Diseño de viga a corte sismo-resistente

Armado a flexión

			3#5	5.94 cm^2			
2#5=	3.96 cm^2					3#5	5.94 cm^2
1#3	0.71 cm^2					1#4	1.27 cm^2
			3#5	5.94 cm^2			

manual

Datos:

Viga:

h =	50.00	cm			
b =	25.00	cm			
d =	44.44	cm	Peralte		
L = Ln =	550.00	cm =	5.50	m	

Materiales:

f'c =	210.97	kg/cm²
Fy = Fyt =	4219.41	kg/cm²

Cargas:

Wv =	850.00	kg/m	Carga viva
Wm =	1000.00	kg/m	Carga muerta
Svd =	0.20	Coeficiente de sismo vertical	
Pu =	0.00	Carga axial en la viga	

Diagrama de corte ultimo

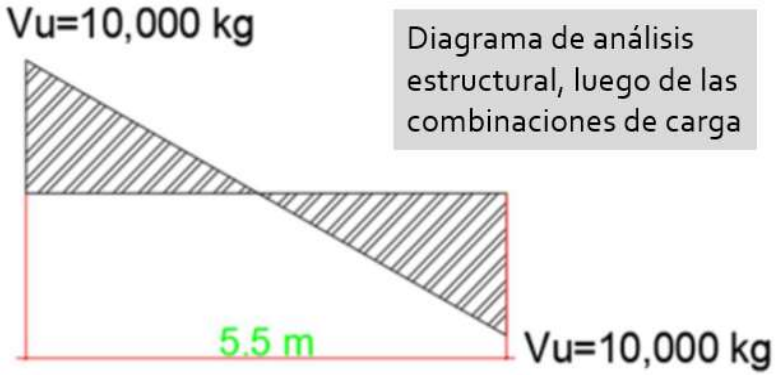
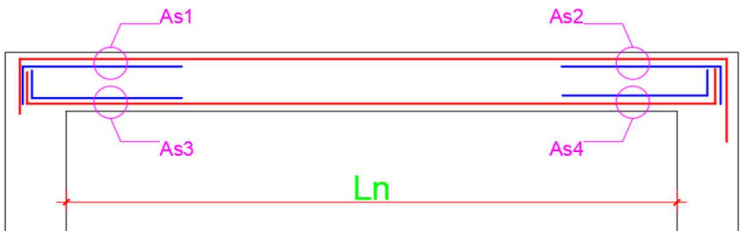


Diagrama de análisis estructural, luego de las combinaciones de carga

1) Momentos probables



As1 = 9.90 cm² As2 = 11.88 cm²

As3 = 6.65 cm² As4 = 7.21 cm²

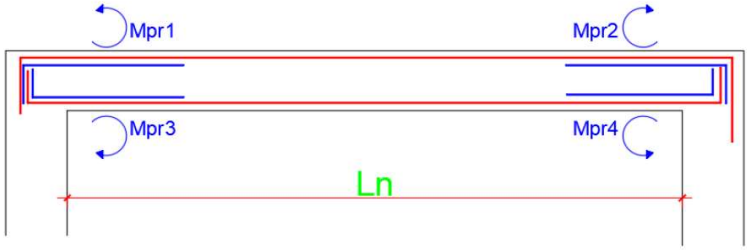


TABLE A3.1
R_y and R_t Values for Steel and Steel Reinforcement Materials

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M	1.5	1.2
• ASTM A1043/1043M Gr. 36 (250)	1.3	1.1
• ASTM A572/572M Gr. 50 (345) or 55 (380), ASTM A913/A913M Gr. 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M	1.1	1.1
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
• ASTM A529 Gr. 50 (345)	1.2	1.2
• ASTM A529 Gr. 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500/A500M (Gr. B or C), ASTM A501	1.4	1.3
Pipe:		
• ASTM A53/A53M	1.6	1.2
Plates, Strips and Sheets:		
• ASTM A36/A36M	1.3	1.2
• ASTM A1043/1043M Gr. 36 (250)	1.3	1.1
• A1011/A1011M HSLAS Gr. 55 (380)	1.1	1.1
• ASTM A572/A572M Gr. 42 (290)	1.3	1.0
• ASTM A572/A572M Gr. 50 (345), Gr. 55 (380), ASTM A588/A588M	1.1	1.2
• ASTM 1043/1043M Gr. 50 (345)	1.2	1.1
Steel Reinforcement:		
• ASTM A615, ASTM A706	1.25	1.25

$$Mpr = As * \alpha y * Fy * \left(d - \frac{a}{2}\right) \qquad a = \frac{As * \alpha y * Fy}{0.85 * f'c * b}$$

αy = 1.25 f'c = 210.97 kg/cm²

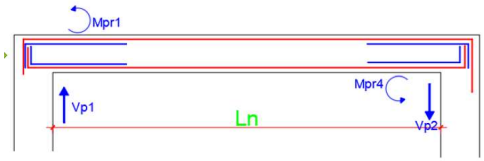
Fy = 4219.41 kg/cm² b = 25.00 cm

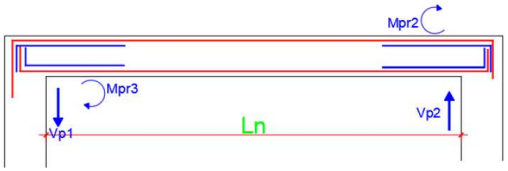
d = 44.44 cm

Mpr1 = 20,163.66 kg-m Mpr2 = 23,466.61 kg-m

Mpr3 = 14,214.81 kg-m Mpr4 = 15,286.58 kg-m

2) Corte probable sismico



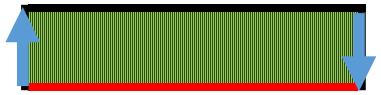


$$Vp1 = Vp2 = \frac{Mpr1 + Mpr4}{Ln} \qquad Vp1 = Vp2 = \frac{Mpr2 + Mpr3}{Ln}$$

Vp1 = 6,445.50 kg Vp1 = 6,851.17 kg

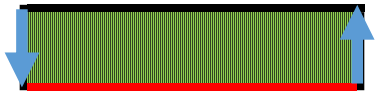
Vp2 = 6,445.50 kg Vp2 = 6,851.17 kg

Vp1 = 6,445.50



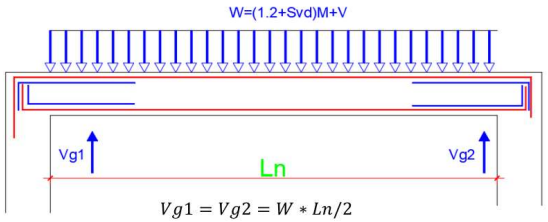
Vp2 = 6,445.50

Vp1 = 6,851.17



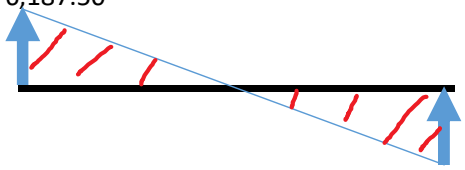
Vp2 = 6,851.17

3) Corte probable gravitacional



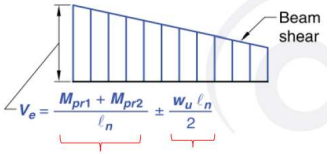
Vg1 = Vg2 = 6,187.50 kg

6,187.50



6,187.50

4) Corte total



$$Ve = Vp + Vg$$

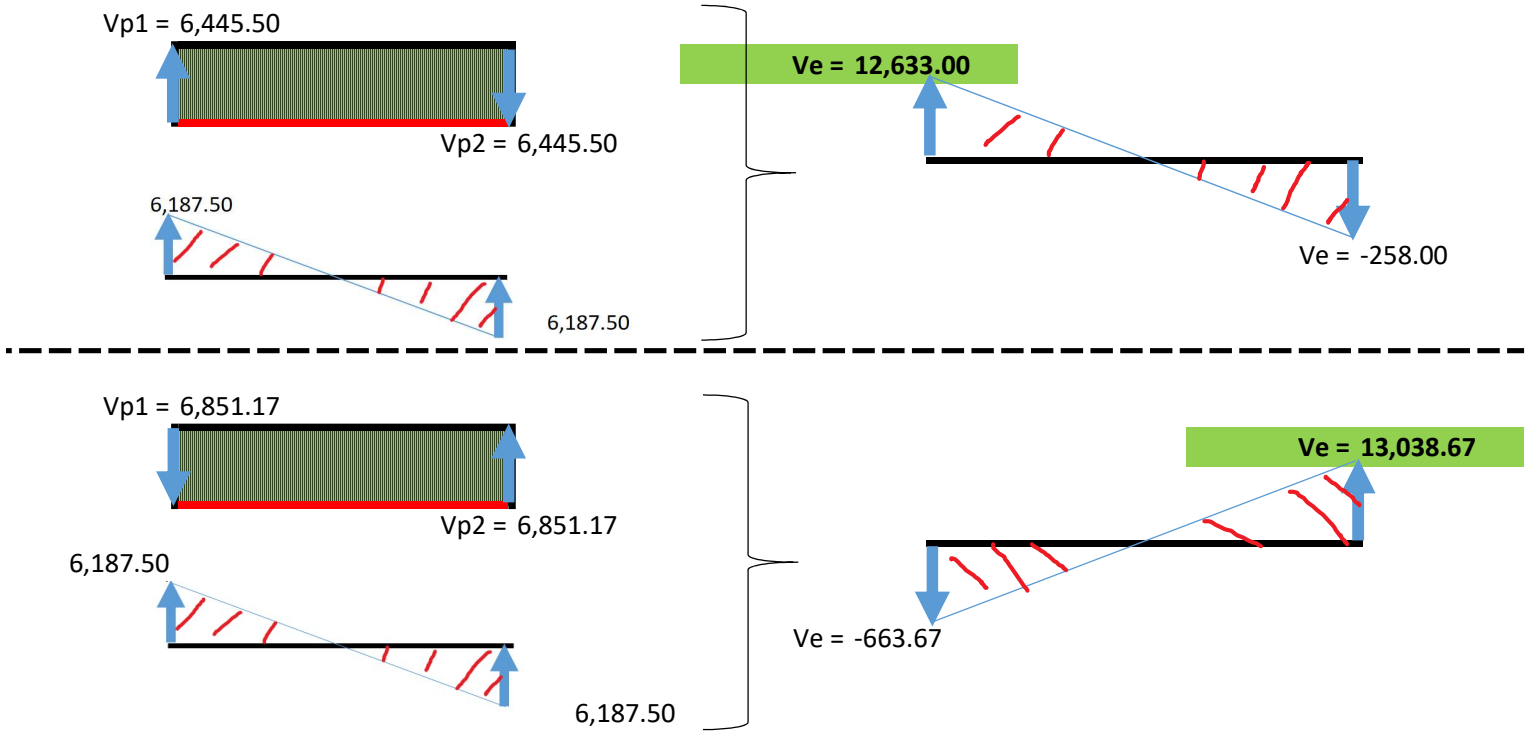
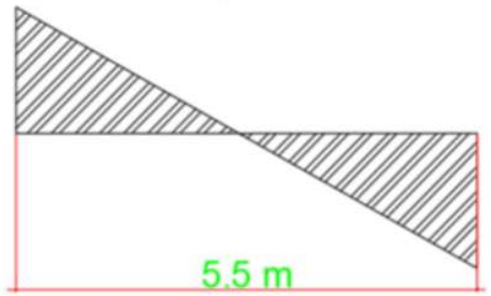


Diagrama de corte ultimo Por caga externa

Vu = 10,000.00 kg



vu = 10,000.00 kg

$$Vu = Vmax = Ve = \text{corte acutante maximo ultimo}$$

Para el diseño a corte, se toma el valor mayor entre Ve y Vu

DISEÑO A CORTE

5) Resistencia nominal a corte del concreto para viga

- **Resistencia nominal a corte del concreto para vigas:**

$$Vc = 0.53 * \lambda * \sqrt{f'c} * b * d$$ 22.5.5.1 ACI 318-19

Vc = 8,552.64 kg

Donde:

λ=	1	por Wc = 2400 kg/m³

6) Resistencia ultima a corte del concreto

ϕVc = 5,131.59 kg

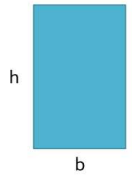
ϕ = 0.6

ϕ = factor de reducción de capacidad por cortante
- ϕ = 0.75 (Gravitacionales)
- ϕ = 0.60 zona sísmica

7) Revision de seccion de la viga

$Vu > \phi Vcmax$ Sección muy pequeña para resistir la fuerza cortante ultima

$Vu < \phi Vc/2$ Sección muy grande para soportar la fuerza cortante



$$\phi Vmax = \phi * (Vc + 2.2\sqrt{f'c} * b * d)$$ 22.5.1.2 ACI 318-19

$\phi Vmax$ = 26,432.50 kg

$Vu > \phi Vcmax$ Sección muy pequeña para resistir la fuerza cortante ultima

13,038.67 > 26,432.50 La seccion NO es pequeña, ok

$Vu < \phi Vc/2$ Sección muy grande para soportar la fuerza cortante

13,038.67 < 2,565.79 La seccion No es grande, OK

Calculo de estribos: 9.6.3 ACI 318-19

Si $\phi Vc/2 < Vu < \phi Vc$ → Colocar Avmin ó Estribo No. 3 @ d/2

$Vu > \phi Vc$ → Se diseñan estribos

12,633.00
13,038.67 > 5,131.59 DISEÑE ESTRIBOS

Se debe de omitir la resistencia del concreto Vc=0 para el calculo de refuerzo de vigas si:

a) Vp > 0.5*Ve (si se, cumple Vc = 0, si no se cumple considera Vc)

b) Pu < Ag*f'c/20

(Considerar carga axial Pu = 0 en la viga)

a) Vp = 6,445.50
Ve = 12,633.00 } Vc =0

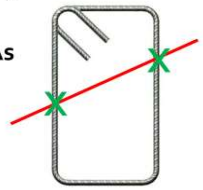
a) Vp = 6,851.17
Ve = 13,038.67 } Vc =0

b) Pu = 0.00 kg
Ag*f´c/20 = 13,185.63 kg } Vc = 0

8) Area de varilla

Estribos:

Av = 2As



Suponiendo usar, estribo # 3

As = 0.713 cm²

Av = 1.425 cm²

9) Calculando Vs

$$Vs = \frac{(V_u - \phi * Vc)}{\phi}$$

Vu = 12,633.00 kg

$Vs = \frac{(12633 - 0)}{0.60}$

Vs = 21,055.00 kg

Vu = 13,038.67

$Vs = \frac{(13038.67 - 0)}{0.60}$

Vs = 21,731.11 kg

10) Separacion de estribos "S"

$$S = \frac{A_v F_{yt} d}{V_S}$$

$S = \frac{(1.425 * 4219.41 * 44.44)}{21055}$ $S = \frac{(1.425 * 4219.41 * 44.44)}{21731.11}$

S = 12.69 cm S = 12.30 cm

11) Confinamiento de estribos

Longitud de confinamiento L = 2h 18.6.3.3 ACI 318-19

Separación de estribos (S_1) En zona confinada 18.6.4.4
- * d/4
- * 6ϕmin (Acero grado 60°)
- * 15 cm

Menor varilla longitudinal

Longitud = 2h = 2*50 = 100.00 cm ⟹ 1.00 m

Separacion de estribos en zona confinada

d/4 = 44.44/4 = 11.11 cm

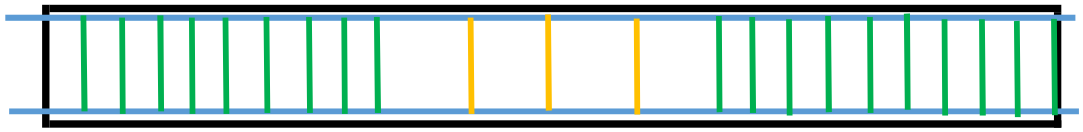
S1 < 6ϕmin 6*1.59 9.53 cm Menor varilla longitudinal # 5

15 cm 15.00 cm Diam = 1.59 cm

9 cm

S1 = 9.53 cm

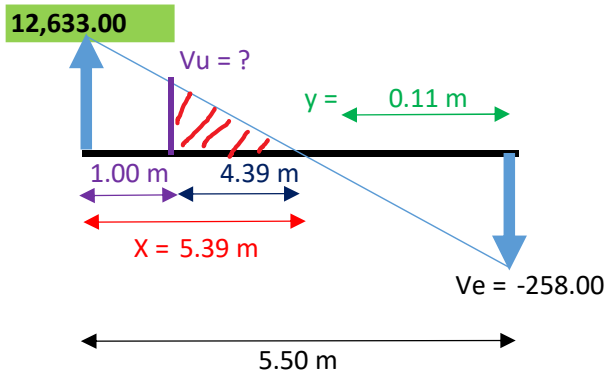
10 cm



Est. #3 @ 0.10	Est #3 @???	Est. #3 @ 0.10
1.00 m	3.50 m	1.00 m

Separacion de estribos en zona no confinada

Smax = d/2 = 44.44/2 = **22.22 cm**



Vu = **10289.18 kg**

Calculando Vs

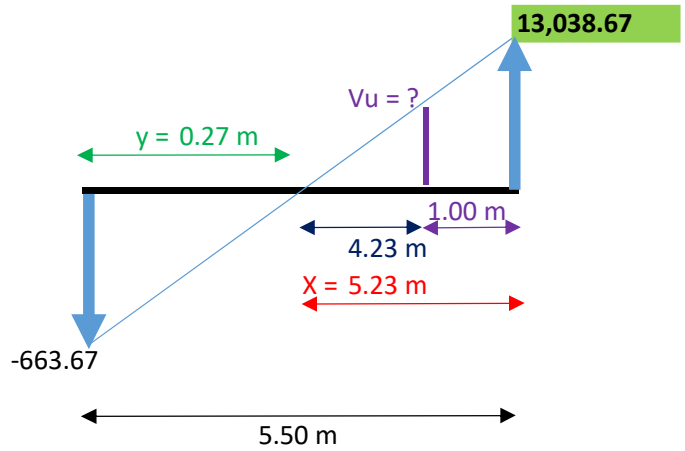
$$Vs = \frac{(V_u - \phi * Vc)}{\phi}$$

$$Vs = \frac{(10289.18 - 0)}{0.60}$$

Vs = **17148.63 kg**

$$S = \frac{(1.425 * 4219.41 * 44.44)}{17148.63}$$

S = **15.58** cm → 16.00 cm / 15.00 cm



Vu = **10547.33 kg**

Calculando Vs

$$Vs = \frac{(V_u - \phi * Vc)}{\phi}$$

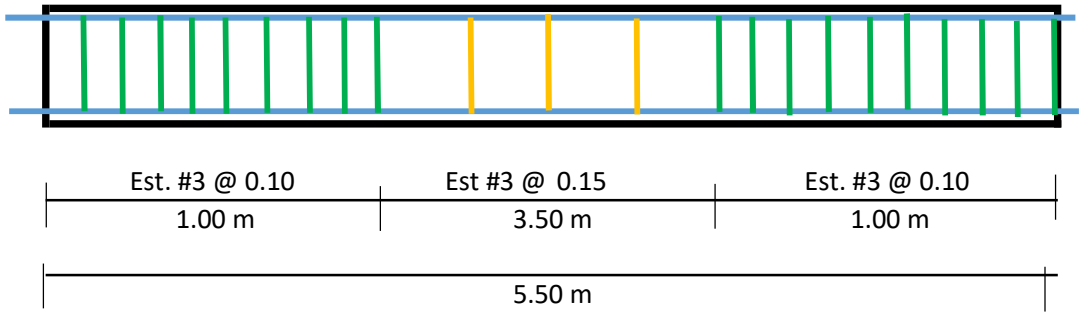
$$Vs = \frac{(10547.33 - 0)}{0.60}$$

Vs = **17578.89 kg**

$$S = \frac{(1.425 * 4219.41 * 44.44)}{17578.89}$$

S = **15.20** cm → 16.00 cm / 15.00 cm

R//



12) Calculo de Avmin

$$A_{v,min} \geq 0.2\sqrt{f'_c}\frac{b_w s}{f_{yt}}$$ Avmin ≥ $0.2*\sqrt{210.97}*\frac{(25*15)}{4219.41}$ 0.258 cm2

$$A_{v,min} \geq 3.5\frac{b_w s}{f_{yt}}$$ Avmin ≥ $3.5*\frac{(25*15)}{4219.41}$ 0.311 cm2

Avmin = 0.311 cm2

Chequeo:

$A_v > Av_{min}$

1.425 > 0.311 El estribo propuesto OK, # 3