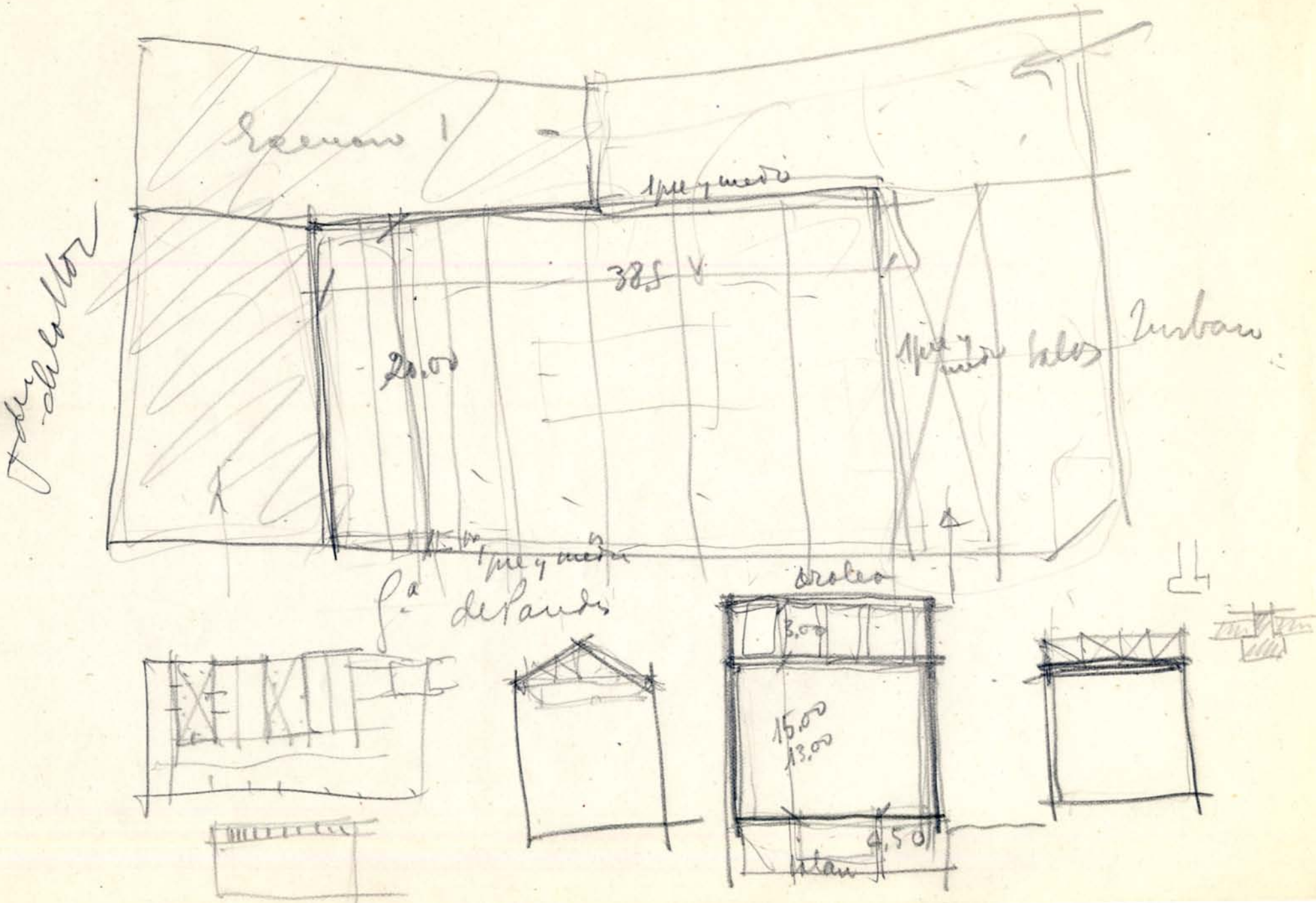

Frente 1
1 pie y medio
38.5
20.00
1 pie y medio
Trabes
1 pie y medio
Fa de Pared
Arcos
3.00
15.00
13.00
4.50

VIGA VIERENDEL

PRIMER TANTEO : BARRAS DE MOMENTO DE INERCIA CONSTANTE

Separación : 4 metros

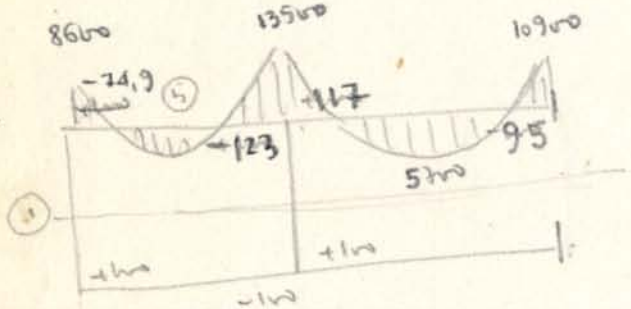
Cargas: peso propio: $2 \times 0.70 \times 0.4 \times 2500 = 1400$
$0.35 \times 0.4 \times 2500 = 350$
1750

forjado : $2 \times 4 \times 250 = 2000$

total 3750 kg/ml

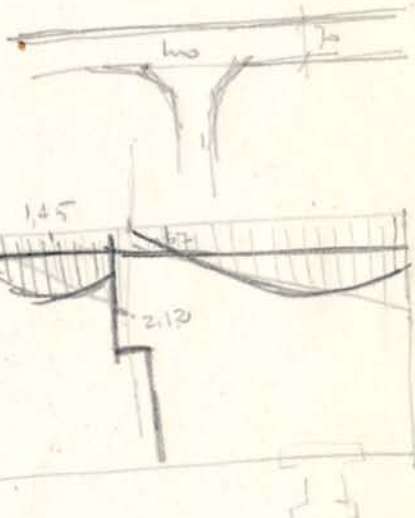
sobrecarga: $2 \times 4 \times 250 = 2000$ kg/ml

5750 "

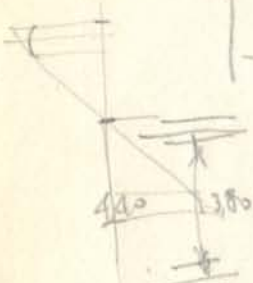


$I_{vo} \Rightarrow \frac{1}{12} 5750 \times 5^2 = \sim 11500$ m/kg

$P = 5.750 \times 5 = 28750$ kg



$19,3\alpha =$	$+37,9\alpha$	$-28,8\alpha$	$-14,4\alpha$
$-5,7\beta =$	$-38,5\beta$	$+52,7\beta$	$+76,7\beta$
+1,87	+3,66	-2,78	-1,39
-0,23	-1,54	+2,11	+3,07
+1,64	+2,12	-0,67	+1,68



$M_{máx} = 1.68 = 48300$: $\alpha = 1,50$ " $M_{1a} = 32,200$ Tm

$M_{máx} = 2,12 = 61300$ $\frac{60}{120} \cdot 42 \cdot 1,5 = 63 cm^2$

$1,4\beta = 41500$ " $M_{1a} = 1m \cdot Tm$ $50 \cdot 1,5 = 75 cm^2$

$\frac{13500}{28000}$: 40 Tm 10 ϕ 30

$M_{1a} = 40$ " 7 T/m

47500

43m

1300 Pilar

882

2750

00525

1540

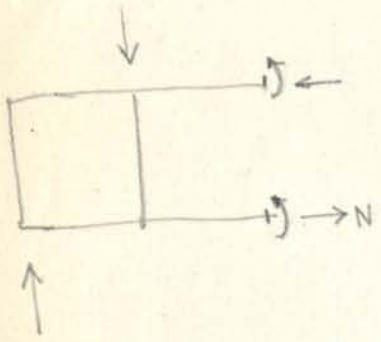
2108

3068

40 mm m/kg : 0.4 = Inercia $\sqrt{M_{1a}} = 316$ $\frac{c}{i} = \frac{80}{316}$

1000 mm $0.15 \times 1.8 = 0.40 = 5$ cm

SEGUNDO TANTEO = BARRAS CON ACARTELAMIENTO



$N \cdot H + \Sigma M_2 + M = 0 \qquad N = \frac{M - \Sigma M_2}{H}$

$M_2 = \frac{1}{8} 5750 \times 10^2 = 71800$ mkg $\qquad 2M_2 = 2 \times 37400 = 74800$

$M_1 = \frac{1}{10} 5750 \times 10^2 = 57500 \qquad 2M_1 = 2 \times 47500 = 89000$

$N_1 = \frac{89000 - 57500}{4} = \frac{31500}{4} = 8000$ kg

COMPROBACION PILARES

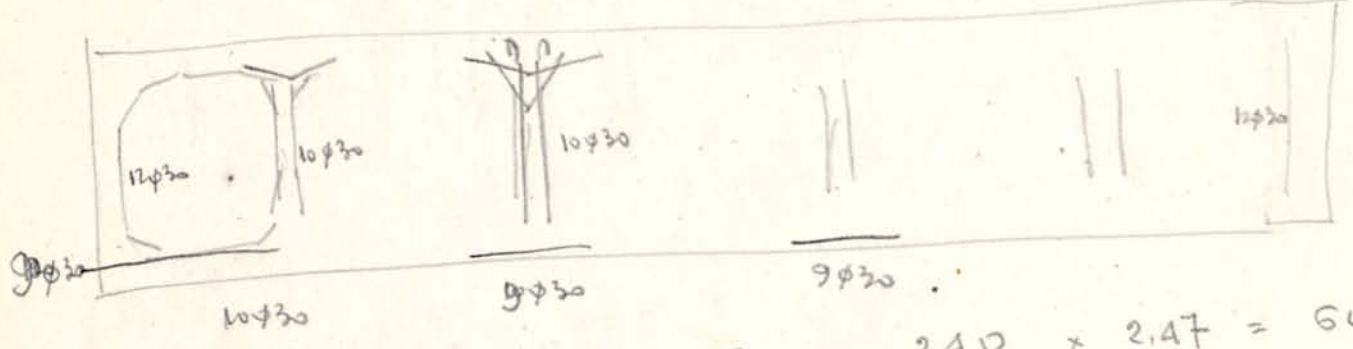
Mex. momento sin redondeamiento: 37 000 mkg

sección 120 x 40 : $M_{ia} = \sim 90$ .. $\sqrt{M_{ia}} = 300$ .. $\xi = \frac{110}{300} = 0.37$ .. $\rho = 0.008$ $\omega = 35\ cm^2$ 5φ30

sin acartelamiento : sección 40×40 Mex M = 21 000 mkg

$M_{ia} = 50$ .. $\sqrt{M_{ia}} = 224$ .. $\xi = \frac{40}{224} = 0,18$ .. $\rho = 0.030$ $\omega = 48\ cm^2$

Hormigón: $20,50 \times 4,50 \times 0,40 = 37\ m^3 - 5 + 3,4 \times 3 \times 0,4 = 20,4 = 17\ m^3$



5+5 φ20	= 10 φ20	+ 20 + 1.20	= 240	× 2,47 = 600
10+10 φ30	= 20 φ30	+ 20 + 1.20	= 480	} 530 × 5,55 = 3050
20+20 φ30	= 4 φ30	× 5 × 1.20	= 50	3650
Cercos: 14 × 20 + 2 × 2.20			= 1230	} 1470 × 0,62 = 880
Cercos 5 × 4 × 6 × 2.00			= 240	4530

kg/m³ = $\frac{4530}{178}$ = 25 kg/m³

56 112 1232

Piso INFERIOR:

Superficie: $38.5 \times 20 = 770$

Volumen: $770 \times 0.25 = 170\ m^3$

Tn de hierro: $170 \times 130 = 22100$ kgs

VIGAS VIERENDEEL $n° = 10$

Hormigón: $170\ m^3$

Hierro: 45 Tons

PILARES $n° = 20$

40T ↓

Hormigón $15 \times 0.40 \times 0.50 = 3 m^3 \times 20 = 60\ m^3$

Hierro: $100\ kg/m^3$ 6 Tons

VIGAS DE ARRIOSTRAMIENTO: $n° = 6$

Hormigón: $6 \times 20 \times 0.50 \times 0.4 =$ $24\ m^3$

Hierro: 3 Tons

		Hormigón	Acero
Total:	Piso inf:	170	22
	Vigas:	170	45
	Pilares sup:	84	9
		$424\ m^3$	76 Tons
	Pts	127,2	228 ores

355 ores pts

Con un solo piso:

relación de cargas: $\frac{3750}{5750} = 0.65$

En viga: $0.65 \times 61\,000 = 40\,000$ .. M:e = 27 Tm

$38.7 + 1.5 = 57\ cm^2$

$60/75 = 0.80$

Ahora en tiempo: $0.20 \times 45 = \sim 10$ Tm.

VIGA VIERENDEL Luz 16.00

Carga por mt. Muro: 1 × 0.4 × 1800 = 5000 kg/mt

Techo: 6.00 × 300 = 1800

6800

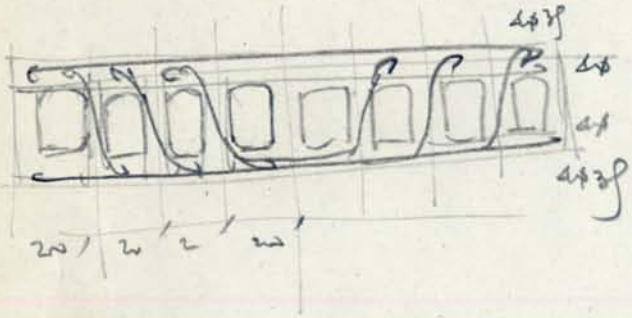
Al luz 2.00 Mzximo momento flector $\frac{1}{10} pL^2 = \frac{1}{10} 6800 \times 16^2 = 174000$

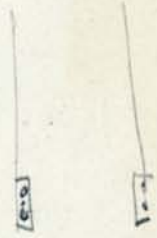
4300000

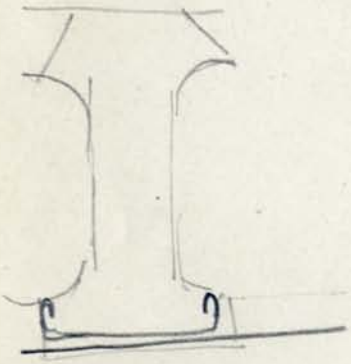
e = 2.00 a = 0.4

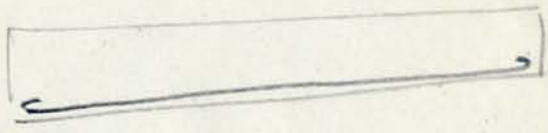
50 / 1200 } 1.0 × 80 = 80 cm² → 8 φ 3/4

0.5 = 40 cm² → 4 φ 3/4

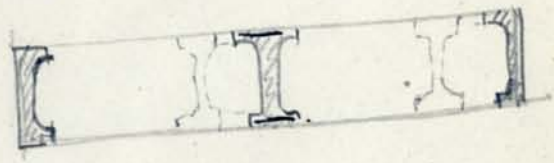






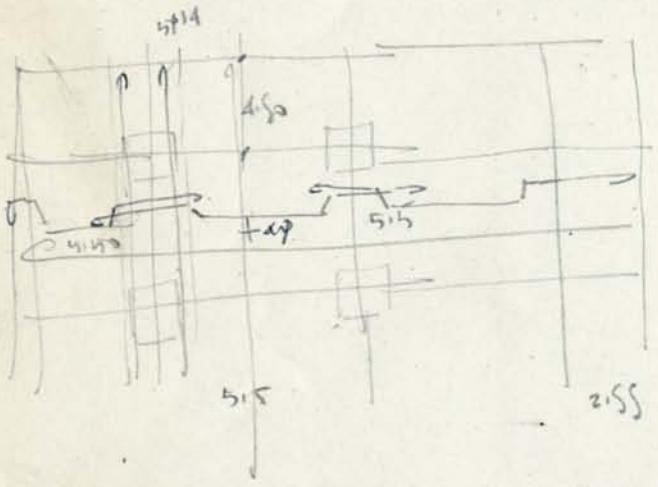


nota
pue vetas
un φ 3/4 por
cada lado



Apoyos Vierendel $\frac{1}{2}$ 16.00 × 6800 = 52 tn

LOSA SOBRE COLUMNAS



cargas p.p.		500
Sobrecarga		1000
Total		1500

Formulas de Ritz : Centro $\frac{1}{30} 1500 \cdot 5.5^2 = 1500$ mkg

Bandas cabeza : $\frac{1}{25} 1500 \cdot 5.5^2 = 1800$ "

Armadura : Centro : $8.00\ cm^2$ $\left.\begin{matrix}25 \\ 120\end{matrix}\right\}$ 6φ14 $19.3\ cm^2$

Bandas : $9.5\ cm^2$ $\left.\begin{matrix}38 \\ 120\end{matrix}\right\}$ 8φ14 $18.9\ cm^2$

