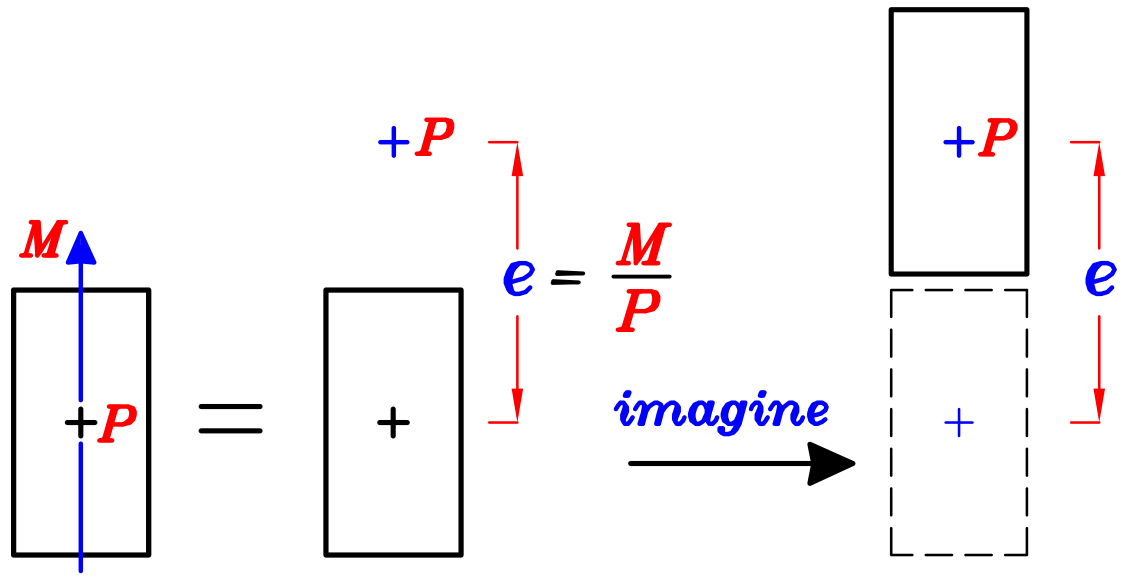
Parabolic Slab

Arch Slab

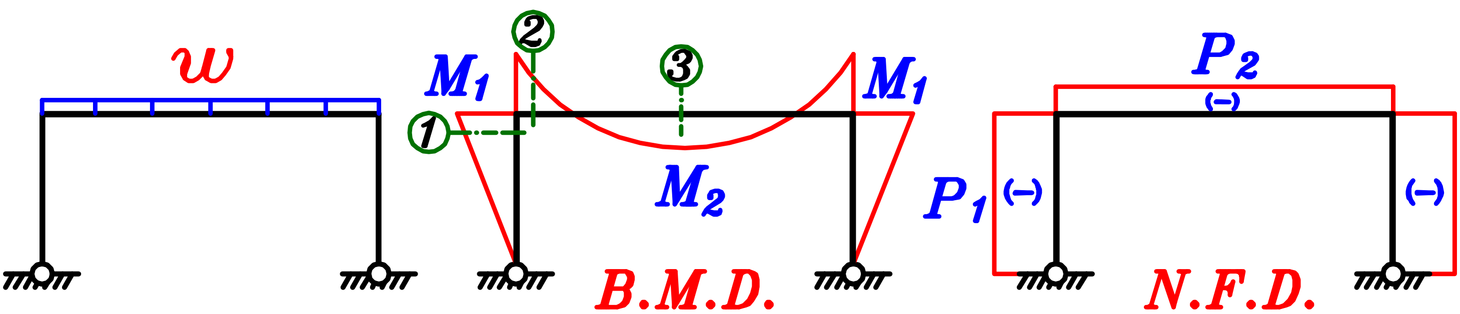
Introduction.

Thrust Line. (Pressure Line).

للقطاعات المؤثر عليها M , P اذا تخيلنا أنه تم ترحيل القطاع مسافه e عكس اتجاه ال moment سيكون القطاع المرحل عليه Normal Force فقط وبالتالى عند تصميمه سيحتاج ابعاد قطاع اقل و كميه حديد تسليح اقل .



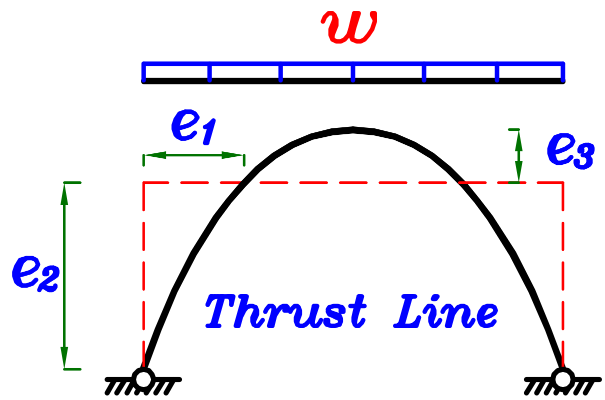
اذا استطعنا لاى structure ان نرحل كل قطاعاته عكس اتجاه ال moment مسافه e سنضمن ان ال structure الجديد كل قطاعاته سيؤثر عليها Normal Force فقط.
و بالتالى تكون ابعاد قطاعاته و كميات حديد تسليحه اقل فتكون تكلفته أقل .
و يسمى ال structure الجديد Thrust Line أو Pressure Line .



Sec. 1 $\quad e_1 = \dfrac{M_1}{P_1}$

Sec. 2 $\quad e_2 = \dfrac{M_1}{P_2}$

Sec. 3 $\quad e_3 = \dfrac{M_2}{P_2}$



المنشأت التى شكلها نفس شكل (Thrust Line)

و لان فى هذه المنشأت تكون قيمه (axial Force) تقريبا ثابته على جميع القطاعات .

أى أن $\left(e = \frac{M}{P} = \frac{M}{constant} \right)$

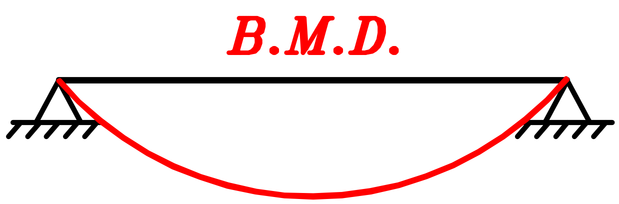
لذا اذا رسمنا شكل ال (structure) عكس شكل ال (B.M.D.) يكون هو نفسه

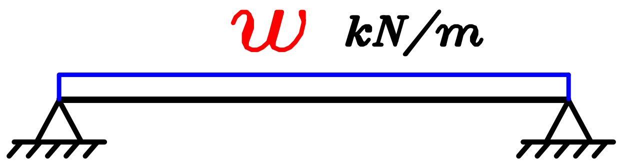
شكل ال (Thrust Line) أى لا يكون عليه (Bending moment)

و لكن يؤثر عليه فقط (axial Force) .

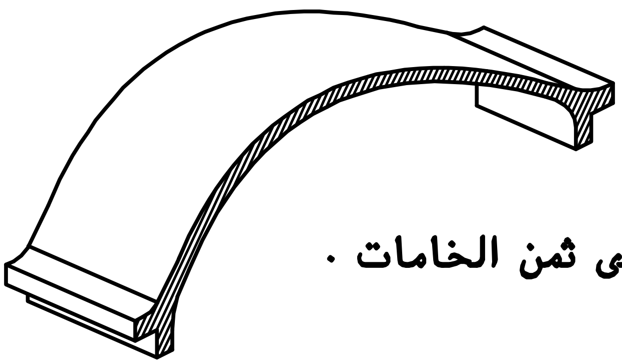
و هذه تعتبر ميزه اقتصاديه لان هذا يوفر فى كميات كلا من الخرسانه و حديد التسليح .

لان البلاطه تحمل احمال منتظمه فيكون شكل ال (Bending moment) عباره عن parabola





فيفضل اخذ البلاطه parabola و لكن لاعلى لكى يكون عكس ال B.M.D.



حتى يكون على البلاطه compression فقط

و يكون deflection البلاطه أقل بكثير

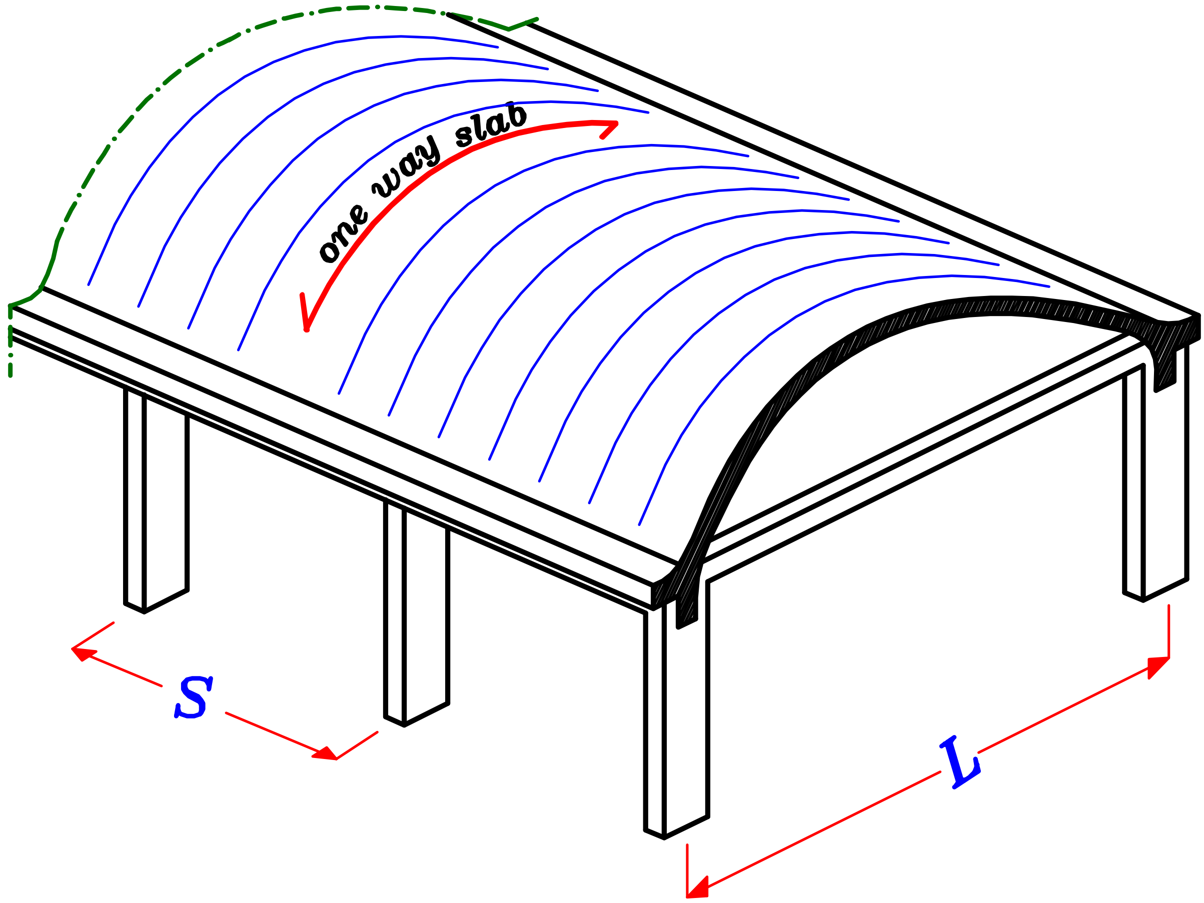
فتكون القطاعات و التسليح أقل و بالتالى أوفر فى ثمن الخامات .

ملحوظه parabolic slabs تكون فى الاسطح النهائيه فقط و ليست فى الادوار المتكرره .

ملحوظه

لان الاحمال على ال parabolic slab قليله فيكون ال tension على ال tie نسبياً قليل

لذلك ممكن للتسهيل اهمال ال extension of tie .



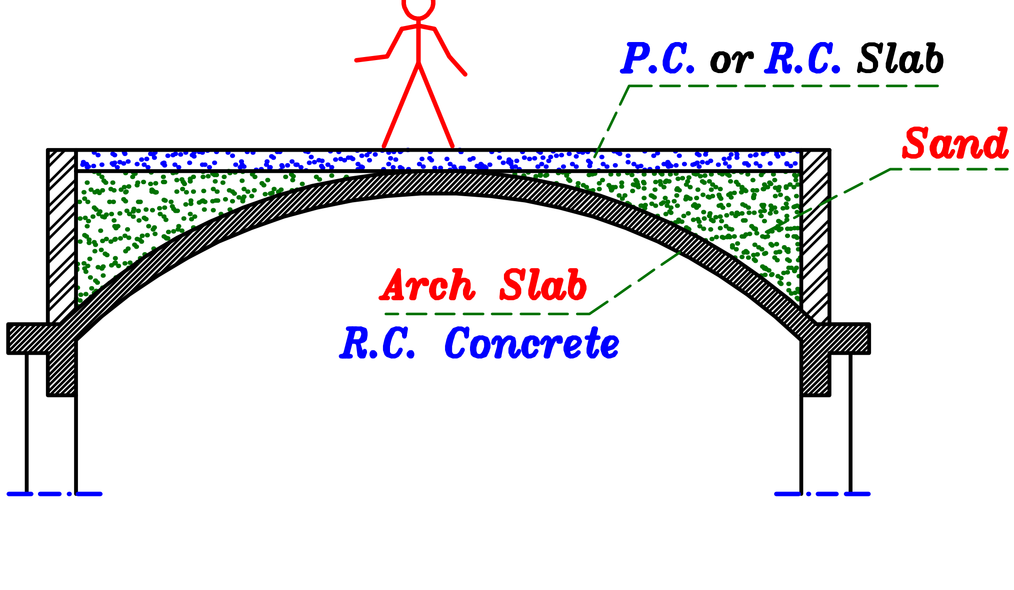
هى عباره عن بلاطه solid و تكون one way لانها محموله على كمرتين فقط.

مميزاتها : لان شكلها عكس ال bending moment
لا يكون عليها moment و يكون عليها compression Force فقط
و لا يكون لها deflection مما سيؤدى عند التصميم الى ان تكون
كميات الخرسانه و الحديد المطلوبين قليله أى تكون البلاطه أرخص .

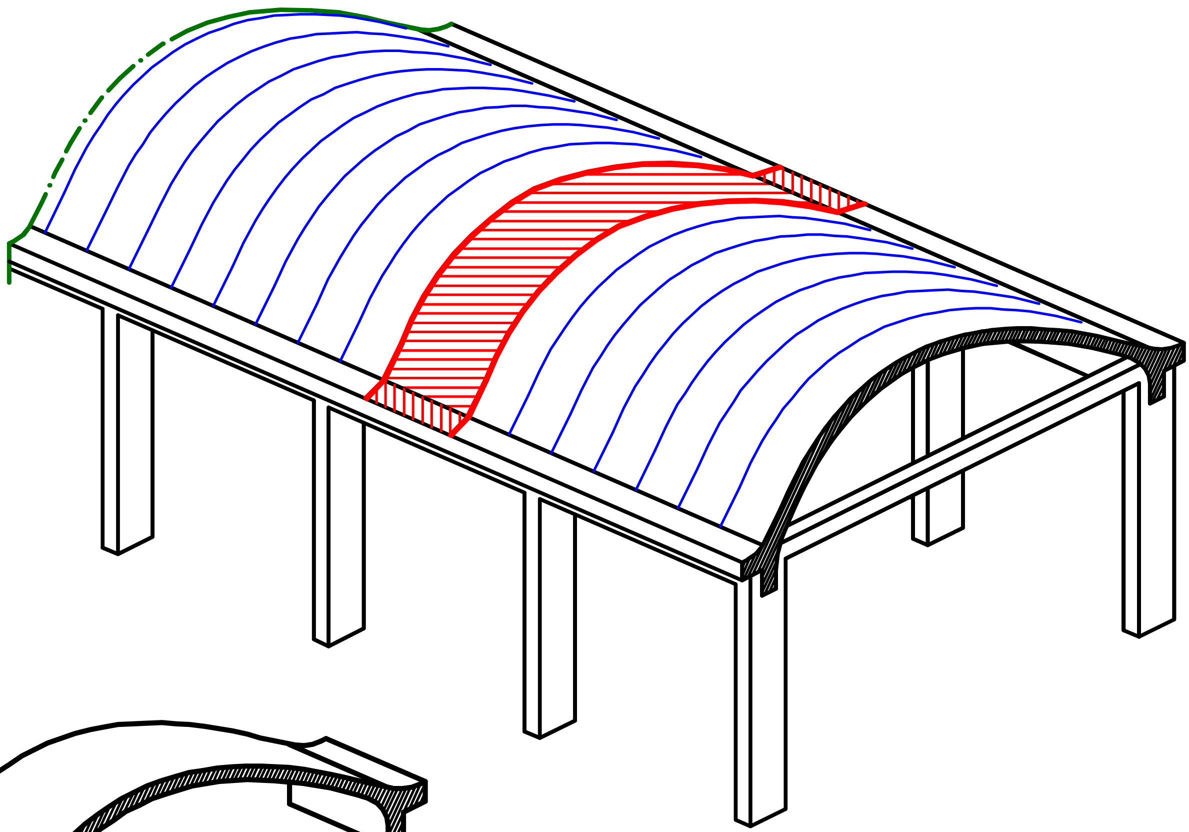
عيوبها : ١ـ تكون الشده فى التنفيذ منحنيه و يكون الحديد منحنى مما يصعب عمليه التنفيذ .

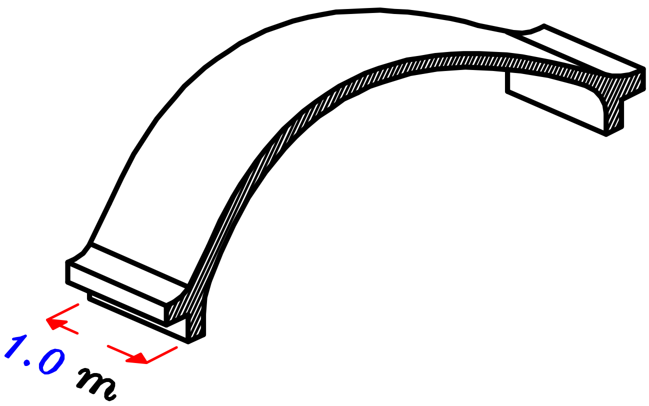
٢ـ يجب أن تكون دور أخير أى لن نستطيع عمل دور فوقها الا بشروط خاصه .

طريقه استخدام ال
parabolic slab
فى حاله الادوار المتكرره

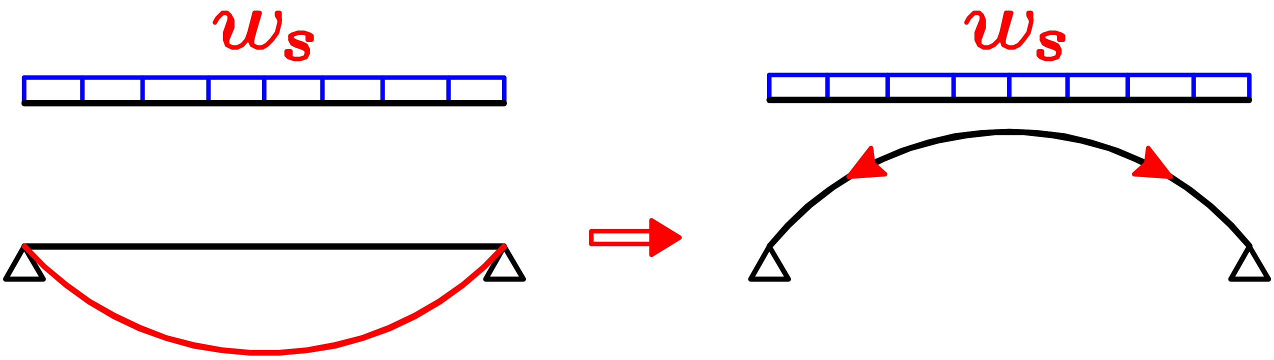


Concept of Parabolic Slab.





بأخذ شريحه فى البلاطه عرضها - ١,٠ م



و لان عاده البلاطات تكون الاحمال عليها *Distributed Loads*

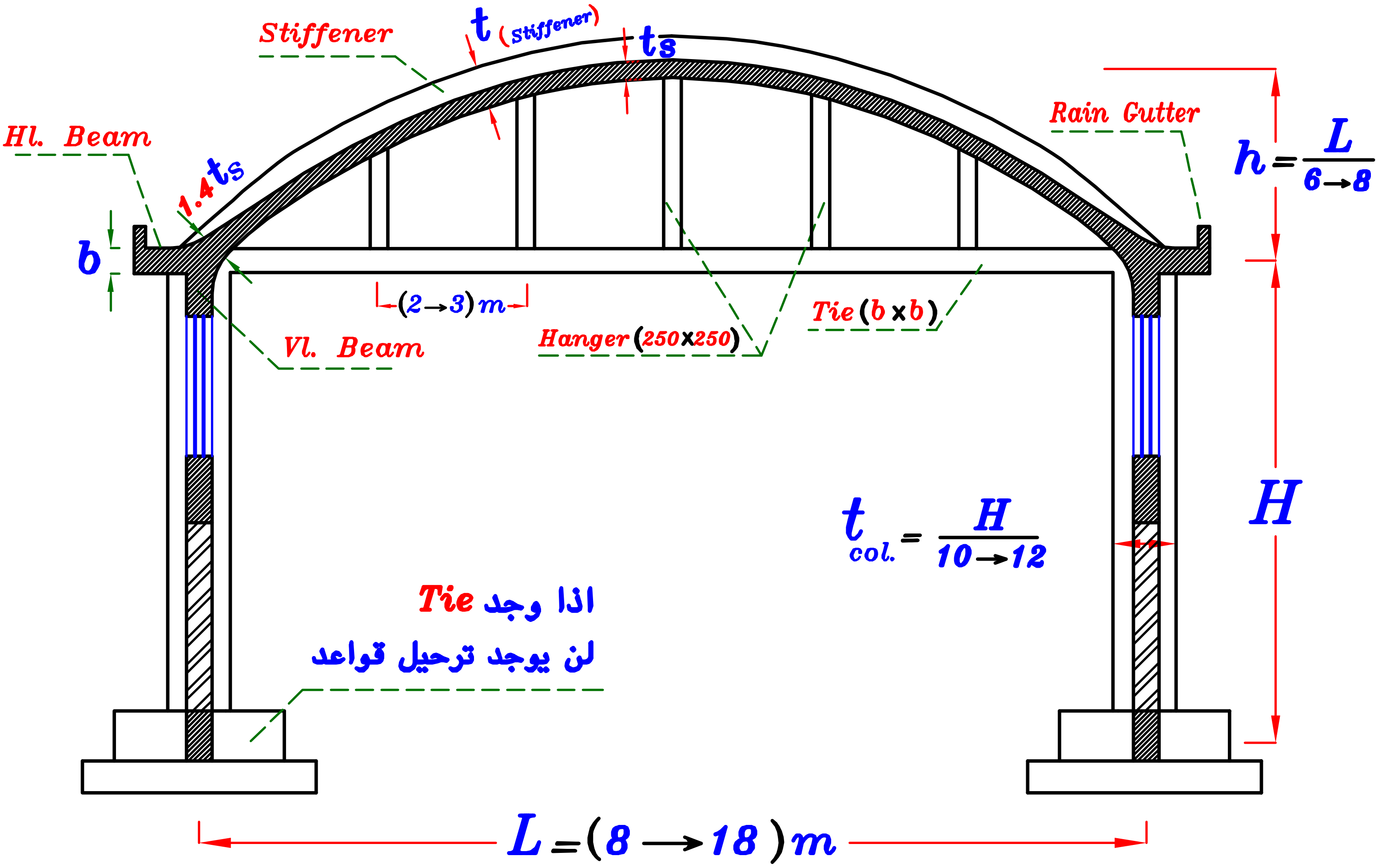
فسيكون الـ *moment* المفروض أن يحدث عليها شكله *Parabola* لاسفل

لذا اذا اخذنا شكل البلاطه *Parabola* لاعلى سيكون شكل البلاطه عكس الـ *moment*

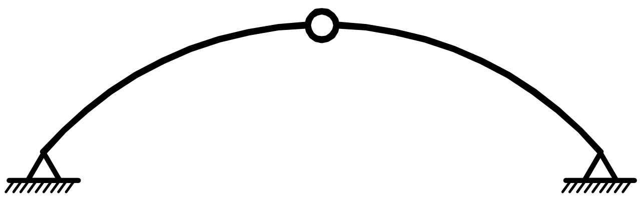
أى أن الشكل الحقيقى للبلاطه يجب ان يكون *Parabola* و ليس *Arch*

أى أن معادلته $\boxed{Y = a X^2 + b\ X + c}$

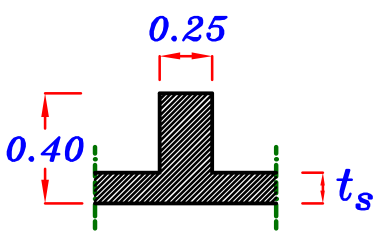
Concrete Dimensions.



* Span $(L) = (8 \rightarrow 18)$ m
* Hieght $(h) = \dfrac{L}{6 \rightarrow 8}$
* $t_s = (8 \rightarrow 14)$ cm.
* b = width of HL. Beam
  $= (0.25$ OR $0.30)$ m
* Tie $(b \times b)$
* Hanger (250×250)
* $t_{col.} = \dfrac{H}{10 \rightarrow 12}$
* Stiffener (250×400)

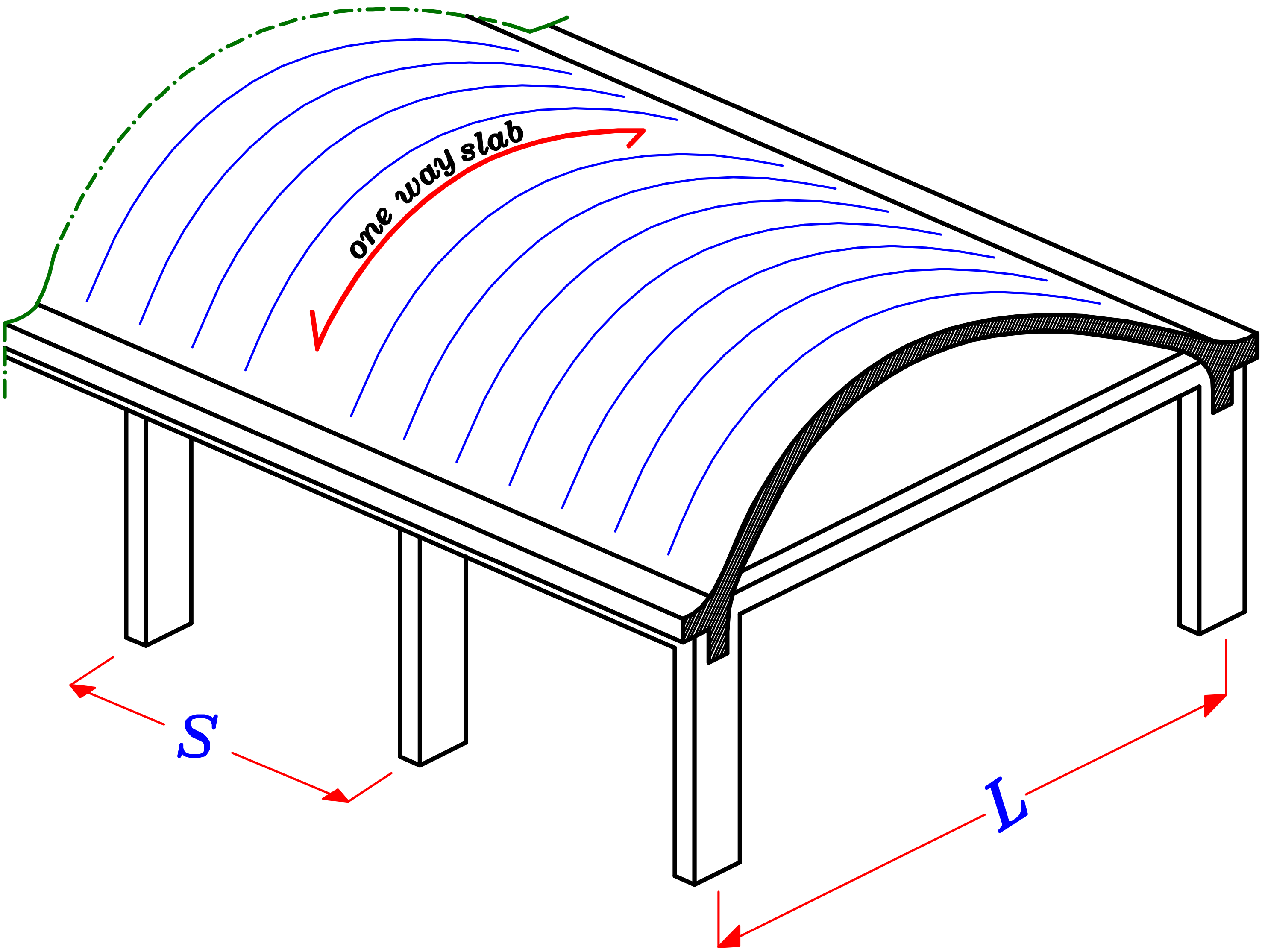


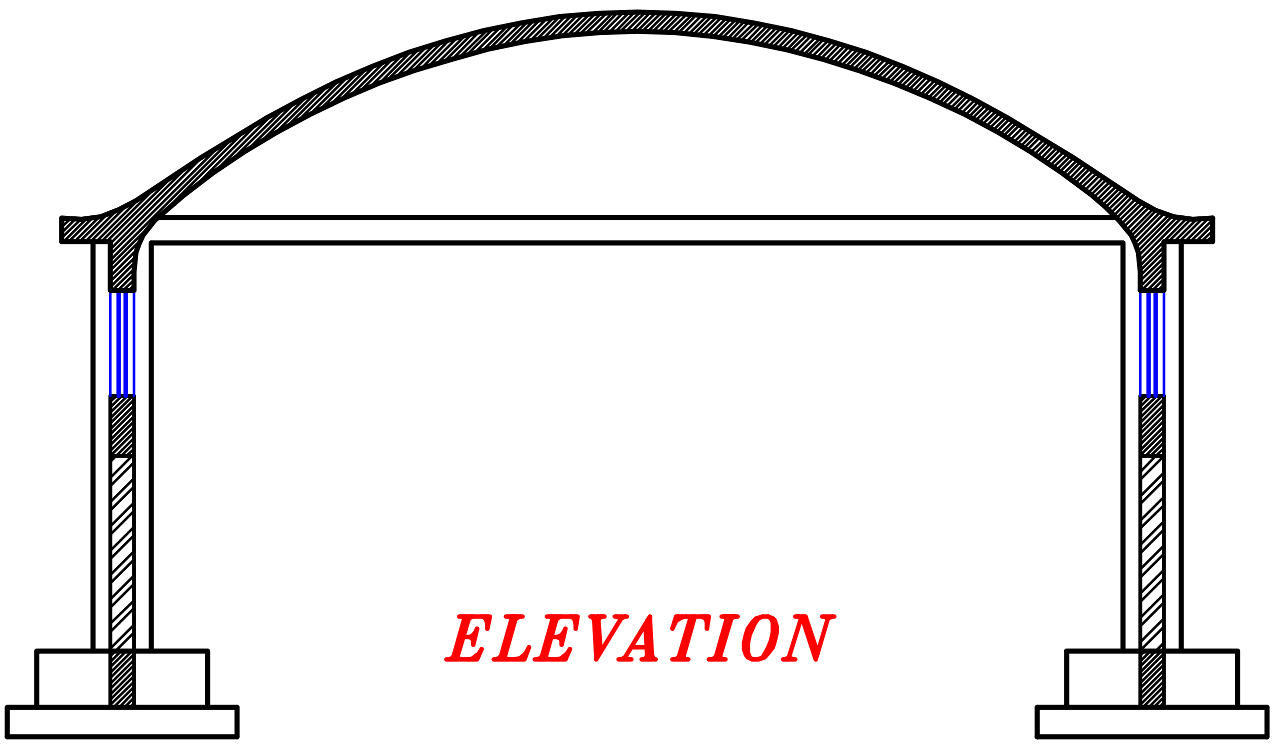
Statical System



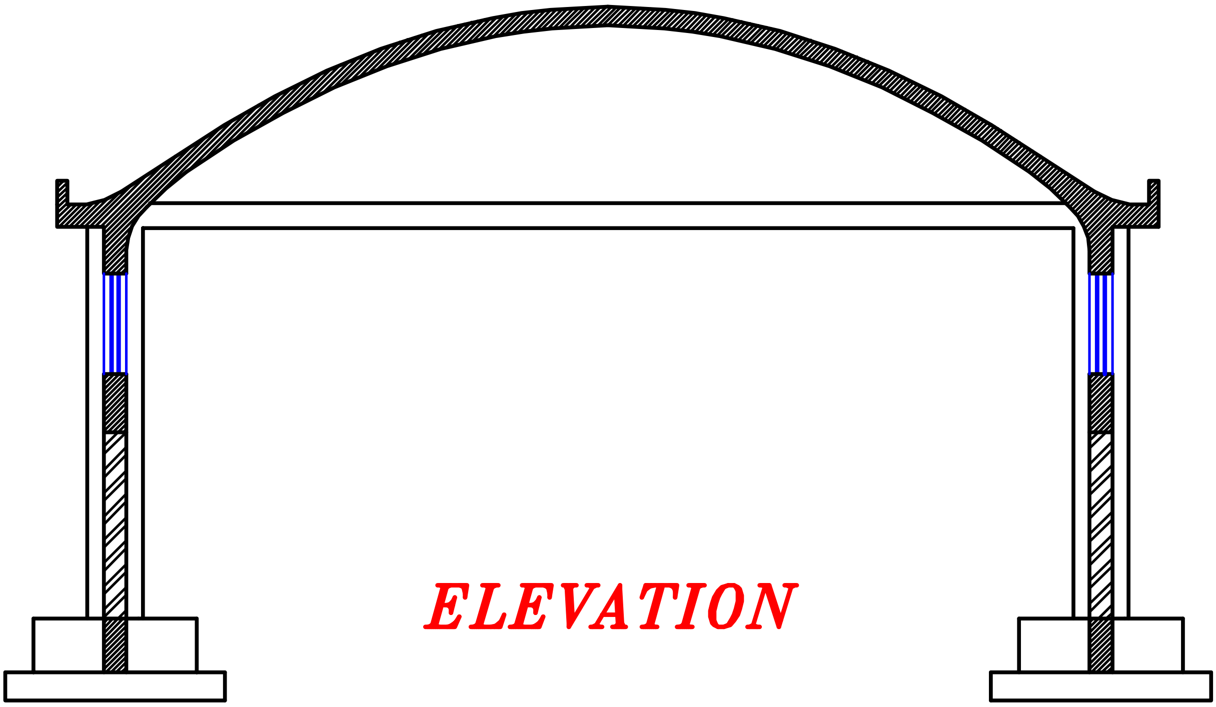
Comp. Force توضع لتقويه البلاطه وتقليل ال Buckling حيث أن البلاطه معرضه ل
و يفضل وضعها فوق ال Hangers حتى يدخل تسليح ال Hangers بها .

Arch Slab. Without Stiffeners & Without Hangers.

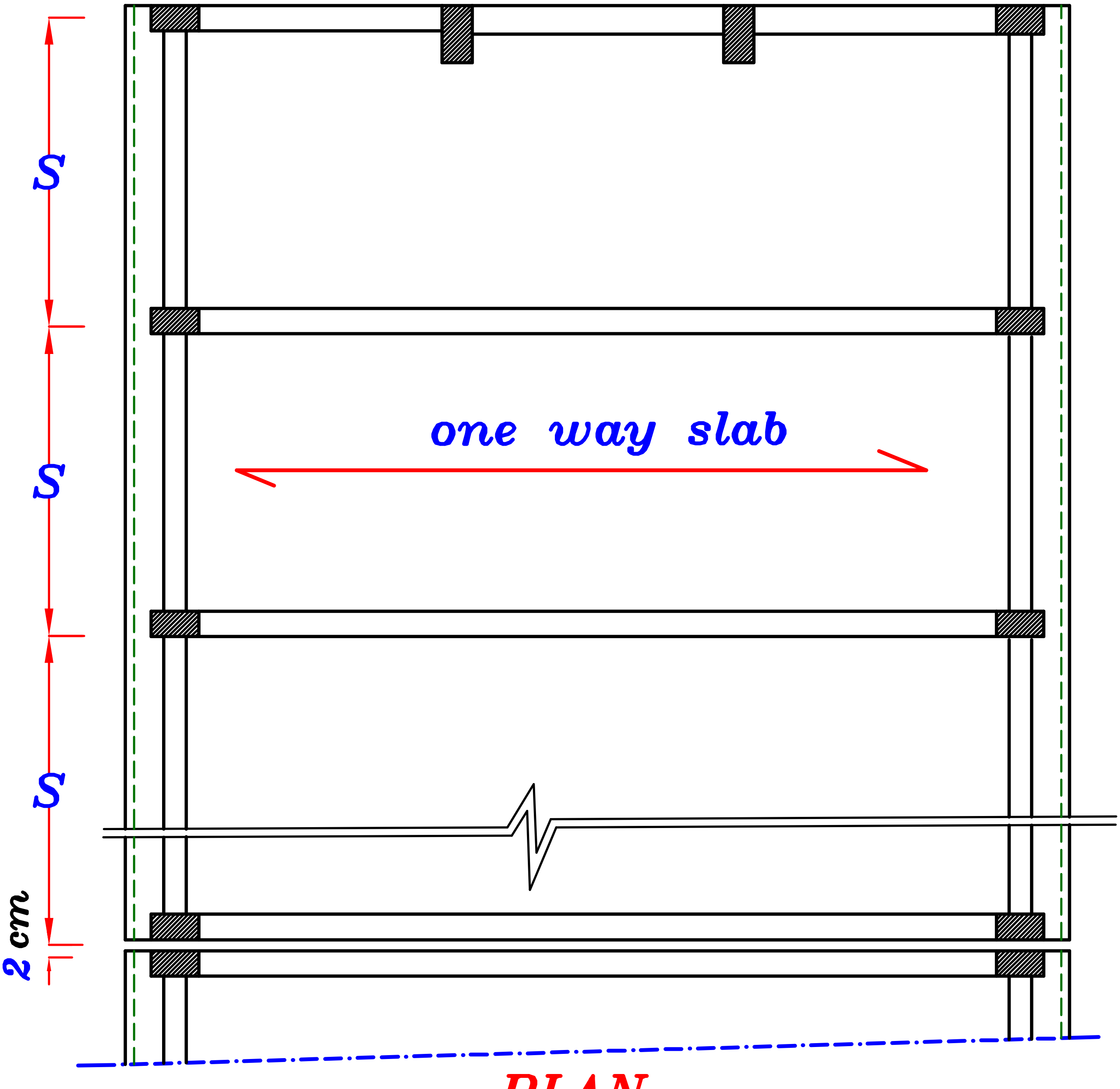




ELEVATION

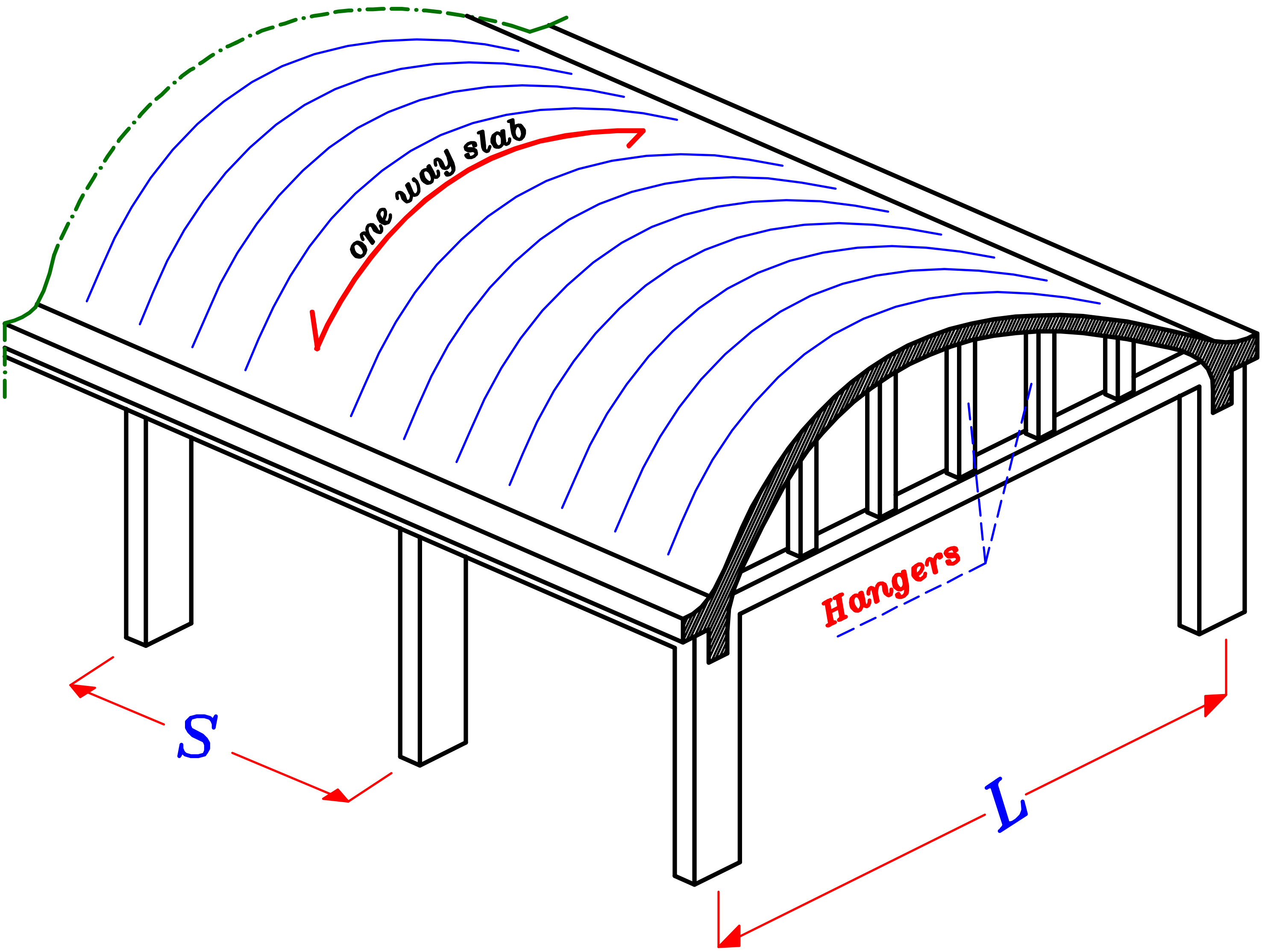


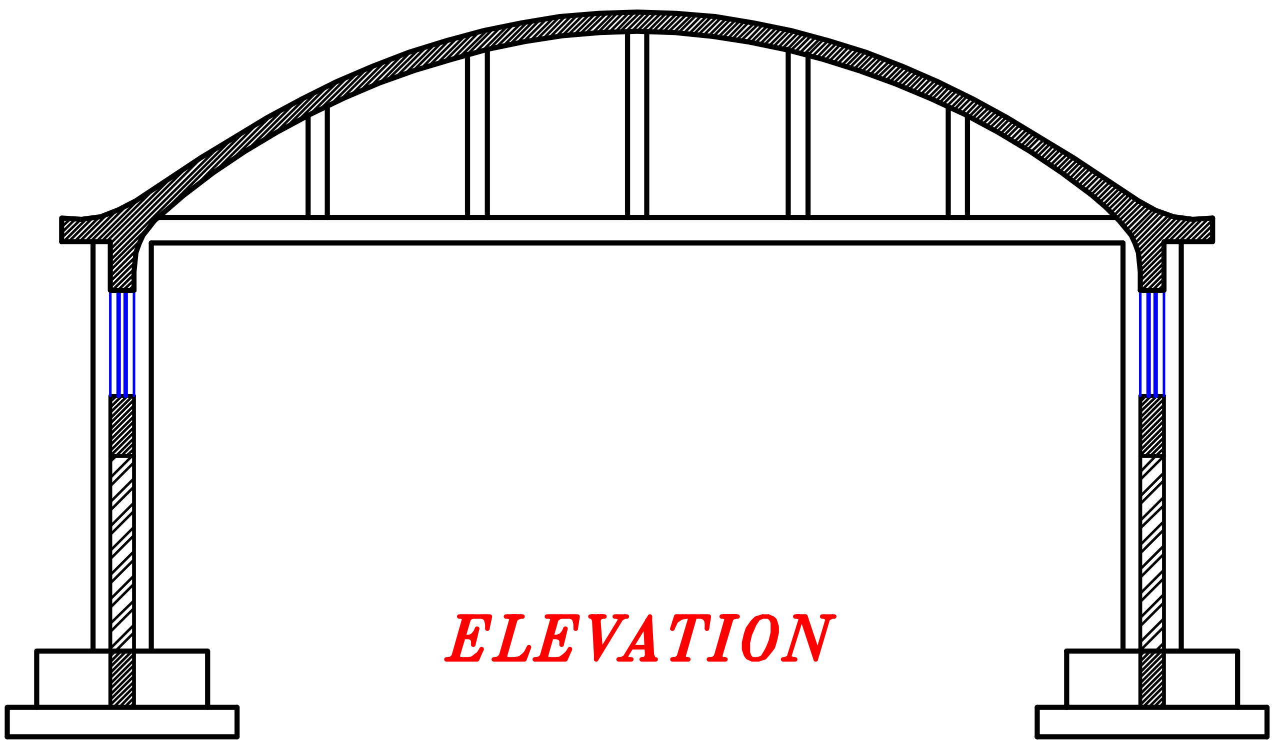
ELEVATION



PLAN

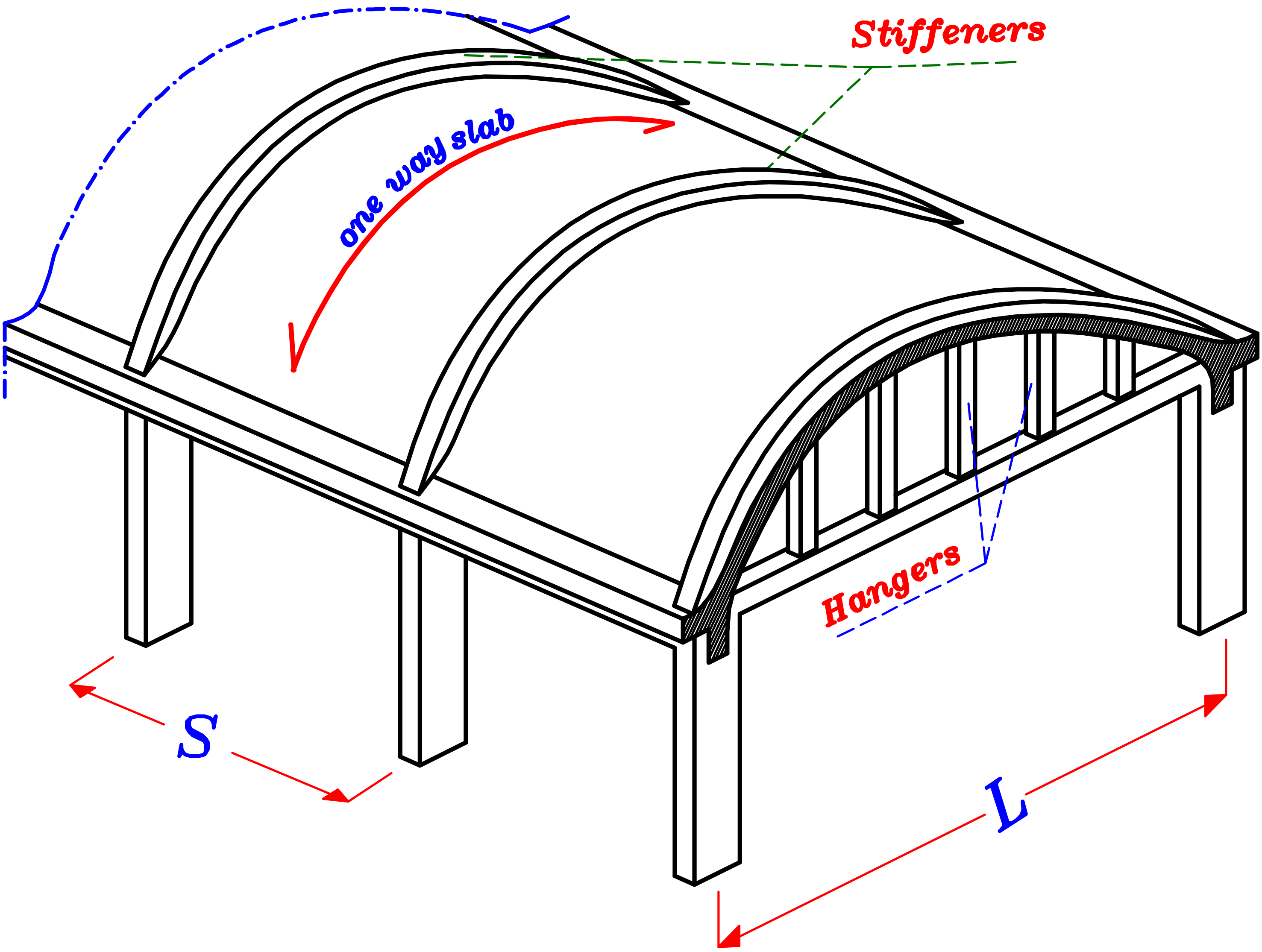
Arch Slab. Without Stiffeners & With Hangers.

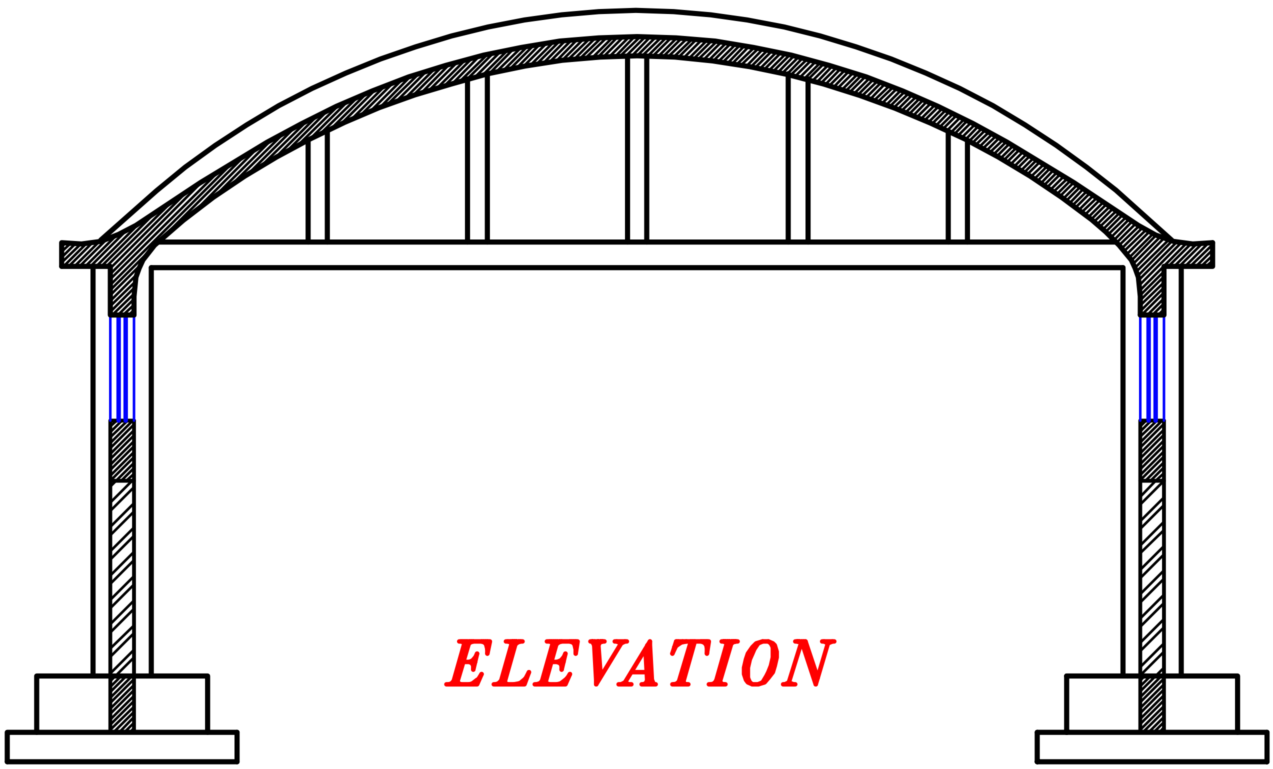




ELEVATION

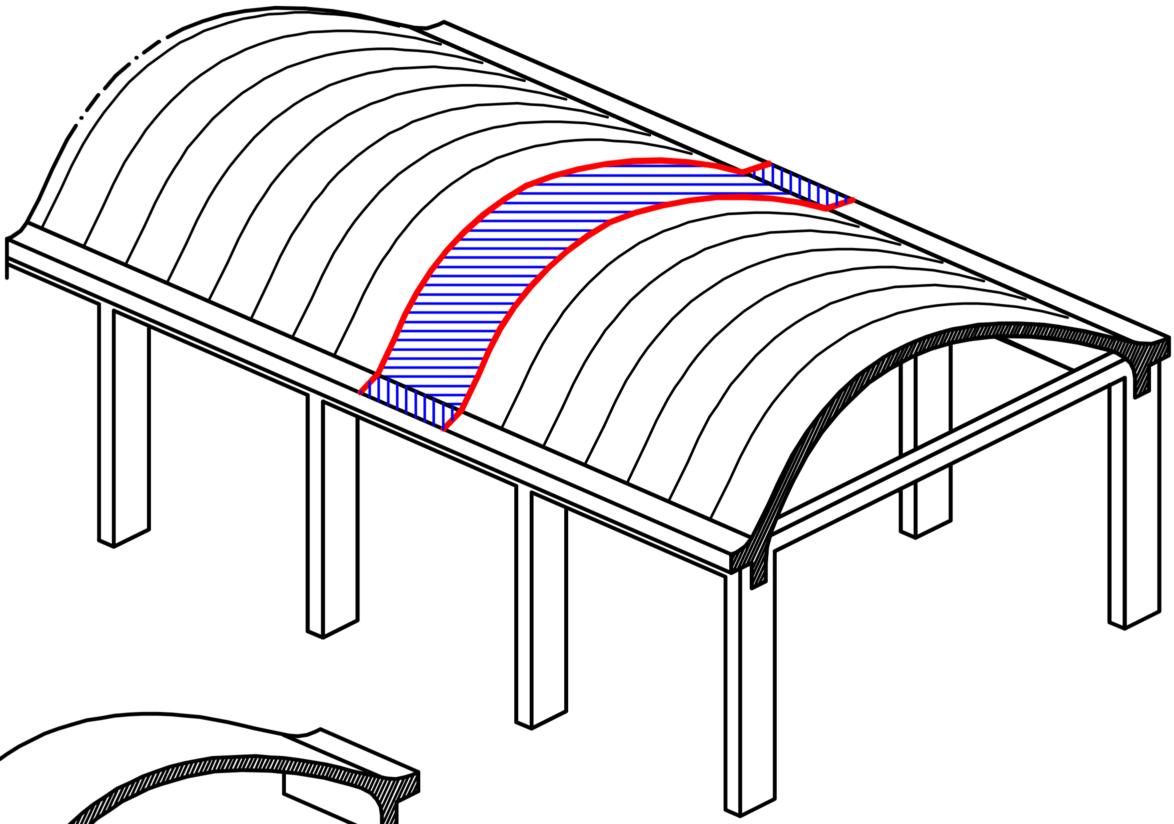
Arch Slab. With Stiffeners & With Hangers.

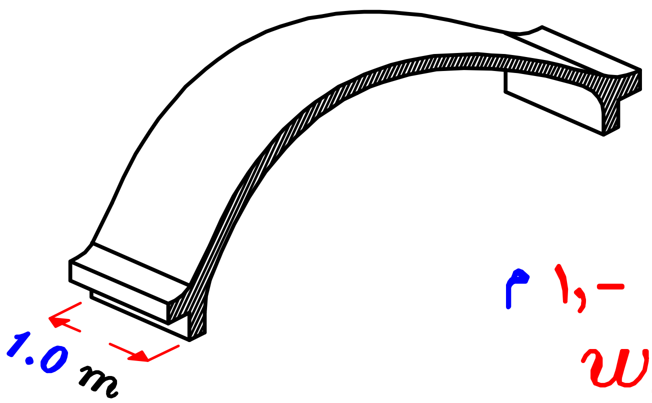




ELEVATION

Analysis of Arch Slab.





بأخذ شريحه فى البلاطه عرضها ١,٠ م

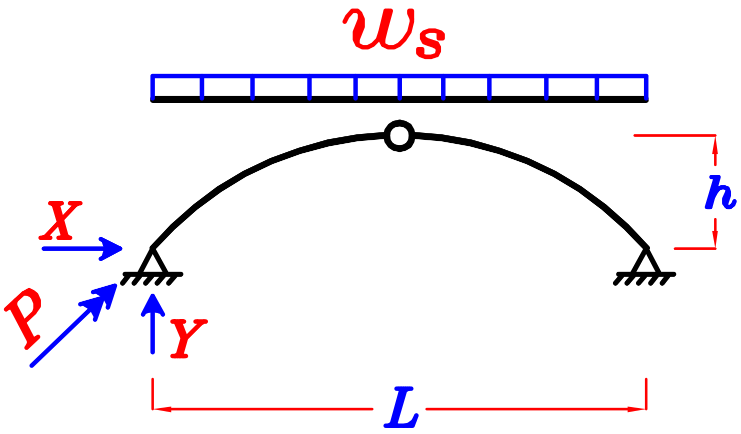
و نضع عليها حمل منتظم w_s

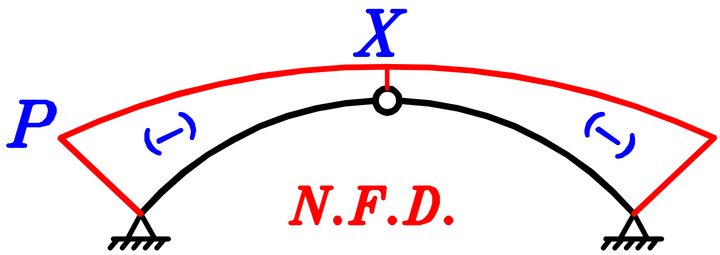
Take $t_s = (100 \rightarrow 140)\ mm$ $\boxed{t_s \simeq 120\ mm}$

assume $F.C. \simeq 0.50\ kN\backslash m^2$, $L.L. \simeq 0.50\ kN\backslash m^2$

$$w_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.) \simeq 5.0\ kN\backslash m^2$$

To get max Normal



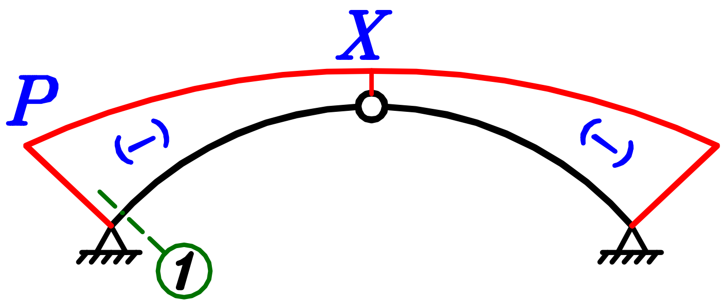


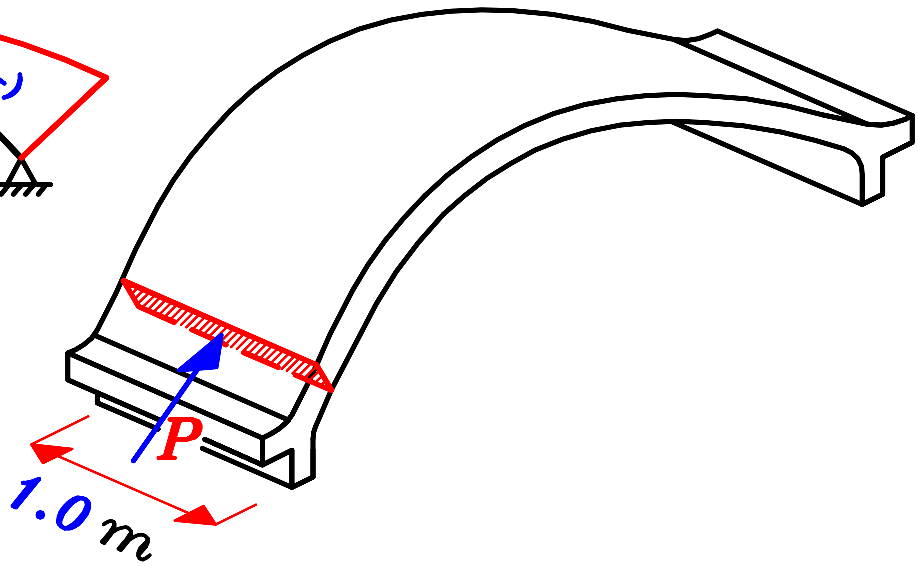
$$Y = \frac{\Sigma Loads}{2} = \frac{w_s\ L}{2}$$

$$X = \frac{w_s\ L^2}{8\ h}$$

$$P = \sqrt{X^2 + Y^2}$$

Design Critical Section of Arch Slab.

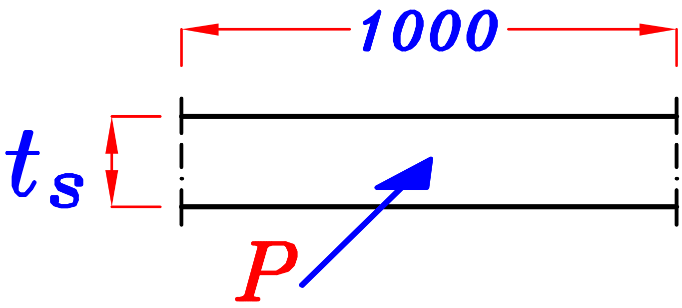




$$\boxed{P = \sqrt{X^2 + Y^2}}$$

Design on N.F. only.

Sec (1−1)



$$\therefore \boxed{P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y}$$

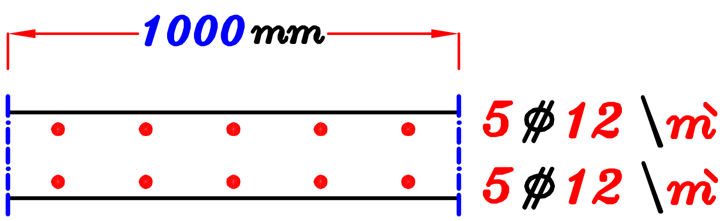
$P_{U.L.} = P$, $A_c = t_s * 1000 \longrightarrow$ Get $A_s = \checkmark\ mm^2$

عاده تكون A_s أقل من $A_{s\,min}$

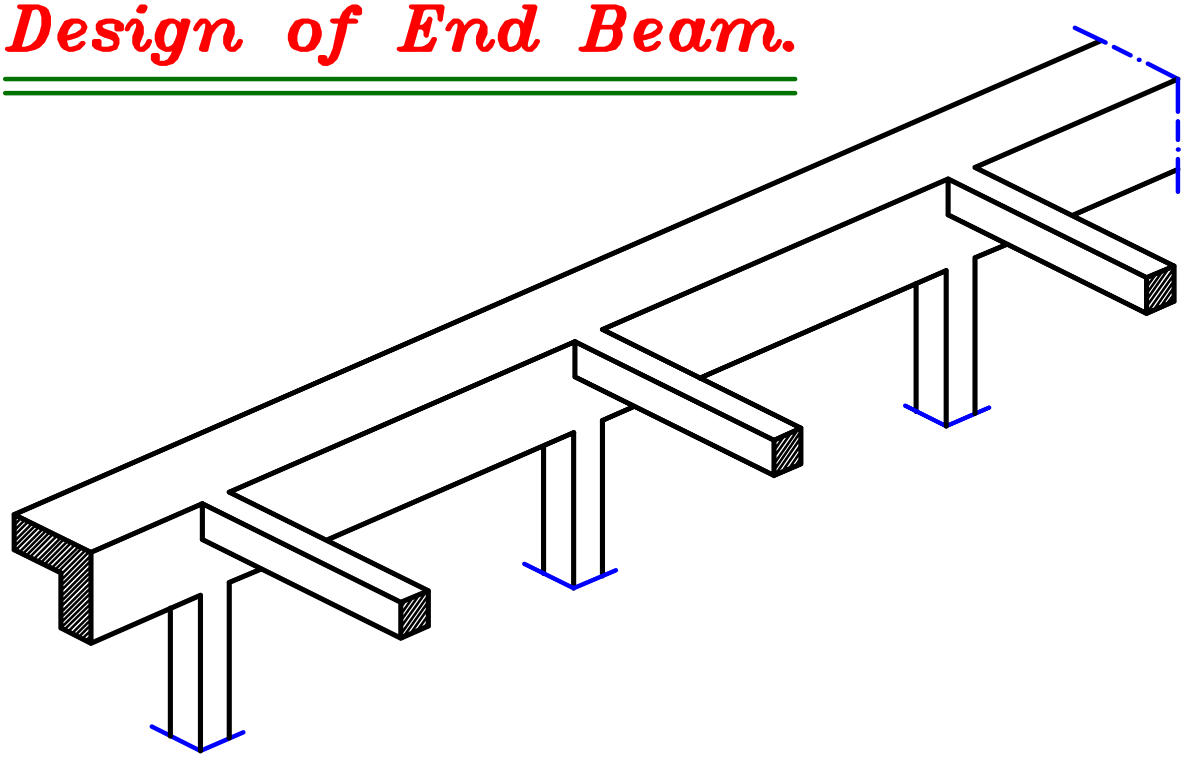
$$\therefore \text{Take } A_s = A_{s\,min} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 120 * 1000$$

$= 960\ mm^2 \simeq 10\phi 12 \backslash m'$ مجموع الحديد السفلى و العلوى

$$\boxed{A_s = A_s' \simeq 5\phi 12 \backslash m'}$$



* Design of End Beam.

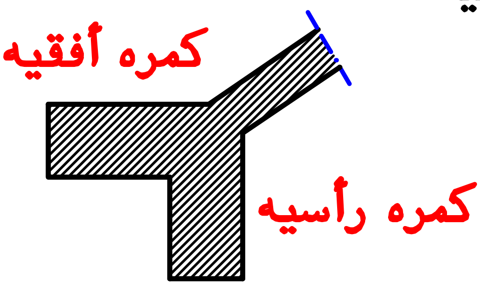


- الكمره الطرفيه *End beam* يوجد عليها قوه أفقيه

لذا تتكون من كمرتين

كمره رأسيه لتحمل الاحمال الرأسيه

كمره أفقيه لتحمل الاحمال الافقيه .

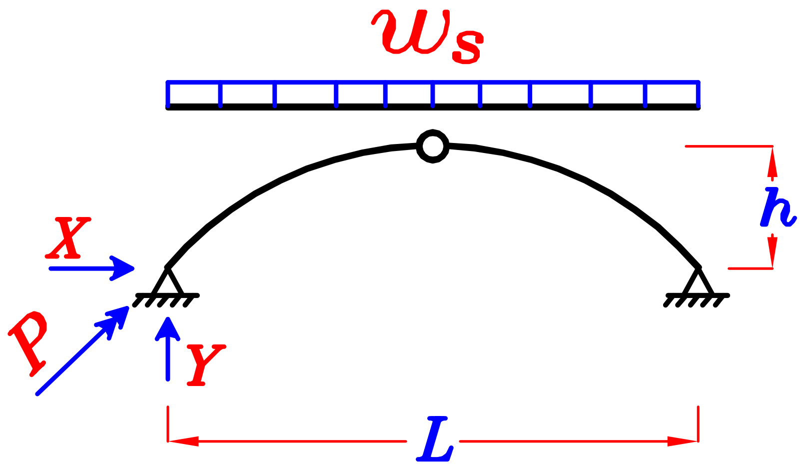


- أى قوى رأسيه تذهب الى الكمره الرأسيه

أى قوى أفقيه تذهب الى الكمره الافقيه .

- وزن الكمرتين هو حمل رأسى لذا يذهب الى الكمره الرأسيه فقط.

$$\underset{(beam)}{O.W.} \quad (VL.+HL.)$$



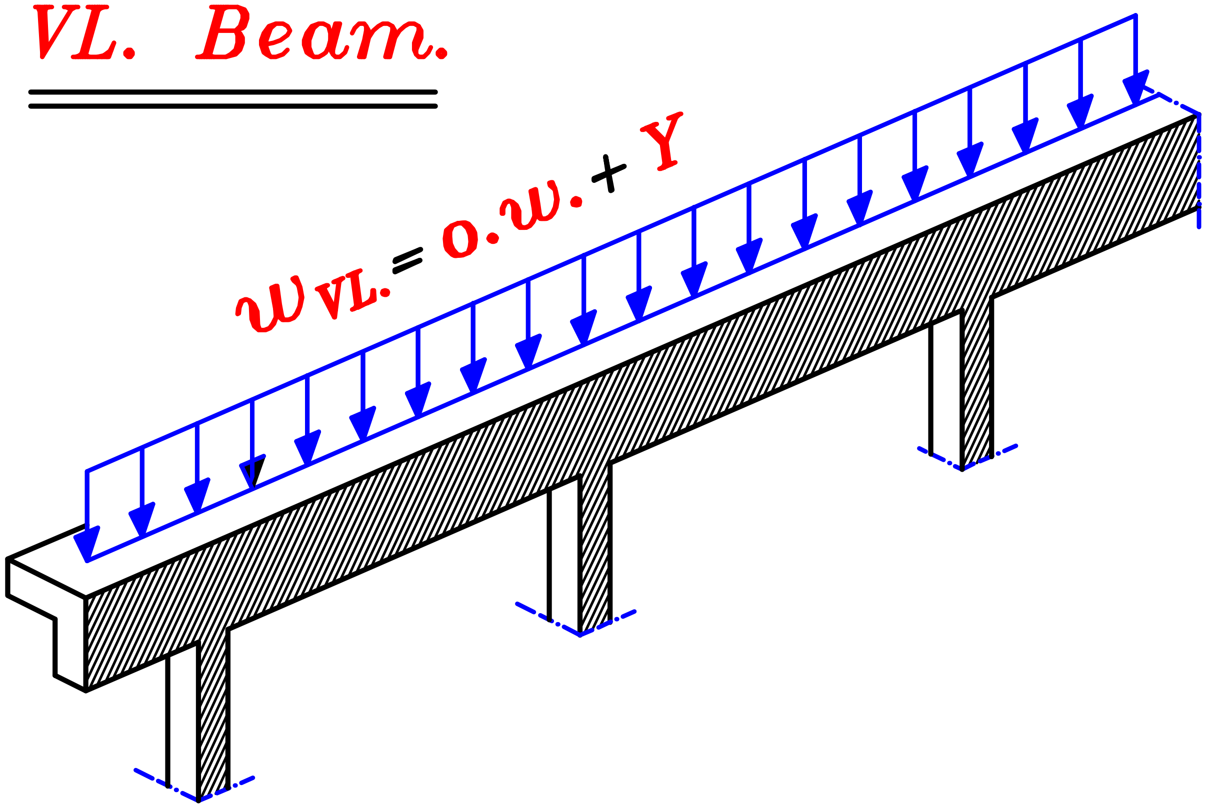
- X, Y من شريحه البلاطه

تنتقل على الا *End beam*

Y تذهب الى الكمره الرأسيه .

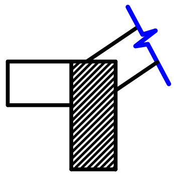
X تذهب الى الكمره الافقيه .

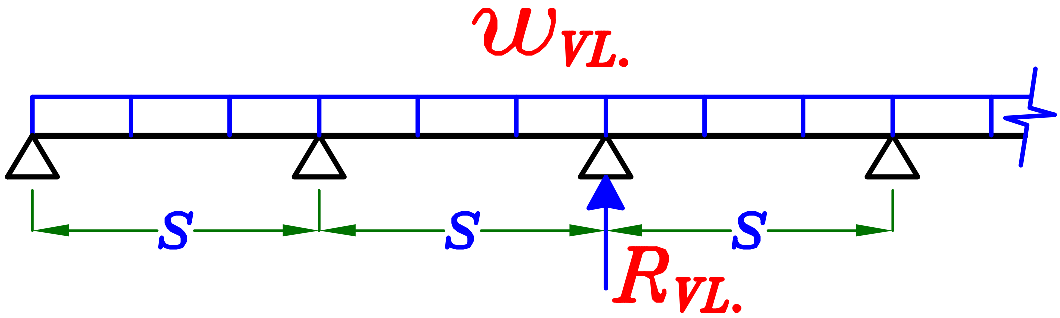
VL. Beam.

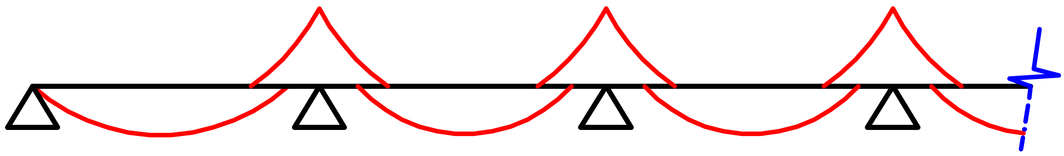


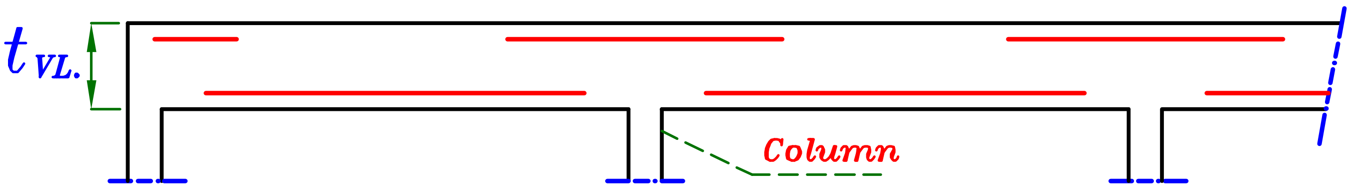
$$w_{VL.} = O.W._{(beam)} + Y \quad kN\backslash m`$$

Designed as R-Sec.



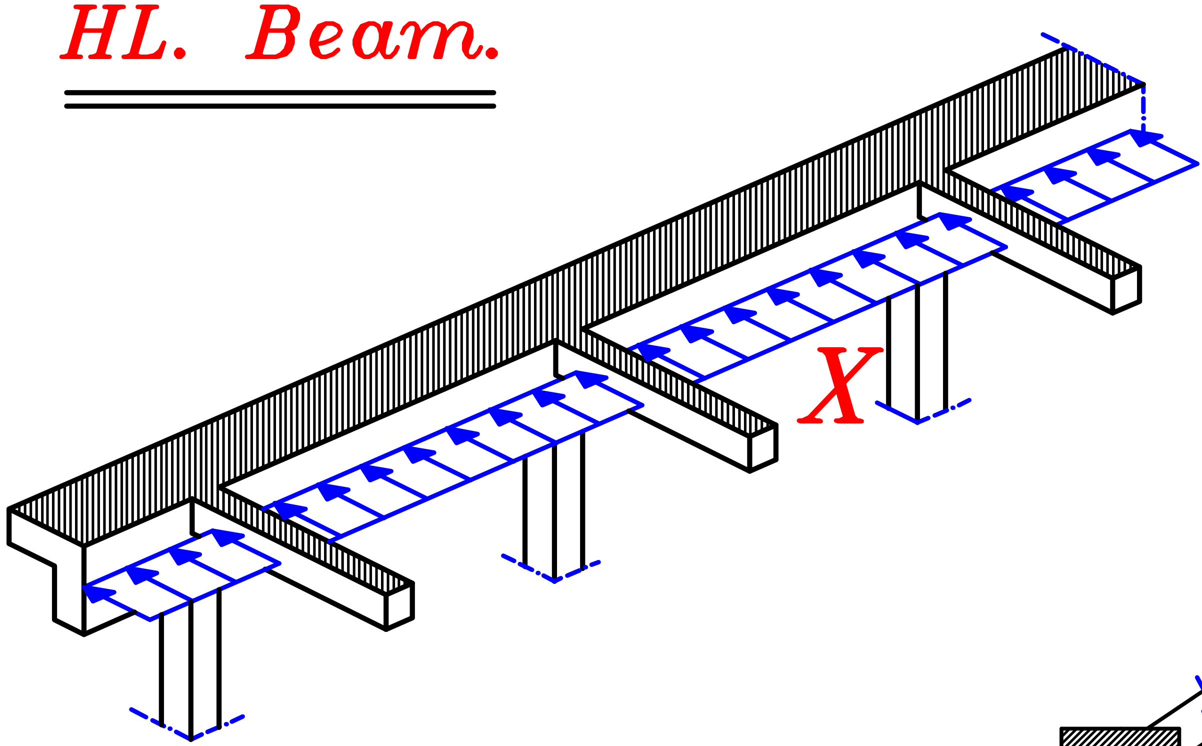




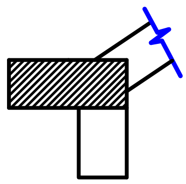


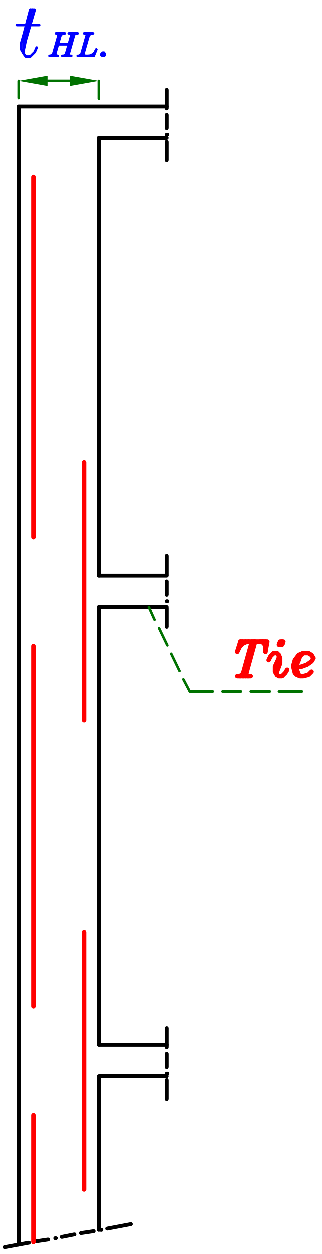
$$\boxed{R_{VL.} = (o.w. + Y) * S}$$ تنقل الى العمود

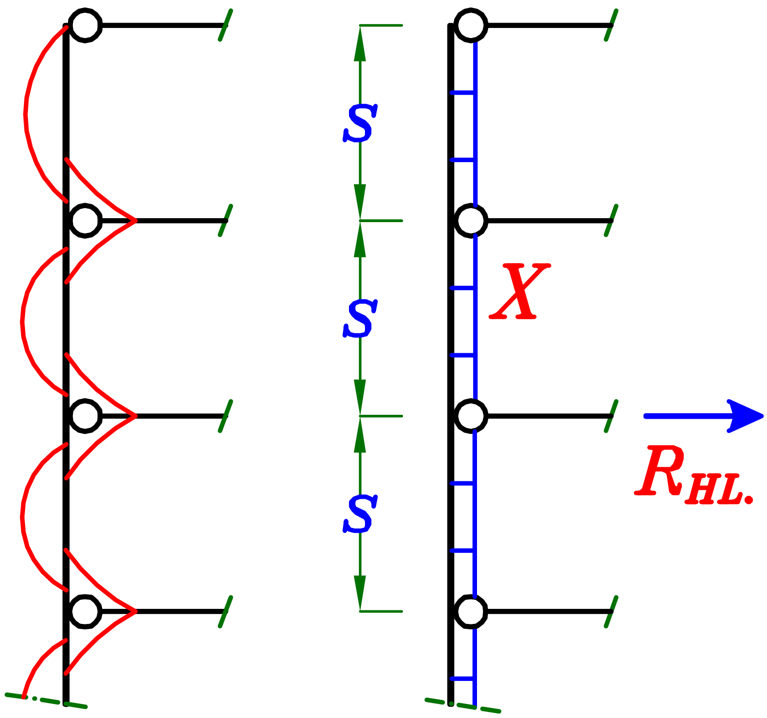
HL. Beam.



Designed as R–Sec.





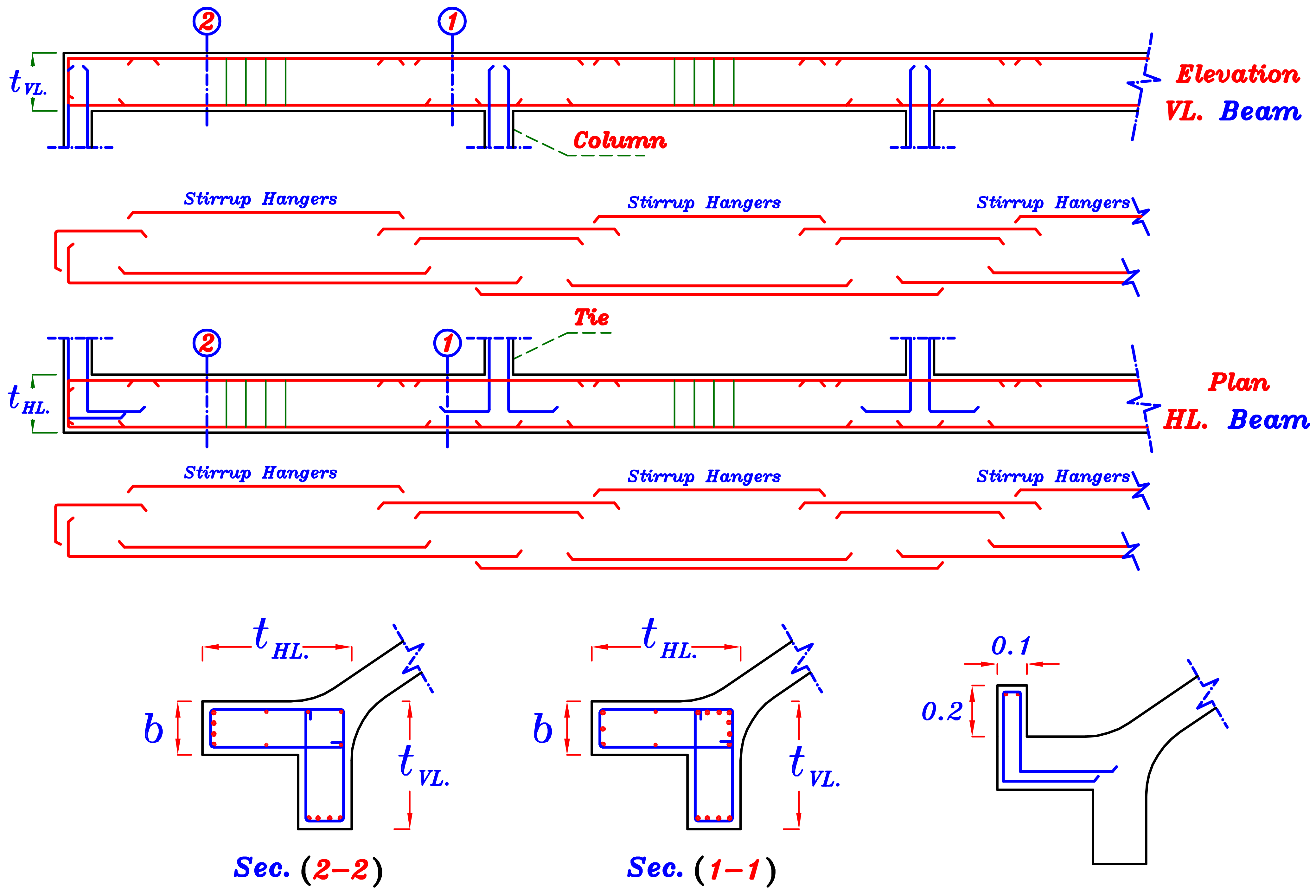


plan

$$\boxed{R_{HL.} = X * S}$$

تنتقل الى ال Tie

RFT. of End Beam.



* Design the Tie. $(b \times b)$

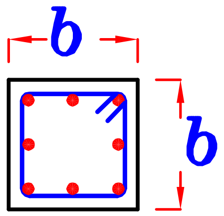
المقصود بـ **b** هو العرض الاصغر من عرض العمود و عرض الكمره الافقيه لان تسليح الـ **tie** سيدخل فى الاثنين .

Neglect O.W. $\therefore$ *B.M.* $\simeq$ *Zero*

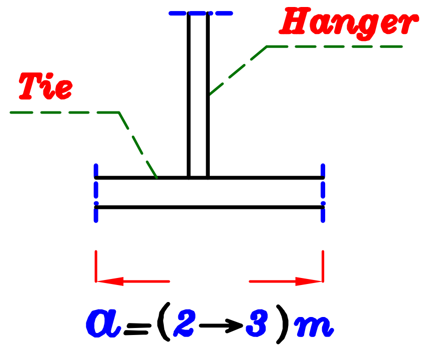
$$\boxed{T_{(Tie)} = R_{HL.} = X * S}$$

$$A_S = \frac{T_{(Tie)}}{F_y \backslash \gamma_s} = (\text{Total area of steel})$$

$$A_C = (b \times b)$$

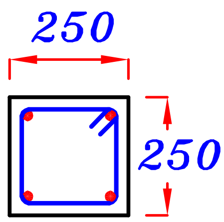


* Design the Hanger. (250×250)



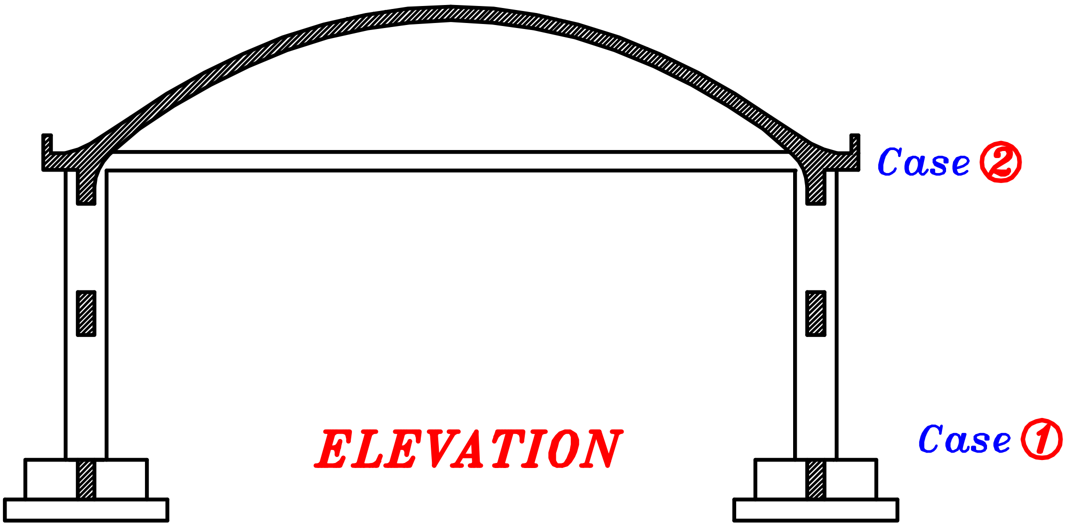
$$T = O.W._{(hanger)} + O.W._{(Tie)} * a$$

$$A_S = 4\phi 12$$



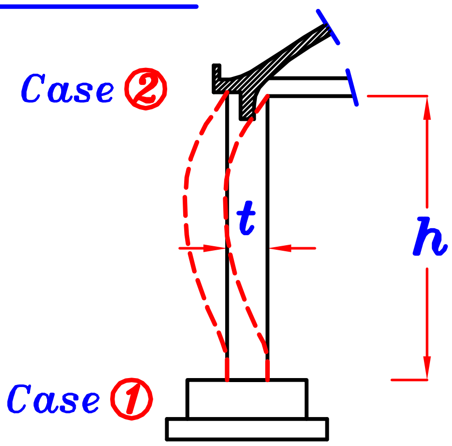
* Design the Column

$$P = R_{VL.} = (o.w. + Y) * S$$



Check Buckling.

① In Plane.

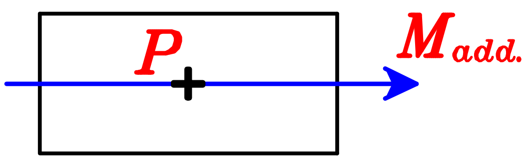


$$H_o = h$$

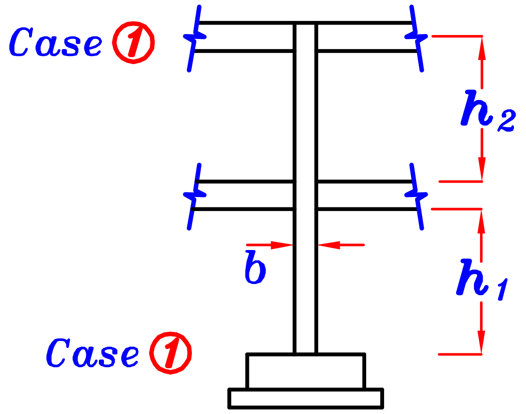
$$\lambda_b = \frac{1.3 * H_o}{t}$$

IF $\lambda_b \leq 10 \xrightarrow{\text{Designed}} P$ only

$\lambda_b > 10 \xrightarrow{\text{Designed}} P\ ,\ M_{add.}$



② Out of Plane.

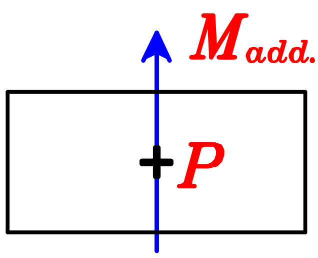


$H_o =$ The bigger of h_1 , h_2

$$\lambda_b = \frac{1.2 * H_o}{b}$$

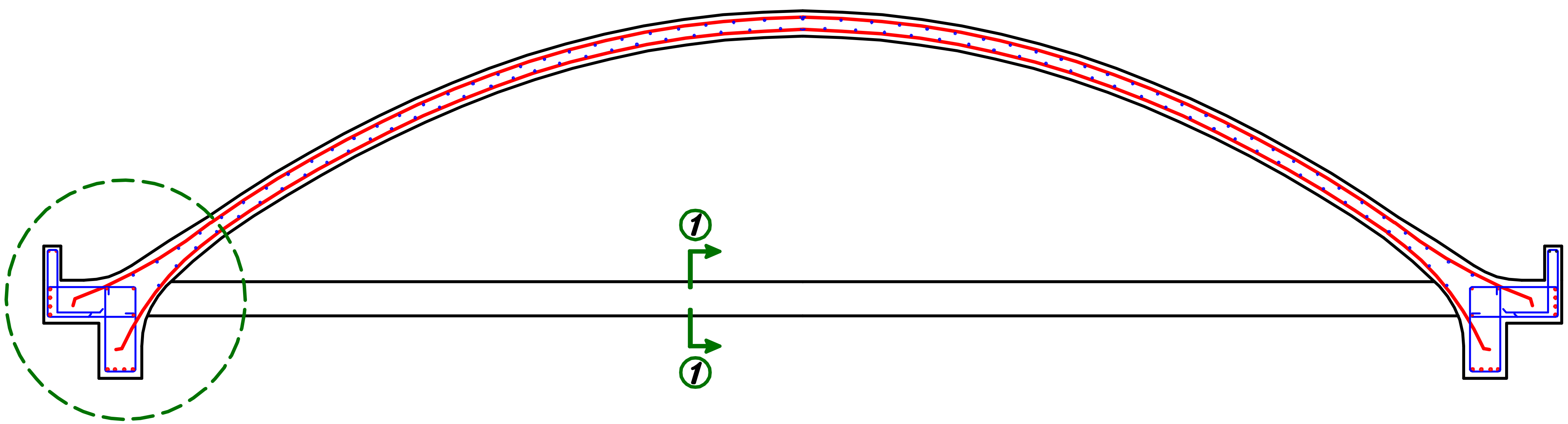
IF $\lambda_b \leq 10 \xrightarrow{\text{Designed}} P$ only

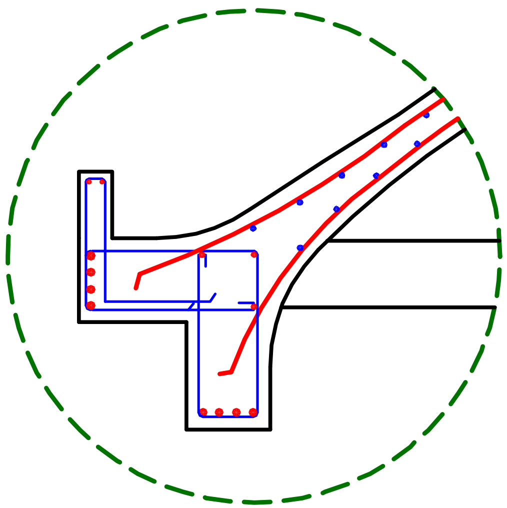
$\lambda_b > 10 \xrightarrow{\text{Designed}} P\ ,\ M_{add.}$



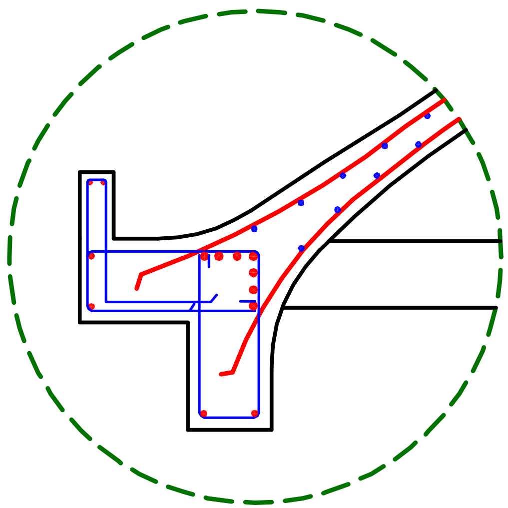
Reinforcement of Arch Slab.

Without hangers or Stiffener

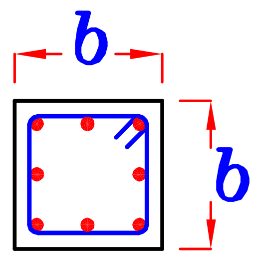




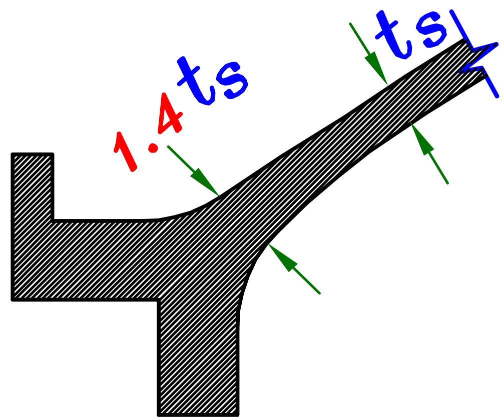
Sec. at
the Mid Span.



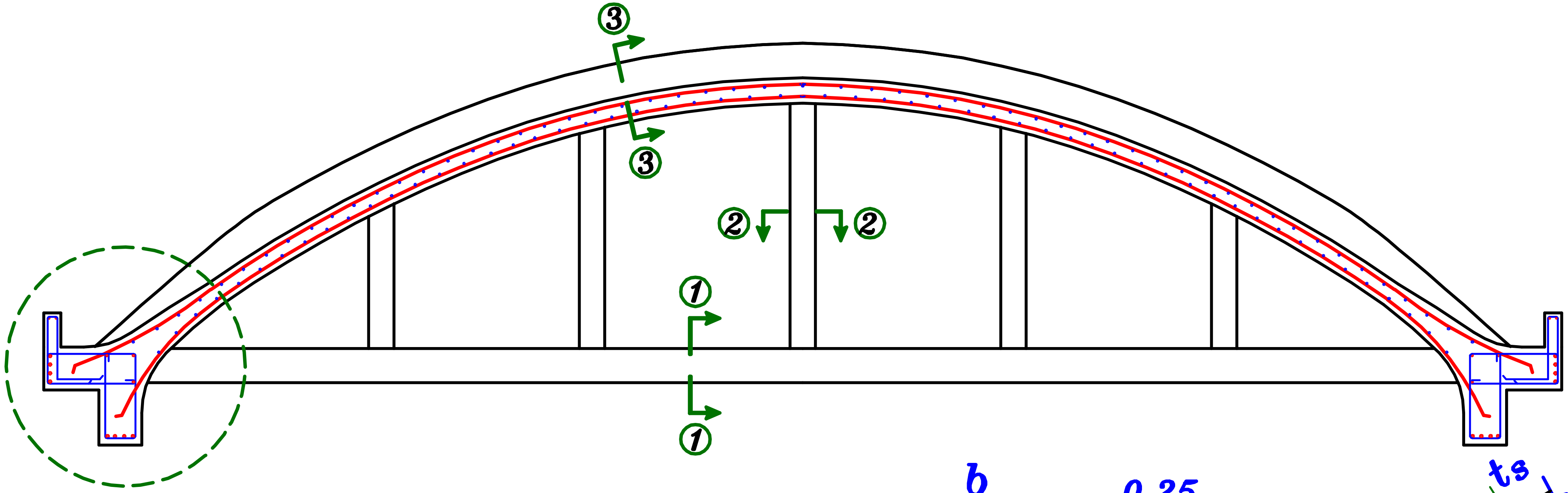
Sec. at
the Column.

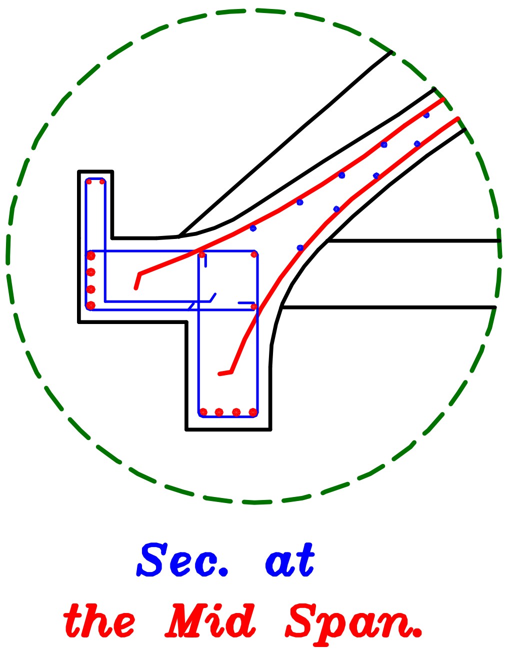


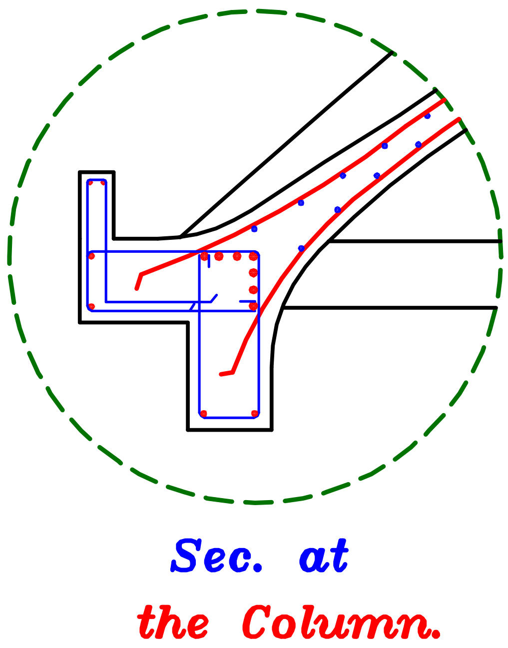
Sec. (1-1)

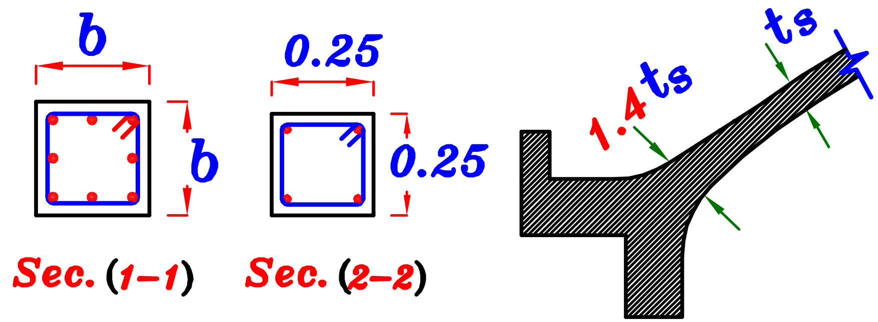


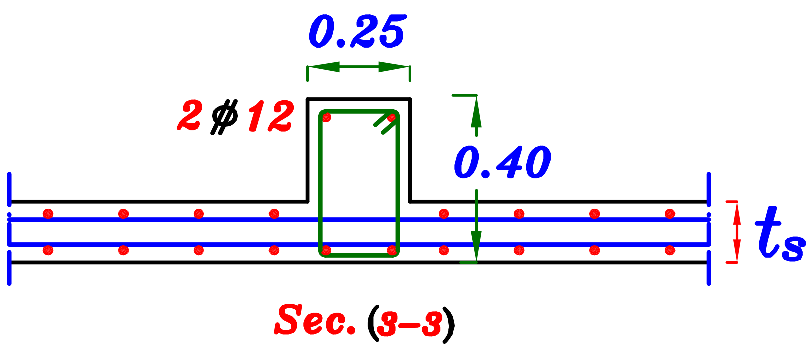
Reinforcement of Arch Slab. With hangers and Stiffener





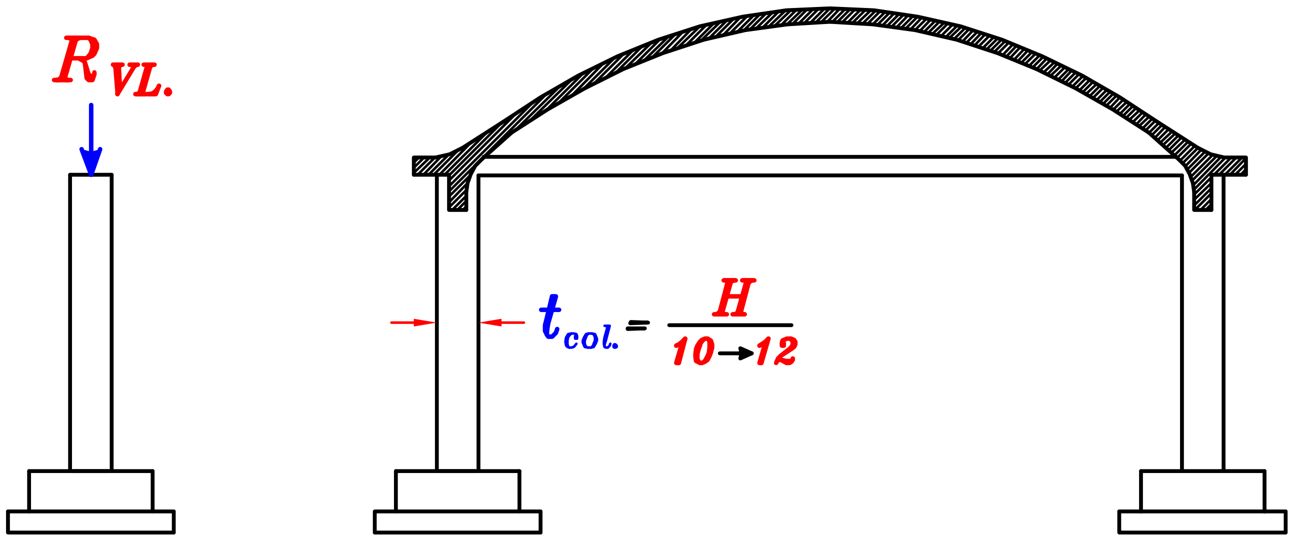




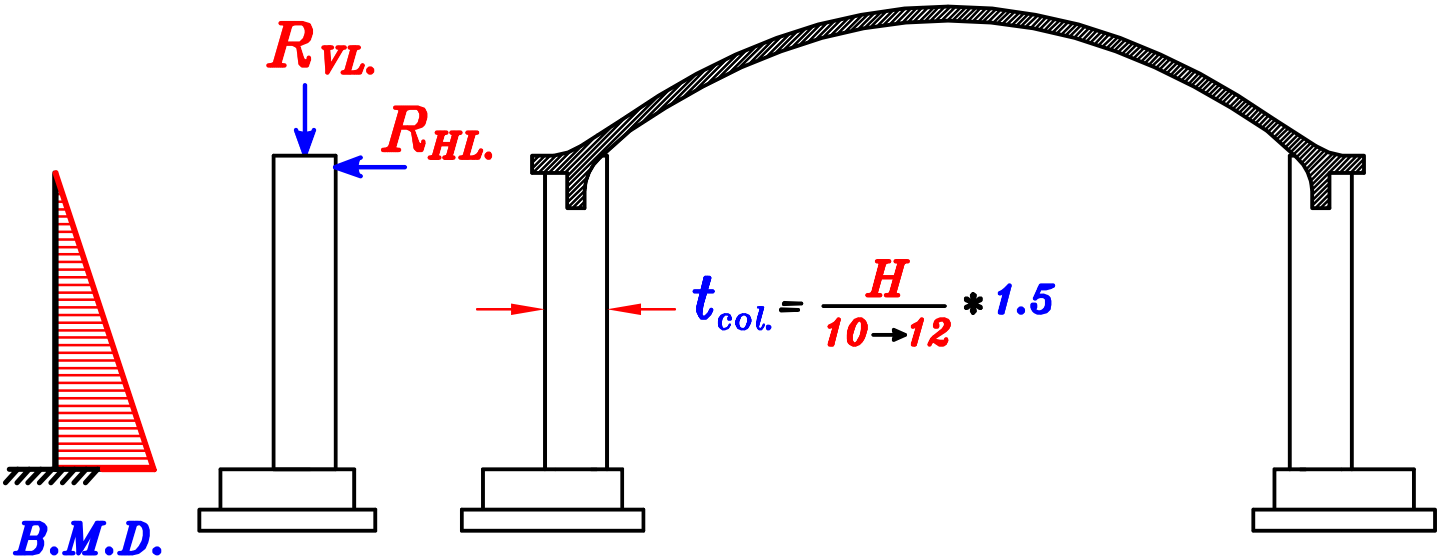


Special Cases.

Arch Slab without Tie.



اذا لم نضع *Tie* مع ال *Arch Slab* سينتقل الحمل من الكمره الرأسيه $R_{VL.}$
الى العمود ليعمل *Normal Force* على العمود .
و ستنتقل القوى الافقيه من الكمره الافقيه $R_{HL.}$ الى العمود أيضا لتعمل
Bending Moment على العمود .
فيتم تصميم العمود على M, N و يتم ترحيل القاعده عكس ال *moment* .



Arch Slab Examples.

Example.

$F_{cu.} = 25$ N\mm^2

$F_y = 360$ N\mm^2

L.L. = 0.50 kN\m^2 H.P.

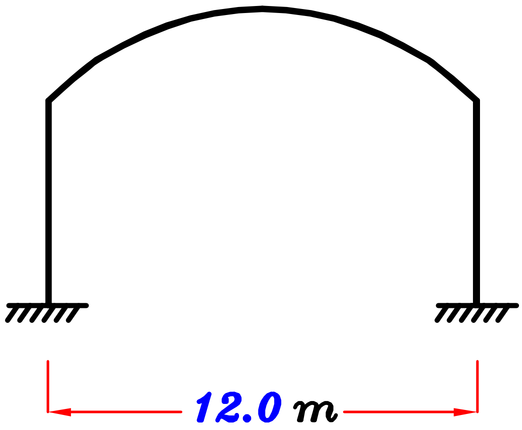
F.C. = 0.50 kN\m^2 H.P.

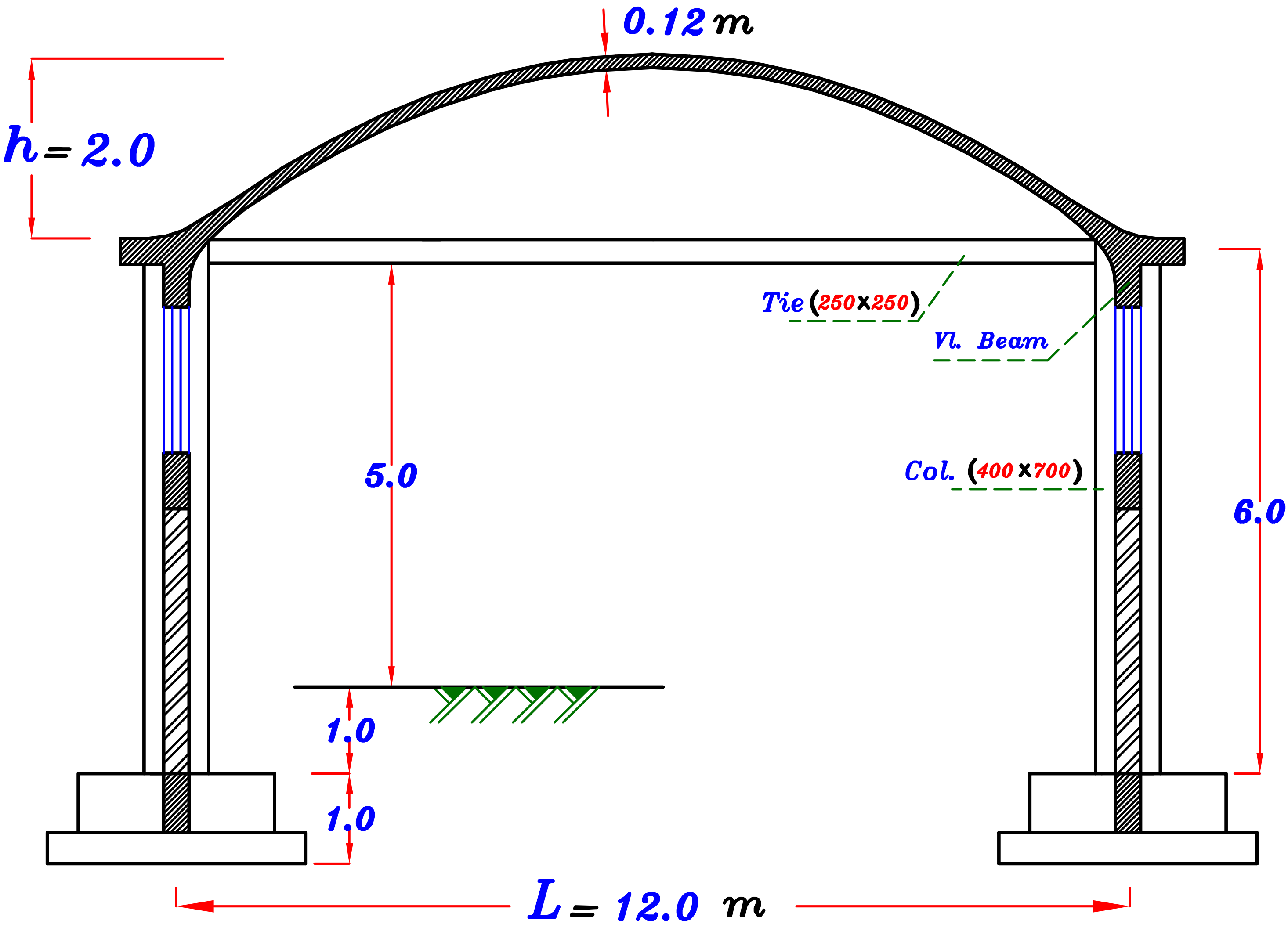
Clear hieght = 5.0 m

Foundation Level. = – 2.0 m

Window hieght = 1.5 m

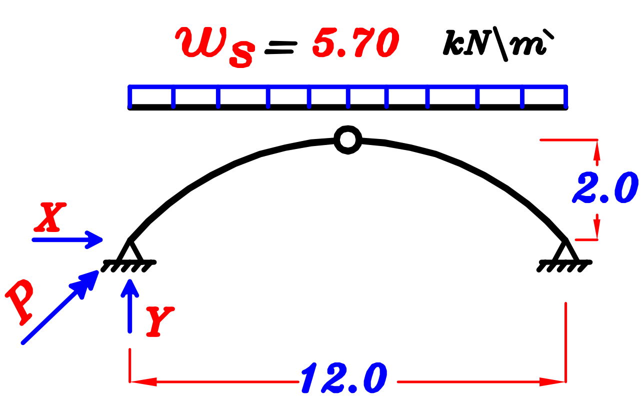
Spacing between Columns = 6.0 m





Design the Arch Slab.

Take $t_s = 120$ mm



$$(w_s)_{U.L.} = 1.4\,(t_s\,\delta_c + F.C.) + 1.6\,(L.L.)$$

$$(w_s)_{U.L.} = 1.4\,(0.12 * 25 + 0.50) + 1.6\,(0.50)$$

$$= 5.70 \ \ kN\backslash m^2 \ \ (H.P.)$$

To Get N.F.

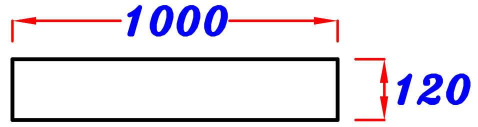
$$Y = \frac{w\,L}{2} = \frac{5.70 * 12}{2} = 34.2 \ \ kN\backslash m$$

$$X = \frac{w\,L^2}{8\,h} = \frac{5.70 * 12^2}{8 * 2.0} = 51.3 \ \ kN\backslash m$$

$$P = \sqrt{X^2 + Y^2} = \sqrt{34.2^2 + 51.3^2} = 61.65 \ kN$$

* Design the Arch Slab.

Neglect B.M. & Design on N.F. only.



∴ Designed as a Column.

$$\therefore P_{U.L.} = 0.35\,A_c\,F_{cu} + 0.67\,A_s\,F_y$$

Take $A_c = 120 * 1000 = 120000 \ \ mm^2$

$$\therefore 61.65 * 10^3 = 0.35\,(120000)(25) + 0.67\,A_s\,(360)$$

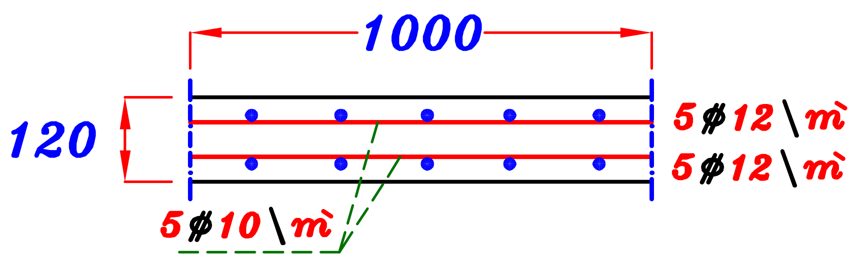
$$\therefore A_s = -\ 4097 \ \ mm^2 = -\ (Ve)\ Value$$

$$\therefore \text{Take } A_s = A_{s_{min.}} = \frac{0.8}{100} * b * t$$

$$\therefore A_s = \frac{0.8}{100} * 120 * 1000 = 960 \ \ mm^2 = A_{s\ total}$$

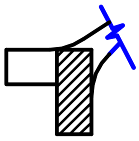
$$\therefore \text{Upper Steel \& Lower Steel} = \frac{A_{s\ total}}{2} = \frac{960}{2} = 480\ mm^2$$

$5 \phi 12 \backslash m`$



Design of End Beam.

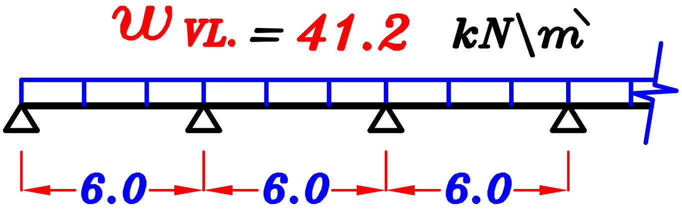
VL. Beam.

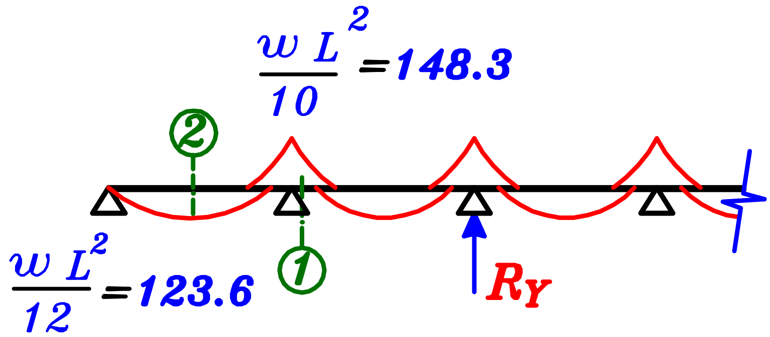


Take $O.W._{(beam)}$ (VL.+HL.) $= 7.0$ kN\m` (U.L.)

$w_{VL.} = O.W._{(beam)} + Y = 7.0 + 34.2 = 41.2$ kN\m`

$R_Y = w_{VL.} * S = 247.2$ kN





Sec. 1

$M_{U.L.} = 148.3$ kN.m R-Sec.

- Take $\boxed{d = 550 \text{ mm}}$, $\boxed{t = 600 \text{ mm}}$

- Get $A_s = 979.0 \text{ mm}^2$

Check $A_{s\,min.}$

$A_{s_{req.}} = 979.0 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 550 = 429.7 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d$ $\therefore$ Take $A_s = A_{s_{req.}} = 979.0 \text{ mm}^2$ 5ϕ16

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{16 + 25} = 5.48 = 5.0 \text{ bars}$$

Sec. ② $M_{U.L.} = 123.6$ kN.m

$d = 550$ mm (the same depth of sec. ①)

$$A_s = 779.3 \ mm^2$$

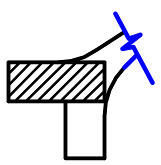
Check $A_{s\,min.}$ $\qquad A_{s_{req.}} = 779.3 \ mm^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 550 = 429.7 \ mm^2$$

$\therefore A_{s_{req.}} > \mu_{min.}\, b\, d \quad \therefore$ Take $A_s = A_{s_{req.}} = 779.3 \ mm^2$ $\quad$ $4 \phi 16$

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\, 779.3$ $\quad$ $2 \phi 10$

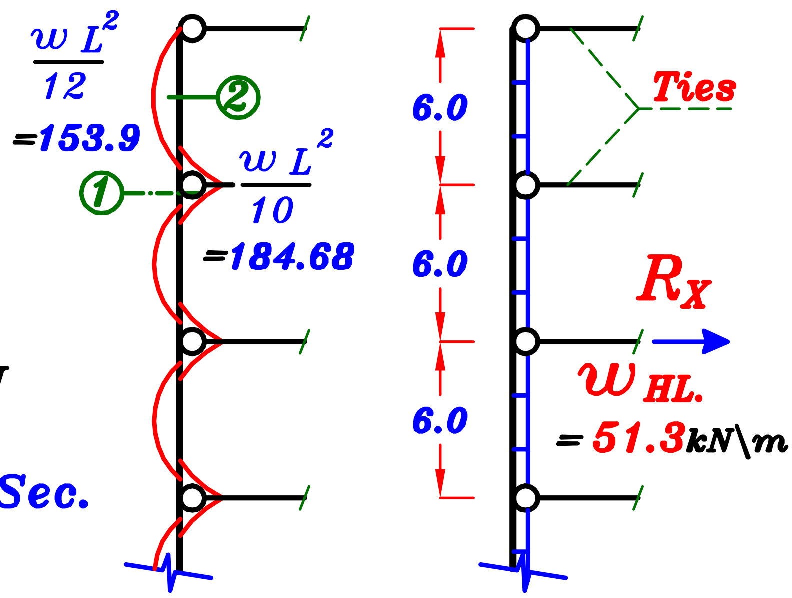
HL. Beam.



$w_{HL.} = X = 51.3$ kN\m

$R_X = w_{HL.} * S = 307.8$ kN

Design all Sections as R-Sec.



Sec. ① $M_{U.L.} = 184.68$ kN.m R–Sec.

– Take $\boxed{d = 650 \text{ mm}}$, $\boxed{t = 700 \text{ mm}}$

– Get $A_s = 1093.2 \text{ mm}^2$

Check $A_{s\,min.}$ $A_{s_{req.}} = 1093.2 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8 \text{ mm}^2$$

$\therefore A_{s_{req.}} > \mu_{min.} b\ d$ $\therefore$ Take $A_s = A_{s_{req.}} = 1093.2 \text{ mm}^2$ 5 ϕ 18

$$\therefore n = \frac{b - 25}{\phi + 25} = \frac{250 - 25}{18 + 25} = 5.23 = 5.0 \text{ bars}$$

Sec. ② $M_{U.L.} = 153.9$ kN.m

$d = 650$ mm (the same depth of sec. ①)

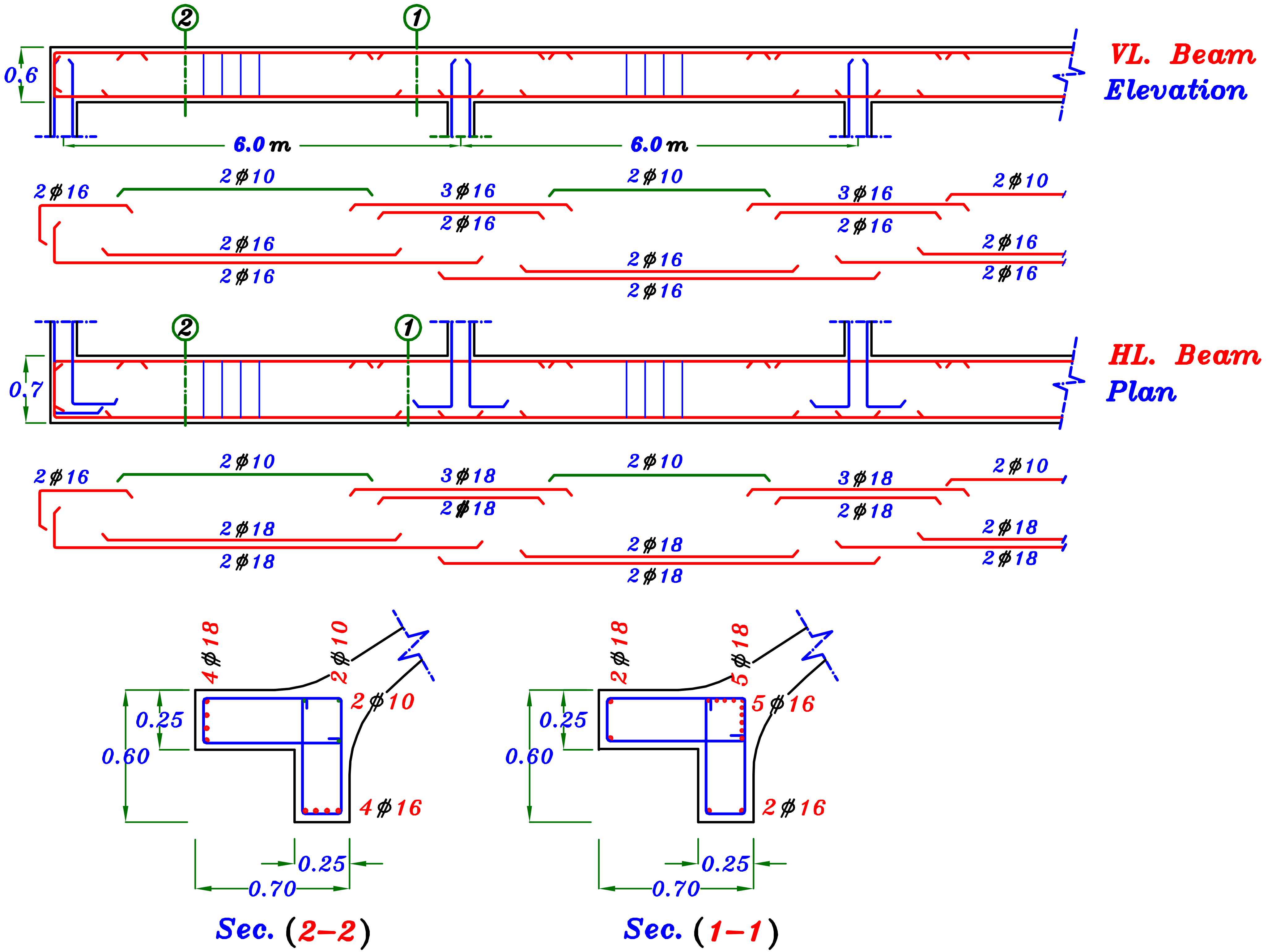
$A_s = 813 \text{ mm}^2$

Check $A_{s\,min.}$ $A_{s_{req.}} = 813 \text{ mm}^2$

$$\mu_{min.}\, b\, d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\, d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 250 * 650 = 507.8 \text{ mm}^2$$

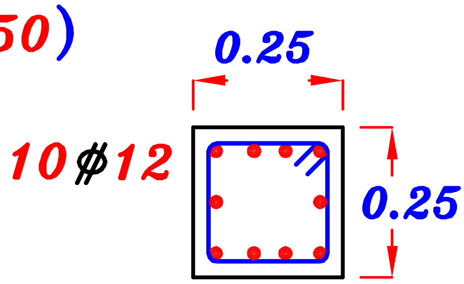
$\therefore A_{s_{req.}} > \mu_{min.} b\ d$ $\therefore$ Take $A_s = A_{s_{req.}} = 813 \text{ mm}^2$ 4 ϕ 18

Stirrup Hangers $= (0.1 \rightarrow 0.2)\, A_s = (0.1 \rightarrow 0.2)\ 813$ 2 ϕ 10



VL. Beam
Elevation
0.6
6.0 m
6.0 m
2 ϕ16
2 ϕ10
3 ϕ16
2 ϕ16
2 ϕ10
3 ϕ16
2 ϕ16
2 ϕ10
2 ϕ16
2 ϕ16
2 ϕ16
2 ϕ16
2 ϕ16
2 ϕ16
HL. Beam
Plan
0.7
2 ϕ16
2 ϕ10
3 ϕ18
2 ϕ18
2 ϕ10
3 ϕ18
2 ϕ18
2 ϕ10
2 ϕ18
2 ϕ18
2 ϕ18
2 ϕ18
2 ϕ18
2 ϕ18
4 ϕ18
2 ϕ10
2 ϕ10
4 ϕ16
0.25
0.60
0.25
0.70
Sec. (2–2)
2 ϕ18
5 ϕ18
5 ϕ16
2 ϕ16
0.25
0.60
0.25
0.70
Sec. (1–1)

* Design the Tie. $(250 * 250)$



Neglect O.W. $\therefore$ B.M. $\simeq$ Zero

$$T_{(Tie)} = R_X = 307.8 \text{ kN}$$

$$A_s = \frac{T_{(Tie)}}{F_y \backslash \delta_s} = \frac{307.8 * 10^3}{360 \backslash 1.15} = 983 \text{ mm}^2$$

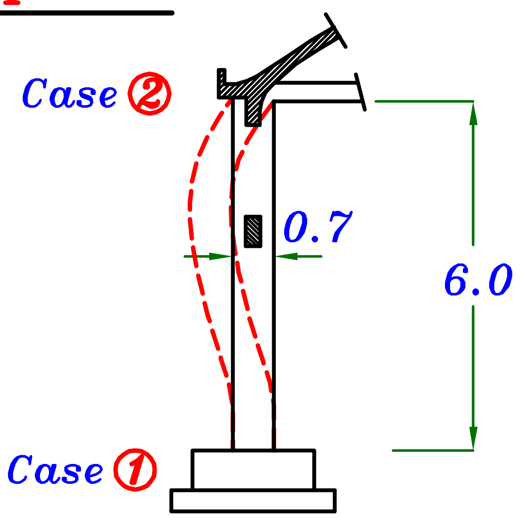
10ϕ12

* Design the Column. $(250 * 700)$

$$N.F. = R_Y = 247.2 \text{ kN}$$

Check Buckling.

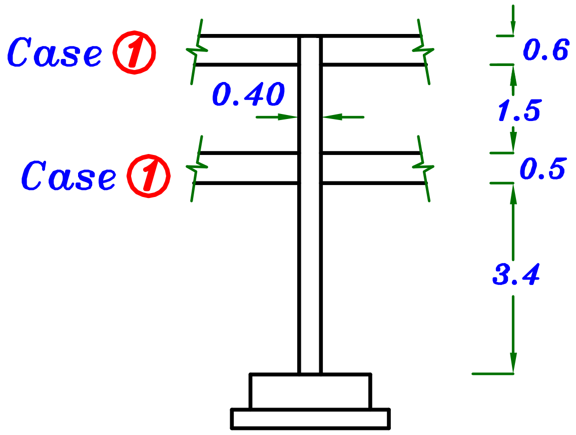
① In plane.



$$H_o = 6.0 \text{ m}$$

$$\lambda_b = \frac{K * H_o}{t} = \frac{1.3 * 6.0}{0.7} = 11.14 > 10$$

② Out of plane.



$$H_o = 3.4 \text{ m}$$

$$\lambda_b = \frac{K * H_o}{b} = \frac{1.2 * 3.4}{0.40} = 10.2 > 10$$

$$\delta = \frac{(\lambda_b)^2 * t}{2000} = \frac{11.14^2 * 0.70}{2000} = 0.043 \text{ m}$$

$$M_{add.} = P * \delta = 247.2 * 0.043 = 10.62 \text{ kN.m}$$

$$e = \frac{M}{P} = \frac{10.62}{247.2} = 0.043 \ m \quad \therefore \frac{e}{t} = \frac{0.043}{0.70} = 0.061 \ m < 0.5 \xrightarrow{use} I.D.$$

$$\zeta = \frac{0.7-0.1}{0.7} = 0.80 \xrightarrow{use} \text{Tables Page } 4-24$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{247.2 * 10^3}{25*400*700} = 0.035 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{10.62 * 10^6}{25*400*700^2} = 0.0021 \end{aligned}\right\} \rho < 1.0 \xrightarrow{Take} \rho = 1.0$$

$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

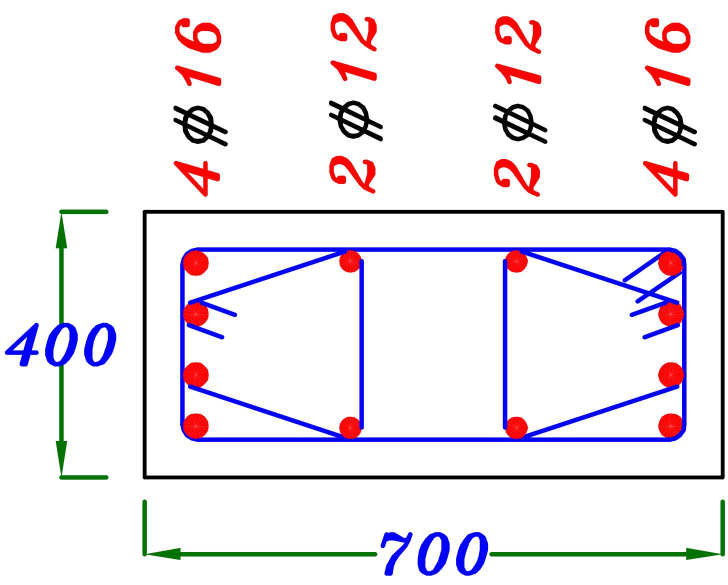
$$A_s = A_s` = \mu * b * t = 2.5 * 10^{-3} * 400 * 700 = 700 \ mm^2$$

$$A_{s\ total} = A_s + A_s` = 1400 \ mm^2$$

$$A_{s_{min}} = \frac{0.25 + 0.052\ \lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052\ (11.14)}{100} * 250 * 700 = 1451.2 \ mm^2$$

$$A_s = A_s` = \frac{A_{s_{min}}}{2} = \frac{1451.2}{2} = 725.6 \ mm^2 \quad 4\phi16$$



RFT. of the Arch slab.