

## الباب الرابع

### CHAPTER4

#### القطاعات الخرسانية المسلحة المعرضة لقوي غير متمركزة

#### Reinforced Concrete Sections Subjected to Eccentric Forces

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#### ٤-١: مقدمة:

تتعرض القطاعات الخرسانية المسلحة في غالب الأحوال لقوي غير متمركزة وقلما يكون القطاع معرضاً لقوي كاملة التمرکز .

ويعزي ذلك إلى عدة أسباب منها بعض عيوب الصناعة حيث يتم تنفيذ القطاع بأبعاد هندسية بها بعض الاختلاف عن الأبعاد التصميمية ، أو يكون ذلك راجعاً لنحافة العمود أو طبيعة وضعه الهندسي داخل المنشأ مما يعرضه لعزوم انحناء .

#### ٤-٣: التحليل الإيشائي:

شكل رقم ( ١ ) يوضح قطاعاً معرضاً لقوي ضغط متمركزة تماماً حيث ينطبق خط عمل القوة  $P_u$  على مركزه اللدن Plastic Center ( النقطة التي إذا أثرت فيها محصلة القوي ينتج عنها انفعالاً منتظماً على القطاع مقداره 0.002 ) ، وبالتالي لا توجد أي لا مركزية (  $e = 0$  ) ولا توجد أي قوي شد أو عزوم على القطاع . ولدراسة اتزان القطاع في هذه الحالة شكل ( ١ ) يمكن استنتاج :

$$P_u = C_c + C_{s1} + C_{s2}$$

$$M_u = 0.00$$

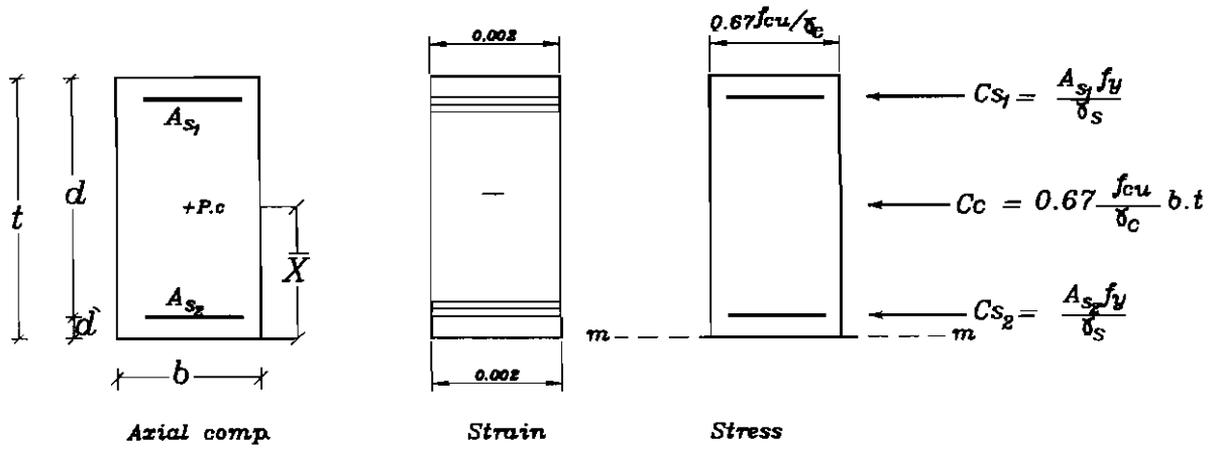
ويمكن تحديد المركز اللدن Plastic Center للقطاع عن طريق حساب العزوم حول أي نقطة ومساواته بعزم محصلة القوي ، فإذا طبقنا ذلك عند الخط  $m - m$  ، شكل ( ١ )

$$\therefore C_c * \frac{t}{2} + A_{s2} \frac{f_y}{\gamma_s} (d^-) + A_{s1} f_y (d - d^-) = \sum c(x^-)$$

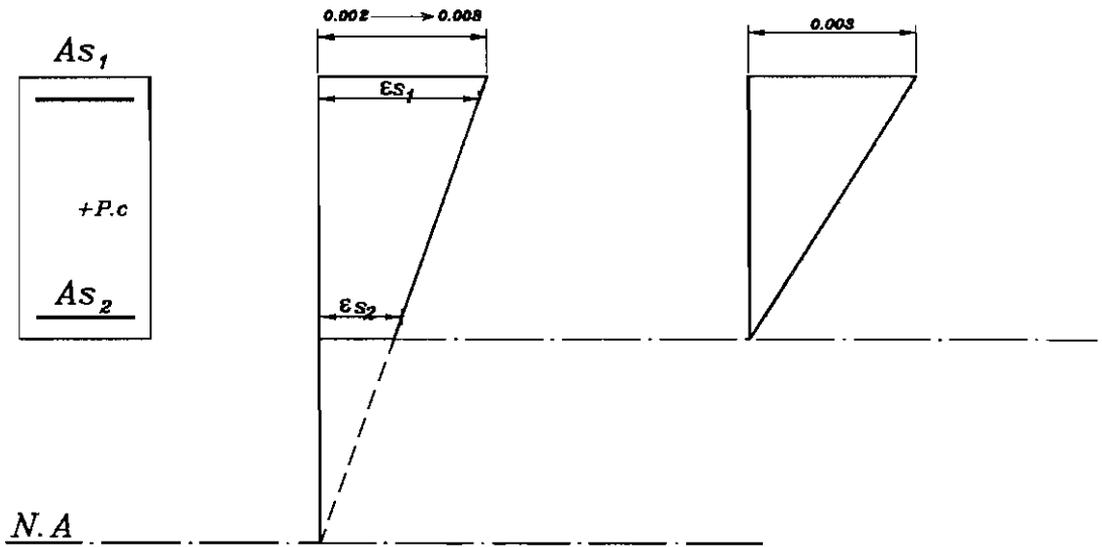
$$\text{Where: } \sum c = C_c + A_{s1} f_y / \gamma_s + A_{s2} f_y / \gamma_s$$

ويمكن في هذه الحالة تخيل أن محور التعادل ( الذي يفصل بين منطقتي الضغط والشد ) يقع جدلاً في اللانهاية .

ولنفرض الآن أن قوي الضغط المؤثرة في مركز القطاع  $P_u$  بدأت في الحركة إلى أعلى ، حيث يظل القطاع معرضاً لإجهاد ضغط فقط إلا أنها بقيمة أقل عند الحافة السفلي وبقيمة أعلى عند الحافة العليا ( شكل ٢ ، ٣ ) ، فإذا ما واصل الحمل  $P_u$  الحركة بعيداً عن مركز القطاع تظل إجهادات الضغط نقل حتى تتلاشى عند الحافة السفلي ( في حالة وقوع محور التعادل مماساً



شكل (١) قطاع خرساني معرض لقوى ضغط متمركزة



شكل (٢) حركة محور التعادل و أثره على قيم الانفعالات

لحافة القطاع السفلية حيث تصبح قيمة الانفعال علي الحافة السفلي = صفراً ، وقيمة الانفعال علي الحافة العليا = 0.003 كحد أقصى ) وتصبح معادلات الاتزان هي :

$$P_u = C_c + C_{s1} + C_{s2}$$

$$M_u = \Sigma M @ P.C.$$

فإذا ما تحرك الحمل  $P_u$  بعيداً عن مركز القطاع يتحول الجزء السفلي من القطاع إلي منطقة شد شكل ( ٤ ) وتصبح معادلات الاتزان كما يلي :

$$P_u = C_c + C_{s1} - T$$

$$M = \Sigma M @ P.C.$$

الآن يمكن تصور قطاع خرساني ( شكل ٦ ) معرضاً لقوة ضغط غير متمركزة Eccentric Compression Force تبعد عن مركز القطاع مسافة  $e$  وتسبب عزم انحناء علي هذا المركز قيمته  $M$  ، وعليه تكون معادلات الاتزان لهذا القطاع :

$$P_u = \frac{0.67 f_{cu} . ab}{\gamma_c} + A_s^- f_s^- - A_s f_s$$

$$M_u = \Sigma M @ P.C.$$

$$= P_u . e$$

$$= \frac{0.67 f_{cu} . ab}{\gamma_c} \left( \frac{t}{2} - \frac{a}{2} \right) + A_s^- f_s^- \left( \frac{t}{2} - d^- \right) + A_s f_s \left( d - \frac{t}{2} \right)$$

$$\text{Where : } f_s = f_s^- = f_y / \gamma_s$$

### ٣-٤ : منحنيات التفاعل : Interaction Diagrams :

من المعلوم أن القطاع الخرساني يمكن أن يتعرض لقوي ضغط متمركزة تفوق قدرته علي التحمل وتكون سببا في انهياره .

وأيضاً يمكن أن ينهار القطاع في حالة تعرضه إلي أحمال غير متمركزة وتكون كل من إجهادات الشد والضغط ويكون القطاع حينئذ معرضاً لـ  $M_u$  ,  $P_u$  . والحالة الأخيرة ليست حالة وحيدة بمعنى إنه لا توجد قيمة وحيدة بعينها لكل من  $M_u$  ,  $P_u$  ، بحيث إذا وصلت قيمتها إلي حدود معينة انهار القطاع ، بل إنه بقليل من التأمل نكتشف أن هناك عدداً لانتهائي من قيم  $M_u$  ,  $P_u$  التي ينهار عندها القطاع ، وبمعنى آخر يمكن أن ينهار القطاع بسبب زيادة قيمة  $P_u$  رغم أن  $M_u$  ضئيلة والعكس صحيح ، أي أن هناك قيمتين مترافقتين لكل من  $M_u$  ,  $P_u$  إذا تحققنا ينهار عندهما القطاع ، فإذا صغرت إحداهما

وكبرت الأخرى يمكن أن ينهار القطاع أيضا عند رقمين مترافقين جديدين لكل من  $P_u$  ,  $M_u$  ولقد أمكن جمع قيم  $P_u$  ,  $M_u$  التي يحدث عندها انهيار للقطاع في منحنى سمي منحنى التفاعل Interaction Diagram ، حيث يحدث تفاعل بين قيمتي  $P_u$  ,  $M_u$  ويتغيران بقيم عديدة يحدث عند كل قيمة ( كل نقطة على المنحنى ) حالة انهيار للقطاع ، شكل ( ٧ ) .

#### ٤-٤ : ملاحظات عامة على منحنى التفاعل : Interaction Diagrams :

- ١ - المحور الرأسي يمثل قيمة مختلفة للحمل الأقصى  $P_u$  والتي يمكن أن ينهار عندها القطاع .
- ٢ - المحور الأفقي يمثل قيمة مختلفة للعزم الأقصى  $M_u$  المناظر للحمل الأقصى  $P_u$  والذي يمكن أن ينهار عنده القطاع أيضا .
- ٣ - المنحنى في مجمله يمثل المحل الهندسي لقيم  $P_u$  ,  $M_u$  لكل حالات الانهيار الممكنة للقطاع .
- ٤ - نلاحظ أنه كلما زادت  $e$  يزيد العزم ونقل القوة  $P_u$  المسببة للانهيار ، وكلما قلت  $e$  قل العزم وتزيد القوة  $P_u$  المسببة للانهيار .
- ٥ - كما سبق دراسته في باب تصميم الكمرات فإن معاملات تخفيض المقاومة  $\gamma_s$  ,  $\gamma_c$  تتغير وليست ثابتة ، ويعتمد تغيرها على قيمة اللامركزية (  $e$  ) ونسبتها إلى (  $t$  ) حيث :

$$\gamma_c = 1.75 - 0.5 \left( \frac{e}{t} \right) \geq 1.5 \quad [E.C.3-16-a]$$

$$\gamma_s = 1.36 - 0.43 \left( \frac{e}{t} \right) \geq 1.15 \quad [E.C.3-16-b]$$

وعليه فإنه عندما  $e = 0$  تكون  $\gamma_c = 1.75$  ,  $\gamma_s = 1.36$  .

- ٦ - نلاحظ أن منحنى التفاعل يشكل علاقة تناسب عكسية بين المتغيرين (  $P_u$  ,  $M_u$  ) ( المرحلتين A , B ) ، ويستمر ذلك حتى نقطة انقلاب تحدث عن قيمة  $P_b$  وتسمى حمل الاتزان وتسمى الحالة حالة اتزان Balanced Case ، حيث ينعطف المنحنى للداخل عند هذه النقطة وتصبح العلاقة بين  $M_u$  ,  $P_u$  طردية تناقصية ( في المرحلتين C , D ) .

٧ - نلاحظ أن منحنى التفاعل من الممكن أن يتضخم للخارج موازيا لنفسه بمجرد تغيير نسبة وجود الحديد ، شكل ( ٧ ) ، حيث :

$$\mu = \frac{\mu}{f_{cu} * 10^{-5}}$$

$$\mu = \frac{A_s}{A_c}$$

حيث :

$f_{cu}$  = Ultimate Concrete Comp. Strength.

$A_s$  = Steel Cross Section Area .

$A_c$  = Concrete Cross Section Area = b.t .

٨ - نلاحظ أنه يمكن التعامل مع منحنيات التفاعل عن طريق متغيرين مرادفين لكل من  $P_u, M_u$  حيث يمكن قسمة  $P_u$  على  $f_{cu}.b.t$  ، وقسمة  $M_u$  على  $f_{cu}.b.t^2$  ، وعليه يمكن القول أن المحور الرأسي يصبح :

$$K = \frac{P_u}{f_{cu}.b.t} = Dimensionless$$

والمحور الأفقي يصبح :

$$K.e = \frac{M_u}{f_{cu}.b.t^2} = \frac{P_u.e}{f_{cu}.b.t^2} = Dimensionless$$

وتصبح العلاقة على منحنى التفاعل بين المتغيرين  $K, K.e$  ، ويمكن وصف أي نقطة على هذا المنحنى بالخط الواصل بين نقطة الصفر وبين المنحنى نفسه ، حيث يعرف هذا الخط بميله  $\frac{e}{t}$  وهو ظل الزاوية المحصورة بين هذا الخط والمحور الأفقي ، وعليه تم تعريف الخط الواصل بين نقطة الأصل ونهاية المرحلة A بالخط  $\frac{e}{t} = 0.05$  وتم تعريف الخط الواصل بين نقطة الأصل ونقطة انعطاف المنحنى (نهاية المرحلة B) بالخط  $\frac{e_b}{t}$  وهكذا .

٩ - يمكن رسم وتكوين منحنى التفاعل بمجموعة من النقاط أهمها :

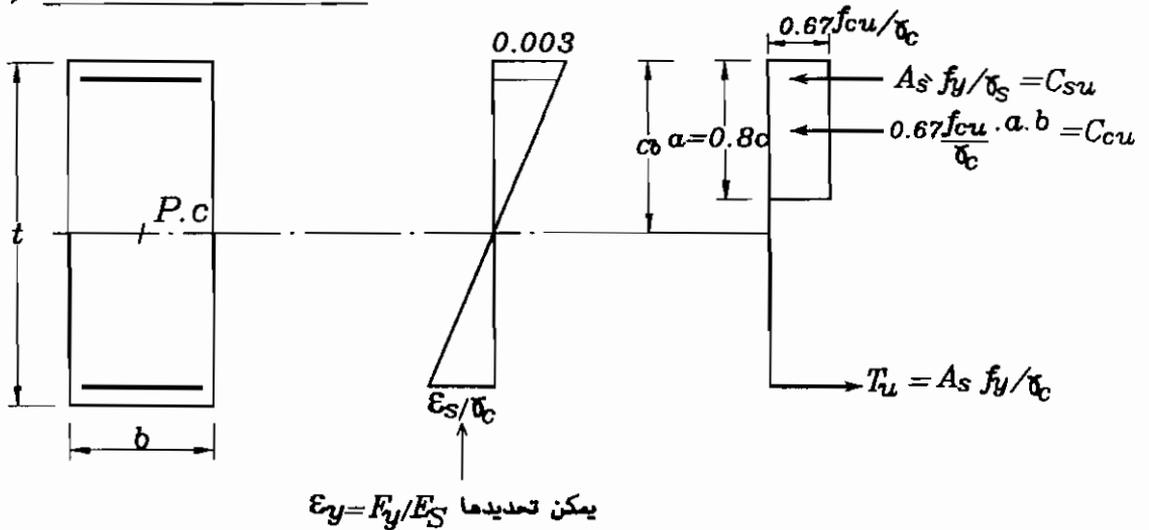
٩ - ١ : قيمة  $P_u$  كقوة محورية متمركزة .

$$P_u = 0.35 f_{cu} A_c + 0.67 f_y A_{sc} .$$

٩ - ٢ : قيمة  $P_b$  وهي القوى المتزنة والتي يحدث عندها انهيار في كل من

الخرسانة والحديد في آن واحد وتحليلها الإنشائي كالاتي :

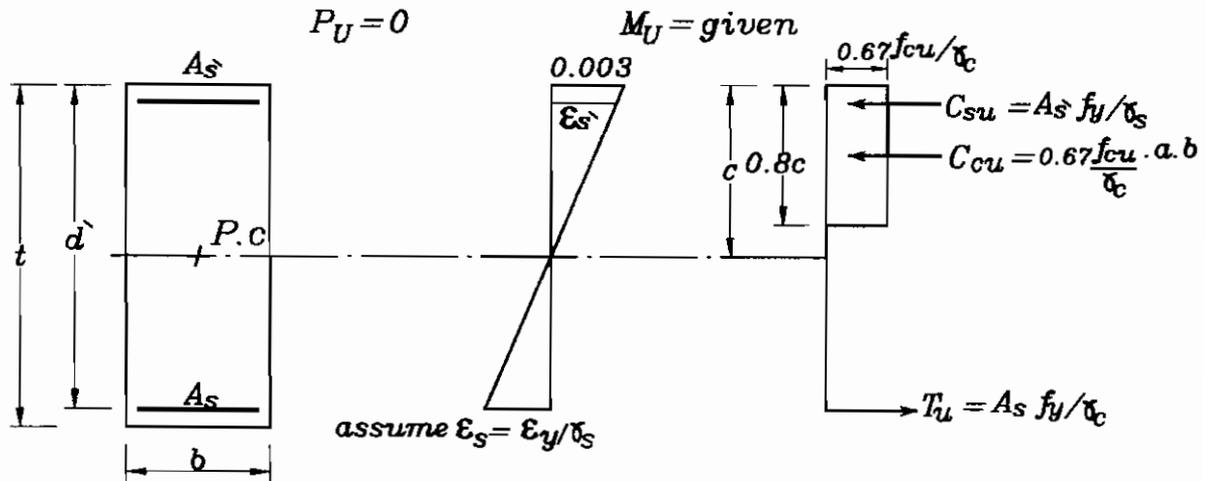
b) Balanced Section

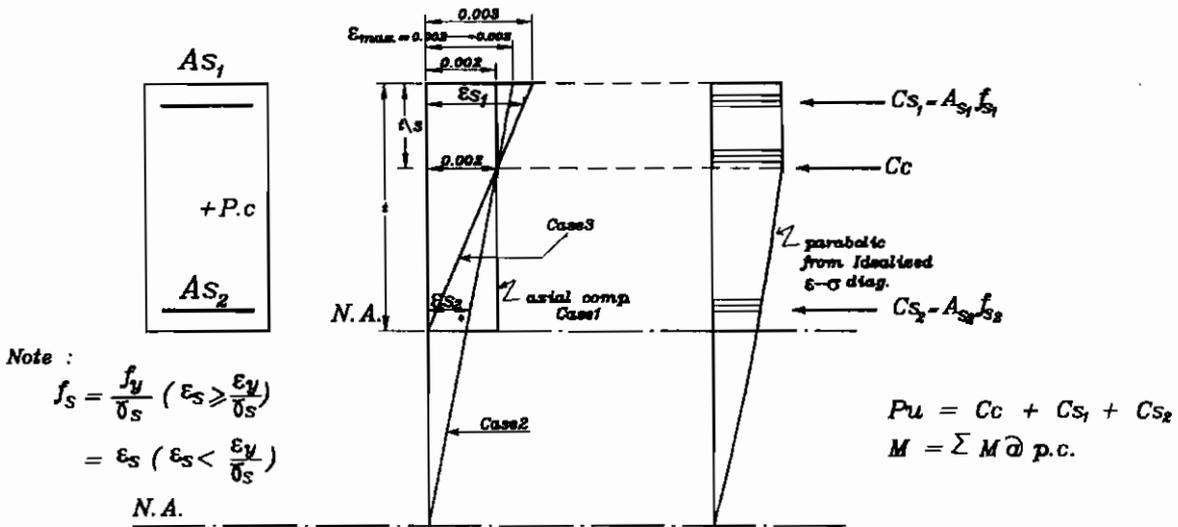


$M_u @ P.C.$  يتم أخذ العزوم حول P.C.  
 يتم أخذ محصلة القوي  
 $P_u = C_{su} + C_{cu} - T_u$   
 وبهاتين القيمتين ( يتم تحديد نقطة علي المنحني ) .

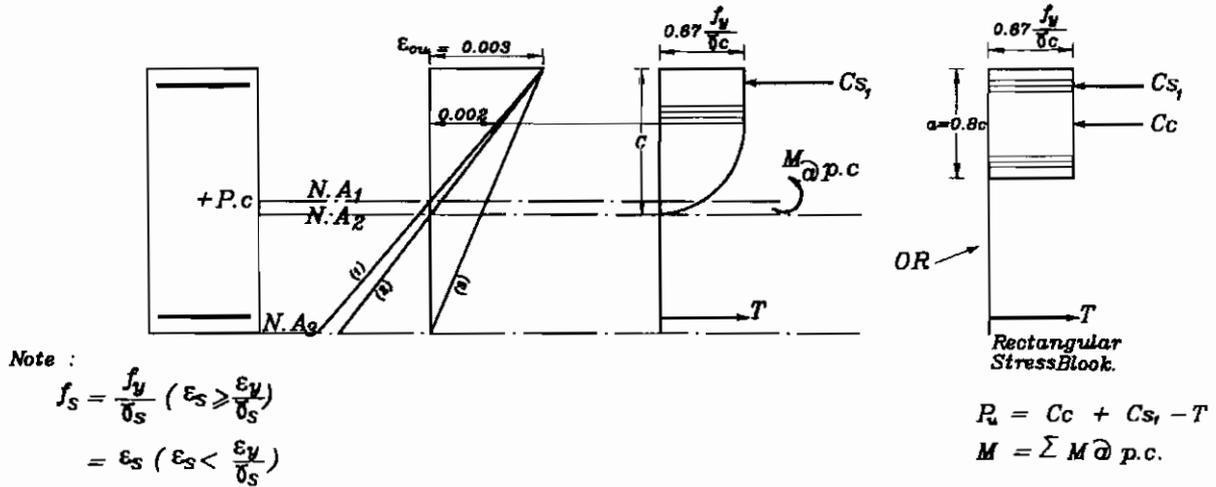
3 - 9 قيمة  $M_u$  المناظرة لحمل أقصى  $P_u = 0$   
 وتحليلها الإنشائي كالآتي :

c) Flexure Design

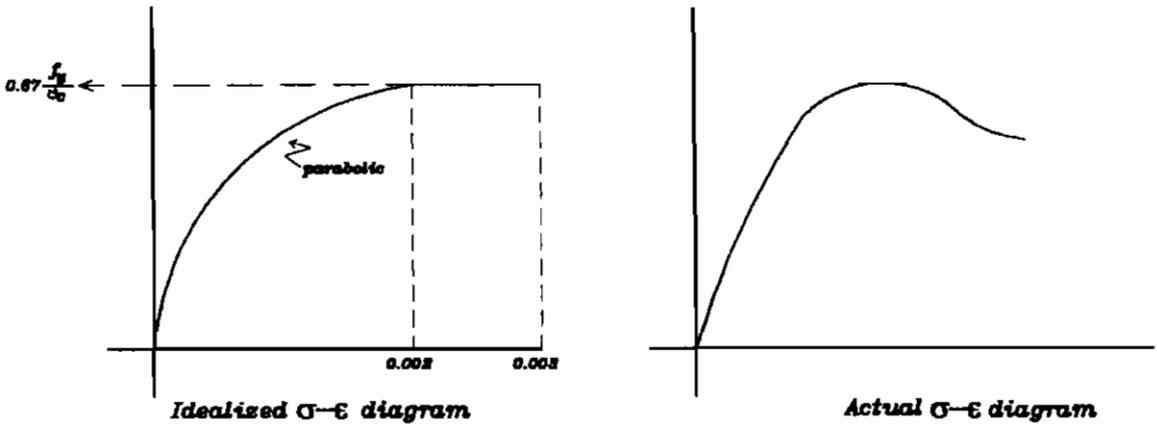




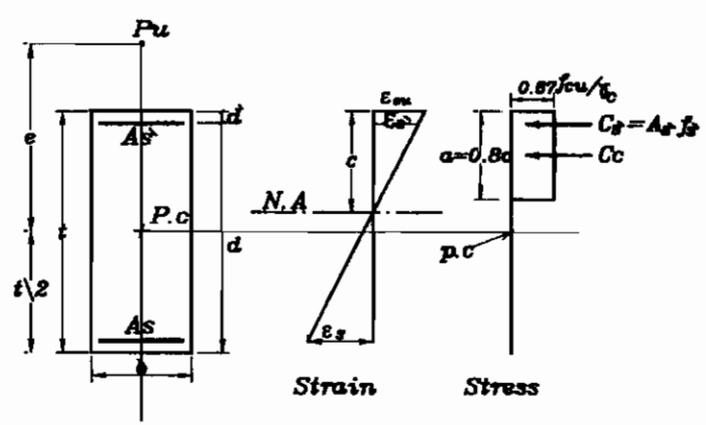
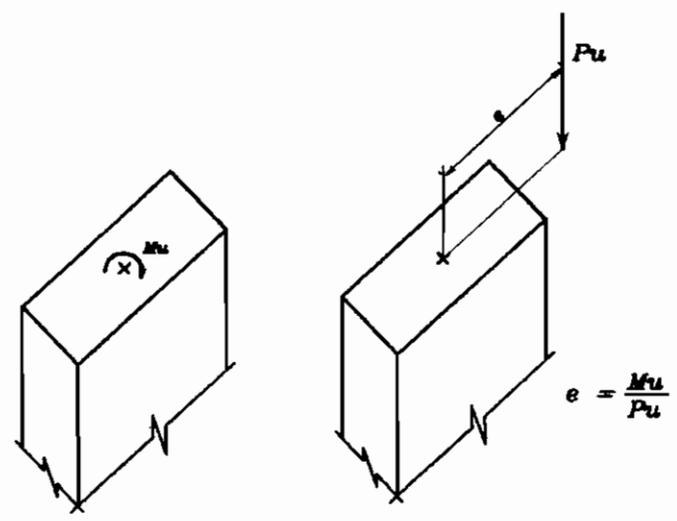
شكل (٣) حركة محور التعادل و أثره على قيم الاجهادات



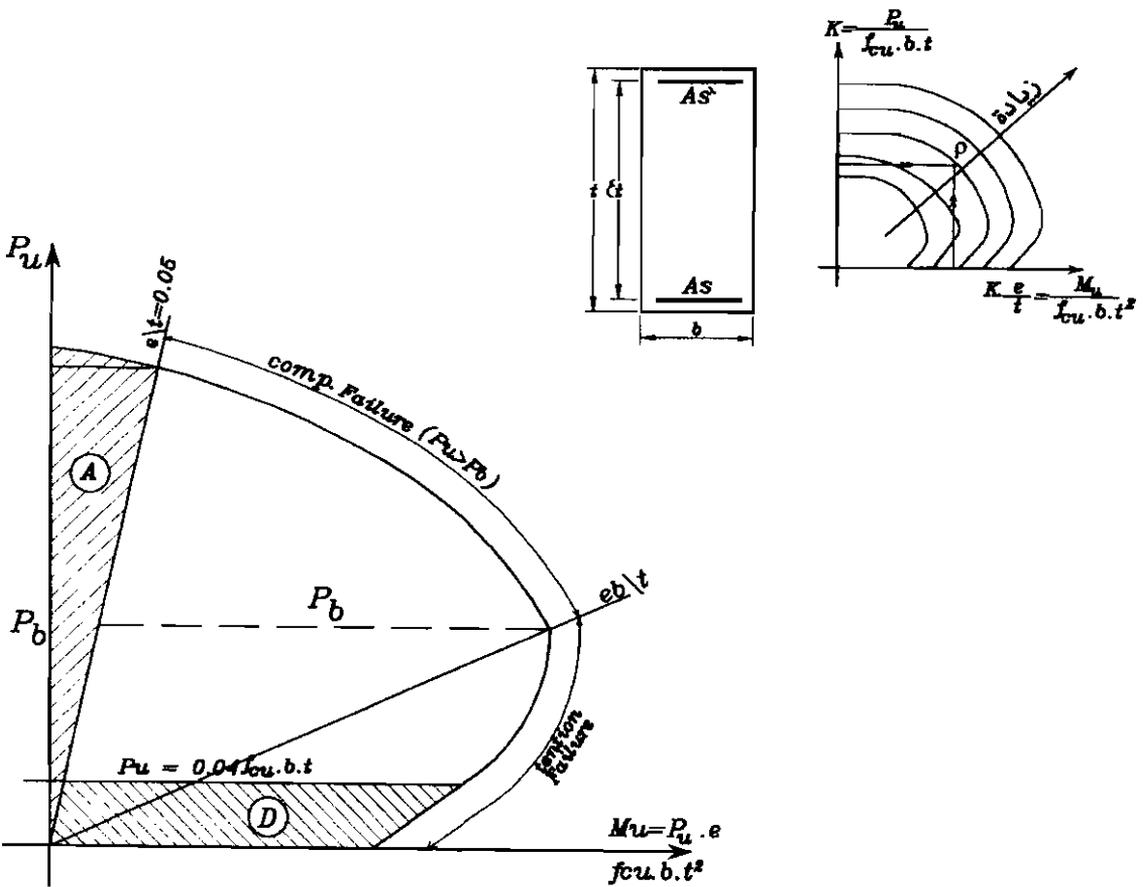
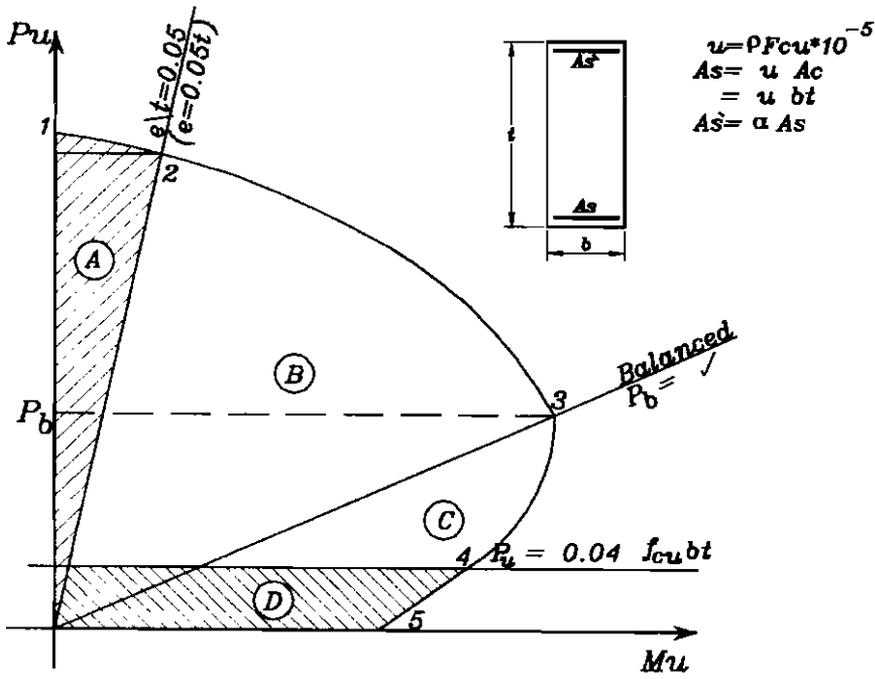
شكل (٤) القطاع و هو معرض لكلا نوعى الاجهادات  
( الضغط و الشد ) نتيجة القوى غير المتمركزة



شكل (٥) المنحنى الفعلي و المثالي المعدل  
 لسلوك الخرسانة في الضغط



شكل (٦) تحليل القطاعات ذات الاحمال  
 غير المتراكزة



شكل (٧) منحنى التفاعل  
 Interaction Diagram .

نفرض أن الحديد وصل إلى مرحلة لخصوع ومن ثم يمكن استنتاج أن :

$$T_u = C_{su} + C_{cu}$$

$$1 - A_s f_y / \gamma_s = 0.67(f_{cu} / \gamma_s) (a) (b) + A_s^- f_y / \gamma_s$$

get ( a ) from this equation then make check that :

$$1 - \varepsilon_s > \varepsilon_y / \gamma_s \text{ if not take}$$

$$f_y / \gamma_s = f_s = \varepsilon_s^- E_s \text{ where } \varepsilon_s^- = 0.003 \frac{c-d^-}{c}$$

$$C = a / 0.8$$

$$2 - \varepsilon_s^- > \varepsilon_y / \gamma_s \text{ if not take}$$

$$f_y / \gamma_s = f_s = \varepsilon_s^- E_s \text{ where } \varepsilon_s^- = 0.003 \frac{c-d^-}{c}$$

$$2 - M_u = C_u (d - C / 2) + C_{su} (d - d^-)$$

get  $M_u$  .

١٠ - المنحى يمثل أربع مراحل مختلفة :

### مرحلة ( A ) المنطقة من ( ١ - ٢ ) : Axial Compression. Failure :

تمثل سيادة  $P_u$  على الانهيار ، حيث تكون قيمتها كبيرة وقيم  $M_u$  المناظرة إما صفرا أو قيمة صغيرة نسبيا ، وفي هذه المرحلة يمكن إهمال قيمة  $M_u$  ويمكن تصميم القطاع على  $P_u$  فقط ، وقد تم وضع حدود لقيمة  $M_u$  التي يمكن إهمالها عندها وهي القيمة التي تناظر لا مركزية  $e$  حيث إن :

$$e / t \leq 0.05$$

حيث يظهر القطاع وعليه اجهادات ضغط متساوية ، ويكون الانهيار تحت تأثير اجهادات الضغط المتمركزة ويصمم القطاع على أنه عمود .

### مرحلة B المنطقة بين ( ٢ ، ٣ ) : Compression. Failure ( شكل ٧ ) :

وهي منطقة يلاحظ فيها زيادة قيمة  $M_u$  مع نقص في قيمة  $P_u$  ( مع بقائها كبيرة ) وبالتالي يظهر القطاع وعليه قوي ضغط ، إلا أنها غير متساوية أو تظهر اجهادات شد غير جوهرية ويحدث الانهيار أيضا تحت تأثير قوي الضغط الغير متمركزة Eccentric Compression Failure .

### مرحلة ( C ) المنطقة من ( ٣ - ٤ ) : Tension Failure :

حيث تتزايد اجهادات الشد وتصبح هي السائدة وتتحسر اجهادات الضغط وبالتالي ترتفع قيم  $M_u$  ونقل قيم  $P_u$  ويحدث الانهيار نتيجة اجهادات الشد Tension Failure .

### مرحلة ( D ) المنطقة بين ( ٥ - ٤ ) : Flexure Failure :

وهي مرحلة تتضاعل فيها قيم  $P_u$  وتصبح قيمة  $M_u$  هي الغالبة ، ويمكن إهمال قيمة  $P_u$  لصغرهما وتصميم القطاع باعتباره معرضا لعزوم انحناء فقط والذي يسبب انهياره Flexure Failure ، وقد تم تحديد قيمة  $p_u$  التي يمكن بعدها إهمالها والتصميم على عزوم خالص Pure Bending بالمعادلة الآتية :

$$P_u \leq 0.04 f_{cu} . b . t .$$

ويصمم القطاع على أنه كمره

### ٤-٥: تصميم القطاعات الخرسانية المعرضة لقوى ضغط غير متمركزة :

#### منطقة ( A ) :

إذا كانت قيم  $P_u$  كبيرة وقيمة  $M_u$  صغيرة وقيم اللامركزية لا تزيد عن ٥ % (  $\frac{e}{l} \leq 0.05$  ) فإنه يمكن إهمال قيمة  $M_u$  ويمكن التصميم باعتبار قيمة  $P_u$  فقط بحيث يمكن اعتبار القطاع عمودا :  
 $P_u = 0.35 f_{cu} A_c + 0.67 f_y A_{sc}$

#### منطقة ( D ) :

حيث تتضاعل قيمة  $P_u$  وتصبح أقل من  $0.04 f_{cu} . b . t$  ، ويتم إهمال حمل الضغط  $P_u$  ويجري التصميم على  $M_u$  فقط كما سبق شرحه في باب تصميم الكمرات .

#### منطقة ( C ) :

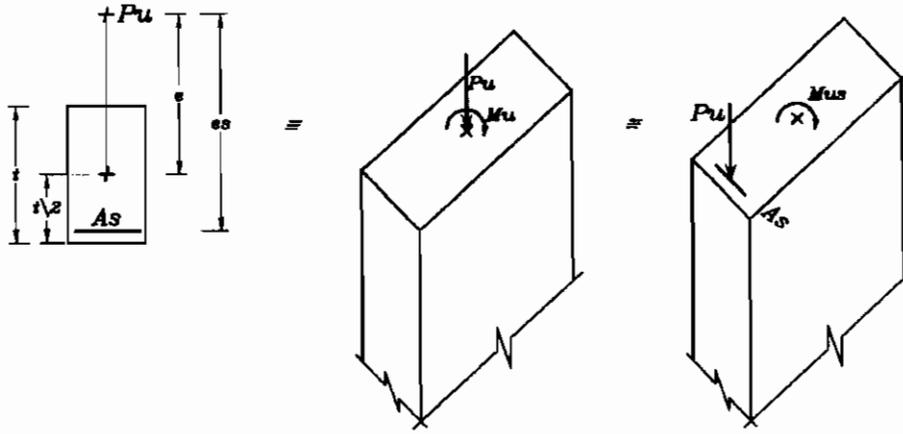
حيث ينهار القطاع نتيجة إجهادات الشد ويكون  $P_u$  أقل من حمل الاتزان  $P_b$  ، ويمكن تحليل القطاع إنشائيا كما هو موضح بشكل ( ٨ ) ، حيث يتم حساب العزوم  $M_{us}$  باعتبار أن  $P_u$  يؤثر على صلب التسليح وعليه يكون :

$$e = \frac{M_u}{P_u}$$

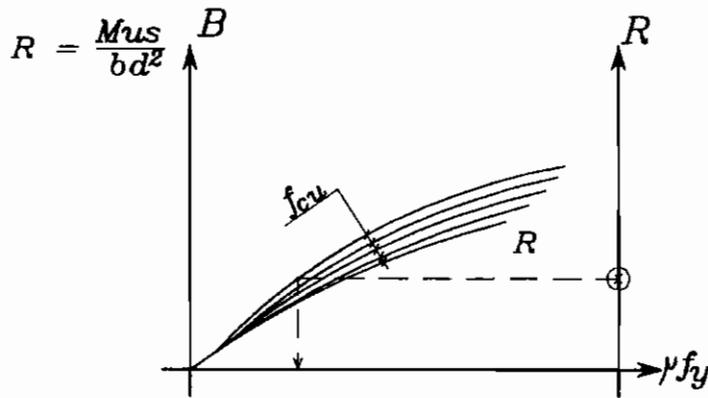
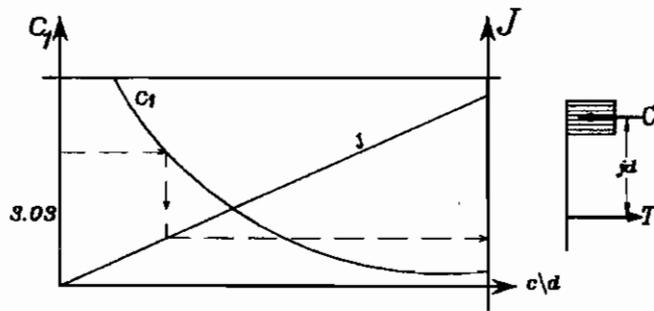
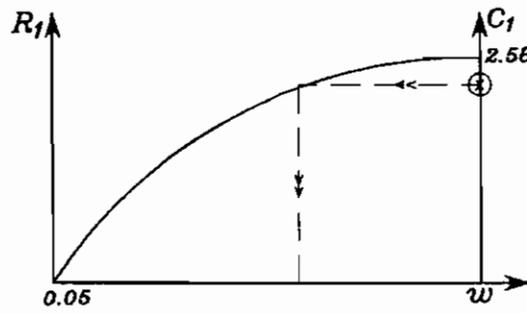
$$e_s = e + \frac{t}{2} - cover$$

$$M_{us} = P_u \cdot e_s$$

ويتم تصميم القطاع على قيمة  $M_{us}$  باستخدام المخطط البياني ( ١ ، ٢ ، ٣ ) شكل ( ٩ ) والسابق بيانها في باب الكمرات .



شكل (٨) قطاع يقع في منطقة  $C$  (انصيار بالشد)



شكل (٩)

### منطقة (B) :

تعتبر هذه المنطقة هي الاستخدام الحقيقي لمنحني التفاعل Interaction Diagram ، حيث يتم في هذه الحالة فرض قيم تقريبية لأبعاد القطاع وحساب قيمتي  $M_u$  ،  $P_u$  ، ومن ثم حساب قيمتي  $K, K \frac{e}{l}$  ومن منحني التفاعل يتم حساب  $(\rho)$  ،  $As^-$  ،  $As$  شكل (٧) .

### ٤-١ : القطاعات المعرضة لقوى شد غير متوازنة : Eccentric Tension :

#### هناك حالتان :

الأولى : أن تكون القوة المؤثرة لها لا مركزية صغيرة ( داخل حدود أبعاد القطاع ) شكل (١٠) حيث يمكن تحليل القطاع باعتبار أن صلب التسليح هو الذي يتحمل القوة المؤثرة بالكامل .

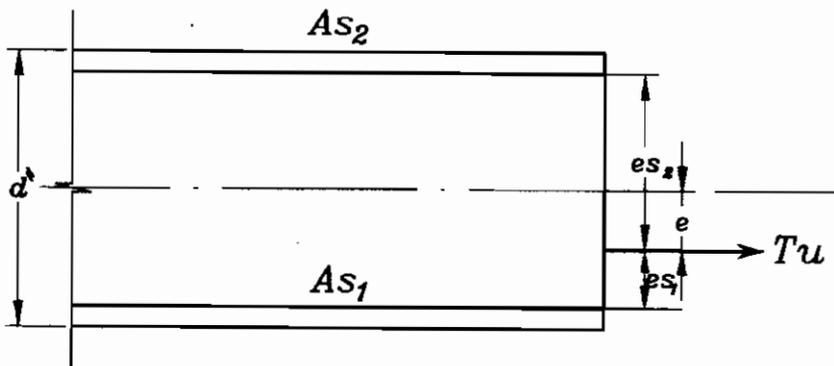
$$e = \frac{M_u}{T_u} \quad \text{Where : } e \leq \frac{d - d'}{2}$$

$$e_{s1} = \frac{t}{2} - e - \text{cover.}$$

$$A_{s1} = \left( \frac{T_u \cdot e_{s2}}{e_{s1} + e_{s2}} \right) / \frac{f_y}{\gamma_s}$$

$$e_{s2} = \frac{t}{2} + e - \text{cover}$$

$$\therefore A_{s2} = \left( \frac{T_u \cdot e_{s1}}{e_{s1} + e_{s2}} \right) / \left( \frac{f_y}{\gamma_s} \right)$$



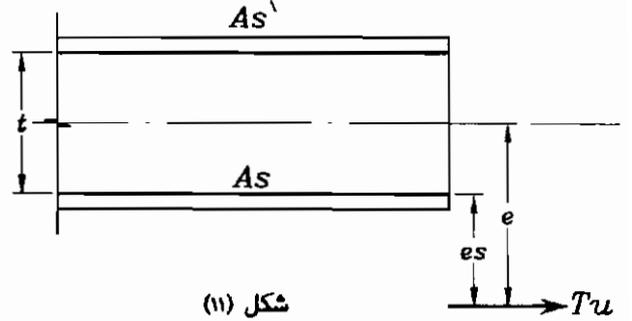
شكل (١٠)

الثابتية : أن تكون القوة المؤثرة لها لا مركزية كبيرة خارج أبعاد القطاع ( شكل ١١ ) ، حيث يمكن نقل نقطة تأثير القوة لمركز الحديد الذي يتحمل هذه القوة ، و يمكن أن تتحمل الخرسانة نصيباً منها أو يمكن الاستعانة بحديد ثانوي  $A_s'$  باعتبار القطاع ذا تسليح مزدوج double reinforced section .

$$e = M_u / T_u$$

$$e_s = e - \frac{t}{2} + \text{cover.}$$

$$M_{us} = T_u \cdot e_s$$



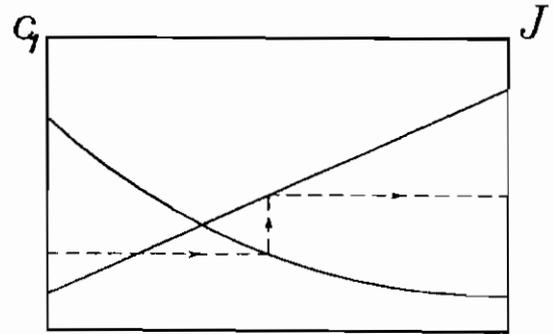
و يمكن استخدام المخطط البياني رقم ( ١ ) إيجاد صلب التسليح  $A_s$  ومن ثم  $A_s'$  ، كما يأتي :

أولاً : باستخدام المخطط البياني C - J :

$$d = C_1 \sqrt{\frac{M_{us}}{f_{cu} b}} \rightarrow \text{get } C_1$$

from Chart  $\rightarrow$  get J

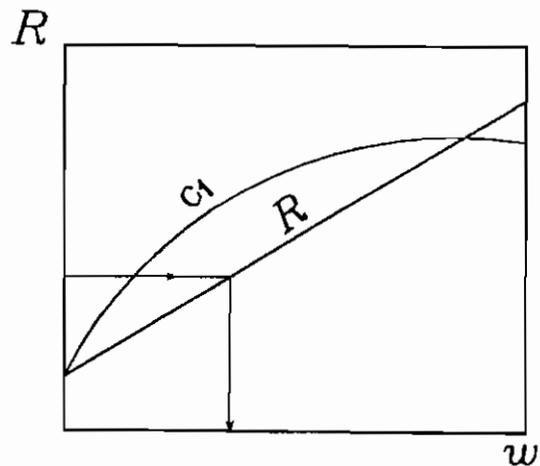
$$A_s = \frac{M_{us}}{J * d * f_y} + \frac{T_u}{f_y / \gamma_s} \quad (\gamma_s = 1.15)$$



ثانياً : باستخدام المخطط البياني R - w :

$$R = \frac{M_{us}}{bd^2 f_{cu}} \rightarrow \text{get } w$$

$$A_s = wbd \frac{f_{cu}}{f_y} + \frac{T_u}{f_y / \gamma_s}$$



ملء الفراغات بتصميم القطاعات غير المتحركة :

Steps of Design of sec. subjected to  $M_u, P_u$  (comp) :

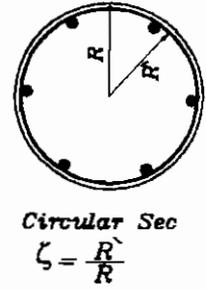
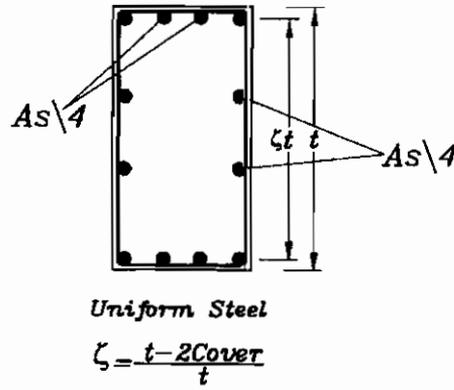
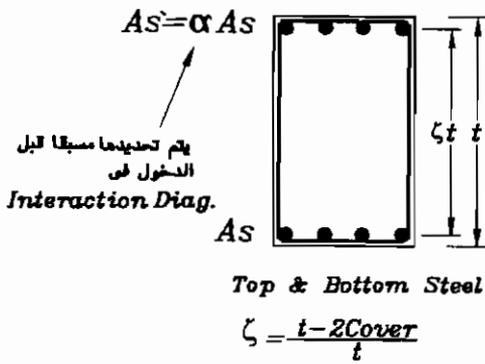
- يتم فرض أبعاد تقريبية بالنسبة للقطاع باعتبار عدم وجود عزوم ( معادلة الكود ) :
- إذا كان مستطيل ( b ) عرض القطاع .
- طول القطاع ( t ) .
- إذا كان هذا العمود دائرياً يتم فرض القطر .

وهذا الفرض يمكن زيادته بنسبة ٤٠ % لاستيعاب قيمة العزوم أيضاً .

- يمكن الآن الدخول في منحنيات التفاعل :

1 -  $f_y = \text{Given}$

2 -  $\zeta = \frac{t - 2 \text{ cover}}{t}$



- من الأبعاد الافتراضية للقطاع يتم حساب :

( المحور الأفقي ) :  $k \frac{e}{t} = \frac{M_u}{f_{cu} b t^2}$  ، ( والمحور الرأسى ) :  $k = \frac{P_u}{f_{cu} b t}$

ومن ثم نوجد  $\rho \leftarrow \mu \leftarrow A_s \leftarrow A_s$  .

ويمكن أن تكون حالة القطاع إحدى الحالات الآتية :

- ١ - منطقة ( A ) حيث يتم تصميم القطاع على أن تتحمل قوي ضغط فقط .  
 ٢ - منطقة ( B ) حيث يتم إيجاد (  $\rho$  ) من المخططات البيانية ثم نوجد (  $\mu$  ) .

$$\mu = \rho f_{cu} * 10^{-5} \quad \text{ثم نوجد}$$

$$A_s \mu b t$$

$$A_s^- = \alpha A_s$$

- ٣ - منطقة ( C ) حيث :

$$e = \frac{M_u}{P_u}$$

$$e_s = e + \frac{t}{2} - \text{cover}$$

$$M_{us} = P_u \cdot e_s$$

$$R = \frac{M_{us}}{f_{cu} b d^2} < \frac{R_{\max}}{\gamma_c} \quad (\text{table 4-1}) \quad E.C$$

Using R -  $\omega$  chart , get  $\omega$  :

$$A_s = \omega \frac{f_{cu}}{f_y} \cdot b \cdot d - \frac{P_u}{f_y / \gamma_s}$$

- ٤ - منطقة ( D ) حيث : (  $P_u \leq 0.04 f_{cu} A_c$  )  
 و يتم التصميم على أساس وجود عزم فقط .

**Example ( 1 ) :**

Design a R.C Section:

$$P_u = -200 \text{ t}$$

$$M_u = 5 \text{ m.t.}$$

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$f_y = 3600 \text{ kg/cm}^2$$

$$b = 30 \text{ cm}$$

**Solution :**

$$e = \frac{M_u}{P_u} = \frac{5}{200} = 0.025 \text{ m} = 2.5 \text{ cm.}$$

Assume  $\frac{e}{t} \leq 0.05$  min ecc

Zone(A) :

$$P_u = 0.35 A_c F_{cu} + 0.67 A_s f_y$$

Assume  $A_s = 1\% A_c$

$$200 * 10^3 = A_c (0.35 * 250 + 0.67 * \frac{3600}{100})$$

$$A_c = 1791.79 \text{ cm}^2 = 30 * t \text{ cm}^2$$

$$t = 59.7 \text{ cm}$$

$$t = 60 \text{ cm}$$

$$\frac{e}{t} = \frac{2.5}{60} = 0.04167 < 0.05$$

The assumption is correct (zone A)

$$A_s = 0.01 * 1791.79$$

$$\cong 18 \text{ cm}^2$$

Choose : 10ϕ16 ( $A_s = 20.11 \text{ cm}^2$ )

Check :

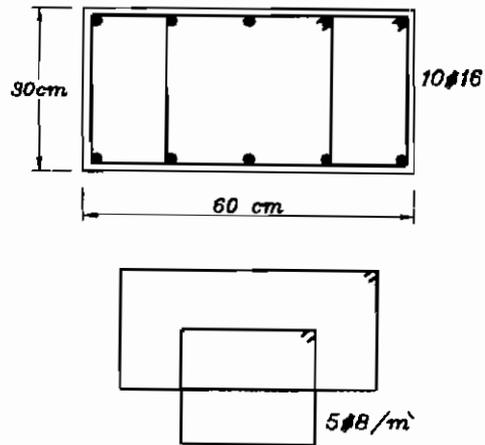
$$A_{s_{min}} = \frac{0.8}{100} * 1791.79 = 14.33 \text{ cm}^2 < A_s \Rightarrow (OK)$$

$$= \frac{0.6}{100} * 30 * 60 = 10.8 \text{ cm}^2 < A_s \Rightarrow (OK)$$

Use 5ϕ8/m as stirrups :

$$V_{s_{min}} = \frac{0.25}{100} * 30 * 60 * 100 = 450 \text{ cm}^3$$

$$V_{s_{act}} = 5 * \frac{\pi}{4} (0.8)^2 [25 * 4 + 55 * 2 + 27.5 * 2] = 666 \text{ cm}^3 > V_{s_{min}} \Rightarrow (OK).$$



**Example (2):** (شكل ١٢)

Design a R.C Section to carry:

$P_u = -200 \text{ t.}$                        $M_u = 25 \text{ m.t.}$   
 $b = 30 \text{ cm.}$                                $f_{cu} = 250 \text{ kg/cm}^2$

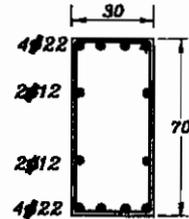
**Solution:**

$$e = \frac{M_u}{P_u} = \frac{25 * 10^5}{200 * 10^3} = 12.5 \text{ cm.}$$

Assume  $t = 70 \text{ cm}$

$$\frac{e}{t} = 0.178 > 0.05 \text{ assume Zone(B)}$$

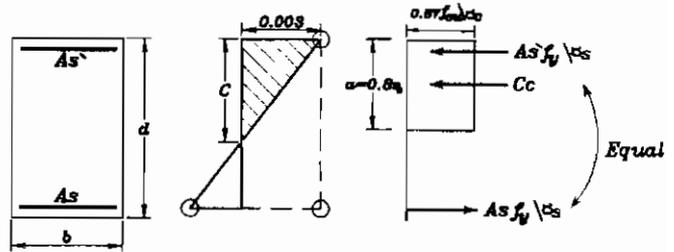
$P_u > P_b \dots \text{Comp.}$



Zone(B):

$$K = \frac{P_u}{F_{cu} b t} = \frac{200 * 10^3}{250 * 30 * 70} = 0.381$$

$$K \frac{e}{t} = 0.381 * \frac{12.5}{70} = 0.068$$



شكل (١٢) مثال (٢)

Take  $\alpha = 1.0$                        $f_y = 3600 \text{ kg/cm}^2$

$$\zeta = \frac{t - 2 \text{ cover}}{t} = \frac{70 - 2 * 2.5}{70} = 0.9$$

Chart No. 27

$$\rho = 2.4$$

$$\mu = \rho f_{cu} \times 10^{-5}$$

$$= 2.4 * 250 * 10^{-5} = 0.006$$

$$A_s = A_{s'} = \mu b t$$

$$= 0.006 * 30 * 70 = 12.6 \text{ cm}^2$$

(4φ22) for each side

$$e = \frac{M_u}{P_u} = 0.125 \quad , \quad \frac{e}{t} = 0.178$$

$$\gamma_c = 1.75 - 0.5 \frac{e}{t} = 1.6607 < 1.5$$

$$\gamma_s = 1.36 - 0.43 \frac{e}{t} = 1.283 < 1.15$$

$$a = 0.8 C$$

$$= 0.8 \left( \frac{0.003}{0.003 + \epsilon_y / \gamma_s} \right) d$$

$$\frac{C_b}{d} = \frac{0.003}{0.003 + \epsilon_s / \gamma_s}$$

$$P_b = \frac{0.67 f_{cu}}{\gamma_c} a_b b + A_s f_y / \gamma_s - A_s f_y / \gamma_s$$

$$= (0.67 f_{cu} / \gamma_c) b d * 0.8 \left( \frac{0.003}{0.003 + \epsilon_y / \gamma_s} \right)$$

$$= 0.536 \frac{f_{cu}}{\gamma_c} b d \left( \frac{6000}{6000 + f_y / \gamma_s} \right)$$

$$\cong 114.47 t < 200$$

∴ Compression failure

**Example (3):** (شكل ١٣)

Design a R.C Section:

$$P_u = 40 t$$

$$M_u = 40 m.t.$$

$$b = 30 \text{ cm}$$

**Solution :**

Assume Zone C :

$$d = C_1 \sqrt{\frac{M_{us}}{f_{cu} b}}$$

Assume  $C_1 = 3$

$$= 3 \sqrt{\frac{40 * 10^5}{250 * 30}}$$

$$d = 69.28 \cong 70 \text{ cm}$$

$$t = 1.2 * 70 = 84 \cong 90 \text{ cm}$$

$$t = 90 \text{ cm}$$

$$e = \frac{M_u}{P_u} = 1 \text{ m}$$

$$e/t = \frac{1}{0.9} = 1.11 > 0.05$$

$$P_b = \frac{0.67 f_{cu} b * 0.8 C_b}{\gamma_c}$$

$$= \frac{0.67 * 250}{1.5} * 30 * 0.8 \frac{6000}{6000 + \frac{3600}{1.15}} * 85$$

$$P_b = 149.7 \text{ t} > P_u \therefore \text{tension failure}$$

$$0.04 f_{cu} b t = 0.04 * 250 * 30 * 90 = 27 \text{ t}$$

$$27 < P_u < 149$$

$$\therefore 0.04 f_{cu} b t < P_u < P_b$$

(Zone C)

$$e = \frac{M_u}{P_u} = 100 \text{ cm}$$

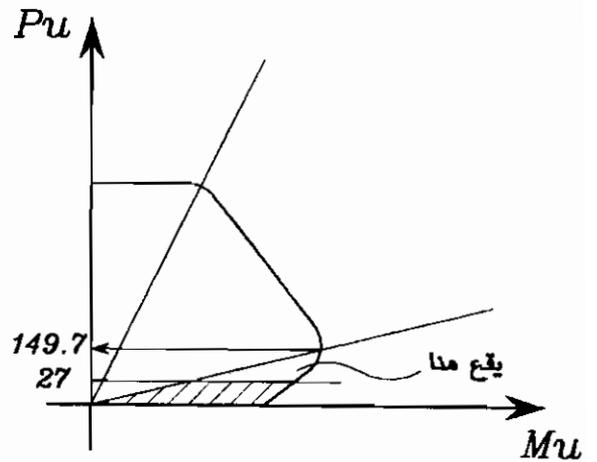
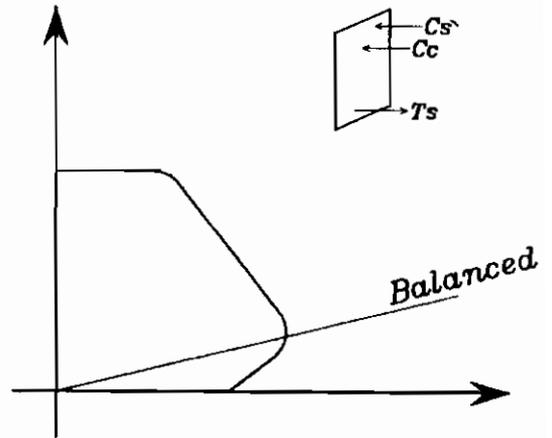
$$e_s = e + \frac{t}{2} - \text{cover} = 100 + \frac{90}{2} - 5 = 140 \text{ cm}$$

$$M_{us} = P_u * e_s = 56 \text{ mt}$$

$$R = \frac{M_{us}}{f_{cu} b d^2} = \frac{56 * 10^5}{250 * 30 * 85^2} = 0.1033$$

Chart(1):  $w = 0.138$

$$A_s = w b d \frac{f_{cu}}{f_y} - \frac{P_u}{f_y / \gamma_s} = 0.138 * 30 * 85 * \frac{250}{3600} - \frac{40 * 10^3}{\frac{3600}{1.15}} = 11.70 \text{ cm}^2 \quad (6 \text{ \#} 16)$$



شكل (١٣) مثال (٣)

:  $C_1$  ممكن بطريقة

$$M_{us} = 56 \text{ m.t}$$

$$d = C_1 \sqrt{\frac{M_{us}}{f_{cu} \cdot b}} \rightarrow \text{get } C_1 = 3.11$$

$$\rightarrow \text{get } J = 0.75$$

$$A_s = \frac{M_u}{f_y J d} - \frac{P_u}{\frac{f_y}{\gamma_s}} = \frac{56 \times 10^5}{3600 \cdot 0.75 \cdot 85} - \frac{40 \times 10^3}{3600/1.15} = 11.623 \text{ cm}^2$$

Check :

$$\mu_{\max} = 5 \times 10^{-5} f_{cu} = 0.0125 \text{ .....(Eq}^n \text{ 4-1 code)}$$

$$A_{s_{\max}} = 0.0125 \times 30 \times 85 = 31.875 \text{ cm}^2 > A_{s_{act}} \rightarrow (OK)$$

$$\mu_{\min} = \frac{11}{f_y} = \frac{11}{3600} \rightarrow A_{s_{\min}} = \frac{11}{3600} \times 30 \times 85 = 7.8 \text{ cm}^2 < A_{s_{act}} \rightarrow (OK)$$

**Example (4):**

( شكل ١٤ )

Design a R.C Section to carry:

$$P_u = 15 \text{ t}$$

$$M_u = 60 \text{ m.t.}$$

$$b = 30 \text{ cm}$$

**Solution:**

It may be zone D because  $M_u > P_u$

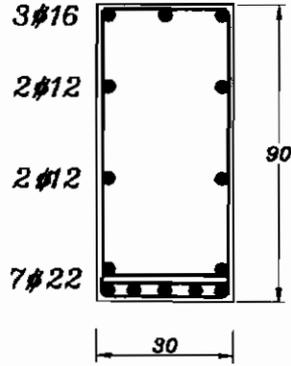
$$d = c \sqrt{\frac{M_u}{F_{cu} b}}$$

assume  $C = 3$

$$d = 3 \sqrt{\frac{60 * 10^5}{250 * 30}}$$

$$d = 84.9 \text{ cm}$$

Take  $t = 90 \text{ cm}$



شكل (١٤) مثال (٤)

Code item (4-2-1-3)

$$0.04 f_{cu} bt = 0.04 * 250 * 30 * 90 = 27 \text{ t}$$

$$P_u < 0.04 f_{cu} bt$$

∴ Zone D

∴ Neglect  $p_u$  & design as a beam (flexure failure)

$$R = \frac{60 * 10^5}{30 (85) (250)^2} = 0.11$$

$$W = 0.15$$

$$A_s = 0.15 * 30 * 85 * \frac{250}{3600} = 26.5625 \text{ cm}^2 \quad (7\#22)$$

**Example ( 5 ) :**

( شكل ١٥ )

Design a R.C Section to carry:

$$P_u = + 50 \text{ t}$$

$$M_u = 5 \text{ m.t.}$$

$$b = 30 \text{ cm}$$

$$t = 80 \text{ cm}$$

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$f_y = 3600 \text{ kg/cm}^2 .$$

**Solution :**

**Method ( 1 ) :**

$$e = 5 / 50 = 0.1 \text{ m} < 0.5 \text{ t} \quad \text{small sec.}$$

$$e_{s1} = \frac{t}{2} - e - \text{cover}$$

$$= 0.4 - 0.1 - 0.025 = 0.275$$

$$e_{s2} = \frac{t}{2} + e - \text{cover}$$

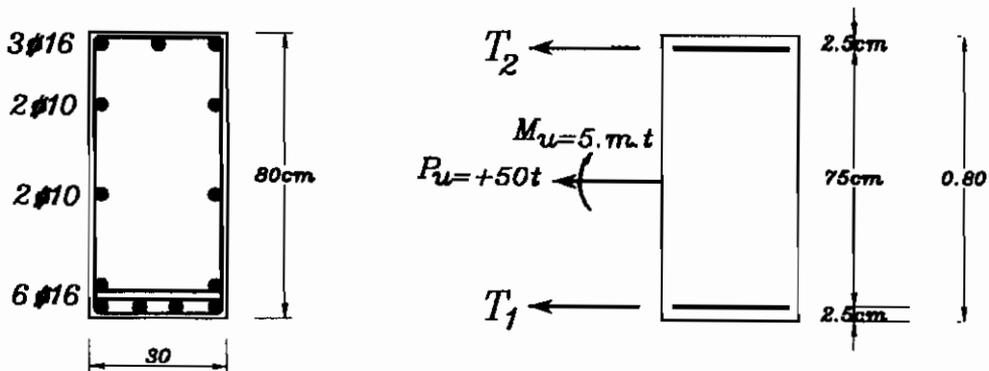
$$= 0.4 + 0.1 - 0.025 = 0.475$$

$$e_{s1} + e_{s2} = 0.75 \text{ m}$$

$$A_{s1} = \frac{50 * 0.475 * b^3}{0.75} / \frac{3600}{1.15} = 10.12 \text{ cm}^2 \quad (6\phi 16)$$

$$A_{s2} = \frac{50 * 0.275 * 10^3}{0.75} / \frac{f_y}{\gamma_s}$$

$$= \frac{18333.33}{3600/1.15} = \frac{18333.3}{3130.4} = 5.86 \text{ cm}^2 \quad (3\phi 16)$$



شكل (١٥) مثال (٥)

**Example (6):**

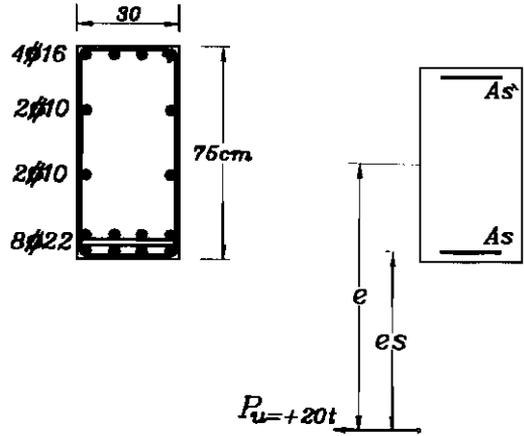
(شكل ١٦)

Design a R.C Section to carry:

$$P_u = +20 \text{ t}$$

$$M_u = 50 \text{ m.t.}$$

$$b = 30 \text{ cm}$$



شكل (١٦) مثال (٦)

**Solution :**

$$d_o = c_1 \sqrt{\frac{M_u}{f_{ck} b}}$$

assume  $c = 3$

$$= 3 \sqrt{\frac{50 * 10^5}{250 * 30}} = 77.5 \text{ cm}$$

$$d = 0.9 d_o \cong 70 \text{ cm}$$

نقل مشاركة الخرسانة في tension .

$$t = 75 \text{ cm}$$

$$e = \frac{50}{20} = 250 \text{ cm}$$

$$e/t = 3.33 > 0.5 \quad \therefore \text{big ECC}$$

$$e_s = e - \frac{t}{2} + \text{cover}$$

$$e_s = 250 - \frac{75}{2} + 5 = 217.5 \text{ cm}$$

$$M_{u_s} = P_u \cdot e_s = 43.5 \text{ m.t}$$

$$R = \frac{M_{u_s}}{f_{ck} b d^2} = \frac{43.5 * 10^5}{250 * 30 * (70)^2} = 0.118$$

$$R = 0.118 \quad , \quad \frac{d^-}{d} = \frac{5}{70} = 0.07$$

$$w = 0.15 \quad , \quad \alpha = 0.3 \quad (\text{from table 1 design Aids})$$

$$A_s = 0.15 * 30 * 70 \frac{250}{3600} + \frac{20 * 10^3}{\frac{3600}{1.15}} = 28.26 \text{ cm}^2$$

$$(8 \text{ } \phi 22) = 30.4 \text{ cm}^2$$

$$A_{s_1} = 0.3 (0.15 * 30 * 70 \frac{250}{3600}) = 6.5625 \text{ cm}^2$$

$$(4 \text{ } \phi 22) = 8.04 \text{ cm}^2.$$

**Example ( 7 ) :** ( شكل ١٧ )

**Case ( A ) :**

Design the following for the given cases

Case	$M_u (m.t)$	$N_u (t)$	$t (cm)$	$f_{cu}$	$f_y$	$a$
A	53	-16	70	250	3600	0.4
B	42	-35	70	250	3600	?
C	42	-100	70	250	3600	0.8
D	23	-150	60	300	3600	1.0
E	100	-19	?	300	3600	0.0
F	45	-210	80	300	3600	1.0

A ) given  $N_u = - 16 t$   $M_u = 53 m.t$

$$0.04 f_{cu} bt = 0.04 * 250 * 25 * 70 = 17500 kg$$

$$N_u < 0.04 f_{cu} bt \quad (\text{neglect } N_u)$$

Design  $n$  for moment only

$$M_u = 53 m.t$$

$$R = \frac{M_u}{f_{cu} bd^2} = 0.2 \quad , \alpha = 0.4$$

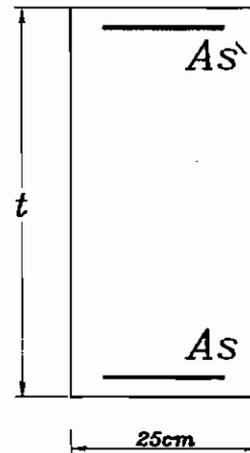
using  $R - w$  table (table 2) page 13

$$w = 0.27$$

$$A_s = wbd f_{cu} / f_y = 0.27 * 25 * 65 * \frac{250}{3600}$$

$$= 30.47 cm^2 = 8\phi 22$$

$$A_s^- = 0.4 A_s = 12.19 cm^2 = 5\phi 18$$



شكل (١٧) مثال (٧)

**Case ( B ) :**

Given  $N_u = -35 \text{ t}$

$M_u = 42 \text{ m.t}$

$N_u > 0.04 f_{cu} bt = 17.5 \text{ t}$

(calculated before)

$$e = \frac{M_u}{N_u} = \frac{42}{35} = 1.2 \text{ m}$$

$$e_s = \frac{M_u}{N_u} + t/2 - 0.05 = 1.5 \text{ m}$$

$$M_{us} = N_u \cdot e_s$$

$$M_{us} = 52.5 \text{ m.t}$$

using (R-W)table (2) page (13)

$$R = \frac{52.5 * 10^5}{250 * 25 * (65)^2} = 0.198 > (R_{max} / \gamma_c = 0.129)(\text{table 4 - 1})$$

use comp steel :

$$\alpha = 0.4$$

$$w = 0.27$$

$$A_s = wbd \frac{f_{cu}}{f_y} - \frac{N_u}{f_y / \gamma_s}$$

$$= 0.27 * 25 * 65 * \frac{250}{3600} - \frac{35 * 10^3}{3600 / 1.15} = 19.29 \text{ cm}^2 \quad (8\text{ff}18)$$

$$A_s^- = \alpha wbd f_{cu} / f_y = 12.18 \text{ cm}^2. \quad (5\text{ff}18)$$

**Case ( C ) :**

$$M_u = 42 \text{ m.t}$$

$$N_u = -100 \text{ t}$$

$$t = 70 \text{ cm}$$

$$\alpha = 0.8$$

$$e = \frac{M_u}{N_u} = 0.42$$

Using interaction diag . (Chart No.26)

$$\zeta = 0.857 \cong \frac{60}{70} = 0.9$$

$$f_y = 3600 \text{ kg/cm}^2$$

$$\alpha = 0.8$$

$$K = \frac{P_u}{f_{cu} b t} = \frac{100 * 10^3}{250 * 25 * 70} = 0.229$$

$$K \cdot \frac{e}{t} = 0.137$$

$$\rho = 3.2$$

$$\mu = \rho * f_{cu} * 10^{-5} = 3.2 * 250 * 10^{-5} = 0.008$$

$$A_s = \mu b t = 14 \text{ cm}^2 \quad (7 \text{ \#} 16)$$

$$A_s^- = \alpha A_s = 11.2 \text{ cm}^2. \quad (6 \text{ \#} 16)$$

**Case ( D ) :**

Given:

$$N_u = -150 \text{ t}$$
$$f_{cu} = 300 \text{ kg/cm}^2$$

$$M_u = 23 \text{ m.t}$$
$$f_y = 3600 \text{ kg/cm}^2$$

$$e = \frac{23}{150} = 0.153 \text{ m} > e_{\min} \quad \text{code (4-2-1-3)} \quad \alpha = 1$$

use interaction diag (chart No.17) (uniformly distributed steel)

$$\zeta = \frac{50}{60} = 0.83 \cong 0.8 = 0.8 \quad f_y = 3600 \text{ kg/cm}^2$$

$$f_{cu} = 300 \text{ kg/cm}^2$$

$$K = \frac{P_u}{f_{cu} b t} = \frac{150 * 10^3}{300 * 25 * 60} = 0.33$$

$$K \cdot \frac{e}{t} = 0.085$$

$$\rho = 8$$

$$\mu = \rho f_{cu} * 10^{-5} = 0.024$$

$$A_s = \mu b t = 0.024 * 25 * 60 = 36 \text{ cm}^2 \quad (10\phi 22)$$

$$A_s^- = \alpha A_s = 36 \text{ cm}^2 \quad (10\phi 22)$$

**Case ( E ) :**

$$M_u = 100 \text{ m.t}$$

$$N_u = -19 \text{ t}$$

$$t = ?$$

$$f_{cu} = 300 \text{ kg/cm}^2$$

$$f_y = 3600 \text{ kg/cm}^2$$

We can assume ( t ) to design as member as subjected to moment only .

$$d = C_1 \sqrt{\frac{M_u}{b f_{cu}}} \quad \text{take } C_1 = 3$$

$$\therefore d = 109.5 \text{ cm} = 110 \text{ cm}$$

To make it single Rfd

$$R = \frac{100 * 10^5}{300 * 110^2 * 25} = 0.11 < R_{\max} \quad (O.K)$$

$$W = 0.148$$

$$A_s = w.b.d.f_{cu} / f_y = 33.917 \text{ cm}^2$$

$$9\phi\phi 22$$

Check that the section is designed to carry moment only:

$$0.04 f_{cu} bt = 34.5 \text{ t} > P_u \quad (O.K)$$

**Case ( F ) :**

Given:

$$t = 80 \text{ cm}$$

$$\alpha = 1.00$$

$$\zeta = \frac{70}{80} = 0.875 \cong 0.9$$

$$M_u = 45 \text{ m.t}$$

$$N_u = -210 \text{ t}$$

Case ( 1 ) : Using interaction diag. ( chart No. 27 ) :

$$K = \frac{P_u}{f_{cu} b t} = 0.35$$

$$K \cdot \frac{e}{t} = \frac{M_u}{f_{cu} b d^2} = 0.107$$

$$\rho = 3.5$$

$$\mu = \rho * f_{cu} * 10^{-5} = 0.0105$$

$$A_s = A_s^- = \mu b t = 21 \text{ cm}^2$$

Case(2) : using interaction diag chart N° .25

$$\alpha = 0.6 \quad \zeta = \frac{70}{80} = 0.9$$

$$\rho = 4.4$$

$$\mu = 0.0132$$

$$A_s = 26.4 \text{ cm}^2$$

$$A_s^- = \alpha A_s = 15.84 \text{ cm}^2$$

**Example (8)**

( شکل ۱۸ )

Case	$M_u (m.t)$	$N_u (t)$	$t (cm)$	$f_{cu}$	$f_y$
A	65	-16	70	250	3600
B	65	+16	70	250	3600

**Case ( A ) :**

$M_u = 65 \text{ m.t.}$

$N_u = - 16 \text{ t.}$

$t = 70 \text{ cm}$

$f_{cu} = 250 \text{ kg/cm}^2$

$f_y = 3600 \text{ kg/cm}^2 .$

$t_s = 10 \text{ cm.}$

assuming the C.G of the cross section is at middle of the depth ( t )

assume  $a < t_s$

$$e = \frac{M_u}{N_u} = \frac{65}{16} = 4.06 \text{ m}$$

$$e_s = e + \frac{t}{2} - \text{cover}$$

$$= 4.06 + \frac{0.7}{2} - 0.05 = 4.36 \text{ m}$$

$$M_{us} = 16 * 4.36 = 69.76 \text{ t.m}$$

USING ( C - J ) chart(2) (page 4)

$$d = C_1 \sqrt{\frac{M_{us}}{f_{cu} \cdot B}}$$

$C_1 = 4.26$

$(c/d) = 0.163$

$c = 10.6 \text{ cm}$

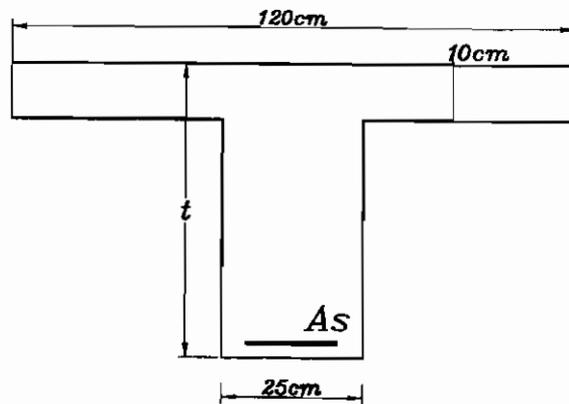
$a = 0.8 \text{ c}$

$a = 8.48 \text{ cm} < t_s$

then the assumption that  $(a < t_s)$  is valid

$J = 0.81$

$$A_s = \frac{M_{us}}{f_y \cdot J \cdot d} - \frac{N_{us}}{f_y / 1.15} = \frac{69.76 * 10^5}{3600 * 0.81 * 65} - \frac{16 * 1000}{(3600 / 1.15)} = 31.7 \text{ cm}^2$$



شکل (۱۸) مثال (۸)

**Case ( B ) :**

$$M_u = 65 \text{ m.t.}$$

$$t = 70 \text{ cm}$$

$$f_y = 3600 \text{ kg/cm}^2 .$$

$$N_u = + 16 \text{ t.}$$

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$e = \frac{65}{16} = 4.06 \text{ m}$$

$$e_s = \frac{M_u}{N_u} - \frac{t}{2} + \text{cover} = 3.7625 \text{ m}$$

$$M_{us} = N_u \cdot e_s = 60.2 \text{ m.t}$$

Using C – J curve:

$$d = C_1 \sqrt{\frac{M_{us}}{f_{cu} \cdot B}}$$

$$C_1 = 4.59$$

$$\frac{c}{d} = 0.14$$

$$C = 9.1 \text{ cm}$$

$$a = 0.8 c$$

$$a = 7.28 \text{ cm} < t_s \rightarrow (O.K)$$

$$j = 0.82$$

$$A_s = \frac{M_{us}}{f_y \cdot j \cdot d} + \frac{N_u}{(f_y / \gamma_s)} = 36.46 \text{ cm}^2$$

**Example (9)**

(شكل ١٩)

Case (A) :

Case	$M_u$ (m.t)	$N_u$ (t)	$t$ (cm)	$f_{cu}$	$f_y$
A	15	+ 90	60	250	3600
B	65	+ 90	60	250	3600

$M_u = 15 \text{ m.t.}$

$N_u = + 90 \text{ t.}$

$t = 60 \text{ cm}$

$f_{cu} = 250 \text{ kg/cm}^2$

$f_y = 3600 \text{ kg/cm}^2$

$$e = \frac{M_u}{N_u} = \frac{15}{90} = 0.166 \text{ m}$$

$$\frac{d - d'}{2} = \frac{50}{2} = 25$$

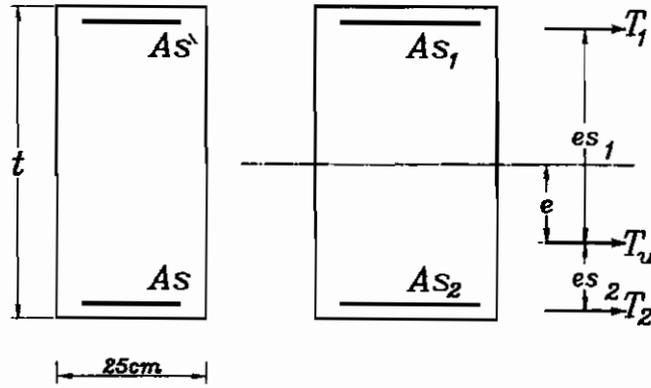
$$e < \frac{d - d'}{2}$$

$$e_{s1} = 60/2 - 16.6 - 5 = 8.4 \text{ cm}$$

$$e_{s2} = 60 + 16.6 - 5 = 71.6 \text{ cm}$$

$$T_1 = \frac{T_u \cdot e_{s2}}{e_{s1} + e_{s2}} = \frac{90 \cdot 71.6}{80} = +80.55 \text{ t}$$

$$T_2 = \frac{T_u \cdot e_{s1}}{e_{s1} + e_{s2}} = \frac{90 \cdot 8.4}{80} = 9.45 \text{ t}$$



شكل (١٩) مثال (٩)

$T_2 = 60 \text{ t}$

$$A_{s1} = T_1 / (f_y / \gamma_s) = \frac{80.55}{(3.6/1.15)} = 25.73 \text{ cm}^2$$

$$A_{s2} = \frac{9.45}{(3.6/1.15)} = 3.02 \text{ cm}^2$$

**Case ( B ) :**

$$M_u = 55 \text{ m.t.}$$

$$t = 60 \text{ cm}$$

$$f_y = 3600 \text{ kg/cm}^2 .$$

$$N_u = + 90 \text{ t.}$$

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$e = \frac{M_u}{N_u} = 0.61 \text{ m} > \left( \frac{d - d'}{2} = \frac{50}{2} = 25 \right)$$

$$e_s = e - \frac{t}{2} + \text{cover}$$

$$= 61.1 - \frac{60}{2} + 5 = 36.1 \text{ cm}$$

$$M_{us} = N_u e_s = 32.5 \text{ m.t}$$

using (R - W) table (2) page (13) :

$$R = \frac{M_{us}}{f_{cu} b d^2} = 0.172$$

$$\alpha = 0.3$$

$$w = 0.225$$

$$A_s = w b d \frac{f_{cu}}{f_y} + \frac{N_u}{f_y \gamma_s}$$

$$= 0.225 * 25 * 55 * \frac{250}{3600} + \frac{90 * 10^3}{(3600/1.15)} = 50.23 \text{ cm}^2$$

$$A_s^- = \alpha \left( w b d \frac{f_{cu}}{f_y} \right) = 6.44 \text{ cm}^2 .$$

**Example ( 10 )**

( شكل ٢٠ )

Design the exterior column shown in figure (A)

Data:

$F_{cu} = 200 \text{ kg/cm}^2$

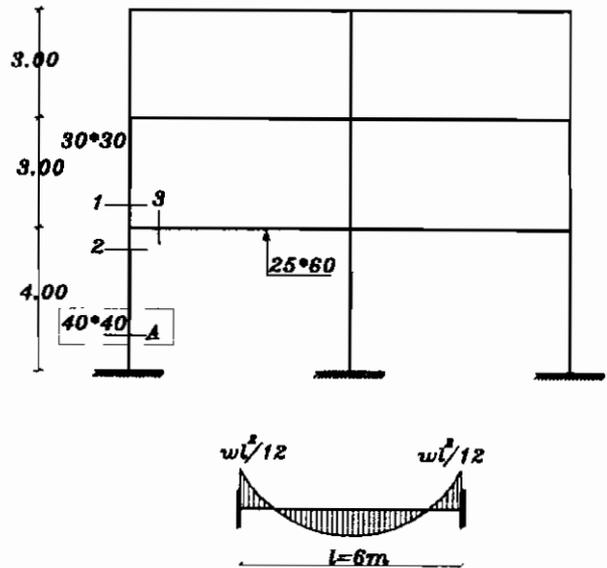
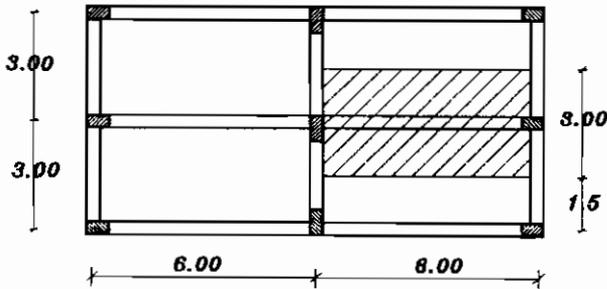
$L.L = 215 \text{ kg/cm}^2$

$P_{L.L} = 50 \text{ ton}$

Assume that the building is unbraced.

$f_y = 2400 \text{ kg/cm}^2$

$P_{D.L} = 50 \text{ ton}$



(شكل ٢٠)

**Solution:**

$W_{us} = 1.4[0.10(2500) + 200] + 1.6(215) = 975 \text{ kg/cm}^2$

$W_{ub} = 1.4 (2.5) 0.25 (0.6) + 3 (0.975) = 3.45 \text{ t/m}$

$M_t = \frac{3.45 (6)^2}{12} = 10.35 \text{ mt}$

$K =$  stiffness of the member  $= 4 EI/L$  ( for member fixed from both ends )

We can also use relative stiffness as all member from concrete ( having the same E )

Relative stiffness  $= I/L$  .....( Fixed – Fixed )

$= 3 I / 4 L$  .....( Fixed – Hinged )

From code page 154 and for two bays or more , the moment over column ends can be calculated as follows:

$$M_n = \frac{K_n M_f}{\Sigma K} = K_r M_f \dots \text{where } K_r = \frac{K_n}{\Sigma K}$$

These calculations are illustrated in a tabulated form

Nº	Section	I	L	K	$K_r = K / \Sigma K$	$M = K_r (M_f)$
1	30 * 30	67500	300	225.0	0.149	1.55
2	40 * 40	213333	400	533.3	0.353	3.65
3	25 * 60	450000	600	750.0	0.498	5.15**
$\Sigma K$				<b>1508.3</b>	$\Sigma M = 10.35$	

$$H_o = 4.0, H_e = 0.75 (4.0) = 3.0 \text{ m}$$

$$\lambda = \frac{3.0}{0.40} = 7.5 < 10 \quad ((\text{short column}))$$

$$P_u = 1.4(50) + 1.6(50) = 150 \text{ t}$$

$$K = \frac{150 \times 1000}{200 (40) 40} = 0.468$$

$$K \cdot \frac{e}{t} = \frac{3.65 \times 10^5}{200 (40) 40^2} = 0.028$$

Use chart Nº.11 page 16:

$$\rho = 8.0 \dots \dots \dots \mu_{tot} = 8(200)10^{-5} = 0.016$$

$$A_{s(\text{total})} = 0.016 (40) (40) = 25.6 \text{ cm}^2$$

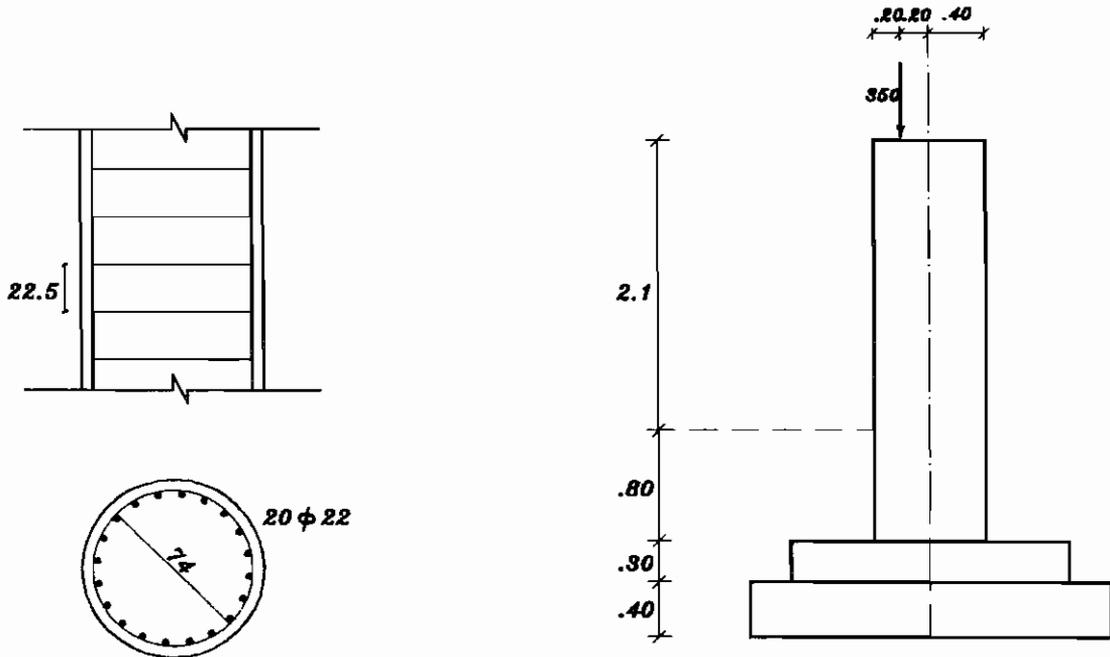
$$A_{s(\text{chosen})} = 27.32 \text{ cm}^2 (4\phi 19 + 8\phi 16)$$

**\*\* Note:** the value of the moment transferred to beam (5.15 m.t) is in a good agreement with the value of the assumed - ve bending moment on beam edges suggested by the code  $(WL^2 / 24)$   
 $= 3.45 (6)^2 / 24 = 5.17 \text{ m.t.}$

**Example (11)**

( شکل ۲۱ )

Check the column shown in figure if  $f_{cu} = 250 \text{ kg/cm}^2$  and  $f_y = 2400 \text{ kg/cm}^2$   
 $P_u = 350 \text{ ton}$ .



شکل ( ۲۱ )

The column is circular with closed stirrups.

$$\zeta = 0.74 / 0.8 = 0.92$$

the column fixed at the foundations and free from the top (unbraced)

From table ( 6-10 ) .....K = 2.2

$$H_e = 2.2 ( 2.1 + 0.8 ) = 6.38 \text{ m}$$

Use chart No . 46 page 51

$$\lambda = \frac{6.38}{0.8} = 7.98 < 8 \dots\dots\dots(\text{table 6-7}) \text{ short column}$$

$$R = D / 2 = 40 \text{ cm}$$

$$\text{from fig., } e = 0.2 \text{ m, } e / R = 20 / 40 = 0.5$$

$$K = \frac{P_u}{f_{cu} R^2} = \frac{350 \times 1000}{250 \times 40^2} = 0.875$$

Enter the chart with  $e / R = 0.5$  and  $k = 0.875$

$$\text{Or } M_u = 350 (0.2) = 70 \text{ m.t}$$

$$k \cdot \frac{e}{R} = \frac{M_u}{f_{cu} R^3} = \frac{70 \times 10^5}{250 \times 40^3} = 0.4375$$

Enter the chart with  $k = 0.875$  and  $k(e/R) = 0.4375$

$$\rho = 7$$

$$\mu = 7(250) 10^{-5} = 0.0175 > \mu_{\min} (1\%)$$

$$A_{s(\text{required})} = 0.0175(\pi) 40^2 = 87.9 \text{ cm}^2$$

$$\text{but } A_{s(\text{provided})} = 20 \sqrt[3]{22} (76 \text{ cm}^2)$$

$a_s A_{s(\text{required})} > A_{s(\text{provided})}$  .....the column considered unsafe

Also the spacing between stirrups (22.5 cm ) is bigger than the value allowed by the code (20 cm ) article ( 6 – 4 – 7 ).

**Example (12)**

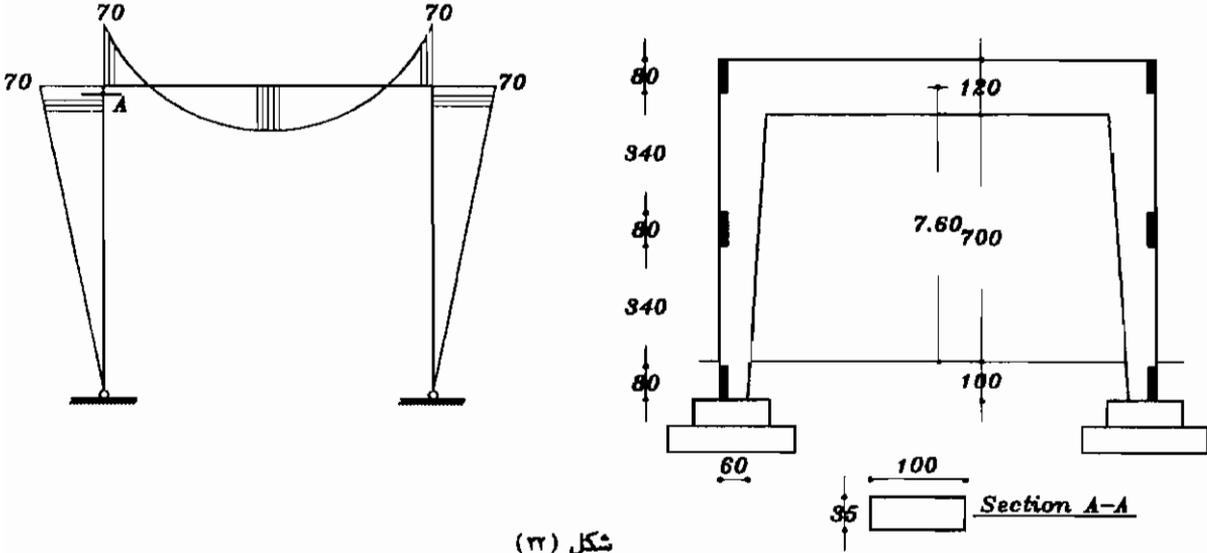
( شکل ۲۲ )

Design sec A – A in the frame shown in. Assume the frame is braced in y direction (out of plane ) and unbraced in x – direction ( its plane ).

Data:

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$f_y = 2400 \text{ kg/cm}^2 .$$



**Solution:**

As the column has a variable cross section, so an average value (  $t_{avg}$  ) will be used for buckling calculation, while the actual section (  $t = 100 \text{ cm}$  ) is used in designing sec A-A the average thickness for buckling may be taken at  $2/3 h$  .

$$T_{avg} = 60 + 40 (2/3) = 86.7 \text{ cm} .$$

Refer to table ( 6-10 ) as the frame is unbraced in x – direction

Case ( 3 ) at bottom and case ( 1 ) at top .....  $K = 1.6$

$$H_o = 7.6 + 1 - 0.6 = 8.0 \text{ m} .$$

$$H_e = 1.6 ( 8 ) = 12.8 \text{ m} .$$

$$\lambda = \frac{1280}{86.7} = 14.76$$

$$\delta = \frac{(14.76)^2 (0.867)}{2000} = 0.0944$$

$$M_{uy( add )} = 0.0944 ( 156.5 ) = 14.7 \text{ m.t} .$$

$$M_{uy (tot)} = 70 + 14.7 = 84.7 \text{ m.t.}$$

In most cases the additional moment in – Y – direction equal to zero to small values and can be neglected .

$$K = \frac{156.5 \times 1000}{250 (35) 100} = 0.179$$

$$K \cdot \frac{e}{t} = \frac{84.7 \times 10^5}{250 (35) 100^2} = 0.097$$

$$\zeta = 0.9, f_y = 2400 \text{ kg/cm}^2, \alpha = 0.6 \dots \dots \dots \text{chart No 19 page 24}$$

$$\rho = 2.5$$

$$\mu = 2.5(250)10^{-5} = 0.00625$$

$$A_s = 0.00625(35)100 = 21.87 \text{ cm}^2 \quad (5\text{d}25)$$

$$A_s^- = 0.6(21.87) = 13.125 \text{ cm}^2 \quad (3\text{d}25)$$

$$A_{s(\min)} = [0.25 + 0.052(14.76)]35(100)/100 = 35.5 \text{ cm}^2 \quad (\text{Eq.6 - 45})$$

$$A_{s(\text{total})} = 39.28 \text{ cm}^2 (8\text{d}25) > A_{s(\min)}$$

**Another solution:**

Use chart with uniform steel N<sub>o</sub> .11 page 16,  $\zeta = 0.9, f_y = 2400 \text{ kg/cm}^2$  .

$$k = 0.179, k ( e / t ) = 0.097$$

$$\rho = 5 \dots \dots \dots \mu = 5(250)10^{-5} = 0.0125$$

$$A_{s(\text{total})} = 0.0125(35)100 = 43.75 \text{ cm}^2$$

$$A_{s(\text{chosen})} = (45.6 \text{ cm}^2) 12\text{d}22.$$

\*in Y – direction :

refer to table ( 6-9 ) as the frame is braced in Y – direction .....K = 0.90

$$H_c = 0.9 ( 3.4 ) = 3.06 \text{ m} .$$

$$\lambda = \frac{3.06}{0.35} = 8.74 < 15 \quad (\text{short Column}).$$

**Example ( 13):**

A spirally reinforced round column is braced against sidesway at both ends. Its diameter  $D = 60$  cm. And unsupported length  $H_o = 7.5$  m . Elastic analysis indicates that for dead load  $P = 91$  ton,  $M_2 = 23$  m.t ,  $M_1 = 11.5$  m.t, and for maximum effect of live load  $P = 96$  ton ,  $M_2 = 23$  m.t ,,  $M_1 = 11.5$  m.t. End moments  $M_1$  and  $M_2$  have the same sign and thus tend to bend the column single curvature. If  $f_{cu} = 350$  kg/cm<sup>2</sup> ,  $f_y = 3600$  kg/cm<sup>2</sup> ,  $k = 1.0$  and maximum nominal size of concrete aggregate is the equivalent of 1.9 cm.

Determine the necessary longitudinal reinforcement and spirals.

**Solution:**

$$\text{Design } P_u = 1.4 P_{D.L} + 1.6 P_{L.L} = 1.4 ( 91 ) + 1.6 ( 96 ) = 281 \text{ ton}$$

$$\text{Design } M_1 = 1.4 M_{D.L} + 1.6 M_{L.L} = 1.4 (11.5) + 1.6 (11.5) = 34.5 \text{ m.t}$$

$$\text{Design } M_2 = 1.4 M_{D.L} + 1.6 M_{L.L} = 1.4 (23) + 1.6 (23) = 69 \text{ m.t}$$

Effective length factor ,  $K = 1.0$

$$\text{Slenderness ratio } \lambda_i = \frac{KH_o}{i} = \frac{1.0(7.5) \times (100)}{0.25 \times 60} = 50 \leq 50 \text{ and}$$

$$\lambda_b = \frac{KH_o}{D} = \frac{1.0 \times 7.5 \times 100}{60} = 12.5 > 12$$

Must consider slenderness effects:

$$\delta = \frac{\lambda_b^2 D}{2000} = \left( \frac{(12.5)^2 \times 60}{2000} \right) = 4.7 \text{ cm}$$

$$M_{add} = P_u \cdot \delta = 281 \cdot 0.047 = 13.21 \text{ m.t .}$$

Design moment is the biggest value of:

$$M_2 = 69.0 \text{ m.t}$$

$$M_1 + \frac{M_{add}}{2} = 34.5 + \frac{13.21}{2} = 41.1 \text{ m.t}$$

$$M_i = 0.4 M_1 + 0.6 M_2 = 0.4(34.5) + 0.6(69) = 55.2 \text{ m.t} \geq 0.4(69) = 27.6$$

$$M_i + M_{add} = 55.2 + 13.21 = 68.41 \text{ m.t}$$

$$P.e_{min} = 281 \times 0.03 = 8.43 \text{ m.t}$$

$\therefore$  Design moment = 69.0 m.t.

$$e = \frac{M_u}{P_u} = \frac{69 \times 100}{281} = 24.6 \text{ cm}$$

$$\frac{e}{R} = \frac{24.6}{30} = 0.8$$

Assuming bars  $\phi$  22 mm , spirals 10 mm, 2.5cm cover

$$\therefore D_s = 60 - 5 - 2(1.0) - 2.2 = 50.8 \text{ cm}$$

$$\zeta = \frac{D_s}{D} = 0.85$$

From design charts No. 48 with

$$\zeta = 0.9, f_y = 3600 \text{ kg/cm}^2 \quad \text{and} \quad \frac{e}{R} = 0.8$$

$$K = \frac{P_u}{f_{cu} \cdot R^2} = \frac{281 \times 1000}{350 \times (30)^2} = 0.892$$

$$\therefore \rho = 10$$

$$\mu = P f_{cu} \times 10^{-5} = 0.035 < \mu_{\max} = 0.06 > \mu_{\min} = 0.01$$

$$\therefore \text{Required } A_{st} = \frac{0.035\pi D^2}{4} = \frac{0.035\pi(60)^2}{4} = 98.91 \text{ cm}^2 \text{ say } 27 \phi 22 \text{ mm.}$$

Check of clear spacing between longitudinal bars:

$$\text{Available center to center spacing of bars} = \frac{\pi D_s}{27} = 6.1 \text{ cm}$$

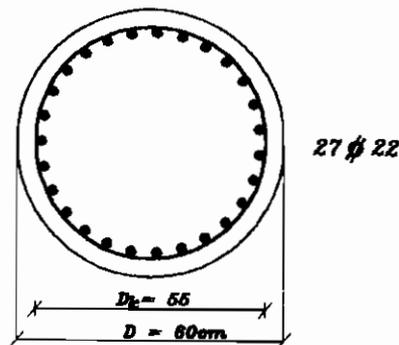
Minimum, allowable clear spacing according to code provisions, is the largest of:

Maximum bar diameter = 2.2 cm or

1.5 maximum nominal aggregate size = 1.5 \* 1.9 = 2.85 cm

$\therefore$  Least center to center of  $\phi$  22 mm = 2.85 + 2.2 = 5.05 cm.

Available 6.1 > 5.05



**Design of spirals:**

$$\mu_{sp(\min)} = 0.36 \left( \frac{f_{cu}}{f_{yp}} \right) \left( \frac{A_c}{A_k} - 1 \right)$$

$$A_c = \frac{\pi D^2}{4} = \frac{\pi (60)^2}{4} = 2826 \text{ cm}^2$$

$$A_k = \frac{\pi (60 - 2 \times 2.5)^2}{4} = 2374.6 \text{ cm}^2$$

$$\therefore \mu_{sp(\min)} = 0.36 \left( \frac{350}{2400} \right) \left( \frac{2826}{2374.6} - 1 \right) = 0.0094$$

$$\mu_{sp} = \frac{V_{sp}}{A_k}, \quad V_{sp} = 0.0094(2374.6) = 22.38 \text{ and}$$

$$V_{sp} = \pi A_{sp} D_k / P$$

USING  $\phi$  10 mm as a spiral

$$\therefore A_{sp} = 0.78 \text{ cm}^2, \quad P = \frac{3.14 \times 0.78 \times 55}{22.38} = 6.05 \text{ cm}$$

Choose  $p = 6 \text{ cm}$  (O.K)

**Example (14):**

Design a spirally reinforced round column for member BC shown in the figure:  
Design  $P_u = 305$  ton, Design  $M_u = 37.2$  m.t,  $f_{cu} = 350$  kg/cm<sup>2</sup>, and  $f_y = 3600$  kg/cm<sup>2</sup>  
columns AB, BC and CD are identical and not braced against sidesway. the stiffness of beams at C and B are 3113.5 cm<sup>3</sup> and 2785.8 respectively. Maximum size of concrete aggregate is 2.5 cm.  $M_u$  is due to wind load.

**Solution:**

Assume 60 cm round column,

$$i = 0.25(60) = 15 \text{ cm}$$

$$I_c = \pi D^4 / 64 = 635850 \text{ cm}^4$$

$$K_c = \frac{I_c}{H_o} = \frac{635850}{400} = 1589.6 \text{ cm}^2$$

At joint B,

$$\alpha_1 = \frac{\sum K_{col}}{\sum K_{beams}} = \frac{2(1589.6)}{2785.8} = 1.14$$

At joint C,

$$\alpha_2 = \frac{2(1589.6)}{3113.5} = 1.02$$

$$k = (1.0 + 0.15 (\alpha_1 + \alpha_2)) = 1.32 \text{ or}$$

$$= (2 + 0.3 \alpha_{min}) = 2.300$$

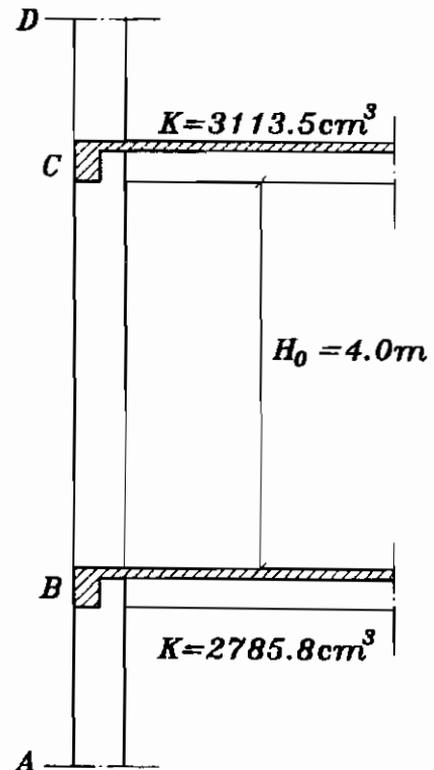
Take the smaller  $K = 1.32$ ,  $KH_o = 1.32 * 400 = 528$  cm

$$\lambda_1 = \frac{KH_o}{i} = \frac{528}{15} = 35.2 > 35$$

$$\lambda_b = \frac{KH_o}{D} = \frac{528}{60} = 8.8 > 8 \therefore \text{slenderness effects must be considered}$$

$$\delta = \frac{\lambda_b^2 \cdot D}{2000} = \frac{(8.8)^2 \cdot 60}{2000} = 2.3 \text{ cm}$$

$$M_{add} = P_u \cdot \delta = 305 \times 2.3 = 7.08 \text{ m.t}$$



Design moment is the greater of:

$$M_{\text{applied}} + M_{\text{add}} = 37.2 + 7.08 = 44.28 \text{ m.t}$$

$$P.e_{\text{min}} = 305 * 0.03 = 9.15 \text{ m.t}$$

$\therefore$  Design moment = 44.28 m.t

$$e = \frac{M_u}{P_u} = \frac{44.28 \times 100}{305} = 14.5 \text{ cm}$$

$$\frac{e}{R} = \frac{14.5}{30} = 0.48$$

assume bars  $\phi$  22 mm spirals 10 mm, 2.5 cm cover

$$\therefore D_s = D - 2(\text{cover}) - 2(\text{spiral diam}) - \text{bar diam} \\ = 60 - 5 - 2 - 2.2 = 50.8 \text{ cm, and } \zeta = 0.9$$

from design chart No 48 :

$$K = \frac{305 * 1000}{350 * (30)^2} = 0.96$$

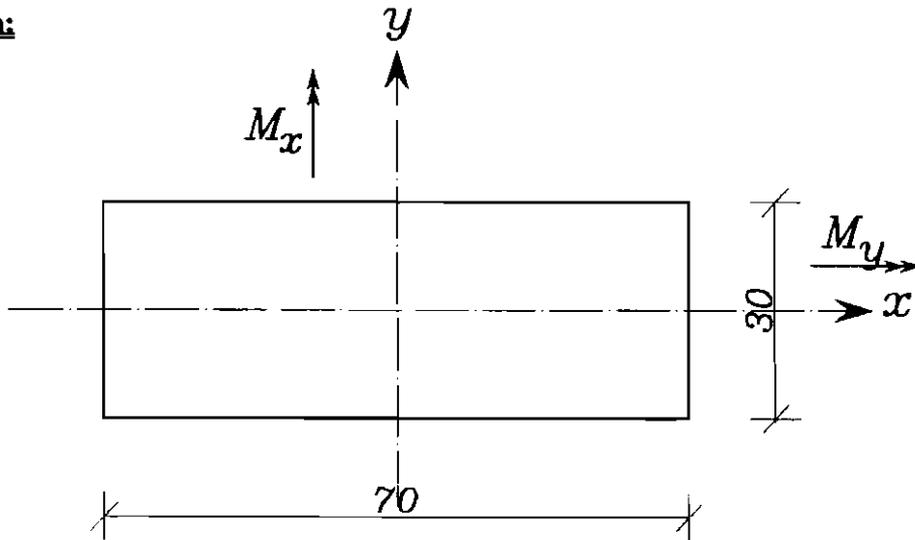
$$\rho = 5.8, \mu = 5.8 * 350 * 10^{-5} = 0.02 < \mu_{\text{max}} = 0.06$$

$$\text{required } A_{st} = 0.02 \frac{\pi(60)^2}{4} = 57.4 \text{ cm}^2 > \mu_{\text{min}} = 0.01 \quad (16\phi 22)$$

**Example(15) :**

It is required to design a column with  $H_o = 5.0$ , the system can be considered unbraced system , applied ultimate load = 300 t (D.L+L.L) ,  $f_y = 2400 \text{ kg/cm}^2$  ,  $f_{cu} = 300 \text{ kg/cm}^2$  , column dimension ( 30 \* 70 ) and the end condition is fixed.

**Solution:**



**Load due to buckling:**

**In X direction:**

From table ( 6 – 10 ) E.C ( unbraced system )

Case ( 1 ) , case ( 1 ) K = 1.2

$$H_e = K H_o = 1.2 * 500 = 600 \text{ cm}$$

$$\lambda = \frac{600}{70} = 8.57 < 10 \text{ (table 6 – 7)}$$

No additional moment In X direction ( short Col ) ( 1 )

**In Y direction :**

$$K = 1.2$$

$$H_e = 1.2 * 500 = 600 \text{ cm.}$$

$$\lambda = \frac{600}{30} = 20 > 10 \text{ table(6 – 7) E.C}$$

N.B The Col is long in this direction < 23 table ( 6 – 8 ) E.C.

$$\delta = \frac{\lambda^2 b}{2000} = \frac{(20)^2 \cdot 30}{2000} = 6 \text{ cm}$$

$$M_{u \text{ add}(x)} = P_u \cdot \delta = 300 \cdot 0.06 = 18 \text{ m.t.}$$

$$P_u \cdot e_{\text{min}} = 300 \cdot (0.05 \cdot 0.3) = 4.5 \text{ m.t}$$

$$\therefore M_{u \text{ design}} = 18 \text{ m.t}$$

Design using interaction diagram:

(Chart No. 32)

$$\zeta = 0.8$$

$$f_y = 2400 \text{ kg/cm}^2$$

$$\alpha = 1.00$$

$$K = \frac{P_u}{f_{cu} \cdot b \cdot l} = 0.476$$

$$K \cdot \frac{e}{l} = 0.095 \quad \rho = 8 \quad \mu = \rho \cdot f_{cu} \cdot 10^{-5} = 0.024$$

$$A_s = A_r = 0.024 \cdot 30 \cdot 70 = 50.4 \text{ cm}^2 \quad 14\phi 22$$

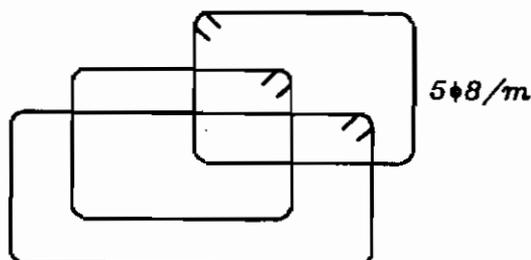
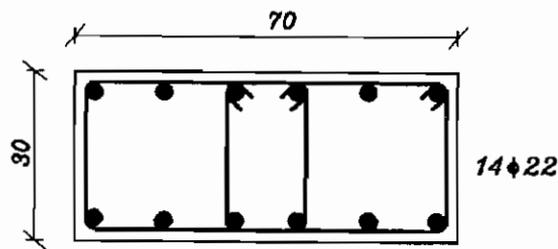
Check of min stirrups:

$$V_{s \text{ min}} = \frac{0.25}{100} \cdot (b \cdot l \cdot 100) = 525 \text{ cm}^3 / \text{m}$$

$$V_s = (6 \cdot 25 + 2 \cdot 65 + 4 \cdot 39) (0.504) (5) = 1098.72 \text{ cm}^3 / \text{m}$$

$$V_s > V_{s \text{ min}}$$

$$\therefore \text{using } 5\phi 8 / \text{m}$$



**Example (16) :**

It required to design a spirally circular column :

Unbraced Col . fixed in upper end & hinged at lower

$$P_{D.L} = 150 \text{ t .}$$

$$P_{L.L} = 100 \text{ t .}$$

$$H_0 = 80 \text{ m .}$$

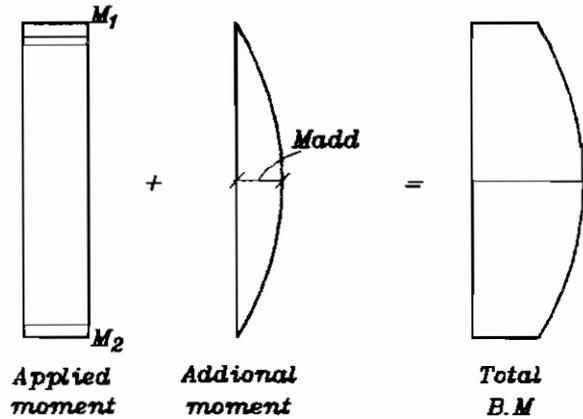
$$\text{Moment due to D.L} = 15 \text{ m.t}$$

$$\text{Moment due to L.L} = 10 \text{ m.t}$$

$$\text{Single curvature } M_1 = M_2$$

$$K = 1.6$$

$$\text{Diameter} = 80 \text{ cm , } f_{cu} = 300 \text{ kg/cm}^2 \text{ , } f_y = 3600 \text{ kg/cm}^2 \text{ .}$$



**Solution:**

$$L.L < 0.75 \text{ D.L}$$

$$P_u = 1.5 ( 150 + 100 ) = 375 \text{ t}$$

$$M_{u1} = 1.5 ( 15 + 10 ) = 37.5 \text{ m.t}$$

Load from buckling :

$$H_e = KH_0 = 1.6 * 800 = 1280 \text{ cm}$$

$$\lambda = \frac{1280}{80} = 16 > 8 \text{ (table 6-7)}$$

$$< 18 \text{ (table 6-8)} \quad \text{E.C.}$$

$$\delta = \frac{\lambda^2 . D}{2000} = \frac{(16)^2 80}{2000} = 10.24 \text{ cm}$$

$$M_{add} = P_u . \delta = 375 * \frac{10.24}{100} = 38.24 \text{ m.t}$$

$$M_1 = M_{u1} + M_{add} = 37.5 + 38.24 = 75.74 \text{ m.t}$$

**Design:**

Using interaction diagram ( chart No 48 ) cover 5 cm.

$R^- = 35 \text{ cm}$

$R = 40 \text{ cm}$

$$\zeta = \frac{R^-}{R} \cong 0.9$$

$$f_y = 3600 \text{ kg/cm}^2$$

$$K = \frac{P_u}{f_{cu} R^2} = \frac{375 \cdot 10^3}{300 \cdot (40)^2} = 0.78$$

$$\rho = 3.5$$

$$\frac{M_u}{f_{cu} R^3} = \frac{75.74 \cdot 10^5}{300 \cdot (40)^3} = 0.394$$

$$\mu = \rho \cdot f_{cu} \cdot 10^{-5} = 0.0105 < \mu_{\max} \text{ (table 4-1) E.C..}$$

$$A_s = \mu \pi R^2 = 52.77 \text{ cm}^2 > 1\% A_c = \frac{1}{100} \cdot \frac{\pi}{4} \cdot (80)^2 = 50.26 \text{ cm}^2$$

$$1.2\% A_k = \frac{1.2}{100} \cdot \frac{\pi (70)^2}{4} = 46.18 \text{ cm}^2$$

using 20  $\phi$  19

Check of min volume of spiral:

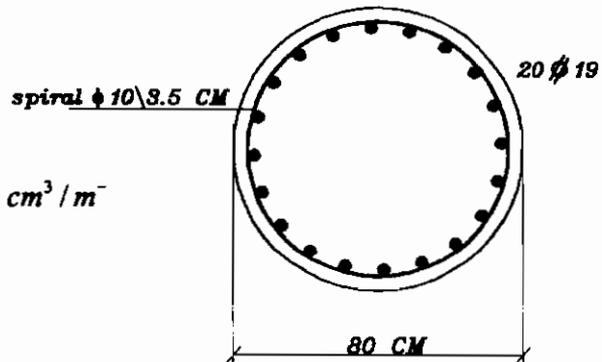
Use min volume of spiral

$$V_{sp} = \pi \cdot \frac{(1.0)^2}{4} \cdot \pi \cdot \frac{70}{P} = 172.72 / p \quad \text{cm}^3 / \text{m}$$

$$\begin{aligned} \mu_{sp \min} &= 0.34 \left( \frac{f_{cu}}{f_{yp}} \right) \left( \frac{A_c}{A_k} - 1 \right) \\ &= 0.34 \left( \frac{300}{2400} \right) \left( \frac{(80)^2}{(70)^2} - 1 \right) = 1.301 \cdot 10^{-2} \end{aligned}$$

Take  $M_{sp} = M_{sp \min}$ 

$$3 \text{ cm} < P_{\max} = 3.5 \text{ cm} < 8 \text{ cm} \quad (\text{O.K})$$



**Example (17) :**

The figure shows that column is braced two direction.

Width of the column  $b = 25$  cm.

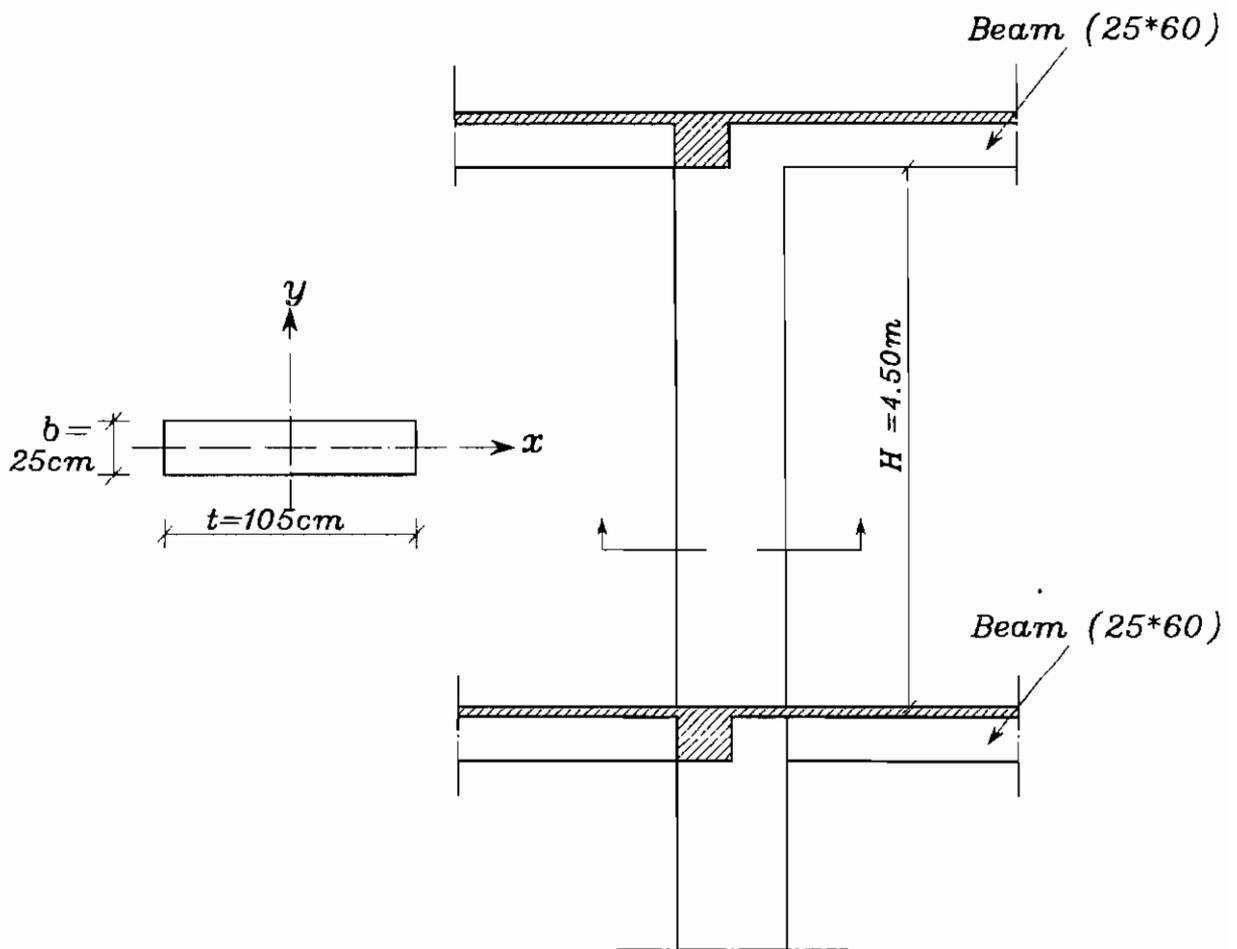
Length of the column  $t = 105$  cm.

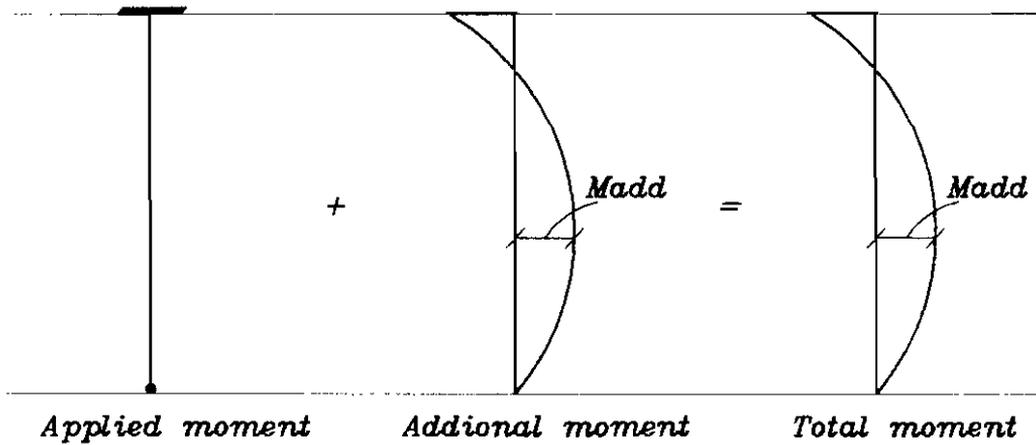
Beam Dim.  $25 * 60$

$P_u = 290$  t

$f_{cu} = 250$  kg/cm<sup>2</sup>.

$f_y = 3600$  kg/cm<sup>2</sup>.





**Solution:**

**Buckling load**

**In X direction: ( about y ):**

Top and bottom  $t_c = 105 > ( t_b = 60 )$

Case ( 2 )

$K = 0.85$  ( table 6 - 9 ) E . C

$$\lambda_y = \frac{KH_o}{t} = \frac{0.85 * 5.4}{1.05} = 4.37 < 15 \quad (\text{table 6 - 7})$$

No additional B . M due to buckling in this direction

**In Y direction : ( about X ):**

Top and bottom  $b_c = 25 < t_b ( 60 \text{ cm} )$

Case ( 1 )

$K = 0.75$  ( table 6 - 9 ) E.C

$$H_e = kH_o = 405 \text{ cm}$$

$$\lambda_b = \frac{H_e}{b} = \frac{405}{25} = 16.2 > 15 \quad (\text{table 6 - 7})$$

$$< 30 \quad (\text{table 6 - 8})$$

$\therefore$  the cal is long in this direction

$$\delta = \frac{\lambda_b^2 \cdot b}{2000} = \frac{(16.2)^2 \cdot 25}{2000} = 3.28 \text{ cm} > 0.05 t \quad , > 2 \text{ cm}$$

$$M_{add} = P_u \delta = 290 * 0.0328 = 9.51 \text{ m.t} \quad \text{about } X \text{ axis}$$

$$P_u = 290 \text{ t}, \quad M_{u \text{ add}} = 9.51 \text{ m.t} \quad b = 25 \text{ cm}$$

(using interaction diagram chart No 38)

$$\zeta = \frac{20}{25} = 0.8 \quad f_y = 3600 \text{ kg/cm}^2 \quad \alpha_c = 1.00$$

$$K = \frac{P_u}{f_{cu} t b} = \frac{290 * 1000}{250 * 105 * 25} = 0.442$$

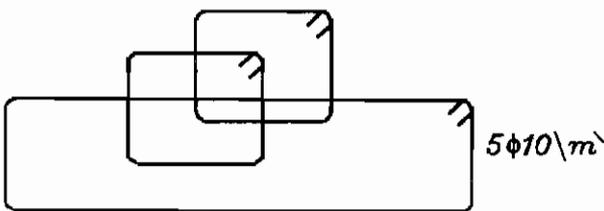
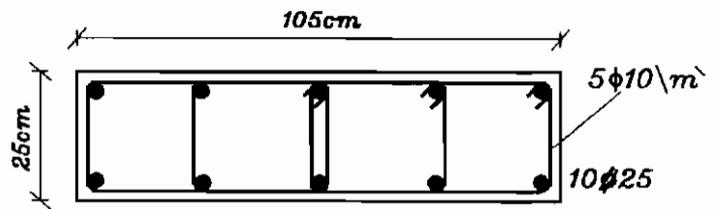
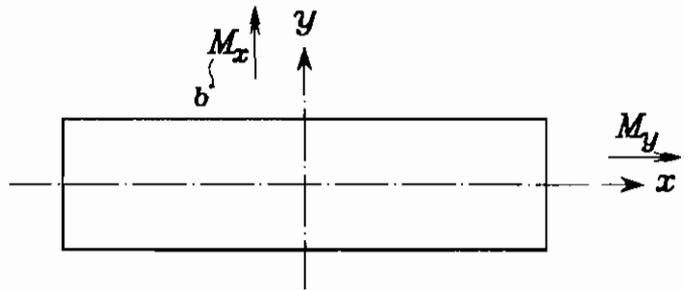
$$\frac{M_u}{f_{cu} t b^2} = \frac{9.51 * 10^5}{250 * 105 * (25)^2} = 0.058$$

$$\rho = 3.6$$

$$\mu = \rho f_{cu} 10^{-5} = 0.009$$

$$A_s = A_s^- = 0.009 * 25 * 105 = 23.63 \text{ cm}^2$$

use 5  $\phi$  25 on each side



**Example ( 18 ):**

Design a circular spirally reinforced concrete column and sketch the full reinforcement detailings having:

- Diameter of column = 50 cm .
- Unsupported length of column = 5 m .
- End conditions gives  $K = 1.5$
- service loads are:
  - $f_{cu} = 300 \text{ kg/cm}^2$
  - $f_y = 3600 \text{ kg/cm}^2$  .
- Moment due to D.L = 6 m.t  $f_{y(stirps)} = 2400 \text{ kg/cm}^2$  .
- Moment due to L.L = 8 m.t
- Single curvature:  $M_1 = M_2$

**Solution:**

Additional load due to buckling:

$$H_e = KH_o = 1.5 * 5 = 7.5 \text{ m}$$

$$\lambda_b = \frac{H_e}{b} = \frac{7.5}{0.5} = 15$$

$\lambda > 8$  (table 6-7) E.C

$$\delta = \lambda_b^2 . D / 2000 = 5.625 \text{ cm}$$

$$M_{L.L.add} = 40 * 0.05625 = 2.25 \text{ m.t}$$

$$M_{D.L.add} = 60 * 0.05625 = 3.375 \text{ m.t}$$

**Ultimate load for Design:**

$$P_{D.L} = 60 * 1.4 = 84 \text{ t}$$

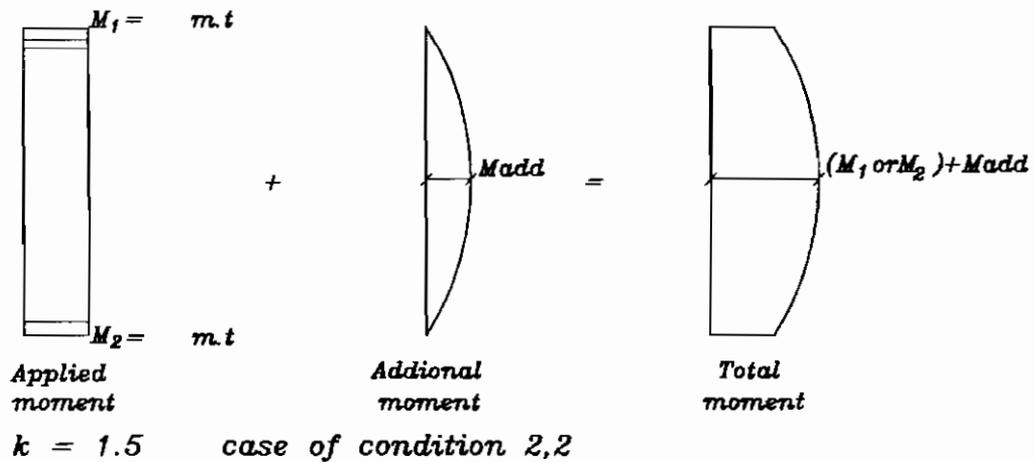
$$P_{L.L} = 40 * 1.6 = 64 \text{ t}$$

$$P_{u \text{ total}} = P_{d.l} + P_{L.L} = 148 \text{ t}$$

$$M_{D.L} = 1.4(6 + 3.375) = 13.125 \text{ t.m}$$

$$M_{L.L} = 1.6(8 + 2.25) = 16.4 \text{ tm}$$

$$M_{total} = M_{D.L} + M_{L.L} = 29.525 \text{ t.m}$$



### Column is spirally col:

Use interaction diag ( chart No . 48 )

$$\frac{R^-}{R} = 0.9$$

$$f_y = 3600 \text{ kg/cm}^2$$

$$\frac{P_u}{f_{cu} R} = 0.79 \quad \frac{M_u}{f_{cu} R^3} = 0.63$$

$$\frac{g_{el}}{\rho} \rightarrow \rho = 8.0$$

$$\mu = \rho f_{cu} * 10^{-5} = 0.024 \quad \geq 1.0\% A_c$$
$$\geq 1.2\% A_k \rightarrow (o.k)$$

$$A_s = 47.12 \text{ cm}^2$$

use 10  $\phi$  25

Check of min volume of spiral

use min volume of stirrups

$$V_{sp} = \frac{\pi * (0.8)^2}{4} * \pi * 45 = 71.06 / \rho \text{ cm}^3 / m$$

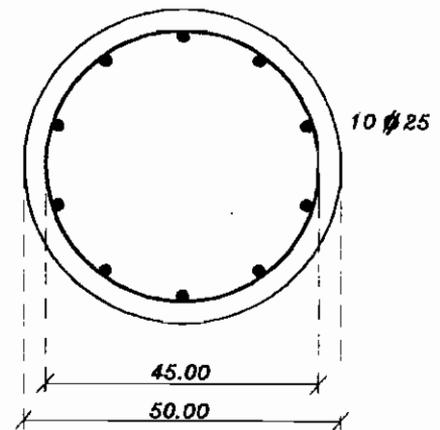
$$\mu_{sp \text{ min}} = 0.34 \left( \frac{f_{cu}}{f_{yp}} \right) \left( \frac{A_c}{A_k} - 1 \right)$$
$$= 0.34 \left( \frac{300}{2400} \right) \left[ \left( \frac{0.5}{0.45} \right)^2 - 1 \right] = 9.96 * 10^{-3}$$

$$\therefore \mu_{sp} = \mu_{sp \text{ min}}$$

$$\mu_{sp} = \frac{V_{sp}}{A_k} = \frac{71.06}{P \pi (45)^2 / 4}$$

$$\therefore P_{\text{max}} = 4.5 \text{ cm}$$

use  $\phi$  8 / 4.5 cm.



**Example( 19):**

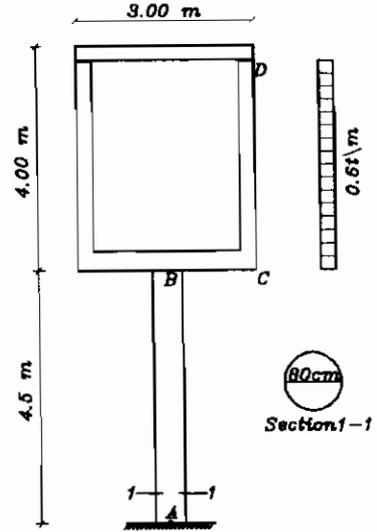
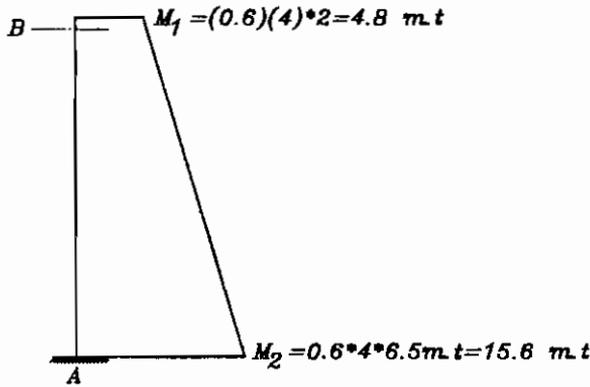
Figure shows an elevated R.C. water tank supported on a circular spiral column AB. The column is 80 cm diameter .Lateral sway at top of column is possible while end rotation is not. Design lateral load on tank side can be assumed 0.6 t/m along the height of the tank CD. Design vertical load at point B of the column can be assumed 140 t. It is required to design a suitable longitudinal and spiral reinforcement for the column AB as sec B – B according to the EC – Code provisions .

$$f_{cu} = 300 \text{ kg/cm}^2$$

$$f_y = 3600 \text{ kg/cm}^2 .$$

$$f_{yp} = 2400 \text{ kg/cm}^2 .$$

**Solution:**



B.M due to buckling

The system is unbraced

Condition ( 1 ) at bottom

Condition ( 4 ) at top ( free ) ( from table 6 – 10 )

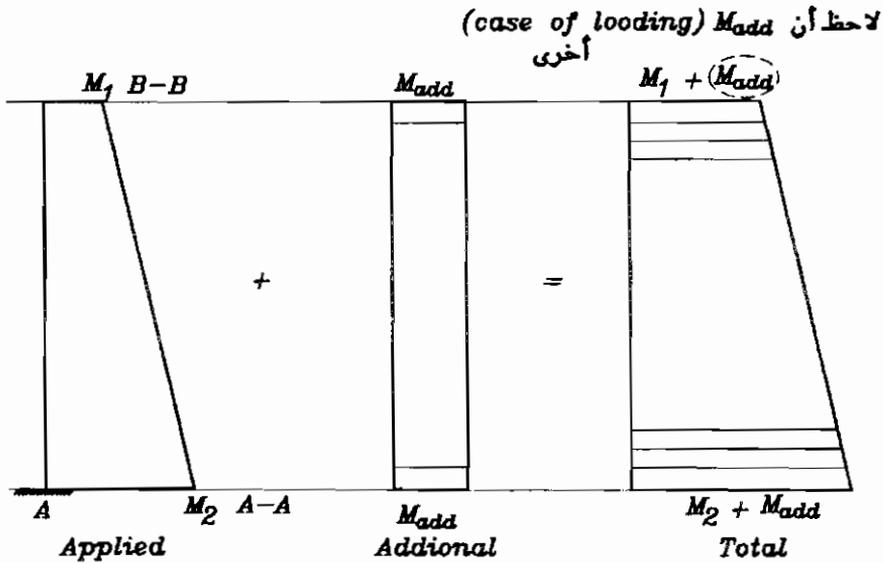
$$K = 2.2$$

$$H_e = K H_o = 4.5 * 2.2 = 9.9 \text{ m}$$

$$\lambda_b = \frac{H_e}{D} = \frac{9.90}{0.8} = 12.375 > 8 \text{ (long col)}$$

$$\delta = \lambda_b^2 . D / 2000 = 6.125 \text{ cm}$$

$$M_{add} = \rho * \delta = 140 * 0.06125 = 8.575 \text{ t,m}$$



في حالة وجود ( lateral load ) نجد أن :

$$u = 0.8 ( 1.4 D. L + 1.6 L . L + 1.6 w.l )$$

$$\text{or } U = 1.4 D. L + 1.6 L . L$$

**Case of loading:**

**Case ( 1 ):** No wind according to ( 3 - 1 ) ( E . C )

$$P_u = 140 \text{ t}$$

$$M_{u \text{ add}} = 8.575 \text{ t.m}$$

**Case ( 2 ):**

$$P_u = 0.8 ( 140 + 0 ) = 112 \text{ t}$$

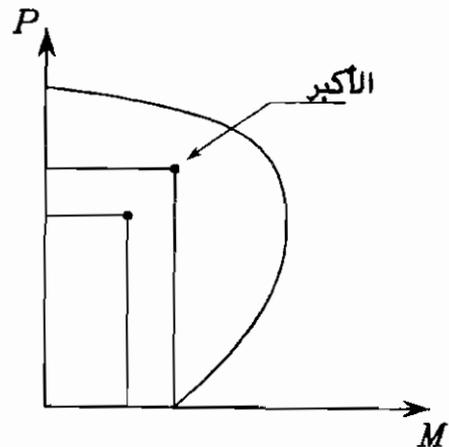
$$M_u = 0.8 ( 8.575 + 1.6 * 15.6 ) = 26.828 \text{ t.m}$$

**For case 1 :**

$$\frac{P_u}{f_{cu} R^2} = \frac{140 * 10^3}{300 * (40)^2} = 0.292$$

$$\frac{M_u}{f_{cu} R^3} = \frac{8.575 * 10^5}{300 * (40)^3} = 0.0446$$

$\therefore \rho < 1$  use min  $A_s$



**For case 2:**

$$\frac{P_u}{f_{cu} R^2} = \frac{112 * 10^3}{300 * (40)^2} = 0.233$$

$$\frac{M_u}{f_{cu} R^3} = \frac{26.828 * 10^5}{300 * (40)^3} = 0.139$$

∴ ρ < 1 use min A<sub>s</sub>

**ملاحظة :**

يمكن تحديد أي الحالتين أولى بالدخول في منحنى رقم ( ٤٨ ) واختيار أكبرهما .

$$use \min A_s = \frac{1}{100} \pi R^2 = 50.26 \text{ cm}^2 \quad 14 \phi 22.$$

Check of M<sub>sp</sub> :

$$\begin{aligned} \mu_{sp \min} &= 0.34 \left( \frac{f_{cu}}{f_{yp}} \right) \left( \frac{A_c}{A_k} - 1 \right) \\ &= 0.34 * \left( \frac{300}{2400} \right) \left[ \left( \frac{0.8}{0.75} \right)^2 - 1 \right] = 5.856 * 10^{-3} \end{aligned}$$

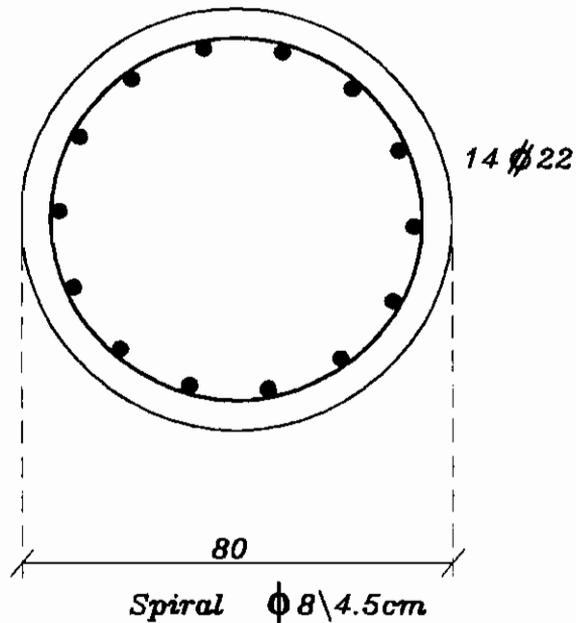
$$V_{sp} = \frac{\pi * (0.8)^2}{4} * \pi * \frac{75}{P} = 118.4 / p$$

$$\mu_{sp} = \frac{V_{sp}}{\left[ \pi D_k^2 / 4 \right]} = 0.0268 / p$$

$$\mu_{sp} = \mu_{sp \min}$$

$$\rho_{\max} = 4.5 \text{ cm}$$

use  $\phi 8 / 4.6 \text{ cm}$



**Example(20):**

Design a square column ( 50 \* 50 ) and sketch the full reinforcement detailing . having unbraced system :

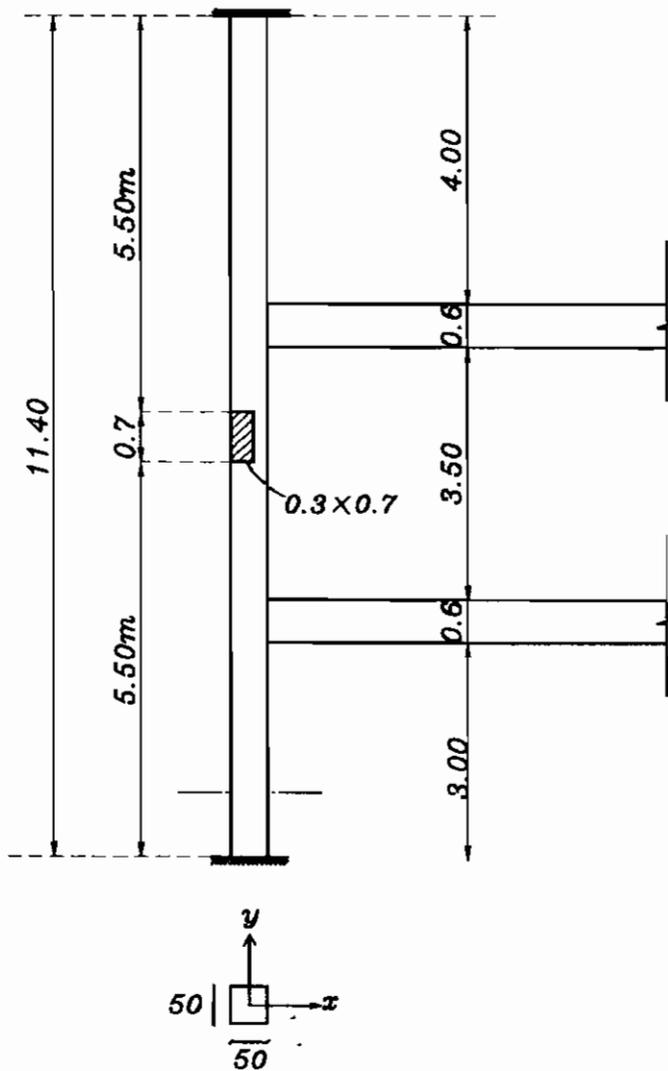
$$P_u = 200 \text{ t.}$$

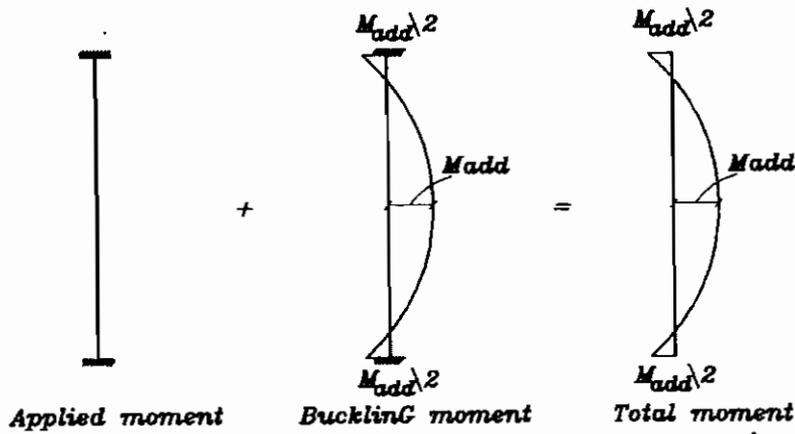
End condition can considered fixed .

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$f_y = 2400 \text{ kg/cm}^2$$

Neglect the effect of moment which transferred from beams:





We have to study tow direction:

**X direction : ( about Y )**

Take the max unsupported length

$$H_{o \max} = 4.00 \text{ m .}$$

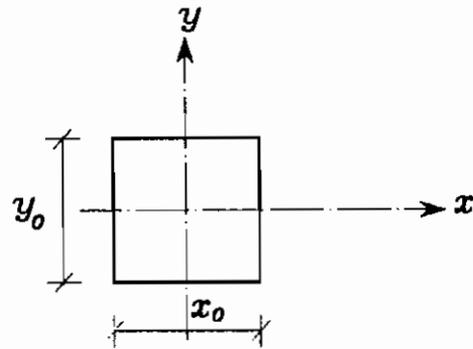
Top condition ( 1 )

bottom condition ( 1 )

$$K = 1.2 \text{ ( table 6 - 10 ) E.C.}$$

$$H_e = 4 * 1.2 = 4.8 \text{ m}$$

$$\lambda = \frac{H_e}{X_o} = \frac{4.8}{0.5} = 9.6 < 10 \text{ (table 6.7) E.C}$$



( no buckling occur in the direction ) .  $M_y = 0$

**Y direction : ( about X )**

Take the max unsupported length

$$H_o = 5.50 \text{ m .}$$

$K = 1.2$  ( unbraced system ) ( top and bottom case 1 )

$$H_e = 5.50 * 1.2 = 6.6$$

$$\lambda = \frac{He}{b} = \frac{6.6}{0.5} = 13.2 > 10 \quad (\text{table 6-7})E.C$$

$$\delta = \frac{\lambda^2 \cdot b}{2000} = \frac{(13.2)^2 \cdot 50}{2000} = 4.36 \text{ cm}$$

$$\frac{e}{t} = \frac{\delta}{t} = \frac{4.36}{50} = 0.0872 > 0.05$$

(since  $e_{\min} = \text{bigger}(0.05t \gamma 2 \text{ cm})$ )

$$M_u = P_u \cdot \delta = 200 * \frac{4.36}{100} = 8.72 \text{ mt}$$

using interaction diag ( chart N<sub>o</sub> 11 ) ( uniform steel )

$$K = \frac{P_u}{f_{cu} \cdot bt} = \frac{200 * 10^3}{250 * 50 * 50} = 0.32$$

$$K \cdot \frac{e}{t} = 0.0279 \quad \rho = 2$$

$$\mu = \rho f_{cu} * 10^{-5} = 2 * 250 * 10^{-5} = 0.005$$

$$A_s = \mu bt = 0.005 * 50 * 50 = 12.5 \text{ cm}^2$$

$$\mu_{\min} = (0.25 + 0.052 \lambda_b) = (0.25 + 0.052 * 13.2) = 0.936\%$$

$$A_{s, \min} = \frac{0.936}{100} * 50 * 50 = 23.41 \text{ cm}^2 \quad 8\phi 20$$

**Stirrups:**

$$S \leq (20 \text{ cm} , b = 50 \text{ cm} , 15\phi_{col})$$

$$\phi_{st} \geq \left( \frac{\phi_{col}}{4} , 8 \text{ mm} \right) \geq 8 \text{ mm}$$

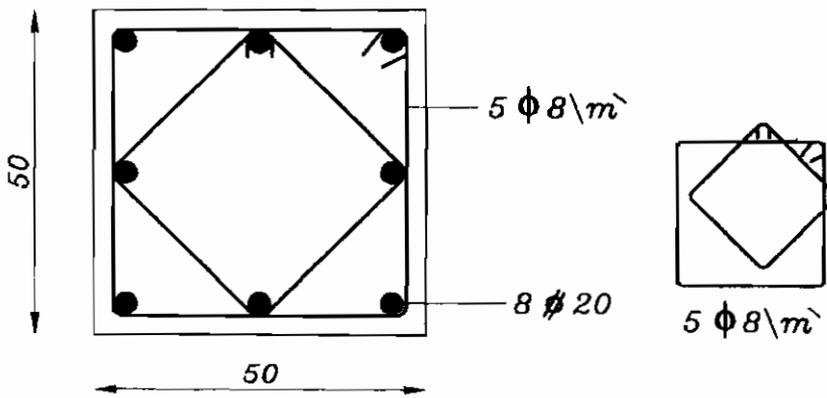
use  $\phi 8 \text{ mm}$

$$V_{st \text{ min}} = \frac{0.25}{100} (\text{Volume of one meter from longitudinal length})$$

$$= \frac{0.25}{100} * 50 * 50 * 100 = 625 \text{ cm}^2$$

$$V_{st} = 4(45 + 31.8) * 0.503 * 5 = 772.6 \text{ cm}^2$$

$$V_{st} > V_{st \text{ min}} \rightarrow (O.K)$$



**Example ( 21 ) :**

**Given:**  $M_u = 25 \text{ m.t.}$

$P_u = 150 \text{ t (comp)}$

$b = 25 \text{ cm}$

$t = 50 \text{ cm}$

$f_{cu} = 300 \text{ kg/cm}^2$

$f_y = 3600 \text{ kg/cm}^2$

**Required:**

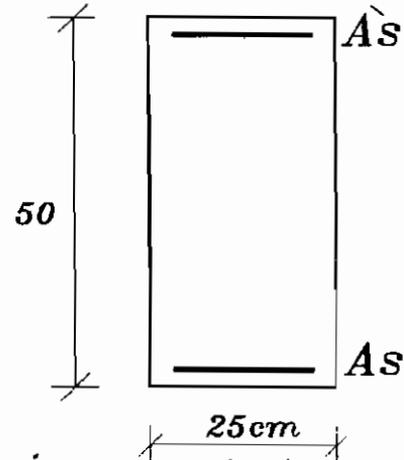
$A_s, A_s^-$  for the following cases:

1 -  $\alpha = 0.6.$

2 -  $\alpha = 0.8.$

3 -  $\alpha = 1.0.$

**Solution:**



كما هو واضح أن الأبعاد محددة مسبقاً والعزم صغير بالنسبة للقوة ( سوف نتوقع وجود النقطة في Zone B ) .

Using the interaction diagrams ( chart N<sub>o</sub> 36 , 37 and 38 ) . we can get :

Case	$\alpha$	K	e (m)	e / t	K(e/t)	$\rho$	$\mu$	$A_s$	$A_s^-$
1	0.6	0.4	0.167	0.334	0.133	7.6	0.0228	28.5	17.1
2	0.8	0.4	0.167	0.334	0.133	7.0	0.021	26.25	21.0
3	1.0	0.4	0.167	0.334	0.133	5.8	0.0174	21.75	21.75

**Example ( 22. ) :**

**Given:**  $M_u = 5 \text{ m.t.}$   $P_u = 200 \text{ t}$   
 $b = 30 \text{ cm}$   $f_{cu} = 250 \text{ kg/cm}^2$   
 $f_y = 3600 \text{ kg/cm}^2$

**Design Sec:**

$$e = \frac{M_u}{P_u} = \frac{5}{200} = 0.025$$

let  $\frac{e}{t} < 0.05$  (zone A) min ecc

$$P_u = 0.35 f_{cu} A_c + 0.67 f_y A_{sc} \quad A_{sc} = 0.01 A_c$$

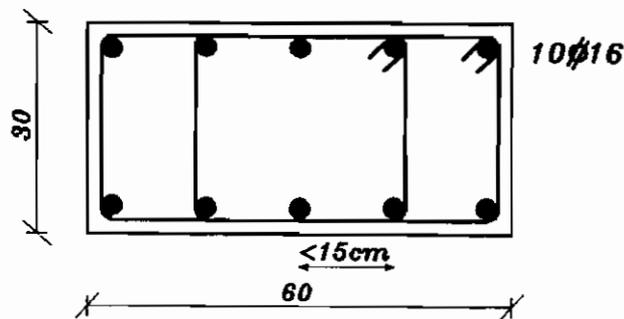
$$200000 = 0.35 * 250 * A_c + 0.67 * 3600 * \frac{A_c}{100}$$

$$A_c = 1791.79 \text{ cm}^2 \quad t = \frac{A_c}{30} = 59.73 \text{ cm}$$

take  $t = 60 \text{ cm}$

$$\frac{e}{t} = \frac{0.025}{60} = 0.0417 < 0.05 \text{ (the assumption is true)}$$

$$\therefore A_s = \frac{A_c}{100} = \frac{1791.79}{100} = 18 \text{ cm}^2 \quad (10 \text{ } \phi \phi 16)$$



**Example (23) :**

**Given:** T – sec. With

$$B = 130 \text{ cm.}$$

$$M_u = 70 \text{ m.t.}$$

$$b = 25 \text{ cm}$$

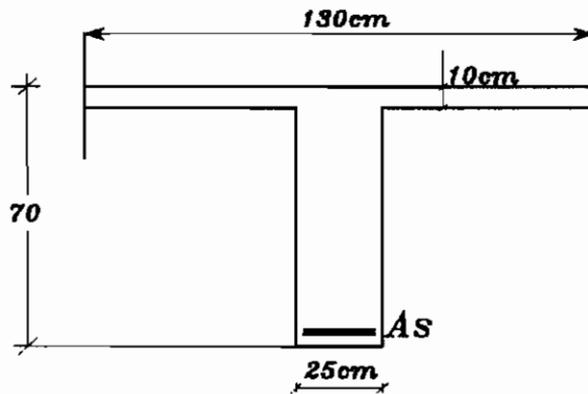
$$f_y = 3600 \text{ kg/cm}^2$$

$$P_u = 10 \text{ t. ( comp )}$$

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$d = 65 \text{ cm.}$$

**Required As:**



Assume  $a < t_s$

$$e = \frac{70}{10} = 7 \text{ m}$$

**Assuming that the C.G of the cross sec. Is at the middle of the depth ( t )**

$$\therefore e_s = e + (t/2) - \text{cover}$$

$$= 7 + \frac{0.7}{2} - 0.05 = 7.3 \text{ m}$$

$$M_{us} = 10 * 7.3 = 73 \text{ m.t}$$

Using chart ( 2) page ( 4 ) :

$$d = C_1 \sqrt{\frac{M_{us}}{f_{cu} \cdot B}}$$

$$\therefore C_1 = 4.34 \quad \therefore c/d = 0.16$$

$$\therefore C = 0.16 * 65 = 10.4 \text{ and } a = 0.8 * 10.4 = 8.32 \text{ cm}$$

then the assumption that (  $a < t_s$  ) is valid

$$j = 0.815, A_s = \frac{73 * 10^5}{3600 * 0.815 * 65} - \frac{10 * 1000}{\left( \frac{3600}{1.15} \right)} = 35.1 \text{ cm}^2.$$

**Example (24) :**

**Given:** T – sec. With

$$B = 130 \text{ cm.}$$

$$M_u = 70 \text{ m.t.}$$

$$P_u = 10 \text{ t. ( tens )}$$

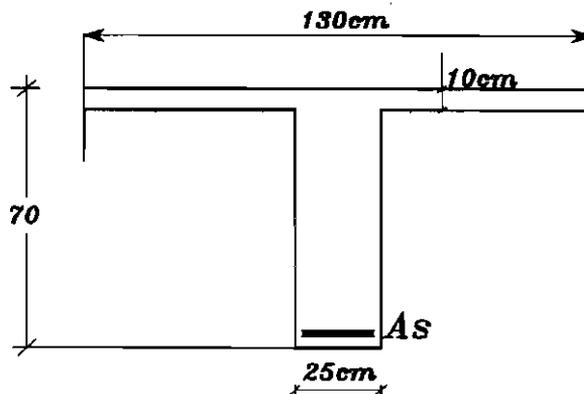
$$b = 25 \text{ cm}$$

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$f_y = 3600 \text{ kg/cm}^2$$

$$d = 65 \text{ cm.}$$

**Required As:**



Assume  $a < t_s$

$$e = \frac{70}{10} = 7 \text{ m}$$

$$\therefore e_s = e - (t/2) + \text{cover}$$

$$= 7 - \frac{0.7}{2} + 0.05 = 6.7$$

$$M_{us} = 10 * 6.7 = 67 \text{ mt}$$

using chart ( 2 ) page ( 4 ) :

$$d = C_1 \sqrt{\frac{M_{us}}{f_{cu} \cdot B}}$$

$$\therefore C_1 = 4.53 \quad \therefore c/d = 0.146$$

$$\therefore C = 0.146 * 65 = 9.49 \text{ and } a = 0.8 * 9.49 = 7.59 \text{ cm}$$

Then the assumption that  $( a < t_s )$  is valid

$$j = 0.82, \quad A_s = \frac{67 * 10^5}{3600 * 0.82 * 65} + \frac{10 * 1000}{\left( \frac{3600}{1.15} \right)} = 38.11 \text{ cm}^2.$$

**Example (25) :**

**Given:** T – sec. With:

$$B = 120 \text{ cm.}$$

$$M_u = 80 \text{ m.t.}$$

$$b = 25 \text{ cm}$$

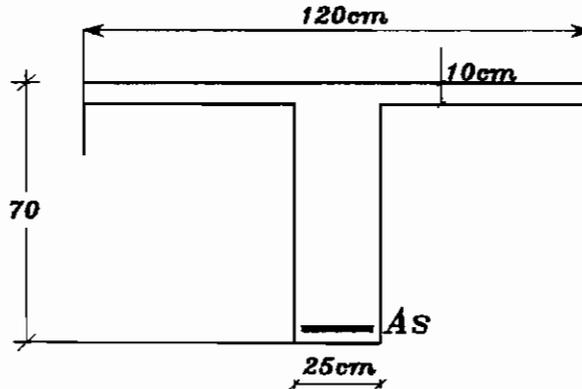
$$f_y = 3600 \text{ kg/cm}^2$$

$$P_u = 18 \text{ t. ( comp )}$$

$$f_{cu} = 250 \text{ kg/cm}^2$$

$$d = 65 \text{ cm.}$$

**Required As:**



Assume  $a < t_s$

$$e = \frac{80}{18} = 4.44 \text{ m}$$

$$\therefore e_s = 4.44 + \frac{0.7}{2} - 0.05 = 4.74$$

$$M_{us} = 18 * 4.74 = 85.32 \text{ m.t}$$

Using chart ( 2 ) page ( 4 ) :

$$d = C_1 \sqrt{\frac{M_{us}}{f_{cu} \cdot B}}$$

$$\therefore C_1 = 3.85 \quad \therefore c/d = 0.205$$

$$\therefore C = 0.205 * 65 = 13.33 \text{ and } a = 0.8 * 13.33 = 10.66 \text{ cm}$$

Then the assumption that  $( a < t_s )$  is not valid

Then the chart used in the previous two example ( chart 2 page 4 ) is not .

Valid and the design of the cross sec. ought to be carried out from the 1<sup>st</sup> principles.

The contribution of the compressive force in the web below the flange.

may be neglected, in this case the lever arm will be .

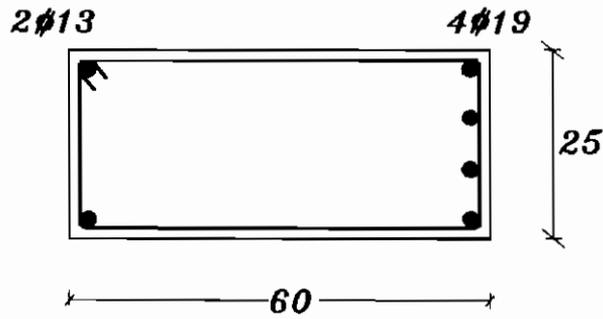
$$Y_{CT} = (d - \frac{t_s}{2}) = 65 - 5 = 60 \text{ cm}$$

$$\text{then : } T_u = \frac{M_u}{y_{ct}} + P_u = \frac{85.32}{0.6} - 18 = 124.2 \text{ t}$$

$$\therefore A_s = \frac{124.2 \times 1000}{\frac{3600}{1.15}} = 39.68 \text{ cm}^2$$

**Example (26):**

Construct the interaction diagram for the following cross section:



Take  $f_{cu} = 250 \text{ kg/cm}^2$  and  $f_y = 3600 \text{ kg/cm}^2$ .

1 – if the section is subjected to axial load only

$$\gamma_c = 1.75 \text{ \& } \gamma_s = 1.36$$

$$P_{u1} = 2.65 \times \frac{3600}{1.36} = 7014.7 \text{ kg.}$$

$$P_{u2} = 0.67 \times \frac{250}{1.75} \times 60 \times 25 = 143571.4 \text{ kg}$$

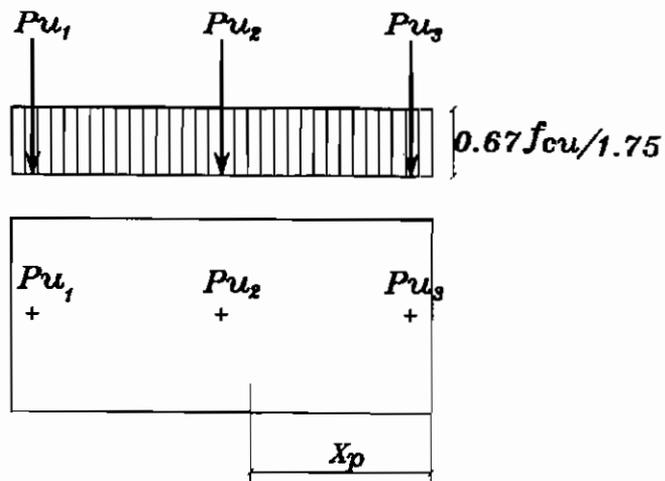
$$P_{u3} = 11.34 \times \frac{3600}{1.36} = 30017.9 \text{ kg}$$

$$P_u = 7014.7 + 143571.4 + 30017.6 = 180603.7 \text{ kg}$$

$$\therefore P_u = 180.6 \text{ t}$$

plastic Centroid :

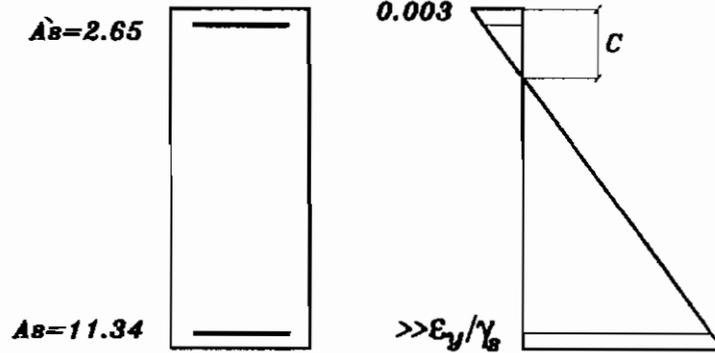
$$X_p = \frac{30.0176 \times 5 + 143.571 \times 30 + 7.014 \times 55}{180.6} = 26.82 \text{ cm}$$



2 - if the section is subjected to pure B.M:

$$\gamma_c = 1.5 \text{ and } \gamma_s = 1.15$$

Assume  $A_s^-$  is yielded:



$$T_u = C_u$$

$$\frac{11.34 \times 3600}{1.15} = \frac{0.67 \times 250}{1.5} \times 25 \times a + 2.65 \times \frac{3600}{1.15}$$

$$\therefore a = 9.74 \text{ cm} \quad \therefore c = 12.18 \text{ cm}$$

$$\varepsilon_{s^-} = 0.003 \times \frac{12.18 - 5}{12.18} = 1.77 \times 10^{-3}$$

$$\frac{\varepsilon_y}{\gamma_s} = \frac{3600}{2000000 \times 1.15} = 1.56 \times 10^{-3}$$

$$\therefore \varepsilon_{s^-} > \frac{\varepsilon_y}{\gamma_s}$$

$\therefore$  The assumption that  $A_s^-$  is yielded is correct one.

$$\therefore M_u = \left( \frac{0.67 \times 250}{1.5} \times 25 \times 9.74 \right) \times \left( 55 - \frac{9.74}{2} \right) + 2.65 \times \frac{3600}{1.15} \times 50$$

$$\therefore M_u = 17.78 \text{ mt}$$

3 - if the section is subjected to  $M_u$  and  $P_u$  at the balanced condition :

First we can assume that  $\gamma_c = 1.5$   $\gamma_s = 1.15$

From triangular similarity we can get  $C_b$  as following:

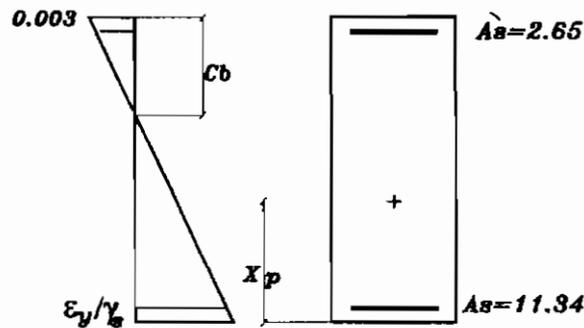
$$\frac{0.003}{0.003 + 0.00156} = \frac{C_b}{55}$$

$$\therefore C_b = 36.18 \text{ cm}$$

$$\therefore \varepsilon_{s^-} = 0.003 \times \frac{36.18 - 5}{36.18}$$

$$\therefore \varepsilon_{s^-} = 2.586 \times 10^{-3} > \frac{\varepsilon_y}{\gamma_s}$$

$\therefore A_s^-$  is yielded



$$P_u = \frac{0.67 \times 250}{1.5} \times 25 \times 0.8 \times 36.18 + 2.65 \times \frac{3600}{1.15} - 11.34 \times \frac{3600}{1.15} = 53598.5 \text{ kg}$$

$$\therefore P_{u,b} = 53.6 \text{ t}$$

$M_u = \Sigma$  moments @ the plastic centroid of the cross section

$$= \frac{0.67 \times 250}{1.5} \times 25 \times 28.944 \times \left( 60 - 26.82 - \frac{28.944}{2} \right) + 2.65 \times \frac{3600}{1.15} \times (55 - 26.82)$$

$$+ 11.34 \times \frac{3600}{1.15} \times (26.82 - 5)$$

$$M_{u,b} = 25.2 \text{ mt}$$

Check on assumed  $\gamma_c$  and  $\gamma_s$ :

$$e_b = \frac{25.2}{53.6} = 0.47 \quad \frac{e_b}{t} = 0.78$$

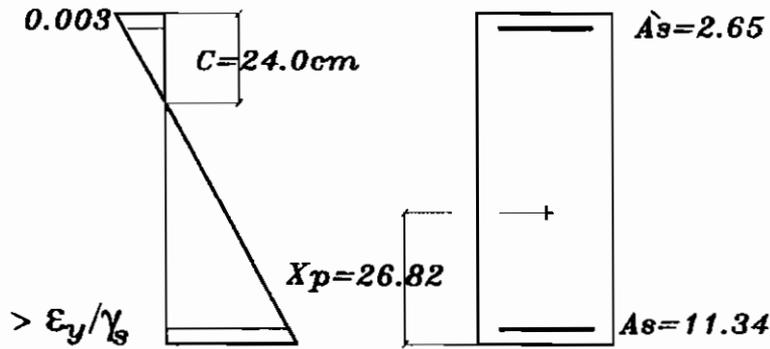
$$\gamma_c = 1.75 - 0.5(e/t) = 1.36 < 1.5 \quad \therefore \gamma_c = 1.5 \dots \dots \dots (\text{Eq. 3-16-a})$$

$$\gamma_s = 1.36 - 0.43(e/t) = 1.02 < 1.15 \quad \therefore \gamma_s = 1.15 \dots \dots \dots (\text{Eq. 3-16-b})$$

It is obvious that the assumed values are quite acceptable.

#### 4 – Locating a point below the balanced point ( A tension failure point )

we will locate point in the tension failure zone by assuming a value of ( e ) that lies between ( c ) at the pure moment condition and ( c ) at the balanced , i.e. between 12.18 cm and 36.18 cm say 24 cm.



This case represent a transitional case between the pure moment case and the balanced case, therefore we can make use of those two cases and deduce that both  $A_s$  and  $A_s'$  are yielded.

$$\therefore P_u = 0.67 \times \frac{250}{1.5} \times 25 \times 0.8 \times 24 + 2.65 \times \frac{3600}{1.15} - 11.34 \times \frac{3600}{1.15}$$

$$\therefore P_u = 26.4 \text{ t}$$

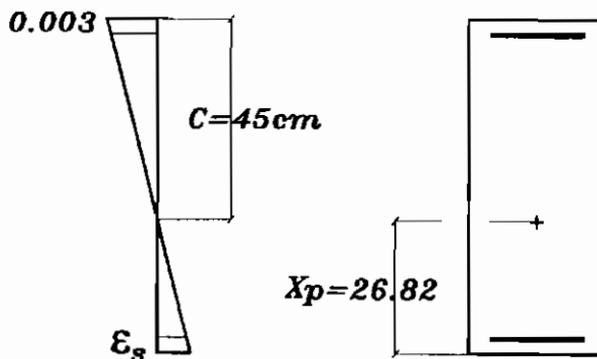
$$\& M_u = 0.67 \times \frac{250}{1.5} \times 25 \times 19.2 \times \left( 60 - 26.82 - \frac{19.2}{2} \right) + 2.65 \times \frac{3600}{1.15} \times (55 - 26.82)$$

$$+ 11.34 \times \frac{3600}{1.15} (26.82 - 5)$$

$$\therefore M_u = 22.72 \text{ mt}$$

5 – locating a point above the balanced point ( A compression failure point ):

This case lies between the balanced condition and the pure axial load condition i.e (c) lies between 36.18 cm and  $\infty$ . Say  $c = 45$  cm.



$$\varepsilon_s = 0.003 \times \frac{45-5}{45} = 2.67 * 10^{-3} > \frac{\varepsilon_y}{\gamma_s}$$

∴ As is yielded

$$\therefore A_s \text{ is yielded \& } f_s = 0.67 * 10^{-3} \times 2000000 = 1340 \text{ kg/cm}^2$$

Assume  $\gamma_c = 1.5$  and  $\gamma_s = 1.15$

$$\therefore P_u = 0.67 \times \frac{250}{1.5} \times 25 \times 0.8 \times 45 + 2.65 \times \frac{3600}{1.15} - 11.34 \times 1340$$

$$\therefore P_u = 93.6 \text{ t}$$

$$M_u = 0.67 \times \frac{250}{1.5} \times 25 \times 36 \times \left( 60 - 26.82 - \frac{36}{2} \right) + 2.65 \times \frac{3600}{1.15} \times (55 - 26.82) \\ + 11.34 \times 1340 \times (26.82 - 5)$$

$$M_u = 20.91 \text{ mt.}$$

Check on assumed  $\gamma_c$  &  $\gamma_s$ :

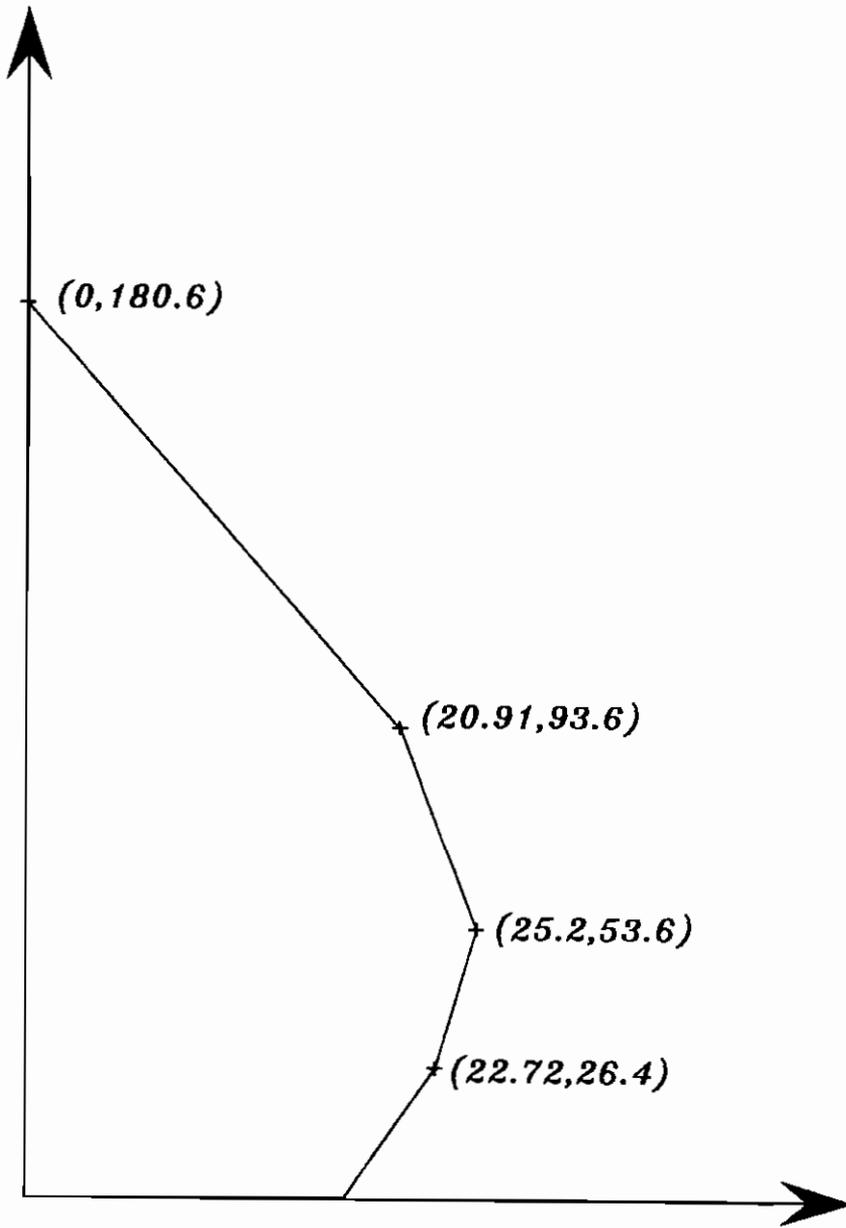
$$e = \frac{20.91}{93.6} = 0.22$$

$$\gamma_c = 1.75 - 0.5(e/t) = 1.75 - 0.5 \times \frac{0.22}{0.6} = 1.57$$

$$\gamma_s = 1.36 - 0.43(e/t) = 1.36 - 0.43 \times \frac{0.22}{0.6} = 1.20$$

Thus, the assumed values are acceptable.

Generally, as long as the neutral axis lies inside the cross sec. the previous assumed values are quite accepted.



*The Interaction Diagram*