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U.N.R.R.A. - C.A.S.A.S.

COPIA

LEGGE 173 - QUARTIERE COORDINATO DI BRINDISI

"RIONE PARADISO"

RELAZIONE DI CALCOLO DELLE STRUTTURE IN CALCESTRUZZO ARMATO

IMPRESA ING: RUGGERO LECCISI

[Signature]

VISTO: si approva

L'INGEGNERE CAPO
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STUDIO TECNICO
Dott. Ing. Leonardo POTT
BRINDISI

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ANALISI DEI CARICHI UNITARIAnalisi solaio

peso proprio	250 Kg/mq	
incidenza tramezzi	100 "	
pavimento	80 "	
intonaco	30 "	
sovraccarico	250 "	460
		710 Kg/mq
balconi 460 + 400	860 Kg/mq	

scale*Analisi di carico riportata a mq di pianta -*

p.p. rampa $(0.20 \times 0.00 + 0.18 \times 0.00) \frac{1.00}{0.28} \times 2500$	330 Kg/mq	
rivestimento $(0.02 \times 0.18 + 0.025 \times 0.28) \frac{1.00}{0.28} \times 2000$	75 "	
intonaco	30 "	
sovraccarico	400 "	
	835 Kg/mq	
835 / 0.84		990 Kg/mq

Muratura a

Foderò interno	70 Kg/mq	
" esterno	100	
intonaco interno	30	
	200 Kg/mq	
200 x 3		600 Kg/ml
160 x 3		480 "
Muretto d'attico		200 Kg/ml

Copertura a tetto vano scala

soletta p.p.	300 Kg/mq	
intonaco	30 "	
lastre d'acciaio + ginocchio	30 "	
carico accidentale	150 "	
	510 Kg/mq	

Pilastri	$0.15 \times 0.25 \times 3.20 \times 2500$	=	300 Kg/mq
	$0.15 \times 0.50 \times 3.20 \times 2500$	=	600 "
	$0.50 \times 0.50 \times 1.00 \times 2500$	=	650 "
Solaio copertura terrazzo			
p.p. solaio		=	570 "
sovraccarico		=	250 "
			820 Kg/mq

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ANALISI DEI CARICHI

TIPO IN LINEA

PILASTRO 1 - 48 - 14 - 61

carico a quota + 7.51

solaio	$\frac{4.50}{2} \times \frac{1.70}{2} \times 820$	=	1570 Kg
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muratura	$\frac{1.70}{2} \times 600$	=	500 "
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"	$\frac{4.50}{2} \times 480$	=	1080 "
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muretto d' attico	$\frac{1.70}{2} \times 200$	=	200 "
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travi	$\frac{1.70 + 4.50}{2} \times 250$	=	800 "
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pilastro		=	300 "
			4450 Kg

C.q. 4.27

solaio	$\frac{4.50}{2} \times \frac{1.70}{2} \times 710$	=	1400 Kg
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muratura	$\frac{1.70}{2} \times 600$	=	500 "
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	$\frac{4.50}{2} \times 480$	=	1080 "
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travi	=	800 Kg	
pilastro	=	300 "	
		<u>4000 Kg</u>	8.530
C.q. + 1.03		4000	12.530
C.q. - 0.25			
Solaio	$\frac{4.50}{2} \times 1.70 \times 710$	= 2715 Kg	
muratura	1.70×300	= 500 "	
"	$\frac{4.50}{2} \times 200$	= 450 "	
travi	$\frac{(4.50 + 1.70)}{2} \times 250$	= 1000 "	
pilastro	=	650 "	
		<u>5165 Kg</u>	17.695
a sommare $\frac{1}{2} P_2$ bis *			<u>7.250</u>
			24.945

PILASTRO 3 - 10 - 52 - 59

C.q. + 7.51			
solaio	$\frac{(3.10+4.50)}{2} \times \frac{1.70}{2} \times 820$	= 2600 Kg	
muratura	1.70×600	= 1000 "	
muretto d' attico	1.70×210	= 700 "	
travi	1.70×400	= 360 "	
pilastro	=	300 "	
		<u>4960 Kg</u>	

C.q. 4.27

solaio	$\frac{(3.10+4.50)}{2} \times \frac{1.70}{2} \times 710$	= 2200	
muratura		= 1000	
travi		= 360	
balcone	$0.55 \times 0.80 \times 860$	= 600	
pilastro	=	300	
		<u>4460</u>	9.420
C.q. 1.03		4460	13.880

* N.B.: Vedere pag. 14

C.q. - 0.25

$$\text{solaio } \left(\frac{3.10}{2} \times \frac{1.70}{2} \right) \times 710 = 3600$$

$$\text{muratura } \frac{3.40}{2} \times \frac{4.50}{2} \times 710 = 500$$

$$\text{travi} = 360$$

$$\text{balcone} = 600$$

$$\text{pilastro} = 650$$

5710

19.590

da aggiungere $\frac{1}{2} P_9$ bis

7250

26.840

PILASTRO 4 - 11 - 51 - 58

C.q. + 7.51

$$\text{solaio } \frac{2.98}{2} \times \frac{1.70}{2} \times 820 = 1050$$

$$\text{muratura } \frac{1.70 + 2.98}{2} \times 600 = 1400$$

$$\text{muretto d' attico } \frac{1.70 + 2.98}{2} \times 210 = 500$$

$$\text{travi } \frac{1.70 + 2.98}{2} \times 400 = 950$$

$$\text{pilastro} = 300$$

4200

C.q. + 4.27

$$\text{solaio } \frac{2.98}{2} \times \frac{1.70}{2} \times 710 = 900$$

$$\text{muratura} = 1400$$

$$\text{balcone } 0.85 \times 0.80 \times 860 = 600$$

travi	=	950	
pilastro	=	<u>300</u>	
		4150	8.350
C.q. - 1.03		4150	12.500
C.q. - 0.25		3800	16.300

PILASTRO 5 - 12 - 50 - 57

C.q. + 7.51			
solaio $820 \times \frac{1.70}{2} \times 2.25$	=	1600	
muratura $\frac{1.70 + 2.25}{2} \times 600$	=	1200	
travi $\frac{1.70 + 2.25}{2} \times 400$	=	800	
pilastro	=	300	
muretto d'attico $\frac{1.70 + 2.25}{2} \times 210$	=	<u>420</u>	
		4320	
C.q. + 4.27			
solaio $\frac{1.70}{2} \times 2.25 \times 710$	=	1400	
muratura	=	1200	
travi	=	800	
pilastro	=	<u>300</u>	
		3700	8.050
C.q. + 1.03		3700	11.750
C.q. - 0.25			
solaio	=	2400	
muratura $(1.70 + \frac{2.25}{2}) \times 160$	=	450	
travi	=	800	
pilastro	=	<u>650</u>	
		3300	15.050
ad aggiungere $\frac{1}{2} P_5$ bis			<u>7.250</u>
			22.300

PILASTRO 6 - 8 - 9 - 53 - 55 - 56 - 2 - 49 - 60 - 13

C.q. + 7.51

$$\text{solaio} \quad \frac{1.70}{2} \times \frac{4.00}{2} \times 8.20 \times 2 = 3200$$

$$\text{muratura} = 1020$$

$$\text{muretto d' attico} = 300$$

$$\text{travi} = 700$$

$$\text{pilastro} = 300$$

5500 5.500

C.q. + 4.27

$$\text{solaio} \quad 3200 \times \frac{710}{820} = 2800$$

$$\text{muratura} = 1020$$

$$\text{travi} = 700$$

$$\text{pilastro} = 300$$

4820 10.330

C.q. + 1.03 4820 15.150

C.q. - 0.25

$$\text{solaio} \quad 2 \times 3200 = 5600$$

$$\text{muratura} = 1020$$

$$\text{travi} = 1400$$

$$\text{pilastro} = 650$$

8670 23.880

da aggiungere $\frac{1}{2} P_5$ bis * 7.750

" $\frac{1}{2} P_6$ bis $\frac{7.750}{39.380}$

PILASTRO 7 - 54

C.q. + 10.95

$$\text{solaio cop. lav.} \quad 510 \times \frac{1.74^5}{2} \times \frac{4.00}{2} \times$$

$$\times \frac{1.00}{4.50} \times 2 = 300$$

$$\text{muratura} \quad 500 \times \frac{1.74^5}{2 \times 2} = 450$$

* N.B.: Vedere pag. 14

muratura	$500 \times \frac{4.50}{2} \times \frac{4.50}{4} \times \frac{1}{4.50} \times 2$	=	300	
trave	$250 \times \frac{1.74^5}{2 \times 2}$	=	230	
muretto d' attico	$210 \times \frac{1.70}{2} \times 2$	=	350	
			1650	1.650

C.q. + 7.51

solaio terrazzo	$\frac{1.70}{2} \times \frac{4.50}{2} \times 820 \times 2$	=	3200	
muratura	$600 \times \frac{1.70}{2} \times 2 + 500 \times \frac{4.34}{2}$	=	2120	
travi	$400 \times \frac{(2 \times 1.70 + 4.50)}{2}$	=	1600	
pilastro		=	300	
			7220	8.870

C.q. + 4.27

solaio	$\frac{1.70}{2} \times \frac{4.50}{2} \times 710 \times 2$	=	2800	
muratura		=	2120	
travi		=	1600	
pilastro		=	300	
			6820	15.700
C.q. + 1.03			6820	22.500

C.q. + 0.25

solaio	$\frac{4.50}{2} \times 3.40 \times 710$	=	5500	
muratura	$200 \times 0.80 \times 3.40$	=	600	
travi	$(3.40 + \frac{4.50}{2}) \times 400$	=	2250	
pilastro		=	650	
			9000	31.500
da aggiungere	$\frac{1}{2}$ P ₆ bis		7750	39.250
"	$\frac{1}{2}$ P ₇ bis		7750	47.000

PILASTRO 15 - 47

meta' pilastro n. 32 dopo aver sottratto carico a quota + 10.95

$$40.450 - 4.250 = 36.200 \text{ Kg}$$

$$P = 18.100 \text{ Kg}$$

PILASTRO 16 - 46

come pilastro n. 33 a sottrarre però carico a quota + 10.95

$$31.750 - 4.750 = 27.000$$

PILASTRO 18 - 17 - 27 - 28 - 29 - 30 - 34 - 35 - 44 - 45

C.q. + 7.51

solaio	=	6500
travi	=	700
pilastro	=	300
		7500

C.q. + 4.27

solaio	=	5500	
travi	=	700	
pilastro	=	300	
		6500	14.000
C.q. + 1.03		6500	
C.q. - 0.25		6500	27.000

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PILASTRO 24 - 20 - 41 - 37

C.q. 10.95

$$\text{c.v.s.} \quad 1.70 \times \frac{1.39^5}{2} \times 510 = 600$$

$$\text{muratura} \quad \frac{1.39^5}{2} + 1.70 \times 600 = 950$$

$$\text{travi} \quad \left(\frac{1.39^5}{2} + 1.70 \right) \times 250 = 600$$

$$\text{pilastro} = \underline{600}$$

2750

C.q. 7.51

$$\text{scala} \quad 1.70 \times \frac{3.65}{2} \times 900 = 2800$$

$$\text{solaio} \quad \frac{3.10^5}{2} \times \frac{1.70}{2} \times 820 = 1100$$

$$\text{muratura} \quad \frac{1.39^5}{2} + 1.70 \times 600 = 950$$

$$\text{travi} \quad \left(\frac{1.39^5}{2} + 1.70 \right) \times 400 = 950$$

$$\text{pilastro} = \underline{600}$$

6400 9.150

$$\text{C.q.} + 4.27 \quad 6400 \quad 15.550$$

$$\text{C.q.} + 1.03 \quad 6400 \quad 21.950$$

$$\text{C.q.} - 0.25 \quad 6000 \quad 27.950$$

$$28.000$$

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PILASTRO 22 - 26 - 39 - 43

C.q. 10.95

$$\text{cop. vano scala} \quad 1.70 \times \frac{2.25}{2} \times 510 = 1000$$

$$\text{muratura} \quad \frac{2.25}{2} \times 600 = 700$$

$$\text{muretto d' attico} \quad \frac{2.25}{2} \times 210 = 240$$

$$\text{travi} \quad \left(\frac{2.25}{2} + 1.70 \right) \times 250 = 750$$

$$\text{pilastro} = 300$$

3000

C.q. + 7.51

$$\text{Scala} \quad \frac{3.40}{2} \times \frac{3.65}{2} \times 900 = 2800$$

$$\text{mura} \quad \frac{4.50}{2} \times 600 = 1400$$

$$\text{travi} \quad (1.70 + 2.25) \times 400 = 1600$$

$$\text{pilastro} = 300$$

6100 9.100

C.q. 4.27 6100 15.200

C.q. 1.03 6100 21.300

C.q. - 0.25 5700 27.000

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PILASTRO 23 - 19 - 40 - 36

C.q. + 10.95

$$\text{cop. vano scala} \quad \frac{2.25 + \overline{1.39}^5}{2} \times 1.70 \times 510 = 1600$$

$$\text{muratura} \quad \frac{2.25 + \overline{1.39}^5}{2} \times 600 = 1100$$

$$\text{travi} \quad \frac{2.25 + \overline{1.39}^5}{2} \times 400 = 750$$

$$\text{pilastro} = 300$$

3750 3.750

C.q. + 7.51

$$\text{solaio} \quad \frac{\overline{4.50} \times 2.25 + \overline{4.40}}{2} \times \frac{1.70}{2} \times 820 = 2300$$

$$\text{muratura} \quad 2.25 \times 600 = 1400$$

$$\text{travi} \quad 2.25 \times 400 = 900$$

$$\text{pilastro} = 300$$

4900 8.650

C.q. 4.27 4900 13.550

C.q. 1.03 4900 18.450

C.q.- 0.25 4500 22.950

23.000

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PILASTRO 25 - 21 - 38 - 42

come pilastro n. 11 a dedurre carico balconi:

$$16.300 - 1.800 = 14.500 \text{ Kg}$$

PILASTRO 32

C.q. - 10.95

$$\text{solaio cop. lav.} \quad 510 \times \frac{1.74^5}{2} \times \frac{4.00}{2} \times$$

$$\frac{3.50}{4.50} \times 4 = 3000$$

$$\text{muratura} \quad 490 \times \frac{1.74^5}{2 \times 2} \times 4 = 850$$

$$\text{trave} \quad 210 \times 1.70 = 400$$

$$4250 \quad 4.250$$

C.q. + 7.51

$$\text{solaio} \quad 4.50 \times 1.70 \times 820 = 6500$$

$$\text{travi} \quad 1.70 \times 400 = 700$$

$$\text{pilastro} = 300$$

$$\text{muratura} \quad 500 \times 4.50 = 2300$$

$$9800 \quad 14.050$$

C.q. + 4.27

$$\text{solaio} \quad 4.50 \times 1.70 \times 710 = 5500$$

$$\text{muratura} = 2300$$

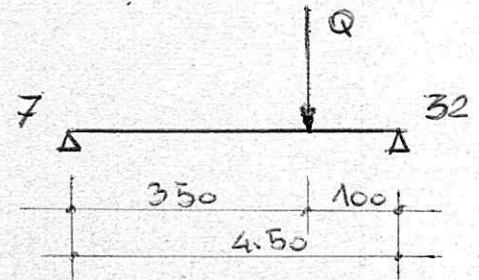
$$\text{travi} = 700$$

$$\text{pilastro} = 300$$

$$8800 \quad 22.850$$

$$\text{C.q.} + 1.03 \quad 8800 \quad 31.650$$

$$\text{C.q.} - 0.25 \quad 8800 \quad 40.450$$



PILASTRO 33 - 31

C.q. + 10.95

$$\text{solaio cop. lav.} \quad 510 \times \frac{1.74^5}{2} \times \frac{4.00}{2} \times \frac{3.37}{4.50} \times 2 = 1500$$

$$\text{muratura} \quad 490 \times \frac{1.74^5}{2 \times 2} \times 2 = 450$$

$$\text{"} \quad 490 \times \frac{4.00}{2} \times \frac{3.37}{4.50} \times 2 = 1700$$

$$\text{trave} \quad 250 \times \frac{1.74^5}{2 \times 2} \times 2 = 250$$

$$\text{"} \quad 250 \times \frac{4.00}{2} \times \frac{3.37}{4.50} \times 2 = 850$$

4750 4.750

C.q. - 7.51

$$\text{solaio} \quad 4.50 \times 1.70 \times 820 = 6500$$

$$\text{travi} \quad 400 \times 1.70 = 700$$

$$\text{pilastro} = 300$$

7500 12.250

C.q. + 4.27

$$\text{solaio} \quad 4.50 \times 1.70 \times 710 = 5500$$

$$\text{travi} = 700$$

$$\text{pilastro} = 300$$

6500 18.750

C.q. + 1.03

6500 25.250

C.q. - 0.25

6500 31.750

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PILASTRO 7 bis - 54 bis

carico a quota + 10.95

$$\text{solaio cop. lav.} \quad 510 \times \frac{1.74^5}{2} \times \frac{4.00}{2} \times \frac{4.50}{4} \times \frac{1.00}{4.50} = 160$$

muratura = 220

$$" \quad 500 \times \frac{4.50}{2} \times \frac{4.50}{4} \times \frac{1.00}{4.50} = 150$$

travi = 110

muretto d'attico = 360

1000

Carico a quota + 4.27

solaio terrazzo = 3200

muratura = 1020

travi 400 x 1.70 = 700

pilastro = 300

5220

6.250

Carico a quota + 4.27

solaio = 2800

muratura = 1020

travi = 700

pilastro = 300

4820

11.100

Carico a quota + 1.03

solaio = 2800

muratura = 510

travi = 700

pilastro = 300

4300

15.400

15.500 Kg

N.B. 1 PILASTRO N° 1 bis - 2 bis - 5 bis - 6 bis

9 bis - 53 bis - 55 bis - 56 bis sono ANALOGHI

AL PILASTRO 7 bis A DEDURRE CARICO q. + 10.95

(COPR. LAVATOIO)

P = 14.500 kg.

ANALISI DEI CARICHI

TIPO STELLA E TIPO MISTO

PILASTRO 1 - 14 - 6 (STELLA)
 44 (MISTO)

Carico a quota + 8.05

Muretto d'attico $(\frac{1.70}{2} + \frac{4.50}{2}) \times 200 = 600$

solaio cop. terrazzo $(\frac{4.50}{2} + \frac{1.70}{2}) \times 820 = 1600$

muratura $(\frac{1.70}{2} + \frac{4.50}{2}) \times 600 = 1850$

trave $(\frac{1.70}{2} + \frac{4.50}{2}) \times 400 = 1250$

pilastri $= \underline{300}$
 4600

C.q. 4.81 4400 9.000

C.q. 1.57 4400 13.400

C.q. - 0.25

solaio 1400

trave 1250

muratura $(\frac{1.70}{2} + \frac{4.50}{2}) \times 160 = 500$

Pilastro $\underline{1000}$
 4150 17.550

PILASTRO 7 - 24 (STELLA)
 69 (MISTO)

C.q. 8.05

Solaio $4.50 \times \frac{1.70}{2} \times 820 = 3150$

muretto d'attico $4.50 \times 200 = 900$

muratura $4.50 \times 600 = 2700$

trave $(4.50 + \frac{1.70}{2}) \times 400 = 2150$

pilastro $= \underline{300}$
 9200

C.q. 4.81

Solaio $4.50 \times \frac{1.70}{2} \times 710 = 2700$

muratura = 2700

trave = 2150

pilastro = 300

7850 17.050

C.q. 1.57 7850 24.900

C.q. 0.25

solaio = 2700

muratura $4.50 \times 160 = 720$

trave = 2150

pilastro = 1050

6650 31.550

PILASTRO 17 (STELLA)
35 (MISTO)

Come carico Pilastro 15 a sommare balcone

$19.300 + 3 \times 1.70 \times 1.16 \times 860$

$19.300 + 5100 = 24.400$

PILASTRO 16 (STELLA)
32 (MISTA)

Come carico Pilastro 15 a sommare balcone

$19.300 + 3 \times \frac{1.70}{2} \times 1.16 \times 860 = 21.900$

PILASTRO 31 - 23

Come Pilastro 1 ^{fibra in linea} a sommare muratura

~~23.200~~ 24.945

P. 24.945

~~4.50 x 160~~

~~750~~

~~23.950~~

~~24.000~~

PILASTRO 37 - 22 (STELLA)
58 - 73 (MISTO)

Come Pilastro 48 a sottrarre balcone

44.000 - 2.600

41.400

PILASTRO 2 - 3 - 4 - 5 - 15 (STELLA)
31 - 34 - 37 - 40 - 29 (MISTO)

C.q. + 8.05

Muretto d' attico 1.70×200 = 350

solaio $\frac{4.50}{2} \times 1.70 \times 820$ = 3150

muratura 1.70×600 = 1000

trave 1.70×400 = 700

pilastro = 300

5500

C.q. 4.81

Solaio $\frac{4.50}{2} \times 1.70 \times 710$ = 2700

muratura = 1000

trave = 700

pilastro = 300

4700 10.200

C.q. 1.57 4700 14.900

C.q. 0.25

Solaio = 2700

muretto 1.70×160 = 300

trave = 700

pilastro = 1000

4700 19.600

PILASTRO 18 (STELLA)
38 (MISTO)

C.q. 8.05

$$\text{Muretto d' attico} \quad \frac{1.70 + 2.10}{2} \times 200 = 400$$

$$\text{solaio cop. terr.} \quad \frac{4.50}{2} \times 1.70 \times 820 = 3150$$

$$\frac{1.40 \times 0.80}{2 \times 2} \times 820 = 250$$

$$\text{muratura} \quad \left(\frac{1.70}{2} + \frac{2.10}{2} \right) \times 600 = 1150$$

$$\text{trave} \quad \frac{1.70 + 1.63}{2} \times 400 = 700$$

$$\text{pilastro} = 300$$

5950

C.q. + 4.81

$$\text{Solaio} \quad \frac{4.50}{2} \times 1.70 \times 710 = 2750$$

$$\frac{1.40 \times 0.80}{2 \times 2} \times 710 = 250$$

$$\text{muratura} = 1150$$

$$\text{trave} = 700$$

$$\text{balcone} \quad \left(\frac{1.70}{2} + 0.50 \right) \times 1.16 \times 860 = 1300$$

$$\text{pilastro} = 300$$

6450 12.400

C.q. 1.57 6450 18.850

C.q. - 0.25

$$\text{Solaio} = 2750$$

$$\text{trave} = 700$$

$$\text{muratura} \quad \left(\frac{1.70}{2} + \frac{2.10}{2} \right) \times 160 = 300$$

$$\text{Balcone} = 1300$$

$$\text{pilastro} = 1000$$

6050 24.900

PILASTRO 47 - 33 (STELLA)
 46 - 61 (MISTO)

Come pilastro 18 a sommare $\frac{1}{2}$ 47 bis

25.000 + 9.800

34.800

PILASTRO 36 - 21 (stella)
57 - 72 (misto)

Come pilastro 47 a sottrarre balconi

35.000 - 3 x 1.300 31.100

PILASTRI 8 - 9 - 10 - 11 - 25 - 26 - 27 - 28 - 40 - 41 - 42 - 43

(tipo a stella) analoghi ai:

30 - 33 - 36 - 39 - 68 - 67 - 66 - 65 - 54 - 53 - 52 - 51 (tipo misto)

corrispondono al pilastro n° 17 tipo in linea

P = 27.000 Kg

PILASTRI 38 - 49 (stella)
48 - 59 (misto)

corrispondono al pilastro n° 1 tipo in linea

P = 24.945 Kg

PILASTRI 37 (stella)
58 (misto)

corrispondono al pilastro n° 2 tipo in linea

P = 38.380 Kg

PILASTRI 39 (stella)
55 (misto)

corrispondono al pilastro n° 15 tipo in linea

P = 36.200 Kg

PILASTRI 48 - 32 (stella)
47 - 62 (misto)

Come pilastro 6 tipo in linea a sottrarre $\frac{1}{2} P_5$ bis e ad aggiungere

carico balconi 31.600

$3 \times \frac{1.70}{2} \times 1.16 \times 860$ 2.600

e a sommare $\frac{1}{2}$ 47 bis 9.800
44.000

PILASTRI 47 bis (stella)
46 bis (misto)

Come pilastro 55 tipo in linea e a sommare carico balconi

14.500

$$3 \times 1.70 \times 1.16 \times 860 \times 860$$

$$\underline{5.100}$$

$$19.600$$

PILASTRI 46 - 34 - 19

(stella)

45 - 60 - 41

(misto)

C.q. + 8.05

$$\text{solaio c.t. } \left(\frac{4.50}{2} \times \frac{1.70}{2} + \frac{1.40 \times 0.80}{2 \times 2} + \frac{3.10}{2} \times 1.10 \right) 810 =$$

$$= 3.200$$

$$\frac{1.20 \times 1.20 \times 900}{2 \times 3}$$

$$= 250$$

$$\text{muretto d'attico } \frac{2.73 + 1.40}{2} \times 200$$

$$= 400$$

$$\text{scala } \frac{3.10}{2} \times 1.10 \times 900$$

$$= 1.550$$

$$\text{muratura } \left(\frac{2.90}{2} + 0.70 \right) \times 600$$

$$= 1.300$$

trave	$\frac{(2.90 + 1.40)}{2} \times 400$	=	1250	
pilaastro		=	300	
			8250	
C.q. + 4.81				
Solaio	$\frac{(4.50 \times 1.70)}{2} + \frac{1.40 \times 0.80}{2 \times 2}$			
	$\times 710$	=	1550	
pianerottolo		=	250	
scala		=	1550	
trave		=	1250	
muratura		=	1300	
pilaastro		=	300	
			6200	14.450
C.q. + 1.57			6200	14.450
C.q. - 0.25				20.650
Solaio		=	1550	
pianerottolo		=	250	
scala		=	1550	
trave		=	1250	
pilaastro		=	1000	
			5600	26.250
<u>PILASTRO</u> 45 - 30 - 13 (STELLA) 49 - 70 - 42 (MISTA)				
C.q. + 8.05				
muretto d' attico	$\frac{3.10 + 1.50}{2} \times 200$	=	450	
muratura	$\frac{3.10 + 1.50}{2} \times 600$	=	1400	
travi	$(1.40 + \frac{3.10 + 1.50}{2}) \times 400$	=	1500	
scala		=	1550	
pianerottolo		=	250	
pilaastro		=	300	
			5450	

C.q. + 4.81 5.450

come sopra a detrarre muretto d' attico

5450 - 450 5000 10.450

C.q. + 1.57 5000 15.450

C.q. - 0.25

travi = 1500

scala = 1550

pianerottolo = 250

pilastro = 1000

4300 19.750

PILASTRO 44-12-29 (STELLA)
50-43-64 (MISTO)

C.q. + 8.05

muretto d' attico $\frac{4.50 + 1.50}{2} \times 200 = 600$

solaio c.t. $4.50 \times \frac{1.70}{2} \times 860 = 3300$

muratura $\frac{4.50 + 1.50}{2} \times 600 = 1800$

pianerottolo = 250

travi $\frac{1.70 + 4.50 + 1.50}{2} \times 400 = 1550$

pilastro = 300
7800

C.q. + 4.81 7.800

solaio $4.50 \times \frac{1.70}{2} \times 710 = 2800$

muratura = 1800

pianerottolo = 250

travi = 1550

pilastro = 300

6700 14.500

C.q. + 1.57 6700 21.200

C.q. - 0.25

solaio	=	2800	
pianerottolo	=	250	
travi	=	1550	
pilastro	=	1000	
		5600	26.800

PILASTRO 20 (STELLA)
 71 (MISTO)

C.q. + 8.05

$$\text{solaio cop. ter. } \frac{1.70}{2} \times \frac{4.50}{2} \times 860 = 1650$$

$$\text{muratura d' attico } \frac{4.50 + 1.70}{2} \times 200 = 650$$

$$\text{muratura } \frac{4.50 + 1.70}{2} \times 600 = 1850$$

$$\text{travi } \frac{1.70 + 4.50}{2} \times 400 = 1250$$

$$\text{pilastro} = 300$$

$$5700$$

C.q. + 4.81

$$\text{solaio } \frac{1.70}{2} \times \frac{4.50}{2} \times 710 = 1350$$

$$\text{muratura} = 1850$$

$$\text{travi} = 1250$$

$$\text{pilastro} = 300$$

4750 10.450

$$\text{C.q. + 1.57} \quad 4750 \quad 15.200$$

C.q. - 0.25

$$\text{solaio} = 1350$$

$$\text{travi} = 1250$$

$$\text{pilastro} = 1000$$

$$\text{muratura} = 1850$$

$$4450$$

19.650

Pilastro Nr. 43 (STELLA)
 51 (MISTO)
 come pilastro n. 16 tipo ~~acanto~~ LINEA

27.000

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CALCOLO DELLE FONDAZIONI

TERRENO:

Data la natura del terreno la sollecitazione unitaria sul piano di posa e' stata contenuta in 1,2 Kg/cm².

SISTEMA DI FONDAZIONE:

La natura del terreno e la vicinanza dei pilastri ha suggerito di adottare travi rovesce continue su sottofondo di calcestruzzo magro, e spianare il piano di posa, dello spessore medio di cm. 10.

Le travi rovesce si sono dimensionate in modo da avere rilevante rigidezza onde permettere una piu' uniforme distribuzione dei carichi sul terreno.

Inoltre la minore dimensione della costola e' stata obbligata dalla dimensione dei pilastri e cio' ha portato a tassi di lavoro per il calcestruzzo non troppo elevati.

PRESCRIZIONE MATERIALI:

Calcestruzzo di cemento tipo 600 avente una resistenza cubica a compressione dopo 28 gg. non inferiore a 160 Kg/cm².

Carico di lavoro: $\sigma_c \text{ max} \leq 40 \text{ Kg/cm}^2$

$$\tau \leq 4 \text{ Kg/cm}^2$$

Acciaio $A_s 42 \div 50 \text{ cm}^2/\text{mq}$

limite di ~~snervamento~~ ^{snervamento} superiore

a 23 Kg/mm², allungamento a rottura superiore al 20°/°

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

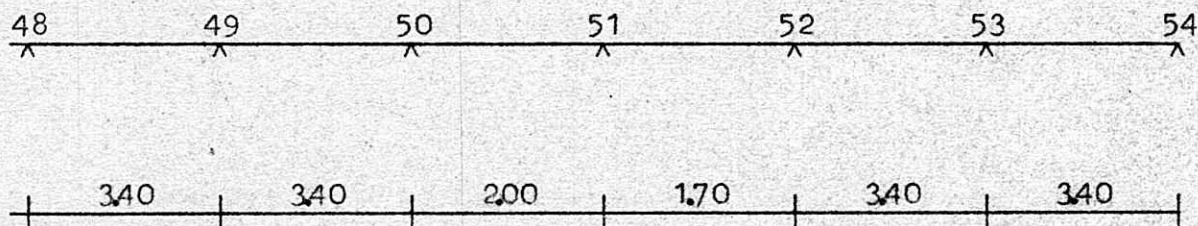
A) Tipo in linea

PILASTRATA 48 - 61/1 - 14

L = 38.00 m

l = 1.15 m

EJ = cost.



Scarichi pilastri:

$$\begin{aligned} \leq P = & \frac{23.2}{2} + 39 + 22.3 + \frac{2}{3} \times 16.3 + 24.5 + 39 + \frac{2}{3} \times 47 + 39 + 38 + \\ & + \frac{2}{3} \times 22.3 + 16.3 + 24.5 + 39 + \frac{23.2}{2} = 362,800 \text{ Kg} \end{aligned}$$

Peso fondazioni al ml:

magro	$0.10 \times 1.15 \times 1.00 \times 2000$	=	230 Kg/ml
ala	$0.30 \times 1.15 \times 1.00 \times 2500$	=	860 "
costola	$0.55 \times 0.70 \times 1.00 \times 2500$	=	950 "

$$2040 \text{ Kg/ml} = 2040 \text{ Kg/ml}$$

Terra sovrastante

$$0.60 \times 1.00 \times 1800 = 1100 \text{ Kg/ml} = 1100 "$$

Scarico al ml. di fondazione

$$3140 \text{ Kg/ml}$$

$$\sigma_t = \frac{362.800 + 3140 \times 38.00}{115 \times 3800} = \frac{482.000}{437.000} = 1,1 \text{ Kg/cm}^2$$

Carico al ml. sulle travate:

$$p_c = \frac{362.800}{38.00} = 9550 \text{ Kg/ml}$$

Data la costante ripidezza delle travate, la uniforme distribuzione dei carichi, si considerano le singole travate come semincastate e si determinano le seguenti sollecitazioni:

$$\begin{aligned} M_{48} = M_{49} = M_{53} = M_{54} = M_{55} = M_{56} = M_{60} = M_{61} &= \frac{1}{12} 9550 \times 3.40^2 = \\ &= 9250 \text{ Kgm} \end{aligned}$$

$$M_{48-49} = M_{49-50} = M_{52-53} = M_{53-54} = M_{54-55} = M_{55-56} = M_{56-57} = \\ = M_{59-60} = M_{60-61} = \frac{1}{14,5} \times 9550 \times 3,40^2 = 7650 \text{ Kgm}$$

$$M_{50} = M_{57} = \frac{1}{12} \left(\frac{3,40+2,00}{2} \right)^2 \times 9550 = 5800 \text{ Kgm}$$

$$M_{50-51} = M_{57-58} = \frac{1}{14,5} \times 2,00^2 \times 9550 = 2650 \text{ Kgm}$$

$$M_{51} = M_{58} = \frac{1}{12} \left(\frac{1,70+2,00}{2} \right)^2 \times 9550 = 2750 \text{ Kgm}$$

$$M_{51-52} = M_{58-59} = \frac{1}{14,5} \times 1,70^2 \times 9550 = 1900 \text{ Kgm}$$

$$M_{52} = M_{59} = \frac{1}{12} \left(\frac{1,70+3,40}{2} \right)^2 \times 9550 = 5200 \text{ Kgm}$$

Si calcolano le armature per:

$$H = 100 \text{ cm}$$

$$h = 95 \text{ cm}$$

$$b = 55 \text{ cm}$$

$$n = 10$$

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

$$M = 9250 \text{ Kgm}$$

$$\sigma_c = 26 \text{ Kg/cm}^2$$

$$A_f = 7,4 \text{ cm}^2$$

$\left\{ \begin{array}{l} 4+4 \phi 10 \text{ d.} \\ 2 \phi 14 \text{ S} \end{array} \right.$

$$M = 5800 \text{ Kgm}$$

$$\sigma_c = 20 \text{ Kg/cm}^2$$

$$A_f = 4,6 \text{ cm}^2$$

$\left\{ \begin{array}{l} 4+4 \phi 10 \\ 2 \phi 14 \text{ S} \end{array} \right.$

$$M = 7650 \text{ Kgm}$$

$$\sigma_c = 23 \text{ Kg/cm}^2$$

$$A_f = 6,0 \text{ cm}^2$$

$\left\{ \begin{array}{l} 2 \phi 14 \text{ S} \\ 3 \phi 12 \text{ d.} \end{array} \right.$

Taglio:

Si arma la travata per il taglio massimo

$$T = 9550 \times \frac{3,40}{2} = 16.300 \text{ Kg}$$

$$\tau_{\max} = \frac{16.300}{0,9 \times 55 \times 95} = 3,45 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

Si adoperano prudenzialmente Staffe $\phi 8/20$

$$\tau_{os} = \frac{1,01 \times 1400}{55 \times 20} = 1,28 \text{ Kg/cm}^2$$

$$\tau_R = 3,45 - 1,28 = 2,17 \text{ Kg/cm}^2$$

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ALA:

$$M = \frac{1}{2} \times 9550 \times 0,30^2 = 430 \text{ Kgm}$$

Data l' esiguita' dell' azione flessionale si arma prudenzialmente con ϕ 8/20 cm

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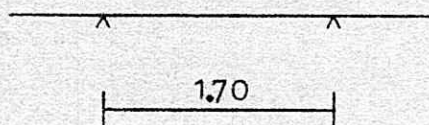
PILASTRATA 36 - 23

$$L = 18,40 \text{ m}$$

$$l = 150$$

Scarichi pilastri:

$$\begin{aligned} \sum P &= \frac{23}{3} + 27 + 27 + 27 + 32 + \frac{40,5}{2} + 32 + 27 + 27 + 27 + 27 + 23 = \\ &= 262,0 \text{ t} \end{aligned}$$



EJ = cost

Peso fondazione al ml.:

magro	$0.10 \times 1,50 \times 1,00 \times 2000$	=	300 Kg/ml
costola	$0.40 \times 0.70 \times 1.00 \times 2500$	=	700 "
ala	$1,50 \times 0.30 \times 1.00 \times 2500$	=	1100 "
			2100 Kg/ml

Terra sovrastante

$$1.10 \times 1.00 \times 1800 = 1980 \text{ Kg/ml}$$

scarico al ml. di fondazione

$$1980 + 2100 = 4080 \text{ Kg/ml}$$

$$\sigma_c = \frac{262.000 + 4080 \times 18.40}{1840 \times 150} = \frac{337.000}{276.000} = 1,21 \text{ Kg/cm}^2$$

Carico al ml. di fondazione

$$p_c = \frac{262.000}{18,4} = 14.200 \text{ Kg/ml}$$

Data la uniforme ripidezza, l' eguaglianza delle luci e la unifor-

mita' dei carichi si considerano le singole campate come semincastrate e si determinano le sollecitazioni

$$M_i = \frac{1}{12} \times 1,7^2 \times 14.200 = 3400 \text{ Kgm}$$

$$M_m = \frac{1}{14,5} \times 1,7^2 \times 14.200 = 2800 \text{ Kgm}$$

Si calcolano le armature:

$$H = 100 \text{ cm}$$

$$h = 95 \text{ cm}$$

$$b = 40 \text{ cm}$$

$$n = 10$$

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

$$M_i = 3400 \text{ Kgm}$$

$$\sigma_c = 20 \text{ Kg/cmq}$$

$$A_f = 3,0 \text{ cmq } 5 \phi 10$$

$$M_m = 2800 \text{ Kgm}$$

$$A_f = 2,5 \text{ cmq } 3 \phi 12$$

$$T = \frac{1}{2} \times 1,7 \times 14.200 = 12.100 \text{ Kg}$$

$$\tau_{\max} = \frac{12.100}{0,9 \times 95 \times 40} = 3,5 \text{ Kg/cmq} < 4 \text{ Kg/cmq}$$

Si adoperano prudenzialmente Staffe $\phi 8/25 \text{ cm}$

$$\tau_s = \frac{1,01 \times 1400}{40 \times 20} = 1,78 \text{ Kg/cmq}$$

$$\tau_R = 1,70 \text{ Kg/cmq}$$

ALA:

$$M = \frac{1}{2} \times 14.200 \times 0,55^2 = 2150 \text{ Kgm}$$

$$H = 30 \text{ cm}$$

$$h = 25 \text{ cm}$$

$$b = 10 \text{ cm}$$

$$n = 10$$

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

$$\sigma_c = 36 \text{ Kg/cmq}$$

$$A_f = 6,6 \text{ cmq}$$

Si adoperano 5 $\phi 10$ + 5 $\phi 10$ al ml.

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PILASTRATA 47 - 40/19 - 15

$$L = 6,80 \text{ m}$$

$$l = 1,50 \text{ m}$$

$$\Sigma P = \frac{1}{3} \times 18,5 + 81 + \frac{23}{3} = 95,0 \text{ t}$$

Scarico al ml. di fondazione

come analisi pilastrata precedente 4080 Kg/ml.

$$\sigma_t = \frac{95.000 + 4080 \times 6,80}{680 \times 150} = 1,2 \text{ Kg/cm}^2$$

$$p_c = \frac{95.000}{6,80} = 14.000 \text{ Kg/ml}$$

$$M_i = \frac{1}{12} \times 1,7^2 \times 14.000 = 3300 \text{ Kgm}$$

$$M_n = \frac{1}{14,5} \times 1,7^2 \times 14.000 = 2700 \text{ Kgm}$$

Per H = 100 cm

h = 95 cm

b = 40 cm

n = 10

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

M = 3300 Kgm

$\sigma_c = 20 \text{ Kg/cm}^2$

$A_f = 3,0 \text{ cm}^2$ 3 ϕ 10

M = 2700 Kgm

$A_f = 2,5 \text{ cm}^2$ 3 ϕ 12

$$T = \frac{1}{2} \times 14.000 \times 1,7 = 12.000 \text{ Kg}$$

$$\tau_{\max} = 3,5 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

Si adoperano prudenzialmente ϕ 8/20"

$$\tau_s = 1,41 \text{ Kg/cm}^2$$

$$\tau_R = 2,09 \text{ Kg/cm}^2$$

ALA:

$$M = \frac{1}{2} \times 14.000 \times 0,55^2 = 2150 \text{ Kgm}$$

H = 30 cm

h = 25 cm

b = 100 cm

n = 10

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

$\sigma_c = 36 \text{ Kg/cm}^2$

$A_f = 6,6 \text{ cm}^2$

Si adoperano 10 ϕ 12 al ml.

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FONDAZIONE 1 - 48/14 - 61

$L = 10.15 \text{ m} \quad l = 67,5 \text{ m}$

$\Sigma P = \frac{23,2}{2} + \frac{18,5}{3} + \frac{23,2}{2} = 30.000 \text{ Kg}$

Peso fondazioni al ml.:

magro	$0.10 \times 0,675 \times 1,00 \times 2000$	=	135 Kg/ml
ala	$0.675 \times 0.30 \times 1.00 \times 2500$	=	510 "
costola	$0.70 \times 0.375 \times 1.00 \times 2500$	=	660 "
			<u>1305 Kg/ml</u>

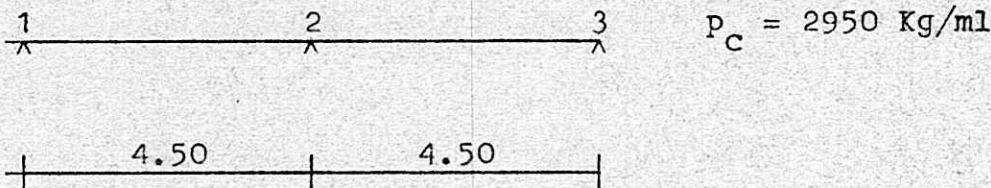
Terra sovrastante

$0.30 \times 1.00 \times 1.00 \times 1800 = 540 \text{ Kg/ml}$

$\sigma_t = \frac{30.000 + 1845 \times 10,15}{1015 \times 67,5} = 0.71 \text{ Kg/ml}$

Scarico sulla pilastrata al ml.

$p_c = \frac{30.000}{10.15} = 29.50 \text{ Kg/ml}$



$M_2 = \frac{1}{10} 2950 \times 4,5^2 = - 6000 \text{ Kgm}$

$M_{\max} = \frac{1}{14} 2950 \times 4,5^2 = + 4300 \text{ Kgm}$

Armature per:

$H = 100 \text{ cm} \quad h = 95 \text{ cm} \quad b = 37,5 \text{ cm} \quad n = 10$

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

$M = 6000 \text{ Kgm} \quad \sigma_c = 25 \text{ Kg/cm}^2$

$M = 4300 \text{ Kgm} \quad \sigma_c = 18 \text{ Kg/cm}^2$

$T = \frac{1}{2} 2950 \times 450 = 6700 \text{ Kg}$

$A_f = 4,8 \text{ cm}^2 \begin{matrix} 4 + 4 + 8 \text{ d} \\ 2 \text{ d } 12 \text{ s} \end{matrix}$
 $A_f = 3,4 \text{ cm}^2 \begin{matrix} 3 + 10 \text{ d} \\ 2 \text{ d } 12 \text{ s} \end{matrix}$

$$\tau_{\max} = \frac{6.700}{0.9 \times 95 \times 37,5} = 2,1 \text{ Kg/cm}^2$$

Si adoperano prudenzialmente

1 ϕ 8/25"

ALA:

Data l' esiguita' delle azioni flettenti si arma prudenzialmente con 1 ϕ 8/25 cm

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PILASTRATA 7 - 54

$$L = 10,15 \text{ m}$$

$$l = 0,90 \text{ m}$$

$$\Sigma P = \frac{1}{3} \times 2 \times 47 + \frac{2}{4} \times 40,5 = 51,75 \text{ t}$$

peso fondazione al ml.

magro	$0.90 \times 0.10 \times 1.00 \times 2000$	=	180 Kg/ml
ala	$0.90 \times 1.00 \times 0.30 \times 2500$	=	670 "
costola	$0.30 \times 0.70 \times 1.00 \times 2500$	=	670 "
			1520 Kg/ml

Terra sovrastante

$$0.60 \times 1.00 \times 1600 = 950 \text{ Kg/ml}$$

$$\sigma_t = \frac{51.750 + (950 + 1520) \times 10,15}{1015 \times 90} = 0.85 \text{ Kg/cm}^2$$

$$P_c = \frac{51.750}{10.15} = 5100 \text{ Kg/ml}$$

$$M_2 = \frac{1}{10} \times 5100 \times 4,5^2 = 10.300 \text{ Kgm}$$

$$M_m = \frac{1}{14} \times 5100 \times 4,5^2 = 7300 \text{ Kgm}$$

Armature per:

$$H = 100 \text{ cm}$$

$$h = 95 \text{ cm}$$

$$b = 30 \text{ cm}$$

$$n = 10$$

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

$$M_i = 10.300 \text{ Kgm}$$

$$\sigma_c = 38 \text{ Kg/cm}^2$$

$$A_f = 8,2 \text{ cm}^2$$

$$M_m = 7300 \text{ Kgm}$$

$$\sigma_c = 31 \text{ Kg/cm}^2$$

$$A_f = 5,8 \text{ cm}^2$$

$\left\{ \begin{array}{l} 4 + 4 \phi 10 d \\ 3 \phi 14 s \\ 3 \phi 10 d \\ 3 \phi 14 s \end{array} \right.$

$$T = \frac{1}{2} 5100 \times 4,5 = 11.300 \text{ Kg}$$

$$\tau_{\max} = \frac{11.300}{0,9 \times 95 \times 30} = 4,4 \text{ Kg/cm}^2$$

Adoperando staffe ϕ 8/20 cm a 2 braccia

$$\tau_s = \frac{1,01 \times 1400}{30 \times 15} = 2,4 \text{ Kg/cm}^2$$

$$\tau_R = 4,4 - 2,4 = 2 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

Mensola ϕ 8/20 cm

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PILASTRATA 39 - 41/20 - 26

$$l = 120$$

$$\sum P = \frac{28}{2} + \frac{2}{3} \times 23 + \frac{1}{2} 27 = 34,0 \text{ t}$$

Fondazione + terra sovrastante = 3140 Kg/ml

$$\sigma_t = \frac{34.000 \times 3140 \times 3,65}{365 \times 120} = 1,1 \text{ Kg/cm}^2$$

$$P_c = \frac{34.000}{3,65} = 9300 \text{ Kg/ml}$$

$$M = \frac{1}{12} 9300 \times 2,25^2 = 3950 \text{ Kgm}$$

per H = 100 cm

h = 95 cm

b = 30 cm

n = 10

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

$\sigma_c = 22 \text{ Kg/cm}^2$

$A_f = 3,0 \text{ cm}^2 \text{ } 3 \phi 12$

$$T = 4450 \text{ Kg}$$

$$\tau_{\max} = 1,1 \text{ Kg/cm}^2$$

Si adoperano staffe ϕ 8/25 cm

MENSOLA ϕ 12/25"

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PILASTRATA 5 - 37/24 - 57

$$l = 0,90 \text{ m}$$

$$\sum P = \frac{1}{3} \times 22,3 + \frac{2}{3} \times 27,3 + \frac{2}{3} \times 23 + \frac{1}{2} \times 28 = 55,00 \text{ t}$$

$$\sigma_t = \frac{55.000 \times (950 + 1520) \times 8,00}{90 \times 800} = 1,1 \text{ Kg/cmq}$$

$$p_c = \frac{55.000}{8,00} = 6800 \text{ Kg/ml}$$

$$M = \frac{1}{12} \times 6100 \times 2,25^2 = 3300 \text{ Kgm}$$

$$H = 100 \text{ cm} \quad h = 95 \text{ cm} \quad b = 30 \text{ cm} \quad n = 10$$

$$\sigma_f = 1400 \text{ Kg/cmq} \quad \sigma_c = 21 \text{ Kg/cmq} \quad A_f = 2,6 \text{ cmq}$$

$$\tau = \frac{6800}{95 \times 0,9 \times 30} = 2,8 \text{ Kg/cmq} < 4 \text{ Kg/cmq}$$

Si adoperano staffe $\phi 8/25$

ALA 1 $\phi 8/25$

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T.I.D.O A STELLA
TIPO MISTO A STELLA E IN LINEA

PILASTRATA 14 - 19/44 - ~~28/26~~ - ~~41/1~~ - 6 (STELLA)
~~44-28~~ / ~~26-41~~ (MISTO)

$$l = 1,30 \text{ m}$$

$$\sum P = \frac{17,6}{2} \times 19,6 + 22 + 24,4 + 25 + \frac{26,5}{2} = 113,1 \text{ t}$$

Peso fondazione:

$$\text{magro } 0,10 \times 1,30 \times 1,00 \times 2000 = 240 \text{ Kg/ml}$$

$$\text{ala } 0,30 \times 1,30 \times 1,00 \times 2500 = 900 \text{ Kg/ml}$$

$$\text{costola } 0,30 \times 0,70 \times 1,00 \times 2500 = 680 \text{ "}$$

$$1820 \text{ Kg/ml}$$

Terra sovrastante

$$0,80 \times 1,00 \times 1,00 \times 1800 = 1600 \text{ Kg/ml}$$

$$\sigma_t = \frac{113.100 + (1820 + 1600) \times 9,70}{130 \times 970} = 1,2 \text{ Kg/cm}^2$$

$$P_c = \frac{113.100}{9,70} = 11.500 \text{ Kg/ml}$$

$$M = \frac{1}{12} 11.500 \times 1,70^2 = 2900 \text{ Kgm}$$

$$T = 9800 \text{ Kg}$$

Armatura per:

$$H = 100 \text{ cm} \quad h = 95 \text{ cm} \quad b = 30 \text{ cm} \quad n = 10$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 21 \text{ Kg/cm}^2 \quad A_f / 3,0 \text{ cm}^2 \quad \left. \begin{matrix} \\ \end{matrix} \right\} 3 \phi 12$$

$$T = 9800 \text{ Kg}$$

$$\tau_{\max} = \frac{9.800}{0,9 \times 95 \times 30} = 3,7 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

Si adoperano prudenzialmente Staffe $\phi 8/20 \text{ cm}$

Mensola:

$$M = 11.500 \times \frac{0,50^2}{2} = 1450 \text{ Kgm}$$

$$H = 30 \text{ cm} \quad h = 25 \text{ cm} \quad b = 100 \text{ cm} \quad n = 10$$

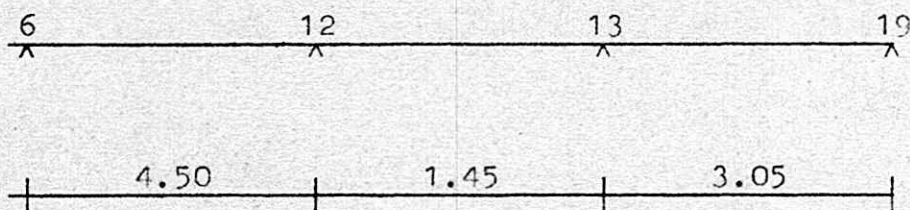
$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 30 \text{ Kg/cm}^2 \quad A_f = 4,5 \text{ cm}^2/\text{ml}$$

$$10 \phi 8$$

TIPO MISTO	Trave 18 - 43 come trave 23 - 36 tipo in linea					
	"	2 - 10	"	"	15 - 19	"
	"	19 - 17	"	"	24 - 57	"
	"	15 - 16	"	"	51 - 38	"
	"	3 - 28	"	"	7 - 14	"
	"	1 - 3	"	"	7 - 54	"
	"	39 - 44	"	"	40 - 47	"
	"	38 - 49	"	"	1 - 48	"

=====

PILASTRATE 6 - 19/41 - ~~41/35~~ - ²⁰⁻³⁴46/20 - ~~34/45~~ - ~~56/60~~ - ~~71~~ STELLA
 41-44 / 60-71 / 45-56
 1 = 120 m MISTO



$$\leq P = \frac{17,6}{2} + \frac{2}{3} 27 + 20 + \frac{26,5}{2} = 60,1 \text{ t}$$

p.p. fondazione

magro	$0.10 \times 0.90 \times 2000$	=	180 Kg/ml
ala	$0.30 \times 0.90 \times 2500$	=	670 "
costola	$0.30 \times 0.70 \times 2500$	=	530 "
			<u>1380 Kg/ml</u>

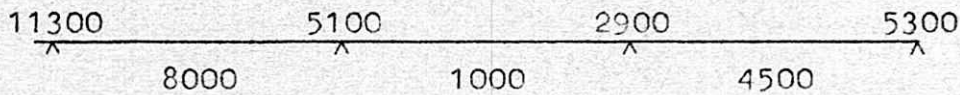
Terra sovrastante

$$0.60 \times 1.00 \times 1.00 \times 1800 = 1080 \text{ Kg/ml}$$

$$\sigma_t = \frac{60.100 + (1380 + 1080) \times 9.00}{90 \times 900} = 1.0 \text{ Kg/cm}^2$$

$$P_c = \frac{60.100}{9.00} = 6700 \text{ Kgm}$$

Sollecitazioni flettenti:



Armature per:

$$H = 100 \text{ cm} \quad h = 95 \text{ cm} \quad b = 30 \text{ cm} \quad n = 10$$

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

σ_c	40	33	26	19	24	26	Kg/cm ²
A_f	9,0	6,4	4,1	2,3	3,6	4,2	cm ²

$$T_{\max} = \frac{1}{2} 6700 \times 4,5 = 15.000 \text{ Kg}$$

$$\tau_{\max} = \frac{15.000}{0,9 \times 95 \times 38} = 5,8 \text{ Kg/cm}^2$$

Adoperando staffe ϕ 8/20 cm a 2 braccia

$$\tau_s = \frac{1,01 \times 1400}{30 \times 20} = 2,4 \text{ Kg/cm}^2$$

$$\tau_R = 5,8 - 2,4 = 3,4 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

=====

PILASTRATA 31 - 23/68 - 24 STELLA

63-7/1 MISTO

$$l = 1,15 \text{ m}$$

$$\leq P = 2 \times \frac{24}{2} + 2 \times \frac{31,6}{3} = 45 \text{ t}$$

p.p. fondazione

magro	$0,10 \times 1,15 \times 2000$	=	230 Kg/ml
ala	$1,25 \times 0,30 \times 2500$	=	870 "
costola	$0,55 \times 0,70 \times 2500$	=	950 "
			<u>2050 Kg/ml</u>

Terra sovrastante

$$0.60 \times 1,00 \times 1800 = 1100 \text{ Kg/ml}$$

$$\sigma_t = \frac{45.000 + (2050 + 1100) \times 9,00}{900 \times 115} = 0,7 \text{ Kg/cm}^2$$

$$p_c = \frac{45.000}{9.00} = 5000 \text{ Kg/ml}$$

$$M_i = \frac{1}{12} 5000 \times 4,50^2 = 8500 \text{ Kgm}$$

$$M_m = \frac{1}{14} 5000 \times 4,5^2 = 7300 \text{ Kgm}$$

$$T = 11.200 \text{ Kg}$$

$$\text{Per } H = 100 \text{ cm}$$

$$h = 95 \text{ cm}$$

$$b = 55 \text{ cm}$$

$$n = 10$$

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

$$M = 8500 \text{ Kgm}$$

$$\sigma_c = 25 \text{ Kg/cm}^2$$

$$M_m = 7300 \text{ Kgm}$$

$$A_f = 6,9 \text{ cm}^2 \left\{ \begin{array}{l} 4 + 4 \phi 10 d \\ 1 \phi 14 S \end{array} \right.$$

$$\sigma_c = 22,5 \text{ Kg/cm}^2$$

$$A_f = 5,9 \text{ cm}^2 \left\{ \begin{array}{l} 3 \phi 12 d \\ 2 \phi 14 S \end{array} \right.$$

$$T = 11.200 \text{ Kg}$$

$$\tau_{\max} = \frac{11.200}{0,9 \times 55 \times 95} = 2,4 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

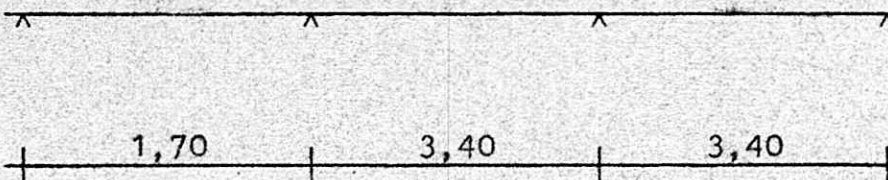
Si adoperano prudenzialmente Staffe $\phi 8/25 \text{ cm}$

Mensola 1 $\phi 10/25 \text{ cm}$

=====

PILASTRATA 20 - 23/24 - 24/31 - 34/35 - 38/56 - 59/45 - 48/46 - 49

1 = 1,25 m 71-74 60-63 / 56-59 / 45-48 STELLA MISTO



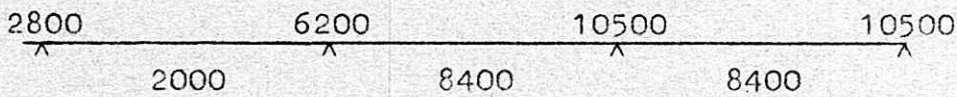
$$\sum P = \frac{26,5}{2} + 35 + 44 + \frac{24}{2} = 104,3 \text{ t}$$

p.p. fondazione + terra sovrastante: 3450 Kg/ml

$$\sigma_t = \frac{104.300 + (3400 \times 9,00)}{900 \times 125} = 1,2 \text{ Kg/cm}^2$$

$$p_c = \frac{104.300}{900} = 11.500 \text{ Kg/ml}$$

Sollecitazione flettenti



Armature per:

H = 100 cm

h = 95 cm

b = 55 cm

n = 10

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

σ_c		21	24	26	24	28	Kg/cm ²
A_f		5,11	6,6	8,5	6,6	8,5	cm ²

T = 18.000 Kg

$$\tau_{\max} = \frac{18.000}{0,9 \times 95 \times 55} = 3,8 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

Si adoperano Staffe ϕ 8/20 cm

ALA:

$$M = \frac{1}{2} \times 11.500 \times 0,35^2 = 750 \text{ Kgm}$$

H = 30 cm

h = 25 cm

b = 100 cm

n = 10

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

$\sigma_c = 20 \text{ Kg/cm}^2$

$A_f = 2,2 \text{ cm}^2/\text{ml}$

Si adoperano 1 ϕ 10/20 cm

=====

PILASTRATA 12 - 7/29 - 24/54 - 60 (44-39) (STELLA)
64-69 / 50-55 (MISTO)

$$l = 1.50 \text{ m}$$

$$\Sigma P = 4 \times 27 + \frac{1}{3} 27 + \frac{2}{3} 31,6 = 138,0 \text{ t}$$

p.p. fondazione + terra sovrastante: 3500 Kg/ml.

$$\sigma_t = \frac{138.000 + (3500 \times 9.00)}{900 \times 150} = 1,24 \text{ Kg/cm}^2$$

$$p_c = \frac{138.000}{9.00} = 15.300 \text{ Kg/ml}$$

$$M_i = \frac{1}{12} 15.300 \times 1,7^2 = 3700 \text{ Kgm}$$

Per H = 100 cm

h = 95 cm

b = 40 cm

n = 10

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

$$\sigma_c = 19 \text{ Kg/cm}^2$$

$$A_f = 3,0 \text{ cm}^2$$

(5φ10)

$$T = 13.000 \text{ Kg}$$

$$\tau_{\max} = \frac{13.000}{0,9 \times 40 \times 95} = 3,9 \text{ Kg/cm}^2$$

Si adoperano Staffe φ 8/20 cm

$$\tau_s = \frac{1,01 \times 1400}{40 \times 20} = 1,8 \text{ Kg/cm}^2$$

$$\tau_R = 2,1 \text{ Kg/cm}^2 < 4 \text{ Kg/cm}^2$$

Mensola:

$$M = \frac{1}{2} 15.300 \times 0,55^2 = 2400 \text{ Kgm}$$

Per H = 30 cm

h = 25 cm

b = 100 cm

n = 10

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

$$\sigma_c = 38 \text{ Kg/cm}^2$$

$$A_f = 7,3 \text{ cm}^2/\text{ml}$$

Si adoperano 5φ10 + 5φ12 / ml.

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STRUTTURE IN ELEVAZIONI

"TIPO IN LINEA"

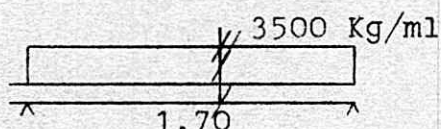
TRAVE 36 - 23/15 - 19/40 - 47

Analisi dei carichi:

p.p. trave	$0.20 \times 0.50 \times 2500$	=	250 Kg/ml
solaio	$4.50 \times 1.00 \times 460$	=	2100 "
sovraccarico	$4.50 \times 1.00 \times 250$	=	1150 "
			3500 Kg/ml

Per le travi continue a luci uguali e carichi uniformi costanti si considera ogni campata semincastrata.

Per le rimanenti a carico uniforme costante ma con luci disuguali o dissimetriche si e' adottato il sistema del Cross sulla trave continua.



$$M_i = \frac{1}{12} 3500 \times 1.70^2 = 850 \text{ Kgm}$$

$$M_{mx} = \frac{1}{14,5} 3500 \times 1.70^2 = \text{Kgm } 750$$

$$T = 3000 \text{ Kgm}$$

$$\text{Armatura per: } H = 20 \text{ cm} \quad h = 18,5 \text{ cm} \quad b = 50 \text{ cm}$$

$$m = 10 \quad \sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M = 850 \text{ Kgm} \quad \sigma_c = 45 \text{ Kg/cm}^2 \quad A_f = 3,3 \text{ cm}^2$$

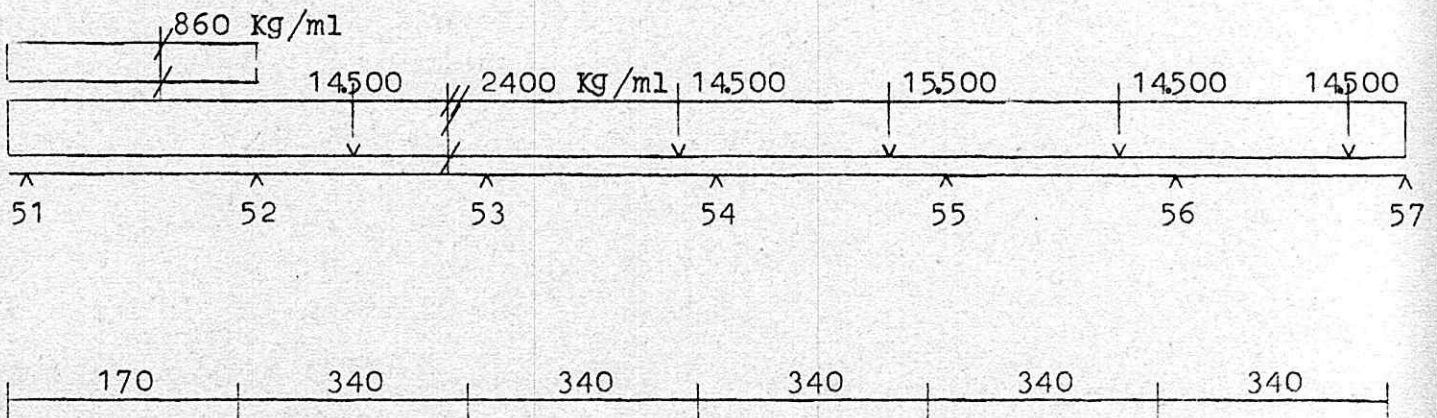
Verifica la taglio:

$$T = 3000 \text{ Kgm}$$

$$\tau = \frac{3000}{0,9 \times 18,5 \times 50} = 3,6 \text{ Kg/cm}^2$$

Adotteremo staffe $1 \phi 8/25''$

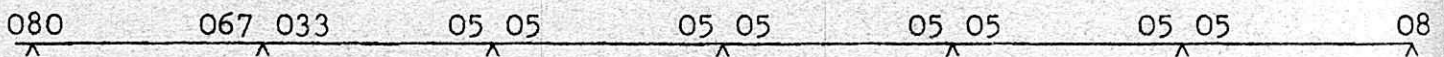
TRAVE 51 - 57/11 - 5



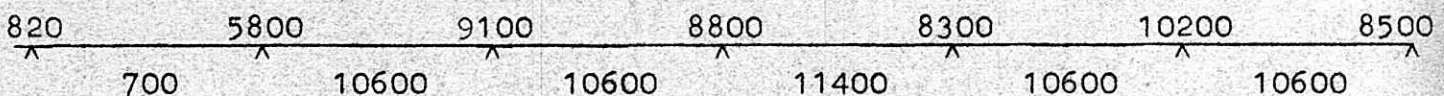
Analisi dei carichi:

peso solaio	$\frac{403}{2} \times 1.00 \times 710$	=	1450 Kg/ml
muratura		=	600 "
p.p. trave	$0.22 \times 0.60 \times 1.00 \times 2500$	=	350 "
			2400 "
carico concentrato			14500 Kg
" "			15500 Kg
balcone	1.00×860		860 Kg/ml

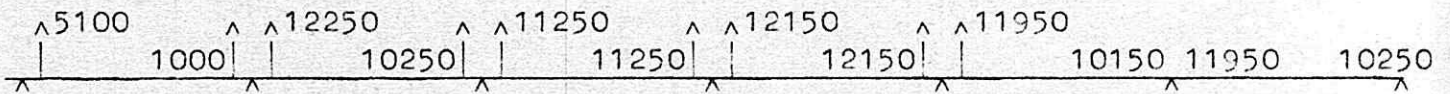
Coefficienti di ripartizione



Sollecitazioni flettenti:



Sollecitazioni taglianti:



Armatura per:

$$H = 60 \text{ cm}$$

$$h = 57 \text{ cm}$$

$$b = 22 \text{ cm}$$

$$\sigma_c = 65 \text{ Kg/cm}^2$$

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

$$m = 10$$

	18	57	65	64	65	63	66	65	65	64	65	61
σ_c												
σ_f	1,5	8,8	15,2	12,6	15,2	12,2	1,6	11,8	15,2	14,3	15,2	11,8
σ_f		2,2	15,2	9,5	15,2	9,2	1,6	5,9	15,2	14,3	15,2	9

Verifica al taglio:

$$T = 12.250 \text{ Kg}$$

$$\tau = \frac{12.250}{0,9 \times 57 \times 22} = 10,9 \text{ Kg/cm}^2$$

$$S_t = 10,9 \times \frac{340}{4} \times 22 = 18.000 \text{ Kg}$$

$$S_s = 1400 \times \frac{340}{2 \times 20} \times 1,01 = 12.000 \text{ Kg}$$

$$\tau_s = \frac{1400 \times 1,01}{22 \times 20} = 3,2 \text{ Kg/cm}^2$$

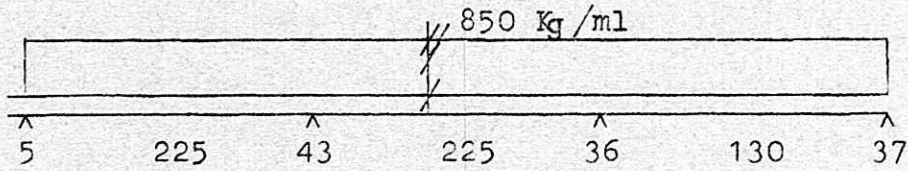
$$\tau_R = 10,9 - 3,2 = 7,7 \text{ Kg/cm}^2$$

$$A f_p = \frac{6000}{1400 \times \sqrt{2}} = 3,2 \text{ cm}^2$$

$$\tau_{f_p} = \frac{1400 \times 2,54 \times \sqrt{2}}{22 \times 30} = 7,6 \text{ Kg/cm}^2$$

=====

TRAVE 5 - 37/41 - 50/24 - 57/12 - 20



Analisi dei carichi:

p.p. trave

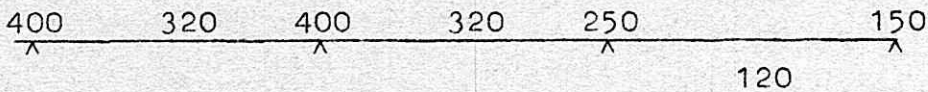
$$= 250 \text{ Kg/ml}$$

muratura

$$= 600 \text{ "}$$

$$850 \text{ "}$$

Sollecitazioni flettenti:



Sollecitazioni taglianti:



Armatura per:

$$H = 60 \text{ cm}$$

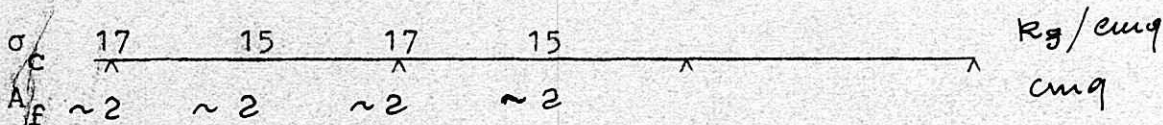
$$h = 57 \text{ cm}$$

$$b = 15 \text{ cm}$$

$$m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2$$

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



Verifica al taglio:

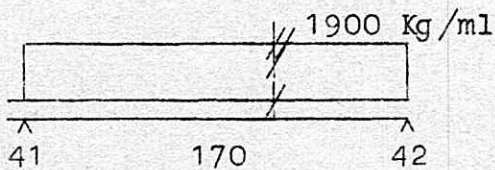
$$\tau = \frac{1250}{0.9 \times 15 \times 57} = 1,7 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

Staffe 1 ϕ 8/25"

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TRAVE 41 - 42/37 - 38/24 - 25/20 - 21

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Analisi dei carichi:

$$\text{solaio} \quad \frac{2.90}{2} \times 1.00 \times 710 = 1050 \text{ Kg/ml}$$

$$\text{p.p. trave} \quad 0.15 \times 0.60 \times 2500 = 250 \text{ "}$$

$$\text{muratura} \quad = 600 \text{ "}$$

$$1900 \text{ "}$$

$$M_i = \frac{1}{12} 1900 \times 1.70^2 = 500 \text{ Kgm}$$

$$M_m = \frac{1}{14.5} 1900 \times 1.70^2 = 410 \text{ Kgm}$$

$$T = 1900 \times \frac{1.70}{2} = 1600 \text{ Kg}$$

Armatura per:

$$H = 60 \text{ cm}$$

$$h = 57 \text{ cm}$$

$$b = 15 \text{ cm}$$

$$\sigma_c = 65 \text{ Kg/cm}^2$$

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

$$M = 500 \text{ Kgm}$$

$$\sigma_c = 19 \text{ Kg/cm}^2$$

$$A_f = 1 \text{ cm}^2$$

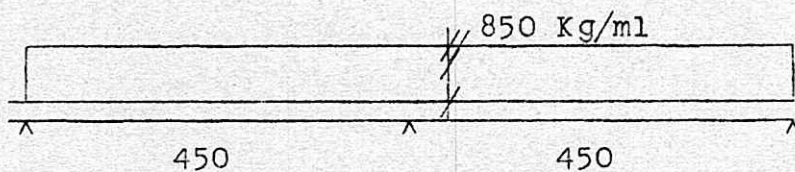
Verifica al taglio:

$$\tau = \frac{1600}{0.9 \times 15 \times 57} = 2.1 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

Staffe 1 ϕ 8/25"

=====

TRAVE 1 - 48/14 - 61



Analisi dei carichi:

$$\begin{array}{rcl}
 \text{p.p. trave} & 0.15 \times 0.60 \times 2500 & = 250 \text{ Kg/ml} \\
 \text{muratura} & & = 600 \text{ "} \\
 & & \hline
 & & 850 \text{ Kg/ml}
 \end{array}$$

$$M_i = \frac{1}{12} 850 \times 4.50^2 = 1500 \text{ Kgm}$$

$$M_{mx} = \frac{1}{14} 850 \times 4.50^2 = 1250 \text{ Kgm}$$

$$T = 850 \times \frac{4.50}{2} = 1900 \text{ Kg}$$

Armatura per:

$$H = 60 \text{ cm} \quad h = 57 \text{ cm} \quad b = 15 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M = 1500 \text{ Kgm}$$

$$\sigma_c = 33 \text{ Kg/cm}^2 \quad A_f = 2 \text{ cm}$$

$$M = 1250 \text{ Kgm}$$

$$\sigma_c = 31 \text{ Kg/cm}^2 \quad A_f = 2 \text{ cm}$$

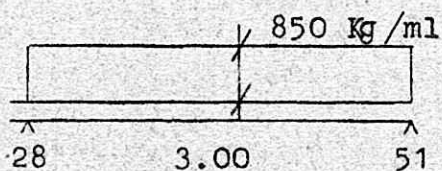
Verifica al taglio:

$$\tau = \frac{1900}{0.9 \times 15 \times 57} = 2,5 \text{ Kg/cm} < 6 \text{ Kg/cm}^2$$

Staffe 1 ϕ 8/25"

=====

TRAVE 38 - 51/4 - 42/11 - 25/21 - 58



Analisi dei carichi:

pp. trave

$$= 250 \text{ Kg/ml}$$

muratura

$$= 600 \text{ "}$$

$$850 \text{ "}$$

$$M_i = \frac{1}{12} 850 \times 3.00^2 = 650 \text{ Kgm}$$

$$M_{mx} = \frac{1}{14,5} 850 \times 3.00^2 = 550 \text{ Kgm}$$

$$T = \frac{3.00}{2} \times 850 = 1300 \text{ Kg}$$

Armatura per:

$$H = 60 \text{ cm}$$

$$h = 57 \text{ cm}$$

$$b = 15$$

$$m. = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2$$

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

$$M = 650 \text{ Kgm}$$

$$\sigma_c = 22 \text{ Kg/cm}^2$$

$$A_f = 1 \text{ cm}^2$$

$$M = 550 \text{ Kgm}$$

$$\sigma_c = 20 \text{ Kg/cm}^2$$

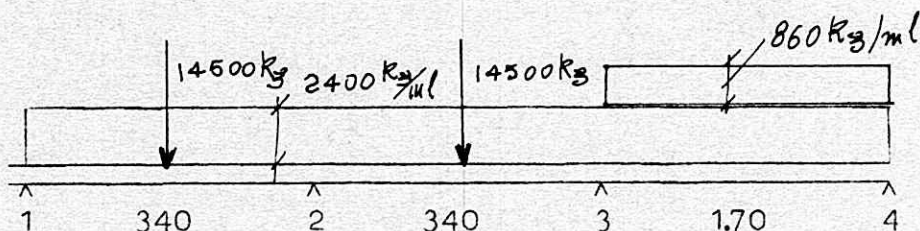
$$A_f = 1 \text{ cm}^2$$

Verifica al taglio:

$$\tau = \frac{1300}{0.9 \times 15 \times 57} = 1,7 \text{ Kg/cm}^2$$

Staffe 1 ϕ 8/25"

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TRAVE 1 - 4/61 - 58

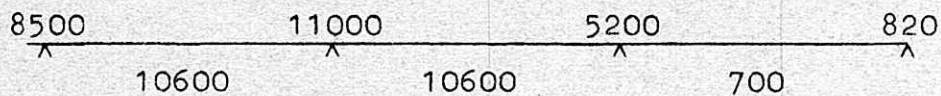
Analisi dei carichi:

peso solaio	=	1450 kg/ml
muratura	=	600 "
p.p. trave	=	350 "
		2400 "
carico concentrato		14500 kg
balcone 1.00 x 860		860 "

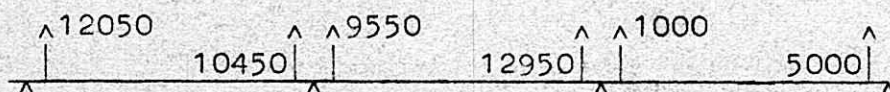
Coefficienti di ripartizione



Sollecitazioni flettenti:



Sollecitazioni taglianti:



Armature per:

H = 60 cm	h = 57 cm	b = 22 cm	m = 10
$\sigma_c = 65 \text{ Kg/cm}^2$	$\sigma_f = 1400 \text{ Kg/cm}^2$		

σ_c	61	65	66	65	53	18	20	Kg/cm ²
A_f	12	15,2	16	15,2	6,8	1,5	2	cm ²
A'_f	9	15,2	16	15,2	2			cm ²

Verifica la taglio:

$$T = 12.050 \text{ K}$$

$$\tau = \frac{12.050}{0.9 \times 22 \times 57} = 9,4 \text{ Kg/cm}^2$$

Adotteremo staffe 1 ϕ 8/20"

$$\tau_s = \frac{1.01 \times 1400}{22 \times 20} = 3,2 \text{ Kg/cm}^2$$

$$\tau_R = 9,4 - 3,2 = 6,2 \text{ Kg/cm}^2 > 6 \text{ Kg/cm}^2$$

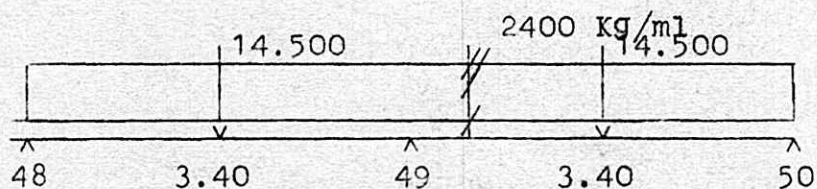
$$S_t = 9,4 \times \frac{3.40}{4} \times 22 = 17.600 \text{ Kg}$$

$$S_s = 1400 \times \frac{3.40}{2.20} \times 1.01 = 12.000 \text{ Kg}$$

$$A_{f_p} = \frac{5600}{1400 \times \sqrt{2}} = 2,9 \text{ cm}^2$$

=====

TRAVE 48 - 50/12 - 14



Analisi dei carichi:

$$\text{p.p. solaio} \quad \frac{4,0}{2} \times 1,00 \times 710 = 1450 \text{ Kg/ml}$$

$$\text{muratura} = 600 \text{ "}$$

$$\text{p.p. trave} \quad 0,22 \times 0,60 \times 1,00 \times 2500 = 350 \text{ "}$$

$$2400 \text{ "}$$

$$\text{carico concentrato} \quad 14500 \text{ Kg}$$

$$M_{48} = M_{49} = M_{50} = \frac{1}{12} 2400 \times 3,40^2 + \frac{14,500 \times 3,40}{8} = 8500 \text{ Kgm}$$

$$M_{mx} = 10.600 \text{ Kgm}$$

Taglio:

$$T = 11.350 \text{ Kgm}$$

Armatura per:

$$H = 60 \text{ cm} \quad h = 57 \text{ cm} \quad b = 22 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M_i = 8500 \text{ Kgm} \quad \sigma_c = 61 \text{ Kg/cm}^2 \quad A_f = 11,8 \text{ cm}^2$$

$$A'_f = 9 \text{ cm}^2$$

$$M_{mx} = 10.600 \text{ Kgm} \quad \sigma_c = 65 \text{ Kg/cm}^2 \quad A_f = A'_f = 15,2 \text{ cm}^2$$

Verifica al taglio:

$$\tau = \frac{11.350}{0,9 \times 22 \times 57} = 10 \text{ Kg/cm}^2$$

Adotteremo Staffe 1 ϕ 8/20"

$$\tau_s = \frac{1,01 \times 1400}{22 \times 20} = 3,2 \text{ Kg/cm}^2$$

$$\tau_R = 10 - 3,2 = 6,8 \text{ Kg/cm}^2 > 6 \text{ Kg/cm}^2$$

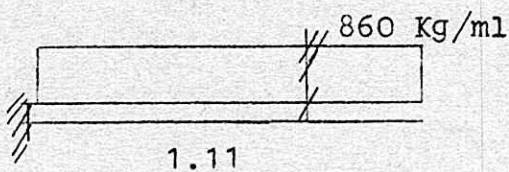
$$S_t = 10 \times \frac{3,40}{4} \times 22 = 19.000 \text{ Kgm}$$

$$S_s = 1400 \times \frac{3.40}{2.20} \times 1.01 = 12.000$$

$$A_{fP} = \frac{7000}{1400 \times \sqrt{2}} = 3,6 \text{ cmq}$$

=====

LOGGIA



Analisi dei carichi:

860 Kg/ mq

$$M_i = 860 \times \frac{1.11^2}{2} = 540 \text{ Kgm}$$

$$T = 860 \times 1.11 = 950 \text{ Kgm}$$

Armatura per:

$$H = 16 \text{ cm}$$

$$h = 14,5 \text{ cm}$$

$$b = 100 \text{ cm}$$

$$m = 10$$

$$\sigma_c = 65 \text{ Kg/cmq}$$

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

$$M = 540 \text{ Kgm}$$

$$\sigma_c = 30 \text{ Kg/cmq}$$

$$A_f = 2,8 \text{ cmq/ml}$$

Verifica al taglio:

$$\tau = \frac{950}{0,9 \times 100 \times 14,5} = 0,75 \text{ Kg/cm} < 6 \text{ Kg/cm}$$

=====

COPERTURA VANO SCALA

$$L = 3.25 \times 1.05 = 3.40 \text{ m}$$

$$\text{---} \quad \text{3.40} \quad \text{---}$$

$$\text{carico} = 510 \text{ Kg/ml}$$

$$M_i = M_m = \frac{1}{12} 510 \times 3.40^2 = 490 \text{ Kgm}$$

$$T = 510 \times \frac{3.40}{2} = 870 \text{ Kgm}$$

Armatura per:

$$H = 10 \text{ cm}$$

$$h = 8,5 \text{ cm}$$

$$b = 100$$

$$m = 10$$

$$\sigma_c = 65 \text{ Kg/cmq}$$

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

$$M = 490 \text{ Kgm}$$

$$\sigma_c = 53 \text{ Kg/cmq}$$

$$A_f = 4,6 \text{ cmq/ml}$$

$$A' f = 2 \text{ cmq/ml}$$

$$\tau = \frac{870}{0.9 \times 8.5 \times 100} = 0,12 \text{ Kg/cm} < 6 \text{ Kg/cm}$$

=====

TRAVE COPERTURA VANO SCALA

$$\text{---} \quad \text{3.40} \quad \text{---}$$

Analisi dei carichi:

p.p.

$$= 150 \text{ Kg/ml}$$

$$\text{solaio copertura} \quad \frac{1}{4} 510 \times 3.25$$

$$= \frac{420}{570} \text{ "}$$

$$M_{mx} = M_i = \frac{1}{12} 570 \times 3.40^2 = 550 \text{ Kgm}$$

$$T = 570 \times \frac{3.40}{2} = 980 \text{ Kgm}$$

Armatura per:

$$H = 40 \text{ cm}$$

$$h = 37 \text{ cm}$$

$$b = 15 \text{ cm}$$

$$m = 10$$

$$\sigma_c = 65 \text{ Kg/cmq}$$

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

$$M = 550 \text{ Kgm}$$

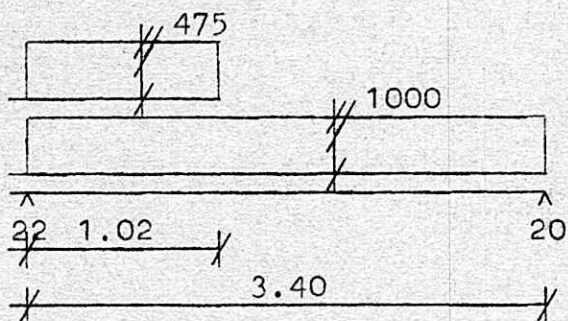
$$\sigma_c = 32 \text{ Kg/cmq}$$

$$A_f = 1,3 \text{ cmq}$$

$$\tau = \frac{980}{0.9 \times 15 \times 37} = 2 \text{ Kg/cmq} < 6 \text{ Kg/cmq}$$

=====

TRAVE 22 - 20



Analisi dei carichi:

$$\text{p.p. } 0.15 \times 0.40 \times 2500$$

$$= 150 \text{ Kg/ml}$$

pianerottolo

$$= 850 \text{ "}$$

$$1000 \text{ "}$$

meta' rampa scala

$$475 \text{ "}$$

$$M_{mx} = \frac{1}{12} 1000 \times 3.40^2 + \frac{1}{475 \cdot 2} \left(\frac{475 \times 1.02}{2 \times 3.40} (2.38 + 3.40) - \left[\frac{-475 - 1.02^2}{3.40} \right] \right)$$

$$\left(\frac{1}{2} - \frac{2.04}{3 \times 3.40} + \frac{1.02^2}{4 \times 3.40^2} \right) + 475 \times 1.02^2 \left(\frac{1.02}{3 \times 3.40} - \frac{1.02^2}{4 \times 3.40^2} \right) \Bigg] \Bigg)^2 -$$

$$- 475 \times 1.02^2 \left(\frac{1}{2} - \frac{2 \times 1.02^2}{3 \times 3.40} + \frac{1.02^2}{4 \times 3.40^2} \right) = 1040 + 130 = 1170 \text{ Kgm}$$

$$M_{22} = \frac{1}{12} 1000 \times 3.40^2 + 740 = 1780 \text{ Kg M}$$

$$M_{20} = \frac{1}{12} 1000 + \overline{3.40}^2 + 510 = 1550 \text{ Kg m}$$

$$T = 1700 + 350 = 2050 \text{ Kg m}$$

Armatura per:

$$H = 40 \text{ cm} \quad h = 37 \text{ cm} \quad b = 15 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cmq} \quad \sigma_f = 1400 \text{ Kg/cmq}$$

Sollecitazioni flettenti:

$$\begin{array}{ccc} 1800 & 1200 & 1600 \\ \wedge & & \wedge \end{array}$$

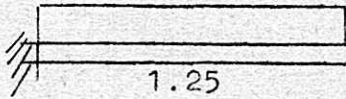
σ_c	$\frac{63}{\wedge}$	50	$\frac{59}{\wedge}$	Kg/cmq
A_f	3,4	2,7	2,9	cmq

Verifica al taglio:

$$T = 2050 \text{ Kg}$$

$$\tau = \frac{2050}{0.9 \times 15 \times 37} = 4,15 \text{ Kg/cmq} < 6 \text{ Kg/cmq}$$

=====

MENSOLA

$$\text{carico} = 850 \text{ Kg/ml}$$

$$M_i = 850 \times \frac{1.25^2}{2} = 660 \text{ Kgm}$$

$$T = 850 \times 1.25 = 1050 \text{ Kg}$$

Armatura per:

$$H = 8 \text{ cm} \quad h = 6.8 \text{ cm} \quad b = 100 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M = 660 \text{ Kgm} \quad \sigma_c = 61 \text{ Kg/cm}^2 \quad A_f = A'_f = 7 \text{ cm}^2/\text{ml}$$

$$\tau = \frac{1050}{0.9 \times 6.8 \times 100} = 1.7 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

=====

SCALA

Analisi di carico riportata a ml di pianta.

$$\text{p.p. rampa } (0.28 \times 0.08 + 0.18 \times 0.08) \frac{1.00}{0.28} \times 2500 = 330 \text{ Kg/mq}$$

$$\text{rivestimento } (0.02 \times 0.18 + 0.025 \times 0.28) \frac{1.00}{0.28} \times 2000 = 75 \text{ "}$$

$$\text{intonaco} = 30 \text{ "}$$

$$\text{sovraccarico} = 400 \text{ "}$$

$$835 \text{ "}$$

$$\text{carico ripartito lineare del parapetto} = 90 \text{ "}$$

carico normale alla rampa per gradino

$$835 \times 0.28 \times \frac{0.28}{0.334} = 200 \text{ Kg/ml}$$

$$90 \times 0.28 \times \frac{0.28}{0.334} = 21 \text{ "}$$

$$M_i = 200 \times \frac{1.07^2}{2} + 21 \times 1.07 = 170 \text{ Kgm}$$

$$T_i = (835 \times 1.07 + 90) \times 0.28 \times \frac{0.28}{0.334} = 235 \text{ Kgm}$$

ARmatura per:

$$H = 26 \text{ cm}$$

$$h = 24.5 \text{ cm}$$

$$b = 8 \text{ cm}$$

$$\sigma_c = 36 \text{ Kg/cm}^2$$

$$A_f = 0.6 \text{ cm}^2$$

$$\tau = \frac{235}{0.9 \times 8 \times 24.5} = 1.3 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

Soletta rampante semincastrata agli estremi

$$l = 3.27 \times 1.05 = 3.45 \text{ m.}$$

$$\text{carico a ml. } 835 \times 1.02 = 850 \text{ Kg/ml}$$

$$M_{mx} = \frac{1}{10} 850 \times 3.45^2 = 1020 \text{ Kgm}$$

$$M_i = \frac{1}{12} 850 \times 3.45^2 = 850 \text{ Kgm}$$

$$T = 850 \times \frac{3.45}{2} = 1500 \text{ Kgm}$$

Armature per:

$$H = 8 \text{ cm}$$

$$h = 6.8 \text{ cm}$$

$$b = 100$$

$$m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2$$

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

$$M_{mx} = 1020 \text{ Kgm}$$

$$r = \frac{6.8}{\sqrt{\frac{102000}{100}}} = \frac{6.8}{32} = 0.212$$

$$\sigma_c = 67 \text{ Kg/cm}^2$$

$$t' = 0.00414$$

$$Af = A' f = 0.00414 \times 100 \times 32 = 13.250 \text{ cm}^2$$

Verifica al taglio:

$$\tau = \frac{1500}{0.9 \times 6,8 \times 100} = 2,45 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

Trave a ginocchio semincastrata agli estremi.

carico a ml. di trave

$$\text{rampa } 835 \times 1.02 = 850 \text{ Kg/ml}$$

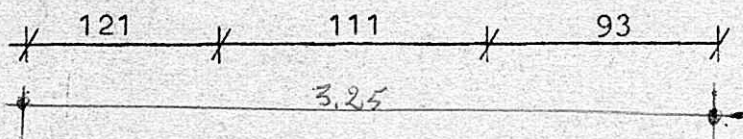
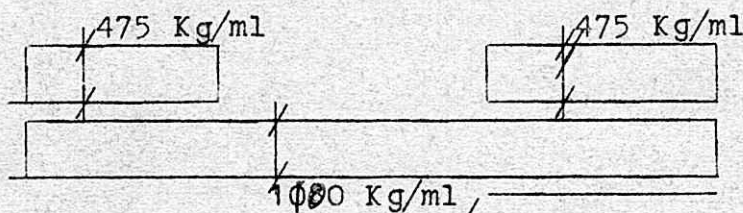
$$\text{trave } 0,15 \times 0,50 \times 1,00 \times 2500 \times \frac{0,334}{0,28} = 230 \text{ "}$$

$$\text{carico dovuto al parapetto} = 1080 \text{ "}$$

$$\text{carico dovuto alla rampa antistante} = 20 \text{ "}$$

$$1100 \text{ "}$$

$$835 \times 0.56 = 475 \text{ "}$$



$$L = 3,25 \text{ m}$$

$$M_i = M_1 = \frac{1}{2} \times \frac{1}{12} \times 3,25^2 \times 1100 + \frac{1}{12} \times 475 \times 1,21^2 \left[6 - 8 \times \frac{1,21}{3,25} + 3 \times \frac{1,21^2}{3,25^2} \right] +$$

$$+ \frac{1}{12} \times 475 \times \frac{1,21^2}{3,25} (1 + 3 \times \frac{2,31}{3,25}) = 1000 + 310 + 70 = 1380 \text{ Kgm}$$

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$$T = 1100 \times \frac{3,25}{2} + \frac{1}{2} 475 \times 1,21 \left[2 - \frac{1,21^2}{3,25^2} \left(1 + \frac{2,03}{3,25} \right) \right] + \frac{1}{2} 475 +$$

$$+ 3,25 \times \frac{1,21^2}{3,25^2} \times \left(1 + \frac{2,03}{3,25} \right) = 1800 + 510 + 70 = 2380 \text{ kgm}$$

Armatura per:

$$H = 50 \text{ cm} \quad h = 47 \text{ cm} \quad b = 15 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M = 1400 \text{ Kg} \cdot \text{m} \quad \sigma_c = 41 \text{ Kg/cm}^2 \quad A_f = 2,4 \text{ cm}^2$$

Verifica al Taglio:

$$T = 2380 \text{ Kg}$$

$$\tau = \frac{2380}{0,9 \times 47 \times 15} = 3,6 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

Torsione

$$M_i = (850 \times \frac{1,02^2}{2} + 90 \times 1,00) \times \frac{0,28}{0,334} = 435 \text{ Kg} \cdot \text{m}$$

$$T_i = (850 \times 1,02 + 90) \times \frac{0,28}{0,334} = 800 \text{ Kg} \cdot \text{m}$$

Momento torcente rispetto all'asse della trave

$$M_t = 435 \times 800 \times 0,075 = 495 \text{ Kg} \cdot \text{m}$$

$$\tau = \frac{49500}{50 \times 15^2} \times 3,90 = 14,6 \text{ Kg/cm}^2 < 16 \text{ Kg/cm}^2$$

Armatura a torsione:

$$M_{tmx} = 495 \times \frac{3,25}{2} = 81.000 \text{ Kg} \cdot \text{cm}$$

$$A = 15 \times 50 = 750 \text{ cm}^2$$

$$C = 2(50 + 15) = 130 \text{ cm}$$

$$A_{f1} = \frac{81.000 \times 130}{2 \times 1400 \times 750} = 5 \text{ cm}^2 \quad A_{fs} = \frac{81.000 \times 15}{2 \times 1400 \times 750} = 0,6 \text{ cm}^2$$

TRAVE II° PIANO TIPO IN LINEA

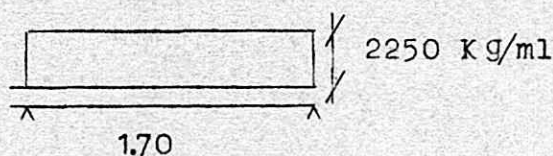
TRAVE 51 - 57/11 - 5/1 - 4/48 - 50/58 - 61/12 - 14

Analisi dei carichi:

peso solaio	=	1450 Kg/ml
muratura	=	600 "
p.p. trave 0.20 x 0.40 x 2500	=	200 "
		2250 "
balcone		860 "

data l'eguaglianza delle luci e dei carichi, la costante
rigidezza delle travate:

Si considera ogni campata come sem incastrata



$$M_i = M_{mx} = \frac{1}{12} \cdot 1.70^2 \times 2250 = 570 \text{ Kg m}$$

$$T = 2250 \times \frac{1.70}{2} = 1900 \text{ Kg m}$$

Armature per:

$$H = 20 \text{ cm} \quad h = 18,5 \text{ cm} \quad b = 40 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M = 570 \text{ Kg m} \quad \sigma_c = 40 \text{ Kg/cm}^2 \quad A_f = 2,4 \text{ cm}^2$$

Verifica al taglio:

$$\tau = \frac{1900}{0.9 \times 18.5 \times 40} = 2,9 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

stiffe $1\phi 8/25''$

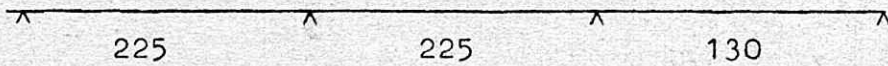
TRAVE 36 - 23/40 - 47/19 - 15

come travi I° piano.

TRAVI 1 - 48/14 - 61

come travi I° piano.

TRAVE 41 - 50/5 - 37/24 - 57/12 - 20



Analisi dei carichi:

p.p. trave

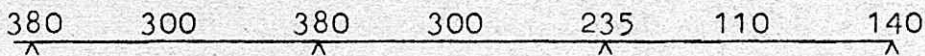
= 200 Kg/ml

muratura

= 600 "

800 "

Sollecitazioni flettenti:



Sollecitazioni taglianti:



Arlature per:

H = 20 cm

h = 18,5 cm

b = 40 cm

m = 10

$\sigma_c = 65 \text{ Kg/cm}^2$

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

σ_c	32	28	32	28	25	16	19	Kg/cm ²
A_f	1,6	1,3	1,6	1,3	1,2	1	1	cm ²

Verifica al Taglio:

$$\tau = \frac{1200}{0,9 \times 18,5 \times 40} = 1,8 \text{ K /cmq} < 6 \text{ K /cmq}$$

=====

TRAVE 4 - 42/38 - 51/11 - 25/21 - 58

Λ Λ
3.00

Analisi dei carichi:

p.p. trave	= 200 Kg/ml
muratura	= 600 "
	800 Kg/ml

$$M_i = M_{mx} = \frac{1}{12} 800 \times 3.00^2 = 610 \text{ Kgm}$$

$$T = 1200 \text{ Kgm}$$

Armatatura per:

$$H = 20 \text{ cm} \quad h = 18,5 \text{ cm} \quad b = 40 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cmq} \quad \sigma_f = 1400 \text{ Kg/cmq}$$

$$M = 610 \text{ Kgm} \quad \sigma_c = 42 \text{ Kg/cmq} \quad A_f = 2,6 \text{ cmq}$$

Verifica al taglio:

$$\tau = \frac{1200}{0,9 \times 18,5 \times 40} = 1,8 \text{ Kg/cmq} < 6 \text{ Kg/cmq}$$

=====

TRAVE 41 - 42/38 - 37/24 - 25/20 - 21

1.70

Analisi dei carichi:

solaio	=	1050 Kg/ml
p.p. trave	=	200 "
muratura	=	600 "
		<u>1850 Kg/ml</u>

$$M_i = M_{mx} = \frac{1}{12} 7850 \times 1.70^2 = 465 \text{ Kgm}$$

$$T = 1850 \times \frac{1.70}{2} = 1600 \text{ Kgm}$$

Armature per:

H = 20 cm	h = 18,5 cm	b = 40 cm	m = 10
$\sigma_c = 65 \text{ Kg/cmq}$	$\sigma_f = 1400 \text{ Kg/cmq}$		
M = 465 Kgm	$\sigma_c = 36 \text{ Kg/cmq}$	$A_f = 2 \text{ cmq}$	

Verifica al taglio:

$$\tau = \frac{1600}{0.9 \times 18.5 \times 40} = 2,5 \text{ Kg/cmq} < 6 \text{ Kg/cmq}$$

=====

TRAVI ULTIMO SOLAIO

TRAVE 51 - 57/5 - 11/1 - 4/48 - 50/12 - 14/58 - 61

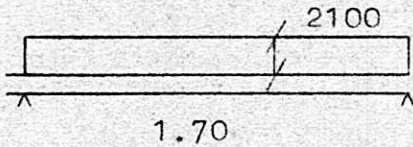
Analisi dei carichi:

p.p. trave	$0.22 \times 0.31 \times 2500$	=	170 Kg/ml
solaio	$\frac{4,21}{2} \times 820$	=	1730 "

muretto attico

$$= \frac{200 \text{ Kg/ml}}{2100 \text{ Kg/ml}}$$

Si considera ogni campata sem incastrata



$$M_i = M_{mx} = \frac{1}{12} 2100 \times 1.70^3 = 530 \text{ Kgm}$$

$$T = 2100 \times \frac{1.70}{2} = 1800 \text{ Kgm}$$

Armatura per:

$$H = 31 \text{ cm}$$

$$h = 29,5 \text{ cm}$$

$$b = 22 \text{ cm}$$

$$m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2$$

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

$$M = 530 \text{ Kgm}$$

$$\sigma_c = 32 \text{ Kg/cm}^2$$

$$A_f = 1n4 \text{ cm}^2$$

Verifica al taglio:

$$\tau = \frac{1800}{0.9 \times 29.5 \times 22} = 3,1 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

staffe $1\phi 8 / 25''$

=====

TRAVE 23 - 36/40 - 47/10 - 15

Analisi dei carichi:

$$\text{p.p. trave } 0.25 \times 0.31 \times 2500$$

$$= 200 \text{ Kg/ml}$$

$$\text{solaio } 4.21 \times 820$$

$$= 3500 \text{ "}$$

$$3700 \text{ Kg/ml}$$

$$M_i = M_{mx} = \frac{1}{12} 3700 \times 1.70^2 = 930 \text{ Kgm}$$

$$T = 3700 \times \frac{1.70}{2} = 3150 \text{ Kgm}$$

Armatura per:

$$H = 31 \text{ cm} \quad h = 29,5 \text{ cm} \quad b = 25 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M = 930 \text{ Kgm} \quad \sigma_c = 41 \text{ Kg/cm}^2 \quad A_f = 2,42 \text{ cm}^2$$

Verifica al taglio:

$$\tau = \frac{3150}{0,9 \times 29,5 \times 25} = 4,8 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

stette $1\phi 8/20''$

=====

TRAVE 41 - 50/5 - 37/24 - 57/12 - 20

Analisi dei carichi:

$$\text{p.p. trave } 0,15 \times 0,31 \times 2500 = 120 \text{ Kg/ml}$$

$$\text{muretto attico} = \frac{200}{320 \text{ Kg/ml}}$$

$$M_{mx} = M_i = 140 \text{ Kgm}$$

$$T = 320 \times \frac{2,25}{2} = 360 \text{ Kgm}$$

Armatura per:

$$H = 31 \text{ cm} \quad h = 29,5 \text{ cm} \quad b = 15 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M = 140 \text{ Kgm} \quad \sigma_c = 19 \text{ Kg/cm}^2 \quad A_f = 1 \text{ cm}^2$$

Verifica la taglio:

$$\tau = \frac{360}{0,9 \times 29,5 \times 15} \cong 1 < 6 \text{ Kg/cm}^2$$

=====

TRAVE 11 - 25/21 - 58/38 - 51/4 - 42

Λ Λ
3.00

Analisi dei carichi:

p.p. trave	= 120 Kg/ml
muretto attico	= 200 "
	320 Kg/ml

$$M_i = M_{mx} = \frac{1}{12} 320 \times 3.00^2 = 240 \text{ Kgm}$$

$$T = 320 \times \frac{3.00}{2} = 480 \text{ Kgm}$$

Armatura per:

$$H = 31 \text{ cm} \quad h = 29,5 \text{ cm} \quad h = 15 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

$$M_i = 240 \text{ Kgm} \quad \sigma_c = 25 \text{ Kg/cm}^2 \quad A_f \approx 1 \text{ cm}^2$$

Verifica la taglio:

$$\tau = \frac{480}{0.9 \times 15 \times 29,5} = 1,2 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

=====

TRAVE 38 - 37/41 - 42/24 - 25/20 - 21

Λ Λ
1.70

Analisi dei carichi:

solaio	$\frac{2.76}{2}$	$\times 820$	= 1140 Kg/ml
p.p. trave	$0.25 \times 0.31 \times 2500$		= 200 "
muretto attico			= 200 "
			1540 Kg/ml

$$M_i = M_{mx} = \frac{1}{12} \times 1.70^2 \times 1540 = 385 \text{ Kgm}$$

$$T = 1540 \times \frac{1.70}{2} = 1300 \text{ Kgm}$$

Armatura per:

$$H = 31 \text{ cm} \quad h = 19,5 \text{ cm} \quad b = 25 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cmq} \quad \sigma_f = 1400 \text{ Kg/cmq}$$

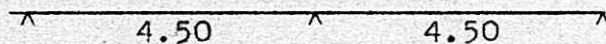
$$M = 385 \text{ Kgm} \quad \sigma_c = 25 \text{ Kg/cmq} \quad A_f = 1 \text{ cmq}$$

Verifica la taglio:

$$\tau = \frac{1300}{0.9 \times 29,5 \times 25} = 2 \text{ Kg/cmq} < 6 \text{ Kg/cmq}$$

=====

TRAVE 1 - 48/14 - 61



Analisi dei carichi:

p.p. trave	=	250 Kg/ml
muretto attico	=	200 "
		450 Kg/ml

$$M_i = M_{mx} = \frac{1}{12} \times 450 \times 4.50^2 = 770 \text{ Kgm}$$

$$T = 450 \times \frac{4.50}{2} = 1020 \text{ Kgm}$$

Armature per:

$$H = 60 \text{ cm} \quad h = 57 \text{ cm} \quad b = 15 \text{ cm} \quad m = 10$$

$$\sigma_c = 65 \text{ Kg/cmq} \quad \sigma_f = 1400 \text{ Kg/cmq}$$

$$M = 770 \text{ Kgm}$$

$$\sigma_c = 23 \text{ Kg/cm}^2$$

$$A_f = 1 \text{ cm}^2$$

Verifica al taglio:

$$\tau = \frac{1020}{0.9 \times 15 \times 57} = 1,4 \text{ Kg/cm}^2 < 6 \text{ Kg/cm}^2$$

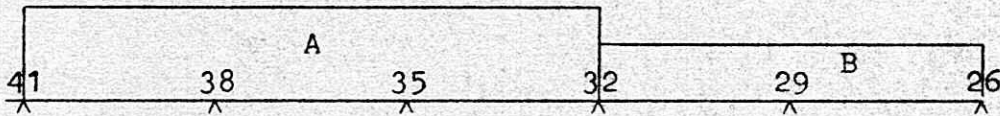
=====

"TIPO A STELLA" - TIPO HISTO

La muratura è riferita al tipo misto

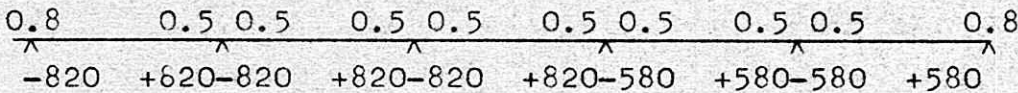
Primo solaio di copertura.

Trave 41 ÷ 26 (lev. A B C)

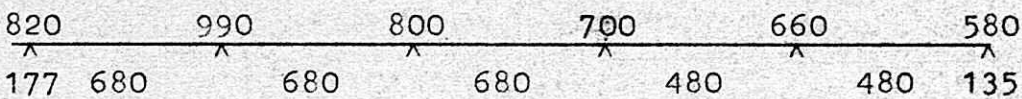


Analisi dei carichi:

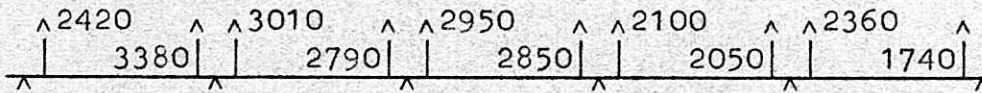
A) solaio	c. anal. preced.	=	1450 Kg/ml
muratura	" "	=	600 "
p.p. trave	" "	=	350 "
balcone	860 × 1,17	=	1000 "
			3400 Kg/ml
B) solaio		=	1450 Kg/ml
muratura		=	600 "
p.p. trave		=	350 "
			2400 Kg/ml



Azioni flettenti:



Azioni taglianti:



Calcolo delle armature per:

$$H = 60 \text{ cm} \quad h = 57 \text{ cm} \quad b = 22 \text{ cm} \quad m = 10$$

$$\sigma_{c \text{ max}} = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

Data l'esiguità delle azioni flettenti in rapporto alla notevole ripidezza della trave si arma questa rispetto alla massima azione flettente:

$$M = 1000 \text{ Kgm} \quad \sigma_c = 22 \text{ Kg/cm}^2 \quad A_f = 1,33 \text{ cm}^2$$

Si adopera perciò una doppia armatura simmetrica costituita da 3 ϕ 8 sup. e 3 ϕ 8 inf.

$$T_{\text{max}} = 3400 \text{ Kg} \quad \sigma_{\text{max}} = 3,05 \text{ Kg/cm}^2 < 6$$

Si adoperano Staffe ϕ 6/25 cm.

Data la modestia delle luci per il piano tipo in riferimento alle stesse azioni flettenti e per i seguenti dati:

$$H = 20 \text{ cm} \quad h = 18,5 \text{ cm} \quad b = 40 \text{ cm} \quad n = 10 \text{ cm}$$

$$\sigma_{c \text{ max}} = 65 \text{ Kg/cm}^2 \quad \sigma_f = 1400 \text{ Kg/cm}^2$$

si hanno:

σ_c	50	45	56	45	49	45	45	36	44	36	40	Kg/cm ²
A_f	3,45	2,96	4,17	2,96	3,37	2,96	2,96	2,03	2,78	2,03	2,45	cm ²

$$T_{\text{max}} = 3400 \text{ Kg} \quad \sigma_{\text{max}} = \frac{3400}{0,9 \times 18,5 \times 40} = 5,1 \text{ Kg/cm}^2 < 6$$

Adoperando staffe ϕ 8/20 cm

$$\tau_{os} = \frac{1.07 \times 1400}{20 \times 40} = 1,77 \text{ Kg/cm}^2$$

$$\tau_R = 5,1 - 1,77 = 3,33 \text{ Kg/cm}^2$$

Balcone

$$M = 860 \times \frac{1.17^2}{2} = 590 \text{ Kgm}$$

$$H = 17 \text{ cm}$$

$$h = 15,5 \text{ cm}$$

$$b = 100 \text{ cm}$$

$$n = 10$$

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

$$\sigma_c = 30 \text{ Kg/cm}^2$$

$$A_f = 2,9 \text{ cm}^2$$

Si adoperano 5 ϕ 10 al ml

a 3,93 cm

=====

TRAVE 44 - 29 al Piano Terra (Tov. 4 B C)

Data l'eguaglianza delle luci e dei carichi, la costante ripidezza delle travate, si considerano le singole campate come semincastrate.

$$L = 1,70 \text{ m}$$

$$p = 2400$$

$$M_i = M_{mx} = \frac{1}{12} 2400 \times 1,7^2 = 580 \text{ Kgm}$$

$$H = 60 \text{ cm}$$

$$h = 57 \text{ cm}$$

$$b = 22 \text{ cm}$$

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

$$\sigma_c = 15 \text{ Kg/cm}^2$$

$$A_f = 0,8 \text{ cm}^2$$

Si adoperano 2 ϕ 10 inf. e 2 ϕ 10 sup.

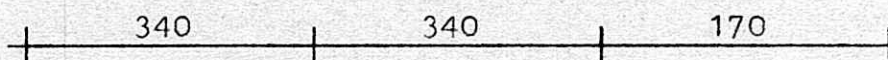
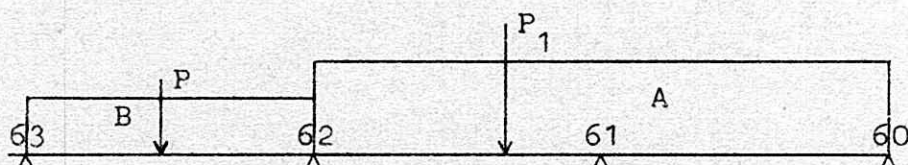
$$T = 2050 \text{ Kg}$$

$$\tau_{max} = 1,8 \text{ Kg/cm}^2 < 6$$

Si arma il taglio con 1 ϕ 8/30 cm

TRAVE $60 \div 63 \equiv 45 \div 48$ (Trav. h B C)

Primo solaio di copertura:



Dalle analisi precedenti si hanno i seguenti calcoli

Carico concentrato $P = 14.500 \text{ Kg}$
 c.v.d. B $= 2.400 \text{ Kg/ml}$
 c.v.d. A $= 3.400 \text{ Kg}$

Carico concentrato P_1

come carico P $= 14.500 \text{ Kg}$

balconi $1000 \times 1,7 \times 2 = 3.400 \text{ "}$

$P_1 = 17.900 \text{ Kg}$

$$M_{63-62} = \frac{1}{12} 2400 \times 3,4^2 + \frac{1}{8} 14.500 \times 3,4 = 8450 \text{ Kgm}$$

$$M_{62-61} = \frac{1}{12} 3400 \times 3,4^2 + \frac{1}{8} 14.500 \times 3,4 =$$

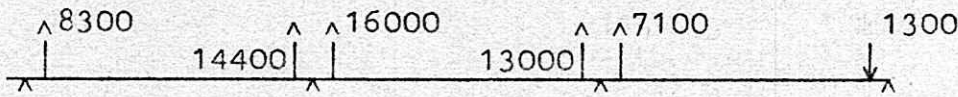
$$M_{61-60} = \frac{1}{12} = 3400 \times 1,7^2 =$$

0.80	0.5 0.5	0.33 0.67	0.80
-8450	+8450-10850	+10850-820	+820

Azioni flettenti

1660	12260	7540	820
10.100	12.000	700	382

Azzioni taglianti



Armature per:

$$H = 60 \text{ cm}$$

$$h = 57 \text{ cm}$$

$$b = 22 \text{ cm}$$

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

$$\sigma_{c \text{ max}} = 65 \text{ Kg/cm}^2$$

$$n = 10$$

σ_c	65	61	67	67	62		Kg/cm ²
Af	11,4	17,8	16,3	16,0	10,3		cm ²
A' f	5,7	17,8	16,3	16,0	5,15		cm ²
x	18,1	17,2	18,3	18,1	16,6		cm

$$T = 16.000 \text{ Kg}$$

$$\tau_{\text{max}} = \frac{16.000}{22 \times (57 - \frac{16,3}{3})} = 14,1 \text{ Kg/cm}^2 < 16$$

$$S_t = 14,1 \times \frac{340}{4} \times 22 = 21.400 \text{ Kg}$$

Adoperando Staffe ϕ 6/10 cm a 2 braccia

$$S_s = 0,57 \times 18 \times 1400 = 11.500 \text{ Kg}$$

$$A_{f_p} = \frac{21.400 - 11.500}{\sqrt{2} \times 1.400} = 4,91 \text{ cm}^2$$

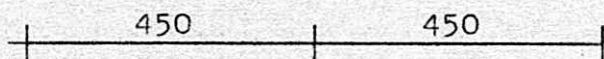
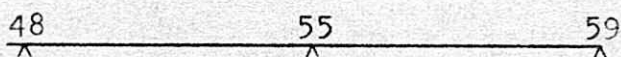
$$\tau_s = \frac{0,57 \times 1400}{10 \times 22} = 3,60 \text{ Kg/cm}^2$$

$$\tau_p = \frac{254 \times 1400 \times \sqrt{2}}{22 \times 30} = 7,6 \text{ Kg/cm}^2$$

$$\tau_R = 14,1 - (3,60 + 7,60) = 2,9 \text{ Kg/cm}^2$$

=====

TRAVE 48 ÷ 59; 63 ÷ 74; 1 ÷ 3 (Tor. 4 Bc)



Analisi dei carichi:

$$\begin{aligned} \text{p.p. trave } 0.15 \times 0.60 \times 2500 &= 230 \text{ Kg/ml} \\ \text{muratura} &= 600 \text{ "} \\ &= 830 \text{ Kg/ml} \end{aligned}$$

$$M_{55} = -\frac{1}{8} 830 \times 4,5^2 = 2000 \text{ Kgm}$$

$$M_{\text{max}} = \frac{1}{14,4} 830 \times 4,5^2 = 1150 \text{ Kgm}$$

Armature per:

$$H = 60 \text{ cm} \quad h = 57 \text{ cm} \quad n = 10$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c \text{ max} = 65 \text{ Kg/cm}^2$$

σ_c	29	41	29	Kg/cm ²
Af	1,6	4,7	1,6	cm

$$T = 1900 \text{ Kg} \quad \tau_{\text{max}} = 2,4 \text{ Kg/cm}^2$$

Staffe ϕ 6/25 cm

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N.B.: Le strutture mancanti sono analoghe a quelle del tipo in linea. corrispondenti:

- 71 -
SVILUPPO DEI CROSS.

TRAVE 51-57/M-5 (fig. 39)

080	067	033	05	05	05	05	05	05	08
+ 600	- 600	+ 8500	- 8500	+ 8500	- 8500	+ 8950	- 8950	+ 8500	- 8500
- 2650	← - 5300	+ 49	← + 97	- 194	← - 3875	+ 325	← + 650	- 850	← - 1700
+ 1640	→ + 820	- 2649	→ - 1324	+ 97	→ + 49	- 3875	→ - 194	+ 650	→ + 325
- 387	← - 774	+ 340	← + 6795	- 35	← - 695	+ 90	← + 180	- 166	← - 3325
+ 310	→ + 155	- 386	→ - 193	+ 6795	→ + 340	- 695	→ - 35	+ 180	→ + 90
- 113	+ 71	← + 1415	- 90	← - 1795	+ 19	← + 37	- 39	← - 78	+ 66
- 113		+ 1415		- 1795		+ 37		- 78	

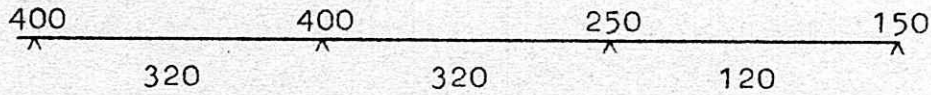
Sollecitazioni flettenti:

820	5800	9100	8800	8300	10200	8500
700	10600	10600	11400	10600	10600	

TRAVE 5 - 37/41 - 50/24 - 57/12 - 20 (fig. 41)

080	05	05	039	061	080
+ 1550	- 1550	+ 1550	- 1550	+ 600	- 600
- 1240	→ - 620	+ 310	→ + 155	+ 480	→ + 240
+ 155	← + 310	+ 158	← + 315	+ 145	← + 290
- 124	→ - 62	- 48	→ - 24		
- 48		- 47	- 74	→	

Sollecitazioni flettenti:

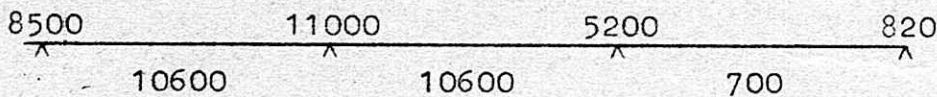


TRAVE 1 - 4/61 - 58

(fig. 45)

080		05	050		033	067		08
^		^		^		^		^
+8500	-8500	+8500	-8500	+ 600	- 600			
<u>-6800</u> →	-3400	<u>+1700</u> →	+ 850	<u>+4700</u> →	+2350			
+ 850←	<u>+1700</u>	+1175←	<u>+2350</u>	- 700←	<u>-1400</u>			
<u>- 680</u> →	- 340	<u>-4175</u> →	+ 209	<u>+ 610</u> →	+ 305			
- 209←	<u>-4175</u>	+ 150←	<u>+ 299</u>					
<u>+ 170</u> →	+ 85	<u>-1175</u>						
	<u>-1175</u>							

Sollecitazioni flettenti:



TRAVE 41 ÷ 26

(Ref-65)

08	05 05	05 05	05 05	05 05	08
- 820	+ 820- 820	+ 820- 820	+ 820- 580	+ 580- 580	+ 580
+ 660	+ 330				
- 82	- 165- 165	- 82			
+ 65	+ 33+ 20	+ 41+ 41	+ 20		
	- 26- 26	- 13- 65	- 130- 130	- 65	
		+ 39+ 39	+ 20+ 16	+ 32+ 32	+ 16
			- 18- 18	- 9- 235	- 470
				+ 122+ 122	+ 61
					- 48

Azioni flettenti:

820		990		800		700		660		580
177	680	680	680	680	480	480	139			

TRAVE 60 ÷ 63 ≡ 45 ÷ 48 (pag. 68)

080	05 05	033 067	080
-8450	+8450 -10850	+10850- 820	+ 820
+6750	+3375		
- 244	- 488 - 488	- 244	
+ 196	+ 98 - 1615	- 3230-6556	-3278
+ 428	+ 856 + 856	+ 428+ 980	+1960
- 340	- 170 - 118	- 236-1172	- 586
	+ 144 + 144	+ 72+ 235	+ 470
		- 102- 205	

Azioni flettenti:

1660	12260	7540	820
10100	12000	700	382

PRESCRIZIONE MATERIALI
STRUTTURE IN ELEVAZIONE

Calcestruzzo di cemento tipo 730 avente una resistenza cubica *compres.*
dopo 28 gg. non inferiore a 225 Kg/cm².

Carichi di lavoro: $\sigma_c \leq 65 \text{ Kg/cm}^2$

$\tau \leq 6 \text{ Kg/cm}^2$

Acciaio Aq 42 ÷ 50 avente uno snervamento superiore a 24 Kg/mm²
allungamento a rottura superiore al 20 %.

Carico di lavoro $\sigma_f \leq 1400 \text{ Kg/cm}^2$

IL CALCOLATORE
(Dott. Ing. Leonardo POTT)

Brindisi, Giugno 1967.