p = 9.15$ $\rightarrow$ Compact ✓

$L_b = 15$, $L_p = 3.81$, $L_r = 11.1$

$L_b > L_r$ (Zone 3)

$\phi M_n = \phi F_{cr} S_x$

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2} \sqrt{1 + 0.078 \frac{J c}{S_x h_o} \left(\frac{L_p}{r_{ts}}\right)^2}$$

(all from Manual Part 1)

لاتنسى تعوض قيم Lb, Lp بالإنشات

$F_{cr} = 26.2$ ksi

$\phi M_n = 0.9(26.2)(35.3)/12 = 69.4$ kip-ft $< \underset{150}{M_u}$

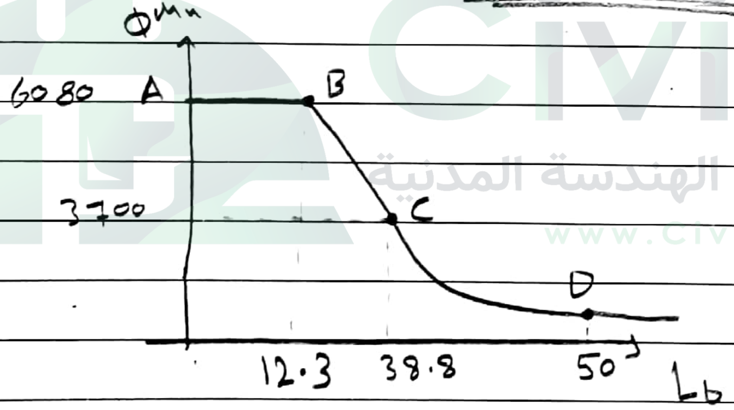
NO it's not adequate.

سنوات سابقه

Q6: (9 Points) The curve of the nominal Flexural strength ΦM_n for W44x335 for grade 50 of steel $F_y = 50$ is Plotted in the figure, The section is compact & $C_b = 1$.

⇒ Sketch the curve to the same section but for grade (65), $F_y = 65$, by finding these values. (Show your calculations)

① New L_p
② New L_r
③ New Point A (ΦM_p).
④ New nominal strength at new L_b (New B).
⑤ " " " " " L_r (" C)
⑥ " " " at $L_b = 50$ ft. (New D)



Solution: ① $L_p = 1.76\, r_y \sqrt{E/F_y} = 1.76(3.49)\sqrt{\frac{29,000}{65}} \Big/ 12$

$L_p = 10.81$ ft

② $L_r = 1.95(4.24)\frac{29,000}{0.7(65)}\sqrt{\frac{74.7(1)}{1410(42.3)}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7*65*1410*42.3}{29,000(74.7)}\right)^2}}$

$\frac{L_r}{12} \Rightarrow \quad L_r = 32.63$ ft

③ $\Phi M_p = \Phi f_y Z_x / 12 \rightarrow 0.9 (65) (1620) / 12$

$\Phi M_p = 7897.5$ kip-ft

④ Same $\Phi M_p = 7897.5$ kip-ft.

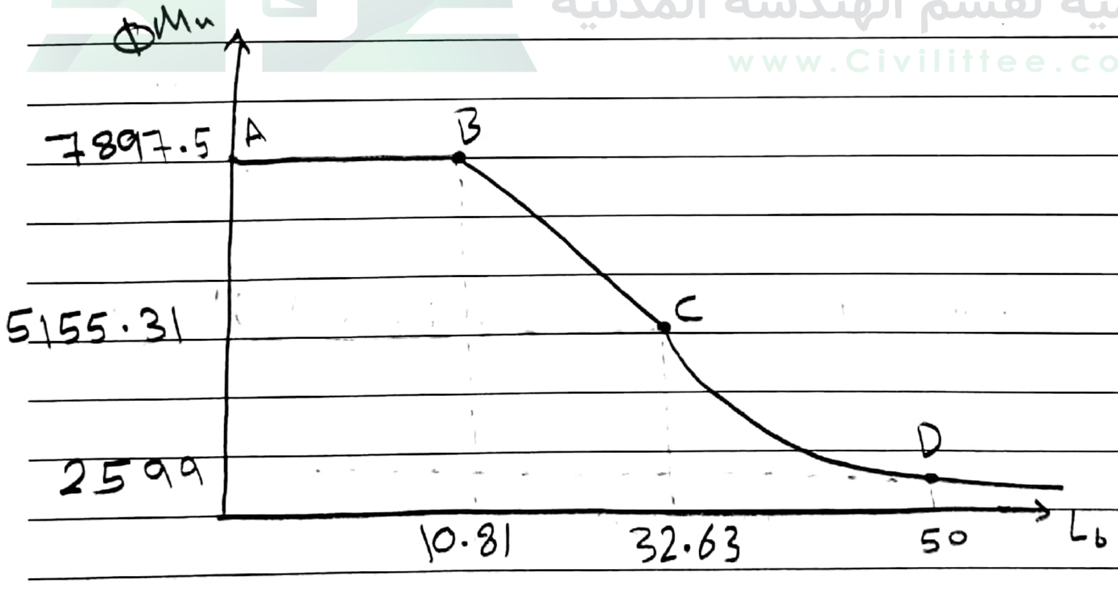
⑤ at $\underline{\underline{C}} \Rightarrow \Phi M_n = C_b \left[\Phi M_p - (\Phi M_p - \Phi 0.7 F_y S_x) \overset{\rightarrow L_b = L_r}{(1)} \right] / 12$

$\Phi M_n = 5155.31$ kip-ft.

⑥ at D , $\Phi M_n = \Phi F_{cr} S_x \cdot / 12$

$$F_{cr} = \frac{1 (\pi)^2 (29,000)}{(50*12 / 4.24)^2} \sqrt{1 + 0.078 * \frac{74.7}{1410 (42.3)} \left(\frac{50*12}{4.24}\right)^2}$$

$$\Phi M_n = \frac{0.9 F_{cr} S_x}{12} = 2599 \text{ kip-ft.}$$



#

Final

⊛ Shear (V)

(90 - 95)% of the shear stress is on the web.

Shear capacity → $\phi V_n \geq V_u$

$$\boxed{\phi V_n = \phi(0.6 F_y A_w) C_v}$$

$$\boxed{A_w = d\, t_w}$$

Area of the web

⇒ if it's W-shape ⇒ $\phi = 1$, $C_v = 1$, $\frac{h}{t_w} < 260$ ← check.

⇒ if not W-shape ⇒ $\phi = 0.9$ $\boxed{K_v = 5}$ always

① $h/t_w \leq 1.1\sqrt{\frac{K_v E}{F_y}}$ → $C_v = 1$

② $1.1\sqrt{\frac{K_v E}{F_y}} < h/t_w \leq 1.37\sqrt{\frac{K_v E}{F_y}}$ → $C_v = \frac{1.1\sqrt{E K_v / F_y}}{h/t_w}$

③ $h/t_w > 1.37\sqrt{\frac{K_v E}{F_y}}$ → $C_v = \frac{1.51\, E K_v}{(h/t_w)^2\, F_y}$

لما يكون W-section ⊛ : Check also $\frac{h}{t_w} < 2.24\sqrt{\frac{E}{F_y}}$ (if ok✓) use $\boxed{\phi = 1}$

*من جداول ال Zx لأي سكشن أمريكي (w-shape) بتقدر تجيب

قيمة ϕV_n جاهزة.

* ملاحظة: لو كان عندي Shear بالسكشن ، ما بنغير السكشن
وانما بنحط plate في مكان الـ max-shear .

Example: a simply supported beam with a span length of 45 ft is laterally supports at it's ends, & subjected to the following service loads:

$D_L = 400$ lb/ft (include self weight of beam).

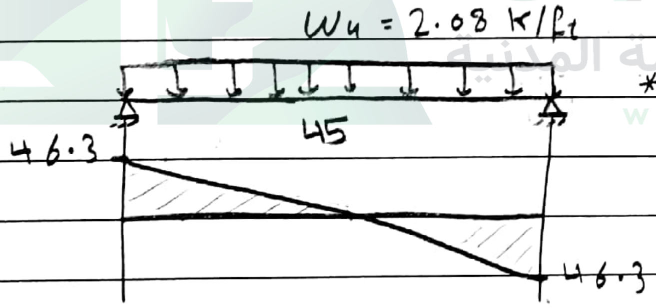
$L_L = 1000$ lb/ft.

if $F_y = 50$ Ksi, is W14x90 adequate? (in Shear)

Solution: W-Section $\rightarrow \Phi = 1$, $C_v = 1$

$$*\text{Check}: \frac{h}{t_w} = 25.9 < 2.24\sqrt{\frac{29{,}000}{50}} = 54 \quad (\text{ok } \Phi = 1)$$

$\Rightarrow$



$* W_u = 1.2(0.4) + 1.6(1)$
$= 2.08$ Kip/ft

$\Rightarrow \{V_u = 46.3\}$ Kips.

$$\Phi V_n = \Phi(0.6 F_y A_w) C_v = 1(0.6 * 50 * 14 * 0.44) * 1$$

$$\Phi V_n = 184.5 \text{ Kips}$$

or from Z_x tables $\Rightarrow \Phi V_n = 185$ Kips.

$\Rightarrow \Phi V_n > V_u$ ok ✓ adequate

⊛ Deflection :

* get allowable deflection : $\Delta_{allow} = \frac{L}{360}$ → inches

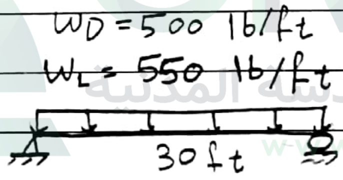
* get Δ_{max} From manual (cases) (كل اشي بالانش)
↓
بعد ال charts

if ⇒ $\Delta_{max} < \Delta_{allow}$ OK ✓

! مهم جدا ⇒ in deflection ⇒ use only Service live loads.

* Design (No section) → assume $\Delta_{max} = \Delta_{allow}$
get I_x → get a section.

Example :



W18 x 35

* بس live بنوخذ وبدونه ضرب بـ ١.٦

Sol: $\Delta_{allow} = \frac{30*12}{360} = 1$ in

from case ① : $\Delta_{max} = \frac{5 W L^4}{384 E I_x}$

$= \frac{5(0.55/12)(30*12)^4}{384(29,000)(510)} = 0.678$ in.

عشان نحول كل اشي لـ inches

510 → I_x

⇒ $0.678 < 1$ OK ✓

*if not ok (change the Section)

⊛ Biaxial Bending

وهو لما يكون عندنا مومنت بال (x , y)

if: $\boxed{\dfrac{M_{ux}}{\phi M_{nx}} + \dfrac{M_{uy}}{\phi M_{ny}} \leq 1}$ (Safe)

* M_{ux} & M_{uy} ⇒ get them from loads on structure

X: check compactness ⟶ check which Zone ⟶ get ϕM_{nx}

y: always Zone 1 .... if non-compact .. ϕM_{ny}
($\phi M_{ny} = \phi F_y Z_y$)

$$\downarrow$$

$$\phi\left[M_P - (M_P - 0.7 F_y S_x/12) \frac{(\lambda - \lambda_P)}{(\lambda_r - \lambda_P)}\right]$$

Example: a W21X68 used as simply supported beam with span length of 12 feet, lateral supports at the ends only, loads are through shear center, & have moments about x, y axes:
Service load moments: $M_{Dx} = 48$ Kip-ft
$M_{Lx} = 144$ Kip-ft
$M_{Dy} = 6$ K-ft
$M_{Ly} = 18$ K-ft

A992 Steel ($F_y = 50$), assume that all the moments are uniform over the length of the beam. ⟶ $\boxed{C_b = 1}$ يعني

* Does the beam satisfy Provisions of AISC Specifications.

Solution: $M_{u_x} = 1.2(48) + 1.6(144) = 288$ kip-ft

$M_{u_y} = 1.2(6) + 1.6(18) = 36$ kip-ft

*Check compactness: $\lambda f = 6.04$ $\Rightarrow$ compact

$\lambda p = 0.38\sqrt{E/F_y} = 9.15$

x:

* $L_b = 12$, $L_p = 6.36$, $L_r = 18.7$

$\Rightarrow$ Zone 2 , $BF = 18.8$, $\phi M_p = 600$

$\phi M_{n_x} = C_b[\phi M_p - BF(L_b - L_p)] = 494$ kip-ft

y: Zone 1: $\phi M_n = \phi M_p = \phi \cdot F_y Z_y / 12 = 0.9(50)(24.4)/12$

$\phi M_{n_y} = 91.5$ kip-ft

$$\Rightarrow \frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{uy}} = \frac{288}{494} + \frac{36}{91.5} = 0.976 < 1 \checkmark \text{ ok}$$

* Safe & economy

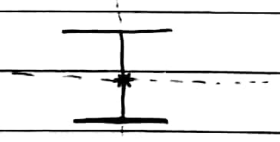
لانها بين (0.9 - 1)

⊛ Shear Center:

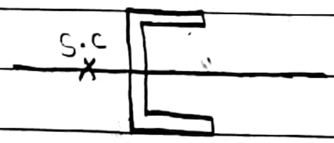
* هي النقطة اللي اذا كانت محملة ال load عليها ما بيكون عندي تورشن (دورانه للسكشن).

* اذا كان الشكل Symmetry بال (x & y) بيكون ال Shear center في مكانه السنترويد بالزبط

* واذا كان Symmetry على محور واحد بيكون shear center على هذا الخط

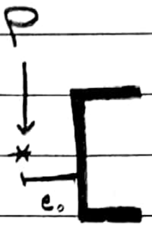


W-section ,



C-section

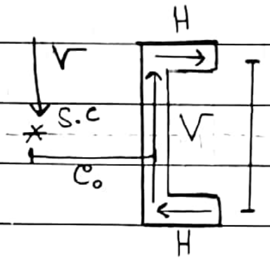
No torsion



No torsion.

e_0 : بعد الـ S.C عن الـ Center line للسكشن.

* Location of shear center.



لإيجاد قيمة e_0

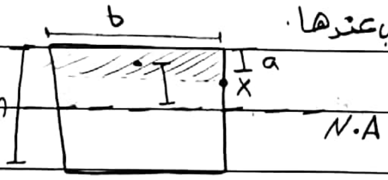
↳ $\Sigma M_{s.c} = 0 \Rightarrow \boxed{H h = V e_0}$

e_0 : ممكن تجيبها جاهزة من المانيوال (Part 1) اذا كان المقطع امريكي.

Shear flow (at certain Point) $\boxed{q_V = \frac{Q V}{I_x}}$, $Q = A d$

d ↓ المسافة بين سنترويد الشكل كامل وسنترويد القطعة فوق أو تحت النقطة المراد الحساب عندها.

Example on Q :

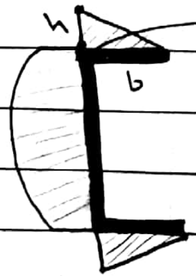


find Q at $\underline{\underline{x}}$.

Sol: $Q_x = A d = (b * a)\left(\frac{h}{2} - \frac{a}{2}\right)$

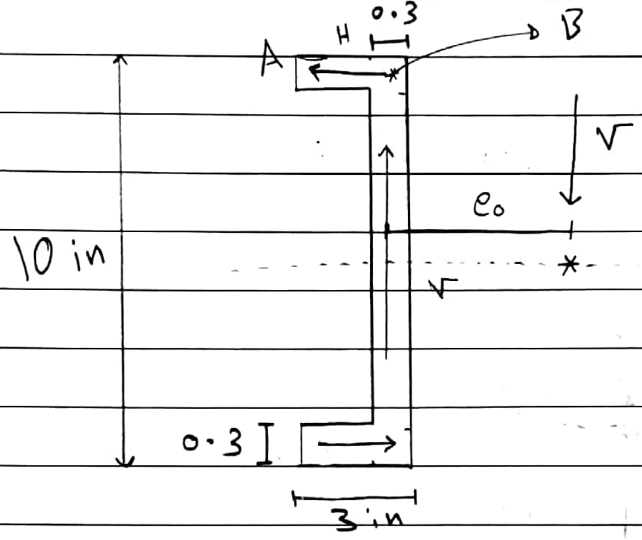
مهم ↳ H: محصلة الـ Shear flow (مساحة)

توزيع الـ Shear flow على C-section



→ $H = \frac{1}{2} b h$ مساحة مثلث

Example: find the location of shear center.



10 − 0.15 − 0.15

h = 9.7

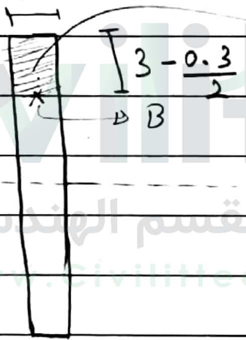
5 − 0.15
= 4.85

Sol: Shear flow at B

* هو بيحددلك تحسب عندها واذا ماحددلك انت احسبها عندها

$$q_V = \frac{Q\,V}{I_x}$$

* لو فتحنا ال C

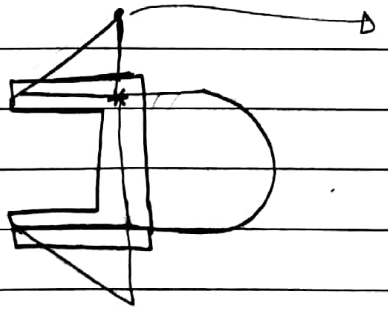


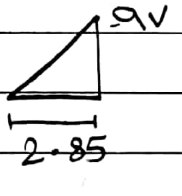
هاي المنطقة اللي رح احسبلها Q

$$Q = Ad = (0.3 * 2.85)(4.85)$$

$$I_x = \underbrace{2\frac{(3)(0.3)^3}{12} + 2(3*0.3)(4.85)^2}_{\text{two flanges}} + \underbrace{\frac{(0.3)(9.4)^3}{12}}_{\text{web}} = 63.12$$

$$\Rightarrow \boxed{q_V = 0.0657\,V}$$
at B





$$H = 0.5(2.85)(0.0657\,V)$$

$$H = 0.0936$$

$\Rightarrow Ve = Hh$

$Ve = 0.0936 \cancel{V} * 9.7$

$\boxed{e = 0.91 \text{ in}}$ from the center of the web.

* * *

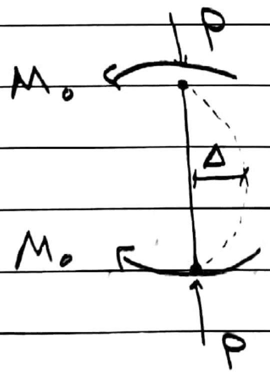
Chapter 6

Analysis & Design of beam-columns

beam-column : وهو ال member الي يكون عليه مومنت مثل ال beam وعليه compression مثل ال column.

* First order : Calculations & Analysis without deformation.

* Second order : Calculations & Analysis with respect to the deformation of the member



$$\Rightarrow M = \underset{\substack{\downarrow \\ \text{1st} \\ \text{order}}}{M_0} + \underset{\substack{\downarrow \\ \text{2nd} \\ \text{order}}}{P\delta}$$

* قيمة Δ حسابها معقد جداً .. لذا نستخدم ما يسمى ب
⇒ Interaction Formula.

if: $\boxed{1}$ $\frac{P_u}{\phi_c P_n} \geq 0.2$

use ⇒ $\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right) \leq 1$

if: $\boxed{2}$ $\frac{P_u}{\phi_c P_n} < 0.2$

use ⇒ $\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right) \leq 1$

* $\phi_c P_n$: from manual, (table 4-1) using KL_y (or equivalent)

* ϕM_{nx} : check compactness → check which Zone
→ get ϕM_{nx} → FLB
→ LTB

* ϕM_{ny} : always Zone 1 ($\phi M_n = \phi f_y Z_y$)

مهم: $M_{ux} = B_1 \underset{\text{from load on structure}}{\underset{\downarrow}{M_{nt}}} + B_2 M_{nt}$

لغايات الامتحان

* قيمة B_2 دائماً Zero لانه كل الاسئلة Braced يعني
ما فيها sides-way.

B_1 : Amplification factor : بسبب تأثير الـ deformation

$$B_{1x} = \frac{C_m}{1-\left(\frac{P_u}{P_e}\right)} \geq 1$$

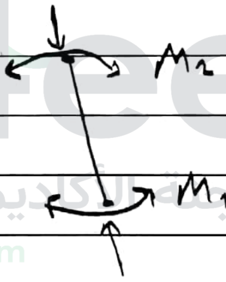
ـ اذا أقل نأخذها 1

$$P_e = \frac{\pi^2 E I_x}{(KL)^2}$$

$(KL)^2$ → inches not ft

* إذا كان عندك مومنت بالـ y يجب أنه تحسب M_{uy} وتحسب B_{1y} جديدة وعند حساب P_e نعوض قيمة I_y (إنتبه)

* Cm



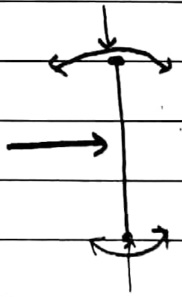
① → if there is no transverse loads

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right)$$

M_1 : الأصغر

M_2 : الأكبر

② → There is transverse loads →



$$C_m : 1 + \psi\left(\frac{P_u}{P_e}\right)$$

ψ → From Manual

بعد جداول الـ U ووو

قبل جدول قيمة K

* حسب شكل الـ load والتثبيت

مهم
صح

* النسبة $\frac{M_1}{M_2}$ نعوضها موجبة أو سالبة حسب اتجاهات المومنت

* اذا كانه المومنت على جهة واحدة فقط تكونه قيمة $M_1 = 0$

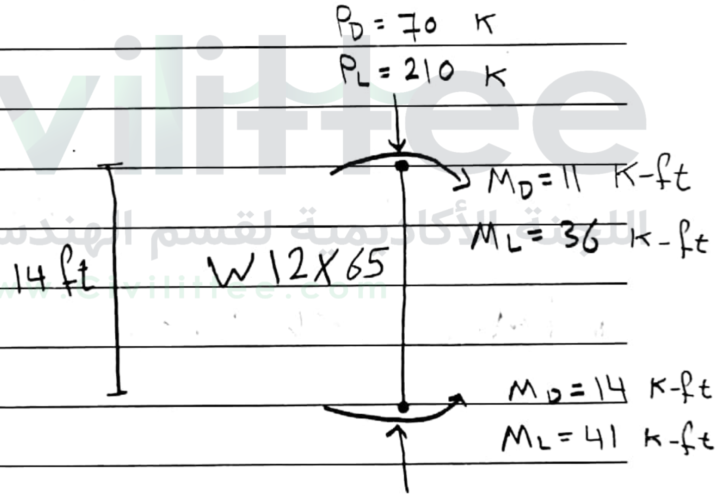
Example 1:

* braced frame $(B_2 = 0)$
* Service loads & Moments are about Strong axis (x)

A572 steel $(F_y = 50)$

$K_x L_x = K_y L_y = 14$ ft

is W12X65 adequate ?



Sol: $P_u = 1.2(70) + 1.6(210) = 420$ K.

$\phi_c P_n = 685$ K (Manual table 4-1)

$$\frac{P_u}{\phi_c P_n} = \frac{420}{685} = 0.6131 > 0.2$$

$$\Rightarrow \text{use} \left[\frac{P_u}{\phi_c P_n} + \frac{8}{9} \left(\frac{M_{ux}}{\phi M_{nx}} + \cancel{\frac{M_{uy}}{\phi M_{uy}}}^{\,0} \right) \right] \leq 1$$

$$* M_{ux} = B_1 M_{nt} + B_2 \overset{\to 0}{\cancel{M_{lt}}}$$

$1.2(11) + 1.6(36)$

$M_{nt} = 70.8$

$M_{nt} = 82.4$ K-ft نأخذ الأكبر

$1.2(14) + 1.6(41)$

$M_{nt} = 82.4$

$$* B_1 = \frac{C_m}{1 - \left(\frac{P_u}{P_e}\right)} \geq 1$$

$$P_e = \frac{\pi^2 E I_x}{(KL)^2} = \frac{\pi^2 (29{,}000)(533)}{(14 * 12)^2} = 5405.13$$

No transverse load $* C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right)$

$$= 0.6 - 0.4\left(-\frac{70.8}{82.4}\right) = 0.9437$$

$\Rightarrow B_1 = 1.0232$ $\Rightarrow M_{ux} = 84.31$

$* \phi M_{nx}$: Check compactness $\left(\overset{\text{Manual}}{W12x65\ \underline{f}}\right)$ أو احسب λ_f و λ_r

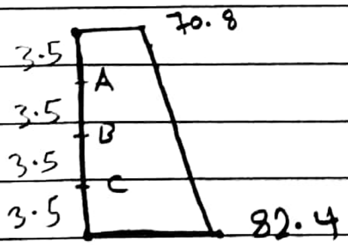
$\downarrow$

Non-compact

Check Which Zone :

$L_b = 14$, $L_p = 11.9$, $L_r = 35.1$ $\Rightarrow$ Zone $\underline{\underline{2}}$ BF = 5.41

① LTB $\Rightarrow$ $\phi M_p = 0.9(50)(\overset{Z_x}{96.8})/12 = 363$ K-ft



$M_A = 73.7$
$M_B = 76.6$
$M_C = 79.5$

⇐ من تشابه المثلثات

$$\Rightarrow C_b = \frac{12.5(82.4)}{2.5(28.4)+3(73.7)+4(76.6)+3(79.5)}$$

$$\boxed{C_b = 1.06}$$

$$\Rightarrow \text{Zone 2: } \phi M_n = C_b\left(\phi M_p - BF(L_b - L_p)\right) < \phi M_p$$

$$= 1.06\left(363 - 5.41(14 - 11.9)\right) = 372 \nless 363$$

$$\phi M_n = 363$$

$\Rightarrow$ FLB $\Rightarrow$ $\phi M_n = 356$ Kip-ft   جاهزة من المانيوال جداول Z_x

لأنه non-compact

$$\Rightarrow \boxed{\phi M_{nx} = 356 \text{ Kip-ft}}$$

* حلها هيك وسيبك من طريقة الـ Chart اللي حالل عليها بالسلايدات

هيك أسهل.

$\Rightarrow$ نعوض في المعادلة

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{nx}} + 0\right) \leqslant 1$$

$$0.6131 + \frac{8}{9}\left(\frac{84.31}{356}\right) = 0.824 < 1$$

Safe

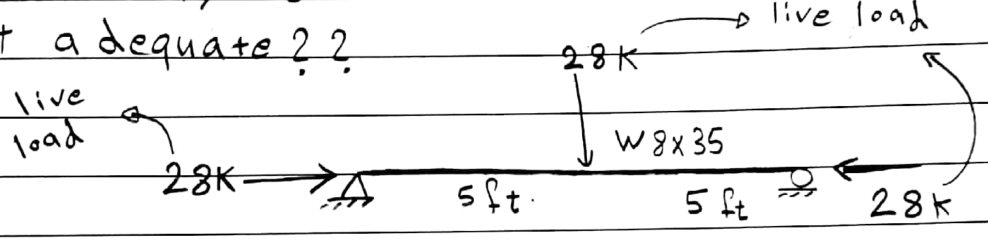
↙

Adequate

Example 2: * Service loads are shown.

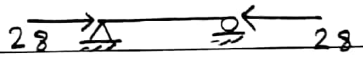
* laterally braced at the ends
* bending about x-axis.
* $F_y = 50$

* is it adequate ??



Sol: $P_u = 1.6(28) = 44.8$ Kips.

* The axial force * ⇒



$\phi_c P_n = 358$ Kip.

$$\frac{P_u}{\phi_c P_n} = 0.1251 < 0.2 \Rightarrow \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi M_{n_x}} + \overset{\text{zero}}{\cancel{\frac{M_{uy}}{\phi M_{ny}}}}\right)$$

$$* \; M_{ux} = B_1 M_{nt} + \cancel{B_2 M_{lt}}$$

dead load الذاتي

$$M_{nt} = \frac{PL}{4} + \frac{WL^2}{8} = \frac{1.6(28)(10)}{4} + \frac{1.2(0.035)(10)^2}{8}$$

The transverse load

Self weight

$$\boxed{M_{nt} = 112.525 \text{ k-ft}}$$

$$B_1 = \frac{C_m}{1 - \left(\frac{P_u}{P_e}\right)} \quad , \quad P_e = \frac{\pi^2 (29{,}000)(127)}{(10 * 12)^2} = 2524 \text{ Kip}$$

I_x

$$C_m = 1 + \psi \frac{P_u}{P_e} = 1 - 0.2\left(\frac{44.8}{2524}\right) = 0.99645$$

from manual, Case 4.

$B_1 = 1.0145$ ⇒ $\boxed{M_{ux} = 114.15 \text{ k-ft}}$

$*\phi M_{nx}$ ⇒ check compactness → W8x35 (No $\underline{\underline{f}}$) → Compact.

⇒ $L_b = 10$, $L_p = 7.17$, $L_r = 27$, $BF = 2.43$

$\phi M_p = 130$

⇒ Zone 2:

$C_b = 1.32$ (from the sheet of C_b cases)

$$\Rightarrow \phi M_n = C_b\left[\phi M_p - BF(L_b - L_p)\right] < \phi M_p$$

$$= 1.32\left[130 - 2.43(10 - 7.17)\right] < 130$$

$$162.5 < 130 \; \times .$$

$$\Rightarrow \phi M_{nx} = 130 \text{ k-ft}$$

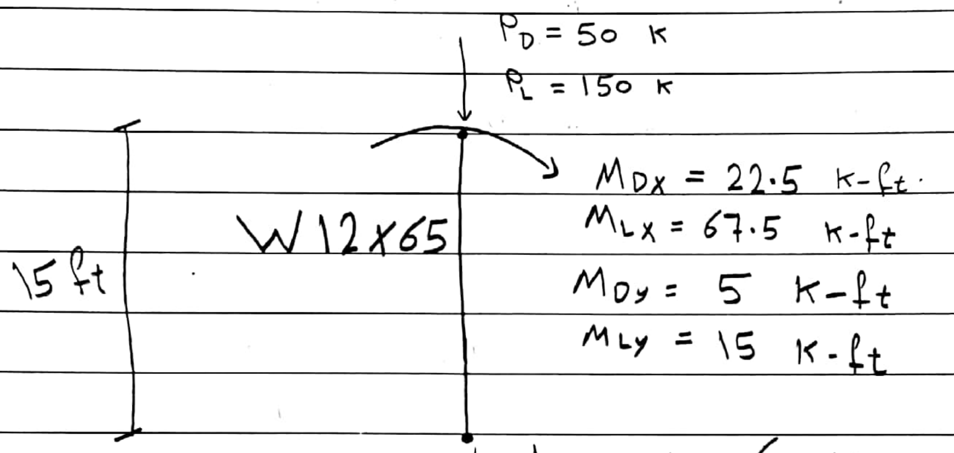
$$\Rightarrow \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi M_{nx}} + 0\right) \leq 1$$

$$\frac{0.1251}{2} + \frac{114.15}{130} = 0.941 < 1 \quad ✓$$

Yes it's adequate.

Example 3: * Service loads & Moments are shown.

* $K_x = K_y = 1$
* $F_y = 50$ ksi
* is it adequate?



↳ hinged (M = 0 يعني).

Sol: $P_u = 1.2(50) + 1.6(150) = 300$ kips

$\phi_c P_n = 662$ kips

$$\frac{P_u}{\phi_c P_n} = \frac{300}{662} = 0.4532 \rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{ux}} + \frac{M_{uy}}{\phi M_{uy}}\right) \leq 1$$

X

* $M_{ux} = B_1 M_{nt} + 0$

$$M_{nt} = 1.2(22.5) + 1.6(67.5) = 135$$

$$B_1 = \frac{C_m}{1 - (P_u/P_e)} \quad , \quad P_e = \frac{\pi^2 (29{,}000)(533)}{(15 * 12)^2} = 4708.5$$

No transverse loads: $C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6$ $\left\{ \begin{array}{l} M_1 = 0 \\ M_2 = 135 \end{array} \right.$

$\Rightarrow B_1 = 0.641 < 1$ x $\boxed{B_1 = 1}$ $\Rightarrow \boxed{M_{ux} = 135 \text{ k-ft}}$

ϕM_{n_x} → Check compactness , $(W12\times65 \overset{f}{=})$ → Non-compact

LTB FLB

① LTB , $L_b = 15$, $L_p = 11.9$, $L_r = 35.1$

, $BF = 5.41$

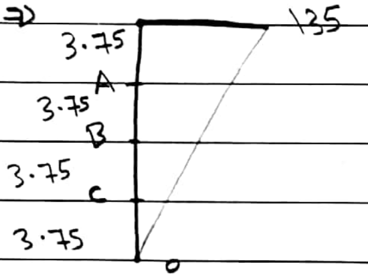
Zone 2 . , $\phi M_p = 0.9 f_y Z_x/12 = 0.9(50)(96.8)/12$

$\phi M_p = 363$ k-ft

non-compact نحسبها حساب لأنه ◄

$\rightarrow * M_{max} = 135$

رسمة المومنت ⇒



من تشابه المثلثات ⇒ $M_A = 101.25$

$M_B = 67.5$

$M_C = 36$

$\Rightarrow \boxed{C_b = 1.656}$

$\phi M_n = C_b [\phi M_p - BF(L_b - L_p)] < \phi M_p$

$= 1.656[363 - 5.41(15-11.9)] < 363$

$\phi M_n = 573 < 363$ X ⟶ $\phi M_n = 363$ k-ft

② FLB ⇒ $\phi M_n = 356$

$\Rightarrow \boxed{\phi M_{n_x} = 356 \text{ k-ft}}$

y : $M_{u_y} = B_1 M_{nt} + 0$

$M_{nt} = 1.2(5) + 1.6(15) = 30$

$B_{1y} = \dfrac{C_m}{1 - P_u/P_e}$, $P_e = \dfrac{\pi^2(29{,}000)(174)}{(15\times12)^2} = 1537$

$C_m = 0.6 \rightarrow B_1 = 0.746$ X $\rightarrow B_{1y} = 1 \rightarrow M_{u_y} = 30$

مهم دير بالك تنساها (اكتبها عالشيت).

$$\Phi M_{ny} = \Phi\left[M_p - \left(M_p - 0.7 f_y \frac{S_y}{12}\right)\left(\frac{\lambda-\lambda_p}{\lambda_r-\lambda_p}\right)\right] \rightarrow$$ لانه non compact.

$$M_p = f_y Z_y = 50(44.1)/12 = 183.75$$

$$\lambda = 9.92, \quad \lambda_p = 0.38\sqrt{\frac{E}{f_y}} = 9.15, \quad \lambda_r = \sqrt{\frac{E}{f_y}} = 24.08$$

$$\Phi M_{ny} = 0.9\left[183.75 - \left(183.75 - 0.7 * 50 * \frac{29.1}{12}\right)\left(\frac{9.92-9.15}{24.08-9.15}\right)\right]$$

$$\Phi M_{ny} = 160.79 \text{ k-ft}$$

$$\Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\Phi M_{nx}} + \frac{M_{uy}}{\Phi M_{ny}}\right) \leq 1$$

$$\Rightarrow \frac{300}{662} + \frac{8}{9}\left(\frac{135}{356} + \frac{30}{160.79}\right)$$

$$= 0.956 < 1$$

✓ adequate.

---

*Design of beam-column (Amin Mansour)

* قام بتبسيط المعادلات الى ٣ متغيرات P , bx , by لتقليل عدد المجاهيل و part 6 من المانيوال موجود فيه هذه القيم لكل سكشن لكن مضروبة بـ (١٥$^{-3}$).

- $P = \frac{1}{\phi_c P_n}$
- $b_x = \frac{8}{9\Phi M_{nx}}$
- $b_y = \frac{8}{9\Phi M_{ny}}$

* جميع القيم في المانيوال مبنية على $C_b = 1$

⇒ Design Procedure (مهم جدا)

① get ultimate loads & moments.

② assume $B_{1x} = 1$, $B_{1y} = 1$

$$M_{ux} = B_{1x} M_x \quad , \quad M_{uy} = B_{1y} M_y .$$

③ assume KL_y controls. → Select any section (عشوائي)
tables 6-1

get $P = ☁ * 10^{-3}$, $b_x = ☁ * 10^{-3}$, $b_y = ☁ * 10^{-3}$

$$\Rightarrow \text{if} \rightarrow P P_u > 0.2 \overset{\text{use}}{\Longrightarrow} P P_u + b_x M_{ux} + b_y M_{uy} < 1 \quad \text{ok}$$

$$\rightarrow P P_u < 0.2 \Rightarrow 0.5 P P_u + \frac{9}{8}\left(b_x M_{ux} + b_y M_{uy}\right) < 1 \quad \text{ok}$$

* if not ok → sellect a larger section

⇒ Start Analysis :

*check which axis controls, $\left(\frac{KL_x}{r_x} , \frac{KL_y}{r_y}\right)$ & get equivalent

if x-controls, equivalent $= \frac{KL_x}{r_x/r_y}$ → select new section.

* Check $B_{1x} = \frac{C_m}{1-\left(\frac{P_u}{P_e}\right)} \rightarrow P_e = \frac{\pi^2 E I_x}{(KL_x)^2}$

in

* Check $B_{1y} = \frac{C_m}{1-\frac{P_u}{P_e}} \rightarrow P_e = \frac{\pi^2 E I_y}{(KL_y)^2}$

* Check which Zone (L_b, L_p, L_r) → calc C_b

$$\Rightarrow C_b \phi M_{nx} = C_b * \frac{8}{9} * \frac{1}{b_x}$$

from Z_x tables, get ϕM_p of the section & compare with $C_b \phi M_n$ ⇒ select smaller (نأخذ الأصغر)

→ New ϕM_{nx}

→ New $b_x = \frac{8}{9}\left(\frac{1}{\phi_b M_{nx}}\right)$, Same P & b_y

⇒ Check Amin Mansour equation again. < 1 ok or not

to get an economy result ⇒ (0.9 − 1)

ناتج تعويض معادلة أمين منصور في النهاية.

Example 1: braced frame (No sides-way)

إرجع لنفس السؤال في السلايدات

have service loads & Moment :-

$P_D = 25k$, $P_L = 75k$

$M_{Dx} = 12.5$ k-ft , $M_{Lx} = 37.5$

$M_{Dy} = 5$, $M_{Ly} = 15$

$M_1 = 0$ ⇑

* all the moment are at one end, the other is Pinned.
* The effective length for each axis is 15 ft = KL
* use A992 steel ($F_y = 50$, $F_u = 65$)
* Select a W10 shape.

Sol: $P_u = 1.2(25) + 1.6(75) = 150$ kips

$M_{ntx} = 1.2(12.5) + 1.6(37.5) = 75$ k-ft

$M_{nty} = 1.2(5) + 1.6(15) = 30$ k-ft

assume $B_{1x} = 1$, $B_{1y} = 1$

$M_{ux} = 75$ Kip-ft , $M_{uy} = 30$ Kip-ft.

assume KL_y controls ⇒ KL = 15 ft

عشوائي → Try W10x49 from tables 6-1

$p = 2.22 * 10^{-3}$, $b_x = 4.35 * 10^{-3}$, $b_y = 8.38 * 10^{-3}$

$\Rightarrow P * P_u = (2.22 * 10^{-3} * 150) = 0.333 > 0.2$

$\Rightarrow$ use $P P_u + b_x M_{ux} + b_y M_{uy} < 1$

عوض ⇒ $0.9107 < 1$ ok ✓

*ملاحظة: لو طلع هذا الرقم قليل كثير مثلاً 0.5 , 0.6 لا تكمل.. اختار سكشن أصغر منه

⇒ No bracing ⇒ y-controls ✓

*Check $B_{1x} = \dfrac{C_m}{1 - \frac{P_u}{P_e}}$, $P_e = \dfrac{\pi^2 (29,000) (\overset{I_x}{272})}{(15 * 12)^2} = 2403$ K

, $C_m = 0.6 - 0.4 \left(\dfrac{0}{M_2}\right) = 0.6$

$B_{1x} = 0.64 < 1$ ✗

$\boxed{B_{1x} = 1}$ ok عادي لو اكبر من 1 خذ القيمة وكمل.

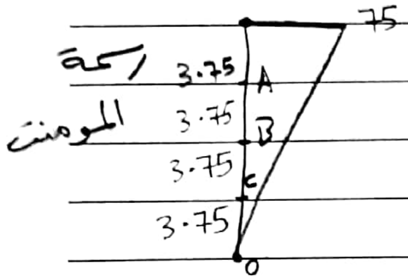
*Check $B_{1y} = \dfrac{C_m}{1 - \frac{P_u}{P_e}}$, $P_e = \dfrac{\pi^2 (29,000) (93.4)}{(15 * 12)^2} = 825.1$ K

$B_{1y} = 0.73 < 1$ ✗ , $C_m = 0.6$

$\boxed{B_{1y} = 1}$ ok

*Check Which Zone : $L_b = 15$, $L_p = 8.97$, $L_r = 31.6$

Zone 2 → Calc C_b



$M_{max} = 75$

$\Rightarrow$

تشابه مثلثات $\Rightarrow M_A = 56.25$

$M_B = 37.5$

$M_c = 18.75$

$\Rightarrow C_b = 1.67$

$\Rightarrow C_b \Phi M_{n_x} = C_b * \frac{8}{9} * \frac{1}{b_x}$

$= 1.67 * \frac{8}{9} * \frac{1}{4.35*10^{-3}} = 341.25$

$\Rightarrow$ From Z_x tables : $\Phi_b M_p = 227 < 341.25$

$\Rightarrow \Phi M_{nx} = 227$ k-ft

$\Rightarrow$ New $b_x = \frac{8}{9}\left(\frac{1}{\Phi M_{nx}}\right) = \frac{8}{9}\left(\frac{1}{227}\right) = 3.916 * 10^{-3}$

Amin Mansour eqn $\Rightarrow PP_u + \underset{\&\ New}{b_x M_{ux}} + b_y M_{uy} < 1$

$0.878 < 1$ Safe ✓ ولكن يجب أن نعمل check على سكشن أصغر منه لنحصل على ناتج بين $(0.9 - 1)$ يعني economy أكثر.

* Try W10 x 45

$P = 3.01 * 10^{-3}$, $b_x = 5.07 * 10^{-3}$, $b_y = 11.7 * 10^{-3}$

*check $PP_u = 0.45 > 0.2 \Rightarrow$ use $(PP_u + b_x M_{ux} + b_y M_{uy} < 1)$

$0.831 < 1$ ok

$B_{1x} = \frac{C_m}{1 - P_u/P_e}$, $P_e = \frac{\pi^2 (29{,}000)(248)}{(15 * 12)^2} = 2191$ Kips.

$B_{1x} = 0.64$ X , $C_m = 0.6$

$\boxed{B_{1x} = 1}$

* $B_{1y} = \frac{C_m}{1 - P_u/P_e}$, $P_e = \frac{\pi^2 (29{,}000)(53.4)}{(15 * 12)^2} = 471.7$ K.
, $C_m = 0.6$

$B_{1y} = 0.88$ X

$\boxed{B_y = 1}$

$\Rightarrow L_b = 15$, $L_p = 7.1$, $L_r = 26.9$

Zone 2 $\longrightarrow$ $C_b = 1.67$

* $C_b \phi M_n = 1.67 * \frac{8}{9} * \frac{1}{b_x} = 292.8$ k-ft.

→ from Z_x tables : $\phi M_p = 206 < 292.8$

$\Rightarrow \phi M_n = 206$ k-ft

→ New $b_x = \frac{8}{9}\left(\frac{1}{206}\right) = 4.32 * 10^{-3}$

$\Rightarrow pP_u + \underset{\overset{\downarrow}{\text{New}}}{b_x} M_{ux} + b_y M_{uy} < 1$

1.13 > 1 X Neglect.
(Not safe).

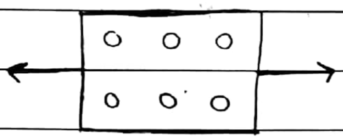
$\Rightarrow$ USE W10X49

Chapter 7

* Bolted connections *

* يوجد نوعين من الـ connections :

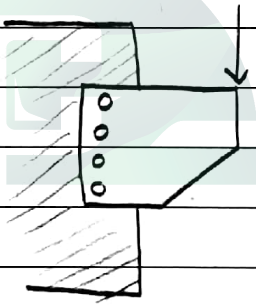
1) Simple connections :



* محصلة اللود في السنترويد لهذا الـ connection .
(No moment)

* كل برغي من هذه البراغي عليه نفس الـ force .. (تتوزع بالتساوي)

2) Eccentrical loaded connections :



* محصلة اللود ليست في السنترويد

* طريقة حل هذه الاسئلة مشروحة آخر إشي بالتشابتر.

* * *

* Types of bolted shear connections :

1) Bearing type :
* مسموح الحركة للبرغي بشكل قليل جداً. (وهو المستخدم دائماً)

2) Slip critical type:
* الحركة غير مسموحة أبداً (مشدود عالآخر). (أقل استخدام)

⇒ Types of failure in Connections :

*أشياء لازم تعمل عليها بال check connections :-

1 Shear Strength

$\boxed{\phi R_n = \phi F_{nv} A_b}$ * for one bolt

* $\phi R_{n\,\text{total}} = \phi R_n *$ عدد البراغي

ϕR_n : nominal strength for one bolt

$\boxed{\phi = 0.75}$

A_b : area of the bolt $= \frac{\pi}{4} d_b^2$

F_{nv} : nominal shear stress : جاهزة من المانيوال table J3.2

صفحة ١٥٤ بعد جداول U ووو

*نأخذ القيمة حسب نوع البراغي وحسب الـ Plane of Shear

*ملاحظة: (عند أخذ قيمة Fnv من الجدول) :- (اكتبهم عالمانيوال)

حسب شو معطيك بالسؤال :-

١- (in Plane of shear) نأخذ القيمة الأصغر (not excluded)

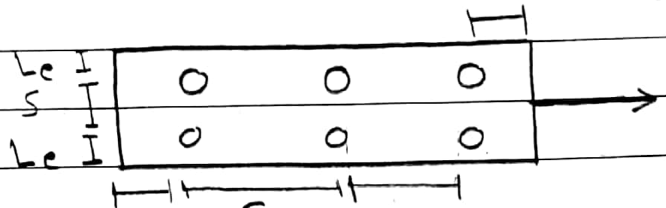
٢- (not in Plane of Shear) نأخذ القيمة الأكبر (excluded)

*مشكلة الـ Shear يعني انه البرغي نفسه ينكسر (ينقطع)

Note ⊛ gusset plate : هي الـ plate التي يكون مثبت عليها الـ tension member

[2] Bearing Strength :

L_e : edge distance



(edge) L_c S from center to center. L_c : Clear distance (other)

* مشكلة الـ bearing يعني انه البرغي بجزع الـ Plate

$$\boxed{R_n = 1.2 L_c t F_u \leq 2.4 d_b t F_u}$$ (for 1 bolt)

t : thickness .

$\phi R_n = 0.75$.

* نقسم البراغي الى مجموعتين :- ١- edge : البراغي اللي ع الحفة
٢- other : باقي البراغي الموجودة . ونحسب ϕR_n لكل منهم

* بدي أحسب ϕR_n مرتين مرة للـ tension member ومرة للـ guesset plate

اذا كانت المسافات الأفقية والعامودية بين البراغي مختلفة اما اذا كانت متساوية نحسب فقط للأقل سماكة (t) بينهم

⊛ Edge bolts : $L_c = L_e - \frac{h}{2}$

⊛ other bolts : $L_c = S - h$

h: diameter of the hole
$h = d_b + \frac{1}{16}$

* L_e : نفس مكانه الـ Fnv بعد جداول لا .. حسب قطر البرغي (نختار الأكبر)

$* \; S \geq \frac{8}{3} d_b$

$$\Rightarrow \phi R_{n_{total}} = \underbrace{n_e \quad (\phi R_{n_e}) + n_o}_{\text{عدد براغي}} (\phi R_{n_o})$$

$\boxed{3}$ Yielding : (for the tension member)

$$\phi R_{tot} = 0.9 \, f_y A_g$$

$\boxed{4}$ Fracture : (tension member)

$$\phi R_{n_{tot}} = 0.75 \, F_u A_e$$

for plate $U = 1$

$A_e = U A_n$

$A_n = A_g - A_{holes}$

$\boxed{5}$ Block Shear : (tension member)

$$\phi R_{n_{tot}} = 0.75 \left[0.6 F_u A_{nv} + F_u A_{nt} < 0.6 F_y A_{gv} + F_u A_{nt} \right]$$

$\boxed{6}$ Slip critical : (من المانيوال عند Fnr و Le)

for one bolt

$$R_n = 0.35 * 1.13 * T_b * N_s$$

Single shear : $N = 1$

double shear : $N = 2$

* $\phi = 1$: إذا حسبت ال 5 اللي قبلها كلهم.

* $\phi = 0.85$: اذا ما بدك تحسب غير Slip-critical فقط.

$$\phi R_{tot} = \phi R_n * \text{عدد البراغي}$$

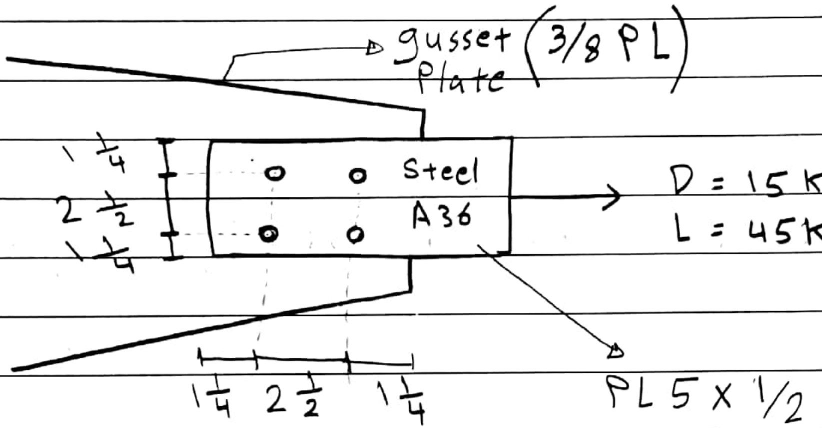
Example: Check bolt spacing $\underline{\underline{S}}$,
edge distance Le,

* bearing in connection — يعني (حسب ١، ٢، ٣، ٤، ٥

slip-critical بدونه

ناخذ الأكبر

* bolts used A325 With threads <u>not</u> in plane of Shear.



$\boxed{d_b = 3/4}$

<u>Solution:</u> - Check $S \geq \frac{8}{3} d_b$

$2.5 \geq \frac{8}{3}\left(\frac{3}{4}\right) \rightarrow 2.5 > 2$ ok ✓

- Check $Le = 1\frac{1}{4}$

* from manual Le (Minimum) $= 1\frac{1}{4}$ ok ✓

$P_u = 1.2(15) + 1.6(45) = 90$ K.

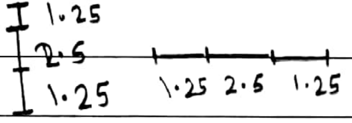
① Shear strength: $\phi R_n = 0.75\ F_{nv} A_b$

$= 0.75(60)\left(\frac{\pi}{4}\left(\frac{3}{4}\right)^2\right) = 19.88$ K

$\phi R_{n_{tot}} = 19.88(4) = 79.52$ K

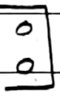
② Bearing strength:

$R_n = 1.2 * L_c * t * F_u \leqslant 2.4 * d_b * t * F_u$

* بما أن المسافات متساوية  نحسب فقط للأقل t

(فقط هذا السؤال حسب الشنتين للتوضيح فقط)

وهي ال gusset plate ← $t = 3/8$

لل gusset البرغيين  على اليمين هم ال edge

edge: $L_c = L_e - \frac{h}{2} = 1.25 - \frac{(\frac{3}{4} + \frac{1}{16})}{2} = 0.8438$ in

$R_{n_e} = 1.2(0.8438)(3/8)(58) \leqslant 2.4(3/4)(3/8)(58)$

$= \boxed{22.02} \leqslant 52.2$

$\longrightarrow \phi R_{n_e} = 16.52$ K

other: $L_c = S - h = 2.5 - (\frac{3}{4} + \frac{1}{16}) = 1.688$ in

$R_{n_o} = 1.2(1.688)(3/8)(58) \leqslant 52.2$

$44.06 \leqslant 52.2$

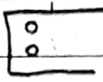
$\longrightarrow \phi R_{n_o} = 33.05$ K

$\phi R_{n_{tot}} = 2(16.52) + 2(33.05) = 99.14$ k

}

for gusset plate

* For tension member : $(t=1/2)$:



البرغيين على اليسار هم الـ edge

<u>edge</u> : $L_c = L_e - \frac{h}{2} = 0.8438$

$R_n = 1.2(0.8438)(1/2)(58) \leqslant 2.4(3/4)(1/2)(58)$

$= 29.36 \leqslant 54.38$

$\hookrightarrow \phi R_{n_e} = 22.02$ K

<u>other</u> : $L_c = S - h = 1.688$

$R_n = 1.2(1.688)(1/2)(58) \leqslant 2.4(3/4)(1/2)(58)$

$58.74 \leqslant \boxed{54.38}$

$\hookrightarrow \phi R_{n_o} = 40.79$ K

$\phi R_{n_{tot}} = 2(22.02) + 2(40.79) = 125.62$ K

اكبر من القيمة اللي حسبناها
للـ gusset عشانه هيك احنا
بنحسب فقط للي سماكتها أقل.

$\boxed{3}$ Yielding : $\phi R_n = 0.9(36)(5 * 1/2)$
$= 81$ K

$\boxed{4}$ Fracture : $\phi R_n = 0.75 F_u A_e$

لانه $U=1$

$A_e = A_n = (5 * 1/2) - 2 * (1/2)(3/4 + \frac{1}{8}) = 1.625$

$\phi R_n = 70.69$ K

$\boxed{5}$ Block shear : $\phi R_n = 0.75 [0.6 F_u A_{nv} + F_u A_{nt} < 0.6 F_u A_{gv} + F_u A_{nt}]$

* سوف يحدث في الـ gusset لانها أقل سماكة.

block

*توضيح أرقام فرع 5 الـ Shear موجود

في المثال التالي فرع 5 .

$*A_{gv} = 2*3.75*3/8 = 2.8125\,in^2$

$*A_{nv} = 2.8125 - 3\,(3/8)(3/4+1/8) = 1.828$

$*A_{nt} = 2.5*3/8 - 1\,(3/8)(3/4+1/8) = 0.609$

$$\phi R_n = 0.75\left[\,98.94 < \boxed{96.07}\,\right]$$

$$\Rightarrow \boxed{\phi R_n = 72.05\ K}$$

*بالأخير أقل قيمة تكون هي الـ control ولكن في السلايدات

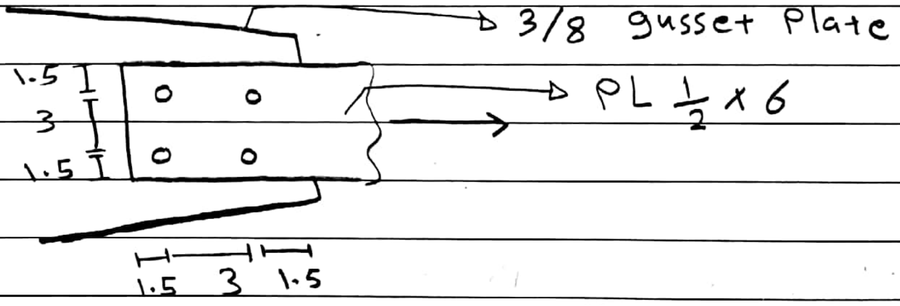
ما حسب ٣،٤،٥ فأخذ $\phi R_n = 79.38$

$P_u > \phi R_n$ ✗ Not ok

Example: $d_b = 3/4$ in

A325 bolts with threads in plane of shear.

بدك تحسبهم الـ٦ كلهم. ← No Slip is Permitted , Steel A36 .



find ϕR_n

Sol: ① Shear : $\phi R_n = 0.75\,(48)\left(\frac{\pi}{4}(3/4)^2\right) = 15.9\ K$

$$\phi R_{n_{tot}} = 4\,(15.9) = 63.6\ K$$

② Bearing : $R_n = 1.2\,L_c\,t\,F_u \leq 2.4\,d\,t\,F_u$

* only for gusset $\Rightarrow t = 3/8$ (same spacings)

edge: $L_c = 1.5 - \frac{(3/4 + 1/16)}{2} = 1.094$

$R_n = 1.2(1.094)(3/8)(58) < 2.4(3/4)(3/8)(58)$

$28.55 < 39.15$

$\phi R_n = 0.75(28.55) = 21.41\ K$

other: $L_c = 3 - (3/4 + 1/16) = 2.1875$

$R_n = 57.1 < 39.15$

$\phi R_n = 0.75(39.15) = 29.36\ K$

$\Rightarrow \phi R_{n_{tot}} = 2(21.41) + 2(29.36) = 101.54K$

$\boxed{3}$ Yielding : (for the tension plate)

$\phi R_n = 0.9(36)(6 * 1/2) = 97.2\ K$

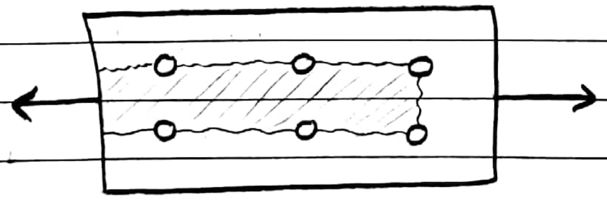
$\boxed{4}$ Fracture : (for the tension member) : $\phi R_n = 0.75\ F_u A_e$

$A_e = A_n = (6 * 1/2) - 2(1/2)(3/4 + 1/8) = 2.125$

$\phi R_n = 92.44\ K$

$\boxed{5}$ Block Shear : $0.75[0.6 A_{nv} F_u + F_u A_{nt} < 0.6 F_y A_{gv} + F_u A_{nt}]$

*مش مشروح في الفيرست شكل الكسر في الـ block shear لما يكونه عند two lines of bolts



هيك بيكونه

~~* عدّل أرقام المثال السابق بفرع الـ block shear~~

$A_{gv} = 2 * 4.5 * 3/8 = 3.375$

$A_{nv} = 3.375 - \underbrace{3}_{\text{عدد البراغي}}(3/8)(3/4 + 1/8) = 2.39$

$3 = (1 + 1 + 0.5 + 0.5)$

$A_{gt} = 3 * 3/8 = 1.125$

$A_{nt} = 1.125 - 1(3/8)(3/4 + 1/8) = 0.7969$

$1 = (0.5 + 0.5)$

* الدكتور في السلايدات معوض قيمة t للـ gusset وهي 3/8 وليس 0.5 لأنه الـ block shear يحدث في الـ plate التي لها أقل سماكة (t) .

$\Rightarrow \phi R_n = 0.75 [129.39 < \boxed{119.12}]$

$\phi R_n = 89.3\ K$

$\boxed{6}$ Slip-critical: $R_n = 0.35 * 1.13 * T_b * N_s$

$= 0.35 * 1.13 * (28)(1) = 11.074$

لأني حسبت الباقيين

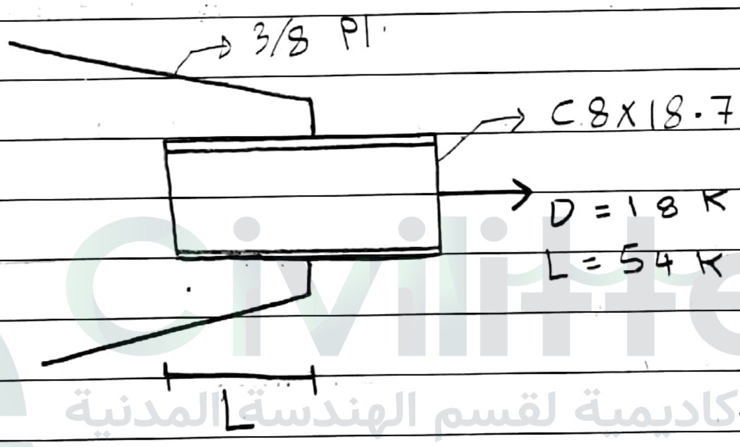
$\Rightarrow \phi R_{n\,tot} = 1 * 11.074 * \underbrace{4}_{\text{عدد البراغي}}$

$\phi R_n = 44.296\ K$

$\Rightarrow$ Design $\phi R_n = 44.3\ K$ الأقل

Example (Design): The C8X18.7 Shown has been selected to resist a service dead load of 18K & service live load 54 K, it's attached to a 3/8 in gusset plate with 7/8 in bolt diameter (A325 bolts), assume that the threads are in plane of shear & that the slip of the connection is permissible, Determine number and required lay-out of bolts such that the length of the connection (L) is minimum, A36 steel used.

bearing type يعني



لاحظ أنه التثبيت في الـ web

$t_w = 0.487$

Sol: أول خطوة $P_u = 1.2(18) + 1.6(54) = 108\ K$

① Shear strength: $\Phi R_n = 0.75\ F_{nv}\ A_b$

$= 0.75(48)\left(\frac{\pi}{4}(7/8)^2\right) = 21.65\ K$

(one bolt)

② Bearing strength: use upper limit: (use least $\underline{\underline{t}}$)

$t_{gusset} = 3/8 < t_w = 0.487$

$\Rightarrow \Phi R_n = 0.75 * 2.4 * d * t * F_u$

$= 0.75 * 2.4 * 7/8 * 3/8 * 58 = 34.26\ K$

Check which one controls $\left(\begin{matrix}\text{Shear} \\ \text{or} \\ \text{bearing}\end{matrix}\right)$ (الأصغر)

$*$ Shear controls : $\phi R_n = 21.65$

$*$ $\text{No of bolts} = \dfrac{P_u}{\phi R_n} = \dfrac{108}{21.65} \approx 5$ bolts

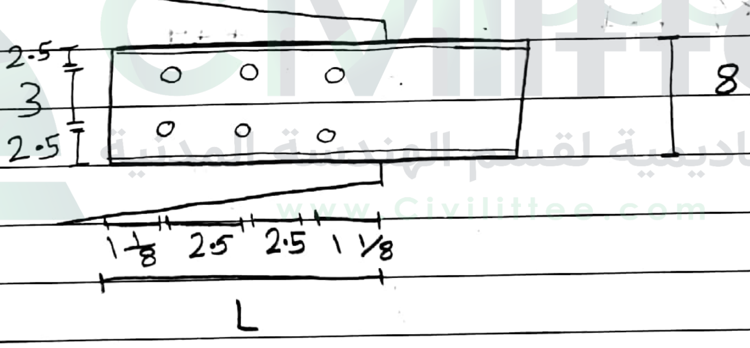
use $\boxed{6 \text{ bolts}}$

* عشان ما نستخدم staggered دائماً خذ عدد زوجي.

$S \geq 8/3\, d_b \rightarrow S \geq 2.33 \Rightarrow S = 2.5$ in

Le min from manual $= 1\frac{1}{8}$ in $= Le$

هدول بنحطهم من عنا لانه محصورين بارتفاع السكشن (نستخدم قيم اكبر من ال minimum)



edge: $L_c = L_e - h/2 = 1\frac{1}{8} - \dfrac{(7/8 + 1/16)}{2} = 0.656$

$R_n = 1.2(0.656)(3/8)(58) < 2.4(7/8)(3/8)(58)$

$\boxed{17.12} < 45.675$

$\phi R_{ne} = 12.84$ K

other: $L_c = S - h = 2.5 - \left(\frac{7}{8} + \frac{1}{16}\right) = 1.563$

$R_n = 1.2(1.563)(3/8)(58) < 45.675$

$\boxed{40.68} < 45.675 \longrightarrow \phi R_{no} = 30.51$

$\phi R_{n\,tot} = 2(12.84) + 4(30.51) = 147.72$ K

bearing $\phi R_n = 147.72 > \underset{108}{P_u}$ ok ✓.

والـ Shear اكيد OK لاني صممت عليه أصلاً.

③ Yielding : $\phi R_n = 0.9(36)(5.51) = 178.56$ K > 108

مانيوال OK ✓.

C8×18.7

④ Fracture : $\phi R_n = 0.75\, F_u A_e$

$A_n = 5.51 - 2(0.487)(7/8 + 1/8) = 4.536$ in²

مانيوال $U = 1 - \bar{X}/L = 1 - \frac{0.565}{2.5} = 0.774$

(case $\underline{\underline{2}}$)

* في السلايدات استخدم case 8 وأخذ U=0.6 لانه ما كان ماحسب قيمة S .. هيك الصح

$\Rightarrow \phi R_n = 152.7$ K $> P_u$ OK ✓.

⑤ Block Shear Strength: (in gusset $t = 3/8 < 0.487$)

$\phi R_n = 0.75\left[0.6\, F_u A_{nv} + F_u A_{nt} < 0.6 F_y A_{gv} + F_u A_{nt}\right]$

$A_{gv} = \overset{\text{two lines}}{\underline{\underline{2}}}(3/8)\left(2.5 + 2.5 + 1\tfrac{1}{8}\right) = 4.594$

$A_{nv} = 4.594 - \underset{\text{عدد البراغي }(2+0.5+0.5+2)}{(5)} * (3/8)(7/8 + 1) = 2.719$

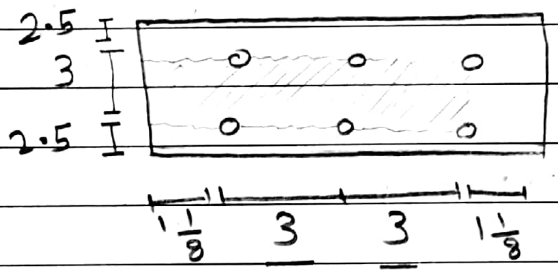
$A_{gt} = 3(3/8) = 1.125$

$A_{nt} = 1.125 - \underset{\text{عدد البراغي }(0.5+0.5)}{1}(3/8)(7/8 + 1/8) = 0.75$

$\phi R_n = 0.75\left[\boxed{138.1} < 142.7\right]$

$\phi R_n = 103.6\ K < P_u = 108$ Not OK!

* اذا كان عندك مشكلة block shear نقوم بتكبير المسافة الافقية S حيث أننا بذلك نقوم بتقوية ال shear Plane.



كيف أجا الرقم

$\phi R_n = P_u$

$\longrightarrow 0.75\left[0.6 * 58 * A_{nv} + 58(0.75)\right] = 108$

A_{nt} نفسها

$\Rightarrow A_{nv} = 2.888\ in^2 = A_{gv} - A_{holes}$

$2.888 = 2(S + S + 1.125)(3/8) - 5(3/8)(7/8 + 1/8)$

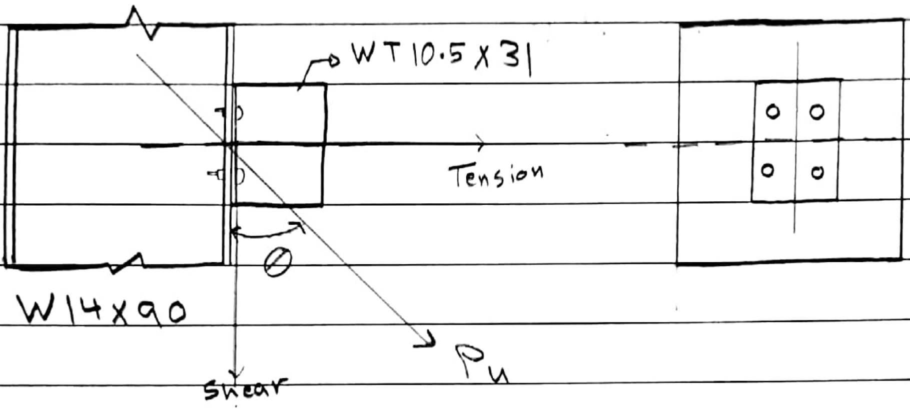
$\rightarrow S = 2.888 \Rightarrow$ use $\boxed{S = 3\ in}$

$\Rightarrow \phi R_n = 0.75\left[\boxed{144} < 158.9\right]$

$\phi R_n = 108\ K \geq P_u$ OK ✓

☒ Bolts subjected to shear & tension.

* هاي الحالة لما يكون ال load مائل.



* في الرسمة .. التربيط بين two flanges

*Procedure:

① نحلل القوة P_u → $V_{u\,tot} = P_u \cos\theta$ → use in:
1-Shear
2-Bearing
3-yielding
4-fracture
5-block shear
6-Slip critical

→ $T_{u\,tot} = P_u \sin\theta$

$\Downarrow$

$$\boxed{F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}}(f_v) < F_{nt}}$$

F_{nt} → من المانيوال من نفس جدول F_{nv}

ϕ → 0.75

$f_v = \frac{V_u \text{ (on one bolt)}}{A_{bolt}}$

F'_{nt} : nominal tensile stress with shear.

$\phi R_n = 0.75 F'_{nt} A_b$ → for one bolt.

*if > T_u (on one bolt) ✓ ok.

* Slip critical with tension in equation:

$$\boxed{K_s = 1 - \frac{T_{u\,(tot)}}{1.13\, T_b\, N_b}}$$ → عد البراغي

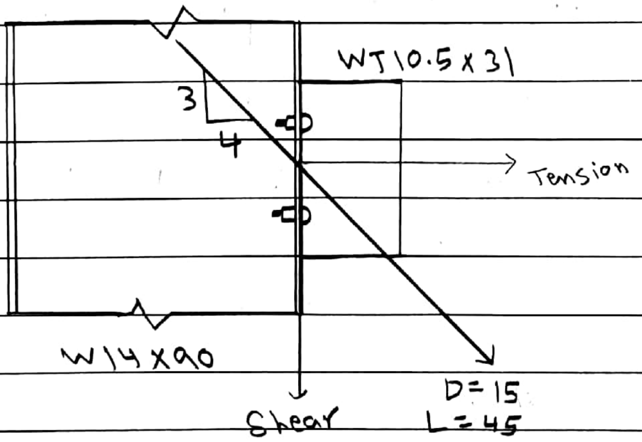
$K_s * \phi R_n > V_{u\,(tot)}$ OK ✓

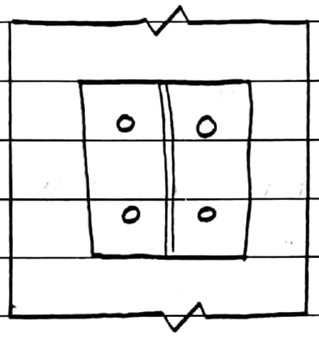
↓ ع

from slip critical $\underline{\underline{6}}$.

<u>**Example:**</u> a WT10.5x31 is used as a bracket to transmit a 60-kip service load to W14x90 column, The load consists of 15K dead load & 45K live load, Four bolts (7/8) in diameter, A325 bolts used,
The column is A992 steel (Fy = 50, Fu = 65)
The bracket is A36 steel (Fy = 36, Fu = 58)
*assume all edge distances & spacings are satisfying all requirements, including those necessary for the use of the maximum nominal bearing strength (2.4 d t Fu)

⇒ determine the adequacy of bolts:





مش معطيني أرقام
عشان استخدم الـ upper-limit
في الـ bearing.

* طبعاً في الامتحان بيحددلك بكل فرع شو تحسب ما بتركك تحسب كل اشي مثل هيك .. هذا فقط توضيح ومراجعة.

Solution: $P_u = 1.2(15) + 1.6(45) = 90\ K$

* $V_{u\ tot} = 90 * (3/5) = 54\ Kips$

* V_u on one bolt $= 54/4 = 13.5\ K$

* $T_{u\ tot} = 90 * (4/5) = 72\ K$

* T_u on one bolt $= 72/4 = 18\ K$

① Shear strength: $\Phi R_n = 0.75\ f_{nv}\ A_b$ (for 1 bolt).

$\Phi R_n = 0.75(48)\left(\frac{\pi}{4}(7/8)^2\right) = 21.65\ K > 13.5$ ok ✓.

② Bearing strength: $\Phi R_n = 0.75(2.4\, d\, t\, F_u)$ for 1 bolt

$\Phi R_n = 0.75(2.4(7/8)(0.615)(58)) = 56.18\ K > 13.5$ ok ✓

W14x90
WT10.5x31 الأقل من بين t_f

③ Yielding: $\Phi R_n = 0.9\, f_y\, A_g$ → for the bracket

$= \frac{0.9(36)(9.13)}{4} = 73.95\ K > 13.5$ ok ✓

4 ← لكي تصبح للبرغي الواحد.

④ Fracture: $\Phi R_n = 0.75\ F_u\ A_e$ (for the bracket)

$A_n = 9.13 - 2(0.615)(7/8 + 1/8) = 7.9$ (take $U = 0.9$)

$\Rightarrow \Phi R_n = 309.3 \rightsquigarrow \underset{\text{one bolt}}{\Phi R_n} = 77.3 > 13.5$ ok ✓.

⑤ block shear: * لا يمكن حسابه بدون المسافات.

ملاحظة : لو طلب منك تحسب slip critical لازم تحسبه
الفرعين ⑥ و ⑧ مع بعض . (فرع 6 غير كافي لوحده)

⑥ Slip critical : $R_n = 0.35 * 1.13 * 1 * T_b * N_s$ (for 1 bolt)

$\phi R_n = 1 * (0.35 * 1.13 * 39 * 1) = 15.4\ k > 13.5$ ok ✓.

Now using T_u :

⑦ Tension:

$$f'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} (f_v) < F_{nt}$$

$$= 1.3(90) - \frac{90}{0.75(48)} \left(\frac{13.5}{\frac{\pi}{4}(7/8)^2} \right) = \boxed{60.9} < 90$$

$$\Rightarrow \phi R_n = 0.75\, f'_{nt} A_b = 27.45\ k > \underset{(T_u \text{ on bolt})}{18\ k} \quad \text{ok} ✓$$

⑧ Slip-critical (with tension) . * المثال اللي بعده عبارة عن فرع 8 ←

$$K_s = 1 - \frac{T_u\,(tot)}{1.13\, T_b\, N_b} = 1 - \frac{72}{1.13(39)(4)} = 0.592$$

$$K_s \underset{\text{from } \underline{\underline{6}}}{\underbrace{\phi R_n}}{}_{tot} = 0.592(15.4 * 4) = 36.4 < V_u = 54\ k$$

✗ Neglect

* not adequate in slip-critical.

*Eccentric connections

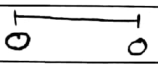
* موضوع الـ elastic محذوف من سلايد 30 ← 33 .

* هذا الموضوع عبارة عن قسمين:

الأول وهو عبارة عن حل طويل جداً وهي الطريقة التي تستخدمها الـ (Softwares) في الحل ولكن يدوياً أي سؤال على هذا الموضوع وحتى لو اعطاك قيمة Ro الصحيحة في السؤال فإنه السؤال سيكون طويل جداً ومستحيل وما بحياته اجا سؤال عليه مثل المثال الأول بالسلايدات (Slide 35)

الثاني: عن طريق إستخدام Part 7 من المانيوال لحساب ØRn والمثال الثاني في Slide(38) عليها .. وهي الطريقة المشروحة في الدوسية فقط.

* عشان أوجد الصفحة الصح في المانيوال بتطلع على ٣ شغلات في السؤال:

1) No of vertical rows of bolts.
2) Angle of load
3) Spacing between vertical rows 

} الرسمة اللي فوق عاليمين في المانيوال.

* get ex (distance from P to the center of bolts)

S : vertical distance between bolts

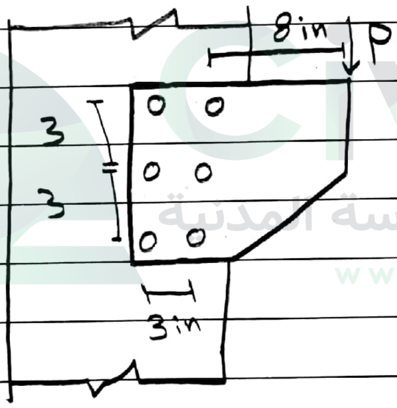
using $\left\{ \begin{array}{l} S \\ e_x \\ \text{No of Bolts in one vertical row} \end{array} \right\} \Rightarrow$ get $\underline{\underline{C}}$ (interpolation is ok)

$\boxed{r_n = F_{nv} * A_b}$ r_n: strength of 1 bolt.

$\boxed{\phi R_n = 0.75 * C * r_n}$ ϕR_n: strength of the connection.

Example: * 3/4 in bolts.

* A325 bearing with threads in plane of shear.
* Bolts are in single shear.



دائما عليه سؤال في الفاينل

Sol: ① two rows (vertical)
② Angle = 0
③ Spacing = 3
$\Rightarrow$ Page 7-38

* $e_x = 8 + 3/2 = 9.5$ in (from $\underline{P}$ to center).
* 3 bolts in one vertical row.

$\Rightarrow$

e_x	C
9	1.6
10	1.46

$\Rightarrow$ at $e_x = 9.5 \Rightarrow C = 1.53$

$r_n = F_{nv} * A_b = 48\left(\frac{\pi}{4}(3/4)^2\right) = 21.21$ k (1 bolt)

$\Rightarrow \phi R_n = 0.75 * C * r_n = 0.75(1.53)(21.21)$

$\phi R_n = 24.34$ Kips

شيت فاينل

Shear

$$\phi V_n = \phi(0.6\, F_y A_w)\, C_v$$

W Shape $\phi = 1$ $C_v = 1$

$K_v = 5$

check $h/t_w < 260$

* Not W-section : $\phi = 0.9$

① $h/t_w \leqslant 1.1\sqrt{K_v E/F_y}$ → $C_v = 1$

② $1.1\sqrt{K_v E/F_y} \leqslant h/t_w \leqslant 1.37\sqrt{K_v E/F_y}$ → $C_v = \dfrac{1.1\sqrt{K_v E/F_y}}{h/t_w}$

③ $h/t_w > 1.37\sqrt{K_v E/F_y}$ → $C_v = \dfrac{1.51\; K_v E}{(h/t_w)^2\; F_y}$

* $\boxed{A_w = d\, t_w}$

Deflection

كل اشي بالانش

get Δ_{max} (from manual cases), get $\Delta_{allow} = L/360$ → inches.

if $\Delta_{max} < \Delta_{allow}$ → OK ✓.

* using only service live loads

* if No section (design)
$\Delta_{max} = \Delta_{allow}$
get I_x → get section

Biaxial bending (Moment in x, y)

$$\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}} \leqslant 1$$

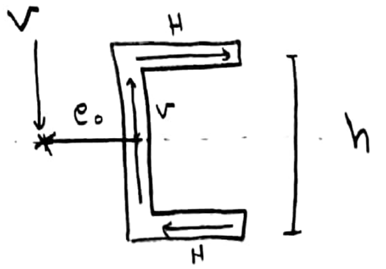
(M_{ux} & M_{uy} from structure)

x: check compactness → check which zone → get ϕM_{nx}

y: always Zone 1 ($\phi M_n = \phi M_p = \phi f_y Z_y$) → if non-compact → $\phi M_{ny} = \phi\left[M_p - \left(M_p - \frac{0.7 F_y S_x}{12}\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right)\right]$

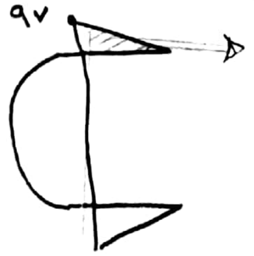
Shear-Center

*Shear flow $q_v = \dfrac{QV}{I_x}$ (at certain point), $Q = A d$



* $Hh = Ve$ (if american, get e from manual)

H: محصلة ال Shear flow (مساحة)



$H = \frac{1}{2} bh$ مساحة مثلث

Beam - column

axial →
$$\frac{P_u}{\phi_c P_n} \geq 0.2 \Rightarrow \left[\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right)\right] \leq 1$$

$$\frac{P_u}{\phi_c P_n} < 0.2 \Rightarrow \left[\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right)\right] \leq 1$$

* get $\phi_c P_n$ from Manual, $\underset{F_y=50}{KL}$ → table 4-1

* $M_{ux} = B_1 M_{nt} + \cancelto{0}{B_2 M_{lt}}$

$$B_1 = \frac{C_m}{1-\left(\frac{P_u}{P_{e1}}\right)} \geq 1 \quad , \quad P_{e1} = \frac{\pi^2 E I_x}{\underset{\hookrightarrow \text{inches}}{(KL)^2}}$$

* $M_{uy} = B_1 M_{nt} + \cancelto{0}{B_2 M_{lt}}$

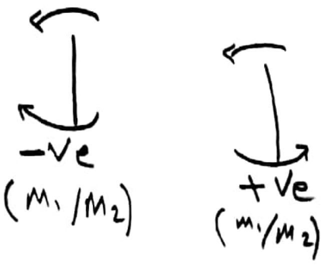
$$B_1 = \frac{C_m}{1-\left(\frac{P_u}{P_{e1}}\right)} \quad , \quad P_{e1} = \frac{\pi^2 E I_y}{\underset{\hookrightarrow \text{inches}}{(KL)^2}}$$

* ϕM_{nx} : Check compactness → check zone → get ϕM_n $\begin{cases} \to \text{LTB} \\ \to \text{FLB (if non-compact)} \end{cases}$

* ϕM_{ny} : always zone 1 ($\phi M_n = \phi F_y Z_y$) but if non-compact ↴

$$\phi M_{ny} = \phi\left[M_p - \left(M_p - 0.7\frac{F_y S_y}{12}\left(\frac{\lambda-\lambda_p}{\lambda_r-\lambda_p}\right)\right)\right]$$

* if No transverse loads : $C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right)$,



* if there is → manual (get C_m)

Design of beam column (Amin Mansour)

* get ultimate loads & Moments

→ assume $B_{1x} = 1$, $B_{1y} = 1$

⇒ $M_{ux} = B_{1x} M_x$, $M_{uy} = B_{1y} M_y$

* assume KL_y controls ⇒ select any section (6-1) عشوائي

⇒ get $P = \bigcirc * 10^{-3}$, $b_x = \bigcirc * 10^{-3}$, $b_y = \bigcirc * 10^{-3}$, of the section.

1) $P P_u > 0.2 \rightarrow P P_u + b_x M_{ux} + b_y M_{uy} < 1$ ok.

2) $P P_u < 0.2 \rightarrow 0.5 P P_u + \frac{9}{8}\left(b_x M_{ux} + b_y M_{uy}\right) < 1$ ok

* if not ok sellect a larger Section

* Try to have section ⇒ (0.9 → 1)

⇒ Start analysis:

Check which axis is control, $\left(\frac{KL_x}{r_x}, \frac{KL_y}{r_y}\right)$, if x-controls → get equivalent $eq = \frac{KL_x}{r_x/r_y}$ → Select new Section

* check $B_{1x} = \frac{C_m}{1-\left(\frac{P_u}{P_{e1}}\right)}$, $P_{e1} = \frac{\pi^2 E I_x}{(KL)^2}$

$L_{b\,in}$

* check $B_{1y} = \frac{C_m}{1-\left(\frac{P_u}{P_{e1}}\right)}$, $P_e = \frac{\pi^2 E I_y}{(KL)^2}$

⇒ L_b , L_p , L_r → (Zone ?) → Calc $\underline{C_b}$

⇒ $C_b \Phi M_{nx} = C_b * \frac{8}{9} * \frac{1}{b_x}$

* from Z_x tables → get $\Phi_b M_p$ of section ... compare with & select smaller.

→ New $\Phi_b M_{nx}$.

→ New $b_x = \frac{8}{9}\left(\frac{1}{\Phi_b M_{nx}}\right)$, P & by still the same.

⇒ Check Amin mansour eqn again.. < 1 (ok or not)....

get P_u, then check :

Connections

$\boxed{5}$ شيت فاينل

① Shear strength : $\phi R_n = \phi F_{nv} A_b$ (for 1 bolt) → $\phi R_{n_{tot}} = \phi R_n *$ عدد البراغي

0.75 ↑ table ↑ $\frac{\pi}{4} d_b^2$

---

② Bearing strength :

$R_n = 1.2 L_c t F_u \leqslant 2.4 d t F_u$ (for 1 bolt).

L_c ↑ clear, t ↑ thick, F_u ↑ for plate not bolt.

$\phi R_n = 0.75 R_n$.

R_n for gusset & tension plate if different spacings

⇒ for same ... calc only for the least t.

L_e: tables

⇒ get ϕR_n for → edge bolts → $L_c = L_e - \frac{h}{2}$

→ other bolts → $L_c = S - h$

$h = d_b + \frac{1}{16}$

* $S \geqslant \frac{8}{3} d_b$

$\phi P_n = n_e (\phi R_{n_e}) + n_o (\phi R_{n_o})$ → عدد براغي

---

③ Yielding : $\phi P_n = 0.9 f_y A_g$

---

④ fracture : $\phi P_n = 0.75 f_u A_e$ } $A_e = U A_n$ ($A_n = A_g - A_{holes}$) ↳ $h * t * n$

U=1 plate

---

⑤ Shear block : $\phi P_n = 0.75 [0.6 F_u A_{nv} + F_u A_{nt} < 0.6 F_y A_{gv} + F_u A_{nt}]$

---

⑥ Slip critical : $R_n = 0.35 * 1.13 * 1 * T_b * N_s$ (for 1 bolt).

T_b ↓ Manual, N_s ↓ (No of Plates -1)

$\phi = 1$

$\phi = 0.85$ (only slip critical) $\phi P_n = \phi R_n * n_{bolts}$

Design of bolted connections

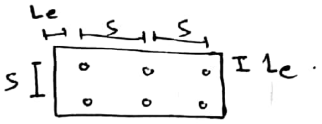
get $\underline{P_u}$.

① Shear strength $\longrightarrow \phi R_n = 0.75 f_{nv} * A_b$

② Bearing upper limit $\longrightarrow$ ~~2~~$\phi R_n = 0.75 * 2.4 * d * t * F_u$ → least for two plates.

* Check which controls → shear or bearing $\Rightarrow$ No of bolts $= \dfrac{P_u}{\phi R_n} =$ خذ عدد زوجي.

* $S \geqslant \frac{8}{3} d_b$
L_e : table



$\Rightarrow$ get ϕR_n for edge & other bolts $\Rightarrow$ get total bearing strength.

③ yielding $\longrightarrow$ ④ fracture $\longrightarrow$ ⑤ block shear

Bolts subjected to shear & tension

$\Rightarrow$ get total ultimate shear load $(V_{u\,tot})$ $\left(\begin{smallmatrix}\sin\\ \cos\end{smallmatrix}\right) \longrightarrow$ use in :
1 - shear
2 - bearing
3 - yield , 4 fracture
5 - block shear
6 - slip critical.

$\Rightarrow$ get total ultimate tension load $(T_{u\,tot})$

$$f'_{nt} = 1.3 \underbrace{F_{nt}}_{\text{manual}} - \frac{F_{nt}}{\underbrace{\phi}_{0.75} F_{nv}} (f_v) < f_{nt}$$

tensile stress with shear

$f_v \rightarrow \dfrac{V_u \text{ (on bolt)}}{A_{bolt}}$.

$\Rightarrow \phi R_n = 0.75 f'_{nt} A_b \longrightarrow$ (for 1 bolt) $> T_u$ (on bolt) ok.

* Slip critical with tension in question.

$$K_s = 1 - \frac{T_{u\,(tot)}}{1.13\, T_b\, N_b}$$

N_b → عدد البراغي.

$K_s * \underset{\text{from slip critical } \underline{6}}{\phi P_{n\,tot}} > V_{u\,(tot)}$ ok.

Eccentric connection

Tables Part 7 : to get page $\Rightarrow$ (No of vertical rows , Angle , spacing)

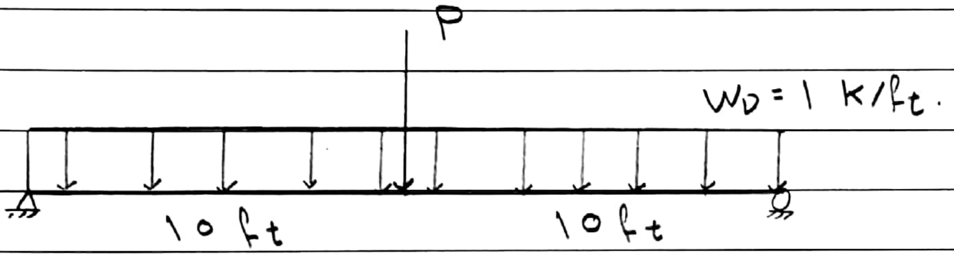
→ get $\underline{e_x}$ (from $\underline{P}$ to center of bolts) ⟶ calc (interpolation is ok).

r_n : strength of 1 bolt. $\longrightarrow r_n = F_{nv} * A_b$

R_n : " of connection $\longrightarrow \phi R_n = 0.75 * C * r_n$

Q1: (6 Pts): The beam section is W14x90, The beam is carrying a uniformly distributed service dead load of 1 k/ft and one concentrated service live load P, Steel (Fy = 50)

*Neglect self weight.

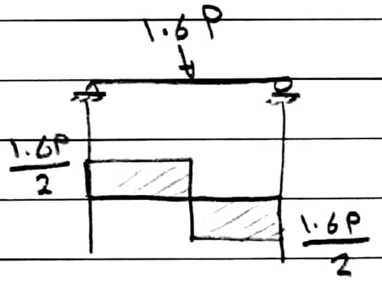


a) Calculate Max service live load P that the beam can sustain (3 Pts)

Sol: Shear strength: $\phi V_n = \phi(0.6 F_y A_w) C_v$ (W-section $\phi = 1$, $C_v = 1$).

$\phi V_n = 1(0.6 * 50 * 14 * 0.44) = 184.8$ Kips.

* Maxium Shear $= \frac{1.6P}{2}$ ⇒



$\Rightarrow \frac{1.6P}{2} = 184.8$

$\boxed{P = 231 K}$

b) Calculate maximum service live load P that the beam can sustain in deflection. (use live load only & assume $\Delta_{allow} = L/360$) (2 Pts).

Sol: $\Delta_{allow} = \frac{20*12}{360} = 0.67$

$$\Delta_{max} = \frac{PL^3}{48EI} \Rightarrow \Delta_{max} = \Delta_{allow}$$

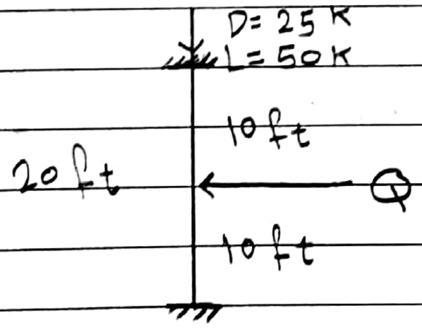
$$\frac{P(20*12)^3}{48*29{,}000*999} = 0.67 \Rightarrow \boxed{P = 67.4\ K}$$

c) Based on above, what is the max P that the beam can sustain?

Sol: the lower one $\rightarrow \boxed{P = 67.4\ K}$

Q2: (16 Pts): The steel braced member shown is W21×48 of A992 steel (F_y=50, F_u=65), supports service axial dead load of 25 kips & live of 50 kips, and a concentrated live load Q at the mid of span, bending is about the strong axis only, lateral supports in x & y are at the ends only.

* Neglect self weight of the beam in your calcs.



a) Determine the ultimate axial load P_u (1 Pt):

Sol: P_u = 1.2(25) + 1.6(50) = 110 K

b) Calculate compressive design strength ($\phi_c P_n$), Neglect the effect of local buckling in your calcs. (2 Pts)

Sol: $\phi_c P_n = \phi F_{cr} A_g$.

$$\frac{KL}{r_y} \text{ (controls)} \Rightarrow \frac{0.65(20*12)}{1.66} = 93.98 < \overset{(113.4)}{4.71\sqrt{\frac{E}{F_y}}}$$

$$F_{cr} = \left[0.658^{f_y/f_e}\right] f_y \quad , \quad f_e = \frac{\pi^2 (29{,}000)}{(93.98)^2} = 32.4 \text{ ksi}$$

$$F_{cr} = 26.21 \text{ ksi} \Rightarrow \phi_c P_n = 0.9(26.21)(14.1) = \underline{332.6 \text{ Kips}}$$

c) according to a & b, which one of the interaction formulas will be used: (1 Pt).

Sol: $\frac{P_u}{\phi_c P_n} = \frac{110}{332.6} = 0.331 > 0.2$

$$\rightarrow \text{equ } ① : \left[\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right)\right] \leq 1$$

d) is W21x48 compact section or not. (show your calculations). $\left(\frac{1}{Pt}\right)$

Sol: $\lambda_f = 9.47$, $\lambda_p = 0.38\sqrt{E/F_y} = 9.15$, $\lambda_r = \sqrt{E/F_y} = 24.08$

$\lambda_p < \lambda < \lambda_r \rightarrow$ Non-compact.

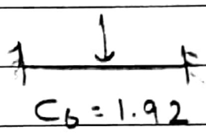
e) Calculate the Flexural design strength (ϕM_{nx}) of the beam. (4 Pts).

Sol: 1) LTB : $L_b = 20$, $L_p = 6.09$, $L_p = 16.6$

$L_b > L_r$: $\underline{\text{Zone 3}}$: $\phi M_n = \phi f_{cr} S_x / 12$

$$f_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2}\sqrt{1+0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$

$C_b = 1.92$ (from the sheet cases)



$$F_{cr} = \frac{1.92(\pi)^2(29{,}000)}{\left(20*12/2.05\right)^2}\sqrt{1+0.078*\frac{0.803*1}{93*20.2}\left(\frac{20*12}{2.05}\right)^2}$$

$F_{cr} = 48.396$ ksi $\Rightarrow \Phi M_n = 0.9(48.396)(93)/12$

$\Rightarrow \Phi M_{n_x} = 337.56$ K-ft

② FLB $\Rightarrow \Phi M_{n_x} = 398$ K-ft (ΦMp from manual)

$\Rightarrow$ use $\boxed{\Phi M_{n_x} = 337.56}$

f) Calculate the first order Maximum ultimate moment Mntx in terms of service live load Q. (1 pt)

Sol: $M_{ntx} = \frac{PL}{8} = \frac{\underbrace{1.6Q}_{\text{live load.}}(20)}{8} = 4Q$

g) Calculate the amplification factor B_{1x}? (3 pts)

Sol: $B_{1x} = \frac{C_m}{1-P_u/P_e}$, $P_e = \frac{\pi^2(29{,}000)(\overset{I_x}{959})}{(20*12)^2} = 4765.34$

from manual: $C_m = 1-0.2\left(\frac{110}{4765.34}\right) = 0.9954$

$\Rightarrow B_1 = 1.01892 > 1$ ✓.

h) Calculate the second order Maximum ultimate moment M_{ux} in terms of Q. (1Pt).

Sol: $M_{ux} = B_{1x} M_{ntx} = 1.01892 (4Q) = 4.076 Q$

i) Determine the maximum Q that can placed on the beam column. (2 Pts)

$$\frac{P_u}{\Phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{nx}} + \cancel{\frac{M_{uy}}{\phi M_{ny}}}\right) = 1$$

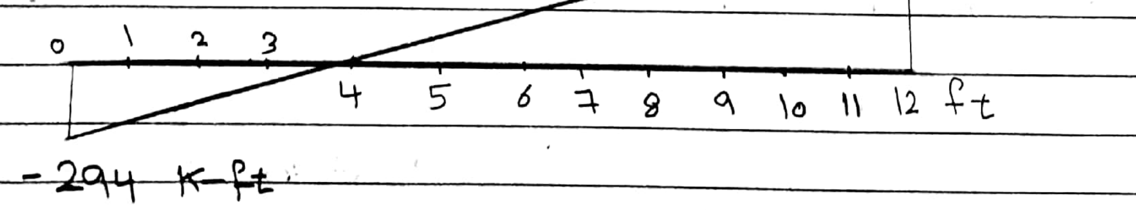
$$\frac{110}{332.6} + \frac{8}{9}\left(\frac{4.076Q}{337.56}\right) = 1 \Rightarrow \boxed{Q = 62.36 \text{ Kips}}$$

Q3: (10Pts) The moment diagram below was found from a first order analysis of beam-column in braced frame, bending is about the strong axis, bracing in x & y is the same (at ends only), $KL_x = KL_y = 12$ ft, The Moments shown are already factored, the column is also subjected to factored axial load $P_u = 89$ K, No lateral forces on the member, Determine the lightest W12 section that is adequate.

* Neglect Self weight, $F_y = 50$ ksi.

Design by

Amin Mansour



سنوات فاينل

Sol: M_x only , $KL_x = KL_y = 12$, $P_u = 89$ kip.

assume $B_{1x} = 1$, $M_{ux} = 412$ k-ft

⇒ Try W12x106 , $P = 0.834 \times 10^{-3}$, $b_x = 1.46 \times 10^{-3}$

بشكل عشوائي

$*PP_u = 0.0742 < 0.2 \Rightarrow \left[0.5PP_u + \frac{9}{8}(b_x M_{ux} + 0) \right] \leq 1$

(0.9 → 1) ولكنها بعيدة عن ⇒ $0.714 < 1$ ok

نجرب سكشن أصغر.

⇒ Try W12x96 , $P = 0.924 \times 10^{-3}$, $b_x = 1.63 \times 10^{-3}$

$PP_u = 0.0822 < 0.2 \Rightarrow [0.8377 < 1]$ ok قريبة :

$*check \; B_{1x} = \frac{C_m}{1 - \frac{P_u}{P_e}}$, $P_e = \frac{\pi^2 (29,000)(833)}{(12 * 12)^2} = 11497.88$

$C_m = 0.6 - 0.4 \left(\frac{294}{412} \right) = 0.31456$

عكس الاتجاهات

النسبة موجبة

$\Rightarrow B_{1x} = 0.317 < 1$ X → assume is ok , $B_{1x} = 1$

$* L_b = 12$, $L_p = 10.9$, $L_r = 46.6$ { $\Phi M_p = 551$

Zone 2 → calc C_b

* من الرسمة المعطاة وتشابه المثلثات أوجد M_A, M_B, M_C وجد قيمة C_b

$\Rightarrow C_b = 2.215$

$C_b \phi M_{nx} = 2.215 * \frac{8}{9}\left(\frac{1}{1.63 * 10^{-3}}\right) = 1207.9 > \underset{\phi M_p}{551}$

→ New $\phi M_n = 551$ k-ft

→ New b_x ⇒ $b_x = \frac{8}{9}\left(\frac{1}{551}\right) = 1.6132 * 10^{-3}$

⇒ Check Amin.M equ again ⇒ $[0.747 < 1]$

safe but not economy.

⇒ Try W12x87, $P = 1.02 * 10^{-3}$, $b_x = 1.82 * 10^{-3}$

$PP_u < 0.2$ ⇒ $[0.889 < 1]$.

كمل نفس الخطوات

* $D_{1x} = 1$ (assume is ok)

$L_b = 12$, $L_p = 10.8$, $L_r = 43$, $\{ \phi M_p = 488$

Zone 2: , $C_b = 2.215$,

$C_b \phi M_{nx} = C_b * \frac{8}{9} * \frac{1}{b_x} = 1081.8 > 488$ ⇒ $\underline{\phi M_n = 488}$

→ New $b_x = \frac{8}{9}\left(\frac{1}{488}\right) = 1.821 * 10^{-3}$

⇒ Check equ: $[0.889 < 1]$

Try W12x79 → $P = 1.13 * 10^{-3}$, $b_x = 2.02 * 10^{-3}$

$PP_u < 0.2$ ⇒ $[0.987 < 1]$

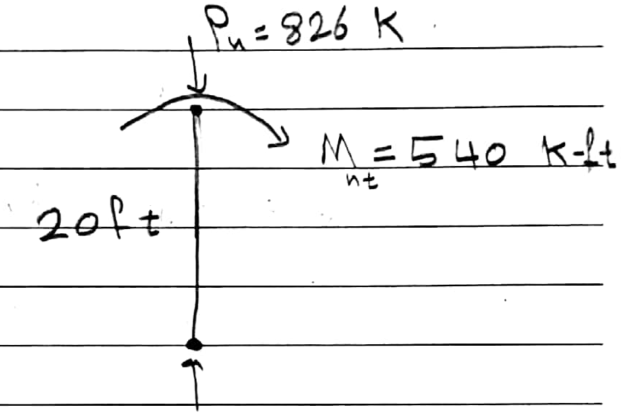
* كمل...

بالأخير ⇒ $[0.974 < 1]$ ✓ safe & economy ✓

Use W12x79

سنوات فاينل

Q4: use $F_y = 50$ & select the lightest W14 Shape for
(9 Pts) the beam column Shown using Amin Mansour method, The member is part of a braced frame,
* The axial load & bending Moment given are factored.
* Bending is about the strong axis.
* $K_x = K_y = 1$.



* هذا السؤال أجا على دوري وكانه معطى رقم السكشن الذي نبدأ به

Sol: $f_y = 50$, $KL_y = 20$.

assume $B_{1x} = 1$, $M_{ux} = 540$

Try W14x145, $P = 0.679 * 10^{-3}$, $b_x = 0.956 * 10^{-3}$.

$PP_u = 0.5609 > 0.2 \rightarrow (PP_u + b_x M_{ux} + 0) < 1$

$(1.077 > 1)$ Not safe

* نأخذ سكشن اكبر

Try W14X159, $P = 0.619 * 10^{-3}$, $b_x = 0.863 * 10^{-3}$.

$PP_u > 0.2 \longrightarrow [0.9773 < 1]$

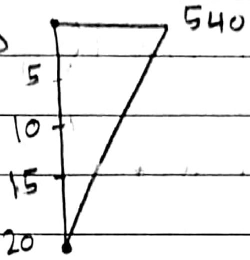
$\Rightarrow B_{1x} = \frac{C_m}{1 - P_u/P_e}$, $P_e = \frac{\pi^2 (29,000)(1900)}{(20*12)^2} = 9441.24$

$C_m = 0.6 - 0.4\left(\frac{0}{(540)}\right) = 0.6 \rightarrow B_{1x} = 0.658$ ✗

$B_{1x} = 1$ ok.

* $L_b = 20$, $L_p = 14.1$, $L_r = 66.7$, } $\phi M_p = 1080$

Zone 2: Calc C_b ⇒



⇒ $C_b = 1.667$

$$C_b \phi M_{nx} = 1.667 * \frac{8}{9} * \frac{1}{0.863 * 10^{-3}} = 1717 > \boxed{1080}$$

⇒ $\phi M_{nx} = 1080$ k-ft.

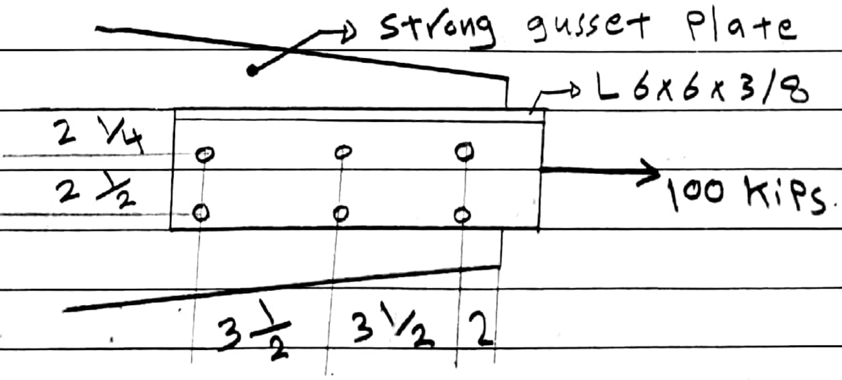
⇒ New $b_x = \frac{8}{9}\left(\frac{1}{1080}\right) = 0.823 * 10^{-3}$

⇒ check eqn ⇒ $[0.956 < 1]$

safe & most economy ✓.

$\boxed{\text{Use W14 x159}}$

Q5 (11 Pts): an angle shape L6x6x3/8, is connected to a 1/2 inch gusset plate with 3/4in diameter A325 bolts with threads not in plane of shear, Steel A572 grade50 ($F_y = 50$, $F_u = 65$) used for both angle & gusset, The connection is bearing type.



a) Determine the shear strength of the connection. (1Pt)

Sol: $\phi R_n = 0.75 F_{nv} A_b = 0.75(60)\left(\frac{\pi}{4}(3/4)^2\right) = 19.88$ K

⇒ for the connection: $\phi R_{n\,tot} = 19.88(6) = 119.28$ K

b) Determine the critical bearing strength for the connected parts. (4 Pts)

Sol: only for the Angle .. (least $t = 3/8$).

$R_n = 1.2 L_c t F_u \leq 2.4 d t F_u$

edge: $L_c = L_e - \frac{h}{2} = 2 - \frac{(3/4 + 1/16)}{2} = 1.5938$

$R_n = \left[46.62 \leq \boxed{43.875}\right] \rightarrow \phi R_{n_e} = 32.91$ K

other: $L_c = 3.5 - (3/4 + 1/16) = 2.6875$

$R_n = \left[78.61 \leq \boxed{43.875}\right] \rightarrow \phi R_{n_o} = 32.91$ K

⇒ Total: $\phi R_n = 2(32.91) + 4(32.91) = 197.46$ K.

c) Determine yielding strength of tension member (1Pt).

$\phi R_n = 0.9 f_y A_g = 0.9(50)(4.38) = 197.1$ K

d) determine the fracture strength for the tension member. (3 Pts)

Sol: $\phi R_n = 0.75 F_u A_e$

$A_n = A_g - A_{holes}$

$= 4.38 - 2(3/8)(3/4 + 1/8) = 3.723$

$U \rightarrow$ from case ②: $1 - \bar{X}/L = 1 - \frac{1.62}{7} = 0.768$

case ⑧: $U = 0.6$

$\Rightarrow$ take $U = 0.768$ $\Rightarrow$ $\phi R_n = 0.75\, F_u (U A_n)$.

$\phi R_n = 139.4$ kips.

e) based on above, the capacity strength of the connection is ? (1 Pt).

Sol: the lowest value $\Rightarrow$ $\phi R_n = 119.28$ K

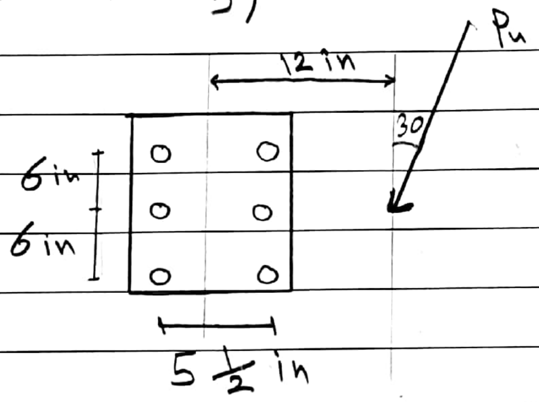
f) Based on above, is the connection adequate ?

$\phi R_n = 119.28 > P_u$ (100k).

✓ adequate.

Q6: Determine the nominal strength of bearing type connection shown, using the ultimate strength method (Tables), bolts are A325 with threads in plane of shear with 1/2 in diameter (the shear is controlling).

(3 Pts)



Sol: 1) two vertical lines
2) angle = 30°
3) spacing = 5 1/2 in
→ Page 7-46.

$e_x = 12$
No of bolts = 3
Per line
$S = 3$
→ $C = 1.76$

$r_n = F_{nv} A_b = 48 \left(\frac{\pi}{4} (1/2)^2\right) = 9.425 K$ (1 bolt).

$\phi R_n = 0.75 * C * r_n$

$$\boxed{\phi R_n = 12.44 \text{ Kip}}$$

♡ ** The End ** ♡