1

COLUMNS

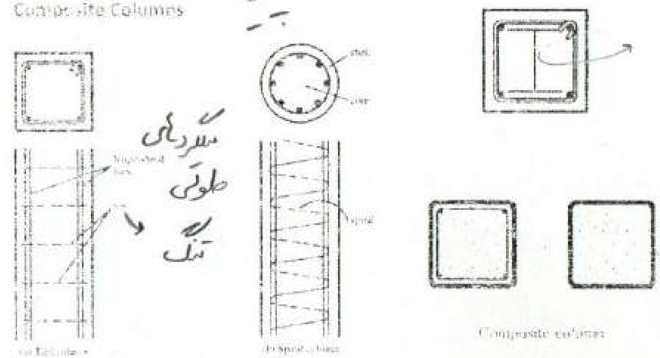
Axial Strength

میلگردها تا $\phi 12$ بصورت roll شده وجود دارد (در بازار)
می توان از آن استفاده کرد

شکل و نوع آرماتورگذاری

Types of Columns (Geometry & Reinforcement)

- Tied Columns
- Spiral Columns → ستون های دورپیچ
- Composite Columns ← بتن و میلگرد + مقطع فولادی مانند قوطی



میلگردهای طولی

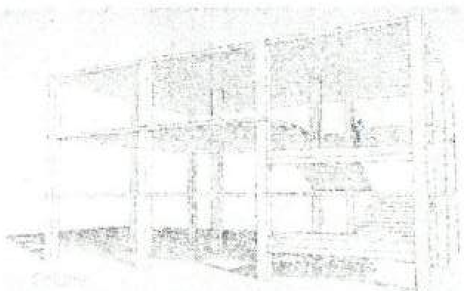
تنگ

I شکل

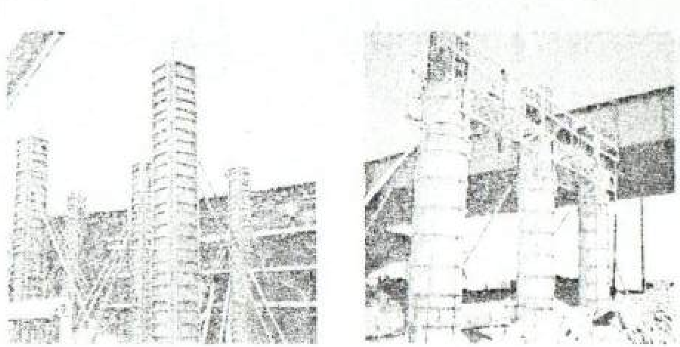
قوطی box
داخلش بتن

Definition

- Columns are vertical structural members supporting axial loads with or without moments.
- Columns transmit loads from the floors to the foundation.



Column Formwork



Rectangular Columns

Round Columns

در عرف ستون ها یا مستطیلی هستند یا دایروی

جاهایی که نمای معماری می خواهیم و یا ستون spiral ی می خواهیم اجرا کنیم (در پایه های پل ها) استفاده می گردد.

در حالت عادی نیروی 'P آستانه‌ای هست که در اثر کمانش دیواره را بشکافد.

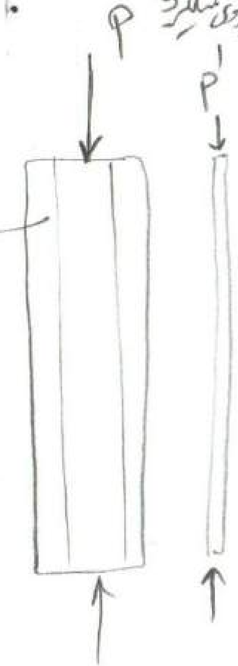
کمانش $P_{cr} = \frac{\pi^2 EI}{L^2}$

نقش tie

tie می‌توانند دایره‌ای هم باشند.

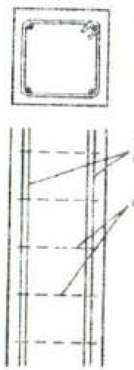
نیروی سوی میلگرد

سوی میلگرد



Tied Columns

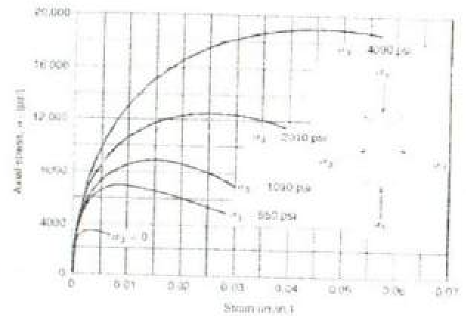
- About 95% of columns in buildings are tied
- What are ... ties?

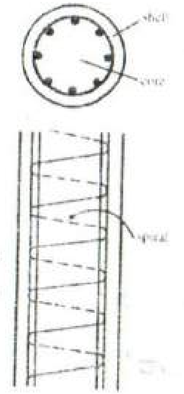


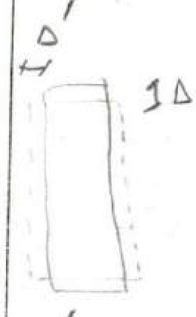
spiral مزیت بزرگ آن این است که جلوی تغییر شکل جانبی را می‌گیرد.

Spiral Columns

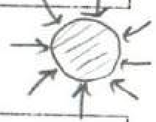
- Pitch generally varies from 35 mm to 100 mm.
- Spiral restrains lateral deformations.







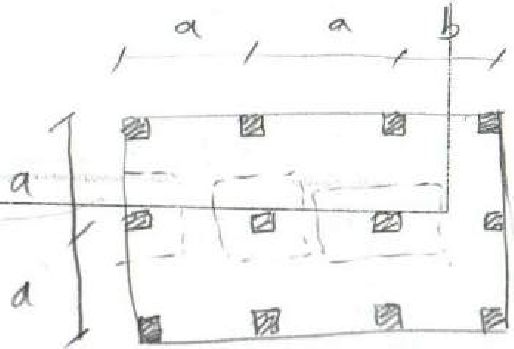
$\frac{\Delta'}{\Delta} = \nu$



spiral روی هسته‌ی بتن اثر confinement ایجاد می‌کند که عامل افزایش مقاومت بتن است.

Do We Really Need Ties for Columns?

- Let's see an experiment.

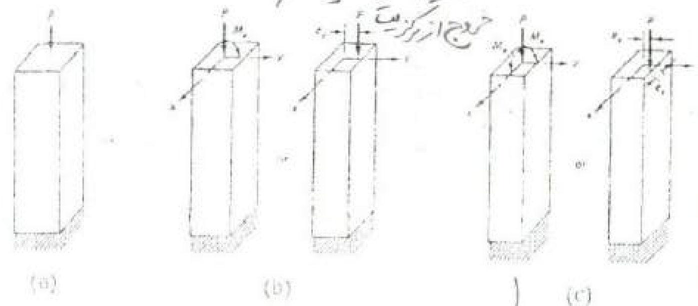


انواع مختلف بارگذاری روی ستون با توجه به چشمه‌ی باربر

Types of Columns (Load Position)

a) Concentrically loaded columns
b) Axial load plus uniaxial moment
c) Axial load plus biaxial moment

تبدیل به یک P با خروج از مرکزیت



خروج از مرکزیت حول دو محور داشته باشیم.

WSD → working stress design

Types of Columns (Slenderness)

- Short (stocky) columns → $\frac{kl}{r}$ کم
  - No concern about buckling
  - secondary moments are negligible.
- Long (slender) columns
  - Buckling and secondary moments must be considered.

لنگرهای ثانویه قابل صرف‌نظر هستند. (اثر P-Δ) در لیمات این اثر قابل صرف‌نظر کردن است.

Secondary moment P-δ

Secondary moment P-Δ

برای ستون‌های بلند این دو تا قابل ملاحظه است.

Elastic Analysis of Axially Loaded Columns (cont.)

- From transformed section analysis

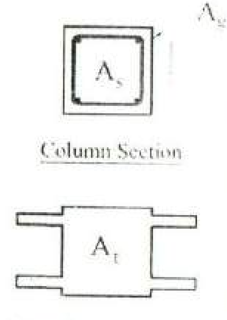
transformed ← از مقطع استفاده می‌شود.

$$A_t = A_g + (n-1)A_s$$

$$n = \frac{E_s}{E_c}$$

$$f_c = \frac{P}{A_t}$$

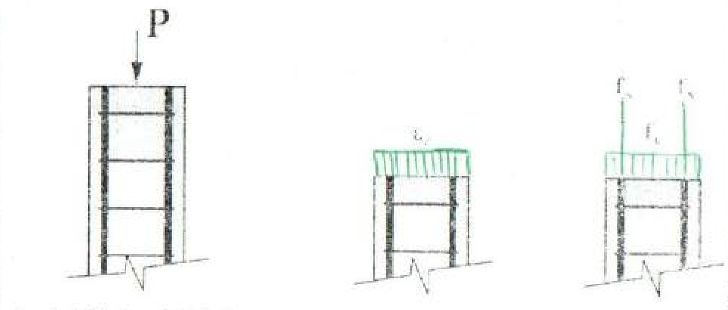
$$f_s = nf_c = n\frac{P}{A_t}$$



- In WSD method

$$f_c \le f_{c,all}$$
$$f_s \le f_{s,all}$$
$\Rightarrow$ *Acceptable Design* (Unreliable!) ← روش غیرقابل اطمینان

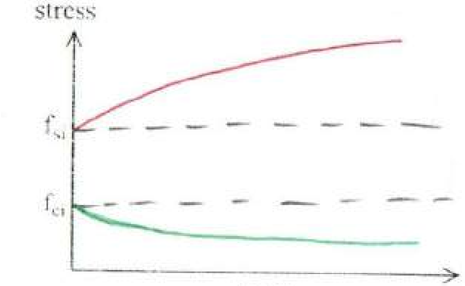
Elastic Analysis of Axially Loaded Columns



An Axially Loaded Column — Strain Profile — Stress Profile

Elastic Analysis of Axially Loaded Columns (cont.)

- The stress level in both concrete and steel changes with respect to time. (Why?)



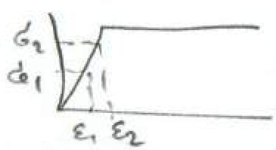
در واقعیت سطح تنش در فولاد و بتن در طول زمان تغییر می‌کند و تنش بتن کم می‌شود و فولاد زیاد می‌گردد.

- Since 1940, all reinforced concrete columns are designed by the "strength" method.

برای ستون‌ها نمی‌توان از آنالیز الاستیک استفاده کرد.

دو علت دارد ۱- خزش ۲- shrinkage

در اثر خزش ← $\frac{1}{\Delta}$ ، ۱۵٪

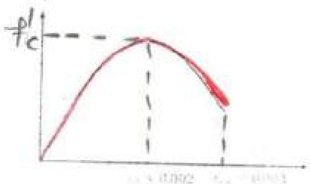


تنش در فولاد زیاد می‌گردد ← به دلیل تعادل در بتن تنش کم می‌شود چون جمع آن ثابت است.

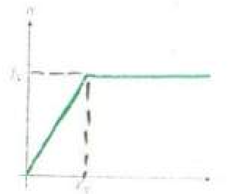
اثر سایز است به دلیل آنکه در ستون های بزرگ به خود f'_c (مقاومت) نمی رسیم بلکه به $0.85f'_c$ می رسیم.

Analysis Assumptions

- Strain varies linearly across the section.
- Strain-stress compatibilities
- Concrete crushes at $\varepsilon_{cu} = 0.003$
- No slippage between concrete and steel.
- Concrete has no tensile strength.



Strain-stress diagram of concrete



Strain-stress diagram of steel

Nominal Capacity of Concentrically Loaded Short Columns

$$C_c = 0.85f'_c(A_g - A_{st})$$

$0.85f'_c$ or f'_c?

$$C_s = A_{st} f_y$$

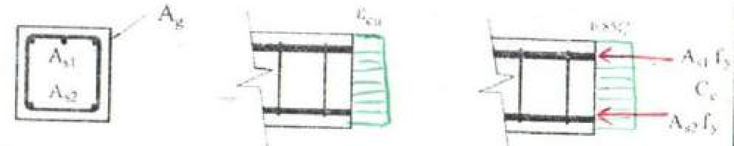
where $A_{st} = A_{s1} + A_{s2}$, i.e. the total area of longitudinal reinforcement.

From equilibrium, we have

Why P_{n0} not P_n?

$$P_{n0} = C_c + C_s$$

$$\Rightarrow P_{n0} = 0.85f'_c(A_g - A_{st}) + A_{st}f_y$$



Column Section

Strain

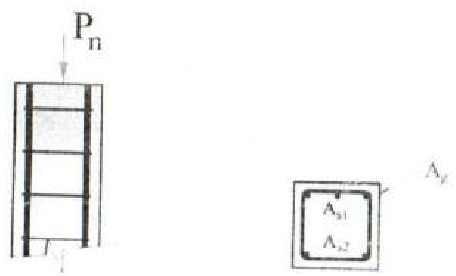
Internal Forces

P_{n0} ← یعنی بار دقیقاً concentric است

Nominal Capacity of Concentrically Loaded Short Columns

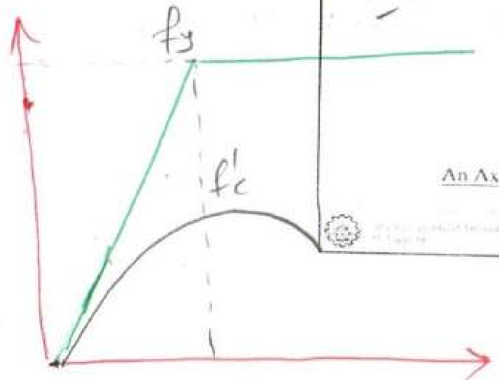
- Consider a reinforced concrete column subjected to a concentric axial load. Let's find the nominal axial capacity of this column.

← مقاومت اسمی ، مقاومتی که در آزمایشگاه ستون تحت بار به ما می دهد را مقاومت اسمی گویند.



An Axially Loaded Column

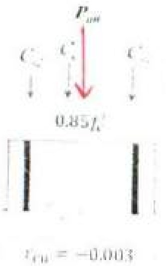
Column Section



Plastic Centroid of Column Section

- The point in the column cross section through which the resultant column load must pass to produce uniform strain at failure.
- For symmetric sections, the plastic centroid coincides with the centroid of the column cross section.

plastic centroid جایی است که اگر بار را در آن قرار دهیم به ε 0.003 می رسیم.



Symmetric Section

Asymmetric Section

Effects of Accidental Eccentricity

- To account accidental eccentricities, the maximum nominal axial capacity is defined as a factor of P_{n0}.

- For tied columns ($e = 0.1h$)

$$P_{n,max} = 0.8P_{n0}$$

- For spiral columns ($e = 0.05h$) → معادل eccentricity ٪5 بُعد مقطع است.

$$P_{n,max} = 0.85P_{n0}$$

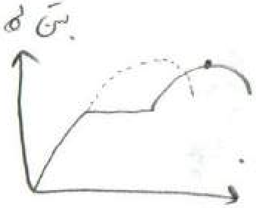
Question

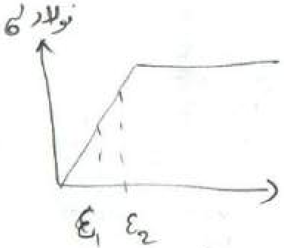
Although we showed that both creep and shrinkage phenomena change the stress levels of concrete and steel, we did not consider their effects in the calculation of P_{n0}. Why?

creep روی مقاومت نهایی بتن تاثیر بدی نمی گذارد

زیرا طبق نمودار رو به رو بتن در اثر خزش تاثیر روی مقاومت نهایی اش نیست.

و مقاومت نهایی میلگرد نیز همان f_y خواهد بود و بیشتر نمی شود.





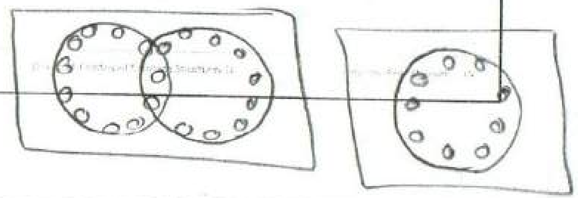
ممکن است در اثر خزش میلگرد yield کند

Reinforcement Requirements for Columns (cont.)

2- Minimum Number of Longitudinal Bars

- $n = 3$ for triangular tied columns → دایره ای تنگ دار
- $n = 4$ for rectangular or circular tied columns
- $n = 6$ for spiral columns

- Can we have spiral columns with rectangular cross sections? → میتوان هسته را به صورت دایره ای قرار داد و چید و کار مستطیلی است



Reinforcement Requirements for Columns

1- Longitudinal Steel Ratio, ρ_g

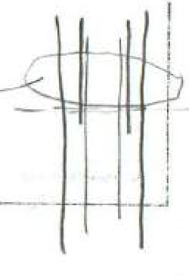
$$\rho_g = \frac{A_{st}}{A_g}$$

$1\% \leq \rho_g \leq 8\%$ → درجاهایی که میلگردها را داریم lap می کنیم این مقدار به ٪4 تقلیل پیدا می کند.

- Common practical steel percentage

$$1\% \leq \rho_g \leq 3\%$$

- See Clause 10.8.4 of ACI Code.



lap zone ناحیه

Reinforcement Requirements for Columns (cont.)

3- Minimum Clear Spacing Between Longitudinal Bars

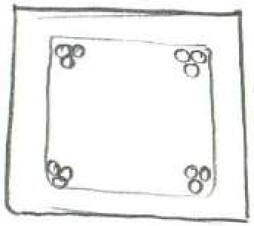
$$s_{min} = \max(40\ mm, 1.5d_b, 1.33D_{max})$$

where

d_b = longitudinal bar diameter, → قطر میلگرد

D_{max} = maximum size of coarse aggregate. → اندازه ی سنگدانه

- The minimum clear spacing must be satisfied at lap zones as well.



Reinforcement Requirements for Columns (cont.)

3- Lateral Ties

خاموت بعنوان مسلح‌کننده برشی و پیچشی هم در ستون کاربرد دارد.

- Size, d_t
  - $d_t \geq 10\ mm$ if $d_b \leq 32\ mm$
  - $d_t \geq 12\ mm$ if $d_b > 32\ mm$
  - $d_t \geq 12\ mm$ if bundled bars used
- Maximum vertical spacing, s_{max}

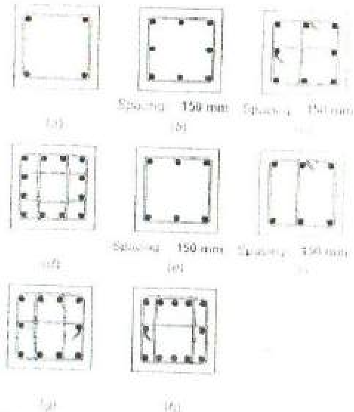
$$S_{max} = min(16d_b, 48d_t, h_{min})$$

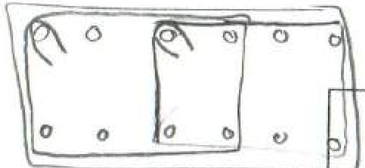
where h_{min} is the least dimension of the column section.

Reinforcement Requirements for Columns (cont.)

3- Lateral Ties

- Typical tie arrangements





Reinforcement Requirements for Columns (cont.)

3- Lateral Ties

- Tie arrangement in section 7.10.5
  - Let's see Clause 7.10.5.3 of ACI Code.

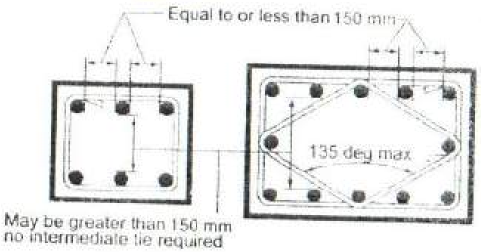


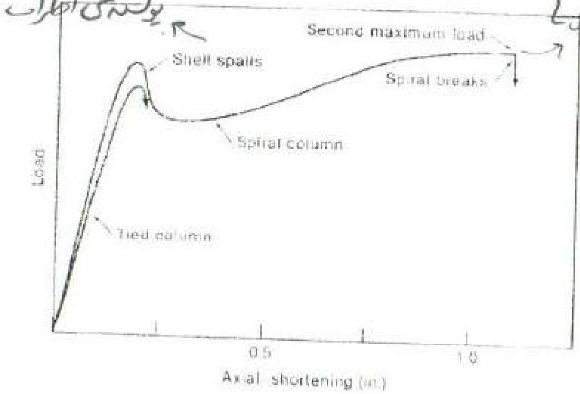
Fig. R7.10.5—Sketch to clarify measurements between laterally supported column bars

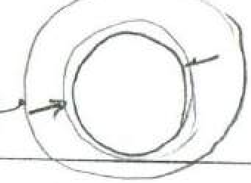
زاویه بیش از 135° با حالت اول خیلی فرقی ندارد

Tied Columns vs. Spiral Columns (Axial Loading)

پوسته‌ی اطراف حداره پوسته می‌کند و می‌ریزد

spiral می‌شکند



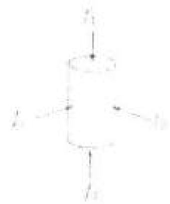


Behavior of Spiral Columns

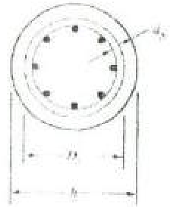
- The compressive strength of a triaxially loaded concrete specimen is

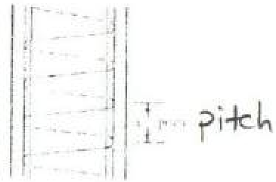
$$f_c^* = f_c' + 4.1 f_2 \qquad (1)$$

$f_c^* = f_c' + 4.1 f_2$



- Consider a spiral column.





Behavior of Spiral Columns (cont.)

- The nominal capacity of whole cross section

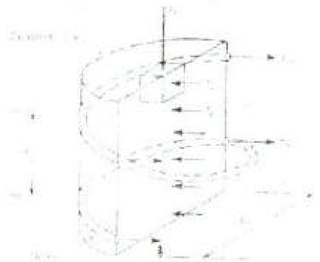
$$P_{n1} = 0.85 f_c'(A_g - A_{st}) + f_y A_{st} \qquad (3)$$

- The nominal capacity after concrete shell spalls

$$P_{n2} = 0.85 f_c^*(A_{ch} - A_{st}) + f_y A_{st} \qquad (4)$$

$$P_{n1} = P_{n2} \Rightarrow f_c^* = f_c' \frac{A_g - A_{st}}{A_{ch} - A_{st}} \approx f_c' \frac{A_g}{A_{ch}} \qquad (5)$$

Behavior of Spiral Columns (cont.)



نیروی کششی در spiral

$f_2 = \frac{2f_{sp}}{sD_c}$

$f_{sp} = a_s f_{yt}$

$\Longrightarrow f_2 = \frac{2a_s f_{yt}}{sD_c}$

Behavior of Spiral Columns (cont.)

درصد حجمی spiral

- The volumetric spiral reinforcement ratio, ρ_s is defined as

$$\rho_s = \frac{\text{volume of spiral in one turn}}{\text{volume of core}}$$

محیط دایره

$$= \frac{P \times a_s(D_c - d_s)}{\pi D_c^2 s/4} \approx \frac{4a_s}{D_c s} \qquad (6)$$

- The nominal capacity after concrete shell spalls

$$(2) \text{ and } (6) \Rightarrow f_2 = \frac{\rho_s f_{yt}}{2} \qquad (7)$$

$$(1), (5), \text{ and } (7) \Rightarrow \rho_s \approx 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\left(\frac{f_c'}{f_{yt}}\right) \qquad (8)$$

Requirements for Spiral Columns

- Size, d_s

$$d_s \geq 10\ mm$$

- Clear spacing between successive turns, s_c

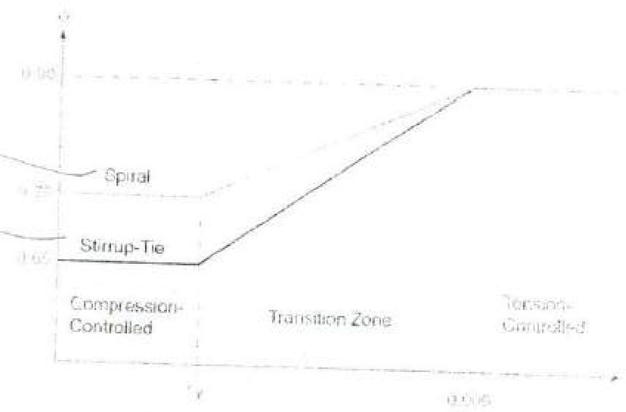
$$\max(25\ mm, 1.33\ D_{agg}) \leq s_c \leq 75\ mm$$

- Minimum volumetric spiral reinforcement ratio, $\rho_{s,min}$

$$\rho_{s,min} = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\left(\frac{f'_c}{f_{yt}}\right)$$

Strength Reduction Factor, ϕ

نوع ستون



الان بتن در ستون در این محدوده می‌شکند.

2

Short Columns

Design for Axial Loads

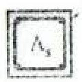
Interaction Diagram

Design of Axially Loaded Columns: General Concept

- For a satisfactory design

$$P_u \le \phi P_n$$

- Design strength, ϕP_n



Column Section

$$\phi P_n = \phi r[0.85 f'_c(A_g - A_{st}) + A_{st} f_y] \ge P_u$$

where r is the reduction factor due to accidental eccentricity, $r = 0.8$ for tied columns, and $r = 0.85$ for spiral column.

مساحت میلگردهای طولی

- Introducing $\rho_g = A_{st}/A_g$, we can rewrite ϕP_n as

$$\phi P_n = \phi r A_g[0.85 f'_c + \rho(f_y - 0.85 f'_c)] \ge P_u$$

Design of Axially Loaded Columns: Known Dimensions

- Given
  - P_u
  - Material properties, f'_c and f_y
  - Type of lateral reinforcement, tied or spiral
  - Cross sectional area, A_g

Column Section

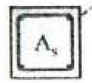
- Wanted مساحت میلگردهای طولی مورد نیاز
  - A_{st} →
  - Details of reinforcement

$$A_{st} \ge \frac{1}{(f_y - 0.85 f'_c)}\left[\frac{P_u}{\phi r} - 0.85 f'_c A_g\right]$$

un

Design of Axially Loaded Columns: Known Dimensions

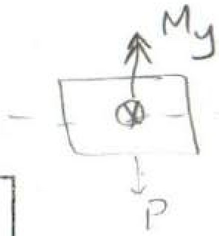
- Given
  - P_u
  - Material properties, f'_c and f_y
  - Type of lateral reinforcement, tied or spiral نوع میلگرد
  - ρ_g



طولی Column Section

- Wanted
  - A_g
  - Details of reinforcement → جزئیات آرماتورگذاری

$$A_g \ge \frac{P_u}{\phi r[0.85 f'_c + \rho_g(f_y - 0.85 f'_c)]}$$



$$\sigma_{max} = \frac{P}{A} \pm \frac{M_y C}{I}$$

Example 1- Design of Tied Columns

- Design a tied rectangular column subjected to the following axial loads:
  - $P_D = 100\ tons,$
  - $P_L = 70\ tons,$
  - $P_W = \pm 30\ tons,$
  - $P_E = \pm 110\ tons.$

{ + فشار
− کشش }

The architectural aspects limits the width of the column cross section to 400 mm. Take C25 concrete and grade AIII reinforcement. The earthquake load is extracted from the 4th edition of Iranian seismic code 2800.

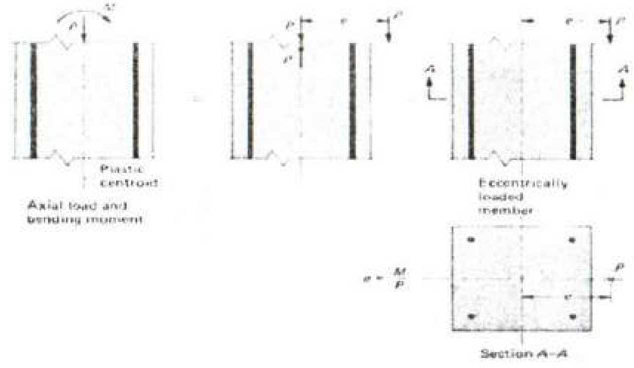
a) Use the minimum reinforcement, ضریب زلزله همان ۱ است.

b) Consider a square cross section.

طبق آیین نامه P_{min} **(Left to you)**

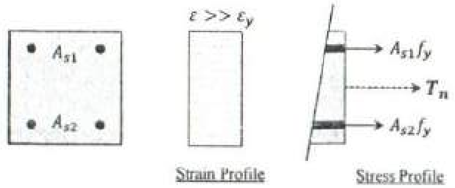
ستون مربعی

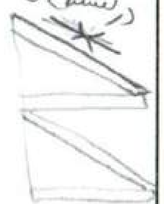
Moment & Load Eccentricity



Pure Tension Capacity

- Concrete is cracked and does not provide any tensile strength.
- Uniform tensile strain all over the section.
- All reinforcement yields.



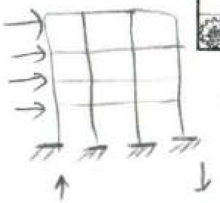


در کشش می افتد.

- Nominal Pure Tension Capacity

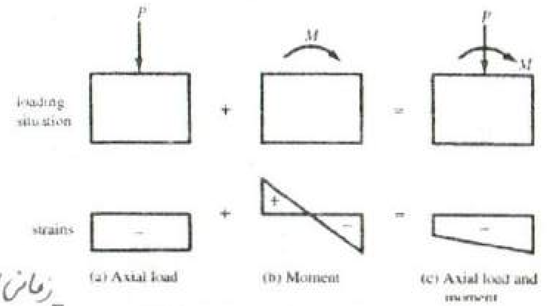
$$T_n = A_{st} f_y$$

- When might we have tensile forces in RC members?



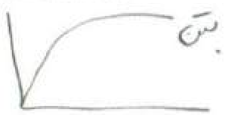
بارجانبی باد یا زلزله تعدادی ستون را در فشار و تعدادی را در کشش می اندازد.

Strain Profile of Columns under Eccentric Axial Load

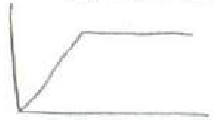


- Is this superposition technique valid for finding stress distribution across the column section?

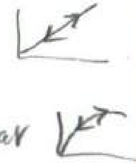
زمانی از برهم نهی می توان استفاده کرد که رفتار ماده خطی باشد چون تنش در فولاد و بتن خطی نیست نمی توان باهم جمع کرد.

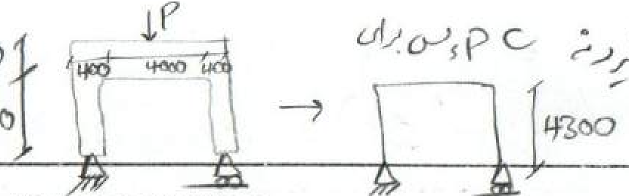


بتن



Material < Elastic < linear / Non-linear





plastic centroid ← چون در تحلیل سازه‌ای، تحلیل بر اساس centroid مقطع صورت می‌گیرد، پس برای PC

برهم‌نهی تنش‌های خمشی و ... از همین

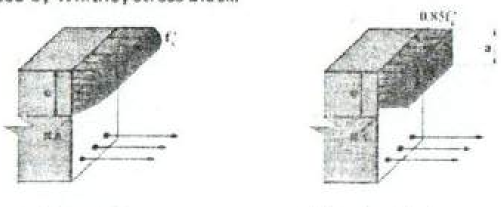
اندرکنش لنگر

چرا از CG است نه PC

center of Gravity

Analysis Assumptions

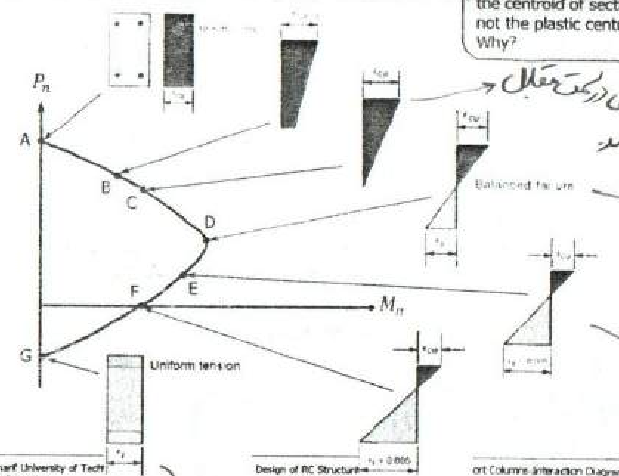
- Strain varies linearly across the section.
- Strain-stress compatibilities
- Concrete crushes at $\varepsilon_{cu} = 0.003$.
- No slippage between concrete and steel.
- Concrete has no tensile strength.
- Nonlinear stress distribution acting on the concrete can be equivalently replaced by Whitney stress block.



$a = \beta_1 c$

P_n-M_n Interaction Diagram

$M_n = P_n e$, where e is measured from the centroid of section not the plastic centroid. Why?



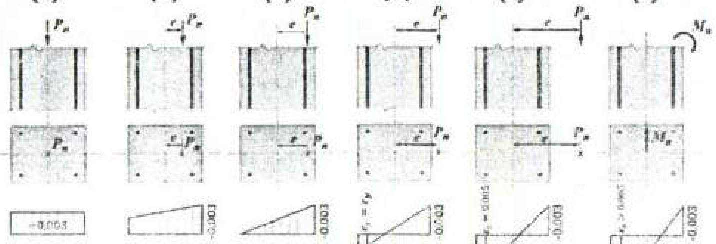
اولین ترک کششی در بتن مقطع به وجود می‌آید

نقطه‌ی بالانس

میلگردها yield می‌کنند

مقطع تحت کشش خالص

Behavior of Column Section under Axial Load & Bending

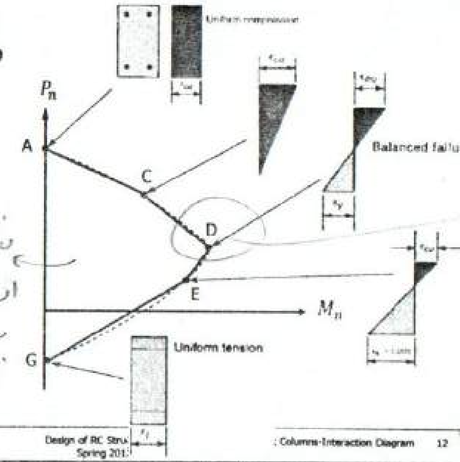


(A)	(B)	(C)	(D)	(E)	(F)
Large concentric axial load	Large axial load	Large axial load	Balanced loading condition	Large moment	Large moment
Zero eccentricity	Small moment	Moment larger than B	Reinforcement yields in tension at the same time as concrete crushes	No appreciable axial load	Infinite eccentricity
Uniform strain	Concrete crushes	Concrete crushes		Failure initiated by yielding of steel bars	Failure occurs as in beams
Concrete crushes	All bars in compression	Concrete cracks on tension side			
Reinforcement yields in compression		All bars in compression			

کرنش‌ها خطی است ولی تنش نه در این حالت کرنش خطی را با خروج از کرنش‌های مختلف بررسی می‌کنیم.

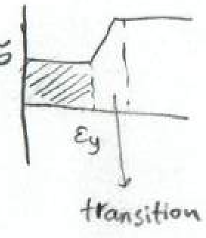
Essential Points for Construction of P_n-M_n Interaction Diagram

- Pure Compression, A
- First Tension Crack, C
- Balanced Condition, D
- $\varepsilon_t = 0.005$, E
- Pure Tension, G



نقطه‌ای که داخل diagram است OK است ستون fail می‌کند.

نقطه‌ی بالانس حائز اهمیت است زیرا φ مقطع از آنجا به بعد زیاد می‌شود.



حالت بالانس که همزمان با fail کردن بتن میلگردها هم yield می‌کنند.

مقطع در برابر شکست نرم قرار دارد و میلگردها زودتر از بتن yield می‌کنند.

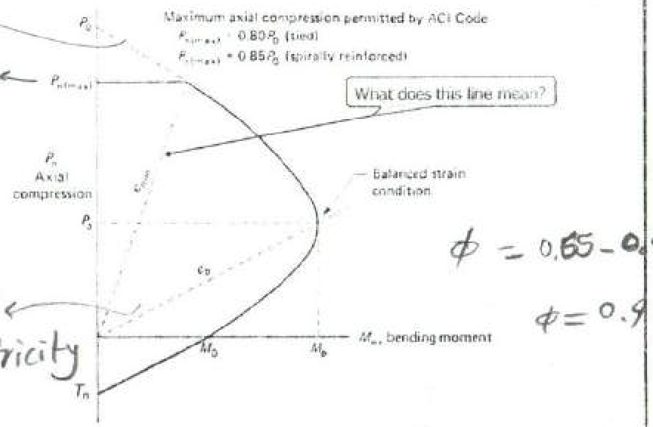
آنقدر خروج از مرکزیت زیاد می‌شود که عملاً خمش داریم و دقیقاً مثل تیر طوری عمل می‌کند.

چون ستون بررسی می‌کنیم ماهیت ستون کلاً ترد شکل است پس این محدودیت را نداریم.

Maximum Axial Compression Permitted by ACI Code, $P_{n,max}$

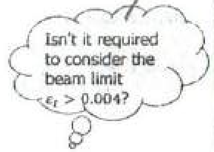
بعد از دستی خوردن

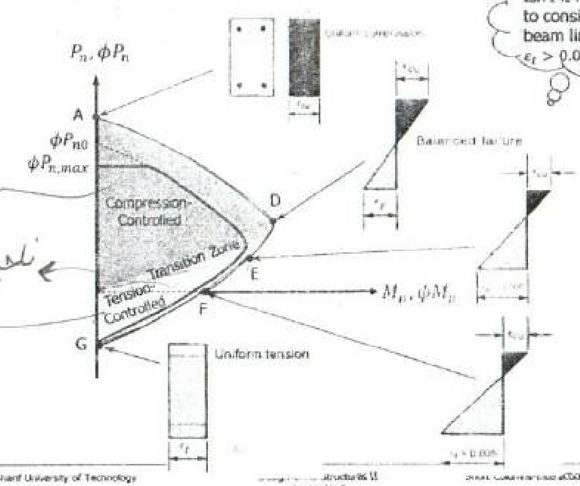
حد max ای که برای فشار در نظر می‌گیریم.



شیب این خط معکوس eccentricity است یا $\frac{1}{e}$

$\phi P_n - \phi M_n$ Interaction Diagram



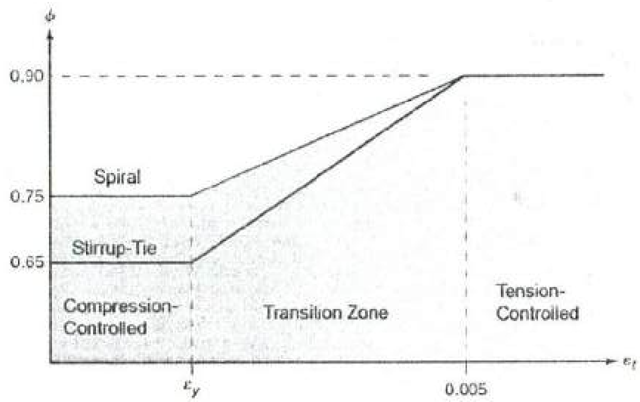


$\phi = 0.65$

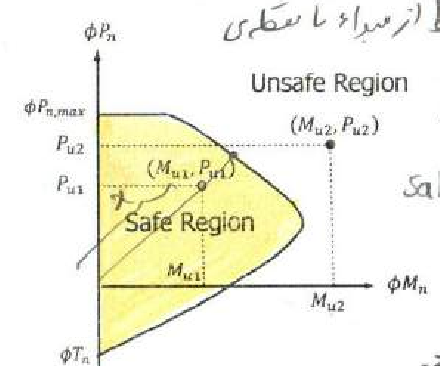
$\phi = 0.65 - 0.9$ بین transition ناحیه

$\phi = 0.9$ tension controlled ناحیه

Strength Reduction Factor



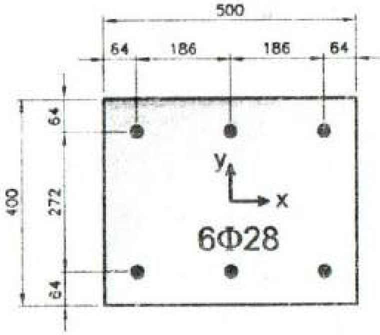
How to Check Adequacy of a Column by Using Interaction Diagram?



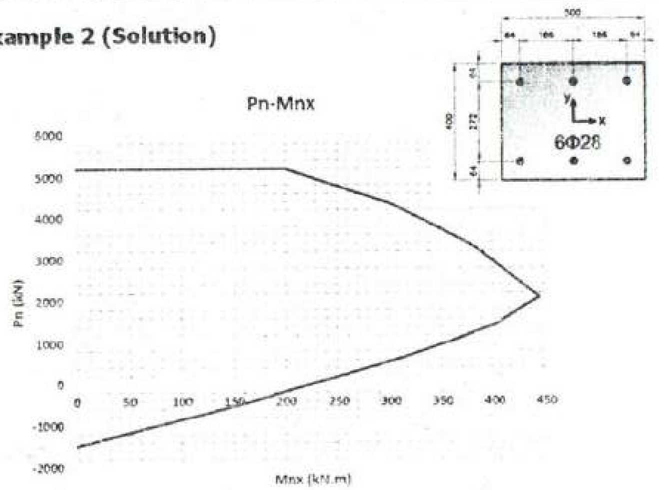
برای مشخص کردن safety یک خط از مبداء تا نقطه (M_{u1}, P_{u1}) رسم می‌کنند پس نسبت x به طول خط برابر میزان safety است و هرچه به ۱ نزدیک‌تر باشد به Failure نزدیک‌تر می‌شود.

Example 2- Nominal Interaction Diagram $P_n - M_n$

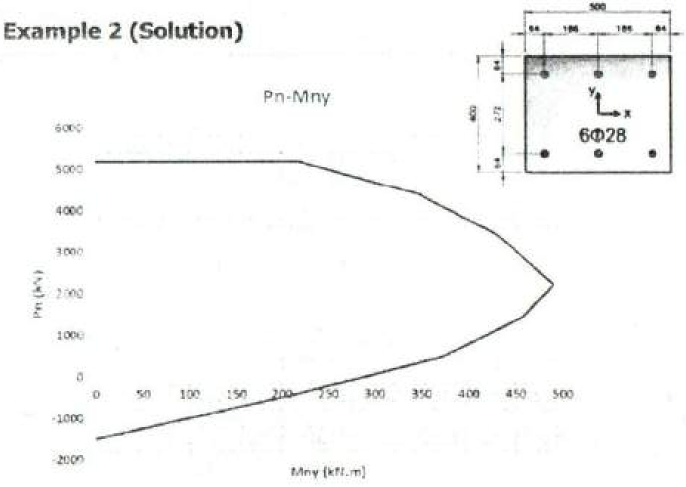
- Construct both $P_n - M_{nx}$ and $P_n - M_{ny}$ interaction diagrams for the given column section. Take C30 concrete and grade AIII reinforcement.



Example 2 (Solution)



Example 2 (Solution)



Example 3- Design Interaction Diagram $\phi P_n - \phi M_n$

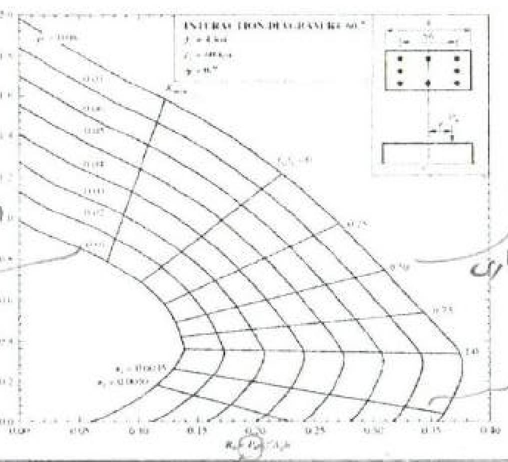
- Apply the strength reduction factor on the interaction diagrams of Example 1. (Left to you)
- Can the given column section support an eccentric ultimate axial compressive load of 300 tons with $e_y = 10$ cm?
- Determine the maximum ultimate axial load that the column can support with an eccentricity $e_x = 10$ cm.

نقاط حساس $\varepsilon = 0.005$ و سایر نقاط را پیدا می‌کنیم.

نقطه را روی نمودار مشخص می‌کنیم داخل است ok است.

خط $e_x = 10$ cm را رسم می‌کنیم می‌بینیم نمودار را کجا قطع کرده است.

Nondimensional Interaction Diagrams



P_n

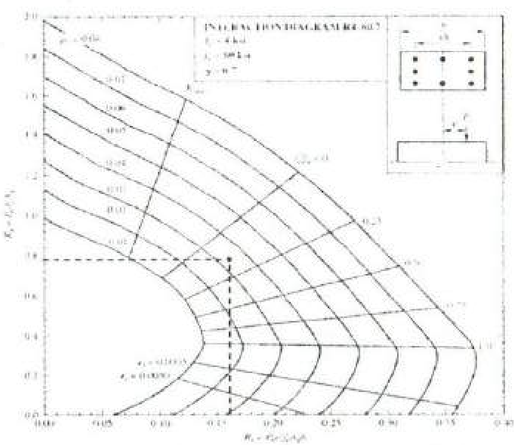
interaction diagram یک به ازای هر ρ را دارد.

برای به کار بردن وصله‌ها از آن استفاده می‌شود که کجا وصله فشاری داریم و کجا کششی

یک خط میانی است به عنوان کمکی استفاده می‌شود

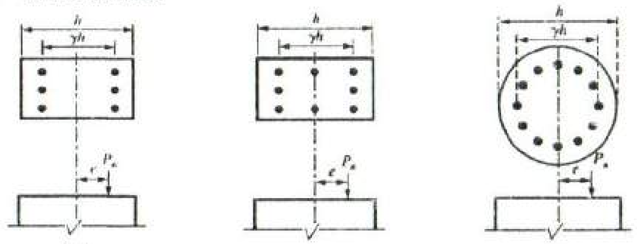
$M_n = P_n e$

Design of Eccentrically Loaded Columns by Interaction Diagrams



Typical Reinforcement Layouts of Rectangular and Circular Columns

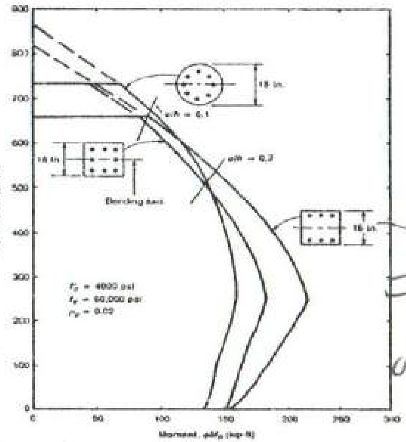
a) Rectangular tied columns with bars on end faces only
b) Rectangular tied columns with bars uniformly distributed on all four faces
c) Circular columns



(a) (b) (c)

Nondimensional interaction diagrams are available for different material properties and $\gamma = 0.6, 0.7, 0.8,$ and 0.9.

Effects of Column Type on Interaction Diagram

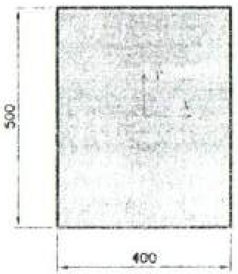


زمانیکه میلگردگذاری‌ها در دو طرف متمرکز است می‌تواند مقاومت بیشتری بدهد ولی در طراحی سازه‌ها توصیه نمی‌شود زیرا افرادی که اجرا می‌کنند خیلی دقت نمی‌کنند ممکن است به جای ، را اجرا کنند که خطرناک است!

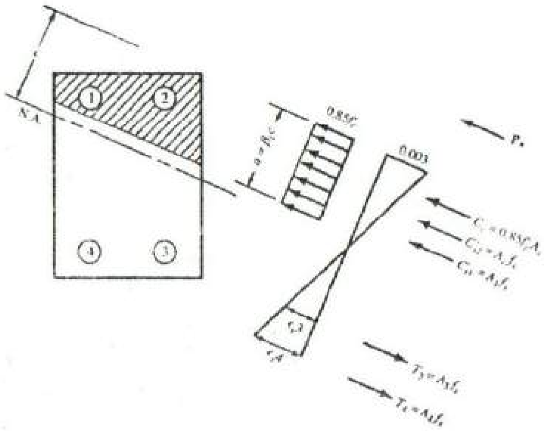
Example 4- Design of an Eccentrically Loaded Column

- A short 400x500 mm column is subjected to the following loads and moments:
  - P_D=800 kN,
  - P_L=1150 kN,
  - M_{xD}=70 kN.m, and
  - M_{xL}=120 kN.m.

For $f_c' = 28$ MPa, and $f_y = 420$ MPa, select the reinforcing bars to be placed at its end faces only.



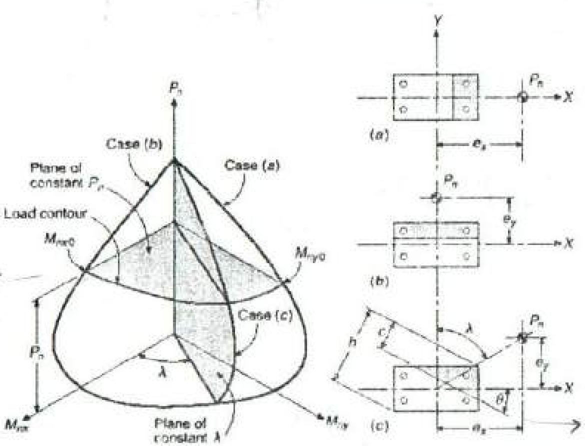
Analysis Procedure of Columns under Biaxial Bending



یک روش تقریبی داریم که با داشتن ۳ نقطه از نمودار ۳ بعدی بتوان شکل ۳ بعدی را رسم کرد ← روش معکوس برسلر

Biaxial Bending

خمش‌های دو محوری M_{ux} و M_{uy}

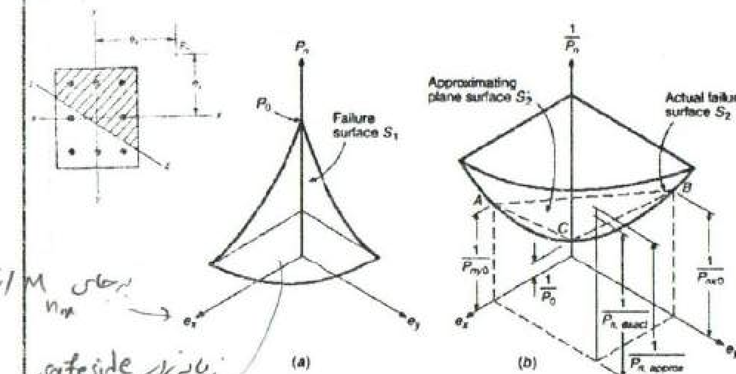


حجم ۳ بعدی فندقی

بجای M از e_x در رسم استفاده می‌شود.

Neutral axic دچار چرخش می‌شود

Bresler's Reciprocal Load Method



زمانی در safeside هستیم که نقاط ما در داخل پوسته و حجم باشند.

$$A = \left(e_x, 0, \frac{1}{P_{ny0}}\right)$$
$$B = \left(0, e_y, \frac{1}{P_{nx0}}\right)$$
$$C = \left(0, 0, \frac{1}{P_{n0}}\right)$$

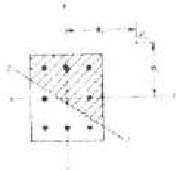
Bresler's Reciprocal Load Method

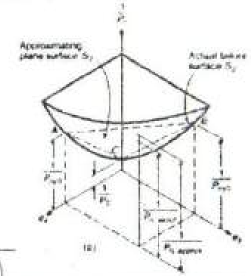
P_{n0} = nominal axial load strength under pure axial compression (corresponds to point C) $M_{nx} = M_{ny} = 0$

P_{nx0} = nominal axial load strength under uniaxial eccentricity e_y (corresponds to point B) $M_{nx} = P_n e_y$

P_{ny0} = nominal axial load strength under uniaxial eccentricity e_x (corresponds to point A) $M_{ny} = P_n e_x$

P_n = nominal axial load strength under biaxial eccentricity, e_x & e_y

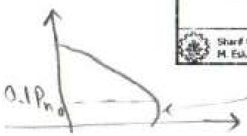




ترسیمی تو را اثبات کنید

$$\frac{1}{P_n} = \frac{1}{P_{nx0}} + \frac{1}{P_{ny0}} - \frac{1}{P_{n0}}$$

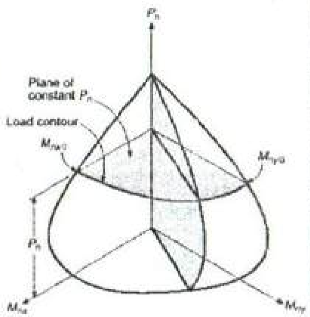
This method is only applicable for $P_n > 0.1P_{n0}$.



چون $0.1P_{n0}$ به بعد تقعر برمی‌گردد

Bresler-Parme Method (cont.)

$$\left(\frac{M_{nx}}{M_{nx0}}\right)^{\frac{\ln 0.5}{\ln \beta}} + \left(\frac{M_{ny}}{M_{ny0}}\right)^{\frac{\ln 0.5}{\ln \beta}} = 1$$



$M_{nx} = P_n e_y$

$M_{ny} = P_n e_x$

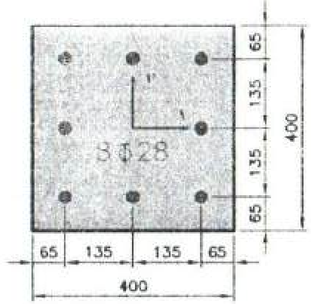
$M_{nx0} = M_{nx}$ when $M_{ny} = 0$

$M_{ny0} = M_{ny}$ when $M_{nx} = 0$

$0.55 < \beta < 0.70$ (Recommended value for design $\boldsymbol{\beta = 0.65}$)

Example 5- Biaxial Moment

- **Determine the adequacy of a 400 mm square column with 8Φ28 bars as shown. The section is to carry loads of P_u=120 tons, M_{ux}=14 ton.m, and M_{uy}=18 ton.m. Take f'_c=28 MPa and f_y=420 MPa. Use both discussed methods.**



Splice of Longitudinal Reinforcement

Compressive	-	
Tensile $f_s \le 0.5f_y$	> 50	Tension Class B
	≤ 50	Tension Class A (staggered by l_d)
Tensile $f_s > 0.5f_y$		Tension Class B

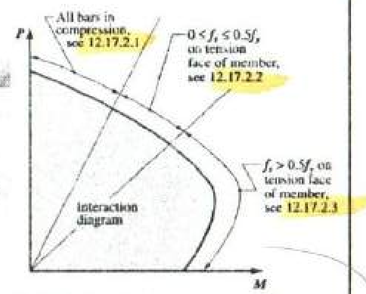
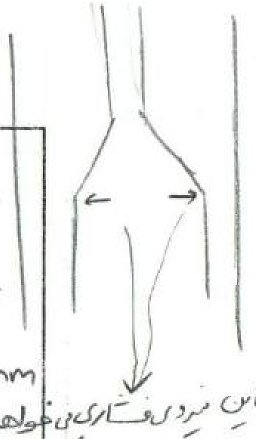


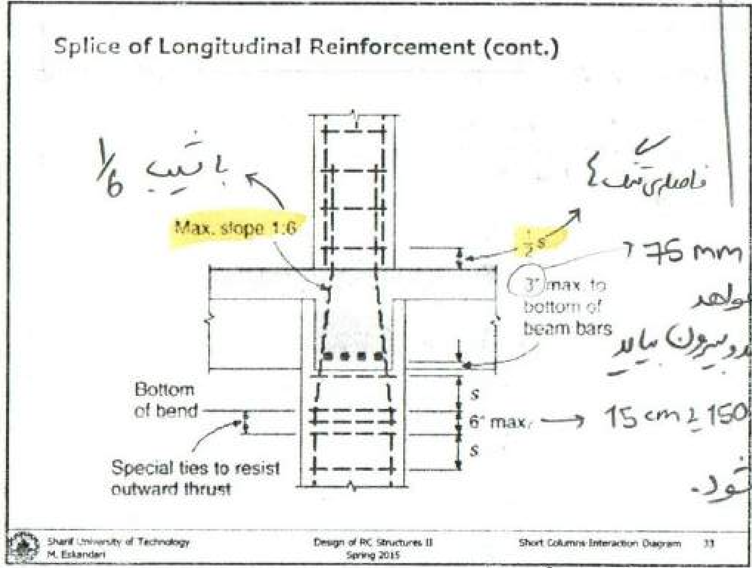
Fig. R12.17—Special splice requirements for columns

نمودار را به ۳ ناحیه تقسیم می‌کند و می‌گوید در هر ناحیه از چه نوع وصله‌ای استفاده گردد.

میلگرد کرنک

cranked Bars





$\frac{1}{6}$ با شیب

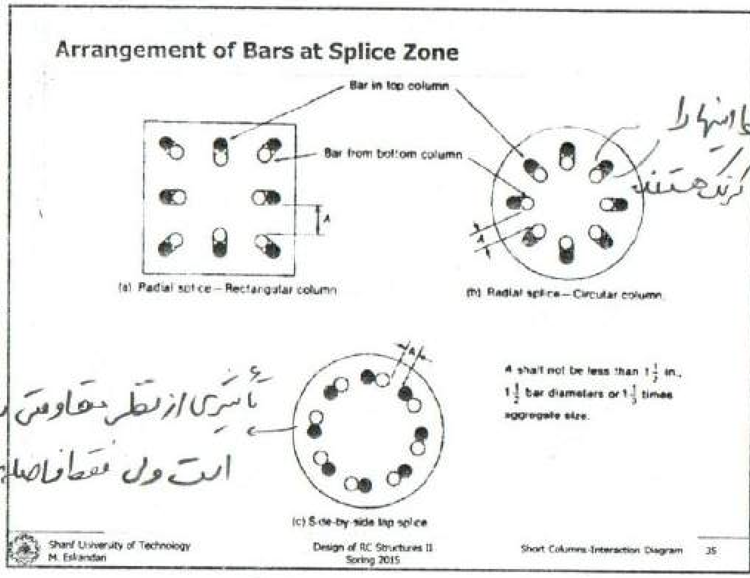
فاصله‌ی تنگ‌ها

> 75 mm

15 cm یا 150 mm

این نیروی فشاری می‌خواهد پوسته‌ی بتن را خورد کند و بیرون بیاید.

با شیب از نظر مقاومتی ندارد و خوب است ولی فقط فاصله‌ی دو آرماتور کم می‌شود.



آرماتورها اینها را بهم وصل کند یعنی کرنک هستند

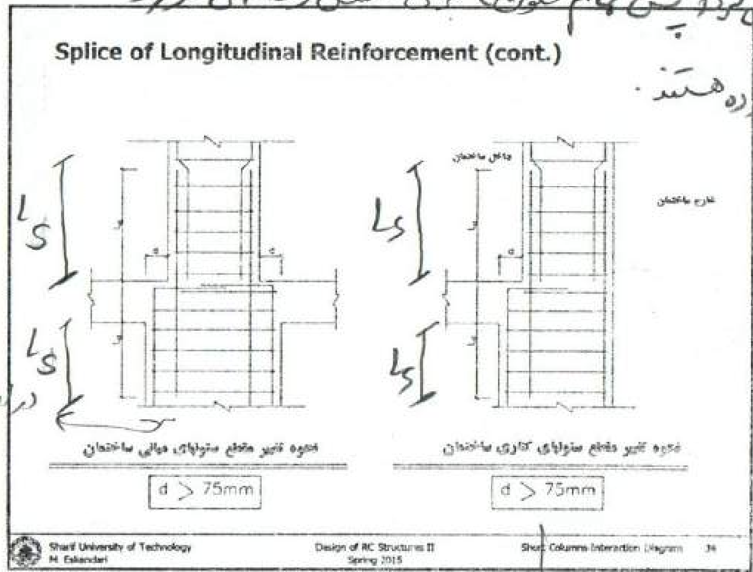
در مقاطع مدور

آرماتورها

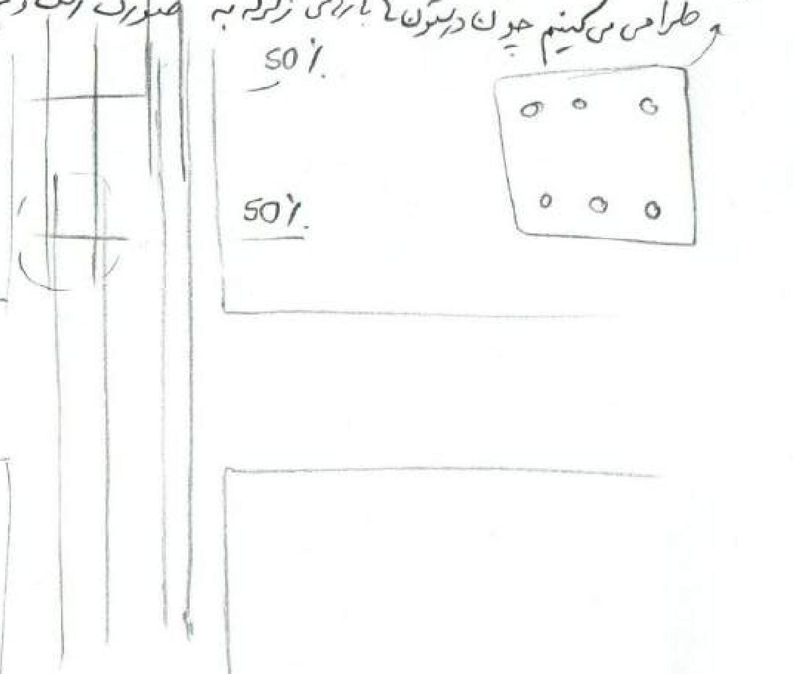
صورت رفت و برگشتی است که باعث تخریب ستون می‌گردد پس تمام ستون‌ها برای کشش وصله می‌خورند.

میلگردهای ستون را همگی برای وصله‌ی کششی (class B) طراحی می‌کنیم چون در ستون‌ها بارهای زلزله به

نصف بقیه میلگردها در این محدوده هستند.



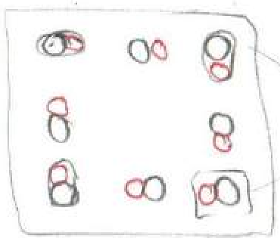
نصف میلگردها در این محدوده وصله می‌شوند.

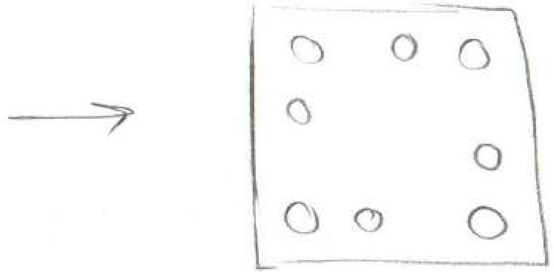


در این محدوده وصله‌ی تماسی داریم.

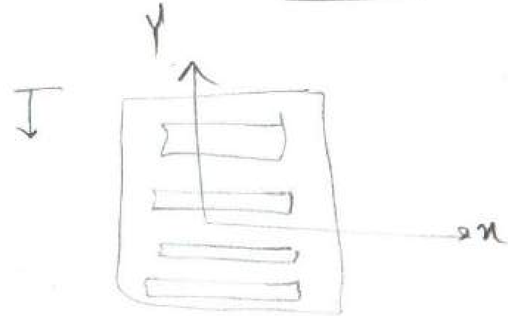
در ستون‌های کناری مقطع سمت کوچک می‌گردد و کوچک شدن از یک سمت رخ می‌دهد.

درمقاطع مستطیلی باید حتماً میلگردهای گوشه را crancked کرد چون خاموت را از دست می‌دهیم.

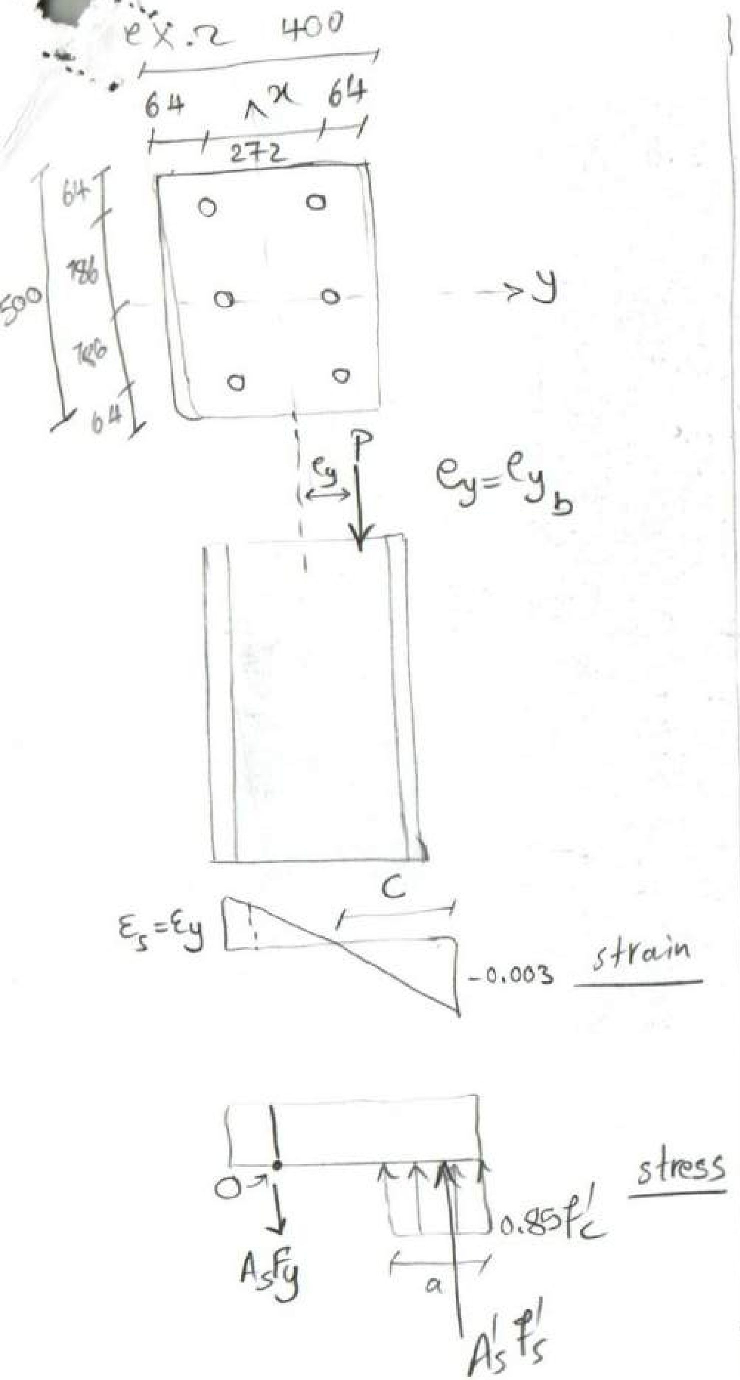




پروژه مقطع ۱



$\Phi P_n - \Phi M_{nx}$



$P_n - M_{nx}$

balanced condition

$\sum F_z = P_n$

Depth of Netural Axis

$$\frac{c}{0.003} = \frac{326-C}{0.002} \rightarrow C = 202 \text{ mm}$$

$\beta_1 = 0.84$

$a = \beta_1 C = (0.84)(202) = 167$

$$\frac{\varepsilon'_s}{c-64} = \frac{0.003}{c} \rightarrow \varepsilon'_s = 0.002 = \varepsilon_y \rightarrow f'_s = f_y$$

$\sum F_z = P_n$

$$\rightarrow P_n = (0.85)\underbrace{(30)}_{f'_c}\underbrace{(167)}_{a}\underbrace{(500)}_{b} + \overbrace{3 \times \frac{\pi}{4} \times (28)^2}^{A_s}\overbrace{(400)}^{f_y} - 3\left(\frac{\pi}{4}\right)(28)^2(400) \rightarrow P_n = 2130 \text{ kN}$$

$\sum M_o = M_n$

$$M_n = (0.85)(30)(167)(500)\left(336 - \frac{167}{2}\right) + 3 \times \frac{\pi}{4} \times 28^2 \times 400\,(336-64) - P_n(200-64)$$

$= 449$

چون لنگر را حول o گرفتیم نه مرکز سطح وسط
مقطع باید لنگر P_n حول o را از آن کم کنیم

ex 4.

$P_u = 1.2D + 1.6P_L = (1.2)(800) + (1.6)(1150) = 2800$ kN

$M_{ux} = 1.2 M_{Dx} + 1.6 M_{Lx} = (1.2)(70) + (1.6)(120) = 276$ kN.m

$$K_n = \frac{P_u}{\phi A_g f'_c}$$

$$= \frac{2800 \times 10^3}{(0.65)(400)(500)(28)} = 0.77$$

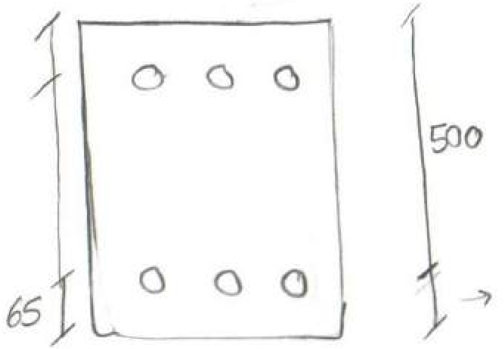
$$\frac{e}{h} = \frac{M_u}{P_u h} = \frac{276}{(2800)(0.5)} = 0.2 \longrightarrow \text{Assume } \phi = 65$$

ستون tied است و

در ناحیه هستیم.

$$R_n = K_n \frac{e}{h} = 0.77 \times 0.2 = 0.15$$

$$\begin{cases} K_n = 0.77 \\ R_n = 0.15 \end{cases}$$



$40 + 10 + 30/2$

$\rightarrow cover + d_t + d_b/2 = 65$ mm

$$\gamma = \frac{500 - 2(65)}{500} = 0.74$$

$\gamma = 0.7 \longrightarrow \rho = 2.6\ \%$

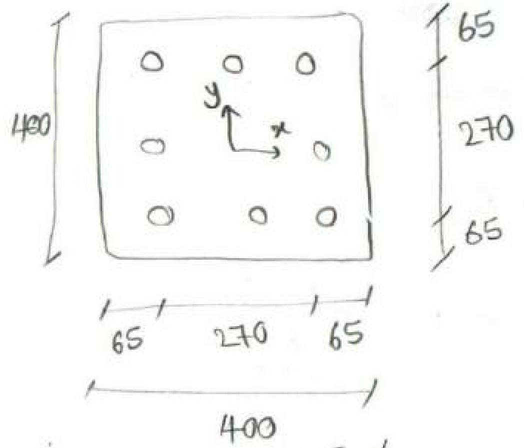
$A_{st} = \rho A_g = \ldots$

بعدش جانمون گذاری را خودمان

OK می‌کنیم.

① روش برسلر Bresler's

$$\begin{cases} P_u = 120 \text{ ton} \\ M_{ux} = 14 \text{ t.m} \\ M_{uy} = 18 \text{ t.m} \end{cases}$$

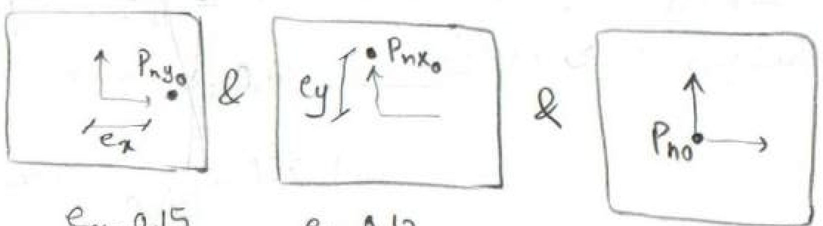


$\boxed{\gamma = 0.675}$

باید ابتدا یک تخمین از eccentricity داشته باشیم که بتوانیم عددی بدانیم ϕ
تقریبا در کجای کار است.

$$M_u = \sqrt{M_{ux}^2 + M_{uy}^2} =$$

$$\begin{cases} e_y = \dfrac{M_{ux}}{P_u} = \dfrac{14}{120} = 0.12 \text{ m} \\ e_x = \dfrac{M_{uy}}{P_u} = \dfrac{18}{120} = 0.15 \text{ m} \end{cases}$$



$e_x = 0.15$ $\quad$ $e_y = 0.12$ $\quad$ $K_{no} = 1.3$

$\dfrac{e_x}{h} = \dfrac{0.15}{0.4} = 0.38$ $\quad$ $\dfrac{e_y}{h} = \dfrac{0.12}{0.4} = 0.3$

از اعتدار خطا 0.03 تا صور K_y
مقدار K_{y_0} بدست می‌آید.

$K_{nxo} = 0.47$ $\quad$ $K_{nyo} = 0.56$

$$\left(\frac{1}{P_n} = \frac{1}{P_{nxo}} + \frac{1}{P_{nyo}} - \frac{1}{P_{no}}\right) \times (A_g f'_c)$$

$$\hookrightarrow \frac{1}{K_n} = \frac{1}{K_{nxo}} + \frac{1}{K_{nyo}} - \frac{1}{K_{no}}$$

$$\frac{1}{K_n} = \frac{1}{0.56} + \frac{1}{0.47} - \frac{1}{1.3} = 3.14 \longrightarrow K_n = 0.32$$

$P_{n\gamma=0.6} = K_n A_g f'_c = (0.32)(400)^2(28) = 143 \text{ ton}$

$P_{n\gamma=0.7} =$ مانند مراحل بالا می‌رویم و حساب می‌کنیم

$P_{n\gamma=0.675}$

$\longrightarrow P_{n\gamma=0.675} > P_{n\,req} = 189 \text{ ton}$

روش دوم ، Bresler - parme

$M_{nx_0} = ?$ $\quad P_n = \frac{P_u}{\phi} = 189 \text{ ton} \quad K_n = \frac{189 \times 10^4}{(400)^2 (28)} = 0.42$

$\Rightarrow R_{nx} = 0.17 \Rightarrow M_{nx_0} = (0.17)(28)(400)^2 \times 400 = 30.5 \text{ ton.m}$

Due to symmetry $\longrightarrow R_{ny} = 0.17 \quad M_{ny_0} = 30.2 \text{ t.m}$

$$\left(\frac{M_{nx}}{M_{nx_0}}\right)^{1.61} + \left(\frac{M_{ny}}{M_{ny_0}}\right)^{1.61} = \left[\frac{(14)}{(0.65)(30.5)}\right]^{1.6} + \left[\frac{18}{(0.65)(30.5)}\right]^{1.6}$$

$M_{nx} = \frac{M_{ux}}{\phi}$

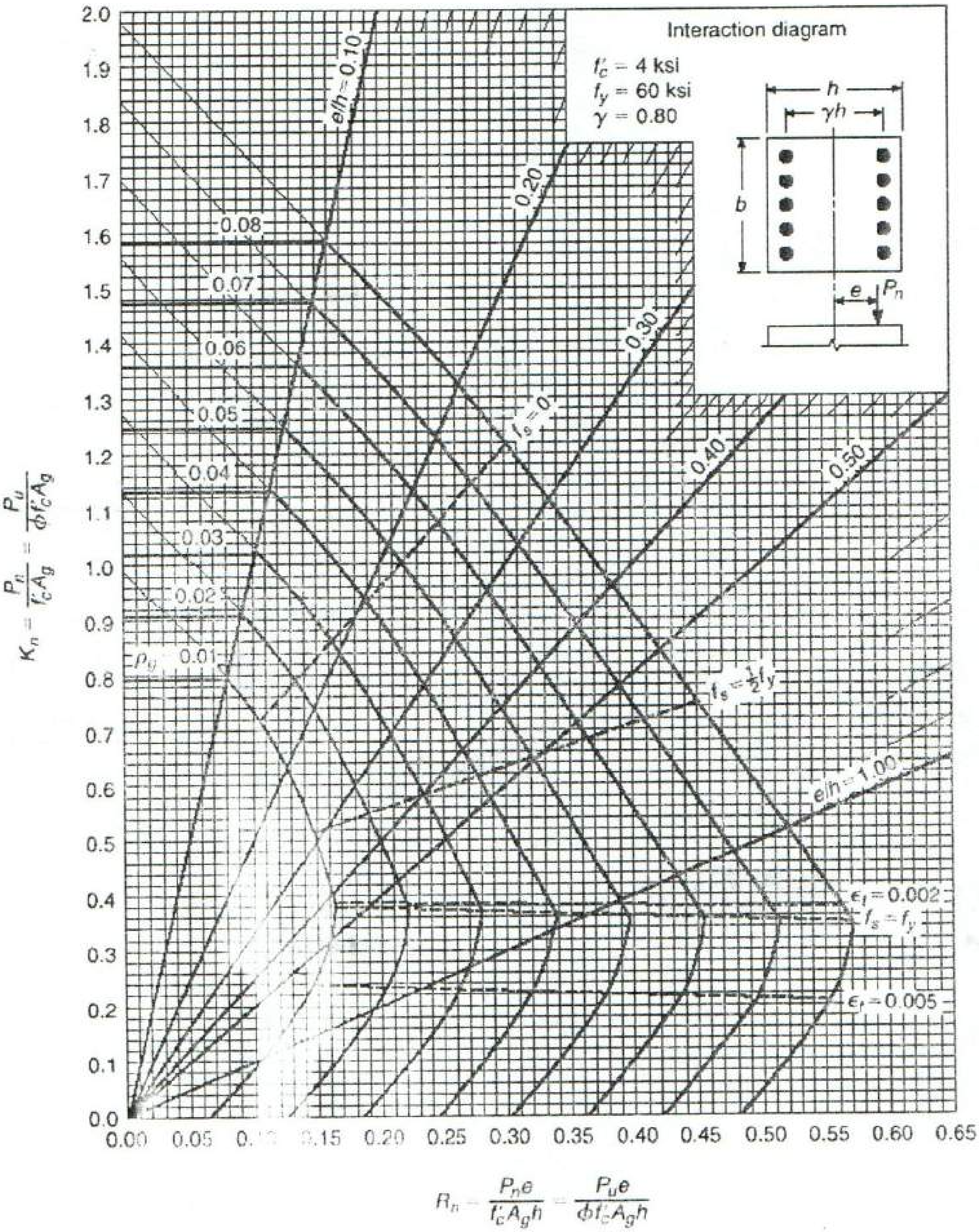
$= 1.43 > 1 \longrightarrow$ not ok

باید مجدداً برای $\gamma = 0.7$ نیز حساب کنیم ، سپس $\gamma = 0.675$ را حساب کنیم

بعد با هم بسنجیم ،

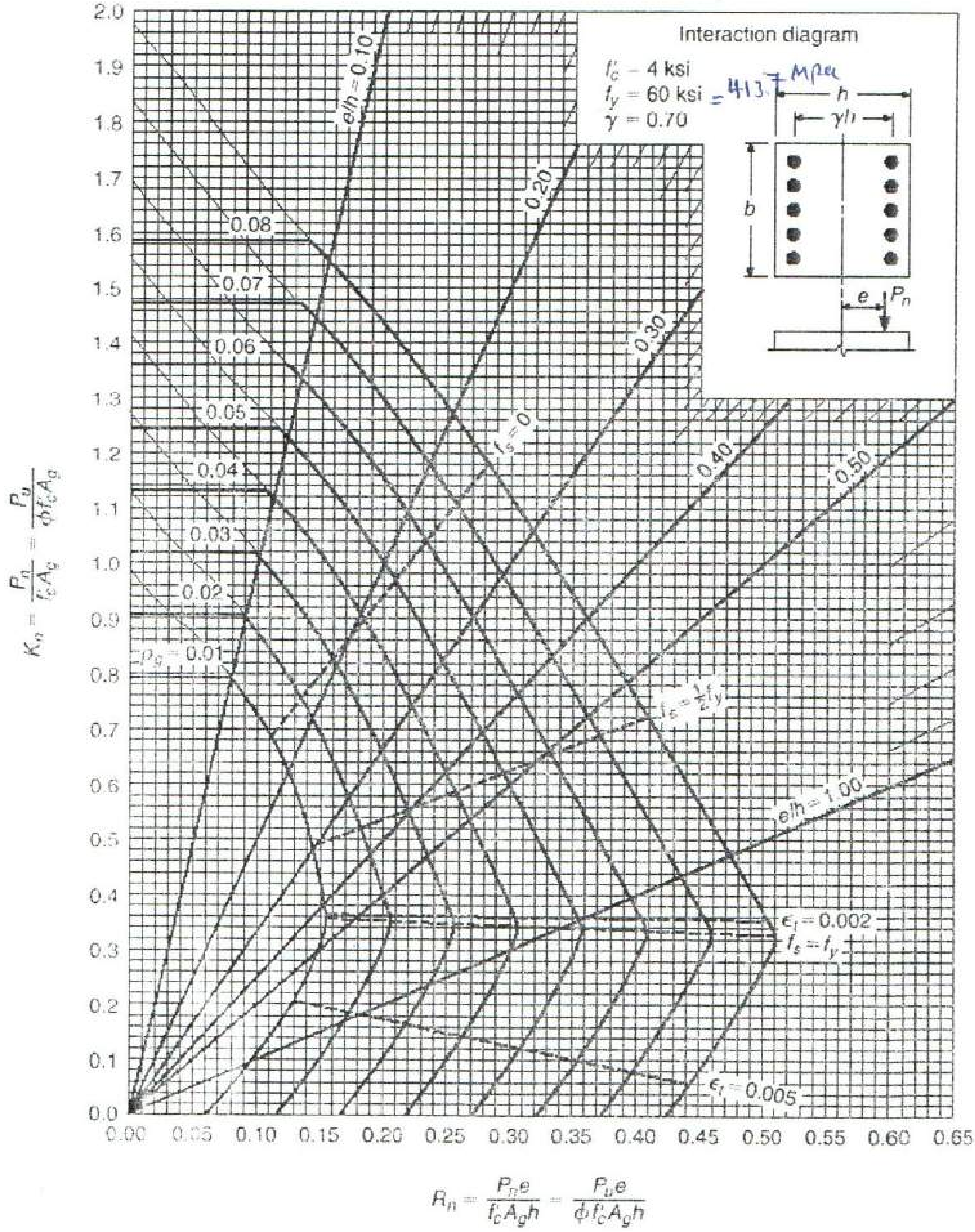
* مسئله‌ی بعدی که حتماً باید check گردد $P_n > 0.1 P_{n0}$ است

که باید ok باشد.



GRAPH A.11
Column strength interaction diagram for rectangular section with bars on end faces and γ = 0.80 (for instructional use only).

1 ksi = 6.895 Mpa



GRAPH A.10

Column strength interaction diagram for rectangular section with bars on end faces and $\gamma = 0.70$ (for instructional use only).

3

Slender Columns

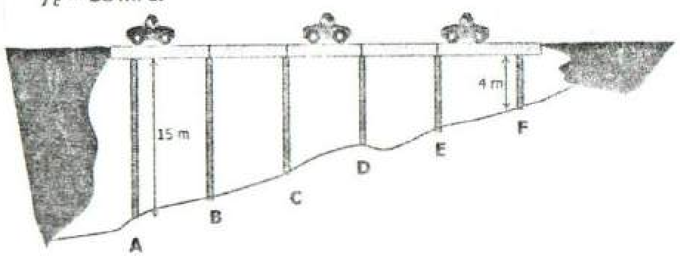
Second-Order Effects

Based on ACI318M-11

Spring 2015

A Simple Design Problem

- The girders of a box-girder bridge simply sit on the 700 mm diameter RC piers locating 10 m apart and their lengths vary from 4 m to 15 m. Based on the applied loads to the bridge in normal conditions, each pier shall carry a factored compressive axial load of 4700 kN with 155 mm eccentricity. Design the required reinforcement. Take $f_y = 420$ MPa and $f'_c = 28$ MPa.



A Simple Design Problem (Cont.)

Assume $\phi = 0.65$

$K_n = 0.671$

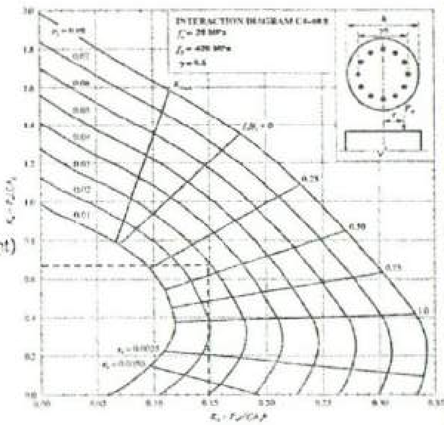
$R_n = 0.148$ $\Rightarrow$ $\rho \approx 3\%$

Select 17Φ30 (12017 mm²) and Φ10@300 for ties

left to you (Control the provided reinforcement)

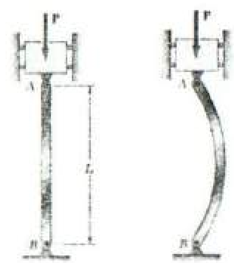
Question:

Do you select the designed section for all columns A-F?



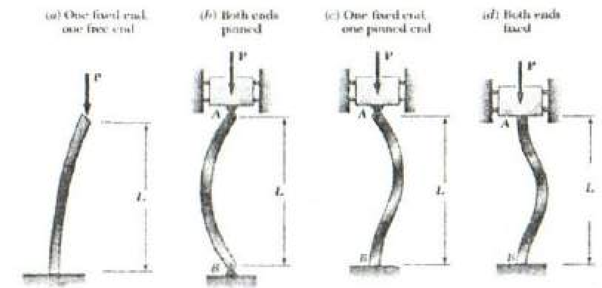
Euler Buckling Load

- For an axially loaded column with both ends hinged, Euler found the critical load in terms of the flexural rigidity (EI) and length of column (L) as



$$P_E = P_{cr} = \frac{\pi^2 EI}{L^2}$$

Individual Columns with Various End Conditions

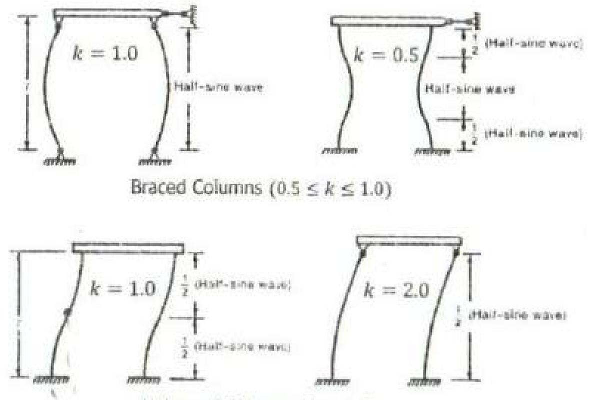


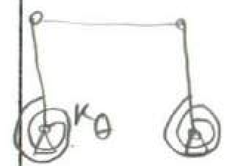
$P_{cr} = \dfrac{\pi^2 EI}{4L^2}$ $\quad$ $P_{cr} = \dfrac{\pi^2 EI}{L^2}$ $\quad$ $P_{cr} = \dfrac{\pi^2 EI}{0.49L^2}$ $\quad$ $P_{cr} = \dfrac{\pi^2 EI}{0.25L^2}$

باید غلتک بذاریم

یک نقطه عطف

Sway (Unbraced) and Nonsway (Braced) Columns





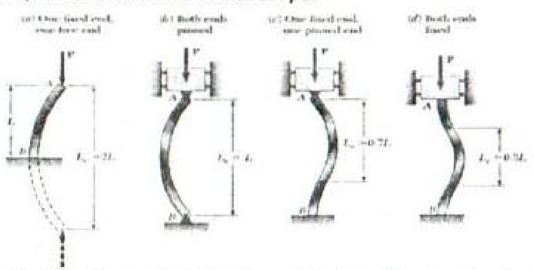
$0 < k_\theta < \infty$

$k_\theta \to \infty \to K = 2$

$k_\theta = 0 \to K = \infty$

Effective Length Factor

- The effective length is the distance between inflection points (zero moments) on the buckled deformed shape.



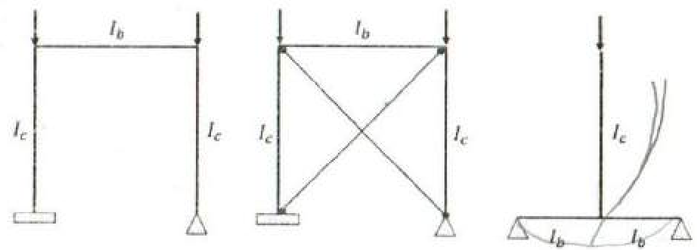
- Introducing the effective length as $L_e = kL$, where k is the effective length factor, the critical load is defined as

$$P_{cr} = \frac{\pi^2 EI}{(KL)^2}$$

L_e فاصله‌ی بین دو نقطه‌ی عطف روی نمودار تغییر شکل یافته

Example 1

- Determine the lower and upper bounds of k-factor for each column.

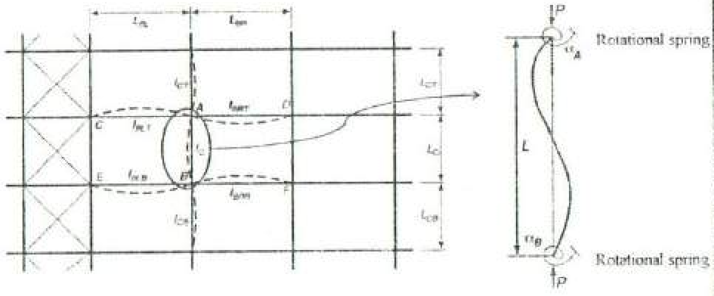


$I_b \to \infty$

$\frac{I_b}{I_c} \to \infty$

$\frac{Pl^3}{48EI}$ $\quad$ $k = \frac{48EI}{l^3}$

Columns in Braced Frames

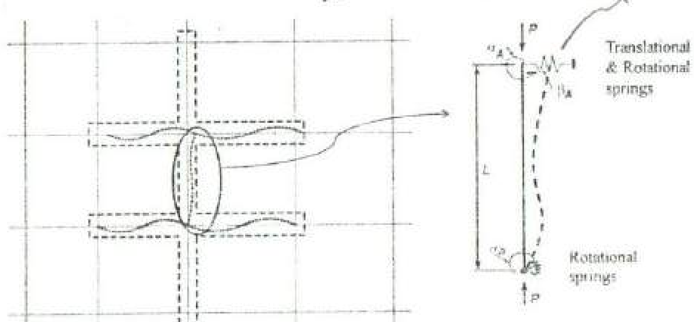


Idealized model

چون تیرها بیشتر تحت خمش اند

Columns in Sway Frames

یک سختی جانبی هم داریم که با یک فنر مدل می‌کنیم.



Idealized model

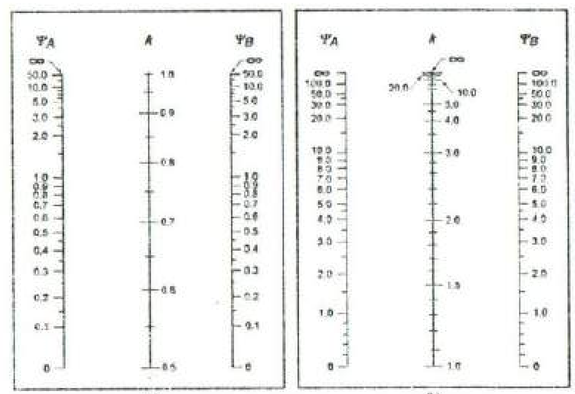
Relative Stiffness of Columns to Beams

- The term Ψ is defined at each end of column and it reflects the ratio of flexural stiffness of columns to the flexural stiffness of beams connecting at the common joint.

$$\Psi = \frac{\Sigma\left(\frac{EI}{l}\right)_{columns}}{\Sigma\left(\frac{EI}{l}\right)_{beams}}$$

- l is measured center-to-center of joints.
- I should reflects the effects of cracking.
  - Simple approach (ACI 10.10.4.1)
    - $I_{beams} = 0.35 I_g$ ← چون تیرها بیشتر تحت خمش اند
    - $I_{columns} = 0.70 I_g$ → چون ستون‌ها بیشتر تحت فشار و خمش هستند
  - More detailed formulas (Refer ACI 10.10.4.1)

K Factor for Columns in Frames (Alignment Charts)



(a) Nonsway Frames — nonsway — braced — $0.5 < K < 1$

(b) Sway Frames — sway — unbraced — $1 < K < \infty$

Notes on Alignment Chart

- General assumptions:
  - Behavior is purely elastic.
  - All joints are rigid.
  - All columns in the story buckle simultaneously.
  - All floor panels are rectangular. ← چون در تیرهای مایل سختی تیرهای عمود را ندارند
  - The end beam moments at a joint are distributed to connected columns based on their relative flexural stiffness.
  - The rotation of beams far ends is equal to the joint rotation. (single curvature in braced frames, double curvature in unbraced frames)
  - No significant axial force exists in beams.
- A cantilever beam provides no flexural rigidity for the related joint.
- Take $\Psi = 1$ for fixed end condition (theoretical value is 0). → به دلیل اینکه در واقعیت هیچ تکیه‌گاه Fix نداریم
- Take $\Psi = 10$ for hinged end condition (theoretical value is ∞).
- If beams far ends are not continuous, appropriate modifications shall be applied on their flexural stiffness.
- In braced frames k can be taken conservatively equal to 1.0.

Slenderness Ratio

- Slenderness ratio reflects the degree of slenderness of the column.
- Slenderness ratio $= \frac{kl_u}{r}$

  where

  k is the effective length factor,

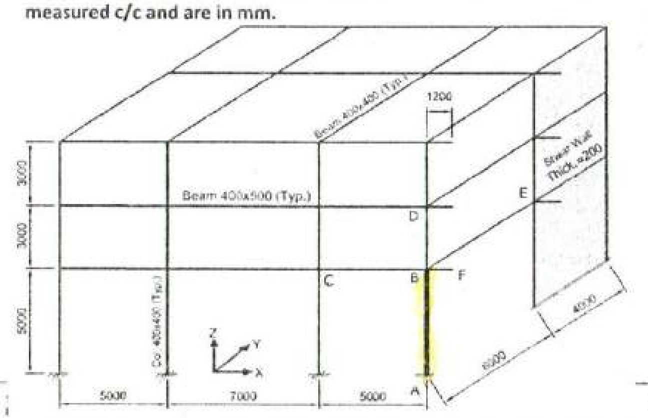
  l_u is the unsupported column length, and

  r is the radius of gyration of column section.

فاصله‌ی مهار شده‌ی اول تا آخر ستون باید
در اندازه‌ی ... دقت گردد که فاصله‌ی ارتفاع تیر بالا و پایین کم گردد.
عمق

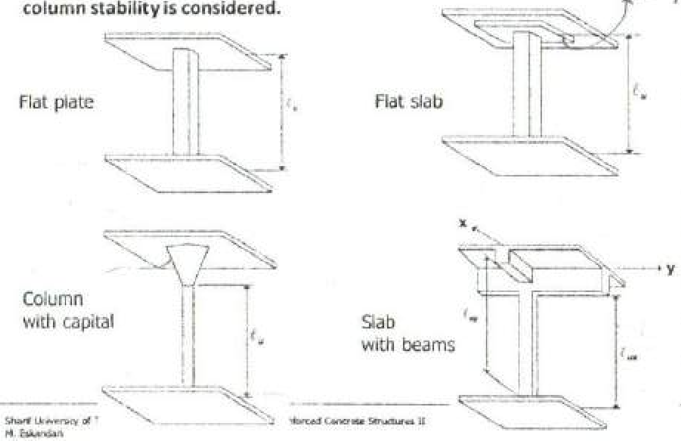
Example 2

- Find the K factor for the column AB. For clarity, the crossing beams and columns at the first and second floors are not shown. All dimensions are measured c/c and are in mm.

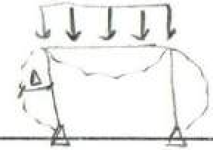


Unsupported Column Length (l_u)

- The clear distance between two column ends in the direction that the column stability is considered. drop-panel



← چون ما همیشه آنالیز مرتبه اول یا First order می‌کنیم و آنالیزما اثرات مرتبه ۲ ندارد.



Radius of Gyration (r)

- The radius of gyration reflects the effects of size and geometry of section on the slenderness ratio.

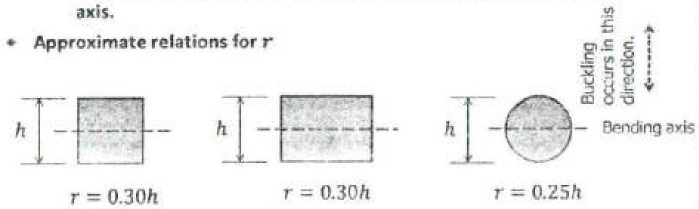
$$r = \sqrt{\frac{I}{A}}$$

where

A is the gross sectional area, and

I is the second moment of inertia of section about the bending axis.

- Approximate relations for r



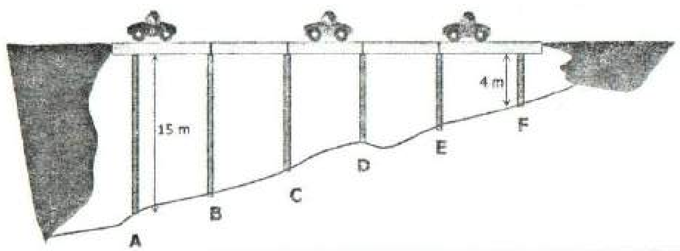
Is Buckling the Only Concern for Design of Slender Columns?

- NO
  - Buckling of concrete columns seldom occurs, unless the slenderness ratio is being very high.
- Secondary effects must also be considered for slender columns.

معمولا در ستون‌ها کمانش رخ نمی‌دهد، آنچیزی که مهم است درنظرگرفتن اثرات P-Δ است.

A Simple Design Problem (Cont.)

- Based on the discussed topics, the designer has revised his/her design and has considered the following criteria:
  - Strength capacity by employing the column interaction diagram
  - Buckling criterion (All columns are designed in such a way that the buckling is prevented, $P_u < P_{cr}$.
- Now, do you accept her/his design?

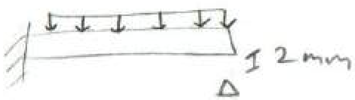


First-Order Analysis vs. Second-Order Analysis

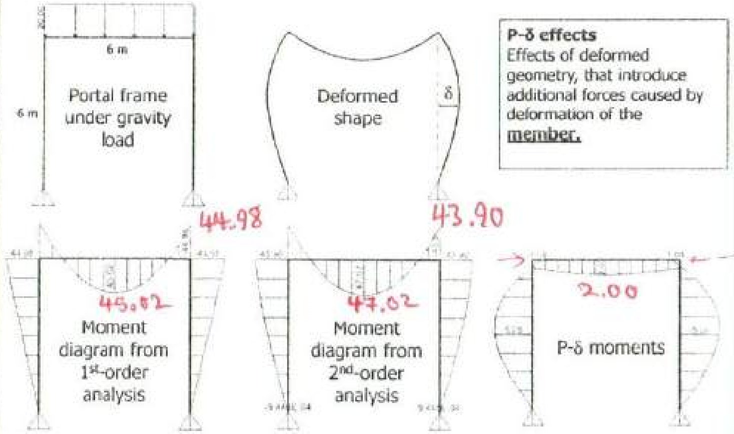
- First-order analysis
  - All equilibrium equations are written based on the undeformed shape of the structure.
- Second-order analysis
  - The deformed shape affects the magnitude of internal forces developed in the structure (secondary effects, P-Δ & P-δ).
  - The effects of axial force on the element stiffness are taken into account (stress stiffening).
- Questions — چون تحت حرکات، زمانی که P-Δ قابل ملاحظه باشد آنالیز می‌کنیم.
  - When is it required to perform a second-order analysis? →
  - What is the difference between nonlinear analysis and second-order analysis?
  - Is the superposition technique valid for a second-order analysis?

non linear { Material nonlinearity →

Genetric nonlinearity →



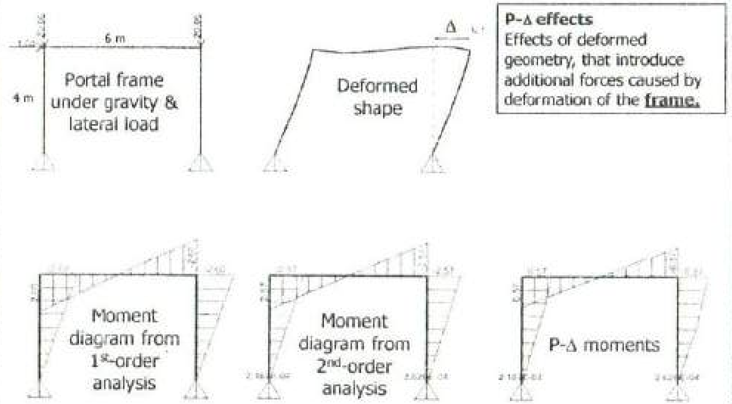
Second-Order Effects (P-δ)



P-δ effects
Effects of deformed geometry, that introduce additional forces caused by deformation of the **member.**

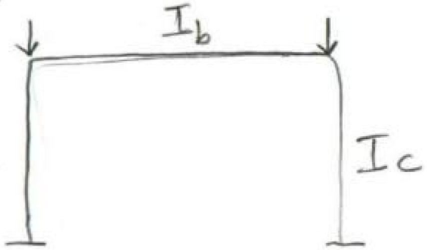
نیروی محوری

Second-Order Effects (P-Δ)

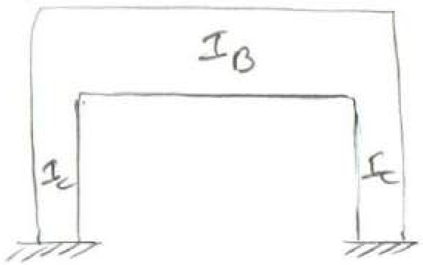


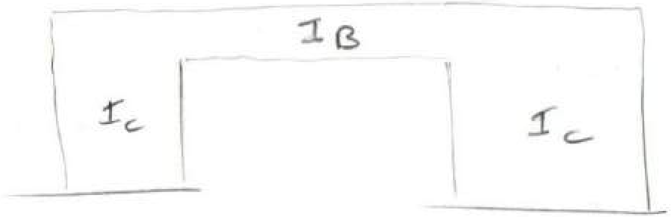
P-Δ effects
Effects of deformed geometry, that introduce additional forces caused by deformation of the **frame.**

Example 1:



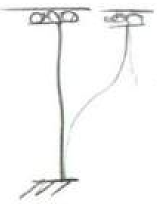
دو حالت دارد $\longrightarrow$



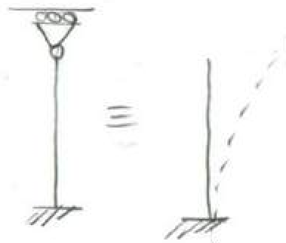


$$\frac{I_b}{I_c} \longrightarrow \infty$$

$$\frac{I_b}{I_c} \longrightarrow 0$$



$\longrightarrow K = 1$



$K = 2$

Example 2

Buckling about X-axis , K_x (nonsway)

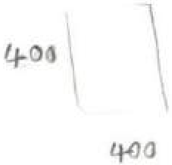
ستون حول محور x buckle کند.

$$\psi_A = \frac{\sum (\frac{EI}{L})_c}{\sum (\frac{EI}{L})_b} = 0 \longrightarrow \psi_A = 1$$

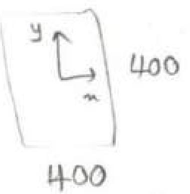
ضریب کاهش I

$$\psi_B = \frac{\sum (\frac{EI}{L})_c}{\sum (\frac{EI}{L})_b} = \frac{(\frac{I}{L})_{AB} + (\frac{I}{L})_{BD}}{(\frac{I}{L})_{BE}} = \frac{\frac{(0.7)(2.13\times10^9)}{5000} + \frac{(0.7)(2.13\times10^9)}{3000}}{\frac{(0.35)(2.13\times10^9)}{6000}} = 6.4$$

$\Big\} K_x = 0.85$ ✓



$I_{beam\,(BE)} = \frac{1}{12}(400)(400)^3 = 2.13\times10^9 \ mm^4$



$I_{column} = 2.13\times10^9 \ mm^4$

I_{Beam}

$\longrightarrow I_{Beam} = \frac{1}{12}(400)(500)^3 = 4.16\times10^9 \ mm^4$

Buckling about y-axis

$\psi_A = 1.0$

$$\psi_B = \frac{\sum (\frac{EI}{L})_c}{\sum (\frac{EI}{L})_b} = \frac{(I/L)_{AB} + (I/L)_{BD}}{(I/L)_{BC}} = \frac{\frac{(0.7)(2.13\times10^9)}{5000} + \frac{(0.7)(2.13\times10^9)}{3000}}{\frac{(0.35)(4.16\times10^9)}{5000}} = 2.73$$

$\Big\} K_y = 1.6$ ✓

4

Slender Columns

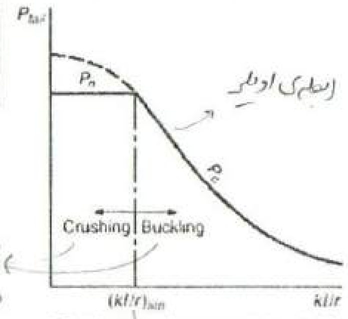
Magnification Factors

Based on ACI318M-11

Spring 2015

Failure Modes of RC Columns under Concentric Loading

- Material failure
- Stability failure (rarely occurs)

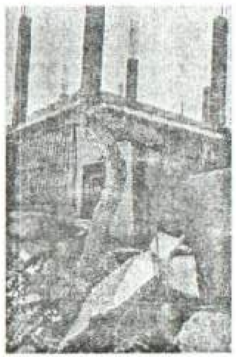


منحنی اویلر

دو حالت خراب شدن ممکن است
یکی کمانش، دیگری تخریب مصالح

RC column under concentric axial load

لاغری حد (مرز)



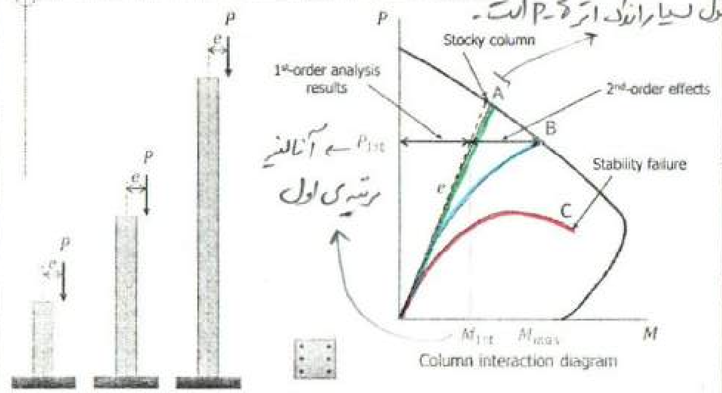
Buckling failure of an RC column
(Algeria Earthquake, 2003)

Failure of Short Columns vs. Slender Columns

Experimental Illustrations

Failure Modes of RC Columns under Eccentric Loading

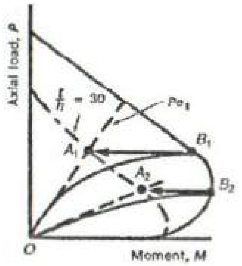
این طول بسیار اندک اثر P-δ است.



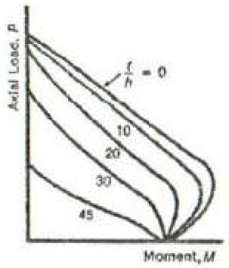
آنالیز مرتبه‌ی اول

Column interaction diagram

Interaction Diagram of Slender Columns



Construction of interaction diagram for a slender column



Typical interaction diagram for a column with various slenderness ratios

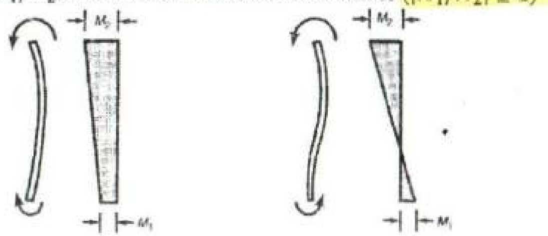
مقدار اینها به چه ستونی کوتاه و به چه ستونی بلند گویند.

Short Columns

- The slenderness effects can be neglected for short columns.
- Short column in an unbraced frame if $\frac{kl_u}{r} \le 22$
- Short column in a braced frame if $\frac{kl_u}{r} \le 34 - 12\left(\frac{M_1}{M_2}\right) \le 40$

where M_1/M_2 is the ratio of moments at column ends $(|M_1/M_2| \le 1)$

این وضعیت بدتر است، چون
Δ آن بیشتر است.

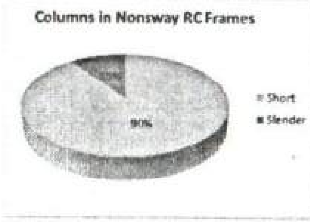


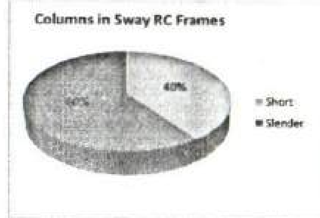
Single curvature column.
$0 \le M_1/M_2 \le 1.0$

Double curvature column.
$-1 \le M_1/M_2 \le 0$

A Survey of Slender Columns in Typical RC Buildings

- The slenderness effects may be neglected for more than 90% of columns in nonsway frames, i.e. 90% of columns are categorized as short columns in braced frames.
- Effects of slenderness may be neglected for about 40% of columns in sway frames.





Braced and Unbraced Columns

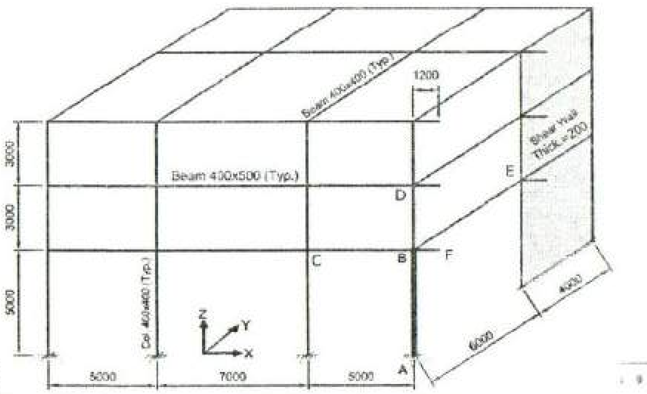
- ACI code states that
  - It shall be permitted to consider compression members braced against sidesway when bracing elements have a total stiffness, resisting lateral movement of that story, of at least 12 times the gross stiffness of the columns within the story.

gross مقطع

- More detailed discussions on ACI criteria for identifying sway and nonsway stories will be given later.

Example 3

- Can the slenderness effects be neglected for the column AB of Ex. 2?
- Check the braced assumption along the y-direction.



ACI Criteria for Sway and Nonsway Frames

- **First Method**

  It shall be permitted to assume a column in a structure is nonsway if the increase in column end moments due to second-order effects does not exceed **5 percent** of the first-order end moments. → nonsway کمتر از ۵٪ باشد قاب nonsway است.

- **Alternative Method** جمع کل بارهای ستون های طبقه

  A story within a structure is assumed to be nonsway if

$$\text{Stability index} \leftarrow Q = \frac{\Sigma P_u \Delta_0}{V_{us} l_c} \le 0.05$$

شاخص پایداری

برش طبقه

where

ΣP_u is the total factored vertical load in the story

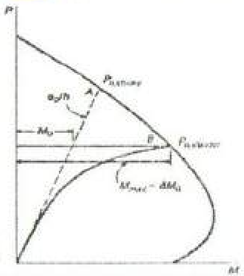
V_{us} is the total factored horizontal story shear

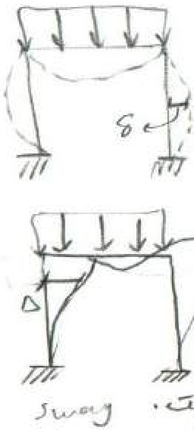
Δ_0 is the first-order relative lateral deflection between the top and the bottom of the story due to V_{us}

l_c: طول ستون از center to center joint

ACI Suggested Methods for Considering Slenderness Effects

- Nonlinear second-order analysis (Complicated, needs computer analysis)
- Elastic second-order analysis (Effective, needs computer analysis)
  - The effects of cracking can be considered by using reduced moment of inertia.
  - The effects of sustained loads can be considered by reducing the flexural rigidity, EI.
- Moment magnification method (Approximate, easy for hand-calculations)
  - Moments obtained from the first-order analysis are simply amplified by a factor to estimate the second-order effects.





Moment Magnification Procedure-Nonsway Columns

$$M_c = \delta_{ns} \times \max[M_2, M_{2,min}]$$

$$M_{2,min} = P_u(15 + 0.03h)$$

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} \ge 1.0 \quad \text{where} \quad \delta_{ns} \le 1.4$$

$$C_m = \begin{cases} 0.6 + 0.4\dfrac{M_1}{M_2} \ge 0.4 & \text{No transverse loading} \\ 1.0 & \text{Transverse loading} \end{cases}$$

$$P_c = \frac{\pi^2 EI}{(kl_u)^2}$$

Understimate EI for $\rho > 2\%$

برای ρ ها ۱۲٪ رابطه خوب نیست.

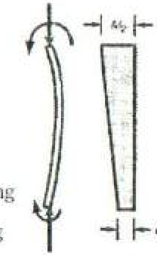
$$EI = \frac{0.4E_c I_g}{1 + \beta_{dns}} \quad \text{or} \quad EI = \frac{0.2E_c I_g + E_s I_{se}}{1 + \beta_{dns}}$$

برای اثر ترک خوردگی

چون فولاد داخل بتن ممکن است تحت اثر creep دچار yield گردد (در حالت سرویس)

$$\beta_{dns} = \frac{\text{maximum factored sustained axial load}}{\text{maximum factored axial load}} \le 1$$

The creep parameter is applied to both $E_c I_g$ and $E_s I_{se}$!!!



اثر ترک خوردگی

مثلاً بار مرده

I_g

$E_s I_{se}$

اگر شد یعنی مقطع خیلی کوچک است باید ابعاد را تغییر داد

تشدید nonsway نیازی به افزایش ندارد

تشدید های sway باید افزایش یابند.

Notes on Moment Magnification of Nonsway Columns

$\delta_{ns} < 1.4$ باید

- If δ_{ns} is greater than 1.4, it is mandatory to enlarge the column section due to the stability requirements.
- What is the role of the factor 0.75 in the δ_{ns} formula?

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} \geq 1.0$$

ثابتی است که باعث کاهش سختی می‌گردد.

$$P_c = \frac{\pi^2 EI}{(kL)^2}$$

- What is the role of C_m factor?
- We cannot use $I_c = 0.7I_g$, as what we did for calculating the k factor, to evaluate P_c. Why? Isn't it a contradiction?
- What does it mean if $\delta_{ns} < 0$?

مقطع دچار Failure نمی‌شود buckle می‌گردد.

Moment Magnification Procedure-Sway Columns

$$M_1 = M_{1ns} + \delta_s M_{1s} \qquad M_2 = M_{2ns} + \delta_s M_{2s}$$

Nonsway moments are not magnified.

$$\delta_s = \frac{1}{1-Q} \geq 1 \quad \text{where} \quad \delta_s \leq 1.5$$

or

$$\delta_s = \frac{1}{1 - \frac{\Sigma P_u}{0.75\Sigma P_c}} \geq 1$$

ΣP_u = the summation for all factored vertical loads in story

ΣP_c = the summation for all sway-resisting columns in a story

$$P_c = \frac{\pi^2 EI}{(kl_u)^2}$$

$$EI = \frac{0.2E_c I_g + E_s I_{se}}{1+\beta_{ds}} \quad \text{or} \quad EI = \frac{0.4E_c I_g}{1+\beta_{ds}}$$

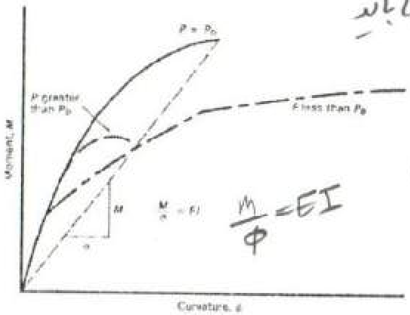
$$\beta_{ds} = \frac{\text{maximum factored sustained shear load}}{\text{maximum factored shear load}} \leq 1.0$$

M_1 M_2

lateral soil pressure ← بارهای جانبی ماندگار

Notes on the Column Rigidity (EI) for Evaluating P_c

- For evaluating P_c one shall use the column flexural rigidity at the time of failure.



$\frac{M}{\phi} = EI$

این مقدار گشتاور اضافه را در بالا و پایین ستون ها باید به اندازه ی مقدار سختی تیر ها بین آنها پخش کنیم در نتیجه گشتاور تیر ها نیز به خاطر P-δ اصلاح می‌گردد.

Notes on Moment Magnification Procedure-Sway Columns

- The ratio of the amplified moment due to the second-order effects to the moment obtained from the first-order analysis shall not exceed 1.4.
- The beams at the column joints shall be designed for a total additional moment of $(\delta_s - 1)M_s$. This additional moment shall be distributed between the responsible beams based on their flexural stiffness.
- For significantly slender columns in sway frames it is possible that the maximum moment occurs along the length of member instead of its ends. In such situations, the amplified sway moments shall be amplified again by employing the amplification factor of nonsway columns, i.e.

$$M_c = \delta_{ns} M_2 = \delta_{ns}(M_{2ns} + \delta_s M_{2s})$$

- For $\frac{l_u}{r} < \frac{35}{\sqrt{P_u/A_g f'_c}}$, it may not be required to consider the previous amplification.

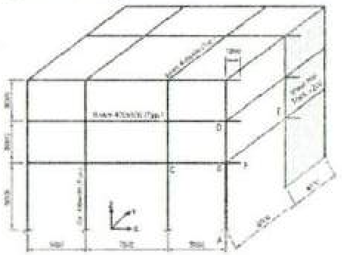
Final Notes

- The moment magnification method gives good results for nonsway columns, but it is not very reliable for sway columns.
- It is recommended to use the second-order analysis (simplified P-Δ or complete P-Δ) for sway frames instead of the ACI moment magnification method.
- The dimensions of members used in the analysis should not be different from those shown on the contract documents by more than 10%. Why? → درهنگام نقشه کشی نمی‌توان برای type کردن تیرها یا ستون‌ها نهایت ۱۵٪ می‌توان تغییر داد
- It is recommended to avoid using too slender columns, $kl_u/r > 100$.

← برای حالت های معماری که کاربرد غیرسازه‌ای دارد می‌توان از این $\frac{kl}{r}$ استفاده کرد.

Example 4

- A first-order analysis gives the following internal forces in the column AB of Ex.2. Design the column section by considering the second-order effects.
  - Dead load
    - $P = 22$ ton
    - $M_{xA} = -0.81$ ton.m
    - $M_{xB} = 1.33$ ton.m
    - $M_{yA} = M_{yB} = 0$
  - Live load
    - $P = 4$ ton
    - $M_{xA} = -0.23$ ton.m
    - $M_{xB} = 0.38$ ton.m
    - $M_{yA} = M_{yB} = 0$
  - Earthquake load
    - $P = 13$ ton
    - $M_{xA} = M_{xB} = 0$
    - $M_{yA} = 22.3$ ton.m
    - $M_{yB} = -11.3$ ton.m



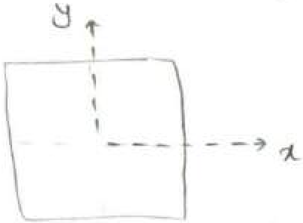
Δ_x of first story $= 32$ mm
$V_x = 95$ ton
$\Sigma P_D = 570$ ton
$\Sigma P_L = 154$ ton

Ex.3 column AB

Buckling about X : $K_x = 0.85$ braced

y : $K_y = 1.6$ unbraced

$$\left(\frac{k\,l_u}{r}\right)_x = \frac{K_x\,l_{ux}}{r_x} = \frac{0.85 \times 4800}{120} = 34.0 \longrightarrow \text{short column}$$



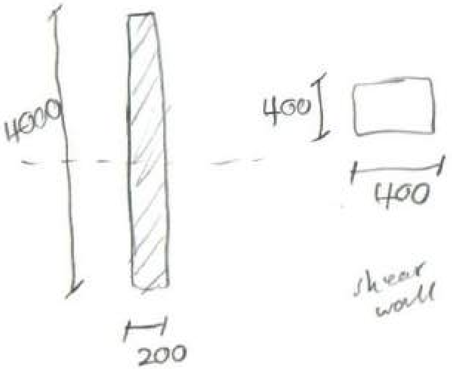
$l_{ux} = 5000 - \overset{400/2}{200} = 4800$

$r_x = 0.3h = 0.3 \times 400 = 120\,mm$

$$\left(\frac{k\,l_u}{r}\right)_y = \frac{k_y\,l_{uy}}{r_y} = \frac{(1.60)(4750)}{120} = 62.9 \geq 22 \longrightarrow \text{slender column}$$

$l_{uy} = 5000 - \frac{500}{2} = 4750\ mm$

$K \alpha EI$



$$\frac{K_{\text{shear wall}}}{K_{\text{columns}}} \propto \frac{(EI)_{shw}}{(EI)_c}$$

shear wall تعداد

$$= \frac{2 \times \frac{1}{12} \times 200 \times 4000^3}{12 \times \frac{1}{12} \times (400)^4} = 83.3 > 12.0$$

column تعداد

braced frame

Sadeghi Slide 4

Solution of Example 4

- Calculation of K_x (Done in the lecture time) $\Rightarrow K_x = 0.85$
- Calculation of K_y (Done in the lecture time) $\Rightarrow K_y = 1.52$
- Is the column slender about the x-axis?

  Done in the lecture time $\Rightarrow$ The column is short about x-axis.

- Is the column slender about the y-axis?

  Done in the lecture time $\Rightarrow$ The column is slender about y-axis. The second-order effects shall be considered.

- Load Combinations

Load case	Load Combination	P_u (ton)	$(M_{ux})_A$ (t.m)	$(M_{ux})_B$ (t.m)	$(M_{uy})_A$ (t.m)	$(M_{uy})_B$ (t.m)
1	1.4D	30.8	−1.13	1.86	0	0
2	1.2D+1.6L	32.8	−1.34	2.20	0	0
3	1.2D+L+1.4E	48.6	−1.20	1.98	31.2	−15.8
4	1.2D+L−1.4E	12.2	−1.20	1.98	−31.2	15.8
5	0.9D+1.4E	38.0	−0.73	1.20	31.2	−15.8
6	0.9D−1.4E	1.6	−0.73	1.20	−31.2	15.8

* Note

Here, it has been assumed that the earthquake is determined based on Iranian Seismic Code 2800. So, the factor 1.4 should be used instead of 1.0. written in

Sway Load Combinations (Moments about y-axis) $M_1 = (M_{uy})_B$, $M_2 = (M_{uy})_A$

Case	P_u (ton)	M_1 (t.m)	M_2 (t.m)	M_{1ns} (t.m)	M_{1s} (t.m)	M_{2ns} (t.m)	M_{2s} (t.m)	K	β_{ds}	EI ($\times 10^{12}$ N.mm^2)	P_c (ton)	ΣP_u (ton)	Q	$\delta_s = (1/1-Q)$	M_1 (t.m)	M_2 (t.m)
3	48.6	-15.8	31.2	0	-15.8	0	31.2	1.52	0	21.2	401	838	0.056	1.06	-16.7	33.1
4	12.2	15.8	-31.2	0	15.8	0	-31.2	1.52	0	21.2	401	838	0.056	1.06	16.7	-33.1
5	38.0	-15.8	31.2	0	-15.8	0	31.2	1.52	0	21.2	401	513	0.035	N.A.	-15.8	31.2
6	1.60	15.8	-31.2	0	15.8	0	-31.2	1.52	0	21.2	401	513	0.035	N.A.	15.8	31.2

$M_1 = M_{1ns} + \delta_s M_{1s}$

$M_2 = M_{2ns} + \delta_s M_{2s}$

$$E_c = 4700\sqrt{f_c'} = (4700)(\sqrt{28}) = 24{,}870 \text{ MPa}$$

$$EI = \frac{0.4\, E_c I_g}{1+\beta_{ds}} = \frac{(0.4)(24{,}870)(\frac{1}{12})(400)^4}{1+0} = 21.2 \times 10^{12} \text{ N.mm}^2$$

$$P_c = \frac{\pi^2 EI}{(k l_u)^2} = \frac{(\pi^2)(21.2 \times 10^{12})}{[(1.52)(4750)]^2} = 4013 \text{ kN} \approx 401 \text{ ton}$$

$$\delta_s = \frac{1}{1-Q} \qquad Q = \frac{\Sigma P_u \Delta_o}{V_{us}\, l_c}$$

Summary of Results

Column End B

Load Case	Pu (ton)	Mux (t.m)	Muy (t.m)	
1	30.8	1.86	0	→ Not critical
2	32.8	2.20	0	→ Not critical
3	48.6	1.98	−16.7	→ Not critical
4	12.2	1.98	16.7	→ Not critical
5	38.0	1.20	−15.8	→ Not critical
6	1.6	1.20	15.8	→ Not critical

Column End A

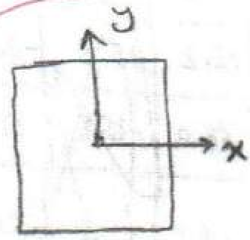
Load Case	Pu (ton)	Mux (t.m)	Muy (t.m)	
1	30.8	−1.13	0	→ Not critical
2	32.8	−1.34	0	→ Not critical
3	48.6	−1.20	33.1	✓
4	12.2	−1.20	−33.1	→ Not critical
5	38.0	−0.73	31.2	→ Not critical
6	1.6	−0.73	31.2	✓

By engineering judgement, you can consider only critical load cases.

Design of Reinforcement

Since M_{uy} is much greater than M_{ux}, we neglect the effect of M_{ux} in the design of reinforcement, ie (Biaxial Bending reduces to Uniaxial Bending)

!!



Case 6 at End A

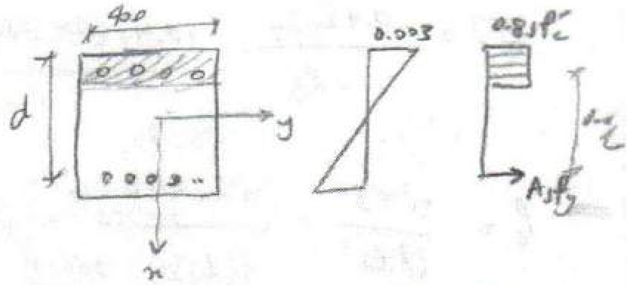
$P_u = 1.6$ ton

$M_{uy} = 31.2$ t.m

??

$$\frac{P_u}{A_g f'_c} = \frac{1.6 \times 10^4 \text{ (N)}}{(400)^2 (28)} = 0.004 < 0.1 \rightarrow \text{Design as a beam.}$$

$$d = 400 - \underbrace{40}_{\text{Cover}} - \underbrace{10}_{d_s} - \overset{d_b}{\frac{20}{2}} = 340 \text{ mm}$$



$\phi = 0.9$

$$A_{s_{req}} = \frac{M_u}{\phi f_y (d - a/2)} \qquad \text{Take } a = 100 \text{ mm}$$

Trial & Error cycles have been terminated. $a_2 = 135$ mm

$$A_{s_{req}} = \frac{31.2 \times 10^7 \text{ (N.mm)}}{(0.9)(400)(340 - \frac{100}{2})} = 2988 \text{ mm}^2 \rightarrow \underline{4\Phi 32 \; (3216 \text{ mm}^2)}$$

Check $A_{s_{min}}$, ϕ (left to you)

Check minimum & maximum spacing between bars

Case 3 at End A

$P_u = 48.6$ ton

$M_{uy} = 33.1$ t.m

$$\frac{P_u}{A_g f'_c} = \frac{48.6 \times 10^4}{(400)^2 (28)} = 0.11 > 0.1$$

$\rightarrow$ Design as a column.

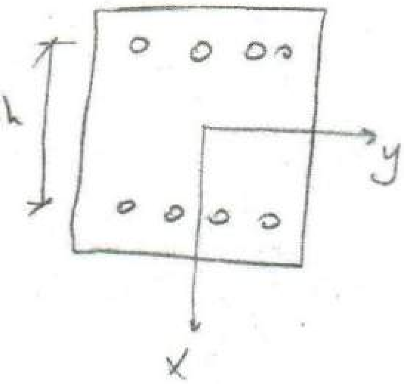
Take $\phi = 0.9$

$$K_n = \frac{P_u}{\phi A_g f'_c} = \frac{48.6 \times 10^4}{(0.9)(400)^2(28)} = 0.12$$

$$R_n = \frac{M_u}{\phi A_g f'_c h} = \frac{33.1 \times 10^7}{(0.9)(400)^2(28)(400)} = 0.21$$

cover tie قطر میلگرد

$$\gamma = \frac{400 - 2(40) - 2(10) - 30}{400} = 0.675$$



By using the interaction diagrams for $f'_c = 28$ MPa and $f_y = 420$ we have

$$\begin{cases} \gamma = 0.6 \Rightarrow \rho = 3.6\% \\ \gamma = 0.7 \Rightarrow \rho = 3.1\% \end{cases} \Rightarrow \gamma = 0.675 \rightarrow \rho = 3.5\%$$

?? How ← $\boxed{\text{Check assumed } \phi = 0.9 \text{ (left to you)}}$

$$A_{s_{req}} = \rho A_g = (0.0035)(400)^2 = 5600 \text{ mm}^2 \quad — \quad \underline{8\Phi 32 \ (6433 \text{ mm}^2)}$$

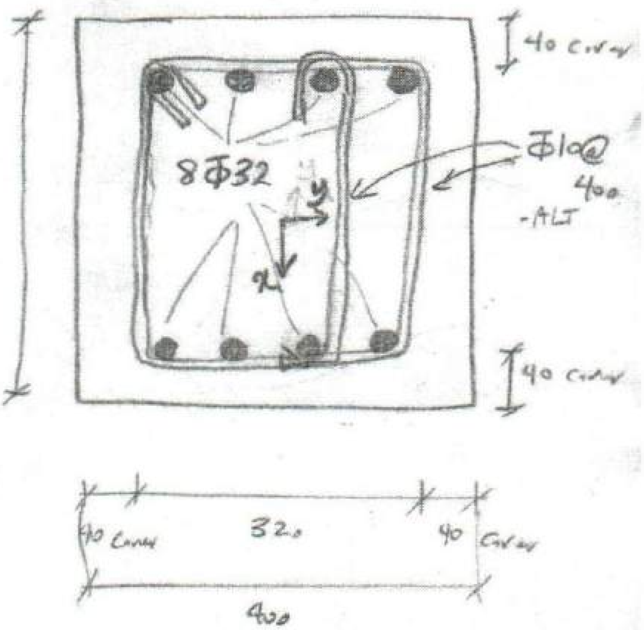
Ties

Use $\Phi 10$ since $d_b < 32$ mm

$S = \min(16 d_b, 48 d_s, h_{min})$

$= \min((16)(30), (48)(10), 400)$

$S = 400$ mm



$\boxed{\text{* Check clear spacing between bars (left to you)}}$

__Notes__:

* It is not recommended to use $\rho > 3\%$ in columns, due to congestion of bars at splice location and the requirement for ductility. Therefore, it is better to enlarge the column section in this example.
* It is recommended to use symmetric reinf. layout for column sections to avoid any construction errors.
* For column sections, we shall always check the shear reinforcement, because the spacing between ties for shear design can be shorter than the required spacing for confinement. $\left(V_u \simeq \frac{(1.4)(95)}{12} = 11 \text{ ton. Check Shear Reinf.} \right)$

Floor Systems

Based on ACI318-11

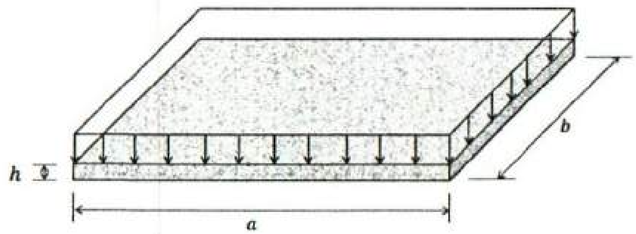
Spring 2015

دال های یک طرفه در یک راستا کنترل شده است و مانند تیر طراحی می گردد-

جهت انتقال بار را نشان می دهد

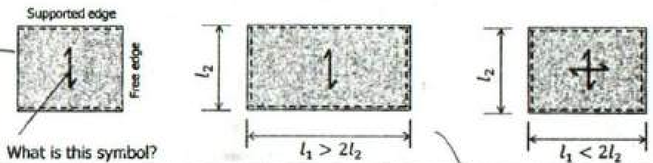
Slab

- Slabs are plane elements used in floors, roofs, and foundations of buildings to carry normal loads applied to their surfaces.
- The thickness of a slab is much smaller than its other dimensions ($h \ll a, b$).



Types of Slabs (Behavior)

- **One-Way Slabs**
  - Supported along one direction only.
    Load is transmitted in the direction normal to the supports.
  - Four-edge supported with $\frac{long\ span}{short\ span} \geq 2.0$.
    The majority of applied load is transmitted along the short span. (Why?)
- **Two-Way Slabs**
  - Four-edge supported with $\frac{long\ span}{short\ span} < 2.0$.
    Load is transmitted along two orthogonal directions.



What is this symbol?

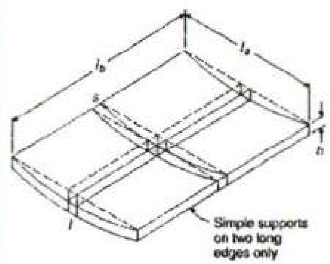
مسیر کوتاه تر را طی می کند.

انتقال در دو راستا

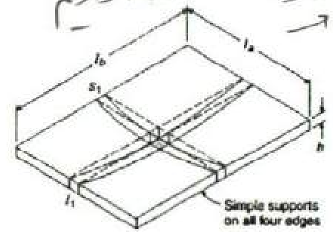
Types of Slabs (Behavior)

- One-way slabs deform in a cylindrical form (single curvature).
- Two-way slabs deform into a bowl shape (double curvature).

در دال دوطرفه در هردو سمت انحنا داریم.



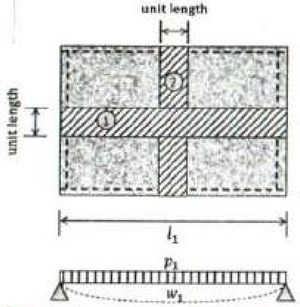
Deformed shaped of a one-way slab under uniform loading



Deformed shaped of a two-way slab under uniform loading

Strip Method

- Consider a four-edge simply supported slab under uniform load p.
- Let's consider the intersecting strips of unit length at the middle of each span. If fact, the slab analysis is simplified to a beam analysis.
- The compatibility of deformation and loads leads to



$$\frac{p_2}{p_1} = \left(\frac{l_1}{l_2}\right)^4$$

For $l_1 = 2l_2$ about 95% of applied load is transmitted along the short direction.

$P_1 + P_2 =$ کل بار

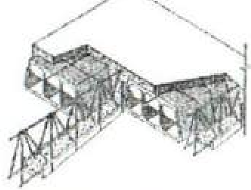
تغییر شکل در وسط تیرها برابر است.

فرض برای دال یک طرفه است

Types of One-Way Slabs

- Supported on edge beams
- Ribbed or joist
  - Pan-joist
  - Block-precast joist
- Four-edge supported ($\frac{l_1}{l_2} \geq 2$)

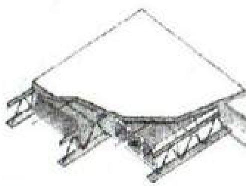
تیرچه ها را سخت می کند — joist پشت بند یا



Block-joist system with precast concrete joists

One-way slab supported on edge beam

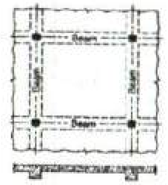
Pan-joist system



Block-joist system with open web steel joists

Types of Two-Way Slabs

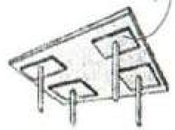
- Flat plate
- Flat slab
- Supported on edge beams
- Grid or waffle
- Voided slab



Two-way slab supported on edge beams

برای جلوگیری از اثر punching روی دال توسط ستون ← drop panel

Flat plate



Flat slab

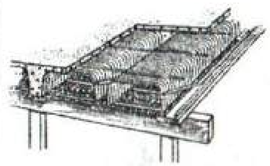
Waffle slab

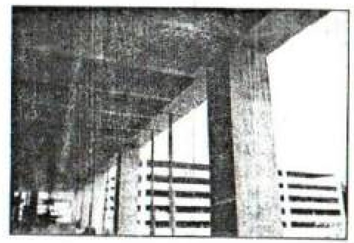
Voided slab

پولیاکس

شیب دارد تا قالب خارج گردد.

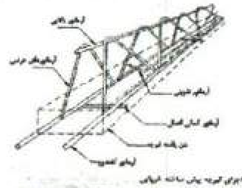
Pan-Joist System

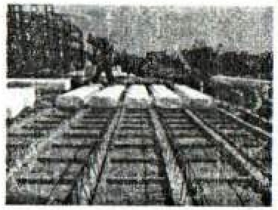


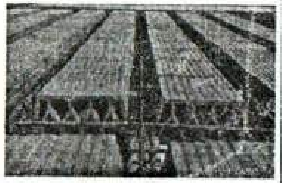


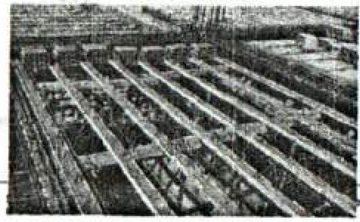
Block-Precast Joist System

- Not recommended
- Fake pan-joist system!

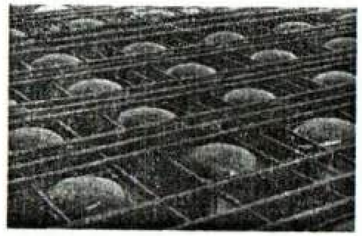


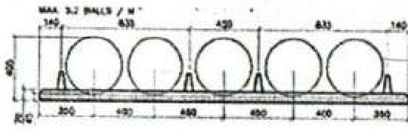






Voided Slab-Cobiax

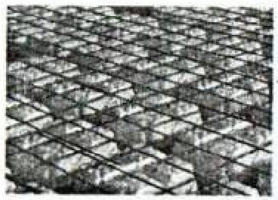




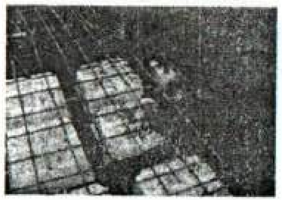
TYP CROSS SECTION

چون چگالی کمتری دارد باید بر آن را با گیره نگهداشت تا بالا نیاید.

Voided Slab (U-Boot)

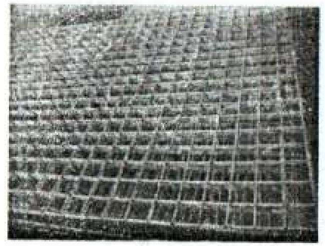


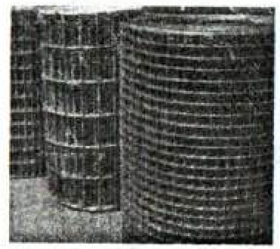
سقف U بوت



Slab Reinforcement Assembly

- Use of Welded Wire Fabric (WWF) facilitates the assembly of slab reinforcement.



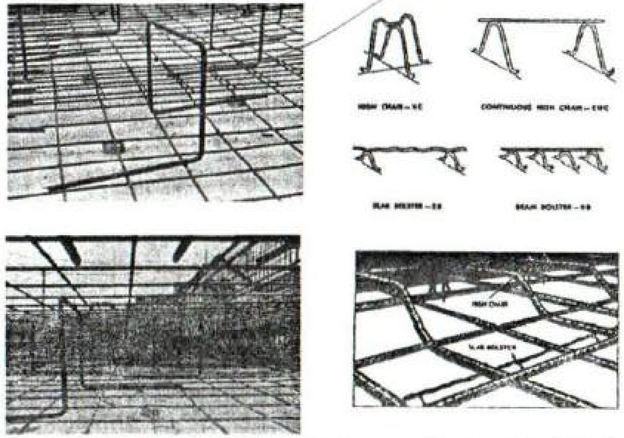


برای سایزهای 6 و 8 به صورت کلاف موجود است.

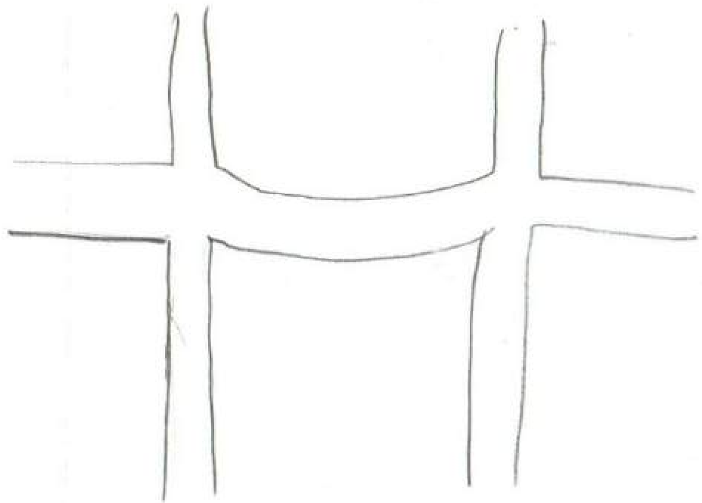
باید در هر دو طرف زیر آن خرک زد وگرنه خمیده می‌شود.

خرک‌هایی هستند که برای بستن آرماتور کار روی آن استفاده می‌شود

Slab Reinforcement Assembly



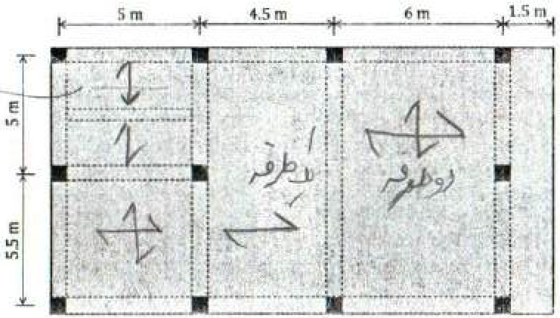
دال‌های دوطرفه هزینه‌ی اجرای بالایی دارند. در ساختمان‌های بسیار بلند اگر ضخامت دال‌های هر طبقه را کمتر کنیم می‌توان در ۲۰-۱۰ طبقه یک طبقه اضافه گرفت که در دال‌های دوطرفه ضخامت کمتری وجود دارد.



Example

- Identify one-way and two-way slabs in the given floor system.

یک طرفه است



تیر دال

در دال‌های یک طرفه می‌توان از همان ضرایب بار دال‌های دوطرفه استفاده کرد.

6

Floor Systems

Design of One-Way Slabs

•

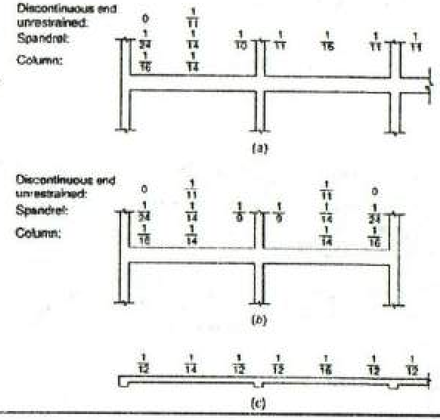
Based on ACI318-11

Spring 2015

Analysis of One-Way Slabs

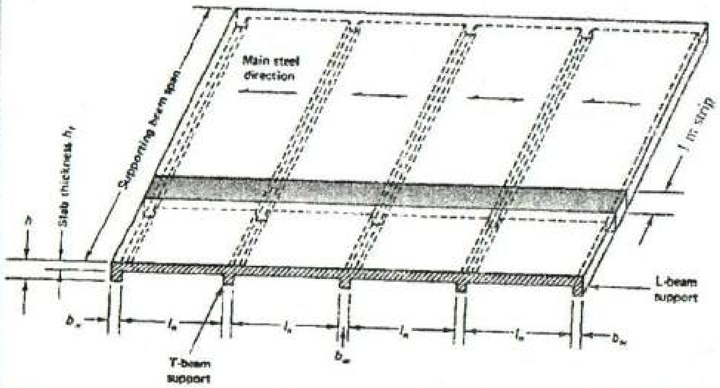
- The approximate shear and moment coefficients given in ACI can be used for the analysis of one-way slabs.

FIGURE 12.10
Summary of ACI moment coefficients: (a) beams with more than two spans; (b) beams with two spans only; (c) slabs with spans not exceeding 3 m



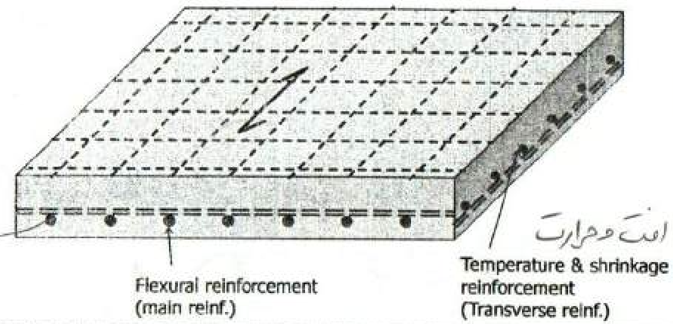
Design of One-Way Slabs

- For analysis and design purposes an imaginary 1 m strip is considered.



Reinforcement Layout

- Flexural reinforcement carries the developed bending moment.
- Transverse reinforcement is required to distribute the shrinkage and temperature cracks. (How?)



آرماتور خمشی
یا اصلی

افت و حرارت

میلگرد حرارتی هرجای دال می‌توانند قرار گیرد

Design Requirements: 1-Thickness

- Slab thickness is usually assumed and the reinforcement is designed based on the assumed thickness.
- Table 9.5(a) of ACI code gives the recommended minimum thickness of beams/one-way slabs not supporting partitions likely to be damaged by large deflections.
- If a slab is dimensioned based on Table 9.5(a) of ACI code:
  - Deflection need not to be checked.
  - Usually, neither flexure nor shear controls the thickness.
- Slab thickness is rounded to the nearest 10 mm.

Design Requirements: 1-Thickness (Cont.)

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

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

Notes:
Values given shall be used directly for members with normalweight concrete and Grade 420 reinforcement. For other conditions, the values shall be modified as follows:
a) For lightweight concrete having equilibrium density, w_c, in the range of 1440 to 1840 kg/m³, the values shall be multiplied by $(1.65 - 0.0003w_c)$ but not less than 1.09.
b) For f_y other than 420 MPa, the values shall be multiplied by $(0.4 + f_y/700)$.

Design Requirements: 2-Concrete Cover

- According to Section 7.7 of ACI318-11, for cast-in-place concrete the minimum clear concrete cover for slabs shall be

Surface Condition	Bar Size (mm)	Minimum Clear Concrete Cover (mm)
Concrete cast against and permanently exposed to earth	-	75
Concrete not exposed to weather or in contact with ground	$\Phi < 40$	20
	$\Phi \geq 40$	40
Concrete exposed to earth or weather	$\Phi < 18$	40
	$\Phi \geq 18$	50

در دال‌ها کاور دال روی ظرفیت تأثیر زیادی دارد زیرا ضخامت دال نسبتاً کم است. h d

Design Requirements: 3- Shear

- Under normal loads shear forces are not critical and shear reinforcement is not required.
- If $V_u \leq \phi V_c$ then no shear reinforcement is required:

روی کل ظرفیت بتن حساب می‌شود

$$V_c = 0.17\sqrt{f'_c}\,b_w d$$

$\frac{\phi V_c}{2}$ نه ϕV_c

- Engineers prefer to design slabs with no shear reinforcement. (Why?)

مهندسین به جای آرماتور برشی معمولاً ضخامت دال را زیادی کنند چون اجرا سخت است و در طول کوتاه میلگرد برشی جاری نمی‌گردد.

- For heavily loaded slabs shear reinforcement is provided but it should not be used in slabs less than 200 mm thickness. (Why?)

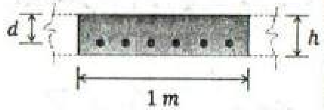
اگر زیر 200 mm باشد مجاز به گذاشتن خاموت نیستیم چون میلگرد مهار نمی‌شود.

Design Requirements: 4- Flexure

- The flexural reinforcement is designed for a rectangular section of unit width.

$$A_s \text{ per meter} = A_b \left(\frac{1000 \text{ mm}}{\text{bar spacing}}\right)$$

← مساحت یک میلگرد



- The area of reinforcement is determined based on singly reinforced sections.
- A good initial guess for a is $0.1d$, or $\left(d - \frac{a}{2}\right) = 0.95d$.
- The minimum steel area is equal to the required temperature and shrinkage reinforcement:

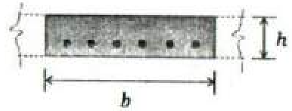
$$A_{s,min} = A_{s,\,temp.\&\,shrink.}$$

در تیرها لق میلگرد به دلیل این بوده که اگر ترک بخورد Fail می‌کند ولی در دالها باز توزیع نیرو صورت می‌گیرد و Fail نمی‌کند.

Design Requirements: 5- Temperature & Shrinkage Reinforcement

- The minimum amount of temperature and shrinkage reinforcement is defined as

$$\frac{A_{s,temp.}}{bh} = \begin{cases} 0.0020 & f_y \le 350\ MPa \\ 0.0018 & f_y = 420\ MPa \\ \dfrac{(0.0018)(420)}{f_y} \ge 0.0014 & f_y > 420\ MPa \end{cases}$$



- The temperature reinforcement can be placed at any depth of the section, i.e. bottom, middle, or top. → میلگرد حرارتی می‌تواند هر جایی قرار گیرد

Design Aid: Area of Equally Spaced Standard Bars

Area of Equally Spaced Standard Bars (mm²/m)

Bar Size (mm)	Center-to-Center Spacing (mm)											
	75	100	125	150	175	200	225	250	300	350	400	450
6	377	283	226	188	162	141	126	113	94	81	71	63
8	670	503	402	335	287	251	223	201	168	144	126	112
10	1,047	785	628	524	449	393	349	314	262	224	196	175
12	1,508	1,131	905	754	646	565	503	452	377	323	283	251
14	2,053	1,539	1,232	1,026	880	770	684	616	513	440	385	342
16	2,681	2,011	1,608	1,340	1,149	1,005	894	804	670	574	503	447
18	3,393	2,545	2,036	1,696	1,454	1,272	1,131	1,018	848	727	636	565
20	4,189	3,142	2,513	2,094	1,795	1,571	1,396	1,257	1,047	898	785	698
22	5,068	3,801	3,041	2,534	2,172	1,901	1,689	1,521	1,267	1,086	950	845
25	6,545	4,909	3,927	3,272	2,805	2,454	2,182	1,963	1,636	1,402	1,227	1,091

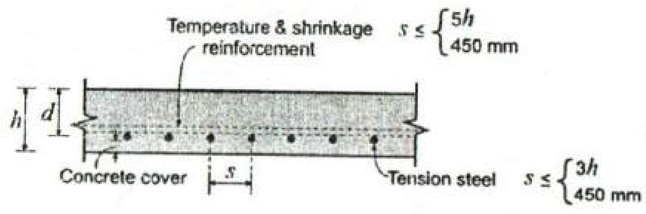
Design Requirements: 6-Maximum Bar Spacing

- Flexural Reinforcement

$$s_{max} = \min(3h, 450 \text{ mm})$$

- Temperature & Shrinkage Reinforcement

$$s_{max} = \min(5h, 450 \text{ mm})$$



برای شبکه‌ی بالا به دلایل اجرایی چون میگردد حین اجرا انحناء پیدا میکند حداقل Φ12 باید باشد.

Design Requirements: 7- Bar Size

- It is preferable to use Φ12 or larger bars for flexure reinforcement, as Φ10 may be bent out of position by labors walking on it. This issue is more critical for top bars.
- Sometimes, Φ10 is used for bottom bars, and Φ12 for top bars.
- It is more practical to use similar bar sizes for both slab directions and if needed change the spacing or use some add bars.

سعی کنید شبکه‌ی اصلی و فرعی یکی باشد.

- Use of welded wire meshes (WWM) facilitates the slab construction.

Design Requirements: 8- Deflection

- If the thickness of slab is selected based on Table 9.5(a) of ACI code, it is not required to control deflection. (Exception?)
- For other conditions, the deflection shall be calculated by using the same procedure discussed for beams.

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

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

Notes:
Values given shall be used directly for members with normalweight concrete and Grade 420 reinforcement. For other conditions, the values shall be modified as follows:
a) For lightweight concrete having equilibrium density, w_c, in the range of 1440 to 1840 kg/m³, the values shall be multiplied by $(1.65 - 0.0003w_c)$ but
b) For f_y other than 420 MPa, the values shall be multiplied by $(0.4 + f_y/700)$.

Why?

Design Requirements: 9- Crack Control

- The spacing of flexural reinforcement shall be less than

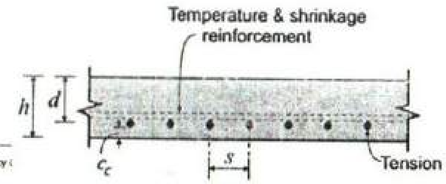
$$s_{max} = 380\left(\frac{280}{f_s}\right) - 2.5c_c \leq 300\left(\frac{280}{f_s}\right)$$

where

c_c = the least distance from the surface of reinforcement to the tension face (mm),

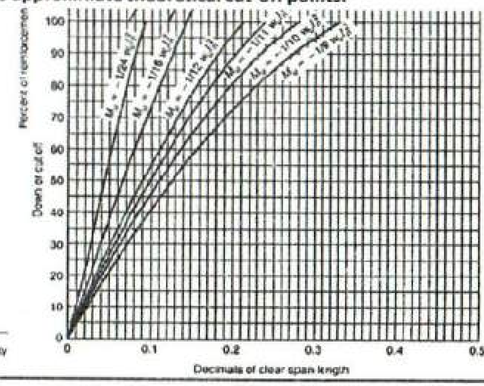
f_s = stress in reinforcement at service condition (MPa)

- It is permissible to take $f_s = \frac{2}{3}f_y$ if no detailed information is available.

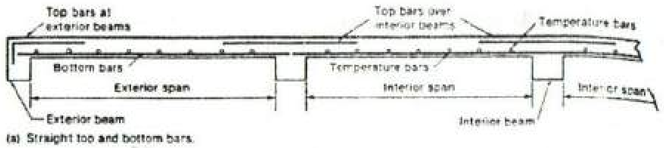


Curtailment of Bars

- Development and curtailment of bars in one-way slabs are similar to beams.
- If moment coefficients are used for analysis, the Graph A.3 of textbook shows approximate theoretical cut-off points.

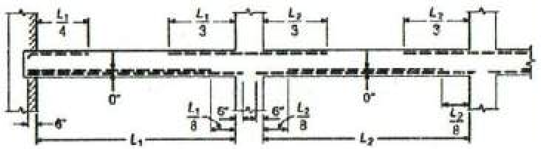


Typical Arrangement of Reinforcement



(a) Straight top and bottom bars.

FIGURE 5.15
Cutoff or bend points for bars in approximately equal spans with uniformly distributed loads.

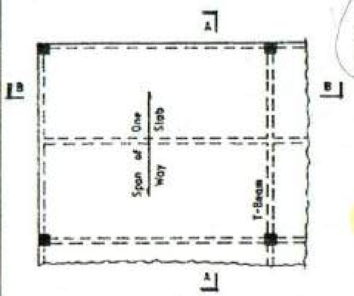


که لنگر داشت تا در آنجا ترک نخورد

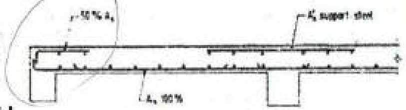
One-Way Slabs with Edge Beams

بالا معمولی As 50%

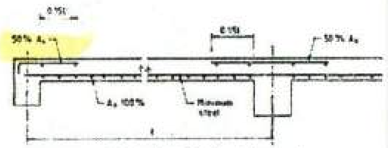
- Two-way action near the edge beams



(a) Typical Plan



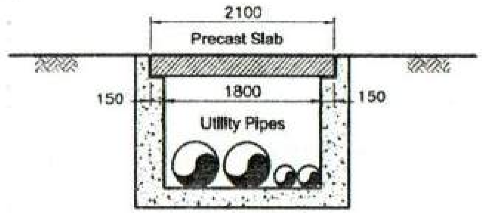
(b) Section A-A (Short Direction)



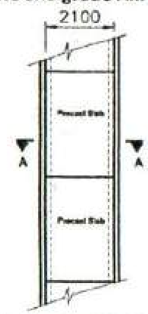
(c) Section B-B (Long Direction)

Example 1

- **A mechanical utility trench to be covered by precast concrete slabs. The slabs simply sit on the walls of the cast-in-situ trench. They may be subjected to a distributed live load on 10 kN/m² during the design life. Design the slab. If the lifting capacity is limited to 2 tons, find the appropriate length of precast units. Use C30 concrete and grade AIII reinforcement.**



Section A-A

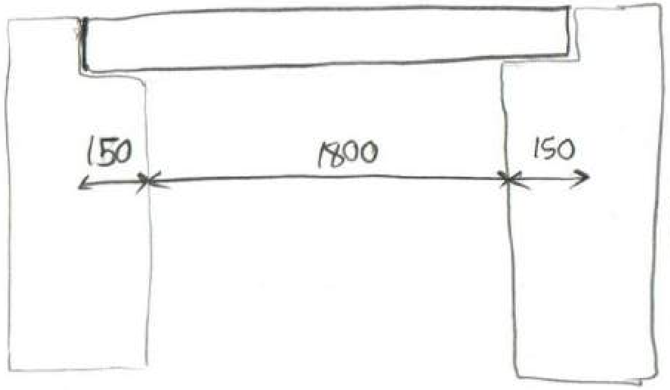


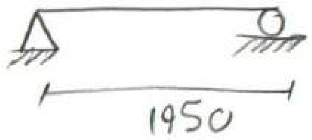
Plan View

Area of Equally Spaced Standard Bars (mm^2/m)

Bar Size (mm)	Center-to-Center Spacing (mm)											
	75	100	125	150	175	200	225	250	300	350	400	450
6	377	283	226	188	162	141	126	113	94	81	71	63
8	670	503	402	335	287	251	223	201	168	144	126	112
10	1,047	785	628	524	449	393	349	314	262	224	196	175
12	1,508	1,131	905	754	646	565	503	452	377	323	283	251
14	2,053	1,539	1,232	1,026	880	770	684	616	513	440	385	342
16	2,681	2,011	1,608	1,340	1,149	1,005	894	804	670	574	503	447
18	3,393	2,545	2,036	1,696	1,454	1,272	1,131	1,018	848	727	636	565
20	4,189	3,142	2,513	2,094	1,795	1,571	1,396	1,257	1,047	898	785	698
22	5,068	3,801	3,041	2,534	2,172	1,901	1,689	1,521	1,267	1,086	950	845
25	6,545	4,909	3,927	3,272	2,805	2,454	2,182	1,963	1,636	1,402	1,227	1,091

• example 1





$\boxed{\text{ACI } 9.9.8.9.1}$

راجعه (دهانه آزاد) تیر دو سر مفصل

$l = l_n + d$

عمق مقطع — دهانه خالص آزاد

1) Thickness:

$9.5\,(a) \rightarrow h = \frac{L}{20} = \frac{1950}{20} \approx 95 \text{ mm}$

select $h = 100$ mm

2) cover: $\rightarrow$ $c = 40$ mm

3) shear: $d = h - c - \frac{d_b}{2} = 100 - 40 - \frac{10}{2} = 55$ mm

$\phi V_c = (0.75)(0.17)(\sqrt{30})(1000)(55) = 52 > V_u = 17.5$

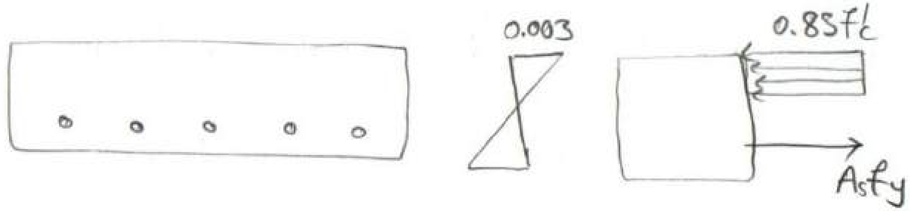
Analysis:

$M_u^+ = \frac{w_u l^2}{8} = \frac{18.9 \times 1.855^2}{8}$

$w_u = 1.2D + 1.6L = 1.2 \times 2.4 + 1.6 \times 10 = 18.9$

$w_D = (24 \text{ KN/m}^3)(1)(0.1) = 2.4 \text{ KN/m}$

$V_u = \frac{w_u l}{2} = \frac{18.9 \times 1.855}{2} = 17.5 \text{ KN}$



$A_{sreq} = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{8.1 \times 10^6}{(0.9)(400)(55-5)} = 450$

$a = 10$ mm

$a = \frac{450 \times 400}{(0.85)(30)(1000)} = 7$ mm ok

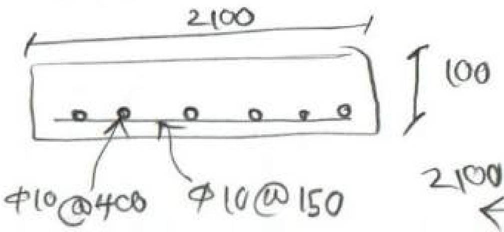
$A_{sreq} = 450 \text{ mm}^2 \longrightarrow \begin{cases} \phi 8 \ @ \ 100 \ (503) \\ \phi 10 \ @ \ 150 \ (524) \\ \phi 12 \ @ \ 250 \ (452) \end{cases}$

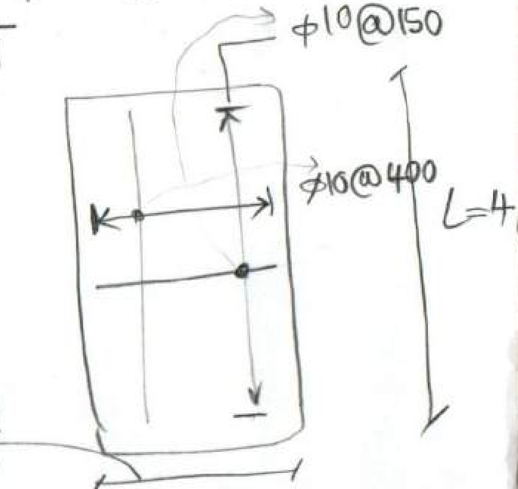
Tempreture & shrinkage Reinf:

$A_{s\,temp} = (0.0018)bh = (0.0018)(1000)(100) = 180 \ \frac{\text{mm}^2}{\text{m}}$

$A_{s\,temp} = 180 \ \frac{\text{mm}^2}{\text{m}} \begin{cases} \phi 8 @ 250 \\ \phi 10 @ 400 \\ \phi 12 @ 250 \end{cases}$

- S_{max} ✓
- bar size ✓ $\phi 8$ ✓
- crack control ✓

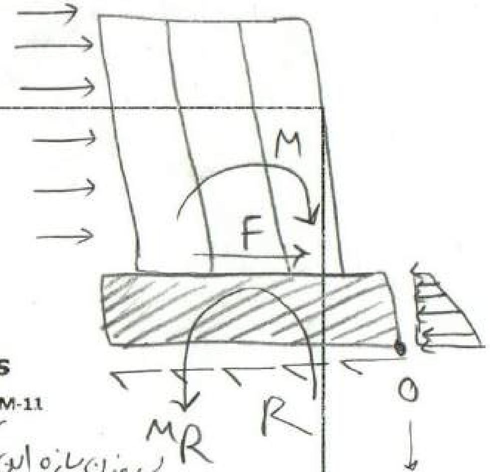




$$W = (2.1)(L)(2.4\ t/m)(0.1) = 2\ ton$$

$$L = 3.95\ m \longrightarrow L = 4\ m$$ ✓

7



Footings

Based on ACI 318M-11

وزن سازه این لنگر مقاوم را ایجاد می‌کند

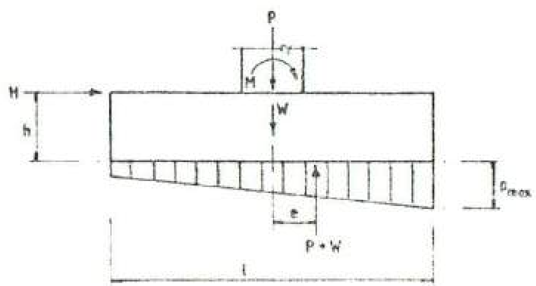
Spring 2015

overturning

حول این نقطه رخ می‌دهد.

Introduction

- Foundations transfer loads from the building members including columns and walls to the earth.



Roles of Foundation

1) Distribute transferred loads in such a way that the soil bearing capacity is not exceeded. → تنش را در خاک به حدی برسانند که قابل تحمل باشد.
2) Limit total uniform settlement (usually specified by design codes) → تغییر شکل سازه در کل نباید از حدی بیشتر گردد.
3) Limit differential settlement (usually specified by design codes, more critical than uniform settlement, why?) → اگر سازه نامعین باشد، تغییر شکل نامتقارن باعث ایجاد نیرو در سازه می‌گردد.
4) Provide safety against sliding
5) Provide safety against overturning → ضریب ایمنی برای برش و لنگر

Bonus Question: Find the specified values by Iranian Codes for items 2 to 5 related to building structures.

		لرزه‌ای
لغزش	0.8	0.9
واژگونی	0.5	0.65

Types of Foundations

- Isolated or pad or single footings
- Combined footings
- Strap or cantilever footings
- Wall footings
- Strip or continuous footings
- Raft or mat or floating foundations
- Pile foundations

در برخی جاها برای اتصال پی های تک از tie beam استفاده می گردد که برای کشش طراحی می شود تا کل پی با هم کار کند و ظرفیت tie beam برابر $0.1 P_{max}$ است و این نوع پی با پی باسکولی متفاوت است.

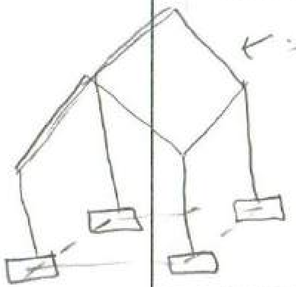
فونداسیون تک

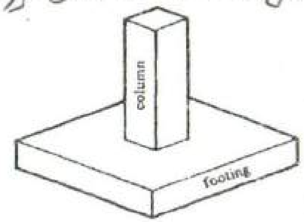
Isolated Footings

- Support a single column
- Very common in low-rise buildings
- Suitable for lightly loaded columns
- Almost ineffective for large uplifts or moments
- Used when columns are not closely spaced

معمولاً در تعداد طبقات بالا از پی تک نمی توان استفاده کرد و پی نواری بهتر و اقتصادی تر است.

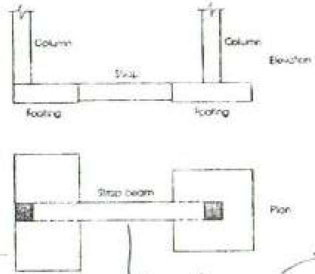
کاربرد پی تک در سازه های صنعتی است که دهانه های بزرگی دارد.





Strap or Cantilever Footings

- Support two columns
- In comparison with the combined footings, two single footings are connected by a tie beam or a strap.
- More economical than combined footings
- In seismic regions, closely spaced single footings should be connected together by strap beams.



پی باسکولی

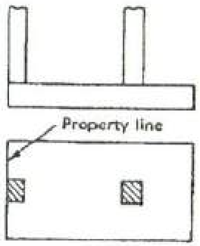
strap

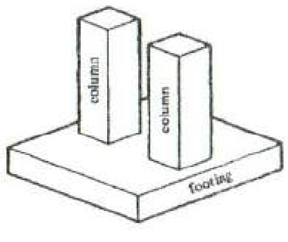
به جای استفاده از پی تک که از یک تیر برای اتصال دو پی با هم استفاده می گردد ولی چون این نوع پی نیاز مند قالب بندی بیشتری نسبت به حالت یکپارچه است و این باعث گران ترشدن این نوع پی می شود.

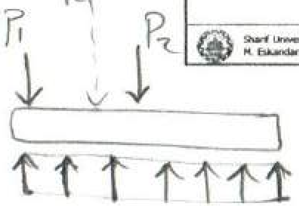
Combined Footings

- Support two or three closely spaced columns
- For columns located at or near the property lines, single footings may extend across those lines.

مثل یک تیر عمل می کند که بالای آن دو تکیه گاه داریم و زیر آن بار خاک است.

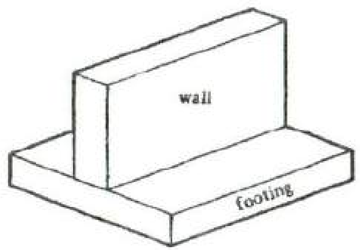






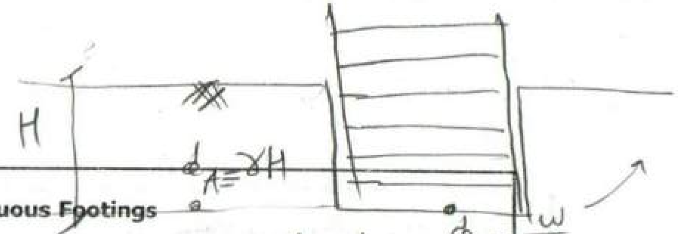
Wall Footings

- A continuous slab along the wall.
- The width of cantilever parts are determined based on the allowable soil pressure.



اگر حجم زیادی خاک برای ساختن پی یک برج برداشته شود ممکن است بار ساختمان از تنش اولیه کمتر شود که به این نوع پی شناور گویند.

پی شناور



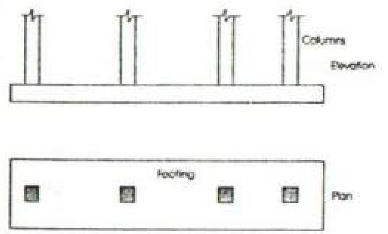
$q_1 = \gamma H$

$q_2 = \frac{W}{A_F}$

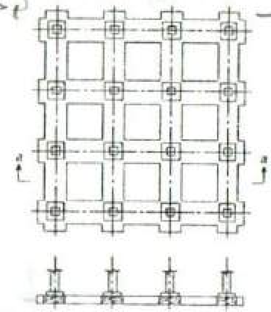
Strip or Continuous Footings

- Support three or more columns in a row
- Limit differential settlement
- More effective than single footings
- Columns can be supported by a network of intersected strips (Grid foundations).

اگر در دو جهت اینها به هم وصل گردد یک شبکه درست می شود که به آن پی شبکه ای گویند و تا ساختمان ۶-۷ طبقه خوب است.



Strip footing

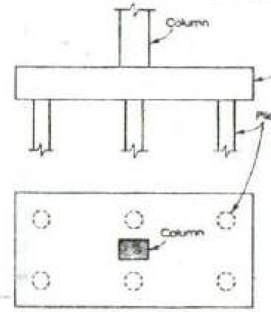


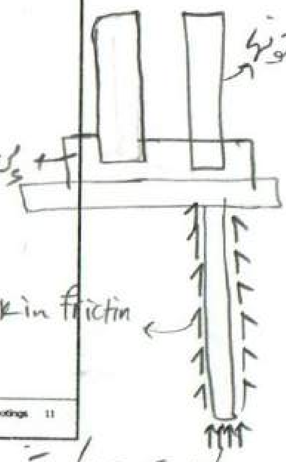
Grid foundation

پی نواری دارای ظرفیت قابل قبولی است.

Pile Foundations

- Used when top soil layers do not have proper bearing capacity.
- Piles-columns interface consists of a thick slab called pile cap.
- Piles bearing capacity may be provided by either skin friction or end bearing.
- Piles are categorized as cast-in-place and driven types.



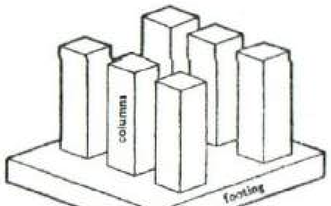


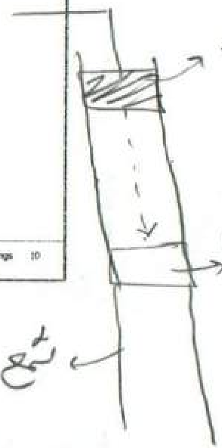
دال های گسترده و پیوسته

Raft or Mat or Floating Foundations

درجاهایی مثل برج ها که بارگذاری خیلی بالاست از این نوع پی ها استفاده می شود

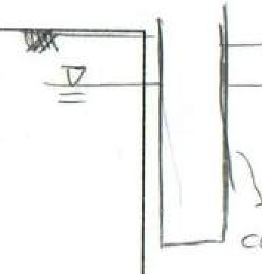
- A continuous slab that supports all columns
- For low bearing capacity or heavily loaded columns
- Reduce differential settlement effectively
- Low formwork costs
- If the total area of single footings is greater than one-half of the building area, use of mat foundations is more economical.
- Why are they called floating foundations?





Parameters Affecting Foundation Type

- Soil properties and conditions
- Structural system and level of loads
- Permissible differential settlements
- Engineer's experience
- Economy → اقتصادی باشد
- Feasibility → قابل اجرا باشد.



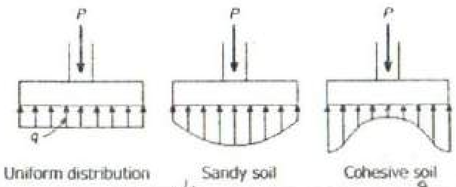
در مجاورت نواحی ساحلی از لوله ی ترمیت استفاده می گردد که درجاهایی که داخل آب است بتن ریزی از پایین صورت می گیرد

Soil Pressure Distribution of Concentrically Loaded Footings

- For concentrically loaded footings, soil pressure is assumed to be uniform.

$$q = P/A$$

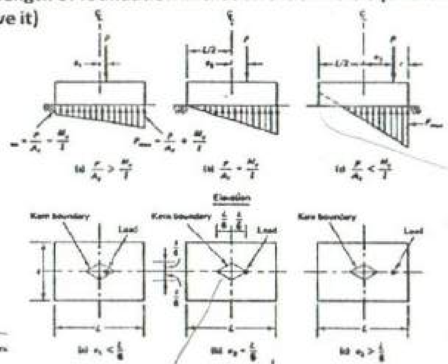
- The actual soil pressure distribution depends on various parameters, mainly:
  - Soil type
  - Foundation rigidity



ماسه
رس

Soil Pressure Distribution of Eccentrically Loaded Footings

- Linear soil pressure distribution is assumed.
- Soil cannot bear any tensile load (uplift pressure).
- If uplift is unavoidable, the length of uplifted zone shall be limited to 0.25 of the length of foundation in that direction. It is equivalent to e<L/4. (Prove it)



در طراحی پی اجازه می‌دهند که $\frac{1}{4}$ پی از جایش بلند شود.

$e_2 = \frac{L}{6}$

kern boundary

Design Considerations

- Soil bearing capacity → size of foundation
- Permissible settlements (uniform and differential)
- Strength requirements
- Frost line consideration → عمق یخ زدگی
- Underground water depth ← سطح آب زیرزمینی

سطح یخ زدگی جایی است که هیچ مقاومتی نمی‌توان برای پی در آنجا در نظر گرفت و پی باید زیر frost line قرار گیرد.

Soil Bearing Capacity

- Allowable soil bearing capacity is determined from geotechnical tests.
- If no data are available for preliminary designs, the following table may be used.
- Iranian engineers like to express q_a in kg/cm².
- What factor of safety is considered for q_a?

Rock or soil	Typical bearing value (kN/m²)
Massive igneous bedrock	10 000
Sandstone	2000 to 4000
Shales and mudstone	600 to 2000
Gravel, sand and gravel, compact	600
Medium dense sand	100 to 300
Loose fine sand	less than 100
Hard clay	300 to 600
Medium clay	100 to 300
Soft clay	less than 75

رس نرم پایین‌ترین ظرفیت را دارد.

ظرفیت را معمولاً برحسب kg/cm^2 اندازه‌گیری می‌کنند.

load controll
displacement controll

در ASTM دیده شود

Strength Requirements

- One-way (beam) shear control
- Punching (two-way) shear control
- Bending moment strength and required steel reinforcement
- Bearing capacity at column base
- Dowel and starter bars — میلگرد انتظار
- Bars development and anchorage

طول مهار و گیرداری

Size of Footing Base

- Size of footing base is determined from the soil stress distribution and allowable soil capacity, i.e. $q_{max} < q_{allowable}$.
- Soil bearing capacity is given at a certain depth. Thus, for other depths where the foundation base locates, the weight of excavated or filled volume as well as imposed surcharge shall be considered (effective bearing capacity q_e).
- For concentrically loaded footings, the required area of footing based is obtained as

کل بار

$$A_{req} = \frac{\text{Total service load including selfweight \& surcharge}}{\text{net allowable soil capacity}} = \frac{P_{service}}{q_e}$$

q_{net}

Size of Footing Base

- The service load is the algebraic sum of existing loads in different load combinations, i.e. set all load factors to unity. For example,

  $D, \quad D + L, \quad D + L \pm E, \quad D + L \pm W$

- In extreme events, it is permitted to increase the soil bearing capacity by 1.33 factor. Why?

مثل زلزله و باد

در بارگذاری سریع مقاومت بتن بالا رفته است و چون بار باد و زلزله سریع رخ می دهد بنابراین می توان تخفیف داد

- For strength calculations we need ultimate soil pressure $q_u = P_u/A$.

Minimum Footing Depth

- Depth of footing above bottom reinforcement shall not be less than
  - 150 mm for footings on soil
  - 300 mm for footings on piles

در ایران پی با ارتفاع زیر 40 cm اجرا نمی شود

- Considering the minimum concrete cover for bottom bars, the overall thickness of shallow footings should be greater than 250 mm.

برای ستون‌های مربعی این را نمی‌خواهد چک کنیم.

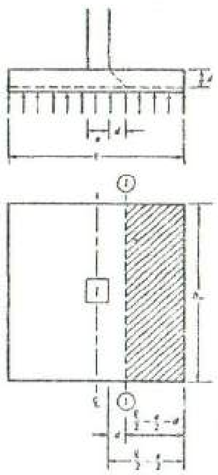
One-Way Shear

- One way shear or beam shear is calculated along a section located at distance d from the column face.
- The design shear capacity must satisfy

$$\phi V_n = \phi V_c = \phi\left(0.17\sqrt{f'_c}\,b_w d\right) \geq V_u$$

where $\phi = 0.75$.

در پی معمولاً میلگرد برشی نمی‌گذارند و آنقدر ضخامت بتن را زیاد می‌کنند که خودش بتواند برش را تحمل کند.



Punching Shear Capacity Provided by Concrete

- Punching shear capacity V_c

$$V_c = \min\begin{cases} 0.17\left(1+\frac{2}{\beta}\right)\sqrt{f'_c}\,b_0 d & \text{Not governing if } \beta < 2.1 \\ 0.083\left(\frac{\alpha_s d}{b_0}+2\right)\sqrt{f'_c}\,b_0 d & \text{Not governing if } d/b_0 > 0.1 \\ 0.33\sqrt{f'_c}\,b_0 d \end{cases}$$

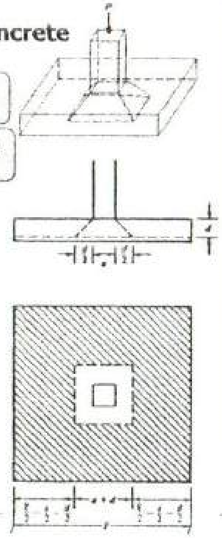
β = the ratio of long side to short side of the column

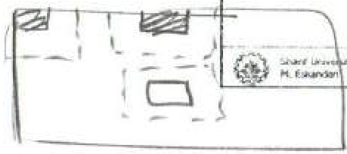
α_s = 20, 30, and 40 for corner, edge, and interior columns, respectively

ضریبی از تعداد وجه‌هایی است که ستون می‌تواند از آنجا بشکند.

b_0 = perimeter of the critical section

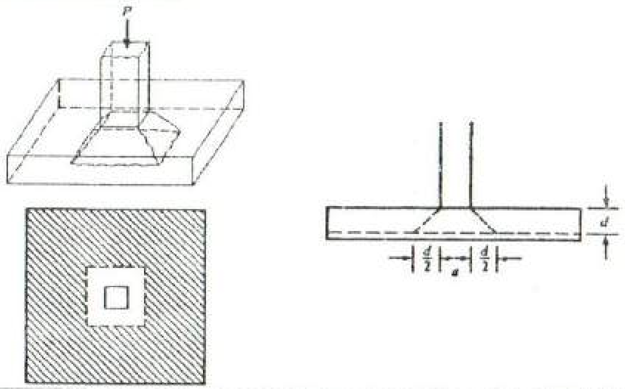
d = effective depth



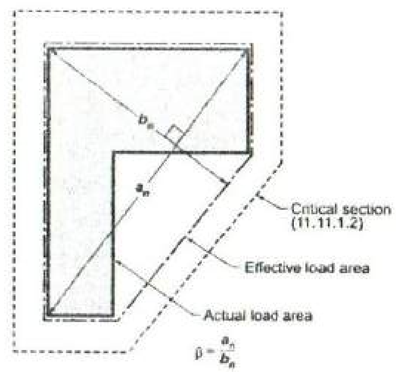


Punching Shear (Two-Way Shear)

- The critical section for two-way shear is located at a distance d/2 from the face of the column.

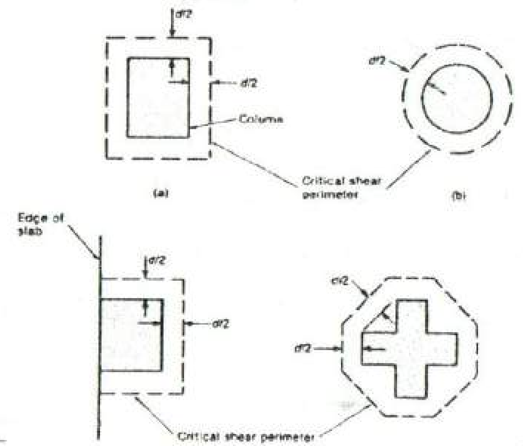


Critical Section of Punching Shear



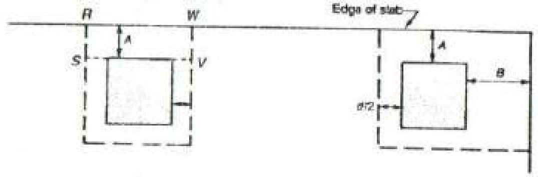
بستگی به آرماتورهای پی دارد مثلاً Φ20 بود l_d را برای Φ20 بدست می‌آوریم.

Critical Section of Punching Shear (Some Examples)

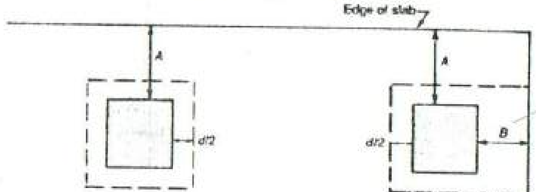


Critical Section of Punching Shear Near Edges

- A and B do not exceed the greater of $4h$ or $2l_d$

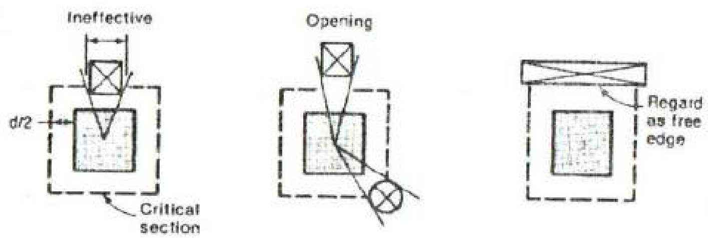


- A exceeds the greater of $4h$ or $2l_d$, but B does not.



چون B کمتر از 4h و l_d است این مسیر برش خورده است.

Critical Section of Punching Shear Near Openings



Punching Shear (Cont.)

- The design punching shear capacity must satisfy

$$\phi V_n = \phi V_c \geq V_u$$

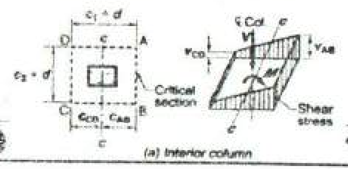
where $\phi = 0.75$.

- V_u is the factored shear force acting on the critical section, e.g. for a square column of side c under axial load P_u we have

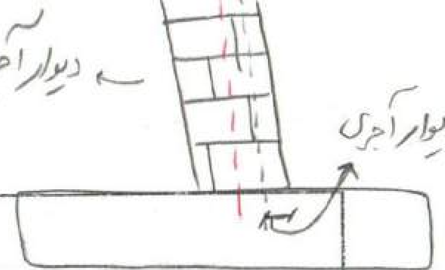
$$V_u = P_u - (c + d)^2 q_u$$

- If bending moment is also present at the column base, the effects of eccentric shear shall be considered.

$$v_{u(AB)} = \frac{V_u}{A_c} + \frac{\gamma_v M_u c_{AB}}{J_c}$$



$$v_{u(CD)} = \frac{V_u}{A_c} - \frac{\gamma_v M_u c_{CD}}{J_c}$$



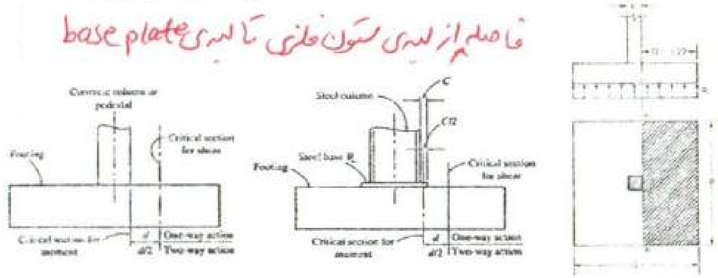
Shear Reinforcement

- In either one-way or punching shear control, if shear reinforcement consisting wires or stirrups is required, the design capacity becomes
$$V_n = V_c + V_s \leq 0.5\sqrt{f'_c}b_0 d$$
where $V_c = 0.17\sqrt{f'_c}b_0 d$.
- Use of shear reinforcement in foundations is not common.
- The depth of foundations is usually controlled by shear capacity.

Critical Sections for Bending Moments

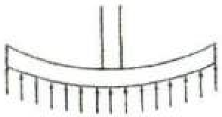
- At the face of concrete columns and pedestals
- Half-way between the middle and edge of masonry walls
- At distance half-way from the edge of base-plate and steel column face for footings supporting steel columns

فاصله از لبه ستون فلزی تا لبه base plate

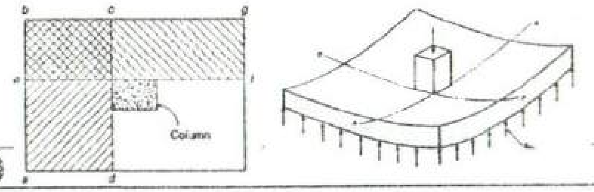


Bending Moments

- Bending moments in a footings are resulted from the ultimate soil pressure acting on the footing base. In fact, the foundation behaves like an inverted beam supported by columns.



- In most types of footings like single or mat foundations bending moments are developed in both directions (two-way action).



Required Area of Flexural Reinforcement

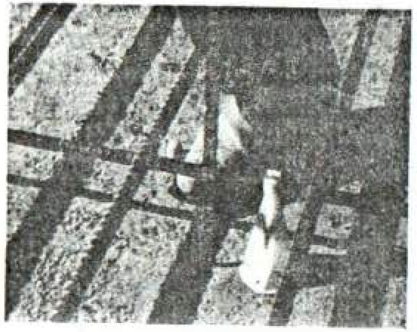
- Required area of reinforcement is calculated like singly reinforced rectangular beams, i.e.

$$A_s = \frac{M_u}{\phi f_y\left(d - \frac{a}{2}\right)}, \qquad a = \frac{A_s f_y}{0.85 f'_c b}$$

اگر پی مربعی باشد توزیع میلگردها یکنواخت است ولی اگر مستطیلی باشد در راستای بلند آن در نزدیکی ستون باید میلگرد بیشتری بگذاریم چون در آنجا تمرکز تنش داریم و برخلاف فرض ما یکنواخت نیست.

Clear Concrete Cover for Cast-in-Place Foundations

- 75 mm for faces in contact with ground
- 50 mm for other faces



Maximum and Minimum Spacing of Bars

- Maximum center-to-center spacing of reinforcing bars shall be not greater than

$$s_{max} = min(3h, 450\ \mathrm{mm})$$

where h is the footing thickness.

- Minimum clear spacing between parallel bars is as follows:

$$s_{min} = max(d_b, 25\ mm, 1.33\ d_c)$$

where

d_b = bar diameter

d_c = nominal maximum size of coarse aggregate

- The minimum clear spacing shall be also satisfied at rebar lap zones.
- It is recommended to select center-to-center spacing of bars greater than 100 mm.

اگر میلگرد حداقل نگذاریم مقطع ترد تشکیل می‌شود

Minimum Reinforcement

حداقل میلگرد طولی همان حداقل میلگرد حرارتی است.

- Based on ACI code, for footings and slabs the provided reinforcement shall be not less than the required shrinkage and temperature reinforcement.
- In spite of ACI code, some engineers believe that the minimum flexural reinforcement of beams should be considered in footings. (Why?)

در تیر می‌خواهیم مقطع ترد نشود

$$A_{s,min} = \frac{0.25\sqrt{f'_c}}{f_y} b_w d \geq \frac{1.4 b_w d}{f_y}$$

- The calculated area of reinforcement based on the developed moments in footing is usually far less than the minimum reinforcement.
- If the provided reinforcement is greater than 1.33 of calculated area, it is permitted to neglect the minimum required reinforcement.

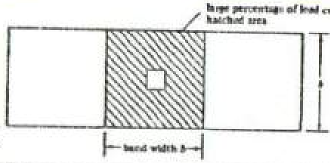
اگر $\frac{1}{3}$ بیشتر از A_{sreq} را تامین کنیم دیگر نیازی به تامین $A_{s\,min}$ نیست.

Distribution of Reinforcement

- In one-way footing and two-way square footings reinforcement shall be distributed uniformly across entire width of footing.
- In two-way rectangular footings, reinforcement shall be distributed as follows:
  - Uniformly in long direction
  - γA_s shall be distributed uniformly across a band width b (centered on the column centerline) in short direction
  - $(1-\gamma)A_s$ shall be distributed uniformly outside the bend width in short direction.

$(1-\gamma)A_s$

$$\gamma = \frac{2}{\beta + 1}, \qquad \beta = \frac{\text{long side of footing}}{\text{short side of footing}}$$



در ستون‌های گوشه احتمال پانچ شدگی بسیار زیاد است.

$\frac{A_2}{A_1} \leq 4$

لهیدگی

Bearing Capacity at Column Base

- Column compressive load is transferred to the footing by bearing on concrete.

$$P_u \leq \phi N_n = \phi(0.85)f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \quad \sqrt{\frac{A_2}{A_1}} \leq 2$$

where

A_1 = loaded area

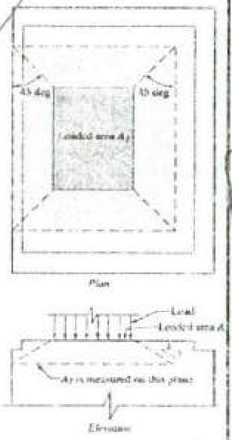
A_2 = area of lower base of load path pyramid

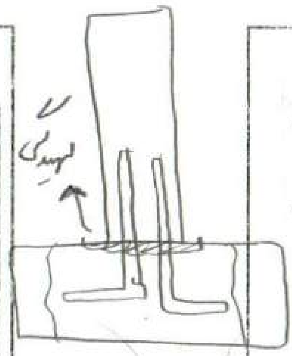
$\phi = 0.65$

- The bearing capacity of column at the footing base shall be also controlled

$$P_u \leq \phi N_n = \phi(0.85)f'_c A_1$$

- It becomes critical when columns made of high strength concrete rest on foundations of low grade concrete.
- If bearing capacity is not sufficient, the remained load shall be transferred by dowels.





$N_{nd} = \frac{P_u - \phi N_n}{\phi}$

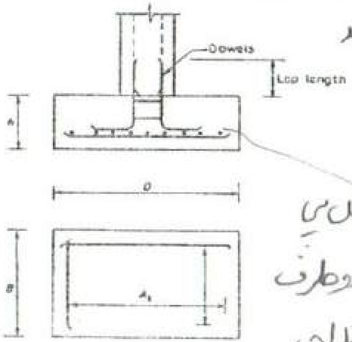
$N_{nd} = f_y A_s$ → باقی مقاومت فشاری را میلگردهای dowel باید تامین کند.

Development and Anchorage of Bars

- Dowels and starter bars shall be properly developed or anchored in both column and foundation.
- Flexural reinforcement shall be properly developed at critical sections.

حداقل dowel bars باید 4 تا باشد

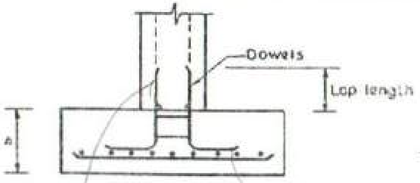
تا 30 cm بالاتر خاموت گذاری کنیم.



باید دقت کرد میلگردهای طولی داخل پی باید به اندازه‌ی l_d گیرداری از دو طرف داشته باشد و طول l_d باید در طراحی ابعاد پی لحاظ گردد.

Dowels and Starter Bars

- For columns under compressive load, the following requirements for dowel bars shall be satisfied:
  - At least four dowel bars shall extend to the column. Dowel bars should be placed at column corners.
  - The area of dowel bars shall be at least 0.005 of gross area of column section
  - Dowels shall extend to columns at distance equal to the development length of dowels in compression or lap length of column bars whichever is greater.
  - It is common and recommended to use the same column bars as starter bars with a minimum extension into column equal to the tension lap length.

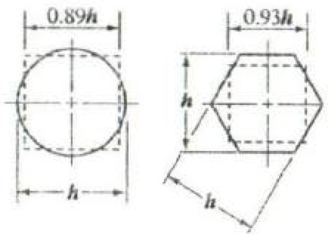


طول بیرون آمدگی آرماتور انتظار l_d کششی در نظر گرفته می‌شود.

اگر ستون معمولی باشد و دارای base plate تا ب فولادی که پای ستون متصل باشد و به بیرون زنند.

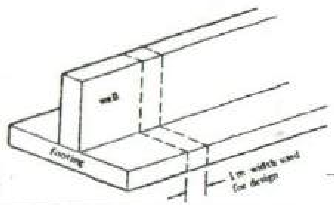
Circular or Regular Polygon Shaped Columns

- In order to determine the critical sections for moment, shear, and development of reinforcement in footings supporting circular or regular polygon shaped columns, it is permitted to treat them as square members with the same area.



Design of Wall Footings

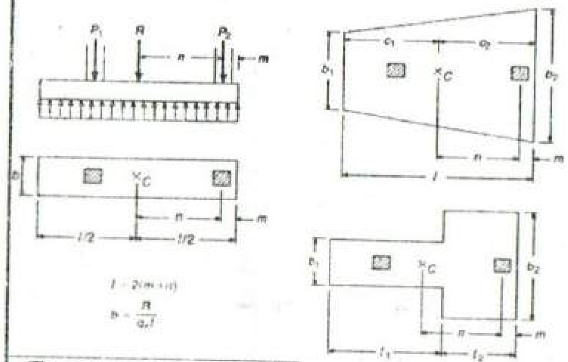
- Wall footings behave like one-way slabs.
- Design controls are done for a unit width of the footing.
- Minimum design controls, but not limited to:
  - Soil bearing capacity
  - One-way shear
  - Flexural reinforcement in short direction
  - Shrinkage and temperature reinforcement in wall direction
  - Dowels: Minimum vertical bars located in walls are sufficient.
  - Development and anchorage of bars



اگر بار لبه‌ی دیوار بیشتر باشد از این نوع پی استفاده می‌شود

Design of Combined Footings

- If the centroid of footing coincides with the resultant of the two column loads, the induced soil pressure will be uniform across the foundation base.



$l = 2(m+n)$

$b = \frac{R}{q_e l}$

$\frac{b_2}{b_1} = \frac{3(n+m)-l}{2l-3(n+m)}$

$(b_1+b_2) = \frac{2R}{q_e l}$

$c_1 = \frac{l(b_1+2b_2)}{3(b_1+b_2)}$

$c_2 = \frac{l(2b_1+b_2)}{3(b_1+b_2)}$

$b_1 = \frac{R}{q_e}\left[\frac{2(n+m)-l_2}{l_1(l_1+l_2)}\right]$

$b_2 = \frac{R}{l_2 q_e} - \frac{l_1 b_1}{l_2}$

$l_1 b_1 + l_2 b_2 = \frac{R}{q_e}$

استخوانی

Design of Single Footings

- Single footings have two-way behavior.
- Minimum design controls, but not limited to:
  - Soil bearing capacity
  - Punching shear
  - One-way shear
  - Flexural reinforcement
  - Distribution of reinforcement
  - Bearing capacity at column base
  - Dowels and starter bars
  - Development and anchorage of bars

هم برای پانچینگ خوب است هم در مقطع بحرانی خمش مقطع بزرگتر بهتر کار می‌کند ولی قالب گذاری آن سخت تر است و گران تر

Design of Combined Footings (Cont.)

- Design controls are similar to single footings.
- In long direction, both shear and moment diagrams should be drawn.
- In short direction, the effective width for each column is considered as the column width plus $d/2$ on each side of the column.

عرض تیرک

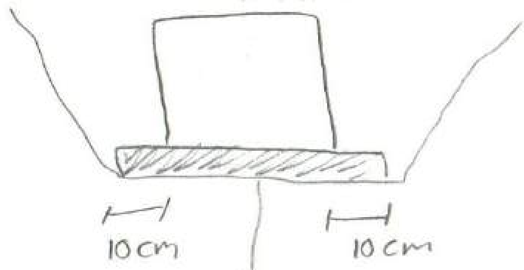
موثر است. این فقط مقطع

شکل تیر طراحی کرده و آرماتورگذاری می‌کنیم

Practical Considerations

- It is very common to level the base of excavated footing by a thin layer of lean concrete of thickness 100 mm. What is lean concrete?

بتن مگر



هم بحث تسطیح سازی است و هم برای رعایت نظافت و جلوگیری از کثیف شدن سطح میلگردها می باشد.

- Low strength concrete is commonly used in footings, say $f'_c = 21$ MPa. However, in special cases, high strength concrete shall be used. Minimum f'_c for structural members?

بتن سازه ای $f'_c=17$ است ولی برای طراحی در سازه ای $f'_c=21$ است و در جاهایی که خوردگی هم داریم f'_c را برای نفوذ پذیری بالا می برند.

- For footings subjected to light compressive loads like those supporting masonry walls, it is permitted to use plain concrete.
- Excavation method shall be considered.
- Round foundation dimensions to multiples of 50 mm.

ابعاد پی به 5cm ها گرد می شوند و حتی به 10 cm مهار می شود.

چون پی سطحی را با فنر مدل می کنند.

Other Topics

- Modeling of soil by equivalent springs (k_s)

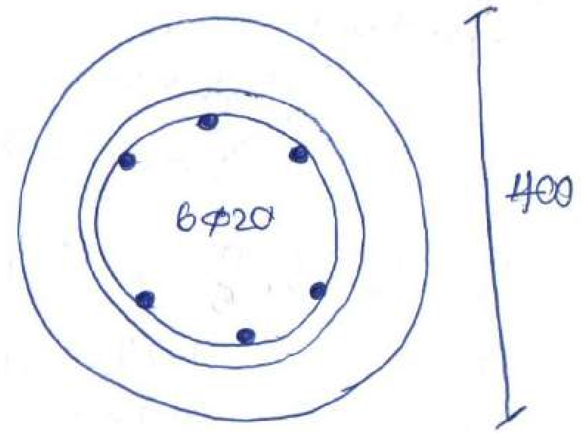
با استفاده مدل plate و بارگذاری plate load test استفاده می شود.

K_s $[N/m^3]$

- Analysis of footing by FEM programs
- Pile foundations and pile caps
- Foundations subjected to horizontal loads
- Design of strap beams
- Seismic provisions for footings

example1 slide 7

A tied 400 mm circular column with $f'_c = 30$ MPa carries a dead load of 800 KN & an imposed live load of 300 KN. the column should be supported by a square footing whose base is 1.2 m below grade. the surcharge acting on the grade is $w_D = 3\ KN/m^2$ & $W_L = 2\ KN/m^2$. Design The footing.

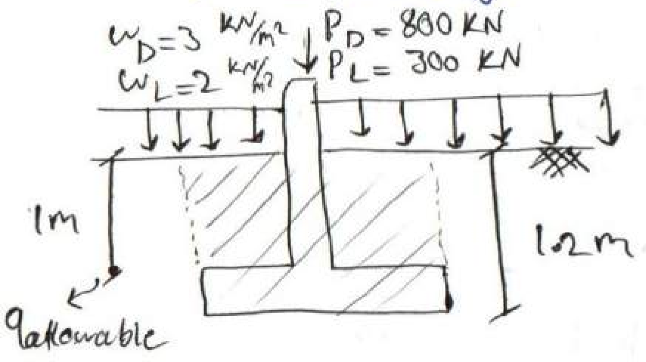


Given : $f'_c = 21$ Mpa

Grade AIII reinf

$q_a = 210\ KN/m^2$ at level 1 m below the grade.

$\gamma_s = 18\ KN/m^3$



* size of Base :

$P_{service} = P_D + P_L = 1100$ KN

$q_e = q_a + (1.2-1)\gamma_s - w_D - W_L - (21)(1.2) = 183.4\ KN/m^2$

$A_{req} = \dfrac{P_{service}}{q_e} = \dfrac{1100}{183.4} = 6.00\ m^2$

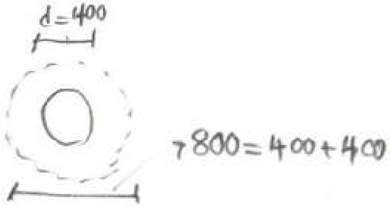
$B = \sqrt{6} = 2.49$ m $\longrightarrow$ select 2.5x2.5 m square footing

ultimate load & pressure

$P_u = 1.2 P_D + 1.6 P_L = 1440$ KN

$q_u = \dfrac{P_u}{A_f} = 230.4\ KN/m^2$

Height of footing, h



- punching shear

Assume $d = 400$ mm b_o = perimeter of the crital section

$b_o = \pi(400+400) = 2512$ mm

$V_u = P_u - q_u \frac{\pi}{6}(0.800)^2 = 1324$ KN

$V_{nreq} = \frac{V_u}{\phi} = \frac{1324}{0.75} = 1765$ KN

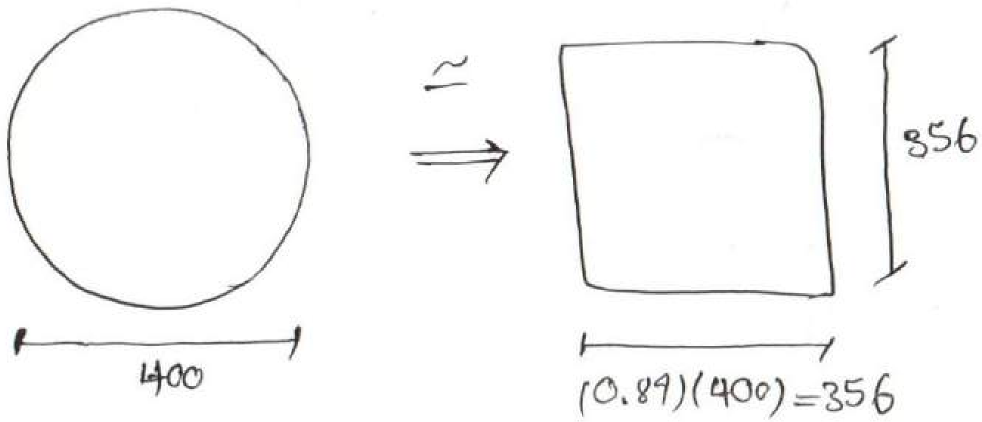
$V_{c_1} = 0.17\left(1+\frac{2}{\beta}\right)\sqrt{f'_c}\, b_o d$ ⟶ برای پی مربع اثر را نیز خواهد

$V_{c_2} = 0.083\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d$ $\alpha_s = 40$

$1765 \times 10^3 = (0.083)\left(\frac{40d}{2512}+2\right)\sqrt{21}\,(2512) \longrightarrow d_{req} = 285$ mm

$V_{c_3} = 0.33\sqrt{f'_c}\, b_o d \Longrightarrow d_{req} = \frac{1765\times10^3}{(0.33)(\sqrt{21})(2512)} = 464$ mm

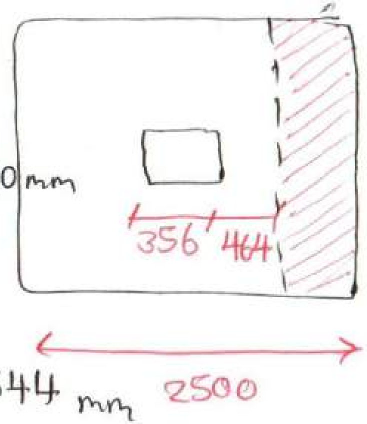
governs



$V_u = (230.4)(2.5)\left(\frac{2.5}{2} - \frac{0.356}{2} - 0.464\right) = 350$ KN

$V_c = 0.17\sqrt{f'_c}\, b_o d$

$d_{req} = \frac{350\times10^3}{(0.75)(0.17)(\sqrt{21})(2500)} = 240$ mm



$h = d + \text{cover} + d_b + \frac{1}{2} d_b$

$= 464 + 50 + 20 + (\frac{1}{2})(20) = 544$ mm

select $h = 550$ mm

Flexural reinf
Temp & shrinkage reinf
Min flexural reinf
spacing of bars
Bearing capacity of footing
Dowels = $0.005 \times Ag_{column}$
L_d → for footing bars
L_{dc} → for dowels
lap or L_s for dowels
sketch

طراحی سازه‌های بتنی ۲ — امتحان پایان‌ترم

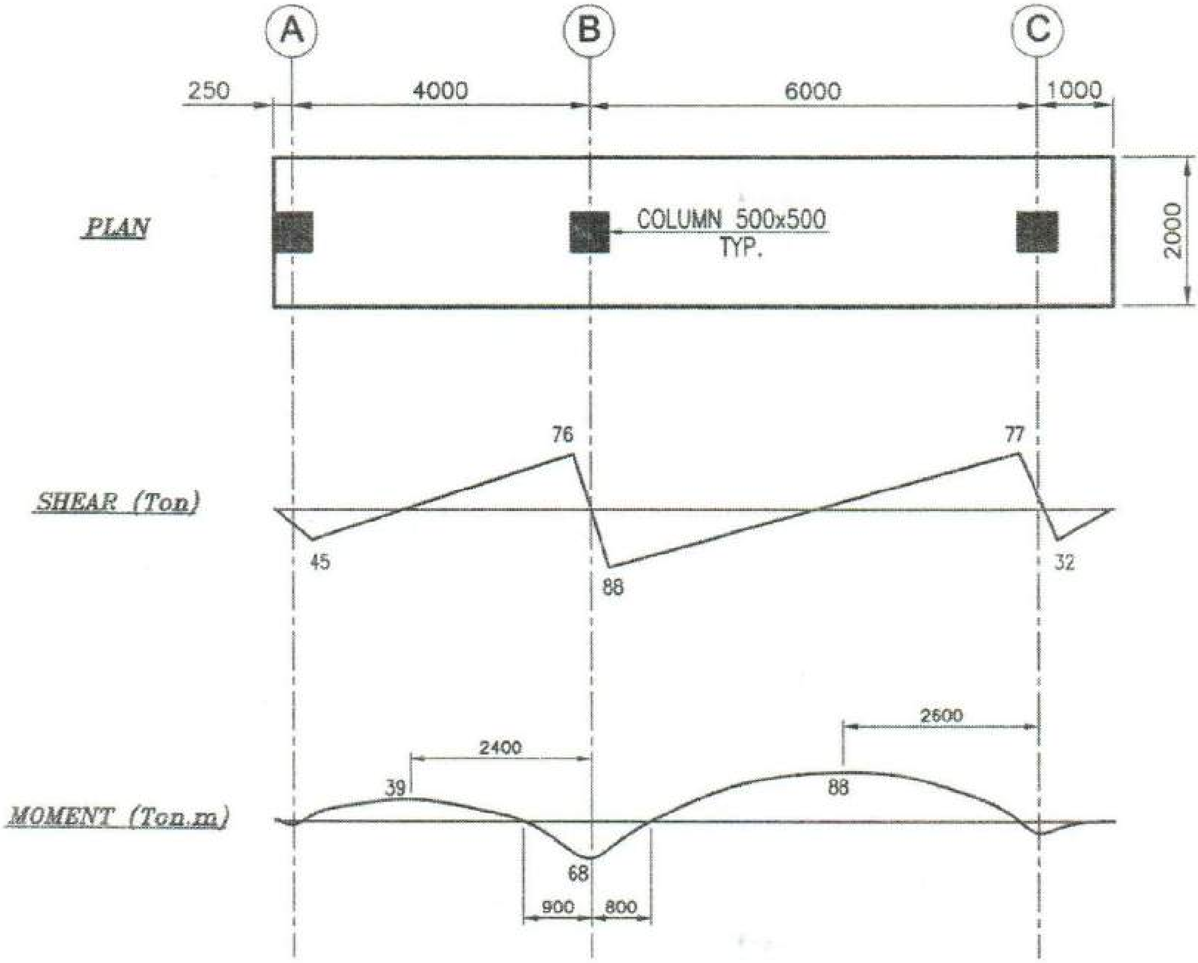
مدرس: مرتضی اسکندری — نیمسال دوم ۹۳-۹۲

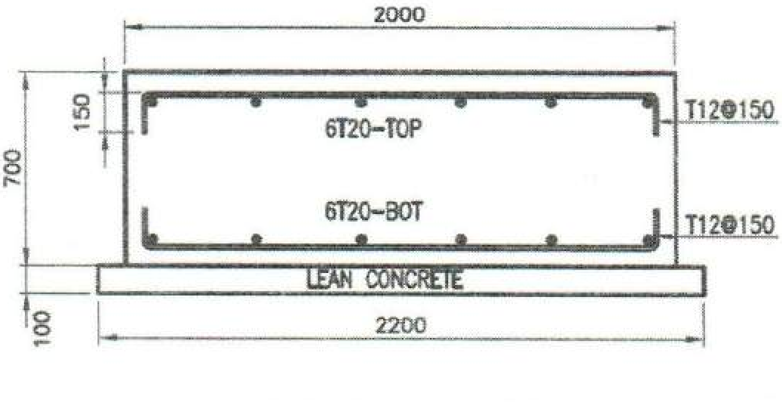
کلیه اندازه‌های نشان داده‌شده بر روی شکل‌ها به میلیمتر است مگر آنکه واحد دیگری ذکر شده باشد. **وقت: ۱۲۰ دقیقه**

اگر اطلاعاتی داده نشده است، با قضاوت مهندسی خود، مقداری را انتخاب کنید.

بتن را از رده C25 و کلیه میلگردها را A3 در نظر بگیرید.

نام:





TYPICAL SECTION

طراحی سازه‌های بتنی ۲ امتحان پایان‌ترم

مدرس: مرتضی اسکندری نیمسال دوم ۹۳-۹۲

کلیه اندازه‌های نشان داده‌شده بر روی شکل‌ها به میلیمتر است مگر آنکه واحد دیگری ذکر شده باشد. **وقت: ۱۲۰ دقیقه**

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بتن را از رده C25 و کلیه میلگردها را A3 در نظر بگیرید.

نام:

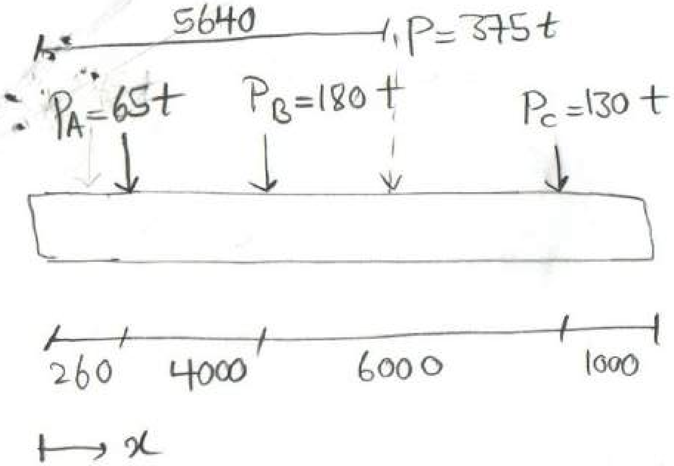
۱- پی نواری نشان داده شده در صفحه ۲ را در نظر بگیرید. نمودار لنگر خمشی و نیروی برشی ضریبدار برای این پی پس از تحلیل در نرم افزار SAFE نشان داده شده است. مهندسی مقطع نشان داده شده را به عنوان مقطع عمومی در نظر گرفته و قصد دارد در جاهایی که لازم است از میلگرد تقویتی استفاده کند. به سوالات زیر پاسخ دهید:

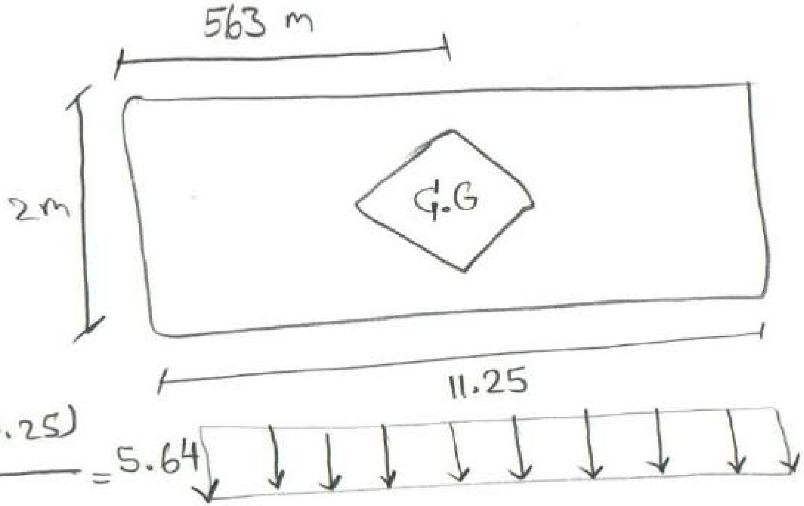
أ. با فرض عملکرد صلب پی، تنش ماکزیمم زیر پی را برای بارهای سرویس داده شده، تعیین کنید. $P_A = 65\ tons,\ \ P_B = 180\ tons,\ \ P_C = 130\ tons$

ب. ظرفیت برش پانچ را زیر ستون C محاسبه کنید.

ت. با فرض کفایت باربری خاک، آیا مقطع ارائه شده توسط مهندس را به عنوان مقطع کلی قبول می‌کنید. توضیح دهید.

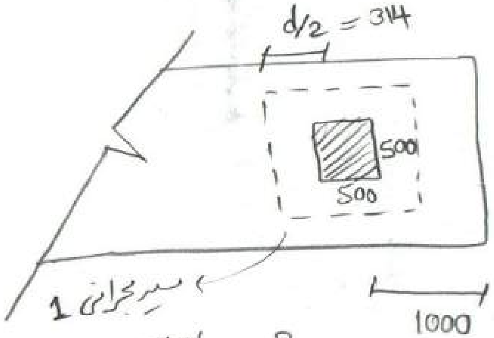
ث. میزان و طول آرماتورهای تقویتی مورد نیاز زیر ستون B را تعیین کنید. از میلگرد ۲۰ استفاده کنید.





$$\bar{x}=\frac{(65)\times(0.25)+(180)(4.25)+(130)(10.25)}{65+180+130}=5.64$$

$$q_{Max}=\frac{375\times10^3}{(11.25)(2)(10^4)}=1.67\ kg/cm^2$$



$\phi V_c = ?$

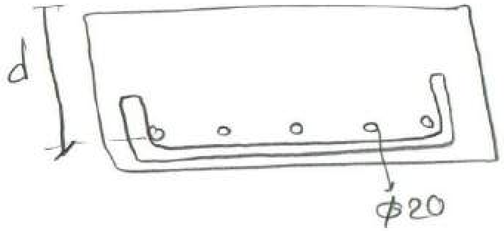
$V_{c_1}=N.A$

$$V_{c_2}=0.083\left(\frac{\alpha_s d}{b_\circ}+2\right)\sqrt{f'_c}\,b_\circ d$$

$$V_{c_3}=0.33\sqrt{f'_c}\,b_\circ d$$

$d=700-150-12-\frac{20}{2}=628\ mm$

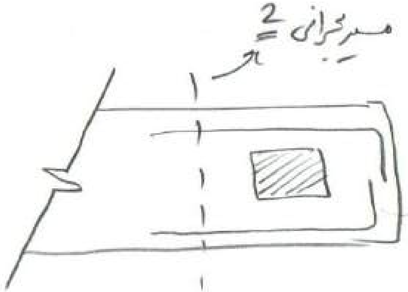
$d/2=314\ mm$



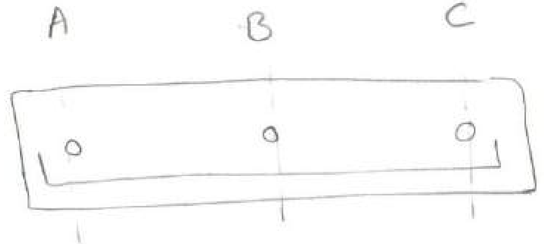
$$l_d\ \alpha\left[\frac{\psi_t\psi_e\psi_s\lambda}{1.1\sqrt{f'_c}\left(\frac{c_b+k_{tr}}{d_b}\right)}\right]f_y d_b$$

(تقریبی می‌گیریم) $l_d \simeq 30d_b = 600\ mm$

حالتی که پانچ 1 → 2 می‌شود



در این‌جا به دلیل ابعاد خاص می‌توان میلگردها را خم کرد بنابراین به جای $2l_d$ از $2l_{dh}$ استفاده می‌گردد.



critical path 1

$$b_o = 4(500+628) = 4512 \text{ mm}$$

$$V_{c_2} = 0.083\left(\frac{(40)(628)}{4512}+2\right)(\sqrt{25})(4512)(628) = 8898 \text{ kN}$$

$$V_{c_3} = (0.33)(\sqrt{25})(4512)(628) = 4672 \text{ kN}$$

$$\phi V_c = 350 \text{ ton}$$

critical path 2

$$b_o = 2000 \rightarrow \phi V_c = 155 \text{ ton}$$

یکبار مسیر مستطیلی را حل می کنیم / یکبار در فاصله ی $\frac{d}{2}$ خط راست را حل می کنیم اما در عمل هیچ کدام از این دو کنترل نمی کند و باید برش یک طرفه را بررسی کرد. ← $0.17\sqrt{f'_c}\, b_w d$

ردیف میلگرد ↑

ج) Transverse reinforcement

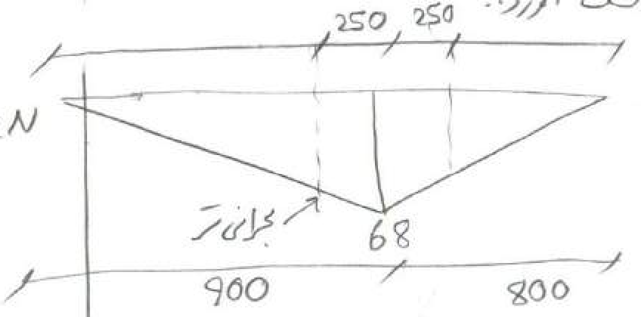
$$2\,\phi 12@150 = 2(754)\text{ mm}^2 = 1508 \text{ mm}^2 > 1260 \quad \text{OK}$$

$$A_{temp} = (0.0018)(1000)700 = 1260 \text{ mm}^2$$

میلگردهای تقویتی باید سمت بالای پی قرار گیرد.

ستون ۴ حکم تکیه گاه برای پی را بازی می کند.

باید M_u را در بر ستون محاسبه گردد فقط باید دقت کرد که به اندازه ی $\frac{d}{2}$ از طرفین فاصله گرفت و لنگر را بدست آورد.



عرض ستون 500 mm است به اندازه ی نصف آن یعنی 250 از وسط ستون فاصله می گیریم.

$$M_u = 68 \text{ t.m}$$

$$\frac{x}{68} = \frac{650}{900} \rightarrow x = 49 \text{ t.m}$$

$$A_{s\,req} = \frac{M_n}{\phi f_y (d-\frac{a}{2})} = \frac{49\times10^7}{(0.9)(400)(628-10)} = 2200 \text{ mm}^2$$

$a = 20$ mm

$$a = \frac{(2200)(400)}{(2000)(0.85)(25)} = 21 \text{ mm OK}$$

$6\phi 20 \rightarrow 1884 \text{ mm}^2$
+
$1\phi 20 \rightarrow 2198 \text{ mm}^2$
OK

باید یک میلگرد $\phi 20$ به قسمت بالای پی اضافه گردد تا به مساحت مورد نیاز برسیم 6 تا کافی نیست باید 7 تا باشد.

8

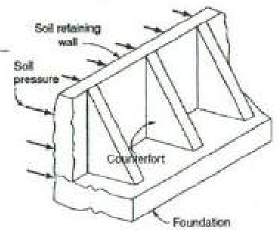
Shear Walls

Based on ACI 318M-11

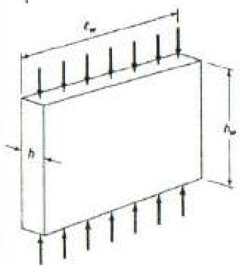
Spring 2015

Main Types of Concrete Walls

- Bearing walls
- Shear walls
- Retaining walls (Nonbearing walls)



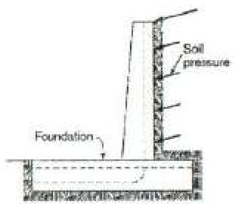
Counterfort retaining wall



Bearing wall

Shear wall

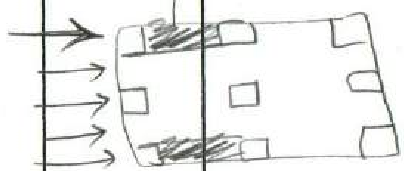
$I_{xx} >> I_{yy}$



Cantilever retaining wall

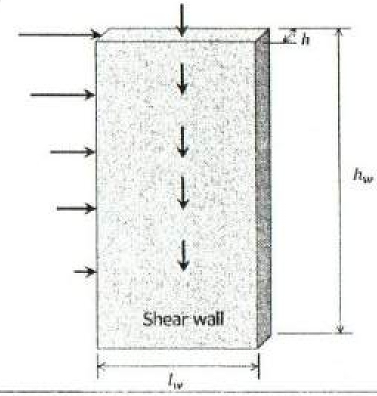
دیوارهای حائل

دیوار برشی



Shear Walls

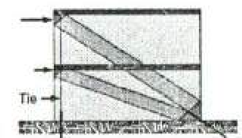
- Shear wall is a structural wall that resists the lateral earthquake or wind loads acting in its plane in the presence of axial gravity loads applied along its longitudinal axis.



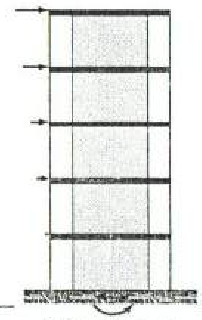
Behavior of Shear Walls

رفتار آنها برشی است و نیروی جانبی را با برش تحمل می کنند

- Short or squat shear walls $\left(\frac{h_w}{l_w} \le 2\right)$
  - The structural behavior is dominated by shear.
- Flexural or slender shear walls $\left(\frac{h_w}{l_w} \ge 3\right)$
  - The lateral loads are resisted mainly by the flexural action like a cantilever beam.

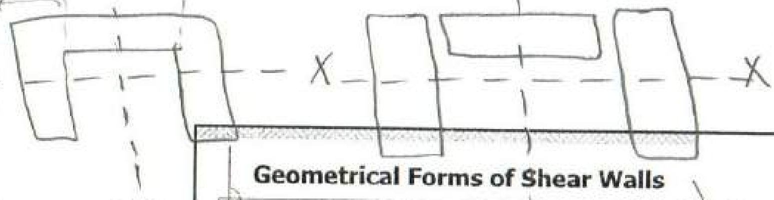


(a) Squat shear wall.



(b) Slender shear wall.

اینو منطق برش تحمل می‌کنه



(1) در این حالت $\frac{VQ}{It}$ را تحمل می کند

(2)

Geometrical Forms of Shear Walls

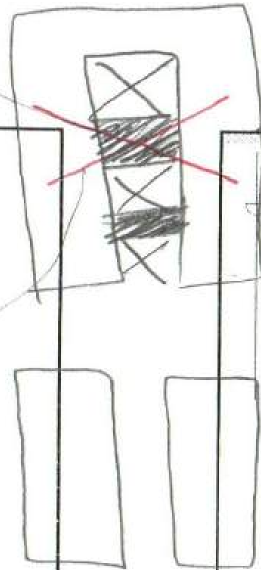
- Rectangular
- Wall Assemblies
  - L-Shaped
  - C-Shaped
  - T-Shaped
  - H-Shaped
  - Hollow Box
- Coupled Shear Walls

Note: For wall assemblies, special controls are required at the walls junctions.

برش را تحمل نمی کند و فقط خمش را می تواند تحمل کند.

و آرماتور ضربدری قرار می دهند تا جدا نگردد.

link beam



دو دیوار برشی مجزا

اگر از لینک beam استفاده گردد دو تا دیوار با هم کار می کنند ولی آن تیر لینک باید برش زیادی را تحمل کند

می توان از دو دیوار برشی جدا استفاده کرد

Minimum Thickness

Chapter 21

- ACI code does not determine the minimum thickness of shear walls.
- It is recommended to consider the thickness of shear walls not less than 1/20 of the unsupported height of walls.

1/15 توصیه بهتر است

- To facilitate the construction of shear walls, the minimum thickness of 150 mm is suggested.

زیر 20 cm عملاً امکان اجرا نمی باشد.

پیچش می دهد.

CG

باید ساختمان متقارن باشد تا پیچش نلرزد.

سختی کم

سختی زیاد

سازه

طبقه

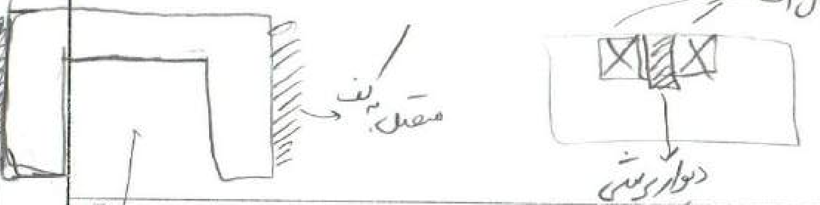
متصل به کف

Layout of Shear Walls in a Building Floor Plan

- It is desirable to consider the following points for the location of shear walls in the plan
  - Symmetric layout about both principal axes of the building
  - Connected to the floor from both sides (at least one side must be connected)
  - Subjected to large gravity loads
  - Providing torsional rigidity
  - Consider the architectural limitations (doors, windows, parkings, etc.)

دیوار برشی باید به کف سازه متصل شود

دیوار برشی های کناری بار ثقلی دارند و تحت لنگر از جا کنده نمی شوند



چال آسانسور

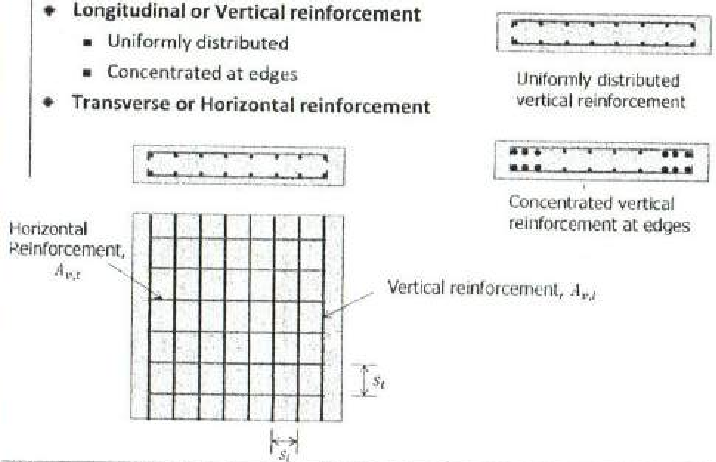
متصل به کف

دیوار برشی

چال آسانسور

Shear Walls Reinforcement

- Longitudinal or Vertical reinforcement
  - Uniformly distributed
  - Concentrated at edges
- Transverse or Horizontal reinforcement



General Requirements of Concrete Walls

- Refer to the Section 14.3 of ACI code to see the requirements of reinforcement in concrete walls. left to you
- These requirements shall be met is all concrete walls including shear walls. in
- The reinforcement of shear walls shall also satisfy other requirements discussed later in this lecture.

Shear Strength

- The shear strength of wall is provided by both the concrete and horizontal shear reinforcement

$$\phi V_n \geq V_u$$

$$V_n = V_c + V_s \leq 0.83\sqrt{f'_c}hd$$

در اینجا اثر نگر دیده شده است.

where $\phi = 0.75$ and d is the effective depth of section.

- It is permitted to take $d = 0.8l_w$.

Shear Strength Provided by Concrete (Simplified Approach)

- For walls under shear and compression

$$V_c = 0.17\sqrt{f'_c}hd$$

- For walls under shear and tension force, N_u (negative in tension)

$$V_c = 0.17\left(1 + \frac{0.29N_u}{A_g}\right)\sqrt{f'_c}hd$$

where A_g is the gross cross sectional area of wall.

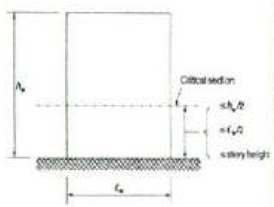
Shear Strength Provided by Concrete (Detailed Approach)

$$V_c = min(V_{c1}, V_{c2})$$

where

$$V_{c1} = 0.27\sqrt{f'_c}hd + \frac{N_u d}{4l_w}$$

$$V_{c2} = \left[0.05\sqrt{f'_c} + \frac{l_w\left(0.1\sqrt{f'_c} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right]hd$$



- If $\frac{M_u}{V_u} - \frac{l_w}{2} < 0$, the second relation shall not apply.
- See clause 11.9.7 of ACI code for the critical section.

اجازه می دهد M_u را کمی بالاتر از پایین ترین نقطه بگیریم.

Shear Strength Provided by Reinforcement

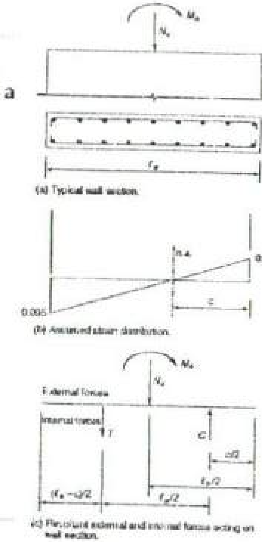
- If $V_u > \phi V_c$, the shear reinforcement is required

$$V_s = \frac{V_u}{\phi} - V_c = \frac{A_{v,t} f_y d}{s_t}$$

where $A_{v,t}$ is the area of transverse shear reinforcement.

Flexural Strength

- When the wall is subjected to pure moment, the flexural strength of wall is determined in a similar way of beams.
- When the wall is under axial force and moment, the wall can be treated as a short column. Either the column interaction diagrams or simplified Tension-Compression methods can be used for analysis or design purposes.



Requirements of Shear Reinforcement

- $\rho_{t,min} = 0.0025, \quad \rho_t = \frac{A_{v,t}}{s_t h}$ مساحت میلگرد افقی — $S \times h$
- $s_{t,max} = min\left(3h, \frac{l_w}{5}, 450\ mm\right)$ فاصله
- $\rho_l = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \geq 0.0025$
- It is not required to have $\rho_l > \rho_t$. → نیازی نیست بیشتر از ρ_t باشد.
- $s_{l,max} = min(3h, \frac{l_w}{3}, 450\ mm)$

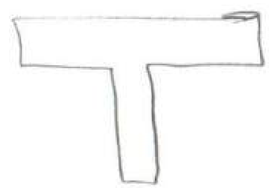
- If $V_u > \frac{\phi V_c}{2}$ the minimum shear reinforcement shall be provided.
- If $V_u < \frac{\phi V_c}{2}$ either the minimum reinforcement of shear walls (given above) or the minimum reinforcement of concrete walls (see Section 14.3 of ACI code) can be provided.

حداقل میلگردی که برای دیوارهای بتنی لازم است را باید قرار دهیم.

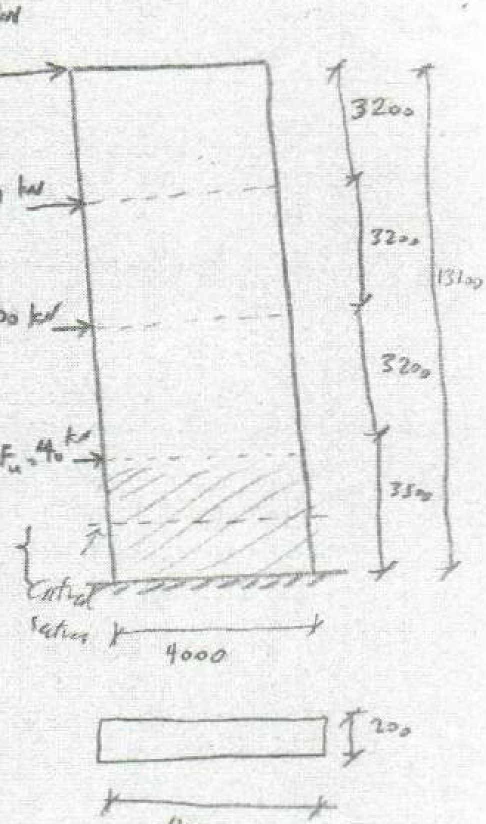
Advanced Topics

- Shear walls with boundary elements → المان های مرزی که دارای ضوابط خاص آرماتورگذاری هستند.
- Coupled shear walls
- Shear wall assemblies
- Seismic provisions

مثلاً



Example (Shear Wall)



$f'_c = 24$ MPa

$f_y = 400$ MPa

Type of Shear Wall

$$\frac{h_w}{l_w} = \frac{13100}{4000} = 3.3 > 3 \Rightarrow \text{flexural shear wall}$$

Factored shear force & Bending Moment

$$V_u = 150 + 140 + 100 + 40 = 470 \text{ kN}$$

$$M_u = (150)(13.1) + (140)(9.9) + (100)(6.7) + (40)(3.5) = 4161 \text{ kN.m}$$

Maximum Permitted Shear Strength

$$(\phi V_n)_{max} = \phi (0.83)\sqrt{f'_c}\, h d \quad , \quad d = 0.8\, l_w = (0.8)(4000) = 3200 \text{ mm}$$

$$= (0.75)(0.83)(\sqrt{24})(200)(3200) = 1952 \text{ kN} > V_u \Rightarrow \text{O.K.}$$

Shear Strength Provided by Concrete (V_c)

* Simplified Approach

$$V_c = 0.17\sqrt{f'_c}\, h d$$

$$V_c = (0.17)(\sqrt{24})(200)(3200) = 533 \text{ kN}$$

* Shear-Flexure Interaction Approach

$$V_c = \min(V_{c_1}, V_{c_2})$$

$$V_{c_1} = 0.27\sqrt{f'_c}\, h d + \frac{N_u d}{4 l_w}$$

$$V_{c_1} = (0.27)(\sqrt{24})(200)(3200) + 0 = 846 \text{ kN}$$

$$V_{c_2} = \left[0.05\sqrt{f'_c} + \frac{l_w\left(0.1\sqrt{f'_c} + 0.2\,\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right] h d$$

Critical Section for V_c

$$x = \min\left(\frac{l_w}{2}, \frac{h_w}{2}, h_{story}\right) = \min\left(\frac{4000}{2}, \frac{15100}{2}, 3500\right) = 2000 \text{ mm}$$

M_u at critical section

$$M_u = (150)(13.1-2) + (140)(9.9-2) + (100)(6.7-2) + (90)(3.5-2)$$

$$\rightarrow \quad M_u = 3301 \text{ kN.m}$$

$$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{3301}{470} - \frac{4}{2} = 5.02 \text{ m} > 0 \quad \text{O.K.}$$

$$V_{c_2} = \left[(0.05)(\sqrt{24}) + \frac{(4000)\left((0.1)(\sqrt{24}) + (0.2)(0)\right)}{5020} \right] (200)(3200)$$

$$\Rightarrow V_{c_2} = 407 \text{ kN} \quad \text{(governs)}$$

$$\Rightarrow V_c = 407 \text{ kN}$$

Horizontal Shear Reinf.

$\phi V_c = (0.75)(407) = 305 \text{ kN} < V_u = 470 \text{ kN} \Rightarrow$ shear reinf is required

$$V_{s\,req} = \frac{V_u}{\phi} - V_c = \frac{470}{0.75} - 407 = 220 \text{ kN}$$

$$\left(\frac{A_{v,t}}{S_t}\right)_{req.} = \frac{V_s}{f_y d} = \frac{220 \times 10^3}{(400)(3200)} = 0.172 \ \frac{\text{mm}^2}{\text{mm}}$$

Try $2\Phi 10 \Rightarrow S_{req} = 913$ mm

$$S_{t,max} = \min\left(\frac{l_w}{5}, 3h, 450\right) = \min\left(\frac{4000}{5}, (3)(200), 450 \text{ mm}\right) = 450 \text{ mm}$$

$\rho_{t,min} = 0.0025$

$A_{t,min} = \rho_{t,min} A_g = (0.0025)(200)(1000) = 500 \text{ mm}^2$

$2\Phi10 \Rightarrow S_{req} = (157)\dfrac{(1000)}{500} = 314 \text{ mm} \Rightarrow$ <u>Select $2\Phi10$ @ 300 for Transverse reinf.</u>

<u>Vertical Shear Reinf.</u>

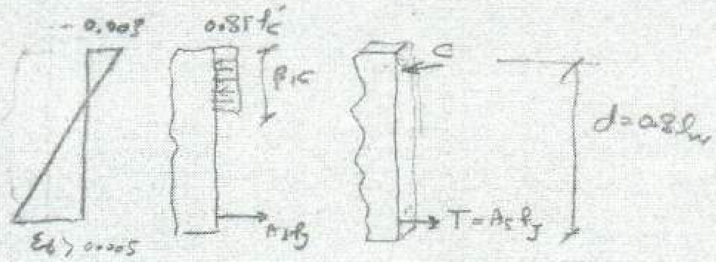
$$\rho_l = 0.0025 + (0.5)\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \geq 0.0025$$

$$\rightarrow \rho_l = 0.0025 + (0.5)(2.5 - 3.3)(0.0025 - 0.0025) = 0.0025$$

$$S_{l,max} = \min\left(3h, \frac{l_w}{3}, 450 \text{ mm}\right) = 450 \text{ mm}$$

$A_{v,min} = 500 \text{ mm}^2 \rightarrow$ <u>Use $2\Phi10$ @ 300 mm</u> as vertical shear reinf.

<u>Design for Flexure</u>



$M_u = 4161 \text{ kN.m}$

$\phi M_n = \phi A_s f_y (d - a/2)$

Assume $a = 300 \text{ mm}$, $d = 3200 \text{ mm}$

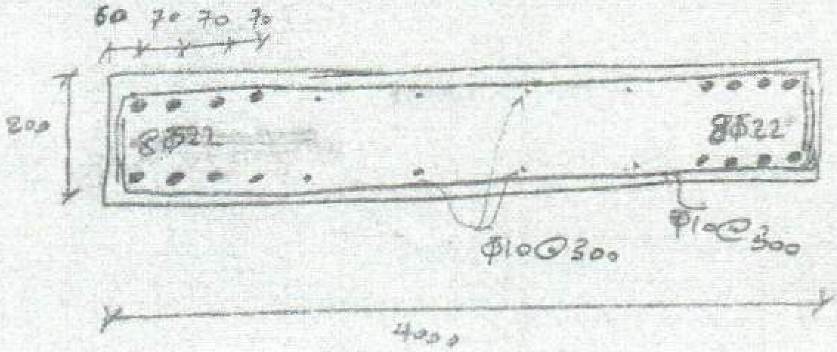
$$A_{s,req} = \frac{4161 \times 10^6}{(0.9)(400)\left(3200 - \frac{300}{2}\right)} = 3790 \text{ mm}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c h} = \frac{(3790)(400)}{(0.85)(24)(200)} = 371 \text{ mm}$$

$$A_{s,req} = \frac{4161 \times 10^6}{(0.9)(400)\left(3200 - \frac{371}{2}\right)} = 3834 \text{ mm}^2$$

<u>Use $8\Phi22$ (3041) ?!</u>

Sketch



Strain Compatibility Analysis

(left to you)

$\phi M_n \simeq 5380$ kN.m

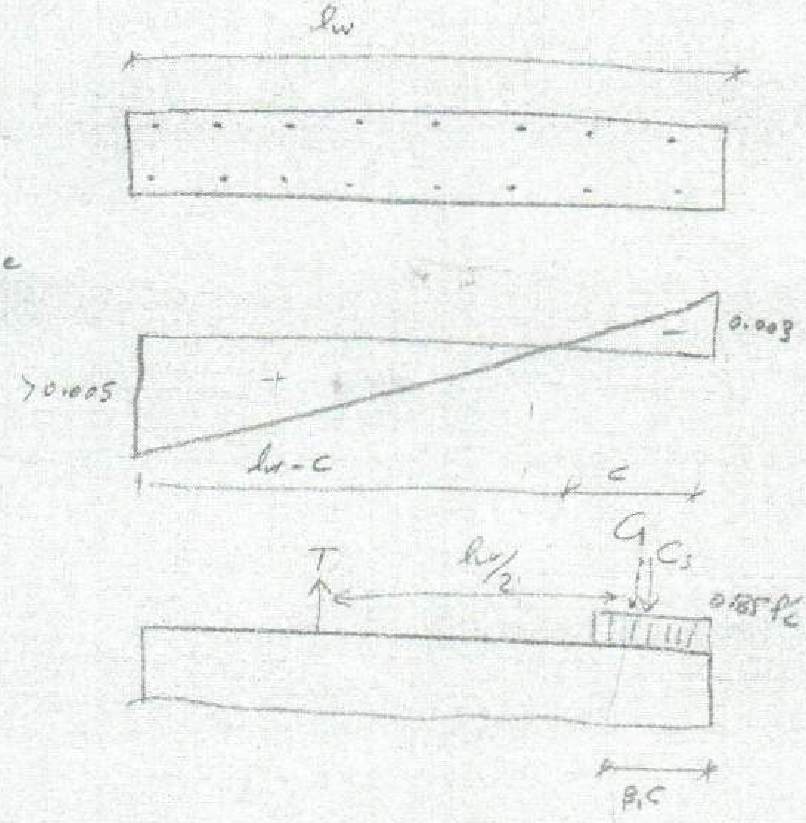
Uniformly Distributed Vertical Reinf.

Assumptions

1- All steel yields

2- The tension force acts at the middepth of the tension zone.

3- The total compression force acts at the middepth of the compression zone.



Let's take A_s as the total area of vertical reinforcement. Then,

$$T = A_s f_y \left(\frac{l_w - c}{l_w} \right)$$

$$C_s = A_s f_y \left(\frac{c}{l_w} \right)$$

$$C_c = 0.85 f_c' h \beta_1 c$$

$$C = C_s + C_c$$

The section equilibrium implies that

$$C_c + C_s - T = 0$$

$$0.85 f_c' h \beta_1 c + A_s f_y \left(\frac{c}{l_w} \right) - A_s f_y \left(\frac{l_w - c}{l_w} \right) = 0 \Rightarrow c = \left[\frac{A_s f_y}{0.85 f_c' h \beta_1 l_w + 2 A_s f_y} \right] l_w$$

The moment equilibrium gives

$$M_n = T \left(\frac{l_w}{2} \right)$$

$$M_n = \frac{4161}{0.9} = 4623 \text{ kN}$$

$$M_n = T \left(\frac{l_w}{2} \right) \Rightarrow T = \frac{4623}{(4/2)} = 2311 \text{ kN}$$

$$T = A_s f_y \left(1 - \frac{c}{l_w} \right)$$

$$\Rightarrow \boxed{A_{s\,req} = \frac{T}{f_y \left(1 - \frac{c}{l_w} \right)}}$$

Assume $c = 700$ mm

$$A_{s\,req} = \frac{2311 \times 10^3}{(400)\left(1 - \frac{700}{4000}\right)} = 7003 \text{ mm}^2$$

$$c = \left[\frac{A_s f_y}{0.85 f'_c \beta_1 h l_w + 2 A_s f_y}\right] l_w$$

$$\Rightarrow c = \left[\frac{(7003)(400)}{(0.85)(24)(0.85)(200)(4000) + (2)(7003)(400)}\right](4000)$$

$c = 575$ mm

Second Try $\quad c = 575$ mm

$$A_{s\,req} = \frac{2311 \times 10^3}{(400)\left(1 - \frac{575}{4000}\right)} = 6747 \text{ mm}^2$$

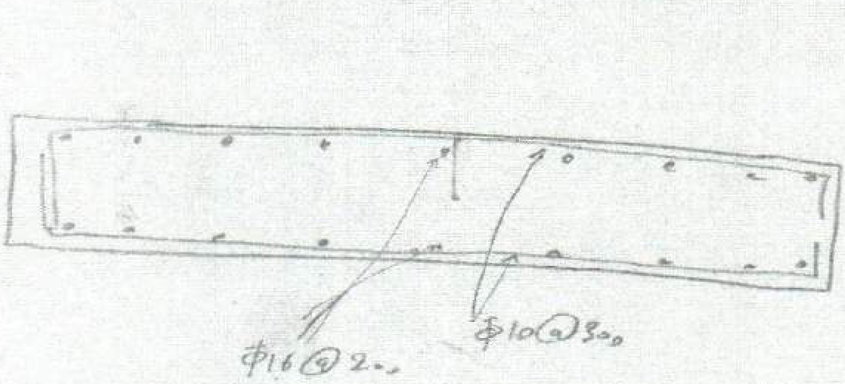
$$\rho_l = \frac{6747}{(200)(4000)} = 0.0084$$

$$A_l \text{ per meter} = (0.0084)(200)(1000) = 1680 \text{ mm}^2/\text{m}$$

Try $2\Phi14 \Rightarrow$ $S_{req} = \frac{(2)(\pi/4)(14)^2(1000)}{1680} = 183$ mm

Try $2\Phi12 \Rightarrow$ $S_{req} = \frac{(2)(\pi/4)(12)^2(1000)}{1680} = 135$ mm

Try $2\Phi16 \Rightarrow$ $S_{req} = \frac{(2)(\pi/4)(16)^2(1000)}{1680} = 239$ mm $\rightarrow$ Select $\Phi16$ @ 200 mm



CODE

11.6.4 — Shear-friction design method

11.6.4.1 — Where shear-friction reinforcement is perpendicular to the shear plane, V_n shall be computed by

$(N_u + A_v f_y)\mu$

نیروی وارده‌ی خود ستون روی سطح

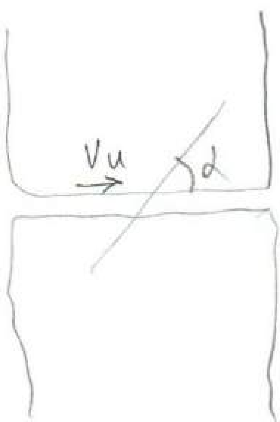
$$V_n = A_{vf} f_y \mu \qquad (11\text{-}25)$$

where μ is coefficient of friction in accordance with 11.6.4.3.

11.6.4.2 — Where shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement, V_n shall be computed by

$$V_n = A_{vf} f_y(\mu \sin\alpha + \cos\alpha) \qquad (11\text{-}26)$$

where α is angle between shear-friction reinforcement and shear plane.



COMMENTARY

In this equation, the first term represents the contribution of friction to shear-transfer resistance (0.8 representing the coefficient of friction). The second term represents the sum of the resistance to shearing of protrusions on the crack faces and the dowel action of the reinforcement.

When the shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in that reinforcement, the nominal shear strength V_n is given by

$$V_n = A_{vf} f_y\,(0.8 \sin\alpha + \cos\alpha) + A_c K_1 \sin^2\alpha$$

where α is the angle between the shear-friction reinforcement and the shear plane (that is, $0 < \alpha < 90$ **degrees**).

When using the modified shear-friction method, the terms $(A_{vf} f_y / A_c)$ or $(A_{vf} f_y \sin\alpha / A_c)$ should not be less than 1.4 MPa for the design equations to be valid.

R11.6.4 — Shear-friction design method

R11.6.4.1 — The required area of shear-friction reinforcement A_{vf} is computed using

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

The specified upper limit on shear strength should also be observed.

R11.6.4.2 — When the shear-friction reinforcement is inclined to the shear plane, such that the component of the shear force parallel to the reinforcement tends to produce tension in the reinforcement, as shown in Fig. R11.6.4, part of the shear is resisted by the component parallel to the shear plane of the tension force in the reinforcement.[11.46]

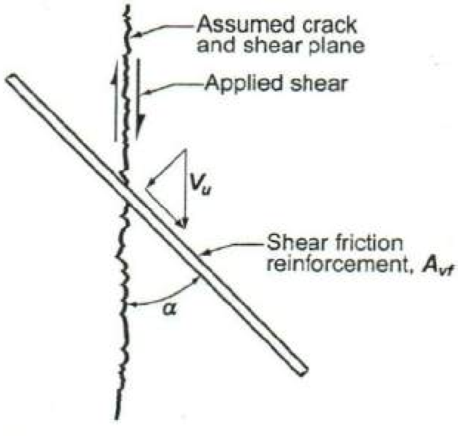


Fig. R11.6.4—Shear-friction reinforcement at an angle to assumed crack.

CODE

11.6 — Shear-friction

11.6.1 — Provisions of 11.6 are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

11.6.2 — Design of cross sections subject to shear transfer as described in 11.6.1 shall be based on Eq. (11-1), where V_n is calculated in accordance with provisions of 11.6.3 or 11.6.4.

11

11.6.3 — A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement A_{vf} across the shear plane shall be designed using either 11.6.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.

11.6.3.1 — Provisions of 11.6.5 through 11.6.10 shall apply for all calculations of shear transfer strength.

COMMENTARY

R11.6 — Shear-friction

R11.6.1 — With the exception of 11.6, virtually all provisions regarding shear are intended to prevent diagonal tension failures rather than direct shear transfer failures. The purpose of 11.6 is to provide design methods for conditions where shear transfer should be considered: an interface between concretes cast at different times, an interface between concrete and steel, reinforcement details for precast concrete structures, and other situations where it is considered appropriate to investigate shear transfer across a plane in structural concrete. (See References 11.43 and 11.44.)

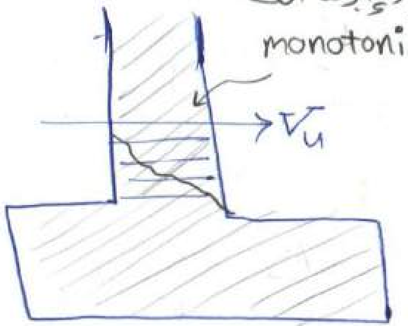
R11.6.3 — Although uncracked concrete is relatively strong in direct shear, there is always the possibility that a crack will form in an unfavorable location. The shear-friction concept assumes that such a crack will form, and that reinforcement must be provided across the crack to resist relative displacement along it. When shear acts along a crack, one crack face slips relative to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. At ultimate, the separation is sufficient to stress the reinforcement crossing the crack to its yield point. The reinforcement provides a clamping force $A_{vf}f_y$ across the crack faces. The applied shear is then resisted by friction between the crack faces, by resistance to the shearing off of protrusions on the crack faces, and by dowel action of the reinforcement crossing the crack. Successful application of 11.6 depends on proper selection of the location of an assumed crack.[11.16,11.43]

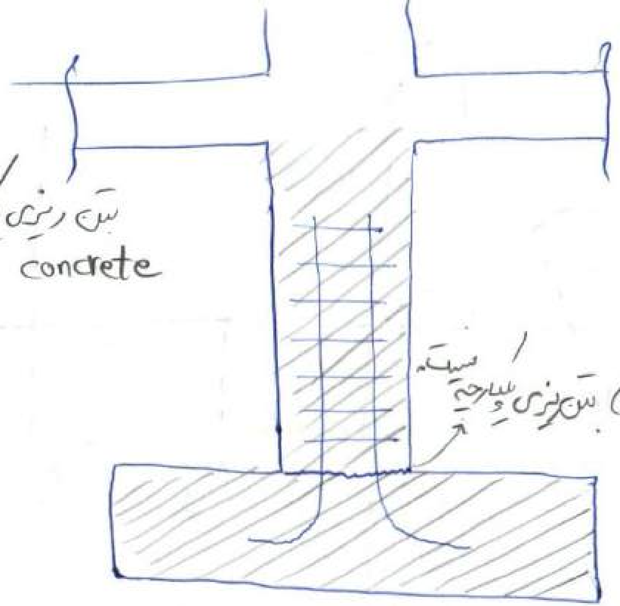
The relationship between shear-transfer strength and the reinforcement crossing the shear plane can be expressed in various ways. Equations (11-25) and (11-26) of 11.6.4 are based on the shear-friction model. This gives a conservative prediction of shear-transfer strength. Other relationships that give a closer estimate of shear-transfer strength[11.16,11.45,11.46] can be used under the provisions of 11.6.3. For example, when the shear-friction reinforcement is perpendicular to the shear plane, the nominal shear strength V_n is given by[11.45,11.46]

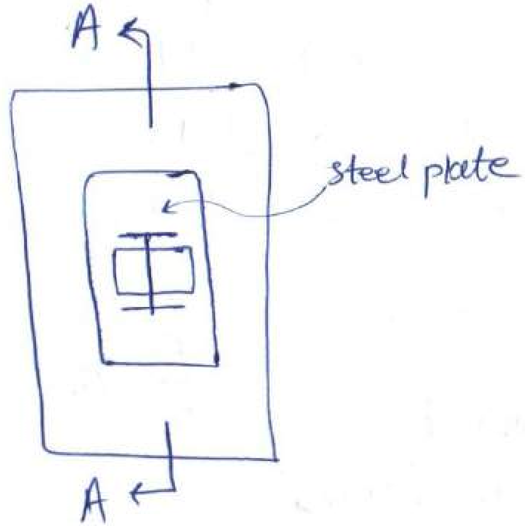
$$V_n = 0.8A_{vf}f_y + A_cK_1$$

where A_c is the area of concrete section resisting shear transfer (mm^2) and K_1 = **2.8 MPa** for normalweight concrete, 1.4 MPa for all-lightweight concrete, and 1.7 MPa for sand-lightweight concrete. These values of K_1 apply to both monolithically cast concrete and to concrete cast against hardened concrete with a rough surface, as defined in 11.6.9.

Shear Friction

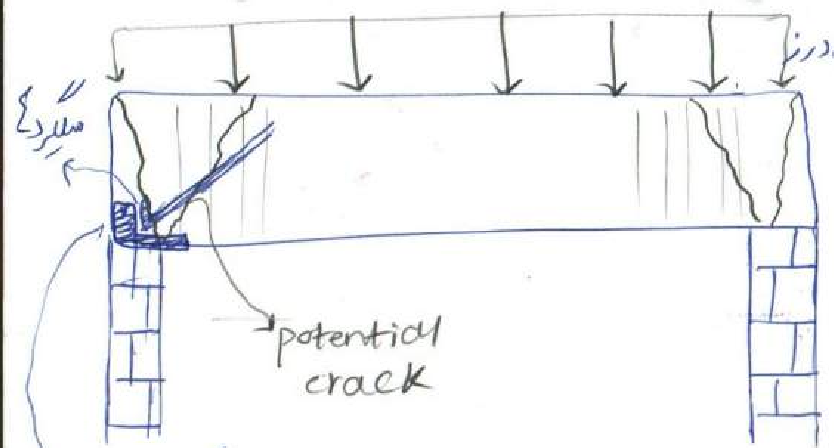


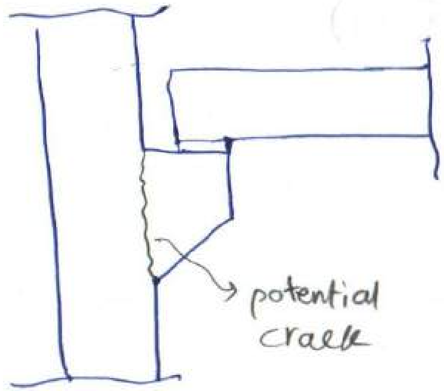




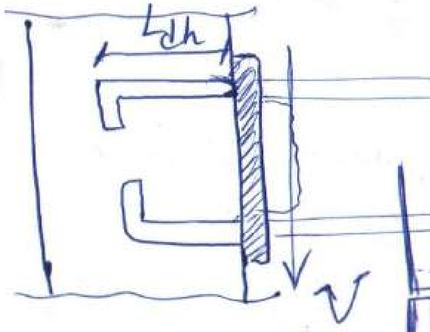
$\phi V_n = \phi(\cancel{V_c} + V_s)$

در محل درز 0

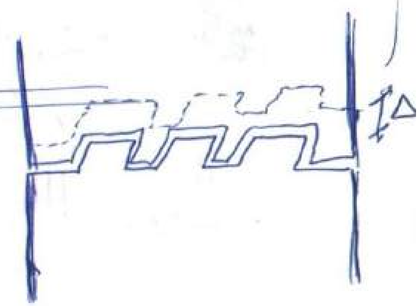


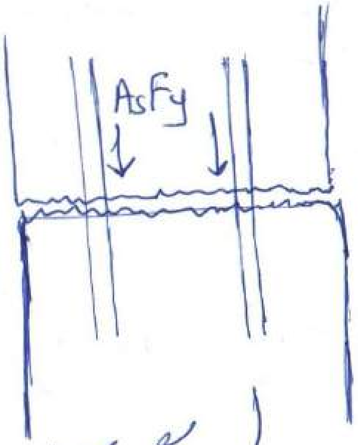


در سازه‌های precast



در اثر برش Δ بوجود می‌آید.





Δ به مقداری است که میلگردها جاری می‌شوند و به yield می‌رسند $\Delta \varepsilon = 0.002$

نبشی نصب می‌گذارند تا میلگردها را بتوان مهار کرد و میلگردها را به نبشی جوش می‌دهند تا مهار شود.

$F_s = \mu_s \overbrace{N}^{A_sF_y}$

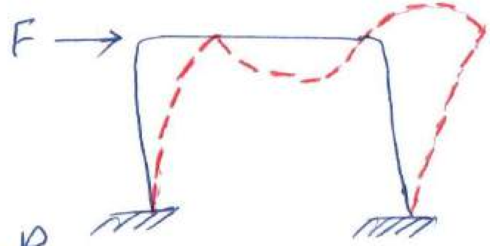
میلگردها تنش F_y دارند و نیروی N که عامل ایجاد مقاومت اصطکاکی است از نیروی میلگردها ناشی می‌گردد؛ جاری می‌شوند.

Ductility

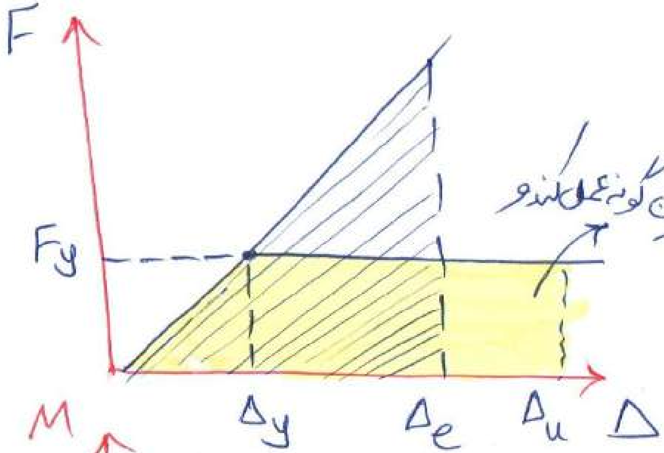
انواع قاب {

- Ordinary Frame → آیین‌نامه ch.21 تا ACI
- Intermediate Frame
- Special Frame

این قاب R بالاتری دارد و ضوابط سخت‌گیرانه‌تری دارد.

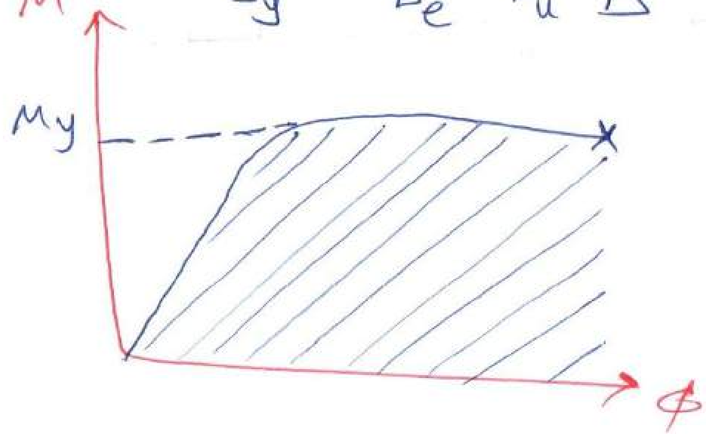


$$\frac{F_y}{F_e} = R$$



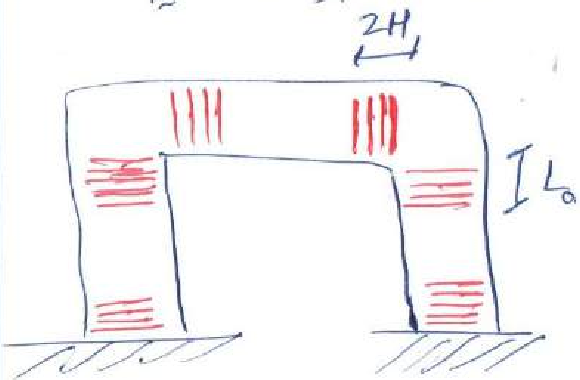
می‌خواهیم سازه این گونه عمل کند و شکل‌پذیرتر گردد.

$$\mu = \frac{\Delta_u}{\Delta_y}$$



برای انطباق بیشتر بحث‌های شکل‌پذیری با ضوابط آیین‌نامه‌ی ایران فصل ۲۳ آیین نامه ایران (20 صفحه)

در این آیین نامه مثلاً می‌گوید در جاهایی که لنگر بیشتر است باید خاموت بیشتری بگذاریم



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11.6.4.3 — The coefficient of friction μ in Eq. (11-25) and Eq. (11-26) shall be taken as:

Concrete placed monolithically **1.4λ**

Concrete placed against hardened concrete with surface intentionally roughened as specified in 11.6.9 .... **1.0λ**

Concrete placed against hardened concrete not intentionally roughened **0.6λ**

Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see 11.6.10) **0.7λ**

where λ = **1.0** for normalweight concrete and 0.75 for all lightweight concrete. Otherwise, λ shall be determined based on volumetric proportions of lightweight and normalweight aggregates as specified in 8.6.1, but shall not exceed 0.85.

11.6.5 — For normalweight concrete either placed monolithically or placed against hardened concrete with surface intentionally roughened as specified in 11.6.9, V_n shall not exceed the smallest of $0.2f_c'A_c$, $(3.3 + 0.08f_c')A_c$ and $11A_c$, where A_c is area of concrete section resisting shear transfer. For all other cases, V_n shall not exceed the smaller of $0.2f_c'A_c$ or $5.5A_c$. Where concretes of different strengths are cast against each other, the value of f_c' used to evaluate V_n shall be that of the lower-strength concrete.

11.6.6 — The value of f_y used for design of shear-friction reinforcement shall not exceed 420 MPa.

11.6.7 — Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane shall be permitted to be taken as additive to $A_{vf}f_y$, the force in the shear-friction reinforcement, when calculating required A_{vf}.

11

COMMENTARY

Equation (11-26) should be used only when the shear force component parallel to the reinforcement produces tension in the reinforcement, as shown in Fig. R11.6.4. When α is greater than 90 degrees, the relative movement of the surfaces tends to compress the bar and Eq. (11-26) is not valid.

R11.6.4.3 — In the shear-friction method of calculation, it is assumed that all the shear resistance is due to the friction between the crack faces. It is therefore necessary to use artificially high values of the coefficient of friction in the shear-friction equations so that the calculated shear strength will be in reasonable agreement with test results. For concrete cast against hardened concrete not roughened in accordance with 11.6.9, shear resistance is primarily due to dowel action of the reinforcement and tests[11.47] indicate that reduced value of $\mu = 0.6\lambda$ specified for this case is appropriate.

The value of μ for concrete placed against as-rolled structural steel relates to the design of connections between precast concrete members, or between structural steel members and structural concrete members. The shear-transfer reinforcement may be either reinforcing bars or headed stud shear connectors; also, field welding to steel plates after casting of concrete is common. The design of shear connectors for composite action of concrete slabs and steel beams is not covered by these provisions, but should be in accordance with Reference 11.48.

R11.6.5 — These upper limits on shear friction strength are necessary as Eq. (11-25) and (11-26) may become unconservative for some cases. Test data[11.49,11.50] on normalweight concrete either placed monolithically or placed against hardened concrete with surface intentionally roughened as specified in 11.6.9 show that a higher upper limit can be used on shear friction strength for concrete with f_c' greater than 28 MPa than was allowed before the 2008 revisions. In higher-strength concretes, additional effort may be required to achieve the roughness specified in 11.6.9.

R11.6.7 — If a resultant tensile force acts across a shear plane, reinforcement to carry that tension should be provided in addition to that provided for shear transfer. Tension may be caused by restraint of deformations due to temperature change, creep, and shrinkage. Such tensile forces have caused failures, particularly in beam bearings.

When moment acts on a shear plane, the flexural tension stresses and flexural compression stresses are in equilibrium. There is no change in the resultant compression $A_{vf}f_y$ acting across the shear plane and the shear-transfer strength is not changed. It is therefore not necessary to provide additional reinforcement to resist the flexural tension stresses, unless the required flexural tension reinforcement exceeds the amount

CODE | **COMMENTARY**

of shear-transfer reinforcement provided in the flexural tension zone. This has been demonstrated experimentally.[11.51]

It has also been demonstrated experimentally[11.44] that if a resultant compressive force acts across a shear plane, the shear-transfer strength is a function of the sum of the resultant compressive force and the force $A_{vf}f_y$ in the shear-friction reinforcement. In design, advantage should be taken of the existence of a compressive force across the shear plane to reduce the amount of shear-friction reinforcement required, only if it is certain that the compressive force is permanent.

11.6.8 — Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop f_y on both sides by embedment, hooks, or welding to special devices.

R11.6.8 — If no moment acts across the shear plane, reinforcement should be uniformly distributed along the shear plane to minimize crack widths. If a moment acts across the shear plane, it is desirable to distribute the shear-transfer reinforcement primarily in the flexural tension zone.

Since the shear-friction reinforcement acts in tension, it should have full tensile anchorage on both sides of the shear plane. Further, the shear-friction reinforcement anchorage should engage the primary reinforcement, otherwise a potential crack may pass between the shear-friction reinforcement and the body of the concrete. This requirement applies particularly to welded headed studs used with steel inserts for connections in precast and cast-in-place concrete. Anchorage may be developed by bond, by a welded mechanical anchorage, or by threaded dowels and screw inserts. Space limitations often require a welded mechanical anchorage. For anchorage of headed studs in concrete, see Reference 11.16.

اگر سند بلاست کنند باعث می‌شود میلگرد

11.6.9 — For the purpose of 11.6, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to **1.0λ**, interface shall be roughened to a full amplitude of approximately 6 mm.

11.6.10 — When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

ورق نوردی فولادی

گل میخ

11.7 — Deep beams

کرنش در آنها خطی نیست

R11.7 — Deep beams

11.7.1 — The provisions of 11.7 shall apply to members with ℓ_n not exceeding **4*h*** or regions of beams with concentrated loads within a distance **2*h*** from the support that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and supports. See also 12.10.6.

R11.7.1 — The behavior of deep beams is discussed in References 11.5 and 11.52 through 11.54. For a deep beam supporting gravity loads, 11.7.1 applies if the loads are applied on the top of the beam and the beam is supported on its bottom face. If the loads are applied through the sides or bottom of such a member, strut-and-tie models, as defined in Appendix A, should be used to design reinforcement to suspend the loads within the beam and transfer them to adjacent supports.

Habib Sadeghi

Notes:

- **Take $f'_c = 28$ MPa and $f_y = 420$ MPa, unless otherwise noted.**
- **For nondimensional interaction diagrams refer to the Appendix of the textbook.**
- **All dimensions shown on figures are in millimeter, unless otherwise noted.**
- **In all problems X and Y are the right figures of your student ID number, i.e. 92150XY.**

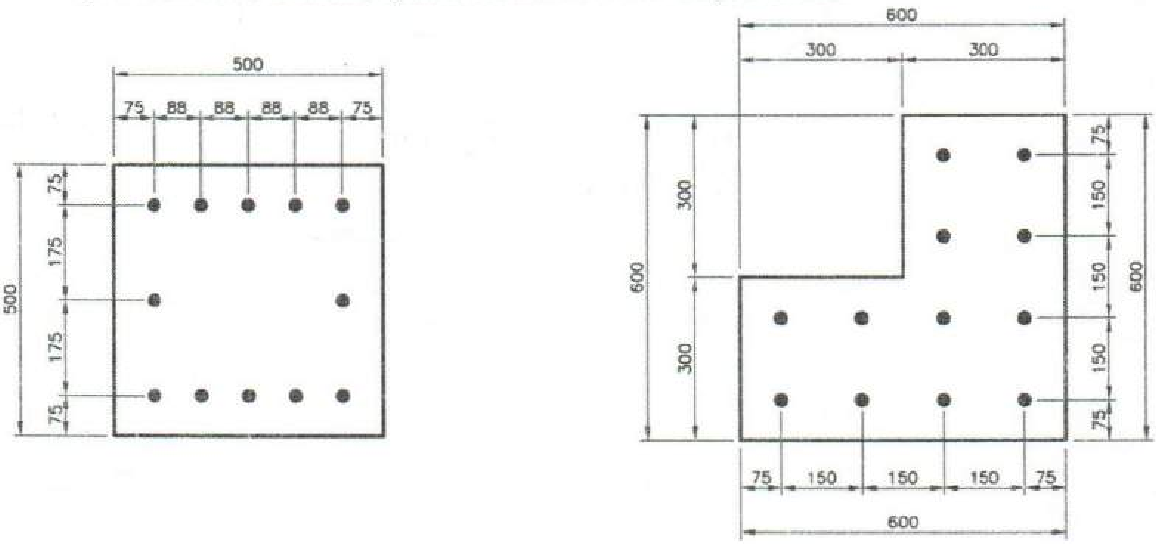
??

1- An interior rectangular short column in an office building with clear height of 3.X m (e.g. 3.7) should be designed for a factored axial compressive load $P_u = 5Y0$ ton (e.g. 500). The lateral force resisting system of the building is provided by shear walls. Architectural aspects limit one side dimension of the column to the maximum value of 300 mm. Design the column for $\rho_g = 2.X\%$ (e.g. 2.7%) and sketch your final design with enough details.

2- The following sections are proposed for RC columns in an ordinary building frame. All reinforcing bars shown are Φ22.

a) Sketch an appropriate tie arrangement for each section. Determine size and spacing of ties.

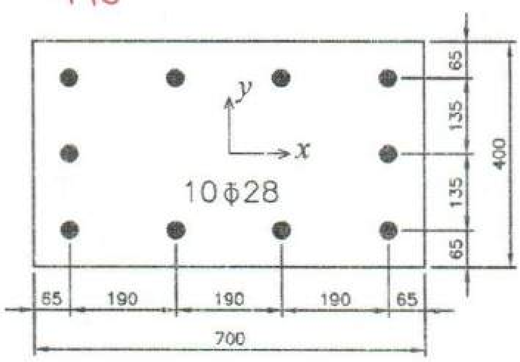
b) Calculate the location of plastic centroid for the L-shaped column.



3- Draw the design interaction diagram $\phi P_n - \phi M_{ny}$ for the given column section. $f'_c = 2X$ MPa (e.g. 27), and $f_y = 4Y0$ MPa (e.g. 400).

27 470

pure moment capacity ??

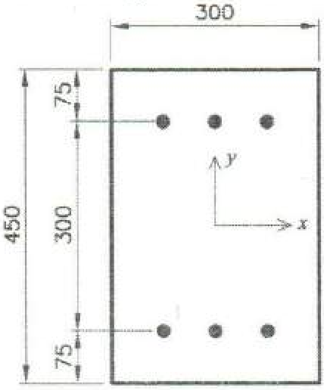


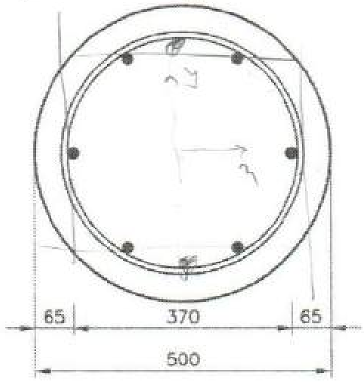
Notes:

- **Take $f'_c = 28$ MPa and $f_y = 420$ MPa, unless otherwise noted.**
- **For nondimensional interaction diagrams refer to the Appendix of the textbook.**
- **All dimensions shown on figures are in millimeter, unless otherwise noted.**
- **In all problems X and Y are the right figures of your student ID number, i.e. 92150XY.**

4- Select reinforcing bars for the following short columns subjected to the applied loads shown.

M_{ux}





(a)

$P_u = 1X0$ tons   140 ton
$M_{ux} = 2Y$ t.m   27 ton.m

(b)

$e_x = 140$ mm $P_u = 2Y$ tons, $P_u = 27$ ton
$e_x = 1X0$ mm, $e_y = 50$ mm $e_y = 50$
(Spirally reinforced column)

5- A nonslender corner column is subjected to a factored compressive axial load $P_u = 900$ kN, a factored bending moment $M_{ux} = 180$ kN.m about the x axis, and a factored bending moment $M_{uy} = 100$ kN.m about the y axis, as shown. Design a rectangular tied column section to resist biaxial bending moments.

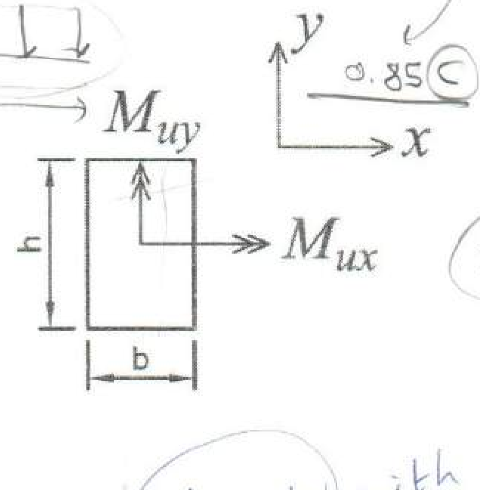
i. Use the Bresler-Parme equation.
ii. Use the reciprocal load method.

Hint: When a rectangular column with unknown dimensions is to be designed under biaxial bending moment, it would be reasonable to select the column dimensions proportional to the applied bending moments, i.e. $\frac{h}{b} \approx \frac{M_{ux}}{M_{uy}}$.

$0.85 f'_c\, a\, b = P_u$

$0.85c$

$\boxed{\frac{M_{ux}}{M_y} = \frac{h}{b}}$



$\frac{h}{1.8} = b$

h = 500
b = 200

$\rho = 2 \sim 3\%$

2.5%

برای ستون‌ها این مقدار را باید فرض کرد

Check with 2 methods

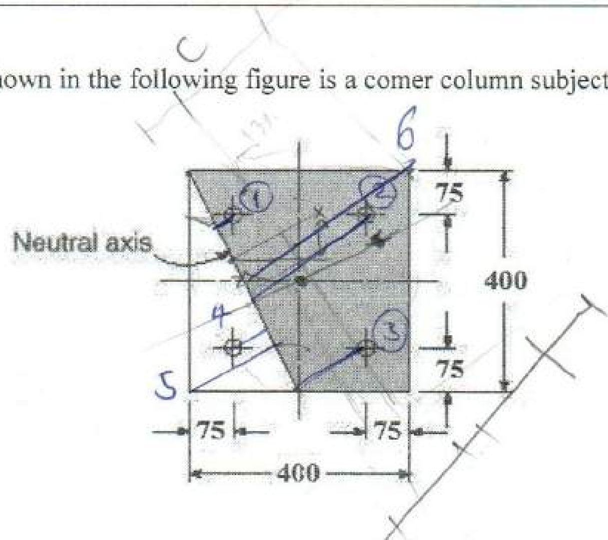
Check

Notes:

- **Take $f'_c = 28$ MPa and $f_y = 420$ MPa, unless otherwise noted.**
- **For nondimensional interaction diagrams refer to the Appendix of the textbook.**
- **All dimensions shown on figures are in millimeter, unless otherwise noted.**
- **In all problems X and Y are the right figures of your student ID number, i.e. 92150XY.**

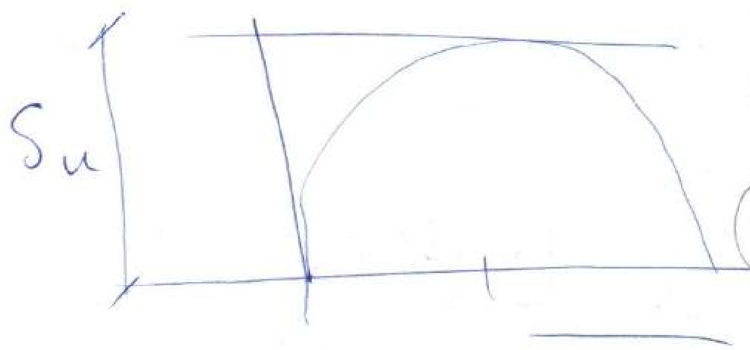
6- The square column shown in the following figure is a corner column subject to axial load and biaxial bending.

determin d_1 to d_6



a) Find the unique combination of P, M_{nx}, and M_{ny} that will produce incipient failure with the neutral axis located as in the figure. The compressive zone is shown shaded. Note that the actual neutral axis is shown, not the equivalent rectangular stress block limit; however, the rectangular stress block may be used as the basis of calculations.

b) Find the angle between the neutral axis and the eccentricity axis, the latter defined as the line from the column center to the point of load.



$$\sqrt{M_x^2 + M_y^2} = M_T$$

$$M_x^2 + M_y^2 = M_T^2$$

$M_T \cos\alpha$

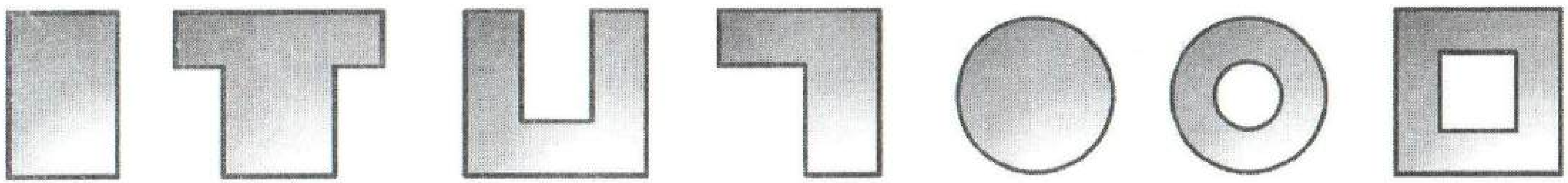
$M_T \sin\alpha$

Project 1: Interaction Diagram

- **Write a worksheet in either Excel or Mathcad for constructing the interaction diagram for the RC column section assigned to you.**
  - The dimensions of the section, area of reinforcement in each layer, and material properties, and type of lateral reinforcement are given.
  - The interaction diagram shall be constructed for both major axes.
  - Verify your results by a commercial program like SAP2000, CSICOL, or ETABS.
  - A short **ENGINEERING** report is required.
  - Due:
    - Group 1: 22nd Farvardin, 1394
    - Group 2: 24th Farvardin, 1394

n از بالا بیار
فاصله از بالا بیار

$\boxed{P=}$



(1) (2) (3) (4) (5) (6) (7)

بسمه تعالی

دانشـــــــگاه صنعتـــی شریف

دانشکـــــــده ی مهندســـی عمران

استاد درس : دکتر اسکندری

93

گروه دوشنبه

تمرین سری ۱ طراحی سازه های بتنی ۲

نام و نام خانوادگی :   حبیب اله صادقی ۹۱۱۰۵۰۴۷

این سری تمرین به همراه جناب آقای ابوالفضل میوه چی به صورت گروهی حل شد.

تاریخ تحویل ۱۳۹۳/۱۲/۱۸

نیمسال دوم سال تحصیلی ۹۴-۱۳۹۳

HW1 DRCS II « بسمه تعالی » حسین الصادقی ۹۱۱۰۵۰۴۷

ج ۱-

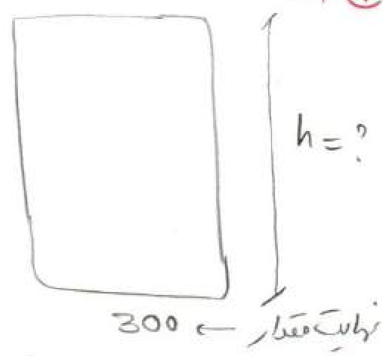
91105047 (x = 4, y = 7)

$$\begin{cases} x = 4 \\ y = 7 \end{cases}$$

rectangular short column
clear height = 3.4 m
$P_u = 570$ ton

$\rho = 2.4\% = 0.024$

$$\begin{cases} f'_c = 28 \text{ Mpa} \\ f_y = 420 \text{ Mpa} \end{cases}$$



$$A_g \geq \frac{P_u}{\phi r\,[0.85 f'_c + \rho_g (f_y - 0.85 f'_c)]}$$

$$\begin{cases} \phi_{spiral} = 0.75 & r_{spiral} = 0.85 \\ \phi_{tied} = 0.65 & r_{tied} = 0.8 \end{cases}$$

$$A_{g_{spirall}} \geq \frac{570 \times 10^3 \times 10 \text{ N}}{0.75 \times 0.85 \left[0.85 \times 28 + \frac{2.4}{100} \times (420 - 0.85 \times 28)\right]} = 268433 \text{ mm}^2$$

حالتی که حاوی spiral باشد

تفاوت spiral سبب خرج زیاد در ستون های اسپیرالی باشد

$$A_{g_{tied}} \geq \frac{570 \times 10^3 \times 10 \text{ N}}{0.65 \times 0.8 \left[0.85 \times 28 + \frac{2.4}{100} (420 - 0.85 \times 28)\right]} = 329088 \text{ mm}^2$$

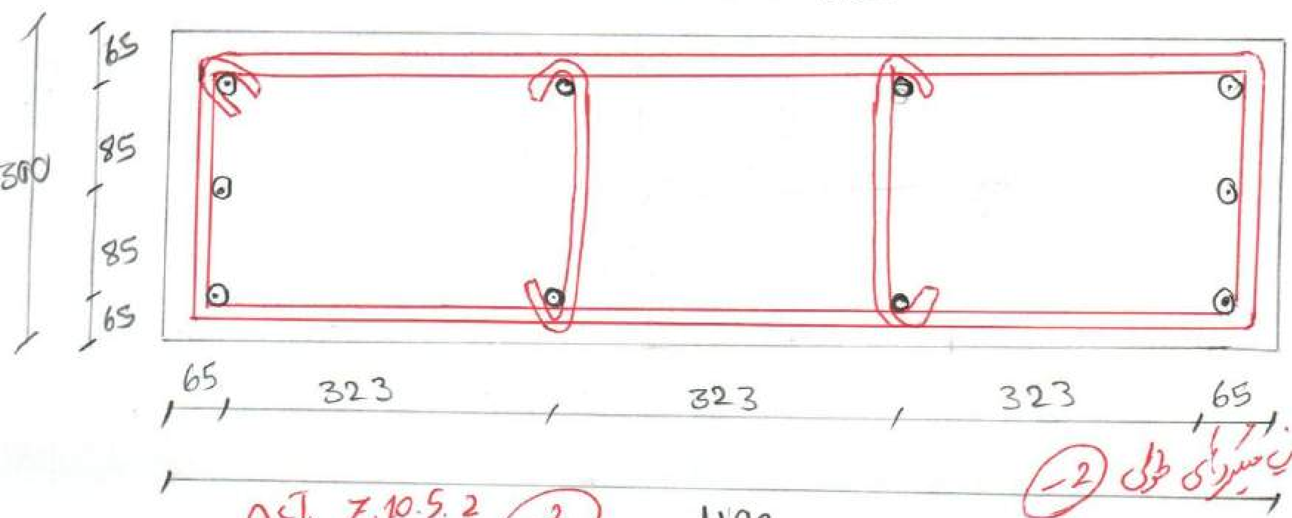
$A_{g_{min\ tied}} = 329088 \longrightarrow 300 \times 1100 \xrightarrow{\rho = 2.4\%} A_{st} = 7920 \longrightarrow 10\phi32 = 8042$ $\boxed{d_t = 10 \text{ mm}}$

$A_{g_{min\ spirall}} = 268433 \longrightarrow 300 \times 900 \xrightarrow{\rho = 2.4\%} A_{st} = 6480 \longrightarrow 14\phi25 = 6870$, $10\phi30 = 7069$ $\boxed{d_t = 10 \text{ m}}$

$S_{min\ tied} = Max(50, 1.5 d_b, 1.33 D_{Max}) = Max(50, \overbrace{1.5 \times 32}^{48}, \overbrace{1.33 \times 25}^{34}) = 50$

$S_{min} = Max(50 \text{ mm}, 1.5 d_b, 1.33 D_{Max}) \overset{Max}{=} (50, \overbrace{1.5 \times 25}^{37.5}, \overbrace{1.33 \times 25}^{34}) = 50$

we Assumed that the column is tied :

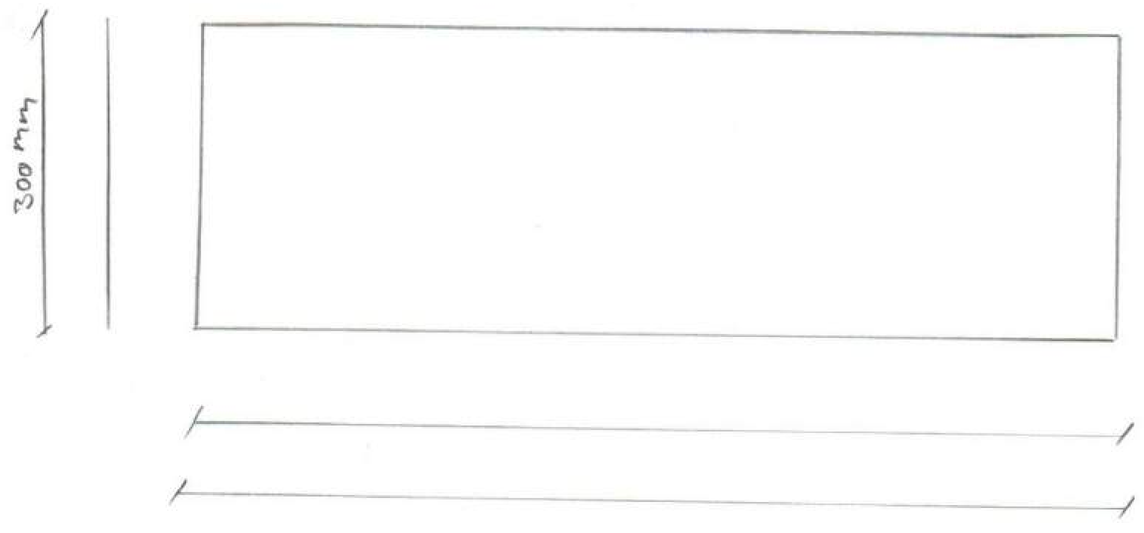


ACI 7.10.5.2
ACI 7.10.5.3 (-2)

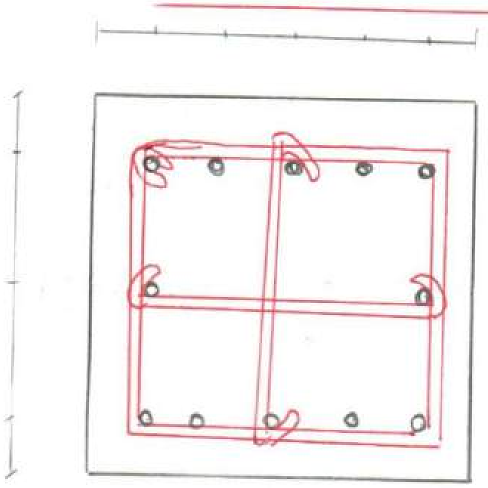
حداقل فاصله بین میلگردهای طولی وجود ندارد (-2)

1

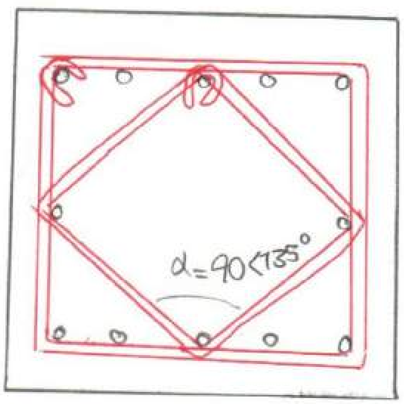
• The column with spirall : → I don't know the detailing of spiral



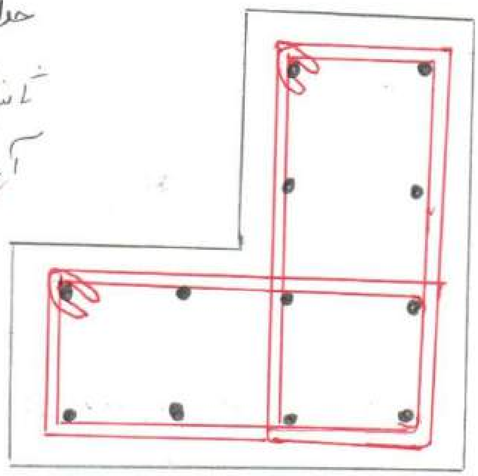
ج-۲



هر دو آرایش
⟷
مناسبند.



کلاً خاموت‌ها باید به نحوی باشند که آرماتورها
حداقل یکی در میان به آرماتور اتکا داشته باشند
تا فاصله‌ی آرماتورهای بدون اتکا از سایر
آرماتورها نباید از 150 mm بیشتر گردد.



← از دو خاموت مستطیلی که در ۲ تا
از آرماتورها اشتراک پوششی دارند
استفاده شده.

برای هر نوع مقطع

$$S_{max}=\min\{16d_b,\ 48d_t,\ h_{min}\}=\min\{\overset{352}{16\times 22},\ \overset{480}{48\times 10},\ \underset{600}{500}\}=352\ \text{mm}$$

HW#1 DRSC II

حسنی الهه صادقی ۹۱۱۰۵۰۴۷

الامرج-۲

$f'_c = 28 \quad f_y = 420$

$$\begin{cases} 12\phi 22 = 4562\ mm^2 \\ 4\phi 22 = 1521\ mm^2 \\ 2\phi 22 = 750\ mm^2 \\ 8\phi 22 = 3041 \end{cases}$$

$P_n = C_c + C_s$

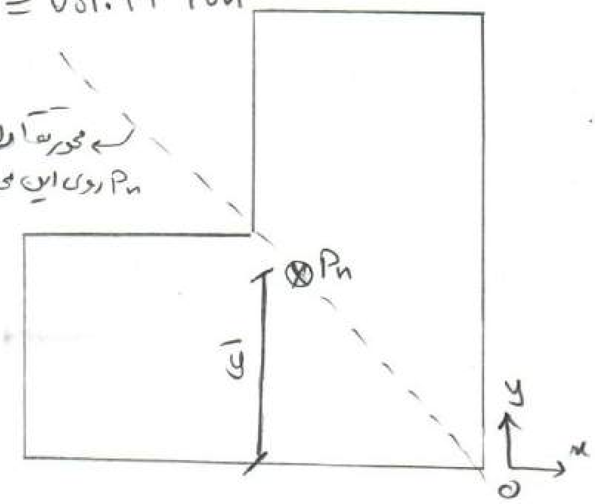
$$C_c = \underbrace{(600\times 600 - 300\times 300 - 4562)}_{A_c}\times 0.85 \times \underbrace{28}_{f'_c} = 631.74\ ton$$

$$C_s = \underbrace{4562}_{A_s}\times\underbrace{420}_{f_y} = 191.604\ ton$$

$P_n = 631.74 + 191.604 = 823.344\ ton$

محور تقارن

P_n روی این محور است.



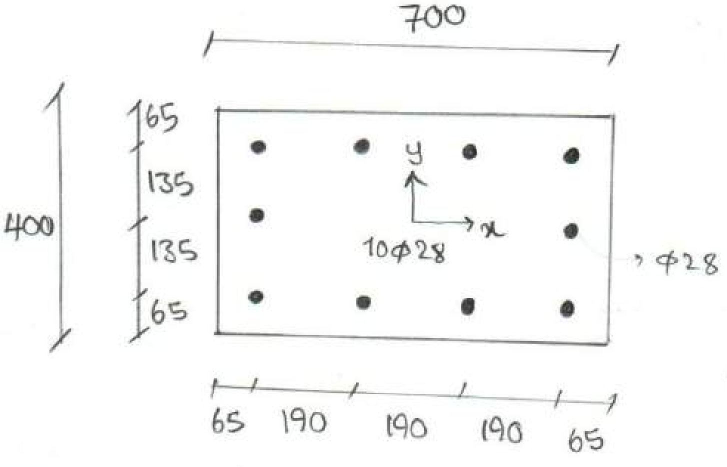
$\sum M_o = 0$

$$10^4 \times 823.334 \times \bar{y} - 750\times 420\times 525 - 750\times 420\times 375 - 1521\times 225\times 420 - 1521\times 75\times 420 -$$

$$- (180000 - \underbrace{3041}_{8\phi 22})\times 28\times 300\times 0.85 - (90000 - 1521)\times 28\times 150\times 0.85 = 0$$

$$\Rightarrow \bar{y} = \bar{x} = 249.53\ mm \checkmark$$

ج-۳



$$\begin{cases} f'_c = 24\ Mpa \\ f_y = 470\ Mpa \end{cases}$$

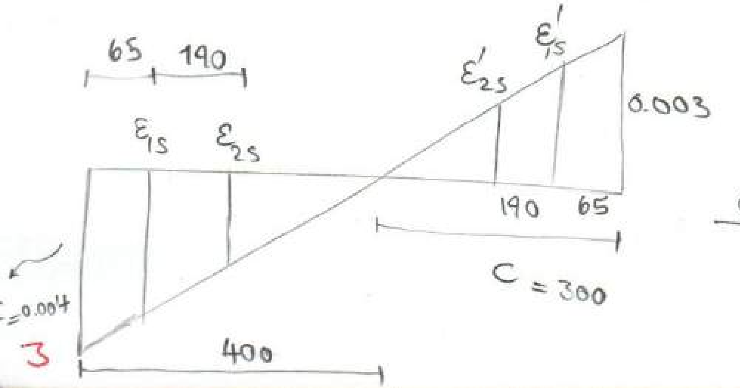
$10\phi 28 = 6158$

$d_t = 10\ mm$

$$\begin{cases} f_y = 470 \\ \varepsilon_y = \dfrac{470}{2\times 10^5} = 0.00235 \end{cases}$$

Balanced point

نقطه بالانس نقطه ای است که فولاد کششی به تسلیم می رسد و همزمان با رسیدن بتن به کرنش $\varepsilon = 0.003$ این اتفاق رخ می دهد.

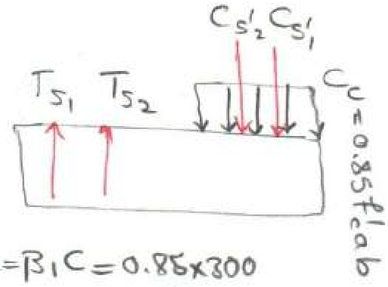


$$\frac{0.003}{C} = \frac{0.00235}{C-65} \longrightarrow C = 300\ mm$$

$$\frac{0.003}{300} = \frac{\varepsilon'_{2s}}{45} \longrightarrow \varepsilon'_{2s} = 45\times 10^{-4} < \varepsilon_y$$

$$\frac{0.004}{400} = \frac{\varepsilon_{2s}}{145} \longrightarrow \varepsilon_{2s} = 1.45 \times 10^{-3} < \varepsilon_y$$

$$\frac{0.004}{400} = \frac{\varepsilon_{1s}}{335} \longrightarrow \varepsilon_{1s} = 0.00335 > \varepsilon_y \rightarrow f_y$$



$f_c' < 28 \longrightarrow \beta_1 = 0.85$ $\quad a = \beta_1 C = 0.85 \times 300 = 255$ mm

$$C_c = 0.85 \times \underbrace{24}_{f_c'} \times \underbrace{255}_{a} \times \underbrace{400}_{b} = 208.08 \text{ ton}$$

$$C_{s_1}' = 470 \times 1847 = 86.804 \text{ ton}$$

$$C_{s_2}' = (4.5 \times 10^{-4} \times 2 \times 10^5) \times 1232 = 11.088 \text{ ton}$$

$$T_{s_1} = 470 \times 1847 = 86.809 \text{ ton}$$

$$T_{s_2} = (1.45 \times 10^{-3} \times 2E5) \times 1232 = 35.728$$

$$P = C_c + C_{s_1}' + C_{s_2}' - T_{s_1} - T_{s_2} = 183.44$$

$$M_O = C_c \times \left(\frac{h}{2} - \frac{a}{2}\right) + C_{s_1}' \times \left(\frac{h}{2} - 65\right) + C_{s_2}' \left(\frac{h}{2} - \overset{190+65}{255}\right) + T_{s_1}\left(\frac{h}{2} - 65\right) + T_{s_2}\left(\frac{h}{2} - 255\right)$$

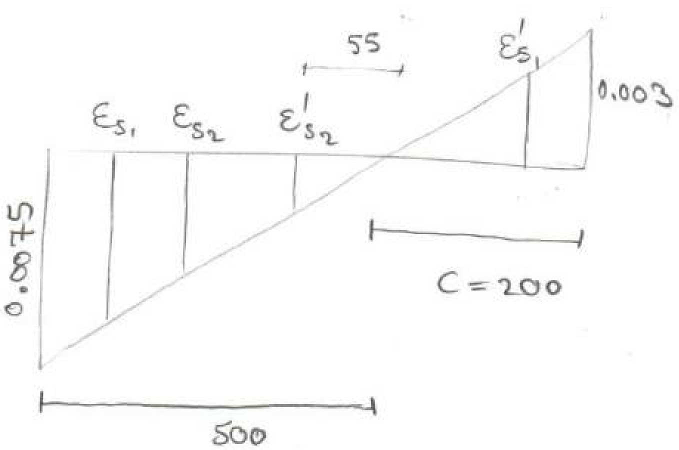
حول وسط
مقطع

$$= 208.08 \times \left(350 - \frac{255}{2}\right) + 86.809(350-65) + 11.088(350-255) + 86.809 \times (350-65)$$

$$+ 35.728(350-255) = 100.23 \text{ ton.m}$$

a point in the tension controlled region of the interaction curve:

select a $\underline{\underline{c}}$ smaller than the one in the balanced point $\longrightarrow C = 200$



$$\frac{0.003}{200} = \frac{\varepsilon_{s_1}'}{200-65} \longrightarrow \varepsilon_{s_1}' = 0.00205 < \varepsilon_y$$

$$\frac{0.0075}{500} = \frac{\varepsilon_{s_2}}{500-255} \longrightarrow \varepsilon_{s_2} = 3.675 \times 10^{-3} > \varepsilon_y$$

$$\frac{0.0075}{500} = \frac{\varepsilon_{s_2}'}{55} \longrightarrow \varepsilon_{s_2}' = 0.825 \times 10^{-3} < \varepsilon_y$$

$$\frac{0.0075}{500} = \frac{\varepsilon_{s_1}}{500-65} \longrightarrow \varepsilon_{s_1} = 6.525 \times 10^{-3} > \varepsilon_y$$

$$a = 0.85 C = 0.85 \times 200 = 170 \text{ mm}$$

حصة الصمادي ٩١١٠٥٠٤٧
الامج٣ -

$C_c = 0.85 f'_c a b = 0.85 \times 24 \times (0.85 \times 200) \times 400 = 138.72$ ton

$C_{s_1}' = A_s f_s = 1847 \times (2.025 \times 10^{-3} \times 2E5) = 74.80$

$T_{s_1} = A_s f_y = 1847 \times 470 = 86.81$ ton

$T_{s_2} = A_s f_y = 1232 \times 470 = 57.904$ ton

$T_{s_2}' = A_s f_s = 1232 \times (0.825 \times 10^{-5} \times 2E5) = 0.2032$

$P_n = C_c + C_{s_1}' - T_{s_1} - T_{s_2} - T_{s_2}' = 68.6$ ton

$M_o = C_c\left(\frac{h}{2} - \frac{a}{2}\right) + C_{s_1}'\left(\frac{h}{2} - 65\right) - T_{s_2}'\left(\frac{h}{2} - 255\right) + T_{s_1}\left(\frac{h_2}{2} - 65\right) + T_{s_2}\left(\frac{h}{2} - 255\right)$

$= 138.72\left(350 - \frac{170}{2}\right) + 74.8(350 - 65) - 0.2032 \times (350 - 255) + 86.81(350 - 65)$

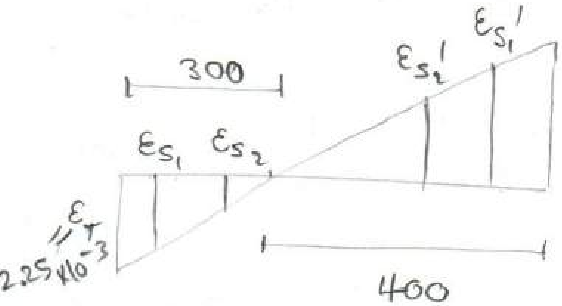
$+ 57.904(350 - 255) = 88.30$ ton.m

$e = \frac{M_u}{P_u} = \frac{88.3}{68.6} = 1.287$ m

---

a point in the compression controlled region of the interaction curve

select a $\underline{\underline{c}}$ larger than the one in the balanced point $c = 400$



$\frac{0.003}{400} = \frac{\varepsilon_{s_1}'}{400 - 65} \longrightarrow \varepsilon_{s_1}' = 0.0025 > \varepsilon_y$

$\frac{0.003}{400} = \frac{\varepsilon_{s_2}'}{400 - 65 - 190} \longrightarrow \varepsilon_{s_2}' = 0.00109 < \varepsilon_y$

$\frac{0.00225}{300} = \frac{\varepsilon_{s_1}}{300 - 65} \longrightarrow \varepsilon_{s_1} = 0.00176 < \varepsilon_y$

$\frac{0.00225}{300} = \frac{\varepsilon_{s_2}}{300 - 65 - 190} \longrightarrow \varepsilon_{s_2} = 0.0003375 < \varepsilon_y$

$a = \beta_1 c = 0.85 \times 400 = 340$

اطسى ٣ -

$C_c = 0.85 \times 24 \times 340 \times 400 = 277.44$ ton

$C_{s_1}' = A_s f_y = 1847 \times 470 = 86.81$ ton

$C_{s_2}' = A_s f_s = 1232 \times (2 \times 10^5 \times 0.00109) = 26.86$ ton $\qquad P_n = 317.78$ ton

$T_{s_1} = A_s f_s = 1847 \times (2 \times 10^5 \times 0.00176) = 65.01$ ton

$T_{s_2} = A_s f_s = 1232 \times (2 \times 10^5 \times 0.0003375) = 8.32$ ton

$$M_o = C_c\left(\frac{h}{2} - \frac{a}{2}\right) + C_{s_1}' \times \left(\frac{h}{2} - 65\right) + C_{s_2}'\left(\frac{h}{2} - 255\right) + T_{s_1}\left(\frac{h}{2} - 65\right) + T_{s_2}\left(\frac{h}{2} - 255\right)$$

$$= 277.44\left(350 - \frac{340}{2}\right) + 86.81(350-65) + 26.86(350-255) + 65.01(350-65) + 8.32(350-255)$$

$$= 96.55 \text{ ton.m}$$

$$e_a = \frac{M_y}{P_n} = \frac{96.55}{317.78} = 0.3038 \text{ m}$$

The point with pure axial stress & no Bending

$\varepsilon_{s_1}' = \varepsilon_{s_2}' = \varepsilon_{s_2} = \varepsilon_{s_1} > \varepsilon_y$

$f_{s_1}' = f_{s_2}' = f_{s_2} = f_{s_1} = f_y = 470$ Mpa

$$P_n = 0.85 f_c'(A_g - A_{st}) + A_{s_1}' f_y + A_{s_2}' f_y + A_{s_2} f_y + A_{s_1} f_y$$

$$= 0.85 \times 24(400 \times 700 - 6158) + 1847 \times 470 \times 2 + 1232 \times 470 \times 2 = 848.06 \text{ ton}$$

The part with pure tension

All steel bars are in tension & yielded

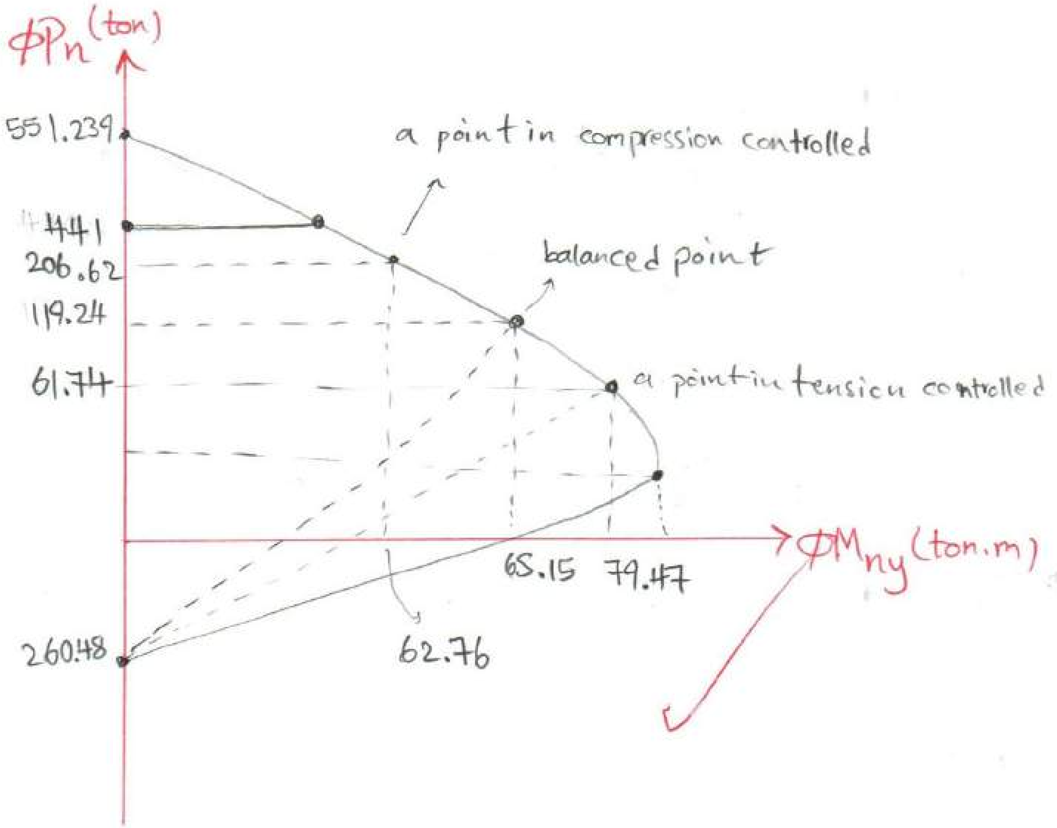
$P_n = A_{s_{total}} \times f_y = 6158 \times 470 = 289.426$ ton

$M_n = 0$

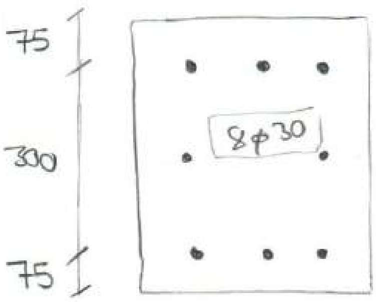
HW1 DRCS II

حسين الصادق ٩١١٥٥٠٤٧

الامرج ٣ـ



ج ٤ـ



$P_u = 140$ ton $\qquad h = 450$

$$P_n = \frac{P_u}{\phi} = \frac{140}{0.65} = 215.385 \text{ ton}$$

$$M_{nx} = \frac{M_{ux}}{\phi} = \frac{27}{0.65} = 41.538 \text{ ton.m}$$

$$e_y = \frac{41.538}{215.385} = 0.193 \text{ m} \qquad \frac{e}{h} = \frac{0.193 \times 10^3}{450} = 0.43$$

$$\gamma = \frac{300}{450} = 0.67 \longrightarrow 0.6 < \gamma < 0.7$$

$$K_n = \frac{P_n}{f'_c A_g} = \frac{215.385 \times 10^4}{28 \times (450 \times 300)} = 0.57$$

$$R_n = \frac{P_n e}{f'_c A_g h} = \frac{215.385 \times 0.193 \times 10^4}{28 \times (450 \times 300) \times \frac{450}{1000}} = 0.24$$

$\gamma = 0.6 \longrightarrow \rho = 0.042$

$\gamma = 0.7 \longrightarrow \rho = 0.034$

$> \gamma = 0.67 \longrightarrow \rho = 0.0396 \rightarrow \rho = 4\%$

$A_g = 450 \times 300 = 135000 \text{ mm}^2 \rightarrow A_{st} = 5400 \text{ mm}^2 \rightarrow 8\phi 30 = 5655 \text{ mm}^2$

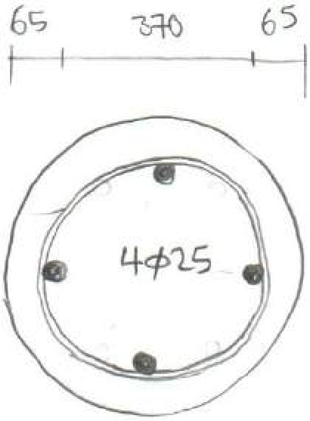
الماسح ٤ -

$$\therefore S_{min} = Max(50\ mm, \overset{45}{1.5 d_b}, \overset{34}{1.33 D_{max}}) = 50\ mm$$

$$S = (300 - 2 \times \overset{cover}{40}) / 3 = 73\ mm > S_{min} \longrightarrow ok$$

tie : $S_{max} = Min(16 d_b, 48 d_t, h_{min}) = Min(16 \times 30, 48 \times 10, 300) = 300\ mm$

pitch مقارنة {



$d_t = 70$

final sketched section

$P_u = 27$ ton

$e_x = 140$ mm

$e_y = 50$ mm

$$P_n = \frac{P_u}{\phi} = \frac{27}{0.75} = 36\ ton$$

$$M_{ny} = \frac{27 \times 140}{0.75} = 5.04\ ton.m$$

$$M_{nx} = \frac{27 \times 50}{0.75} = 1.8\ ton.m$$

$$\gamma = \frac{370}{500} = 0.74 \longrightarrow 0.7 < \gamma < 0.8$$

$$K_n = \frac{P_n}{f'_c A_g} = \frac{36 \times 10^4\ N}{28 \times \pi \times \frac{(500)^2}{4}} = 0.0655$$

$$R_{n_y} = \frac{P_n \cdot e_x}{f'_c \cdot A_g h} = \frac{36 \times 140 \times 10^4}{28 \times \pi \times \frac{500^2}{4} \times 500} = 0.0183$$

$$R_{n_x} = \frac{P_n e_y}{f'_c A_g h} = \frac{36 \times 50 \times 10^4}{28 \times \pi \times \frac{500^2}{4} \times 500} = 0.00655$$

$\left\{ \begin{array}{l} R_{ny} = 0.0183 \\ k_n = 0.655 \end{array} \right. \longrightarrow \begin{array}{l} \gamma = 0.7 \longrightarrow \rho \leq 0.01 \longrightarrow \rho = 0.01 \\ \gamma = 0.8 \longrightarrow \rho \leq 0.01 \longrightarrow \rho = 0.01 \end{array} \Big\rangle \gamma = 0.74 \longrightarrow \rho = 0.01$

$\left\{ \begin{array}{l} R_{nx} = 0.00655 \\ k_n = 0.0655 \end{array} \right. \longrightarrow \begin{array}{l} \gamma = 0.7 \longrightarrow \rho = 0.01 \\ \gamma = 0.8 \longrightarrow \rho = 0.01 \end{array} \longrightarrow \rho = 0.01$

$$A_g = \frac{\pi D^2}{4} = \pi \times \frac{500^2}{4} = 196350\ mm^2 \longrightarrow A_{st} = 1964\ mm^2 \longrightarrow 4\phi 25 = 1963\ mm^2$$

$$S_{min} = Max(50, 1.5 \times 25, 1.33 D_{max}) = 50\ mm < S \quad ok\ ✓$$

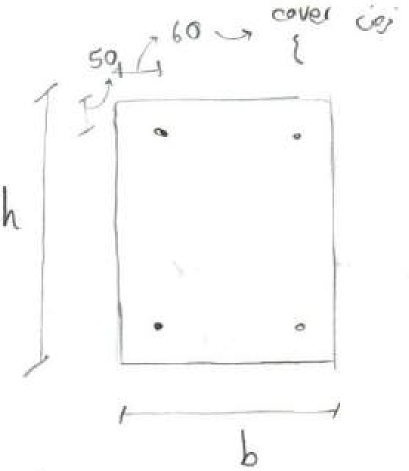
spirall : $S_c \Rightarrow Max(25, \underbrace{1.33 \times 25}_{34}) \leq S_c \leq 75 \longrightarrow 34\ mm \leq S_c \leq 75\ mm$ ok ✓

pitch

8

HW1 DRCS II   مجتبی الصادقی ۹۱۱۰۵۰۴۷

ج ۵-



$P_u = 900$ KN

$M_{ux} = 180$ KN

$M_{uy} = 100$ KN

$$\frac{h}{b} \simeq \frac{M_{ux}}{M_{uy}} = \frac{180}{100} = 1.8 \longrightarrow b = \frac{h}{1.8}$$

for finding a suitable h & b let's use equilibrium equation

$$0.85 f'_c\, ab = \frac{P_u}{\phi} = \frac{900}{0.65} \longrightarrow b = \frac{h}{1.8} \qquad a = 0.85\, \underset{h}{c} = 0.85 h$$

$$0.85 \times 28 \times 0.85 h \times \frac{h}{1.8} = \frac{900}{0.65} \longrightarrow h = 351 \longrightarrow \text{we take } h = 600 \text{ mm}$$

$$b = \frac{h}{1.8} = \frac{500}{1.8} \longrightarrow \text{we take } b = 300 \text{ mm} \qquad \boxed{\text{Assume } \rho = 3\%}$$

$$e_x = \frac{M_{uy}}{P_u} = \frac{100}{900} = 0.11 \text{ m} \qquad e_y = \frac{M_{ux}}{P_u} = \frac{180}{900} = 0.2 \text{ m}$$

i. Bersler-parme :

$$M_{nxo} = ? \quad P_n = \frac{P_u}{\phi} = \frac{900}{0.65} = 1384.62 \quad K_n = \frac{P_n}{f'_c \cdot A_g} = \frac{1384.62 \times 10^3}{28 \cdot (600 \times 300)} = 0.275$$

$$\frac{e_x}{h} = \frac{0.11 \times 10^3}{300} = 0.44 \qquad \frac{e_y}{h} = \frac{0.2 \times 10^3}{600} = 0.33 \qquad \begin{array}{l} \gamma = 0.6 \rightarrow R_{ny} = 0.18 \\ \gamma = 0.8 \rightarrow R_{nx} = 0.23 \end{array}$$

$$M_{nxo} = R_n \cdot f'_c \cdot A_g \cdot h = 0.23 \times 28 \times (300 \times 600) \times 600 = 69.55 \xrightarrow{0.65} M_{uxo} = 45.21 \text{ ton.m}$$

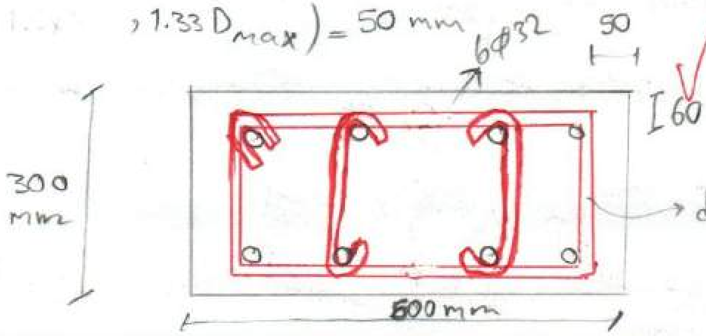
$$M_{nyo} = 0.18 \times 28 \times (300 \times 600) \times 300 = 27.216 \longrightarrow M_{uy_o} = \phi M_{nyo} = 17.69 \text{ ton.m}$$

$$\left(\frac{M_{ux}}{M_{ux_o}}\right)^{1.61} + \left(\frac{M_{uy}}{M_{uy_o}}\right)^{1.61} = \left(\frac{18}{45.21}\right)^{1.61} + \left(\frac{10}{17.69}\right)^{1.61} = 0.68 < 1 \quad \text{OK} \checkmark$$

$$\rho = 3\% \longrightarrow A_{st} = 0.03 \times (300 \times 600) = 5400 \text{ mm}^2 \qquad 8\phi 30 \rightarrow A = 5655 \text{ mm}^2$$

$d_t = 10$ mm

$S_{min} = Max(50\ ,\ \ \ \ \ , 1.33 D_{max}) = 50$ mm

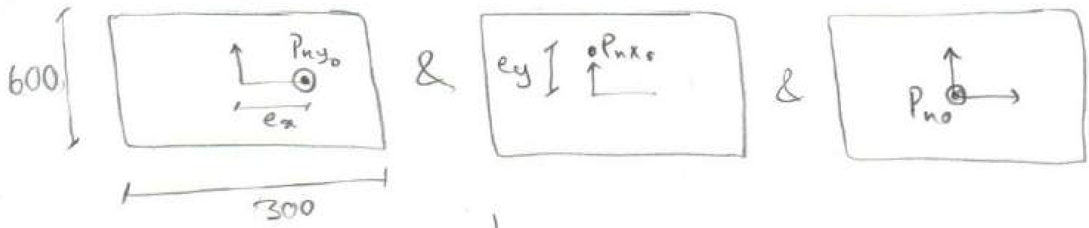


بررسی ضوابط حداقل وحداکثر فاصله
خاموت ها
وطراحی آن

-3

9

ii. use the reciprocal method :



$$e_x = \frac{M_{ux}}{P_u} = \frac{100}{900} = 0.11 \qquad e_y = \frac{180}{900} = 0.2$$

$$\frac{e_x}{h} = \frac{0.11}{300} = 0.00036 \qquad \frac{e_y}{h} = \frac{0.2}{600} = 0.00033$$

$$\gamma_x = 0.6 \rightarrow K_{nx_0} = 1.05 \qquad \left(\frac{1}{P_n} = \frac{1}{P_{nx_0}} + \frac{1}{P_{ny_0}} - \frac{1}{P_{n_0}} \right) \times A_g f'_c$$

$$\gamma_y = 0.8 \qquad K_{ny_0} = 1.05$$

$$\rightarrow \frac{1}{K_n} = \frac{1}{K_{nx_0}} + \frac{1}{K_{ny_0}} - \frac{1}{K_{n_0}}$$

$$\frac{1}{K_n} = \frac{1}{1.05} + \frac{1}{1.05} - \frac{1}{1.3} \Rightarrow K_n = 0.374$$

$$P_{n_r} = K_n A_g f'_c = 0.374 \times 180000 \times 28 = 188.496 \text{ ton} >$$

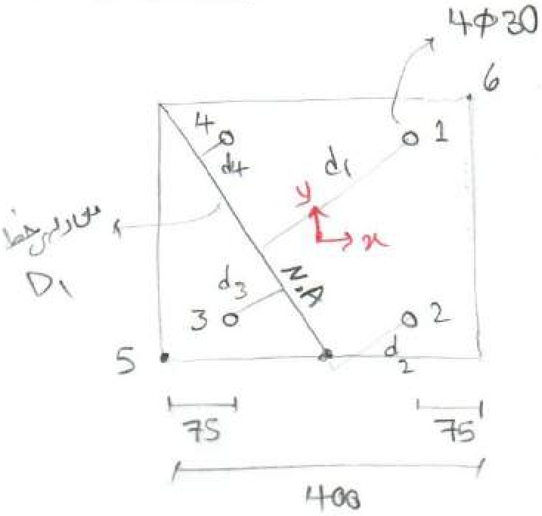
$$P_u = 122 \text{ ton} > \underbrace{90 \text{ ton}}_{P_{ureq}} \qquad \text{OK} \checkmark$$

ابعاد مقطع در این روش نیز با ρ=3٪، OK است و آرایش میلگردها نیز مانند صفحه‌ی قبل است.

HW1 DRC8

[illegible] ۹۱۱۰۵۰۴۷

-۴۶



$C_1 = (125, 125)$

$C_2 = (+125, -125)$

$C_3 = (-125, -125)$

$C_4 = (-125, 125)$

$C_5 = (-200, -200)$

$C_6 = (+200, +200)$

$f_y = 420 \rightarrow \varepsilon_y = 0.0021$

معادله خط D_1 $\Rightarrow y = -2x - 200$

نقاط روی خط $\rightarrow (x, -2x-200)$

شیب خط عمود $= +\frac{1}{2}$

$y = \frac{1}{2}x + B \longrightarrow y_1 = \frac{1}{2}x + 62.5 \qquad y_4 = \frac{1}{2}x + 187.5$

$y_2 = \frac{1}{2}x - 187.5 \qquad y_5 = \frac{1}{2}x - 100$

$y_3 = \frac{1}{2}x - 62.5 \qquad y_6 = \frac{1}{2}x + 100$

$\frac{1}{2}x + 62.5 = -2x - 200 \longrightarrow x_1 = -105 \quad y_1 = 10 \longrightarrow d_1 = 257.15$ mm

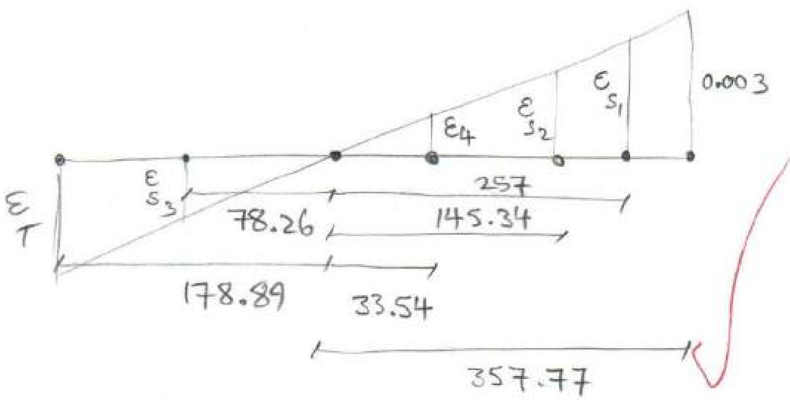
$\frac{1}{2}x - 187.5 = -2x - 200 \longrightarrow x_2 = -5 \quad y_2 = -140 \longrightarrow d_2 = 145.34$ mm

$\frac{1}{2}x - 62.5 = -2x - 200 \longrightarrow x_3 = -55 \quad y_3 = -90 \longrightarrow d_3 = 78.26$ mm

$\frac{1}{2}x + 187.5 = -2x - 200 \longrightarrow x_4 = -155 \quad y_4 = 110 \longrightarrow d_4 = 33.54$ mm

$\frac{1}{2}x - 100 = -2x - 200 \longrightarrow x_5 = -40, \; y_4 = -120 \longrightarrow d_5 = 178.89$ mm

$\frac{1}{2}x + 100 = -2x - 200 \longrightarrow x_6 = -120 \quad y_5 = 40 \longrightarrow d_6 = 357.77$ mm

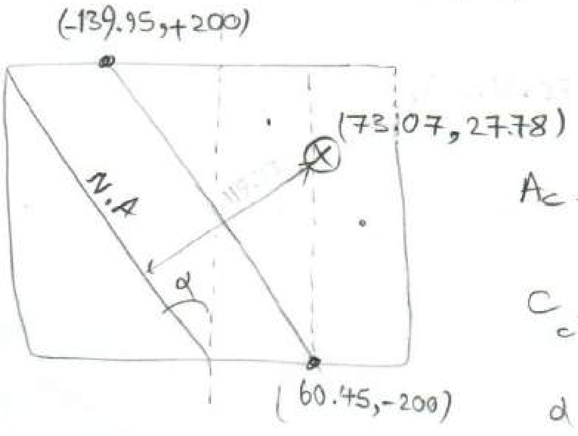


$\frac{\varepsilon_{s_1}}{257} = \frac{0.003}{357.77} \longrightarrow \varepsilon_{s_1} = 0.0022 > \varepsilon_y$

$\frac{\varepsilon_{s2}}{145.34} = \frac{0.003}{357.77} \longrightarrow \varepsilon_{s_2} = 0.00122$

$\frac{\varepsilon_{s4}}{33.54} = \frac{0.003}{357.77} \longrightarrow \varepsilon_{s_4} = 0.00028$

$\frac{\varepsilon_{s3}}{178.89} = \frac{0.003}{357.77} \longrightarrow \varepsilon_{s_3} = 0.0015$



$A_c = \left(\frac{139.55 + 339.95}{2}\right) \times 400 = 95900 \text{ mm}^2$

$C_c = 0.85 f'_c A_c = 228.24$ ton

$\alpha = \tan^{-1}\left(\frac{200}{400}\right) = 26.57°$

$C_{s_1} = A_s f_y = 707 \times 420 = 29.694$ ton

$C_{s_2} = 707 \times (2ES \times 0.00122) = 17.251$ ton

$C_{s_4} = 707 \times (2ES \times 0.00028) = 3.959$ ton

$T_{s_3} = 707 \times (2ES \times 0.0015) = 21.21$ ton

$P_n = C_c + C_{s_1} + C_{s_2} + C_{s_4} - T_{s_3} = 257.934$ ton ✓

$y_7 = \frac{1}{2}x - 8.775 \qquad \frac{1}{2}x - 8.775 = -2x - 200 \rightarrow x = -76.49 \quad y = -47.02$

$d_7 = 167.22$

فاصله‌ی مرکز سطح تا N.A

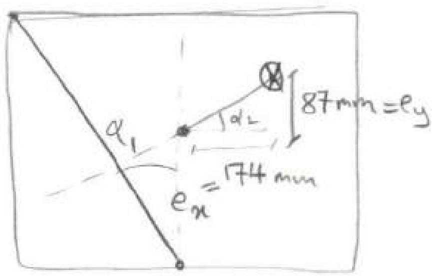
$$M_{total} = 228.24 \times 167.22 + 29.694 \times 257 + 17.251 \times 145.34 + 3.959 \times 33.54 + 21.21 \times 78.26$$

$= 50.097$ ton.m

$$\begin{cases} M_{ny} = M_{total} \cos\alpha = 50.097 \times \cos 26.57 = 44.81 \text{ ton.m} \checkmark \\ M_{nx} = M_{total} \sin\alpha = 50.097 \times \sin 26.57 = 22.41 \text{ ton.m} \checkmark \end{cases}$$

(b

$e_y = \dfrac{M_{nx}}{P} = \dfrac{22.41}{257.934} = 0.087$ m $\qquad e_x = \dfrac{M_{ny}}{P} = \dfrac{44.81}{257.934} = 0.174$ m



$\alpha_2 = \tan^{-1}\left(\frac{e_y}{e_x}\right) = 26.57°$

$\alpha_1 = 26.57°$

The Angle between N.A & eccentricity axis is $\underline{90°}$ ✓

Habib Sadeghi

Notes:

- ***Take $f'_c = 28\ MPa$ and $f_y = 420\ MPa$ for all problems.***
- ***All dimensions shown on the figures are in millimeters, unless otherwise noted.***
- ***X and Y are the right figures of your student ID number, e.g. for 90150307, X=0 and Y=7.*** $X = 4$, $Y = 7$

large, solid, impressive

1- The structure shown in Fig. 1a requires tall slender columns at the left side. It is fully braced by shear walls on the right. All columns are 400 x 400 mm, as shown in Fig. 1b, and all beams are 600 x 450 mm with 150 mm monolithic floor slab, as in Fig. 1c. Trial calculations call for column reinforcement as shown. Alternate load analysis indicates the critical condition with column AB bent in single curvature, and service loads and moments as follows:

- from dead loads, $P = 5X0$ kN (e.g. 500), $M_{top} = 81$ kN.m, $M_{bot} = 52$ kN.m; → Dead (540)
- from live loads, $P = 4Y1$ kN (e.g. 471), $M_{top} = 54$ kN.m, $M_{bot} = 33$ kN.m, → live (471)

Is the proposed column, reinforced as shown, satisfactory for this load condition? Consider the moment of inertia of reinforcing bars in EI calculation.

yes, OK!

nonsway

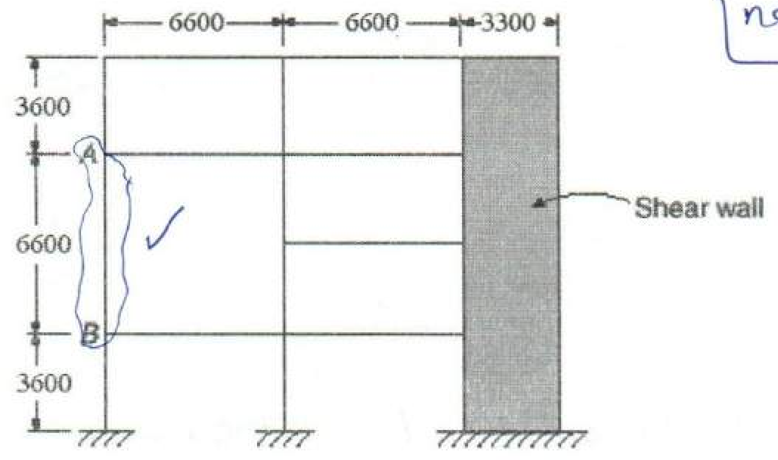


Fig. 1a

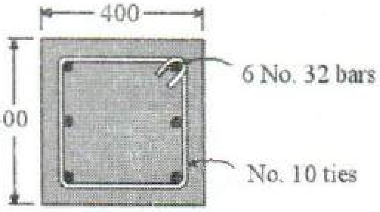


Fig. 1b

مقطع ستون

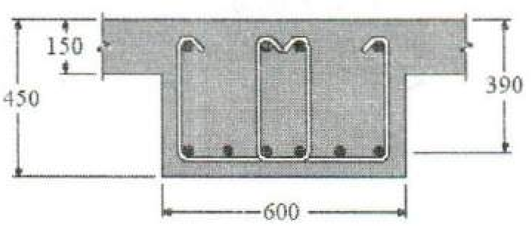


Fig. 1c

مقطع تیر

91105047

Design of RC Structures II – Spring 2015 **HW2** Slender Columns
Due: Group 1: 15th Farvardin, 1394 Related Chapters: Ch. 9
Group 2: 17th Farvardin, 1394

2- The first three floors of a multistory building are shown below. The lateral load–resisting frame consists of 500x500 mm exterior columns, 600x600 interior columns, and 900 mm wide x 600 mm deep girders. The center-to-center column height is 4800 mm. For the second story columns, the service gravity dead and live loads and the horizontal wind loads based on an elastic first-order analysis of the frame are:

→ nonsway or sway

برای حاصل از آنالیز مرتبه اول

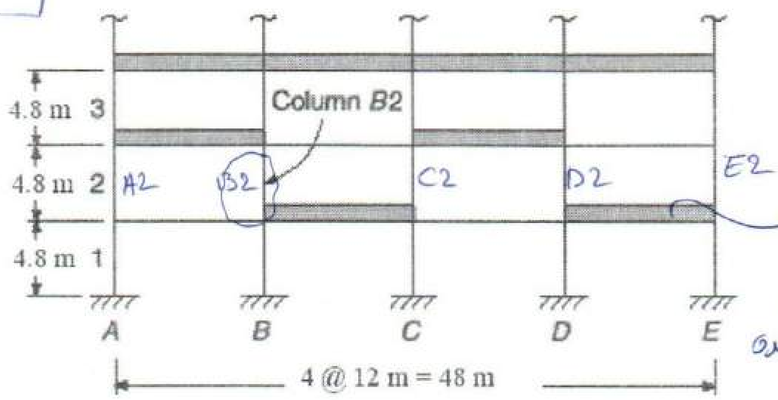
	Columns A2 & E2	Columns B2 & D2	Column C2
P_{dead}	1Y49 kN (e.g. 1794)	3X69 kN (e.g. 3069)	3062 kN
P_{live}	610 kN	1Y66 kN (e.g. 1766)	1313 kN
P_{wind}	±85 kN	40 kN	0 kN
$M_{2,dead}$		41.5 kN.m	
$M_{2,live}$		216 kN.m	
$M_{2,wind}$		141 kN.m	
$M_{1,dead}$		-45.5 kN.m	
$M_{1,live}$		145 kN.m	
$M_{1,wind}$		-131 kN.m	

1749 3469 1766

D L W
+
−

برای تمام ستون‌ها Pu از طبقه اول

Design column B2.



این بارگذاری که نشان داده شده چه کاربردی دارد

ستون داخلی 600x600

ستون خارجی 500x500

تیر 900x600

عرض عمق

$L_c = 4800$ mm

I_{se} ??

??

بسمه تعالی

دانشـــــگاه صنعتـــی شریف

دانشکـــــده ی مهندســـی عمران

استاد درس : دکتر اسکندری

75

گروه دوشنبه

تمرین سری ۲ طراحی سازه های بتنی ۲

نام و نام خانوادگی :   حبیب اله صادقی ۹۱۱۰۵۰۴۷

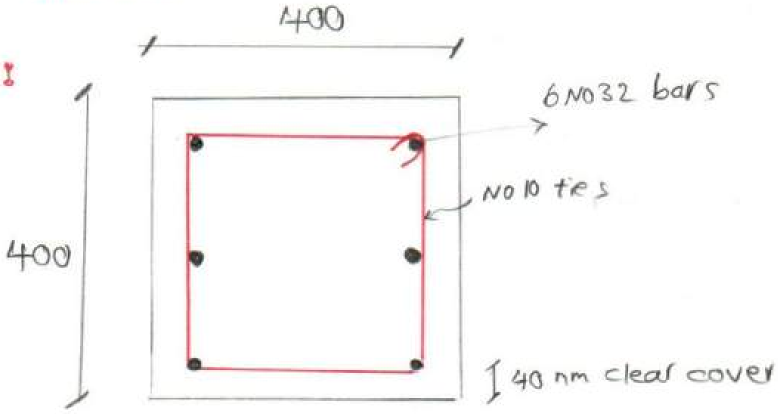
تاریخ تحویل ۱۳۹۴/۱/۱۷

نیمسال دوم سال تحصیلی ۹۴-۱۳۹۳

HW2 DRCS II

٩١١٠٥٠٤٧ حسن الصادق

Q1 :



$f'_c = 28$ Mpa $\quad x = 4$

$f_y = 420$ Mpa $\quad y = 7$

$E_s = 200000$ Mpa

$E_c = 4700\sqrt{28} = 24870.06$ Mpa

$n = \frac{E_s}{E_c} = \frac{200,000}{24870.06} = 8.04$

Dead $\longrightarrow P_d = 540$ KN $\quad M_{top} = 81$ KN.m , $M_{bot} = 52$ KN.m

live $\longrightarrow P_l = 471$ KN $\quad M_{top} = 54$ KN.m , $M_{bot} = 33$ KN.m

The frame is Non-sway due to shear wall. ✓

$$\beta_{ds} = \frac{1.2\times540}{1.2\times540 + 1.6\times471} = 0.462$$

$$I_{g_{column}} = \frac{1}{12}bh^3 = \frac{1}{12}(400)(400)^3 = 2133.33\times10^6 \text{ mm}^4$$

$$I_{se} = Ad^2 = 3217\times\left(\frac{400}{2} - 40 - 10 - \frac{32}{2}\right)^2 = 57.76\times10^6 \text{ mm}^4$$

$$I_{g_{beam}} = \frac{1}{12}bh^3 = \frac{1}{12}\times600\times450^3 = 4556.25\times10^6 \text{ mm}^4$$

$$(E_cI)_{beam} = 24870.06\times4556.25\times10^6 = 1.133\times10^{14} \text{ N.mm}^2$$

$$(EI)_{column} = \frac{0.2E_cI_g + E_sI_{se}}{1+\beta_{ds}} = \frac{0.2\times24870.06\times2133.33\times10^6 + 2\times10^5\times28.87\times10^6}{1+0.462} = 1.121\times10^{13} \text{ N.mm}^2$$

This is for P_c Calculation

load combinations

case 1 : 1.4D $\quad P_u = 756$ KN $\quad M_{u_{top}} = 113.4$ KN.m $\quad M_{u_{bot}} = 72.8$ KN.m

case 2 : 1.2D+1.6L $\quad P_u = 1401.6$ KN $\quad M_{u_{top}} = 183.6$ KN.m $\quad M_{u_{bot}} = 115.2$ KN.m ✓

case 2 is critical

$r_{approximate} = 0.3b = 0.3\times400 = 120$ mm

$$\frac{KL_u}{r} = \frac{1.0\times6000}{120} = 50 \geq 40 \leftarrow \frac{KL_u}{r} \leq 34 - 12\left(\frac{M_1}{M_2}\right) \leq 40$$

calculations of K_y

$$\psi_A = \psi_B = \frac{\sum\left(\frac{EI}{L_c}\right)_{columns}}{\sum\left(\frac{EI}{L_c}\right)_{beams}} = \frac{\cancel{1.121\times10^{13}}\times\left(\frac{1}{6600}+\frac{1}{3600}\right)}{1.113\times10^{14}\times\left(\frac{1}{6600}\right)} = 0.285$$

$0.7I_g$ $\quad$ -5

$0.35I_g$

From braced nomograph $\longrightarrow K_y = 0.62$

$$\frac{KL_u}{r} = \frac{0.62\times6000}{120} = 31 > 34 - 12\left(\frac{M_1}{M_2}\right) = 34 - 12\times\frac{1.2\times52+1.6\times33}{1.2\times81+1.6\times54} = 26.5$$

$\boxed{6600-450}$ $\quad$ -3

$\therefore \longrightarrow$ The column is slender

$\therefore P_c = \frac{\pi^2 EI}{(kl_u)^2} = \frac{\pi^2 \times 1.121\times10^{13}}{(0.62\times6000)^2} = 7.995\times10^3\ KN \rightarrow$ critical buckling load

ok

$C_m = 0.6 + 0.4\frac{M_1}{M_2} = 0.6 + 0.4\times\frac{1.2\times52+1.6\times33}{1.2\times81+1.6\times54} = 0.851 \geq 0.4 \rightarrow$ For case 2

$\delta_{ns} = \frac{C_m}{1 - P_u/0.75P_c} = \frac{0.851}{1-\frac{1401.6}{0.75\times7.995\times10^3}} = 1.111 \geq 1$ ok

$P_u = 1401.6\ KN$ $\qquad M_2 = Max[M_{top}, M_{2min}] =$

$M_u = \delta_{ns}M_2 = 1.111\times183.6 = 203.98\ KN.m$

where is $M_{2,min}$ -2

$\frac{P_u}{\phi f'_c A_g} = \frac{1401.6\times10^3}{0.65\times28\times(400)^2} = 0.483$ $\qquad \gamma = \frac{400-2\times40-2\times10-32}{400} = 0.67$

$\frac{M_u}{\phi f'_c A_g h} = \frac{203.98\times10^6}{0.65\times28\times(400)^2\times400} = 0.175$

$\gamma = 0.6 \longrightarrow \rho = 0.22$

$\gamma = 0.7 \longrightarrow \rho = 0.18$

$\longrightarrow \gamma = 0.67 \longrightarrow \rho = 0.21$

$A_s = \rho A_g = 0.021\times400^2 = 3360\ mm^2 \longrightarrow$ minimum reinf area needed

$A_{s\,6\phi32} = 4825\ mm^2 > 3360 \longrightarrow$ ok ✓

$A_{s\,provided} > A_{s,req}$

tie size : $d_b \leq 32 \rightarrow d_t = 10$ mm ok ✓

$S_{min} = Max(40\ mm, \underset{45}{1.4\times32}, \underset{34}{1.33\times25}) = 45 < S = (400-2\times40-2\times10-32)/2 = 134$

ok ✓

$S_{max} = Min(\underset{512}{16d_b}, \underset{480}{48d_t}, \underset{400}{h_{min}}) = 400\ mm$ ok ✓

The column section reinforcement design is satisfactory.

2

HW2 DRCSII ۷ هو ۸ حنیف الصادقی ۹۱۱۰۵۰۴۷

Q2:

ج ۲-

ستون داخلی $600 \times 600 \longrightarrow$ ستونهای B و C و D $\quad E_s = 200000$ Mpa

ستون خارجی $~~400~~ \times ~~400~~$ 500 x 500 $\longrightarrow$ ستونهای A و E $\quad E_c = 4700\sqrt{f'_c} = 24870$ Mpa

تیر 900×600 $\quad f'_c = 28$ Mpa $\quad f_y = 420$ Mpa $\quad 28$

عمق x عرض

$$\begin{cases} M_1 = M_{1ns} + \delta_s M_{1s} \\ M_2 = M_{2ns} + \delta_s M_{2s} \end{cases} \qquad \delta_s = \frac{1}{1 - \dfrac{\sum P_u}{0.75 \sum P_c}}$$

چون مقدار Δ و درنتیجه Q را نداریم از این رابطه استفاده می‌کنیم.

load combination

case 1 : $1.4D$

case 2 : $1.2D + 1.6L$

case 4 : $1.2D \pm 1.0W + 1.0L$

case 6 : $0.9D \pm 1.0W$

by engineering judgment :

$\rightarrow$ for p_u case 2 governs

for m_u case 4 governs

But you have to Choose one of them at a time.

-5

با توجه به اینکه مجموع بار محوری ستون که ناشی از باد است در یک طبقه به صورت کششی و فشاری و نهایتاً صفر است حتی اگر در یک case ای شامل بار باد از بقیه max می شد نیازی به محاسبه ی آن نبود و به سایر بارهای Dead و live اکتفا می شد.

For column C_2 : $P_u = 1.2 \times 3062 + 1.6 \times 1313 = 5775.2$ KN

For columns B_2 & D_2 : $P_u = 1.2 \times 3469 + 1.6 \times 1766 = 6988.4$ KN

For columns A2 & E2 : $P_u = 1.2 \times 1749 + 1.6 \times 610 = 3074.8$ kN

$$\sum P_u = 5775.2 + 2 \times 6988.4 + 2 \times 3074.8 = 25901.6 \text{ KN}$$

The Frame is sway $\rightarrow K > 1$

because the lateral load is wind $\beta_{ds} = 0$

Is it slender ??

-5

For B2, D2, C2 $\rightarrow (EI)_{column} = \dfrac{0.4 E_c I}{1 + \beta_{ds}} = 0.4 E_c I = 0.4 \times 24870 \times \left(\frac{1}{12} \times 600 \times 600^3\right)$

$= 1.0744 \times 10^{14}$ N.mm^2

For A2 & E2 $\rightarrow (EI)_{column} = 0.4 \times 24870 \times \left(\frac{1}{12} \times 400 \times 400^3\right) = 2.122 \times 10^{13}$ N.mm^2

$$(EI)_{beam} = 24870 \times \left(\frac{1}{12} \times 900 \times 600^3\right) = 4.029 \times 10^{14} \text{ N.mm}^2$$

calculation of K for coulumns

For B2, C2, D2 $\rightarrow \Psi_1 = \Psi_2 = \dfrac{\sum \left(\frac{EI}{l_c}\right)_{column}}{\sum \left(\frac{EI}{L_c}\right)_{beam}} = \dfrac{2 \times \dfrac{1.0744 \times 10^{14}}{4.8}}{2 \times \dfrac{4.029 \times 10^{14}}{12}} = 0.664$

From nomograph $\rightarrow K = 1.2$

3

For A2, E2 $\quad \Psi_1 = \Psi_2 = \dfrac{2 \times \dfrac{2.122 \times 10^{13}}{4.8}}{1 \times \dfrac{4.029 \times 10^{14}}{12}} = 0.263 \longrightarrow K = 1.1$

calculation of P_c :

For B2, C2, D2 $\rightarrow \quad P_c = \dfrac{\pi^2 EI}{(K l_u)^2} = \dfrac{\pi^2 \times (1.0744 \times 10^{14})}{(1.2 \times 4200)^2} = 4.175 \times 10^7 \ N$

For A2, E2 $\rightarrow \quad P_c = \dfrac{\pi^2 EI}{(K l_u)^2} = \dfrac{\pi^2 \times (2.122 \times 10^{13})}{(1.1 \times 4200)^2} = 9.767 \times 10^6 \ N$

$$\Sigma P_c = 3 \times 41750 + 2 \times 9767 = 144.78 \times 10^3 \ kN$$

$$\delta_s = \frac{1}{1 - \dfrac{\Sigma P_u}{0.75 \Sigma P_c}} = \frac{1}{1 - \dfrac{25901.6}{0.75 \times (144.78 \times 10^3)}} = 1.313 \geq 1 \qquad 1 \leq \delta_s < 1.5$$

ok ✓

For column B2 :

$$M = M_{ns} + \delta_s M_s$$

$P_u = 6988.4 \times 10^3 \ N$ ACI 9.2.1(b) -5

$M_{1s} = \cancel{1.0}\ 1.6 \times -131 = -131 \ kN.m$

$M_{2s} = 1.0 \times 141 = 141 \ kN.m$

$M_{1ns} = 1.2 \times -45.5 + 1 \times 145 = 90.4 \ kN.m$

$M_{2ns} = 1.2 \times 41.5 + 1 \times 216 = 265.8 \ kN.m$

$M_1 = 90.4 + 1.313 \times -131 = -81.6 \ kN.m$

$M_2 = 265.8 + 1.313 \times 141 = 450.933 \ kN.m$

$K_n = \dfrac{P_u}{\phi f'_c A_g} = \dfrac{6488 \times 10^3}{0.65 \times 28 \times (600^2)} = 1.0666 \xrightarrow{\text{Assume } \gamma = 0.7} \rho = 0.035$

$R_n = \dfrac{M_u}{\phi f'_c A_g h} = \dfrac{450.933 \times 10^6}{0.65 \times 28 \times 600^2 \times 600} = 0.1147$

$A_{st} = 0.035 \times A_g = 0.035 \times (600^2) = 12600 \ mm^2 \longrightarrow$ take $16\phi32 = 12868 \ mm^2$ ✓

$d_b \leq 32 \longrightarrow d_t = 10 \ mm$ ok ✓

سيخ طولي $S_{min} = Max(40 \ mm, 1.5 \times \overset{d_b}{32}, 1.33 . \overset{34}{D_{max}}) = 48 \ mm$ ✓

[illegible] $S_{max} = Min(16 \times \overset{d_b}{32}, 48 \overset{10}{d_t}, h_{min}) = Min(512, 480, 600) = 480 \ mm$ ✓

take cover = 65 mm ✓

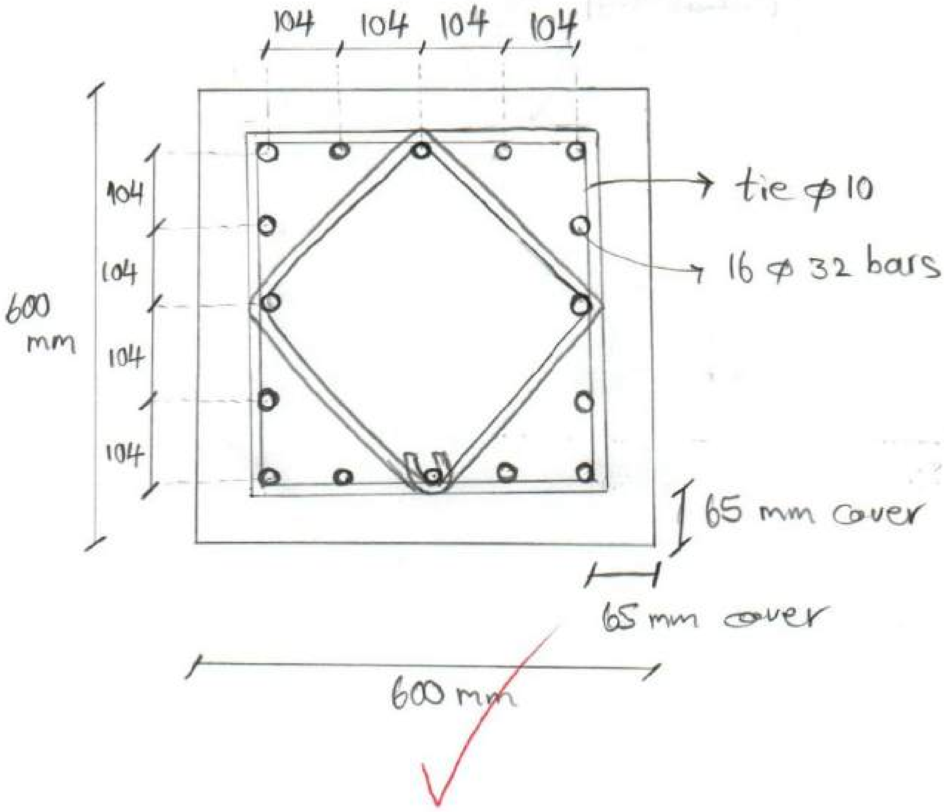
4 " sketch in the Next page "

HW2 DRCS II

۶ هو ۸

حنیف اله صادقی ۹۱۱۰۵۰۴۷

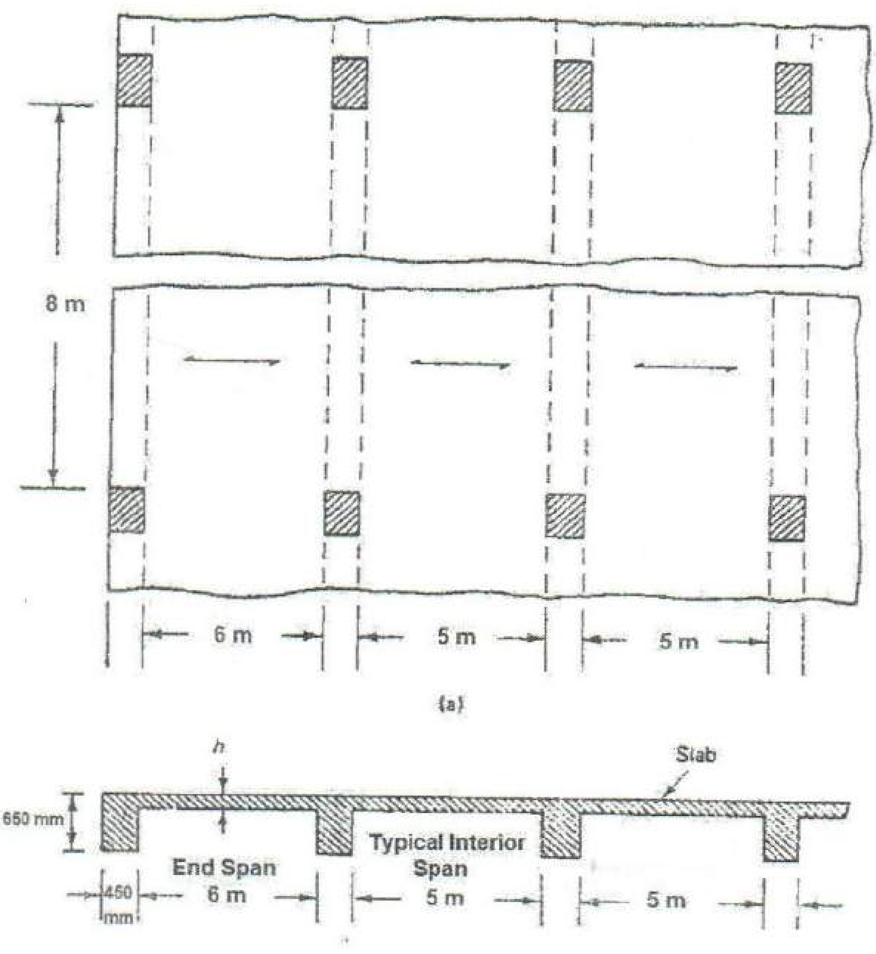
ادامه ج۲- رسم مقطع نهایی بدست آمده برای این سوال :



Habib Sadeghi

1- A reinforced concrete building floor system consists of a continuous one-way slab built monolithically with its supporting beams of dimensions 450 mm wide and 650 mm high, as shown below. Service live load will be 5X0 kg/m^2 (e.g. 570). Dead loads are due to self-weight plus 1Y0 kg/m^2 (e.g. 190) for suspended loads. Using ACI coefficients from Chapter 12, calculate the design moments and shears and design the slab. Use all straight bar reinforcement. Summarize your design with a sketch showing concrete dimensions, and size, spacing, and cutoff points for all reinforcement. Assume steel reinforcement grade AIII and $f_c' = 25$ MPa.

Note: X and Y are the right figures of your student ID number, e.g. X=7 and Y=9 for 9215079.



2- Solve problem 1, if the precast joist and block floor system is used. Use the Bulletin No. 543 and sketch your final design for the precast joists, blocks, and the slab. Polystyrene blocks are used as the filler.

بسمه تعالی

دانشـــــگاه صنعتـــی شریف

دانشکــــــده ی مهندســـی عمران

طراحی سازه های بتنی ۲

استاد درس : دکتر اسکندری

گروه دوشنبه

تمرین سری سوم

نام و نام خانوادگی : حبیب اله صادقی ۹۱۱۰۵۰۴۷

$\min(1.1\times 95, 100) = \underline{100}$

تاریخ تحویل ۱۳۹۴/۲/۱۴

نیمسال دوم سال تحصیلی ۹۴-۱۳۹۳

HW3 DRCS II   ۲ هور ۴   حسین الصادقی ۹۱۱۰۵۰۴۷

Q1 :   ID = 91105047 $x = 4$ $y = 7$

(X = 4, Y = 7)

Live = 540 kg/m2   Dead = 170 kg/m2 + self weight $\gamma_{concrete} = 2400$ kg/m3

AIII reinforcement $f'_c = 25$ Mpa
↳ $f_y =$

for beam self weight assume h = 300 mm = 0.3 m

$w_{self\ weight} = 2400 \times (0.35) = 840$ kg/m2

عمق متوسط فرض

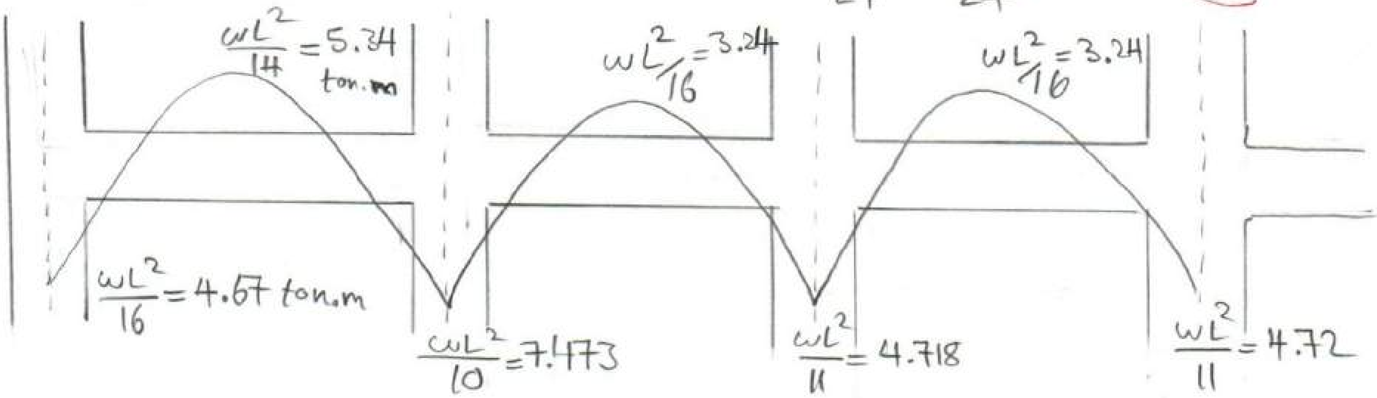
load combination

load 1 : $1.4D = 1.4 \times (170 + 840) = 1414$ kg/m2

load 2 : $1.2D + 1.6L = 1.2(170 + 840) + 1.6 \times 540 = 2076$ kg/m2 ✓

take cover = 20 mm ← not exposed to Earth & weather

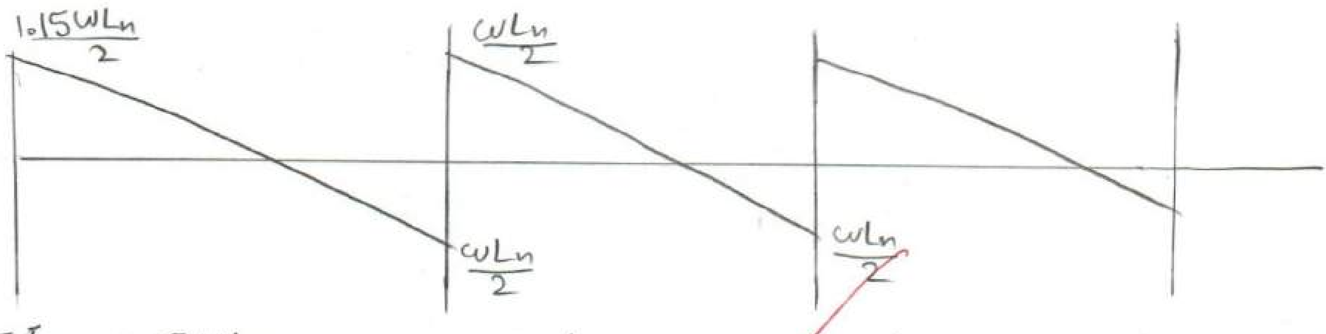
Minimum thickness of slab due to deflection $\frac{L}{24} = \frac{6}{24} = 0.25$ m = 250 mm



$$\begin{cases} A_s = \dfrac{M_u}{\phi f_y (d - \frac{a}{2})} \\ a = \dfrac{A_s \times f_y}{0.85 f'_c b} \end{cases}$$

→ برای هرکدام از لنگرها باید مقدار As را بدست آوریم. (ادامه در صفحه‌ی بعدی)

$\phi V_c = 0.75 \times 0.17 \sqrt{f'_c}\, b w d = 0.75 \times 0.17 \times \sqrt{25} \times 1000 \times (300 - 30) = 39.02$ ton



Max $V_u = 1.15 \frac{wL_n}{2} = 1.15 \times 2.076 \times \frac{6}{2} = 7.162 < \frac{\phi V}{2}$ → no need to shear reinforcement.

1

$d = 270$

$M_{u_1} = 4.67$ ton.m

$A_s = \dfrac{4.67\times10^7}{0.9\times400(270-\frac{a}{2})} \xrightarrow{\text{take } a=10} A_s = 489.5\ mm^2 \longrightarrow \phi 8 @ 100 \rightarrow 503\ mm^2$

$a = \dfrac{A_s\times400}{0.85\times25\times1000} = 9.2$ mm ok ✓

$M_{u_2} = 5.34$ ton.m

$A_s = \dfrac{5.34\times10^7}{0.9\times400(270-\frac{a}{2})} \xrightarrow{\text{take } a=10} A_s = 559.7\ mm^2 \longrightarrow \phi 10 @ 125 \rightarrow 625\ mm^2$

$a = \dfrac{A_s\times400}{0.85\times25\times1000} = 10.5$ mm ✓ ok

---

$M_{u_3} = 7.473$ ton.m

$A_s = \dfrac{7.473\times10^7}{0.9\times400(270-\frac{a}{2})} \xrightarrow{\text{take } a=15} A_s = 814\ mm^2 \longrightarrow \phi 14 @ 175 \rightarrow 880$

$a = \dfrac{A_s\times400}{0.85\times25\times1000} = 15.3$ ok ✓

$M_{u_4} = 3.24$ ton.m

$A_s = \dfrac{3.24\times10^7}{0.9\times400\times(270-\frac{a}{2})} \xrightarrow{\text{take } a=6} A_s = 337\ mm^2 \longrightarrow \phi 8 @ 125 \rightarrow 402$

$a = \dfrac{A_s\times400}{0.85\times25\times100} = 6.7$ ok ✓

$M_{u_5} = 4.718$

$A_s = \dfrac{4.72\times10^7}{0.9\times400\times(270-\frac{a}{2})} = \xrightarrow{\text{take } a=8} A_s = 475\ mm^2 \longrightarrow \phi 10 @ 150 \rightarrow 524$

$a = \dfrac{A_s\times400}{0.85\times25\times1000} = 8.9$ mm ok ✓ ✓

$A_{s\ temperature} = 0.0018\,bh = 0.0018\times1000\times300 = 540\ mm^2 \rightarrow \phi 10 @ 125$
$(f_y = 420)$

Flexural $S_{max} = \text{Min}(3h, 450\ mm) = 450$ mm

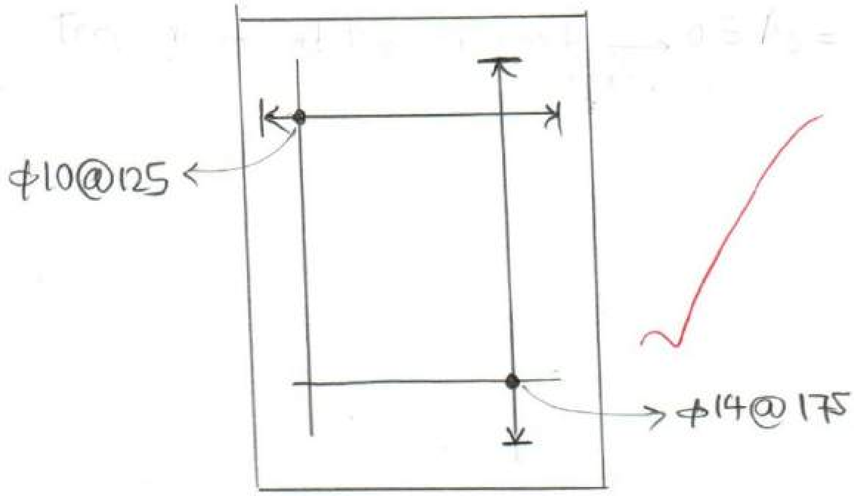
Temperature & shrinkage $S_{max} = \text{Min}(\overset{1500}{5h}, 450) = 450$ mm ✓

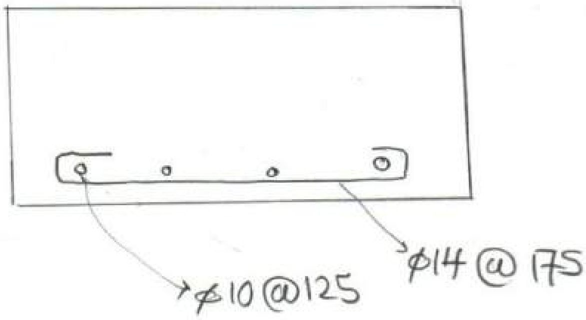
HW3 DRCS II «هو» حشمت‌اله صادقی ۹۱۱۰۵۰۵۰۴۷

الف Q1: crack controll

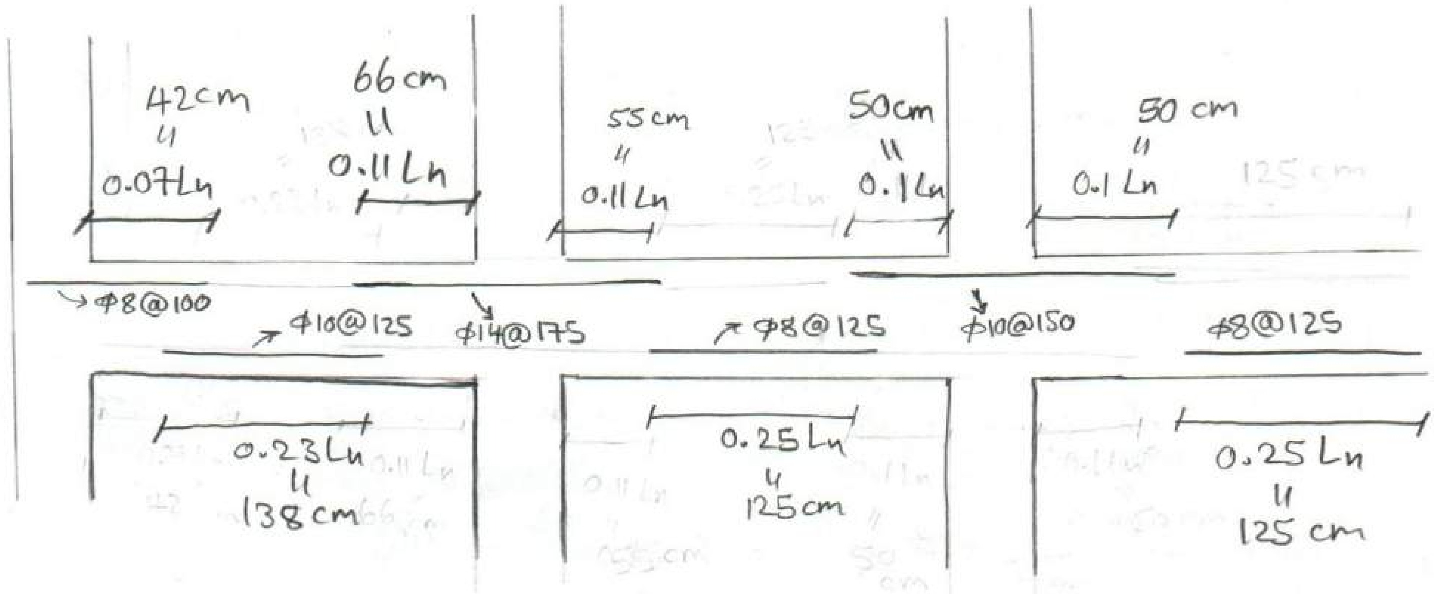
$$S_{max} = 380\left(\frac{280}{\underbrace{\tfrac{2}{3}f_y}_{f_{service}}}\right) - 2.5C_c \leqslant 300\left(\frac{280}{f_s}\right)$$

$$= 380 \times \frac{280}{\frac{2}{3}\times 400} - 2.5\times 20 = 349 \leqslant 315 \longrightarrow S_{max} = 315 \text{ mm}$$





using approximate method for cut off → graph A.3 Nilson



در محل‌های مشخص شده می‌توان 50٪ از میلگردها را cut off زد و بقیه را ادامه داد البته مقادیر نباید از مقادیر کمتر گردد که جلوتر مطرح می‌گردد.

theoritical cut off points Calculations (-5)

Q2:

طراحی اساس تیرچه و بلوک

Minimum thickness due to deflection $\xrightarrow[\text{simply supported}]{\text{oneway slab}}$ $\frac{L}{20} = \frac{6}{20} = 300$ mm

$L = 540$ kg/m²

we use this as H_{min} for the all spans ←

let's calculate Dead load:

block height = 25 cm block mass = 10 kg

From graph ۲-۱ پ $\xrightarrow{\text{فاصله‌ی دو تیرچه } 500}$ $P = 350$ kg/m² + 170 kg/m² = 520 kg/m²

در حالت سرویس ← $q_u = 520 + 540 = 1060$ kg/m²

load combinations based on bulletin No 543
$$\begin{cases} 1.4D = 1.4 \times 520 = 728 \text{ kg/m}^2 \\ 1.25D + 1.5L = 1.25 \times 520 + 1.5 \times 540 = 1460 \text{ kg/m}^2 \checkmark \end{cases}$$

$$M_u = \frac{qL^2}{12} = \frac{1060 \times 0.4^2}{12} = 14.13 \text{ kg.m}$$

$$S = \frac{\overset{\text{عرض واحد}}{100} \times H^2}{6} \quad \text{(ارتفاع مقطع)}$$

$$S = \frac{100 \times H^2}{6} = 16.6 H^2$$

$$f_{ct} = \frac{M_u}{S} = \frac{14.13 \times 10^4}{16.6 H^2} = \frac{851}{H^2}$$

$$f_r = 0.62\sqrt{f'_c} = 0.62\sqrt{25} = 3.1$$

$$f_{ct} \le f_r \quad \frac{851}{H^2} \le 3.1 \rightarrow H \geq 16.5 \text{ mm}$$

← حداقل ضخامت دال بتنی روی بلوک برای ضابطه‌ی ترک خوردگی ۱.۶۵ cm است.

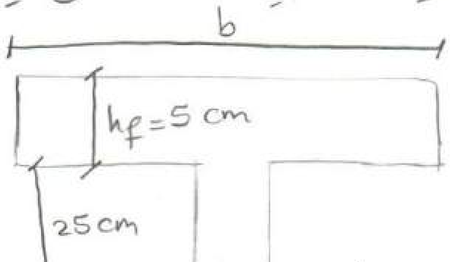
ضابطه‌ی دیگری هم وجود دارد:

$$H_{Min} = Max\left\{50 \text{ mm}, \frac{1}{12} \times \underbrace{\text{فاصله‌ی بین دو تیرچه}}_{500}\right\} = 50 \text{ mm}$$

بنابراین حداقل ضخامت دال بتنی 5 cm است.

$$\begin{cases} P_u = 1460 \times 0.5 = 730 \text{ kg/m} \\ M_u = \frac{P_u L_e^2}{8} = \frac{730 \times 6^2}{8} = 3285 \text{ kg.m} \end{cases}$$

طبق فرضیات نشریه‌ی 543 طرح تیرچه و بلوک باید به صورت T شکل در نظر گرفته شود.



$b_w = 10$ cm ← 10 cm حداقل عرض پاشنه

cover = 20 mm جدول ۱-۲ استاندارد 543 : شرایط تهران ملایم است

$$2 \times 20 + \underbrace{12}_{d_b} = 52 \text{ mm} \rightarrow h' = 6 \text{ cm}$$

$$d = 30 - \frac{6}{2} = 27 \text{ cm}$$

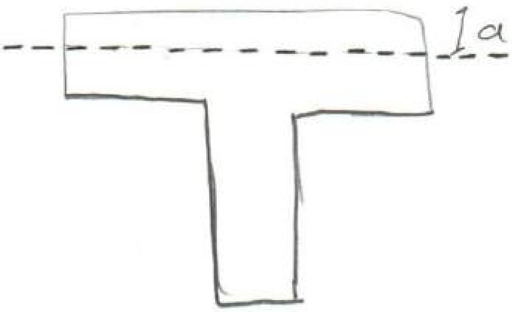
عرض بال = فاصله‌ی آزاد بین دو تیرچه = 50 cm

HW DRCS II   ۸ نمره   حنیف اله صادقی ۹۱۱۰۵۰۲۷

Contine Q2 :

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

$0.9 \times 3285 \times 10^4 = 0.85 \times 25 \times a \times 500 \times (270 - \frac{a}{2}) \rightarrow a = 10.16 \text{ mm} \geqslant h_f = 50 \text{ mm}$



$A_s = \frac{0.85 f'_c ab}{f_y} = \frac{0.85 \times 25 \times 10.16 \times 500}{400} = 269.875 \text{ mm}^2$

$A_{s_{min}} = \max\left\{ \frac{0.25 \sqrt{f'_c} \times b_w \times d}{f_y} , \frac{1.4 b_w d}{f_y} \right\}$

$= \max\left\{ \frac{0.25 \sqrt{25} \times 100 \times 270}{400} , \frac{1.4 \times 100 \times 270}{400} \right\} = 94.5 \text{ mm}^2$

$A_s \geqslant A_{s_{min}} \longrightarrow$ use $1.33 A_{s_{min}} = 94.5 \times 1.33 = 125.69 \text{ mm}^2$

$\downarrow$ use $2\phi 10$

$V_c = 0.17 \lambda \sqrt{f'_c} b_w d = 0.17 \times 1 \sqrt{25} \times 100 \times 270 = 2.295$ ton

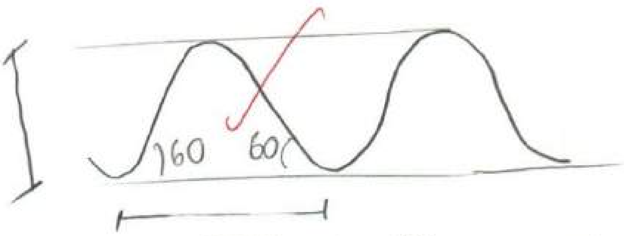
$\phi V_c = 0.75 \times 2.295 = 1.721$ ton

$V_u = \frac{P_u L}{2} = \frac{730 \times 6}{2} = 2190 \text{ kg} = 2.19$ ton

$V_u > \phi V_c$ $\quad \frac{V_u}{\phi} - V_c = V_s = 1.199$ ton $\quad (\frac{A_v}{S})_{req} = \frac{V_{s\,req}}{f_y d}$

$A_{s_{min}} =$

$28.75 = 25 + \frac{5}{2}$
$= 27.5$



با توجه به معیارهای آئین نامه مقدار زاویه را 60° درنظر گرفتیم.

$2 \times 275 \times \tan 30 = 317.5$

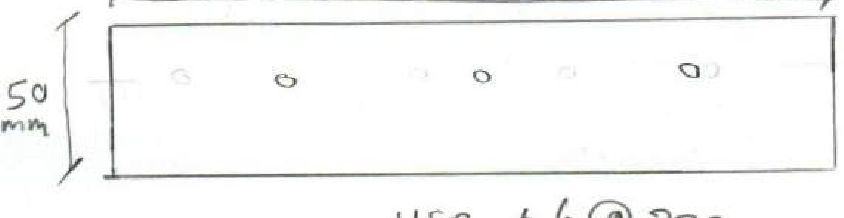
$A_{v\,req} = \frac{1.199 \times 10^4 \times 317.5}{300 \times 270} = 47 \text{ mm}^2 \longrightarrow 2\phi 6 = 56.5 \text{ mm}^2$ ✓

طرح آرماتور فوقانی تیرچه ← جدول 2-2 ← $\phi 8$

آرماتور حرارت و جمع شدگی دال : ۲-۳-۲-۵ نشریه 543

$0.0018 A_g = 0.0018 \times 1000 \times 50 = 90 \text{ mm}^2 / 1000 \text{ mm} \rightarrow$ use $\phi 6$ @ 300

یک متر عرض mm 1000



20 mm → این آرماتورها باید در هر دو جهت و از 20cm از سطح بتن قرار بگیرد

use $\phi 6$ @ 300

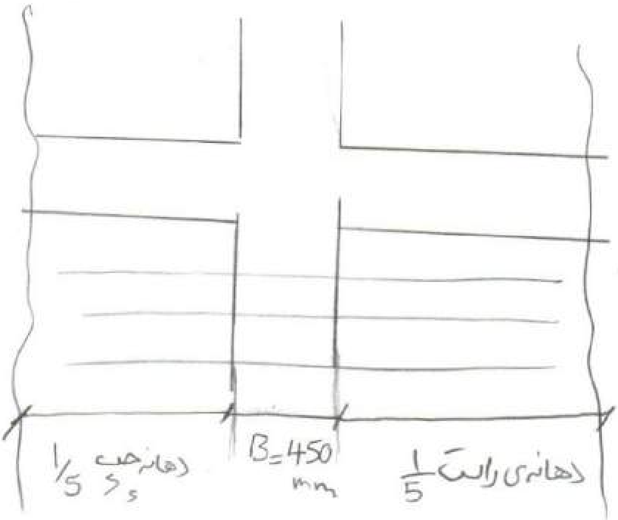
طرح آرماتور منفی در بر تکیه‌گاه ۵ باید 15% از آرماتور کششی تیرچه در فاصله‌ی 1/5 از طول دهانه از بر تکیه‌گاه به عنوان آرماتور منفی در نظر گرفته شود.

$$\frac{15}{100} \times 90 = 13.5\ mm^2$$

این مساحت کاربردی نیست ← از Φ8 استفاده می‌کنیم.

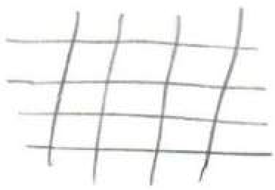
طول کل آرماتور منفی = [1/5 دهانه‌ی چپ + 1/5 دهانه‌ی راست + B (عرض شاهتیر)] $= \left[\frac{6}{5} + \frac{5}{5} + 0.45\right]$

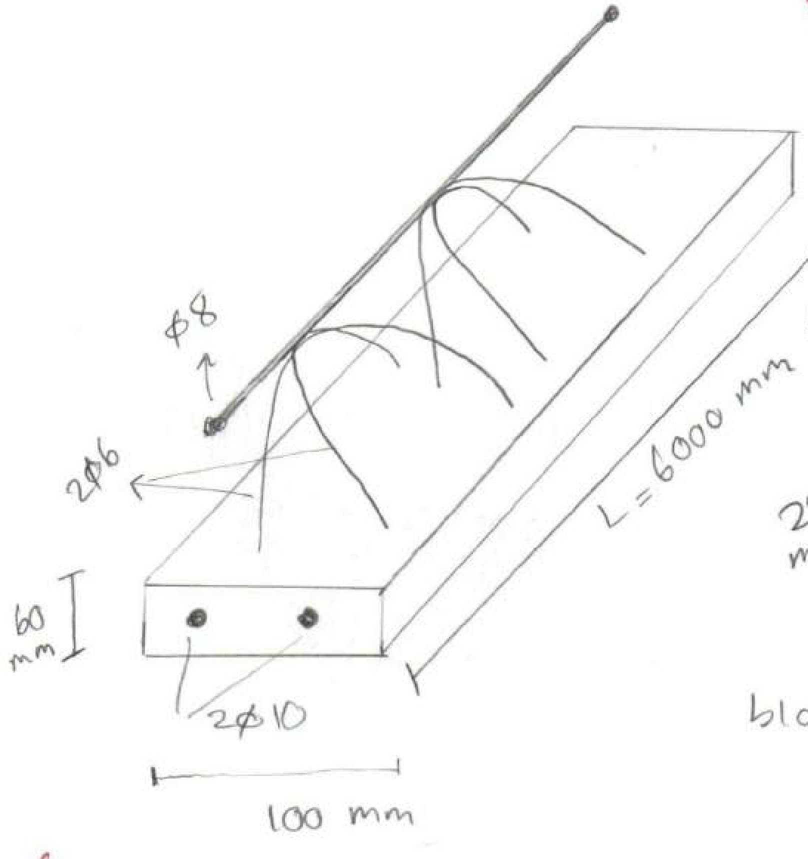
$= 2.65\ m$

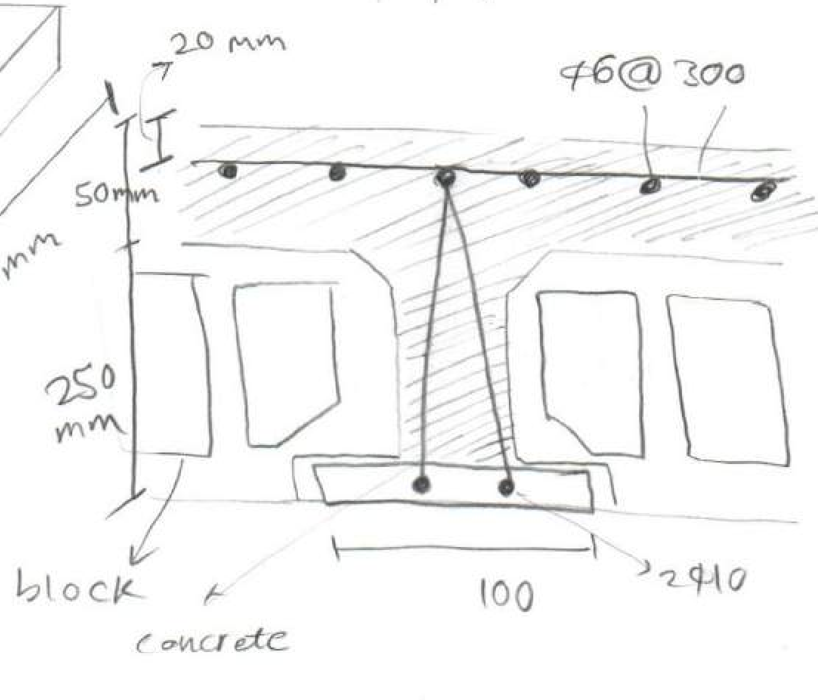


کلاف میانی ۵: براساس بند ۵ ضوابط طرح کلاف میانی چون بار زنده بیشتر از 350 است نیاز به 2 عدد کلاف داریم ← سطح مقطع آرماتورهای کششی وسط دهانه را در داخل کلاف قرار می‌دهیم.

(حداقل یکی در بالا و یکی در پایین آجدار) 2Φ10



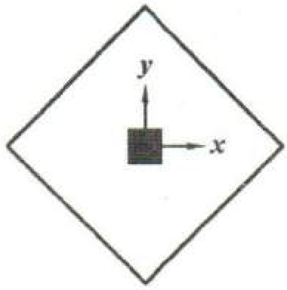




Design of RC Structures II – Spring 2015 | **HW4** | Footings
Due: Group 1: Ordibehesht 26th, 1394 | Related Chapter: Ch. 16
Group 2: Ordibehesht 28th, 1394

1- Design a reinforced concrete wall footing for a 300 mm masonry wall. The intensity of service linear dead load is 45 kN/m and a service linear live load 50 kN/m of wall length. The frost line is assumed to be 600 mm below grade. Given: $\gamma_s = 18\ \mathrm{kN/m^3}$, $q_a = 90$ kPa, $f'_c = 21$ MPa, $f_y = 400$ MPa.

2- A rectangular or square footing is required to support a square 600 mm column located in 45° offset from the footing axes, as shown in the figure. The applied loads are listed in the following table. Design the footing. Your design shall include base dimensions, footing thickness, reinforcement layout. $q_{net} = 1.8\ \mathrm{kg/cm^2}$.



Load	P (ton)	Mx (ton.m)
Dead	85	0
Live	44	0
Earthquake	33	15

3- Two interior columns for a high-rise concrete structure are spaced 4.5 m apart, and each carries service loads D = 220 tons and L = 257 tons. The columns are to be 550 mm square in cross section, and will each be reinforced with twelve Φ32 bars centered 70 mm from the column faces, with an equal number of bars at each face. For the columns, $f'_c = 30$ MPa and $f_y = 400$ MPa. The columns will be supported on a rectangular combined footing with a long side dimension twice that of the short side. The allowable soil-bearing pressure is 3 kg/cm². The bottom of the footing will be 1.8 m below grade. Design the footing for these columns, using $f'_c = 21$ MPa and $f_y = 400$ MPa. Specify all reinforcement, including length and placement of footing bars and dowel steel.

d = 800 mm

Φ

0.9

$\frac{40}{4800} d^2 + 2d - 1.2168$

0.530

بسمه تعالی

دانشـــــــگاه صنعتـــی شریف

دانشکـــــــده‌ی مهندســـی عمران

استاد درس : دکتر اسکندری

100 - 10+5 595

گروه دوشنبه

پروژه‌ی شماره ۲ طراحی سازه های بتنی ۲

تمرین سری چهارم HW#4

نام و نام خانوادگی : حبیب اله صادقی ۹۱۱۰۵۰۴۷

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HW4 DRCS II

"هو"

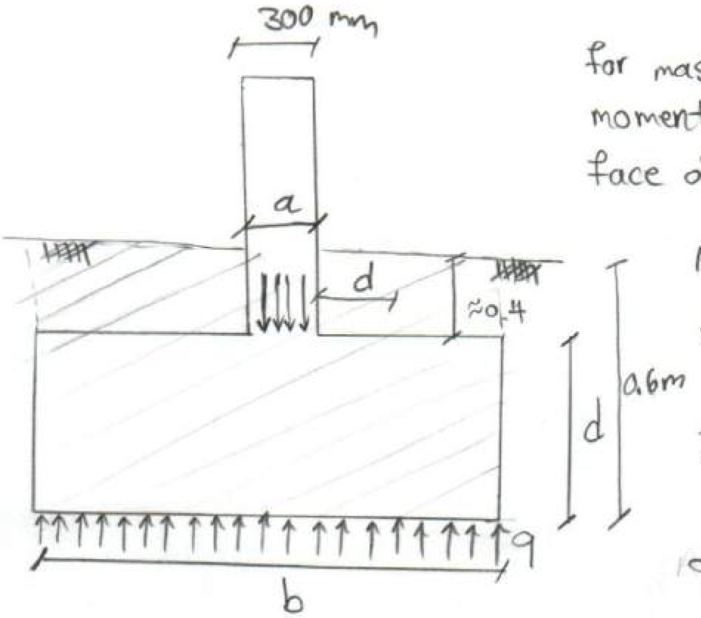
حسنة الصادقی ۹۱۱۰۵۰۶۷

Q1: 300 mm masonary wall

Frost line = 600 mm below ground surface

service loads $\begin{cases} \text{Dead } 45\ kN/m \\ \text{live } 50\ kN/m \end{cases}$ $\gamma_s = 18\ kN/m^3$ $f'_c = 21\ Mpa$

$q_a = 90\ kpa = \frac{kN}{m^2}$ $f'_y = 400\ Mpa$



for masonary wall footing the maximum moment is moment is compted midway between the middle & the face of the wall. (masonary is less rigid than concrete)

$$M_u = q_u \times \left(\frac{b}{2}-\frac{a}{4}\right)\times\left(\frac{b}{2}-\frac{a}{4}\right)\times\frac{1}{2} = \frac{1}{8} q_u \left(b-\frac{a}{2}\right)^2$$

$$V_u = q_u\left(\frac{b-a}{2} - d\right)$$

$$P_{service} = 45+50 = 95\ kN/m \xrightarrow[\text{length of wall}]{\text{for } 1m} P_{service} = 95\ kN$$

$$q_e = 90 + (0.6-0.4)\times\gamma_s - 20 \times 0.6$$

دقیقا منطق این کار مشخص نیست

$\gamma_c = 21$ & $\gamma_s = 18$

See Nilson Ex 16.1

$20\ kN/m^3 =$ γ ناحیه‌ی هاشور خورده

we Assume that B = 4 m

$$q_e = 90 + (0.6-0.4)\times 18 - 20 \times 0.6 = 81.6$$

$$A_{req} = \frac{P_{service}}{q_e} = \frac{95}{81.6} = b \times 1 \longrightarrow b_{min} = 1.16\ m \rightarrow \text{take } b = 2.5\ m$$

(too much & conservative)

ultimate loads & bearing pressure:

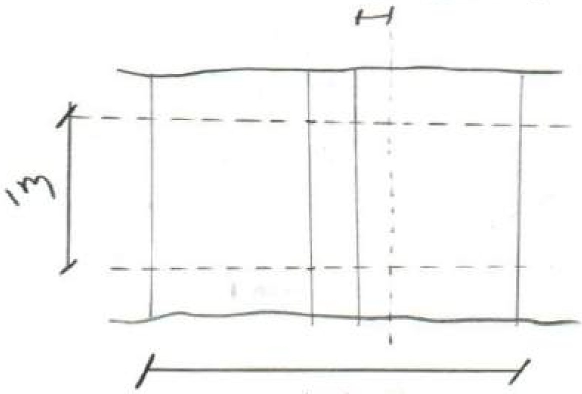
$$P_u = 1.2P_D + 1.6P_L = 1.2\times 45 + 1.6\times 50 = 134\ kN/m$$

$$q_u = \frac{P_u}{A_f} = \frac{134\ kN/m}{\underset{b}{2.5\ m}} = 53.6\ kN/m^2 < q_{all} \text{ OK } ✓$$

* Depth of footing:

one way shear

Assume d = 200 mm ضخامت دیوار



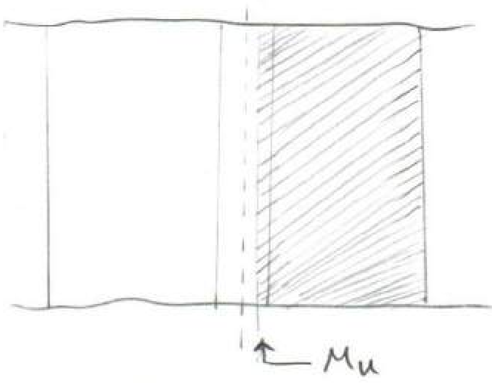
$$V_u = (53.6)\times\left(\frac{2.5}{2} - \frac{0.3}{2} - 0.2\right)\times 1$$

$$= 48.24\ kN$$

$$V_c = 0.17\sqrt{f'_c}\,bd \longrightarrow d_{req} = \frac{48.24\times 10^3}{0.75\times 0.17\times\sqrt{21}\times 1000} = 82.56\ mm \text{ OK } ✓$$

Therefore, select the overal depth of 300 mm is ok. ✓

Flexural Reinforcement in short direction:



$M_u = (53.6)(2.5 - \frac{0.3}{4})^2 \times \frac{1}{2} \times 1 = 157.6$ kN.m

Assume $\phi 20$ rebar, cover 50 mm

$d = 300 - 50 - 20 - \frac{20}{2} = 220$ mm

wrong cover (-3)

$A_{s req} = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{157 \times 10^6}{0.9 \times 400 \times (220 - \frac{a}{2})} = 2236.4$ mm^2 ✓ $(a = 50)$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2236.4 \times 400}{0.85 \times 21 \times 1000} = 50.1$ mm ≈ 50 OK ✓

$A_{s req} = 2236.4$ mm^2 use $8\phi 20$ (2513 mm^2)

Shrinkage & temperature reinf in wall direction:

$A_{s\,temp} = 0.0018\, A_g = 0.0018 \times (2500 \times 300) = 1350$ mm^2 ✓ $5\phi 20$ (1571 mm^2)

↳ better to use $6\phi 20$

minimum flexural reinf

Chaosing reinforcement

$A_{s\,Min} = Max\left(\frac{0.25\sqrt{f'_c}}{f_y} b_w d, \frac{1.4 b_w d}{f_y}\right) = 770$ mm^2 < $A_{s\,pro}$ OK ✓

$A_{s\,min} = 1.33\, A_{s\,req} = 1.33 \times 2236.4 = 2974$ mm^2 (-5) controls $10\phi 20$ 3142 mm^2

only

spacing of bars → This is just for situations where $A_{s,min} > A_{s,req}$.

$S_{max} = Min(3h, 450) = 450$ OK

$S_{min} = Max(d_b, 25\text{mm}, 1.33 d_c) = 34$ mm (20, 34)

$S = \frac{1000 - 2(75)}{10 - 1} = 94$ m $S_{Min} < S < S_{max}$ OK ✓

Bearing capacity at wall base

$\phi N_n = \phi(0.85) f'_c A_1 \sqrt{\frac{A_2}{A_1}} = 0.65 \times 0.85 \times 21 \times (300 \times 1000) \times \sqrt{2} = 4922$ kN > 134 OK

Dowels:

$A_{sd} = 0.005\, A_g = 0.005 \times (300 \times 1000) = 1500$ mm^2 → $5\phi 20$ 1571 mm^2

HW#4 DRCS II

٩١١٠٥٠٤٧ حسن الرصاعي

Q1 : continue ;

development length of flexural reinf

$$L_d = \left[\frac{f_y}{1.1\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \lambda}{\left(\frac{c_b + K_{tr}}{d_b}\right)} \right] d_b \geq 300$$

$\psi_e = \psi_t = \psi_s = \lambda = 1$

$k_{tr} = 0$

$c_b = 75 + \frac{20}{2} = 85$ mm

$$\frac{C_b + K_{tr}}{d_b} = \frac{85+0}{20} = 4.25$$

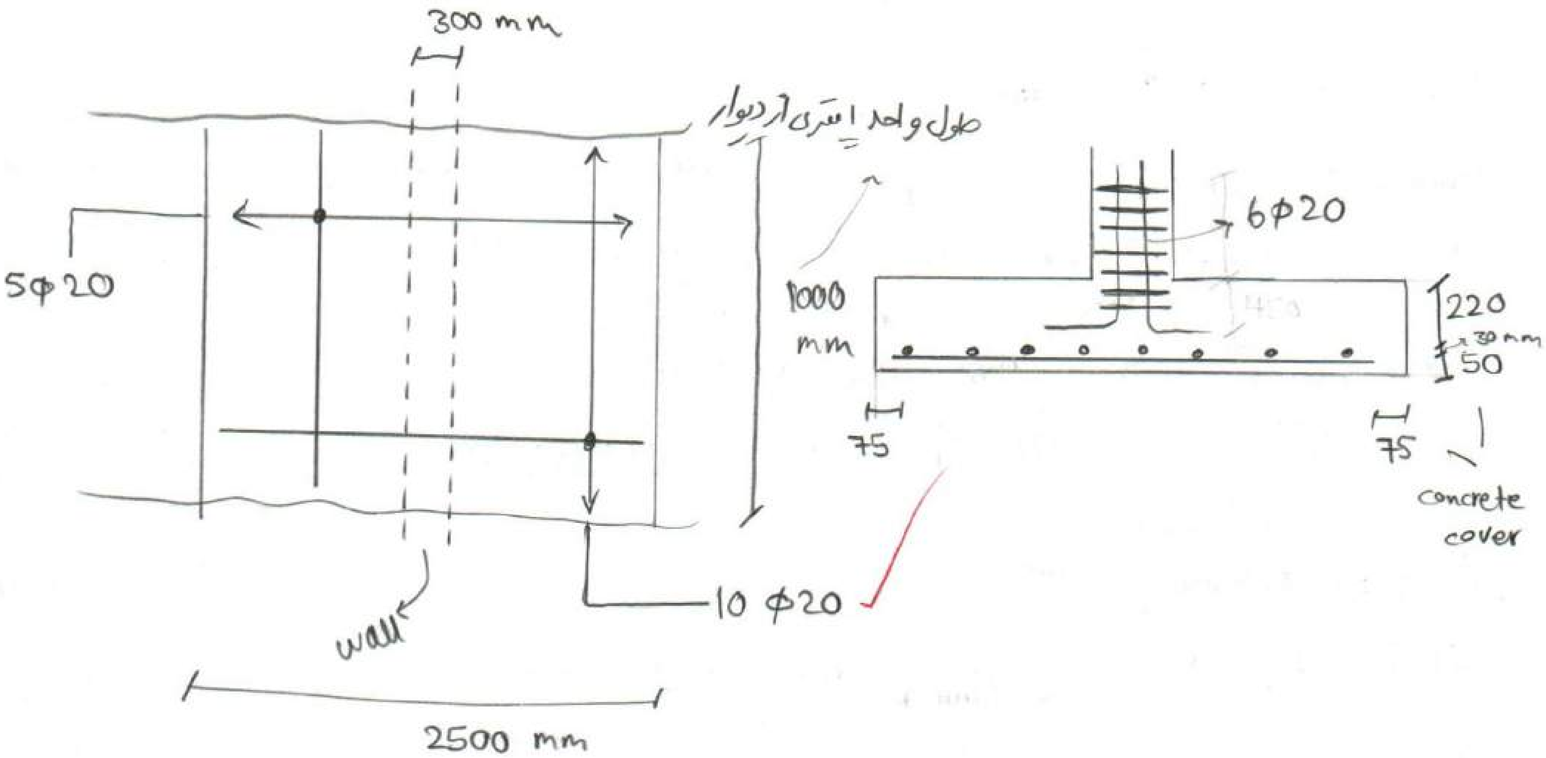
$$L_d = \left[\frac{400}{1.1\sqrt{21}} \cdot \frac{1\times1\times1\times1}{4.25} \right] \times 20 = 373.4 > 300 \quad \text{OK} ✓$$

$$L_{pro} = \frac{2500}{2} - \frac{300}{2} - 75 = 1025 \text{ mm} > 373.4 \quad \text{OK} ✓$$

developement of Dowels in foundation

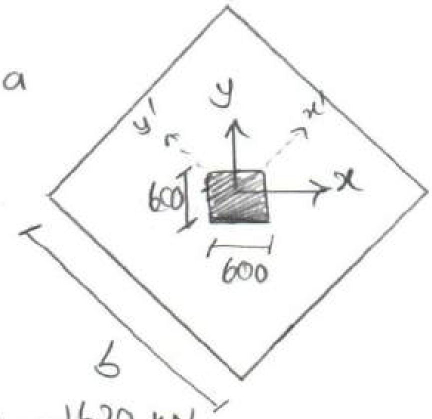
$$L_{dc} = \max\left(\frac{0.24 f_y d_b}{\sqrt{f'_c}}, 0.043 f_y d_b\right) = 419 \text{ mm}$$

Reinforcement



Q2:

Column $600 \times 600 \; mm^2$



$q_{net} = 1.8 \; kg/cm^2 = 180 \; KN/m^2 = 180 \; kpa$

$f'_c = 21 \; Mpa$

$f_y = 400 \; Mpa$

$\gamma_s = 18 \; KN/m^3$

$\gamma_c = 24 \; KN/m^3$

$P_{service} = P_D + P_L \pm P_E = 85 + 44 + 33 = 162 \; ton = 1620 \; KN$

$M_{service} = 150 \; KN.m$

assume that the base is 1.5 m below the ground.

$q_e = 180 + 18 \times 0.5 - 1.5 \times \underbrace{21}_{} = 157.5 \; KN/m^2$

γ معادل γ بتن و γ خاک

لنگر 150 KN.m را در دو جهت x' و y' تجزیه کرده و آن را به طور جداگانه برای این دو حالت بررسی می‌کنیم.

$$\begin{cases} M_{x'} = 15\cos 45 = 106.066 \; KN.m \\ M_{y'} = 15 \sin 45 = 106.066 \; KN.m \end{cases}$$

ابتدا ابعاد اولیه را بدون درنظر گرفتن لنگر بدست آورده سپس اثر لنگر را هم چک می‌کنیم.

$A_{req} = \dfrac{P_{service}}{q_e} = \dfrac{1620}{157.5} = 10.28 \longrightarrow \sqrt{10.28} = 3.2 \longrightarrow b = 3.5 \; m$

select the base 3.5×3.5 m

Ultimate load & bearing pressure:

case 2 : $P_u = 1.2D + 1.6L = 1.2 \times 85 + 1.6 \times 44 = 172.4$ ton

case 5 : $P_u = 1.2D + 1.4E + 1.0L = 1.2 \times 85 + 1.4 \times 33 + 1.0 \times 44 = 192.2$ ton
$M_u = 1.4 M_E = 1.4 \times 106.066 = 148.5 \; KN.m$ } critical

case 7 : $P_u = 0.9D + 1.4E \rightarrow$ not critical

$q_u = \dfrac{P_u}{A_f} \pm \dfrac{Mc}{I} = \dfrac{1922}{3.5^2} + \dfrac{148.5 \times 3.5/2}{\frac{1}{12} 3.5 \times 3.5^3} = 177.68 \; KN/m^2 < q_{net}$ OK

(ابعاد 3.5x3.5 متر مناسب است)

Depth of footing

- punching shear

Assume d = 600

$b_0 = (0.6 + 0.6) \times 4 = 4.8 \; m = 4800 \; mm$

HW#4 DRCS II   ۹۱۱۰۵۰۲۷ حسین الصادقی

Q2: continue

$V_u = 1922\ KN - 177.68 \times (0.6+0.6)^2$

پی را براساس تنش برشی بحرانی که در کناره‌های مربع base رخ می‌دهد طراحی می‌کنیم:

$= 1666.14$

$V_{n\,req} = \frac{V_u}{\phi} = \frac{1666.14}{0.75} = 2221.5\ KN$

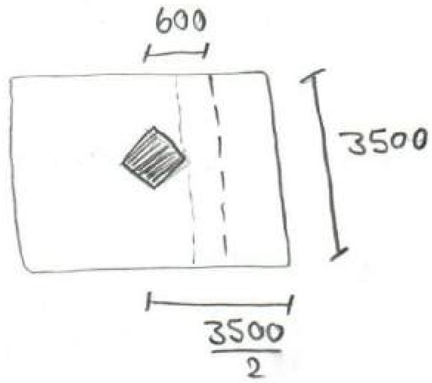
$V_{c_1} = 0.17\left(1+\frac{2}{\beta}\right)\sqrt{f'_c}\, b_o d \qquad \beta = 1 \longrightarrow$ not applicable

$V_{c_2} = 0.083\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d \xrightarrow[\alpha_s = 40]{\text{interior column}} V_{c_2} = 0.083\left(\frac{40 \times d}{4800}+2\right)\sqrt{21} \times 4800 \times d$

$= 2221.5 \times 10^3 \longrightarrow d = 280.5\ mm$ ok ✓

$V_{c_3} = 0.33\sqrt{f'_c}\, b_o d = 0.33\sqrt{21} \times 4800 \times d = 2221.5 \times 10^3 \rightarrow d = 306\ mm$ ok

one way shear:



$V_u = 10^{-6} \times 177.68 \times \left(\frac{3500}{2} - \frac{\sqrt{2} \times 600}{2} - 600\right) \times 3500$

$= 451.3\ KN$

$V_c = 0.17\sqrt{f'_c}\, b d$

$d_{req} = \frac{451.3 \times 10^3}{0.75 \times 0.17 \times \sqrt{21} \times 3500} = 220\ mm$ ok

Therefore select overal depth $H = 700\ mm$

$M_{ux'} = M_{uy'} = (177.68) \times \left(\frac{3.5}{2} - \frac{\sqrt{2} \times 0.6}{2}\right)^2 \times \frac{1}{2} \times 3.5 = 465\ KN.m$ calc

Assume $\phi 20$ rebars ✓wrong cover 50 mm

$d = 700 - 50 - 20 - \frac{20}{2} = 620\ mm$

Assume $a = 14$

$A_{s,req} = \frac{M_u}{\phi f_y \left(d - \frac{a}{2}\right)} = \frac{465 \times 10^6}{0.9 \times 400 \times \left(620 - \frac{a}{2}\right)} = 2107\ mm^2$ ✓

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2107 \times 400}{0.85 \times 21 \times 3500} = 13.5 \approx 14\ mm$ ok ✓

Temperature & shrinkage Reinforcement:

$A_{s,temp} = 0.0018 A_g = (0.0018) \times (3500) \times 700 = 4410 \rightarrow 6\phi 32 \rightarrow 4825\ mm^2$

Minimum flexural reinf: $A_{s,min} = Max\left(\frac{0.25\sqrt{21}}{400}, 3800 \times 620, \frac{1.4 \times 3800 \times 620}{400}\right)$

$= 8246\ mm^2 \longrightarrow$ controls

$A_{s,min} = 1.33\, A_{s,req} = 1.33 \times 2107 = 2803 \; mm^2$

$A_{s,req} = 2803 \; mm^2 \longrightarrow 14\phi16 \quad \underline{2815 \; mm^2}$

$$S = \frac{3500 - 2 \times 75}{14 - 1} = 257.7 \; mm$$

$S_{max} = Min(3h, 450) = 450 \; mm$

$S_{min} = Max(16, 25, 1.33 \times 25) = 33 \; mm$

33 257.7 450

$S_{min} < S < S_{max}$ ok ✓

Bearing capacity of footing at base of column

$$\phi N_n = \phi(0.85) f'_c A_1 \underbrace{\sqrt{\frac{A_2}{A_1}}}_{\geq 2 \rightarrow \text{use } 2} = 0.65 \times 0.85 \times 21 \times 2 \times (0.36 \times 10^6) = 8.353 \times 10^6 \; KN$$

$\phi N_n = 8.533 \times 10^6 > \underbrace{1922 \times 10^3}_{P_u} \rightarrow$ ok ✓

Dowels

$A_{sd} = 0.005 A_g = 0.005 \times 600^2 = 1800 \; mm^2 \longrightarrow 6\phi20 \; (1885 \; mm^2)$

Development length of flexural reinf

$$l_d = \left[\frac{f_y}{1.1\sqrt{f'_c}} \; \frac{\psi_t \psi_e \psi_s \lambda}{\left(\frac{c_b + K_{tr}}{d_b}\right)} \right] d_b \geq 300$$

$\psi_t = \psi_e = \psi_s = \lambda = 1$

$K_{tr} = 0$

$C_b = Min\left(\frac{600}{2}, 75 + \frac{20}{2}\right) = 85 \; mm$

$$L_d = \left[\frac{400}{1.1\sqrt{21}} \times \frac{1 \times 1 \times 1 \times 1}{\left(\frac{85}{20}\right)} \right] \times 20 = 373.4 \; mm \geq 300$$

$$L_{d\,pro} = \frac{3500}{2} - \frac{600\sqrt{2}}{2} - 75 = 1250 \; mm > 373 \; mm \rightarrow \text{ok ✓}$$

Development of dowels in foundation

$$L_{dc} = Max\left(\frac{0.24 f_y d_b}{\sqrt{f'_c}}, 0.043 f_y d_b\right) \geq 200 \; mm$$

$$L_{dc} = Max\left(\frac{0.24 \times 400 \times 20}{\sqrt{21}}, 0.043 \times 400 \times 20\right) = 419 \; mm > 200$$

$L_{dc} = 419 < 620 \; mm$ OK

Development of Dowels in column

This is as same as development of Dowels in foundation due to f'_c equal

HW#4 DRCS II

حسين الصادق ٧٤٠٥٠١١٩

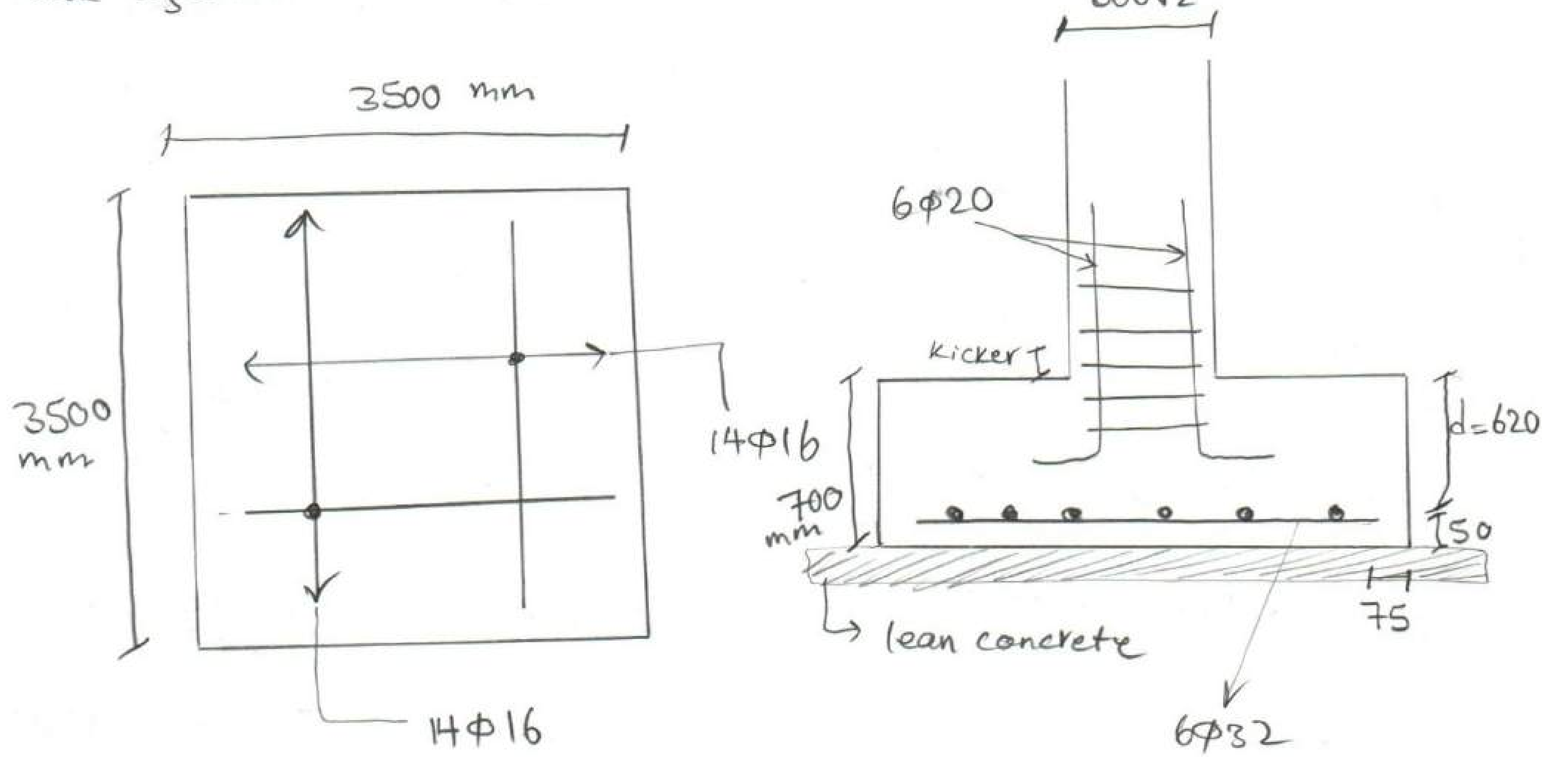
Q2: continue:

lap length of column bars:

$$L_s = 0.071 f_y d_b \geq 300$$

$$L_s = 0.071 \times 400 \times 20 = 568 \text{ mm} \geq 300 \text{ mm} \quad \text{ok}$$

use $L_s = 600$ mm



HW#4 DRCS II "حور" حسن الصرايرة ٩١١٠٥٠٢٧

Q3: $D = 220$ tons
$L = 257$ tons

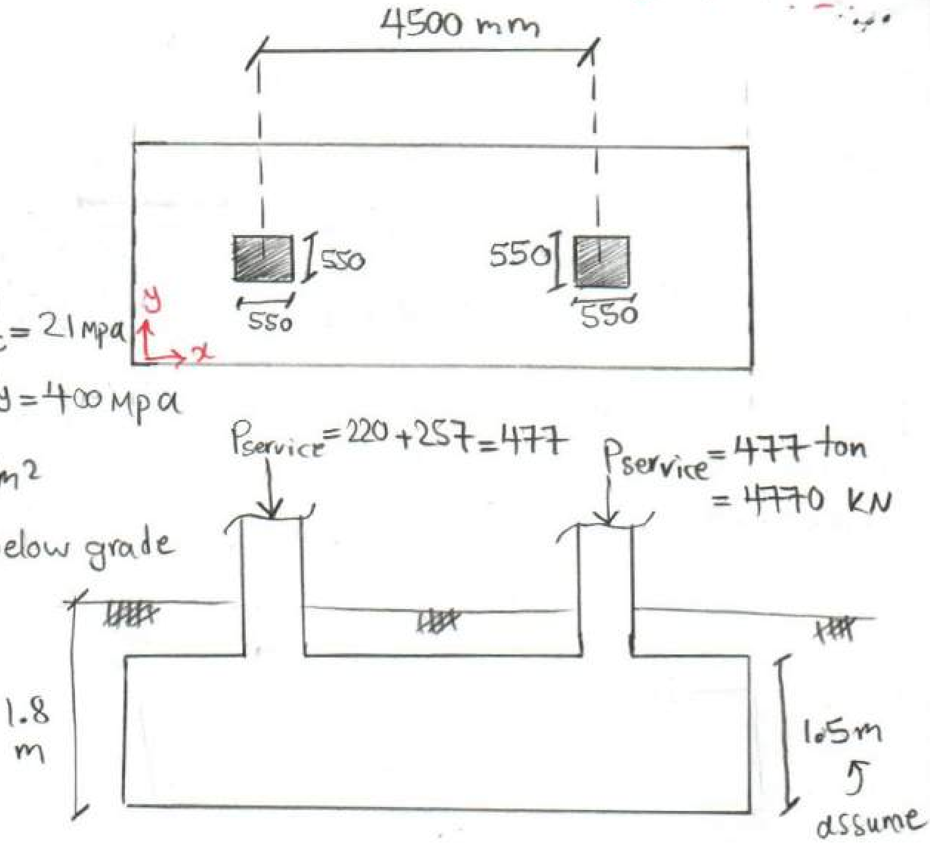
columns reinf with $12\phi 32$
(70 mm from column face)

column $\begin{cases} f'_c = 30 \text{ Mpa} \\ f_y = 400 \text{ Mpa} \end{cases}$ footing $\begin{cases} f'_c = 21 \text{ Mpa} \\ f_y = 400 \text{ Mpa} \end{cases}$

$q_a = 3 \text{ kg/cm}^2 = 300 \text{ kN/m}^2$

bottom of footing 1.8 m below grade

$\gamma_s = 18 \text{ kN/m}^3$



$P_{total} = 477 + 477 = 954 \text{ ton} = 9540$

we assume that the pressure under footing is uniform

$q_{eff} = q_a - \gamma_{ave} \times 1.8 = 300 - 18 \times 1.8 = 267.6 \text{ kN/m}^2$

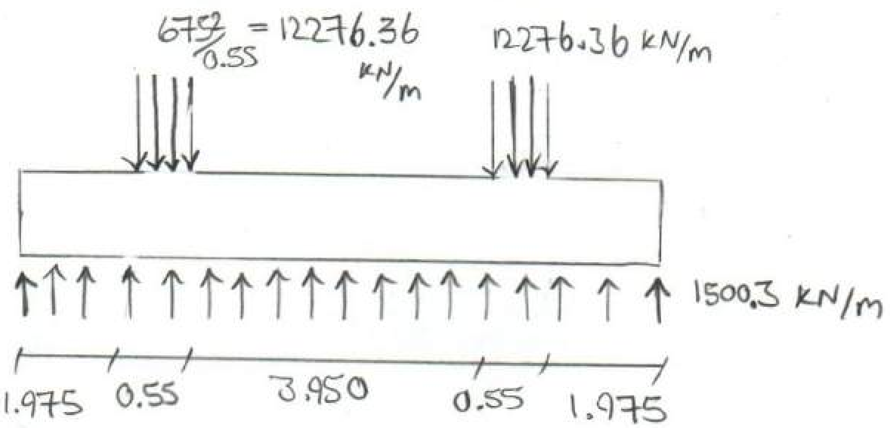
$A_{req} = \dfrac{P_{total}}{q_{eff}} = \dfrac{9540}{267.6} = 35.65 \text{ m} \longrightarrow 4.5 \times 9 = 40.5 \text{ m}^2$

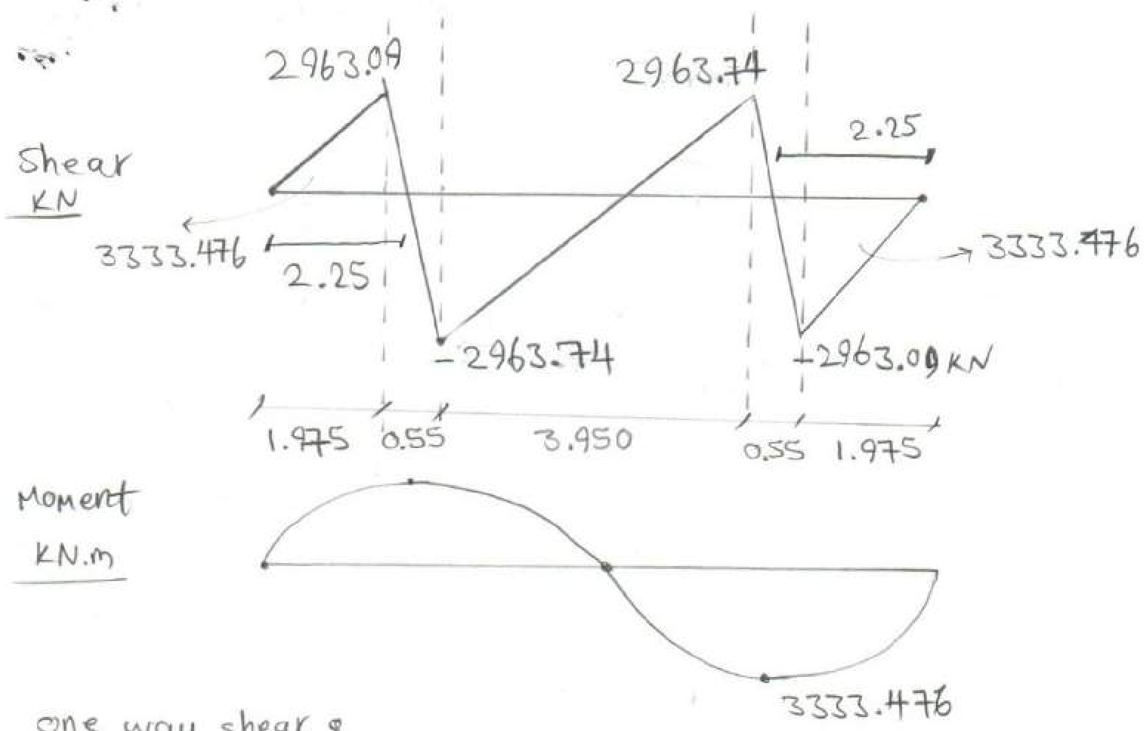
$\bar{x} = 4.5 \text{ m}, \ \bar{y} = 2.25 \text{ m}$ (due to symmetry)

$P_u = 1.2D + 1.6L = 1.2 \times 220 + 1.6 \times 257 = 675.2 \text{ ton} = 6752 \text{ kN}$
↳ (for both columns)

$q_u = \dfrac{2\,P_u}{A_f} = \dfrac{2 \times 6752}{4.5 \times 9} = 333.4 \text{ kN/m}^2 \text{ (kpa)}$

q_u per unit length $= (q_u)(\text{width}) = (333.4) \times 4.5 = 1500.3 \text{ kN/m}$





one way shear :

$h = 1500\,mm \quad d \approx 1400\,mm$

V_u at distance d from the interior column $= 862.86\ KN$

$V_c = 0.17\sqrt{f'_c}\, b_w d \qquad d_{req} = \dfrac{862.86\times10^3}{0.75\times0.17\times\sqrt{21}\times4500} = 328\ mm < 1400\ mm$ ok ✓

punching shear : $V_u = 6752 - 1500.3\times(0.55+1.4)^2 = 1047.1$

$V_{c_1} = 0.17\left(1+\dfrac{2}{\beta}\right)\sqrt{f'_c}\, b_o d \qquad \beta = 1$ (not applicable)

$V_{c_2} = 0.083\left(\dfrac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d \qquad b_o = 4(0.55+1.4) = 7.8\ m \qquad d = 1400 \quad \dfrac{d}{b_o} = \dfrac{1400}{7800} = 0.18 > 0.1$

does not govern

$V_{c_3} = 0.33\sqrt{f'_c}\, b_o d \rightarrow d_{req} = \dfrac{1047\times10^3}{0.75\times0.33\times\sqrt{21}\times7800} = 118\ mm < 1400\ mm$ ok ✓

Moment & reinf in longitudinal direction :

$M_u = 3333.47\ KN.m \qquad$ Assume $a = 62\ mm$

$A_{s\,req} = \dfrac{M_u}{\phi f_y (d-\frac{a}{2})} = \dfrac{3333.47\times10^6}{0.9\times400\times(1400-\frac{a}{2})} = 6764\ mm$

$a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{\times400}{0.85\times21\times2500} = 60.6\ mm \approx 62\ mm$ ok ✓

$A_s\ temp = 0.0018\ A_g = (0.0018)(2500\times1400) = 6300\ mm^2$

Q3: continue

$$A_{s\,min} = \max\left(\frac{0.25\sqrt{f'_c}}{f_y} b_w d,\ \frac{1.4 b_w d}{f_y}\right)$$

$$= \max\left(\frac{0.25\sqrt{21}\times 2500\times 1400}{400},\ \frac{1.4\times 2500\times 1400}{400}\right) = 12250\ mm^2 \rightarrow \text{governs}$$

$$A_s = 1.33\,A_{s\,req} = 1.33\times 6764 = 8996.\ mm^2 \longrightarrow 16\phi 28 \rightarrow 9852$$

$$14\phi 30 \rightarrow 9896\ \checkmark$$

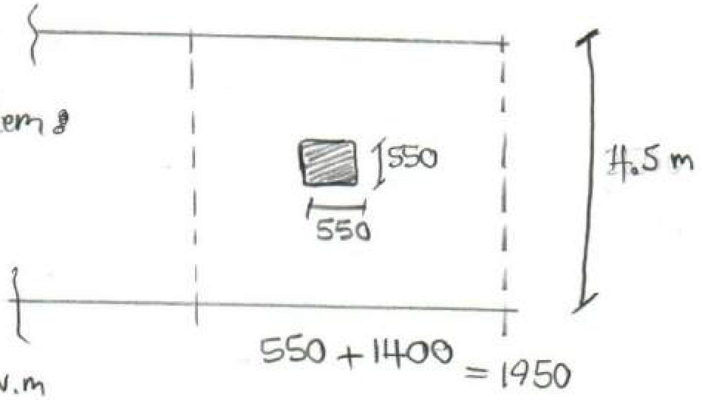
$$S = \frac{2500-2\times 75}{14-1} = 180\ mm$$

$$S_{min} = \max(\overset{30}{d_b}, 25, \overset{34}{1.33 d_c}) = 34\ mm \qquad S_{min} < S < S_{max}$$

$$S_{max} = \min(3h, 450) = 450\ mm$$

Moment & reinf in transverse Direction

both columns are like each other so we calculate for one of them:



$$q_u = \frac{4770}{4.5} = 1060\ kN/m$$

$$M_u = \left(\frac{1060}{2}\right)\left(\frac{4.5}{2} - \frac{0.55}{2}\right)^2 = 2067.3\ kN.m$$

$$A_{s\,min} = \frac{1.4(1950)(1400)}{400} = 9555\ mm^2 \longrightarrow \text{governs}$$

$$A_{s\,temp} = 0.0018\times 1950\times 1400 = 4914 \longrightarrow 8\phi 28 = 4926\ mm^2$$

$$A_{s\,req} = \frac{M_u}{\phi f_y (d-\frac{a}{2})} = \frac{2067.3\times 10^6}{0.9\times 400\times(1400-\frac{a}{2})} \xrightarrow{\text{Assume } a=50} A_s = 4176\ \checkmark$$

$$a = \frac{A_s f_y}{0.85 f'_c\, b} = \frac{4176\times 400}{0.85\times 21\times 1950} = 48\ mm \approx 50\ \text{OK}$$

$$A_s = 1.33\,A_{s\,req} = 1.33\times 4176 = 5554\ mm^2 \longrightarrow 8\phi 30 = 5655\ mm^2\ \text{OK}$$

$$S = \frac{1950}{7} = 279\ mm$$

$$S_{max} = 450 \quad S_{min} = 34 \qquad S_{min} < S < S_{max}\quad \text{OK}\ \checkmark$$

Dowel bars

$A_{sd} = 0.005\,A_g = 0.005 \times (550^2) = 1513\ mm^2 \longrightarrow 5\phi20 \quad 1571\ mm^2$

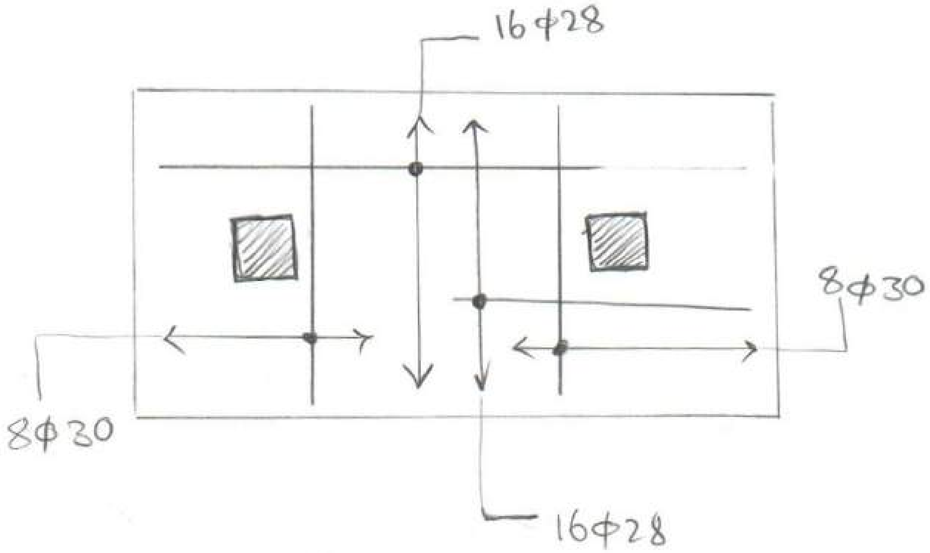
Development of length of flexural reinf

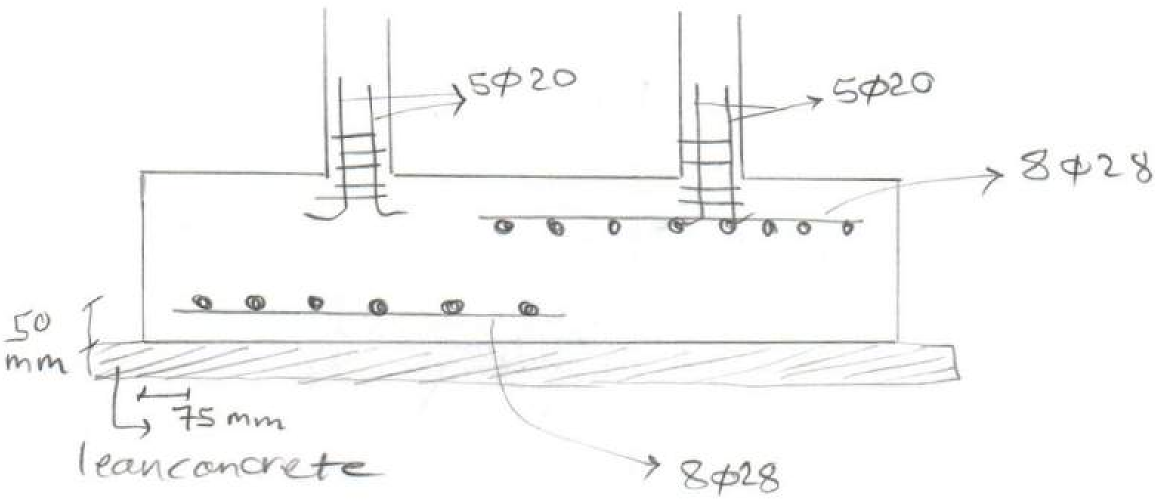
$$L_d = \left[\frac{f_y}{1.1\sqrt{f'_c}}\ \frac{\Psi_t \Psi_e \Psi_s \lambda}{\left(\frac{c_b + k_{tr}}{d}\right)}\right] d_b = \left[\frac{400}{1.1\sqrt{21}} \cdot \frac{1\times1\times1\times1}{\left(\frac{85+0}{30}\right)}\right] \times 30 = 840\ mm$$

$L_{pro} = \frac{4500}{2} - \frac{550}{2} - 75 = 1900\ mm > 840\ mm$ ok ✓

Development of Dowels in foundation :

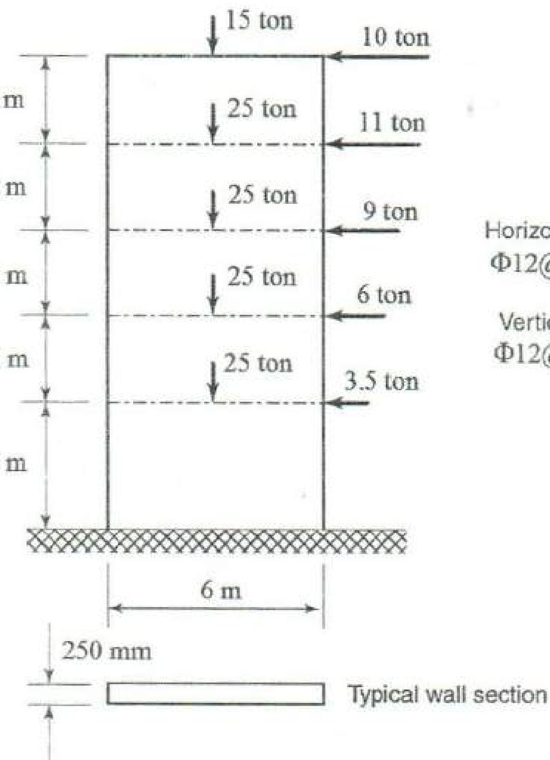
$L_{dc} = Max\left(\frac{0.24 f_y d_b}{\sqrt{f'_c}}, 0.043 f_y d_b\right) = 419\ mm < 1500 - 75 = 1425\ mm$ ok ✓





Sadeghi

1- Check the moment and shear strength at the base of the structural wall shown below. Also, show that the given horizontal and vertical reinforcement satisfies all of the ACI Code requirements regarding minimum reinforcement percentage and maximum spacing. The given lateral loads are equivalent wind forces, considering both direct lateral forces and the effects of any torsion. Use a load factor of 1.6 for the wind load effects. The given vertical loads represent dead loads, and you can assume that the vertical live loads are equal to 60 percent of the dead loads. Assume the wall is constructed with normal-weight concrete that has a compressive strength of 30 MPa. Assume all of the steel is Grade AIII.



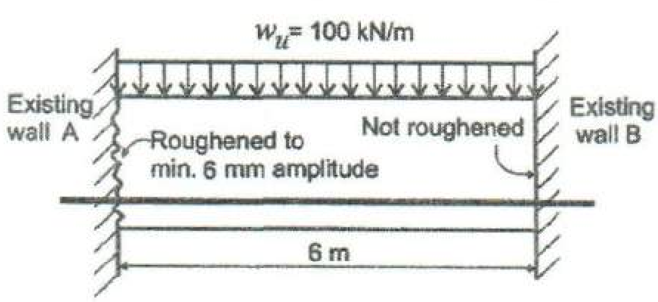
Horizontal reinforcement:
Φ12@400 at each face

Vertical reinforcement:
Φ12@300 at each face

2- A 300 mm wide by 600 mm deep beam shown in the figure below is proposed to be cast between two existing walls A and B. One proposal is to roughen wall A only to minimum 6 mm amplitude at the interface of the new beam. Wall B was originally cast with good quality smooth form ply and will be cleaned prior to casting the new beam. Take $f_c' = 30$ MPa and $f_y = 400$ MPa.

a) Determine the area of shear friction reinforcement required at the beam supports.

b) Do you believe that this solution is good for the right support?



برگرفته از خلاصه بتن II

$$\begin{cases} C_c = 0.85 f'_c (A_g - A_{st}) \\ C_s = A_{st} f_y \end{cases} \qquad P_{no} = C_c + C_s$$

effect of accidental eccentricity $\begin{cases} P_{n_{max}} = 0.8 P_{no} \rightarrow \text{Tied} \\ P_{n_{max}} = 0.85 P_{no} \rightarrow \text{spirall} \end{cases}$

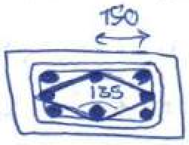
$$\rho_g = \frac{A_{st}}{A_g} \xrightarrow{\text{برای ستون‌ها}} 1\% \leqslant \rho_g \leqslant 8\% \xrightarrow[\text{عملی}]{\text{به صورت}} 1\% \leqslant \rho_g \leqslant 3\%$$

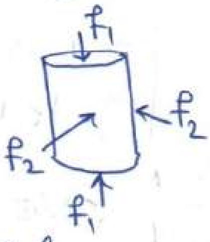
min number of longitudinal bars $\rightarrow \begin{cases} n=3 \rightarrow \text{triangular tied} \\ n=4 \rightarrow \text{rectangular tied} \\ n=6 \rightarrow \text{spiral} \end{cases}$

$$S_{min} = \text{Max}(40\text{ mm}, 1.5 d_b, 1.33 D_{max})$$ → در ناحیه‌ی وصله نیز این فاصله رعایت گردد

حداقل فاصله‌ی میلگردهای طولی

tie size $\begin{cases} d_t \geqslant 10\text{ mm} \rightarrow \text{if } d_b \leqslant 32\text{ mm} \\ d_t \geqslant 12\text{ mm} \rightarrow \text{if } d_b \geqslant 32\text{ mm} \\ d_t \geqslant 12\text{ mm} \rightarrow \text{if bundled bars} \end{cases}$





$$f_c^* = f'_c + 4.1 f_2$$

مقاومت فشاری در بارگذاری سه محوری

حداکثر فاصله‌ی خاموت

$$S_{max} = \min(16 d_b, 48 d_t, h_{min})$$ spirall / سمت مقطع

نیروی جانبی در مقطع $f_2 = \frac{2 f_{sp}}{S D_c}$ $\qquad f_{sp} = a_s f_{yt} \qquad f_2 = \frac{2 a_s f_{yt}}{S D_c} = \frac{\rho_s f_{yt}}{2}$

f_{yt} → اسپیرال f_y

S → گام اسپیرال

D_c → قطر هسته‌ی محصور به اسپیرال

برای میلگردهای اسپیرال $\rho_{s\,min}$

$$P_{no} = 0.85 f'_c (A_g - A_{st}) + f_y A_{st} \qquad P_{n2} = 0.85 f'^{*}_c (A_{ch} - A_{st}) + f_y A_{st}$$

بعد از ترک خوردن پوسته

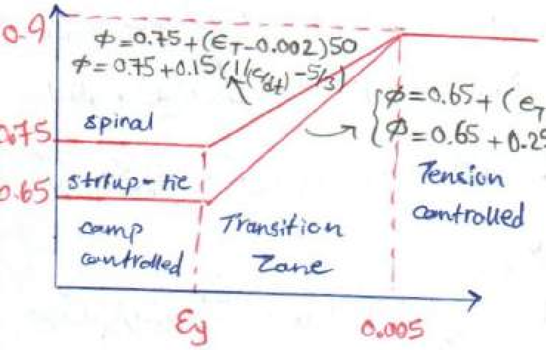
$$P_{no} = P_{n2} \Rightarrow f_c^* = f'_c \times \frac{A_g - A_{st}}{A_{ch} - A_{st}} \approx f'_c \frac{A_g}{A_{ch}}$$

نسبت حجمی میلگرد اسپیرال $\rho_s = \frac{a_s (D_c - d_s)}{\pi D_c^2 S / 4} \approx \frac{4 a_s}{D_c \times S} \approx 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\left(\frac{f'_c}{f_{yt}}\right)$

for spiral columns $\longrightarrow d_s \geqslant 10\text{ mm}$

$$\rho_{s\,min} = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\left(\frac{f'_c}{f_{yt}}\right)$$

$\text{Max}(25\text{ mm}, 1.33 D_{max}) \leqslant S_c \leqslant 75\text{ mm}$ → گام میلگردهای اسپیرال



$$\phi P_n = \phi r [0.85 f'_c (A_g - A_{st}) + A_{st} f_y] \geqslant P_u \qquad r \begin{cases} \text{tied} = 0.8 \\ \text{spiral} = 0.85 \end{cases}$$

$$\text{if } \rho_g = \frac{A_{st}}{A_g} \rightarrow \phi P_n = \phi r A_g [0.85 f'_c + \rho (f_y - 0.85 f'_c)] \geqslant P_u$$

Design of known dimension columns :

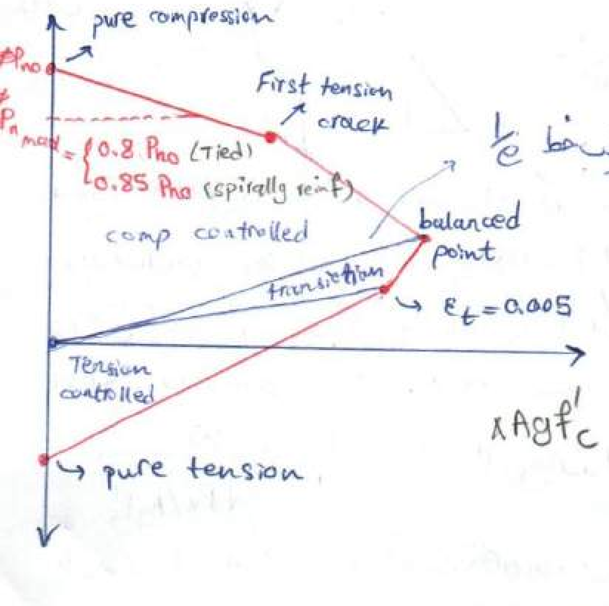
$$A_{st} \geqslant \frac{1}{(f_y - 0.85 f'_c)} \left[\frac{P_u}{\phi r} - 0.85 f'_c A_g\right]$$

Design of unknown dimension :

ستون کوتاه

$$A_g \geqslant \frac{P_u}{\phi r [0.85 f'_c + \rho_g (f_y - 0.85 f'_c)]}$$

pure tension : $T_n = A_{st} f_y$



Bersler's Reciprocal load method : آخرش حتماً این را باید چک کرد

$$\frac{1}{P_n} = \frac{1}{P_{nxo}} + \frac{1}{P_{nyo}} - \frac{1}{P_{no}} \qquad (\text{only when } P_n > 0.1 P_{no})$$

Bersler's-parme method

$$\left(\frac{M_{nx}}{M_{nxo}}\right)^{\frac{\ln 0.5}{\ln \beta}} + \left(\frac{M_{ny}}{M_{nyo}}\right)^{\frac{\ln 0.5}{\ln \beta}} = 1 \qquad 0.55 < \beta < 0.7 \qquad \beta = 0.65$$

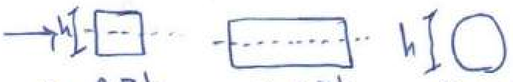
$$\frac{\ln 0.5}{\ln 0.65} = 1.61$$

$$\frac{1}{K_n} = \frac{1}{K_{nxo}} + \frac{1}{K_{nyo}} - \frac{1}{K_{no}}$$

$$P_{cr} = \frac{\pi^2 EI}{L^2} \qquad \Psi = \frac{\sum \left(\frac{EI}{l}\right)_{columns}}{\sum \left(\frac{EI}{l}\right)_{beams}} \longrightarrow \text{simple approach} \begin{cases} I_{beam} = 0.35 I_g \\ I_{column} = 0.70 I_g \end{cases}$$

sway → unbraced → $1 < K < \infty$ nonsway → braced → $0.5 < K < 1$ → we can take $K = 1$

$r = \sqrt{\frac{I}{A}}$ approximate r →



$r = 0.3h$ $r = 0.3h$ $r = 0.25h$

اگر مجموع سختی ستون‌های یک طبقه $\frac{1}{12}$ برابر مجموع سختی ستون‌های مقاوم در برابر بارهای جانبی باشد (دیوار برشی) آنگاه قاب nonsway است

slender columns → $\begin{cases} \text{short column (unbraced)} \rightarrow \frac{KL}{r} \leq 22 \\ \text{short column (braced)} \rightarrow \frac{KL}{r} \leq 34 - 12\left(\frac{M_1}{M_2}\right) \leq 40 \\ \left|\frac{M_1}{M_2}\right| \leq 1 \end{cases}$

\+ تقعر ساده
\- مضاعف

جمع کل بارهای ستون‌های طبقه

stability index $Q = \frac{\sum P_u \Delta_o}{V_{us} L_c} \leq 0.05$

تغییرمکان نسبی بالا و پایین ستون

برش طبقه

center to center طول ستون

$M_c = \delta_{ns} \times \max[M_2, M_{2min}]$ nonsway

check slenderness

$M_{2,min} = P_u(15 + 0.03h)$

$\delta_{ns} = \frac{C_M}{1 - \frac{P_u}{0.75 P_c}} \geq 1.0$ where $\delta_{ns} \leq 1.4$

$$C_M = \begin{cases} 0.6 + 0.4 \frac{M_1}{M_2} \geq 0.4 & \text{No transverse loading} \\ 1.0 & \text{transverse loading} \end{cases}$$

$P_c = \frac{\pi^2 EI}{(K l_u)^2}$

برای $\rho \leq 2\%$ این رابطه خوب نیست. → $EI = \frac{0.4 E_c I_g}{1 + \beta_{dns}}$ or $EI = \frac{0.2 E_c I_g + E_s I_{se}}{1 + \beta_{dns}}$

چون فولاد داخل بتن ممکن است تحت اثر creep دچار yield گردد (در حالت سرویس)

مثلاً بار مرده

$\beta_{dns} = \frac{\text{Max Factored sustained axial load}}{\text{Max Factored axial load}} \leq 1$

اثر ترک خوردگی

δ_{ns} نباید بزرگتر از 1.4 شود و اگر شد باید با افزایش ابعاد مقطع آن را کاهش داد. (یعنی ابعاد مقطع کوچک بوده است) ← stability failure

$\delta_{ns} = \frac{C_M}{1 - \frac{P_u}{0.75 P_c}} \geq 1.0$

ناشی از ستاره است که باعث کاهش سختی می‌گردد.

اگر $\delta_{ns} < 0$ مقطع دچار buckle failure می‌گردد.

---

sway
check slenderness

$M_1 = M_{1ns} + \delta_s M_{1s}$ $M_2 = M_{2ns} + \delta_s M_{2s}$

$\delta_s = \frac{1}{1 - Q} \geq 1$ where $\delta_s \leq 1.5$ $\xrightarrow{or}$ $\delta_s = \frac{1}{1 - \frac{\sum P_u}{0.75 \sum P_c}} \geq 1$

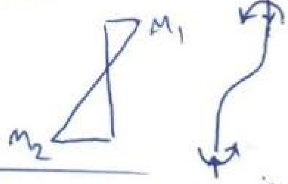
$\sum P_u =$ جمع تمام بار ستونهای طبقه

$\sum P_c =$ جمع تمام بارهای بحرانی ستون جانبی

$P_c = \frac{\pi^2 EI}{(K l_u)^2}$ $EI = \frac{0.2 E_c I_g + E_s I_{se}}{1 + \beta_{ds}}$ or $\frac{0.4 E_c I_g}{1 + \beta_{ds}}$

$\beta_{ds} = \frac{\text{max Factored sustained shear load}}{\text{Maxi Factored shear load}} \leq 1.0$

ناشی از اینکه باعث کاهش سختی می‌گردد.



نسبت لنگر اصلاح شده در اصلاح مرتبه‌ی 2 به لنگر مرتبه‌ی اول نباید از 1.4 بیشتر گردد.

بعد از تحلیل مرتبه‌ی 2 باید مقدار لنگر $(\delta_s - 1) M_s$ را در بالا و پایین ستون بین تیرها به نسبت سختی تقسیم کنیم.

در ستون‌هایی که در قاب‌های sway قرار دارند ممکن است لنگر ماکس در وسط ستون رخ دهد پس باید لنگر آن با ضریب اصلاح nonsway اصلاح گردد. ← check slenderness $\xleftarrow{\text{if slender}}$ $M_c = \delta_{ns} M_2 = \delta_{ns}(M_{2ns} + \delta_s M_{2s})$

برای $\frac{l_u}{r} < \frac{35}{\sqrt{P_u / A_g f'_c}}$ ممکن است لازم نباشد اصلاح قبلی انجام گردد

برای سازه‌های خاص که ستون صرفاً نقش معماری دارد می‌توان از ستون‌هایی با $\frac{Kl}{r} > 100$ نیز استفاده کرد.

حسیب‌الله صادقی برگه ۲ خلاصه بتن II

روش Bersler ← ابتدا $e_x = \frac{M_{uy}}{P_u}$, $e_y = \frac{M_{ux}}{P_u}$ ، $\gamma < \begin{matrix}\gamma_1 \\ \gamma_2\end{matrix}$ انجام با هر دو ، فرض $\rho = 0.03$

$\frac{e_x}{h}$, $\frac{e_y}{h}$ → مقدار K_{nx0} و K_{ny0} ، K_{n0} ← استفاده ٪3 = ρ خود K_n

$$\left(\frac{1}{P_n} = \frac{1}{P_{nx0}} + \frac{1}{P_{ny0}} + \frac{1}{P_{n0}}\right) \times (A_g f'_c) \rightarrow \frac{1}{K_n} = \frac{1}{K_{nx0}} + \frac{1}{K_{ny0}} + \frac{1}{K_{n0}}$$

$$\begin{cases} P_{\gamma_1 = x_1} = K_n A_g f'_c = \checkmark \\ P_{\gamma_2 = x_2} = K_n A_g f'_c = \checkmark \end{cases} \xrightarrow{\text{درون‌یابی}} P_{n\gamma} = \checkmark > P_{n\,req} \leftarrow \text{باید باشد.}$$

Bersler's parme $K_n = \frac{P_u}{\phi f'_c A_g} = \frac{P_n}{f'_c A_g}$ $R_n = \frac{M_u}{\phi f'_c A_g h} = \frac{M_n}{f'_c A_g h}$ → Assume $\phi = 0.65$

$M_{nx0} = ?$ $P_n = \frac{P_u}{\phi}$ ← طرح

↳ Assume $\rho = 0.03$ → calculate K_n → R_n → $M_{nx0} = R_n \times f'_c A_g h$

calculate R_n for y direction → The same way → $M_{ny0} = R_n \times f'_c A_g h$

$$\left(\frac{M_{nx}}{M_{nx0}}\right)^{1.61} + \left(\frac{M_{ny}}{M_{ny0}}\right)^{1.61} < 1$$

$\boxed{\text{This one was for } \gamma_1}$

$\boxed{\text{for } \gamma_2 \text{ the same way}}$ → $\gamma = \checkmark$ $\boxed{\text{check if } P_n > 0.1 P_{n0}}$

For design of column if $\frac{P_u}{A_g f'_c} < 0.1$ → design as a beam

all load comb & Engineering judgment

$A_{s\,req} = \frac{M_u}{\phi f_y (d - \frac{a}{2})}$ $\phi = 0.9$ ...

else design as a column → K → slenderness → sway / nonsway → magnified moment ...

S_{min}, S_{max} tie & design of tie, sketch,

↳ check clear spacing between the bars

For column we shall always check shear reinf beause the spacing of ties may be less than max spacing of colum's ϕV_c

فاصله نقطه تا خط : $d = \frac{|ax_0 + by_0 - c|}{\sqrt{a^2 + b^2}}$

shear $V_{s\,req} = \frac{V_u}{\phi_{\to 0.75}} - V_c$ (V_c → $0.17\sqrt{f'_c}\,b_w d$) $\left(\frac{A_v}{S}\right)_{req} = \frac{V_{s\,req}}{f_y d}$ (A_v → $2\phi10$)

عرضی

$S_{min\ shear} = \min\left(\frac{A_v f_y d}{V_s}, S_{max}\right)$

$\frac{P_2}{P_1} = \left(\frac{L_1}{L_2}\right)^4$ deflection → 9.5(a) ACI

min h →

cover → $\phi < 40$ 20 mm, $\phi > 40$ 40 mm

$V_c = 0.17\sqrt{f'_c}\,b_w d$

As per meter $= A_b \left(\frac{1000}{\text{bar spacing}}\right)$

$$\rho_{s\,temp} = \begin{cases} 0.002 & f_y \leq 350 \text{ Mpa} \\ 0.0018 & f_y = 420 \text{ Mpa} \\ \frac{0.0018(420)}{f_y} \geq 0.0014 & f_y > 420 \text{ Mpa} \end{cases}$$ ($\phi 8$)

$As_{min} = As$ temp & shrinkage

$S_{max} = \min(3h, 450\text{ mm})$ → خمشی

$S_{max} = \min(5h, 450\text{ mm})$ → temp & shrinkage

→ اندازه میلگرد , deflection 9.5(a), crack control

$S_{max} = 380\left(\frac{280}{f_s}\right) - 2.5C_c \leq 300\left(\frac{280}{f_s}\right)$ ($f_s \to 2/3 f_y$), curtailment of bars

بالای دال یک طرفه بر تکیه‌گاه باید بالایش هم میلگرد گذاشت تا ترک نخورد ٪50 As , sketch

Footing پی

1) soil bearing capacity 2) uniform settlement 3) differential settlement
4) sliding 5) overturning

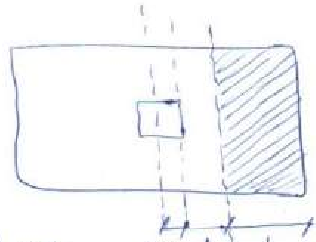
ضریب اطمینان مقاومت:

		لرزه‌ای
لغزش	0.8	0.9
واژگونی	0.5	0.65

تنش زیر پی $\rightarrow q = \dfrac{P}{A_f} \pm \dfrac{MC}{I}$ ± زلزله و باد لحاظ گردد.

$$\text{size of footing base} = A_{req} = \frac{\text{Total } P_{service}}{q_e}$$

minimum footing depth 150 mm on soil
300 mm on pile



$a/2$ d $L/2 - \frac{a}{2} - d$

one way shear $\phi V_n = \phi V_c = \underset{0.75}{\phi}(0.17\sqrt{f'_c}\, b_w d) \geq V_u$

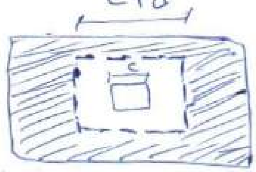
punching shear
$$V_c = \text{Min}\begin{cases} 0.17\left(1+\frac{2}{\beta}\right)\sqrt{f'_c}\, b_o d \leftarrow \beta < 2.1 \text{ not govern} \\ 0.083\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d \leftarrow d/b_o \geq 0.1 \rightarrow \text{not goven} \\ 0.33\sqrt{f'_c}\, b_o d \end{cases}$$

ستون مربعی نمی‌خواهد

محیط مقطع بحرانی

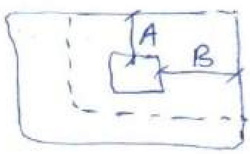
$\beta = \dfrac{\text{طول ستون}}{\text{عرض ستون}}$

α_s = 20 corner, 30 edge, 40 interior



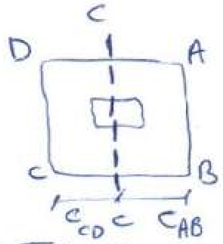
c+d

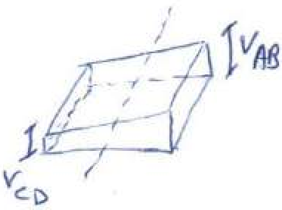
$\rightarrow V_u = P_u - (c+d)^2 q_u$



$A, B \leq 4h, 2L_d$

shear for eccentric loading
$$\begin{cases} \dfrac{V_u}{A_c} + \dfrac{\gamma_v M_u C_{AB}}{I_c} \\ \dfrac{V_u}{A_c} - \dfrac{\gamma_v M_u C_{CD}}{I_c} \end{cases}$$





اگر آرماتور برشی داشته باشیم $\rightarrow V_n = V_c + V_s \leq 0.5\sqrt{f'_c}\, b_o d$ $\quad V_c = 0.17\sqrt{f'_c}\, b_o d$

مقطع بحرانی برای بررسی خمش در خمش یک طرفه به فاصله‌ی d از بر ستون و در خمش دوطرفه $\frac{d}{2}$ از بر ستون هست. در دیوارها و ستون‌های آجری $\frac{1}{4}$ از مرکز دیوار محل بحرانی است.

$A_s = \dfrac{M_u}{\phi f_y (d - \frac{a}{2})}$ $\quad a = \dfrac{A_s f_y}{0.85 f'_c b}$

$A_{s\,min} = \dfrac{0.25\sqrt{f'_c}}{f_y} b_w d \geq \dfrac{1.4 b_w d}{f_y}$

concrete cover
75 mm for faces in contact to ground 50 mm for other faces

$A_{s\,req} < A_{s\,min} \rightarrow$ provide $1.33 A_{s\,req}$ ✓

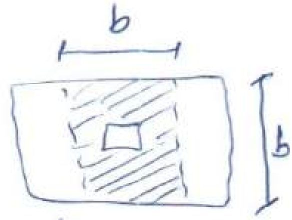
$$\begin{cases} S_{max} = \text{Min}(3h, 450 \text{ mm}) \\ S_{min} = \text{Max}(d_b, 25 \text{ mm}, 1.33 d_c) \end{cases}$$

recomended $S \geq 100$ mm

توزیع میلگردها در خمش $\rightarrow \gamma = \dfrac{2}{\beta+1}$ $\quad \beta = \dfrac{\text{طول پی}}{\text{عرض پی}}$

$\gamma_s A_s \rightarrow$ در فاصله‌ی b از مرکز ستون

$(1-\gamma) A_s \rightarrow$ در خارج b توزیع



b

بیرون عرض

bearing capacity at column base
$$P_u \leq \phi N_n = \phi (0.85) f'_c A_1 \sqrt{\frac{A_2}{A_1}} \leq 2$$
$\phi = 0.65$, A_2 مال پی

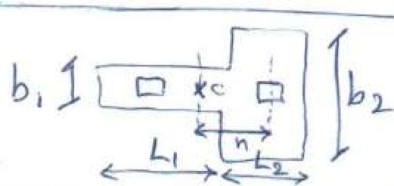
در حالتی که مقاومت بتن ستون و پی فرق دارد $\rightarrow P_u \leq \phi N_n = \phi \times 0.85 \times f'_c A_1$ (مال ستون)

Dowel bars $A_s = 0.005 A_{column} \longrightarrow$ طول وارد شدن به ستون $= L_d \longrightarrow$ اطراف dowel داخل پی تا 300 mm بالا آرماتورگذاری گردد.

این طول باید در طراحی عمق پی نیز لحاظ گردد ✗

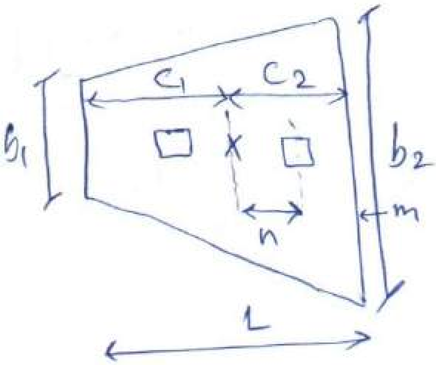
0.89h 0.93h H

نیروی کل وارده



b_1 b_2 L_1 L_2 n c

$$b_1 = \frac{R}{q_e} = \left[\frac{2(n+m) - L_2}{L_1(L_1+L_2)}\right]$$
$$b_2 = \frac{R}{L_2 q_e} - \frac{L_1 b_1}{L_2} \qquad L_1 b_1 + L_2 b_2 = \frac{R}{q_e}$$



$$\frac{b_2}{b_1} = \frac{3(n+m)-L}{2L-3(n+m)}$$

$$(b_1+b_2) = \frac{2R}{q_{el}}$$

$$C_1 = \frac{L(b_1+2b_2)}{3(b_1+b_2)}$$

$$C_2 = \frac{L(2b_1+b_2)}{3(b_1+b_2)}$$

خلاصه فرمول بتن II (برگه‌ی ۳)

$$q_e = P_a - W_D - W_L + (H-H')\gamma_s - 21 \times H$$

ترکیب چگالی بتن و فولاد

$$A_{req} = \frac{P_{service}}{q_e}$$

$B = \sqrt{A_{req}} \rightarrow$ گرد می‌کنیم

$P_u =$ ترکیب بار

$$q_u = \frac{P_u}{A_f}$$

Depth of footing $\rightarrow$ assume d $\rightarrow$ (two way punching shear $d/2$) cal $V_u \rightarrow \frac{V_u}{\phi} = V_n \rightarrow V_1, V_2, V_3 \rightarrow$ واقعی d

one way shear (از d) $\rightarrow$ cal $V_u \rightarrow V_n = \frac{V_u}{\phi}$ ($\phi = 0.75$) $\rightarrow V_c = 0.17\sqrt{f'_c}\,b_w d \rightarrow$ واقعی d مورد نیاز

مقدار h را بدست آورید.

$M_{ux}, M_{uy} \rightarrow$ در دو جهت $\rightarrow$ rebar $\phi 20 \rightarrow d \rightarrow A_{s\,req} \rightarrow a$ } temp & shrinkage $= 0.0018 A_g$

min flex reinf $\rightarrow A_{s\,min} \rightarrow \underline{1.33 A_{s\,req}}$ (اگر لازم بود) $\rightarrow$ (ϕ) تعداد میلگرد و نوع آن $\rightarrow S \rightarrow S_{min}, S_{max} \rightarrow$ bearing footing

$\rightarrow$ bearing column $\rightarrow$ Dowels $= 0.005 A_g \rightarrow$ Development of length of flexural reinf

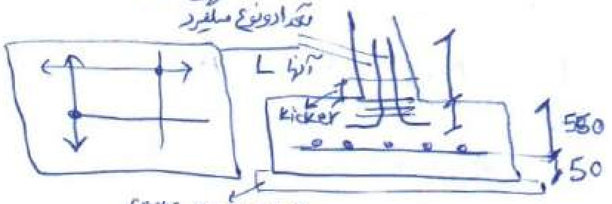
$$L_d = \left[\frac{f_y}{1.1\sqrt{f'_c}}\frac{\psi_t \psi_e \psi_s \lambda = 1}{\left(\frac{c_b + K_{tr}^{\to 0}}{d_b}\right)}\right] \times d_b \quad \boxed{c_b = \min\left(\frac{S}{2}, c_c + \frac{d_b}{2}\right), \; c_c = 75} \rightarrow \text{development of dowels in Foundation}$$

$$L_{dc} = \max\left(\frac{0.24 f_y d_b}{\sqrt{f'_c}}, 0.043 f_y d_b\right) \geq 200 \rightarrow \text{development of } L_{dc} \text{ in the column} \rightarrow \text{lap length of column bar } L_s = 0.071 f_y d_b \geq 300 \text{ mm}$$

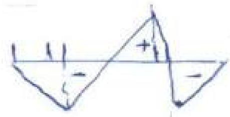
شرایط f'_c

با L_s چک گردد از عمق پی بیشتر نباشد.

sketch

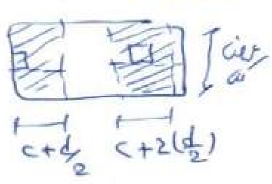


combing footing $\begin{cases} L = 2\bar{x} \\ b = \frac{A}{L} \end{cases} \rightarrow$ بار گسترده که در واحد عرض $\rightarrow$ بار ستون در واحد عرض $\rightarrow$ رسم دیاگرام برش و خمش

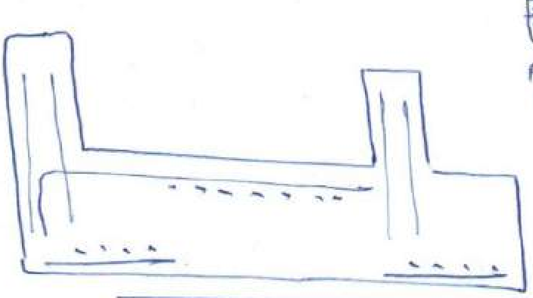


one way shear $\rightarrow$ punching shear $\rightarrow$ لنگر و آرماتور خمشی در دهانه وسط $\rightarrow \underline{A_s, a}$, $A_{s\,temp}, A_{s\,min}, S, S_{min}, S_{max}$

لنگر در جهت عرض $\rightarrow A_s, a, \ldots$



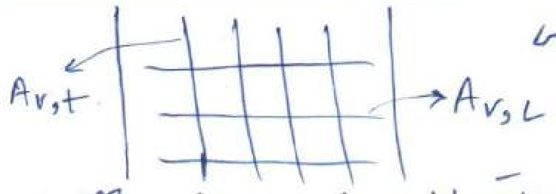
$\rightarrow$ temp reinf (در این جهت هم باید طراحی گردد.)



دیوار برشی

$\frac{h_w}{L_w} \leq 2 \rightarrow$ رفتار برشی $\qquad \frac{h_w}{L_w} \geq 3 \rightarrow$ رفتار خمشی

Thickness $> \frac{1}{20} \times$ طول بدون تکیه‌گاه دیوار



$\phi V_n \geq V_u$ ($\phi = 0.75$) $\qquad V_n = V_c + V_s \leq 0.83\sqrt{f'_c}\,h d$ ($d = 0.8 L_w$)

$$V_c = 0.17\left(1 \pm \frac{0.29 N_u}{A_g}\right)\sqrt{f'_c}\,h d$$

آرماتور حداقل طولی و عرضی دیوار برشی ص 398

$V_c = \text{Min}(V_{c1}, V_{c2})$ $\quad V_{c1} = 0.27\sqrt{f'_c}\,hd + \frac{N_u d}{4L_w}$ $\quad V_{c2} = \left[0.05\sqrt{f'_c} + \dfrac{L_w\left(0.1\sqrt{f'_c} + 0.2\frac{N_u}{L_w h}\right)}{\frac{M_u}{V_u} - \frac{L_w}{2}}\right] hd$

محل مقطع بحرانی $\frac{M_u}{V_u} - \frac{L_w}{2}$

محل مقاسبه ی برش بحرانی را برابر حداقل $\frac{h_w}{2}$ و $\frac{L_w}{2}$ از کف ساختمان می گیرند

وارتفاع طبقه اول

اگر $\frac{M_u}{V_u} - \frac{L_w}{2} < 0$ باشد رابطه ی دوم نیازی نیست.

اگر $V_u > \phi V_c \rightarrow V_s = \frac{V_u}{\phi} - V_c = \frac{A_{vt} f_y d}{s_t}$

مساحت میلگرد عرضی

$\rho_{t,min} = 0.0025 = \frac{A_{v,t}}{s_t h}$ $\quad s_{t,max} = \min(3h, \frac{L_w}{5}, 450\text{ mm})$

فاصله

$\rho_l = 0.0025 + 0.5(2.5 - \frac{h_w}{L_w})(\rho_t - 0.0025) \geq 0.0025$

$S_{l,max} = \text{Min}(3h, \frac{L_w}{3}, 450\text{ mm})$

$\rho_L > \rho_t$ (نیازی نیست) حتما اینجوری نیست

$V_u > \frac{\phi V_c}{2} \rightarrow$ Min shear reinf required $\quad V_u < \frac{\phi V_c}{2} \rightarrow$ Min reinf shear wall or Min wall

aci 14.3

---

type of shear wall $\frac{h_w}{L_w} \geq 3 \rightarrow$ flexural, $< 2 \rightarrow$ shear $\rightarrow$ Factored shear force & moment $\rightarrow$ shear strength concret $0.17\sqrt{f'_c}\,hd$ ($d = 0.8L_w$, 200) $\rightarrow V_c = (V_{c1}, V_{c2}) \rightarrow$

critical section for $V_c \rightarrow M_u$ @ critical section $\rightarrow$ horizontal shear reinf $V_{s\,req} = \frac{V_u}{\phi} - V_c$ $\quad (\frac{A_{v,t}}{s})_{req} = \frac{V_s}{f_y d}$

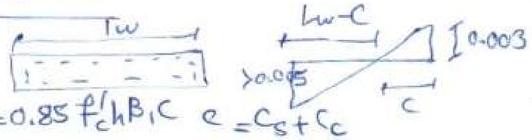
$S_{t\,max}, \rho_{t,min}, A_{t\,min} \rightarrow$ vertical shear reinf $\rho_L, S_{L,max}, A_{v,min} \rightarrow$ design for flexure

calculate M_n with computability eq $\rightarrow$ sketch

$A_{s\,req}$, $a = \frac{A_s f_y}{0.85 f'_c h}$ $\quad \frac{M_u}{0.9 f_y (d - \frac{a}{2})}$

---

uniformly distributed vertical reinf



$T = A_s f_y (1 - \frac{c}{L_w})$ $\quad C_s = A_s f_y (\frac{c}{L_w})$ $\quad C_c = 0.85 f'_c h \beta_1 C$ $\quad C = C_s + C_c$

$C_c + C_s - T = 0 \longrightarrow c = \left[\frac{A_s f_y}{0.85 f'_c h \beta_1 L_w + 2A_s f_y}\right] L_w$ $\quad M_n = T(\frac{L_w}{2})$ $\quad A_{s\,req} = \frac{T}{f_y(1 - \frac{c}{L_w})}$

دوری

$\rightarrow A_{s\,req} \rightarrow \rho_L \rightarrow A_l$ per meter $\rightarrow S_{req} = \frac{A_{2\phi14} \times 1000}{A_l \text{ per meter}} \rightarrow$ sketch

---

تیرچه بلوک ← جرم و ارتفاع بلوک جدول $\frac{1}{5}$ ← بار مرده ی طبقه $\underline{D}$ ← ترکیب بار { ← ضخامت سقف ۸-۱-۲-۳-۲ $\frac{1}{20}$ دهنه

تعیین ضخامت دال روی سقف ← $q_u = (D+L) \times 1^m$ $\quad M_u = q_u L^2$ $\quad$ دهانه 500 تا 400 مربع ← $f_{ct} = \frac{M}{S}$, $f_r = 0.62\sqrt{f'_c}$ ← $f_{ct} < f_r$

$\hookrightarrow h \checkmark$ $\quad S \rightarrow I/h/2$, $\frac{1000h^3}{12}$

$h_{min} = \max\{50, \frac{1}{12} \times$ فاصله دو تیرچه ← طرح آرماتور تحتانی تیرچه (سرویس) مجموع بدون ضریب ← $M_u = \frac{P_u L_e^2}{8}$ ← $P_u = $ بار ضریبدار × عرض / دهنه ← طبق فرضیات طرح تیرچه بلوک و دال روی آن T شکل است

← حداقل ضخامت پاشنه 10 cm ← ارتفاع پاشنه ← جدول 2-1 ← تهران ملایم ← cover = 20 mm

← با توجه به شکل a را با فرض a<hf بدست می آوریم ← $A_s = \frac{0.85 f'_c ab}{f_y}$ ← check $A_{s\,min}$ ← $1.33A_{req}$

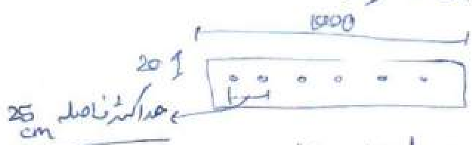
$A_{s\,min} = \max\{\frac{0.25\sqrt{f'_c}\,b_w d}{f_y}, \frac{1.4 b_w d}{f_y}\}$

← آرماتور عرضی $V_c = 0.17\lambda\sqrt{f'_c}\,b_w d$ ← ϕV_c ← V_s

← $A_{s\,min} = \frac{0.35 b_w S}{f_y}$ ← $2h\tan 30$ $\quad$ 60

$\underline{\phi 8}$ $\quad$ جدول 2-2 ← طرح آرماتور فوقانی تیرچه ←

آرماتور حرارت و جمع شدگی ← $0.0018\,A_g$ ← در هر 1000 میلیمتر عرض دال نیاز است.

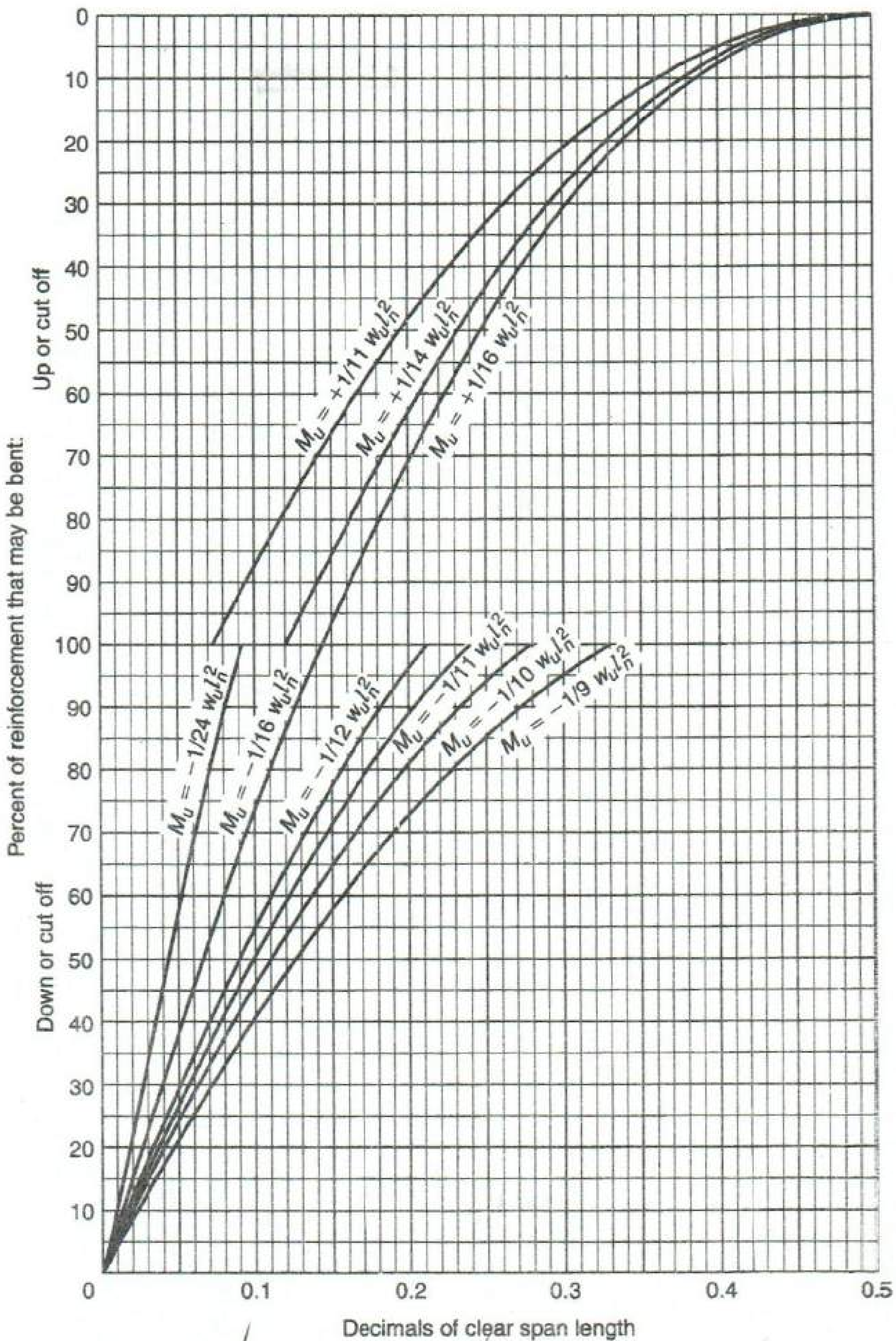


در هر ۲ جهت و 2 cm پایین تر از سطح دال قرار گیرند ← آرماتور منفی ← 0.15 آرماتور کششی و $\frac{1}{5}$ طول دهانه در هر طرف ادامه یابد. $\quad$ عرض × 1000

$\frac{1}{5}$ دهانه راست + $\frac{1}{5}$ دهانه ی چپ + عرض شاهتیر ← کلاف میانی ← بند ۵ بار زنده بیش از 350 نیز به کلاف ← ۲ کلاف نیاز داریم یکی در بالا و یکی در پایین و آجدار

← ترسیم خود تیرچه و بلوک و دال آنها

GRAPH A.3
Approximate locations of points where bars can be bent up or down or cut off for continuous beams uniformly loaded and built integrally with their supports according to the coefficients in the ACI Code.



1) thickness → 4.5(a) 2) concretecover 3) design shear 4) flexure
5) temp & shrinkage 6) max bar spacing 7) bar size 8) deflection
9) crack control 10) cut off 11) arrangment of reinf 12) %50 A_s [illegible]
13) sketch

Area of Equally Spaced Standard Bars (mm^2/m)

Bar Size (mm)	Center-to-Center Spacing (mm)											
	75	100	125	150	175	200	225	250	300	350	400	450
6	377	283	226	188	162	141	126	113	94	81	71	63
8	670	503	402	335	287	251	223	201	168	144	126	112
10	1,047	785	628	524	449	393	349	314	262	224	196	175
12	1,508	1,131	905	754	646	565	503	452	377	323	283	251
14	2,053	1,539	1,232	1,026	880	770	684	616	513	440	385	342
16	2,681	2,011	1,608	1,340	1,149	1,005	894	804	670	574	503	447
18	3,393	2,545	2,036	1,696	1,454	1,272	1,131	1,018	848	727	636	565
20	4,189	3,142	2,513	2,094	1,795	1,571	1,396	1,257	1,047	898	785	698
22	5,068	3,801	3,041	2,534	2,172	1,901	1,689	1,521	1,267	1,086	950	845
25	6,545	4,909	3,927	3,272	2,805	2,454	2,182	1,963	1,636	1,402	1,227	1,091

یه الهه صادقی | خلاصه فرمول بتن 1

$$\begin{cases} E_c = 0.043\, w^{1.5} \sqrt{f'_c} & 1440 < w_c < 2440 \\ E_c = 4700 \sqrt{f'_c} \end{cases}$$

$0.11 < \nu_{\text{بتن}} < 0.21$

ν حالت معمولی $= 0.17$

$Mpa = \frac{N}{mm^2}$

$1kg = 10\,N$

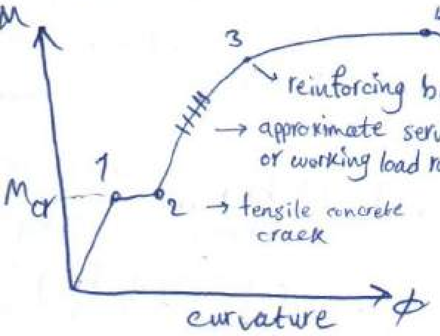
$f_r = 0.62\sqrt{f'_c}$

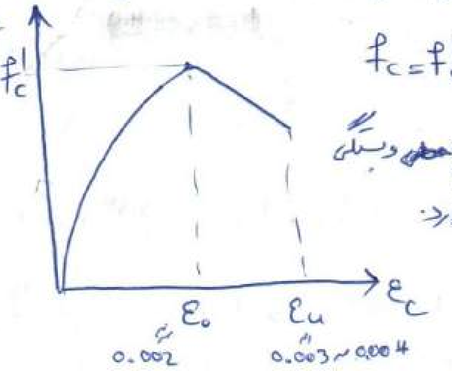
curvature $\leftarrow \phi = \frac{\varepsilon_c}{c}$

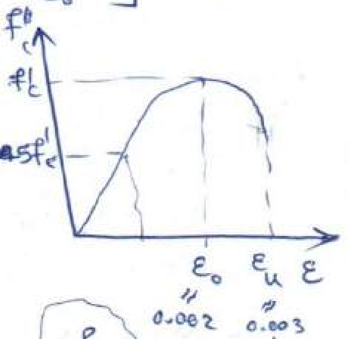
Hognestad model $\rightarrow f_c = f'_c \left[2\left(\frac{\varepsilon_c}{\varepsilon_0}\right) - \left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2 \right] \leftarrow$ غیرخطی

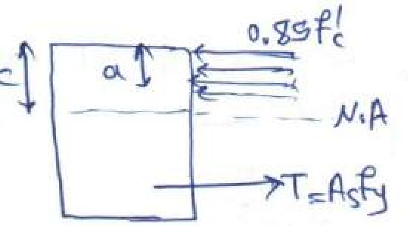
$f_c = f'_c \left[1 - \frac{Z}{1000} \left(\frac{\varepsilon_c - \varepsilon_0}{\varepsilon_0} \right) \right] \leftarrow$ خطی

برای بتن‌های معمولی Z وسیلی 150 به جهت بارگذاری دارد.









$a = \beta_1 c$

$$\beta_1 = \begin{cases} 0.85 & f'_c \le 28\,Mpa \\ 0.85 - 0.05\left(\frac{f'_c - 28}{7}\right) & 28 < f'_c \le 56\,Mpa \\ 0.65 & f'_c > 56\,Mpa \end{cases}$$

$f_{ct} = \frac{2P}{\pi L D}$

درصد میلگرد $\rho = \frac{A_s}{b_w d}$

$A_{s_{min}} = \frac{0.25\sqrt{f'_c}\, b_w d}{f_y} \ge \frac{1.4\, b_w d}{f_y}$

$A_{s_{min}}$ به جای $\rightarrow \rho_{min} = \frac{0.25\sqrt{f'_c}}{f_y} \ge \frac{1.4}{f_y}$

aci provision for $b_{eff} = \min\{b_f, 4b_w\}$

ارتفاع flange حداقل از $\leftarrow h_f \ge \frac{b_w}{2}$ بیشتر باشد.

minimum reinf. of flanged (T) section
- flange in compression → similar for rectangular beams
- flange in tension → $A_{s,min} = \frac{0.25\sqrt{f'_c}\, b_w d}{f_y}$

سرانگشتی $\rightarrow h_{\text{تیر}} = 0.1L$ $\quad d \approx \begin{cases} h - 65 & \rightarrow \text{one layer} \\ h - 95 & \rightarrow \text{two layers} \end{cases}$

$(d - \frac{a}{2}) \approx 0.9d \sim 0.95d \rightarrow A_{s,req} = \frac{M_u}{\phi f_y (d - \frac{a}{2})} \rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{\rho f_y d}{0.85 f'_c} \rightarrow \phi M_n = \phi A_s f_y d \left(1 - \frac{\rho f_y}{1.7 f'_c}\right)$

سرانگشتی $\rightarrow 1 \le \frac{d}{b} \le 2.5$, $\boxed{b \approx 0.5h \sim h}$

$A_{s\,min}$ ✓ $\quad S_{max} = \min\left[380\left(\frac{280}{f_s}\right) - 2.5C_c,\ 300\left(\frac{250}{f_s}\right) \right]$

f_s: $0.66 f_y$; C_c: حداقل فاصله میلگرد از سطح بتن ; f_s: $0.66 f_y$

$S_{min} = \max\left[d_b,\ 25\,mm,\ 1.33 D_{max} \right]$

Design of sections with known dim

1. assume $\phi = 0.9$ $\quad (d - \frac{a}{2}) \approx 0.9d \sim 0.95d$

$A_{s,req} = \frac{M_u}{\phi f_y (d - \frac{a}{2})}$ $\quad a = \frac{A_{s\,req} f_y}{0.85 f'_c d}$

$\varepsilon_t = 0.005 \rightarrow \phi = 0.9$

$\varepsilon_t = 0.004 \rightarrow \min \phi = 0.82$

check S_{min}, S_{max}, S

check $A_{s\,min}$, sketch sect

for unknown sections

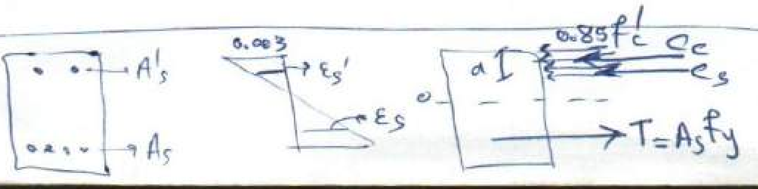
سرانگشتی $h = \frac{L}{16}$, $b = 0.5h \sim h \rightarrow$ self weight $\rightarrow M_u \rightarrow$ Assume $(d - \frac{a}{2})$ $\approx 0.9d \sim 0.95d$

$\rightarrow M_u = \phi A_s f_y (d - \frac{a}{2}) \rightarrow bd^2 = \text{---} \rightarrow b, d \rightarrow$ new self weight $\rightarrow M_u$ جدید $\rightarrow$ تفاوت M_u و M_u جدید زیاد است یا نه

$\rightarrow \rho_{initial} = 0.2\frac{f'_c}{f_y}$ یا 1% $\rightarrow A_s = \rho b d \xrightarrow{\text{جدول میلگرد}} A_{s\,pro} > A_{s\,req} \rightarrow$ check $A_{s\,min}$, S_{max}, S_{min}

$\rightarrow$ $\boxed{\phi = 0.9 \text{ تا } 0.005 \text{ بزرگتر از } \varepsilon_s \text{ گردد}}$ $\rightarrow a = \frac{f_y A_s}{0.85 f'_c b} \rightarrow c = \beta_1 a \rightarrow$ چک $\varepsilon_s > 0.005$ $\rightarrow M_u = \phi M_n = \phi f_y A_s (d - \frac{a}{2})$

sketch section $\leftarrow$



$\Sigma F_x = 0 \rightarrow C_s + C_c = T$

$\Sigma M_0 = 0 \rightarrow M_n = C_c(d - \frac{a}{2}) + C_s(d - d')$

$\phi M_n \ge M_u$

..le reinforced

1\. Mu → (self weight of beam, $0.5h < b < h$) → d → Max M_n singly reinf → nominal capacity of compression bar $M_{n2} = \dfrac{M_u - \phi M_{n1}}{\phi}$

$\phi M_1 + \phi M_{n2} = M_u$

→ $A'_{s\,req} = \dfrac{M_{n2}}{f'_s (d-d')}$ ($\varepsilon_s E_s$) → $A_{s\,req} = A_{s1} + \underbrace{A'_{s,req}\left(\frac{f'_s}{f_y}\right)}_{A_{s2}}$ → select bars A'_s pro $\geq A'_{s,req}$, A_s pro $\geq A_{s,req}$

→ S_{max}, S_{min}, $A_{s\,min}$, chang in effective depth → calculate $\phi M_n \geq M_u$ → sketch section

design of flanged section

1\. if unknown dim select reasonable dims → 2. Mu → d → b_{eff} → $M_{n,rec} = 0.85 f'_c b_e h_f (d - \frac{h_f}{2})$ → check whether T section analysis is req

→ $M_{n,req} = \dfrac{M_u}{\phi} \leq M_{n,\square}$ → not needed
$M_{n,req} = \dfrac{M_u}{\phi} \geq M_{n,\square}$ → needed
→ $A_{sf} = \dfrac{0.85 f'_c (b_e - b_w) h_f}{f_y}$, $M_{nf} = (A_{sf} f_y)(d - \frac{h_f}{2})$

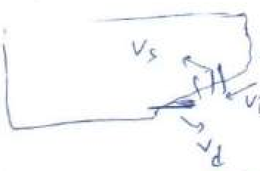
→ $M_{nw} = M_{n,req} - M_{nf}$ $A_{sw} = \dfrac{0.85 f'_c b_w a \rightarrow ?}{f_y}$ → $A_{s\,req} = A_{sf} + A_{sw}$ → جدول میگیرد $A_{s\,pro} \geq A_{s,req}$

→ S_{max}, Min, $A_{s\,min}$, chang in depth → $\phi M_n \geq M_u$ → sketch

$\sigma = \dfrac{-My}{I}$ $\tau = \dfrac{VQ}{It}$ $V_{cr} = 0.29\sqrt{f'_c}$ ← برای ترک web shear $V_{cr} = 0.16\sqrt{f'_c}$ → Diagonal ترک

زاویه میلگرد با افق

$$V_s = \frac{A_v f_y d (\sin\alpha + \cos\alpha)}{S}$$



$\Sigma V_{int} = V_{iy} + V_d + V_s + V_{cz}$

برای 90° → $V_s = \dfrac{A_v f_y d}{S}$ n V

$\rho_w = \dfrac{A_s}{b_w d}$ $\boxed{\dfrac{V_u d}{M_u} \leq 1.0}$

$$\begin{cases} V_c = \left(0.16\sqrt{f'_c} + 17\rho_w \dfrac{V_u d}{M_u}\right) b_w d \leq 0.29\sqrt{f'_c}\, b_w d \\ V_c = 0.17\sqrt{f'_c}\, b_w d \end{cases}$$

زاویه خاموت با افق

S_{max} خاموت قائم $\begin{cases} \min(d/2, 600\ mm) & V_u \leq 0.35\sqrt{f'_c}\, b_w d \\ \min(\frac{d}{4}, 300\ mm) & V_u > 0.35\sqrt{f'_c}\, b_w d \end{cases}$

max spacing inclined → $S_{max} = \min\left(\frac{3}{8}d + \frac{3d}{8}\cot\alpha\right)$, 600, 300

$\alpha \geq 45°$

$\boxed{V_{s\,req} = \dfrac{V_u}{\phi_{0.75}} - V_c}$ $\boxed{\left(\dfrac{A_v}{S}\right)_{req} = \dfrac{V_{s\,req}}{f_y d}}$ $\boxed{\text{مایل} \left(\dfrac{A_v}{S}\right)_{req} = \dfrac{V_{s\,req}}{f_y d (\sin\alpha + \cos\alpha)}}$

برای bentup $A_{v,req} = \dfrac{V_{s\,req}}{f_y \sin\alpha}$ $\alpha \geq 30$ $V_{s,req} \leq 0.25\sqrt{f'_c}\, b_w d$ معیار برای اینکه ترک ترک خورده ولی عرضش زیاد نشود

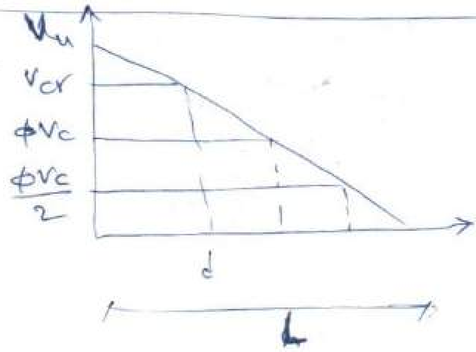
$V_s \leq V_{s\,max} = 0.66\sqrt{f'_c}\, b_w d$

V_u diagram → ϕV_c, $\frac{\phi V_c}{2}$ → $\frac{A_v}{S}$ critical

select striup size $\phi_{خم} \geq 32$, $\phi_{برش} = 14\phi$, spacing

→ S_{req} → uniform spacing stirrup

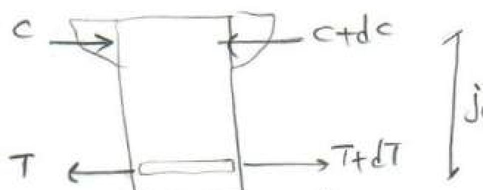
check S_{max}, S_{min}, $V_{s\,max}$, sketch



$A_{v\,min} = \max\left(\dfrac{0.062\sqrt{f'_c}\, b_w S}{f_{yt}}, \dfrac{0.35 b_w S}{f_{yt}}\right)$

$S = \min\left(\underbrace{\dfrac{A_v f_y d}{V_s}}_{S_{req}}, S_{max}\right)$

برگه‌ی خلاصه فرمول DRC از برش به بعد



نیروی bond force بین میلگرد و بتن

$$T = \frac{M}{jd} \qquad \boxed{U = \frac{V}{jd}}$$

$$U = \frac{1}{jd}\frac{dM}{dx}$$

مرکز میلگرد تا نزدیک‌ترین سطح

$$L_d = \left(\frac{f_y}{1.1\lambda\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{C_b + K_{tr}}{d_b}\right)} \right) d_b \geq 300\ \text{mm}$$

ضریب حباب‌های به دام افتاده. $\leftarrow \psi_t \begin{cases} 1.3 \rightarrow \text{زیرش بیش از 300mm بتن باشد} \\ 1.0 \rightarrow \text{other} \end{cases}$

coating factor $\leftarrow \psi_e \begin{cases} 1.5 \rightarrow \text{epoxy coated} \rightarrow \text{cover less than } 3d_b \text{, clear spacing} \leq 6d_b \\ 1.2 \rightarrow \text{other epoxy coated} \\ 1.0 \rightarrow \text{uncoated} \end{cases}$

$$K_{tr} = \frac{40 A_{tr}}{sn} \xrightarrow{\text{simplify}} K_{tr} = 0$$

bar size factor $\leftarrow \psi_s \begin{cases} 0.8 \rightarrow \phi \leq \phi 20 \\ 1.0 \rightarrow \phi \geq \phi 22 \end{cases}$

$\psi_t \psi_e \leq 1.7 \rightarrow$ نیازی به بزرگتر گرفتن نیست.

ضریب بتن سبک $\leftarrow \lambda \begin{cases} 0.75 \rightarrow \text{light weight} \\ \lambda = \frac{f_{ct}}{0.56\sqrt{f'_c}} \leq 1 \rightarrow \text{when we have } f_{ct} \end{cases}$

$\lambda = 1.0 \rightarrow$ for normal concrete

reduction factor : $\frac{A_{s\,req}}{A_{s\,pro}}$ or $\frac{M_u}{M_n} = \frac{M_u/\phi}{M_n}$

$$L_{dc} = \max\left(\frac{0.24 f_y d_b}{\sqrt{f'_c}}, 0.043 f_y d_b\right) \geq 200\ \text{mm}$$

میلگرد فشاری

$$L_{dh} = \frac{0.24 \psi_e f_y}{\lambda\sqrt{f'_c}} d_b \geq \max(8d_b, 150\ \text{mm})$$

$\psi_e \begin{cases} 1.2 \rightarrow \text{epoxy coated} \\ 1.0 \rightarrow \text{other} \end{cases}$

$\lambda = \begin{cases} 0.75 \rightarrow \text{light weight} \\ 1.0 \rightarrow \text{other} \end{cases}$

↳ reduction factors ⟶ 1. cover ≥ 65 mm, reduction factor = 0.7   2. $\frac{A_{s\,req}}{A_{s\,pro}}$

ضوابط مربوط به خم‌های آیین‌نامه لحاظ گردد.

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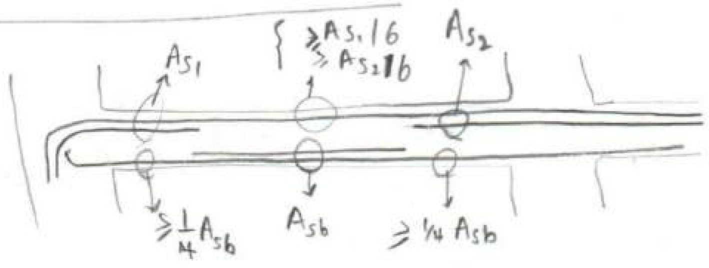
lap splice

$$L_s = \begin{cases} 0.071 f_y d_b & \text{for } f_y \leq 420\ \text{MPa} \\ (0.13 f_y - 24) d_b & \text{for } f_y > 420\ \text{MPa} \end{cases}$$

class $\begin{cases} A \rightarrow \frac{A_{s\,prov}}{A_{s\,req}} > 2 & \text{lap length: } 1.0 L_d \\ B \rightarrow \quad < 2 & \text{lap length: } 1.3 L_d \end{cases}$ ✓

---

perimeter beams



با میلگرد فشاری > 300 mm

Lap length: $1.33 L_s \leftarrow f'_c < 21$

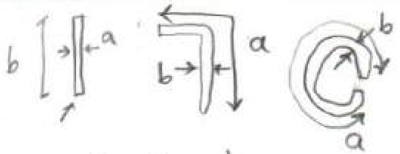
$$\left.\begin{aligned} \phi M_n &= \phi A_s f_y \\ a &= \frac{A_s f_y}{0.85 f'_c b} = \frac{\rho f_y d}{0.85} \end{aligned}\right\} \longrightarrow \phi M_n = \phi A_s f_y d\left(1 - \frac{\rho f_y}{0.85 f'_c}\right) \rightarrow \frac{M_u}{\phi b d^2} = \rho f_y\left(1 - \frac{1}{1.7}\frac{\rho f_y}{f'_c}\right)$$

ϕ: 0.9

$$\boxed{\rho = \frac{0.2 f'_c}{f_y}}$$

Torsion

دایره توپر $J = \frac{1}{2}\pi c^4$ توخالی $J = \frac{1}{2}\pi (c_2^4 - c_1^4)$



$\tau = \frac{T\rho}{J}$ فاصله از مرکز دایره ; برش

زاویه پیچش $\phi = \frac{TL}{GJ}$

$\tau_{max} = \frac{Tc}{J}$ شعاع دایره

دایره‌ای

مرکز ضلع بزرگتر

$\tau_{max} = \frac{T}{c_1 ab^2}$ عرض ، طول

$\frac{a}{b} > 10 \rightarrow c_1 = c_2 = \frac{1}{3}$

$\phi = \frac{TL}{c_2 ab^3 G}$

مستطیلی

شرط جداره نازک $\frac{t}{a}, \frac{t}{b} < 0.1$

جداره نازک $q = t\tau = const$ shear flow

$T = 2qA_0 \rightarrow$ مساحت داخل خط وسط جداره نازک

$\tau_{max} = \frac{q}{t_{min}} = \frac{T}{2t_{min}A_0}$

صرف‌نظر از داشتن یا نداشتن فضای خالی

$T_{cr} = (0.33\sqrt{f'_c})(2A_0 t)$ → rectangular solid sections → $\begin{cases} A_0 = \frac{2}{3}A_{cp} \rightarrow \text{ساحت بتن مقطع} \\ t = \frac{3}{4}A_{cp}/P_{cp} \end{cases}$

$T_{cr,solid} = 0.33\sqrt{f'_c}\,\frac{A_{cp}^2}{P_{cp}}$ $\quad T_{cr,hollow} = 0.33\sqrt{f'_c}\,\frac{A_{cp}}{P_{cp}}A_g$

پیچش آستانه
Threshold Torsion

$\phi T_r = \phi(0.083)\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) \xrightarrow{\text{با نیروی محوری}} \times\sqrt{1+\frac{N_u}{0.33A_g\sqrt{f'_c}}}$

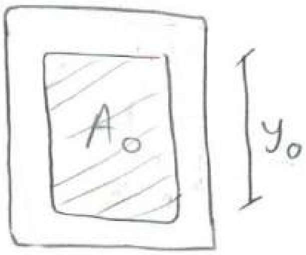
برای Hollow section در روابط فوق به جای A_{cp} از A_g استفاده می‌شود

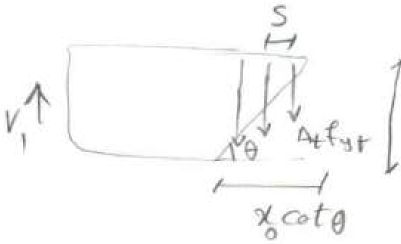
$\phi_{torsion} = 0.75$

---

shear forces in Ties $\quad q = \frac{T_n}{2A_0} \quad V_1 = V_3 = qx_0 = \frac{T_n x_0}{2A_0}$

$V_2 = V_4 = qy_0 = \frac{T_n y_0}{2A_0}$





$\left.\begin{aligned} V_1 &= nA_t f_{yt} \\ n &= \frac{x_0\cot\theta}{S} \\ V_1 &= \frac{T_n x_0}{2A_0} \end{aligned}\right\} \quad \boxed{T_n = \frac{2A_0 A_t f_{yt}}{S}\cot\theta}$

$A_{t,req} = \frac{T_n S}{2A_0 f_{yt}\cot\theta} = \frac{T_u S}{2\phi A_0 f_{yt}\cot\theta}$ خاموت

$\begin{cases} A_0 = 0.85A_{oh} \rightarrow \text{مساحتی که خاموت‌ها محاط کرده‌اند} \\ \theta = 45° \end{cases}$

$\rightarrow A_{t,req} = \frac{T_u S}{1.7\phi A_{oh} f_{yt}} \rightarrow \frac{A_{t,req}}{S} = \frac{T_u}{1.7\phi A_{oh} f_{yt}}$

میلگرد طولی $A_{l,req} = \frac{A_{t,req}}{S}P_h\frac{f_{yt}}{f_y}\cot^2\theta \xrightarrow[A_0 = 0.85A_{oh}]{\theta = 45} A_{l,req} = \frac{T_u P_h}{1.7\phi A_{oh} f_y}\cot\theta$

$P_h \rightarrow$ محیط خاموت $2(x_0 + y_0)$

$\rightarrow A_{l,req} = \frac{T_u P_h}{1.7\phi A_{oh} f_y}$

---

combined Torsion & shear

$\tau_{max} = \tau_v + \tau_t = \frac{V_u}{b_w d} + \frac{T_u}{2A_0 t} = \frac{V_u}{b_w d} + \frac{T_u P_h}{1.7A_{oh}^2} \leq \phi\left(\frac{V_c}{b_w d} + 0.66\sqrt{f'_c}\right)$

Hollow sections

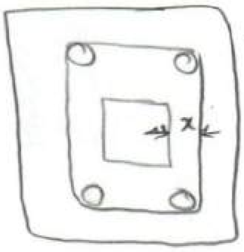
اگر این برقرار نبود یا f'_c را زیاد می‌کنیم یا $b_w d$ را

$\tau_{max} = \sqrt{\tau_v^2 + \tau_t^2} = \sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2} \leq \phi\left(\frac{V_c}{b_w d} + 0.66\sqrt{f'_c}\right)$

solid sections

Torsion + flexure $\quad M_u , T_u \rightarrow$ در یک مقطع حاضر شود

میلگردهای طولی فشاری را می‌توان به اندازه‌ی $\frac{M_u}{0.9 f_y d}$ کاهش داد زیرا اثر خمش و پیچش هم دیگر را خنثی می‌کند.



→ Hollow section

$$\boxed{x > 0.5 \frac{A_{oh}}{P_h}}$$

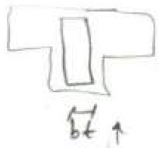
Min reinf عرضی $\rightarrow (A_v + 2A_t)_{min} = 0.062\sqrt{f'_c}\,\frac{b_w S}{f_{yt}} \geq 0.35\frac{b_w S}{f_{yt}}$ (torsional transverse)

Min reinf طولی $\quad A_{l,min} = \frac{0.42\sqrt{f'_c}\,A_{cp}}{f_y} - \max\left(\frac{A_t}{s}, 0.175\frac{b_w}{f_{yt}}\right)P_h\frac{f_{yt}}{f_y}$

محیط خاموت مرکزی دورترین آرماتور پیچش

$S_t < \min\left(\frac{P_h}{8}, 300\ mm\right) \rightarrow$ حل گردد سهم

فاصله‌ی میلگردهای عرضی



قطر میلگردهای $d_{b,min} > \max(0.042 S_t , 10\ mm)$

حداقل طولی که باید میلگرد پیچشی قرار داد $b_t + d$ در طول بعد از جایی است که پیچش آن را حذف کردیم

این رابطه آیین‌نامه برای کنترل عرض ترک می‌باشد $S_{max} = \min\left[380\left(\frac{280}{f_s}\right) - 2.5C_c , 300\left(\frac{280}{f_s}\right)\right]$

$$\Delta_{deflection} = \frac{WL^4}{384EI}$$

(I بعد از ترک)

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \times I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad I_{cr} < I_e < I_g$$

effective, خالص, cracked moment of inertia, cracking moment

$$I_e = I_{cr} + (I_g - I_{cr})\left(\frac{M_{cr}}{M_a}\right)^3$$

---

simply supported $\rightarrow I_e = I_{em}$ $\qquad I_e$

cantilever $\rightarrow I_e = I_{e\,support}$

both end continuous $\rightarrow I_e = 0.7 I_{em} + 0.15(I_{e1} + I_{e2})$

one " " $\rightarrow I_e = 0.85 I_{em} + 0.15(I_{e\,continuous\,end})$

$$\lambda_{t_0,\infty} = \frac{\xi_\infty - \xi_{t_0}}{1 + 50\rho'}$$

$$\Delta = \lambda_{t_0,\infty}\Delta_{iD} + \lambda_\infty \Delta_{i\,20\%L} + \Delta_{iL}$$

$$\Delta_{20\%L} = \Delta_{20\%L+D} - \Delta_D$$

a/b	c_1	c_Y
1	0.208	0.1406
1.2	0.219	0.1661
1.5	0.231	0.1958
2.0	0.246	0.229
2.5	0.258	0.249
3	0.267	0.263
4	0.282	0.281
5	0.291	0.291
10	0.312	0.312
∞	0.333	0.333

$A_{s\,min}$ ch.10.5 ACI

$1\ pa = 1\ \frac{N}{m^2}$

$1\ Mpa = 1\ \frac{N}{mm^2}$

$10\ kg = 1N$

Area of Groups of Standard Bars (mm^2)

Bar Size (mm)	Number of Bars																	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
8	50	101	151	201	251	302	352	402	452	503	553	603	653	704	754	804	855	905
10	79	157	236	314	393	471	550	628	707	785	864	942	1,021	1,100	1,178	1,257	1,335	1,414
12	113	226	339	452	565	679	792	905	1,018	1,131	1,244	1,357	1,470	1,583	1,696	1,810	1,923	2,036
14	154	308	462	616	770	924	1,078	1,232	1,385	1,539	1,693	1,847	2,001	2,155	2,309	2,463	2,617	2,771
16	201	402	603	804	1,005	1,206	1,407	1,608	1,810	2,011	2,212	2,413	2,614	2,815	3,016	3,217	3,418	3,619
18	254	509	763	1,018	1,272	1,527	1,781	2,036	2,290	2,545	2,799	3,054	3,308	3,563	3,817	4,072	4,326	4,580
20	314	628	942	1,257	1,571	1,885	2,199	2,513	2,827	3,142	3,456	3,770	4,084	4,398	4,712	5,027	5,341	5,655
22	380	760	1,140	1,521	1,901	2,281	2,661	3,041	3,421	3,801	4,181	4,562	4,942	5,322	5,702	6,082	6,462	6,842
25	491	982	1,473	1,963	2,454	2,945	3,436	3,927	4,418	4,909	5,400	5,890	6,381	6,872	7,363	7,854	8,345	8,836
28	616	1,232	1,847	2,463	3,079	3,695	4,310	4,926	5,542	6,158	6,773	7,389	8,005	8,621	9,236	9,852	10,468	11,084
30	707	1,414	2,121	2,827	3,534	4,241	4,948	5,655	6,362	7,069	7,775	8,482	9,189	9,896	10,603	11,310	12,017	12,723
32	804	1,608	2,413	3,217	4,021	4,825	5,630	6,434	7,238	8,042	8,847	9,651	10,455	11,259	12,064	12,868	13,672	14,476
40	1,257	2,513	3,770	5,027	6,283	7,540	8,796	10,053	11,310	12,566	13,823	15,080	16,336	17,593	18,850	20,106	21,363	22,619

Bundled Bars

Two, three, or at most four bars can be bundled together to act as a group.

For more details of bundled bars refer to ACI 318-11 Section 7.6.6.

Two bars (Vertical pattern)

Two bars (Horizontal pattern)

Three bars (Triangular pattern)

Three bars (L-shaped pattern)

Four bars (Square pattern)

Habib Sadeghi

Minimum Required Beam Width* (mm)

Bar size (mm)	Number of bars in single layer of reinforcement						
	2	3	4	5	6	7	8
8	181	223	264	305	346	388	429
10	183	227	270	313	356	400	443
12	185	231	276	321	366	412	457
14	187	235	282	329	376	424	471
16	189	239	288	337	386	436	485
18	191	243	294	345	396	448	499
20	193	247	300	353	406	460	513
22	195	251	306	361	416	472	527
25	198	257	315	373	431	490	548
28	201	263	324	385	446	508	569
32	205	271	336	401	466	532	597

* *In the presence of lap splices, the tabulated values may need revision.*

Note:
This table has been prepared based on the following assumptions:

- $d_s = 10\ mm$
- $c = 40\ mm$
- $D_{max} = 25\ mm$

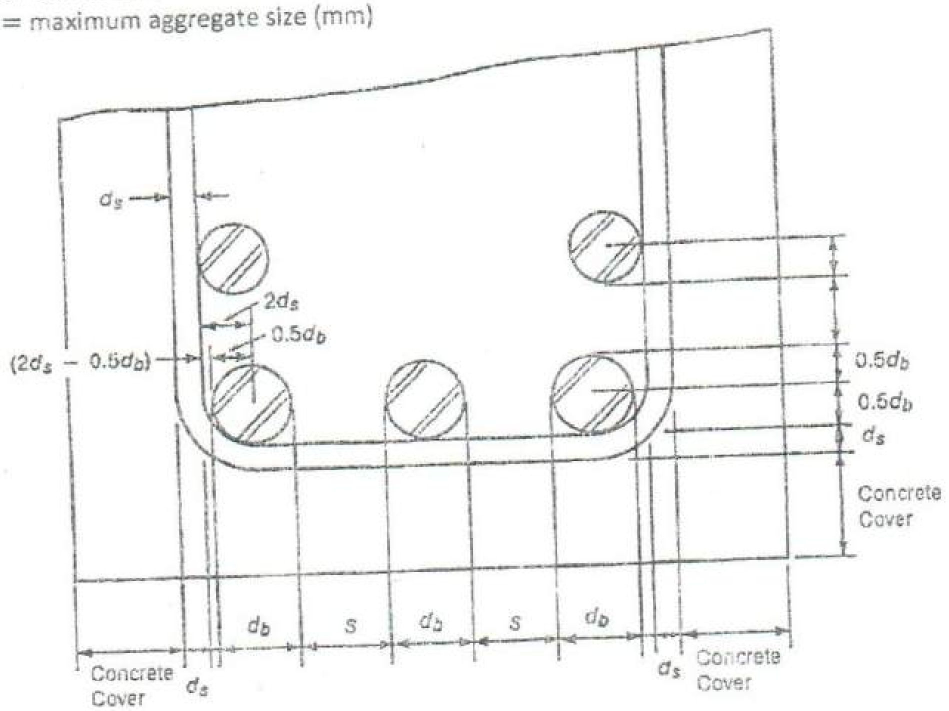
اگر طول تیر خیلی بلند باشد باید رابطه revise شود

For other cases, the following equation can be used:

$$b_{min} = \max(d_b, 25, 1.33D_{max})\,(n-1) + 2\left[c + d_s + \max\left(2d_s, \frac{d_b}{2}\right)\right] + (n-1)d_b$$

where,

b_{min} = minimum required beam width (mm)
c = clear concrete cover (mm)
d_b = bar diameter (mm)
d_s = stirrup diameter (mm)
n = number of bars
D_{max} = maximum aggregate size (mm)



Habib Sadeghi

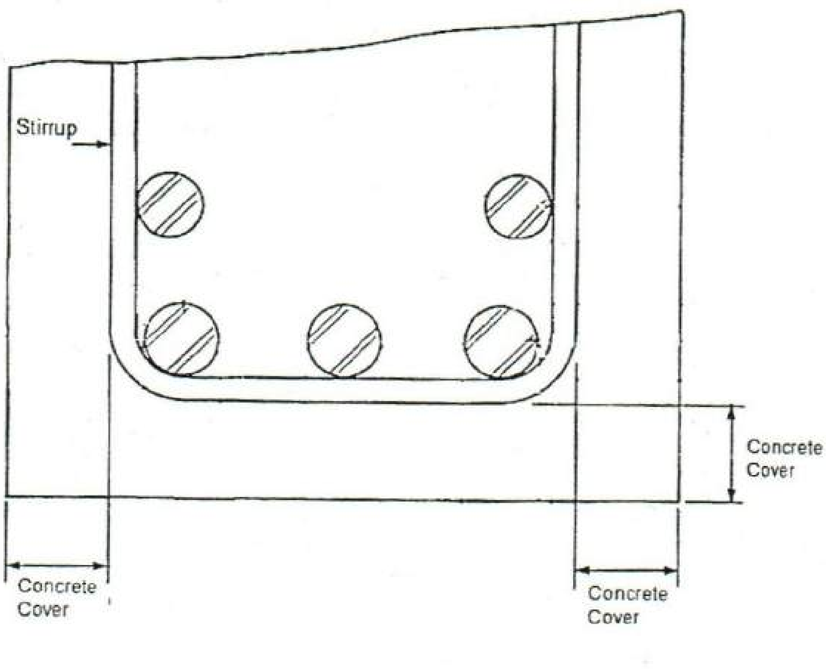
Minimum Concrete Cover* (According to ACI318M-11 7.7)

Casting Type	Surface Exposure Type	Member Type	Bar Size (mm)	Minimum Concrete Cover (mm)
Cast-in-place concrete (nonprestressed)	Concrete cast against and permanently exposed to earth	-	-	75
	Concrete exposed to earth or weather	-	$\Phi \geq 18$	50
			$\Phi < 18$	40
	Concrete not exposed to weather or in contact with ground	Slabs, walls, joists	$\Phi \geq 40$	40
			$\Phi < 40$	20
		Beams, columns	-	40
		Shells, folded plate members	$\Phi \geq 18$	20
			$\Phi < 18$	13
Precast concrete (manufactured under plant control conditions)	Concrete exposed to earth or weather	Wall panels	$\Phi \geq 40$	40
			$\Phi < 40$	20
		Other members	$\Phi \geq 40$	50
			$16 < \Phi < 40$	40
			$\Phi \leq 16$	30
	Concrete not exposed to weather or in contact with ground	Slabs, walls, joists	$\Phi \geq 40$	30
			$\Phi < 40$	16
		Beams, columns (primary reinforcement)	–	$= \Phi$ (bar diameter) But $16 < cover < 40$
		Beams, columns (Ties, stirrups, spirals)	–	10
		Shells, folded plate members	$\Phi \geq 18$	16
			$\Phi < 18$	10

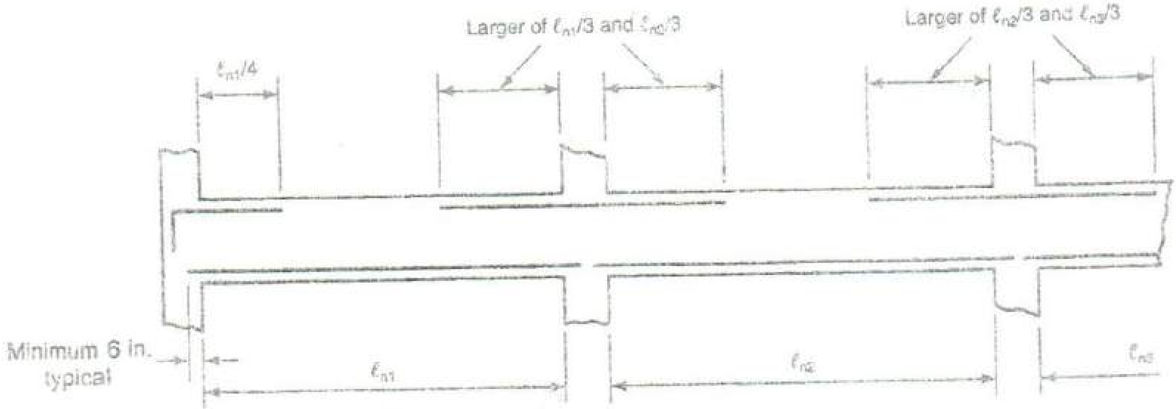
** The listed minimum concrete covers are only for general conditions and in case of corrosive environments and fire protection greater concrete cover may be specified, see Sections 7.7.6 to 7.7.8.*

** For the required concrete covers of prestressed concrete members refer to Section 7.7.2 of ACI318M-11.*

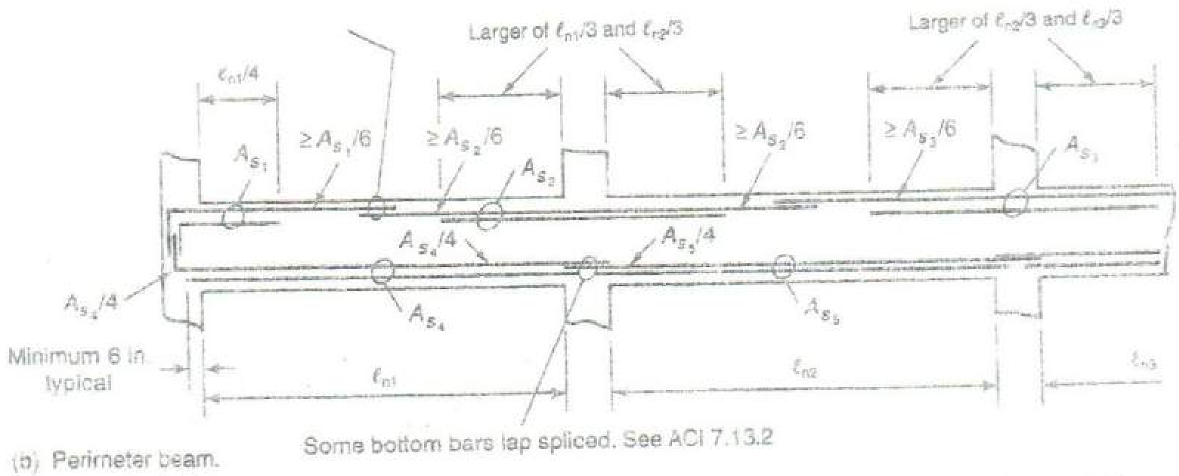
** For bundled bars see Section 7.7.4 of ACI318M-11.*



Approximate Cut-off Rules

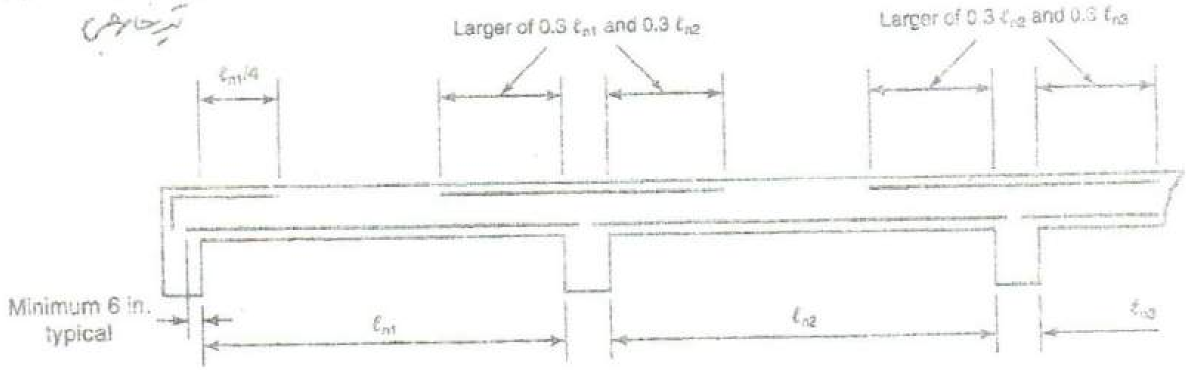


(a) Beam with closed stirrups. If closed stirrups are not provided, see ACI Code Section 7.13.



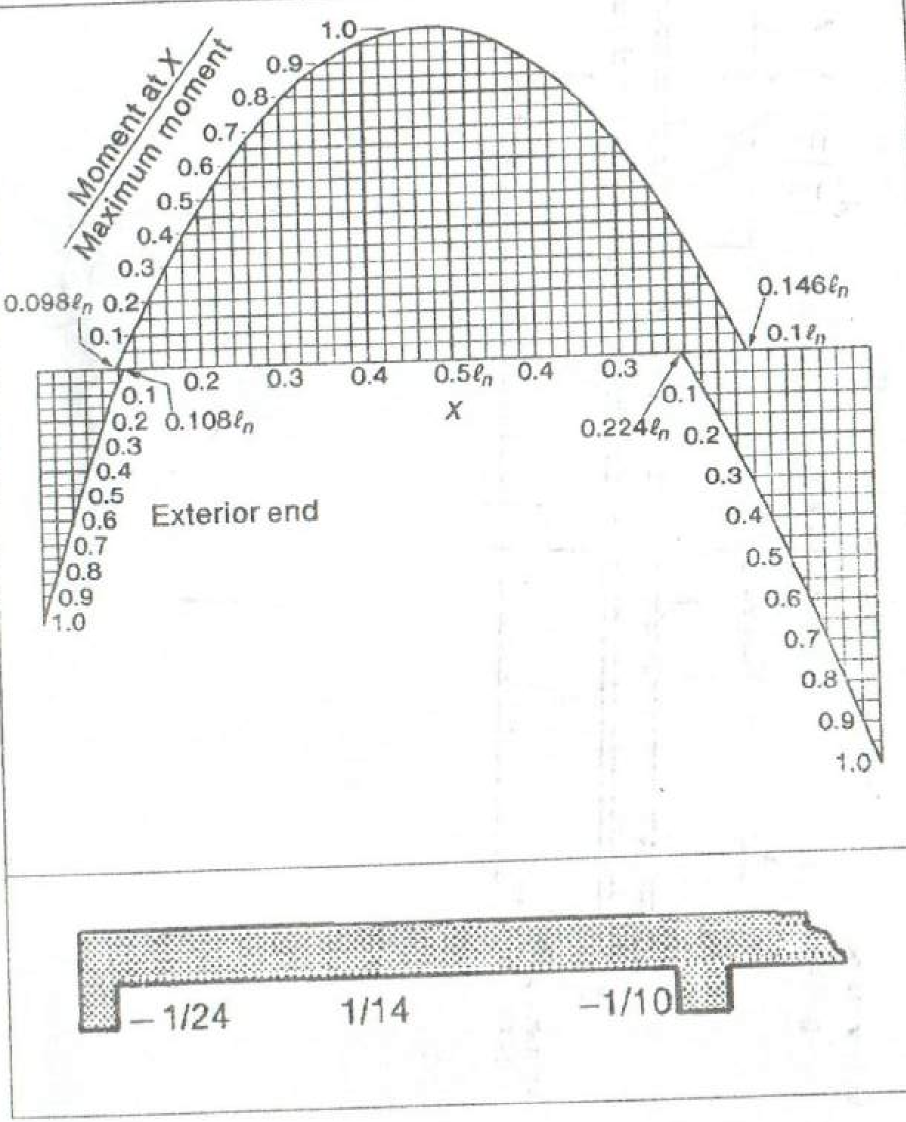
(b) Perimeter beam.

تیر حاشیه‌ای

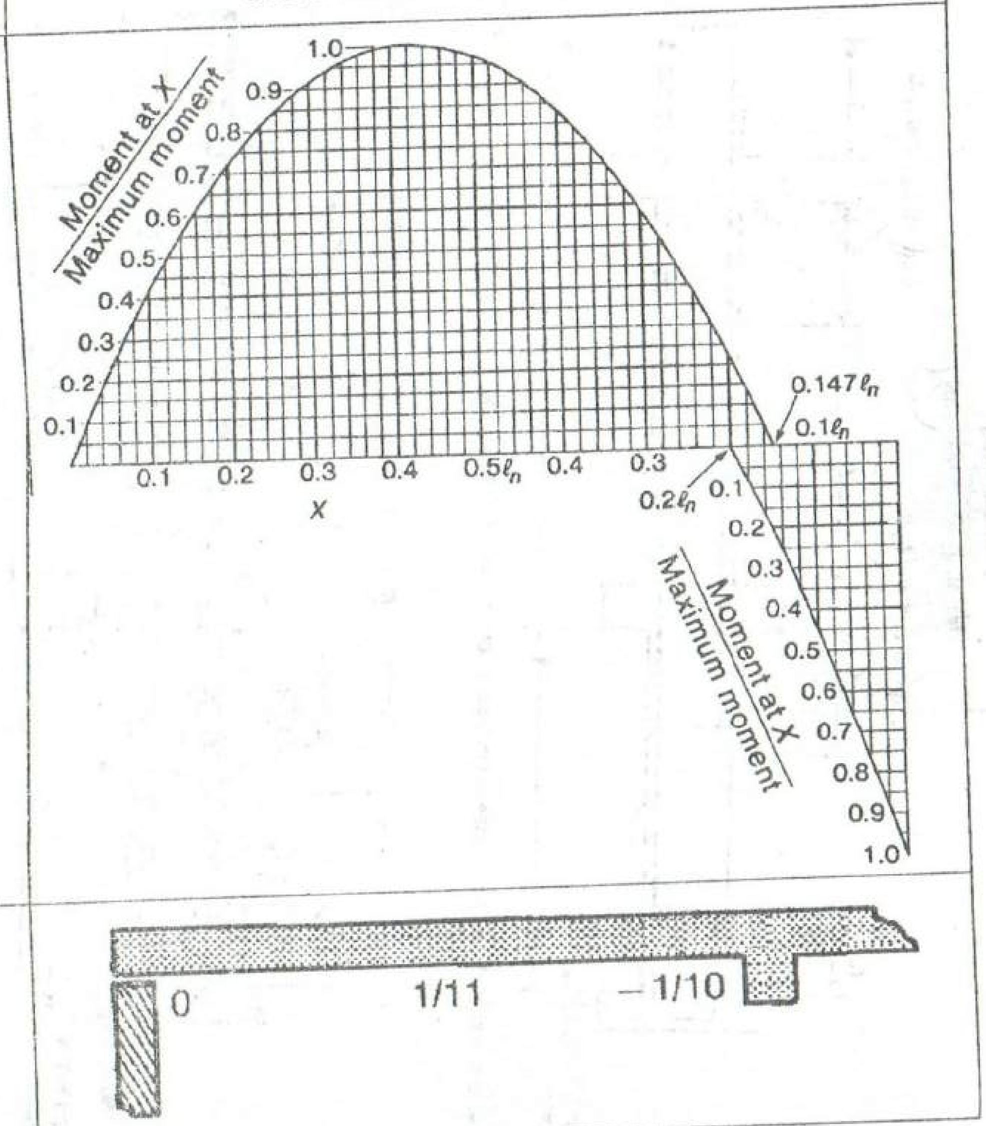


(c) One-way slab.

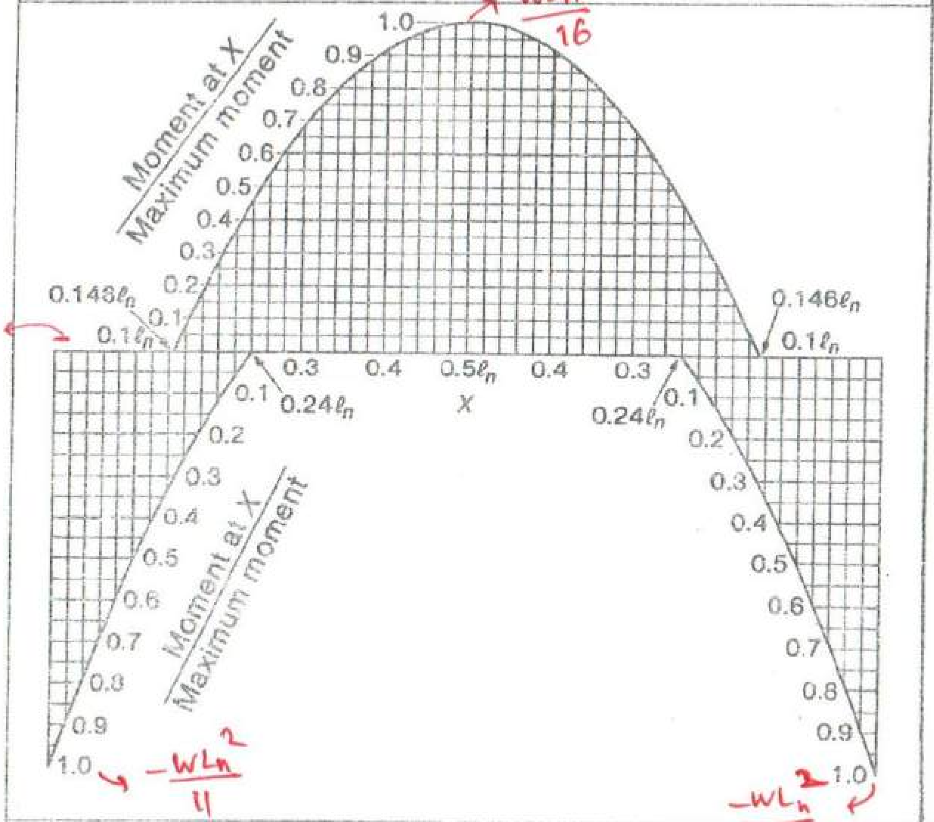
Bending-moment envelope for exterior span with exterior support built integrally with a spandrel beam or girder

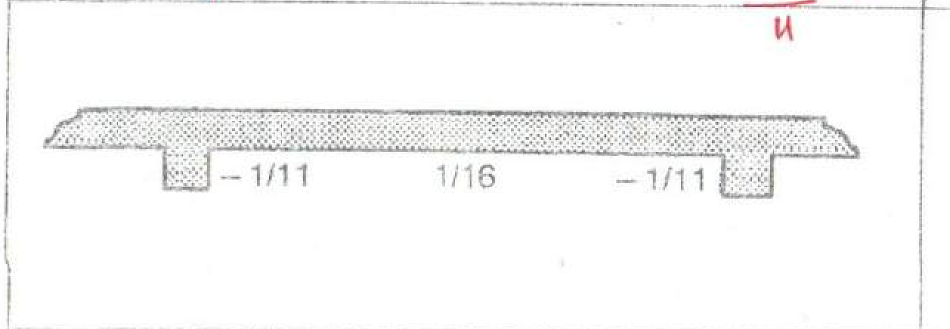


Bending-moment envelope for exterior span with discontinuous end unrestrained

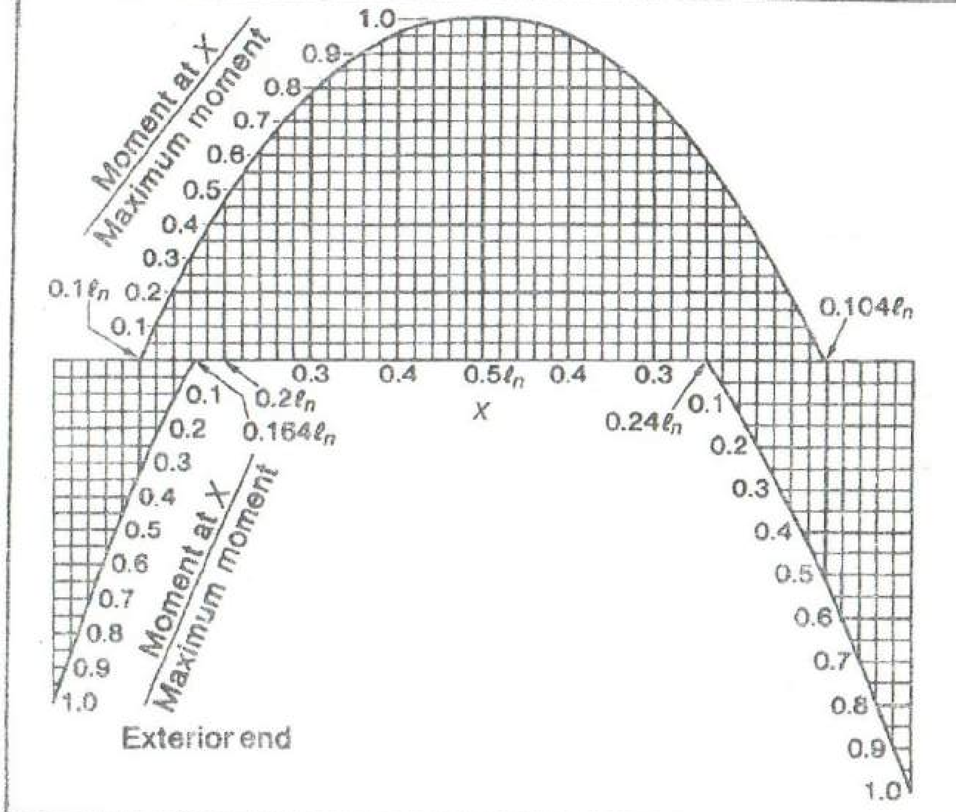


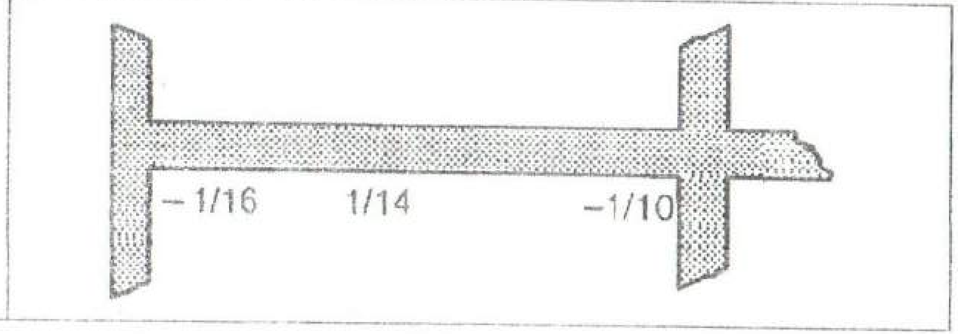
Bending-moment envelope for typical interior span





Bending-moment envelope for exterior span with exterior support built integrally with a column





Project # 1 of designing concrete structures 2 Dr.eskandari 2nd semester of 1393-94

St name : Habib-ollah Sadeghi St ID : 91105047 Group : B

For the following rectagular cross sectional beam let's develope the interaction curve diagram under the given properties :

$b := 400$ dimensions of section $cover := 65$

$h := 500$

$n := 4$ number of layers of bars

Enter the area of the bars of ech layer from top to the bottom in the matrix format :

$As := \begin{bmatrix} 2463 \\ 2463 \\ 2463 \\ 2463 \end{bmatrix}$ $sum := \sum_{i=1}^{n} As_i$ for instance we have 4 layers of $4\phi28$ bars

$sum = 9.852 \cdot 10^3$

$n1 := 4$ number of bars in the first layer $\varepsilon s := \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$ the first input for the εs of different steel layers

$Dstir := 10$ Diameter of the stirrup

$$S := \frac{\left(h - 2 \cdot cover - 2 \cdot Dstir - \left(\frac{As_1}{\pi \cdot n1}\right)^{0.5}\right)}{n} \qquad S = 84$$

$$y := \left\| \begin{array}{l} \text{for } m \in 1..n \\ \quad \left\| \; y_m \leftarrow \left(cover + \frac{\left(\frac{As_1}{\pi \cdot n1}\right)^{0.5}}{2} + Dstir + (m-1) \cdot S \right) \right. \\ y \end{array} \right. \qquad y = \begin{bmatrix} 82 \\ 166 \\ 250 \\ 334 \end{bmatrix}$$

we assumed that the bars were in equal space " s " from each other

$Astotal := sum$ total area of the steel $Astotal = 9.852 \cdot 10^3$

$Atotal := b \cdot h$ total area of the section $Atotal = 2 \cdot 10^5$

$Ac := Atotal - Astotal$ pure area of concrete

$fy := 420$ $\varepsilon y := \frac{fy}{200000}$ $\varepsilon y = 0.0021$ $Es := 200000$

$f'c := 28$

$$d1 := h - cover - Dstir - \left(\frac{As_1}{\pi \cdot n1}\right)^{0.5} \qquad d1 = 411$$

$Ts := 0$ tensile force of bars

$Cs := 0$ compressive force of steel

$Cc := 0$ compressive force of concrete

$\beta 1 :=$ if $f'c \le 28$
  $\beta 1 \leftarrow 0.85$
else if $28 \le f'c \le 56$
  $\beta 1 \leftarrow 0.85 - 0.05 \cdot \dfrac{(f'c - 28)}{7}$
else if $f'c \ge 56$
  $\beta 1 \leftarrow 0.65$
  $\beta 1$

$\beta 1 = 0.85$

let's calculate pure axial load capacity :

$Pno := 0.85 \cdot f'c \cdot Ac + Astotal \cdot fy$ $Pno = 8.6634 \cdot 10^{6}$

lets calculate Pure tensile load capacity :

$Pn1o := -Astotal \cdot fy$ $Pn1o = -4.1378 \cdot 10^{5}$

lets calculate the balanced point of interaction curve :
" At the balanced point the $\varepsilon c := 0.003$ and the tensile steel is yielded "

Solve Constraints Guess Values

$c := 100$

$0.003 \cdot (d1 - c) = \varepsilon y \cdot c$

$c := \text{find}(c)$ $c = 241.7647$

$c = 241.7647$

$a := \beta 1 \cdot c$ $a = 205.5$

```
εs := | for i ∈ 1..n-1
      |   if y_i ≤ c
      |     εs_i ← ((c - y_i) · 0.003) / c
      |   else if y_i ≥ c
      |     εs_i ← (0.003 · (y_i - c)) / c
      | εs_n ← εy
      | εs
```

$$\varepsilon s = \begin{bmatrix} 0.002 \\ 0.0009 \\ 0.0001 \\ 0.0021 \end{bmatrix}$$

```
fs := | for i ∈ 1..n
      |   if εs_i ≥ εy
      |     fs_i ← fy · As_i
      |   else if εs_i ≤ εy
      |     fs_i ← εs_i · Es · As_i
      | fs
```

$$fs = \begin{bmatrix} 9.7657 \cdot 10^{5} \\ 4.6312 \cdot 10^{5} \\ 5.0339 \cdot 10^{4} \\ 1.0345 \cdot 10^{6} \end{bmatrix}$$

$Ts := 0$

```
Ts := | for i ∈ 1..n
      |   if y_i ≥ c
      |     Ts ← Ts + fs_i
      | Ts
```

$Ts = 1.0848 \cdot 10^{6}$

$Cc := 0.85 \cdot f'c \cdot a \cdot b$ $Cc = 1.9564 \cdot 10^{6}$ $Cs := 0$

$$Cs := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } y_i \le c \\ \quad \| Cs \leftarrow Cs + fs_i \end{array} \right. \\ Cs \end{array} \right.$$

$Cs = 1.4397 \cdot 10^6$

$Pb := Cc + Cs - Ts$

$Pb = 2.3112 \cdot 10^6$

$$Mb := Cc \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$$

$$Mb := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } y_i \le c \\ \quad \left\| Mb \leftarrow Mb + fs_i \cdot \left(\frac{h}{2} - y_i\right) \right. \\ \text{else if } y_i \ge c \\ \quad \left\| Mb \leftarrow Mb + fs_i \cdot \left(y_i - \frac{h}{2}\right) \right. \end{array} \right. \\ Mb \end{array} \right.$$

$Mb = 5.7793 \cdot 10^8$

$Mb = 5.7793 \cdot 10^8$ Mb is the moment about the neutral axis at the balanced point
$Pb = 2.3112 \cdot 10^6$ Pb is the total axial force at the balanced point

Let's find a point in the Tension controlled region :
we assume a C smaller than the one in balanced point :

$c := c - 50$

$$\varepsilon s := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } y_i \le c \\ \quad \left\| \varepsilon s_i \leftarrow \frac{\left(c - y_i\right) \cdot 0.003}{c} \right. \\ \text{else if } y_i \ge c \\ \quad \left\| \varepsilon s_i \leftarrow \frac{\left(0.003 \cdot \left(y_i - c\right)\right)}{c} \right. \end{array} \right. \\ \varepsilon s \end{array} \right.$$

$$\varepsilon s = \begin{bmatrix} 0.0017 \\ 0.0004 \\ 0.0009 \\ 0.0022 \end{bmatrix}$$

$a := \beta 1 \cdot c$ $\qquad a = 163$

$$fs := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } \varepsilon s_i \geq \varepsilon y \\ \quad \| fs_i \leftarrow fy \cdot As_i \\ \text{else if } \varepsilon s_i \leq \varepsilon y \\ \quad \| fs_i \leftarrow \varepsilon s_i \cdot Es \cdot As_i \end{array} \right. \\ fs \end{array} \right.$$

$$fs = \begin{bmatrix} 8.4588 \cdot 10^5 \\ 1.9855 \cdot 10^5 \\ 4.4878 \cdot 10^5 \\ 1.0345 \cdot 10^6 \end{bmatrix}$$

$Ts := 0 \qquad Cs := 0$

$$Ts := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } y_i \geq c \\ \quad \| Ts \leftarrow Ts + fs_i \end{array} \right. \\ Ts \end{array} \right.$$

$Ts = 1.4832 \cdot 10^6$

$Cc := 0.85 \cdot f'c \cdot a \cdot b$ $\qquad Cc = 1.5518 \cdot 10^6$

$$Cs := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } y_i < c \\ \quad \| Cs \leftarrow Cs + fs_i \end{array} \right. \\ Cs \end{array} \right.$$

$Cs = 1.0444 \cdot 10^6$

$P1 := Cc + Cs - Ts$

$P1 = 1.113 \cdot 10^6$

$$M1 := Cc \cdot \left(\frac{h}{2} - \frac{a}{2} \right)$$

$$M1 := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } y_i \leq c \\ \quad \left\| M1 \leftarrow M1 + fs_i \cdot \left(\frac{h}{2} - y_i \right) \right. \\ \text{else if } y_i \geq c \\ \quad \left\| M1 \leftarrow M1 + fs_i \cdot \left(y_i - \frac{h}{2} \right) \right. \end{array} \right. \\ M1 \end{array} \right.$$

$M1 = 5.0715 \cdot 10^{8}$ M1 is the moment about the neutral axis at a tension controlled point
$P1 = 1.113 \cdot 10^{6}$ P1 is the total axial force at a tension controlled point

Let's find a point in the compression controlled region :
we assume a C bigger than the one in balanced point :

$$c := c + 100$$

$$\varepsilon s := \begin{array}{l} \text{for } i \in 1..n \\ \quad \text{if } y_i \le c \\ \quad\quad \varepsilon s_i \leftarrow \dfrac{(c - y_i) \cdot 0.003}{c} \\ \quad \text{else if } y_i \ge c \\ \quad\quad \varepsilon s_i \leftarrow \dfrac{(0.003 \cdot (y_i - c))}{c} \\ \quad \varepsilon s \end{array}$$

$$\varepsilon s = \begin{bmatrix} 0.0022 \\ 0.0013 \\ 0.0004 \\ 0.0004 \end{bmatrix}$$

$$fs := \begin{array}{l} \text{for } i \in 1..n \\ \quad \text{if } \varepsilon s_i \ge \varepsilon y \\ \quad\quad fs_i \leftarrow fy \cdot As_i \\ \quad \text{else if } \varepsilon s_i \le \varepsilon y \\ \quad\quad fs_i \leftarrow \varepsilon s_i \cdot Es \cdot As_i \\ \quad fs \end{array}$$

$$Cs := 0$$
$$Ts := 0$$

$$fs = \begin{bmatrix} 1.0345 \cdot 10^{6} \\ 6.37 \cdot 10^{5} \\ 2.1154 \cdot 10^{5} \\ 2.1392 \cdot 10^{5} \end{bmatrix}$$

$$Ts := \begin{array}{l} \text{for } i \in 1..n \\ \quad \text{if } y_i \ge c \\ \quad\quad Ts \leftarrow Ts + fs_i \\ \quad Ts \end{array}$$

$$Ts = 2.1392 \cdot 10^{5} \qquad Cc := 0.85 \cdot f'c \cdot a \cdot b \quad Cc = 1.5518 \cdot 10^{6}$$

$$Cs := \begin{array}{l} \text{for } i \in 1..n \\ \quad \text{if } y_i \le c \\ \quad\quad Cs \leftarrow Cs + fs_i \\ \quad Cs \end{array}$$

$$Cs = 1.883 \cdot 10^{6}$$

$P2 := Cc + Cs - Ts$ $\qquad P2 = 3.2208 \cdot 10^{6}$

$M2 := Cc \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$

$$M2 := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } y_i \le c \\ \quad \left\| M2 \leftarrow M2 + fs_i \cdot \left(\frac{h}{2} - y_i\right) \right. \\ \text{else if } y_i \ge c \\ \quad \left\| M2 \leftarrow M2 + fs_i \cdot \left(y_i - \frac{h}{2}\right) \right. \end{array} \right. \\ M2 \end{array} \right.$$

$M2 = 5.0674 \cdot 10^{8}$ M2 is the moment about the neutral axis at a compression controlled point
$P2 = 3.2208 \cdot 10^{6}$ P2 is the total axial force at a compression controlled point

Let's find the point at the begining of tension controlled Zone where $\varepsilon t = 0.005$:

Guess Values

$c := 100$

Constraints

$0.003 \cdot (h - c) = 0.005 \cdot c$

Solver

$c := \text{find}(c) \qquad c = 187.5$

$c = 187.5$

$$\varepsilon s := \left\| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left\| \begin{array}{l} \text{if } y_i \le c \\ \quad \left\| \varepsilon s_i \leftarrow \frac{(c - y_i) \cdot 0.003}{c} \right. \\ \text{else if } y_i \ge c \\ \quad \left\| \varepsilon s_i \leftarrow \frac{(0.003 \cdot (y_i - c))}{c} \right. \end{array} \right. \\ \varepsilon s \end{array} \right.$$

$$\varepsilon s = \begin{bmatrix} 0.0017 \\ 0.0003 \\ 0.001 \\ 0.0023 \end{bmatrix}$$

$a := \beta 1 \cdot c$ $\quad a = 159.375$

```
fs := for i ∈ 1..n
        if εs_i ≥ εy
           fs_i ← fy·As_i
        else if εs_i ≤ εy
           fs_i ← εs_i·Es·As_i
        fs
```

$$fs = \begin{bmatrix} 8.3151 \cdot 10^5 \\ 1.6945 \cdot 10^5 \\ 4.926 \cdot 10^5 \\ 1.0345 \cdot 10^6 \end{bmatrix}$$

$Cs := 0$ $\quad Ts := 0$

```
Ts := for i ∈ 1..n
        if y_i ≥ c
           Ts ← Ts + fs_i
        Ts
```

$Ts = 1.5271 \cdot 10^6$

$Cc := 0.85 \cdot f'c \cdot a \cdot b$

$Cc = 1.5173 \cdot 10^6$

```
Cs := for i ∈ 1..n
        if y_i ≤ c
           Cs ← Cs + fs_i
        Cs
```

$Cs = 1.001 \cdot 10^6$

$P3 := Cc + Cs - Ts$ $\quad P3 = 9.9115 \cdot 10^5$

$$M3 := Cc \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$$

```
M3 := for i ∈ 1..n
        if y_i ≤ c
           M3 ← M3 + fs_i·(h/2 − y_i)
        else if y_i ≥ c
           M3 ← M3 + fs_i·(y_i − h/2)
        M3
```

$M3 = 4.9923 \cdot 10^{8}$ M3 is the moment about the neutral axis at the begining tension controlled point

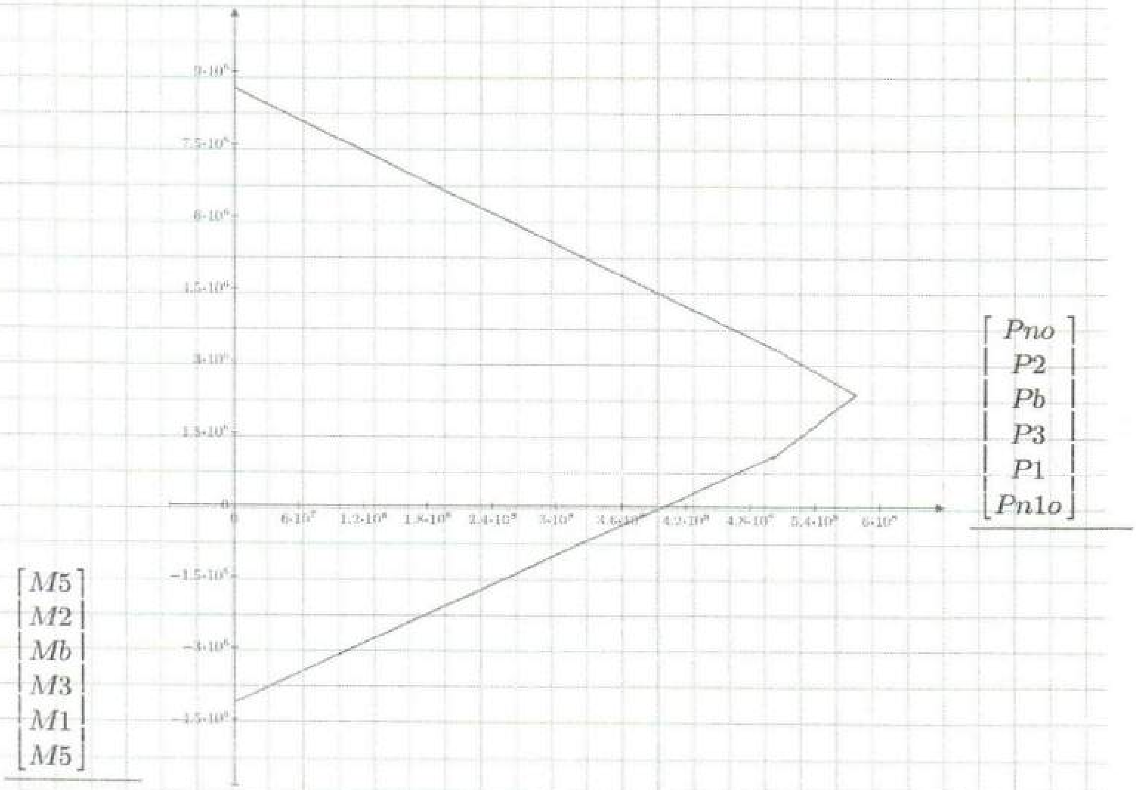
$P3 = 9.9115 \cdot 10^{5}$ P3 is the total axial force at the begining tension controlled point

Now let's plot the interaction diagram based on the points we found :

$M5 := 0$

$$p := \begin{bmatrix} Pno \\ P2 \\ Pb \\ P3 \\ P1 \\ Pn1o \end{bmatrix} \quad p = \begin{bmatrix} 8.6634 \cdot 10^{6} \\ 3.2208 \cdot 10^{6} \\ 2.3112 \cdot 10^{6} \\ 9.9115 \cdot 10^{5} \\ 1.113 \cdot 10^{6} \\ -4.1378 \cdot 10^{6} \end{bmatrix} \quad U := \begin{bmatrix} M5 \\ M2 \\ Mb \\ M3 \\ M1 \\ M5 \end{bmatrix} \quad U = \begin{bmatrix} 0 \\ 5.0674 \cdot 10^{8} \\ 5.7793 \cdot 10^{8} \\ 4.9923 \cdot 10^{8} \\ 5.0715 \cdot 10^{8} \\ 0 \end{bmatrix}$$

interaction diagram



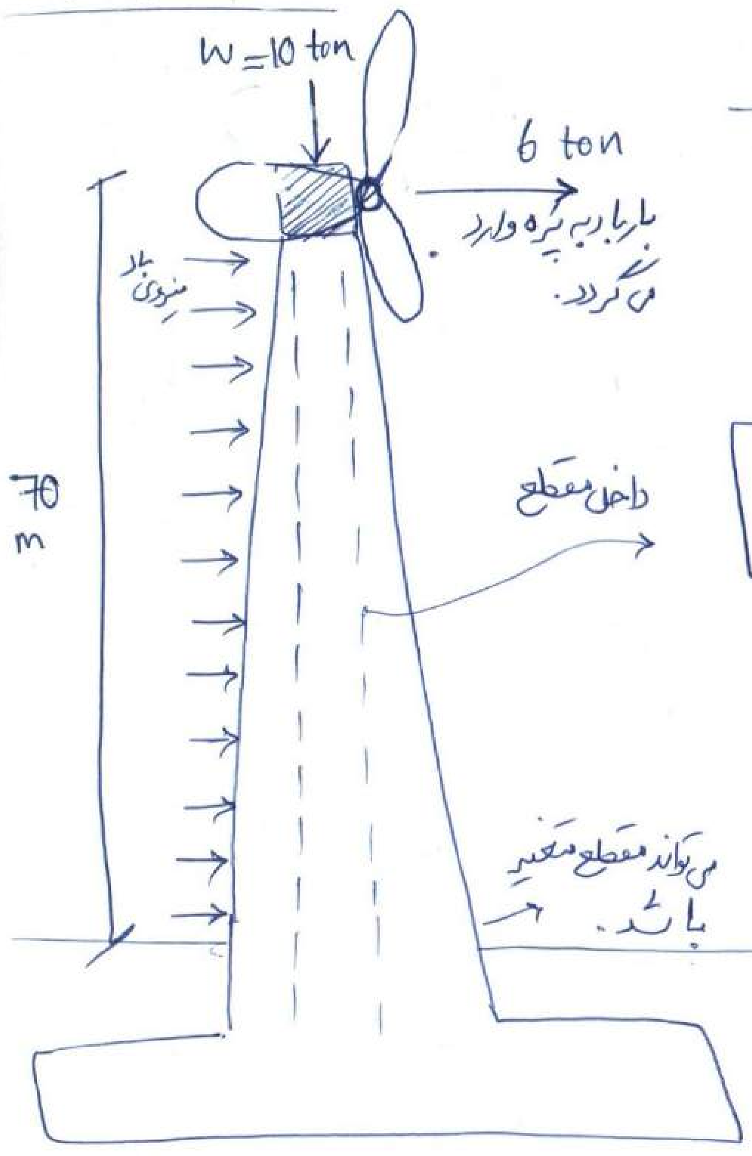
project 2 Deadline 94/4/14 ← شنبه

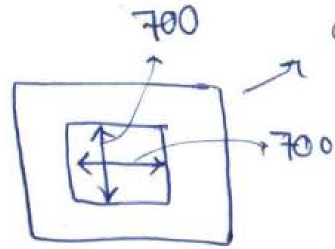
بار باد شهر منجیل

توربین بادی در ارتفاع 50 متر از سطح زمین در دشتهای اطراف شهر منجیل نصب کنیم. پایه بتنی با مقطع دایره‌ای توخالی مورد نظر است. حداقل قطر داخلی 60 سانتی‌متر
وزن توربین و ملحقات آن 15 تن بار وارده به پروانه‌ی توربین در راستای پروانه‌ی 8 تن و عمود بر پروانه‌ی ۳ تن

درب ورودی به ابعاد cm 60 × cm 200

بارگذاری بار باد مبحث ۶





برای بار زلزله نیز می‌توان در نظر گرفت.

۱- بارگذاری را مشخص کنید ← زلزله می‌خواهد یا نه و جهت بار باد

۲- ستون دایره‌ای

۳- فشار باد ← برای محاسبه‌اش می‌شود از یک مقطع مستطیلی معادل رفت.

۴- ستون را در sap مدل کرده و عکس‌العمل تکیه‌گاهی می‌دهیم

۵- با عکس‌العمل تکیه‌گاهی پی را طراحی می‌کنیم.

۶- درب ورودی و خروجی در طراحی ستون لحاظ گردد.
↳ section designer → SD section

۷- Assign → number of output stations $\frac{[illegible]}{5}+1$

برای طراحی پی وزن و لنگر موجود را بدست می‌آوریم.

↳ $q_a = 3$ kg/cm2

بار باد روی بدنه‌ی دکل با مبحث ۶ آیین‌نامه و برای اجزای غیرمسکونی بدست می‌آوریم.

8 – از چه تکنولوژی بتنی استفاده گردد ← نوع بتن ← مقاومت بتن ← نوع آرماتور

۹ – برآورد هزینه ← برآوردن هزینه کل پروژه ← چند کیلو آهن و چه قدر بتن لازم داریم

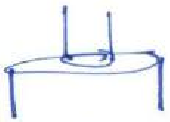
۱۰ – نقشه دستی یا اتوکد

۱۱ – تمامی فرضیات باید مشخص باشد

۱۲ – ضوابط بار باد برای فونداسیون ← آیین نامه ACI

۱۳ – مقاومت خاک ← در صورت نیاز شمع $q_a = 3\ kg/cm^2$

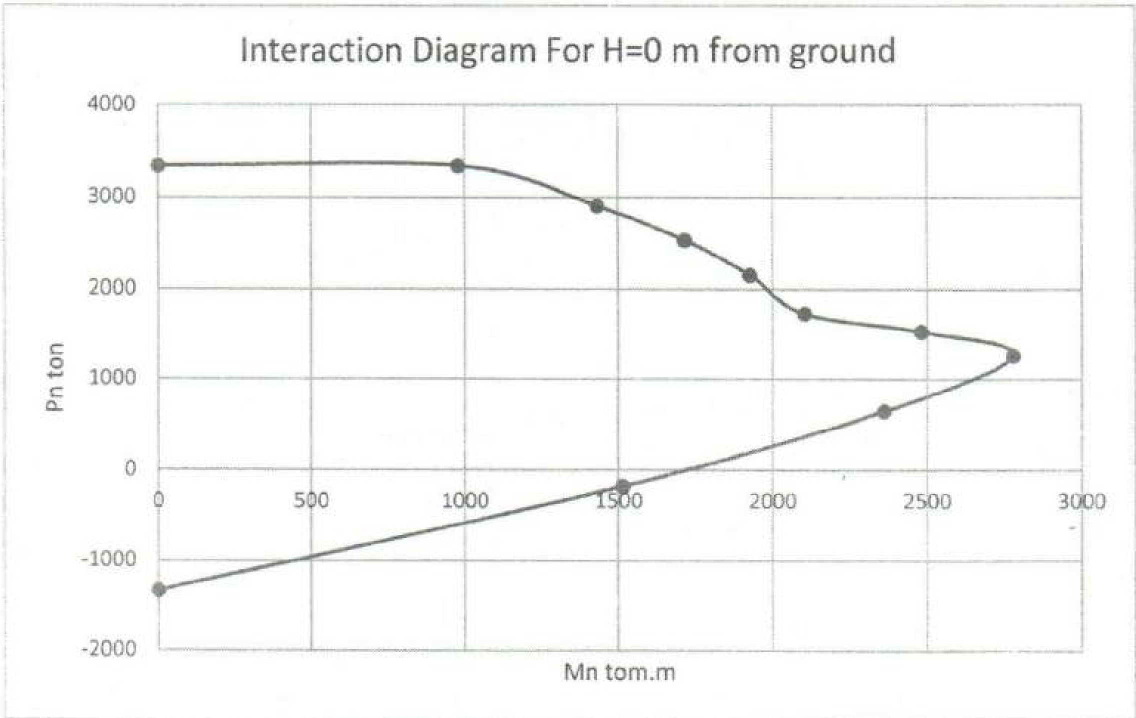
۱۴ – اینکه چند مقطعی طراحی کنیم یا نه



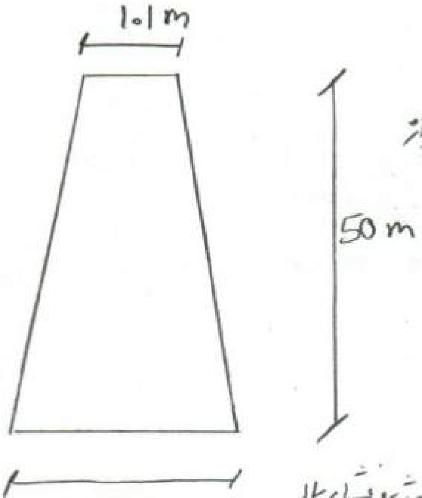
۱۵ – چک کردن موتور را و پره ← بار دینامیکی ← ضربه هنگام شروع چرخش و ایستادن

۱۶ – اثر ارتعاشات روی فونداسیون ← آیین نامه چک شود.

محاسبات مربوط به مقطع دایره توخالی بر تکیه‌گاه



با توجه به بارگذاری باد که در یک طرف سازه به صورت فشار و در طرف دیگر به صورت مکش است یک مقطع متغیر با قطر ۳ متر در تکیه‌گاه تا ۱.۱ متر در بالای ستون پیشنهاد داریم و به کمک نرم افزار sap در فواصل 5 متر - 5 متر مقطع زده و پایداری مقطع را بررسی کردیم.



مرکز سطح توزیع بار گسترده UL $\quad \bar{y} = \frac{h}{3}\left(\frac{2a+b}{a+b}\right) = \frac{50}{3}\left(\frac{2\times3+1.1}{3+1.1}\right) = 28.86\ m$

$\bar{y}$ از پایین $= 21.14\ m$

$$M_w = \left(\frac{3+1.1}{2}\right)\times 50 \times (\overset{\text{فشار}}{0.2859} + \overset{\text{مکش}}{0.1502}) \times 21.14 = 944.96\ ton.m$$

به دلیل مکش و فشار باد

به دلیل دینامیکی بودن

$$M_L = 2\times3\times50 = 300\ ton.m$$

$$M_u = 1.4 M_w + M_L = 1622.94\ ton.m < \phi M_n \approx 1700\ ton.m$$

OK

مقطع مورد استفاده ی ما در قاعده یک دایره ی توخالی به قطر خارجی 3 متر و ضخامت جداره ی 25 سانتیمتر است و از دو ردیف میلگرد استفاده کردیم که جزئیات آن در ادامه موجود است.

کاور خارجی بتن = 50 mm ← تماس با آب و هوا

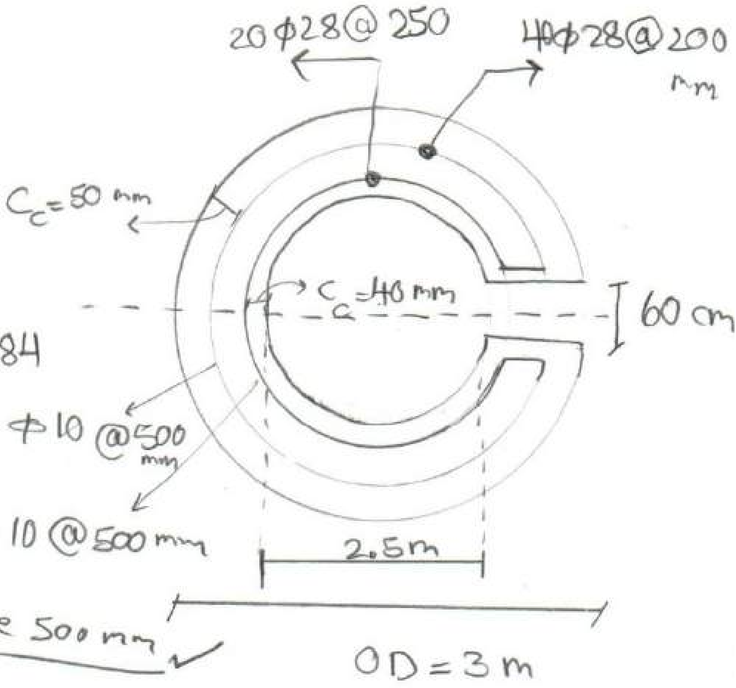
کاور داخلی بتن = 50 mm ← بدون تماس با آب و هوا

$$S_{Min} = Max\left[\overset{28}{d_b}, 25mm, \overset{34}{1.33 D_{max}}\right] = 34\ mm$$

$$S_{max} = Min\left[380\left(\frac{280}{f_s}\right) - 2.5C_c, 300\left(\frac{250}{f_s}\right)\right] = 284$$

$$S_{Min} < S_{provided} < S_{max}$$

خاموت $S_{max} = Min\left(\frac{d}{2}, 600\,mm\right) = 600\ mm \rightarrow$ use 500 mm ✓



با توجه به این نکته که توربین های بادی همواره در جهت باد غالب قرار می گیرند و باد در شهر منجیل در جهت شمالی و شمال غربی می وزد بنابراین می توان درب ورودی پایین توربین بادی را در جهت شرقی - غربی قرار داد تا حداقل تنش روی آن ناحیه قرار گیرد و روی محور خنثی باشد. (اگرچه مقطع اثر در به دلیل کوچک بودن ابعاد آن نسبت به قطر کل مقطع قابل صرفنظر کردن است.)

$$V_u = 1.4V_w + V_L \quad \begin{cases} V_W = \left(\frac{3+1.1}{2}\right)\times 50 \times (0.2859 + 0.1502) = 44.7\ ton \\ V_L = 3\ ton \end{cases}$$

دینامیکی

$$V_u = 1.4 \times 44.7 + 3\times 2 = 68.58$$

$$V_c = 0.17\sqrt{f'_c}\, b_w d \cong 0.17\sqrt{f'_c}\, A_c = 0.17\sqrt{25}\times \pi\frac{(3000^2 - 2500^2)}{4} = 183.5\ ton$$

$$V_u \lesssim \frac{\phi V_c}{2} = \frac{0.75\times 183.5}{2} = 68.5$$ → no need to shear reinf but we provide 2Φ10 @ 50 cm ✓

$$L_d = \left(\frac{f_y}{1.1\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right)d_b \geq 300$$

داخل بتن

$\psi_t = \psi_e = \psi_s = 1$

$K_{tr} = 0$

$c_b = 50$ mm $\quad d_b = 28$ m

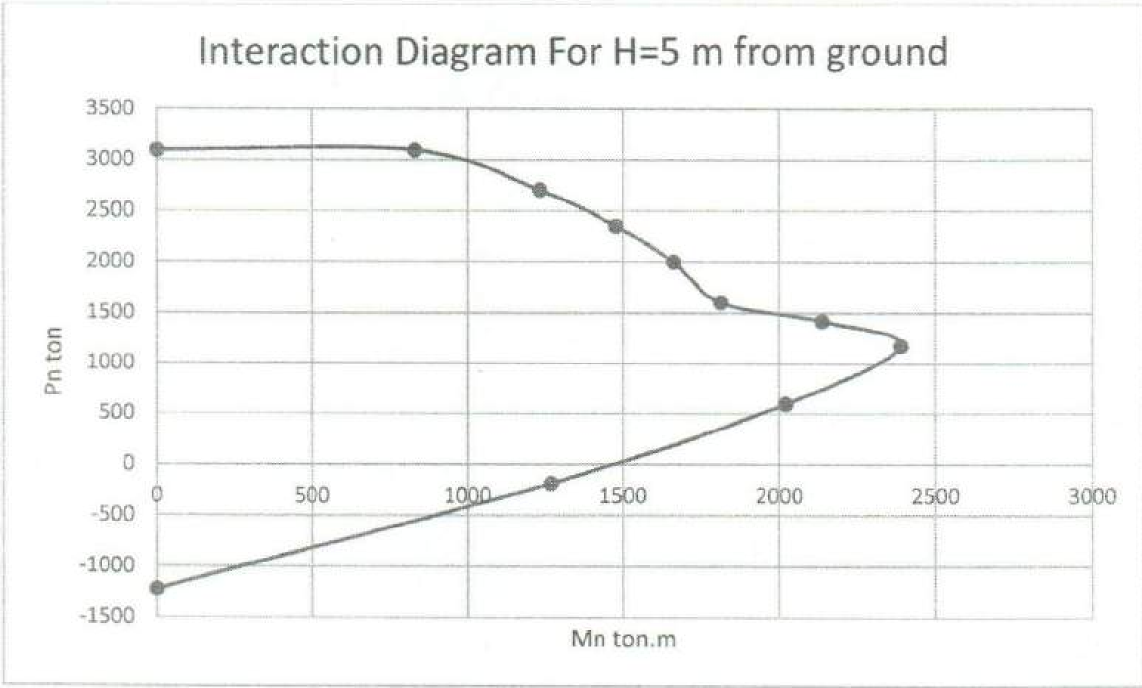
$$L_d = \left(\frac{400}{1.1\times 1 \times\sqrt{25}}\frac{1}{\left(\frac{50}{28}\right)}\right)d_b = 40.7 d_b = 1140\ mm$$ → عمق می باید از این مقدار بیشتر باشد.

$$L_s = 0.071 f_y d_b = 0.71\times 400 \times 28 = 795.2$$ →

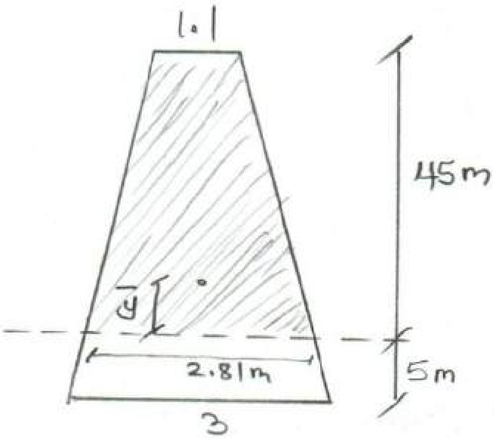
از آنجایی که در تمامی مقاطع از Φ28 استفاده شده است طول وصله 80 cm ای نیاز است.

V

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۵ متری از بر تکیه‌گاه



در ارتفاع 5 متری دیگر بازشو برای درب ورودی وجود ندارد و مقطع دایره‌ای توخالی بدون تکلف است.



$\bar{y}$ از بالا $= \frac{45}{3}\left(\frac{2\times 2.81+1.1}{2.81+1.1}\right) = 25.78$

$\bar{y}$ از پایین $= 19.21$

$$M_w = \left(\frac{2.81+1.1}{2}\right)\times 45\times(0.2859+0.1502)\times 19.21 = 737.01 \text{ ton.m}$$

$$M_L = 2\times 3\times 45 = 270$$

$$M_u = 1.4\,M_w + M_L = 1301.814 \text{ ton.m} < \phi M_n \approx 1500 \text{ ton.m}$$

OK

در این قسمت مقطع مورد استفاده دارای قطر خارجی 2.81 متر و ضخامت 25 cm است از دو ردیف میلگرد که لایه‌ی بیرونی 35 عدد و لایه داخلی 20 عدد φ28 است استفاده شده است.

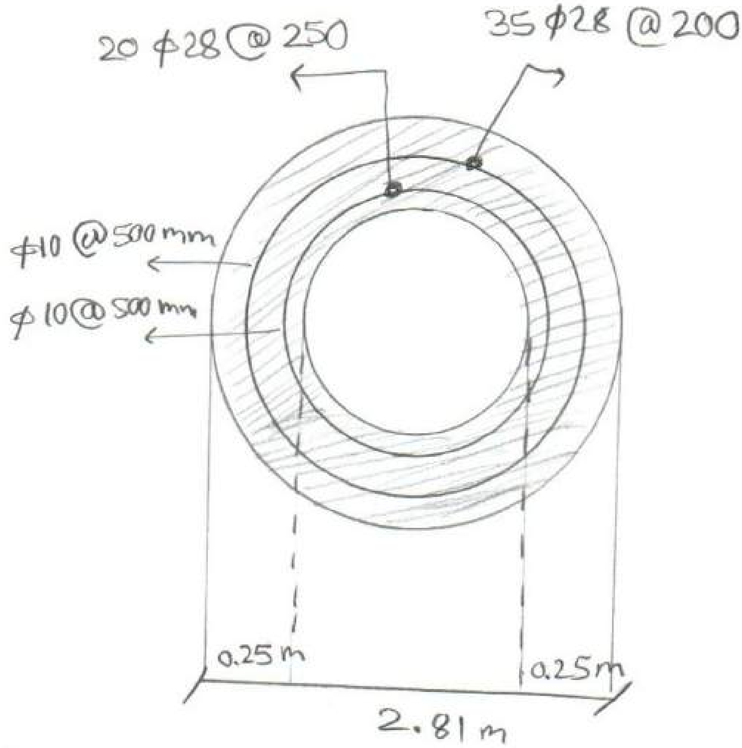
غطاء خارجي = 50 mm

غطاء داخلي = 50 mm

$S_{Min} = 34$ mm

$S_{Max} = 284$ mm

$S_{Min} < S_{provided} < S_{Max}$



Check Shear

$V_u = 1.4 V_w + V_L$

طأ   أساس

$V_w = \left(\frac{2.81+1.1}{2}\right) \times 45 \times (0.2859 + 0.1502) = 38.36$ ton

$V_L = 3$ ton   معامل أمان

$V_u = 1.4 \times 38.36 + 2 \times 3 = 59.71$ ton

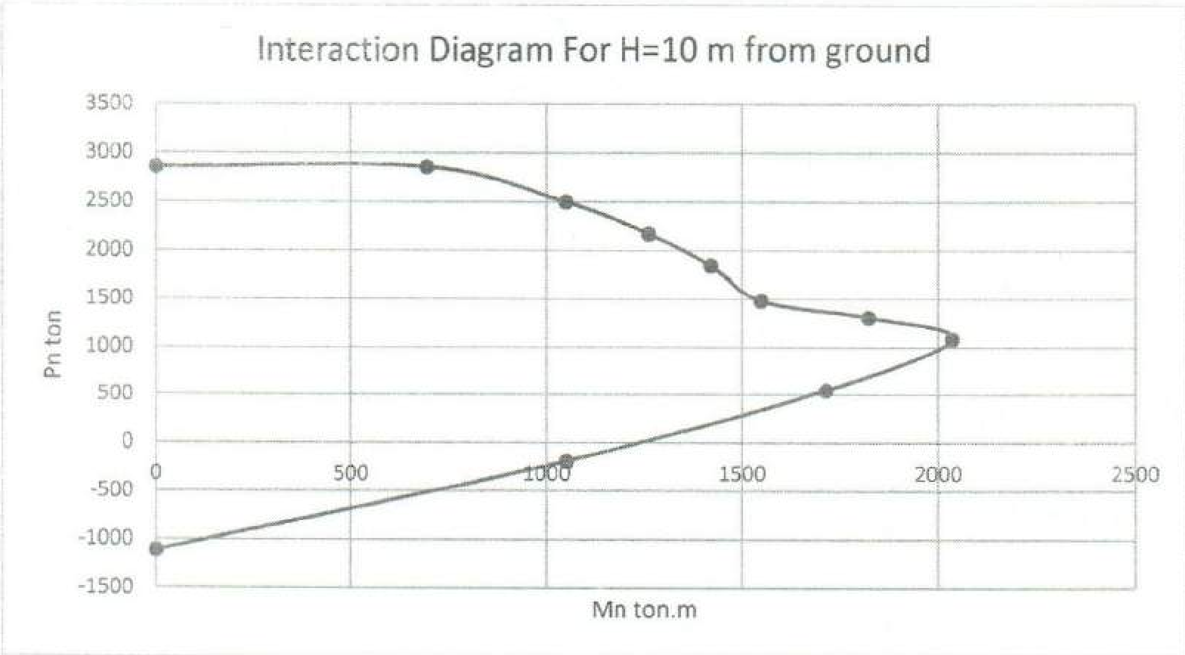
$V_c = 0.17\sqrt{25} \times \pi \times \frac{(2810^2 - 2310^2)}{4} = 170.9$ ton $\frac{\phi V_c}{2} = 64$ ton

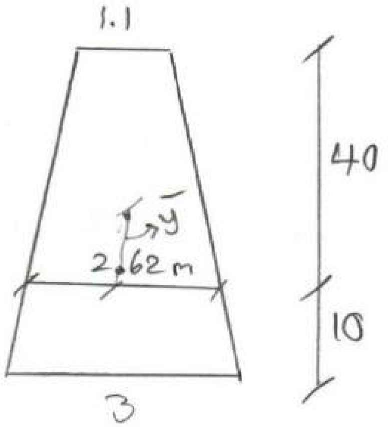
$V_u < \frac{\phi V_c}{2} \rightarrow$ no need to shear reinf but provid $2\phi 10 @ 50$ cm

$L_s = 80$ cm

$L_d = 1140$ mm ≈ 115 cm

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۱۰ متری از بر تکیه‌گاه





$$\bar{y}\ \text{از بالا} = \frac{40}{3}\left(\frac{2\times2.62+1.1}{2.62+1.1}\right) = 22.72$$

$$\bar{y} = 17.27$$

$$M_w = \left(\frac{2.62+1.1}{2}\right)\times40\times(0.2859+0.1502)\times17.27 = 560.33\ \text{ton.m}$$

$$M_L = 2\times3\times40 = 240\ \text{ton.m}$$

$$M_u = 1.4M_w + M_L = 1.4\times560.33+240 = 1024 < \phi M_n = 1200\ \text{ton.m}$$

✓ OK

در این قسمت نیز مقطع دارای قطر خارجی 2.62 m و ضخامت 25 cm است. از دو ردیف میلگرد با آرایش دایره‌ای که لایه‌ی بیرونی 30 عدد و لایه‌ی داخلی 20 عدد Φ28 است استفاده شده است.

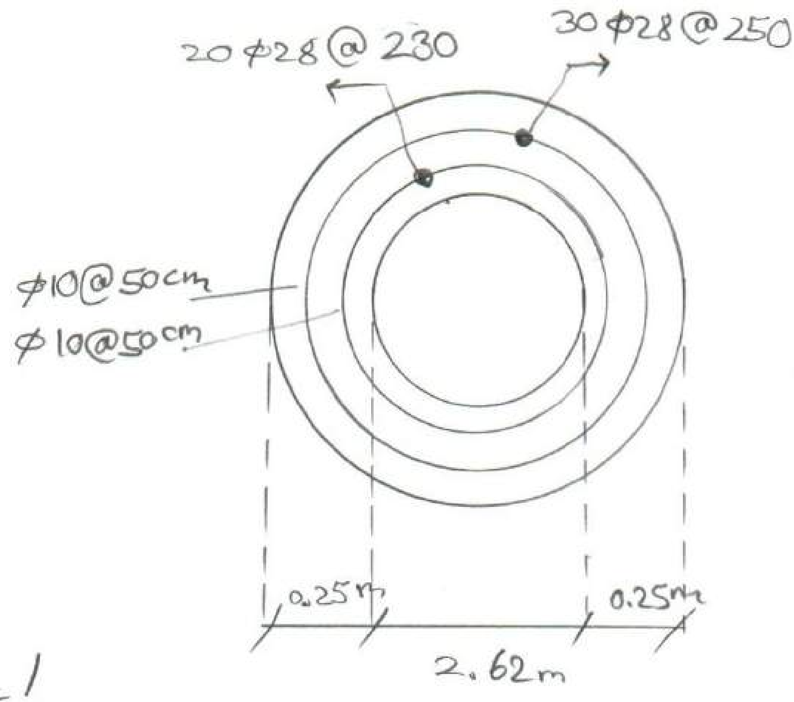
کاور خارجی $= 50$ mm

کاور داخلی $= 40$ mm

$S_{min} = 34$ mm

$S_{max} = 284$ mm

$S_{min} < S < S_{max}$



---

check shear

$V_u = 1.4 V_w + V_L$

فشار مکش

$V_w = \left(\frac{2.62+1.1}{2}\right) \times 40 \times (0.2859 + 0.1502) = 32.45$ ton

$V_L = 3$ ton

دینامیکی اثرات

$V_u = 1.4 \times 32.45 + 2 \times 3 = 51.43$ ton

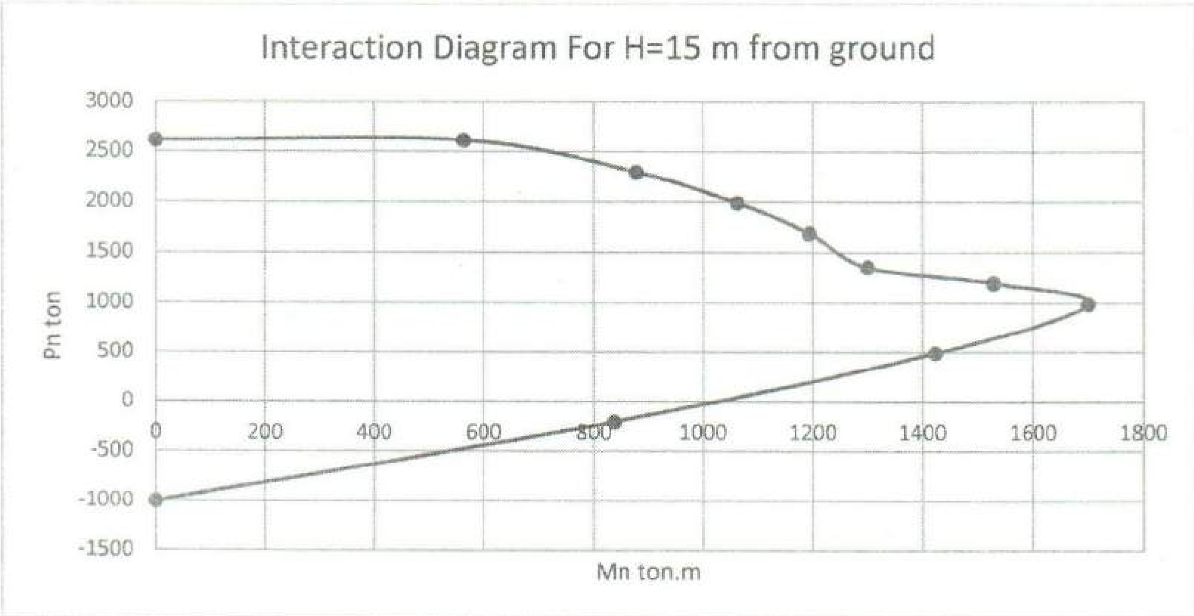
$V_c = 0.17 \times \sqrt{25} \times \pi \times \left(\frac{2620^2 - 2120^2}{4}\right) = 158.22$ ton $\quad \frac{\phi V_c}{2} = 59.33$

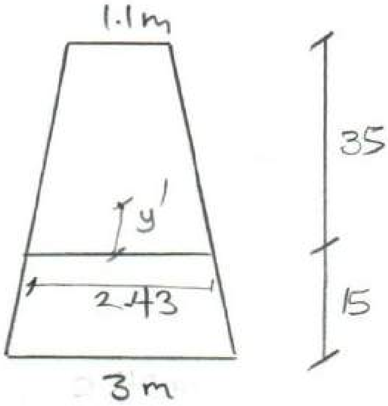
$V_u < \frac{\phi V_c}{2} \longrightarrow$ no need to shear reinf but provide 2Φ10 @ 50 cm

$L_s = 80$ cm

$L_d = 1140$ mm $= 115$ cm

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۱۵ متری از بر تکیه‌گاه





$\bar{y}$ از بالا $= \frac{35}{3}\left(\frac{2\times2.43+1.1}{2.43+1.1}\right) = 19.69$

$\bar{y} = 15.3$

فشار   کشش

$M_W = \left(\frac{2.43+1.1}{2}\right)\times35\times(0.2859+0.1502)\times15.3 = 386.66$ ton.m

$M_L = 2\times3\times35 = 210$ ton.m

$M_u = 1.4M_W + M_L = 1.4\times386.66+210 = 751.3$ ton.m

$< \phi M_n \cong 1000$ ton.m

در این قسمت از مقطع دارای قطر خارجی 2.43 متر و ضخامت 25 cm استفاده شده است. از دو لایه آرماتور با آرایش دایره‌ای به تعداد 30 عدد $\phi28$ در بیرون و 15 عدد $\phi28$ در داخل استفاده شده است.

جزئیات در صفحه‌ی بعدی

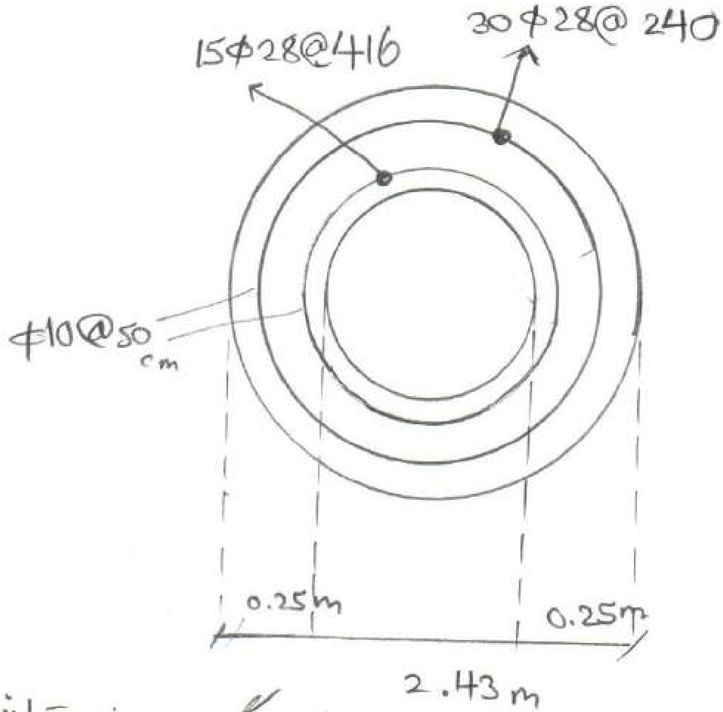
کاور خارجی $= 50\ mm$

کاور داخلی $= 40\ mm$

$S_{Min} = 34\ mm$

$S_{Max} = 284\ mm$

$S_{Min} < S < S_{Max}$ OK ✓



check shear

$V_u = 1.4 V_W + V_L$

آرماتورهای لایه‌ی داخلی بیشتر نقش نگه‌دارنده‌ی خاموت دارند / فشار
بنابراین رعایت نشدن حداکثر فاصله‌ی آنها مشکل چندانی ندارد

$V_W = \frac{(2.43+1.1)}{2} \times 35 \times (0.2859 + 0.1502) = 26.94\ ton$

$V_L = 3\ ton$

دینامیکی است

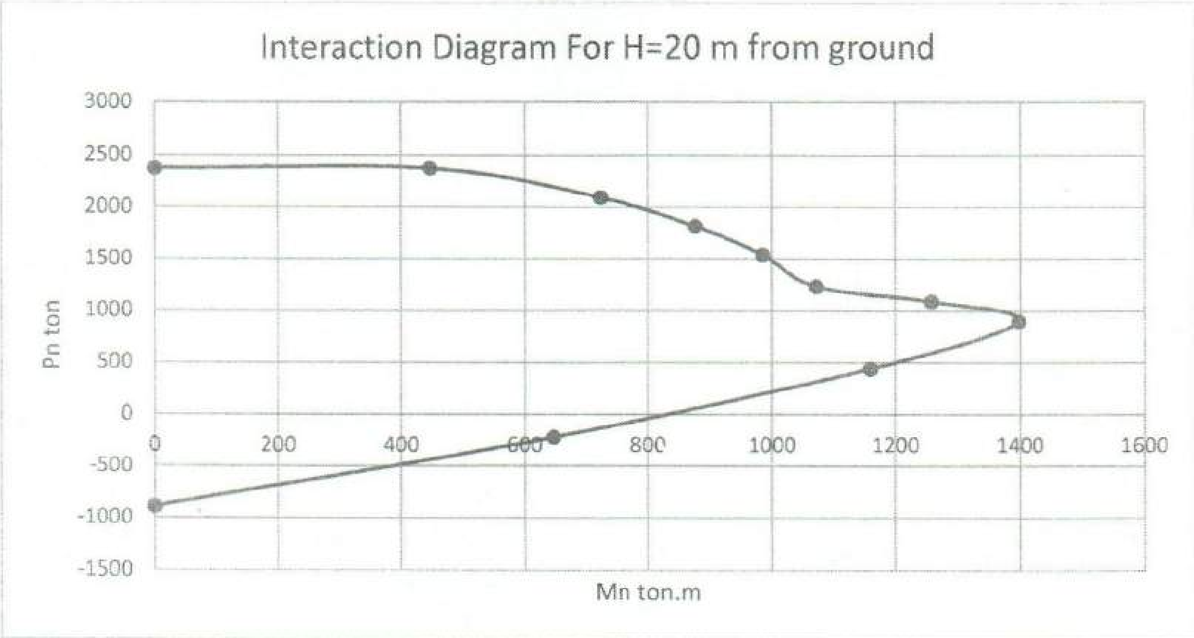
$V_u = 1.4 \times 26.94 + (2 \times 3) = 43.71\ ton$

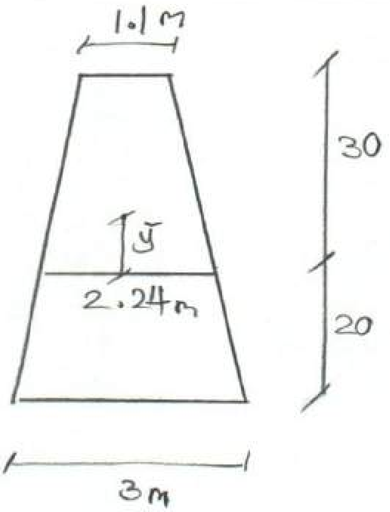
$V_c = 0.17\sqrt{25} \times \pi \times \left(\frac{2430^2 - 1930^2}{4}\right) = 145.5\ ton \qquad \frac{\phi V_c}{2} = 54.57\ ton$

$V_u < \frac{\phi V_c}{2} \longrightarrow$ No need to shear reinf $\longrightarrow$ only use $\phi 10$ @50 cm

$$\begin{cases} L_S = 80\ cm \\ L_d = 115\ cm \end{cases}$$

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۲۰ متری از بر تکیه‌گاه





$$\bar{y}_{\text{از بالا}} = \frac{30}{3}\left(\frac{2\times 2.24+1.1}{2.24+1.1}\right) = 16.71$$

$$\bar{y} = 13.29$$

$$M_w = \left(\frac{2.24+1.1}{2}\right)\times 30\times(0.2859+0.1502)\times 13.29 = 272.39 \text{ ton.m}$$

$$M_L = 2\times 3\times 30 = 180 \text{ ton.m}$$

$$M_u = 1.4M_w + M_L = 1.4\times 272.39+180 = 561.346 < \phi M_n \cong 800 \text{ ton.m}$$

✓ OK

این قسمت از مقطع به قطر خارجی 2.24 متر و ضخامت 25 cm استفاده شده است. همچنین از دولایه آرماتور با آرایش دایره‌ای که لایه‌ی بیرونی دارای تعداد 25 عدد $\phi 28$ و لایه داخلی 25 عدد $\phi 28$ است تشکیل شده است.

جزئیات در صفحه‌ی بعدی

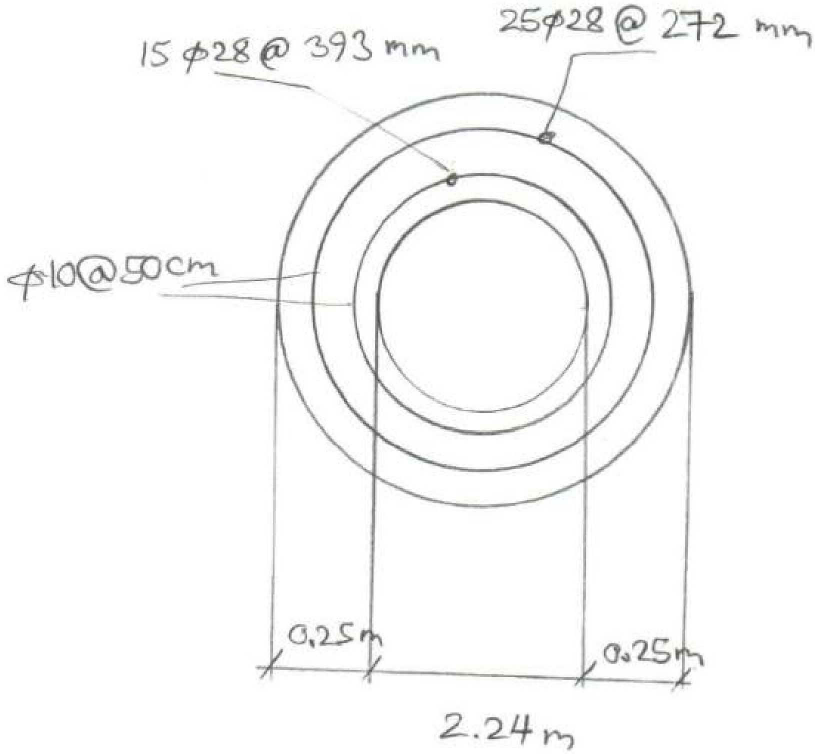
کاور خارجی $= 50$ mm

کاور داخلی $= 40$ mm

$S_{Min} = 34$ mm

$S_{Max} = 284$ mm

$S_{Min} < S < S_{max}$



---

check shear

$V_u = 1.4 V_w + V_L$

$V_w = \frac{(2.24 + 1.1)}{2} \times 30 \times (0.2859 + 0.1502) = 21.85$ ton

$V_L = 3$ ton

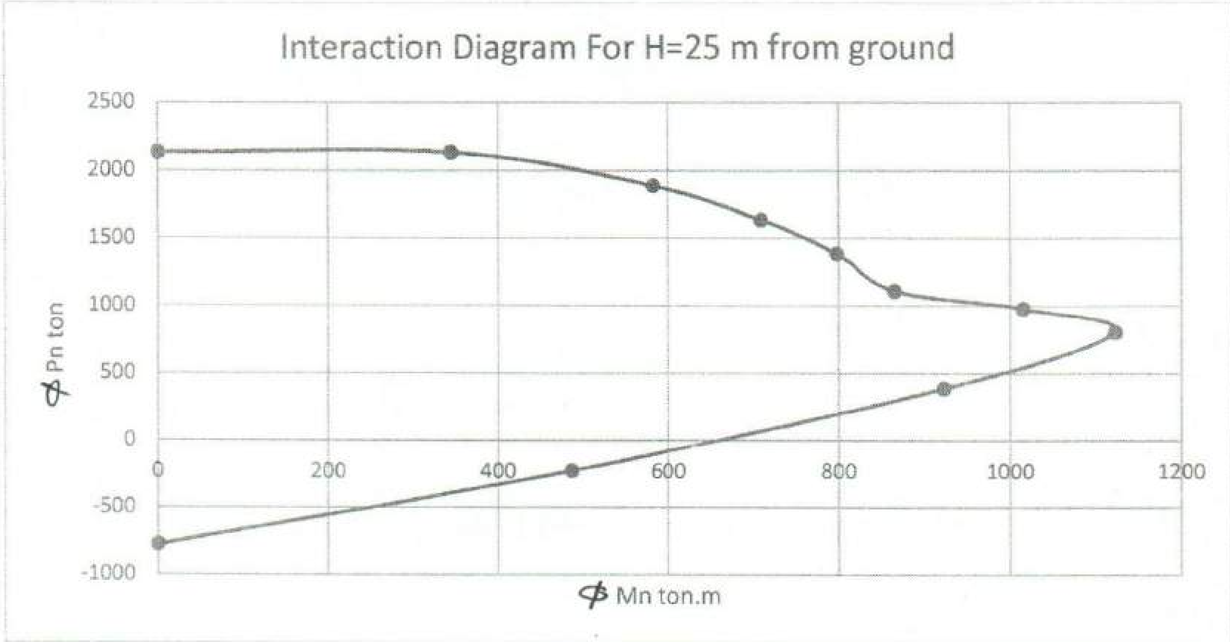
$V_u = 1.4 \times 21.85 + \underbrace{(2 \times 3)}_{\text{اثر ضربه است}} = 36.58$ ton

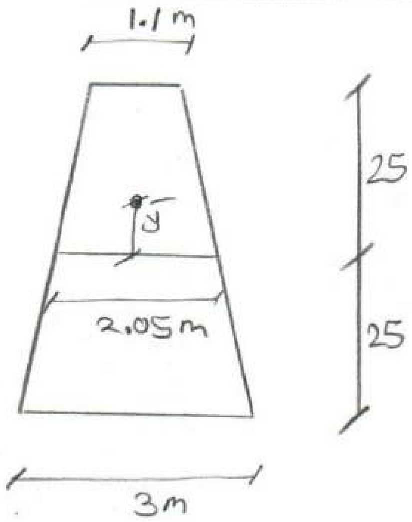
$V_c = 0.17\sqrt{25} \times \pi \times \left(\frac{2240^2 - 1740^2}{4}\right) = 132.85$ ton $\quad \frac{\phi V_c}{2} = 49.8 > V_u$

no need to shear reinf

$\begin{cases} L_s = 80 \text{ cm} \\ L_d = 115 \text{ cm} \end{cases}$

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۲۵ متری از بر تکیه‌گاه





از بالا $\bar{y} = \frac{25}{3}\left(\frac{2\times 2.05+1.1}{2.05+1.1}\right) = 13.75$

$\bar{y} = 11.24$

$M_w = \left(\frac{2.05+1.1}{2}\right)\times 25\times(0.2859+0.1502)\times 11.24 = 143.01$ ton.m

$M_L = 2\times 3\times 25 = 150$ ton.m

$M_u = 1.4 M_w + M_L = 1.4\times 193.01 + 150 = 420.2$ ton.m $< \phi M_n$

$= 620$ ton.m

این قسمت، از مقطع به قطر خارجی 2.05 متر و ضخامت 25 cm استفاده شده است. همچنین از دو لایه ی آرماتور که لایه بیرون 20 عدد φ28 و لایه داخلی 15 عدد φ28 است استفاده شده است.

جزئیات در صفحه ی بعدی

کاور خارجی $= 50$ mm

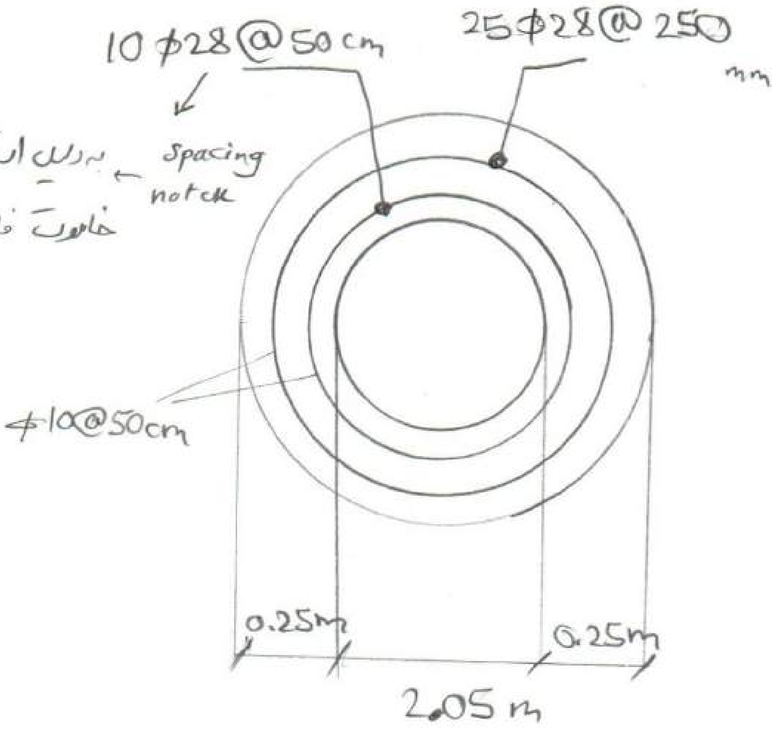
کاور داخلی $= 40$ mm

$S_{Min} = 34$ mm

$S_{Max} = 284$ mm

$$S_{Min} < \underset{250\ mm}{\underset{\parallel}{S}} < S_{Max}$$

به دلیل استفاده برای نگهداری خاموت فاصله تاثیر زیادی ندارد.



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check shear

$$V_u = 1.4V_W + V_L$$

$$V_W = \frac{(2.05+1.1)}{2} \times 25 \times (0.2859 + 0.1502) = 17.17 \text{ ton}$$

$$V_L = 3 \text{ ton}$$

[illegible]

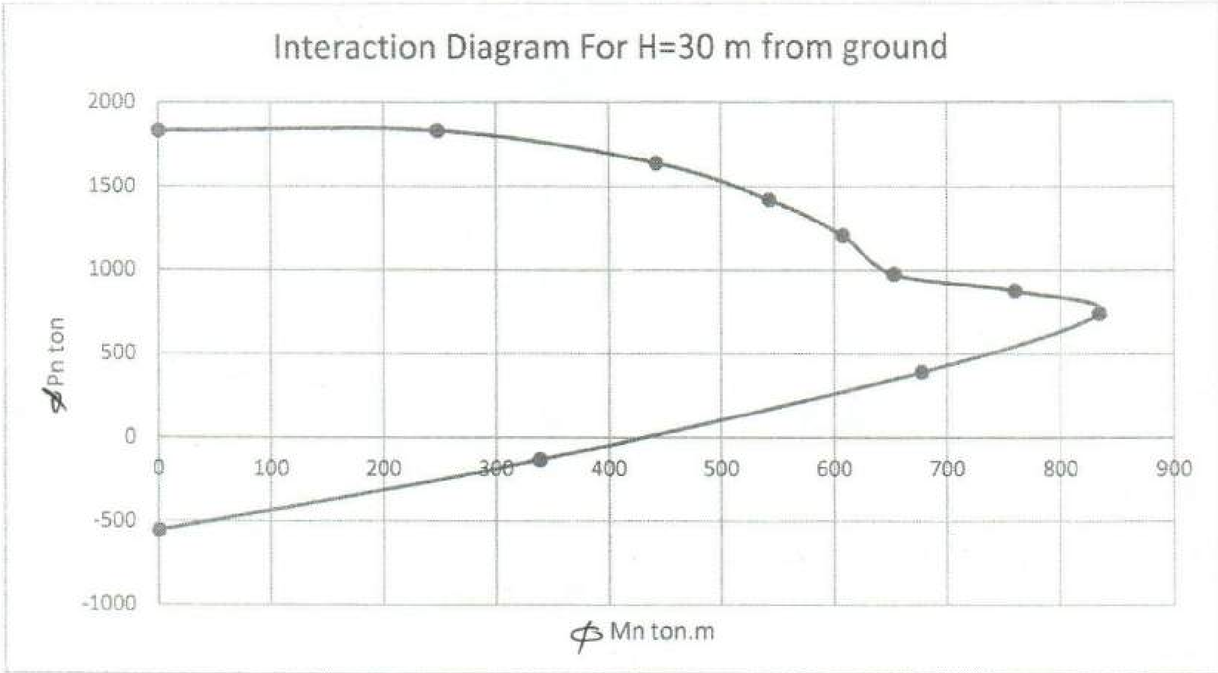
$$V_u = 1.4 \times 17.17 + (3 \times 2) = 30.04 \text{ ton}$$

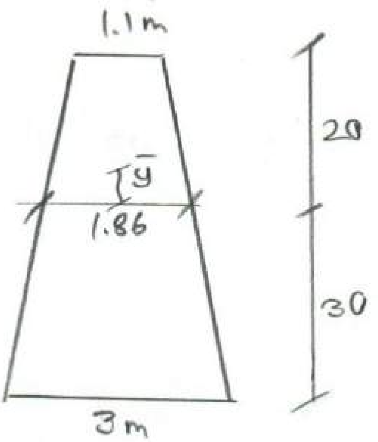
$$V_c = 0.17\sqrt{25} \times \pi \frac{(2050^2 - 1550^2)}{4} = 120.17 \text{ ton} \qquad \frac{\phi V_c}{2} = 45.06 > V_u$$

no need to shear reinf

$$\begin{cases} L_s = 80 \text{ cm} \\ L_d = 115 \text{ cm} \end{cases}$$

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۳۰ متری از بر تکیه‌گاه





$$\bar{y}\ \text{از بالا} = \frac{20}{3}\left(\frac{2\times 1.86+1.1}{1.86+1.1}\right) = 10.85\ m$$

$$\bar{y} = 9.14\ m$$

$$M_w = \left(\frac{1.86+1.1}{2}\right)\times 20\times(0.2859+0.1502)\times 9.14 = 117.98\ \text{ton.m}$$

$$M_L = 2\times 3\times 20 = 120\ \text{ton.m}$$

دینامیکی است.

$$M_u = 1.4M_w + M_L = 1.4\times 117.98 + 120 = 285.18\ \text{ton.m}$$

$$< \phi M_n \simeq 420\ \text{ton.m}$$

✓ ok

در این قسمت از مقطع به قطر خارجی 1.86 m و ضخامت 25 cm استفاده شده است. از دو لایه‌ی میلگرد که لایه‌ی خارجی از 20 عدد φ28 و لایه‌ی داخلی از 5 عدد φ28 تشکیل شده است.

جزئیات در صفحه‌ی بعدی

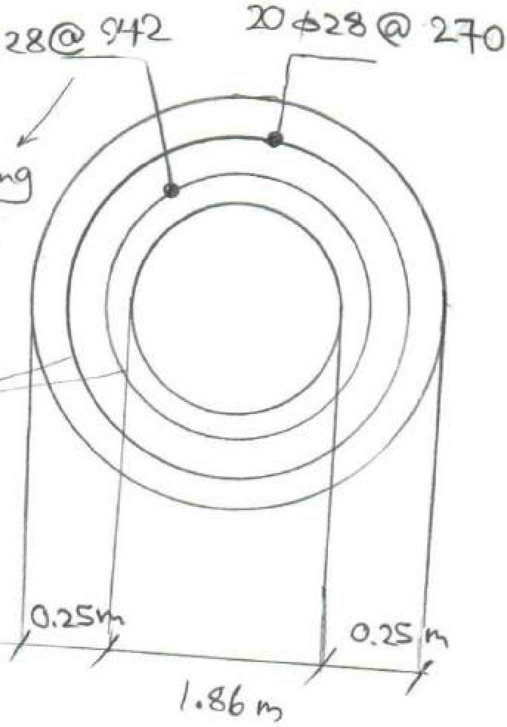
کاور خارجی $= 50\ mm$

کاور داخلی $= 40\ mm$

$S_{min} = 34\ mm$

$S_{max} = 284\ mm$

spacing رعایت نشده است اما خیلی مهم نیست، بیشتر نقش نگهدارندگی آرماتور خاموت را دارد.



Check shear

$V_u = 1.4V_w + V_L$

$V_w = \left(\frac{1.86+1.1}{2}\right)\times 20\times(0.2859+0.1502) = 12.91\ ton$

$V_L = 3\ ton$

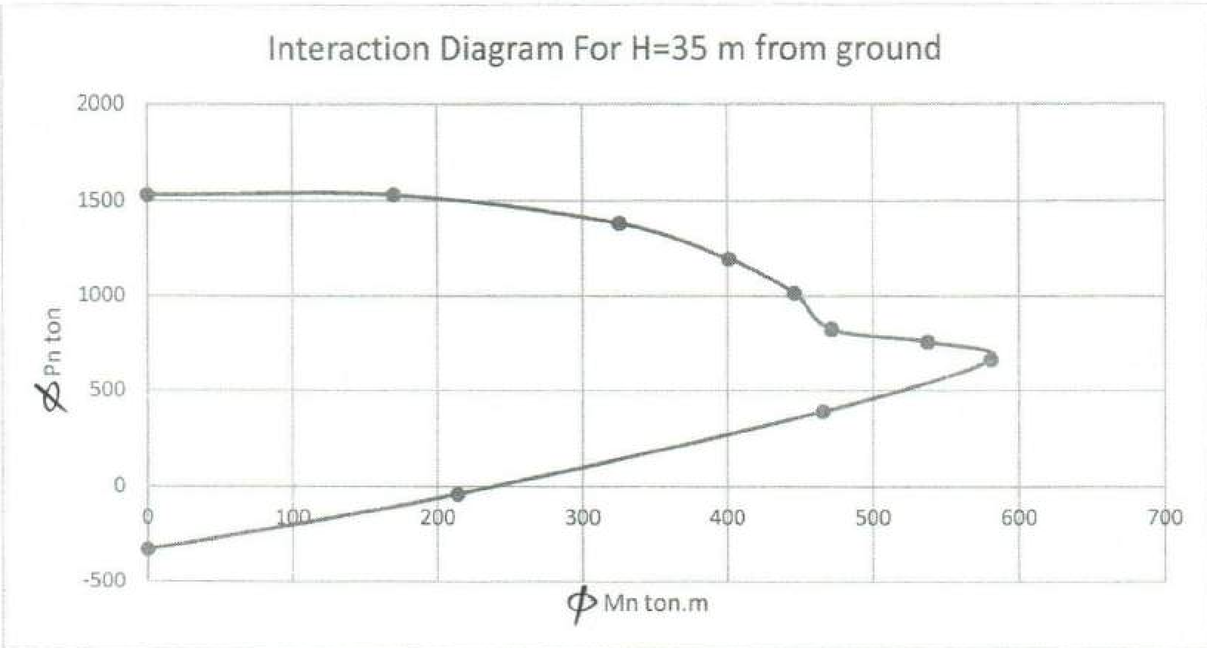
$V_c = 0.17\sqrt{25}\times\pi\times\left(\frac{1860^2-1360^2}{4}\right) = 107.48\ ton \qquad \frac{\phi V_c}{2} = 40.3$

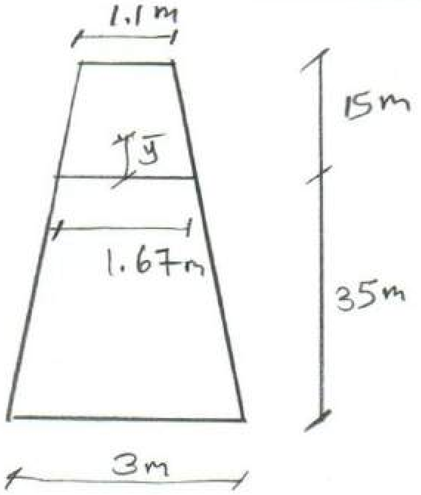
$V_u = 1.4\times 12.91 + (2\times 3) = 24.074 < \frac{\phi V_c}{2} \longrightarrow$ no need to shear reinf

رنا خیلی

$\begin{cases} L_s = 80\ cm \\ L_d = 115\ cm \end{cases}$

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۳۵ متری از بر تکیه‌گاه





$\bar{y}$ از بالا $= \frac{15}{3}\left(\frac{2\times1.67+1.1}{1.67+1.1}\right) = 8.014$

$\bar{y} = 6.985$

$M_w = \left(\frac{1.67+1.1}{2}\right)\times15\times(0.2859+0.1502)\times6.985 = 59.7$ ton.m

$M_L = 2\times3\times15 = 90$ ton.m

$M_u = 1.4M_w + M_L = 59.7\times1.4 + 90 = 173.58$ ton.m

$< \phi M_n = 230$ ton.m

در این قسمت از مقطع به قطر خارجی 1.67 متر و ضخامت 25 سانتی‌متر استفاده شده است. از دو لایه میلگرد که لایه‌ی خارجی 20 عدد $\phi28$ و داخلی 5 عدد $\phi28$ است نیز به عنوان آرماتور استفاده شده است.

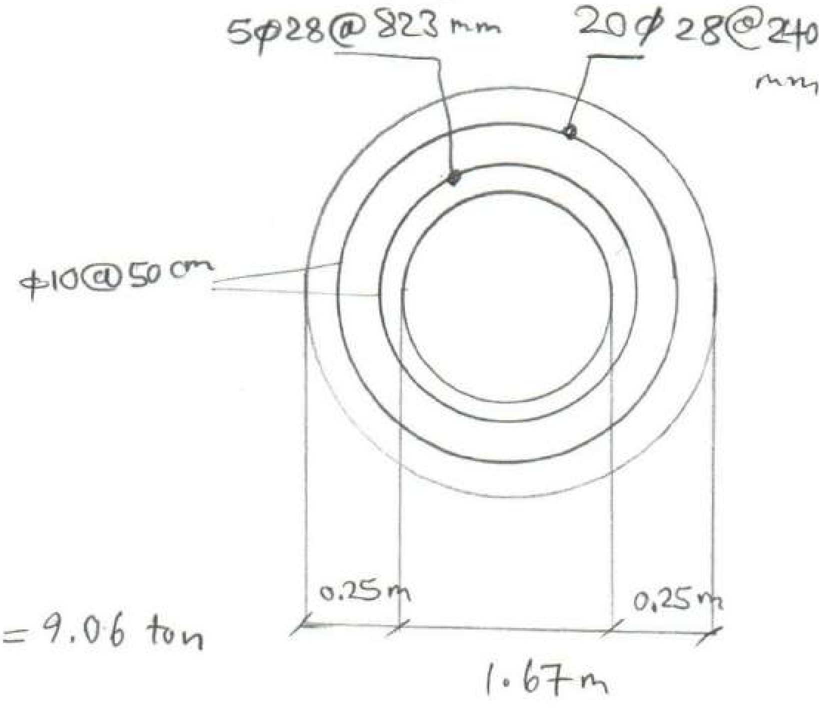
جزئیات در صفحه‌ی بعدی

کاور خارجی $= 50$ mm

کاور داخلی $= 40$ mm

$\delta_{min} = 34$ mm

$\delta_{max} = 284$ mm



---

check shear

$$V_u = 1.4 V_w + V_L$$

$$V_w = \frac{(1.67+1.1)}{2} \times 15 \times (0.2859+0.1502) = 9.06 \text{ ton}$$

$$V_L = 3 \text{ ton}$$

$$V_c = 0.17\sqrt{25} \times \pi \times \left(\frac{1670^2 - 1170^2}{4} \right) = 94.8 \qquad \frac{\phi V_c}{2} = 35.55 \text{ ton}$$

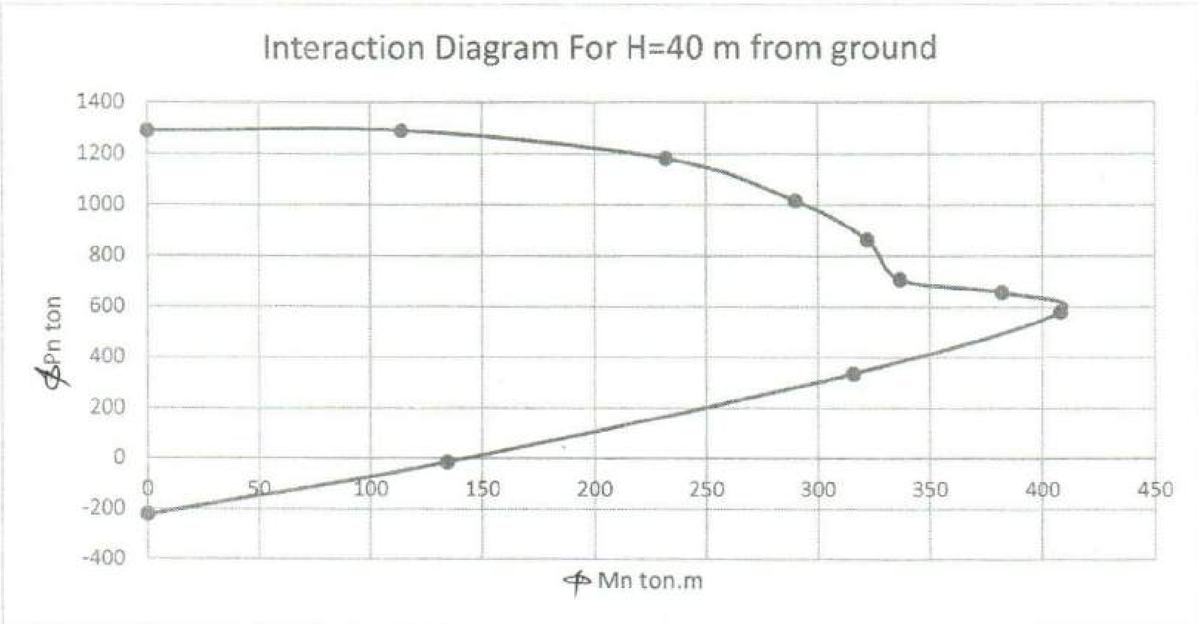
زن کل

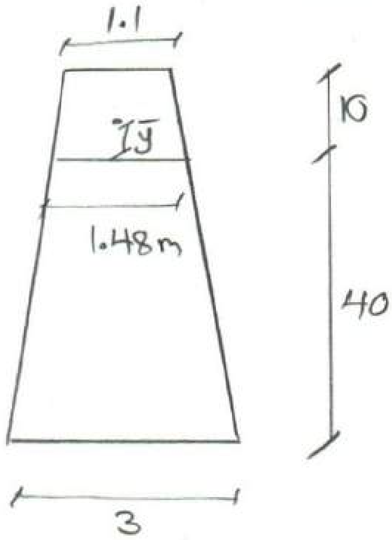
$$V_u = 1.4 \times 9.06 + (2 \times 3) = 18.7 \text{ ton} < \frac{\phi V}{2} \rightarrow$$ no need to shear reinf

just use 2φ10 @ 50 cm

$$\begin{cases} L_s = 80 \text{ cm} \\ L_d = 115 \text{ cm} \end{cases}$$

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۴۰ متری از بر تکیه‌گاه





$$\bar{y}_{\text{از بالا}} = \frac{10}{3}\left(\frac{2\times1.48+1.1}{1.48+1.1}\right) = 5.245$$

$$\bar{y} = 4.7545\ m$$

$$M_w = \frac{(1.48+1.1)}{2}\times 10\times(0.2859+0.1502)\times 4.76 = 25.12\ ton.m$$

$$M_L = 2\times3\times10 = 60\ ton.m$$

$$M_u = 1.4M_w + M_L = 1.4\times25.12+60 = 95.168\ ton.m$$

$$< \phi M_n = 140\ ton.m$$

در این مقطع از دایره‌ای توخالی به قطر خارجی 1.48 و ضخامت 25 سانتی‌متر استفاده شده است. همچنین از دو لایه میلگرد که لایه‌ی خارجی دارای 5 عدد و داخلی 5 عدد $\phi28$ است استفاده شده است.

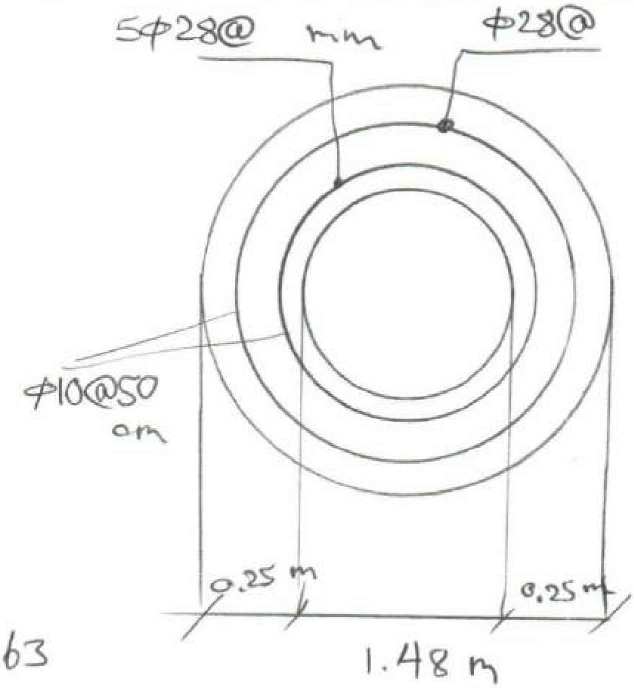
جزئیات در صفحه‌ی بعدی

کاورخارجی $= 50$ mm

کاورداخلی $= 40$ mm

$S_{min} = 34$ mm

$S_{max} = 284$ mm



---

check shear

$V_u = 1.4 V_w + V_L$

$V_w = \dfrac{(1.48+1.1)}{2} \times 10 \times (0.2859 + 0.1502) = 5.63$

$V_L = 3$ ton

ضریب دینامیکی

$V_u = 1.4 \times 5.63 + (2 \times 3) = 13.88$ ton

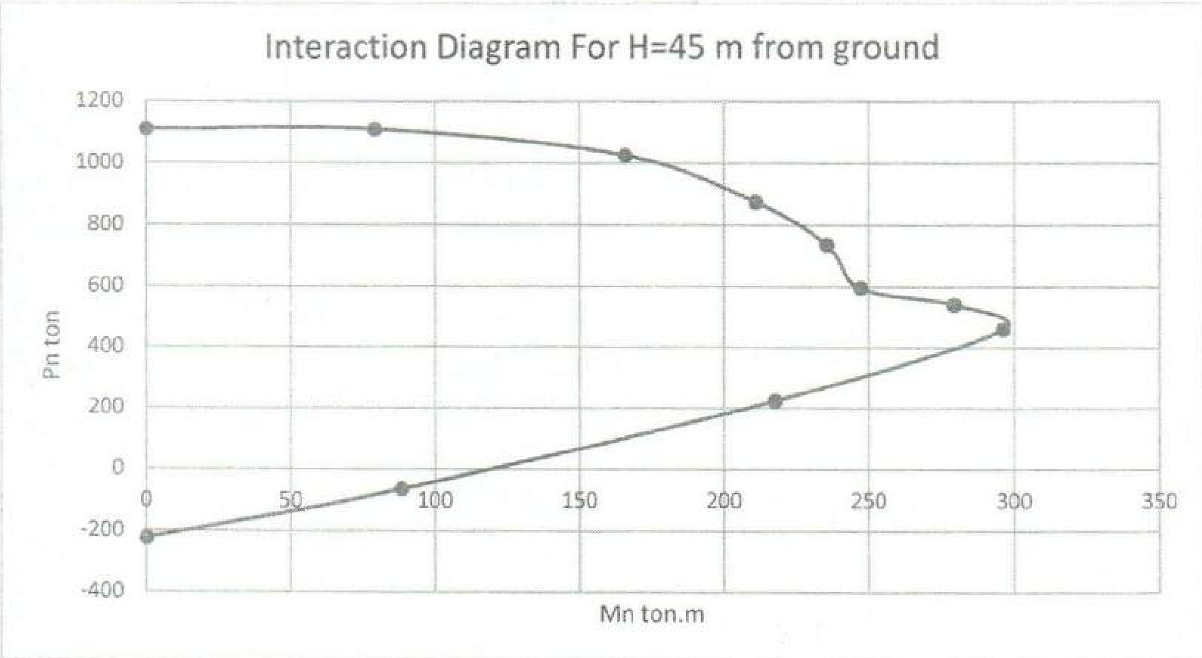
$V_c = 0.17\sqrt{25} \times \pi \times \dfrac{(1480^2 - 980^2)}{4} = 82$ ton $\qquad \dfrac{\phi V_c}{2} = 30.8$ ton $> V_u$

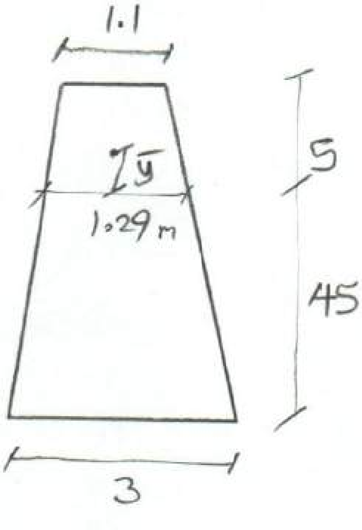
no need to shear reinf

$L_s = 80$ cm

$L_d = 115$ cm

محاسبات مربوط به مقطع دایره توخالی در ارتفاع ۴۵ متری از بر تکیه‌گاه





$\bar{y}$ از بالا $= \frac{5}{3}\left(\frac{2\times1.29+1.1}{1.29+1.1}\right)=2.566$

$\bar{y}=2.43$

$M_w=\left(\frac{1.29+1.1}{2}\right)\times5\times(0.2859+0.1502)\times2.43=5.939$ ton.m

$M_L=2\times3\times5=30$ ton.m

$M_u=1.4M_w+M_l=1.4\times5.939+30=38.31$ ton.m

$<\phi M_n\simeq120$ ton.m

در این مقطع قطر خارجی 1.29 متر و ضخامت 0.25 متر است. دولایه آرماتور که هر یک شامل 5 عدد $\phi28$ است نیز مورد استفاده قرار گرفته است.

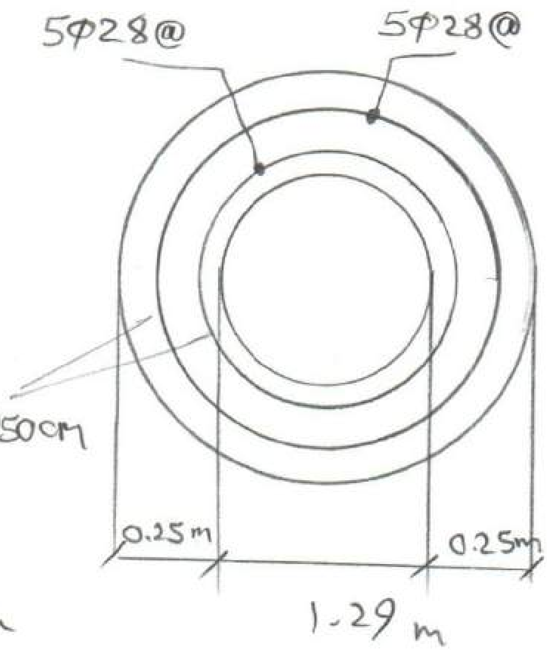
جزئیات در صفحه‌ی بعدی

کاور خارجی $= 50$ mm

کاور داخلی $= 40$ mm

$S_{min} = 34$ mm

$S_{max} = 284$ mm



---

check shear

$V_u = 1.4 V_W + V_L$

$V_W = \frac{(1.29+1.1)}{2} \times 5 \times (0.2859 + 0.1502) = 2.605$ ton

$V_L = 3$ ton

$V_u = 1.4 \times 2.605 + (2 \times 3) = 9.65$ ton

$V_c = 0.17 \sqrt{25} \times \pi \times \frac{(1290^2 - 790^2)}{4} = 69.42$ ton

$\frac{\phi V_c}{2} = 26$ ton $> V_u$

no need to shear reinf

$L_s = 80$ cm

$L_d = 115$ cm

در ارتفاع 50 متری از تکیه گاه هیچ لنگری نداریم و صرفاً برش 3 تن بار زنده دینامیکی موجود است که با دید مهندسی این برش نیز توسط بتن گرفته می شود بنابرین در کل استفاده از خاموت φ10 در فواصل 50 cm جوابگوی برش در کل سازه است.

به دلیل وجود بار محوری کمتر از 0.1 ظرفیت کل ستون از بار محوری در طراحی ستون صرف نظر کردیم

در جاهایی که نیاز به cut off داریم باید آرماتور به اندازه ی L_d ادامه یابد، همچنین در فواصل طولانی نیاز به وصله کردن آرماتور داریم که حداقل به طول $L_s = 80$ cm باید آرماتورهای وصله شده روی یکدیگر قرار گیرند. ضمنا فواصل spacing در محل های وصله نیز دارای مشکلی نیست و در برخی از مقاطع از spacing مقداری از S_{max} بیشتر شد که با توجه به شرایط مقطع ما از آنها اغماض کردیم.

طراحی پی

نیروی مکش روی سقف سازه   15 ton   8 ton

$$P_{service} = -P_w + P_{Turbine\ weigh} + P_{blades\ of\ turbine} + P_{column\ self\ weight}$$

$$P_w = 0.3004\ \frac{ton}{m^2} \times \left(\pi \times \frac{1.1^2}{4}\right) = 0.2855\ ton \uparrow$$

$$V_{column} = \frac{1}{3}\pi h(R^2 + r^2 + R.r) = \frac{1}{3}\pi \times 50\left(\left(3^2 + 1.1^2 + 3\times 1.1\right) - \left(2.5^2 + 0.6^2 + 2.5\times 0.6\right)\right)$$

$$= 282.75\ m^3$$

$$\gamma_{concret} = 2400\ kg/m^3$$

$$P_{column\ weight} = 282.75 \times 2400 = 678.58\ ton$$

$$P_{service} = -0.2855 + 15 + 8 + 678.58 = 701.3\ ton$$

با توجه به خاک شهر مبین مقدار $q_a = 3\ kg/cm^2$ پیشنهاد می گردد.

با فرض اینکه سازه‌ی پی ۱ متر زیر سطح زمین قرار می گیرد و عمق آن 120cm است داریم:

$$q_e = 300 + 18\times 1 - 2.2\times 21 = 271.8\ kN/m^2$$

$$A_{req} = \frac{P_{service}}{q_e} = \frac{701.3\times 10^3}{271.8\times 100} = 25.8\ m^2$$

$$\pi R^2 = 25.8 \longrightarrow R = 2.865\ m \longrightarrow \text{use } R = 3\ m$$

برای طراحی پی دایره‌ای به قطر H می توان از یک مربع با مساحت معادل با ضلع 0.89H استفاده کرد.

$$\pi \times 3^2 = 28.27\ m^2 > 25.8\ m^2 \qquad H = 2\times 3 = 6\ m \qquad 0.89H = 5.34\ m$$

Ultimate Loads :

وزن توربین   وزن بتن   به خاطر دینامیکی بودن 2×

$$P_u = 1.2D + 1.4W + 1L = 1.2\times(15 + 678.58) + 1.4\times(-0.2855) + 1\times(2\times 8)$$

$$= 847.9\ ton$$

$\cdot M_u = 1.2M_D + 1M_L + 1.4M_W = 1\times300 + 1.4\times944.46 = 1622.944$ ton.m

$M_L = 2\times3\times50 = 300$ ton.m

ضریب بار زنده ملی

$M_W = 944.46$ ton.m

$$q_u = \frac{P_u}{A_f} \pm \frac{MC}{I} = \frac{847.9}{28.27} \pm \frac{1622.944\times\left(\frac{5.34}{2}\right)}{\frac{1}{12}\times5.34^4} = 29.99 \pm 63.94 \begin{cases} 93.93 \\ -33.95 \end{cases}$$

با توجه به اعداد بدست آمده در حدود $\frac{1}{4}$ از پی به کشش می‌افتد این مسئله اگرچه ممکن است برای سازه‌های ساختمانی خیلی بحرانی نباشد ولی برای سازه‌ی پراهمیتی مثل توربین بادی باید کنترل گردد. برای این کار می‌توان از شمع جهت ایجاد پایداری پی سازه استفاده کرد که از توانایی این پروژه با توجه به محدودیت اطلاعات خارج است. پس ما از کشش صرفنظر می‌کنیم.

تنش زیر پی را برابر بیشینه تنش می‌گیریم (محافظه‌کارانه)

depth of the footing

assume $d = 1200$ mm

$b_o = \pi(1200 + 3000) = 13194.7$ mm

$V_u = 93.93\times\left(28.27 - \frac{\pi}{4}(1.2+3)^2\right) = 1354$ ton

$V_{n\,req} = \frac{V_u}{\phi} = \frac{1354}{0.75} = 1805$ ton

$V_{c_1} = 0.17\left(1+\frac{2}{\beta}\right)\sqrt{f'_c}\,b_o d \qquad \beta = 1 \rightarrow$ Not applicable

$V_{c_2} = 0.083\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\,b_o d \quad , \alpha_s = 40$

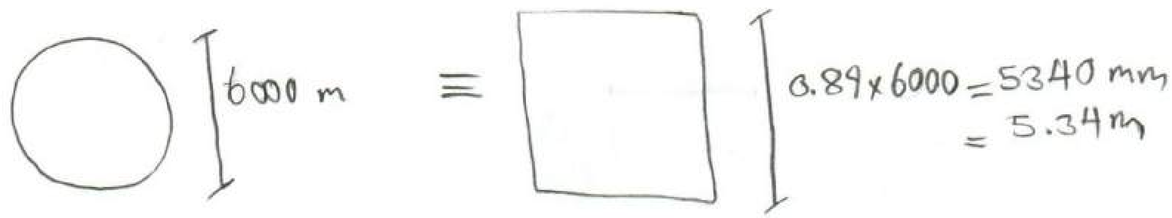
$1354\times10^4 = 0.083\left(\frac{40d}{13194.7} + 2\right)\sqrt{25}\times13194.7\times d \longrightarrow d_{req} = 632$ mm

$V_{c_3} = 0.33\sqrt{f'_c}\,b_o d \longrightarrow d_{req} = \frac{1354\times10^4}{0.33\times\sqrt{25}\times13194.7} = 621.9$ mm

governs

use $d = 622$ mm

One-way shear



$$V_u = 93.93\ \frac{ton}{m^2} \times \left(5.34 \times \underbrace{\left(\frac{5.34}{2} - \frac{3}{2} - 0.622\right)}_{0.548}\right) = 274.4\ ton$$

$$V_c = 0.17\sqrt{f'_c}\, b d = 0.17\sqrt{25} \times 5340 \times d = 274.9 \times 10^4 \rightarrow d = 605.64\ mm$$

Therefore select the overal depth 606 mm, but because L_d of $\phi 28$ bars of column is 1150 mm we should take a depth of <u>1.2 m</u> finally!

Flexural Reinforcement

$$M_{ux} = M_{uy} = 93.93\ \frac{ton}{m^2} \times \frac{(1.17)^2}{2} \times 5.34 = 343.31\ ton.m = 343.31 \times 10^7\ N.mm$$

Assume $\phi 20$ rebar, cover = 50 mm

$$d = 1200 - 50 - 20 - \frac{20}{2} = 1120\ mm$$

$$As_{req} = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{343.31 \times 10^7}{(0.9)(400)(1120 - \frac{a}{2})} = 8632\ mm^2 \checkmark$$

Assume $a = 30.5$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{8632 \times 400}{0.85 \times 25 \times 5340} = 30.42 \approx a \text{ OK} \checkmark$$

Temp & shrinkage reinforcement:

$$A_{s,temp} = 0.0018\, A_g = (0.0018)(1200)(5340) = 11535\ mm$$

Min Flexural reinf

$$As_{min} = Max\left(\frac{0.25\sqrt{f'_c}}{f_y} b_w d,\ \frac{1.4\, b_w d}{f_y}\right)$$

$$As_{\substack{min \\ max}} = \left(\frac{(0.25)\sqrt{25}}{400}(5340)(1200) \,,\, \frac{1.4 \times 5340 \times 1200}{400} \right)$$

$$= 22428 \ mm^2$$

$As_{min} = 1.33\, As_{req} = 1.33 \times 8632 = 11481 \ mm^2 \longrightarrow$ controls

$$As_{req} = 11481 \ mm^2 \left| \begin{array}{l} 17 \phi 30 \ (12017) \ \checkmark \\ 15 \phi 32 \ (12064) \\ 10 \phi 40 \ (12566 \end{array} \right.$$

$S = \dfrac{5340 - 2(75)}{17 - 1} = 324. \longrightarrow$ use $17 \phi 30$ in each direction

Spacing of bars

$S_{max} = min(3h, 450) = 450$ OK

$S_{min} = Max(22, 25, 1.33(25)) = 33$ OK ✓

Bearing capacity of footing at column base

$$\phi N_n = \phi (0.85) f'_c A_1 \sqrt{\frac{A_2}{A_1}} \geq P_u$$

$A_2 = 28.27$

$A_1 = \frac{\pi}{4} \times 3^2 = 7.069$ use $\sqrt{\frac{A_2}{A_1}} = 2$

✓

$\phi N_N = 0.65 \times 0.85 \times 25 \times (28.27 \times 10^6) 2 = 78095.8$ ton $> P_u = 847.9$ ton

Bearing capacity of column

$\phi N_u = 0.65 \times 0.85 \times 25 \times 2.16 \times 10^6 = 2983.5$ ton $> P_u$ OK ✓

Dowels

$As_d = 0.005\, A_g = 10800 \rightarrow 16 \phi 30 \qquad As_d = 11310 \ mm^2$

use $16 \phi 30$

Development length of flexural reinf

$$L_d = \left[\frac{f_y}{1.1\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \lambda}{\left(\frac{c_b + K_{tr}}{d_b}\right)} \right] d_b \geq 300$$

$\psi_t = \psi_e = \psi_s = \lambda = 1.0$

$K_{tr} = 0$

$c_b = \min\left(\frac{324}{2}, 75 + \frac{30}{2}\right) = 90$

$$\left(\frac{c_b + K_{tr}}{d_b}\right) = \frac{90 + 0}{30} = 3 > 2.5 \rightarrow \text{use } 2.5$$

$$L_d = \left[\frac{400}{1.1\sqrt{25}} \frac{1 \times 1 \times 1 \times 1}{2.5} \right] d_b = 29 d_b^{30} = 872.7 \geq 300 \text{ mm}$$

$$L_{d\,pro} = \frac{5340}{2} - \frac{3000}{2} - 75 = 1095 \text{ mm} > 872 \text{ mm} \quad \text{ok } \checkmark$$

Development of Dowels in Foundation

$$L_{dc} = \max\left(\frac{0.24 f_y d_b}{\sqrt{f'_c}}, 0.043 f_y d_b \right) \geq 200 \text{ mm}$$

$$L_{dc} = \max\left(\frac{(0.24)(400)(30)}{\sqrt{25}}, (0.043)(400)(30) \right) = 576 \geq 200 \text{ ok}$$

$L_{dc} \leq 1200$ ok ✓

Development of Dowels in column

$$L_{dc} = \max\left(\frac{0.24 f_y d_b}{\sqrt{f'_c}}, 0.0043 f_y d_b \right)$$

$$= \max\left(\frac{(0.24)(400)(30)}{\sqrt{25}}, 0.0043 \times 400 \times 30 \right) = 576 \quad \text{ok } \checkmark$$

Lap length of column bars was checked previously.

. Reinforcement