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ISTITUTO AUTONOMO CASE POPOLARI PER LA
PER LA PROVINCIA DI BRINDISI

COSTRUZIONE CASE POPOLARI IN BRINDISI - QUARTIERE S. ELIA

RELAZIONE DI CALCOLO DELLE STRUTTURE IN CALCESTRUZZO ARMATO

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STUDIO TECNICO
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B R I N D I S I

QUARTIERE SANT' ELIA

Carichi unitari:

Solaio

p.p. solaio	250 Kg/mq
inc. tramezzi	100 "
pav. + int.	100 "
	450 Kg/mq
c.a.	250 "
	700 Kg/mq

balconi

p.p. solaio	250 Kg/mq
pav. + int.	100 "
	350 Kg/mq
c.a.	400 "
	750 Kg/mq

muratura a cassetta

mattoni forati	120 Kg/mq
	150 "
int. int.	30 "
int. est.	50 "
	350 Kg/mq
$350 \times 2,80 =$	980 Kg/ml

scale

sottogrado $0,05 \times 2500$	120 Kg/mq
gradino $\frac{0,16 \times 0,30 \times 3 \times 2500}{2}$	200 "
rivestimento	100 "
intonaco	30
	450 Kg/mq
c.a.	400 "
	850 Kg/mq

PILASTRI 1 - 9 - 19 - 27

m. attico	$350 \times \frac{2,60+4,8}{2}$	1300 Kg	
travi	$400 \times \frac{2,3 + 5,0}{2}$	1450 Kg	
solaio	$700 \times \frac{2,45 \times 5,0}{4}$	2150 Kg	
muratura	$980 \times \frac{2,3 + 5,0}{2}$	3550 Kg	
pilastro		<u>750 Kg</u>	
c.a. quota 11.10		9200 Kg	9200 Kg

Piano tipo:

travi		1450 Kg	
solaio		2150 Kg	
muratura		3550 Kg	
pilastro		<u>1000 Kg</u>	
		8150 Kg	
8150 x 4		32600 Kg	32600 Kg
a detrarre murat. piano cantinato			<u>- 3550 Kg</u>
scarico pilastro			38250 Kg

PILASTRI 2 - 20 - 7 - 24 - 26

m. attico	$350 \times 2,6$	910 Kg	
travi	$400 \times 2,3$	920 Kg	
solaio	$700 \times 2,6 \times \frac{5,0}{2}$	4600 Kg	
muratura	$980 \times 2,3$	2300 Kg	
pilastro		<u>750 Kg</u>	
c.a. quota 11.10		9480 Kg	9480 Kg

Piano tipo:

travi		920 Kg	
solaio		4600 Kg	
balcone	$750 \times \frac{2,6}{2} \times 1,2$	1200 Kg	

muratura	2300 Kg	
pilastro	<u>1000 Kg</u>	
	10020 Kg	
10.020 x 3	30060 Kg	30.060 Kg

Piano cantinato:

travi	920 Kg	
solaio	4600 Kg	
pilastro	<u>1000 Kg</u>	
	6520 Kg	<u>6.520 Kg</u>
scarico pilastro		46.060 Kg

PILASTRI 3 - 6 - 21 - 25

m. attico 350 x 2,6	910 Kg	
travi 400 x 2,3	920 Kg	
solaio 700 x 2,6 x $\frac{5,0}{2}$	4600 Kg	
muratura 980 x 2,3	2300 Kg	
pilastro	<u>750 Kg</u>	
c.a. quota 11,10	9480 Kg	9.480 Kg

Piano tipo:

travi	910 Kg	
solaio	4600 Kg	
muratura	2300 Kg	
balconi 750 x 2,6 x 1,2	2300 Kg	
pilastro	<u>1000 Kg</u>	
	11.110 Kg	
11.110 x 3 =		33.330 Kg

Piano cantinato:

travi	910 Kg	
solaio	4600 Kg	
pilastro	<u>1000 Kg</u>	
	6510 Kg	<u>6.510 Kg</u>
scarico pilastro		49.320 Kg

PILASTRI 4 - 5

cop. scala	$600 \times \frac{2,6}{2} \times \frac{5,0}{2}$	1950 Kg	
trave	$400 \times \frac{2,3 + 5,0}{2}$	1450 Kg	
m. attico	$350 \times 2,6$	900 Kg	
travi	$400 \times \frac{2,3 + 5,0}{2}$	1900 Kg	
solaio	$700 \times \frac{2,6 \times 5,0}{4}$	2300 Kg	
scala	$850 \times \frac{2,6 \times 5,0}{4}$	2800 Kg	
scala	$850 \times 0,5 \times 2,6$	1100 Kg	
muratura	$0,80 \times \frac{3,3 + 2,3 + 5,0}{2}$	5200 Kg	
pilastro		<u>750 Kg</u>	
carico a quota 11,10		18350 Kg	18.350 Kg

Piano tipo:

solaio		2300 Kg	
scala	$2800 + 1100$	3900 Kg	
balcone	$750 \times \frac{2,6}{2} \times 1,2$	1250 Kg	
muratura		5200 Kg	
travi		1900 Kg	
pilastro		<u>1000 Kg</u>	
		15.550 Kg	
15.550×3			46.650 Kg

Piano cantinato:

solaio		2300 Kg	
travi		1900 Kg	
pilastro		<u>1000 Kg</u>	
		5200 Kg	5.200 Kg
scarico pilastro			<u>70.200 Kg</u>

PILASTRO 8

m. attico	350 x 2,6	910 Kg	
travi	400 x 2,3	920 Kg	
solaio	700 x 2,6 x $\frac{5,0}{2}$	4600 Kg	
muratura	980 x 2,3	2300 Kg	
pilastro		<u>750 Kg</u>	
carico a quota 11.10		9480 Kg	9.480 Kg

Piano tipo:

travi	920 Kg	
solaio	4600 Kg	
muratura	2300 Kg	
pilastro	<u>1000 Kg</u>	
	8820 Kg	
8820 x 3		26.460 Kg

Piano cantinato:

solaio	4600 Kg	
travi	920 Kg	
pilastro	<u>1000 Kg</u>	
	6520 Kg	6.520 Kg
scarico pilastro		<u>42.460 Kg</u>

PILASTRO 22

come analisi pilastro n. 20	46.100 Kg
muratura da cm. 20	
0,20 x 1400 x 3,00 x $\frac{4,6}{4}$ x 4	<u>3.900 Kg</u>
	50.000 Kg

PILASTRO 23

come analisi pilastro n. 8	42.500 Kg
muratura da cm. 20	
0,20 x 1400 x 3,00 x $\frac{4,6}{4}$ x 4	<u>3.900 Kg</u>
	46.400 Kg

PILASTRI 10 - 18

attico	350 × 5,45	1900 Kg	
solaio	700 × $\frac{2,45}{2}$ × 5,0	4300 Kg	
travi	400 × $\frac{2,3}{2}$ + 5,0	2450 Kg	
muratura	980 × 5,0	4900 Kg	
pilastro		<u>750 Kg</u>	
carico a quota 11.10		14300 Kg	14.300 Kg

Piano tipo:

solaio	4300 Kg	
travi	2450 Kg	
muratura	4900 Kg	
pilastro	<u>1000 Kg</u>	
	12650 Kg	
12.650 × 3 =		37.950 Kg

Piano cantinato:

solaio	4300 Kg	
travi	2450 Kg	
pilastro	<u>1000 Kg</u>	
	7750 Kg	7.750 Kg
scarico pilastro		60.000 Kg

PILASTRI 11 - 12 - 15 - 16 - 17

Piano tipo:

solaio	700 × 2,6 × 5,0	9100 Kg	
travi	400 × 2,3	900 Kg	
pilastro		<u>1000 Kg</u>	
		11.000 Kg	
11.000 × 4			44.000 Kg

Piano cantinato:

solaio	9100	
travi	<u>900</u>	10.000 Kg
	10000	44.000 Kg

PILASTRI 14 - 13

copertura scala	$600 \times \frac{2,6 \times 6,5}{4}$	2500 Kg	
travi	$400 \times \frac{2,3 + 5,0}{2}$	1450 Kg	
muratura	$\frac{2}{3} 980 \times \frac{2,6 + 5,0}{2}$	2500 Kg	
pilastro		<u>750 Kg</u>	
		7200 Kg	7.200 Kg

Piano tipo:

solaio	$700 \times \frac{2,6 \times 5,0}{2}$	4500 Kg	
	$700 \times \frac{2,6 \times 5,0}{4}$	2300 Kg	
scala	$850 \times \frac{2,6 \times 5,0}{4}$	2750 Kg	
travi	$400 \times \frac{(2,3 + 5,0)}{2}$	1900 Kg	
muratura	$980 \times \frac{2,6 + 5,0}{2}$	3700 Kg	
pilastro		<u>1000 Kg</u>	
		16.150 Kg	
	$16.150 \times 4 =$		64.600 Kg

Piano cantinato:

solaio	$4500 + 2300$	6800 Kg	
travi		1900 Kg	
scala	$\frac{1}{2} 2700$	<u>1350 Kg</u>	
		9050 Kg	9.050 Kg
			<u>80.850 Kg</u>

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	CARICO (t)	DIMENSIO- NI (m)	Ac (cmq)	ϕ (mm)	Af (cmq)	σ_c (Kg/cmq)	STAFFE
1	38,3	30 x 30	900	4 ϕ 16	8.04	39,0	ϕ 6 / 15
2	46,1	30 x 35	1050	4 ϕ 16	8.04	40,9	ϕ 6 / 15
3	49,3	30 x 35	1050	4 ϕ 16	8.04	43,5	ϕ 6 / 15
4	70,2	35 x 40	1400	4 ϕ 20	12.57	46,1	ϕ 6 / 17,5
5	70,2	35 x 40	1400	4 ϕ 20	12.57	46,1	ϕ 6 / 17,5
6	49,3	30 x 35	1050	4 ϕ 16	8.04	43,5	ϕ 6 / 15
7	46,1	30 x 40	1050	4 ϕ 16	8.04	40,9	ϕ 6 / 15
8	42,5	30 x 40	900	4 ϕ 16	8.04	43,3	ϕ 6 / 15
9	38,3	30 x 30	900	4 ϕ 16	8.04	39,0	ϕ 6 / 15
10	60,0	30 x 40	1200	4 ϕ 18	10.18	43,9	ϕ 6 / 15
11	44,0	30 x 30	900	4 ϕ 16	8.04	44,8	ϕ 6 / 15
12	44,0	30 x 30	900	4 ϕ 16	8.04	44,8	ϕ 6 / 15
13	80,9	35 x 45	1575	6 ϕ 16	12.06	47,7	ϕ 6 / 17,5
14	80,9	35 x 45	1575	6 ϕ 16	12.06	47,7	ϕ 6 / 17,5
15	44,0	30 x 30	900	4 ϕ 16	8.04	44,8	ϕ 6 / 15
16	44,0	30 x 30	900	4 ϕ 16	8.04	44,8	ϕ 6 / 15
17	44,0	30 x 30	900	4 ϕ 16	8.04	44,8	ϕ 6 / 15
18	60,0	30 x 40	1200	4 ϕ 18	10.18	43,9	ϕ 6 / 15
19	38,3	30 x 30	900	4 ϕ 16	8.04	39,0	ϕ 6 / 15
20	46,1	30 x 35	1050	4 ϕ 16	8.04	40,9	ϕ 6 / 15
21	49,3	30 x 35	1050	4 ϕ 16	8.04	43,5	ϕ 6 / 15
22	50,0	30 x 35	1050	4 ϕ 16	8.04	44,3	ϕ 6 / 15
23	46,4	30 x 35	1050	4 ϕ 16	8.04	41,0	ϕ 6 / 15
24	46,1	30 x 35	1050	4 ϕ 16	8.04	40,9	ϕ 6 / 15
25	49,3	30 x 35	1050	4 ϕ 16	8.04	43,5	ϕ 6 / 15
26	46,1	30 x 35	1050	4 ϕ 16	8.04	40,9	ϕ 6 / 15
27	38,3	30 x 30	900	4 ϕ 16	8.04	39,0	ϕ 6 / 15

FONDAZIONI

PILASTRATA 1 - 9

L = 1.80 m

Scaridi pilastri:	$\frac{1}{2} 38,3 + 42,5 + 49,3 + 2 \times \frac{2}{3} 70,2 +$	
	$+ 49,3 + 46,1 + \frac{1}{2} 38,3 =$	365,1 t
muratura P.C.:	$8 \times 2,30 \times 1400 =$	25,0 "
p.p. fondazioni:	$0,96 \times 2500 \times 21,10 =$	50,8 "
terra sovrast.:	$(1,80 - 0,40) \times 1,60 \times 1600 \times$	
	$\times 21,10 =$	<u>75,2 "</u>
	Sommano	516,2 t

$$\sigma_t = \frac{516.200}{2110 \times 180} = 1,36 \text{ Kg/cm}^2$$

Carico al ml di trave rovescia:

$$p_c = \frac{365.200}{21,10} = 17.300 \text{ Kg/ml}$$

Data la uniformita' dei carichi, l'uguaglianza delle luci, la elevata rigidezza della trave rovescia, si calcolano le armature considerando la singola travata come incastrata sugli estremi:

$$M_a = \frac{1}{12} 17.300 \times 2,60^2 = 9700 \text{ Kgm}$$

$$M_m = \frac{1}{14,4} 17.300 \times 2,60^2 = 8100 \text{ Kgm}$$

H = 110 cm

h = 105 cm

b = 40 cm

n = 10

$$\sigma_f = 1400 \text{ Kg/cm}^2$$

Per $M_a = 9700 \text{ Kgm}$ si ha: $\sigma_c = 28 \text{ Kg/cm}^2$ Af = 6,95 cmq

si adoperano 5 ϕ 10 correnti 3.93

2 ϕ 14 monconi 3.08

6.93

Per $M_m = 8100 \text{ Kgm}$ si ha: $\sigma_c = 25 \text{ Kg/cm}^2$ Af = 5,75 cmq

si adoperano 3 ϕ 16 correnti = 6.03 cmq

Verifica la Taglio:

$$T = 17.300 \times \frac{2,60}{2} = 22.500 \text{ Kg}$$

$$\tau_{\max} = \frac{22.500}{0,9 \times 105 \times 40} = 5,95 \text{ Kg/cmq}$$

$$S = 5,95 \times 40 \times \frac{2,60}{4} = 15.500 \text{ Kg}$$

Adoperando Staffe $\phi 8 / 20$ a due braccia si ha:

$$N_s = 7 \quad A_s = 7 \times 1,01 = 7,1 \text{ cmq}$$

$$S_s = 7,1 \times 1400 = 9950 \text{ Kg}$$

$$A_{f_p} = \frac{15.500 - 9.950}{1400 \times \sqrt{2} \cos 15^\circ} = 2,9 \text{ cmq}$$

(si adoperano 2 $\phi 14 = 3,08 \text{ cmq}$)

MENSOLA:

$$M = 17.300 \times \frac{0,70^2}{2} = 4230 \text{ Kgm}$$

per $H = 40 \text{ cm}$

$b = 100 \text{ cm}$

$h = 37 \text{ cm}$

$\sigma_f = 1400 \text{ Kg/cmq}$

$n = 10$

si ha

$\sigma_c = 34 \text{ Kg/cmq}$

$A_f = 8,70 \text{ cmq/ml}$

Si adoperano 1 $\phi 14/40 \text{ cm} + 1 \phi 16/40 = 9,11 \text{ cmq/ml}$

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PILASTRATA 10 - 18

Scaridi pilastri:	$\frac{2 \times 1}{3} 60 + 5 \times 44 + 2 \times 80,9 =$	421,8 t
p.p. fondazione :	(come pilast. $1 \div 9$) =	50,8 t
terra sovrastante:	(" " ") =	<u>75,2 t</u>
	Sommano	547,8 t

$$\sigma_t = \frac{547.800}{2110 \times 190} = 1,35 \text{ Kg/cm}^2$$

Carico al ml. di trave rovescia:

$$P_c = \frac{421.800}{21,10} = 20.000 \text{ Kg/ml}$$

$$M_a = \frac{1}{12} 20.000 \times 2,6^2 = 11.120 \text{ Kgm}$$

$$M_m = \frac{1}{14,4} 20.000 \times 2,6^2 = 9.400 \text{ Kgm}$$

$$H = 110 \text{ cm} \quad h = 105 \text{ cm} \quad b = 40 \text{ cm} \quad n = 10$$

$$\sigma_f = 1400 \text{ Kg/cm}^2$$

$$\text{Per } M_a = 11.120 \text{ Kgm si ha: } \sigma_c = 30 \text{ Kg/cm}^2 \quad A_f = 7,9 \text{ cm}^2$$

si adoperano:	5 ϕ 10 correnti	3.93 cm ²
	2 ϕ 16 monconi	<u>4.02 "</u>
		7.95 cm ²

$$\text{Per } M_m = 9.400 \text{ Kgm si ha: } \sigma_c = 27.5 \text{ Kg/cm}^2 \quad A_f = 6,8 \text{ cm}^2$$

si adoperano:	3 ϕ 18 correnti	7.63 cm ²
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Verifica al taglio:

$$T = 20.000 \times \frac{2,6}{2} = 26.000 \text{ Kg}$$

$$\tau_{\max} = \frac{26.000}{0,9 \times 105 \times 40} = 6,9 \text{ Kg/cm}^2$$

$$S = 6,9 \times 40 \times \frac{260}{4} = 17.900 \text{ Kg}$$

Adoperando Staffe ϕ 10 / 25 cm a due braccia

$$A_s = 6 \times 1,57 = 9,4 \text{ cmq}$$

$$S_s = 9,4 \times 1400 = 13.100 \text{ Kg}$$

$$Af_p = \frac{17.900 - 13.100}{1400 \sqrt{2} \cos 15^\circ} = 2,55 \text{ cmq}$$

Si adoperano 2 ϕ 16 = 4.02 cmq

MENSOLA:

$$M = 20.000 \times \frac{0,75^2}{2} = 5600 \text{ Kgm}$$

per H = 40 cm

h = 37 cm

b = 100 cm

$\sigma_f = 1400 \text{ Kg/cmq}$

n = 10

$\sigma_c = 41 \text{ Kg/cmq}$

Af = 12,01 cmq/ml

Si adoperano 1 ϕ 18/20 = 12,7 cmq/ml

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PILASTRATA 19 - 27

Scaridi pilastri:	$\frac{1}{2} \times 2 \times 38,3 + 3 \times 46,1 + 2 \times 49,3 +$	
	$+ 50 + 46,4 =$	371,6 t
muartura P.C.:	(come al pilastro 1÷9)	25,0 "
p.p. fondazione:	" " "	50,8 "
terra sovrastante:	" " "	<u>75,2 "</u>
		522,6 t

$$\sigma_t = \frac{522.600}{2110 \times 180} = 1,37 \text{ Kg/cm}^2$$

$$p_c = \frac{371.600}{21,10} = 17.600 \text{ Kg/ml}$$

$$M_a = 17.600 \times \frac{2,6^2}{2} \times \frac{1}{12} = 9.900 \text{ Kgm}$$

$$M_m = 17.600 \times \frac{2,6^2}{2} \times \frac{1}{14,4} = 8.300 \text{ Kgm}$$

$$H = 110 \text{ cm} \quad h = 105 \text{ cm} \quad b = 40 \text{ cm} \quad n = 10$$

$$\sigma_f = 1400 \text{ Kg/cm}^2$$

$$\text{per } M_a = 9.900 \text{ Kgm si ha: } \sigma_c = 28,5 \text{ Kg/cm}^2 \quad A_f = 7,2 \text{ cm}^2$$

si adoperano	4 ϕ 12 correnti	4,52 cm ²
	2 ϕ 14 monconi	<u>3,08 "</u>
		7,50 cm ²

$$\text{per } M_m = 8.300 \text{ Kgm si ha: } \sigma_c = 25 \text{ Kg/cm}^2 \quad A_f = 5,9 \text{ cm}^2$$

$$\text{si adoperano 3 } \phi 16 \text{ correnti} = 6,03 \text{ cm}^2$$

Si arma al taglio come la travata 1 ÷ 9 con staffe ϕ 8 / 20 a due braccia e 2 ϕ 14 monconi in corrispondenza dei pilastri.
Si arma la mensola come la travata 1 ÷ 9 con 1 ϕ 14 + 1 ϕ 16 alternati ogni 20 cm.

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TRAVATA 1 - 19

Scaridi pilastri:	$\frac{2 \times 1}{2} 38,3 + \frac{1}{3} 60 =$	58,3 t
muratura P.C.:	$10,0 \times 0,30 \times 2,30 \times 1400 =$	9,7 t
pp. fondazione:	$0,61 \times 10,9 \times 2500 =$	16,5 t
terra sovrastante:	$0,60 \times 10,9 \times 1,6 \times 1600 =$	<u>16,8 t</u>
Sommano		101,3 t

$$\sigma_t = \frac{101.300}{100 \times 1232} = 0,82 \text{ Kg/cm}^2$$

$$p_c = \frac{58.300}{12,32} = 4700 \text{ Kg/ml}$$

$$M_m = \frac{1}{14,4} 4700 \times 5,35^2 = 9300 \text{ Kgm}$$

$$M_a = \frac{1}{12} 4700 \times 5,35^2 = 11.100 \text{ Kgm}$$

$$H = 110 \text{ cm} \quad h = 105 \text{ cm} \quad b = 40 \text{ cm} \quad n = 10$$

$$\sigma_f = 1400 \text{ Kg/cm}^2$$

$$M_m = 9300 \text{ Kgm} \quad \sigma_c = 27 \text{ Kg/cm}^2 \quad A_f = 6,6 \text{ cm}^2$$

si adoperano	2 ϕ 14 diritti	3.08 cm ²
	2 ϕ 16 sagomati	<u>4.02 cm²</u>
		7.10 cm ²

$$M_a = 11.100 \text{ Kgm} \quad \sigma_c = 30 \text{ Kg/cm}^2 \quad A_f = 8,0 \text{ cm}^2$$

si adoperano	4 ϕ 10 correnti	3,14 cm ²
	3 ϕ 16 sagomati	<u>6,03 cm²</u>
		9.17 cm ²

Verifica la Taglio:

$$T = 4700 \times \frac{5,35}{2} = 12.500 \text{ Kg}$$

$$\tau_{\max} = \frac{12.500}{0,9 \times 105 \times 40} = 3,3 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

Si adoprano Staffe ϕ 8 / 20 a due braccia

MENSOLA:

$$M = 4700 \times \frac{0.3^2}{2} = 2100 \text{ Kgm}$$

Per H = 40 cm

h = 37 cm

b = 10 cm

n = 10

$\sigma_f = 1400 \text{ Kg/cmq}$

$\sigma_c = 25 \text{ Kg/cmq}$

$A_f = 4,3 \text{ cmq}$

Si adoperano 1 ϕ 12/25 cm

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TRAVI 5 - 14 $\equiv$ 4 - 13

$$\text{Scarico pilastri} \quad \frac{1}{3} \times 80,9 + \frac{1}{3} \times 70,2 = 50,4 \text{ t}$$

$$\text{p.p. fondazione} \quad 0,61 \times 5,7 \times 2500 = 8,7 \text{ t}$$

$$\text{muratura P.C.} \quad 5.00 \times 0,30 \times 2,30 \times 1400 = 4,8 \text{ t}$$

$$\text{terra sovrastante} \quad 0,60 \times 5,00 \times 1600 \times 1.60 = 7,7 \text{ t}$$

$$\text{Sommano} \quad 71,6 \text{ t}$$

$$\sigma_t = \frac{71.600}{710 \times 100} = 1,01 \text{ Kg/cmq}$$

$$P_c = \frac{50.400}{7.10} = 7100 \text{ Kg/cmq}$$

$$M_a = \frac{1}{12} 7100 \times 5.35^2 = 16.900 \text{ Kgm}$$

$$M_m = \frac{1}{14,4} 7100 \times 5.35^2 = 14.100 \text{ Kgm}$$

Per H = 110 cm

b = 40 cm

h = 105 cm

n = 0

$\sigma_f = 1400 \text{ Kg/cmq}$

$M_m = 14.100 \text{ Kgm}$

$\sigma_c = 34 \text{ Kg/cmq}$

$A_f = 10,1 \text{ cmq}$

Si adoperano 2 ϕ 18 diritti 5.08

2 ϕ 18 sagomati 5.08

10.16 cmq

$$M_a = 16.900 \text{ Kgm} \quad \sigma_c = 38 \text{ Kg/cm}^2 \quad Af = 12,0 \text{ cm}^2$$

$$\text{si adoperano } 5 \phi 12 \text{ correnti} \quad 5.65 \text{ cm}^2$$

$$2 \phi 18 \text{ sagomati} \quad 5.08 \text{ "}$$

$$1 \phi 18 \text{ moncone} \quad 2.54 \text{ "}$$

$$\underline{\hspace{1.5cm}} \\ 13.27 \text{ cm}^2$$

Verifica al Taglio:

$$T = 7100 \times \frac{5.35}{2} = 19.000 \text{ Kg}$$

$$\tau_{\max} = \frac{19.000}{0.9 \times 40 \times 105} = 5,02 \text{ cm}^2$$

$$S = 5.02 \times \frac{5.35}{4} \times 40 = 26.500 \text{ Kg}$$

Adoperando Staffe $\phi 8 / 20 \text{ cm}$

$$S_s = 14 \times 1.01 \times 1400 = 19.800 \text{ Kg}$$

$$Af_p = \frac{26.500 - 19.800}{\sqrt{2} \times 1400 \times \cos 15^\circ} = 3,5 \text{ cm}^2$$

$$\text{Si hanno } 2 \phi 18 \text{ sagomati} = 5.08 \text{ cm}^2$$

MENSOLA:

$$M = 7100 \times \frac{0.3^2}{2} = 3200 \text{ Kgm}$$

$$H = 40 \text{ cm} \quad h = 37 \text{ cm} \quad b = 100 \text{ cm}$$

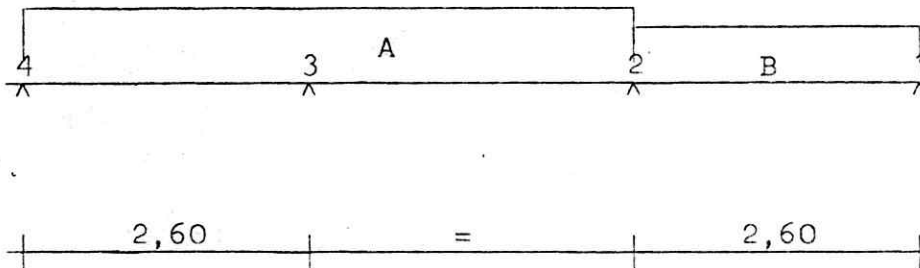
$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 29 \text{ Kg/cm}^2 \quad Af = 6,6 \text{ cm}^2$$

$$\text{Si adoperano } 1 \phi 14/20 \text{ cm}^2$$

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COPERTURA PIANO CANTINATO

TRAVE 1 - 2 - 3 - 4



EJ = cost

Analisi dei carichi:

A) carico permanente:

$$\text{solaio} \quad 450 \times \frac{5,00}{2} = 1100 \text{ Kg/ml}$$

$$\text{trave} \quad 0.30 \times 0.40 \times 2500 = 300 \text{ "}$$

$$\text{muratura} = 980 \text{ "}$$

$$\text{balcone} \quad 350 \times 1,2 = 420 \text{ "}$$

$$A_p = 2800 \text{ Kg/ml}$$

carico accidentale:

$$A_q = 250 \times \frac{5,00}{2} + 400 \times 1,2 = 1100 \text{ Kg/ml}$$

B) Carico permanente:

$$\text{solaio} = 1100 \text{ Kg/ml}$$

$$\text{muratura} = 980 \text{ "}$$

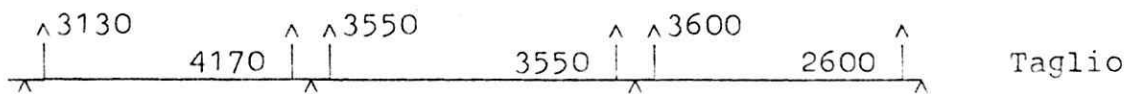
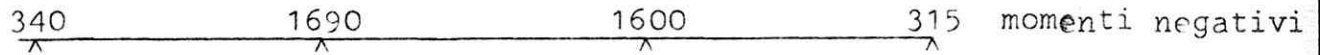
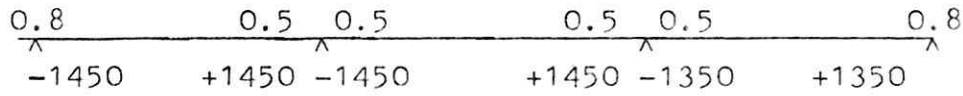
$$\text{trave} = 300 \text{ "}$$

$$B_p = 2380 \text{ Kg/ml}$$

carico accidentale:

$$B_q = 250 \times \frac{5,00}{2} = 625 \text{ Kg/ml}$$

1) Effetto carichi permanenti

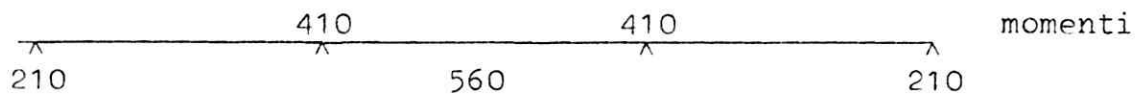
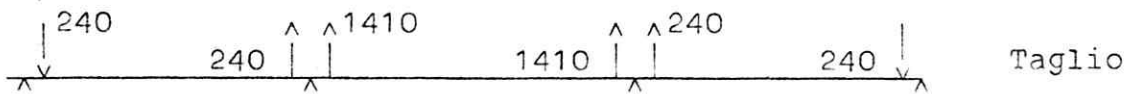


2) Carico accidentale sulla campata intermedia

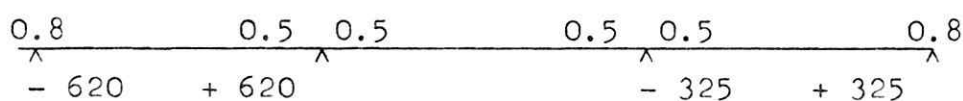
$$M_1 = M_4 = \frac{1}{36} \cdot 1100 \times 2,6^2 = + 210 \text{ Kgm}$$

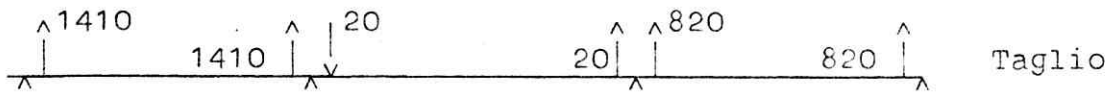
$$M_2 = M_3 = -\frac{1}{18} \cdot 1100 \times 2,6^2 = - 410 \text{ Kgm}$$

$$M_{2-3} = \frac{1}{13,4} \cdot 1100 \times 2,6^2 = 560 \text{ Kgm}$$

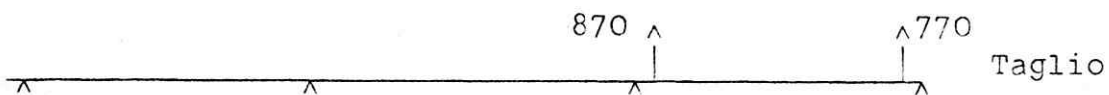
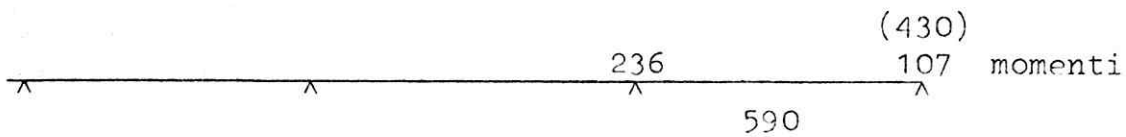


3) Carico accidentale sulla prima e ultima campata





4) Carico accidentale sull' ultima campata

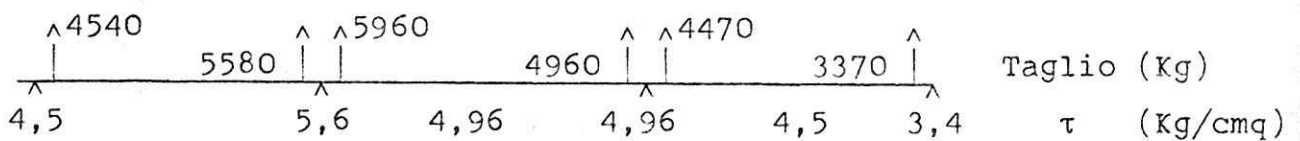
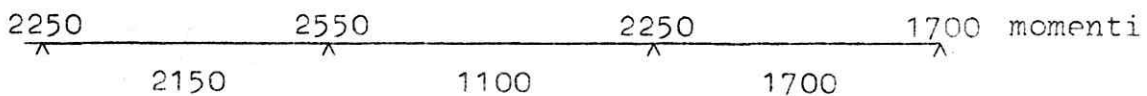


5) Carico accidentale sulle prime due campate

$$M_4 = \frac{1}{12,8} \cdot 1100 \times 2,6^2 = - 580 \text{ Kgm}$$

$$M_3 = - \frac{1}{9} \cdot 1100 \times 2,6^2 = - 830 \text{ Kgm}$$

Sovrapponendo gli effetti del carico permanente con quelli dei carichi accidentali posti nelle posizioni piu' sfavorevoli si hanno le seguenti sollecitazioni massime:



Le τ sono state tutte minori di 6 Kg/cm²; si arma prudenzail-
mente con staffe ϕ 4 / 20 cm

Per H = 40 cm

b = 30 cm

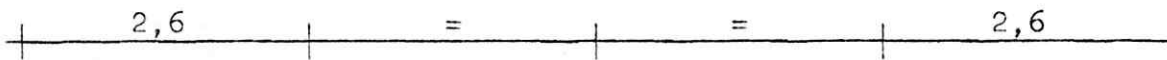
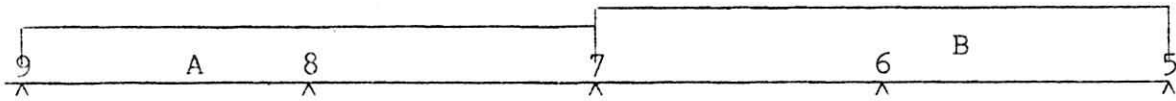
h = 37 cm

$\sigma_f = 2000$ Kg/cm²

n = 10

σ_c	$\frac{55}{\lambda}$	52	$\frac{59}{\lambda}$	37	$\frac{55}{\lambda}$	47	$\frac{47}{\lambda}$
Af	3,29	2,96	3,74	1,6	3,29	2,5	2,5

TRAVE 5 - 6 - 7 - 8 - 9



EJ = cost

Analisi dei carichi (vedi travata precedente)

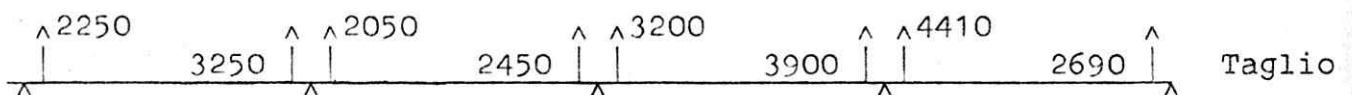
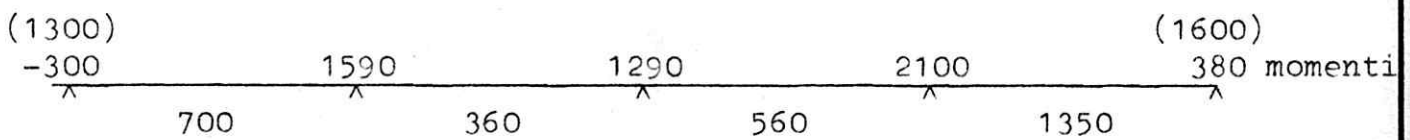
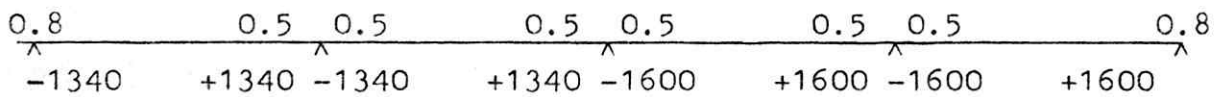
$$A_p = 2380 \text{ Kg/ml}$$

$$A_q = 625 \text{ Kg/ml}$$

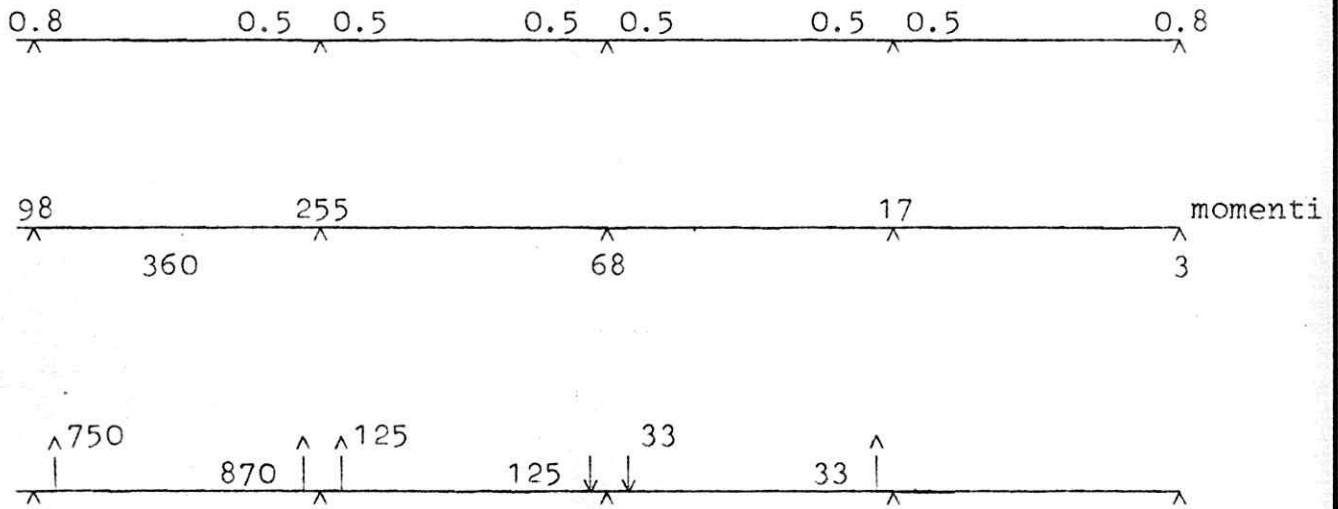
$$B_p = 2800 \text{ Kg/ml}$$

$$B_q = 1100 \text{ Kg/ml}$$

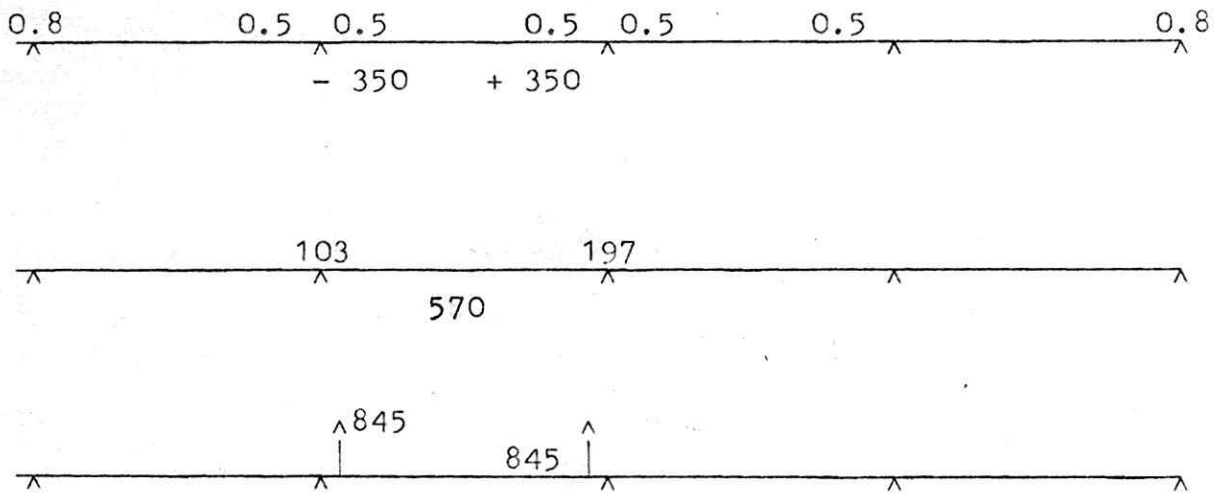
1) Carico permanente:



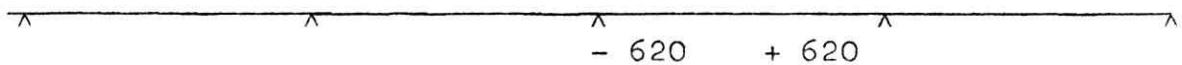
2) Carico accidentale sulla prima campata

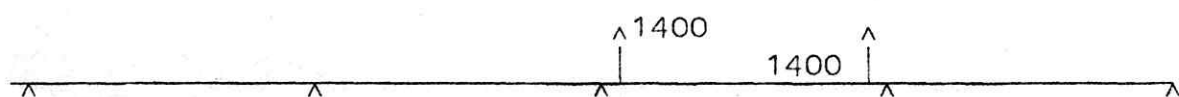
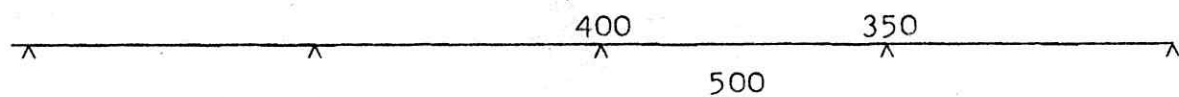


3) Carico accidentale sulla seconda campata

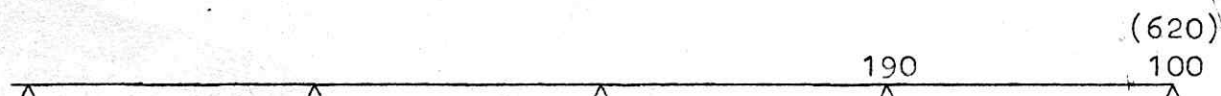
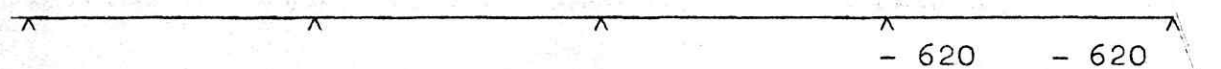


4) Carico accidentale sulla terza campata

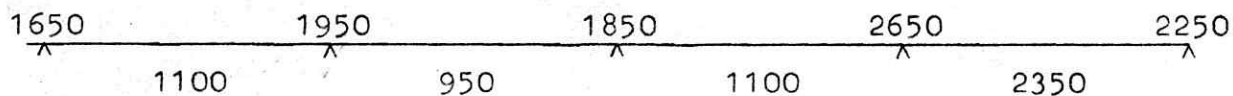




5) Carico accidentale sulla quarta campata



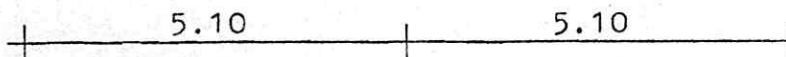
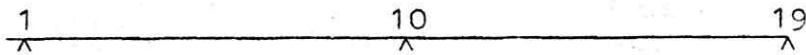
Sovrapponendo gli effetti si ha:



	9		8		7		6		5
σ_c	45	35,5	49	33	47,5	35,5	58	54,5	53
A_f	2,35	1,54	2,75	1,32	2,6	1,54	3,75	3,5	3,16

$\tau < 6$ quindi Staffe 1 ϕ 4 / 20 cm

TRAVE 1 - 19



Analisi dei carichi:

trave	300 Kg/ml
muratura	980 "
	<u>1280 Kg/ml</u>

$$q_1^2 = 1280 \times 5.10^2 = 33.200$$

2780	2300	3350	2300	2780	= M
60	54	66	54	60	= σ_c
4	3,3	4,7	3,3	4	= A_f

$$T = 1280 \times \frac{5.10}{2} = 3260$$

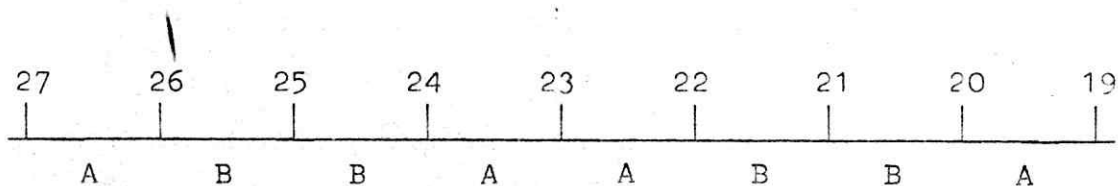
$$\tau_{\max} = \frac{3260}{1030} = 3,15 < 6$$

Staffe 1 ϕ 4 / 25

minima A_{f_p}

=====

TRAVE 27 - 19



Analisi dei carichi:

$$A \left| \begin{array}{l} A_p = 2380 \text{ Kg/ml} \\ A_q = 625 \text{ "} \end{array} \right.$$

$$B \left| \begin{array}{l} B_p = 2800 \text{ Kg/ml} \\ B_q = 1100 \text{ "} \end{array} \right.$$

a) 1) $M_{27} = M_{26} = M_{25} = \frac{1}{12} l^2 = 1350 \text{ Kgm}$

$$M_{27-26} = M_{26-25} = \frac{1}{14,5} l^2 = 1150 \text{ Kgm}$$

2) $M_{27} = \frac{1}{30} 400 \times 2,60^2 = 90 \text{ Kgm}$

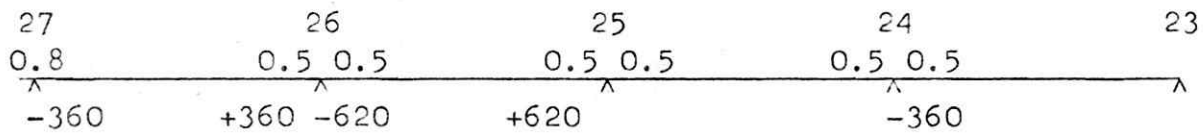
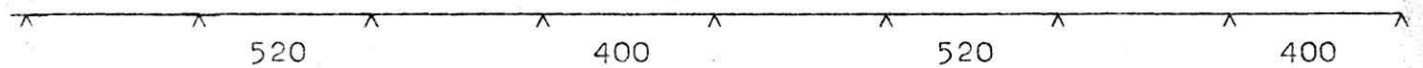
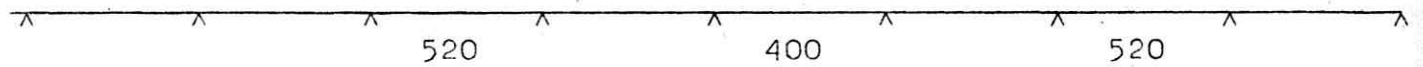
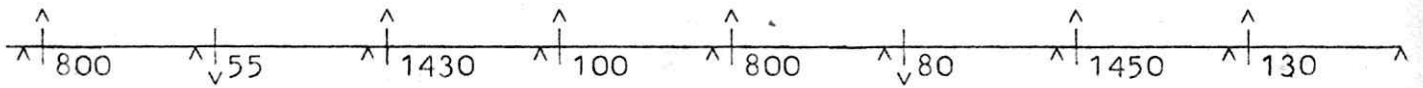
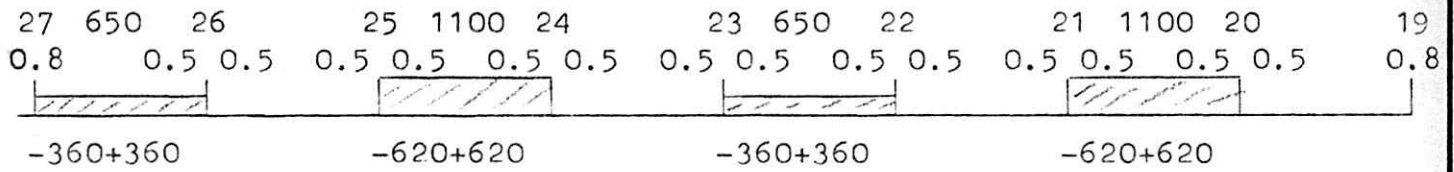
$$M_{26} = M_{26-25} = \frac{1}{15} 400 \times 2,60^2 = 180 \text{ Kgm}$$

$$M_{25} = \frac{1}{10} 400 \times 2,60^2 = 270$$

Sommando

27 ^		26 ^		25 ^
1450	1150	1550	1350	1620

b) Momenti massimi sugli appoggi



$$c) T_{27} = 625 \times \frac{2.60}{2} - \frac{184 - 110}{2.60} = 820 - 25 = 800 \text{ Kg}$$

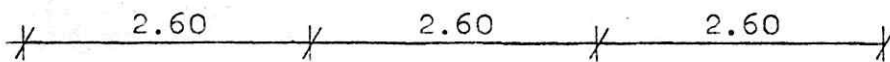
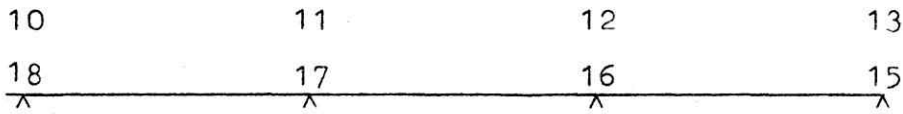
$$T_{26} = - \frac{318 - 184}{2.60} = - 55 = 55 "$$

$$T_{25} = 11.00 \times \frac{2.60}{2} - \frac{365 - 315}{2.60} = 1450 - 20 = 1430 "$$

$$T_{24} = + \frac{365 - 115}{2.60} = + 100 = 100 "$$

$$T_{23} = \frac{625 \times 2.60}{2} - \frac{179 - 116}{2.60} = 820 - 25 = 800 "$$

TRAVE 18 - 15 ; 13 - 10



Analisi dei carichi:

a) Carico permanente

trave	300 Kg/ml
solaio 450 x 5.50	2500 "
	<u>2800 Kg/ml</u>

2800 Kg/ml

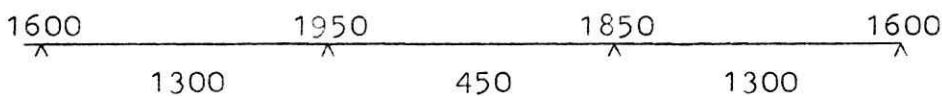
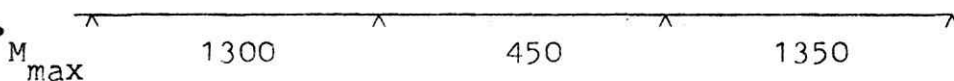
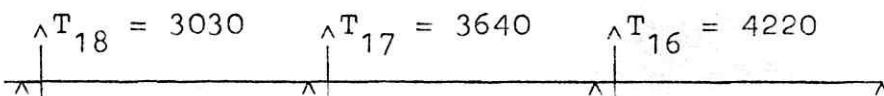
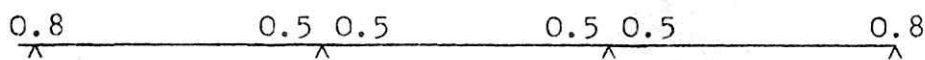
b) Carico accidentale

250 x 5.50

1400 "

Effetto carico permanente

$$= \frac{1}{12} q l^2 =$$



$$T_{18} = 2800 \times \frac{2.60}{2} - \frac{1932 - 352}{2.60} = 3640 - 620 = 3030$$

$$T_{17} = 3640 + \frac{1932 - 1834}{2.60} = 3640 = 3640$$

$$T_{16} = 3640 + \frac{1834 - 364}{2.60} = 3640 + 580 = 4220$$

$$\text{dalla } M_{\max} = M_2 + \frac{T_2^2}{2q}$$

$$2q = 2 \times 2800 = 5600$$

$$M_{18-17} = -352 + \frac{3030^2}{5600} = -352 + 1650 = 1300$$

$$M_{17-16} = -1932 + \frac{3640^2}{5600} = -1932 + 2380 = 450$$

$$M_{16-15} = -1834 + \frac{4220^2}{5600} = -1834 + 3200 = 1370$$

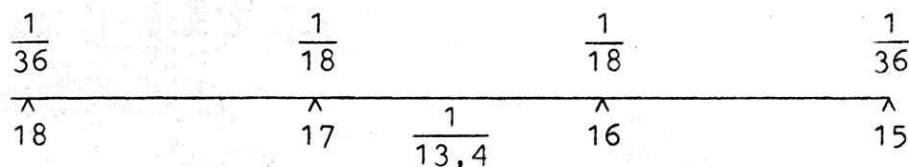
$$\frac{1}{15} q l^2 = 1260 \text{ Kgm}$$

"

"

Effetti carico accidentale

1) Carico accidentale sulla campata intermedia

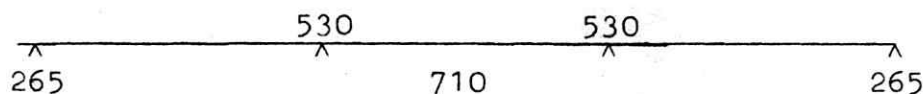


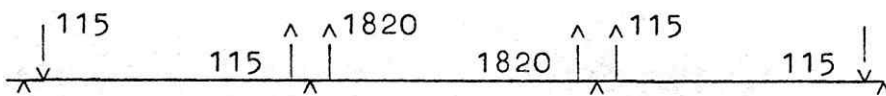
$$q l^2 = 1400 \times 2.60^2 = 9500$$

$$M_{18} = M_{15} = + \frac{1}{36} 9500 = + 265 \text{ Kgm}$$

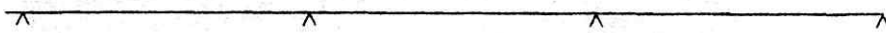
$$M_{17} = M_{16} = - \frac{1}{18} 9500 = - 530 \text{ Kgm}$$

$$M_{17-16} = + \frac{1}{13,4} 9500 = + 710 \text{ Kgm}$$

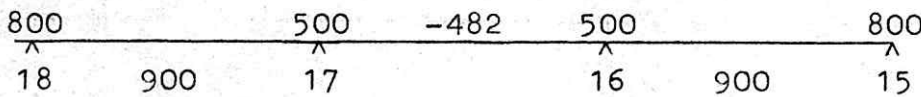
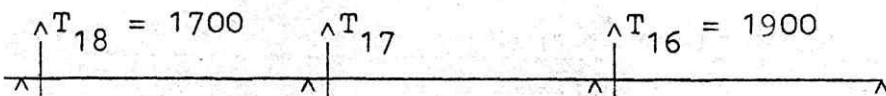
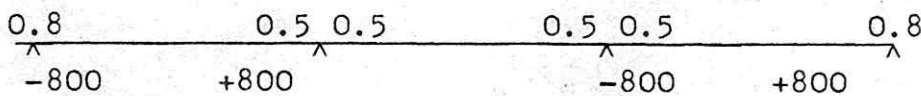




2) Carico accidentale sulla prima ed ultima campata



$$\frac{1}{12} q l^2 = 800 \text{ Kgm}$$



$$T = 1400 \times \frac{2.60}{2} - \frac{482 + 263}{2.60} = 1820 - 85 = 1700$$

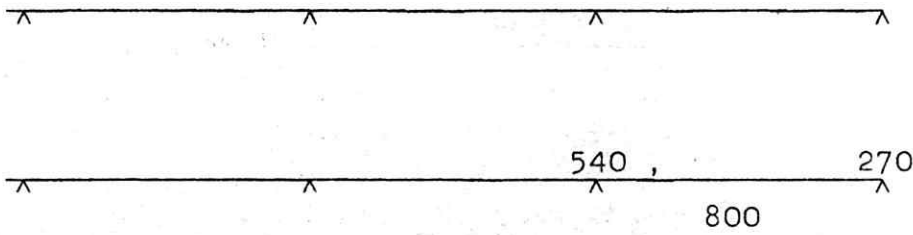
$$T_{17-16} = \frac{482 - 427}{2.60}$$

$$T_{16} = + \frac{427 - 240}{2.60} = 1820 + 72 = 1900$$

$$M_{18-17} = - 263 + \frac{1700^2}{2800} = - 263 + 1100 = 850$$

$$M_{16-15} = -427 + \frac{1900^2}{2800} = - 427 + 1300 = 900$$

3) Carico accidentale sull' ultima campata



4) Carico accidentale sulle prime due campate

$$M_{18} = - \frac{1}{12,8} 1400 \times 2,6^2 = - 740 \text{ Kgm}$$

$$M_{17} = - \frac{1}{9} 1400 \times 2,6^2 = - 1050 \text{ Kgm}$$

Sovrapponendo gli effetti

10		11		12		13
18		17		16		15
2875	2200	4050	1200	3500	3000	2400
62	52,5	71	37	69	63,5	55
4,15	3,15	6,45	1,68	5,00	4,35	3,4

$$M = 4100$$

$$\sqrt{\frac{41000}{3}} = \sqrt{13700} = 117$$

$$r' = \frac{38}{117} = 0,325$$

$$F_f = F_f \quad \sigma_c = 60 \quad t' = 0,00184$$

$$A_f = A'_f = 0,00184 \times 30 \times 117 = 6,45$$

$$T = 4200 \times \frac{2,60}{2} = 5500 \text{ Kg}$$

$$\tau_{\max} = \frac{5500}{1030} = 5,3 < 6$$

Staffe 1 ϕ 4/20

$A_{fp} = \text{minima.}$

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COPERTURA PIANO CANTINATO

Travi di bordo:

Analisi dei carichi:

solaio: $700 \times \frac{5,00}{2} = 1750 \text{ Kg/ml}$

trave $= 350 \text{ "}$

muratura $= 980 \text{ "}$

3080 Kg/ml

Data la simmetria delle luci, la conformita' dei carichi e la costante rigidezza delle travate si considera la singola campata come sem incastrata agli estremi:

$$M = \pm \frac{1}{12} 3080 \times 2,6^2 = 1750 \text{ Kgm.}$$

per $H = 20 \text{ cm.}$

$h = 17,5 \text{ cm.}$

$\sigma_f = 2000 \text{ Kg/cm}^2$

$b = 70 \text{ cm.}$

$n = 10$ si ha:

$\sigma_c = 70 \text{ Kg/cm}^2$

$A_f = 5,55 \text{ cm}^2$

Si arma la trave con

- armatura inferiore $4 \phi 14$ pari a $6,16 \text{ cm}^2$.

diritti correnti

- armatura superiore $4 \phi 10$ pari a $3,14 \text{ cm}^2$.

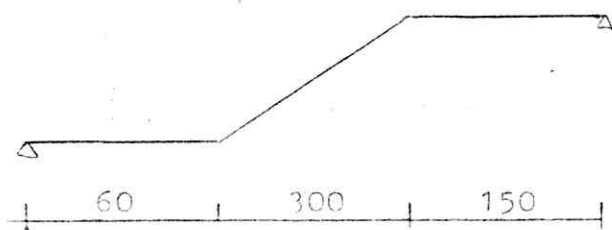
che si sovrappongano in corrispondenza dei pilastri per $4,30 \text{ m.}$

$T = 4000 \text{ Kg.}$

$\tau_{\max} = 3,6 \text{ Kg/cm}^2 < 6$

Si adoperano prudenzialmente staffe $\phi 6/30 \text{ cm.}$

Trave a ginocchio a sostegno rampe scale.



con $\gamma = 0.876$

Carico al ml. di rampa:

rampa	850×1.10	=	950 Kg/ml
trave	$0.20 \times 0.60 \times 2500$	=	300 "
muratura		=	700 "
			1950 Kg/ml

$$l = 0.60 + \frac{3.00}{0.876} + 1.50 = 5.52 \text{ m.}$$

$$M = \frac{1}{12} \cdot 5.52^2 \times 1700 = 4.300 \text{ Kg.m.}$$

Per $B = 60 \text{ cm.}$

$h = 57 \text{ cm.}$

$b = 20 \text{ cm.}$

$\sigma_f = 2000 \text{ Kg/cm}^2.$

$b = 20 \text{ cm.}$

$n = 10$

si ha:

$\sigma_c = 62 \text{ Kg/cm}^2$

$A_f = 4,1 \text{ cm}^2.$

Si arma la trave con 3 ϕ 14 superiori e 3 ϕ 14 inferiori correnti.

Si aggiungono 2 ϕ 14 lati in corrispondenza della rampa onde assorbire le azioni *torzionali*

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