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جزوه درس:

فولاد ۲

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با استفاده از جزوات اسکن شده، به محیط زیست کمک کنیم...

هر آنچه که در این جزوه می خوانید حاصل زحمات دانشجویان دانشگاه صنعتی شریف می‌باشد که دانسته‌های خود از حضور در کلاس استاد محترم، دکتر احمدی‌زاده را مکتوب کرده اند.

استفاده از این جزوات برای تمامی دانشجویان کاملا رایگان می‌باشد.

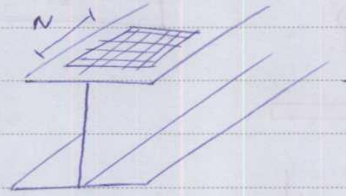
«بسمه تعالی»

Subject: دکتر احمدی زاده

Year. Month. Date. ( )

کهنم شرافت. (فولاد 2) 1

webs and flanges with Concentrated Loads:

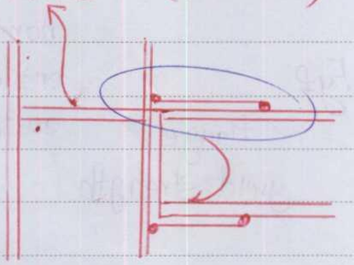


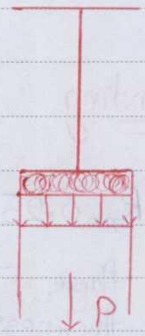
→ have web and flanges with sufficient strength for the concentrated loads

Possible modes of failure:

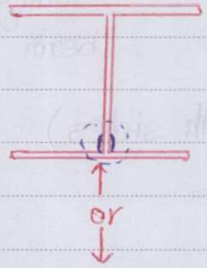
- Local flange Bending

Stiffener (سخت کننده)



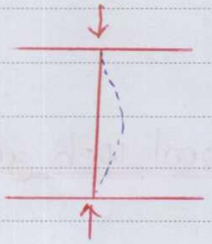


- Local web yielding

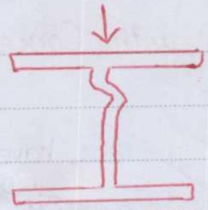


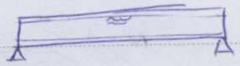
- Compression buckling of web

کمانش فشاری جان

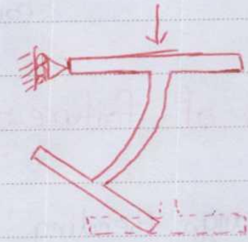


- Web Crippling
  کوفتگی جان





- Sidesway web buckling
  کمانش جانبی جان



- Local Flange Bending

بارکششی الی روی بال

nominal tensile load on the flange

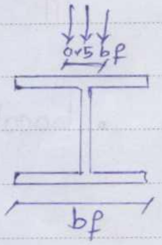
$$R_n = 6.25\, t_f^2\, F_{yf}$$

t_f flange thickness

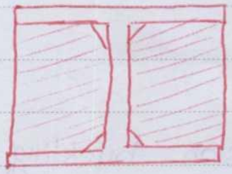
F_{yf} ~ flange yield strength

$\phi = 0.9$ ضریب کاهش مقاومت

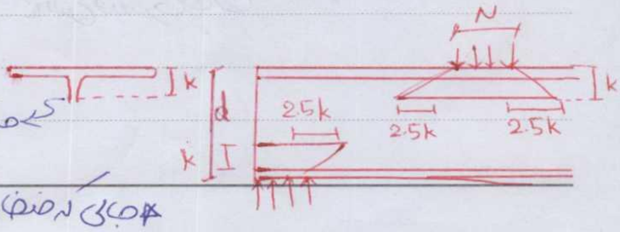
* no need to check if width of loading across beam flange $\leq 0.15\, b_f$



* Stiffeners (both sides)



- Local web yielding



حداقل ضخامت

* جایی که ضخامت جان به حداقل می رسد.

Concentrated Load @ distance greater than d or h from member ends

سطح مقطع بار (N)
ضخامت × طول

$$\underbrace{R_n=(N+5k)}\, t_w F_{y_w}$$
$\varphi=1.0$

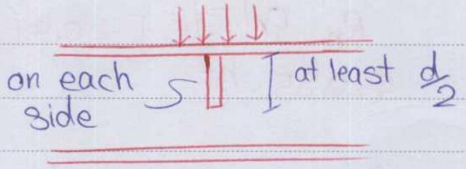
↓ web yield strength

Load @ d or less from member end

web thickness

$$R_n=(N+2.5k)\, t_w F_{y_w}$$
$\varphi=1.0$

* no need to check if



- **Web Crippling** ① Concentrated load at a distance not less than $\frac{d}{2}$ from ends ($\geq \frac{d}{2}$)

$$R_n=0.80\, t_w^2\left[1+3\frac{N}{d}\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E f_y t_f}{t_w}}$$
$\varphi=0.75$

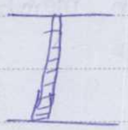
② Concentrated at a distance less than $\frac{d}{2}$ from ends ($<\frac{d}{2}$)

$$\frac{N}{d}\leq 0.2 \longrightarrow R_n=0.4\, t_w^2\left[1+3\frac{N}{d}\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E f_y t_f}{t_w}}$$
$\varphi=0.75$

$$\frac{N}{d}>0.2 \longrightarrow R_n=0.4\, t_w^2\left[1+\left(4\frac{N}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E f_y t_f}{t_w}}$$
$\varphi=0.75$

* no need to check if stiffners (one or two)

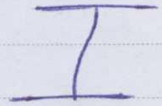
with height $\geq \frac{d}{2}$ are used.



* ~ ~ ~ ~

_ Sidesway Web buckling

* if loaded flange is braced against rotation and $(h/t_w)/(l/b_f) \leq 2.3$ Ⓘ



$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1+0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right]$$

$\varphi = 0.85$

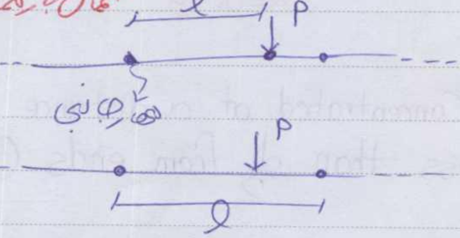
* if loaded flange is not braced against rotation: and $(h/t_w)/(l/b_f) \leq 1.7$ ⓘⓘ

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right]$$

$\varphi = 0.85$

l: بزرگترین طول بدون مهار جانبی هر دو بال در محدوده اعمال بار متمرکز

lm بیشترین فاصله نقطه بارگذاری تا اولین مهار جانبی



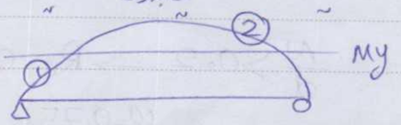
از تحلیل بدست می آید

C_r: 6.62×10^7 kg/cm² if $M_u < M_y$ at the location of force ①

$M_y \to E_y S \to$ مدول مقطع (ISHTAL جدول)

3.31×10^7 kg/cm² if $M_u \geq M_y$ ②

* $h = d - 2k$



* no need to check if:

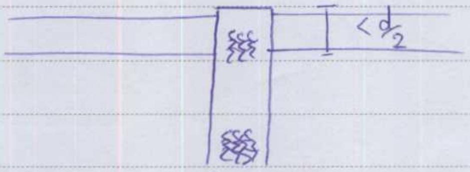
- stiffeners with at least $\frac{d}{2}$ depth (designed for entire load) exist
- relations Ⓘ or Ⓘⅈ not satisfied

- Compression buckling of web

$$R_n = \frac{24 t_w^3 \sqrt{E F y_w}}{h}$$

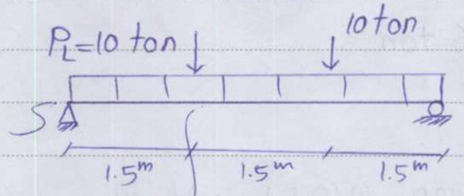
$\varphi = 0.9$

* Reduce R_n by 50% if the concentrated force are applied at less than $\frac{d}{2}$ from member end ($\frac{d}{2}$)



* if a pair of stiffeners extending full web depth are used, there is no need to check.

Example: Select an IPE section



end bearing length : 9 cm

load bearing length : 7.5 cm

مهار جانبی در تکیه‌گاه‌ها و زیر بارها

Check: web yielding
web crippling
Sidesway web buckling

let beam weight : 60 $\frac{kg}{m}$

$M_u = 1.2 \times \frac{1260 \times 4.5^2}{8} + 1.6 \times 10000 \times 1.5 = 27827$ kg.m

$Z = \frac{2782700}{0.9 \times 2333} = 1325 \longrightarrow$ try IPE 450

$L_p = 1.76(4.12)\sqrt{\frac{2 \times 10^6}{2333}} = 212$ cm $\longrightarrow$ zone 1

$Z = 1698\ cm^3$ $d = 45cm$
$b_f = 19cm$ $t_w = 0.94cm$
$t_f = 1.46\ cm$
$k = 3.55cm$ $r_y = 4.12cm$
$G = 77.6\ kg/m$

End Reactions:

$R_u = 1.2(1278 \times 4.5)/_2 + 1.6 \times 1000 = 19.45 ton$

Concentrated

$P_u = 1.6 \times 10 = 16$ ton

Local web yielding

end reactions : $\varphi R_n = \varphi(2.5k + N) t_w F_{y_w}$

$= 1.0 \times (2.5 \times 3.55 + 9) 2333 \times 0.94 = 39.2 \overset{19.45}{>}$ O.K.

concentrated Load O.K.

web Crippling

end reactions: $\frac{N}{d}=\frac{9.0}{45}=0.2$

$$\varphi R_n=\varphi\times0.4t_w^2\left[1+3\frac{N}{d}\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

O.K.

$$=0.75\times0.4\times0.94^2\left[1+3\times0.2\times\left(\frac{0.94}{1.46}\right)^{1.5}\right]\sqrt{\frac{2\times10^6\times2333\times1.46}{0.94}}=29.56\text{ ton}$$

Concentrated load:

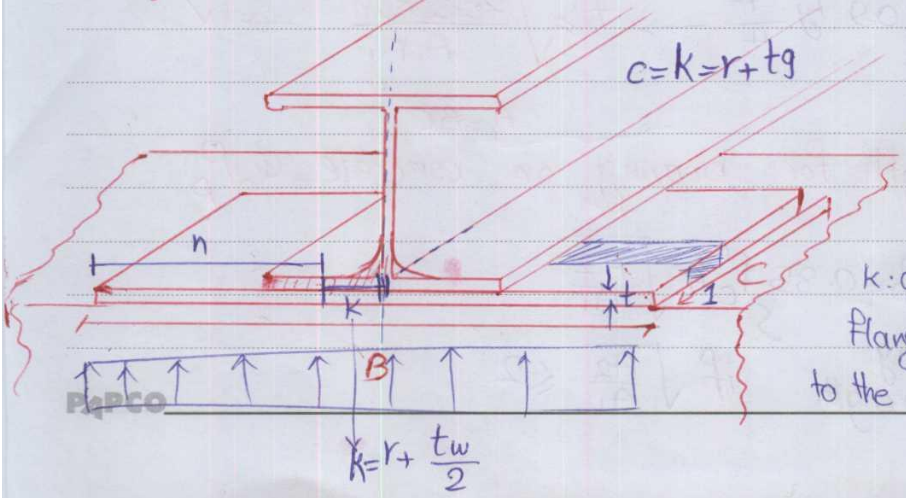
$$\varphi R_n=\varphi\times0.8t_w^2\left[1+3\frac{\overset{7.5^{cm}}{N}}{d}\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\quad}=56.79\text{ ton}\quad O.K.$$

Sidesway web buckling

$$\frac{(h/t_w)}{(l/t_f)}=\frac{\frac{45-2\times3.55}{0.94}}{\frac{150}{19}}=5.11>2.3$$ sidesway web buckling does not occur

$\boxed{\text{USE IPE 450}}$

Design of Beam Bearing Plates: طراحی ورق های زیرسری برای تیرها



A= area of the plate

k: distance from flange outer face to the toe of the filler

Assumptions:

- Uniform spread of reaction through the bearing plate
- Uniform pressure under the plate

LRFD: ورق نشیمن کل ممان را می‌گیرد و هیچی به بال نمی‌رسد.

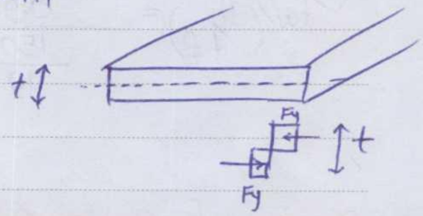
- bearing plate takes the entire moment
- Critical section for moment is at $\underline{k}$ from beam centerLine

- thickness

unit length plate (along C)

? Reaction: $R_u \longrightarrow M_u = \frac{R_u}{A_1} \times 1 \times n \times \frac{n}{2} = R_u \frac{n^2}{2A_1}$

thickness t: $Z = (1) \times \frac{t}{2} \times \frac{t}{4} \times 2 = \frac{t^2}{4}$



$M_n = F_y Z \longrightarrow \varphi_b M_n = \varphi_b F_y Z \quad \varphi_b = 0.9$

$\longrightarrow \frac{R_u n^2}{2A_1} \leq 0.9\, F_y \frac{t^2}{4} \longrightarrow t \geq \sqrt{\frac{2.22 R_u n^2}{A_1 F_y}} \longrightarrow t \checkmark$

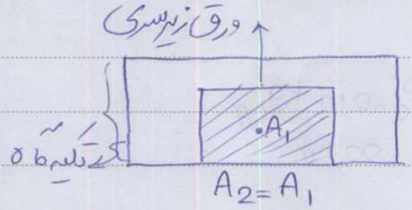
$A_1 = B \times C$

Design strength for bearing on concrete $= \varphi_c P_p$

$\varphi_c = 0.6 \quad P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}$

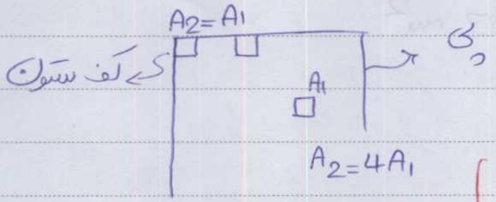
28 day concrete strength

if $\sqrt{\frac{A_2}{A_1}} \leq 2$



A_2 : بزرگترین مستطیلی که هم مرکز مستطیل A_1 و
و نسبت طول و عرض مشابه به A_1 طوری
که شامل تکیه‌گاه باشد

A_2: maximum area of supporting surface that is geometrically similar to and concentric with the loaded area



$$\text{Design} \quad R_u \leq \varphi_c P_p = \begin{cases} \varphi_c 0.85 f'_c A_1 \sqrt{\dfrac{A2}{A_1}} & A_2 \leq 4A_1 \\ \varphi_c 1.7 f'_c A_1 & A_2 > 4A_1 \end{cases}$$

$\overset{0.6}{\uparrow}$

$$\longrightarrow A_1 \geq \begin{cases} \dfrac{1}{A2}\left(\dfrac{Ru}{\varphi_c 0.85 f'_c}\right)^2 & A_2 \leq 4A_1 \\ \dfrac{Ru}{\varphi_c 1.7 f'_c} & A_2 > 4A_1 \end{cases}$$

قرار است $A_1 \longrightarrow B, C$ ✓ بشروط زیر $C \geq \begin{cases} N \text{ to prevent web yielding or crippling} \\ 10 \text{ cm (Practical)} \end{cases}$

$C \leq$ support thickness

Example: Beam : INP 40

reinforced concrete $f'_c = 210\ \frac{kg}{cm^2}$

thickness 20 cm

$R_u = 50$ ton

_ Plate Area (let $A_1 = A_2$)

$$A_1 = \frac{R_u}{\varphi_c 0.85 f'_c} = \frac{50000}{0.6 \times 0.85 \times 210} = 467\ cm^2$$

_ Plate dimensions

* to prevent local web yielding

$$R_u \leqslant \varphi (N + 2.5k) t_w F_{yw} \longrightarrow N \geqslant 5.26\ cm$$

* to prevent web crippling

$$R_u \leqslant \varphi\, 0.4\, t_w^2 \left[1 + \left(4\frac{N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right] \sqrt{\frac{E F_{yw} t_f}{t_w}} \longrightarrow N \geqslant 0$$

$\downarrow$

web Crippling does not occur

take $C = 20 \longrightarrow$ PL 20×24 cm ($A_1 = 480\ cm^2$)

$$k_{INP40} = 3.85 \longrightarrow n = \frac{24}{2} - 3.85 = 8.15\ cm$$

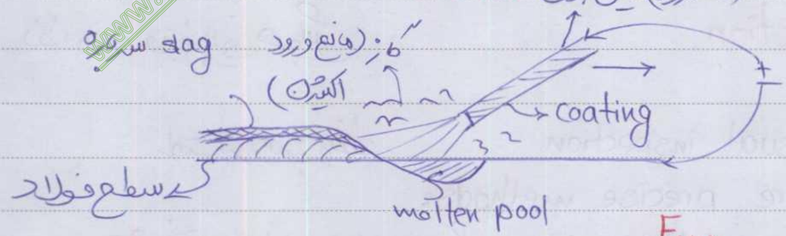
$$t \geqslant \sqrt{\frac{2.22 \times 50000 \times 8.15^2}{480 \times 2333}} = 2.57\ cm$$

$\boxed{\text{Use PL: } 20 \times 24 \times 2.5\ cm}$

Design of welded connections: طراحی اتصالات جوشی

Heat Source → Flow of Base Metal → Joint

به تمام این روش های زیر (Prequalified)



* Shielded Metal Arc welding (SMAW)

E60 xx ↑ F_{uw} (ksi)

	F_{uw}	F_y base metal
E60	4440 kg/cm²	≤ 2950
E70	4830 "	≤ 3860
E80	5520 "	≤ 4570

* Submerged Arc Welding (SAW) جوش توسط قوس الکتریکی غوطه ور

به جای این که coating داشته باشیم یک لایه نازک پودر روی سطح می ریزیم و الکترود را داخل آن قرار می دهیم و همان کار coating رو می کند.

* Gas Metal Arc Welding (GMAW)

خودمان گاز را به محل جوش تزریق می کنیم.

* Flux-cored Arc welding (FCAW) مثل همان پوشش و فولاد است ولی جای پوشش و فولاد را عوض کنیم

Structural Steel Weldibility جوش پذیری فولاد ساختمانی

Carbon 0.06 ~ 0.25 %

Welding Inspection نظارت و بازبینی جوش

- Visual inspection راحت تر والبته بصرفه
- More precise methods:
  - * Ultrasonic تشخیص سوراخ، تخلخل در جوش
  - * Radigraphic (X & γ ray)
  - * Magnetic Particles

Weld Classifications:

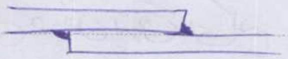
Weld Joints:

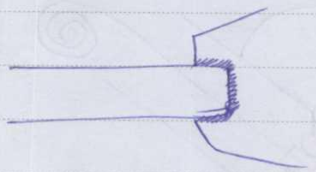
جوش لب بلب Butt Weld 

از لحاظ ظاهر قشنگتر ولی باید درز را آماده کنیم مثلاً فارسی بر کنیم

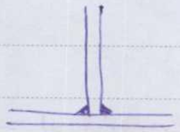
Lap Joints.

جوش روی هم



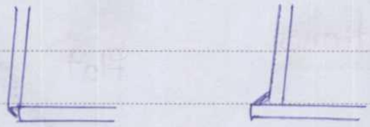


Tee Joints:



Subject:
Year. Month. Date. ( )

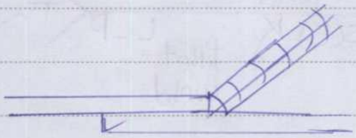
Corner Joint:
درز کنج



Edge Joint: (nonstructural components)
درز

Weld Types:

جوش گوشه Fillet weld



جوش لب Groove weld

square

ورق نگهدارنده

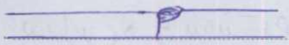
double vee

(جناغی) Single vee

(bevel) نیم جناغی

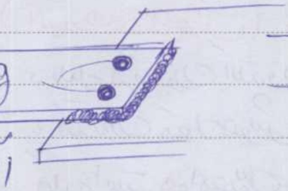
Full penetration
نفوذ کامل

U (لاله ای)



Partial Penetration
نفوذ ناقص

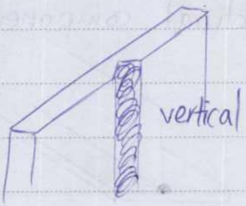
Plug and Slot weld

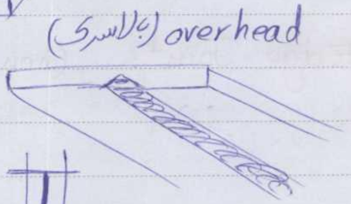


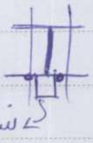
انگشتانه

Slot weld
کام

Welding Positions:

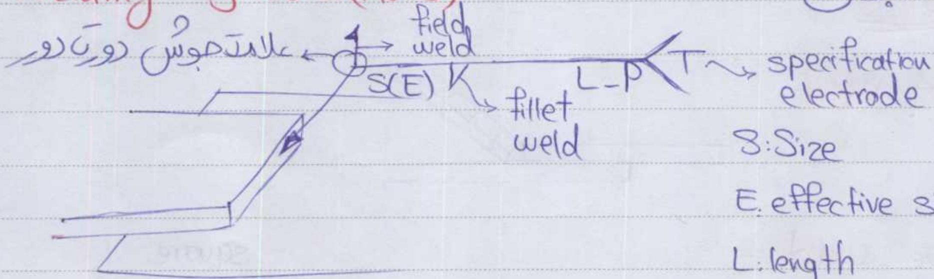






Welding Symbols: (AWS)

نمادهای جوش



S: Size

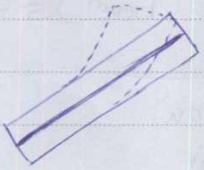
E. effective size

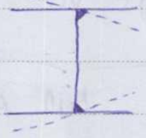
L: length

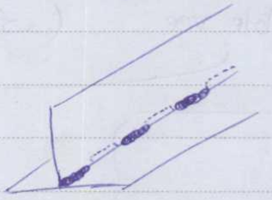
p: pitch

Warping Due to Welding:

جوش بعد از خنک شدن دچار انقباض می‌شود





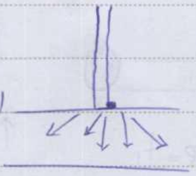


بنابراین هر 5 سانت بصورت منقطع جوش می‌دهند.

Welding Defects: (آسیب‌های جوش)

به دلیل بزرگ بودن الکترود
یا تنگ بودن محل جوش
ممکن است هوا محبوس
شود

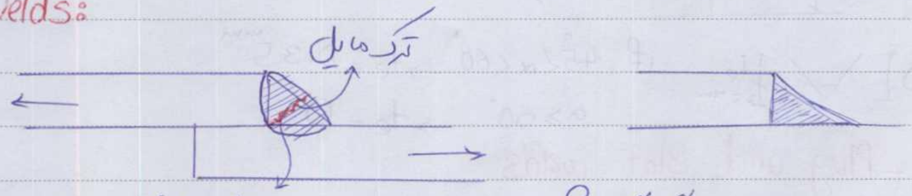
welding Size limitations:

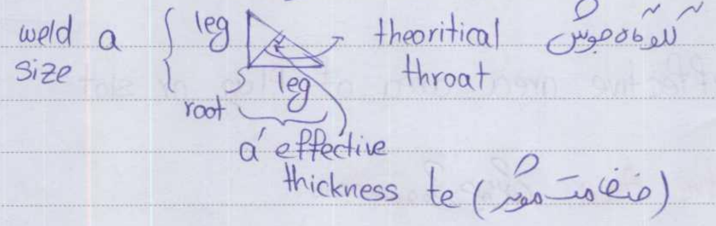


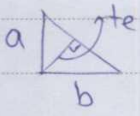
اینجا خوب جوش نمی خورد
بنابراین محدودیت حداقل دارد.

باعث سوراخ شدن ورق می‌شود
بنابراین محدودیت حداکثر دارد

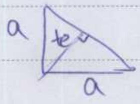
Fillet welds:



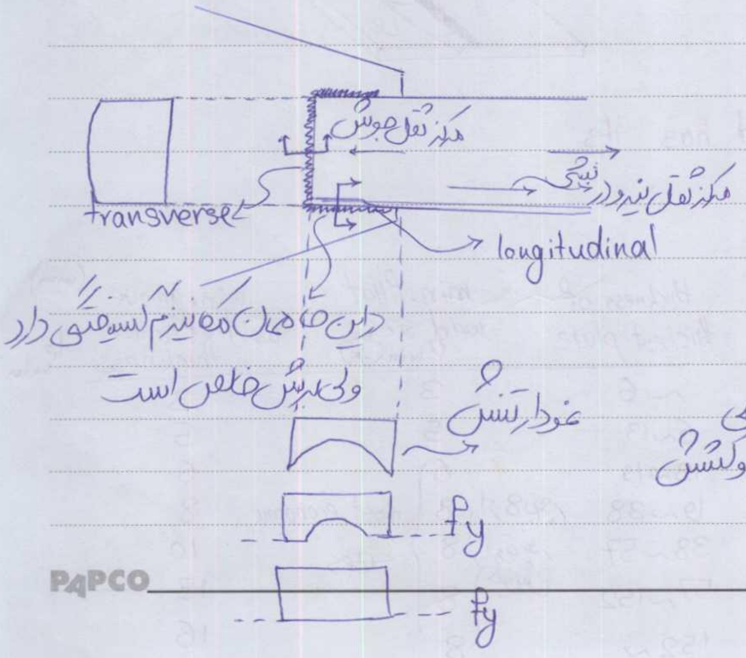




$$t_e = \frac{ab}{\sqrt{a^2+b^2}}$$



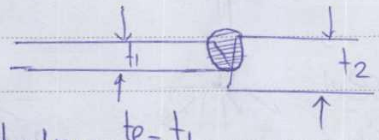
$$t_e = \frac{a}{\sqrt{2}} = 0.707\,a$$



* معمولا جوش عرضی مقاومت بیشتری نسبت به طولی دارند.

* در جوش عرضی سطح گسیختگی به دلیل وارد شدن همزمان برش و کشش زاویه گسیختگی 45° است.

_Groove welds



full penetration

$t_e = t_1$

Partial Penetration

if $45^\circ < \alpha < 60^\circ \rightarrow t_e = D - 3.5^{mm}$

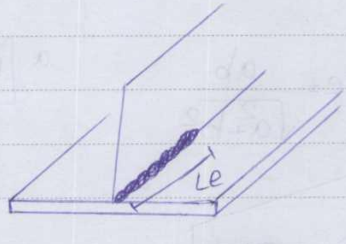
$\alpha \geqslant 60^\circ \longrightarrow t_e = D$

_Plug and Slot welds

: effective area = area of Plug or slot

Weld effective Area مساحت موثر جوش

_Fillet & Groove welds



$A_e = t_e \cdot l_e$

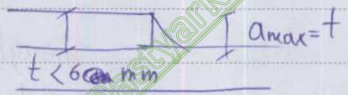
↳ the length along the weld has its full size

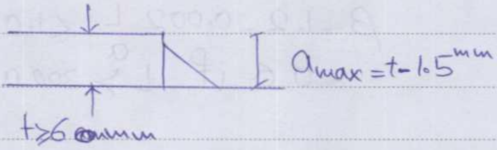
Weld size limitations:

⊛ does not need to be greater than the thickness of the thiner plate

thickness of thickest plate		min. fillet weld size a_{min} ⊛		min. groove weld effective thickness $t_{e\,min}$ (mm)
~6		3		3
6~13		5		5
13~19		6	most economic sizes	6
19~38	از 8 بیشتر (روی فلز جوش)	8		8
38~57		8		10
57~152		8		13
152~		8		16

Max fillet weld size:





Minimum fillet weld length L_e

اگر مجبور شدیم

$L_{e_{min}}=4a\geq 4$ cm otherwise : $L_e<4a\longrightarrow$ take $a=\frac{L_e}{4}$

Effect of Welding Process:

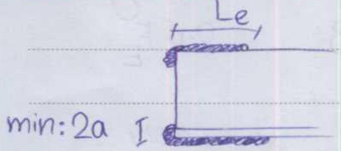
اثر پروسه جوشکاری

SMAW	SAW
$t_e=0.707a$	if $a\leq 1cm\longrightarrow t_e=a$ $a>1cm\longrightarrow t_e=0.707a+2.75^{mm}$

2005 → tests required

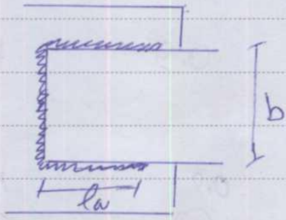
_Minimum fillet weld end returns

(min) برگشت جوش



Minimum fillet weld length (flat bar & plates)

ورق ها و تسمه ها



$l_w>b$

_minimum overlap in lap joints

overlap ≥ max $\begin{cases}2.5^{cm}\\ 5\times \text{ thickness of thinnest plate}\end{cases}$

روی هم قرار گیری

$$\boxed{\overset{0.75}{\varphi} \times F_w \times t_e \times l}$$:مقاومت جوش

_ if in fillet welds

$L > 100a \longrightarrow L_e = \beta L$

$\beta = 1.2 - 0.002 \frac{L}{a} \leq 1.0$

$= 0.6$ if $L > 300a$

F_E: strength of weld metal

F_y: Yield stress of BM

LRFD requirements:

	F_E	$F_{y\,max}$ $\frac{kg}{cm^2}$
E60	4140	2950
E70	4830	3860
E80	5520	4570

$\varphi F_E (\varphi F_w)$ Electrode

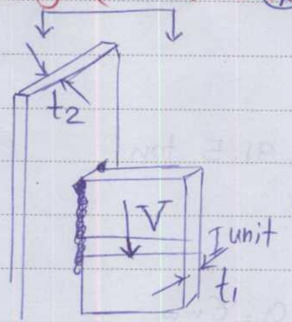
φF_{BM} Base Metal

$\boxed{J2.5}$

Type of weld and stress	Material	Risistance Factor φ	Nominal Strength F_{BM} or F_w
* Groove welds (Complete penetration)			
در امتداد محور جوش عمود بر صفحه موثر _ T or C normal to eff. area or along axis	Base	0.9	F_y
برش _ S on eff. area	Base	0.9	$0.6F_y$
	weld	0.8	$0.6F_E$
* Fillet welds			
برش _ S on effective area	Base	1.0 0.75 ⊛	$0.6F_y$ $0.6F_u$ ⊛
	Weld	0.75	$0.6F_E$
_ T or C parallel to weld axis	Base	0.9	F_y
* Plug or Slot welds	Base	0.9 ⊛	F_y ⊛
سطح مورد اتصال _ S parralel to faying surface	weld	0.75	$0.6F_E$

LRFD Design (Fillet welds)

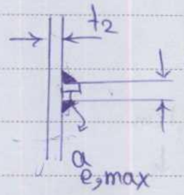
باید مقاومت جوش حداقل برابر مقاومت BM باشد.



$$\varphi R_n = \begin{cases} 1.0\,(0.6F_y)\,A_{BM} & \text{yielding} \\ 0.75\,(0.6F_u)\,A_{BM} & \text{rupture} \end{cases}$$

Area

مقاومت اسمی BM (کششی)



مینیمم اندازه موثر ($a_{min,eff}$) جوش گوشه

minimum effective fillet weld size:

برای طول واحد

$$t_1: \begin{cases} 1.0\times 0.6\times F_y \times t_1 = 0.75\times 0.6F_E\times 0.707\,a\times 2 \\ 0.75\times 0.6\times F_u \times t_1 = 0.75\times 0.6F_E\times 0.707\,a\times 2 \end{cases}$$

$$\rightarrow a_{max,eff} = \begin{cases} 0.943\,\dfrac{F_y t_1}{F_E} \\ 0.707\,\dfrac{F_u t_1}{F_E} \end{cases}$$

* ارزش جوش: مقاومت 1cm از جوش

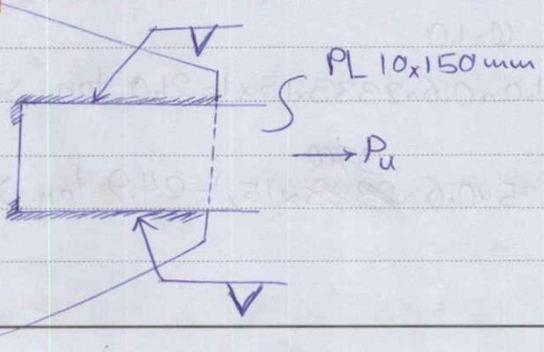
$$t_2: \Big\{ \qquad \rightarrow a_{max,eff} = \begin{cases} 1.89\,F_y\,\dfrac{t_2}{F_E} \\ 1.41\,F_u\,\dfrac{t_2}{F_E} \end{cases}$$

Design of simple fillet welds:

طراحی جوش های گوشه ساده

Example:

Electrode: E70



Design for full capacity of flat bar:

Solution:

فرض ورق $P_u = \varphi F_y A_g = 0.9 \times 2333 \times (1 \times 15) = 31.5$ ton

تسمه ↑

$t = 10^{mm}$ $\begin{cases} a_{max} = 10 - 1.5 = 8.5 \text{ mm} \\ a_{min} = 5^{mm} \end{cases}$ take $a = 6$ mm

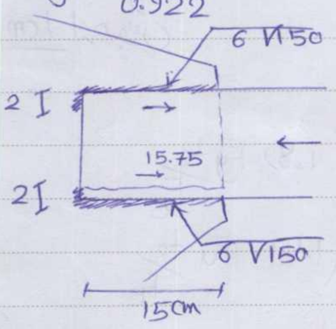
ضخامت مؤثر ↑

$t_e = \frac{a}{\sqrt{2}} = \frac{0.707a}{1} = 4.24$ mm

از برش جوش

Capacity per cm weld $= R_w = \varphi F_w = 0.75 \times 0.6 \times \overset{E70}{4830} \times \overset{t_e}{0.42} \times \overset{1cm}{1.0} = 922$ kg

Required effective length: $\frac{\overset{P_u}{31.5}}{0.922} = 34.2$ cm



provided = 34 cm ≃ 34.2 ✓

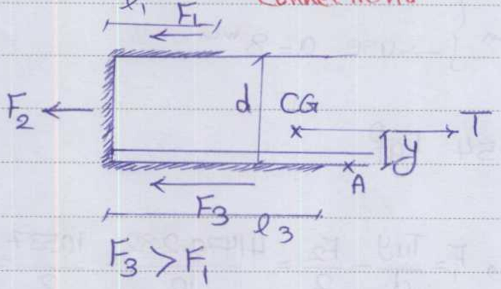
Checking Plate: $\varphi = 1.0$ O.K.

$1.0 \times 0.6 \times 2333 \times 15 \times 1 = 21.0$ ton > 15.75 ton ✓

O.K.

$0.75 \times 0.6 \times \overset{3700}{\cancel{2333}} \times 15 \times 1 = 24.9$ ton > 15.75 ton ✓

Reduction of Eccentricity in Truss Connection: کاهش خروج از مرکزیت در اتصالات خرپایی



$F_3 > F_1$

$F_2 = R_w \times d \rightarrow F_2$ ✓
$F_1 = l_1 \times R_w$
$F_3 = l_3 \times R_w$

$\overset{+}{\circlearrowleft}\Sigma M_A = 0 \quad -Ty + F_1 d + F_2 \frac{d}{2} = 0$

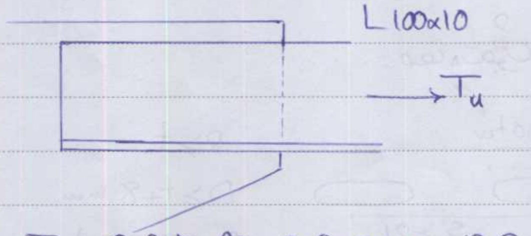
$\rightarrow F_1 = \frac{Ty}{d} - \frac{F_2}{2} \rightarrow F_1$ ✓ $\rightarrow l_1$ ✓

$\Sigma F_x = 0 \rightarrow F_3 = T - F_1 - F_2 \rightarrow F_3$ ✓ $\rightarrow l_3$ ✓

$R_w = 0.75 \times 0.6 F_E \times 0.707 a \times 1 = \begin{cases} 1317\,a & E60 \\ 1537\,a & E70 \\ 1756\,a & E80 \end{cases}$

مقاومت طراحی
1cm از جوش

Example:



E60 , $F_y = 2400$
$F_u = 3700$ $\phi = 0.9$

L 100×10 : $T_u = 0.9 F_y A_g = 0.9 \times 2400 \times 19.2 = 41.47$ ton ✓ It controls
$= 0.75 F_u A_e = 0.75 \times 3700 \times 0.9 \times 19.2 = 47.95$ ton

$$\left.\begin{array}{l} y=2.82\,cm \\ d=10\ cm \\ t=10\ mm \end{array}\right\} \left.\begin{array}{l} a_{min}=5^{mm} \\ a_{max}=8.5^{mm} \end{array}\right\} \rightarrow use\ \ a=8^{mm}$$

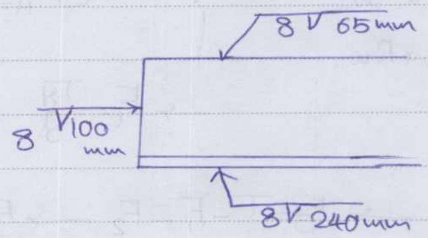
$R_w=1317\,a=1317\times0.8=1054\ kgf$

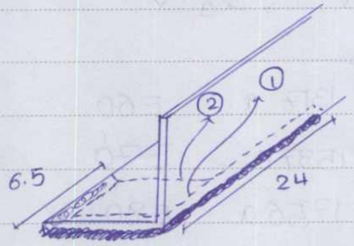
$F_2=1054\times10=10537\ kg$ ✓ , $F_1=\frac{T_u y}{d}-\frac{F_2}{2}=\frac{41470\times2.82}{10}-\frac{10537}{2}\Rightarrow F_1=6426\ kgf$ ✓

$F_3=T_u-F_1-F_2=41470-10537-6426=24507\ kgf$ ✓

$l_1=\frac{F_1}{R_w}=\frac{6426}{1054}=6.1\ cm$

$l_3=\frac{F_3}{R_w}=\frac{24507}{1054}=24.0\ cm$





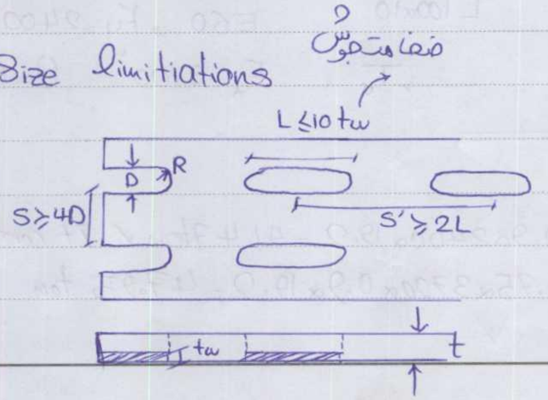
① , ② : مکانیزم های گسیختگی

البته حالت ② نیروی بیشتری می‌برد.

Plug & slot welds:

طراحی جوش های انگشتانه و کام

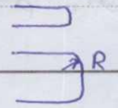
* Size limitations



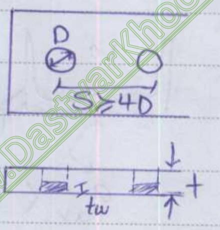
$R\geq t$

$D\geq t+8\ mm$

$D\leq2.25\ t_w$



Subject:
Year. Month. Date. ( )

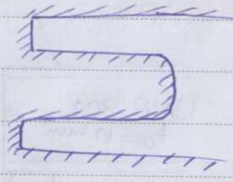


$D \geq t + 8$ mm

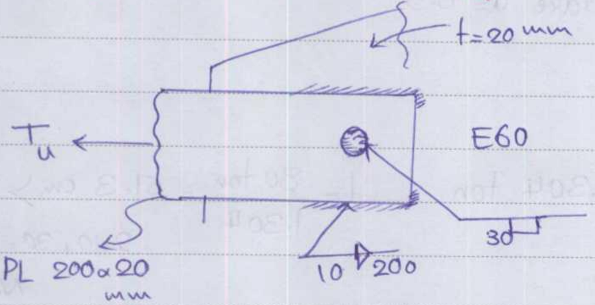
$D \leq 2.25\, t_w$

if $t < 15$ mm $t_w = t$

$t \geq 15$ mm $t_w = \frac{t}{2} \geq 15$ mm



Example: Ultimate force T_u



Fillet welds:

بالا و پایین

$T_1 = R_w L = 1317 \times 2 \times 20 = 52.68$ ton ✓

$R_w = 1317 \times 1 = 1317$ kgf

Plug weld:

$T_2 = \text{area} \times \text{ultimate stress} = \frac{\pi \times 3^2}{4} \times 0.75 \times 0.6 \times 4140 = 31.65$ ton ✓

$T_u = T_1 + T_u = 84.29$ ton

Flat bar:

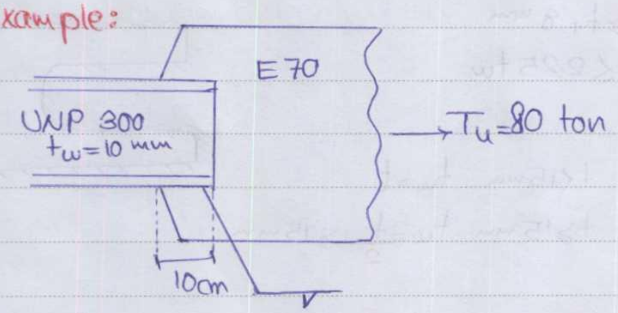
$T_u = \varphi F_y A_g = 0.9 \times 2400 \times 2 \times 20 = 86.4$ ton $\longrightarrow T_u = 84.29$ tons

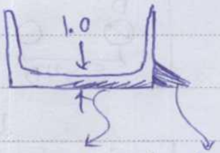
* لازم نیست برای block shear چک کردن چون فولاد مرغوبتر بریده است

PAPCO

Subject:
Year. Month. Date. ( )

Example:





$a_{max} < a'_{max}$: درواقع

$a_{max} = t_w - 1.5 = 8.5$ mm
$a_{min} = 5$ mm
$\Big\} \rightarrow$ take $a = 8.5^{mm}$

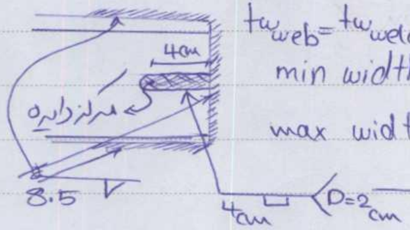
$t_e = 0.707\,a = 0.707 \times 8.5 = 6$ mm

$R_w = 0.75 \times 0.6 \times 4830 \times 0.6 \times 1 = 1.304$ ton $\longrightarrow L = \dfrac{80 \text{ ton}}{1.304} = 61.3$ cm $>$

$2 \times 10 + 30 = 50$

N.G

* بنابراین با اضافه کردن جوش کام کمبود مقاومت را جبران می کنیم.



$t_{w_{web}} = t_{w_{weld}} = 1.0$

min width $= t_w + 8 = 10 + 8$ 18 mm

max width $= 2.25\, t_w = 22.5$ mm

$\longrightarrow$ take $D = 2$cm

fillet weld: $\Sigma l = 48$cm $T_{u_1} = 48 \times 1304 = 62.6$ ton

$T_{u2} = 80 - 62.6$ ton $= 17.4$ ton

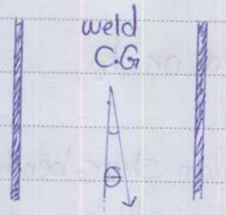
$$\text{required slot weld length} = \frac{T_{u2}}{\text{width} \times \underset{\downarrow\, 0}{\text{design stress}}} = \frac{17400}{2 \times 0.75 \times 0.6 \times 4830} = 4 \text{ cm}$$

$\rightsquigarrow$ فاصله تا مرکز دایره $< 10^{cm}$

O.K.

max slot length $= 10\, t_w = 10$cm > 4cm ✓

Trasversely Loaded Fillet welds: جوش های گوشه‌ای که بصورت جانبی بارگذاری شده اند



Design Strength $=\varphi(0.6F_E)$(effective throat t_e)

$$(\text{weld length } l_e)\times\underbrace{(1.0+0.5\sin^{1.5}\theta)}_{\text{قسمت اضافه شده}}$$

Example: Strength per cm

$a=8$ mm

$\theta=0^\circ; 45^\circ$

E 70

$t_e=0.707a=0.57$cm

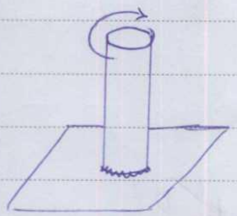
$\theta=0 \Rightarrow \varphi(0.6)F_E\times t_e\times l=0.75\times0.6\times4830\times0.57\times1=1229$ kg

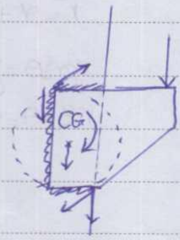
$\theta=45^\circ \Rightarrow 1+0.5\sin^{1.5}45=1.3 \longrightarrow 30\%$ increase in strength

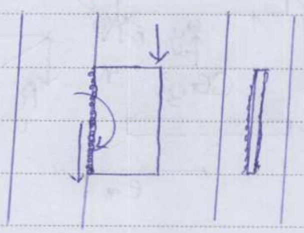
$\longrightarrow R_w=1595$ kg

Welded Connections with Eccentricity اتصالات جوشی با خروج از مرکزیت

(welds subjected to shear, torsion and bearing)







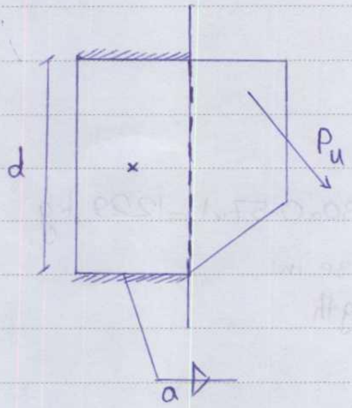
Elastic Design Approach:

روش طراحی الاستیک در طراحی جوش

1. Calculate effective weld area based on t_e
2. Determine of centroid of welds
3. Calculate the streeses resulting from shear, bending, torsion
4. Calculate the vector sum of above stresses at critical

در بحرانی ترین نقطه جوش

Shear and Torsion:

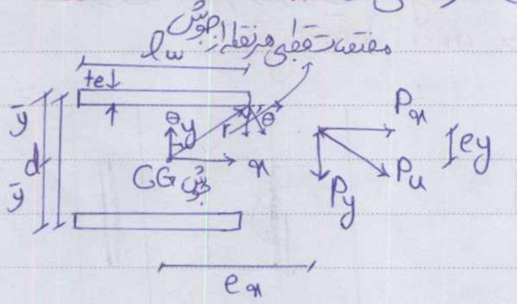


Assumptions:

- Rigid members اعضا صلب
- No friction بین اعضا اصطکاک نباشد
- Elastic weld اعضا و جوش ارتجاعی
- Welds at plate edges

هم جوش در لبه های ورق

Idealized Connection: اتصال ایده آل سازی می شود



$r^2 = x^2 + y^2$

$r\cos\theta = y$

$r\sin\theta = x$

Stress due to direct shear $f' : = \frac{P}{A}$

Stress due to torsional moment $f'' : = \frac{Tr}{J = I_p}$

پیچش کل

فاصله نقطه مورد نظر

Polar moment of Inertia

ممان اینرسی قطبی

$T = P_x e_y + P_y e_x$ $\qquad f'_x = \frac{P_x}{A}$ $\qquad f'_y = \frac{P_y}{A}$ $\qquad A = 2 t_e l_w$

$I_p = I_x + I_y$

$$f''_x = \frac{Ty}{I_p} = \frac{(P_x e_y + P_y e_x) y}{I_p}$$

$$f''_y = \frac{Tx}{I_p} = \frac{(P_x e_y + P_y e_x) x}{I_p}$$

$$I_p = I_x + I_y = \sum I_{xx} + \sum A\bar{y}^2 + \sum I_{yy} + \sum A\bar{x}^2$$

$$I_p = 2\left(\frac{1}{12} l_w t_e^3\right) + 2\left(l_w t_e \bar{y}^2\right) + 2\left(\frac{1}{12} l_w^3 t_e\right) + 20 = \frac{t_e}{6} \cdot \left(l_w t_e^2 + 12 l_w \bar{y} + l_w^3\right)$$

if t_e is assumed to be small $\Rightarrow t_e^2 \simeq 0$

$\longrightarrow I_p = \frac{t_e l_w}{6}\left(12\bar{y}^2 + l_w^2\right)$

* f_r فرم کلی : $f_r = \frac{1}{t_e}(\text{----}) \leq \varphi F_w$

حداکثر تنش

$\varphi F_w \to 0.6 F_E$

* total stress in x-dir : $\vec{f_x} = \vec{f'_x} + \vec{f''_x}$

$f_r t_e \leq \varphi F_w t_e = R_w \rightsquigarrow$ ارزش جوش

* total stress in y-dir : $\vec{f_y} = \vec{f'_y} + \vec{f''_y}$

$f_r t_e = f_r \big|_{t_e=1}$

* resultant stress : $f_r = \sqrt{f_x^2 + f_y^2}$

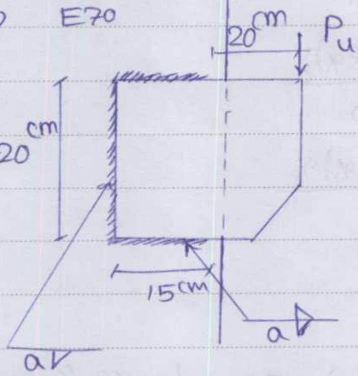
* ابتدا $t_e = 1$ مسئله را حل می‌کنیم و f_r را بدست می‌آوریم

وسپس با ارزش جوش (R_w) مقایسه می‌کنیم و می‌بینیم که t_e بدست آمده مناسب است یا خیر.

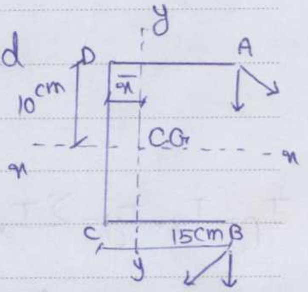
$$\Rightarrow \boxed{t_e \geqslant \frac{f_r|_{t_e=1}}{\varphi F_w} = \frac{f_r(1)}{\varphi F_w}} \longleftarrow \text{فرمول طراحی}$$

Example: Determine the fillet weld size (a)

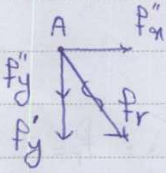
دستی E70



idealized weld section

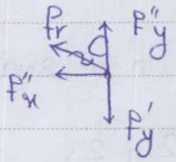


Point A:



higher stress
stronger weld

Point C:



lower stress
weaker weld

let $t_e=1$

$$\bar{x}=\frac{2\times15\times\frac{15}{2}}{2\times15+20}=4.5\ \text{cm}$$

$$I_p=\frac{1}{12}\times1\times20^3+\overset{\text{کوچک}}{\cancel{2\times\frac{1}{12}\times15\times1^3}}+2\times15\times1\times10^2+\overset{\text{کوچک}}{\cancel{\frac{1}{12}\times20\times1^3}}+2\times\frac{1}{12}\times1\times15^3+20\times1\times4.5^2$$

$$+2\times15\times1+(7.5-4.5)^2=\underbrace{4904\ (\text{cm}^4)}=4904\,t_e\ (\text{cm}^4)$$

$$A=2\times1\times15+20\times1=50\ \text{cm}^2$$

$f'_x = 0 \quad f'_y = \frac{9000}{50} = \frac{P_y}{A} = 180 \ \frac{kg}{cm^2}$

$T = P_y e_x + P_x e_y = 9000(20 + 15 - 4.5) = 274500 \ kg.cm$

$f''_x = \frac{Ty}{I_p} = \frac{274500 \times 10}{4904} = 559.7 \ (\frac{kg}{cm^2})$ (Point A)

$f''_y = \frac{Tx}{I_p} = \frac{274500 \times (\overset{15}{\cancel{35}} - 4.5)}{4904} = 587.7$ (Point A)

$f_r(1) = f_r\big|_{t_e=1} = \sqrt{(559.7)^2 + (587.7 + 180)^2} = 950.1 \ \frac{kg}{cm}$

$E70 : \varphi F_w = 0.75 \times 0.6 \times 4830 \times \underbrace{\overset{1.5}{(1 + 0.5 \sin 90)}}_{\text{overstimated}} = 3260.3 \ \frac{kg}{cm^2}$

$t_e \geqslant \frac{f_r(1)}{\varphi F_w} = \frac{950.1}{3260.3} = \boxed{0.29 \ cm}$ چون زاویه کمتر از 90° است*

Point C:

$f'_y = 180$

$f''_x = \frac{Iy}{I_p} = 559.7 \ \frac{kg}{cm^2}$

$f''_y = \frac{Tx}{I_p} = \frac{274500 \times 4.5}{4904} = 251.9$

$f_r(1) = \sqrt{(559.7)^2 + (251.9 - 180)^2} = 564.3 \ \frac{kg}{cm}$

understimated چون زاویه بیشتر از صفر است*

$\varphi F_w = 0.75 \times 0.6 \times 4830 \times 1 = 2173.5 \ \frac{kg}{cm^2}$

vertical weld

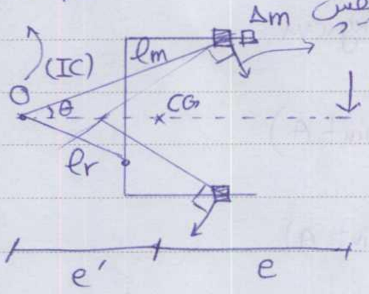
$t_e \geqslant \frac{564.3}{2173.5} = \boxed{0.26 \ cm}$ does not control $\Longrightarrow a = \frac{0.29}{0.707} = 0.41 cm \longrightarrow \boxed{\text{use } a = 5^{mm}}$

Ultimate Strength Method: روش مقاومت نهایی در طراحی جوش

(Shear & Torsion)

مرکز آنی دوران

تنش ناشی از برش هم پیچش



IC: somewher on the
horizontal axis passing
through C.G, when weld is symetric about this axis and
P_u is perpendicular

Trial & error to find IC:

weld deformation @ max stress: $\Delta_m = 0.209(\theta + 2)^{-0.32} a$ ✓

element deformation: $\Delta_r = \frac{\ell_r}{\ell_m} \Delta_m$ ✓

Nominal Strength in a weld element:

$$R = 0.6F_E(1 + 0.5 \sin^{1.5}\theta)[p(\overset{1.9}{\cancel{}} - 0.9p)]^{0.3}$$ ✓

P: ratio of deformation of an element to its deformation at ultimate state

→ at ultimate state: $p = 1.0 \rightarrow [----]^{0.3} = 1$

* فرض می کنیم که تمام المان ها به حداکثر تغییر شکل رسیده باشند

Procedure:

- * Assume a location for I.C.
- * Calculate R, ΣR_x, ΣR_y
- * Check equilibrium
  - ∘ $\Sigma F_x = 0$
  - ∘ $\Sigma F_y = 0$
  - ∘ $\Sigma M = 0$

Trial & error

→ Practical Approach

LRFD Manual (Part 8)

		C_1
ایران ←	E60	0.857
آمریکا	E70	1.000
	E80	1.03

مقاومت نهایی اتصال ←

$$P_u = \varphi R_n = C C_1 D \ell$$

ℓ ← length of vertical weld

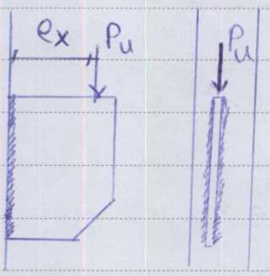
C: tabular coef. (include φ)

C_1: electrode coef.

D: weld size in sixteenth of an inch

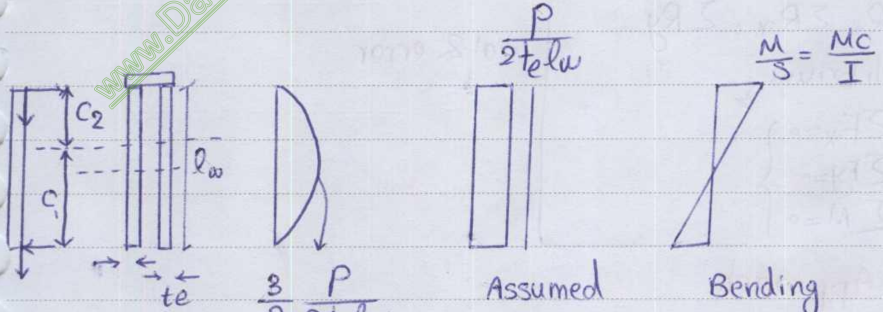
$$D = 16 \frac{a\,(cm)}{2.54}$$

Shear and bending Moment:



Elastic Design Method:

Idealized Connection



Actual shear stress disturbution

Assumed Shear Stress

Bending Stress

due to Shear $f'_y=\frac{P_y}{A}=\underline{\frac{P}{2te\,\ell w}}$

due to bending $f''_x=\frac{MC}{I}=\frac{Pe_x(\frac{\ell w}{2})}{2\frac{1}{12}t_e\ell_w^3}=\underline{\frac{3Pe_x}{t_e\ell_w^2}}$

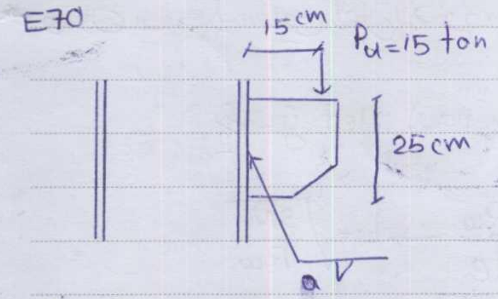
resultant Stress $f_r=\sqrt{f'^2_y+f''^2_x}=\underline{\frac{P}{2t_e\ell w}\sqrt{1+\frac{36e_x^2}{\ell w^2}}}$

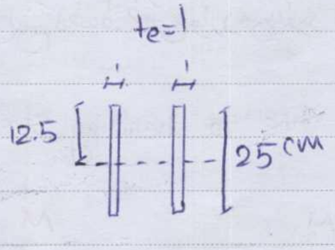
$f_r\leq\varphi F_w=\frac{Rw}{te}$ ارزش جوش

$f_r\ t_e=f_r|_{te=1}\Rightarrow f_r|_{te=1}\leq R_w\longrightarrow t_e\ \checkmark$

* در بیش تر موارد C را فاصله از محور تیر کششی می گیرند چون بحرانی تر است زیرا در فشار قطعه مشکلی ندارد ولی در کشش جوش کشش را تحمل می کند.

Example. Determine weld size





$$f'_y = \frac{P}{A} = \frac{15000}{2\times1\times25} = 300 \ \mathrm{kg/cm^2}$$

$$I_x = 2\,\frac{1}{12}\times1\times25^3 = 2604 \ \mathrm{cm^4}$$

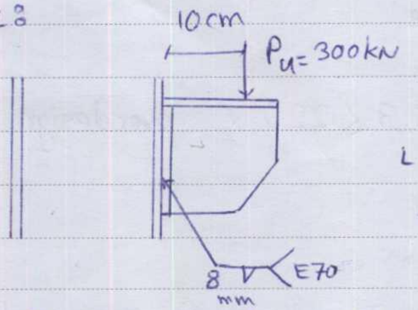
$$f''_x = \frac{MC}{I} = \frac{15000\times15\times12.5}{2604} = 1080 \ \mathrm{kg/cm^2}$$

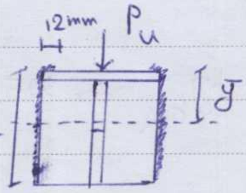
$$f_r(1) = f_r\big|_{te=1} = \sqrt{300^2+1080^2} = 1121 \ \mathrm{kg/cm^2}$$

$$\text{required } t_e = \frac{f_r(1)}{\varphi F_w} = \frac{1121}{0.75\times0.6\times4830} = 0.52 \ \mathrm{cm}$$

$$\text{required } a = \frac{t_e}{0.707} = 0.73 \ \mathrm{cm} \longrightarrow \underline{\text{use } a = 8 \ \mathrm{mm}}$$

Example:





$$R_w = t_e \varphi F_w = 0.707\times0.8\times0.75\times0.6\times4830 = 1229 \ \mathrm{kg/cm}$$

Subject
Date

$M = 300\ kN \times 10\,cm = 300\ ton\_cm$

* می‌توان یک معادله سنگین را حل کرد ولی فرض می‌کنیم که فقط خمش وارد می‌شود (برای راحتی)

let L: Only due to bending stress (for now) ; let $\bar{y} = \frac{l}{2}$

$$f_r = \frac{M}{S} = \frac{M}{2\frac{1}{12}\times te\times l^3/\frac{l}{2}} = \frac{M}{\frac{1}{3}l^2 te} = \frac{R_w}{te} \rightarrow L = \sqrt{\frac{3M}{R_w}}$$

$$L = \sqrt{\frac{3\times 300000}{1229}} = 27.1\,cm$$

Take $L = 30\,cm$ ⇝ برای این که بتواند برش را هم تحمل کند

$$\bar{y} = \frac{2\times 30\times\frac{30}{2}}{2\times 30 + 2\times 1.2} = 14.42\,cm$$

$$f'_y = \frac{30000}{\underset{te=1.0}{2\times 30}} = 500\ kg/cm^2$$

$$I_x = 2\frac{1}{12}\times 30^3 + 2\times 30\times\left(\frac{30}{2} - 14.2\right)^2 + 2\times 1.2\times 14.42^2 = 5019\,cm^4$$

$$f''_x = \frac{MC}{I} = \frac{30000\times 14.4}{5019} = 860.6\ kg/cm^2$$

$$f_r^{(1)} = \sqrt{500^2 + 860.6^2} = 995.3 = f_r\,te$$

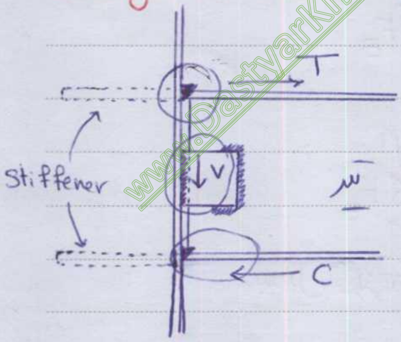
$f_r te \leq R_w = 1229\ kg/cm \Rightarrow 995.3 \leq 1229$ ✓ overdesign

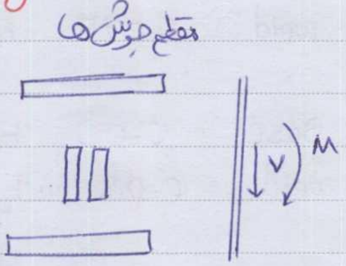
Try
L = 28cm

$\bar{y} = 13.42$ $f'_y = 535.7$ $I_x = 4110$ $S = \frac{I}{c} = 306$ $f''_x = 979.6$ $f_r(1) = 116.5 < 1229$

$$\rightarrow \boxed{take\ L = 28\,cm}$$

Design of Moment Resisting Connections: طراحی اتصالات گیردار

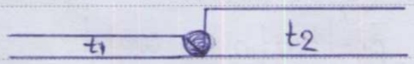




Groove welds:

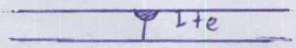
- Full penetration groove welds

strength = Strength of thinner/weaker plate



- Partial Penetration Groove welds جوش لب ب نفوذ ناقص

do not extend through the full thickness



Strength:

T or C
parallel to weld axis   Base $\varphi=0.9$ $F_{BM}=F_y$

Subject
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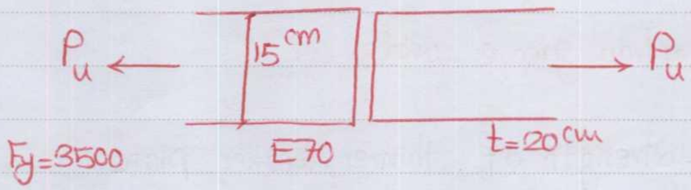
Shear on effective area	Base weld	⊛ $\varphi = 0.75$	⊛ $F_w = \overbrace{0.6}F_E$
T or C normal to eff. area	Base weld	$\varphi = 0.9$ $\varphi = 0.8$	$F_{BM} = F_y$ $F_w = \underbrace{0.6}F_E$

به دلیل برش است

به دلیل تمرکز تنش نه برش

Example:



a) Tensile strength w/ full penetration groove weld
b) 〃 〃 〃 〃 6 mm partial penetration weld

a) $\varphi P_n = \varphi A F_y = 0.9 \times 3500 \times 15 \times 2 = 94.5$ ton

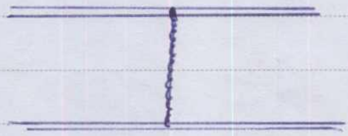
b) base : $\varphi P_n = \varphi F_y A = 0.9 \times 3500 \times 15 \times 0.6 = 28.35$ ton

weld : $\varphi P_n = \varphi (0.6 F_E) A = 0.8 \times 0.6 \times 4830 \times 15 \times 0.6 = 20.87$ ton

$\underline{\varphi P_n = 20.87 \text{ ton}}$

Subject
Date

Example: IPB 240 (two parts to be splited together)



$F_y = 3500\ kg/cm^2$
$F_y = 4550$
E70

$d = 24cm$ $t_w = 10\ mm$

a) Shear design strength of full penetration welds
b) 〃 〃 〃 〃 with two partial penetration welds with $t_e = 3\ mm$

a) base metal $V_u = 0.9\times 0.6 F_y \times A_w = 0.9\times 0.6\times 3500\times 24\times 1.0 = \boxed{45.36\ ton}$

weld material $V_u = 0.8\times 0.6\, F_{\cancel{g}E} \times A_w = 0.8\times 0.6\times 4830\times 24\times 1.0 = 55.64\ ton$

b) base material
Shear fracture
$\varphi F_{uv} A_{nv} = 0.75\times \underbrace{0.6\times 4550}\times \underbrace{24\times 0.6} = \boxed{29.5\ ton}$

Shear yielding
$\varphi (0.6 F_y) A_{gv} = 0.9\times 0.6\times \cancel{45}3500\times 24\times 1 = 45.36\ ton$

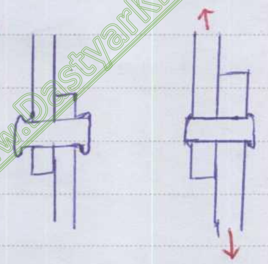
weld $\varphi (0.6 F_E) \underset{weld}{A_w} = 0.75\times 0.6\times 4830\times 24\times 0.6 = 31.3\ ton$

PAPCO

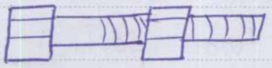
Subject
Date

Bolted and Riveted Connections: اتصالات پیچی و پرچی

Rivets:



Bolts:



Types:

- Unfinished (Ordinary/common) پیچ های معمولی، ساده، خام
  - ASTM : A307
  - mild structural steel فولاد نرمه ساختمانی
  - $\frac{5}{8}'' \sim 1\frac{1}{2}''$ قطر
  - Designed Similar to rivets
- High strength bolts پیچ های پر مقاومت

ASTM: A325 A490 A449

A325 ← medium-carbon steel کربن متوسط

A490 ← alloy steel فولاد آلیاژی

A449 ← diameters greater than $1\frac{1}{2}''$

- 2 or more strength of ordinary bolts
- $\frac{1}{2}'' \sim 1\frac{1}{2}''$ (A 325, A490)

- Advantages of High-Strength bolts
  - construction speed سرعت ساخت و ساز
  - Changes to the structure قابل تغییر در سازه

- better quality control کنترل کیفیت بهتر
- less required crew training آموزش کمتر برای کارگر

Types of Bolted Joints: انواع اتصالات پیچی

① _ Snug-Tight bolts (bearing_Type Connections) ⭑برای پیچ های معمولی به کار می رود

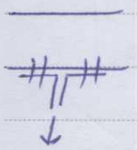
② _ Pretensioned Bolts ⭑برای پیچ های پر مقاومت باید پیش تنیدگی باشد

③ _ Slip_ Critical ⭑اتصالات که برای آن ها لغزش بحرانی باشد
یعنی لغزش کنترل کننده است.

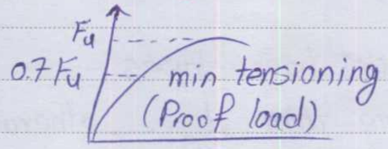
درواقع Joints: { Bearing-Type اتکایی
Slip_Critical اصطکاکی

⭑وقتی از پیچ های پر مقاومت استفاده می کنیم جدا از این که چه نوع پیچی باشد باید پیش تنیده شود.

Slip_Critical {
① when bolts are used in oversized holes
② Connections subjected to load reversals بارهای متغیر
③ Joints subjected to fatigue loads بارهای خستگی آور
④ when A490 bolts are subject to tension ←



Pretensioning: پیچ در کشش قرار می گیرد که حدوداً 70% حد تسلیم پیچ است



- Proof Load (KN) : T_b

(D) قطر / size (mm)	A325	A490
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

روش تشخیص پیش تنیدگی
برای ایجاد نیروی مورد نظر

- Turn-of-the-nut method

- Calibrated Wrench Method

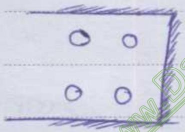
- Direct tension indicator * وقتی برجستگی های روی مهره از بین بروند نشان
می دهد که پیچ به مرحله پیش تنیدگی رسیده است

μN = resistance $\geq$ applied load → slip-critical
$<$ " " → bearing Type

Faying Surfaces (Treatment) سطوحی که قرار است به هم متصل شوند

- Cleanliness تمیز بودن
- Surfaces must have slopes of not more than $\frac{1}{20}$ → otherwise use beveled washers (واشرهای گوه‌ای)

* Mixed_Joints اتصالات ترکیبی

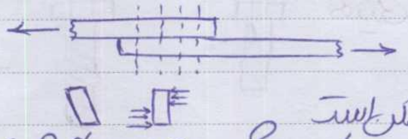


only slip-critical bolts can be mixed with welds or rivets

* فقط پیچ های اصطکاکی اجازه استفاده هم زمان با جوش ها و پرچ ها را دارند.

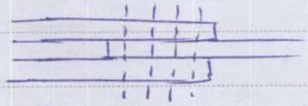
Types of Bolted Joints: انواع اتصالات پیچی

_ Lap Joint



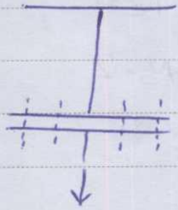
* خروج از مرکزیت ناشی از tilt ممکن است مشکل زا شده (باعث ایجاد لنگر شود)

_ Double_Plane Lap Joints



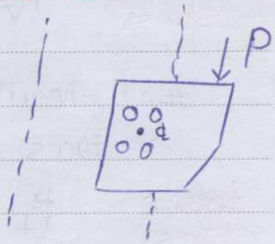
(Butt Joint)

* هر دو این اتصالات shear-connection هستند.

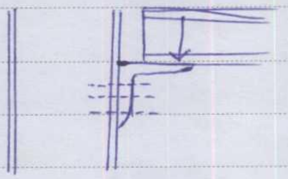


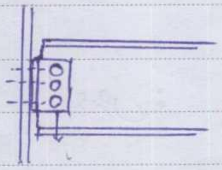
Tension-Connection

Shear Connection with eccentricity



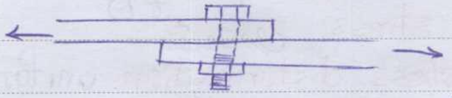
Combined Shear and Tension Connections

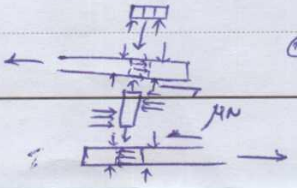




* اتصال ساده
چون برش کم تر شد
از طریق بال منتقل می شود

High-strength bolted Connection

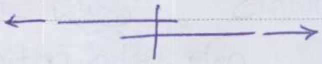


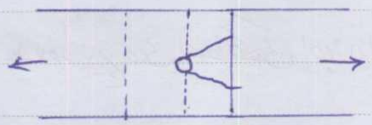


برای پیچ
N=T

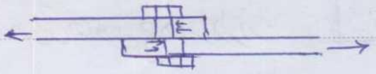
Failure of Bolted Joints:

5 مود گسیختگی

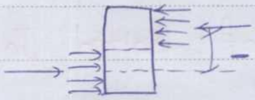




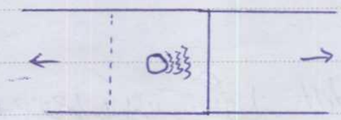
پاره شدگی
— tearing



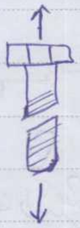
— shearing of bolt



— bending failure of bolt



لهیدگی

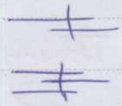


→ tension failure of bolts

→ bearing between bolt and member

Nominal Stresses

تنش های اسمی



$N_s = 1$

$N_s = 2$

Shear stress $f_v = \frac{P}{N_s \frac{\pi D^2}{4}}$

load

\# of shearing surfaces

cross-sectional area

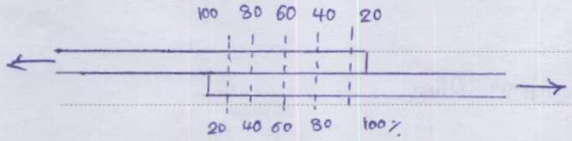
tensile stress $f_t = \frac{P}{\frac{\pi D^2}{4}}$ tensile load

bearing stress $f_p = \frac{P}{tD}$

ضخامت ورق

stress disturibution: uniform : فرض

Subject
Date



* سهم پیچ های کناری به دلیل تفاوت توزیع بین دو صفحه تنش بیشتری می افتد.

Bolted Connections:

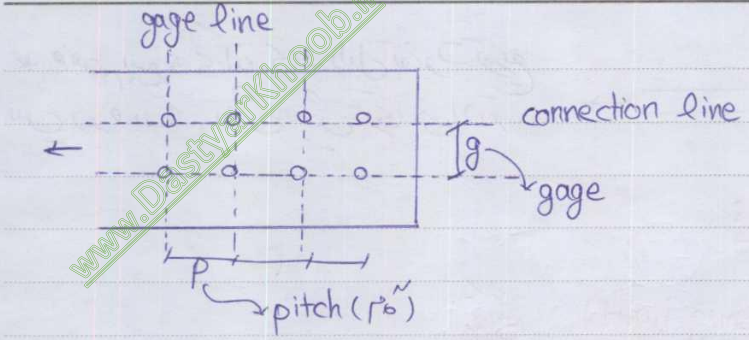
- Bearing Type (snug tight) اتکایی
- Friction Type (Slip_critical) اصطکاکی

Sizes of bolt holes:

Bolt Diam.	استاندارد standard	بزرگ شده Oversized	اتصال لوبیایی کوتاه short slotted hole	اتصال لوبیایی بلند long slotted holes
M24	27	30 ?	27×32	27×60
⋮				
≥M36	D+3	D+8	(D+3)×(D+8)	(D+3)×(2.5D)
		Only for friction-type connections	if the load is normal to the slot can be used in bearing Type connections	in only one of the plies of connection only if the load is normal.

* ~~وقتی~~ سوراخ لوبیایی بلند فقط در یکی از لایه ها می تواند باشد آن هم وقتی که بار عمود باشد.

Spacing and Edge distance of bolts:



Minimum Spacing:

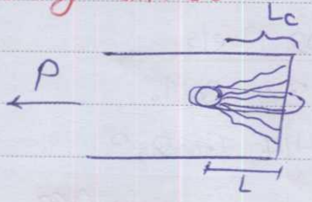
* C/C : 2.67 d → in practce 3d
(center to center) or p

* C/e :
(center to edge)

Bolt Diam.	گوشه‌های برش خورده C/e sheared edges	گوشه‌های نورد شده یا بریده شده C/e Rolled or Gas cut edges
16	28	22
20	34	26
22	38	28
24	42	30
27	48	34
30	52	38
36	64	46
>36	1.75d	1.25d

Tearing failure:

گسیختگی ناشی از پارگی :



shear

$$P_u = 2\varphi F_u^{s} t \underbrace{(L - \frac{D}{2})}_{L_c} \quad , F_u^{s} = 0.62 F_u$$

ورق

$$\boxed{P_u = 1.24\,\varphi F_u t L_c} \quad \varphi = 0.75$$

min edge distance:

(practical) $L_c = 1.5 \sim 2.5\ d$

$$P_u = \underset{1.27\times0.75\times1.5}{(1.4} \sim \underset{1.24\times0.75\times2.5}{2.3)} F_u d t$$

Maximum Spacing and edge distance:

- reduce corrosion کاهش خوردگی
- reduce warping/separation of plates کاهش اعوجاج

C/e max: $12t \leq 150$ mm ← Painted / not subject to corrosion

C/C max: $24t_{min} \leq 305$ mm

$14t_{min} \leq 180$ mm ← subject to atmosphere corrsion

Design Strength of Fasteners:

مقاومت طراحی اتصالات (وسایل اتصال)

- Fasteners Tensile Strength φF_{nt} $\varphi = 0.75$

$$F_{nt} \simeq \underbrace{0.75}_{} F_u$$

به فرض حذف شدن قسمت های دندانه دار

$$0.75 < \frac{\text{net bolt area (threaded)}}{\text{total bolt area}} < 0.79$$

Subject
Date

45

F_{nt} (MPa)

قطعات دنانه دار ↑

A307	A325	A490	Threaded	A502 Rivets
310	620	780	0.75Fu	310 Gr.1.
				414 Gr.2,3

only static loading

$$\varphi R_n = \varphi \times \underbrace{\overset{0.75}{0.75} F_u}_{F_{nt}} \times A_b \rightarrow \text{gross Area}$$

F_{nv} (MPa)

_ Fasteners Shearing Strength in bearing-Type connections

connection for connection length → $0.\overset{8}{\cancel{75}} \times 0.62$

X	165	414	520	0.5Fu	172 Gr.1 228 Gr.2,3
N	165	330	414	0.4Fu	~

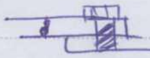
threaded part excluded from shear planes

threaded parts not excluded from shear plates

$0.\underset{8}{\cancel{75}} \times 0.62 \times 0.8$

A490_X

$$\varphi R_n = 0.75 \times F_{nv} \times \frac{\pi D^2}{4} \times N_s \qquad F_{nv} = 0.62 F_u$$



×0.8

Bearing Type Connections _ Loads Passing Through Center of Gravity of connections: اتصالات اتکائی وقتی بارها از مرکز سطح پیچ ها عبور می کنند.

- Shearing strength (above tables)
- Bearing strength (مقاومت لهیدگی) / Tearing Stregth (مقاومت پارگی ورق)
  - Spacing & edge distance of bolts
  - F_u of the connected parts (ورق ها)

Design Strength = φR_n $\varphi = 0.75$

* if the deformation must be limited to 0.25 in (6 mm)

$$R_n = \underbrace{1.2 L_c t F_u}_{\text{(tearing) ناشی از پارگی}} \leqslant \underbrace{2.4 d t F_u}_{\text{حدی برای لهیدن مصالح (bearing)}} \quad (L_c \leqslant 2d)$$

L_c: clear distance between the hole edges, or edges of the material in the direction of loading

* if the deformation is not a concern

$$R_n = 1.5 L_c t F_u \leqslant 3.0 d t F_u \qquad (L_c \leqslant 2d)$$

* if the connection is made with slotted holes (long) (slots being normal to load)

$$R_n = 1.0 L_c t F_u \leqslant 2.0 d t F_u \qquad (L_c \leqslant 2d)$$

وقتی $L_c > 2d$ بشود، tearing پارگی کنترل کننده نیست و فقط bearing ؛ لهیدگی کنترل کننده است.

Minimum Connection Strength: $\phi R_n \geqslant 4.5$ ton

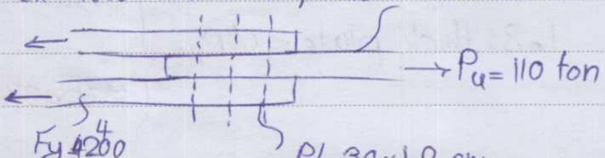
* Abbrevation

SC slip-critical (fully-tensioned)
N snug-tight (threads included in shear planes)
X snug-tight (threads excluded in shear plates)

Example: threads excluded from shear planes

$\frac{3}{4}$" A325

$d = \frac{3}{4} \times 2.54 = 1.905$ cm



Fy 4200
Fu = 3700 kg/cm²

Subject

Date

فرض: Use 2~3 bolts in a connection Line $U=0.85$

C/C: $3d \simeq 6.0$ cm (6~7.5 cm)

C/e: 4.0 (usually taken 4~5 cm)

Bearing / Tearing Strength Per bolt: مقاومت ورق و کشیدگی پیچ

$$L_c = \min\left\{(6-2.2),\ 4-\frac{2.2}{2}\right\} = 2.9 \text{ cm}$$

$$1.2 L_c t F_u = 1.2\times2.9\times1.8\times3700 = 23.1 \text{ ton}$$

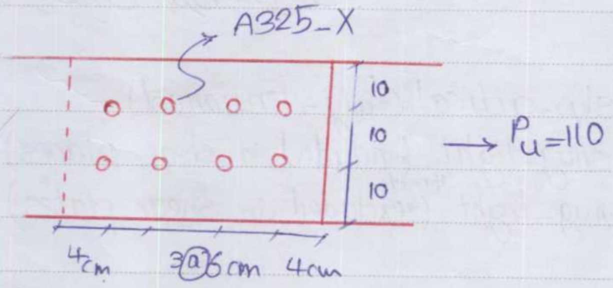
$$2.4 d t F_u = 2.4\times1.905\times1.8\times3700 = 30.45 \text{ ton}$$

$$\varphi R_n = 0.75\times23.1 = 17.32 \text{ ton} \geq 4.5 \text{ ton O.K.}$$

Shearing strength Per bolt:

$$\varphi R_n = 0.75\times\frac{\pi\times1.905^2}{4}\times\overset{N_s}{2}\times4140 = 17.7 \text{ ton} \Rightarrow \varphi R_u = 17.3 \text{ ton}$$

$$\text{bolts required} = \frac{110}{17.3} = 6.35 \rightsquigarrow \text{take 8 bolts}$$



Check:

$$1.8\text{-thick plate: } \phi R_n = \begin{cases} 0.9\times2400\times1.8\times30 = 116.6 \text{ ton} > 110 \text{ tons} \checkmark \text{ controls} \\ 0.75\times3700\times\underbrace{1.8[30-2\times2.2]}_{A_n=45.91} = \underline{127.9} \text{ ton} \end{cases}$$

O.K.

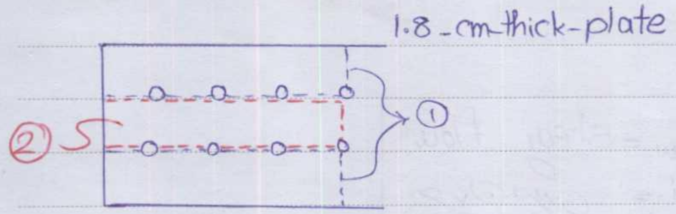
$A_n \leq 0.85 A_g$

$A_g = 30 \times 1.8 = 54 \text{ cm}^2 \longrightarrow \text{use } 45.9 \text{ cm}^2$

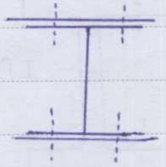
$0.85 A_g = 45.9$

$= 127.4 \text{ ton}$

Block Shear failure:



Bolted Cover plates for I-shaped Members. ورق های تقویت پیچی



shear stress $f_v = \frac{VQ}{Ib}$

Shear force across flange per unit length $= b \times 1 \times \frac{VQ}{Ib} = \frac{VQ}{I}$ shear flow جریان برشی

نیروی برشی در عرض بال در طول واحد

bolts spacing in longitudinal direction $= \frac{\text{Bolt(s) design strength}}{\text{shear force per unit length (shear flow)}}$

فاصله پیچ‌ها در امتداد تیر

Subject

Date

* bolt spacing $\le \min \left\{ \frac{1050}{\sqrt{F_y}} t_{min} , 30 \text{ cm} \right\}$

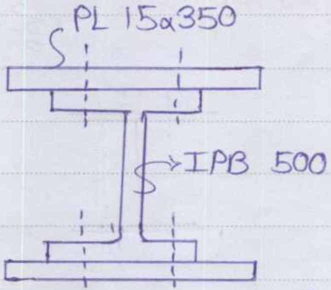
t_{min} → ورق یا مقطع

* $A_{\text{cover plates}} \le 0.70 \, A_{\text{Flange}}$

* Welded Cover plates

$$\boxed{\begin{array}{c} R_w = \text{shear flow} \\ \text{جریان برشی} = \text{ارزش جوش} \end{array}}$$

Example:



Determine bolt spacing

$\frac{7}{8}$" A325-X

$V_u = 120$ ton

$A_{\text{cover plate}} = 1.5 \times 35 = 52.5 \text{ cm}^2$

$A_{\text{Flange}} = 2.8 \times 30 = 84 \text{ cm}^2$

$\frac{A_{\text{cover plate}}}{A_{\text{Flange}}} = \frac{52.5}{84} = 0.62 < 0.7$ O.K.

$I = \underbrace{107180}_{\text{IPB 500}} + 2(35 \times 1.5) \times 25.75^2 = 176802 \text{ cm}^4$ بفرض کوچک بودن I ورق ها

$\frac{V_u Q}{I} = \frac{120000 \times 35 \times 1.5 \times 25.75}{176802} = 810.65 \text{ kg/cm}$

two bolts in each gage line:

$\hookrightarrow 2 \phi A_b F_{nv} = 2 \times 0.75 \times 3.88 \times 4140 = 24.09$ ton

Subject
Date

assuming $L_c > 2d$

فقط این کنترل کننده می‌شود $\leftrightarrow 2\varphi(2.4dtF_u) = 2 \times 0.75 \times 2.4 \times 2.22 \times 1.5 \times 3700 = 44.36$ ton

Design strength of each gage line: 24.1 ton

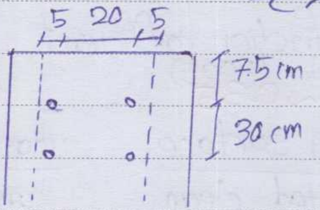
spacing: $\frac{24092}{811} = 29.7$ cm → use $p = 30$ cm

assuming c/e = 7.5 cm

@ end $L_c = 7.5 - \frac{1}{2} \times (2.22 + 0.3) = \overset{6.23}{\cancel{6.35}}$ cm $> 2d = 4.44$ cm

$L_c > 2d$ ✓

$\frac{1050}{\sqrt{F_y}}\ t_{min} = \frac{1050}{\sqrt{2400}} \times 1.5 = 32$ cm ✓



Slip-Critical Connections - Loading Passing through center of gravity of Connections:

اتصالات اصطکاکی - بار به مرکز سطح اتصالات وارد می‌شود.

* هرچند در این نوع اتصالات پدیده برشی و لهیدگی کنترل کننده نیست ولی آیین نامه می‌گه این باید چک بشه.

Resistance by Friction ⇝ no bearing and no shear in bolts

but: the code requires checking of bearing strength

- the code provides "design shear strength" which is in fact "design friction strength"

Design shear strength

Design slip Resistance: $\boxed{\varphi R_n = \varphi D_u \mu T_b N_s}$ ← 2005 ~ 2009 →

Subject
Date

$\varphi=1.0$	0.85	0.75
standard holes or short slotted holes perpendicular to load	Oversized holes or short slotted holes parralel to load	long-slotted holes

حداقل پیش تنیدگی پیچ

T_b: minimum fastener tension ($0.7F_u$)

در آیین نامه ایران: $\boxed{0.55 A_b F_u}$

D_u: ratio of mean bolt pretension to the minimum = 1.13

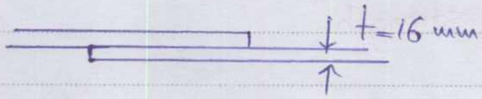
μ: mean friction coefficient

(2010) (2005) 0.3 ~ 0.35	0.5
class A surface	class B surface
- unpainted clean mill scale - galvanized and roughened surface	unpainted blast-cleaned surface

N_s: # of shear surfaces (اگر تعداد پیچ‌ها بیش تر بود باید در تعداد پیچ ضرب شود)

Example: 24 mm A325_SC
standard-sized holes
$\mu=0.33$

$P_D=13$ ton
$P_L=22$ ton



$P_u=1.2P_D+1.6P_L=50.8$ ton

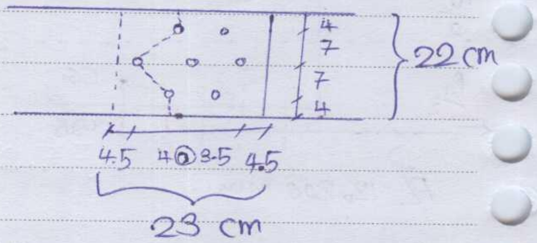
$\varphi=1.0$

$T_b=205$ kN ≃ 20.5 Ton

Subject
Date

$\varphi R_n = 1.0 \times 1.13 \times 0.33 \times 20.5 \times 1 = 7.64$ ton

$\#$ bolts required $= \frac{50.8}{7.64} = 6.64$



bearing: holes: 27 cm

$L_c = \min\left\{7.0 - 2.7, 4.5 - \frac{2.7}{2}\right\} = 3.15$ cm

$$\left\{\begin{aligned} &\varphi R_n = \varphi(1.2) L_c t F_u = 0.75 \times 1.2 \times 3.15 \times 1.6 \times 3700 = \boxed{16.78 \text{ ton}} \overset{\text{controls}}{} \\ &\varphi(2.4) d t F_u = 0.75 \times 2.4 \times 2.4 \times 1.6 \times 3700 = 25.57 \text{ ton} \end{aligned}\right.$$

$\varphi R_n = 16.78 > 7.64$ ton does not control

other checks:

- plate tensile strength $\left\{\begin{array}{l}\text{yielding} \\ \& \\ \text{Rupture}\end{array}\right.$ $-\frac{S^2}{4g}$ نیاز است

- Block shear strength

Design result:

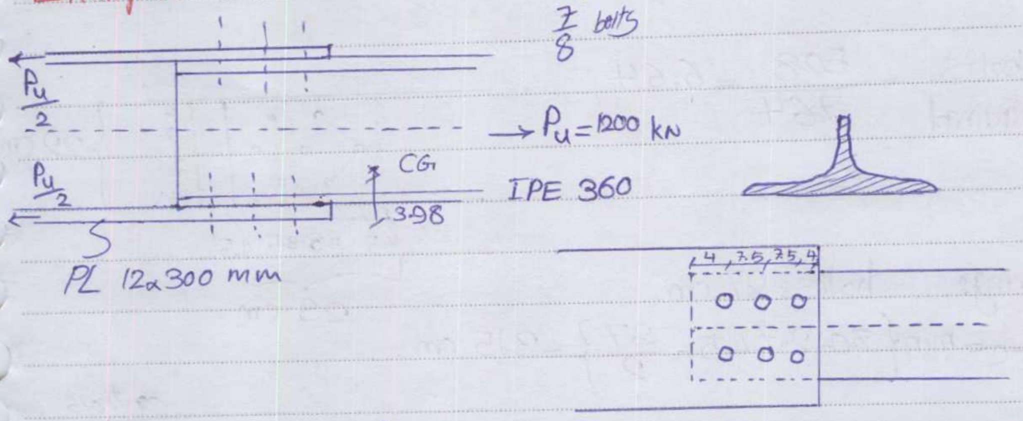
7ō24mm A325-SC

Arrangement shown

min plate overlap: 23 cm

PAPCO

Example:



a) Tensile strength of IPE360

$A_g = 72.7 cm^2$ $t_f = 1.27$ cm

*yielding $\phi_t P_n = 0.9 \times 2400 \times 72.7 = 157.03 > 120$ ✓

$A_n = 72.7 - 4 \times (\frac{7}{8} \times 2.54 + 0.3) \times 1.27 = 59.9 \ cm^2$

Checking 12 bolts spaced @ 7.5 cm

$\longrightarrow L = 15$ cm

$$\bar{x}_{\text{profile half}} = \frac{17 \times 1.27 \times \frac{1.27}{2} + 18 \times 0.8 \times \frac{18}{2}}{17 \times 1.27 + 18 \times 0.8} = 3.98 \ cm$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{3.98}{15} = 0.735$$

$A_e = 0.735 \times 59.9 = 44.0 \ cm^2$

*Fracture $\phi_t P_n = 0.75 \times 3700 \times 44.0 = 122.1$ ton > 120 ✓

b) Tensile Strength of plates

$A_g = 2 \times 1.2 \times 30 = 72\ cm^2$

$\phi_t P_n = 0.9 \times 2400 \times 72 = 155.5$ ton ✓ ≥ 120 ✓

$$\left.\begin{array}{l} A_n = 72 - 4(\frac{7}{8} \times 2.54 + 0.3) \times 1.2 = 59.9\ cm^2 \\ 0.85 A_g = 0.85 \times 72 = 61.2\ cm^2 \end{array}\right\} A_n = 55.9\ cm^2$$

$\phi_t P_n = 0.75 \times 3700 \times 599 = 166.2$ ~~cm²~~ ton > 120 ✓ o.k.

c) bolts in single shear and bearing on $\min\{t_{plate}, t_{flange}\} = 1.2$

shearing strength:

تعداد پیچ ها

$0.75 \times 4140 \times \frac{2.22^2 \pi}{4} \times 12 = 144.2$ ton ✓

let edge distance: 4 cm

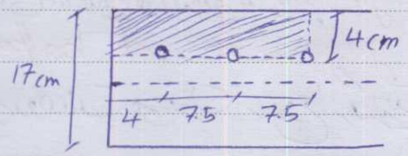
$L_c = \min\left\{7.5 - 2.5, 4.0 - \frac{2.5}{2}\right\} = 2.75\ cm$

Bearing Strength:

$$\underbrace{0.75 \times 1.2 \times 2.75 \times 1.2 \times 3700 \times 12}_{131.9} \leq \underbrace{0.75 \times 2.4 \times 2.2 \times 1.2 \times 3700 \times 12}_{211.0}$$

$= 131.9$ ton ✓

d) Block shear strength - IPE 360



$A_{gt} = 4 \times 1.27 = 5.08\ cm^2$

$A_{gv} = 19 \times 1.27 = 24.13\ cm^2$

$A_{nt} = 5.08 - \frac{1}{2} \times 2.5 \times 1.27 = 3.49\ cm^2$

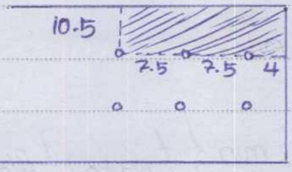
$A_{nv} = 24.13 - 2.5 \times 2.5 \times 1.27 = 16.19\ cm^2$

$0.6\,F_y\,A_{gv} = 34.7$ ton
$0.6\,F_u\,A_{nv} = 35.9$

$T_n = 0.6\,F_y\,A_{gv} + F_u\,U_{bs}\,A_{nt} = \overset{34.7}{\cancel{37.4}} + 3700 \times 1 \times 3.49$

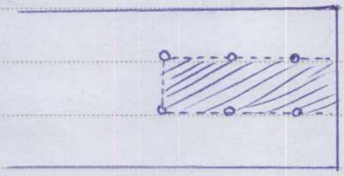
$\varphi T_n = 0.75 \times 47.7$ ton $> \frac{P_u}{4}$ ✓

e) block shear strength - (Plate)



$T_n = 73.9$

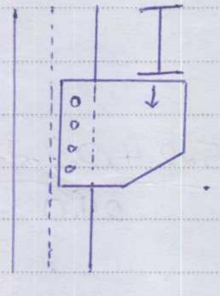
$\varphi T_n = 0.75 \times 73.9 = 55.4 > \frac{P_u}{4}$ ✓ O.K.



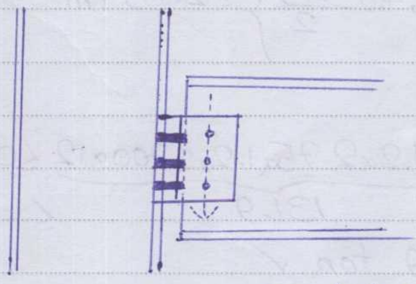
does not Control

$\frac{P_u}{2}$ مقایسه.

Eccentrically Loaded Bolted Connections: اتصالات پیچی با بار خارج از مرکزیت



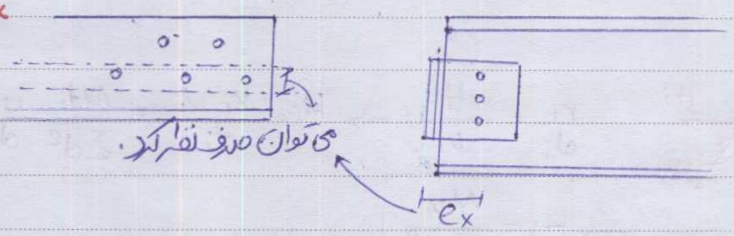
bolts in pure shear



bolts subjected to combined tension and shear

* Statically Loaded Single or Double Angles: بارهای استاتیکی وارد بر نبشی تک و دوبل
* Eccentricity between gravity axis and gage lines of bolted members (statically Loaded) در این دو حالت می توان از خروج از مرکزیت صرف نظر کرد.

مثال *



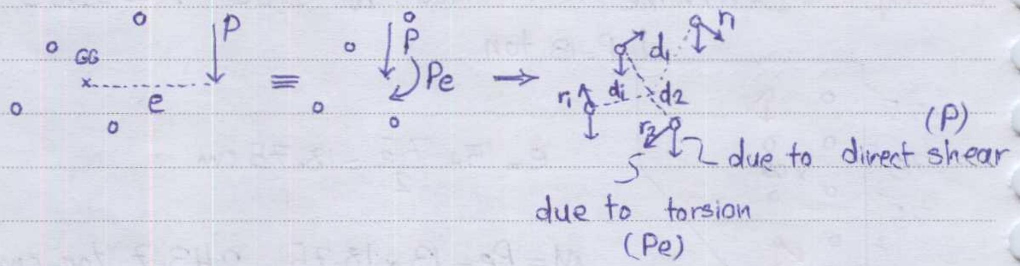
می توان صرف نظر کرد.

In all other cases: eccentricity must be considered

Methods of Analysis:

- * Elastic Method (very conservative) → مثلاً از 8 پیچ، 6 پیچ overdesign هستند بنابراین از روش دوم استفاده می کنیم
- * Reduced Eccentricity Method
- * Ultimate Strength method

Elastic Analysis:



$$M_{CG} = Pe = r_1 d_1 + r_2 d_2 + \cdots$$

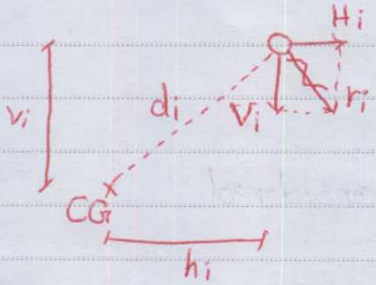
$$\frac{r_1}{d_1} = \frac{r_2}{d_2} = \cdots$$

$$r_1 = \frac{r_1 d_1}{d_1},\ r_2 = \frac{r_1 d_2}{d_1} \quad \text{or} \quad r_i = \frac{r_1 d_i}{d_1}$$

$$M_{GG} = r_1 \frac{d_1^2}{d_1} + \frac{r_1 d_2^2}{d_1} + \cdots = \frac{r_1}{d_1}(d_1^2 + d_2^2 + \cdots) = \frac{r_1 \sum d_i^2}{d_1} \Rightarrow r_1 = \frac{M d_1}{\sum d_i^2}$$

$$r_2 = \frac{M d_2}{\sum d_i^2}$$

$$\Rightarrow r_i = \frac{M d_i}{\sum d^2}$$



$$\frac{r_i}{d_i} = \frac{H_i}{v_i} \Rightarrow H_i = \frac{r_i v_i}{d_i} = \frac{M d_i}{\sum d^2}\,\frac{v_i}{d_i}$$

$$\rightarrow \boxed{H_i = \frac{M v_i}{\sum d^2}}$$

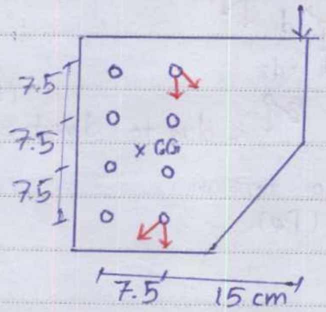
$$\boxed{V_i = \frac{M h_i}{\sum d^2}}$$

due to direct shear:

$H_i' = \frac{P_x}{n}$, $V_i' = \frac{P_y}{n} \longrightarrow$ تعداد پیچ‌ها

total bolt force $= P_u = \sqrt{(H_i+H_i')^2 + (V_i+V_i')^2}$

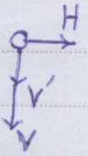
Example: Determine the minimum bolt size A325_SC



$e = 15 + \frac{7.5}{2} = 18.75$ cm

$M = Pe = 13 \times 18.75 = 243.7$ ton-cm

Top right bolt:



$$\sum d^2 = \sum h^2 + \sum v^2 = 8\times 3.75^2 + 4\times 3.75^2 + 4\times(7.5+3.75)^2 = 675$$

$$H = \frac{Mv}{\sum d^2} = \frac{243700\times 11.25}{675} = 4061.7 \text{ kgf}$$

$$V = \frac{Mh}{\sum d^2} = \frac{243700\times 3.75}{675} = 1353.9 \text{ kgf}$$

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$V' = \frac{P_u}{n} = \frac{13000}{8} = 1625$ kgf

$\Rightarrow R_u = \sqrt{(4061.7)^2 + (1353.9 + 1625)^2} = 5037$ kgf

$\varphi R_n = \varphi 1.13 \mu T_b N_s = 1.0 \times 1.13 \times 0.33 \times T_b \times 1 = 5037$ kgf

$\to T_b = 13508$ kgf $\xrightarrow{\text{از جدول}}$ M 20 - A325 - SC

$(T_b = 142$ kN$)$

Reduced (Effective) Eccentricity Method :

*One gage line of fasteners

$e_{eff} = e - 0.635(1+2n)$ (cm) n: # of bolts in a gage line

*Two or more gagelines, symetrically placed

$e_{eff} = e - 1.25(1+n)$ (cm)

Example (Previous): $e = 18.75$

2 gage lines
n=4

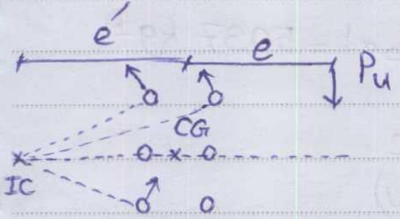
$e_{eff} = 18.75 - 1.25(1+4) = 12.5$ cm

$\Rightarrow e = 12.5$ cm طبق محاسبات قبل $R_u = 3704$ kg

$T_b = 9.93$ ton $\longrightarrow$ M18 - A325 - SC

Ultimate Strength Method:

at Ultimate load: rotation and translation ≡ an equivalent rotation about a single point (instantaneous center of rotation مرکز آنی دوران)



deformations ∝ distance from IC

Ultimate Shear force $R = \underbrace{R_{ult}}_{\text{ultimate strength}}\left(1-e^{-10\Delta}\right)^{0.55}$ → برای یک پیچ

bolt deformation

$\min\{\text{bearing strength, design shear strength}\}$ $\Delta_i = \dfrac{d_i}{d_{max}}\Delta_{max} \rightarrow \simeq 0.86\text{ cm}$

Assume IC Location → e' → Δ_i → R_i → Check equilibrium ↺

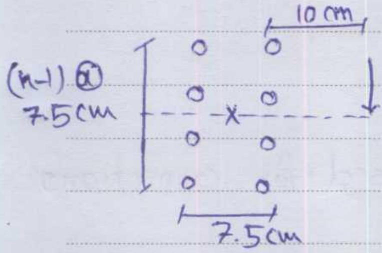
Practical Approach - Using LRFD Tables

$\phi R_n = C \cdot \phi r_n$ ← nominal strength of one bolt

C: tabulated values based on eccentricity and bolt pattern

ϕ: ضریب کاهش مقاومت

Example (~~Previous~~): number of required $\frac{7}{8}$" bolts (A325-X) bolts: single shear, bearing on t=1.2 cm



$e_x = 10 + \frac{7.5}{2} = 13.75$ cm

Shearing strength:

$\varphi R_n = 0.75 \times \frac{2.2^2 \pi}{4} \times 4140 = 11.8$ ton controls

bearing strength:

$\varphi r_n = 0.75 \times 2.4 \times \overset{d}{2.2} \times \overset{t}{1.2} \times \overset{Fu}{3700} = 17.58$ ton

$L_c = 7.5 - (2.2 + 0.3) = 5 \text{ cm} > \underset{=4.4}{2d}$ ✓

$\varphi r_n = 11.8$ ton

required $C = \frac{\varphi R_n}{\varphi r_n} = \frac{P_u}{\varphi r_n} = \frac{45}{11.8} = 3.81$

using tables:

$\left.\begin{array}{l} n = 4 \\ e \simeq 5.5 \text{ inch} \\ S = 3 \text{ inch} \end{array}\right\} C = 3.95 > 3.81 \text{ O.K.}$

$\Rightarrow 8 \ \frac{7}{8}$" in A325-X

Spacing

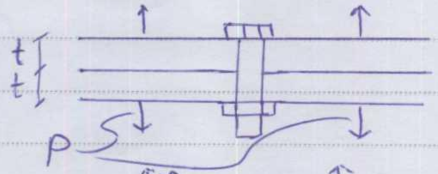
Fasteners Acting in Axial tension:

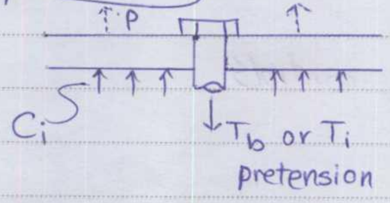
$R_u = \varphi R_n = \varphi F_{nt} A_b N_b$

$F_{nt} \rightarrow 0.75 \sim 0.8 F_u$

$\varphi \rightarrow 0.75$

*Only fully tensioned bolts should be used for connections with axial tension





→ Apply Tension load P → $T_b \rightarrow T_f$, $C_i \rightarrow C_f$

$C_i = T_b$ (I)

$C_f + P = T_f$ (II)

plate deformation $\delta_p = \frac{C_i - C_f}{A_p E_p} t$

bolt deformation $\delta_b = \frac{T_f - T_b}{A_b E_b} t$

$\delta_p = \delta_b$

$$\rightarrow \frac{T_f - T_b}{E_b A_b} = \frac{C_i - C_f}{E_p A_p} \Big\} \xrightarrow{\text{I, II}} \underbrace{\frac{T_f - T_b}{A_b E_b} = \frac{T_b - T_f + P}{A_p E_p}}_{S_{glass}}$$

$$\rightarrow T_f = T_b + \underbrace{\frac{P}{1 + A_p/A_b}}_{\text{increase in bolt tension}}$$

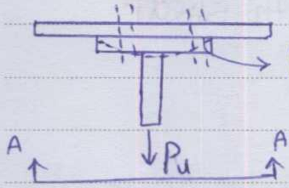
example : $\frac{7}{8}''$

Spacing : 7.5cm $\quad \frac{A_p}{A_b} = \frac{7.5^2}{2.2^2\pi/4} \longrightarrow T_p = T_b + 0.063P$

$\Rightarrow$ Small variation in stress if $P < T_b$

$C_f = T_b - 0.937P \Longrightarrow$ design for $P < T_b$

Example: the number of $\frac{7}{8}''$ bolts required for $P_u = 140$ ton
A490



→ Prying Action (عمل اهرمی)

می توان از سفت کننده ← ابتدا ردیف اول از فراگیر خارج
استفاده کرد می شود سپس ردیف های بعدی

$\varphi = 0.75$ $\quad \rightarrow N_b = 6.29$

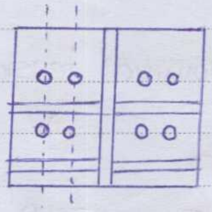
$P_u = 140$ ton $\quad \longrightarrow$ use 8 bolts

$P_u = \varphi P_n = \varphi A_b F_{ut} N_b$

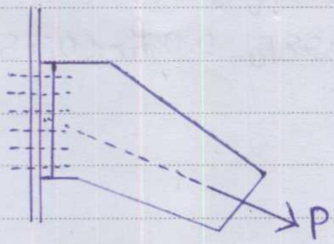
$A_b = \frac{\pi \times 2.2^2}{4} = 3.8\,cm^2$

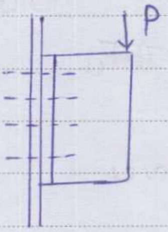
$F_{ut} = 7800\ kgf/cm^2 = F_{nt} = 0.75 \sim 0.8 F_u$

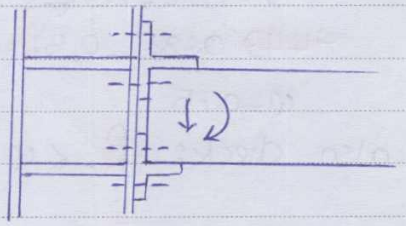
AA



Bolts subjected to shear and Tension:



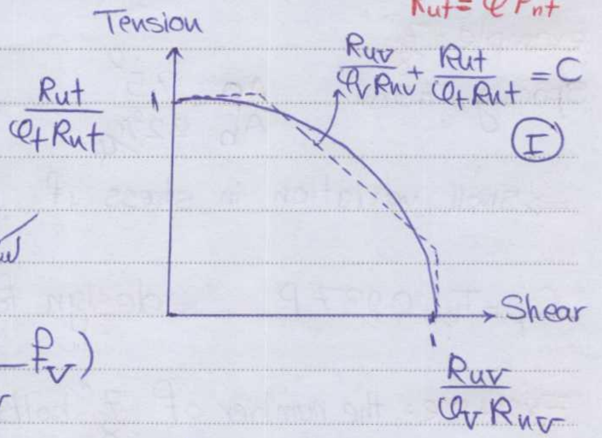




$R_{ut} = \varphi F'_{nt}$

Bearing Type Connections:

$$\left(\frac{R_{ut}}{\varphi_t R_{nt}}\right)^2 + \left(\frac{R_{uv}}{\varphi_v R_{nv}}\right)^2 \leq 1.0$$



Ⓘ $f_{ut} \leq \varphi F'_{nt}$ ⟶ کشش در حضور برش

تنش کششی

$$= \varphi\left(0.75\,\overset{F_{nt}}{\overbrace{C\,F_u}} - \frac{F_{nt}}{\varphi F_{nv}} f_v\right)$$

F'_{nt}: Nominal tensile strength in combination with shear

$C = 1.3$

F_u: ultimate strength of bolt

F_{nt}: nominal tensile strength $\simeq 0.75 F_u$

F_{nv}: nominal shear strength

f_v: ultimate shear strength $= R_{uv} \div F_{uv}$ (پایین صفحه 41)

⟶ Nominal tension in bearing-type connections $\boxed{F'_{nt}}$ (MPa)

	N	X
A307	--- $171 - 2.5 f_v \leq 310$ ---	
A325	$807 - 2.5 f_v \leq 621$	$807 - 2.0 f_v \leq 621$
A490	$1010 - 2.5 f_v \leq 779$	$1010 - 2.0 f_v \leq 779$
A449	$0.98 F_u - 2.5 f_v \leq 0.75 F_u$	$0.98 F_u - 2.0 f_v \leq 0.75 F_u$

$\varphi = 0.75$

also check: $f_v \leq \varphi F_{nv}$

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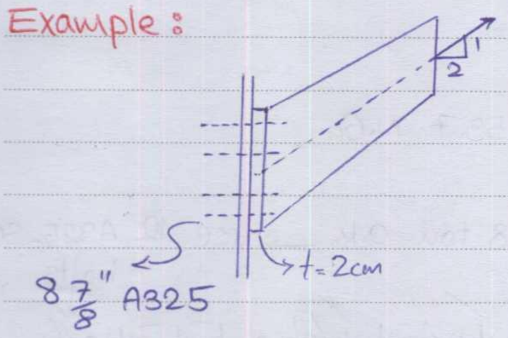
Slip-Critical Connections:

* shear strength reduction factor → $1-\dfrac{T_u}{1.13T_bN_b}$ Ultimate tension resisted by N_b bolts

* $F'_{nv}=\left(1-\dfrac{T_u}{1.13T_bN_b}\right)F_{nv}$

Example:



Check the adequacy:
a) bearing Type connection
(X)
b) Slip-critical connection
(Class B , μ=0.5)

$P_{uy}=53.7$ ton $=V_u$

$P_{ux}=107.3$ ton $=T_u$

$A_b=\left(\frac{7}{8}\times2.54\right)^2\frac{\pi}{4}=3.87\ cm^2$

a) bearing type connection

$f_v=\dfrac{V_u}{N_bA_b}=\dfrac{53700}{8\times3.87}=1734\ kg/cm^2$

$\varphi F_{nv}=0.75\times4140=3105>f_v$ O.K. → Check shear strength

* حال چک کشش و برش توأم:

$f_{ut}=\dfrac{T_u}{N_bA_b}=\dfrac{107300}{8\times3.87}=3466\ kg/cm^2$

$\varphi F'_{nt}=\varphi(8070-2.0f_v)\leq6210\varphi \Rightarrow 0.75(8070-2.0\times1734)=3452\ kgf\simeq f_t$ O.K.

(Check bearing strength)

$f_p=\dfrac{T_u}{N_btd}=\dfrac{107300}{8\times2\times2.22}=3021\ kg/cm^2$ $R_n=\overset{0.75\times}{2.4}\times\cancel{2.22\times2}\times8277=\overset{26407}{\cancel{19865}}$ O.K.

b) Slip-Critical Connection

$T_u = 107.3$ ton

$$1-\frac{T_u}{1.13T_bN_b} = 1-\frac{107.3}{1.13\times17.6\times8} = 0.326$$

$$\varphi R_n = 0.326\times(\underset{0.75}{\varphi}\times1.13\times\underset{0.5}{\mu}\underset{17.6}{T_b}\overset{1}{N_s}) = 3.24 \text{ for each other}$$

for 8 bolts : $\varphi R_n = 8\times3.24 = 25.9$ ton < 53.7 N.G.

Try 12 bolts:

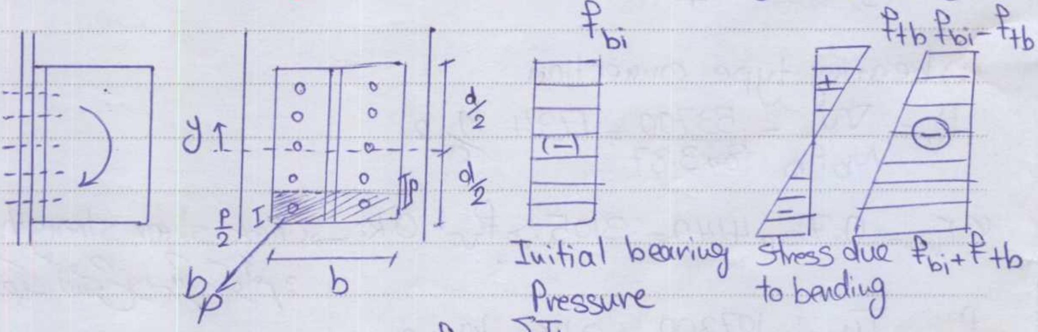
$$1-\frac{T_u}{1.13T_b\underset{12}{N_b}} = 0.55 \Rightarrow \varphi R_n = 65.68 \text{ ton O.K.} \longrightarrow \text{use 12 A325-SC bolts}$$

اگر خواهیم اتصال اصطکاکی استفاده کنیم.

Shear and Tension from Eccentric Loading:

*Tension from bending moment

تنش کششی ناشی از خمش



$$f_{bi} = \frac{\sum T_b}{bd}$$

$$f_{tb} = \frac{M\frac{d}{2}}{I} = \frac{6M}{bd^2}$$

load T on Top bolt. $T = f_{tb} \underbrace{bp}_{\text{tributary area}}$

$$\longrightarrow T = \frac{6M}{bd^2} bp = \frac{6Mp}{d^2}$$

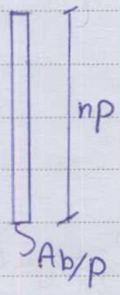
Assuming the top bolts is $\frac{p}{2}$ below the top $T = \frac{6Mp}{d^2}\left(\frac{d-p}{d}\right)$

* Simplified Procedure: considering bolt cross sections

$$f_t = \frac{My}{I} = \frac{My}{\sum A_b y^2} \longrightarrow \underline{T = A_b f_t = \frac{My}{\sum y^2}}$$ (same A_b for all bolts)

Ⓘ ≃ Ⓘⅰ : $d = \underset{\substack{\#\text{bolts} \\ \text{in a gage line}}}{n}p$

$$T = \frac{6Mp}{d^2}\left(\frac{d-p}{d}\right) = \frac{6Mp}{d^2}\left(\frac{np-p}{np}\right) = \frac{6Mp}{n^2p^2}\left(\frac{np-p}{np}\right) = \frac{12M}{n^3p^2}\underbrace{\left(\frac{p(n-1)}{2}\right)}_{\simeq y \text{ for outermost bolt}}$$



$$I = \frac{1}{12}\frac{A_b}{p}(np)^3 \simeq \sum A_b y^2$$

$$\text{or } \frac{n^3p^2}{12} \simeq \sum y^2$$

* To use the above relation in design: first only consider bearing

$$\varphi R_{nt} = \frac{6M}{n^2p}\underbrace{\left(\frac{n-1}{n}\right)}_{\simeq 1} = \frac{6M}{n^2p} \longrightarrow n = \sqrt{\frac{6M}{\varphi R_{nt} p}}$$ $6M \rightarrow \frac{1}{2}M_u$ for one gageline

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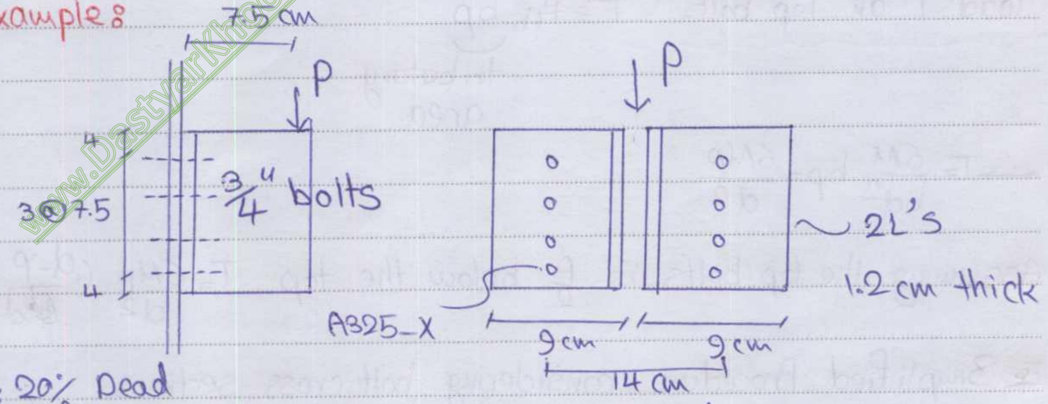
Example:



P: 20% Dead
80% Live → Determine the service load capacity P

$P_u = 1.2(0.2P) + 1.6(0.8P) = 1.52P$

$M_u = 1.52Pe = 11.4P$

$p = 7.5$ cm

$d = 3 \times 7.5 + 2 \times 4 = 30.5$ cm

$$T_u = \frac{6M_u p}{d^2}\left(\frac{d-p}{d}\right) = \frac{6 \times 11.4P \times 7.5}{\underbrace{2 \times 30.5^2}_{}}\left(\frac{30.5-7.5}{30.5}\right) = 0.208P$$

two gagelines of fasteners

$V_u = \frac{P_u}{N_b} = \frac{1.52P}{8} = 0.19P$

Bolts: in shear & tension

* pure shear

$\phi R_{nv} = 0.75 \times 4140 \times \frac{\pi \times 1.9^2}{4} = 8850 \text{ kgf} > V_u = 0.19P$

* Tension & shear

$\phi R'_{nt} = 0.75(8070 - 2.0f_v)A_b \leq 6210\phi A_b$

$T_u = 0.75\left(8070 \times \frac{\pi \times 1.9^2}{4} - 2.0V_u\right) \leq 13200$

$= 17160 - 1.5V_u \leq 13200$

$0.208P = \underset{Ⅱ}{17160} - 1.5(0.19P)$, $\underset{Ⅲ}{0.208P} \leq 13200$

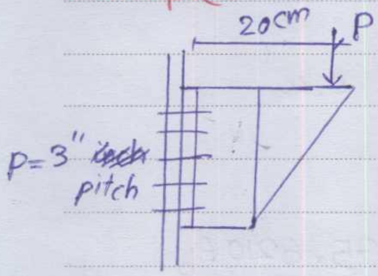
① P = 46.58 ton,
② P = 28.72 ton
③ P = 63.46 ton

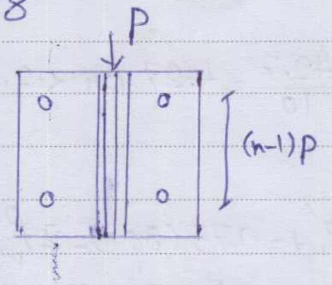
→ P = $\boxed{28.72 \text{ ton}}$ service load

Subject

Date

Example: # of $\frac{7}{8}$" A325 bolts





$P_D = 3.6$ ton $\quad P_L = 24$ ton

a) A325_X

b) A325_SC $\mu = 0.5$

a) A325_X

$P_u = 1.2 \times 3.6 + 1.6 \times 24 = 42.7$ ton

$A_b = \frac{\pi \times 2.2^2}{4} = 3.87 \text{ cm}^2$

$\varphi R_{nv} = 0.75 \times 4140 \times 3.87 = 12.0$ ton فقط برش

$\varphi R_{nt} = 0.75 \times 6200 \times 3.87 = 18.0$ ton 〃 کشش

$M_u = P.e = 42.7 \times 20 = 854$ ton.cm

each gageline: $M_u = \frac{854}{2} = 427$ ton.cm

$n = \sqrt{\frac{6Mu}{P\varphi R_{nt}}} = \sqrt{\frac{6 \times 427000}{7.5 \times 18000}} = 4.4 \rightsquigarrow$ based on moment alone

$n = \frac{\frac{P_u}{2}}{\varphi R_{nv}} = \frac{\frac{42.7}{2}}{12} = 1.8 \rightsquigarrow$ based on shear alone

try: 5 bolts in each gageline (total: 10)

Subject
Date

$\sum y^2 = 4 \times (7.5^2 + (2 \times 7.5)^2) = 11.25 \text{ cm}^2$

check shear: $\frac{P_u}{n} = \frac{42.7}{10} = 4.27$ ton < 12.0 ton O.K.

check tension combined with shear : $\varphi R'_{nt} = 0.75(8070 - 2f_v)A_b \leq 0.75 \times 6210 A_b$

$= 23.42 - 1.5 \times 4.27 = 17 \leq 18.0$

$\hookrightarrow \varphi R'_{nt} = 17.0$ ton

$T_u = \frac{M_u y}{\sum y^2} = \frac{2 \times 427000 \times 15.0}{11.25} = 11.37$ ton $\ll 17.0$ ton O.K.

Try 8 bolts:

$\sum y^2 = 562.5 \text{ cm}^2$

$\frac{P_u}{n} = 5.34$ ton < 12.0 ton O.K.

$\varphi R'_{nt} = \underbrace{23.42 - 1.5 \times 5.34}_{15.4} \leq 18$ $\hookrightarrow \varphi R'_{nt} = 15.4$ ton

$T_u = \frac{2 \times 427000 \times 11.25}{562.5} = 17.08 > 15.4$ N.G.

$\longrightarrow$ Use 10 $\frac{7}{8}''$ A325_X

b) A325_SC

Checking 10 bolts

$T_u = 11.37$ ton

$T_b = 17.6$ ton $T_u < T_b$ ✓

حداکثر نیروی کششی که توسط پیچ تحمل می شود.

$$1-\frac{T_u}{1.13T_b}=1-\frac{11.37}{1.13\times17.6}=0.428$$

shear alone : $\phi R_n=\phi\times1.13\times\mu\times T_b\times N_s=1\times1.13\times0.5\times17.6\times1=9.94$ ton

shear strength with tension : $\phi R_n'=0.428\times9.94=4.26$ ton

$P_u=4.27$ ton $=\frac{42.7}{10}$

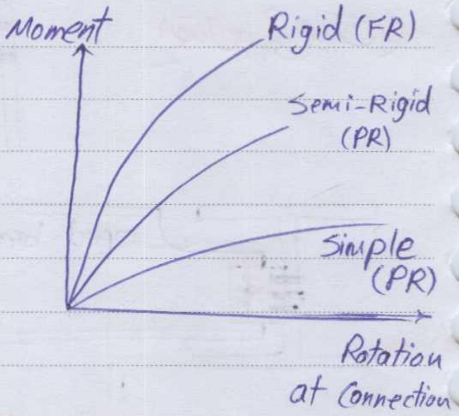
almost O.K.

$\rightarrow$ Use 10 $\frac{7}{8}$" A325-SC

Building Connections اتصالات ساختمانی

Types of Beam Connections:

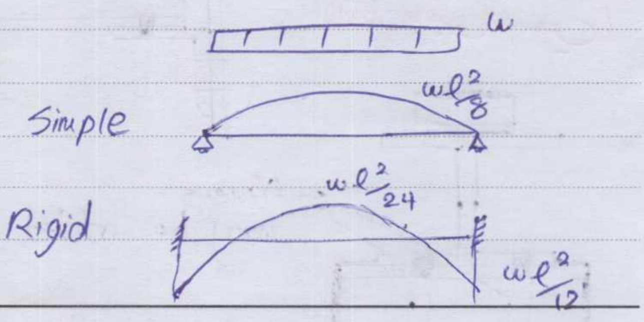
- Fully Restrained کاملاً گیردار
- Partly Restrained } Semi-Rigid, Simple



نسبت صلبیت

Rigidity Ratio $RR=\frac{\text{moment developed at Connection}}{\text{moment developed by a completely rigid connection}}$

RR 0~20% $\rightarrow$ simple
20~90% $\rightarrow$ semi-rigid
90~100% $\rightarrow$ rigid



Subject
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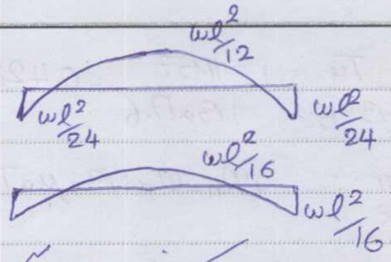
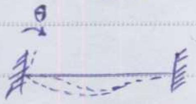
50% Rigid

✓ 75% rigid ازنظر اقتصادی تیر مناسب تر بدست می آید.

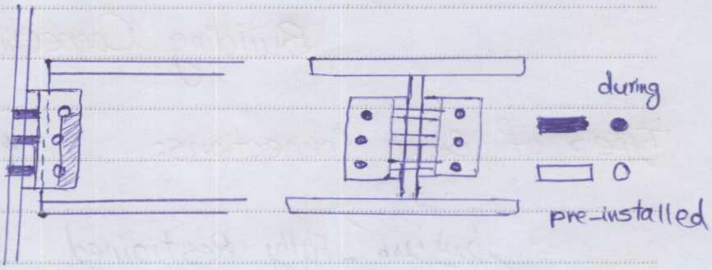


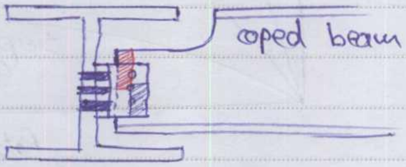


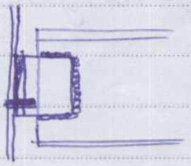
* معیار ما برای این که بفهمیم اتصال گیردار است یا نه این است که ببینیم اتصال ما قابلیت چرخش دارد یا نه ؟

Simple Connections اتصالات ساده

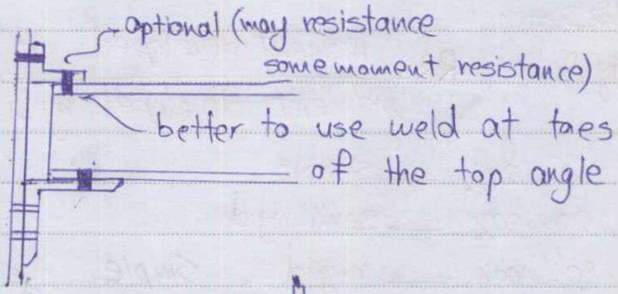
1_Framed Simple Connection
(اتصال نبشی جان)

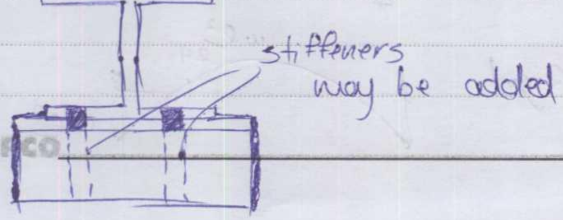


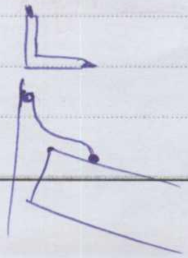




2_Seated Angle Connection
(اتصال نبشی نشیمن)

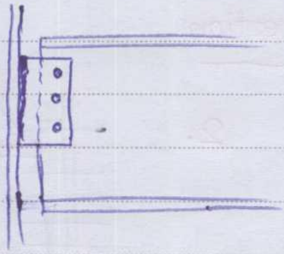






Subject
Date

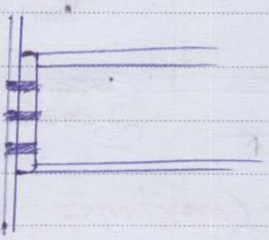
3_ Single Plate or shear tab Connection



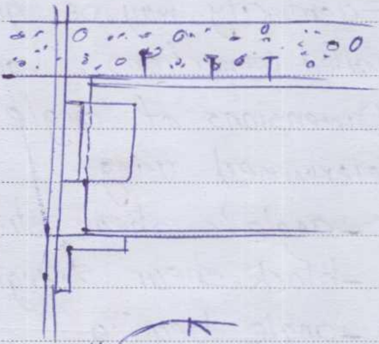
Semi-Rigid Connections

1- Top and Seat Angle Connection

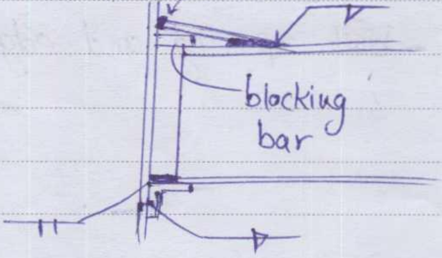
2_ End plate Connection



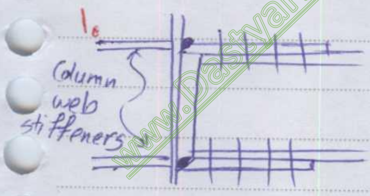
3_ Composite Connection with seat (and /or web) angles

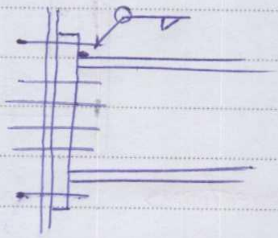


4_ Connection with seat angle and Top plate

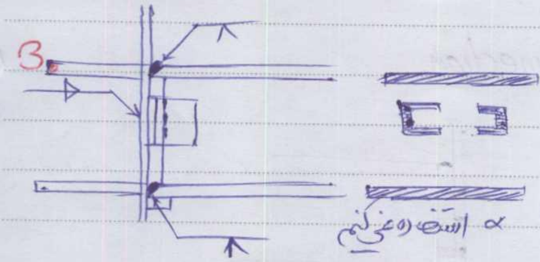


Moment_Resisting Connection

1.

2.



3.

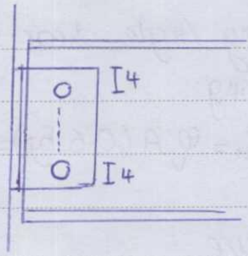


Framed Simple Connections: (اتصالات نبشی جان)

- Eccentricity may be ignored in pretensioned bolted connections, (Loading: static, no fatigue)
- Dimensions of angle and its thickness is determined using:
  - angle's shear strength
  - block shear strength of the angle
  - angle bearing
  - bolt spacing and edge distance

Subject
Date

Framed Simple Connection:



Example: All-bolted double-angle framed Simple Connection

Beam: IPB 70

$t_w = 17$ mm

$R_u = 70.0$ ton $\frac{3}{4}''$ A 325-N

number of bolts required:

$$A_b = \frac{\pi 1.9^2}{4} = 2.85 \text{ cm}^2$$

$$\varphi R_n = 0.75 \times 3300 \times 2.85 = 7.05 \text{ ton}$$ مقاومت برشی اتصال اتکایی برای یک پیچ

$$\varphi R_n = 2.4 \times \frac{3}{4} \times 2.54 \times t \times 3700 = 16916\, t$$ مقاومت لهیدگی

$(L_c > 2d)$

$$16916 t \geq 7050 \rightarrow t \geq 0.42 \text{ cm}$$

for beam web $t_w \geq 2 \times 0.42 = 8.4$ mm < 17 mm o.k. ✓

$\#$of bolts: $\frac{70}{7.05} = 9.9 \longrightarrow$ 10 Single shear (5 double shear)

select $8 \times 8 \times 0.8$ cm

Angle length: $4 \times 2 + 4 \times 7.5^{\text{cm}} = 38$ cm

Checking Angle shear strength:

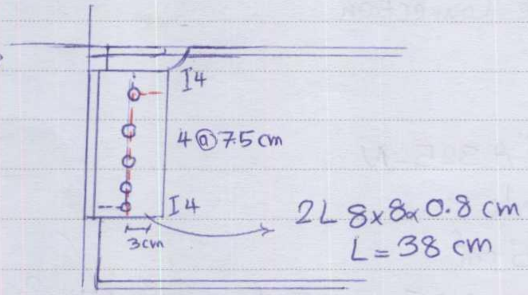
yielding

$$\varphi V_n = \varphi_v A(0.6F_y) = 0.9(0.8\times 38)\times 0.6\times 2400 = 39.4 \text{ ton} > \frac{V_u}{2} = 35 \text{ ton}$$

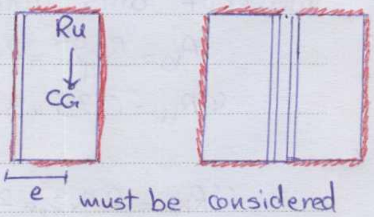
Rupture

$$\varphi V_n = \varphi_v A_n(0.6F_u) = 0.75\times(38-5\times 2.2)\times 0.8\times 0.6\times 3700 = 35.96 > \frac{V_u}{2} \checkmark$$

Result:

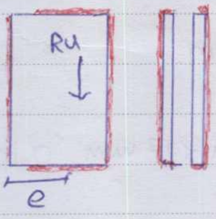


remaing checks:
block shear



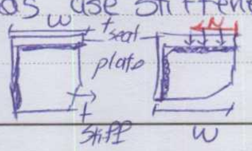
must be considered in design

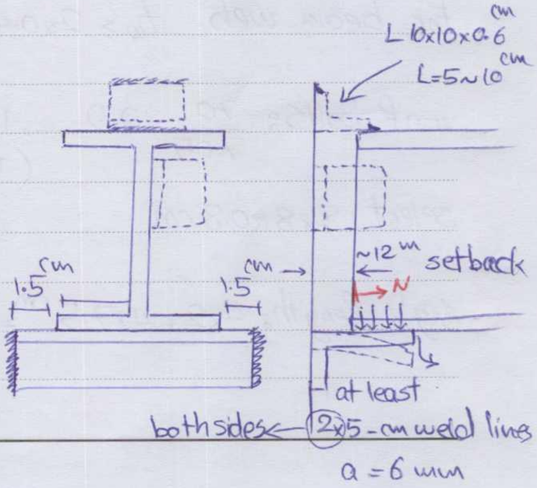
Shear Tab Connection



Seated Beam Connections:

- for loads up to 60 ton angle is sufficient
- for more loads use stiffeners





Subject
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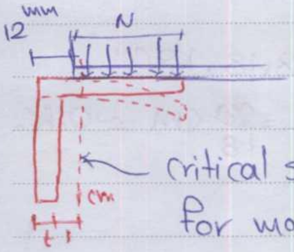
Selection of N:

$N \geq 10cm$ برای اجرا

web yielding $N \geq \frac{R_u}{\varphi F_y t_w} - 2.5k \quad \varphi = 1.0$

web crippling $R_u \leq \varphi 0.4 t_w^2 \left[1 + 3\frac{N}{d}\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_y t_f E}{t_w}} \quad \varphi = 0.75$

$\left(\frac{N}{d} < 0.2\right)$ check



critical section
for moment in angle

Connection to Column $e_f = \text{setback} + \frac{N}{2}$

angle to bending $e = e_f - 1 - t_{angle}$ (cm) → قسمت toe نبشی مثل تکیه‌گاه فرض می شود.

$$M_u = R_u e = \varphi_b M_n$$

$$R_u e = \varphi_b M_p = \varphi_b Z F_y = \underset{0.9}{\varphi_b} \frac{Lt^2}{4} F_y$$

P: نبشی از نشیمن → $t^2 = \frac{4 R_u e}{\varphi_b F_y L}$

Example: Beam: IPE 300 $R_u = 10$ ton
Column: IPB 200

$*N:$ $N \geq 10$ cm

web yielding $N = \frac{10000}{1 \times 2333 \times 0.71} - 2.5 \times 2.55 < 0$

web crippling $\xrightarrow{N=10}$ 19.5 ton > 10 ton
↓
does not occur

* Angle thickness

$$e_f = \frac{N}{2} + \text{setback} = 5 + 1.2 = 6.2 \text{ cm}$$

$$e = e_f - t - 1 = 5.2 - t$$

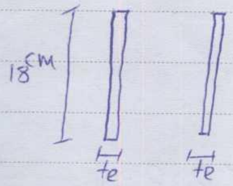
$$L = b_f + 3 = 15 + 3 = 18 \text{ cm}$$

$$t^2 = \frac{4 \times 10000 \times (5.2 - t)}{0.9 \times 2333 \times 18} \longrightarrow t = 1.88 \text{ cm}$$

select L 18x18x1.8 cm

$L = \cancel{20}\ 18$ cm ≃ O.K.

*weld (E70)



let $t_e = 1$

$I = 972 \text{ cm}^4/\text{cm}$

$f'_x = 574 \text{ kg/cm}^2$

$f''_y = 278 \text{ kg/cm}^2$

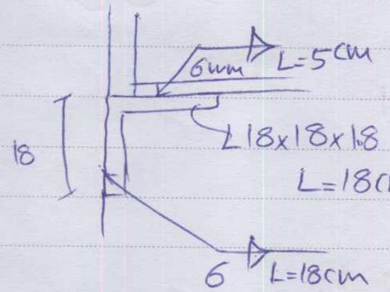
$$f_r = \sqrt{574^2 + 278^2} = 638 \text{ kg/cm}^2 = f_{r(1)} = f_r t_e$$

$$f_{r(1)} \leq \overset{0.75}{\varphi} (0.707a) 0.6 \times 4830$$

$\longrightarrow a = 0.42$ cm

$\longrightarrow$ use $a = 6$ mm

check min weld size



Stiffened Seated Beam Connections

Design: - select N
- stiffener design:

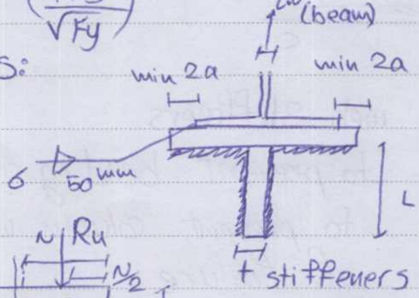
$t_{stiffener} \geq t_w(beam)$

$t_{seat\ plate} \geq t_f(beam)$

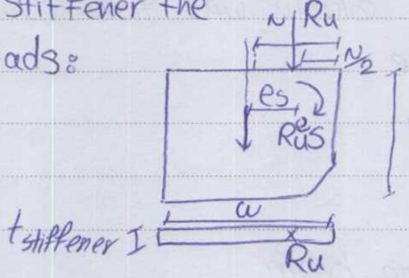
stiffener

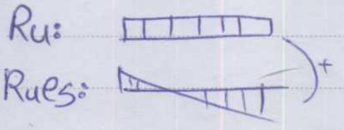
Compactness : $t_{stiffener} \geq \frac{w}{\left(\frac{795}{\sqrt{F_y}}\right)}$

- dimensional requirements:



- design the stiffener the applied loads:





$)^{+} \leq \varphi_b F_y (w \times t_s)$?

$e_s = \frac{w}{2} - \frac{N}{2}$

ظرفیت جوش — ظرفیت برشی ورق

- weld capacity < plate shearing capacity

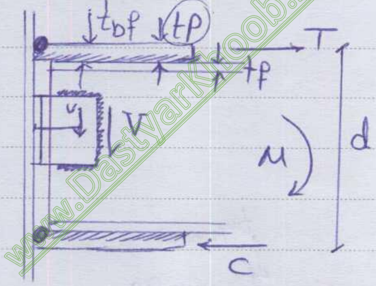
Moment Resisting Connections:

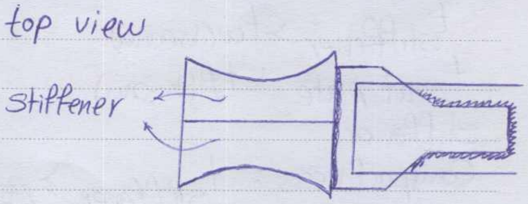
ممان ← $\frac{M}{d} = C = T$ کشش، فشار

عمق تیر ← d

Subject
Date

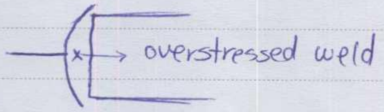
برای ستون این ضخامت مهم است



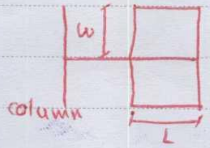


* Column web stiffeners

- to prevent bending of column and rebase of moment
- to prevent column web from its possible modes of failure



* connection to only one beam



$L \geq \frac{d}{2} - t_f$

factored load from beam flange: P_{bf}
column web height: $h = d - 2k$
thickness of beam flange: t_{bf}
column flange: t_{cf}

* Necessity of web stiffeners
if P_{bf} > any of the following values
$\longrightarrow$ stiffeners needed

Subject
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- Stiffness of column

$$R_u = 6.25\, t_{cf}^2 F_{yf}$$

$$\varphi = 0.9$$

- local web yielding $N = t_{bf}$

$$P_u = \varphi F_{yw}\, t_{cw} (t_{bf} + 5k)$$

$$\varphi = 1.0$$

- compression buckling of unstiffened web

$$P_u = \varphi \frac{24 t_{cw}^3 \sqrt{E F_{yw}}}{h}$$

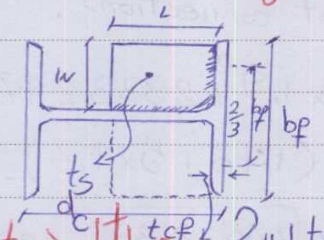
$$\varphi = 0.9$$

Subject
Date

Column Web Stiffeners

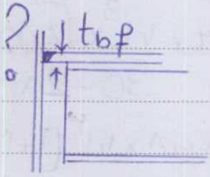
- Necessity: * Stiffness of column flange
  * Column web { yielding
    Buckling in compression
    Crippling

- Selection and design



1. $W + \frac{1}{2} t_{cw} \geqslant \frac{1}{3} b_f$ ↳ beam flange width

column web thickness



2. $\begin{cases} t_s \geqslant \frac{1}{2} t_{bf} \\ t_s \geqslant 1.79 \sqrt{\frac{F_y}{E}} \\ t_s \geqslant \frac{W}{15} \end{cases}$

3. if moment connection is made only to one column flange : $L \geqslant \frac{d_c}{2} - t_{cf}$
otherwise : extend the stiffener to fill in the column web

4. the weld to column web should have enough strength to carry the forces caused by unbalanced moments
اختلاف نیروهای دو طرف ستون را با جوش سخت کننده به ستون تحمل کند .

5. stiffeners must be designed to carry the excess applied load

Example: column IPB 300

moment → T = C = 69 ton

$b_f = 30cm$ $t_{bf} = 1,46cm$

→ design stiffeners
for moment connections

$$* P_{bf} = \min \begin{cases} -6,25 \times 0,9 \times 1,9^2 \times 2333 = 47374 kg \\ -2333 \times 1,1 \times (1,46 + 5 \times 4,6) = 62771 \\ -0,9 \dfrac{24 \times 1,1^3 \times \sqrt{2 \times 10^6 \times 2333}}{30 - 2 \times 4,6} = ۹۹۴۱۴ \\ -۰,۷۵ \times ۰,۸ \times ۱,۱^۳ \left[۱ + ۳ \left(\frac{۱,۴۶}{۳۰} \right) \left(\frac{۱,۱}{۱,۹} \right)^{۱,۵} \right] \sqrt{\frac{۲ \times ۱۰^۶ \times ۲۳۳۳ \times ۱,۹}{۱,۱}} \\ = ۴۹۳۴۸ \end{cases}$$

① stiffeners are needed / A longer column must be used

② try IPB ۲۴۰ $t_{cf} = ۱,۷cm$ $t_{cw} = ۱,۵۵cm$

$K = ۹,۴cm$ $h = ۱۵,۲cm$ $d = ۲۴cm$

$$P_{bf} \begin{cases} ۸۸,۷ \text{ ton} \\ ۸۴,۸ \text{ ton} \\ ۲۴۱,۵ \text{ ton} \\ ۱۳۸,۲ \text{ ton} \end{cases} > ۴۹ \text{ ton} ✓$$

① required area of stiffeners $= \dfrac{۴۹۰۰۰ - ۴۷۳۷۴}{\underset{\swarrow ۰,۹}{\phi} \ ۲۳۳۳} = ۱۰,۳ cm^۲$
(۲ stiffeness)

min width $= \frac{1}{2} b_f - \frac{t_{cw}}{2} = 10 - \frac{1.1}{2} = \overset{٩٫٤٥}{۶٫۷۳}$ cm

min thickness $= \begin{cases} t_{bf}/2 = ۱٫۴۶/۲ = ۰٫۷۳ \text{ cm} \\ w/15 = \frac{٩٫٤٥}{١٥} = ۰٫۶۳ \text{ cm} \end{cases}$

t_s required $= \frac{\text{area}}{\text{width}} = \frac{۱۰٫۳ \text{ cm}^۲}{٩٫٤٥ \times ۲} = ۰٫۵۴$ cm

take $t_s = ۸$ mm
w = 10cm for each stiffeners

$t_{cf} = ۱٫۹$ cm $\longrightarrow a_{min} = 6$ mm $\rightarrow t_s = 8$ mm OK✓
(weld)

Stiffener length:
- Symetric Connection: $L = d_c - 2t_{cf} = 30 - 2 \times 1.9 = 26.2$ cm
- Single beam connection: $L = \frac{d_c}{2} - t_{cf} = \frac{30}{2} - 1.9 = 13.1$ cm

take L = 14 cm

weld a = 6 mm

required weld length $= \frac{(69000 - 47374)/2}{0.75 \times 0.6 \times 4830 \times 0.707 \times 0.6} = 11.73$ cm each stiffener

provided weld length:
stiffener to flange: 10×2 = 20 cm
" " web: 14×2 = 28 cm

Result: 2×PL 14×10×0.8 cm
$L_w = 48$ cm
a = 6 mm
E70

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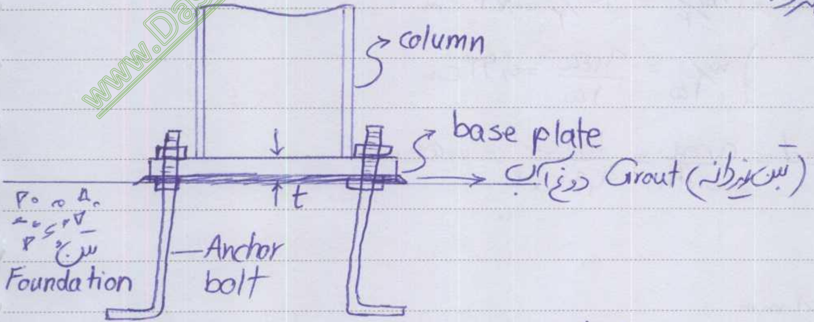
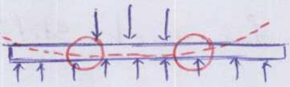
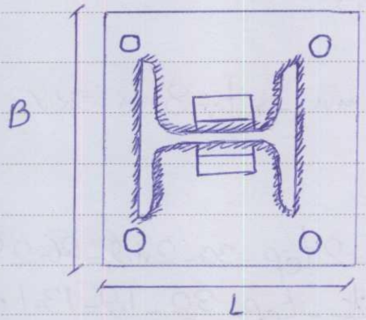
Base Plates: طراحی کف ستون‌ها

* چون مقاومت فولاد از بتن بیشتر است پس بتن خرد می‌شود پس باید بار را باید جوری به کف ستون پخش کرد.







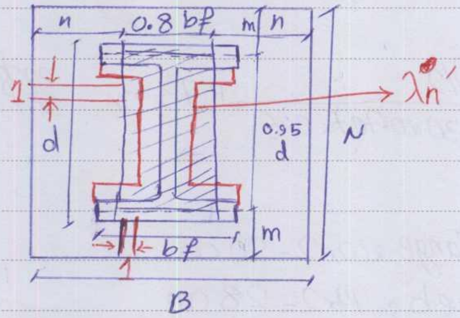


*Plate Area (BN)

base plate design strength $= \varphi_c(0.85f'_c)\sqrt{\frac{A_2}{A_1}}A_1$, $A_1 \geq \frac{P_u}{\varphi_c(0.85f'_c)\sqrt{\frac{A_2}{A_1}}}$

φ_c ↑ 0.6

f'_c ↑ 28-day chlindral test result

A_1 ↖ BN base plate area

$\sqrt{\frac{A_2}{A_1}} \leq 2.0$

let $\Delta = \frac{1}{2}(0.95d - 0.8b_f) \rightarrow N \simeq \sqrt{A_1} + \Delta$

$B \simeq \frac{A_1}{N}$

Stress under base plate: $\frac{P_u}{BN}$

$$\left.\begin{array}{l} \text{in N-dir: } \frac{P_u}{BN} \times m \times \frac{m}{2} \\ \text{in B-dir: } \frac{P_u}{BN} \times n \times \frac{n}{2} \end{array}\right\} M_u = \max\left\{\frac{P_u m^2}{2BN}, \frac{P_u n^2}{2BN}\right\}$$

$\underset{0.9}{\varphi} M_n = \varphi F_y Z$ bending about weak axis

* $Z = \frac{t^2}{4} \Rightarrow \varphi F_y \frac{t^2}{4} \geq \max\left\{\frac{P_u n^2}{2BN}, \frac{P_u m^2}{2BN}\right\}$

$\longrightarrow t \geq \max\{m, n\}\sqrt{\frac{2P_u}{0.9F_y BN}}$

* Code Relation

let $\ell = \max\{m, n, \lambda n'\}$

$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \leq 1$ $\quad X = \frac{4b_f d}{(d+b_f)^2}\,\frac{P_u}{\varphi_c P_p}$

$\varphi_c P_p = \varphi_c(0.85 f'_c) A_1 \sqrt{\frac{A_2}{A_1}}$

$n' = \frac{\sqrt{d b_f}}{4}$

Finally: $t = \ell\sqrt{\frac{2P_u}{0.9F_y BN}}$

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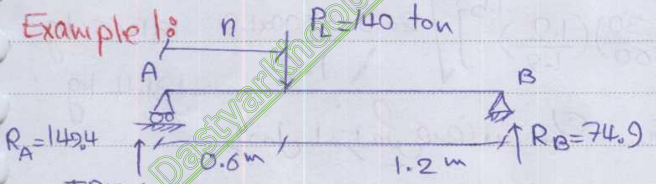
① TA فولاد II

Example 1: حلسه اول



IPE 600

$l_b=N=20\,cm \longrightarrow$ left support

under concentrated load $N=30$ cm

$F_y=3500\ \frac{kg}{cm^2}$

$h=d-2c$, $c \to k$

Shear check:

$C_v=1$

$\varphi_v=1.0$

$$\frac{h}{t_w}=\frac{51.4}{1.2}=42.8 \leq 2.24 \times \sqrt{\frac{E}{F_y}}=54.1$$

O.K.

$$\varphi V_n=1.0\times 0.6\,F_y\times A_w=0.6\times 3500\times(\underset{d}{60}\times \underset{t_w}{1.2})=151.2>149.4$$

Web yielding @ support

$k=4.3$

$N=20$ cm $\longrightarrow N>k$ O.K. ✓

$\varphi=1.0 \longrightarrow \varphi R_n=F_{yw}\cdot t_w\,(2.5k+N)=3500\times 1.2\,(2.5\times 4.3+20)=129950\,kg$

$=130<R_A=149.4$

(N.G)

*under Concentrated load

$k=4.3$

$N=30$ cm $\longrightarrow \varphi R_n=1.0\times 3500\times 1.2\times(5\times 4.3+30)=216.3$

$P_u=1.6\times 140=224>\varphi R_n \longrightarrow$ N.G.

Web local crippling:

*under concentrated load $n>\frac{d}{2}$

$0.6>\frac{0.6}{2}$ ✓

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②

$$\varphi R_n = 0.75 \times 0.8 \times 1.2^2 \left[1+3\left(\frac{30}{60}\right)\left(\frac{1.2}{1.9}\right)^{1.5}\right]\sqrt{\frac{2\times10^6\times3500\times1.9}{1.2}} = 181366 \text{ kg}$$

$= 181.4$ kg

$\varphi R_n < 1.6\times140 = 224$ N.G. راه حل: افزایش ضخامت جان

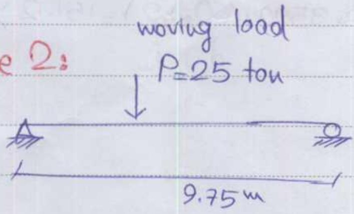
* ⓐ support

$\frac{N}{d} = \frac{0.2}{0.6} = 0.33 \geqslant 0.2$

$$\varphi R_n = 0.75\times0.4\times1.2^2\left[1+\left(\frac{4\times20}{60}-0.2\right)\left(\frac{1.2}{1.9}\right)^{1.5}\right]\sqrt{\frac{2.04\times10^6\times3500\times1.9}{1.2}}$$

$\varphi R_n = 72.1$ ton < 149.4 ton N.G. $\times$

Example 2:



Laterally supported

$F_y = 3500 \ \frac{kg}{cm^2}$

$M_{max} = 1.6\times\frac{25\times9.75}{4} = 97.5$ (t.m)

$\varphi M_n = 0.9 F_y Z = M_u \Rightarrow 0.9\times3500\times Z = 97.5\times10^5$

$\rightarrow Z = 3095 \ cm^3$ is required

IPE 600 $\rightsquigarrow G = 122\left(\frac{kg}{m}\right)$ ✓

IPB 600 $\rightsquigarrow G = 155\left(\frac{kg}{m}\right)$

Use IPE600 $\begin{cases} t_f = 1.9\text{ cm} \\ t_w = 1.2\text{ cm} \end{cases}$

Shear Check

$\varphi_v = 1.0$

$\varphi = 1.0 \longrightarrow \varphi V_n = 0.6 \times 3500 \times (1.2 \times 60) = 151.2$ ton

$V_u = 1.6 \times \frac{25}{2} = 20$ ton $< \varphi V_n$ O.K.

Minimum length of bearing web yelding

$\varphi R_n = 1.0 \times 3500 \times 1.2\,(2.5 \times 4.3 + N) = 4200\,(10.75 + N)$

$R_u = 40$ ton

$4200\,(10.75 + N) = 40 \times 10^3 \longrightarrow N < 0 \Longrightarrow N = k \simeq 5$ cm

web local grippling

if $N = 5$ cm

$\frac{N}{d} = \frac{5}{60} = 0.083 < 0.2$

$$\longrightarrow \varphi R_n = 0.75 \times 0.4 \times 1.2^2 \left[1 + 3\left(\frac{5}{60}\right)\left(\frac{1.2}{1.9}\right)^{1.5}\right] \times \sqrt{\frac{2 \times 10^6 \times 3500 \times 1.9}{1.2}} = 52 > 40 \text{ ton}$$

o.k.✓

دستیارخوب را به دوستانتان معرفی کنید...