البـــاب الخامـــس

CHAPTER 5

الإطـــــارات

Frames

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الباب الخامس

الإطارات Frames

٥ – ١ : مقدمة :

في معظم منشآتنا يتم استخدام النظام التقليدي (البلاطة – الكمرة – العمود) لتغطية المساحات المطلوبة بالأسقف الخرسانية ، أما في حالة ما إذا كانت المساحات المطلوب تغطيتها ذات بحور واسعة (Too Large Spans) ولا يسمح باستخدام أعمدة داخل المساحة نفسها فإننا نلجأ إلي استخدام نظام الإطارات (Frames) .

إذن تعتبر الإطارات أحد النظم الإنشائية المستخدمة في تغطية مساحات واسعة دون الحاجة إلي أعمدة داخلية ، ونحن عادة نبدأ في التفكير في نظام الإطارات حين يزيد البحر عن ١٢ م تقريباً .

وللإطارات أشكال هندسية كثيرة إلا أن أكثرها مثالية واقتصاداً هو ما أخذ شكل خط الضغط (Pressure Line) وهو في العادة يكون مقلوب مخطط عزوم الانحناء(B.M.D) ، ويبين شكل (١) فلسفة هذا الاعتبار في الحالات الثلاث الآتية :

أ – حالة حمل مركز واحد .

ب – حالة حملين مركزين .

جـ – حالة حمل موزع .

٥ – ٢ : تصنيف الإطارات طبقاً لدرجة التحدد الإستاتيكي :

Classification of frames according to indetermincy:

يمكن تقسيم الإطارات إلي نوعين :

١ – الإطارات المحددة استاتيكياً : Statically determinate frames .

يبين شكل (٢) مجموعة من الإطارات المحددة أستاتيكياً وهي تلك التي يمكن إيجاد قيم ردود أفعالها من خلال معادلات الاتزان دون حاجة لطرق حل خاصة ، وتستخدم الإطارات المحددة استاتيكياً في حالة التربة الأضعف نسبياً وكلما قويت التربة كلما كان استخدام الإطارات غير المحددة إستاتيكياً أنسب .

٢ – الإطارات غير المحددة استاتيكياً Statically indeterminate frames :

يبين شكل (٣) بعض أشكال الإطارات غير المحددة استاتيكياً ، ويلاحظ أنه كلما زادت درجة عدم التحدد كلما كان ذلك أنسب لأنواع التربة ذات المقاومة العالية .

٥ – ٣ :مزايا استخدام نظام الإطارات Advantages of R.C frame as a structural system:

الفرق بين الإطار والنظام العادي التقليدي (الكمرة والعمود) أنه يوجد وصلة قوية بين العنصرين الكمرة والعمود rigid connection تستطيع أن تتحمل عزوم الانحناء مما يساهم في تخفيض قيمة العزوم الموجبة في منتصف بحر الكمرة ، ويتولد عزوم سالبة عند أطرافها وينعكس علي قمة الأعمدة (أرجل الإطار) لتحقيق اتزان الوصلة .

وهذا السلوك يعيد توزيع العزوم ويقلل من قيمتها مما ينعكس علي كميات حديد التسليح والتكلفة .

شكل رقم (٤) يوضح كيف أن تحويل الكمرة الرئيسية من حالتها البسيطة إلي نظام الإطار يقلل من قيمة عزم الانحناء الموجب بدرجة كبيرة .

٥ – ٤ : الأبعاد الهندسية لعناصر الإطار Dimensioning of R.C frame :

من المعلوم أن عزم الانحناء المتولد عند الوصلة (connection) بين الكمرة الرئيسية (Girder) والعمود (column) يعتمد علي تناسب أبعادهما ، ويبين شكل (٥) ثلاث حالات رئيسية ذات أبعاد كبيرة وصغيرة ومتوسطة بالنسبة للعمود (رجل الإطار) ومدي تأثير ذلك علي قيم عزوم الانحناء ، وعليه فإنه من الأنسب أن يكون عمق الكمرة = طول قطاع رجل الإطار عند التقائهما عند الوصلة .

- **من الخبرات العملية السابقة يمكن وضع العلاقات الآتية شكل (٦) .**

L : span of frame = (12 – 25 m) .
H : height of frame .
h_c : clear height of frame = H - $\frac{1}{2}$ t_g .
b: breadth of frame = 30 – 50 cm .

$$t_g = \frac{span}{12-16} \cong \frac{L}{14}.$$

$$t_u = (0.6 \rightarrow 1)t_g \cong t_g.$$

$$t_t = 0.6tg.$$

$$t_3 \cong t_g \Rightarrow for.simplicity.$$

$$t_4 = (0.5 \rightarrow 0.6)t_g.$$

$t_5 =$ from design (OR) = 0.6 t_g.

$$t_6 = 0.5t_5$$

$t_7 = t_{link} =$ greater of 0.4 t_g or L / 20

٥ – ٥ : ترتيب أماكن الإطارات :

يبين شكل (٧) طريقة ترتيب الإطارات متتالية الواحد تلو الآخر بهدف تغطية إحدى الصالات حيث ، يمكن ملاحظة الآتي :

١ – المسافات البينية بين الإطارات تكون في حدود من ٤ : ٦ م .

٢ – يراعي عند وضع الكمرات الثانوية أن تكون بينها بلاطات ثنائية توزع الأحمالtwo ways slabs

٣ – يراعي أن يتم عمل فاصل تمدد كل مسافة (٣٠ : ٤٠ م) بعرض في حدود ٢ سم .

٤ – من المناسب أن يكون ارتفاع الإطار مساويا لنصف بحره حيث يساعد ذلك علي تكوين قيم عزوم موجبة وسالبة مناسبة .

٥ – تنتقل الأحمال من البلاطات للكمرات الثانوية ومنها لكمرة الإطار Frame girder .

٦ – في العادة تكون أبعاد الكمرات الثانوية في حدود ٢٥ × ٥٠ .

– Loads on frame:

٥ – ٦ : حســاب الأحمــال :

مثـــال :

الموضح بشكل(٨) إطار له الخصائص الآتية :

سمك البلاطات = ١٢ سم .

الحمل الحي = ٠,٢ طن / م^{٢} .

أبعاد الكمرات الثانوية = ٢٥ × ٥٠ .

احسب قيم الأحمال علي الإطار .

الحـــل :

$$t_g = \frac{L}{12-16} \cong \frac{L}{14} = 114.29 \cong 120cm.$$

For cantilever $t_{bigger} = 120$ cm , $t_{smaller} = 60$ cm , $t_{av} = 90$ cm .

$W_{su} = 1.4\, g_s + 1.6\, p_s = 1.4\,(0.12 * 2.5 + 0.15) + 1.6 * 0.2 = 0.95$ t / m^2 .

شكل رقم (٩) يبين شكلا مجردا statical system للإطار وعليه الأحمـــال بنوعيـــها المـــوزع والمركز ، ويلاحظ أن الحمل الموزع ينتج من الوزن الخاص بكمرة الإطار girder بالإضافـــة لأحمال المثلثات الثمانية في ناحيتي الكمرة ، أما الأحمال المركزة فهي نتيجة ارتكـــاز الكمـــرات الثانوية علي كمرة الإطار .

Distributed load:

w_1 = o.w of frame girder * 1.4 + slab load

$$= \gamma_c \times t_g \times b \times 1.4 + (\sum area\ of\ slabs)\frac{W_{su}}{span}$$

$$\therefore w_1 = 2.5 \times 1.2 \times 0.35 \times 1.4 + \frac{W_{su}}{16} \times 8[4 \times 2 \times 0.50] = 3.37 t/m^{-}.$$

$$w_2 = o.w.of\ cantilever \times 1.4 + slab\ load$$

$$= 2.5 \times 0.90 \times 0.35 \times 1.4 + \frac{0.95}{4} \times 2[4 \times 2 \times 0.5] = 3\, t/m^{-}.$$

Concentrated loads :

P = spacing * (o.w of sec. beam *1.4 + slab load) .

$$P = 5 \times (\gamma_c * 0.25 * 0.5 * 1.4 + \sum \text{area of slabs on sec beam} * \frac{w_{sl}}{span})$$

$$\therefore P_1 = 5\left[2.5*0.25*0.5*1.4+\frac{0.95}{5}*2\left[\frac{5+1}{2}*2\right]\right] = 13.59 \text{ ton}$$

$$p_2 = 5\left[2.5*0.25*0.5*1.4+\frac{0.95}{5}\left[\frac{5+1}{2}*2\right]\right] = 7.89 \text{ ton}$$

تبسيـــط :

في حالة وجود عدد من الأحمال المركزة (ثلاثة فأكثر) يوصي بتوزيع هـــذه الأحمـــال باعتبارها حملا موزعا مكافئا (شكل ١٠) .

$$weq. = \frac{\sum p}{span} * 1.10$$

الحمل المكافئ $= \frac{3*13.59}{16.0} * 1.1 = 2.8 \text{ t/m}^{-}$.

حســـاب مؤثـــرات القـــوي الداخليـــة :

Straining actions:

والمقصود بها قيم :

١ – عزوم الانحناء . - B . M . D

٢ – قوي القص . - N . F . D

٣ – القوي العمودية . - S . F . D

- يتم عمـل جـدول كالتالـي :

Sec N_o	M_u	N_u	Sec is designed for
1		بالإشارة	M_u only
2			N_u only
3			M_u & N_u
⋮	⋮	⋮	⋮

ويجب رسم منحنيات مؤثرات القوي الداخلية بصورة صحيحة وتجنب الخطأ فيها لاعتماد شـكل التسليح عليها ، كما سيتضح فيما بعد : -

- تصميم قطاعات الإطار : **Design of frame sections :**

يتم تصميم جميع القطاعات التي تتغير عندها (M_u , N_u) طبقاً لما سبق شرحه فـي الأبـواب السابقة .

1 – Section subjected to M_u only:

use R – ω curve :

$$R = \frac{M_u}{fcu.bd^2} \rightarrow \omega$$

$$A_s = \omega bd \frac{f_{cu}}{f_y}$$

Note:

If section subjected to M_u & N_u (- ve) but $N_u \leq 0.04\ f_{cu}\ A_c$.

$\therefore$ neglect N_u and design for M_u only .

2 – Section subjected to M_u & N_u (-ve : comp.) , e / t < 0.05 :

∴ Neglect M_u and design for N_u only as short column.

$N_u * 1000 = 0.35\ f_{cu}\ A_c + 0.67\ f_y\ A_{sc}$.

3 – Section subjected to M_u & N_u (-ve : comp.) : But e / t > 0.05 .

∴ Design the section for M_u & N_u as ecc . sec .

∴ use interaction diagram .

∴ use chart of : $\zeta = 0.9$ & $\alpha = 0.60$

فإذا وقع القطاع في منطقة الضغط (شكل ١١) يتم الحل باستخدام منحنيات التفاعـــل (Interaction diagrams) .

$$curve \xrightarrow{get} \rho \rightarrow \mu = \rho f_{cu} \times 10^{-5}$$

$$\therefore A_s = \mu bt$$

$$\& \ As^- = \alpha\ As.$$

أما لو وقع القطاع في منطقة الشد فلا تستخدم منحنيـــات التفـــاعل(Interaction diagrams) ، ولكن يمكن استخدام المخطط البياني $R-\omega$.

$$e_s = e + t/2 - cover \rightarrow Mu_s = Nu.es$$

$$\therefore R = \frac{Mus}{f_{cu} bd} \xrightarrow{\alpha=0.4} get.w$$

$$As = \omega bd \frac{f_{cu}}{f_y} - \frac{Mu}{f_y / \gamma_s} \ \& \ As^- = \alpha\ As.$$

وغالبا ما يكون هذا القطاع في الجزء العلوي من رجل الإطار .

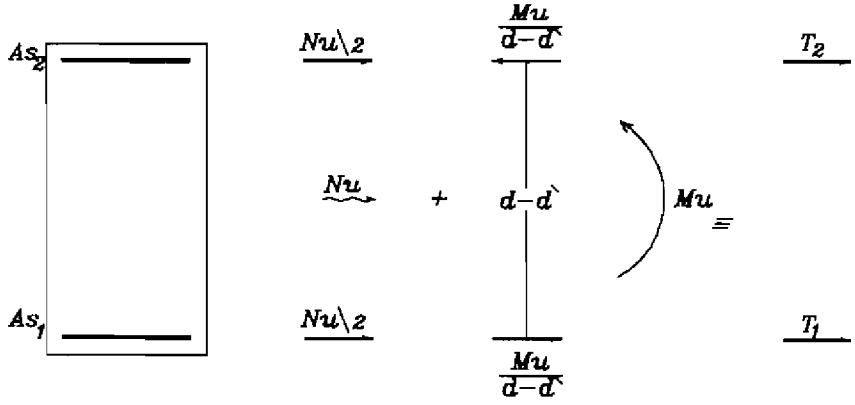
4 – Section subjected to M_u & N_u (+ve : tension) : & $e/t < 0.5 \therefore$ *Small.ecc.*

$$T_1 = \frac{N_u}{2} + \frac{M_u}{d - d'}.$$

$$T_2 = \frac{N_u}{2} - \frac{M_u}{d - d'}$$

$$\therefore As_1 = \frac{T_1}{f_y / \gamma_s}$$

$$As_2 = \frac{T_2}{f_y / \gamma_s}$$



شكل (١٢)

5 – Section subjected to M_u & N_u (+ve : tension) : & $e/t > 0.5 \therefore$ *L*arg.*ecc.*

Use.R ω (*curve*) :

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

$$\therefore M_{us} = N_u \times e_s.$$

$$R = \frac{M_{us}}{f_{cu} b d^2} \xrightarrow{curve} \omega$$

$$\therefore A_s = \omega\, bd \frac{f_{cu}}{f_y} + \frac{N_u}{f_y / \gamma_s}.$$

٥ – ٧ : تصميم أرجل الإطار

Design of frame legs :

شكل (١٣) يبين إطاراً له رجلان إحداهما عنصر وصل link member ، كما يلاحظ أن هناك كمرات متعامدة علي الإطار في منتصف الارتفاع تقريباً للمساهمة في حمل الحوائـــط الطـــوب والتي يجب أن لا تزيد المساحة المستمرة من المباني عن حد معين (من ٢٠– ٢٥ م٢) .

– ولتصميم الرجل اليمني شكل (١٤) : نصمم قطاعاً له عمق = t_{av} يقع عند ثلثـــي ارتفـــاع الرجل ، وهنا يجب دراسة القطاع في كلا الاتجاهين (x , y) .

$$\therefore t_{av} = t_L + \frac{2}{3}(t_u - t_L).$$

x – direction : -

Top end condition case 1 ($t_g \geq t_u$) .

bottom end condition case 3 (hinged support) .

$He = K.\ clear\ Height\ (H_c)$

$\lambda_{av.} = He / t_{av}$

$If : \lambda_{av.} \leq 10(braced), \lambda_{av.} \leq 15(unbraced)$

$\therefore$ no additional M i.e. (short column) .

$, If : \lambda_{av.} > 10(braced), \lambda_{av.} > 15(unbraced)$

$\therefore M_{addy}$ exist (long column) .

$M_{addy} = \delta . N_u .$

$$\delta = \frac{\lambda_{av.}^2 * t_{av.}}{2000}.$$

y – direction : (١٥) شكل

Top end condition :

$t_b > b \rightarrow \therefore case(1)$

$t_b < b \rightarrow \therefore case(2)$

Bottom end condition fixed at support $\therefore$ (case 1) .

$\therefore$ He = K * (larger of H_1 & H_2) .

$\lambda_b = \frac{He}{b}$.

$If : \lambda_b \le 10.$ braced $or : \lambda_b \le 15$ unbraced

$\therefore$ No M_{add} (short column) .

$, If : \lambda_b > 10$ braced . $or : \lambda_b > 15$ unbraced

$\therefore M_{addx}$ .exists (long column) .

$$, M_{addx} = \delta . N_u \ \& \ \delta = \frac{\lambda_b^2 * b}{2000}$$

يتم تصميم قطاع العمود (sec.1)علي أبعاد (b * t_u) شكل (١٦) وعلي القوى الآتية :

$N_u = \text{Re}action$

$M_{u.design} = M_{u.1-1} + M_{addy}(in.x - dir)$ حيث M_{u1-1} هو العزم الأصلي علي القطاع

$M_{u.design} = M_{addx}(in.y - dir)$

لا حظ لا يوجد عزم أصلي علي القطاع في هذا الاتجاه .

٥ – ٨ : تصميم عنصر الوصل: **– Design of link member:**

نفس الخطوات المتبعة سابقا عدا :

١ – لا يوجد t_{avr} لأن t_{link} ثابتة علي طول العنصر .

٢ – $M_{odd} = Mu_{design}$ لأنه لا يوجد عزم أصلي علي القطاع .

$e = M_u / N_u$

If $e / t < 0.05$ ..

$\therefore$ Design as short column & neglect M_u .

$N_u * 10^3 = 0.35 f_{cu} A_c + 0.67 f_y * A_{sc}$.

get A_{sc} .

If A_{sc} = -ve .

$\therefore$ Use $As_{min} = 0.006 A_c$.

٥ – ٩ : تصميم عنصر الربط : **- Design of tie member:**

شكل (١٧) حيث يتم فرض أبعاد الشداد كالتالي : –

$b_{tie} = b_{girder} = 30 – 40$ cm .

$t_{tie} = 50 – 60$ cm .

يتم تصميم الشداد علي قيمة N_u بالإضافة إلي $M_{u.o.w}$ (العزم الناتج عن الوزن) .

$$M_{uo.w} = \frac{Wuo.wL^2}{8}$$

ملاحظة : يمكن تصميم الشداد علي N_u فقط في حالة استعمال شمعات الشد (posts) فيكـــون منحني العزوم للشداد تحت تأثير وزنه قيمة صغيرة يمكن إهمالها (شكل ١٨) .

$\therefore Design.tie.for.N_u only$

$$As_{tie} = \frac{N_u * 1000}{f_y / \gamma_s} cm^2.$$

ويتم توزيع As_{tie} علي القطاع بالتساوي .

٥ - ١٠ : التحقـق مـــن القـص : **Check of shear :**

القطاع الحرج في القص يكون علي بعد $\frac{d}{2}$ من وجه العمود (شكل ١٩) .

$$\therefore Qured = Q_u - W_u(\frac{column.\dim}{2} + \frac{d}{2})$$

$$q_u = \frac{Qu * 10^3}{bd}$$

$$q_{cu_{max}} = 2.2\sqrt{\frac{f_{cu}}{\gamma_c}}$$

$$q_{cu} = 0.75\sqrt{\frac{f_{cu}}{\gamma_c}}$$

$$q_{su} = q_u - 0.5q_{cu}$$

If $q_u > q_{cu\,max}$.

$\therefore$ concrete dimensions. are unsafe

$\therefore$ Increase b or d .

If $q_u < q_{cu}$

$\therefore$ use $A_{st.min}$

$\therefore A_{st.\min} = \frac{3.5b.s}{f_{y_{st}}/\gamma_s}$ area of all branches

If $q_{cu} < q_u < q_{cu.max}$ $\therefore$ we need stirrups $\therefore A_{st} = \frac{q_{su}.b.s}{f_{y.st}/\gamma_s}(area.of.all.branches)$.

Design of hinged Base: **٥ – ١١ : تصميم الركيزة المفصلية شكل (٢٠) :**

Bearing check:

$$f = \frac{y_A}{A_1} \nmid f_b .$$

$$f_b = 0.67 f_{cu} / \gamma_c \nmid 0.67 f_{cu} / \gamma_c \sqrt{\frac{A_2}{A_1}}$$

$A_1 = b . t_f / 3.$

$A_2 = (2 * h + t_f / 3) . (2h + b) .$

If $f < f_b$ $\therefore$ safe .

If $f > f_b$ increase lead plate width to $\frac{t_L}{2}$

2 – Area of dowels : تصميم عدد وقطر (الأشاير):

$$As_{dowels} = \frac{X}{0.8 f_y / \gamma_s}$$

AS_{dowels} = area of all dowels.

3 – Horizontal stirrups: **الكانات الأفقية شكل (٢١) :**

distributed through height t_t .

$$T = \frac{y_A}{4} \rightarrow \frac{y_A}{6}$$

$$As_h = \frac{T}{f_y / \gamma_s}$$

$As_h = A_{req}$ for all stirrups both branches Number of stirrups = $\frac{As_h}{2 \times 0.785}$

Where 0.785 = A_s of 1ϕ10

4 – Inclined closed stirrups : **الكانات الركنية شكل (٢٢) :**

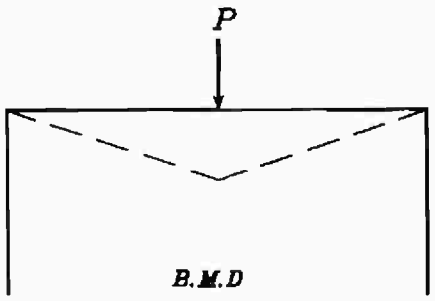
$T = \frac{A_s f_y}{\gamma_s}$ Closed stirrups.

$A_{st} = \frac{\sqrt{2}T}{4 f_{y,st} / \gamma_s} = 0.35 \frac{f_y}{f_{yst}} . As = 0.53 A_s .$ where T = splitting force .

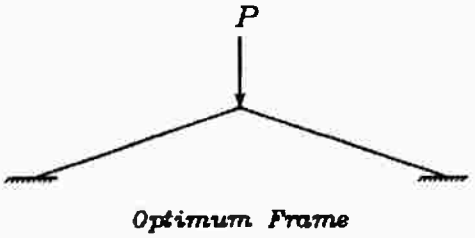
A_{st} = area of all stirrups for both branches.

Using ϕ10

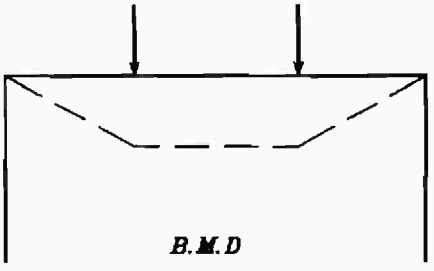
∴ Number of closed stirrups = $\frac{A_{st}}{2 * 0.785}$.



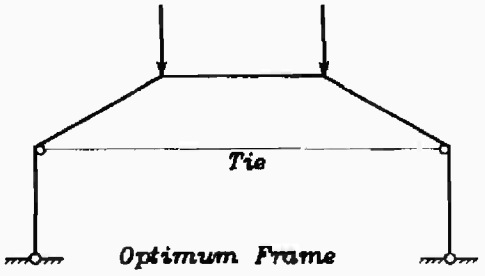
For one concentrated load



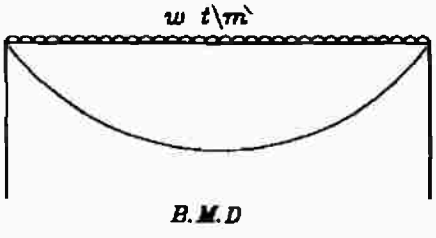
أ-فى حالة حمل مركز واحد



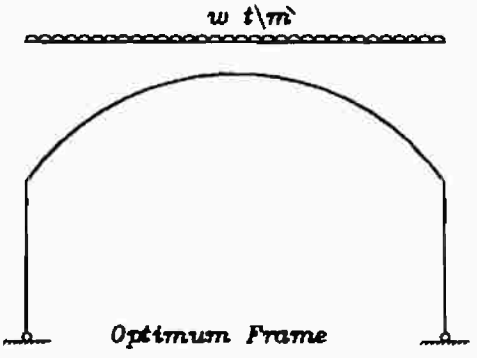
For two concen. Loads



ب - فى حالة حملين مركزين

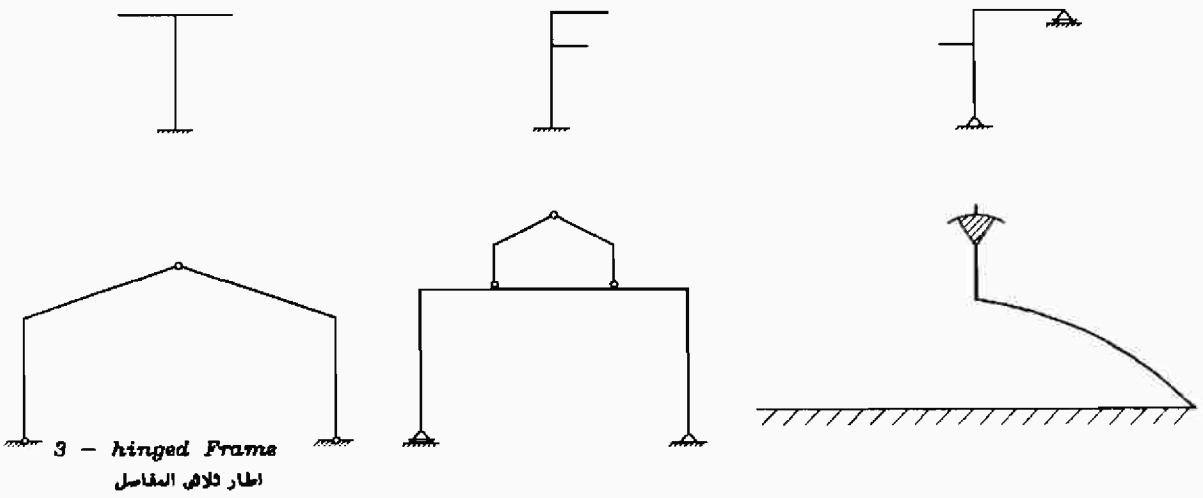


Distributed Load



جـ - فى حالة حمل موزع

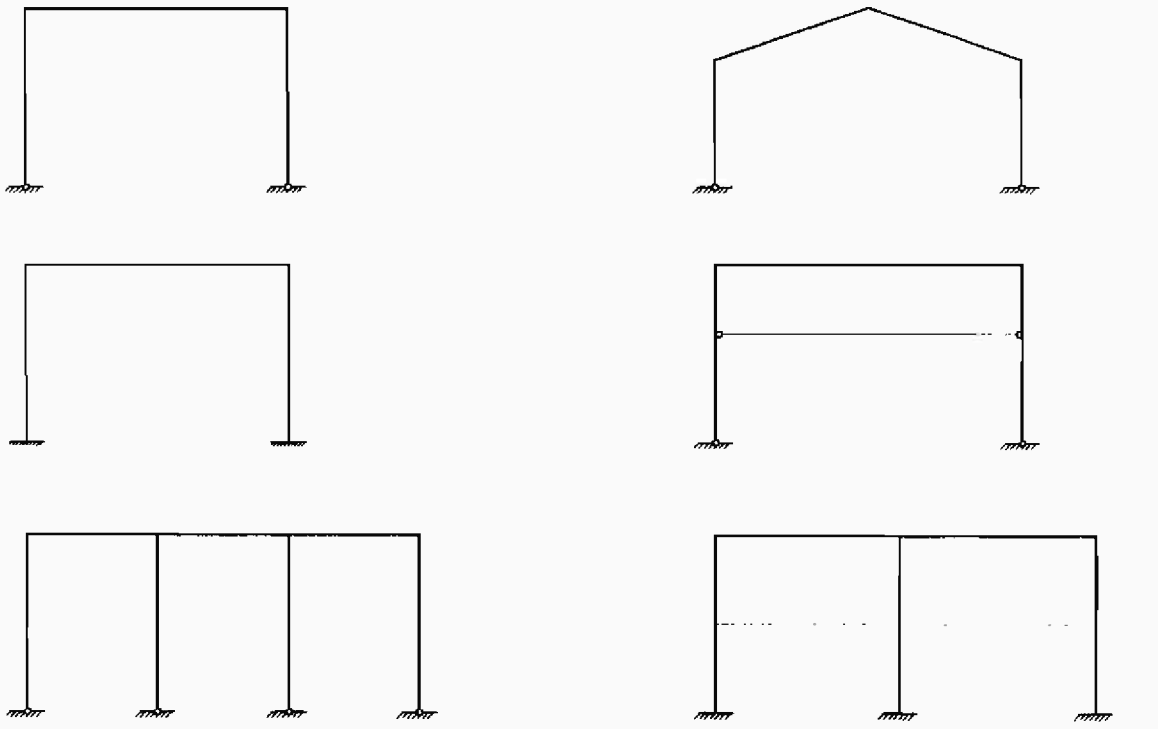
شكل (١) الشكل المثالى للاطار



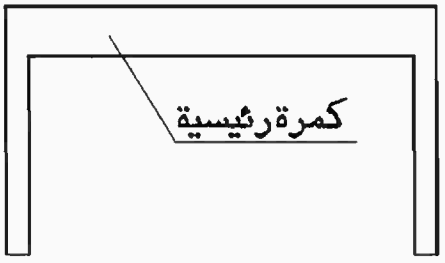
Statically Determinate Frames .

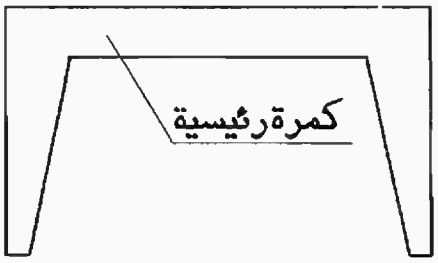
الاطارات المحددة استاتيكيا

شكل (٢) أمثلة للاطارات المحددة استاتيكيا



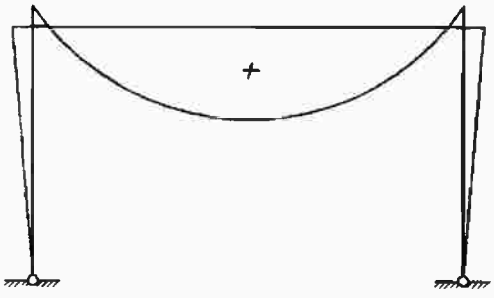
شكل (٣) أمثلة للاطارات الغير محددة استاتيكيا

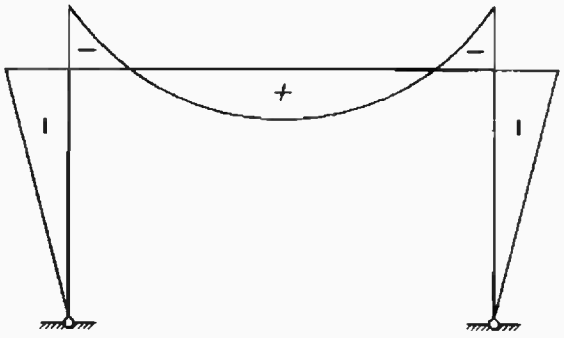




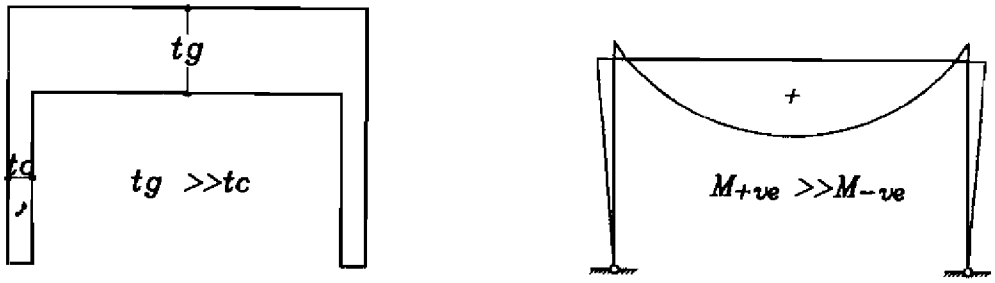
imple beam with Columns

R.c Frame

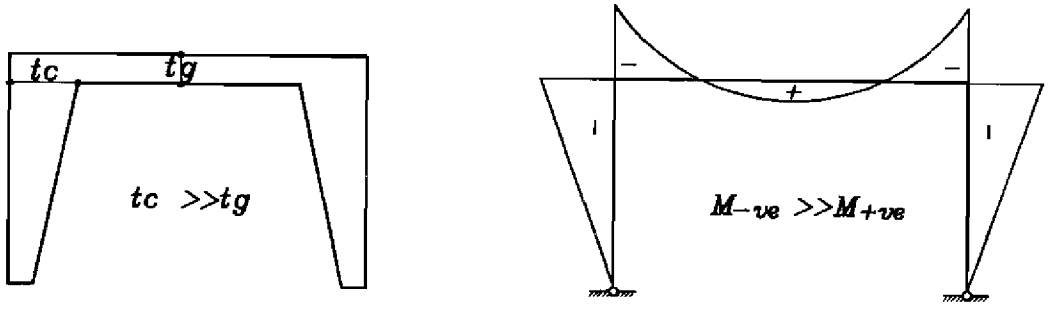




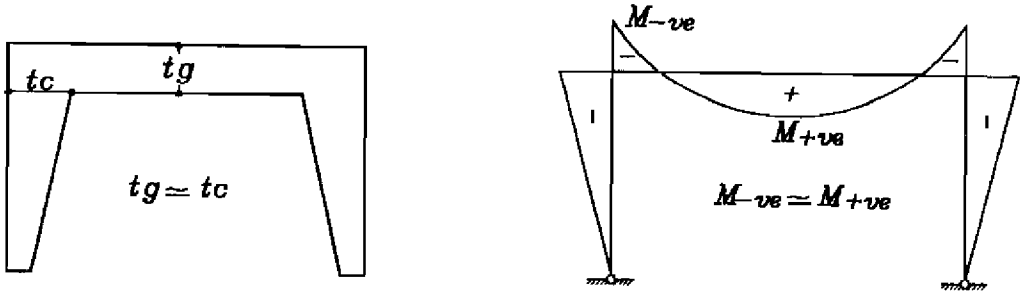
شكل (٤) الكمرة الرئيسية فى حالتها البسيطة و الاطار .



Case of too large Girder

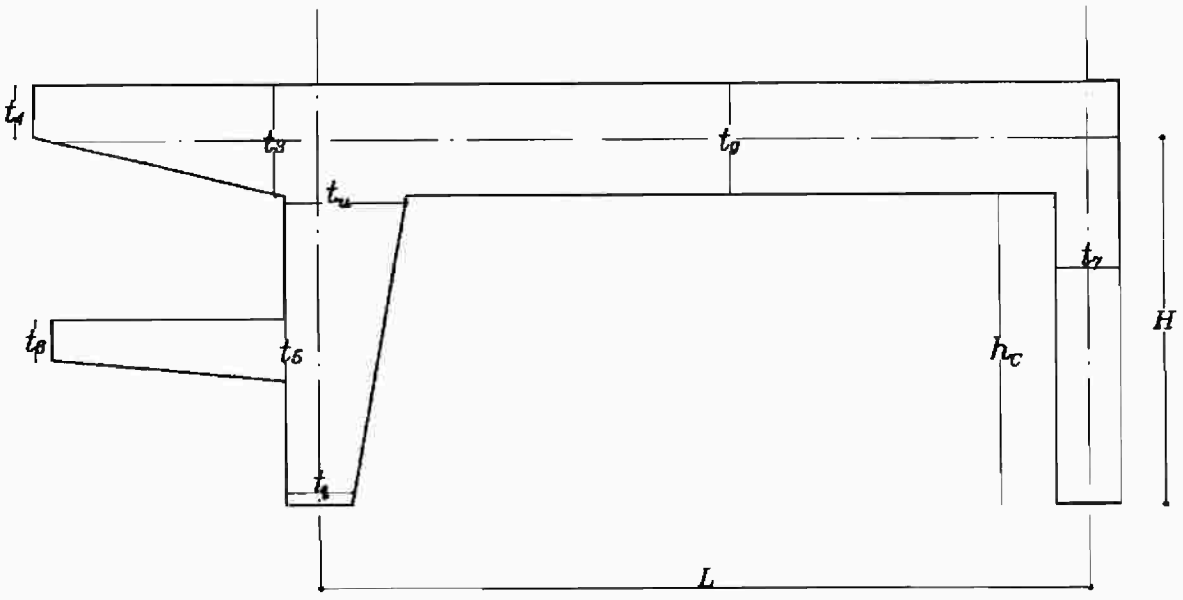


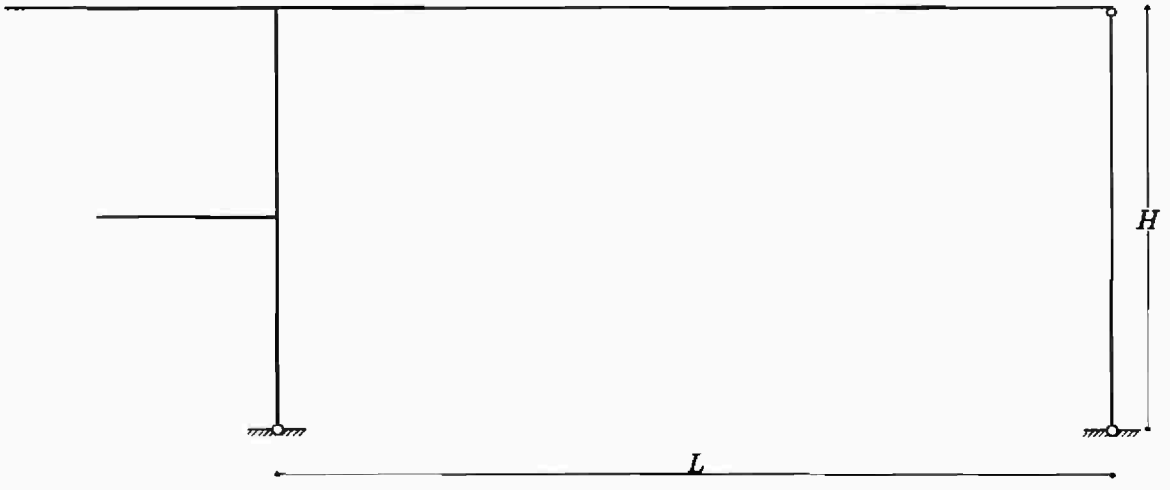
Case of too large Column



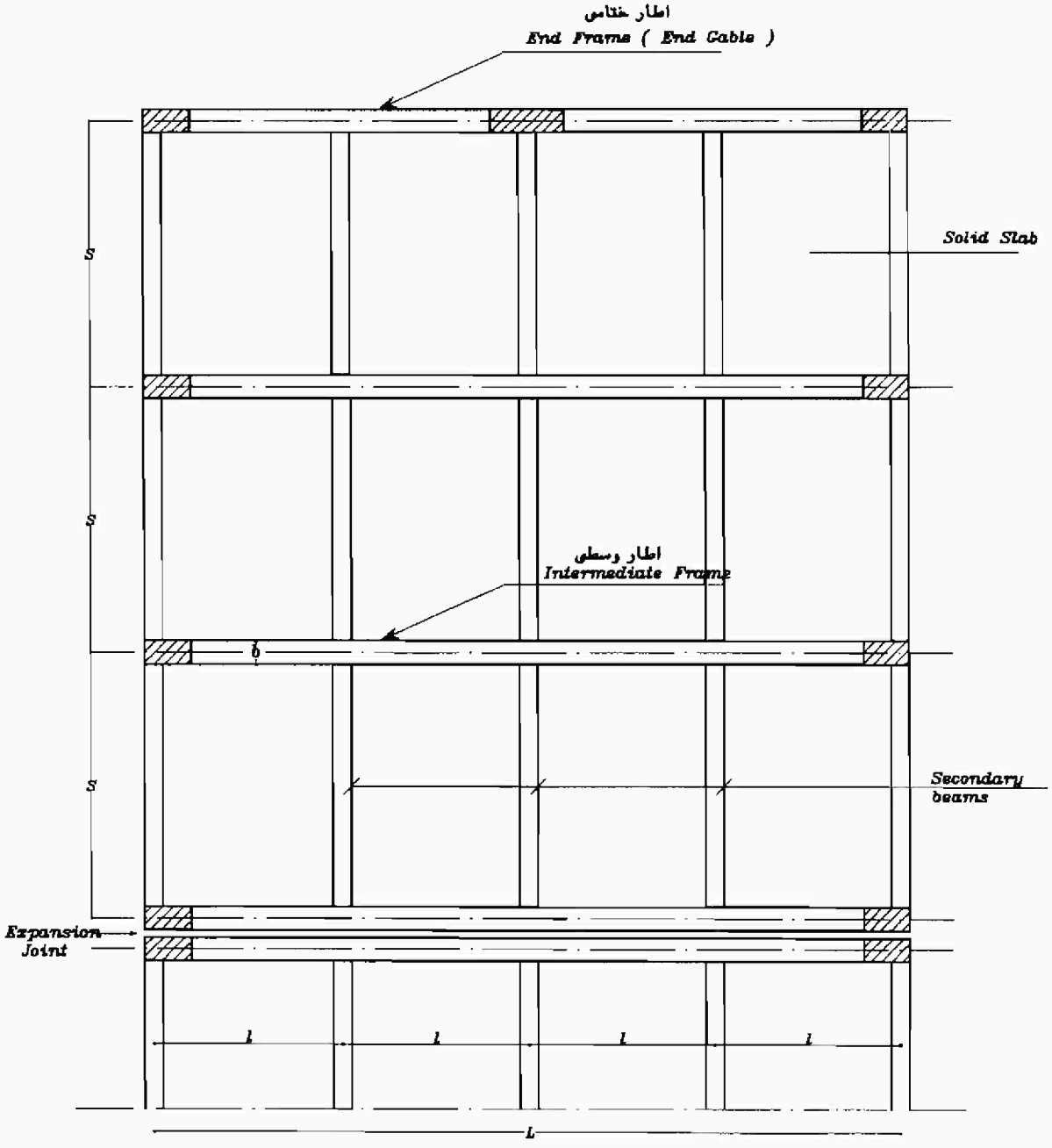
Case of relatively Equal dimensions *(The best case)*

شكل (٥) علاقة أبعاد الكمرة بأبعاد رجل الاطار.

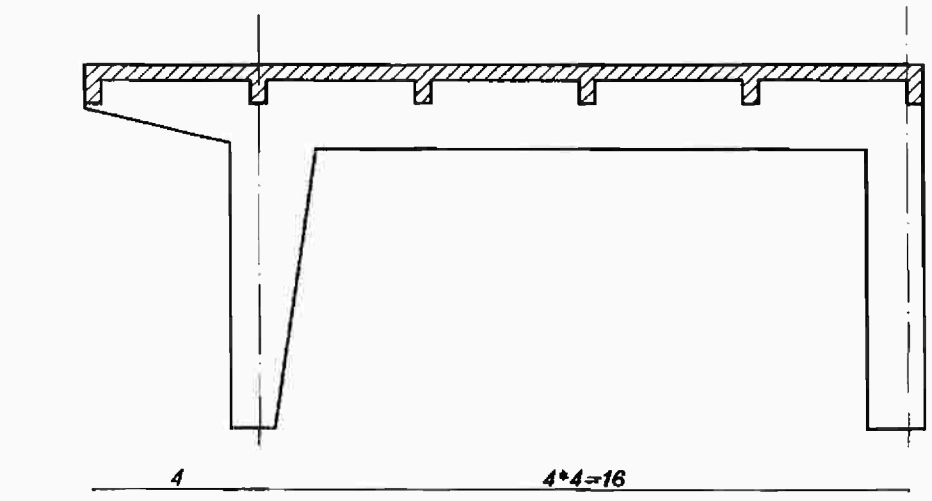


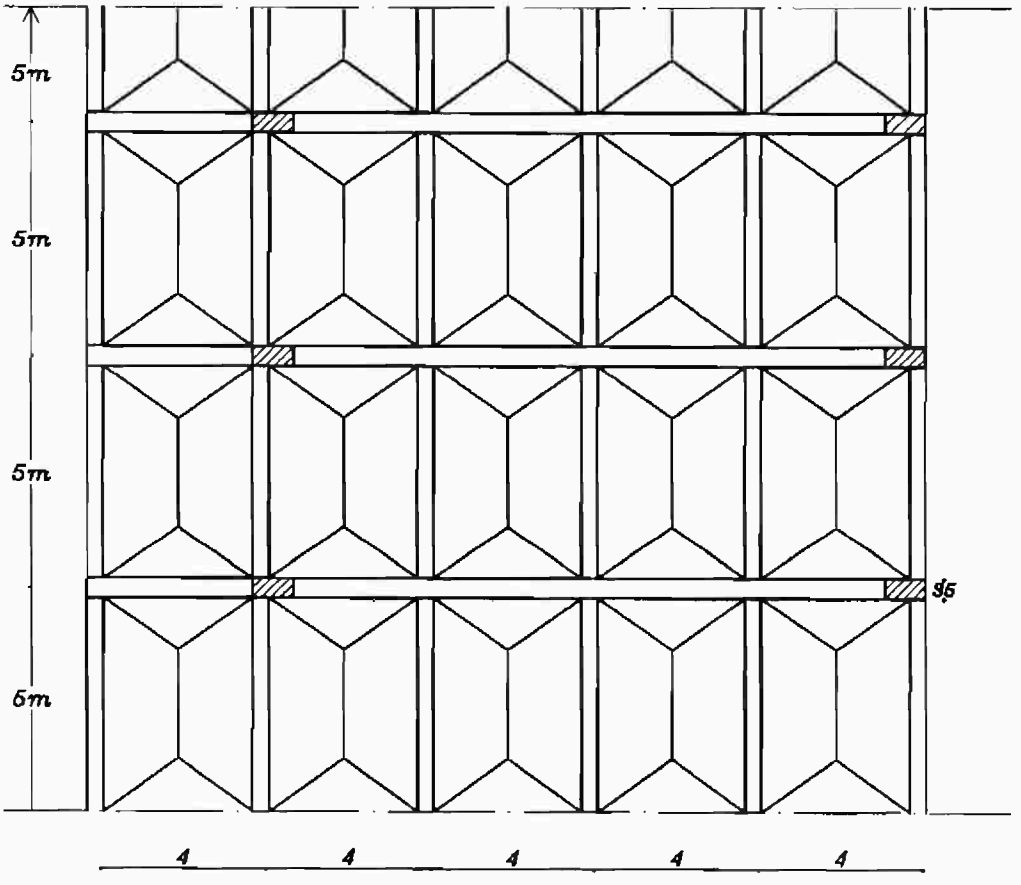


شكل (٦) علاقات الابعاد العندسية لعناصر الاطار

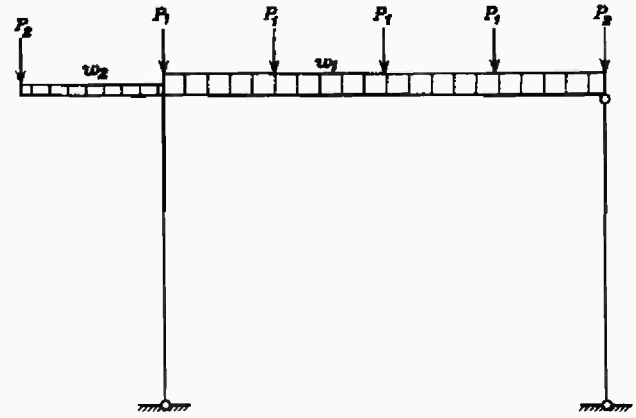


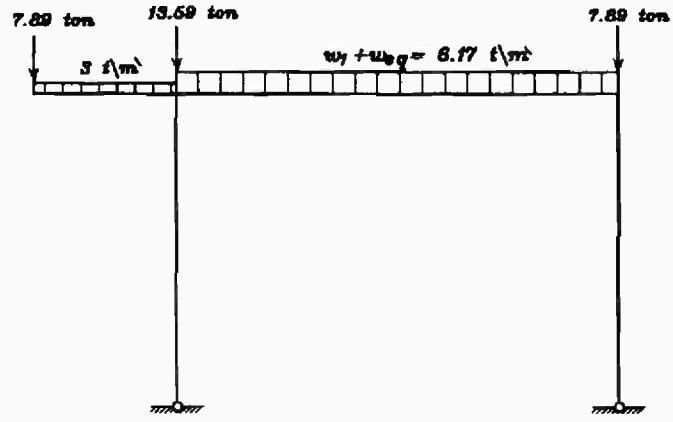
شكل (٧) طريقة ترتيب الاطارات .



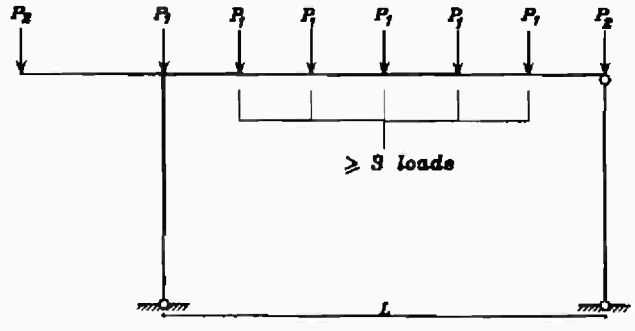


شکل (۸) اطار دو کابولی





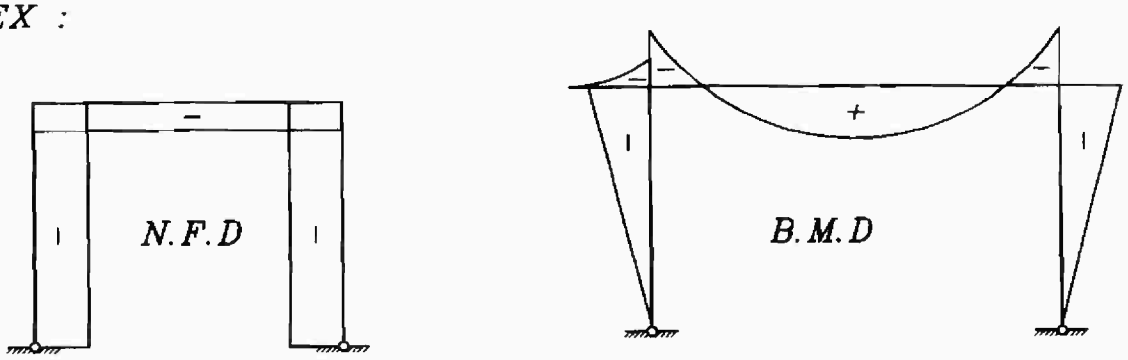
شكل (٩)

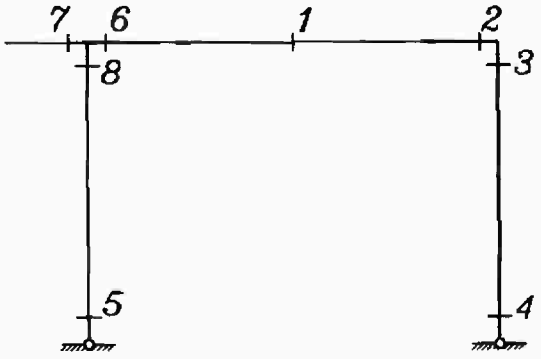


$$Weq = \frac{\Sigma P}{span} * 1.10$$

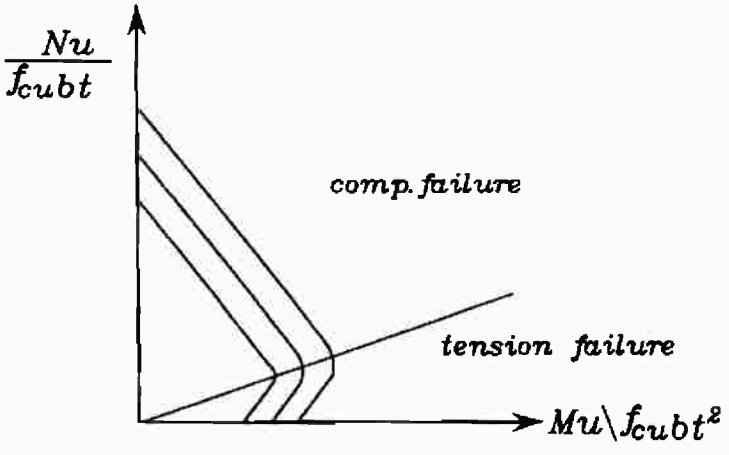
شكل (١٠) حالة عامة

EX :

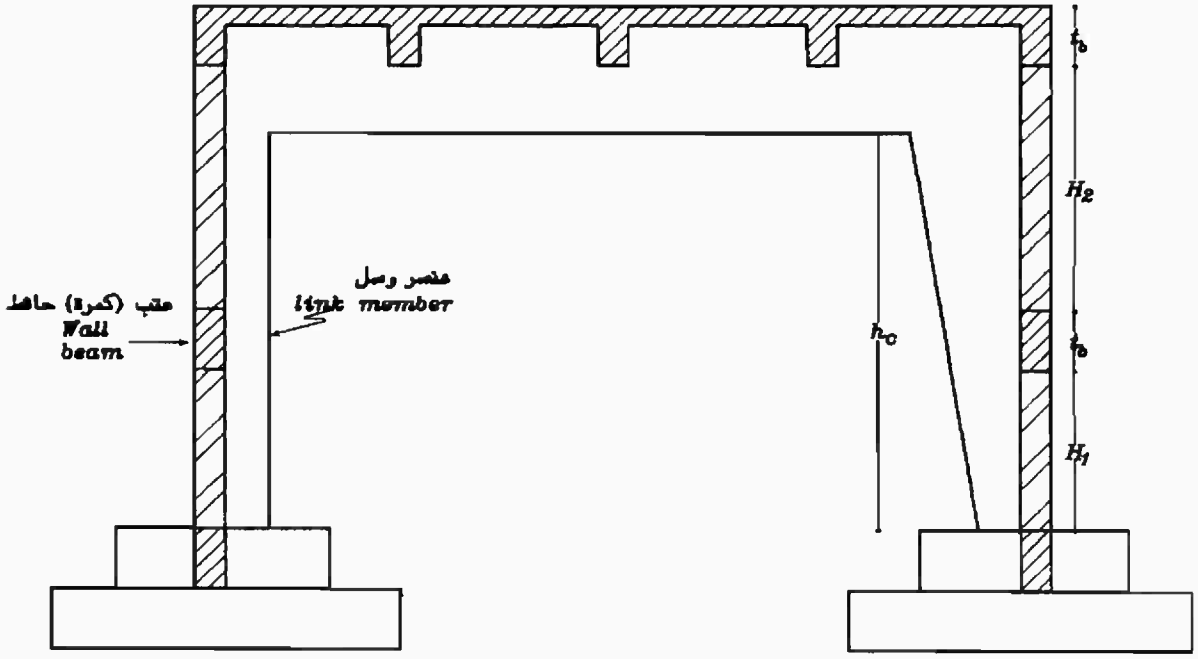


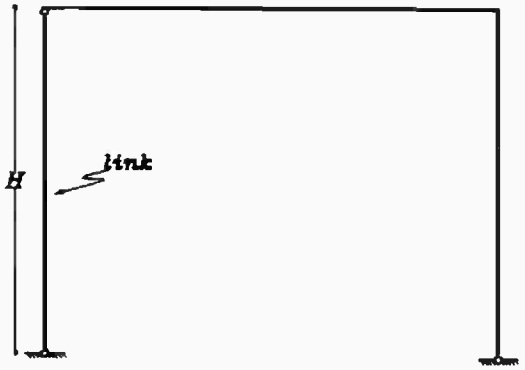


يجب تصميم الثمانية قطاعات الموضحة

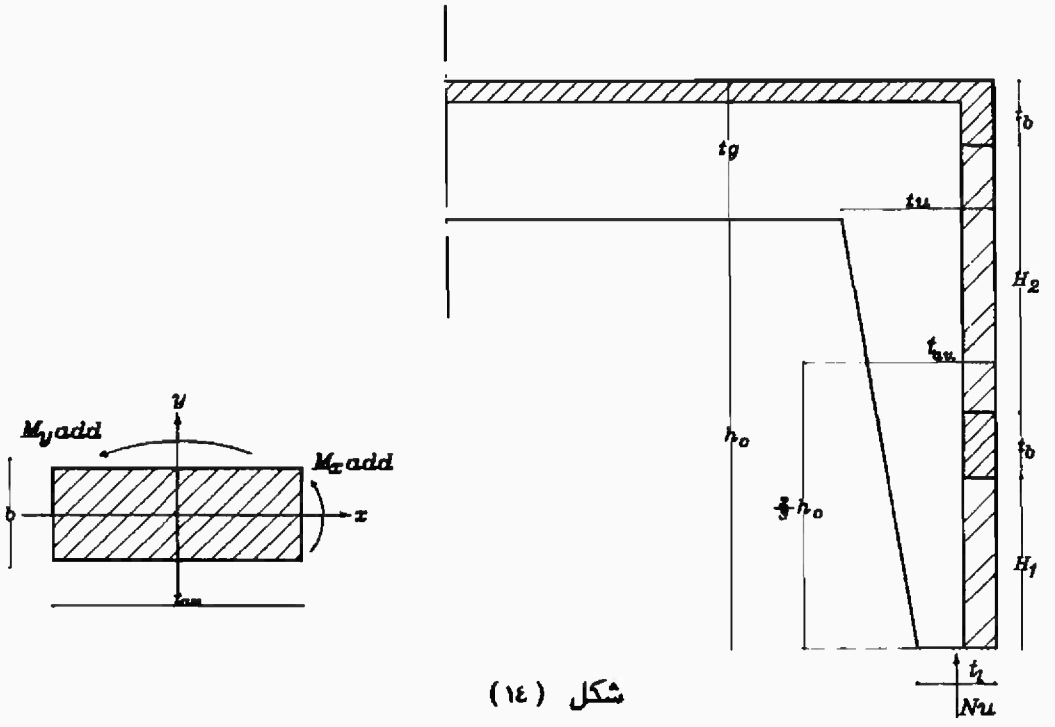


شكل (١١)

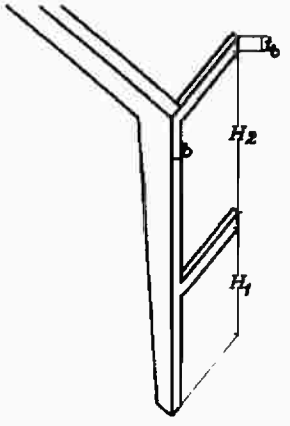

t_b
H_2
عنصر وصل
link member
عتب (كمرة) حائط
Wall
beam
h_c
t_b
H_1


H
link

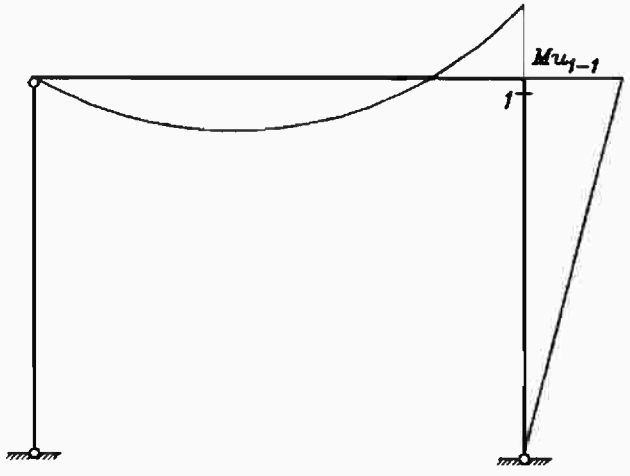
شكل (١٣)



شكل (١٤)



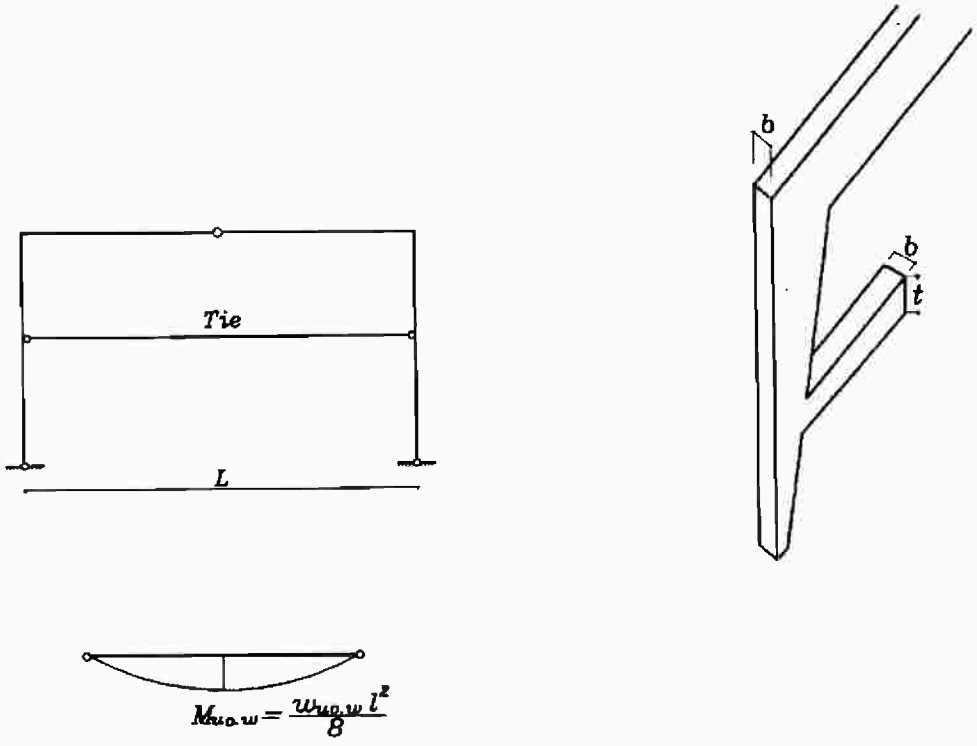
شكل (١٥)



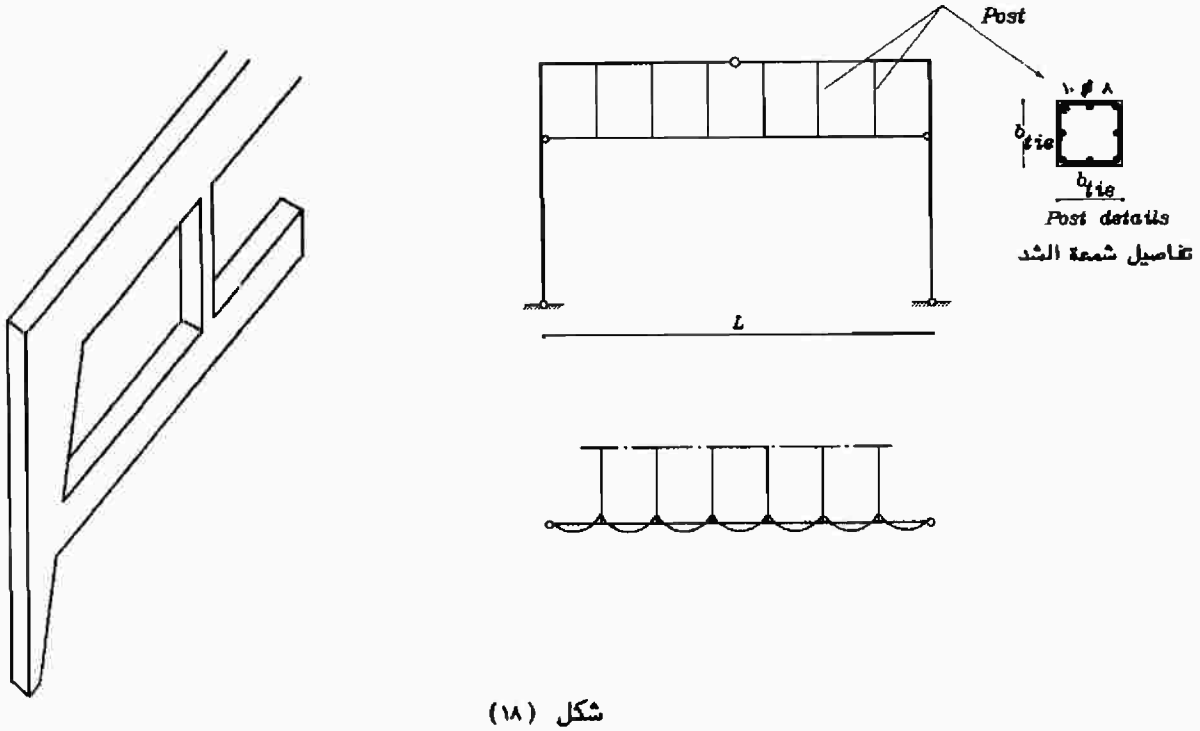
لاحظ لا يوجد عزم اصلى على القطاع

فى هذا الاتجاه ←

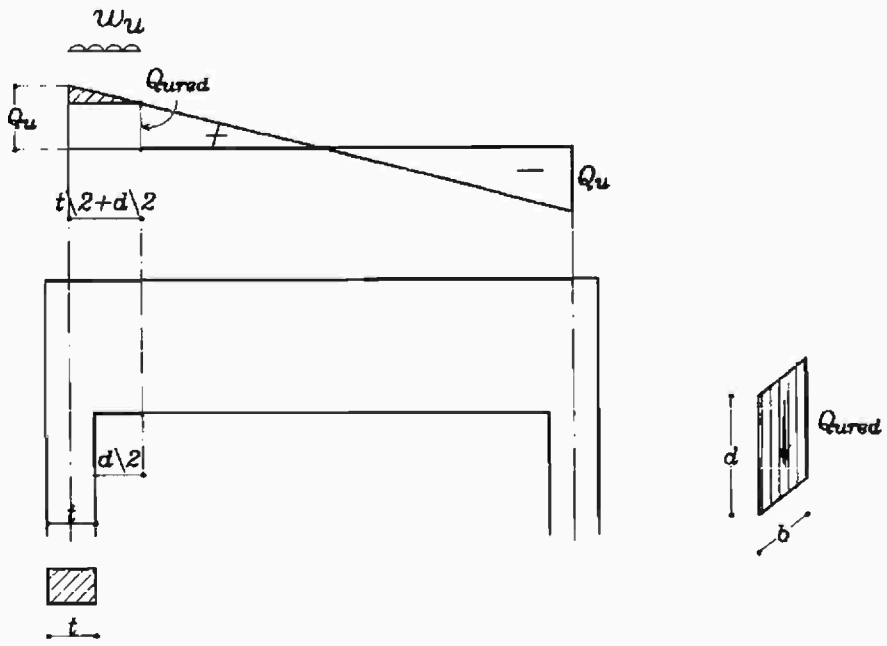
شكل (١٦)


Tie
L
b
b
t
$M_{u\,o.w} = \frac{w_{u\,o.w}\, l^2}{8}$

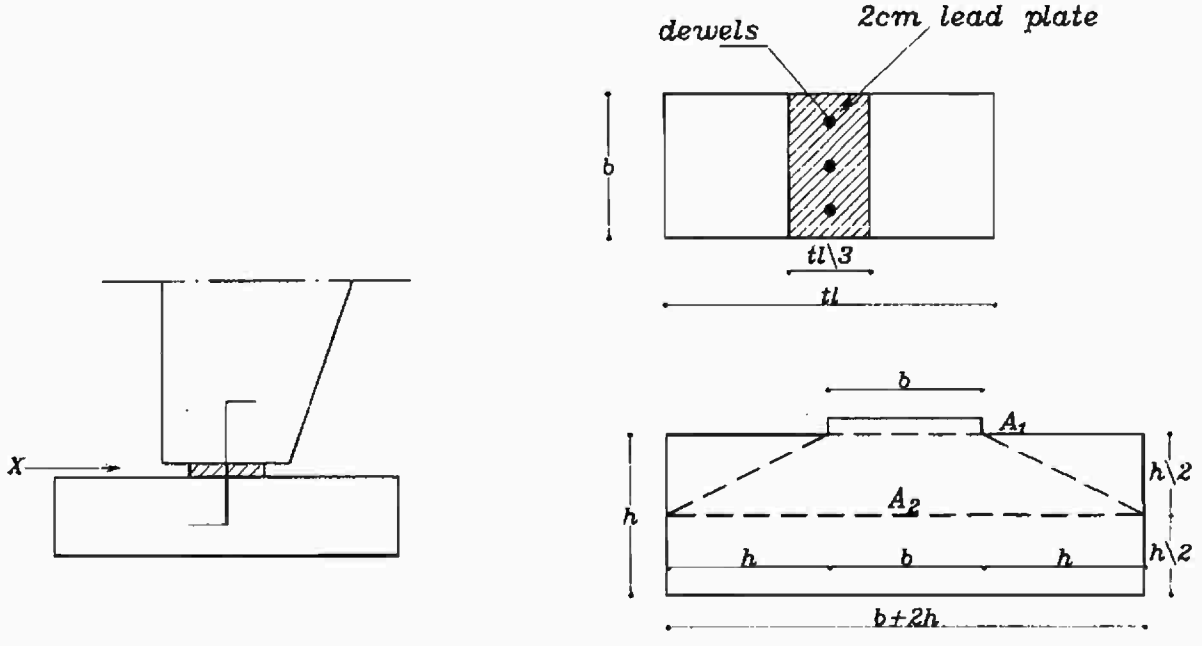
شكل (١٧)


Post
١٠ ϕ ٨
b_{tie}
b_{tie}
Post details
تفاصيل شمعة الشد
L

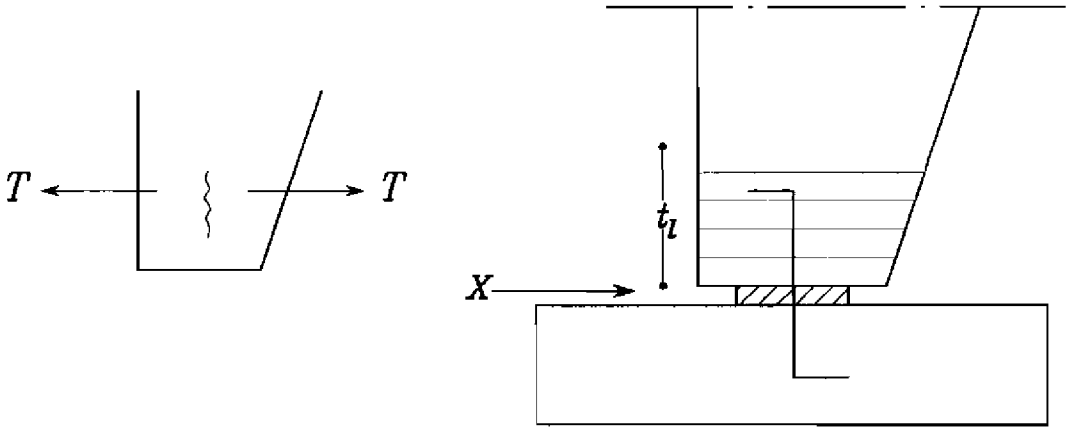
شكل (١٨)


w_u
Q_{ured}
Q_u
+
−
Q_u
$t\2+d\2$
$d\2$
t
t
Q_{ured}
d
b

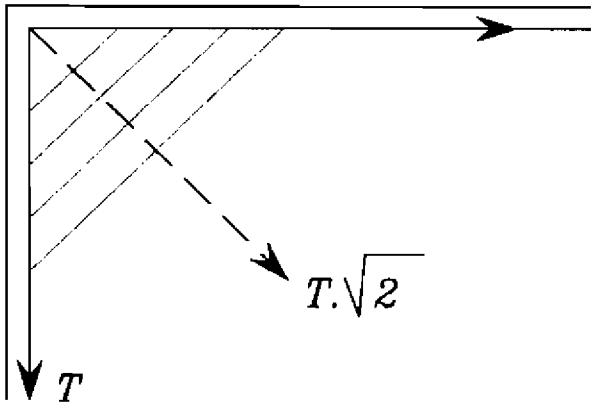
شکل (۱۹)


dewels
2cm lead plate
b
$tl\3$
tl
X
b
A_1
A_2
h
h
b
h
$h\2$
$h\2$
b+2h

شکل (۲۰)



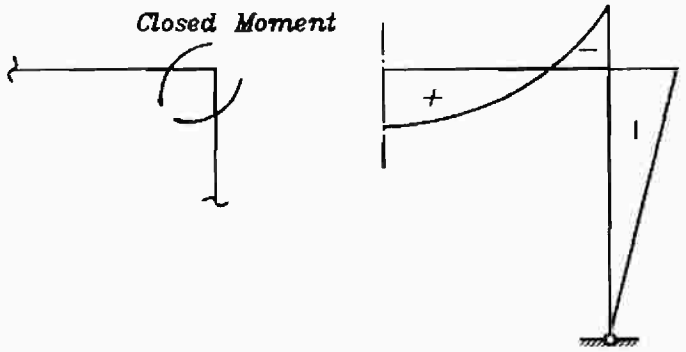
شکل (۲۱)

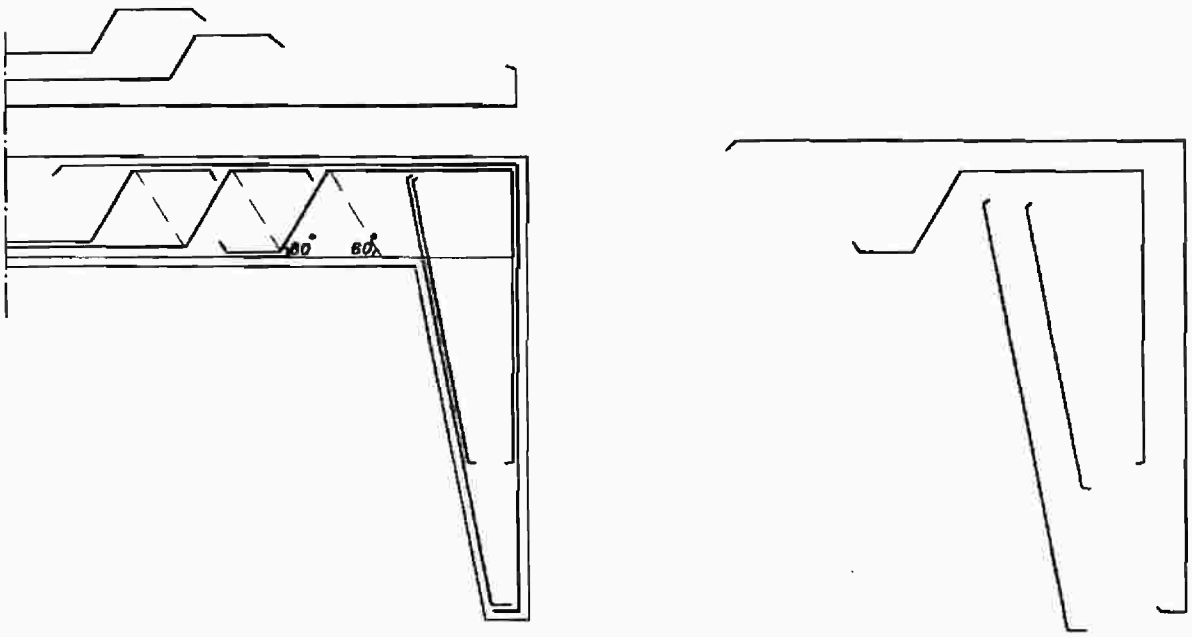


شکل (۲۲)

٥ - ١٢ تفاصيل تسليح بعض الوصلات المشهورة :-

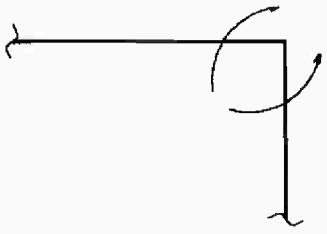
1- Connection (1) وصلة ١

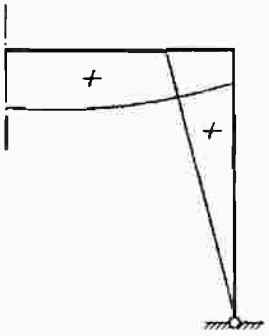


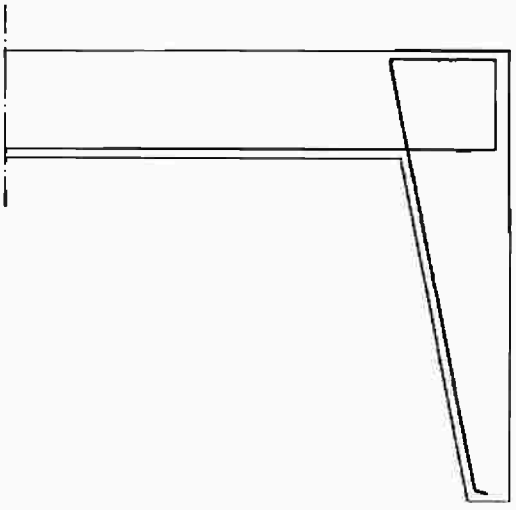


شكل (٢٣)

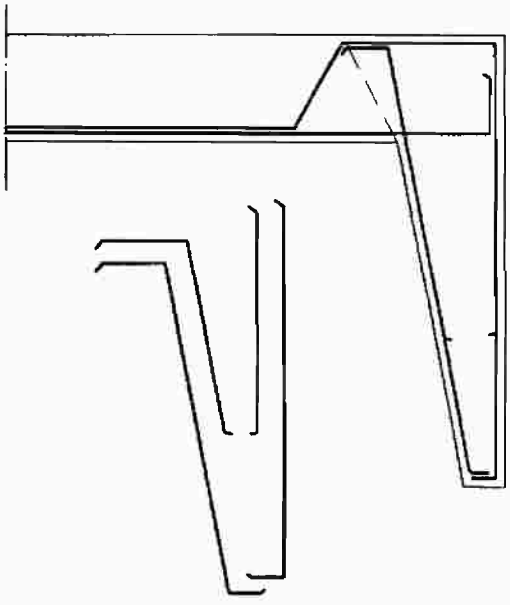
Opening Moment

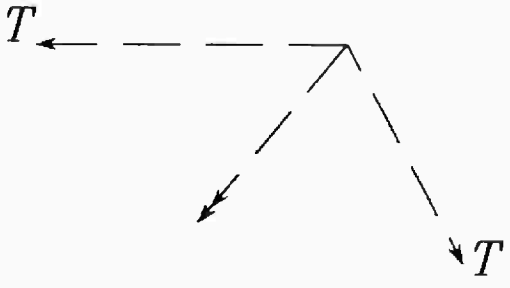






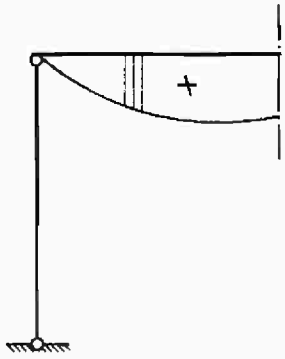
Or

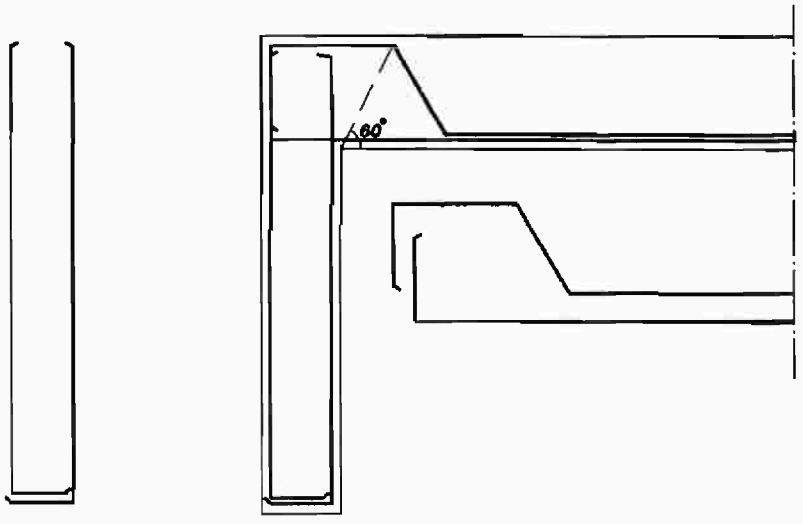




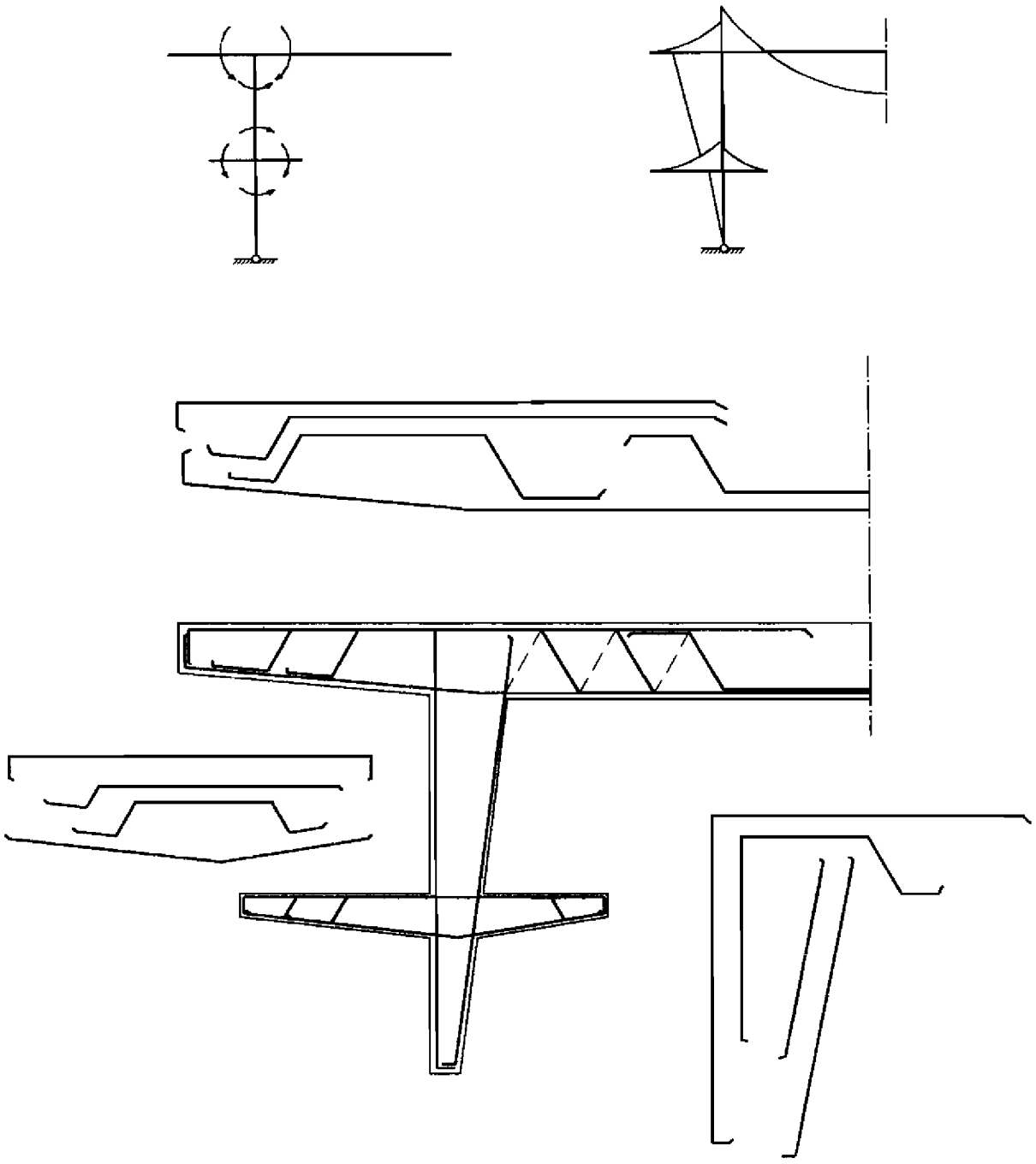
محصلة الشد يمكن أن تسبب انهيار الغطاء الخرسانى .

شكل (٢٤)



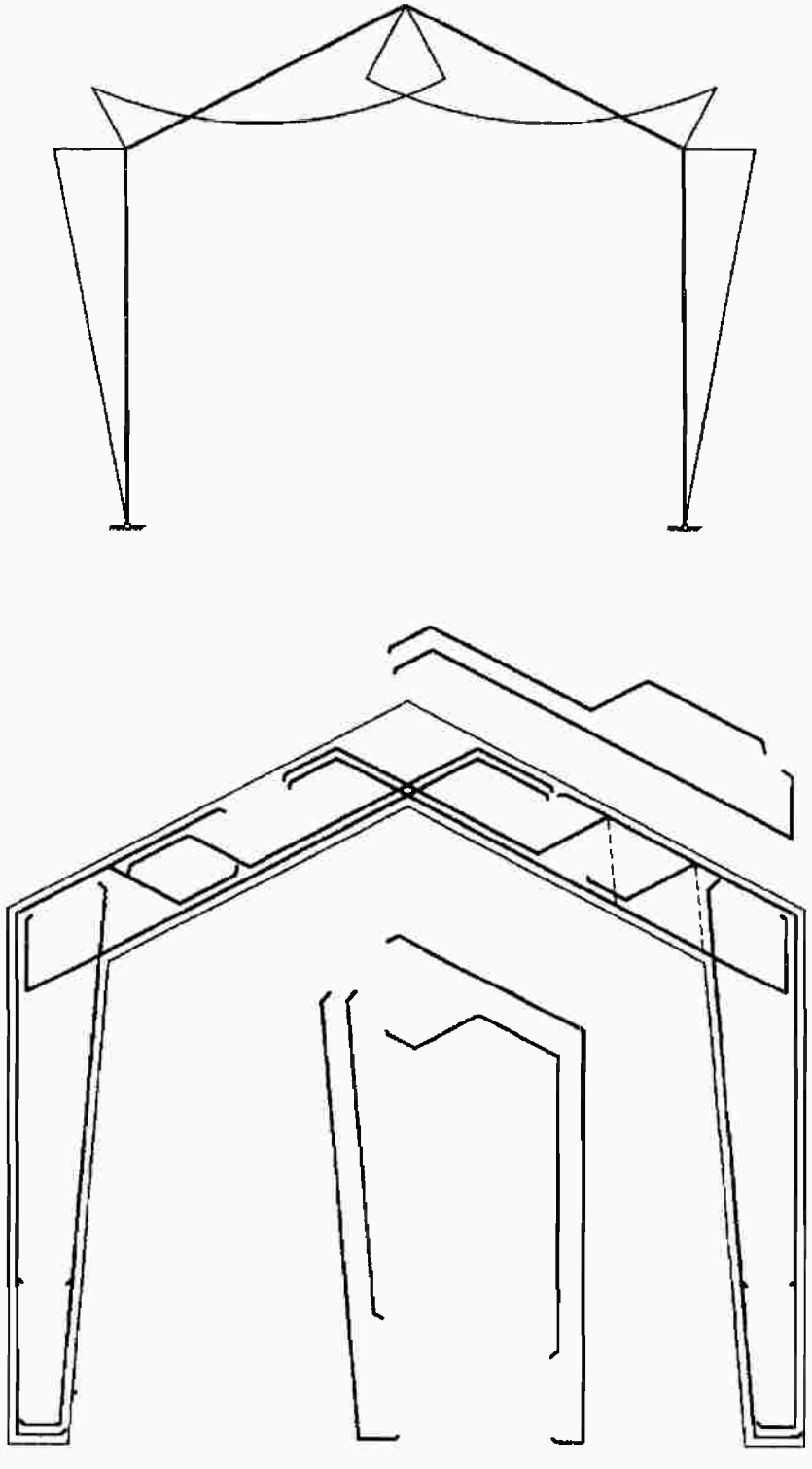


شكل (٢٥)

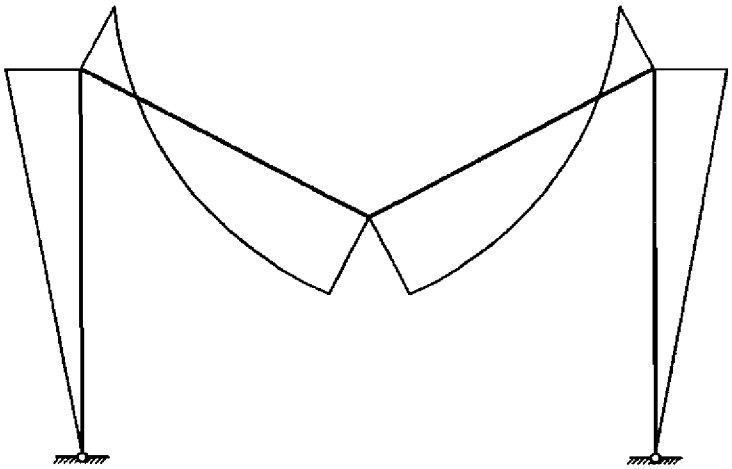


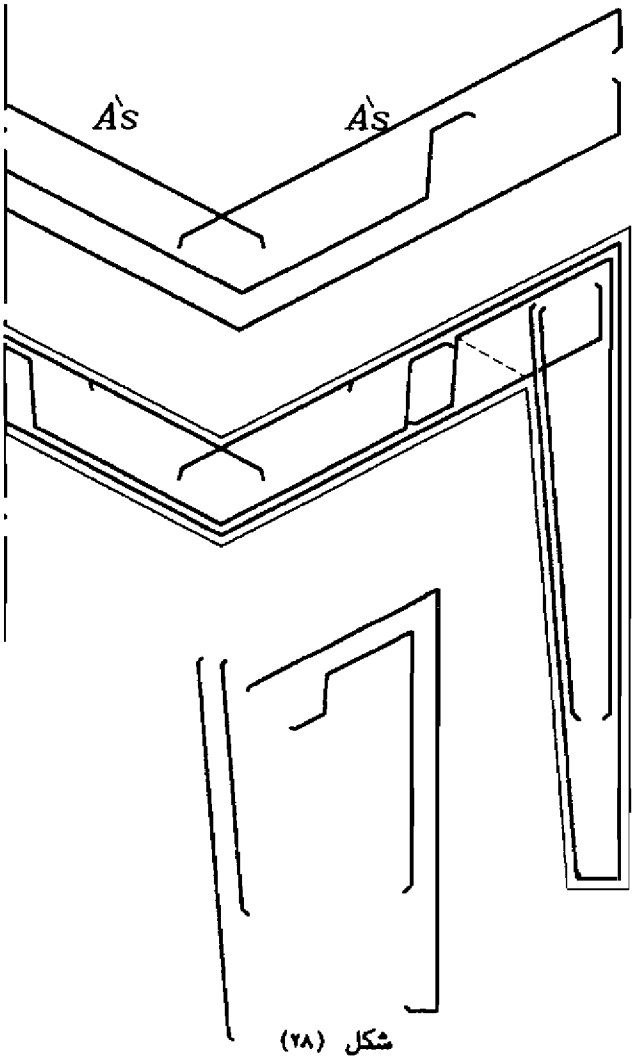
شكل (٣٦)

5- Connection 5 :- وصلة ٥



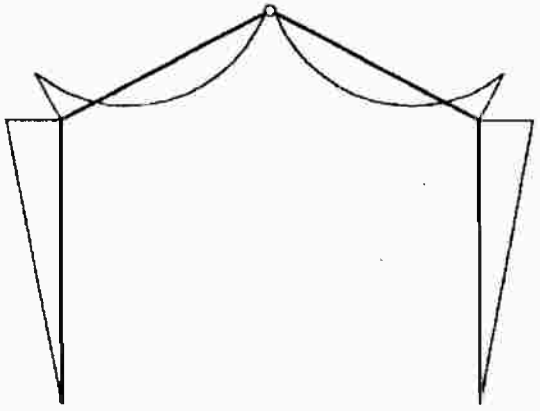
شكل (٣٧)

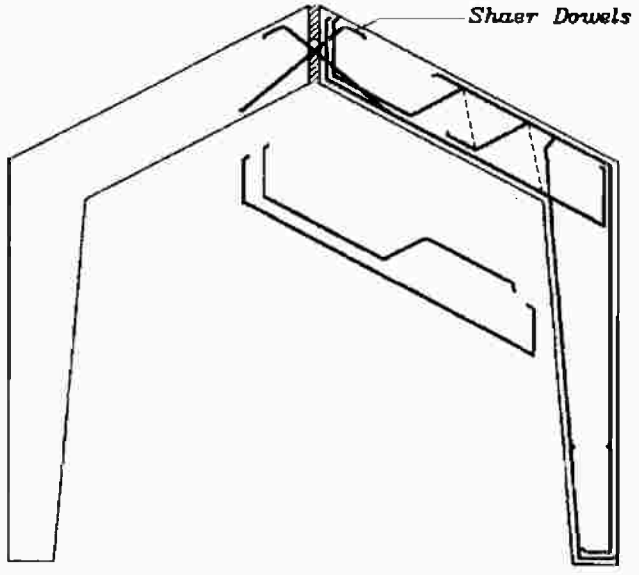


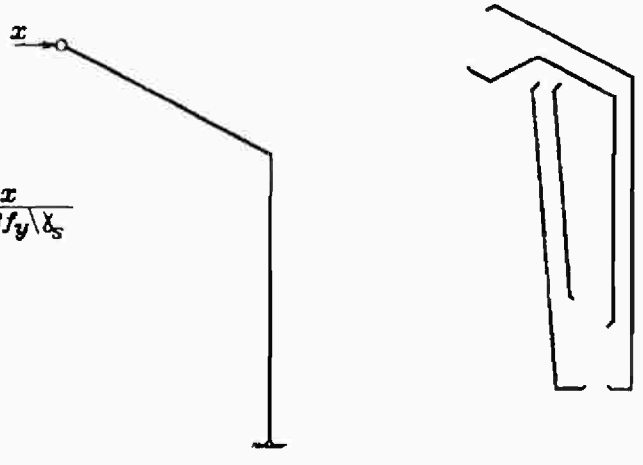


شكل (٢٨)

7- Connection 7 :- Intermediate hinge Connection وصلة ٧





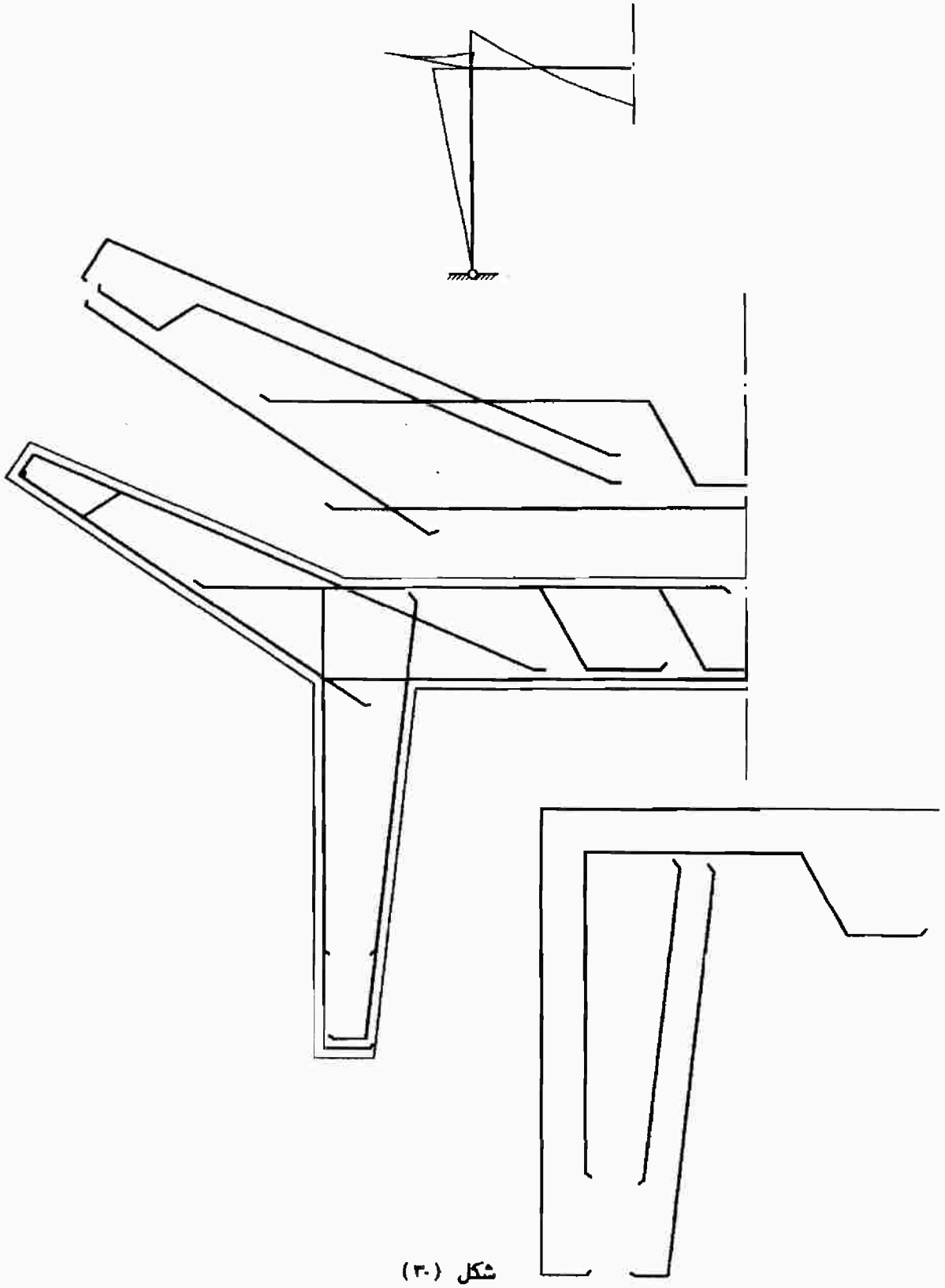


$$A_{dowels} = \frac{x}{0.8 f_y \backslash \gamma_s}$$

شكل (٣٩)

8- Connection 8 :-

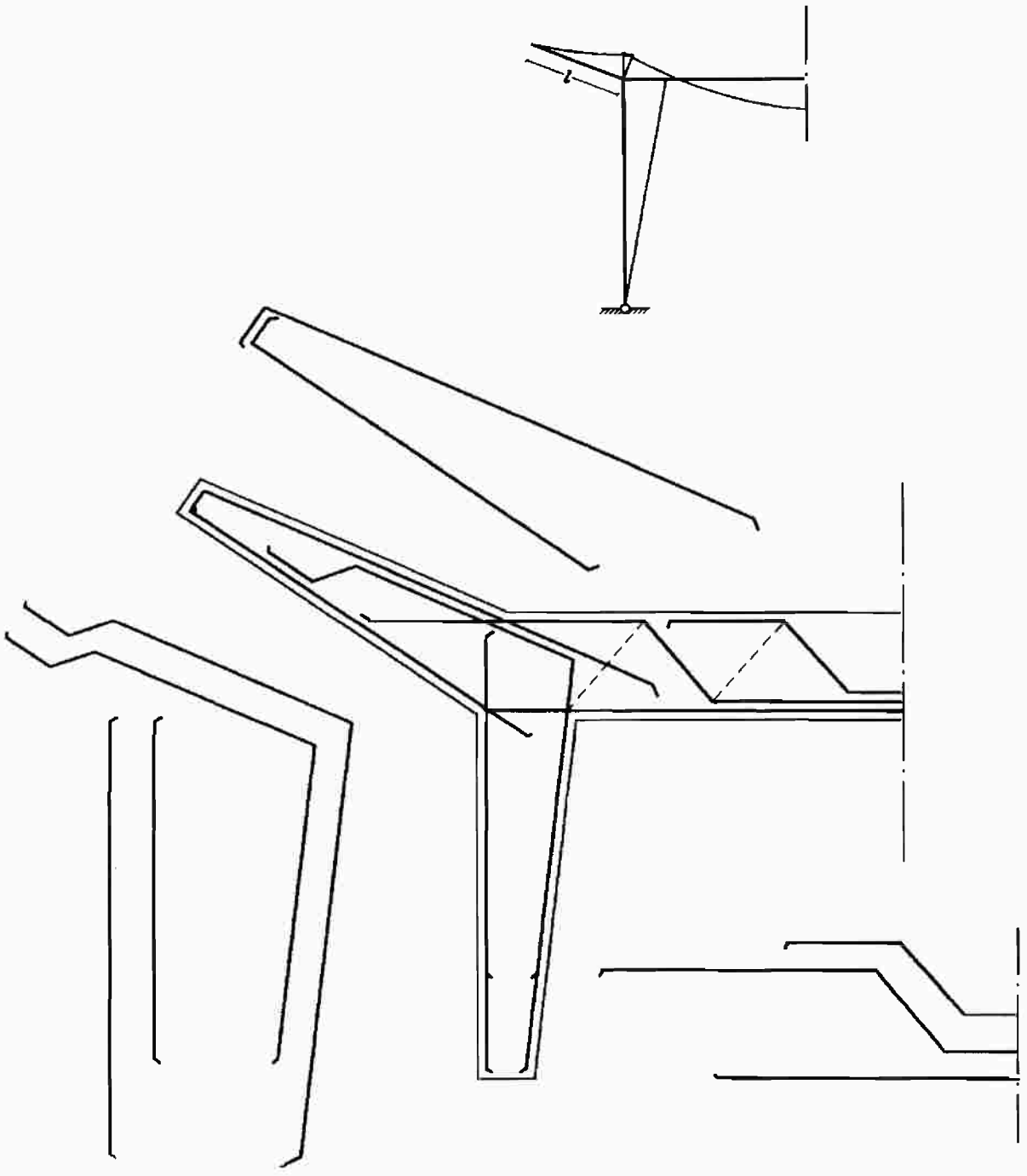
وصلة ٨



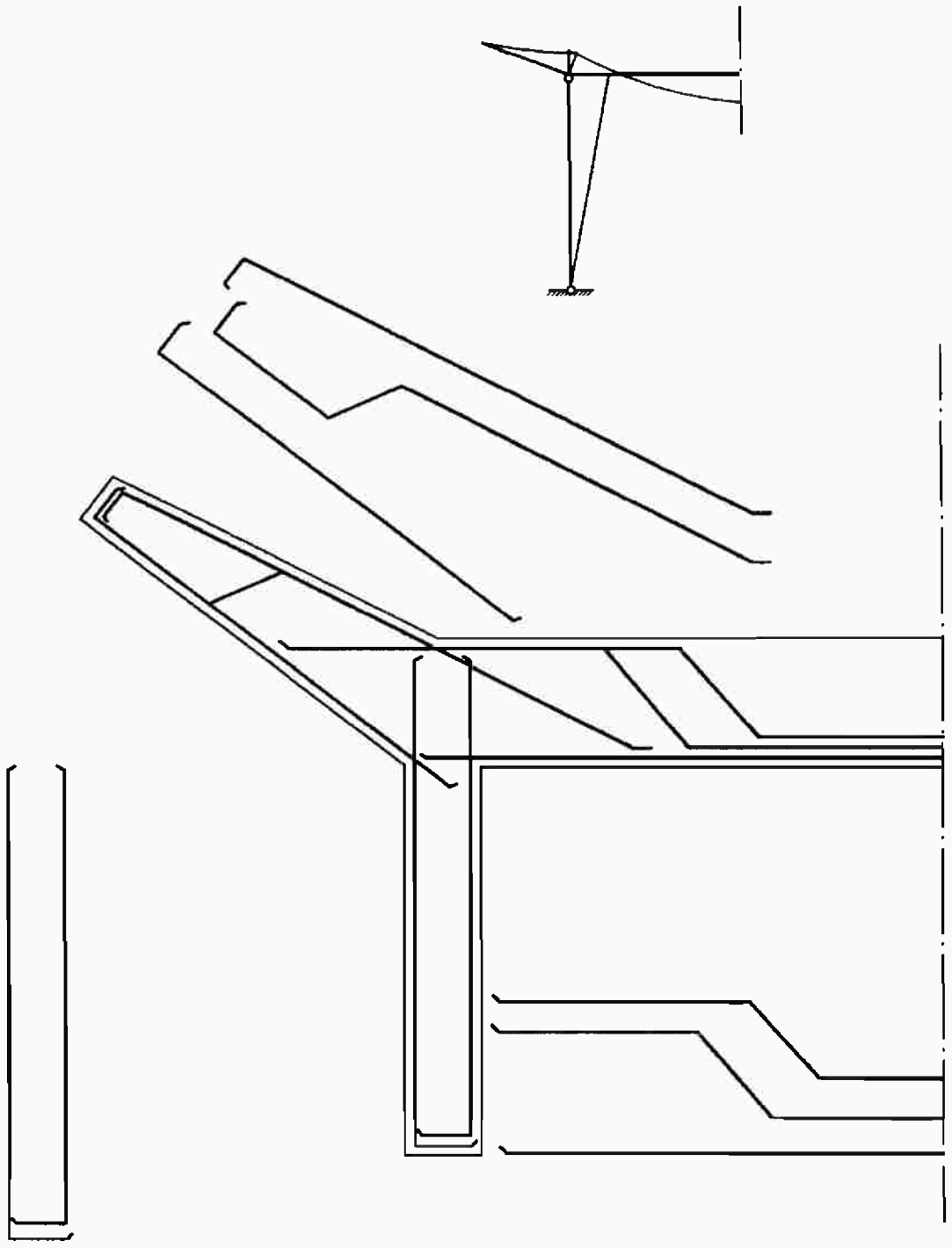
شكل (٣٠)

9- Connection 9 :-

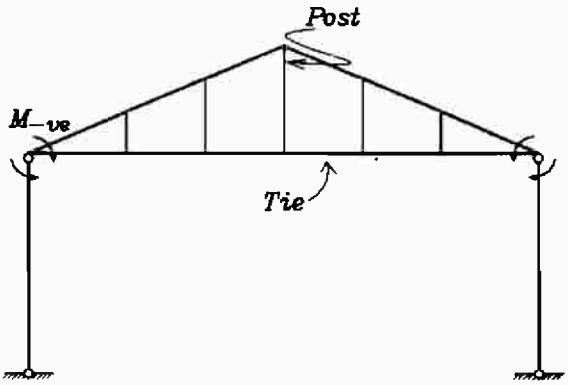
وصلة ٩

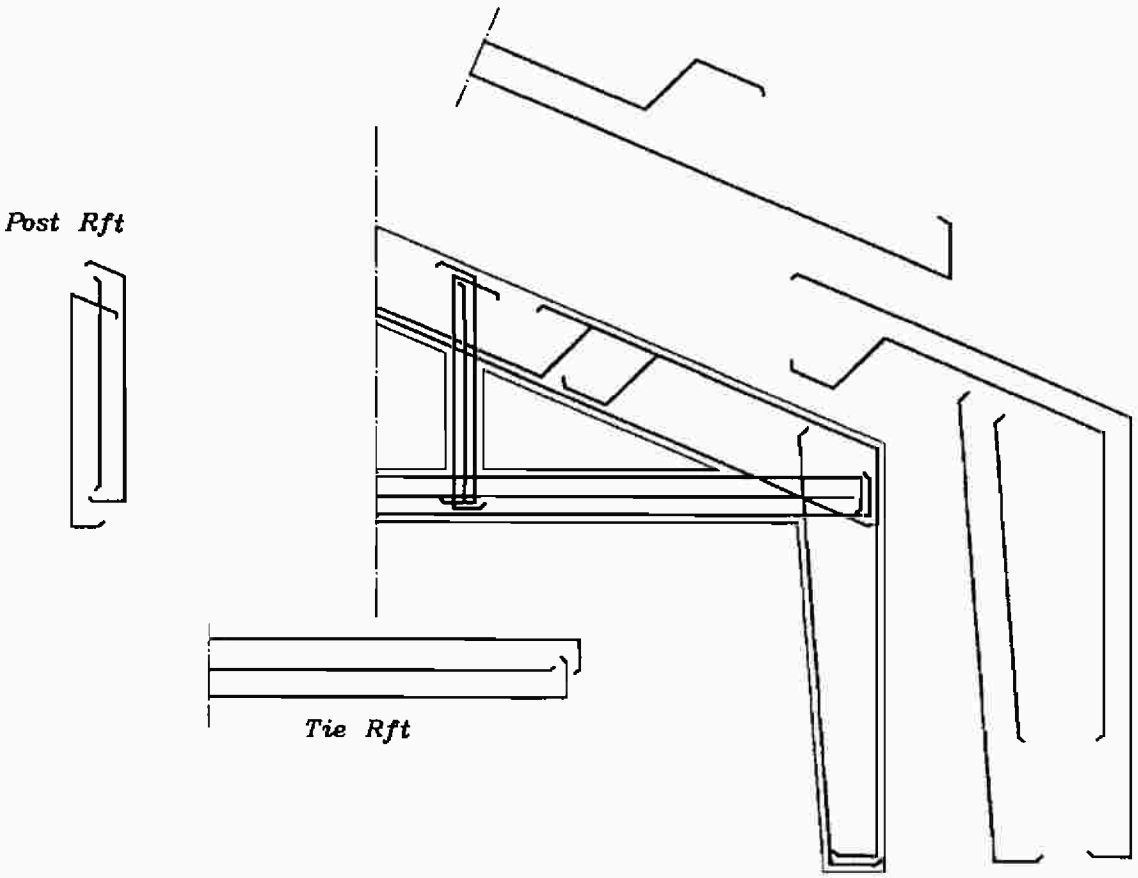


شكل (٣١)



شكل (٣٢)





شكل (٣٣)

٥ - ١٣ : امتحانـــات وأسئلـــة محلولـــه :

السؤال الأول : (٨ %) :

صمم عموداً مستطيلاً مقيداً (braced) مثبتاً عند كلا طرفيه بياناته كالتالي ، ثم ارسم تفـــاصيل القطاع :

$P_{D.L}$=100 t $P_{L.L}$ = 63 t

f_{cu} = 250 kg/cm^2 f_y = 2400kg/cm^2

H_o = 4.0 m .

السؤال الثاني (١٢ %) :

صمم عموداً دائرياً قصيراً ذا كانة حلزونية بياناته كالتالي ، ثم أرسم تفاصيل القطاع :

$P_{D.L}$=400 ton . $P_{L.L}$ = 400 ton .

f_y = 3600 kg/cm^2 f_{cu} = 250 kg/cm^2

f_{yp} = 2400 kg/cm^2

السؤال الثالث (١٠ %) :

أحسب العزم التصميمي للعمود غير المقيد (Unbraced) ذي البيانات الآتية ، علماً بأن طرفه العلوي مثبت Fixed وطرفه السفلي مفصلي hinged :

M_u = 30 m.t H_o = 4.0 m

N_u = 150 t .

السؤال الرابع (١٠ %) :

صمم قطاعاً خرسانياً معرضاً للأحمال الآتية :

Nu = + 20t . (tension force) .

M_u = 50 m.t .

علماً بأن بيانات القطاع كما يلي :

b = 30 cm .

f_{cu} = 250 kg/cm^2 .

f_y = 3600 kg/cm^2 .

السؤال الخامس (٤٠ %) :

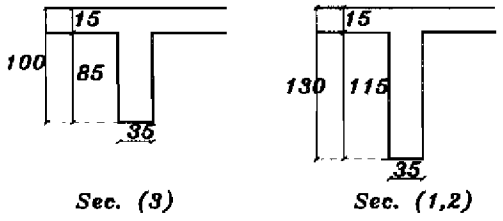
الإطار (Frame) الموضح معرض لقوي عمودية وعزوم انحناء :

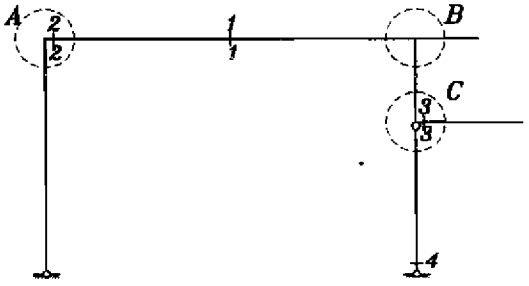
Normal force & Bending Moments.

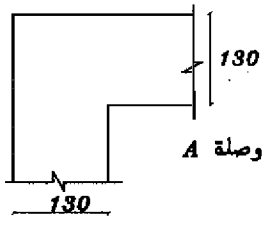
طبقا لما هو معطي علي الرسم والمطلوب :

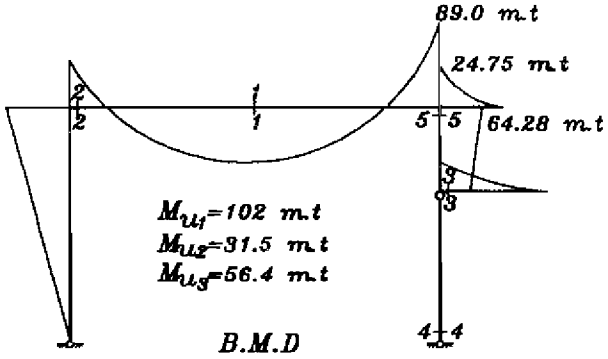
١ – تصميم القطاعات أرقام (1 , 2 , 3 , 4 , 5) فقط .

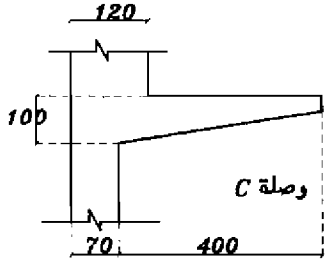
٢ – التوضيح بمقياس رسم مناسب تفاصيل الوصلات (Joints) A , B , C .

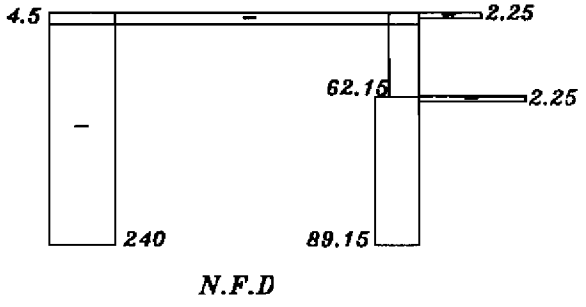




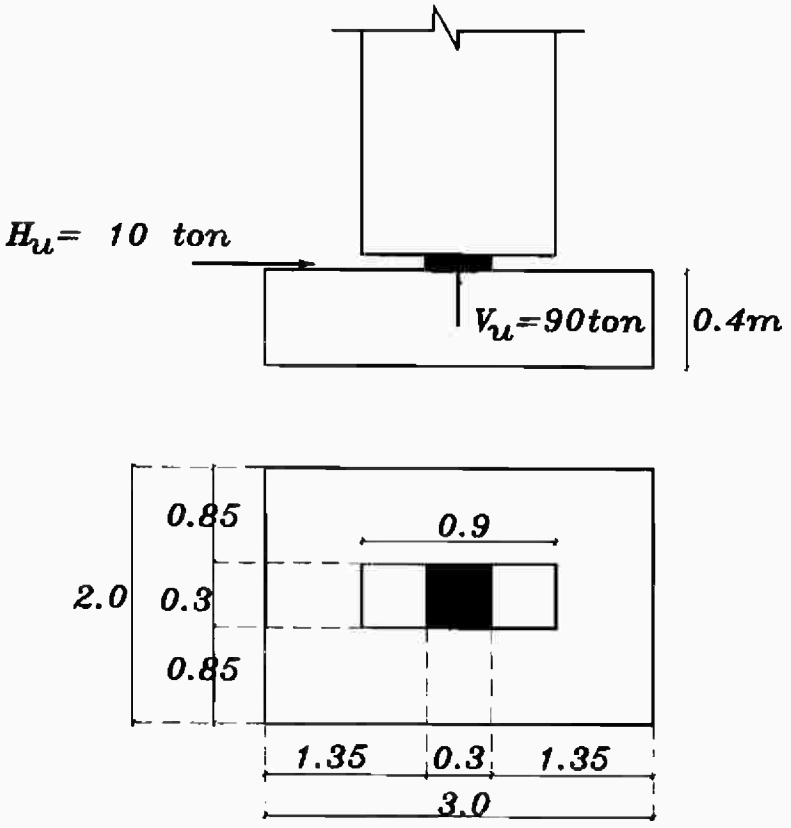








السؤال السادس (٢٠ %) :



- الشكل الموضح لوح رصاص Lead Plate يمثل الركيزة المفصلية لرجل إطار Frame :

١ - تحقق من قيمة إجهادات التحميل Bearing Stresses .

٢ - أحسب مساحة وعدد الأشاير المطلوبة dowels .

٣ - أحسب مساحة وعدد الكانات الأفقية اللازمة عند أسفل رجل الإطار .

٤ - أرسم تفصيلة لما سبق .

(حـــل الامتحـــان)

السؤال الأول :

$P_u = 1.4$ d.L. $+ 1.6$ L.L

$= 1.4 * 100 + 1.6 * 63.$

$= 240.8$ ton.

$\because$ Braced

$\because$ Fixation

$\therefore K = 0.75$

$H_e = 0.75 * 4.0 = 3.0$ m.

$$\lambda = \frac{H_e}{b} = \frac{3}{0.25} = 12 < 15$$

$\therefore$ *Short.*

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

Assume $A_s = 1\ \%\ A_c$.

$$240.8 * 10^3 = 0.35 * 250 * A_c + 0.67 * 2400 * \frac{A_c}{100} .$$

$= A_c\ (\ 87.5 + 16.08\ ) = 103.58\ A_c$

$\therefore A_c = 2324.77\ cm^2$.

$$\therefore t = \frac{2324.77}{25} = 93\ cm$$

Take $25 * 95$

$$\therefore A_{sc} = \frac{1}{100} \times 25 \times 95 = 23.25 cm^2$$

Choose: $12\ \phi\ 16$

$A_{\text{schooen}} = 24.12\ cm^2.$

Check of $A_{s\,min}$

$$As_{min} = \frac{0.8}{100} * 25 * 93 = 18.6\ cm^2$$

$$or = \frac{0.6}{100} * 25 * 95 = 14.25\ cm^2$$

وكليهما أقل من المساحة المختارة :

$A_{s\,chooen} = 24.12 > 18.6$ (O.K)

Choose: 5 $\phi 8/m^-$ as stirrups.

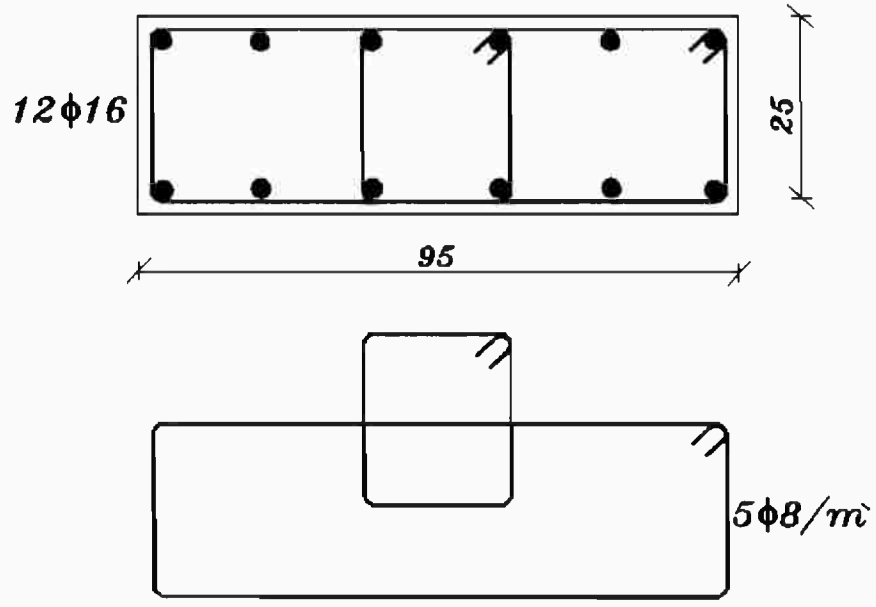
Check of volume of stirrups

V_{st} = area * length = 0.503 * (4 * 20 + 2 * 90 + 2 * 15) * 5 = 729.35 cm^3 .

Check:

$$V_{min} = \frac{0.25}{100} * 25 * 95 * 100$$

$$= 593.75 cm^3 < V_{st} \rightarrow O.K$$



السـؤال الثانـــي :

$P_u = 1.4\ d.L + 1.6\ L.L$

$= 1.4 * 400 + 1.6 * 400$

$= 1200\ t$.

$\because$ *Short*

$\therefore$ No add moment.

$P_u = 0.4\ f_{cu}\ .\ A_c + 0.76\ A_{sc}\ f_y \rightarrow (I)$

$= 0.35\ f_{cu}\ .\ A_k + 0.67\ A_{sc}\ f_y + 1.38\ V_{sp}\ f_{yp} \rightarrow (II)$

بالتطبيق في المعادلة (II) وفرض :

$$V_{sp} = 1.0\ \%\ A_k\ .$$

$$A_{sc} = 1.2\ \%\ A_k\ .$$

$$\therefore 1200 * 10^3 = A_k\ (\ 0.35 * 250 + 0.67 * \frac{1.2}{100} * 3600 + 1.38 * \frac{1}{100} * 2400$$

$$\therefore A_k = \frac{1200 \times 10^3}{149.56} = 8023.5\ cm^2$$

$$= \frac{\pi}{4} D_k^2$$

$\therefore D_k = 101.09$ cm

$= 105$ cm.

$$\therefore A_k = \frac{\pi(105)^2}{4} = 8659\ cm^2$$

$$\therefore As_c = \frac{1.2}{100} * 8659 = 103.9\ cm^2 = 22\phi\!\!\!\phi 25$$

$$\because D = 110cm$$

$$A_c = \frac{\pi D^2}{4} = 9503.3\ cm^2.$$

هنا يتم التأكد من أن المعادلة (I) مساوية :

$P_u = 0.4\ f_{cu}\ .\ A_c + 0.76\ A_{sc}\ .\ f_y$

$= 0.4 * 250 * 9498.5 + 0.76 * 103.86 * 3600 = 1234.6\ t > P_{applied}$ (O.K) .

Check of V_{sp} :

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

$$= 0.36 \frac{250}{2400} \left(\frac{9503.3}{8659} - 1 \right) = 0.00366$$

$$V_{sp.min} = \mu_{sp.min} * A_k$$

$$= 0.00345 * 8659 = 31.66$$

$$But : V_{sp}act = \frac{1}{100} A_k = 86.6 > \min \rightarrow (O.K).$$

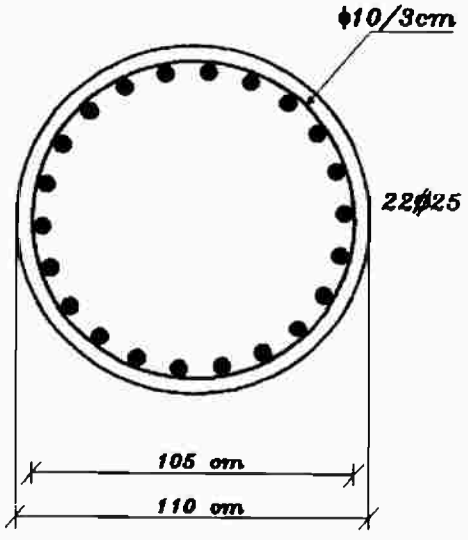
$$V_{sp} = \frac{\pi A_{sp} Dk}{P}$$

$$86.6 = \frac{\pi * 105 * A_{sp}}{P}$$

$use\ \phi 10$

$$\therefore A_{sp} = 0.785cm^2$$

$$P = \frac{3.14 \times 105 \times 0.785}{86.6} = 3\ cm.$$



السـؤال الثالـث :

K = 1.6 (case 1 , case 3) E.C.P.

$H_e = KH_o$

$= 1.6 * 4$

$= 6.4$ m .

b = 25 cm

$\frac{H_e}{b} = \frac{6.4}{0.25} = 25.6 > 23$ وهي قيمة أكبر من المسموح

Increase (b)upto 30 cm .

$\lambda_b = \frac{H_e}{b} = \frac{6.4}{0.3} = 21.33 < 23 \rightarrow (O.K)$

$\because 10 < \lambda_b < 23$ *long col.*

$\therefore M_{add} = P \times \delta_{av}$

$\delta_{av} = \frac{\lambda^2 b}{2000} = \frac{(21.33)^2 * 0.3}{2000} = 0.068m$

$M_{add} = P.\delta_{av}$

$= 150 * 0.068$

$= 10.24$ *t.m*

$\therefore M_{design} = M + M_{add}$

$= 30 + 10.24$

$= 40.24$ t.m

(or) : $M_{desy} = P.\ e_{min} = 150 * (0.05\ t \text{ or } 2)$.

} أيهما أكبر

∴

P =	150 t
M =	40.24

السـؤال الرابـــع :

$$d_o = C_1\sqrt{\frac{M}{f_{cu}b}}$$

$$= 3\sqrt{\frac{50\times10^5}{250\times30}} = 77cm.$$

d = 0.9 do = 0.9 * 77 = 70 cm

$\therefore t = 75\ cm$

$$e = \frac{M_u}{P_u} = \frac{50}{20} = 2.5m.$$

$$\frac{e}{t} = \frac{2.5}{0.75} = 3.33 > 1/2\ \ (Big.ec)$$

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

$$= 2.5 - \frac{0.75}{2} + 0.05 = 2.175\ m$$

$M_u = N_u.es = 20 * 2.175 = 43.5$ m.t.

$$R = \frac{M_{us}}{f_{cu}b\times d^2} = \frac{43.5*10^5}{250\times30\times(70)^2} = 0.118$$

$\therefore w = 0.15$

$\alpha = 0.3$

$$A_s = wbd\ f_{cu}/f_y + p_u/(f_y/\gamma_s)$$

$$= 0.15(30)\ (70)\ (250/3600) + 20*10^3/(3600/1.15)$$

$$= 28.26\ cm^2 \qquad (14\phi 16)$$

$$As^{-} = 0.3\ (0.15*30*70\frac{250}{3600}) = 6.56cm^2 = 4\phi 16$$

السـؤال الخامـس :

تصميم القطاعات :

given:

f_{cu}= 180 kg/cm^2 .

f_y = 2400 kg/cm^2 .

sec . 1 :

t – Section.

M_u = 102 t.m.

N_u = - 4.5 ton.

من الواضح صغر قيمة N_u ويمكن التأكد من ذلك بحساب 0.04 f_{cu} . b.t .

$0.04\ f_{cu}\ .\ b.t = 0.04 * 180 * 35 * 130$

$= 32760\ kg = 32.76$

$\because N_u < 0.04 f_{cu} bt$

$\therefore neglect(N_u)$

$B = b_o + 16 * t_s$

$= 35 + 16 * 15 = 275\ cm$.

$d = 130 - 5\ cm = 125\ cm$

$$\because d = C_1 \sqrt{\frac{M_u}{f_{cu} B}}$$

$$\therefore C_1 = \frac{125}{\sqrt{\frac{102 \times 10^5}{180 \times 275}}} = 8.708$$

$$\therefore \frac{c}{d} = \frac{8.708}{125} = 0.069 < 0.125 < \left(\frac{c}{d}\right)_{min}$$

$$\therefore Take : \frac{c}{d} = \min = 0.125$$

$$\therefore J = 0.825$$

$a = 0.8\,C$.

$\because \frac{C}{d} = 0.125$

$\therefore C = 0.125 * 125 = 15.625$

$\because a = 0.8C$

$= 0.8\,(15.625)$

$= 12.5 < t_s$. (O.K)

This section is designed as $\square^{ler}$ sec

$A_s = \frac{M_u}{J.d.fy}$

$= \frac{102 * 10^5}{0.825 * 125 * 2400}$

$= 41.21 cm^2 \rightarrow 12\phi 22.$

Sec.2:

$M_u = 31.5$ m.t

$N_u = -4.5$ t .

$0.04 f_{cu} bt = 32.76$

$\because 4.5 << 32.76$

$\therefore neglect(Nu)$

$d = C_1 \sqrt{\frac{M_u}{f_{cu}.B}}$

$C_1 = 6$

$C/d = 0.044 < (c/d)_m$ $\therefore take\ c/d = \min = 0.125$

$J = 0.825$

$A_s = \frac{31.5 * 10^5}{0.825 * 125 * 2400} = 12.72\ cm^2$

$\mu = 12.72/35*125 = 0.003$

Check:

$$\mu_{min} = \frac{11}{f_y} = 0.00458$$

$$Smaller\ y : 1.3\mu = 1.3\frac{As_{req}}{b.d} = 0.00378$$

$$\& \nmid \frac{0.25}{100} A_c \rightarrow 24/35$$

$$\therefore A_s = 0.00378\ (35)\ (125) = 16.545\ cm^2 = 3\phi 19 + 3\phi 22$$

Sec. 3:

Rectangle section:

t = 35 cm .

M_u = 56.4 m.t

N_u = - 2.25 t → Small (neglected) .

$$\therefore R = \frac{M}{f_{cu} b d^2} = \frac{56.4*10^5}{180*35*95^2} = 0.099$$

$$\therefore w = 0.13$$

$$A_s = w.b.d\frac{f_{cu}}{f_y} = 0.13*35*95*\frac{180}{2400}$$

$$= 32.42 cm^2 \rightarrow 9\phi 22$$

Sec .4 :

N_u = - 89.15 t

Design as a column 35 * 70

H = 350 cm

Assume braced

H_e = 1 * 350 = 350

$$\lambda = \frac{H_e}{b} = \frac{350}{35} = 10 < 15 \rightarrow \text{No additional moment}$$

assume $\mu = 0.8\% = \frac{A_s}{A_c} = A_s = \frac{0.8}{100} A_c$

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

$$89.1 * 10^3 = (0.35 * 180 + 0.67 * \frac{0.8}{100} * 2400)\ A_c$$

$$\therefore A_c = 1174.47\ cm^2$$

$$A_s = \frac{0.8}{100} * 1174.47 = 9.4\ cm^2$$

Check:

$$A_s < \frac{0.6}{100} * 35 * 70 = 14.7\ cm^2$$

$\therefore$ *use* $A_s = 14.7 cm^2$ 6ϕ19

Answer Q 5:

1 - Bearing check:

$A_1 = b.\ t_e / 3 = 30(90/3) = 900\ cm^2$

$f = V_u / A_1 = 90 * 10^3 / 900 = 100\ kg/cm^2$

$f_b = 0.67\ f_{cu} / \gamma_c = 111.67\ kg/cm^2$

$A_2 = (2h + t_e / 3)\ (2h + b)$

$= (2 * 40 + 90/3)\ (2 * 40 + 30) = 12100\ cm^2$

$0.67\ f_{cu} / \gamma_c\ \sqrt{A_2 / A_1} = 405.4\ kg/cm^2$

$\because f_b = 111.67 \not> 0.67\ f_{cu} / \gamma_c \sqrt{A_2 / A_1}$ $(O.K)$

$\because f < f_b$ Safe Dimension

2 – Area of dowles:

$A_{s\ dowels} = X / (0.8\ f_y / \gamma_s)$

$= 10 * 10^3 / (0.8 * 3600 / 1.15)$

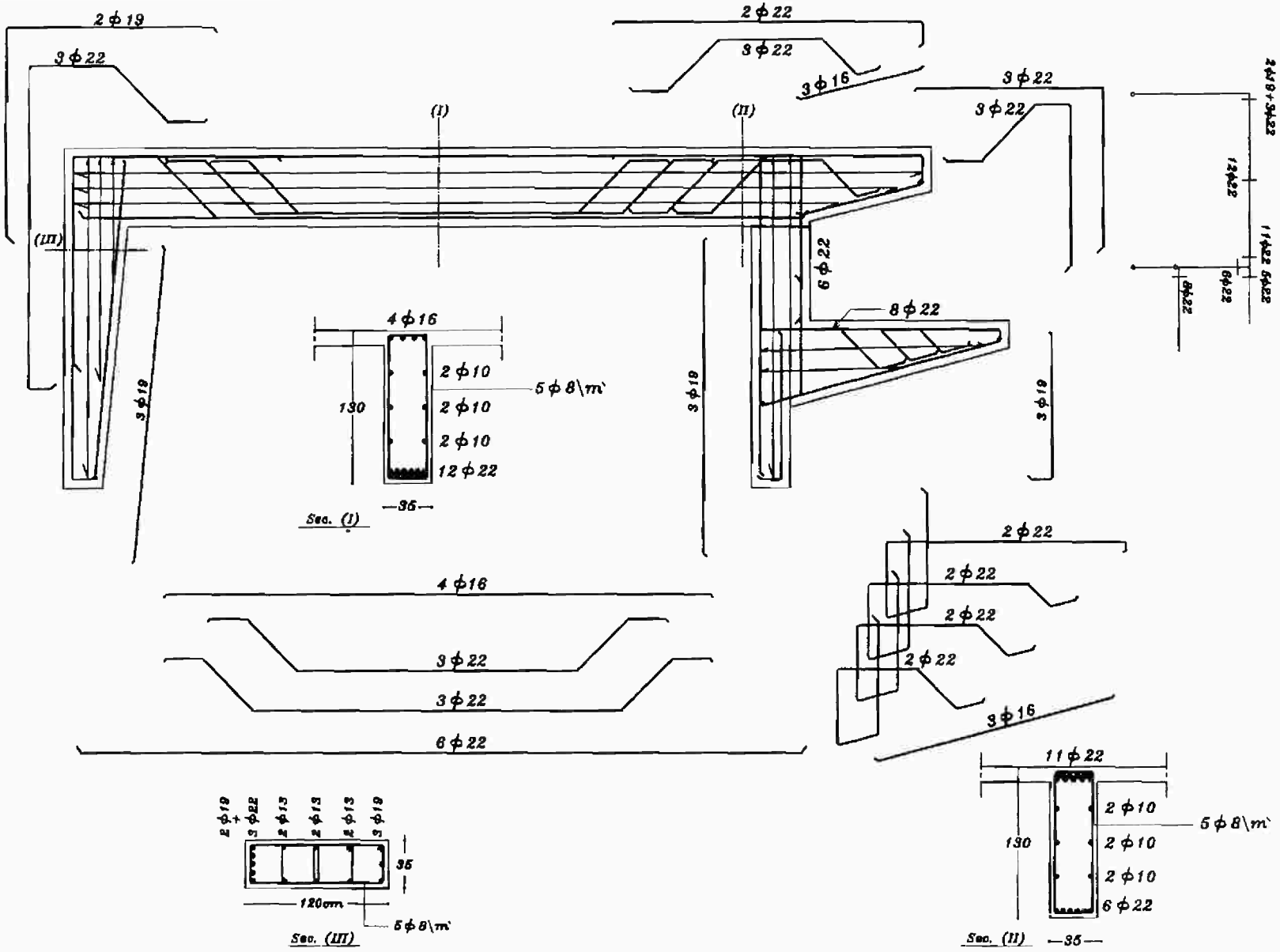
$= 4\ cm^2$ use $3\phi\phi 16)$ $(6\ cm^2)$

3 – Horizontal Stirrups : it will be distributed to height

$t = V/5 = 18\ t$

$A_{sh} = t / (f_y / \gamma_s) = 1.8 * 10^3 / (2400 / 1.15)$

$= 8.625\ cm^2 = 7\phi 13 / t_e$


2 ϕ 19
3 ϕ 22
2 ϕ 22
3 ϕ 22
3 ϕ 16
3 ϕ 22
3 ϕ 22
(I)
(II)
(III)
6 ϕ 22
8 ϕ 22
3 ϕ 19
3 ϕ 19
3 ϕ 19
4 ϕ 16
2 ϕ 10
5 ϕ 8\m`
130
2 ϕ 10
2 ϕ 10
12 ϕ 22
35
Sec. (I)
4 ϕ 16
3 ϕ 22
3 ϕ 22
6 ϕ 22
2 ϕ 22
2 ϕ 22
2 ϕ 22
2 ϕ 22
3 ϕ 16
11 ϕ 22
2 ϕ 10
5 ϕ 8\m`
130
2 ϕ 10
2 ϕ 10
6 ϕ 22
Sec. (II)
35
2 ϕ 19
+
3 ϕ 22
2 ϕ 13
2 ϕ 13
2 ϕ 13
3 ϕ 19
35
120cm
5 ϕ 8\m`
Sec. (III)

Example (1) :

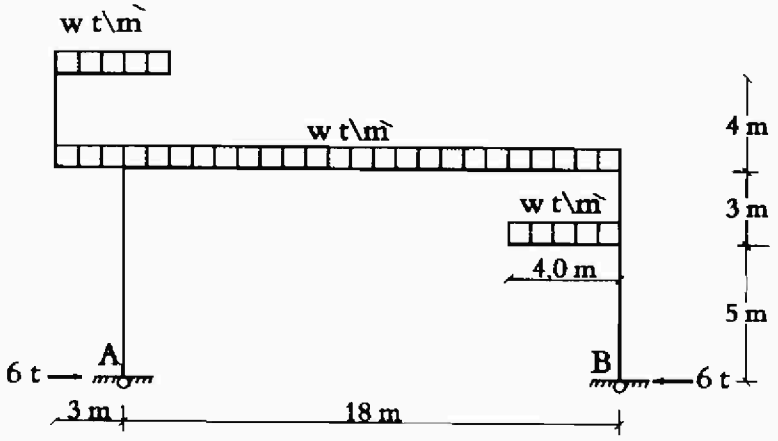
Draw straining action diagrams, and design the critical section, then give full details for the following frames:

Frame (1) :

Data:

t_s = 12 cm
b = 35 cm
$w_{D.L}$ = 3 t/m$^-$
f_y = 3600 kg/cm^2 .
$w_{L.L}$ = 1 t/m$^-$
o.w = 1 t/m$^-$
f_{cu} = 250 kg/cm^2

The frame may be considered braced in each direction:



Solution:

$$t_g = \frac{1800}{12 \rightarrow 16} = 140\ cm$$

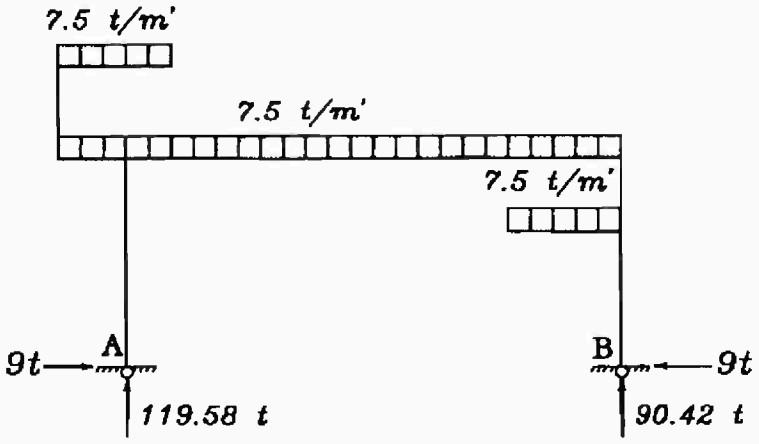
$$t_{c\ (upper)} = 1 \rightarrow 0.8\ tg = 110\ cm$$

$$t_{c\ (lower)} = 80\ cm$$

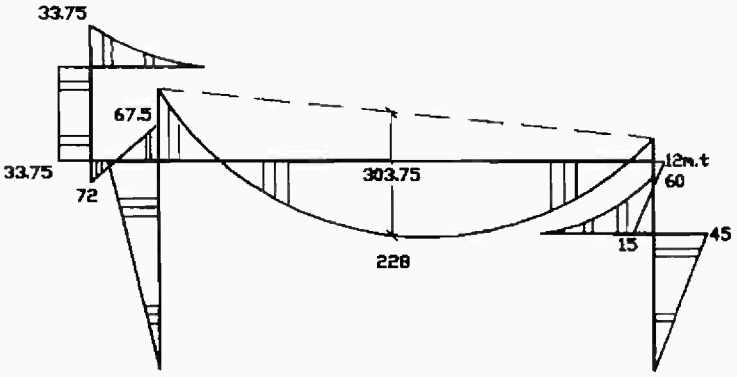
$$t_{cantilver} = 100\ cm$$

Loads:

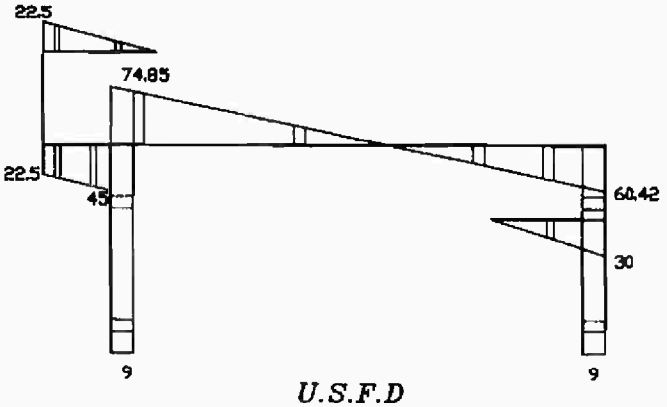
w = 3 + 1 + 1= 5 t/m$^-$.
w_u = 1.5 * 5 = 7.5 t/m$^-$.



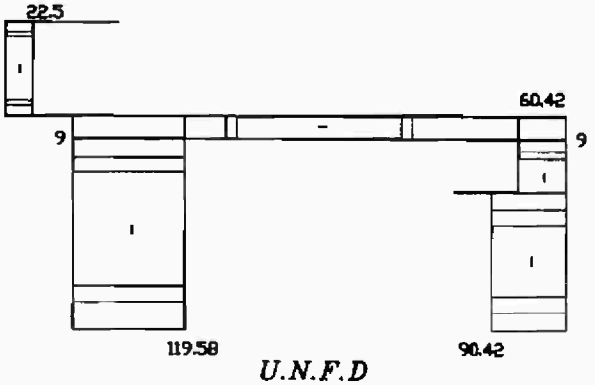
Straining Actions



U.B.M.D



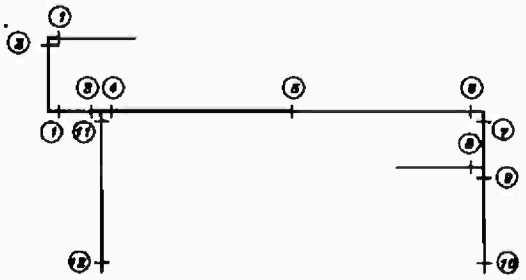
U.S.F.D



U.N.F.D

Design of section:

Sec	M_u	N_u
1	33.75	0
2	33.75	-22.5
3	67.5	0
4	139.5	-9
5	228	-9
6	12	-9
7	12	-60.4
8	60	0
9	45	-90.42
10	0	-90.42
11	72	-119.58
12	0	-119.58



Sec (1) : 35 * 100 cm $\square^{er}$

$M_u = 33.75$ m.t
$N_u = 0$

$$R = \frac{M_u}{f_{cu}\, bd^2} = 0.0427 \rightarrow \omega = 0.053$$

$$A_s = \omega bd \frac{f_{cu}}{f_y} = 12.24\ cm^2$$

$$A_{s\,min} = \frac{11}{f_y} bd = 10.16\ cm^2 < A_s \quad o.k$$

(*use* 4ϕϕ22)

Sec (2) : 35 * 100 cm $\square^{er}$
$M_u = 33.75$ m.t
$N_u = -22.5$ t

$$\frac{N_u}{f_{cu\ bt}} = 0.026 < 0.04 \rightarrow neglect\ \ N_u$$

$A_s = 12.24\ cm^2 \rightarrow A_s\ of$ sec(1)

(*use* 4ϕϕ22)

Sec (3) : 35 * 140 cm $\square^{er}$

$M_u = 67.5$ m.t

$N_u = 0$

$$R = \frac{M_u}{f_{cu}\, bd^2} = 0.0423 \rightarrow w = 0.051$$

$$A_s = wbd\,\frac{f_{cu}}{f_y} = 16.73\ cm^2 \rightarrow use\ (6\phi\!\!\!\phi 22)$$

Sec (4) : 35 * 140 cm $\square^{er}$

$M_u = 139.5$ m.t

$N_u = -9$ t

$$\frac{N_u}{f_{cu} bt} = 0.007 < 0.04 \rightarrow neglect\ N_u$$

$$R = \frac{M_u}{f_{cu}\, bd^2} = 0.087 \rightarrow w = 0.113$$

$$A_s = wbd\,\frac{f_{cu}}{f_y} = 37.1\ cm^2 \rightarrow use\ (6\phi\!\!\!\phi 22)$$

Sec (5) : 35 * 140 cm T – sec

$M_u = 228$ m.t

$N_u = -9$ t $\rightarrow$ *neglected as* sec (4)

B = Smallest of

$16\, t_s + b = 227$ cm

C_L to $C_L = 500$ cm

$$\frac{0.8\,L}{S} + b = 323\ cm$$

$\therefore$ B = 227 cm

$$d = C_1\sqrt{\frac{M_u}{f_{cu}\, B}} \rightarrow C_1 = 6.74 \qquad take\ \frac{C}{d}_{min} = 0.125$$

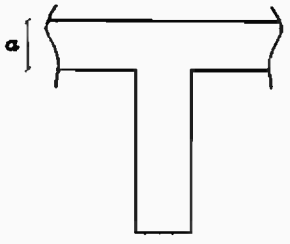
$\therefore C = 16.875\ cm$ $\qquad\qquad$ $\therefore a = 0.8\,C = 13.5\ cm > t_s$

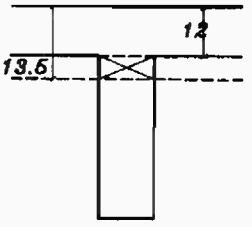
$\therefore$ *take* $a = t_s$

$$M_u = \frac{A_s f_y}{\gamma_s}(d - \frac{a}{2})$$

$\therefore A_s = 56.46\ cm^2$

$\therefore$ *use* $(12\phi\!\!\!\phi 25)$ *or* $(6\phi\!\!\!\phi 25 + 8\phi\!\!\!\phi 22)$





Sec (6) : 35 * 140 cm $\square^{er}$

M_u = 12 m.t

N_u = -9 t → neglected

$$R = \frac{M_u}{f_{cu}\, bd^2} = 0.0075 \rightarrow w = 0.01$$

$$A_s = 3.3\ cm^2$$

$$use\ A_{s\,min} = \frac{11}{f_y} \rightarrow bd = 14.4 cm^2 \quad use\ (4\phi\!\!\!\phi 22)$$

Sec (7) : 35 * 110 cm $\square^{er}$

M_u = 12 m.t

N_u = -60.42 t

$$\frac{N_u}{f_{cu} bt} = 0.063 > 0.04$$

$$e = \frac{M_u}{N_u} = 0.1986\ m \rightarrow \frac{e}{t} = 0.18 > 0.05$$

$$\frac{M_u}{f_{cu} bt^2} = 0.011$$

$Interaction\ Diag \quad \alpha = 0.6 \quad \zeta = 0.9 \qquad f_y = 3600\ kg/cm^2$

$\rho < 1 \rightarrow Tension\ failure$

$$e_s = e + \frac{t}{2} - 0.05 = 0.6986\ m$$

$$M_{us} = N_u .es = 42.42\ mt$$

$$R = \frac{M_{us}}{f_{cu}\, bd^2} \xrightarrow{\alpha=0} w = 0.054$$

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

$$\therefore use\ A_{s\,min} = \frac{11}{f_y} bd = 11.23 cm^2$$

$(use\ 4\phi\!\!\!\phi 22)$

Sec (8) : 35 * 100 cm $\square^{er}$

$M_u = 60$ m.t

$N_u = 0$ t

$$R = \frac{M_u}{f_{cu}\, bd^2} = 0.076 \rightarrow w = 0.097$$

$$A_s = wbd\,\frac{f_{cu}}{f_y} = 22.11\ cm^2 \rightarrow use\ (5\phi\!\!\!

Sec (10) : 35 * 80 cm □er

$M_u = 0$
$N_u = -90.42\ t$ (A_s short col.)
$N_u = P_u = 0.35\ f_{cu}\ A_c + 0.67\ f_y\ A_{sc}$

$90.42 * 1000 = 0.35 * 250 * 35 * 80 + 0.67 * 3600\ A_{sc}$

$A_{sc} = -\ V_e$

$\therefore use\ A_{s\,min} = 0.6\%\ A_c = 16.8\ cm^2$

Sec (11) : 35 * 110 cm □er

$M_u = 72\ t$
$N_u = -119.58\ t$

$$\frac{N_u}{f_{cu}\,bt} = 0.124 > 0.04$$

$$e = \frac{M_u}{N_u} = 0.6\ m \quad \rightarrow \frac{e}{t} = 0.547 > 0.05$$

$$\frac{M_u}{f_{cu}\,bt} = 0.068$$

$Interaction\ diag\ \propto = 0.6 \qquad \zeta = 0.9 \qquad f_y = 3600\ kg/cm^2$

$\rho < 1 \rightarrow Tension\ failure$

$$e_s = e + t/2 - 0.05 = 1.102\ m$$

$$M_{us} = N_u.e_s = 131.8\ m.t$$

$$R = 0.137 \xrightarrow{\propto=0.1} w = 0.19$$

$$A_s = wbd\,\frac{f_{cu}}{f_y} - \frac{N_u}{f_y/\gamma_s} = 10.3\ cm^2$$

$$A_{s\,min} = \frac{11}{f_y}bd = 11.23\,cm^2 \qquad (4\oint\!\!\!\oint 22)$$

Sec (12) : 35 * 80 cm □er

N_u = -119.58t
N_u = P_u = 0.35 f_{cu} A_c + 0.67 f_y A_{sc}

A_{sc} = - ve

$\therefore$ *use* $A_{s\,min} = 0.6\ \%\ A_c = 16.8\ cm^2$

Check of shear:

Critical sec is sec (4) 35 * 140 cm

Q_u = 74.85 t.

$$\therefore q_u = \frac{Q_u}{bd} = 15.84\ kg/cm^2$$

$$q_{cu} = 0.75\sqrt{\frac{f_{cu}}{\gamma_c}} = 9.68\ kg/cm^2$$

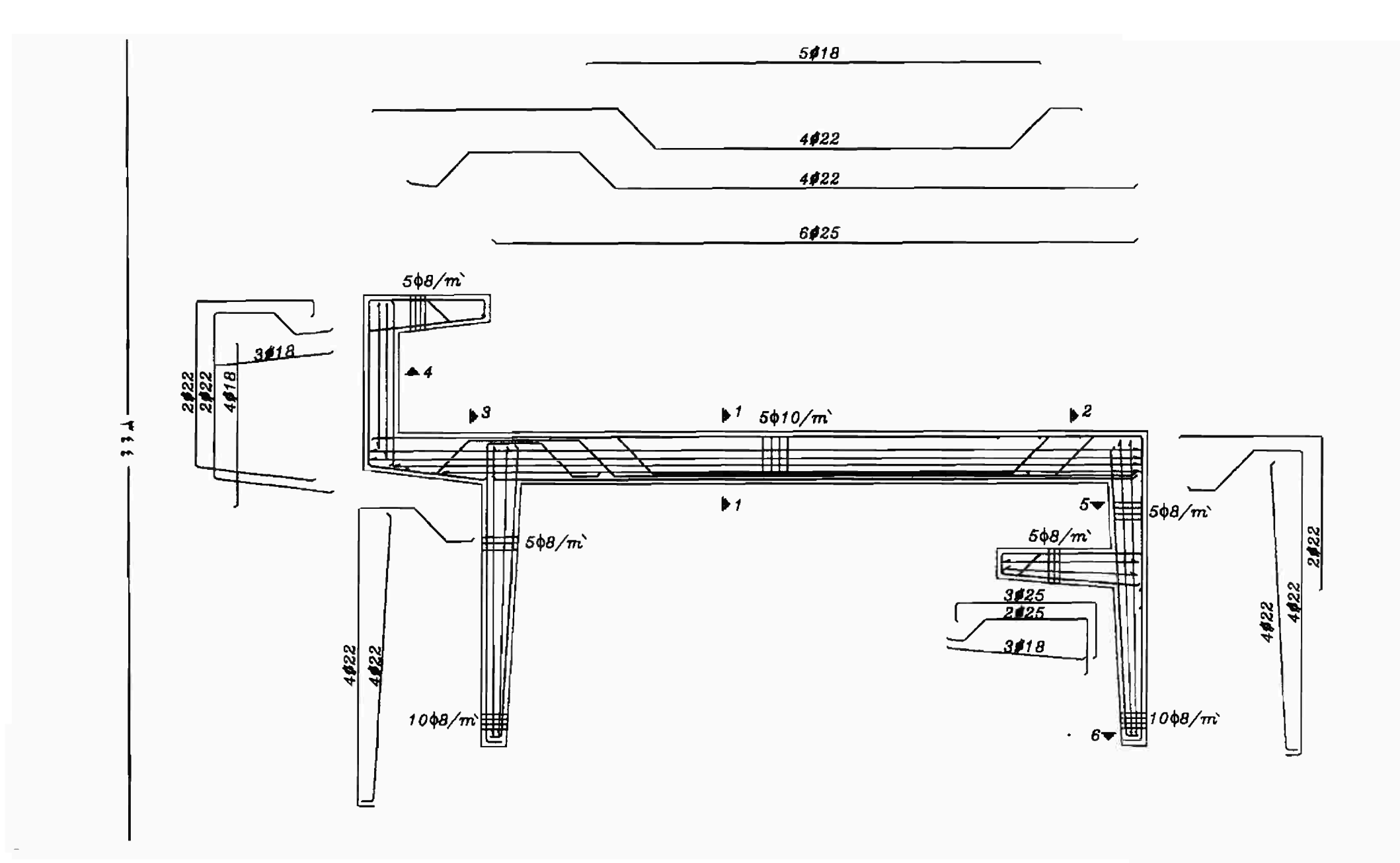
$$\therefore q_{su} = q_u - \frac{1}{2}q_{cu} = 11.0\ kg/cm^2$$

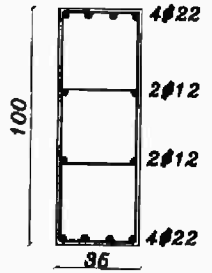
Assume using stirrups $5\oint 10/m^-$ (2 – *branches*)

$$q_{st} = \frac{A_{st}\ f_y/\gamma_s}{b.s} = \frac{2(0.785)*2400/1.15}{35*20} = 4.68\ kg/cm^2$$

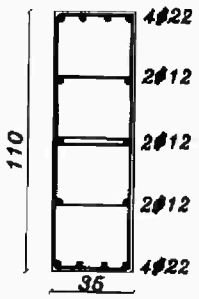
$$\therefore q_{sub} = 114.68 = 6.32\ kg/cm^2$$

$$A_{sb} = \frac{q_{sub}.b.d}{f_y/\gamma_s\ \sin_\alpha} = 13.5\ cm^2/raw \qquad (use\ 4\oiint 22)$$

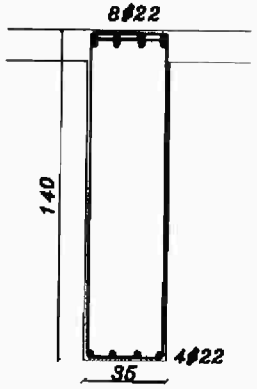

5ϕ18
4ϕ22
4ϕ22
6ϕ25
5ϕ8/m`
3ϕ18
2ϕ22
2ϕ22
4ϕ18
4
3
1
5ϕ10/m`
2
1
5
5ϕ8/m`
5ϕ8/m`
5ϕ8/m`
3ϕ25
2ϕ25
3ϕ18
4ϕ22
4ϕ22
10ϕ8/m`
10ϕ8/m`
6
4ϕ22
4ϕ22
2ϕ22



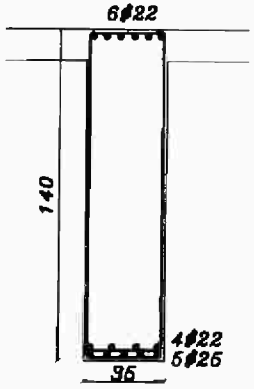
Sec. (4-4)



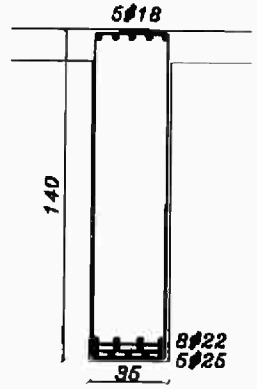
Sec. (5-5)



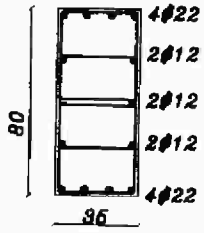
Sec. (3-3)



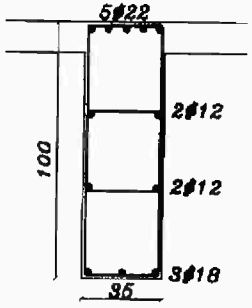
Sec. (2-2)



Sec. (1-1)



Sec. (6-6)



Sec. (7-7)

Example (2) :

Draw straining action diagrams, and design the critical section, then give full details for the following frames:

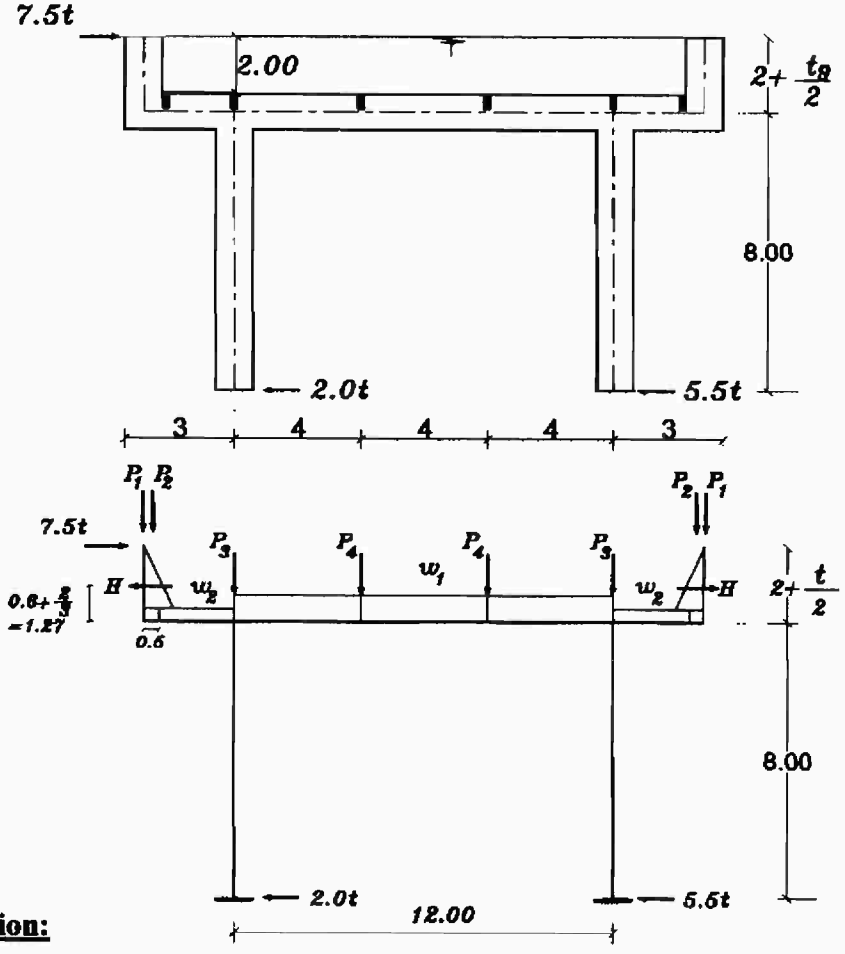
Frame (2) :

Data:

t_s = 14 cm
b = 35 cm
Spacing bet frames = 5 m.

o.w of sec beams – 0.5 t\ m^{-} .
o.w of frame = 1 t\ m^{-}

The frame is unbraced in its direction and braced in the other direction.



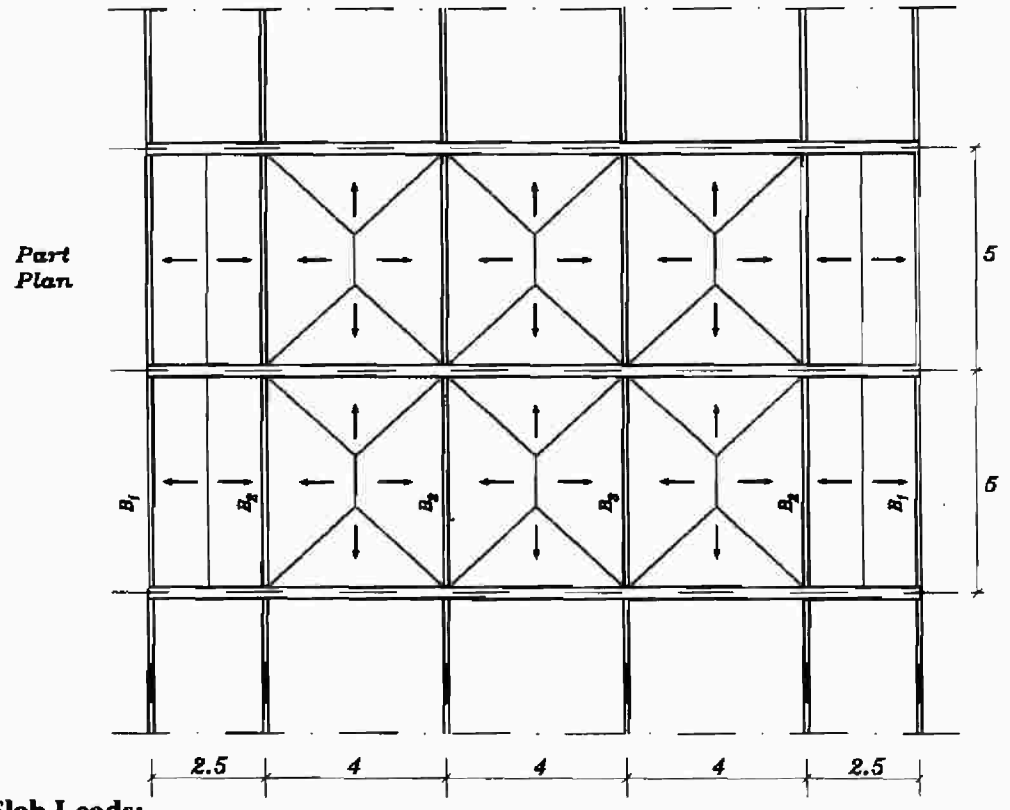
Solution:

$$t_g = \frac{1200}{12 \rightarrow 16} = 100cm \quad \textit{in it to } 120 \textit{ cm for add loads}$$

$t_c(upper) = 100\ cm$ $\qquad$ $t_c(lower) = 70\ cm$

$t_{we(stiff)} = 80\ cm$

Ultimate Loads:



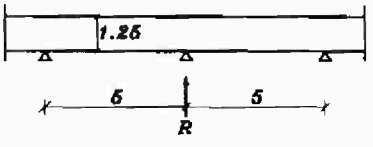
Slab Loads:

$w_s = o.w + \gamma h$

$= 0.14 * 2.5 + 2 = 2.35\ t/m^2$

$= 4.76\ t/m^-$

Beam B_1 :



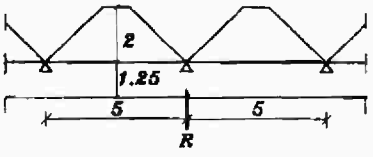
$w_{u1} = o.w + slab\ load$

$= 0.25 * 4.4 + 1.25 * 3.525$

$= 4.76\ t/m^-$

$P_2 = R = w_u * 5 = 23.8\ t$

Beam B_2 :



$w_{u2} = o.w + slab\ load$

$\frac{L}{2x} = \frac{5}{4} = 1.25 \rightarrow B = 0.6$

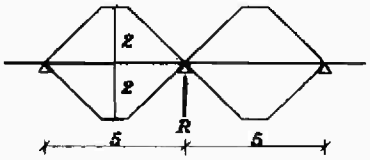
$\therefore w_{u2} = 0.25 \times 1.4 + 1.25 \times 3.525 + 0.6 * 3.525 * 2 = 8.99\ t/m^-$

$P_3 = R = w_{u2} * 5 = 44.9\ t$

Beam B_3 :

$w_{u3} = o.w + slab\ load$

$\frac{L}{2x} = 1.25 \rightarrow B = 0.6$



$w_{u3} = 0.25 * 1.4 + 2 * 0.6 * 3.525 * 2 = 8.81\ t/m^{-}$

$P_4 = R = w_{u3} * 5 = 44.1\ t$

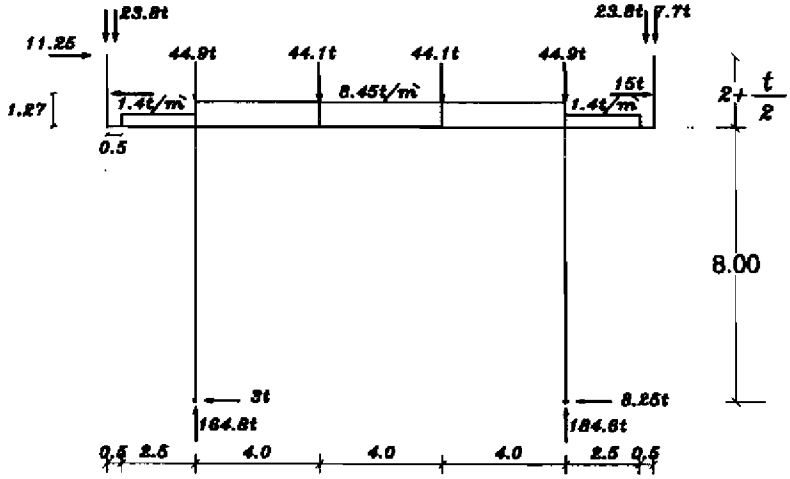
$P_1 = o.w\ of\ wall = 1.4\left[1 * 2 + 0.14 * 2.5 * 2 * 5\right] = 7.7t$

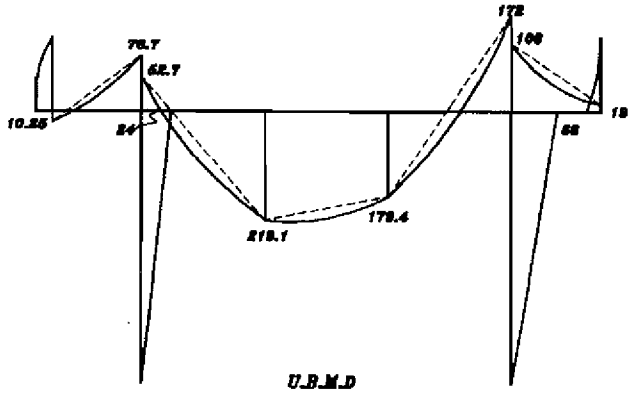
$$w_1 = o.w + slab\ load = 1.4 * 1 + \frac{6 * \frac{1}{2} * 4 * 2}{12} * 3.525 = 8.45\ t/m^{-}$$

$w_2 = o.w = 1 * 1.4 = 1.4\ t/m^{-}.$

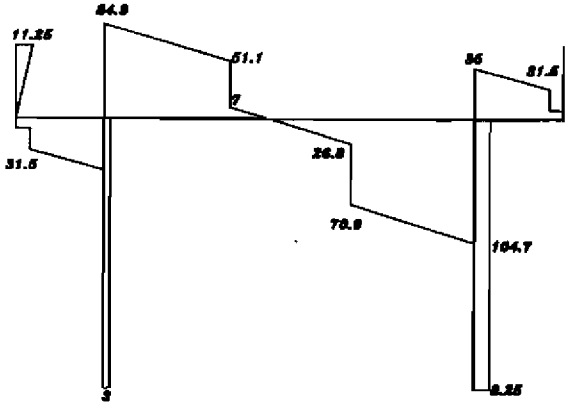
$H = \frac{\gamma h^2}{2} * spacing = \frac{1(2)^2}{2} * S = 10\ t$

$H_u = 1.5 * 10 = 15t$

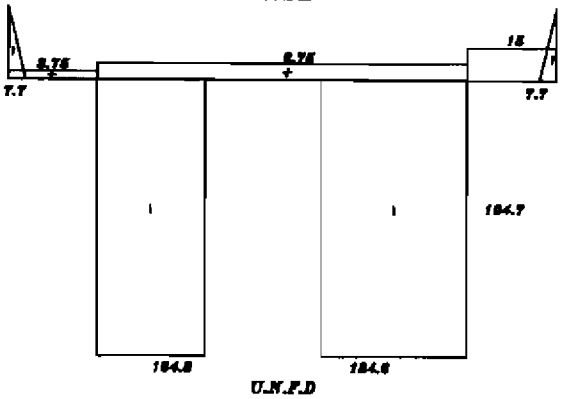




U.B.M.D

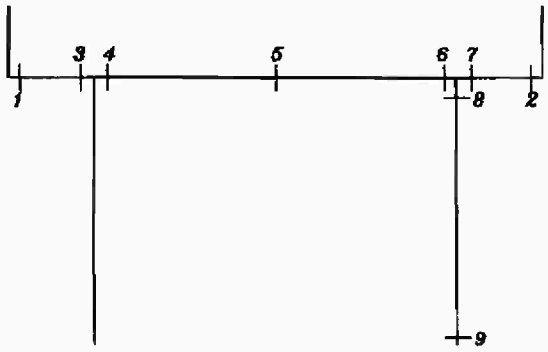


U.S.F.D



U.N.F.D

Design of sections:



Sec (1) 35 *100 cm (I)

$M_u = 10.25 mt \quad N_{us} + 3.75\ t$

$$e = \frac{M_u}{N_u} = 2.73\ m$$

$e/t = 2.73 > 0.5$ *Big ecc*

$e_s = e - t/2 + 0.05 = 2.28 m$

$M_{us} = N_u - e_s = 8.56\ mt$

$$R = \frac{M_{us}}{f_{cu} bd^2} = 0.011 \rightarrow w = 0.013$$

$$A_s = wbd \frac{f_{cu}}{f_y} + \frac{N_u}{f_y/\gamma_s} = 4.2\ cm^2$$

$$A_{s\ min} = \frac{11}{f_y} bd = 10.16 cm^2$$

$\therefore$ *use* $A_{s\ min} = (4\phi 22)$

Sec (2) 35 * 100 □ Ler

$M_u = 19$ m.t $\qquad N_u = +15$ t

$e = 1.27\ m \qquad \frac{e}{t} = 1.27 > 0.5 \qquad$ *Big ecc*

$e_s = e - t/2 + 0.05 = 0.817\ m$

$M_{us} = N_u e_s = 12.25\ mt$

$R = 0.016 \rightarrow w = 0.019$

$$A_s = wbd \frac{f_{cu}}{f_y} + \frac{N_u}{f_y/\gamma_s} = 9.18\ cm^2 < A_{s\ min}$$

$\therefore$ *use* $A_{s\ min} = 10.16 cm^2 \qquad use(4\phi 22)$

Sec (3) 35 * 1200 m R :

$M_u = 76.7\ mt$, $N_u = 13.75\ t$

$e = 20.45\ m \qquad \frac{e}{t} > 0.5 \qquad$ *big ecc*

$e_s = e - t/2 + 0.05 = 19.9\ m$

$M_{us} = N_u e_s = 74.6\ mt$

$R = 0.064 \rightarrow w = 0.81$

$$A_s = wbd \frac{f_{cu}}{f_y} + \frac{N_u}{f_y/\gamma_s} = 23.84\ cm^2 \qquad (6\phi 25)$$

Sec (4) 35 * 120 cm R

$M_u = 52.7\ m.t$ $\qquad N_u = 6.75\ t$

$e = 7.81\ m$ $\qquad e/t = 6.5 > 0.5$ *Big ecc*

$e_s = e - t/2 + 0.05 = 7.26\ m$

$M_{us} = N_u . e_s = 48.99\ m.t$

$R = 0.042 \rightarrow w = 0.051$

$$A_s = wbd\frac{f_{cu}}{f_y} + \frac{N_u}{f_y/\gamma_s} = 16.4\ cm^2 \qquad (4\phi 25)$$

Sec (5) 35 * 120 cm. T

$M_u = 219.1\ m.t$ $\qquad N_u = +6.75\ t$

$e = 32.46\ m$ $\qquad e/t > 0.5$

$e_s = e - t/2 + 0.05 = 31.9\ m$

$M_{us} = N_u . e_s = 215.4\ m.t$

$B = 16\ t_s + b = 259\ cm$

$$d = C_1\sqrt{\frac{M_{us}}{f_{cu}\ B}} \rightarrow C_1 = 6.3 \rightarrow take\ c/d_{min} = 0.125$$

$C = 14..375\ cm$ $\qquad a = 0.8C = 11.5\ cm < t_s$ $\qquad (O.K)$

$J = 0.826$ $\qquad \therefore A_s = 65.15\ cm^2$

$\therefore use\ (14\phi 25)$

Sec (6) 35 * 120 □ Lec

$M_u 172\ m.t$ $\qquad N_u = +6.75\ t$

$e = 25.48\ m$ $\qquad e/t > 0.5$

$e_s = e - t/2 + 0.05 = 24.93\ m$

$M_{us} = N_u . e_s = 168.3\ m.t$

$R = 0.145 \xrightarrow{\alpha=0.1} w = 0.2$

$A_s = 58.06\ cm^2$ $\qquad (12\phi 25)$

Sec (7) 35 * 120 $\square^{Lor}$

$M_u = 106\ mt$ $N_u = +15\ t$

$e = 707\ m$ $e/t > 0.5$

$e_s = e - t/2 + 0.05 = 6.517\ m$

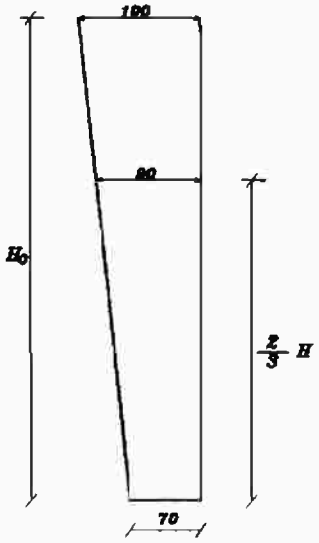
$M_{us} = N_u e_s = 97.75\ mt$

$R = 0.084 \rightarrow w = 0.109$

$A_s = 35.26 cm^2$ (8ϕ25)

Sec (8) 35 * 100 $\square^{Lor}$

$M_u = 66\ mt$ $N_u = -184.64\ t$



Buckling:

$H_o = 8.0 - 0.6 = 7.40$ m .

Top End condition $t_B > t_c$ case (1)
Bottom End condition hinged case (3)

Table (6 – 10) → K = 1.6

$$\lambda = \frac{1.6 * 740}{90} = 13.16 > 10 \qquad Long$$

$$\delta = \frac{\lambda^2 b}{2000} = \frac{(13.16)^2 * 0.9}{2000} = 0.0775\ m$$

$M_{add} = N_u . \quad \delta = 14.4\ mt$

$M_u = 66 + 14.4 = 80.4\ mt$

$$\frac{N_u}{f_{cu}\ bt} = 0.23 > 0.04$$

$e = 0.436\ m \rightarrow e/t = 0.436 > 0.05$

$$\frac{M_u}{f_{cu} bt^2} = 0.092$$

Interaction Diag $\alpha = 0.6$ $\eta = 0.9$ $f_y = 3600$

Comp failure $\rho = 1.8$

$\mu = 0.45\ \% \rightarrow \mu_t = (1+\alpha)\mu = 0.72\%$

$\mu_{min} = 0.85 + 0.052\lambda = 0.93\ \%$

$\therefore$ *use* $A_{s\,min}$

$$A_s = \frac{0.0093}{1.6} * 35 * 100 = 20.44\ cm^2$$

$A_s^- = 0.6\ A_s = 12.26\ cm^2$

sec (9) 35 * 70 cm

$N_u = -184.6\ t$ (A_s *short column*)

$N_u = 0.35\ f_{cu}\ A_c + 0.67\ f_y\ A_{sc}$

$\therefore A_{sc} = -v_e$

$\therefore$ *use* $A_{s\,min} = 0.6\%\ A_c = 14.7\ cm^2$

Check of shear sec (6) 35 * 120 cm

$Q_u = 104.7\ t$

$$q_u = \frac{Q_u}{bd} = 26.\ kg/cm^2$$

$$q_{cu\,(max)} = 2.2\sqrt{\frac{f_{cu}}{\gamma_c}} = 28.4\ kg/cm^2 > q_u \qquad (o.k)$$

$$q_{cu} = 0.75\sqrt{\frac{f_{cu}}{\gamma_c}} = 9.68\ kg/cm^2$$

$$\therefore q_{su} = q_u - \frac{1}{2}q_{cu} = 21.16\ kg/cm^2$$

Assume usin g stirr ($8\phi 10/m^-$)

$$\therefore q_{st} = \frac{A_{st}.f_y/\gamma_s}{b.s} = \frac{2*0.785*2400/1.15}{35*12.5} = 7.49\ kg/cm^2$$

$$\therefore q_{sub} = 21.16 - 7.49 = 13.67\ kg/cm^2$$

$$\therefore A_{sb} = \frac{q_{su}b.\ b.d}{f_y/\gamma_s\ \sin_\alpha} = 24.85\ cm^2 \qquad (5\phi\phi 25)$$

(*OR*) *Assume usin g* $4\phi\phi 25$ *bent bars*

$\therefore q_{sub} = 10.8\ kg/cm^-$

$\therefore q_{st} = 21.16 - 10.8$

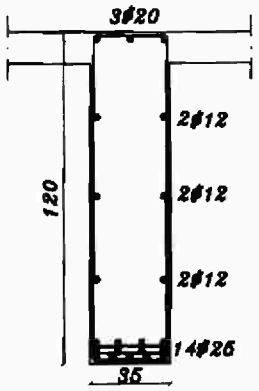
$$A_{st} = \frac{q_{st}\ b.s}{f_y/\gamma_s} \qquad use\ \ f_y = 2800\ kg/cm^2$$

usin g stirrups $\phi 10$

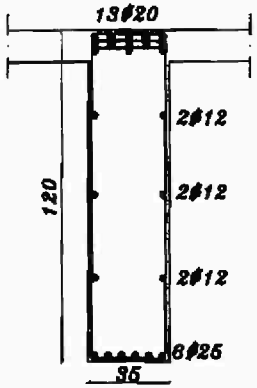
$\because S = 10.55\ cm$

$\therefore$ *usin g stirrups* ($10\phi 10/m$)

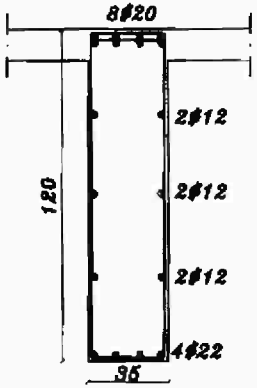
5Φ25
4Φ22
3Φ18
4Φ22
4
8
8Φ8/m`
5Φ8/m`
5
2
3Φ20
4Φ25
4Φ25
6Φ25
10Φ10/m`
1
5Φ8/m`
8Φ8/m`
3
4Φ22
4Φ22
3Φ18
4Φ22



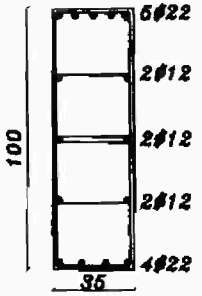
Sec. (1)



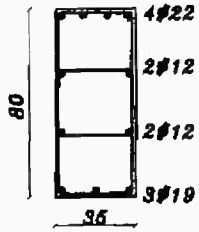
Sec. (2)



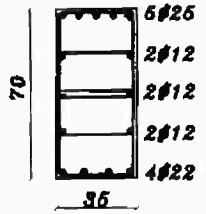
Sec. (3)



Sec. (5)



Sec. (4)



Sec. (8)

Example (3) :

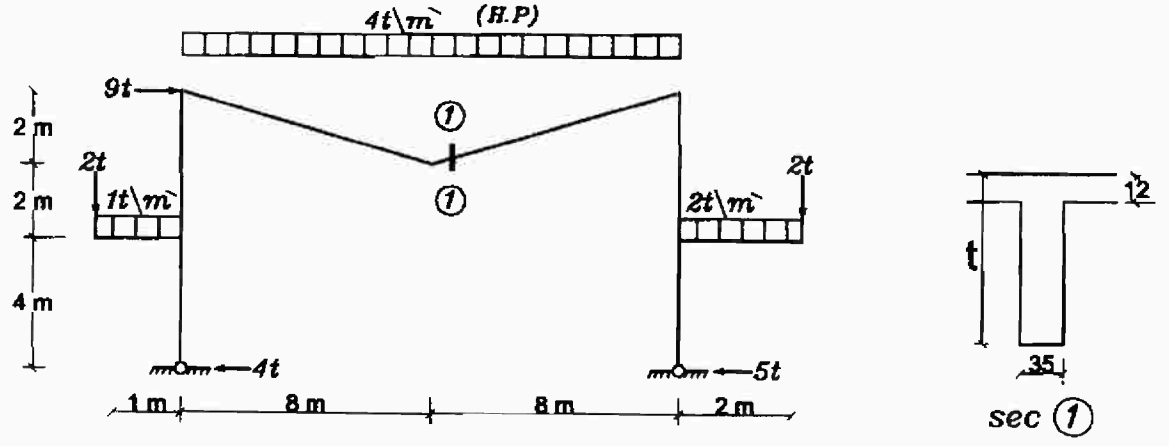
Draw straining action diagrams, and design the critical section, then give full details for the following frames:

Frame (3) :

Data:

$t_s = 12$ cm
$b = 35$ cm
$f_{cu} = 250$ kg/cm^2 .
$f_y = 3600$ kg/cm^2 .

The frame may be considered braced in each direction.



Solution:

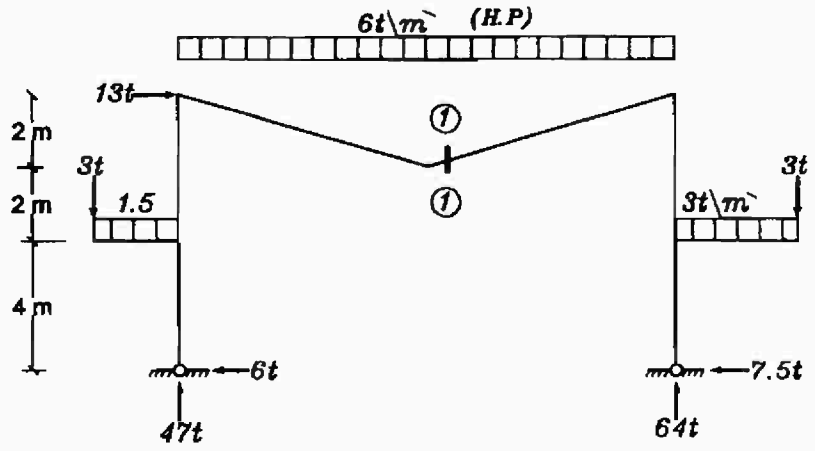
Dimensioning:

$$t_g = \frac{1600}{12 \rightarrow 16} = 120\ cm$$

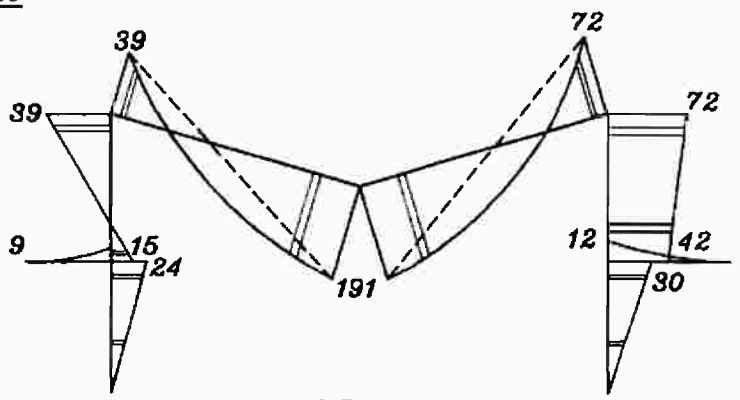
$t_{c\,(upper)} = 100\ cm$

$t_{c\,(lower)} = 70\ cm$

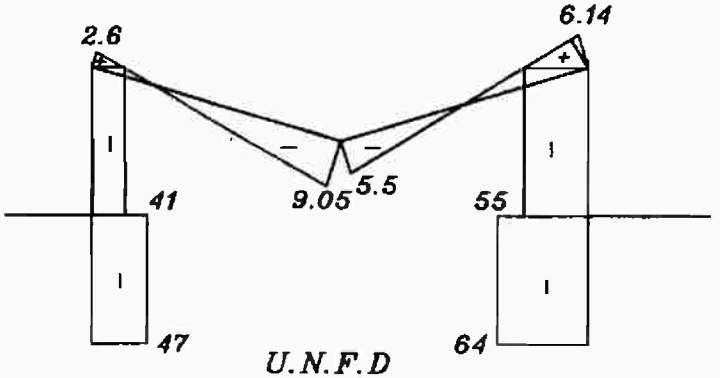
$t_{\ cantliver} = 80\ cm$



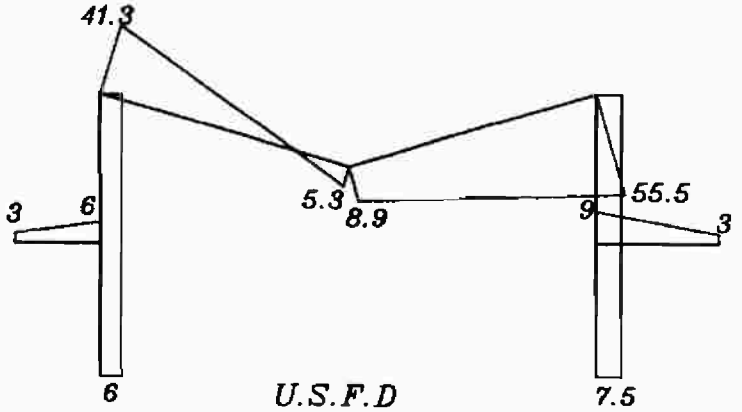
Straining Actions



U.B.M.D



U.N.F.D



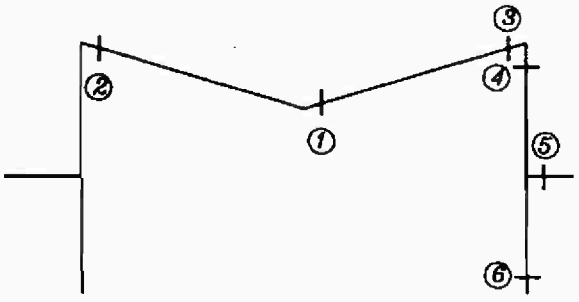
U.S.F.D

Loads:

$w_u = 1.5 * w_{working}$

Design of section:

Sec	M_u	N_u
1	191	- 9.05
2	39	+ 2.6
3	72	+ 6.14
4	72	- 55
5	12	0
6	0	64



Sec (1) 35 * 120 T – sec:

B = Smallest of
- $16\ t_s + b = 227$ cm
- C_L to d
- $\frac{L}{10} + b$

$M_u = 191\ m.t$

$N_u = -9.05\ t$

$\frac{N}{f_{cu} b t} = 0.009 < 0.04 \qquad \therefore neglect\ N_u$

$d = C_1 \sqrt{\frac{M_u}{f_{cu} B}} \rightarrow C_1 = 6.27 \qquad \therefore Take\ c/d_{min} = 0.125$

$\therefore C = 0.125\ d = 14.375 cm \rightarrow a = 0.8\ c = 11.5\ cm < t_s \ \ (O.K)$

$J = 0.826$

$A_s = \frac{M_u}{f_y J d} = 55.7\ cm^2 \qquad (12 \phi 25)$

Sec (2) : 35 * 120 cm □ler

$M_u = 39$ m.t

$N_u = +2.6$ t

$e = \frac{M_u}{N_u} = 15\ m \rightarrow \frac{e}{t} = 12.5 > 0.5 \quad Big.\ ecc$

$e_s = e - t/2 + 0.05 = 14.45 m$

$M_{us} = N_u - e_s = 37.6\ m.t$

$R = \frac{M_u}{f_{cu} b d^2} = 0.032 \rightarrow w = 0.039$

$$A_s = wbd\frac{f_{cu}}{f_y} + \frac{N_u}{f_y/\gamma_s} = 11.73\ cm^2$$

$$A_{s\,min} = \frac{11}{f_y}bd = 12.3\ cm^2$$

$$\therefore use\ A_{s\,min}(4\phi 22)$$

Sec (3) : 35 * 120 cm $\square^{ler}$

$M_u = 72$ m.t
$N_u = 6.14$ t

$$e = \frac{M_u}{N_u} = 11.73\ m \rightarrow \frac{e}{t} = 9.77 > 0.5\ \ Big\ ecc$$

$$e_s = e - t/2 + 0.05 = 11.176\ m$$

$$M_{us} = N_u.e_s = 68.62\ m.t$$

$$R = 0.059 \rightarrow w = 0.074$$

$$\therefore A_s = wbd\frac{f_{cu}}{f_y} + \frac{N_u}{f_y/\gamma_s} = 22.65\ cm^2 \qquad (use\ 6\phi 22)$$

Sec (4) : 35 * 100 cm $\square^{ler}$

$M_u = 72$ m.t
$N_u = -55$ t

$$\frac{N_u}{f_{cu}bt} = 0.063 > 0.04$$

$$e = \frac{M_u}{N_u} = 1.31\ \ m \rightarrow \frac{e}{t} = 1.31 > 0.05$$

$$\frac{M_u}{f_{cu}bt^2} = 0.082$$

$$Interaction\ Diag\ \alpha = 0.6 \qquad \zeta = 0.9 \qquad f_y = 3600\ \ kg/cm^2$$

$$\rho < 1 \rightarrow Tension\ failure$$

$$e_s = e + \frac{t}{2} - 0.05 = 1.76\ m$$

$$M_{us} = N_u.e_s = 96.75\ m.t$$

$$R = \frac{M_{us}}{f_{cu}bd^2} = 0.123 \xrightarrow{\alpha = 0} w = 0.171$$

$$A_s = wbd\frac{f_{cu}}{f_y} - \frac{N_u}{f_y/\gamma_s} = 21.9\ cm^2 \qquad (6\phi 22)$$

Sec (5) : 35 * 80 cm (Rect)

M_u = 12 m.t
R = 0.024 $\rightarrow$ $w = 0.029$

A_s = 5.3 cm^2 .

$$As_{\min} = \frac{11}{f_y}bd = 8\ cm^2$$

$\therefore use\ As_{\min}$ $\qquad (4\phi 19)$

Sec (6) : 35 * 70 cm

N_u = - 64 t

Design as short column

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

64 * 1000 = 0.35 * 250 * 35 * 70 + 0.67 * 3600 A_{sc}

A_{sc} = - ve

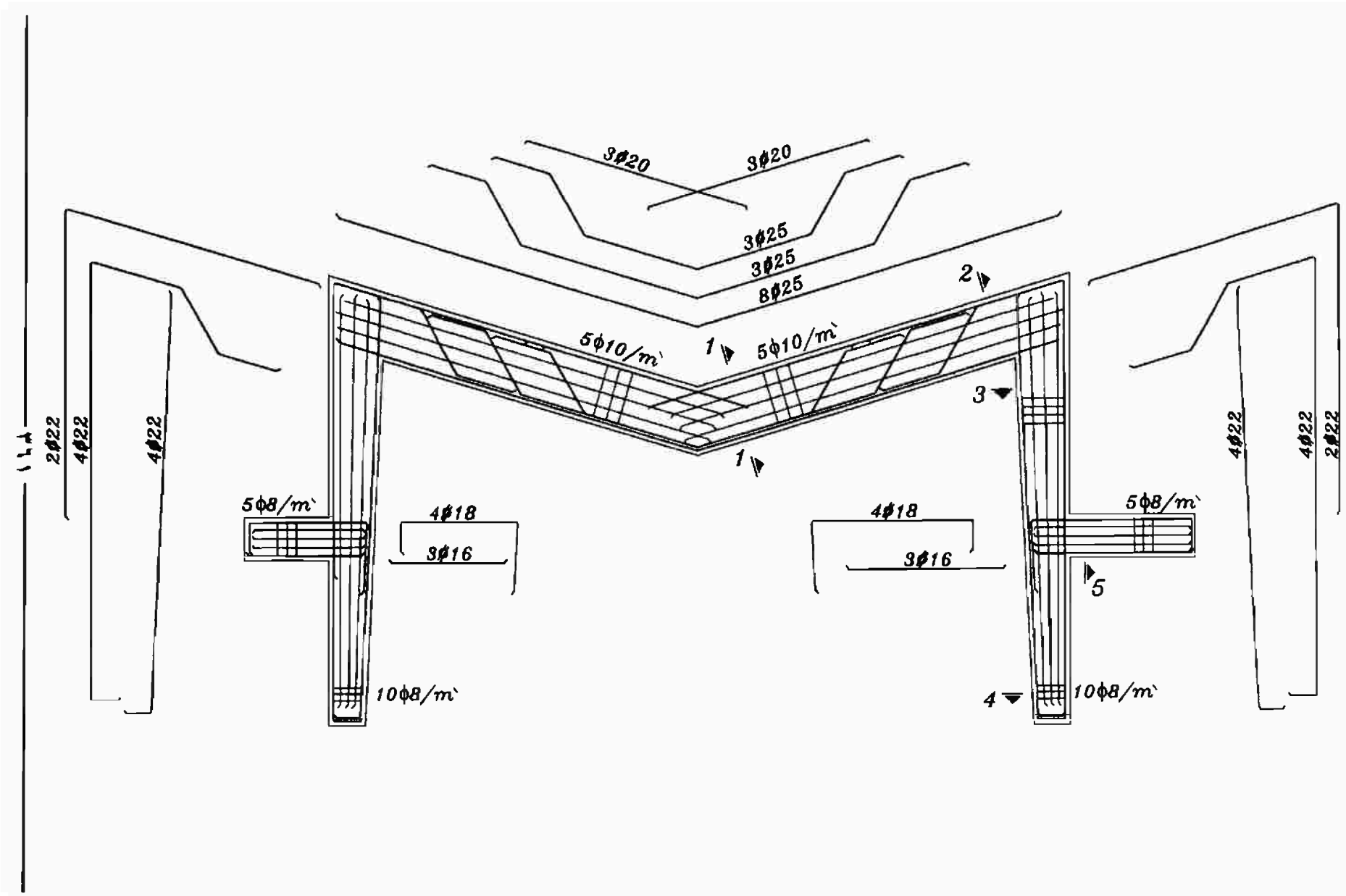
$\therefore$ use $A_{sc\ min}$ = 0.6 % A_c = 14.7 cm^2 .

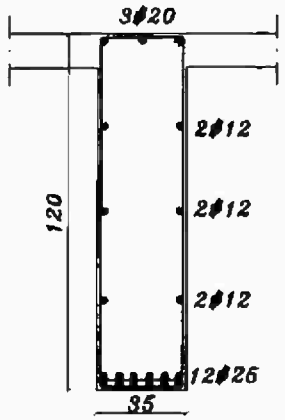
Check of shear:

$$Q_u = 55.5\ t$$
$$q_u = \frac{Q_u}{bd} = 13.79\ kg/cm^2$$
$$q_{cu} = 0.75\sqrt{\frac{f_{cu}}{\gamma_c}} = 9.68\ kg/cm^2$$
$$q_{su} = q_u - \frac{1}{2}q_{cu} = 8.95\ kg/cm^2$$

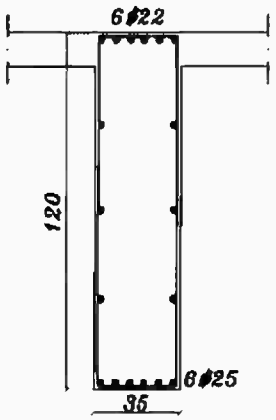
Assume using stirrups $5\phi 10\ /m^-$ $\qquad (2-branches)$

$$\therefore q_{st} = \frac{A_{st}\ f_y/\gamma_s}{b.s} = 4.68\ kg/cm^2$$
$$q_{sub} = 8.95 - 4.68 = 4.24\ kg/cm^2$$
$$A_{sb} = \frac{q_{sub}\ b.d}{f_y/\gamma_s\ \sin_\alpha} = 7.8\ cm^2 \qquad (3\phi 22)$$

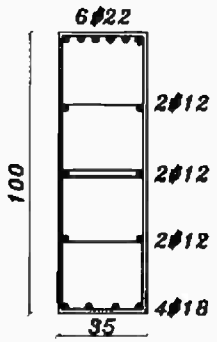

3ϕ20
3ϕ20
3ϕ25
3ϕ25
8ϕ25
2
5ϕ10/m`
1
5ϕ10/m`
3
1
2ϕ22
4ϕ22
4ϕ22
5ϕ8/m`
4ϕ18
3ϕ16
4ϕ18
3ϕ16
5ϕ8/m`
5
4ϕ22
4ϕ22
2ϕ22
10ϕ8/m`
4
10ϕ8/m`



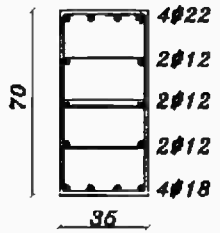
Sec. (1-1)



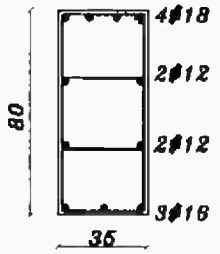
Sec. (2-2)



Sec. (3-3)



Sec. (4-4)



Sec. (5-5)

Example (4) :

Draw straining action diagrams, and design the critical section, then give full details for the following frames:

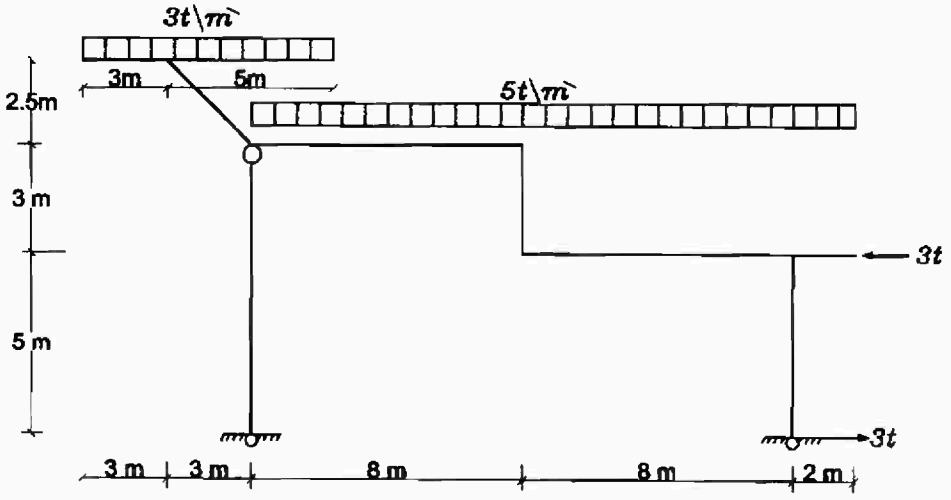
Frame (4) :

Data:

$t_s = 12$ cm
$b = 40$ cm

$f_{cu} = 250$ kg/cm^2 .
$f_y = 3600$ kg/cm^2 .

The frame is unbraced in its direction and braced in the other direction.



Solution:

Dimensioning:

$$t_g = \frac{1600}{12 \rightarrow 16} = 120\ cm$$

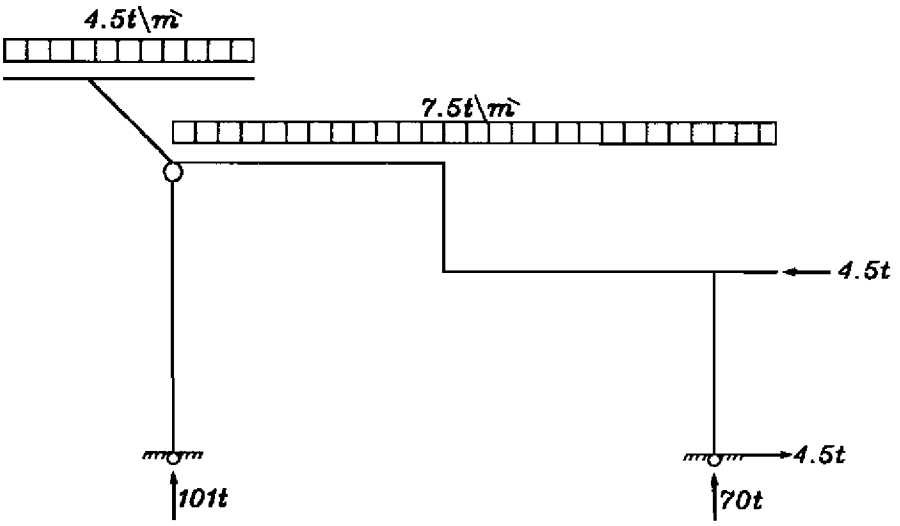
$t_{c\,(upper)} = 100\ cm$ & $t_{c\,(lower)} = 70\ cm$

$t_{(link)} = Bigger\ of$ $0.4\ tg = 48\ cm$

$$\frac{1}{20} = 80\ cm$$

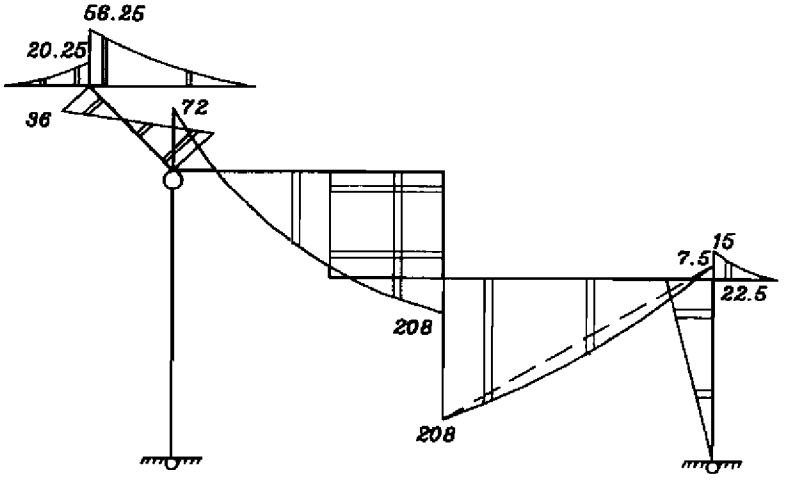
$t_{(Link)} = 80\ cm$

Loads

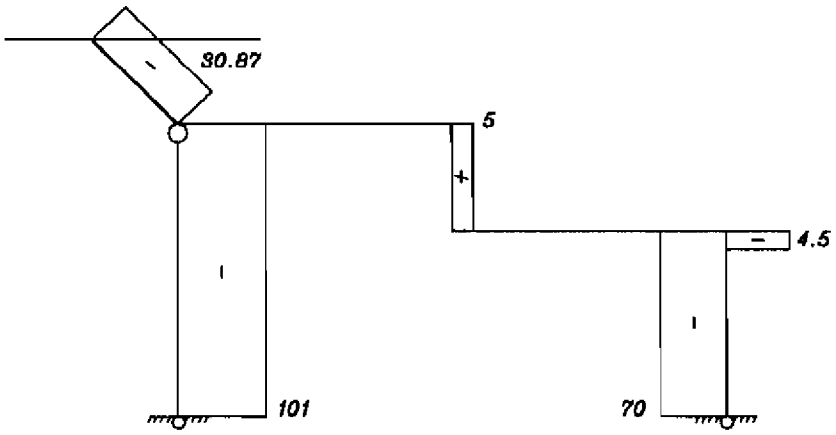


*Ultimate Load = Working Load * 1.5*

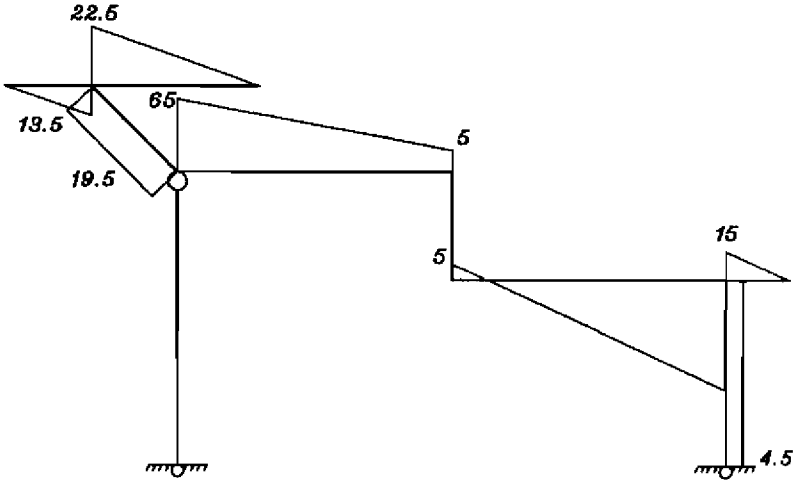
Straining Actions



U.B.M.D



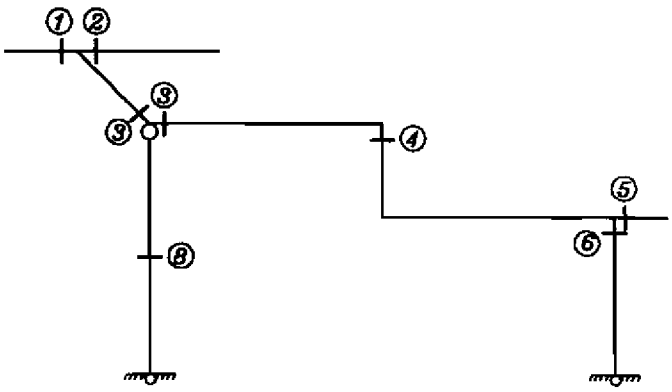
U.N.F.D



U.S.F.D

Design of section:

Sec	M_u	N_u
1	20.25	0
2	56.25	0
3	72	-30.87
4	208	+5
5	15	0
6	22.5	-70
7	0	-70
8	0	-101



Sec (1) 40 * 120 cm $\square^{ler}$

$M_u = 20.25$ m.t
$N_u = 0$

$$R = \frac{M_u}{f_{cu} bd^2} = 0.015 \rightarrow w = 0.02$$

$$A_s = wbd \frac{f_{cu}}{f_y} = 6.39\ cm^2$$

$$A_{s\ min} = \frac{11}{f_y} bd = 14\ cm^2 > A_s$$

$\therefore$ *use* $A_{s\ min}$ (4ϕ22)

Sec (2) 40 * 120 cm $\square^{ler}$

$M_u = 56.25$ m.t

$$R = \frac{M_u}{f_{cu} bd^2} = 0.043 \rightarrow w = 0.053$$

$$\therefore A_s = wbd \frac{f_{cu}}{f_y} = 16.93\ cm^2 \quad (6ϕ22)$$

Sec (3) 40 * 120 cm □ter

M_u = 72 m.t
N_u = 30.87 t

$$\frac{N_u}{f_{cu} bt} = 0.026 < 0.04 \rightarrow neglect\ N_u$$

$$\therefore R = \frac{M_u}{f_{cu}\ bd^2} = 0.054 \quad \rightarrow w = 0.067$$

$$\therefore A_s = 21.4\ cm^2 \quad (6\phi 22)$$

Sec (4) 40 * 120 cm □ter

M_u = 208 m.t
N_u = + 5 t

$$e = \frac{M_u}{N_u} = 41.6\ m \rightarrow \frac{e}{t} > 0.5 \qquad \rightarrow Big\ ecc$$

$$e_s = e - t/2 + 0.05 = 41.05\ m$$

$$M_{us} = N_u . e_s = 205.25\ mt$$

$$R = \frac{M_{us}}{f_{cu} bd^2} = 0.1552 \xrightarrow{\alpha = 0.2} w = 0.21$$

$$A_s = wbd\ \frac{f_{cu}}{f_y} + \frac{N_u}{f_y / \gamma_s} = 68.7\ cm^2 \qquad (14\phi 25)$$

$$As^- = 0.2\ A_s = 13.74\ cm^2 \qquad (4\phi 22)$$

Sec (5) 40 * 120 cm □ker

M_u = 15 m.t

Use $A_{s\,min}$ = 14 cm^2 (4 ⌀22)

Sec (6) 40 * 100 cm □ker

M_u = 22.5 m.t
N_u = 70 t .

Check additional moment due to buckling In plane

H_o = 5 – 0.6 = 4.4 m .

Top End Condition case (1) $t_g > t_c$

Bottom End condition case (3) Hinge

Table (6 – 10) → K = 1.6

$$t_{avr} = 70 + 30 * \frac{2}{3} = 90\ cm$$

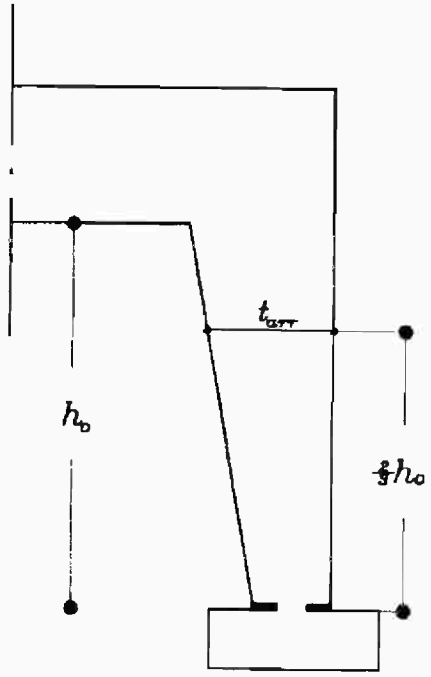
$$\lambda_b = \frac{1.6\ H_o}{t_{avr}} = \frac{1.6 \times 440}{80} = 7.8 < 10 \ \therefore \textit{short}$$

no additional moments

$$\frac{N_u}{f_{cu}\, bt} = 0.07 > 0.04$$

$$e = \frac{M_u}{N_u} = 0.32 \qquad e/t > 0.05$$

$$\frac{M_u}{f_{cu}\, bt^2} = 0.016$$



Interaction Diag:

$\alpha = 0.6$ $\quad \zeta = 0.9$ $\quad f_y = 3600\ kg/cm^2$

$\rho < 1.0 \quad \rightarrow tension\ \ failure$

$$e_s = e + \frac{t}{2} - 0.05 = 0.32 + 0.5 - 0.05 = 0.77$$

$$M_{us} = N_u e_s = 53.9\ mt$$

$$R = \frac{M_{us}}{f_{cu}\, bd^2} = 0.0597 \xrightarrow{\alpha=0.5} w = 0.071$$

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

$$\therefore use\ A_{s\,min} = \frac{0.6\%}{1.6} bt = 18\ cm^2 \quad (6\phi 22)$$

Sec (7) 40 * 70 cm

N_u = - 70 t.

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

A_{sc} = - ve

$$\therefore use\ A_{s\,min} = 0.6\%\ A_c = 16.8\ cm^2$$

Sec (8) 40 * 80 cm

N_u = - 101 t.

Check on additional moment due to buckling. (in plane)

$H_o = 8 - 0.6 = 7.4$ m

Top End condition — case (1) $t_g > t_c$
Bottom End condition — case (3) Hinge.

Table (6 – 10) $\rightarrow K = 1.6$

$$\therefore \lambda_b = \frac{1.6 \times 740}{80} = 14.8 > 10 \; long$$

$$\delta = \frac{\lambda_b^2 b}{2000} = \frac{(14.8)^2 * 0.8}{2000} = 0.088 \; m$$

$$M_{add} = N_{us} = 8.85 \; m.t = M_u$$

$$\frac{N_u}{f_{cu} bt} = 0.126 > 0.04$$

$$e = \frac{M_u}{N_u} = 0.088 \; m \qquad e/t = 0.11 > 0.05$$

$$\frac{M_u}{f_{cu} \, bt^2} = 0.014$$

Interaction diagrams $\alpha = 0.1$ $\zeta = 0.9$ $f_y = 3600$

$\rho < 1.0$

$\therefore$ *use* $A_{s\,min}$

$$M_{min} = 0.25 + 0.052 \, \lambda_b = 1.02 \, \%$$

$$A_{s\,min} = 32.6 cm^2$$

Check of shear:

Sec (3) 40 * 120 cm .

$Q_u = 65\ t$

$\therefore q_u = 14.13\ kg/cm^2$

$q_{cu} = 9.68\ kg/cm^2$

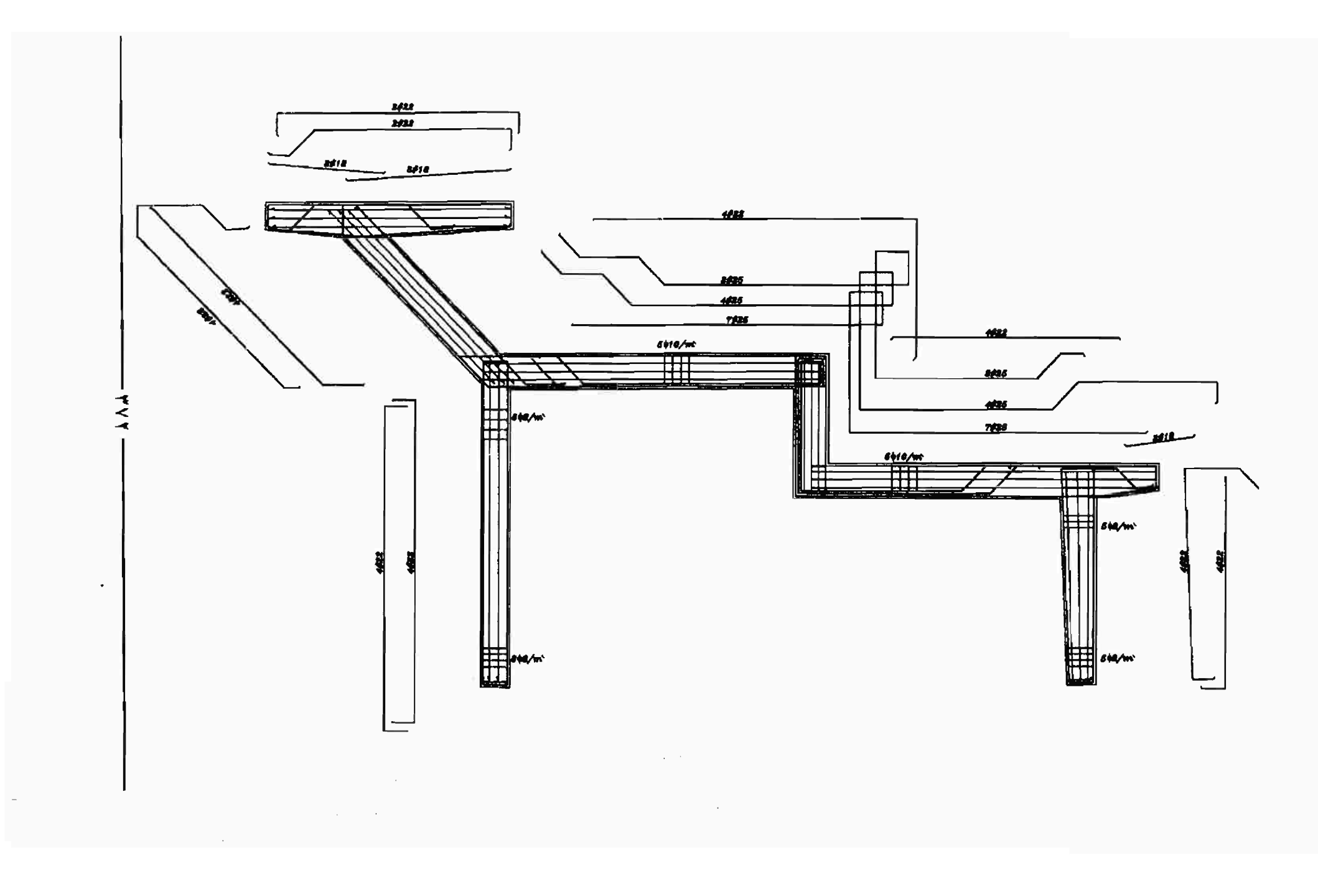
$q_{su} = q_u - 0.5\ q_{cu} = 9.29\ kg/cm^2$

Assume usin g stirr (5∫10/m⁻)

$$\therefore q_{st} = \frac{A_{st}\ f_y/\gamma_s}{b.s} = 5.42\ kg/cm^2$$

$\therefore q_{sub} = 3.87\ kg/cm^2$

$$A_{sb} = \frac{q_{sub}.b.d}{f_y/\gamma_s \sin_{\alpha}} = 8.04\ cm^2 \qquad (3\oint\!\!\!\!\int 22)$$



Example (5) :

* Design curves and Egyptian code are only allowed material.
* Maximum grade is 100 points.

General Data:

Concrete grade : f_{cu} = 250 kg /cm^2.
For reinforcing steel : f_y = 3600 kg/cm^2.

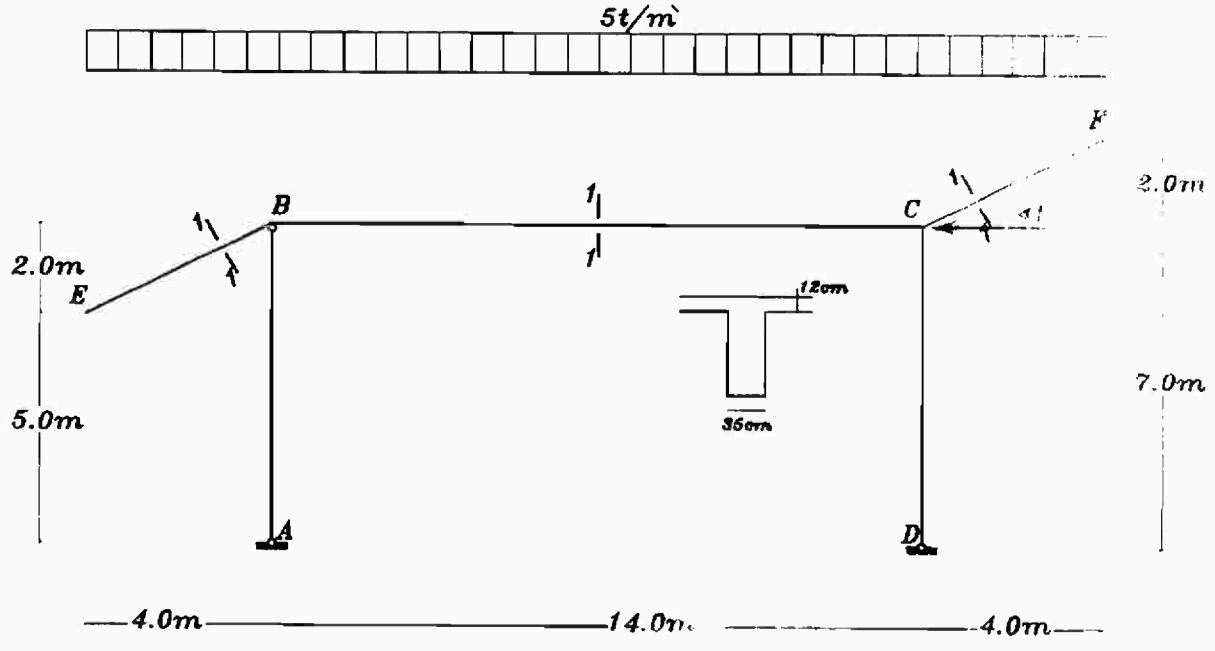
Any data not given may be reasonably assumed.

Question 1 : (65 Points) :

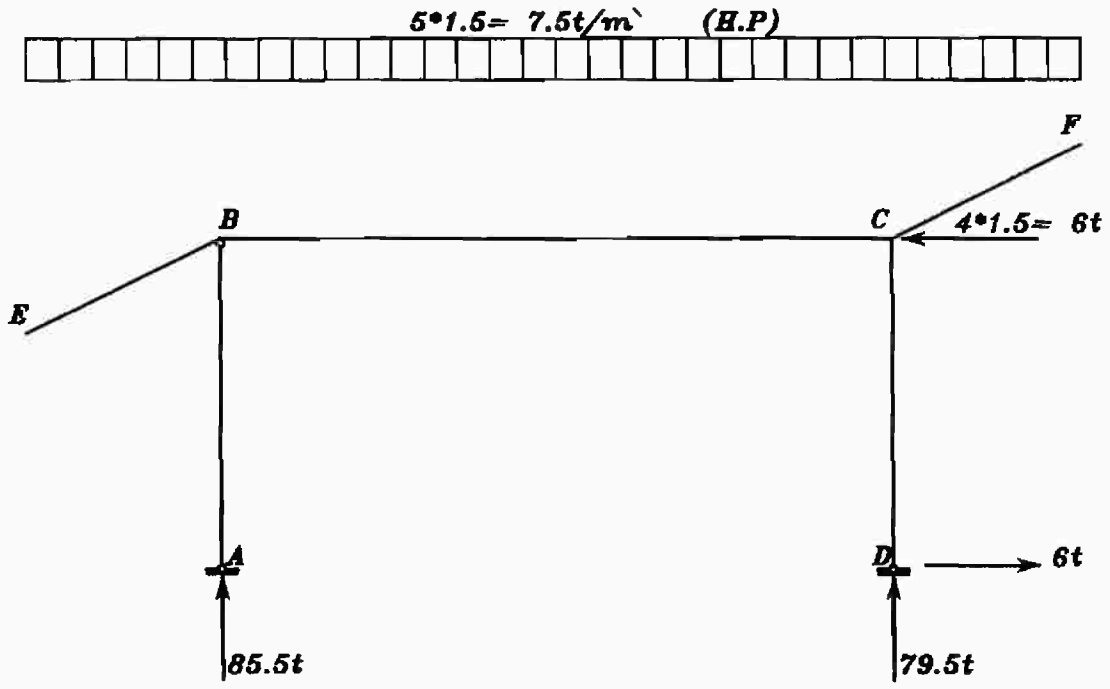
A stadium frame ABCD has two cantilevers BE and CF. The vertical member AB is hinged at booth ends A and B, while the vertical member CD is hinged only at the support D. The frame is assumed braced in both direction. Service loads on the frame are 5 t / m horizontal project (including own weight of the frame) between E and F. A horizontal force of 4 tones is also acting at point C as shown in figure.

It is required to:

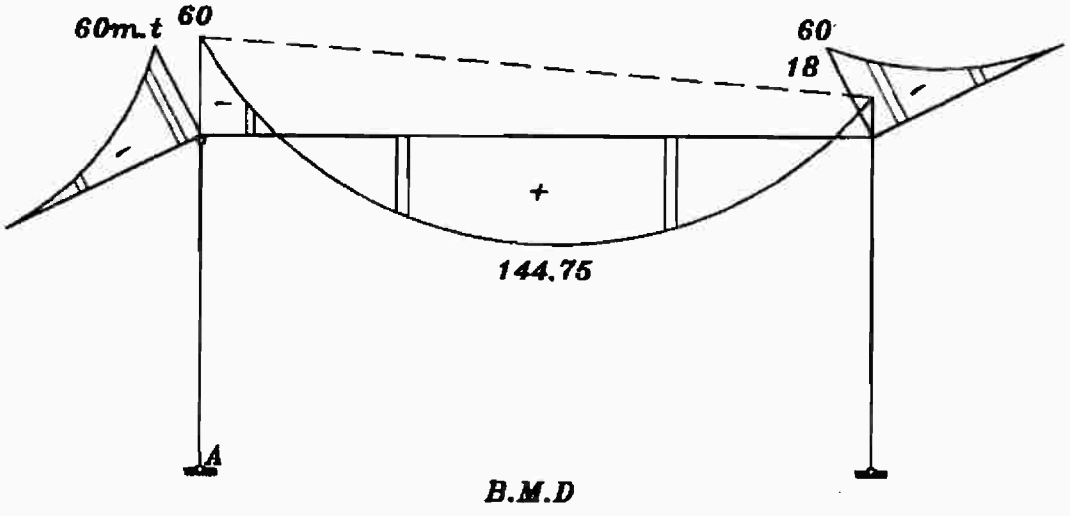
1 – Draw the straining actions (B . M. , NF and S . F)
2 – Design all critical section.
3 – Check shear at section B in girder B.C.
4 – Give full details of reinforcement in elevation and cross section.



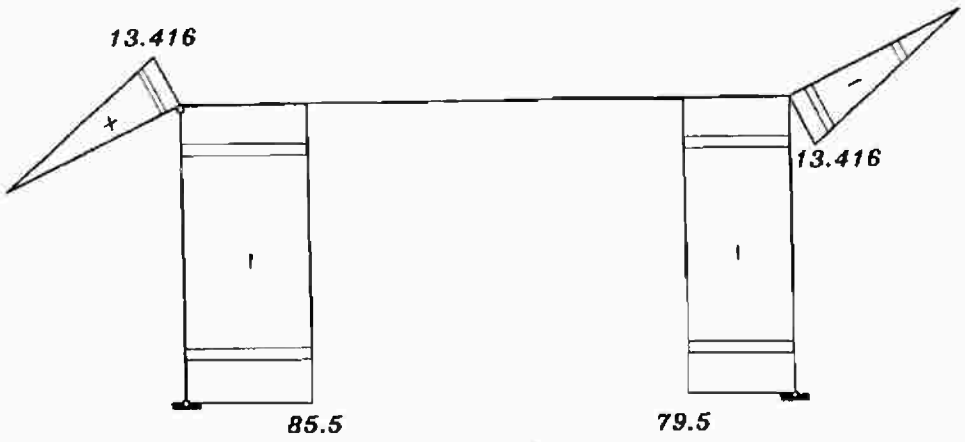
*Ultimate Load = 1.5*Service Load*



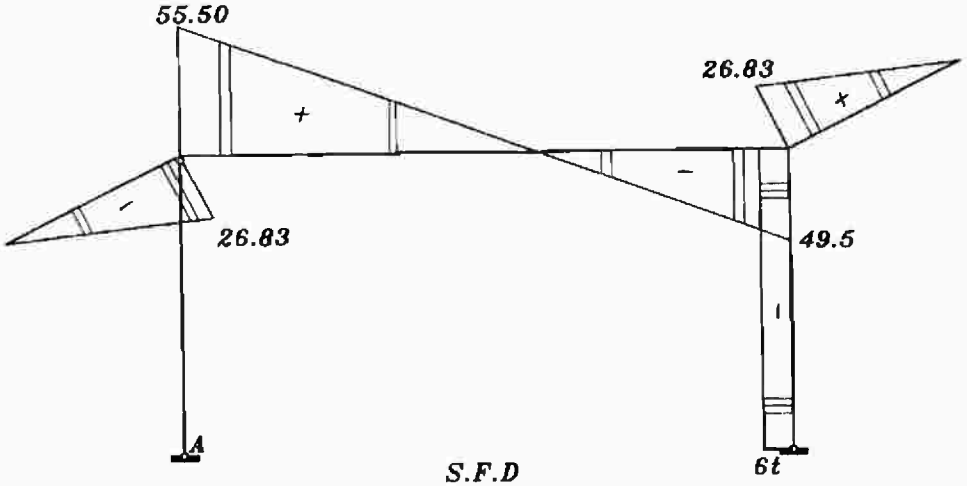
1) Straining Actions



B.M.D



N.F.D



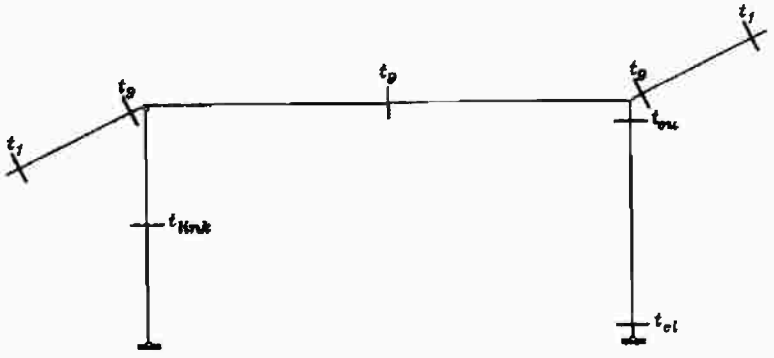
S.F.D

Dimensioning of the Frame

$$t_g = \frac{Span}{12\text{-}16} = \frac{1400}{12} = 117cm \simeq 120cm$$

$t_{cu} = (0.8 \rightsquigarrow 1)\ t_g = 120cm$

$t_{cl} = 0.6 t_{cu} = 75cm$



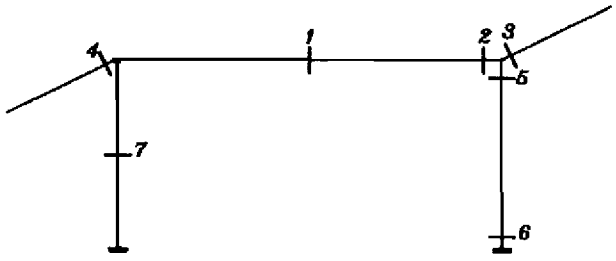
$t_1 \simeq 0.5\, t_g = 60$ cm

$t_{(link)}$ = larger of $\begin{cases} 0.4\, t_g = 48 \text{ cm} \\ L / 20 = 70 \text{ cm} \end{cases}$

$\therefore t_{(link)} = 70\ cm$

Design of critical Section:

Sec	M_u	N_u	Sec .type
1	144.75	--	t- sec
2	18	--	☐ ter
3	60	-13.42	☐ ter
4	60	+13.42	☐ ter
5	42	-79.5	☐ ter
6	--	-79.5	☐ ter
7	--	-85.5	☐ ter



Sec 1 – 1 : (T – sec) :

M_u 144.75 m.t & $N_u = 0.0$

B = Largest of $\begin{cases} 16\, t_s + b = 227 \text{ cm} \\ C_L \text{ to } C_L = 500 \text{ cm (spacing bet frames)} \\ \dfrac{0.7\, L}{5} + b = 231\ cm \end{cases}$

$\therefore B = 227\ cm$

Using (C – J) curve $d = C_1 \sqrt{\dfrac{M_u}{f_{cu} B}} \rightarrow C_1 = 7.2$

$\therefore \dfrac{C}{d} < \dfrac{C}{d_{min}} \rightarrow take \dfrac{C}{d} = \dfrac{C}{d_{min}} \rightarrow J = 0.826$

$A_s = \dfrac{M_u}{f_y J d} = 42.33 cm^2$ (10ϕϕϕ25)

Sec 2 – 2 : (□^{ker} _ sec) **(35 * 120)**

M_u = 18 m.t & N_u = 0.0

Using (C – J) curve :

$$d = C_1\sqrt{\frac{M_u}{f_{cu}\,b}} \rightarrow C_1 = 8.02$$

$$\therefore \frac{C}{d} < \frac{C}{d}_{min} \rightarrow use \frac{C}{d}_{min} = 0.125$$

$J = 0.826$

$$A_s = \frac{M_u}{f_y\,Jd} = 5.26\ cm^2$$

$1.3\ A_{rq} = 6.84\ cm^2$.

A_{smin} = The greater of

$$\frac{11}{f_y}bd = 12.3\ cm^2.$$

$\therefore A_s < A_{s\ min} \rightarrow use\ A_{s\ min}$ (4 ϕ25)

sec (3 – 3)

M_u = 60.00 m.t N_u = -13.44 ton

$$\frac{0.04\ f_{cu}\ A_c}{1000} = 42\ \text{tan} > N_u$$

$\therefore$ *Neglect* N_u

Using (R – W) curve :

$$R = \frac{M_u}{f_{cu} b d^2} = 0.0518 \rightarrow w = 0.064$$

$$A_s = wbd \frac{f_{cu}}{f_y} = 17.9\ cm^2 \qquad (7\phi 22)$$

sec (4 – 4) (35 * 120) (□ ^ler – sec)

M_u = 60 m.t

N_u = + 13.42 ton

$$e = \frac{M_u}{N_u} = 4.47 > \frac{(d - d^{-})}{2} \Rightarrow Large\ ecc$$

$$e_s = e - t/2 + cover = 4.47 - \frac{1.2}{2} + 0.05 = 3.92\ m$$

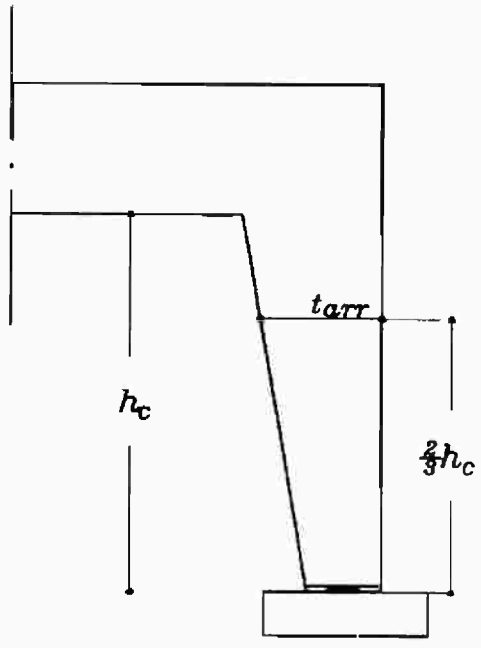
$$M_{us} = N_u - e_s = 52.6\ mt$$

Using (R – W) curve :

$$R = \frac{M_{us}}{f_{cu} b d^2} = 0.0455 \xrightarrow{chart} w = 0.057$$

$$A_s = wbd\ f_{cu} / f_y + \frac{N_u * 1000}{f_y / \gamma_s} = 20.22\ cm^2$$

use $(6\phi 25)$



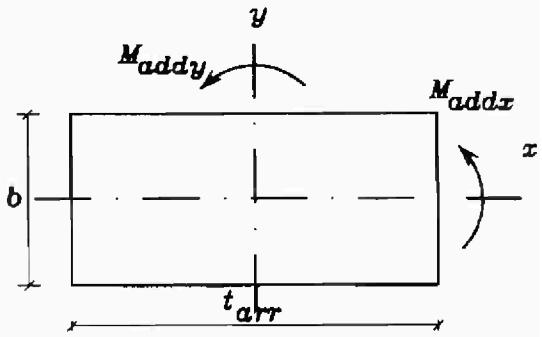
Sec (5-5) (35 * 120) column sec :

M_u = 42.00 m.t

N_u = -79.5 ton

Check additional moment due to buckling

In – plane Buckling:



$$H_o = 7 - \frac{1}{2}\, t_g = 6.40\ m$$

$$t_{avr} = 0.75 + [1.2 - 0.75] * \frac{2}{3} = 1.05\ m$$

* Top End Condition $t_g = t_c \rightarrow$ Case (1)

* bottom End condition hinged $\rightarrow$ Case (3)

$\because$ The frame is braced in both directions

$\therefore$ From table (6- 9)E.C.P $\rightarrow$ K = 0.9

$H_e = 0.9 * 6.40 = 5.76$ m

$$\lambda = \frac{H_e}{tav} = 5.49 < 15 \rightarrow short\ \ column$$

$\therefore$ No add. Moments due to buckling.

Out of plane Buckling:

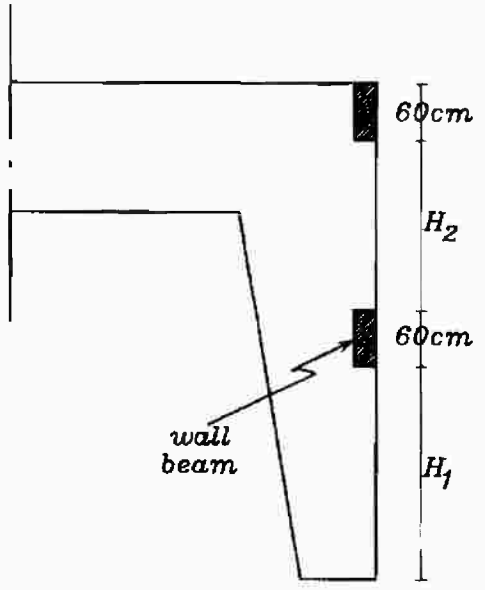
Assuming existence of wall beams at mid – height of frame connecting frames out of plane.

$H_1 + H_2 = 7 - 0.6 - 0.6 = 5.8$

Let $H_1 = 3\ m$ & $H_2 = 2.8\ m$

Top End condition $t_{beam} > b \rightarrow$ case (1).

Bottom End Condition case (1).



From table (6 – 9) E . C . P $\rightarrow$ K = 0.75

$H_e = 0.75 * 3 = 2.25$

$$\lambda = \frac{H_e}{b} = 6.43 < 15 \rightarrow Short\ column$$

$\therefore$ No add. Moment due to buckling

Design of sec (5 – 5) :

M_u = 42 m.t

N_u = - 79.5 ton .

Assuming tension failure

$$e = \frac{M_u}{N_u} = 0.53$$

$$e_s = e + t/2 - cover = 1.08$$

$$M_{us} = N_u . e_s = 85.86\ m.t$$

Use (R – W) cover :

$$R = 0.074 \quad \frac{table}{\alpha = 0.5} \qquad \therefore w = 0.09$$

$$\therefore A_s = wbd \frac{f_{cu}}{f_y} - \frac{N_u * 1000}{f_y / \gamma_s} = - v_e$$

∴ The failure is compression failure.

Using Interaction diagram (chart No . 25) ($\alpha = 0.6$)

$$K = \frac{P_u}{f_{cu} bt} = 0.076 \qquad \& \qquad K.\frac{e}{t} = 0.033$$

$$\rho < 1.00 \qquad \therefore take \quad \rho = 1.00$$

$$\mu = \rho\ f_{cu} * 10^{-5} = 0.0025 \qquad (for\ A_s\ only)$$

$$\mu_{min} = 0.006 \quad \rightarrow short\ column \qquad (for\ A_s + A_s^-)$$

$$\mu_{min} = \frac{0.006}{1.6} = 0.00375 \qquad (for\ A_s\ only)$$

$\therefore A_s = 0.00375 * bt = 15.75\ cm^2$ (5ϕϕ12)

$As^- = \propto A_s = 0.6 + 15.75 = 9.45\ cm^2$ (5ϕϕ22)

Sec (6 – 6) (35*75) column sec

N_u = -79.5 ton

M_u = 0.0

Short column:

$N_u * 1000 = 0.35\ f_{cu}\ A_c + 0.67\ f_y\ A_{sc}$

$A_{sc} = -V_w \rightarrow \therefore use\ A_{s\,min}$

$A_{s\,min} = 0.006\ bt = 15.75\ cm^2$ (6ϕϕ22)

$A_s = As^-$ = (3ϕϕ22)

sec (7 –7) (35 * 70) Link member

N_u = - 85.8 ton M_u = 0.0

Check moment due to buckling

In – Plane Buckling :

Top End condition hinged case (3)

Bottom End condition hinged case (3)

$\therefore$ from table (6 – 9) → K = 1

$$H_o = 7 - \frac{1}{2} t_g = 6.40\ m$$

$$H_e = K.H_o = 6.40\ m$$

$$\lambda = \frac{H_e}{t} = \frac{6.40}{0.7} = 9.14 \times < 15$$

∴ Short Column in t – direction (No – add . moment)

out of plane Buckling :

Assume that awall beam 25 * 60 is used at mid height of link .

So buckling out of plane will be safe and No additional moment (as in sec 5 – 5)

Design as short column:

$N_u * 1000 = 0.35\ f_{cu}\ A_c + 0.67\ f_y\ A_{sc}$

$A_{sc} = -\ v_e$

∴ use $A_{s\ min} = 0.006\ A_c = 14.7\ cm^2$ (14ϕ12)

Check of shear:

$Q_u = 55.50$ ton

Critical section at d/2 from the link face

$$Q_{u\ design} = Q_u - w_u\left[\frac{Link\ \dim}{2} + \frac{d}{2}\right] = 55.5 - 7.5\left[\frac{0.7+1.15}{2}\right] = 48.56\ t$$

$$q_u = \frac{Q_{u\ design}}{bd} = 12.065\ kg/cm^2$$

$$q_{cu} = 0.75\sqrt{\frac{f_{cu}}{\gamma_c}} = 9.68\ kg/cm^2$$

$$q_{cu\ max} = 2.2\sqrt{\frac{f_{cu}}{\gamma_c}} = 28.4 > 30\ kg/cm^2$$

$\therefore q_{cu} < q_u < q_{cu\ max}$

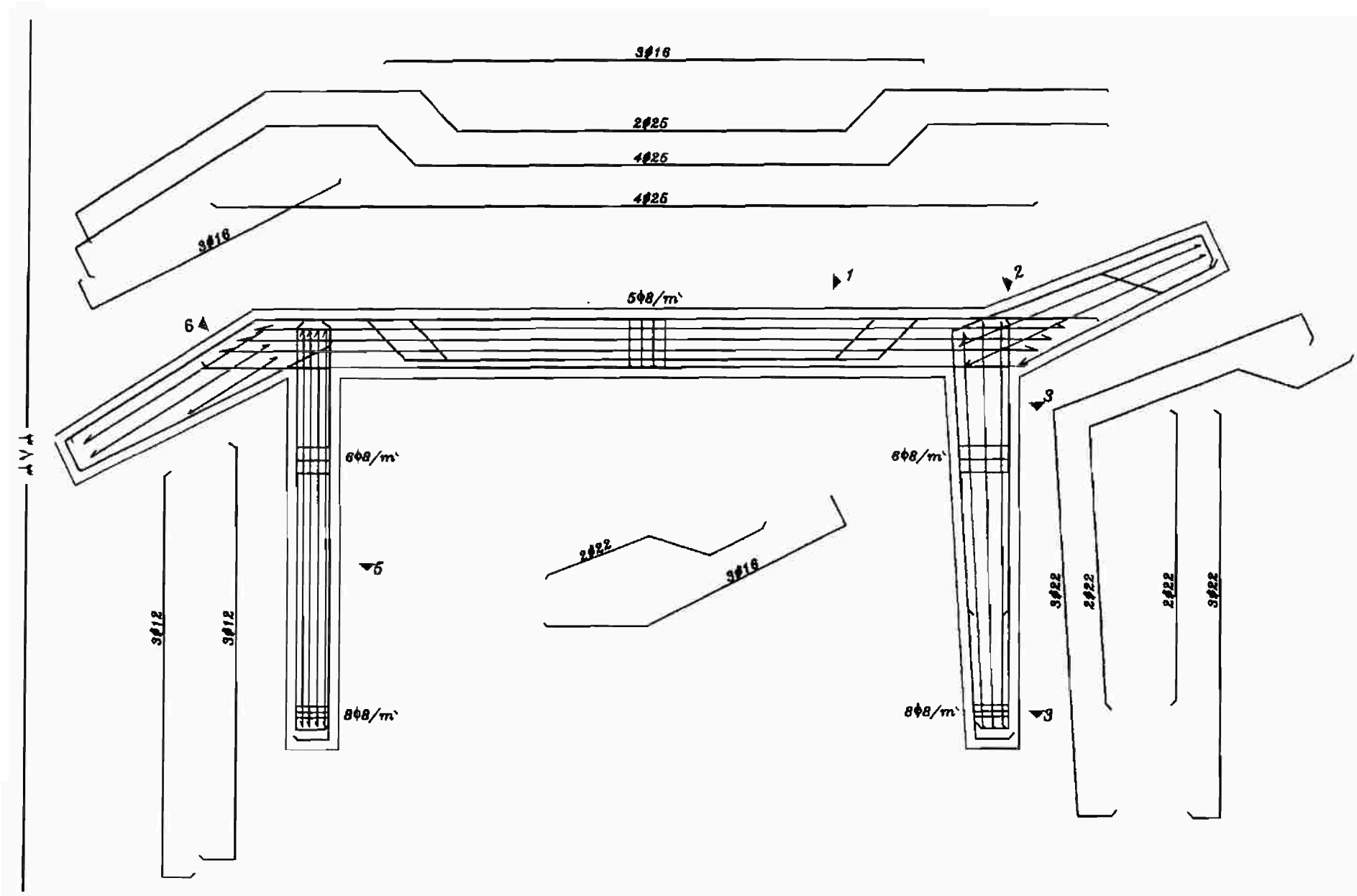
∴ shear reinforcement is needed .

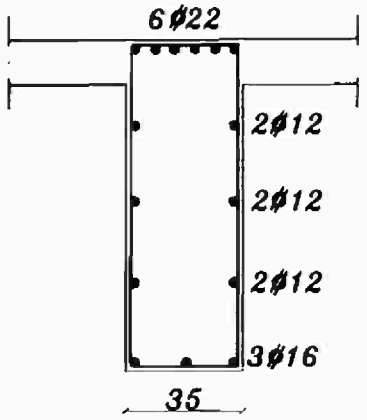
$q_{su} = q_u - 0.5\ q_{cu} = 7.22\ kg/cm^2$.

by using VL stirrups only

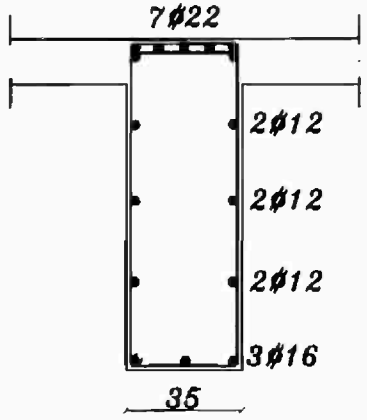
$$A_{st} = \frac{q_{su}\,b\,s}{f_{y\ st}/\gamma_s} = \frac{7.22\times 35\times 10}{2800/1.15} = 1.038\ cm^2$$

use (10ϕ10/m^-)

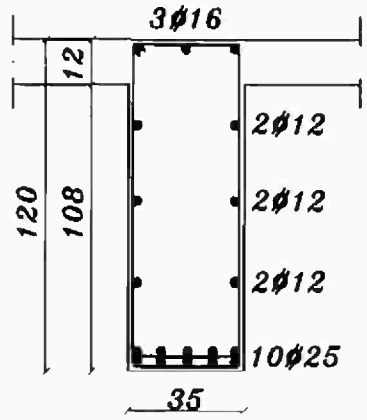

3Φ16
2Φ25
4Φ25
4Φ25
3Φ16
5Φ8/m`
1
2
6
3
6Φ8/m`
6Φ8/m`
2Φ22
3Φ16
3Φ12
3Φ12
5
8Φ8/m`
8Φ8/m`
3
3Φ22
2Φ22
2Φ22
3Φ22



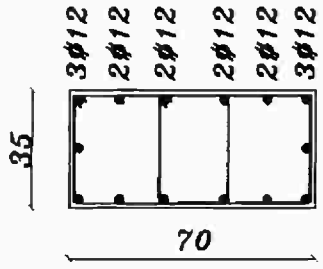
Sec. (6)

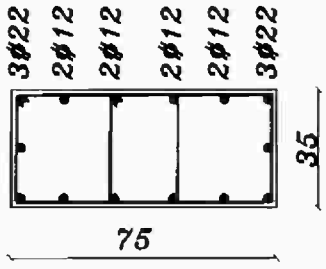


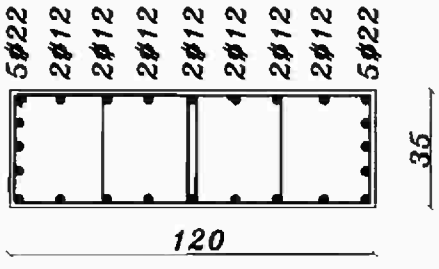
Sec. (2)



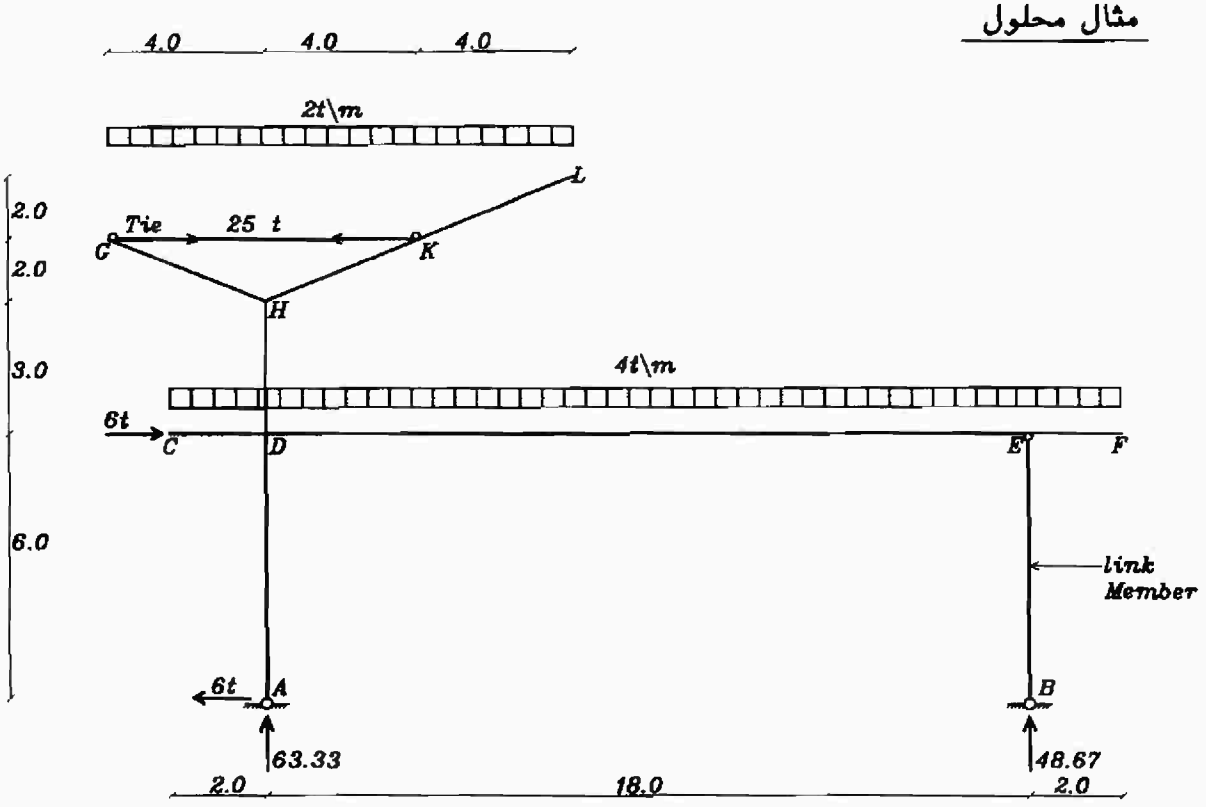
Sec. (1)

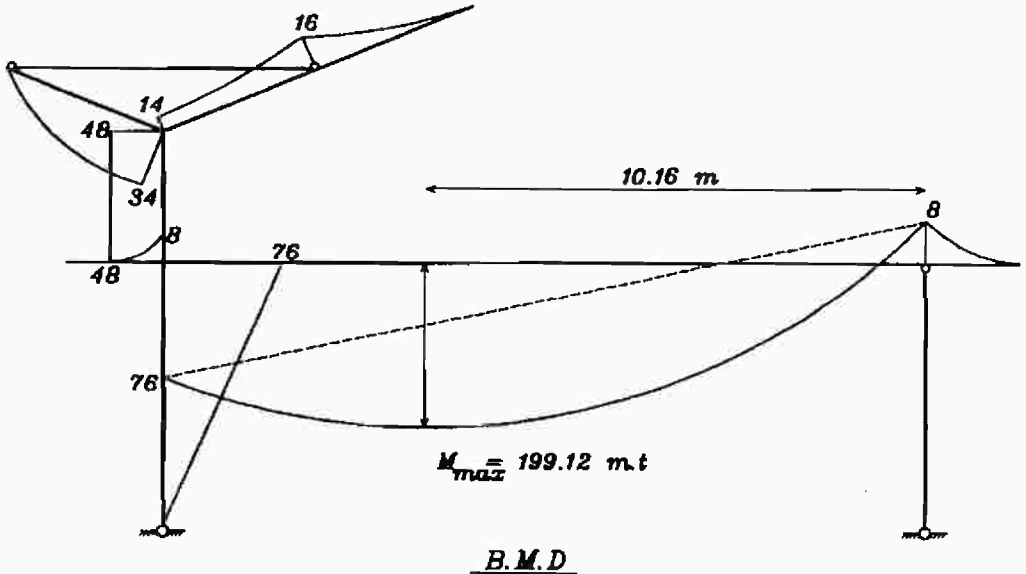




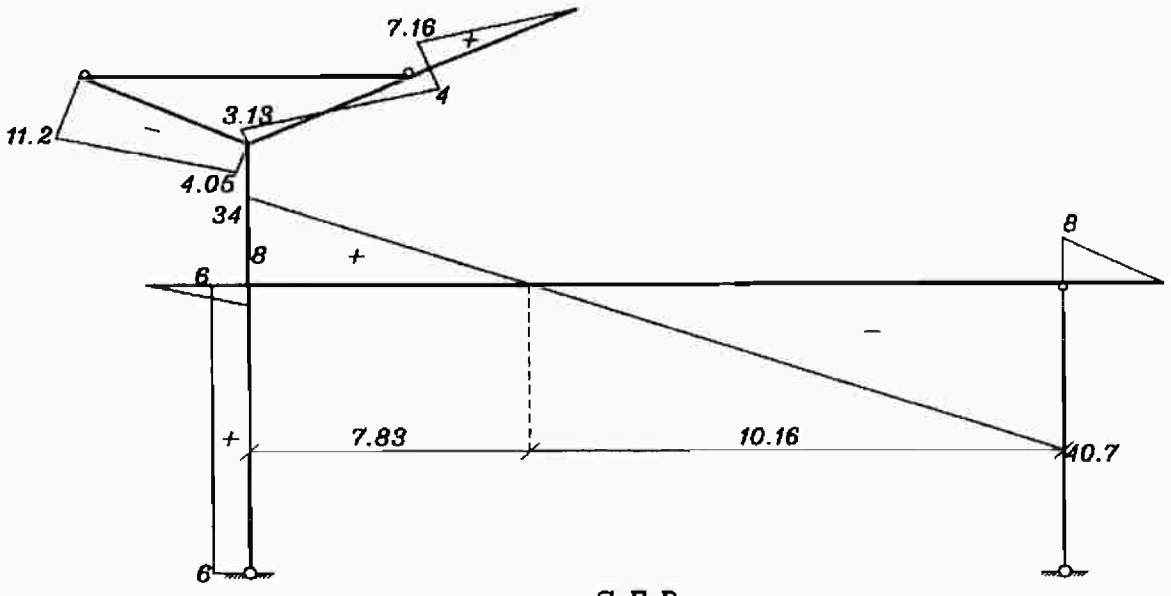


مثال محلول

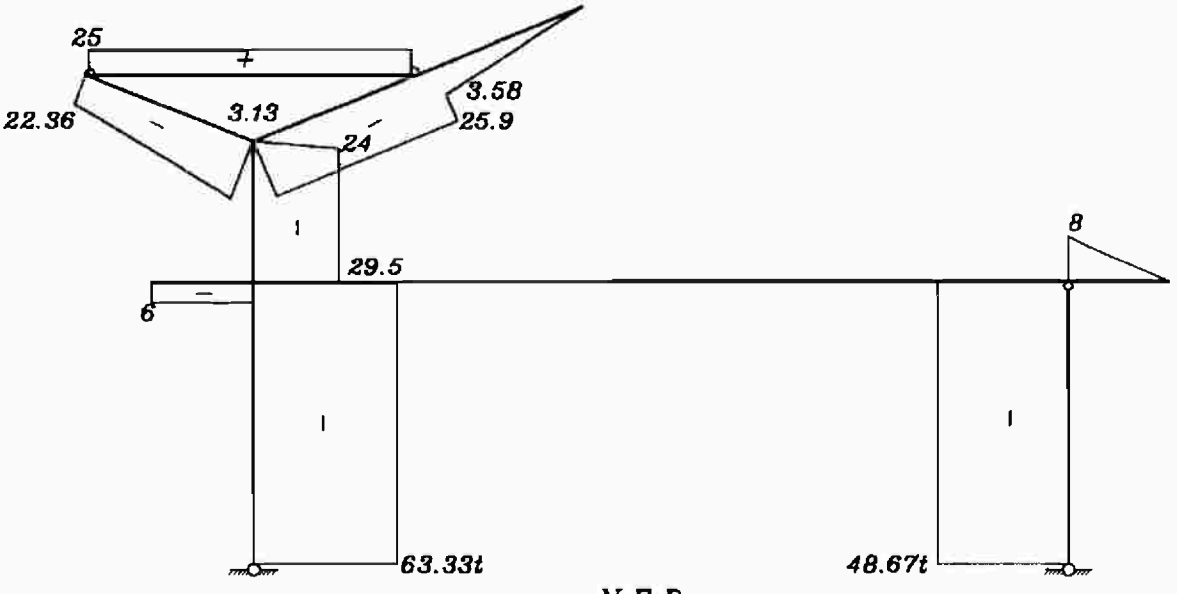




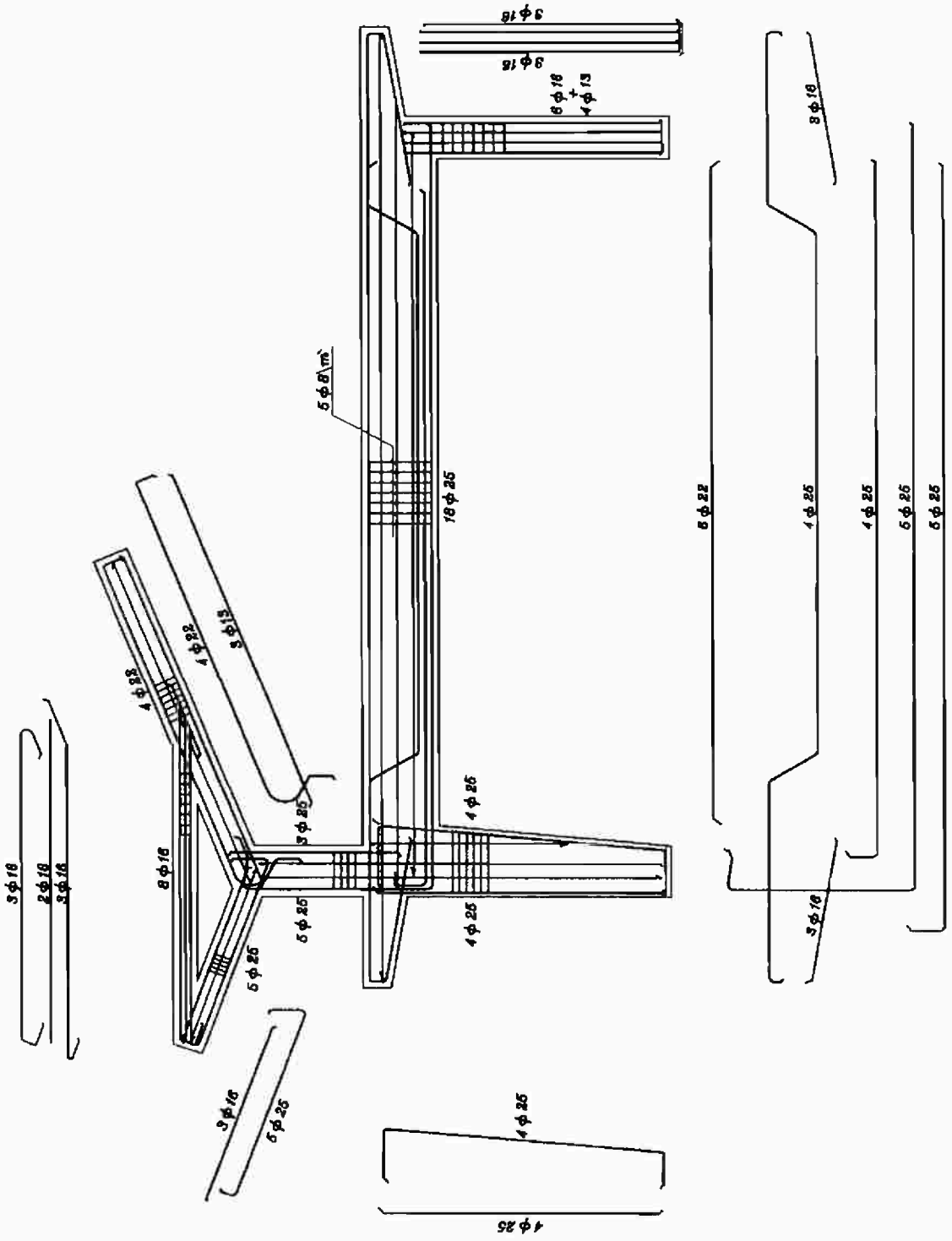
B.M.D

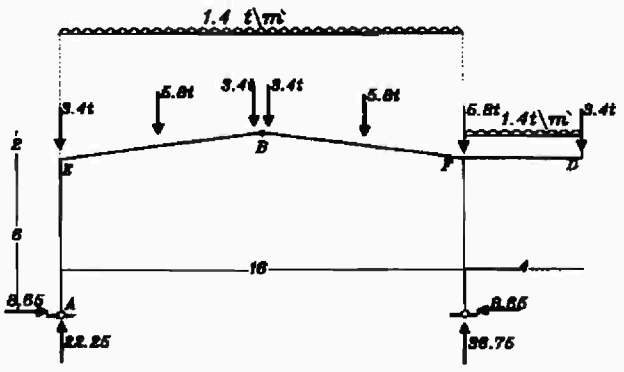

7.16
+
4
3.13
11.2
−
4.06
34
8
+
8
6
−
+
7.83
10.16
40.7
6

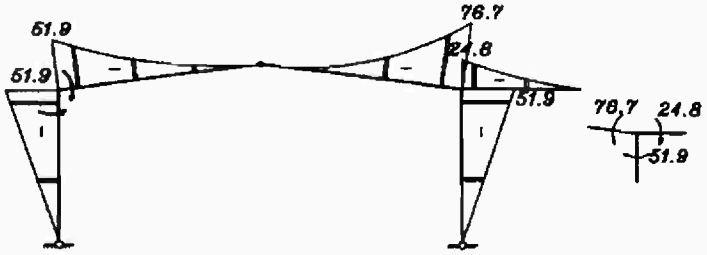
S.F.D


25
+
3.58
22.36
3.13
−
25.9
24
−
1
8
29.5
6
−
1
1
63.33t
48.67t

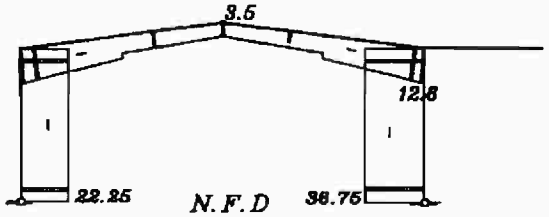
N.F.D


3 φ 18
3 φ 18
6 φ 18 + 4 φ 13
5 φ 8 m'
18 φ 25
3 φ 18
5 φ 22
4 φ 25
4 φ 25
5 φ 25
5 φ 25
3 φ 18
4 φ 22
4 φ 22
3 φ 13
3 φ 25
4 φ 25
8 φ 18
5 φ 25
4 φ 25
5 φ 25
3 φ 18
2 φ 18
3 φ 18
3 φ 18
5 φ 25
4 φ 25
4 φ 25

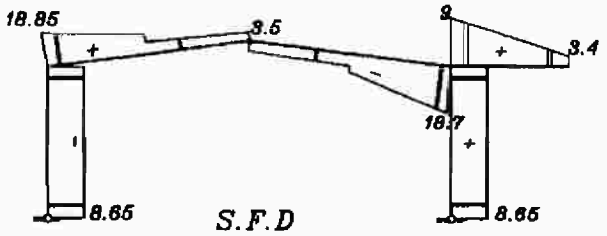




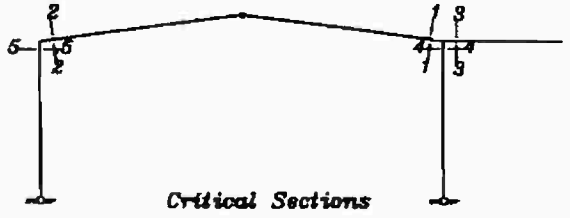
B.M.D



N.F.D

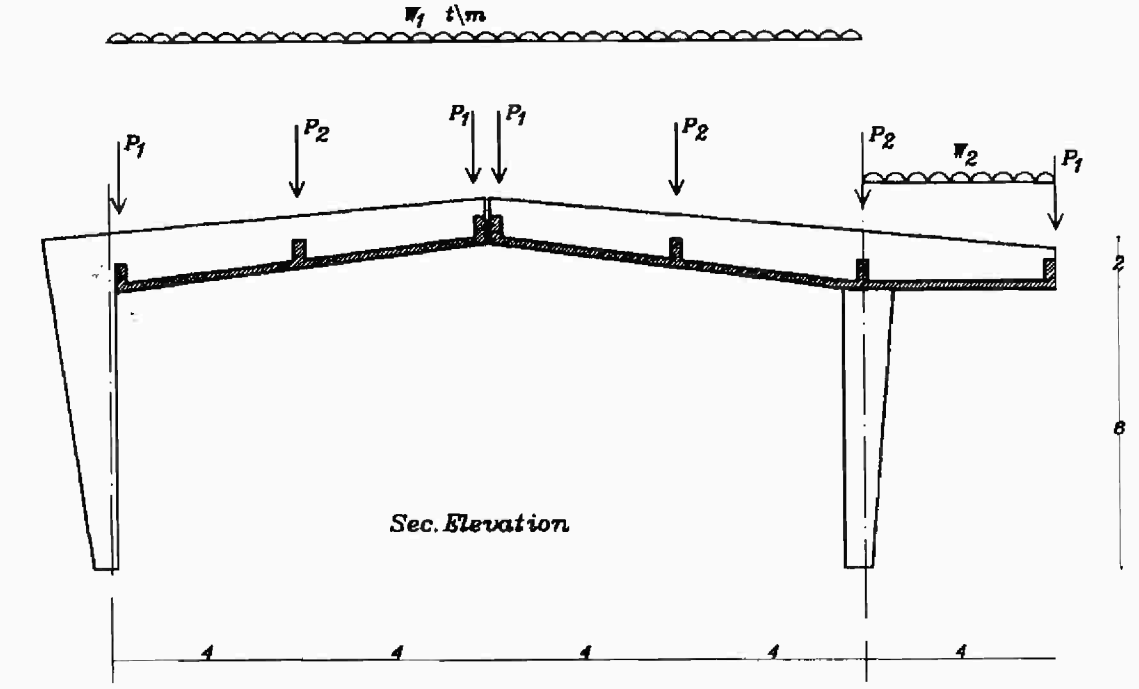


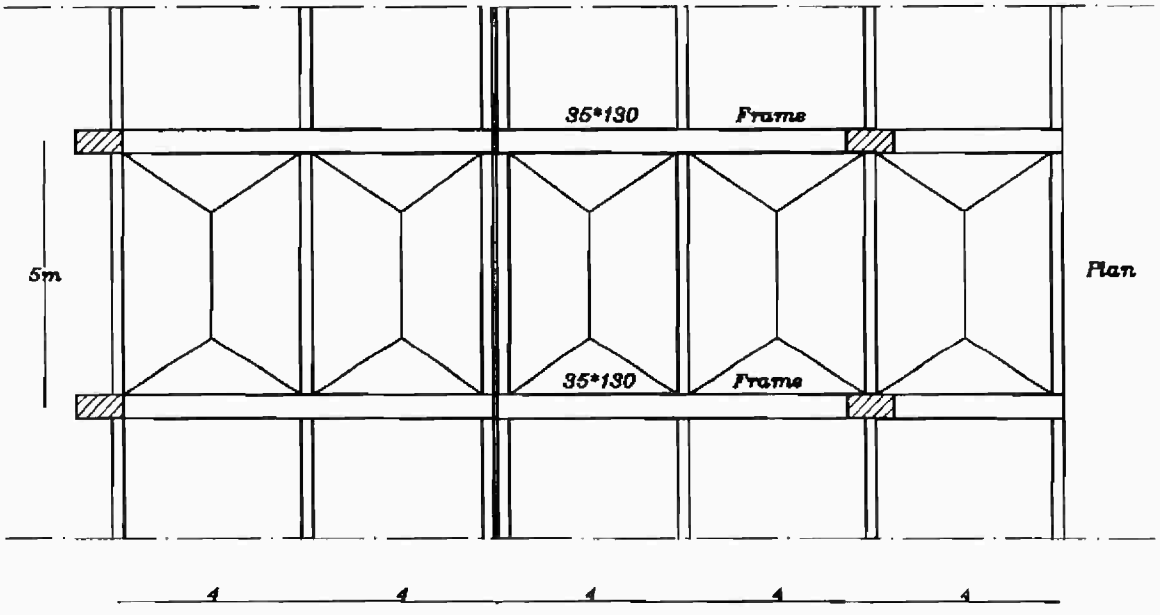
S.F.D

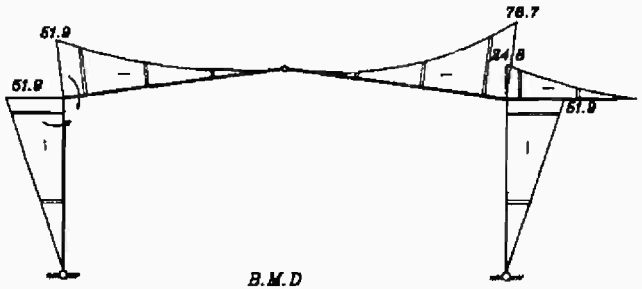


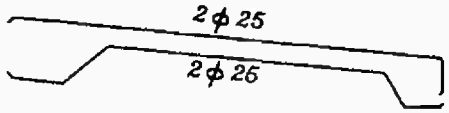
Critical Sections

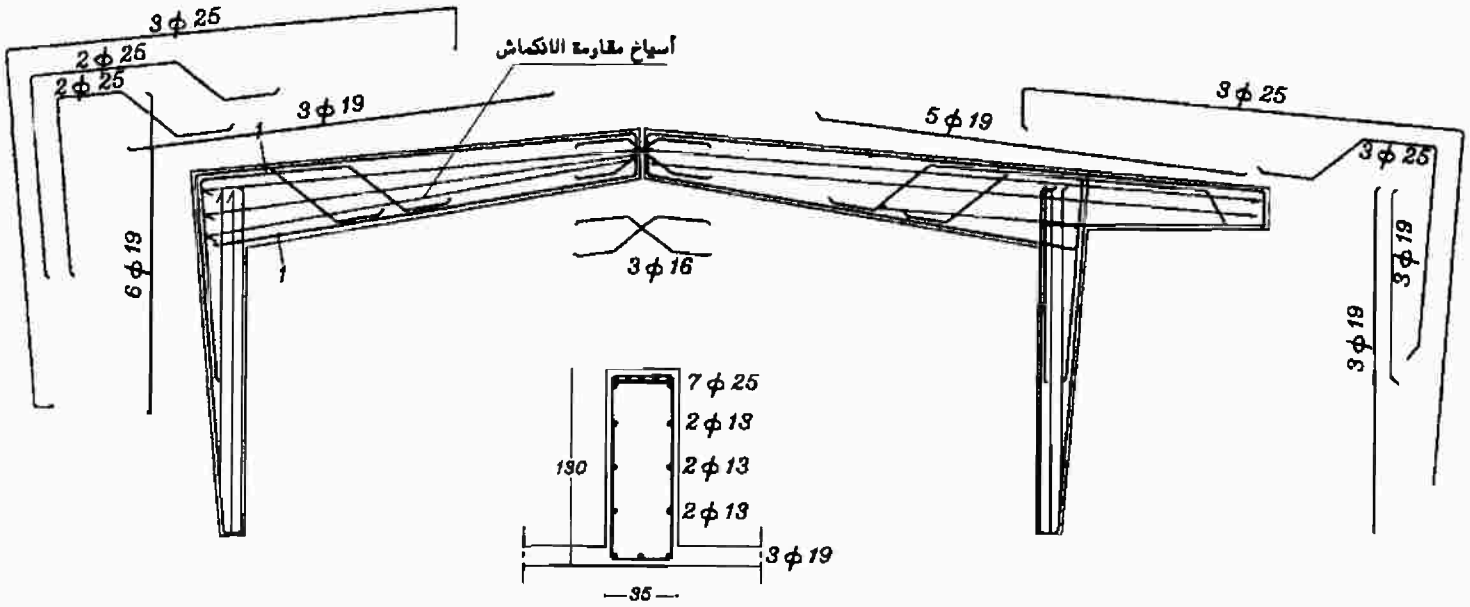
3 – hinged Frame





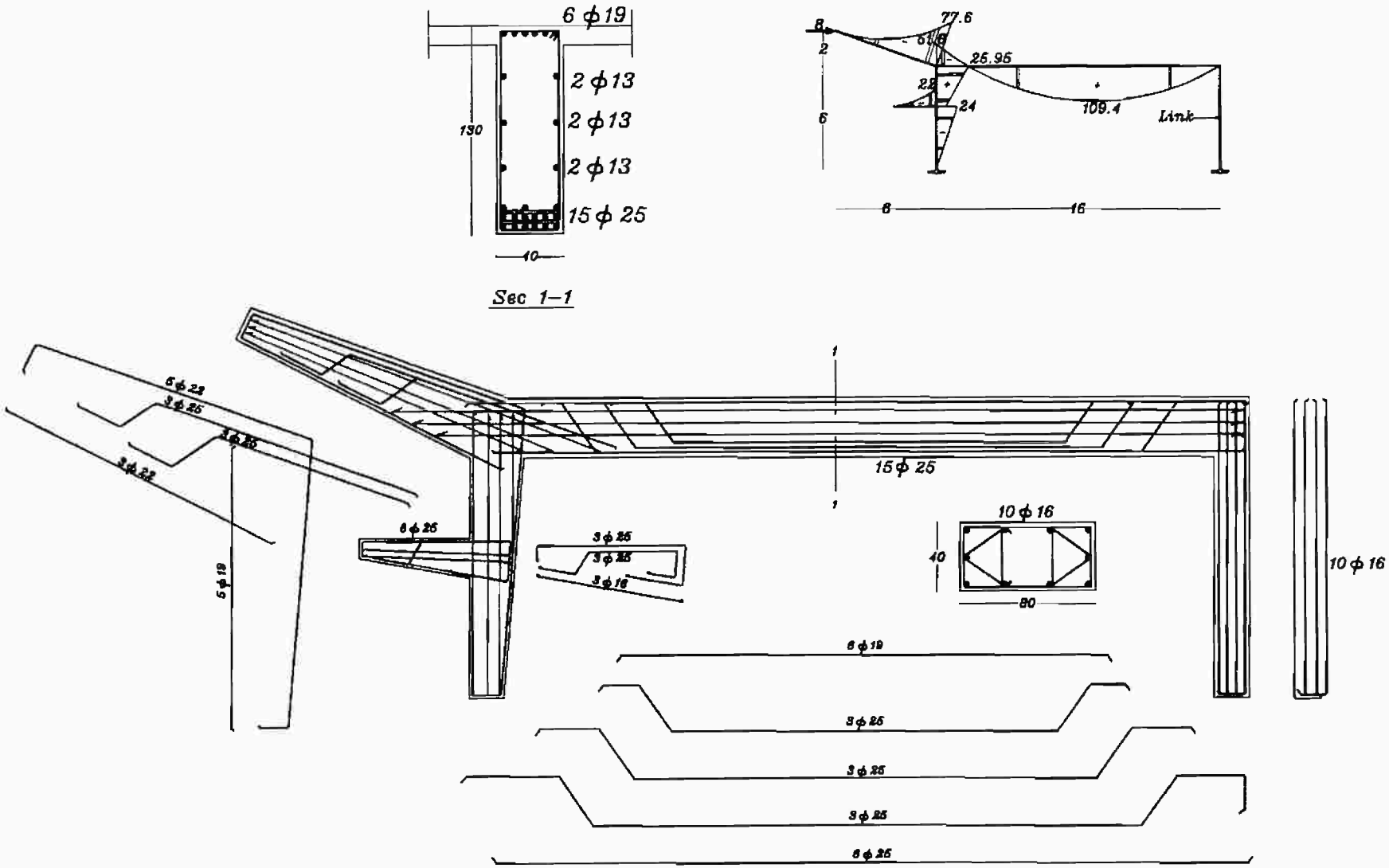

51.9
78.7
24.8
51.9
51.9
B.M.D


2 ϕ 25
2 ϕ 26


3 ϕ 25
2 ϕ 26
2 ϕ 25
3 ϕ 19
أسياخ مقاومة الانكماش
5 ϕ 19
3 ϕ 25
3 ϕ 25
6 ϕ 19
3 ϕ 16
3 ϕ 19
3 ϕ 19
7 ϕ 25
2 ϕ 13
190
2 ϕ 13
2 ϕ 13
3 ϕ 19
35

Sec 1–1

مثال محلول



6 ϕ19
2 ϕ13
2 ϕ13
2 ϕ13
15 ϕ 25
130
40
Sec 1–1
77.6
25.95
109.4
Link
8
2
6
22
24
16
1
1
15 ϕ 25
10 ϕ 16
40
80
10 ϕ 16
5 ϕ 22
3 ϕ 25
3 ϕ 25
3 ϕ 22
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3 ϕ 25
3 ϕ 16
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3 ϕ 25
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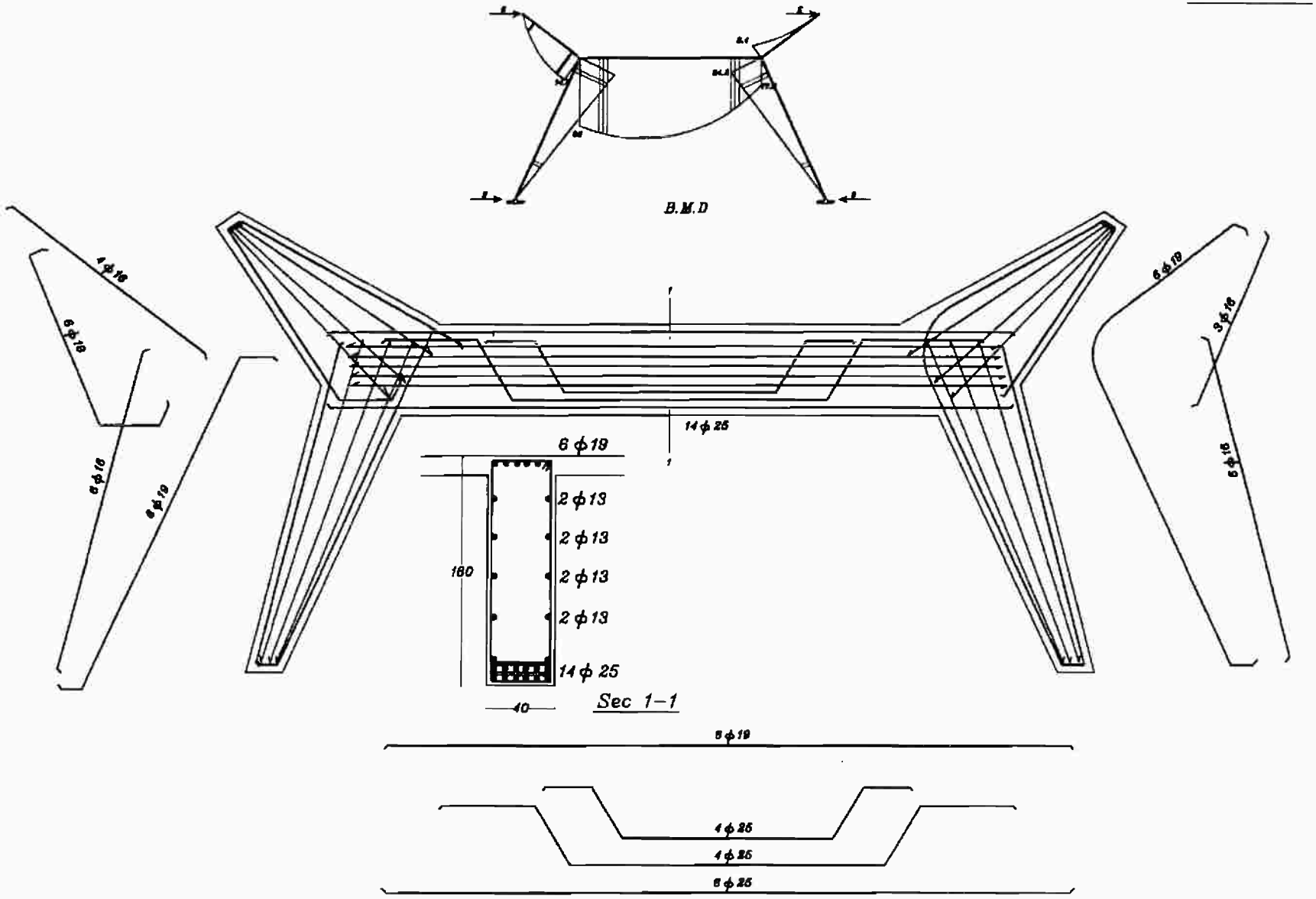
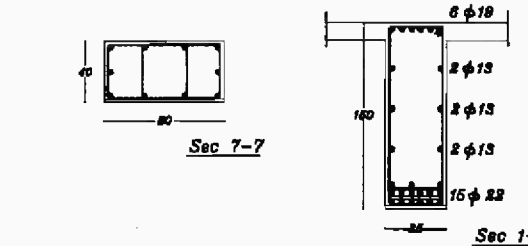
3 ϕ 25
6 ϕ 25


B.M.D
4 ϕ 16
6 ϕ 19
6 ϕ 16
6 ϕ 19
6 ϕ 19
3 ϕ 16
6 ϕ 16
14 ϕ 25
6 ϕ 19
2 ϕ 13
2 ϕ 13
2 ϕ 13
2 ϕ 13
160
14 ϕ 25
40
Sec 1–1
6 ϕ 19
4 ϕ 25
4 ϕ 25
6 ϕ 25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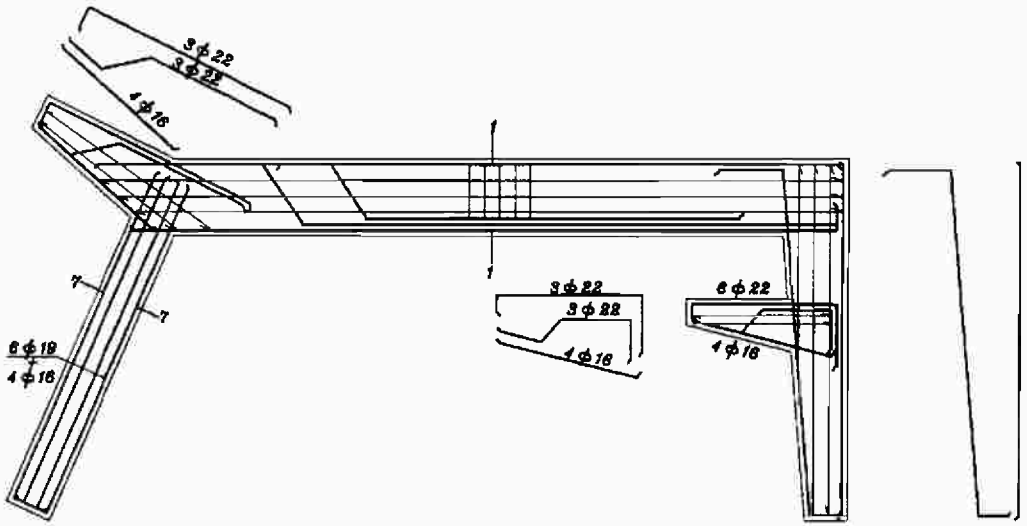
2 ϕ 13
2 ϕ 13
2 ϕ 13
15 ϕ 22
Sec 7-7
Sec 1-1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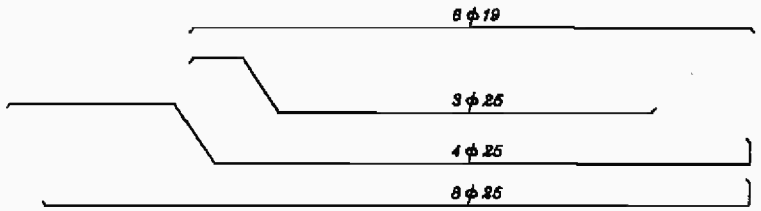
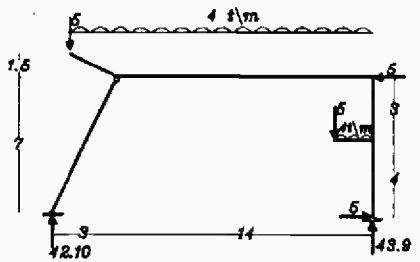
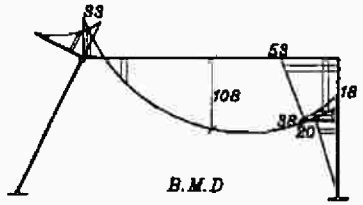
3 ϕ 22
4 ϕ 16
6 ϕ 19
4 ϕ 16
3 ϕ 22
3 ϕ 22
4 ϕ 16
6 ϕ 22
4 ϕ 16


6 ϕ 19
3 ϕ 25
4 ϕ 25
8 ϕ 25


4 t\m
1.5
7
3
4
3
14
43.9
12.10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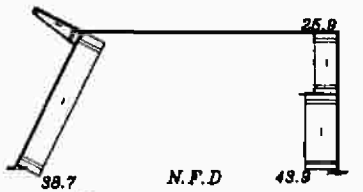
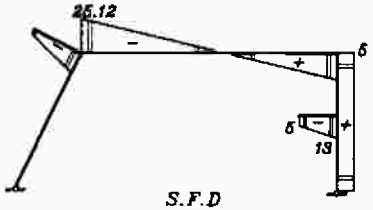
53
108
18
38
20
B.M.D


25.9
38.7
N.F.D
43.9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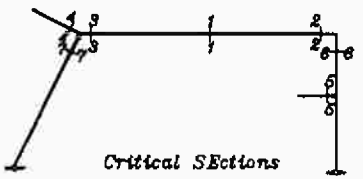
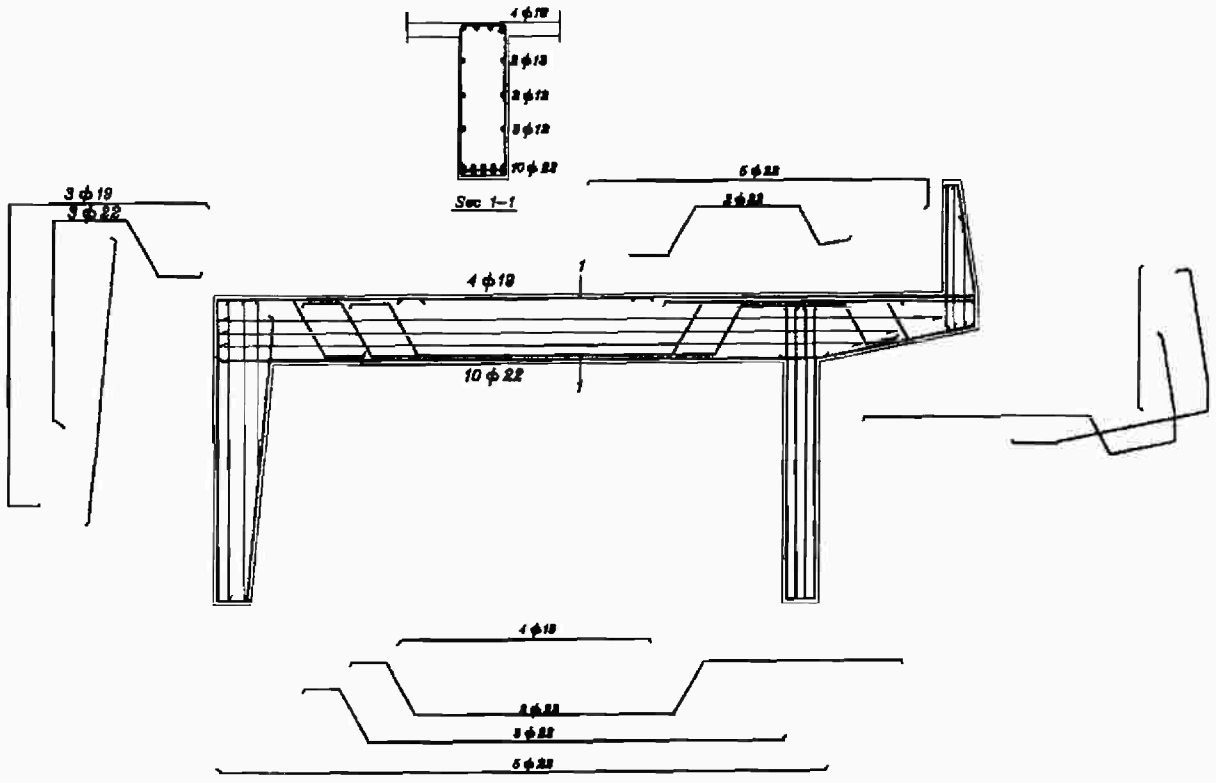
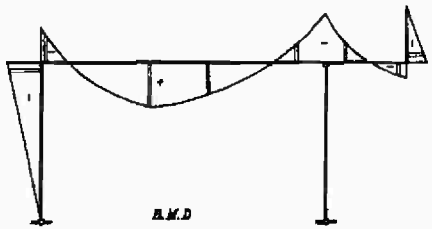
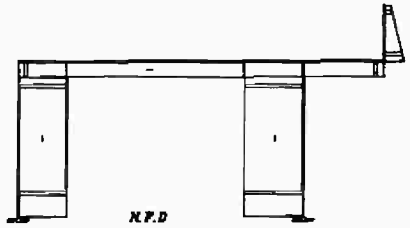

Critical SEctions

مثال محلول


4 ϕ 19
3 ϕ 19
3 ϕ 22
Sec 1–1
5 ϕ 22
4 ϕ 19
10 ϕ 22
4 ϕ 19
5 ϕ 25


B.M.D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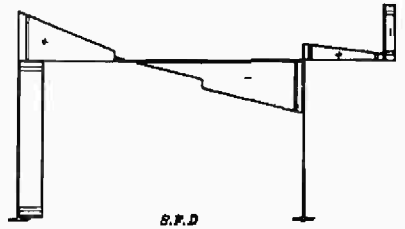
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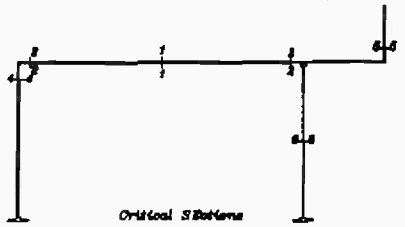
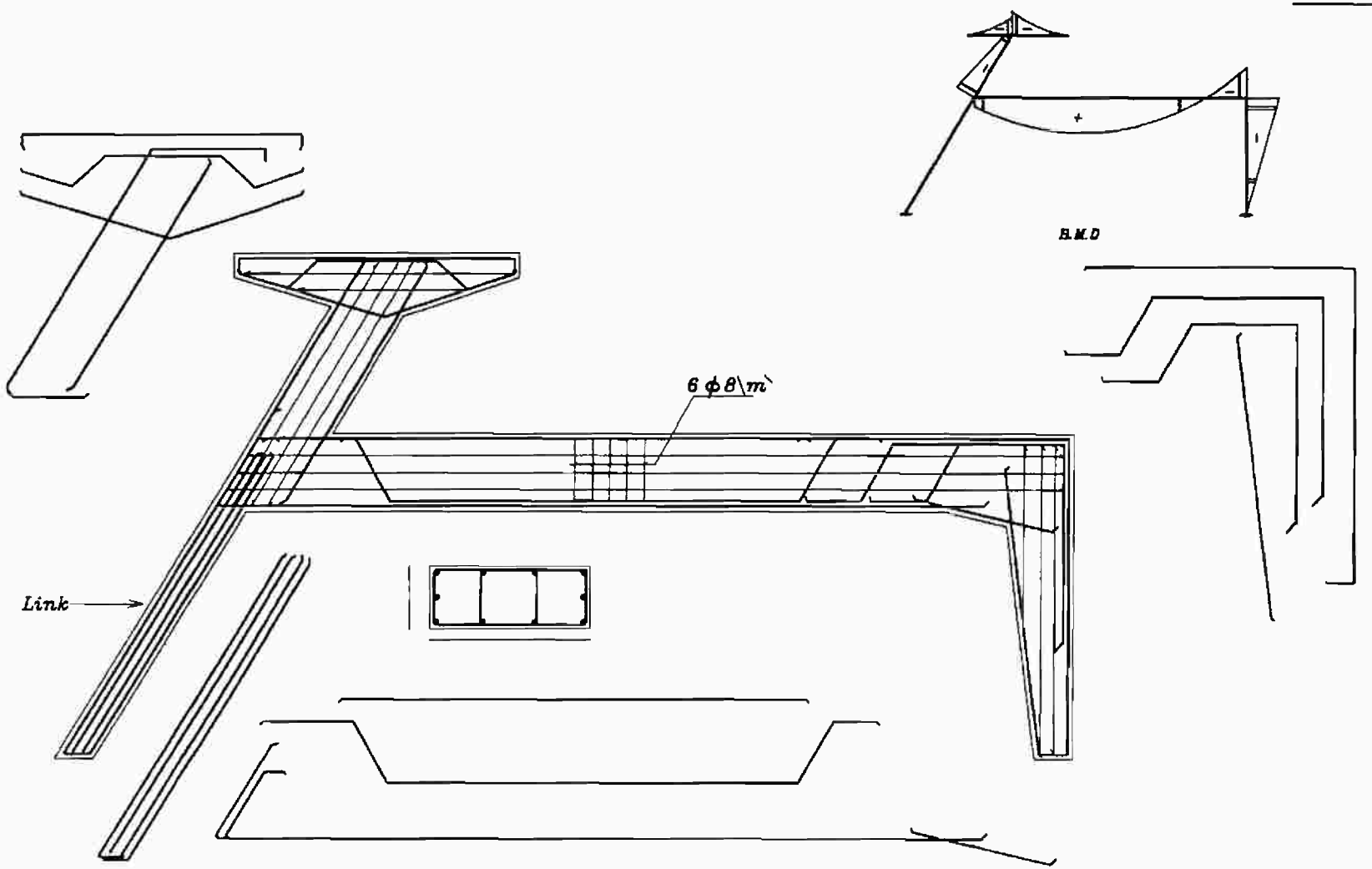

Critical Sections

مثال محلول