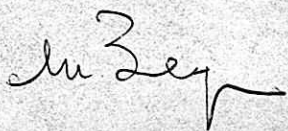


APPROVATO

DAL CONSIGLIO DI AMMINISTRAZIONE
INTEGRATO GISCAL NELLA SEDUTA
DEL 24 GIU. 1971 CON PROVVEDI-
MENTO N. _____

Brindisi, _____

PRESIDENTE

GESTIONE CASE LAVORATORI LEGGE 14/2/63 N° 60		INTERVENTO N 1211 3° TRIENNIO		I.A.C.P.	
QUARTIERE "S. ELIA" BRINDISI Progetto delle strutture				data 21-5-71	N°
				agg	
RELAZIONE DI CALCOLO SECONDA PARTE				rapp	
Dott Arch	E. NESPEGA	Edilizia			
Dott Ing	A. CORRADO	Edilizia			
Dott Ing	F. CARADORINI	Prog.e Costi			
Dott Ing	M. BOLOGNESE	Strutt.e Calc.			
Dott Ing	N. VIVARELLI	Imp.Tecnici			

3-0-0 EDIFICIO TIPO B2

3-1-0 - Analisi di carico sulle travi -

3-1-1 COPERTURA

Trave 12-14-15-16-17-19

Solaio $650 \times 2,70$ = 1750" " $650 \times 2,70$ = 1750p.p. $0,30 \times 0,80 \times 1,00 \times 2500$ = 600

4100 Kg/ml

Trave 10-11

Solaio $650 \times 140/2$ = 450sovracc. $365 \times 0,50$ = 200p.p. $0,50 \times 0,25 \times 1,00 \times 2500$ = 300

950 Kg/ml

Trave 10-4; 11-5

Solaio $650 \times 1,30/2$ = 400sbalzo $650 \times 0,70$ = 450sovracc. $365 \times 0,50$ = 200p.p. $0,50 \times 0,25 \times 1,00 \times 2500$ = 300

1350 Kg/ml3-1-2 COPERTURA PIANO TIPO

Trave 1-2-3-4

Solaio $650 \times 3,90/2$ = 1250sbalzo $650 \times 1,00$ = 650

tompagno = 850

p.p. $1,00 \times 0,25 \times 1,00 \times 2500$ = 635sovracc. $365 \times 1,00$ = 365

3750 Kg/ml

Trave 5-6

Solaio $6,50 \times 3,40/2$

= 1100

sovracc. $365 \times 1,20$

= 450

tompagno

= 850

p.p. $1,20 \times 1,00 \times 0,25 \times 2500$

= 750

3150 Kg/ml

Trave 6-7-8

Solaio $650 \times 3,90/2$

= 1250

sovracc. $365 \times 0,70$

= 250

tompagno

= 850

p.p. $0,70 \times 0,25 \times 1,00 \times 2500$

= 450

2800 Kg/ml

Trave 12-13-14-15; 16-17-18-19-20

Solaio $650 \times 3,65/2$

= 1200

" " $650 \times 3,90/2$

= 1250

sovracc. $365 \times 0,90$

= 350

p.p. $0,90 \times 1,00 \times 0,25 \times 2500$

= 550

3350 Kg/ml

Trave 21-22-23; 26-27-28-29

solaio $650 \times 3,65/2$

= 1200

sovracc. $365 \times 0,70$

= 250

tompagno

= 850

p.p.

= 450

2750 Kg/ml

Trave 23-24; 25-26

Solaio $650 \times 3,65/2$

= 1200

sovracc. $365 \times 0,70$

= 250

sbalzo $800 \times 1,15$

= 900

tompagno

= 850

p.p.

= 450

3650 Kg/ml

Trave 24-25; 10-9-4; 11-9-5

Tompagno

p.p. $0,30 \times 0,40 \times 1,00 \times 2500$

= 850

= 350

1200 Kg/ml

Trave 24-15-10; 25-16-11

Rampa 850 $\times$ 1,30

tompagno

p.p. $0,30 \times 0,60 \times 1,00 \times 2500$

= 1100

= 850

= 450

2400 Kg/ml

Trave 1-12-21; 8-20-29

tompagno

sovracco. 365 $\times$ 0,50p.p. $0,50 \times 0,25 \times 1,00 \times 2500$

= 850

= 200

= 350

1400 Kg/ml

3-2-0 - Analisi di carico sui pilastri -

3-2-1 COPERTURA

Pilastri 4-5

Trave 4-10 $1350 \times 3,10/2$ = 2160p.p. $0,30 \times 0,30 \times 2,70 \times 2500$ = 600

2700 Kg

Pilastri 10-11

trave 10-4 = 2100

" " 10-11 $950 \times 2,40/2$ = 1150p.p. $0,85 \times 0,20 \times 2,70 \times 2500$ = 1150

4400 Kg

Pilastro 12

Trave 12-14 $4100 \times 6,90/2$ = 14150

p.p. = 600

14750 Kg

Pilastro 14

Trave 14-12 = 14150

" " 14-15 $4100 \times 2,00/2$ = 4100

p.p. = 600

18850 Kg

Pilastri 15-16

trave 15-14 = 4100

" " 15-16 $4100 \times 3,00/2$ = 6150

p.p. = 600

10850 Kg

Pilastro 17

trave 17-16 4100 x 2,00/2

= 4100

" " 17-19 4100 x 6,70/2

= 13700

p.p.

= 600

18400 Kg

Pilastro 19

trave 19-17

= 13700

p.p.

= 600

14300 Kg

3-2-2 Copertura piano tipo

Pilastro 1

trave 1-2 3750 x 2,65/2

= 4950

" " 1-12 1400 x 4,60/2

= 3200

p.p. 0,30 x 0,40 x 3,05 x 2500

= 900

9050 Kg

Pilastro 2

trave 2-1

= 4950

" " 2-3 3750 x 3,95/2

= 7400

p.p.

= 950

13300 Kg

Pilastro 3

trave 3-2

= 7400

" " 3-4 3750 x 2,70/2

= 5000

p.p.

= 900

13300 Kg

Pilastro 4

Trave 4-3

= 5050

" " 4-9 1200 x 1,50/2

= 900

p.p.

= 900

6850 Kg

Pilastro 5

trave 5-6 3150 x 5,55/2

= 8750

" " 5-9 1200 x 1,50/2

= 900

p.p.

= 900

10550 Kg

Pilastro 6

trave 6-5

= 8750

" " 6-7 2800 x 3,50/2

= 4900

p.p.

= 900

14450 Kg

Pilastro 7

trave 7-6

= 4900

" " 7-8 2800 x 3,55/2

= 4950

p.p.

= 900

10750 Kg

Pilastro 8

trave 8-7

= 4950

" " 8-20

= 3200

p.p.

= 900

9050 Kg

Pilastro 9

trave	9-4		=	900
" "	9-5		=	900
" "	9-10	1200 x 1,60/2	=	950
" "	9-11		=	950
p.p.	0,20 x 1,95 x 3,05 x 2500		=	3000

6700 Kg**Pilastri 10-11**

trave	10-9		=	950
" "	10-15	2400 x 1,80/2	=	2150
aliquota solaio	650 x 2,40 x 0,60/2		=	500
p.p.	0,20 x 0,83 x 3,05 x 2500		=	1300

4900 Kg**Pilastro 12**

Trave	12-1		=	3200
" "	12-21	1400 x 4,45/2	=	3100
" "	12-13	3350 x 3,35/2	=	5600
p.p.			=	900

12800 Kg**Pilastro 13**

trave	13-12		=	5600
" "	13-14	3350 x 3,55/2	=	5950
p.p.			=	900

12450 Kg**Pilastro 14**

trave	14-13		=	5950
" "	14-15	3350 x 2,00/2	=	3350
p.p.			=	900

10200 Kg

Pilastri 15-16

trave	15-10		=	2150
" "	15-14		=	3350
" "	15-24	2400 x 4,45/2	=	5350
P.p.			=	<u>900</u>
				11750 Kg

Pilastro 17

trave	17-16	3350 x 2,00/2	=	3350
" "	17-18	3350 x 3,70/2	=	6200
P.p.			=	<u>900</u>
				10450 Kg

Pilastro 18

trave	18-17		=	6200
" "	18-19	3350 x 3,00/2	=	5000
P.p.			=	<u>900</u>
				12100 Kg

Pilastro 19

trave	19-18		=	5000
" "	19-20	3350 x 3,55/2	=	6200
P.p.			=	<u>900</u>
				12100 Kg

Pilastro 20

trave	20-19		=	6200
" "	20-8		=	3200
" "	20-29		=	3100
P.p.			=	<u>900</u>
				13400 Kg

Pilastro 21		
trave 21-12		= 3100
" " 21-22	2750 x 3,35/2	= 4600
p.p.		= 900
		8600 Kg

Pilastro 22		
trave 22-21		= 4600
" " 22-23	2750 x 3,55/2	= 4900
p.p.		= 900
		10400 Kg

Pilastro 23		
trave 23-22		= 4900
" " 23-24	3650 x 2,00/2	= 3650
p.p.		= 900
		9450 Kg

Pilastri 24-25		
trave 24-23		= 3650
" " 24-25	1200 x 3,00/2	= 1800
" " 24-15		= 5350
p.p.		= 900
		11700 Kg

Pilastro 26		
trave 26-25		= 3650
" " 26-27	2750 x 3,70/2	= 5100
p.p.		= 900
		9650 Kg

Pilastro 27

trave 27-26

= 5100

" " 27-28 2750 x 3,00/2

= 4100

p.p.

= 900

10100 Kg

Pilastro 28

trave 28-27

= 4100

" " 28-29 2750 x 3,55/2

= 4900

p.p.

= 900

9900 Kg

Pilastro 29

trave 29-27

= 4900

" " 29-20

= 3100

p.p.

= 900

8900 Kg

3-3-0 - Verifica ed armatura pilastri -

3-3-1

Carichi al piede dei pilastri

Pilastri	Copertura	Cop. IV	Cop. III	Cop. II	Cop. I	Cop. portico fondazione
1		9000	18000	27000	36000	45000
2		13300	26600	39900	53200	66500
3		13300	26600	39900	53200	66500
4	2700	9550	16400	23250	30100	36950
5	2700	13250	23800	34350	44900	55450
6		14550	29100	43650	58200	72750
7		10750	21500	32250	43000	53750
8		9000	18000	27000	36000	45000
9		6700	13400	20100	26800	33500
10-11	4400	9300	14200	19100	24000	28900
12	14750	27550	40350	53150	65950	78750
13		12450	24900	37350	49800	62250
14	18850	29050	39250	49450	59650	69850
15-16	10850	22600	34350	46100	57850	69600
17	18400	28850	39300	49750	60200	70650
18		12100	24200	36300	48400	60500
19	14300	26400	38500	50600	62700	74800

Pilastri	Copertura	Cop. IV	Cop. III	Cop. II	Cop. I	Cop. portico fondazione
20		13400	26800	40200	53600	67000
21		8600	17200	25800	34400	43000
22		10400	20800	31200	41600	52000
23		9450	18900	28350	37800	47250
24-25		11700	23400	35100	46800	58500
26		9650	19300	28950	38600	48250
27		10100	20200	30300	40400	50500
28		10000	20000	30000	40000	50500
29		9000	18000	27000	36000	45000

3-3-2 Sezione ed armatura pilastri fondazione e portico

N°	P tn	A' c cmq	A f cmq	Armatura	Sezione cm x cm	A ci cmq	σ c Kg/cmq
1,8,21,29	45	950	7,6	4Ø14 + 4Ø12	30 x 40	1307	34
2,3,20	67	1400	11,2	4Ø14 + 6Ø12	30 x 50	1629	41
4	37	750	6	4Ø14 + 4Ø12	30 x 40	1307	28
5	55	1150	9,2	4Ø14 + 4Ø12	30 x 40	1307	42
6	73	1500	12	4Ø14 + 6Ø12	30 x 50	1629	44
7,22	54	1100	8,8	4Ø14 + 4Ø12	30 x 40	1307	41
9	33	700	5,6	24 Ø 10	20 x 195	4060	<10
10,11	29	600	4,8	14 Ø 10	20 x 82 ⁵	1759	16
12	79	1600	12,8	4Ø14 + 6Ø12	30 x 50	1629	46
13,	63	1300	10,4	4Ø14 + 6Ø12	30 x 50	1629	38
14,15,16,17	70	1450	11,6	4Ø14 + 6Ø12	30 x 50	1629	42
18	60	1250	10	4Ø14 + 6Ø12	30 x 50	1629	36
19	75	1550	12,4	4Ø14 + 6Ø12	30 x 50	1629	45
23,26	48	1000	8	4Ø14 + 4Ø12	30 x 50	1307	36
24,25	58	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	44
27,28	51	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	39

Staffe Ø 8" / 15"

3-3-3 Sezione ed armatura pilastri piano I e II

N°	P tn	A ^c cmq	A ^f cmq	Armatura	Sezione cm x cm	A ^{ci} cmq	σ ^c Kg/cmq
1,8,21,29	36	750	6	8 Ø 12	30 x 30	990	36
2,3,20	54	1100	8,8	4Ø14 + 4Ø12	30 x 40	1307	41
4	30	600	4,8	8 Ø 12	30 x 30	990	30
5	45	900	7,2	8 Ø 12	30 x 30	990	45
6	58	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	44
7,22	43	900	7,2	8 Ø 12	30 x 30	990	43
9	27	550	4,4	24 Ø 10	20 x 195	4060	<10
10,11	24	500	4	14 Ø 10	20 x 82 ⁵	1759	13
12	66	1350	10,8	4Ø14 + 4Ø12	30 x 40	1307	47
13	50	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	38
14,15,16,17	60	1250	10	4Ø14 + 4Ø12	30 x 40	1307	45
18	48	1000	8	4Ø14 + 4Ø12	30 x 40	1307	36
19	63	1300	10,4	4Ø14 + 4Ø12	30 x 40	1307	46
23,26	39	800	6,4	8 Ø 12	30 x 30	990	39
24,25	47	950	7,6	8 Ø 12	30 x 30	990	46
27,28	40	850	6,8	8 Ø 12	30 x 30	990	40

Staffe Ø 6" /15"

3-3-4 Sezione ed armatura pilastri piano III e IV

N°	P	A' c	A f	Armatura	Sezione
	tn	cmq	cmq		cm x cm
1,8,21,29	18	350	2,8	8 Ø 10	30 x 30
2,3,20	27	550	4,4	4Ø12 + 4Ø10	30 x 30
4	16	300	2,4	8 Ø 10	30 x 30
5	24	500	4,0	8 Ø 10	30 x 30
6	29	600	4,8	4Ø12+4Ø10	30 x 30
7,22	21	450	3,6	8 Ø 10	30 x 30
9	13	250	2	24 Ø 8	20 x 195
10,11	14	300	2,4	14 Ø 8	20 x 82 ⁵
12	40	800	6,4	4Ø12 + 4Ø10	30 x 30
13	25	500	4,0	4Ø12 + 4Ø10	30 x 30
14,15,16,17	39	800	6,4	4Ø12 + 4Ø10	30 x 30
18	24	500	4	4Ø12 + 4Ø10	30 x 30
19	39	800	6,4	4Ø12 + 4Ø10	30 x 30
23,16	19	400	3,2	8 Ø 10	30 x 30
24,25	23	450	3,6	8 Ø 10	30 x 30
27,28	20	400	3,2	8 Ø 10	30 x 30

Staffe Ø 6^m / 15"

3-3-5 Sezione ed armatura pilastri piano Volumi tecnici

N°	P tn	A' c cmq	Armatura	Sezione cm x cm
4	3	100	8 Ø 10	30 x 30
5	3	100	8 Ø 10	30 x 30
10,11	5	100	14 Ø 8	20 x 82 ⁵
12	15	300	8 Ø 10	30 x 30
14,15,16,17	19	400	8 Ø 10	30 x 30
19	14	300	8 Ø 10	30 x 30

Staffe Ø 6" / 15"

3-4-0 Fondazioni

3-4-1 -Analisi di carico sul terreno-

pilastri 1595000

trave 222000

1817000 Kg

3-4-2 -Tensione sul terreno-

$$\sigma_t = \frac{1817000}{1429600} = 1,27 \text{ Kg/cm}^2$$

1429600

3-4-3 Travi 12-13-14-15-16-17-18-19-20; 21-22-23-24-25-26-27-28-29

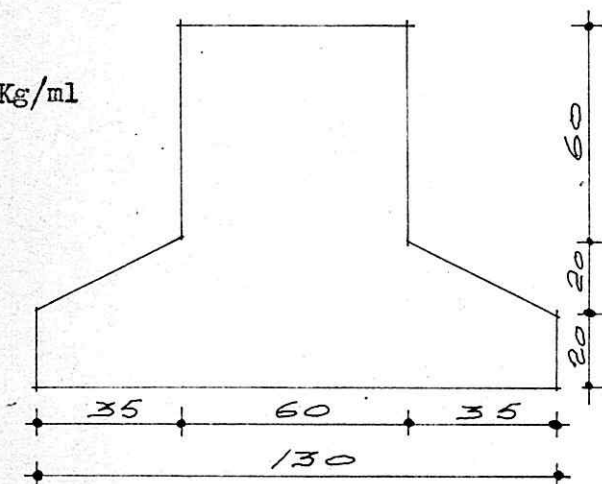
Carico sulla trave

$$q = \frac{1595000}{100 \times 130} = 14500 \text{ Kg/ml}$$

1429600

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 14500 \times 3,70^2 = 16500 \text{ Kgm} \quad H = 100'' \quad b = 60''$$

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

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

$$A_f = 13 \text{ cm}^2$$

$$M_{camp} = 16500 \text{ Kgm}$$

$$H = 100''$$

$$b = 130''$$

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

$$\sigma_c = 21 \text{ Kg/cm}^2$$

$$A_f = 12,5 \text{ cm}^2$$

-Armatura trasversale-

$$M = \frac{1}{2} 14500 \times 0,35^2 = 900 \text{ Kgm} \quad H = 40'' \quad b = 100''$$

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

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

$$A_f = 1,5 \text{ cm}^2$$

3-4-4 Travi 1-2-3-4-5-6-7-8-9-10

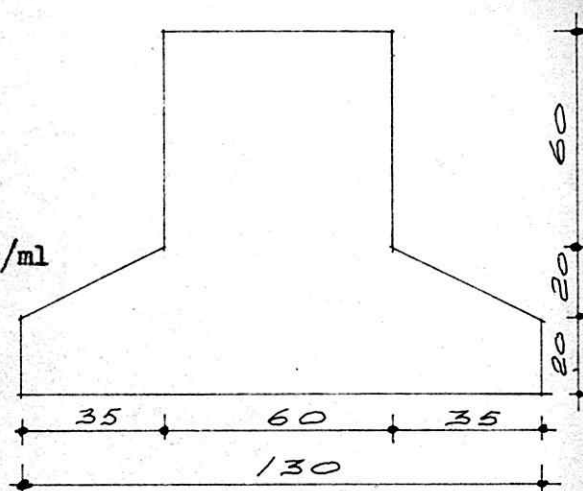
Carico sulla trave

$$q = \frac{1595000}{100} \times 100 \times 130 = 14500 \text{ Kg/ml}$$

1429600

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 14500 \times 5,55^2 = 37200 \text{ Kgm} \quad H = 100'' \quad b = 60''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 52 \text{ Kg/cm}^2 \quad A_f = 29,5 \text{ cm}^2$$

$$M_{camp} = 37200 \text{ Kgm} \quad H = 100'' \quad b = 130''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 33 \text{ Kg/cm}^2 \quad A_f = 29 \text{ cm}^2$$

3-4-5 Travi 1-12-21, 8-20-29

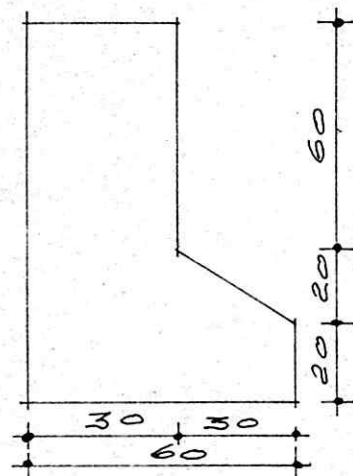
Carico sulla trave

$$q = \frac{1595000}{100} \times 60 \times 100 = 6700 \text{ Kg/ml}$$

1429600

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 6700 \times 4,50^2 = 13400 \text{ Kgm} \quad H = 100'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 43 \text{ Kg/cm}^2 \quad A_f = 10,6 \text{ cm}^2$$

$$M_{camp} = 13400 \text{ Kgm} \quad H = 100'' \quad b = 60''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 29 \text{ Kg/cm}^2 \quad A_f = 10,4 \text{ cm}^2$$

3-4-6 Travi 2-13-22; 3-14-23; 4-9-10-15-24; 5-9-11-16-25; 6-18-27;
7-19-28-26-27

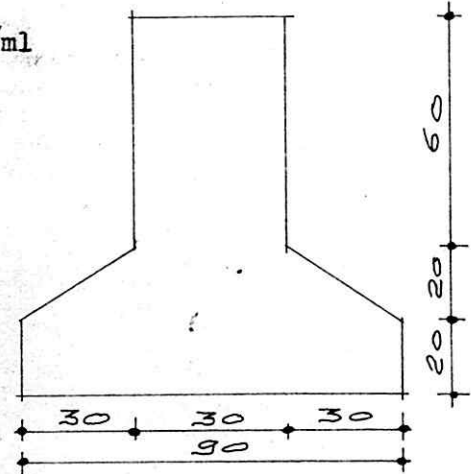
Carico sulla trave

$$q = 1595000 \times 100 \times 90 = 10000 \text{ Kg/ml}$$

$$1429600$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 10000 \times 4,90^2 = 20000 \text{ Kgm} \quad H = 100'' \quad b = 30''$$

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

$$\sigma_c = 54 \text{ Kg/cm}^2$$

$$A_f = 16 \text{ cm}^2$$

$$M_{camp} = 20000 \text{ Kgm}$$

$$H = 100''$$

$$b = 90''$$

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

$$\sigma_c = 29 \text{ Kg/cm}^2$$

$$A_f = 15,5 \text{ cm}^2$$

3-5-0 - Vano Scala -

3-5-1 Calcolo del gradino

Analisi di carico

$$p.p. \quad 0,235 + 0,07 \times 0,30 \times 1,00 \times 2500 = 115$$

sovracc. perm. (²intonaco, grado

$$\text{sottograde e massetto }) = 150 \times 0,30 = 45$$

$$\text{sovracc. accid.} = 400 \times 0,30 = 120$$

280 Kg/ml

$$M = \frac{1}{2} 280 \times 1,20^2 = 200 \text{ Kgm} \quad H = \frac{0,235 \times 0,07}{2} = 0,15 \quad b = 30 \text{ cm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 36 \text{ Kg/cm}^2 \quad A_f = 1,10 \text{ cm}^2$$

3-5-2 Calcolo dei pianerottoliSolaio in letero-cemento $H = 16'' + 4'' \quad i = 50''$

Analisi di carico

$$p.p. = 140$$

$$\text{soletta} \quad 0,04 \times 1,00 \times 1,00 \times 2500 = 100$$

$$\text{sovracc. accid. (intonaco, massetto, pavimento)} = 110$$

$$\text{sovracc. perm.} = 400$$

750 Kg/mq

$$M_{inc} = \frac{1}{12} 750 \times 2,70^2 = 455 \text{ Kgm} \quad H = 20'' \quad b = 100''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 22 \text{ Kg/cm}^2 \quad A_f = 0,96 \text{ cm}^2/50''$$

$$M_{camp} = \frac{1}{14} 750 \times 2,70^2 = 390 \text{ Kgm} \quad H = 20'' \quad b = 100''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 20 \text{ Kg/cm}^2 \quad A_f = 0,82 \text{ cm}^2 / 50''$$

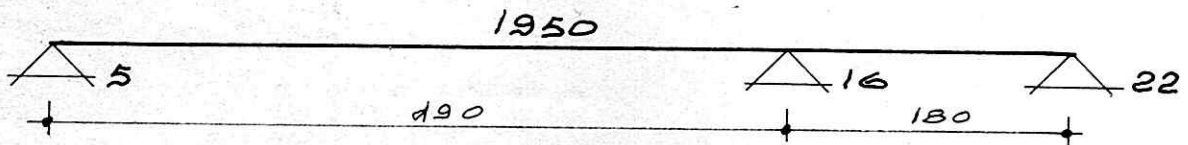
3-5-3 Calcolo delle travi a ginocchio

Analisi di carico

Rampa	850 x 1,20	= 1100
tompagno		= 400
p.p.	0,30 x 0,60 x 1,00 x 2500	= 450
		1950 Kg/ml

Verifica ed armatura a flessione

Schema di calcolo



$$M_5 = M_{5-16} = \frac{1}{14} 1950 \times 4,90^2 = 3350 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 35 \text{ Kg/cm}^2 \quad A_f = 4,4 \text{ cm}^2$$

$$M_{16} = \frac{1}{10} 1950 \times 4,90^2 = 4700 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 43 \text{ Kg/cm}^2 \quad A_f = 6,3 \text{ cm}^2$$

$$M_{16-22} = \frac{1}{8} 1950 \times 1,80^2 = 4700 - 3900 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 39 \text{ Kg/cm}^2 \quad A_f = 5,3 \text{ cm}^2$$

$$M_{22} = \frac{1}{12} 1950 \times 1,80^2 = 550 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c < 15 \text{ Kg/cm}^2 \quad A_f = 1,5 \text{ cm}^2$$

Verifica a torsione della trave 5-16

$$m_i = \frac{1}{2} 850 \times 1,20^2 = 612 \text{ Kgm} \quad m_t = 612 + 1020 \times 0,15 = 765 \text{ Kgm}$$

$$t_i = 850 \times 1,20 = 1020 \text{ Kgm} \quad M_t = 765 \times \frac{4,90}{2} = 1875 \text{ Kgm}$$

$$y = 3 + \frac{2,6}{0,45 + \frac{60}{30}} = 4,061$$

$$0,45 + \frac{60}{30}$$

$$\tau = \frac{4,061 \times 187500}{60 \times 30^2} = 14,1 \text{ Kg/cm}^2$$

- Armatura a torsione -

$$F = 55 \times 25 = 1375 \text{ cm}^2$$

$$V = (55 + 25) \times 2 = 160 \text{ cm}$$

$$\Sigma f_l = \frac{187500}{2 \times 1400 \times 1375} = 160 = 7,8 \text{ cm}^2$$

$$\Sigma f_s = \frac{187500}{2 \times 1400 \times 1375} = 0,0487 \text{ cm}^2/\text{cm}$$

3-5-4 Calcolo della trave al pianerottolo di riposo

$$M = \frac{1}{12} 1200 \times 3,00^2 = 900 \text{ Kgm} \quad H = 40'' \quad b = 20''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 34 \text{ Kg/cm}^2 \quad A_f = 1,8 \text{ cm}^2$$

3-6-0 BALCONI

$$q = 800 \text{ Kg/mq} \quad H = 20 + 5 \text{ cm} \quad b = 100 \text{ cm}$$

$$M = \frac{1}{2} 800 \times 1,05^2 = 440 \text{ Kgm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 17 \text{ Kg/cm}^2 \quad A_f = 1,6 \text{ cm}^2/\text{ml}$$

$$i = 50 \text{ cm} \quad A_f = 0,8 \text{ cm}^2$$

$$M = \frac{1}{2} 800 \times 1,15^2 = 530 \text{ Kgm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 18 \text{ Kg/cm}^2 \quad A_f = 1,7 \text{ cm}^2/\text{ml}$$

$$i = 50 \text{ cm} \quad A_f = 0,85 \text{ cm}^2/50$$

3-7-0 SOLAIO PIANO TIPOCarico per nervatura $q = 650 \times 0,50 = 325 \text{ Kg/ml}$ $\sigma_f = 1400 \text{ Kg/cm}^2$ $b = 50 \text{ cm}$ luce netta $3,90 \times 1,05 = 4,10 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 4,10^2 = 455 \text{ Kgm}$	$H=25\text{cm}$	$\sigma_c = 24\text{Kg/cm}^2$	$A_f = 1,47\text{cm}^2$
		$H=18\text{cm}$	$\sigma_c = 32\text{Kg/cm}^2$	$A_f = 1,93\text{cm}^2$

campata	$M = \frac{1}{14} 325 \times 4,10^2 = 390\text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 22\text{Kg/cm}^2$	$A_f = 1,25\text{cm}^2$
		$H=18\text{cm}$	$\sigma_c = 30\text{Kg/cm}^2$	$A_f = 1,68\text{cm}^2$

luce netta $3,63 \times 1,05 = 3,80\text{m.}$

incastro	$M = \frac{1}{12} 325 \times 3,80^2 = 390\text{Kgm}$	$H = 25\text{cm}$	$\sigma_c = 22\text{Kg/cm}^2$	$A_f = 1,25\text{cm}^2$
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campata	$M = \frac{1}{14} 325 \times 3,80^2 = 335\text{Kgm}$	$H = 25\text{cm}$	$\sigma_c = 20\text{Kg/cm}^2$	$A_f = 1,05 \text{ cm}^2$
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luce netta $2,40 \times 1,05 = 250 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 2,50^2 = 170\text{Kgm}$	$H=25\text{cm}$	$\sigma_c < 15\text{Kg/cm}^2$	$A_f = 0,8 \text{ cm}^2$
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campata	$M = \frac{1}{14} 325 \times 2,50^2 = 145\text{Kgm}$	$H= 25\text{cm}$	$\sigma_c < 15\text{Kg/cm}^2$	$A_f = 0,8 \text{ cm}^2$
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luce netta $358 \times 1,05 = 3,75\text{m.}$

incastro	$M = \frac{1}{12} 325 \times 3,75^2 = 380\text{Kgm}$	$H = 25\text{cm}$	$\sigma_c = 22\text{Kg/cm}^2$	$A_f = 1,25\text{cm}^2$
		$H=18\text{cm}$	$\sigma_c = 29\text{Kg/cm}^2$	$A_f = 1,6 \text{ cm}^2$

campata	$M = \frac{1}{14} 325 \times 3,75^2 = 330\text{Kgm}$	$H= 25\text{cm}$	$\sigma_c = 20\text{Kg/cm}^2$	$A_f = 1,05 \text{ cm}^2$
		$H = 18\text{cm}$	$\sigma_c = 27\text{Kg/cm}^2$	$A_f = 1,4 \text{ cm}^2$

luce netta $408 \times 1,05 = 4,30\text{m.}$

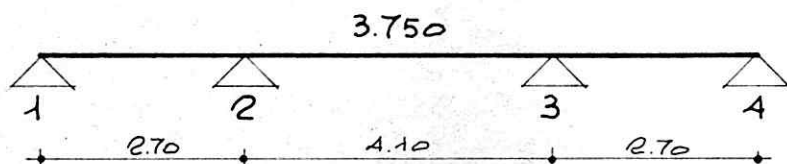
incastro	$M = \frac{1}{12} 325 \times 4,30^2 = 500\text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 26\text{Kg/cm}^2$	$A_f = 1,65 \text{ cm}^2$
		$H=18\text{cm}$	$\sigma_c = 35\text{Kg/cm}^2$	$A_f = 2,2 \text{ cm}^2$

campata	$M = \frac{1}{14} 325 \times 4,30^2 = 430\text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 24\text{Kg/cm}^2$	$A_f = 1,42\text{cm}^2$
		$H=18\text{cm}$	$\sigma_c = 31\text{Kg/cm}^2$	$A_f = 1,8 \text{ cm}^2$

luce netta 100m.

$M = \frac{1}{2} 325 \times 1,00^2 \times 425 = 585\text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 28\text{Kg/cm}^2$	$A_f = 1,92 \text{ cm}^2$
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3-8-1 Travi 1-2-3-4



- Armatura a flessione -

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

$$H = \frac{25}{20} \text{ cm}$$

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

Sezione 1; 1-2; 3-4; 4

$$M = \frac{1}{14} 3750 \times 2,70^2 = 1900 \text{ Kgm}$$

$$\sigma_c = 37 \text{ Kg/cm}^2$$

$$A_f = 6,3 \text{ cm}^2$$

Sezione 2; 3

$$M = \frac{1}{12} 3750 \times 4,10^2 = 5200 \text{ Kgm}$$

$$H = 25 \quad \sigma_c = 67 \text{ Kg/cm}^2$$

$$A_f = 18 \text{ cm}^2$$

$$H = 20 \quad \sigma_c = 93 \text{ Kg/cm}^2$$

$$A_f = 24 \text{ cm}^2$$

Sezione 2-3

$$M = \frac{1}{16} 3750 \times 4,10^2 = 3900 \text{ Kgm}$$

$$H = 25$$

$$\sigma_c = 57 \text{ Kg/cm}^2$$

$$A_f = 13,5 \text{ cm}^2$$

$$H = 20$$

$$\sigma_c = 72 \text{ " "}$$

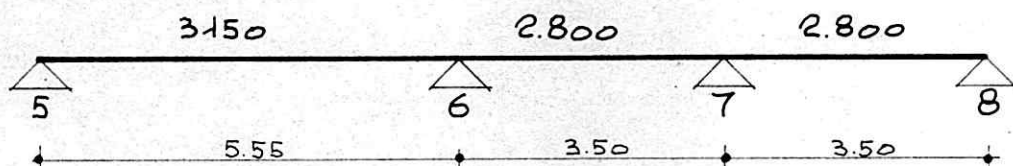
$$A_f = 16,6 \text{ cm}^2$$

$$\text{Taglio massimo } T = 7700 \text{ Kg}$$

$$\tau_{\max} = \frac{7700}{23 \times 0,9 \times 100} = 3,7 \text{ Kg/cm}^2$$

$$23 \times 0,9 \times 100$$

3-8-2 Travi 5-6-7-8



- Armatura a flessione -

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

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

$$b = 120/70 \text{ cm}$$

Sezione 5; 5-6

$$M = \frac{1}{14} 3150 \times 5,55^2 = 6950 \text{ Kgm}$$

$$b = 120 \text{ cm} \quad \sigma_c = 73 \text{ Kg/cm}^2 \quad A_f = 20,5 \text{ cm}^2$$

Sezione 6

$$M = \frac{1}{10} 3150 \times 5,55^2 = 9700 \text{ Kgm} \quad b = 120 \text{ cm} \quad \sigma_c = 90 \text{ Kg/cm}^2 \quad A_f = 29 \text{ cm}^2$$

Sezione 6-7

$$M = \frac{1}{16} 2800 \times 3,50^2 = 2200 \text{ Kgm} \quad b = 70 \text{ cm} \quad \sigma_c = 50 \text{ Kg/cm}^2 \quad A_f = 8,0 \text{ cm}^2$$

Sezione 7

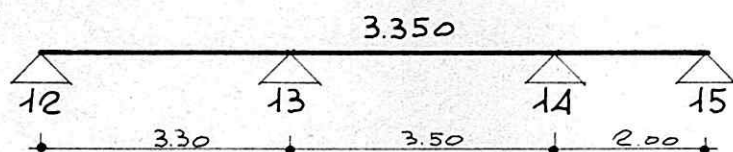
$$M = \frac{1}{10} 2800 \times 3,50^2 = 3450 \text{ Kgm} \quad \sigma_c = 69 \text{ Kg/cm}^2 \quad A_f = 12,5 \text{ cm}^2$$

Sezione 7-8, 8

$$M = \frac{1}{14} 2800 \times 3,50^2 = 2450 \text{ Kgm} \quad \sigma_c = 53 \text{ Kg/cm}^2 \quad A_f = 8,4 \text{ cm}^2$$

Taglio massimo T 8750 Kg $\tau_{\max} = \frac{8750}{23 \times 0,9 \times 120} = 3,5 \text{ cm}^2$

3-8-3 Travi 12-13-14-15



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 90 \text{ cm}$$

Sezione 12, 12-13

$$M = \frac{1}{14} 3350 \times 3,30^2 = 2600 \text{ Kgm} \quad \sigma_c = 47 \text{ Kg/cm}^2 \quad A_f = 8,8 \text{ cm}^2$$

Sezione 13, 14

$$M = \frac{1}{12} 3350 \times 3,50^2 = 3400 \text{ Kgm} \quad \sigma_c = 55 \text{ Kg/cm}^2 \quad A_f = 11,5 \text{ cm}^2$$

Sezione 13-14

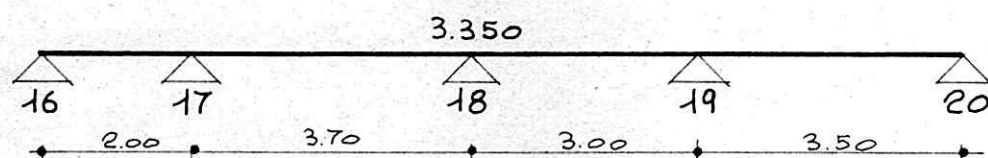
$$M = \frac{1}{16} 3350 \times 3,50^2 = 2550 \text{ Kgm} \quad \sigma_c = 47 \text{ Kg/cm}^2 \quad A_f = 8,8 \text{ cm}^2$$

Sezione 14-15; 15

$$M = \frac{1}{14} 3350 \times 2,00^2 = 950 \text{ Kgm} \quad \sigma_c = 27 \text{ Kg/cm}^2 \quad A_f = 3,2 \text{ cm}^2$$

$$\text{Taglio massimo } T = 5850 \text{ Kg} \quad \tau_{\max} = \frac{5850}{23 \times 0,9 \times 90} = 3,1 \text{ Kg/cm}^2$$

3-8-4 Travi 16-17-18-19-20



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 90 \text{ cm}$$

Sezione 15, 16-17

$$M = \frac{1}{14} 3350 \times 2,70^2 = 1750 \text{ Kgm} \quad \sigma_c = 38 \text{ Kg/cm}^2 \quad A_f = 6 \text{ cm}^2$$

Sezione 17, 18

$$M = \frac{1}{12} 3350 \times 3,70^2 = 3800 \text{ Kgm} \quad \sigma_c = 59 \text{ Kg/cm}^2 \quad A_f = 13 \text{ cm}^2$$

Sezione 17-18

$$M = \frac{1}{16} 3350 \times 3,70^2 = 2850 \text{ Kgm} \quad \sigma_c = 50 \text{ Kg/cm}^2 \quad A_f = 9,7 \text{ cm}^2$$

Sezione 18, 19

$$M = \frac{1}{16} 3350 \times 3,00^2 = 1900 \text{ Kgm} \quad \sigma_c = 39 \text{ Kg/cm}^2 \quad A_f = 6,5 \text{ cm}^2$$

Sezione 19

$$M = \frac{1}{10} 3350 \times 3,50^2 = 4100 \text{ Kgm} \quad \sigma_c = 62 \text{ Kg/cm}^2 \quad A_f = 14 \text{ cm}^2$$

Sezione 19-20; 20

$$M = \frac{1}{14} 3350 \times 3,50^2 = 2950 \text{ Kgm}$$

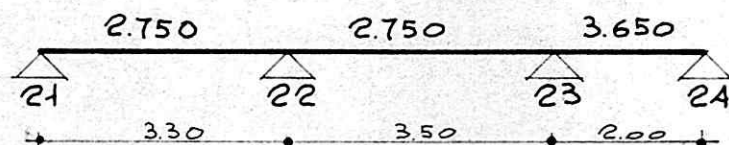
$$\sigma_c = 51 \text{ Kg/cm}^2$$

$$A_f = 10,1 \text{ cm}^2$$

$$\text{Taglio massimo } T = 6200 \text{ Kg}$$

$$\tau_{\max} = \frac{6200}{23 \times 0,9 \times 90} = 3,3 \text{ cm}^2$$

3-8-5 Travi 21-22-23-24



- Armatura a flessione -

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

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

$$b = 70 \text{ cm}$$

Sezione 21; 21-22

$$M = \frac{1}{14} 2750 \times 3,30^2 = 2150 \text{ Kgm}$$

$$\sigma_c = 49 \text{ Kg/cm}^2$$

$$A_f = 7,7$$

Sezione 22; 23

$$M = \frac{1}{12} 2750 \times 3,50^2 = 2800 \text{ Kgm}$$

$$\sigma_c = 58 \text{ Kg/cm}^2$$

$$A_f = 9,7 \text{ cm}^2$$

Sezione 22-23

$$M = \frac{1}{16} 2750 \times 3,50^2 = 2100 \text{ Kgm}$$

$$\sigma_c = 49 \text{ Kg/cm}^2$$

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

Sezione 23-24; 24

$$M = \frac{1}{14} 2750 \times 3,50^2 = 2400 \text{ Kgm}$$

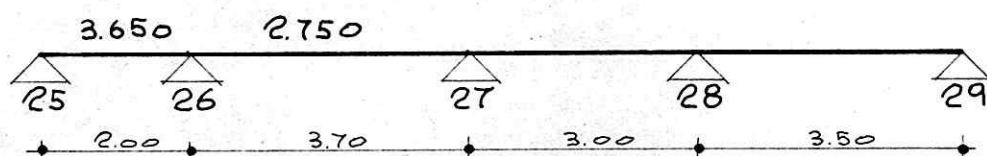
$$\sigma_c = 52 \text{ Kg/cm}^2$$

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

$$\text{Taglio massimo } T = 4800 \text{ Kg}$$

$$\tau_{\max} = \frac{4800}{23 \times 0,9 \times 70} = 3,3 \text{ Kg/cm}^2$$

3-8-6 Travi 25-26-27-28-29



- Armatura a flessione -

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

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

$$b = 70 \text{ cm}$$

Sezione 25; 25-26

$$M = \frac{1}{14} 3650 \times 2,00^2 = 1050 \text{ Kgm}$$

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

$$A_f = 4,9 \text{ cm}^2$$

Sezione 26; 27

$$M = \frac{1}{12} 2750 \times 3,70^2 = 3150 \text{ kgm}$$

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

$$A_f = 10,9 \text{ cm}^2$$

Sezione 26-27

$$M = \frac{1}{15} 2750 \times 3,70^2 = 2350 \text{ Kgm}$$

$$\sigma_c = 52 \text{ Kg/cm}^2$$

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

Sezione 27-28

$$M = \frac{1}{16} 2750 \times 3,00^2 = 1550 \text{ Kgm}$$

$$\sigma_c = 40 \text{ Kg/cm}^2$$

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

Sezione 28

$$M = \frac{1}{10} 2750 \times 3,50^2 = 3350 \text{ Kgm}$$

$$\sigma_c = 64 \text{ Kg/cm}^2$$

$$A_f = 11,6 \text{ cm}^2$$

Sezione 28-29; 29

$$M = \frac{1}{14} 2750 \times 3,50^2 = 2400 \text{ Kgm}$$

$$\sigma_c = 53 \text{ Kg/cm}^2$$

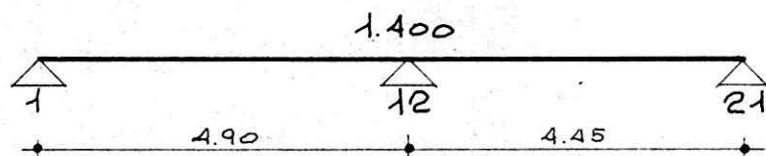
$$A_f = 8,3 \text{ cm}^2$$

$$\text{Taglio massimo } T = 5100 \text{ Kg}$$

$$\tau_{\max} = \frac{5100}{23 \times 0,9 \times 70} = 3,5 \text{ Kg/cm}^2$$

$$23 \times 0,9 \times 70$$

3-8-7 Travi 1-12-21; 8-20-23



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 50 \text{ cm}$$

Sezione 7; 1-12

$$M = \frac{1}{14} 1400 \times 4,90^2 = 2400 \text{ Kgm} \quad \sigma_c = 64 \text{ Kg/cm}^2 \quad A_f = 8,4 \text{ cm}^2$$

Sezione 12

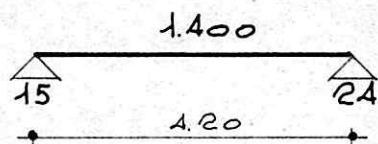
$$M = \frac{1}{10} 1400 \times 4,90^2 = 3350 \text{ Kgm} \quad \sigma_c = 79 \text{ Kg/cm}^2 \quad A_f = 11,8 \text{ cm}^2$$

Sezione 12-21; 21

$$M = \frac{1}{14} 1400 \times 4,43^2 = 2000 \text{ Kgm} \quad \sigma_c = 57 \text{ Kg/cm}^2 \quad A_f = 6,8 \text{ cm}^2$$

$$\text{Taglio massimo } T = 3450 \text{ kg} \quad \tau_{\max} = \frac{3450}{23 \times 0,9 \times 50} = 3,3 \text{ Kg/cm}^2$$

3-8-8 Travi 15-24; 16-25



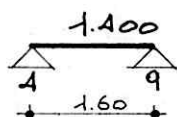
- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 30 \text{ cm}$$

Sezione 15; 15-24; 24

$$M = \frac{1}{12} 1400 \times 4,20^2 = 2050 \text{ Kgm} \quad \sigma_c = 80 \text{ Kg/cm}^2 \quad A_f = 7,2 \text{ cm}^2$$

3-8-9 Travi 4-9; 9-10; 5-9; 9-11

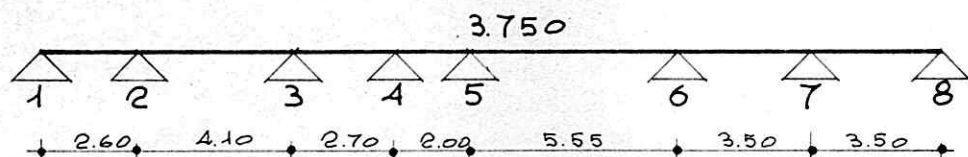


$$H = 24 \text{ cm} \quad b = 20$$

Sezione 4; 4-9; 9

$$M = \frac{1}{12} 1400 \times 1,60^2 = 300 \text{ Kgm} \quad \sigma_c = 32 \text{ Kg/cm}^2 \quad A_f = 1,5 \text{ cm}^2$$

3-8-10 Travi 1-2-3-4-5-6-7-8 Alla copertura IV piano in variante rispetto al piano tipo.



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = \begin{matrix} 100 \\ 120 \end{matrix} \text{ cm}$$

Sezione 1; 1-2

$$M = \frac{1}{14} 3750 \times 2,70^2 = 1900 \text{ Kgm} \quad \sigma_c = 37 \text{ Kg/cm}^2 \quad A_f = 6,3 \text{ cm}^2$$

Sezione 2; 3

$$M = \frac{1}{12} 3750 \times 4,10^2 = 5200 \text{ Kgm} \quad \sigma_c = 67 \text{ Kg/cm}^2 \quad A_f = 18 \text{ cm}^2$$

Sezione 2-3

$$M = \frac{1}{16} 3750 \times 4,10^2 = 3900 \text{ Kgm} \quad \sigma_c = 57 \text{ Kg/cm}^2 \quad A_f = 13,5 \text{ cm}^2$$

Sezione 3 - 4; 4-5

$$M = \frac{1}{18} 3750 \times 2,70^2 = 1500 \text{ Kgm} \quad \sigma_c = 32 \text{ Kg/cm}^2 \quad A_f = 5 \text{ cm}^2$$

Sezione 4

$$M = \frac{1}{12} 3750 \times 2,70^2 = 2250 \text{ Kgm} \quad \sigma_c = 41 \text{ Kg/cm}^2 \quad A_f = 7,6 \text{ cm}^2$$

Sezione 5; 6

$$M = \frac{1}{12} 3750 \times 5,55^2 = 9600 \text{ Kgm} \quad b = 120 \text{ cm} \quad \sigma_c = 89 \text{ Kg/cm}^2 \quad A_f = 34,1 \text{ cm}^2$$

Sezione 5-6

$$M = \frac{1}{18} 3750 \times 5,55^2 = 6400 \text{ Kgm} \quad b = 120 \text{ cm} \quad \sigma_c = 69 \text{ Kg/cm}^2 \quad A_f = 22,3 \text{ cm}^2$$

Sezione 6-7

$$M = \frac{1}{16} 3750 \times 3,50^2 = 2850 \text{ Kgm} \quad \sigma_c = 47 \text{ Kg/cm}^2 \quad A_f = 9,9 \text{ cm}^2$$

Sezione 7

$$M = \frac{1}{10} 3750 \times 3,50^2 = 4600 \text{ Kgm} \quad \sigma_c = 63 \text{ Kg/cm}^2 \quad A_f = 16 \text{ cm}^2$$

Sezione 7-8; 8

$$M = \frac{1}{14} 3750 \times 3,50^2 = 3300 \text{ Kgm} \quad \sigma_c = 51 \text{ Kg/cm}^2 \quad A_f = 12,2 \text{ cm}^2$$

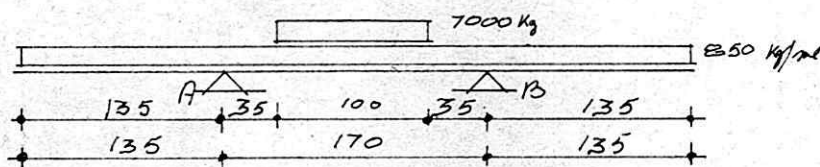
$$\text{Taglio massimo} \quad T = 10400 \text{ Kg} \quad \tau_{\max} = \frac{10400}{23 \times 0,9 \times 120} = 4,1 \text{ Kg/cm}^2$$

3-9-0 - Calcolo della soletta ascensore -

3-9-1 Analisi di carico

p.p.	$0,25 \times 1,00 \times 1,00 \times 2500$	= 625
sovracc. perm.		= 75
sovracc. accid.		= 150
		850 Kg/mq
macchinario		= 6000
basamento		= 1000
		7000 Kg

3-9-2 Condizioni di carico



$$M_A = M_B = 770 \text{ Kgm}$$

$$M_{AB} = \frac{1}{8} 850 \times 1,70^2 \pm \frac{1}{8} 7000 \times 1,00 \times 1,70(1 + 2 \frac{0,35}{1,70}) - 770 = 1630 \text{ Kgm}$$

Armando simmetricamente verifichiamo il momento massimo

$$M = 1630 \text{ Kgm} \quad H = 25 \text{ cm} \quad h = 23 \text{ cm} \quad h' = 2 \text{ cm} \quad b = 100 \text{ cm}$$

$$A_f = A'_f = 10,78 \text{ cm}^2 \quad (1 + 104/15) \quad A_f^x = 21,56 \text{ cm}^2 \quad h^x = 12,5 \text{ cm}$$

$$\gamma = \frac{A_f}{A'_f} = 1 \quad x = 5,47 \text{ cm}$$

$$\sigma_c = 19 \text{ Kg/cm}^2 \quad \sigma_f = 608 \text{ Kg/cm}^2$$

Coefficienti di sicurezza

$$\text{nel calcestruzzo} \quad \gamma_c = \frac{180}{19} = 9,4$$

$$\text{nel ferro} \quad \gamma_f = \frac{4200}{608} = 6,9$$

3-9-3 - Travi portasolette -

Analisi di carico

travi 28-23

soletta $850 \times 1,50/2 \times 7000/2$ = 4150sbalzo $850 \times 1,25$ = 1050

tompagno = 800

p.p. $0,20 \times 0,60 \times 1,00 \times 2500$ = 500

6500 Kg/ml

Travi 23-21

soletta $850 \times 1,50/2$ = 650sbalzo $850 \times 1,25$ = 1050

tompagno = 800

p.p. = 400

2900 Kg/ml

Armatura a flessione

 $M_{\max} = 2100 \text{ Kgm}$ $H = 60 \text{ cm}$ $b = 20 \text{ cm}$ $\sigma_f = 1400 \text{ Kg/cm}^2$ $\sigma_c = 34 \text{ Kg/cm}^2$ $A_f = 2,85 \text{ cm}^2$

3-10-0 - Copertura Volumi tecnici -

3-10-1 -Solai-

Carico per nervatura $q = 650 \times 0,50 = 325 \text{ Kg/ml}$

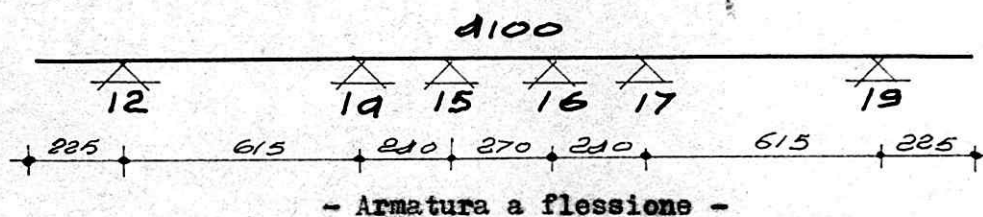
$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 50 \text{ cm}$$

$$M = \frac{1}{2} 325 \times 2,70^2 = 1150 \text{ Kgm} \quad \text{luce netta } 2,70 \text{ m.} \quad \sigma_c = 41 \text{ Kg/cm}^2 \quad A_f = 3,85 \text{ cm}^2$$

$$M = \frac{1}{2} 325 \times 1,50^2 = 360 \text{ Kgm} \quad \text{luce netta } 1,50 \text{ m.} \quad \sigma_c = 21 \text{ Kg/cm}^2 \quad A_f = 1,15 \text{ cm}^2$$

$$M = \frac{1}{12} 325 \times 2,83^2 = 200 \text{ Kgm} \quad \text{luce netta } = 2,70 \times 1,05 = 2,85 \text{ m.} \quad \sigma_c < 15 \text{ Kg/cm}^2 \quad A_f = 1 \text{ cm}^2$$

3-10-2 Travi 12-14-15-16-17-19



$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 80 \text{ cm} \quad b = 30 \text{ cm}$$

Sezione 12, 12-14

$$M = \frac{1}{14} 4100 \times 6,40^2 = 12000 \text{ Kgm} \quad \sigma_c = 54 \text{ Kg/cm}^2 \quad A_f = 12,5 \text{ cm}^2$$

Sezione 14

$$M = \frac{1}{10} 4100 \times 6,40^2 = 16800 \text{ Kgm} \quad \sigma_c = 65 \text{ Kg/cm}^2 \quad A_f = 17,2 \text{ cm}^2$$

Sezione 15

$$M = \frac{1}{12} 4100 \times 2,70^2 = 2500 \text{ Kgm} \quad \sigma_c = 22 \text{ Kg/cm}^2 \quad A_f = 2,45 \text{ cm}^2$$

Sezione 15-16

$$M = \frac{1}{18} 4100 \times 2,70^2 = 1650 \text{ Kgm} \quad \sigma_c = 17 \text{ Kg/cm}^2 \quad A_f = 1,6 \text{ cm}^2$$

$$\text{Taglio massimo } T = 12600 \text{ Kg} \quad \tau_{\max} = \frac{12600}{30 \times 0,9 \times 78} = 5,9 \text{ Kg/cm}^2$$

4-0-0 EDIFICIO TIPO B3

4-1-0 - Analisi di carico sulle travi -

4-1-1 COPERTURA

Trave 12-14-15-16-17-19; 43-45-46-47-48-50

Solaio 650 x 2,70 = 1750

" " 650 x 2,70 = 1750

p.p. 0,30 x 0,80 x 1,00 x 2500 = 600

4100 Kg/ml

Trave 21; 22; 52; 53

Solaio 650 x 1,40/2 = 450

sovracc. 365 x 0,50 = 200

p.p. 0,50 x 0,25 x 1,00 x 2500 = 300

950 Kg/ml

Trave 21- 27; 22-28; 52-58; 53-59;

Solaio 650 x 1,30/2 = 400

sbalzo 650 x 0,70 = 450

sovracc. 365 x 0,50 = 200

p.p. 0,50 x 0,25 x 1,00 x 2500 = 300

1350 Kg/ml

4-1-2 COPERTURA PIANO TIPO

Trave 1-2-3-4; 7-8-9-10; 32-33-34-35; 38-39-40-41

Solaio 650 x 3,65/2 = 1200

sovracc. 365 x 0,70 = 250

tompagno = 850

p.p. 0,70 x 1,00 x 0,25 x 2500 = 450

2750 Kg/ml

Trave 4-5; 6-7; 35-36; 37-38

Solaio 650 x 3,65/2 = 1200

sovracc. 0,70 x 365 = 250

sbalzo 800 x 1,15 = 900

tompagno = 850

p.p. = 450

3650 Kg/ml

Trave 11-12-13-14-15; 16-17-18-19-20; 42-43-44-45-46; 47-48-49-50-51

Solaio 650 x 3,65/2 = 1200

" " 650 x 3,90/2 = 1250

sovracc. 3,65 x 0,90 = 350

p.p. 0,90 x 0,25 x 1,00 x 2500 = 550

3350 Kg/ml

Trave 24-25-26; 29-30-31; 55-56-57; 60-61-62

Solaio 650 x 3,90/2 = 1250

sovracc. 365 x 0,70 = 250

tompagno = 850

p.p. = 450

2800 Kg/ml

Trave 26-27; 28-29; 57-58; 59-60

Solaio 650 x 3,40/2 = 1100

sovracc. 365 x 1,20 = 450

tompagno = 850

p.p. 20 x 0,25 x 1,00 x 2500 = 750

3150 Kg/ml

Trave 5-6; 36-37; 21-23; 22-23; 52-54; 53-54; 23-27; 23-28;
54-58; 54-55

tompagno

= 850

p.p. $0,30 \times 0,40 \times 1,00 \times 2500$

= 350

1200 Kg/ml

Trave 5-15-21; 6-16-22; 36-46-52; 37-47-53

Rampa 850 $\times$ 1,30

= 1100

tompagno

= 850

p.p. $0,30 \times 0,60 \times 1,00 \times 2500$

= 450

2400 Kg/ml

Trave 1-11-24; 10-20-31; 32-42-55; 41-51-62

tompagno

= 850

sovracc. 365 $\times$ 0,50

= 200

p.p. $0,50 \times 0,25 \times 1,00 \times 2500$

= 350

1400 Kg/ml

4-2-0

- Analisi di carico sui pilastri -

4-2-1

COPERTURA

Pilastri 12; 19; 43; 50

trave 12-14 $4100 \times 6,70/2$ = 13750p.p. $0,30 \times 0,30 \times 2,70 \times 2500$ = 600

14350 Kg

Pilastri 14; 17; 45; 48

trave 12-14 = 13750

" " 14-15 $4100 \times 2,00/2$ = 4100

p.p. = 600

18450 Kg

Pilastro 15; 16; 46; 47

trave 15-14 = 4100

" " 15-16 $4100 \times 3,00/2$ = 6150

p.p. = 600

10850 Kg

Pilastri 21; 22; 52; 53

trave 21-22 $950 \times 3,00/2$ = 1400" " 21-27 $1350 \times 3,10/2$ = 2100p.p. $0,85 \times 0,20 \times 2,70 \times 2500$ = 1150

4650 Kg

Pilastri 27; 28; 58; 59

trave 27-21 = 2100

p.p. = 600

2700 Kg

4-2-2 Copertura piano tipo

Pilastrini	1; 10; 32; 41	
trave	1-2	2750 x 3,55/2 = 4900
" "	1-11	1400 x 4,45/2 = 3100
p.p.	0,30 x 0,40 x 3,05 x 2500	= 950
		8950 Kg

Pilastrini	2; 9; 33; 40	
trave	2-1	= 4900
" "	2-3	2750 x 3,00/2 = 4100
p.p.		= 950
		9950 Kg

Pilastrini	3; 8; 34; 39	
trave	3-2	= 4100
" "	3-4	2750 x 3,70/2 = 5100
p.p.		= 950
		10150 Kg

Pilastrini	4; 7; 35; 38	
trave	4-3	= 5100
" "	4-5	3650 x 2,00/2 = 3650
p.p.		= 950
		9700 Kg

Pilastrini	5; 6; 36; 37	
Trave	5-4	= 3650
" "	5-6	1200 x 3,00/2 = 1800
" "	5-15	2400 x 4,45/2 = 5350
p.p.		= 950
		11650 Kg

Pilastri 11; 20; 42; 51

trave 11-1

= 3100

" " 11-24 1400 x 4,90/2

= 3450

" " 11-12 3350 x 3,55/2

= 5950

p.p.= 950

13450 Kg

Pilastri 12; 19; 43; 50

trave 12-11

= 5950

" " 12-13 3350 x 3,00/2

= 5050

p.p.= 950

11950 Kg

Pilastri 13; 18; 44; 49

trave 13-12

= 5050

" " 13-14 3350 x 3,70/2

= 6200

p.p.= 950

12200 Kg

Pilastri 14; 17; 45; 48

trave 14-13

= 6200

" " 14-15 3350 x 2,00/2

= 3350

p.p.= 950

10500 Kg

Pilastri 15; 16; 46; 47

trave 15-14

= 3350

" " 15-5

= 5350

" " 15-21 2400 x 1,80/2

= 2150

p.p.= 950

11700 Kg

Pilastri 21; 22; 52; 53

trave 21-15 = 2150

" " 21-23 1200 x 1,60/2 = 1000

aliquota solaio 650 x 2,10 x 0,65/2 = 450

p.p. 0,85 x 0,20 x 3,05 x 2500 = 1300

4900 Kg

Pilastri 23; 54

trave 23-21 = 1000

" " 23-27 = 1000

" " 23-28 1200 x 1,50/2 = 950

" " 23-22 = 950

p.p. 0,20 x 2,00 x 3,05 x 2500 = 3050

6950 Kg

Pilastri: 24; 31; 55; 62

trave 24-11 = 3450

" " 24-25 2800 x 3,55/2 = 4950

p.p. = 950

9350 Kg

Pilastri 25; 30; 56; 61

trave 25-24 = 4950

" " 25-26 = 4950

p.p. = 950

9850 Kg

Pilastri 26; 29; 57; 60

trave 26-25 = 4950

" " 26-27 3150 x 5,55/2 = 8750

p.p. = 950

14650 Kg

Pilastrini 27, 28, 58, 59

trave 27-26

" " 27-23

p.p.

= 8750

= 1000

= 950

10700 Kg

4-3-0 - Verifica ed armatura pilastri -

4-3-1

Carichi al piede dei pilastri

Pilastri	Copertura	Cop. IV	Cop. III	Cop. II	Cop. I	Cop. portico fondazione
1,10,32,41		9000	18000	27000	36000	45000
2,9,33,40		10000	20000	30000	40000	50000
3,8,34,39		10150	20300	30450	40500	50650
4,7,35,38		9700	19400	29100	38800	48500
5,6,36,37		11650	23300	34950	46600	58250
11,20,42,51		13450	26900	40350	53800	67250
12,19,43,50	14350	26300	38250	50200	62150	74100
13,18,44,49		12200	24400	36600	48800	60000
14,17,45,48	18450	28950	39450	49950	60450	70950
15,16,46,47	10850	22550	34250	45950	57650	69350
21,22,52,53	4650	9550	14450	19350	24250	29150
23,54		17000	14000	21000	28000	35000
24,31,55,62		9350	18700	28050	37400	46750
25,30,56,61		9850	19700	29550	39400	49250
26,29,57,60		14650	29300	43950	58600	73250
27,28,58,59	2700	13400	24100	34800	45500	56200

4-3-2 Sezione ed armatura pilastri fondazione e portico

N°	P tn	A' c cmq	A f cmq	Armatura	Sezione cm x cm	A ci cmq	σ c Kg/cmq
10,32,41	45	950	7,6	4Ø14 + 4Ø12	30 x 40	1307	34
9,33,40	50	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	38
8,34,39	51	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	39
7,35,38	49	1000	8	4Ø14 + 4Ø12	30 x 40	1307	37
6,36,37	58	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	44
1,20,42,51	67	1400	11,2	4Ø14 + 6Ø12	30 x 50	1629	41
2,19,43,50	74	1500	12	4Ø14 + 6Ø12	30 x 50	1629	45
3,18,44,49	60	1250	10	4Ø14 + 4Ø12	30 x 40	1307	45
4,17,45,48	71	1450	11,6	4Ø14 + 6Ø12	30 x 50	1629	43
5,16,46,47	69	1400	11,2	4Ø14 + 6Ø12	30 x 50	1629	42
1,22,52,53	29	600	4,8	14 Ø 10	20 x 85	1809	16
23,54	35	700	5,6	24 Ø 10	20 x 200	4188	<10
1,31,55,62	47	950	7,6	4Ø14 + 4Ø12	30 x 40	1307	35
5,30,56,61	49	1000	8	4Ø14 + 4Ø12	30 x 40	1307	37
6,29,57,60	73	1500	12	4Ø14 + 6Ø12	30 x 50	1629	44
7,28,58,55	56	1150	9,2	4Ø14 + 4Ø12	30 x 40	1307	42

Staffe Ø 8" / 15"

4-3-3 Sezione ed armatura pilastri piano I e II

N°	P tn	A' c cmq	A f cmq	Armatura	Sezione cm x cm	A ci cmq	σ c Kg/cmq
1,10,32,41	36	750	6	8 $\emptyset$ 12	30 x 30	990	36
2,9,33,40	40	850	6,8	8 $\emptyset$ 12	30 x 30	990	40
3,8,34,39	41	850	6,8	8 $\emptyset$ 12	30 x 30	990	40
4,7,35,38	39	800	6,4	8 $\emptyset$ 12	30 x 30	990	40
5,6,36,37	47	950	7,6	8 $\emptyset$ 12	30 x 30	990	44
11,20,42,51	54	1100	8,8	4 $\emptyset$ 14 + 4 $\emptyset$ 12	30 x 40	1307	41
12,19,43,50	62	1300	10,4	4 $\emptyset$ 14 + 4 $\emptyset$ 12	30 x 40	1307	45
13,18,44,49	49	1000	8,0	8 $\emptyset$ 12	30 x 30	990	45
14,17,45,48	60	1250	10	4 $\emptyset$ 14 + 4 $\emptyset$ 12	30 x 40	1307	45
15,16,46,47	58	1200	9,6	4 $\emptyset$ 14 + 4 $\emptyset$ 12	30 x 40	1307	44
21,22,52,53	24	500	4,0	14 $\emptyset$ 10	20 x 85	1809	13
23,54	28	600	4,8	24 $\emptyset$ 10	20 x 200	4188	<10
24,31,55,62	37	750	6	8 $\emptyset$ 12	30 x 30	990	37
25,30,56,61	39	800	6,4	8 $\emptyset$ 12	30 x 30	990	39
26,29,57,60	59	1200	9,6	4 $\emptyset$ 14 + 4 $\emptyset$ 12	30 x 40	1307	44
27,28,58,59	46	950	7,6	8 $\emptyset$ 12	30 x 30	990	44

Staffe $\emptyset$ 6" / 15"

4-3-4 Sezione ed armatura pilastri piano III e IV

N°	P tn	A' c cmq	A f cmq	Armatura	Sezione cm x cm
1,10,32,41	18	400	3,2	8 Ø 10	30 x 30
2,9,33,40	20	400	3,2	8 Ø 10	30 x 30
3,8,34,39	20	400	3,2	8 Ø 10	30 x 30
4,7,35,38	19	400	3,2	8 Ø 10	30 x 30
5,6,36,37	23	450	3,6	8 Ø 10	30 x 30
11,20,42,51	27	550	4,4	4Ø12 + 4Ø10	30 x 30
12,19,43,50	38	800	6,4	4Ø12 + 4Ø10	30 x 30
13,18,44,49	24	500	4,0	8 Ø 10	30 x 30
14,17,45,48	39	800	6,7	4Ø12 + 4Ø10	30 x 30
15,16,46,47	34	700	5,6	4Ø12 + 4Ø10	30 x 30
21,22,52,53	14	300	2,4	14 Ø 8	20 x 85
23,54	14	300	2,4	24 Ø 8	20 x 200
24,31,55,62	19	400	3,2	8 Ø 10	30 x 30
25,30,56,61	20	400	3,2	8 Ø 10	30 x 30
26,29,57,60	29	600	4,8	4Ø12 + 4Ø10	30 x 30
27,28,58,59	24	500	4,0	8 Ø 10	30 x 30

Staffe Ø 6" / 15"

4-3-5 Sezione ed armatura pilastri piano Volumi-tecnici

N°	P tn	A' c cmq	Armatura	Sezione cm x cm
12,19,43,50	14	300	8 $\emptyset$ 10	30 x 30
14,17,45,48	18	400	8 $\emptyset$ 10	30 x 30
15,16,46,47	11	250	8 $\emptyset$ 10	30 x 30
21,22,52,53	5	100	14 $\emptyset$ 8	20 x 85
27,28,58,59	3	100	8 $\emptyset$ 10	30 x 30

Staffe $\emptyset$ 6" / 15"

4-4-0 Fondazioni

4-4-1 -Analisi di carico sul terreno-

Pilastrì	1729000
trave	<u>252900</u>
	1981900 Kg

4-4-2 -Tensioni sul terreno-

$$\sigma_t = \frac{1981900}{1568300} = 1,26 \text{ Kg/cm}^2$$

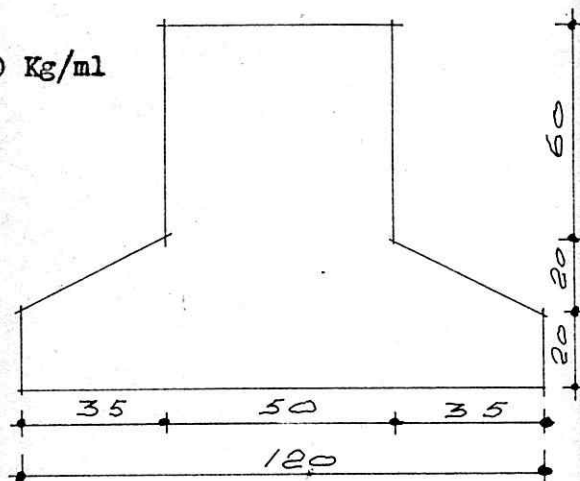
4-4-3 Travi 1-2-3-4-5-6-7-8-9-10; 32-33-34-35-36-37-38-39-40-41

Carico sulla trave

$$q = \frac{1729000}{1568300} \times 100 \times 120 = 13200 \text{ Kg/ml}$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 13200 \times 3,70^2 = 15050 \text{ Kgm} \quad H = 100'' \quad b = 50''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 34 \text{ Kg/cm}^2 \quad A_f = 11,7 \text{ cm}^2$$

$$M_{camp} = 15050 \text{ Kgm} \quad H = 100'' \quad b = 120''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 21 \text{ Kg/cm}^2 \quad A_f = 11,5 \text{ cm}^2$$

-Armatura trasversale-

$$M = \frac{1}{2} 13200 \times 0,35^2 = 800 \text{ Kgm} \quad H = 40'' \quad b = 100''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c < 15 \text{ Kgm} \quad A_f = 1,5 \text{ cm}^2$$

4-4-4 Trave 24-25-26-27-28-29-30-31; 55-56-57-58-59-60-61-62

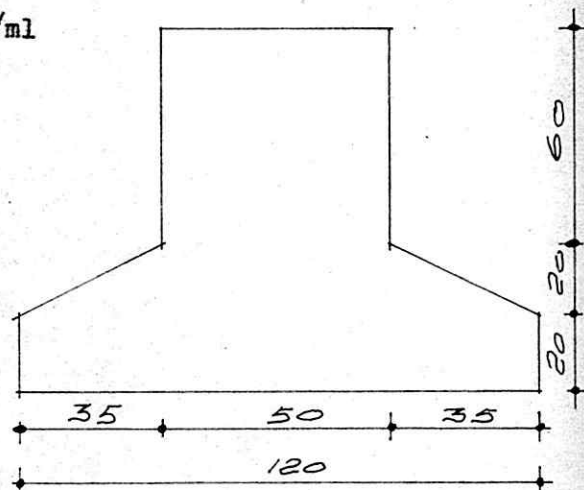
Carico sulla trave

$$q = 1729000 \times 100 \times 120 = 13200 \text{ Kg/ml}$$

$$1568300$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 13200 \times 5,55^2 = 33900 \text{ Kgm}$$

$$H = 100'' \quad b = 50''$$

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

$$\sigma_c = 55 \text{ Kg/cm}^2$$

$$A_f = 27,1 \text{ cm}^2$$

$$M_{camp} = 33900 \text{ Kgm}$$

$$H = 100''$$

$$b = 120''$$

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

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

$$A_f = 25,4 \text{ cm}^2$$

4-4-5 Trave 11-12-13-14-15-16-17-18-19-20; 42-43-44-45-46-47-48-49-50-51

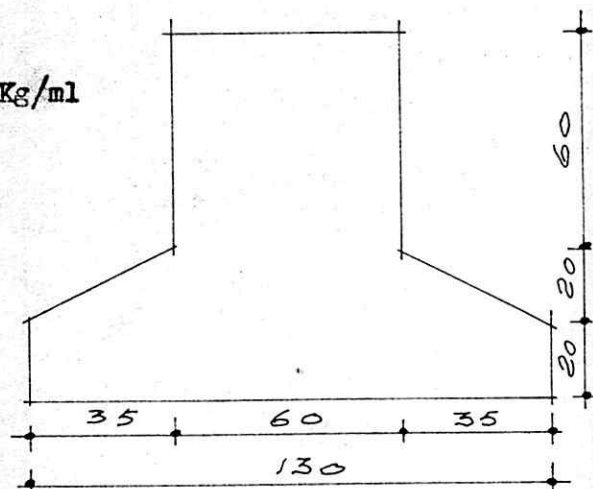
Carico sulla trave

$$q = 1729000 \times 100 \times 130 = 14300 \text{ Kg/ml}$$

$$1568300$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 14300 \times 3,70^2 = 16300 \text{ Kgm}$$

$$H = 100'' \quad b = 60''$$

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

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

$$A_f = 12,8 \text{ cm}^2$$

$$M_{camp} = 16300 \text{ Kgm}$$

$$H = 100''$$

$$b = 130''$$

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

$$\sigma_c = 21 \text{ Kg/cm}^2$$

$$A_f = 12,4 \text{ cm}^2$$

-Armatura trasversale-

$$M = \frac{1}{2} 14300 \times 0,35^2 = 900 \text{ Kgm} \quad H = 40'' \quad b = 100''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 15 \text{ Kg/cm}^2 \quad A_f = 1,5 \text{ cm}^2$$

4-4-6 Trave 1-11-24; 10-20-31; 32-42-55; 41-51-62

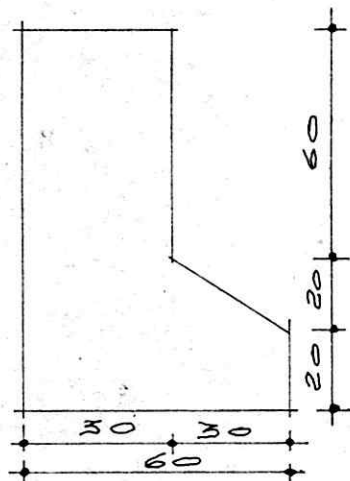
Carico sulla trave

$$q = \frac{1729000}{100} \times 100 \times 60 = 6600 \text{ Kg/ml}$$

1568300

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 6600 \times 4,90^2 = 13200 \text{ Kgm} \quad H = 100'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 43 \text{ Kg/cm}^2 \quad A_f = 10,5 \text{ cm}^2$$

$$M_{camp} = 13200 \text{ Kgm} \quad H = 100'' \quad b = 60''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 29 \text{ Kg/cm}^2 \quad A_f = 10,2 \text{ cm}^2$$

4-4-7 Trave 2-12-25; 3-13-26; 4-14; 5-15-21-23-27; 6-16-22-23-28;
7-17-8-18-29; 9-19-30; 33-43-56; 34-44-57; 35-45; 36-46-52-54
58; 37-47-53-54-59; 38-48; 39-49-60; 40-50-61

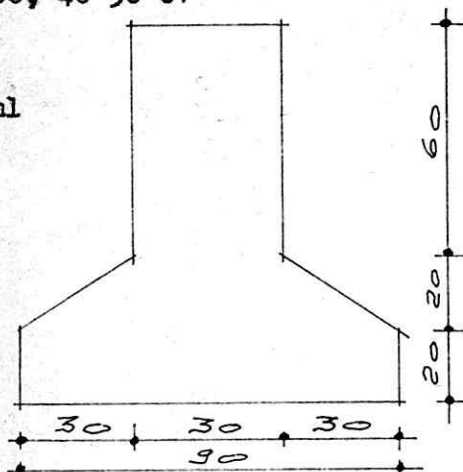
Carico sulla trave

$$q = \frac{1729000}{90} \times 90 \times 100 = 9900 \text{ Kg/ml}$$

1568300

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 9900 \times 4,90^2 = 19800 \text{ Kgm} \quad H = 100'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 54 \text{ Kg/cm}^2 \quad A_f = 15,8 \text{ cm}^2$$

$$M_{\text{camp}} = 19800 \text{ Kgm}$$

$$H = 100''$$

$$b = 90''$$

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

$$\sigma_c = 29 \text{ Kg/cm}^2$$

$$A_f = 15,4 \text{ cm}^2$$

4-5-0 - Vano Scala -

4-5-1 Calcolo del gradino

Analisi di carico

$$\text{p.p.} \quad \frac{0,235 + 0,07}{2} \times 0,30 \times 1,00 \times 2500 = 115$$

sovracc. perm. (intonaco, grado

$$\text{sottogradio, massetto}) = 150 \times 0,30 = 45$$

$$\text{sovracc. accid.} = 400 \times 0,30 = 120$$

280 Kg/ml

$$M = \frac{1}{2} 280 \times 1,20^2 = 200 \text{ Kgm} \quad H = \frac{0,235 \times 0,07}{2} = 0,15 \quad b = 30 \text{ cm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 36 \text{ Kg/cm}^2 \quad A_f = 1,10 \text{ cm}^2$$

4-5-2 Calcolo dei pianerottoliSolaio in latero-cemento $H = 16'' + 4'' \quad i = 50''$

Analisi di carico

$$\text{p.p.} = 140$$

$$\text{soletta} \quad 0,04 \times 1,00 \times 1,00 \times 2500 = 100$$

$$\text{sovracc. accid. (intonaco, massetto, pavimento)} = 110$$

$$\text{sovracc. perm.} = 400$$

750 Kg/mq

$$M_{inc} = \frac{1}{12} 750 \times 2,70^2 = 455 \text{ Kgm} \quad H = 20'' \quad b = 100''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 22 \text{ Kg/cm}^2 \quad A_f = 0,96 \text{ cm}^2 / 50''$$

$$M_{camp} = \frac{1}{14} 750 \times 2,70^2 = 390 \text{ Kgm} \quad H = 20'' \quad b = 100''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 20 \text{ Kg/cm}^2 \quad A_f = 0,82 \text{ cm}^2 / 50''$$

4-5-3 Calcolo delle travi a ginocchio

Analisi di carico

$$\text{Rampa } 850 \times 1,20 = 1100$$

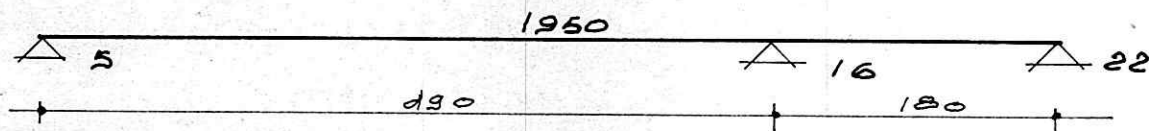
$$\text{tompagne} = 400$$

$$\text{p.p. } 0,30 \times 0,60 \times 1,00 \times 2500 = 450$$

$$1950 \text{ Kg/ml}$$

Verifica ed armatura a flessione

Schema di calcolo



$$M_5 = M_{5-16} = \frac{1}{14} \cdot 1950 \times 4,90^2 = 3350 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 35 \text{ Kg/cm}^2 \quad A_f = 4,4 \text{ cm}^2$$

$$M_{16} = \frac{1}{10} \cdot 1950 \times 4,90^2 = 4700 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 43 \text{ Kg/cm}^2 \quad A_f = 6,3 \text{ cm}^2$$

$$M_{16-22} = \frac{1}{8} \cdot 1950 \times 1,80^2 - 4700 = -3900 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 39 \text{ Kg/cm}^2 \quad A_f = 5,3 \text{ cm}^2$$

$$M_{22} = \frac{1}{12} \cdot 1950 \times 1,80^2 = 550 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c < 15 \text{ Kg/cm}^2 \quad A_f = 1,5 \text{ cm}^2$$

Verifica a torsione della trave 5-16

$$m_i = \frac{1}{2} \cdot 850 \times 1,20^2 = 612 \text{ Kgm} \quad m_t = 612 + 1020 \times 0,15 = 765$$

$$t_i = 850 \times 1,20 = 1020 \text{ Kgm} \quad M_t = 765 \times \frac{4,90}{2} = 1875 \text{ Kgm}$$

$$\psi = 3 + \frac{2,6}{0,45 + \frac{60}{30}} = 4,061$$

$$\tau = \frac{4,061 \times 187500}{60 \times 30^2} = 14,1 \text{ Kg/cm}^2$$

- Armatura a torsione -

$$F = 55 \times 25 = 1375 \text{ cm}^2 \quad V = (55 + 25) \times 2 = 160 \text{ cm}$$

$$\Sigma_{f1} = \frac{187500}{2 \times 1400 \times 1375} = 160 = 7,8 \text{ cm}^2$$

$$\Sigma_{fs} = \frac{187500}{2 \times 1400 \times 1375} = 0,0487 \text{ cm}^2/\text{cm}$$

4-5-4 Calcolo della trave al pianerottolo di riposo

$$M = \frac{1}{12} \cdot 1200 \times 3,00^2 = 900 \text{ Kg} \cdot \text{m} \quad H = 40'' \quad b = 20''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 34 \text{ Kg/cm}^2 \quad A_f = 1,8 \text{ cm}^2$$

4-6-0

BALCONI

$$q = 800 \text{ Kg/mq} \quad H = 20 + 5 \text{ cm} \quad b = 100 \text{ cm}$$

$$M = \frac{1}{2} 800 \times 1,05^2 = 440 \text{ Kgm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 17 \text{ Kg/cm}^2 \quad A_f = 1,6 \text{ cm}^2/\text{ml}$$

$$i = 50 \text{ cm} \quad A_f = 0,8 \text{ cm}^2$$

$$M = \frac{1}{2} 800 \times 1,15^2 = 530 \text{ Kgm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 18 \text{ Kg/cm}^2 \quad A_f = 1,7 \text{ cm}^2/\text{ml}$$

$$i = 50 \text{ cm} \quad A_f = 0,85 \text{ cm}^2/50$$

4-7-0 - S O L A I -Carico nervatura $q = 650 \times 0,50 = 325 \text{ Kg/ml}$ $\sigma_f = 1400 \text{ Kg/cm}^2$ $b = 50 \text{ cm}$ luce netta $3,63 \times 1,05 = 3,80 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 3,80^2 = 390 \text{ Kg} \cdot \text{m}$	$H = 25 \text{ cm}$	$\sigma_c = 22 \text{ Kg/cm}^2$	$A_f = 1,25 \text{ cm}^2$
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campata	$M = \frac{1}{14} 325 \times 3,80^2 = 335 \text{ Kg} \cdot \text{m}$	$H = 25 \text{ cm}$	$\sigma_c = 20 \text{ Kg/cm}^2$	$A_f = 1,05 \text{ cm}^2$
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luce netta $2,40 \times 1,05 = 2,50 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 2,50^2 = 170 \text{ Kg} \cdot \text{m}$	$H = 25 \text{ cm}$	$\sigma_c < 15 \text{ Kg/cm}^2$	$A_f = 0,8 \text{ cm}^2$
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campata	$M = \frac{1}{14} 325 \times 2,50^2 = 145 \text{ Kg} \cdot \text{m}$	$H = 25 \text{ cm}$	$\sigma_c < 15 \text{ Kg/cm}^2$	$A_f = 0,8 \text{ cm}^2$
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luce netta $3,58 \times 1,05 = 3,75 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 3,75^2 = 380 \text{ Kg} \cdot \text{m}$	$H = 25 \text{ cm}$	$\sigma_c = 22 \text{ Kg/cm}^2$	$A_f = 1,25 \text{ cm}^2$
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	$H = 18 \text{ cm}$	$\sigma_c = 29 \text{ Kg/cm}^2$	$A_f = 1,6 \text{ cm}^2$
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campata	$M = \frac{1}{14} 325 \times 3,75^2 = 330 \text{ Kg} \cdot \text{m}$	$H = 25 \text{ cm}$	$\sigma_c = 20 \text{ Kg/cm}^2$	$A_f = 1,05 \text{ cm}^2$
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	$H = 18 \text{ cm}$	$\sigma_c = 27 \text{ Kg/cm}^2$	$A_f = 1,4 \text{ cm}^2$
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luce netta $4,08 \times 1,05 = 4,30 \text{ m.}$

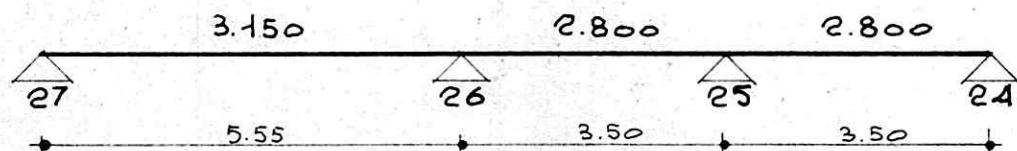
incastro	$M = \frac{1}{12} 325 \times 4,30^2 = 500 \text{ Kg} \cdot \text{m}$	$H = 25 \text{ cm}$	$\sigma_c = 26 \text{ Kg/cm}^2$	$A_f = 1,65 \text{ cm}^2$
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	$H = 18 \text{ cm}$	$\sigma_c = 35 \text{ Kg/cm}^2$	$A_f = 2,2 \text{ cm}^2$
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campata	$M = \frac{1}{14} 325 \times 4,30^2 = 430 \text{ Kg} \cdot \text{m}$	$H = 25 \text{ cm}$	$\sigma_c = 24 \text{ Kg/cm}^2$	$A_f = 1,42 \text{ cm}^2$
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	$H = 18 \text{ cm}$	$\sigma_c = 31 \text{ Kg/cm}^2$	$A_f = 1,8 \text{ cm}^2$
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4-8-1 Travi 24-25-26-27; 28-29-30-31;
55-56-57-58; 59-60-61-62;



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 120/70 \text{ cm}$$

Sezione 27; 27-26

$$M = \frac{1}{14} 3150 \times 5,55^2 = 6950 \text{ Kgm} \quad b = 120 \text{ cm} \quad \sigma_c = 73 \text{ Kg/cm}^2 \quad A_f = 20,5 \text{ cm}^2$$

Sezione 26

$$M = \frac{1}{10} 3150 \times 5,55^2 = 9700 \text{ Kgm} \quad b = 120 \text{ cm} \quad \sigma_c = 90 \text{ Kg/cm}^2 \quad A_f = 29 \text{ cm}^2$$

Sezione 26-25

$$M = \frac{1}{16} 2800 \times 3,50^2 = 2200 \text{ Kgm} \quad b = 70 \text{ cm} \quad \sigma_c = 50 \text{ Kg/cm}^2 \quad A_f = 8,0 \text{ cm}^2$$

Sezione 25

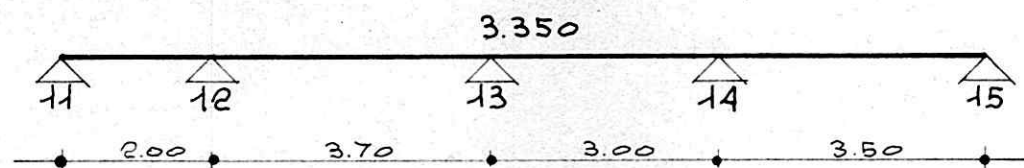
$$M = \frac{1}{10} 2800 \times 3,50^2 = 3450 \text{ Kgm} \quad \sigma_c = 69 \text{ Kg/cm}^2 \quad A_f = 12,5 \text{ cm}^2$$

Sezione 27-24; 24

$$M = \frac{1}{14} 2800 \times 3,50^2 = 2450 \text{ Kgm} \quad \sigma_c = 53 \text{ Kg/cm}^2 \quad A_f = 8,4 \text{ cm}^2$$

$$\text{Taglio massimo } T = 8750 \text{ Kg} \quad \tau_{\max} = \frac{8750}{23 \times 0,9 \times 120} = 3,5 \text{ Kg/cm}^2$$

4-8-2 Travi 11-12-13-14-15; 16-17-18-19-20
42-43-44-45-46; 47-48-49-50-51



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 90 \text{ cm}$$

Sezione 16; 16-17

$$M = \frac{1 \ 3350}{14} \times 2,70^2 = 1750 \text{ Kgm} \quad \sigma_c = 38 \text{ Kg/cm}^2 \quad A_f = 6 \text{ cm}^2$$

Sezione 17; 18

$$M = \frac{1 \ 3350}{12} \times 3,70^2 = 3800 \text{ Kgm} \quad \sigma_c = 59 \text{ Kg/cm}^2 \quad A_f = 13 \text{ cm}^2$$

Sezione 17-18

$$M = \frac{1 \ 3350}{16} \times 3,70^2 = 2850 \text{ Kgm} \quad \sigma_c = 50 \text{ Kg/cm}^2 \quad A_f = 9,7 \text{ cm}^2$$

Sezione 18; 19

$$M = \frac{1 \ 3350}{16} \times 3,00^2 = 1900 \text{ Kgm} \quad \sigma_c = 39 \text{ Kg/cm}^2 \quad A_f = 6,5 \text{ cm}^2$$

Sezione 19

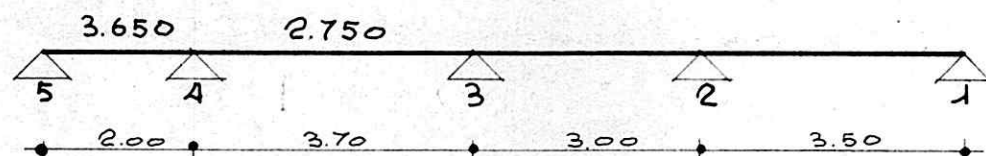
$$M = \frac{1 \ 3350}{10} \times 3,50^2 = 4100 \text{ Kgm} \quad \sigma_c = 62 \text{ Kg/cm}^2 \quad A_f = 14 \text{ cm}^2$$

Sezione 19-20; 20

$$M = \frac{1 \ 3350}{14} \times 3,50^2 = 2950 \text{ Kgm} \quad \sigma_c = 51 \text{ Kg/cm}^2 \quad A_f = 10,1 \text{ cm}^2$$

$$\text{Taglio massimo } T = 6200 \text{ Kg} \quad \tau_{\max} = \frac{6200}{23 \times 0,9 \times 90} = 3,3 \text{ Kg/cm}^2$$

4-8-3 Travi 1-2-3-4-5; 6-7-8-9-10;
32-33-34-35-36; 37-38-39-40-41;



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cmq} \quad H = 25 \text{ cm} \quad b = 70 \text{ cm}$$

Sezione 5, 5-4

$$M = \frac{1}{14} 3650 \times 2,00^2 = 1050 \text{ Kgm} \quad \sigma_c = 32 \text{ Kg/cmq} \quad A_f = 4,9 \text{ cmq}$$

Sezione 4, 3

$$M = \frac{1}{12} 2750 \times 3,70^2 = 3150 \text{ Kgm} \quad \sigma_c = 62 \text{ Kg/cmq} \quad A_f = 10,9 \text{ cmq}$$

Sezione 4-3

$$M = \frac{1}{16} 2750 \times 3,70^2 = 2350 \text{ Kgm} \quad \sigma_c = 52 \text{ Kg/cmq} \quad A_f = 8 \text{ cmq}$$

Sezione 3-2

$$M = \frac{1}{16} 2750 \times 3,00^2 = 1550 \text{ Kgm} \quad \sigma_c = 40 \text{ Kg/cmq} \quad A_f = 5,2 \text{ cmq}$$

Sezione 2

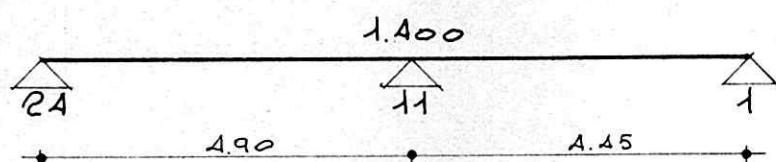
$$M = \frac{1}{10} 2750 \times 3,50^2 = 3350 \text{ Kgm} \quad \sigma_c = 64 \text{ Kg/cmq} \quad A_f = 11,6 \text{ cmq}$$

Sezione 2-1, 1

$$M = \frac{1}{14} 2750 \times 3,50^2 = 2400 \text{ Kgm} \quad \sigma_c = 53 \text{ Kg/cmq} \quad A_f = 8,3 \text{ cmq}$$

$$\text{Taglio massimo} \quad T = 5100 \text{ Kg} \quad \tau_{\max} = \frac{5100}{23 \times 0,9 \times 70} = 3,5 \text{ Kg/cmq}$$

4-8-4 Travi 1-11-24; 10-20-31;
32-42-55; 41-51-62;

**- Armatura a flessione -**

$$\sigma_f = 1400 \text{ Kg/cmq} \quad H = 25 \text{ cm} \quad b = 50 \text{ cm}$$

Sezione 24; 24-11

$$M = \frac{1}{14} 1400 \times 4,90^2 = 2400 \text{ Kgm} \quad \sigma_c = 64 \text{ Kg/cm}^2 \quad A_f = 8,4 \text{ cm}^2$$

Sezione 11

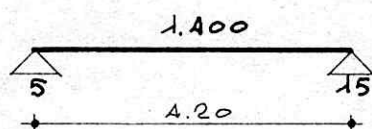
$$M = \frac{1}{10} 1400 \times 4,90^2 = 3350 \text{ Kgm} \quad \sigma_c = 79 \text{ Kg/cm}^2 \quad A_f = 11,8 \text{ cm}^2$$

Sezione 11-1; 1

$$M = \frac{1}{14} 1400 \times 4,45^2 = 2000 \text{ Kgm} \quad \sigma_c = 57 \text{ Kg/cm}^2 \quad A_f = 6,8 \text{ cm}^2$$

$$\text{Taglio massimo } T = 3450 \text{ Kg} \quad \tau_{\max} = \frac{3450}{23 \times 0,9 \times 50} = 3,3 \text{ Kg/cm}^2$$

4-8-5 Travi, 5-15; 6-16; 36-46; 37-47;



- Armatura a flessione -

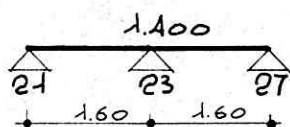
$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 30 \text{ cm}$$

Sezione 15; 15-24; 24

$$M = \frac{1}{12} 1400 \times 4,20^2 = 2050 \text{ Kgm} \quad \sigma_c = 80 \text{ Kg/cm}^2 \quad A_f = 7,2 \text{ cm}^2$$

4-8-6 Travi 21-23-27; 22-23-28;

52-54-58; 53-54-59;



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 20 \text{ cm}$$

Sezione 4; 4-9; 9

$$M = \frac{1}{12} 1400 \times 1,60^2 = 300 \text{ Kgm} \quad \sigma_c = 32 \text{ Kg/cm}^2 \quad A_f = 1,5 \text{ cm}^2$$

4-9-0 - Calcolo della soletta ascensore -

4-9-1 Analisi di carico

p.p. $0,25 \times 1,00 \times 1,00 \times 2500$

= 625

sovracc. perm.

= 75

sovracc. accid.

= 150

850 Kg/mq

macchinario

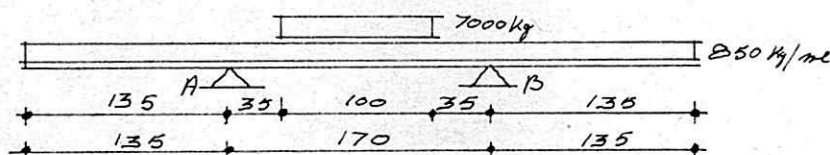
= 6000

basamento

= 1000

7000 Kg

4-9-2 Condizioni di carico



$$M_A = M_B = 770 \text{ Kgm}$$

$$M_{AB} = \frac{1}{8} 850 \times 1,70^2 + \frac{1}{8} 7000 \times 1,00 \times 1,70 \left(1 + 2 \frac{0,35}{1,70}\right) - 770 = 1630 \text{ Kgm}$$

Armando simmetricamente verifichiamo il momento massimo

$$M = 1630 \text{ Kgm} \quad H = 25 \text{ cm} \quad h = 23 \text{ cm} \quad h' = 2 \text{ cm} \quad b = 100 \text{ cm}$$

$$A_f = A'_f = 10,78 \text{ cm}^2 (1+1\phi 14/15) \quad A_f^x = 21,56 \text{ cm}^2 \quad h^x = 12,5 \text{ cm}$$

$$\gamma = \frac{A_f}{A'_f} = 1 \quad x = 5,47 \text{ Kg/cm}^2$$

4-9-3 Coefficienti di sicurezza

nel calcestruzzo $\gamma_c = \frac{180}{19} = 9,4$

nel ferro $\gamma_f = \frac{4200}{608} = 6,9$

4-9-4 Travi portasolette

Analisi di carico

Travi 28-23

soletta	$850 \times 1,50/2 \times 7000/2$	= 4150
sbalzo	$850 \times 1,25$	= 1050
tompagno		= 800
p.p.	$0,20 \times 0,60 \times 1,00 \times 2500$	= 500
		6500 Kg/ml

Travi 23-21

soletta	$850 \times 1,50/2$	= 650
sbalzo	$850 \times 1,25$	= 1050
tompagno		= 800
p.p.		= 400
		2900 Kg /ml

Armatura a flessione

$$\begin{aligned} M_{\max} &= 2100 \text{ Kgm} & H &= 60 \text{ cm} & b &= 20 \text{ cm} & \sigma_f &= 1400 \text{ Kg/cm}^2 \\ \sigma_c &= 34 \text{ Kg/cm}^2 & A_f &= 2,85 \text{ cm}^2 \end{aligned}$$

4-10-0 - Copertura Volumi tecnici -

4-10-1 - Solai -

Carico per nervatura $q = 650 \times 0,50 = 325 \text{ Kg/ml}$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 50 \text{ cm}$$

luce netta 2,70 m.

$$M = \frac{1}{2} 325 \times 2,70^2 = 1150 \text{ Kgm} \quad \sigma_c = 41 \text{ Kg/cm}^2 \quad A_f = 3,85 \text{ cm}^2$$

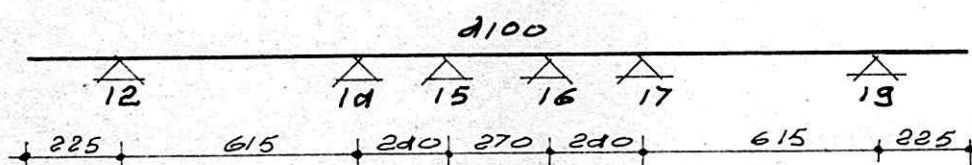
luce' netta 1,50 m.

$$M = \frac{1}{2} 325 \times 1,50^2 = 360 \text{ Kgm} \quad \sigma_c = 21 \text{ Kg/cm}^2 \quad A_f = 1,15 \text{ cm}^2$$

luce netta = $2,70 \times 1,05 = 2,85 \text{ m.}$

$$M = \frac{1}{12} 325 \times 2,83^2 = 200 \text{ Kgm} \quad \sigma_c < 15 \text{ Kg/cm}^2 \quad A_f = 1 \text{ cm}^2$$

4-10-2 Travi 12-14-15-16-17-19



- Armatura a flessione -

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

$$H = 80 \text{ cm}$$

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

Sezione 12; 12-14

$$M = \frac{1}{14} 4100 \times 6,40^2 = 12000 \text{ Kgm} \quad \sigma_c = 54 \text{ Kg/cm}^2 \quad A_f = 12,5 \text{ cm}^2$$

Sezione 14

$$M = \frac{1}{10} 4100 \times 6,40 = 16800 \text{ Kgm} \quad \sigma_c = 65 \text{ Kg/cm}^2 \quad A_f = 17,2 \text{ cm}^2$$

Sezione 15

$$M = \frac{1}{12} 4100 \times 2,70^2 = 2500 \text{ Kgm} \quad \sigma_c = 22 \text{ Kg/cm}^2 \quad A_f = 2,45 \text{ cm}^2$$

Sezione 15-16

$$M = \frac{1}{18} 4100 \times 2,70^2 = 1650 \text{ Kgm} \quad \sigma_c = 17 \text{ Kg/cm}^2 \quad A_f = 1,6 \text{ cm}^2$$

$$\text{Taglio massimo } T = 12600 \text{ Kg} \quad \tau_{\max} = \frac{12600}{30 \times 0,9 \times 78} = 5,9 \text{ Kg/cm}^2$$

$$30 \times 0,9 \times 78$$

5-0-0 EDIFICIO TIPO C

5-1-0 -Analisi di carico sulle travi-

5-1-1 COPERTURA PIANO TIPO

Trave 1-2-3; 6-7-8; 28-29-30; 33-34-35

Solaio 650 x 3,65/2 = 1200

tompagno = 850

sovracc. trave 365 x 0,70 = 250

p.p. 0,70 x 0,25 x 1,00 x 2500 = 450

2750 Kg/ml

trave 3-4; 5-6; 30-31; 32-33

Solaio 650 x 3,65/2 = 1200

sbalzo 800 x 1,15 = 900

tompagno = 850

sovracc. 365 x 0,70 = 250

p.p. = 450

3650 Kg/ml

Trave 9-10-11-12; 13-14-15-16; 36-37-38-39; 40-41-42-43

Solaio 650 x 3,65/2 = 1200

" " 650 x 3,90/2 = 1250

sovracc. 365 x 0,90 = 350

p.p. 0,90 x 1,00 x 0,25 x 2500 = 550

3350 Kg/ml

Trave 20-21-22-23; 24-25-26-27; 47-48-49-50; 51-52-53-54

Solaio 650 x 3,90/2 = 1250

sbalzo 650 x 1,00 = 650

tompagno = 850

sovracc. 365 x 1,00 = 365

p.p. 1,00 x 1,00 x 0,25 x 2500 = 635

3750 Kg/ml

Trave 4-5; 31-32; 17-19-23; 18-19-24; 44-46-50; 45-46-51

tompagno = 850

p.p. $0,30 \times 0,40 \times 1,00 \times 2500$ = 450

1200 Kg/ml

Trave 1-9-20; 8-16-27; 28-36-47; 35-43-54

tompagno = 850

sovracc. $0,50 \times 365$ = 200

p.p. $0,50 \times 0,25 \times 1,00 \times 2500$ = 350

1400 Kg/ml

Trave 4-12-17; 5-13-18; 31-39-44; 32-40-45

ramapa $850 \times 1,30$ = 1100

tompagno = 850

p.p. $0,30 \times 0,60 \times 1,00 \times 2500$ = 450

2400Kg/ml

5-1-2 COPERTURA PIANO TERRA

Trave 1-2-3; 6-7-8; 28-29-30; 33-34-35

Solaio $650 \times 4,25/2$ = 1400

tompagno = 850

p.p. $(0,25 \times 0,90 + 0,25 \times 0,40) \times 1,00 \times 2500$ = 800

3050 Kg/ml

Trave 3-4; 5-6; 30-31; 32-33

Solaio $650 \times 4,25/2$ = 1400

tompagno = 850

sbalzo $800 \times 1,15$ = 900

p.p. = 800

3950 Kg/ml

Trave 9-10-11-12; 13-14-15-16; 36-37-38-39; 40-41-42-43

Solaio $650 \times 4,25/2$ = 1400

" " $650 \times 4,65/2$ = 1500

p.p. = 800

3700 Kg/ml

Trave 20-21-22-23; 24-25-26-27; 47-48-49-50; 51-52-53-54

Solaio $650 \times 4,65/2$ = 1500

sbalzo $650 \times 1,40$ = 900

tompagno = 850

p.p. = 800

4050 Kg/ml

Trave 4-5; 31-32; 17-19-23; 18-19-24; 44-46-50; 45-46-51

tompagno = 850

p.p. $0,30 \times 0,40 \times 1,00 \times 2500$ = 450

1200 Kg/ml

Trave 1-9-20; 8-16-27-36-28-47; 35-43-54

tompagno = 850

p.p. = 800

1650 Kg/ml

Trave 4-12-17; 5-13-18; 31-39-44; 32-40-45

rampa $850 \times 1,30$ = 1100

tompagno = 850

p.p. $0,30 \times 0,60 \times 1,00 \times 2500$ = 450

2400 Kg/ml

5-2-1 - Analisi di carico sui pilastri -

COPERTURA PIANO TIPO

Pilastri 1-8-28-35

Trave 1-12 2750 x 3,35/2 = 4600

" " 1-9 1400 x 4,45/2 = 3100

p.p. 0,30 x 0,40 x 3,05 x 2500 = 900

8600 Kg

Pilastri 2-7-29-34

trave 2-1 = 4600

" " 2-3 2750 x 3,55/2 = 4900

p.p. = 900

12100 Kg

Pilastri 3-6-30-33

Trave 3-2 = 4900

" " 3-4 9650 x 2,00/2 = 3650

p.p. = 900

9450 Kg

Pilastri 4-5-31-32

trave 4-3 = 3650

" " 4-5 1200 x 3,00/2 = 1800

" " 4-12 2400 x 4,45/2 = 5350

p.p. = 900

11700 Kg

Pilastri 9-16-36-43

trave 9-1 = 3100

" " 9-20 1400 x 4,90/2 = 3450

" " 9-10 3350 x 3,35/2 = 5600

p.p. = 900

13050 Kg

Pilastri 10-15-37-42

trave 10-9 = 5600

" " 10-11 3350 x 3,55/2 = 5950

p.p. = 900

12450 Kg

Pilastri 11-14-38-41

trave 11-10 = 5950

" " 11-12 3350 x 2,00/2 = 3350

p.p. = 900

10200 Kg

Pilastri 12-13-39-40

trave 12-11 = 3350

" " 12-4 = 5350

" " 12-17 2400 x 1,80/2 = 2150

p.p. = 900

11750 Kg

Pilastri 17-18-44-45

trave 17-12 = 2150

" " 17-19 1200 x 1,60/2 = 950

aliquota solaio 650 x 2,40 x 0,60/2 = 450

p.p. 0,825 x 0,20 x 3,05 x 2500 = 1250

4800 Kg

Pilastri 19-46

trave 19-17 = 950

" " 19-18 = 950

" " 19-23 1200 x 1,50/2 = 900

" " 19-24 = 900

p.p. 1,90 x 0,20 x 3,05 x 2500 = 2900

6600 Kg

Pilastri 20-27-47-54

trave 20-9

= 3450

" " 20-21 3750x 2,65/2

= 5000

p.p.

= 900

9350 Kg

Pilastri 21-26-48-53

trave 21-20

= 5000

" " 21-22 3750 x 4,10/2

= 7700

p.p.

= 900

13600 Kg

Pilastri 22-25-49-52

trave 22-21

= 7700

" " 22-23 3750 x 2,70/2

= 5050

p.p.

= 900

13650 Kg

Pilastri 23-24-50-51

trave 23-22

= 5050

" " 23-19

= 900

p.p.

= 900

6850 Kg**5-2-2** COPERTURA PIANO TERRA

Pilastri 1-8-28-35

trave 1-2 3050 x 3,35/2

= 5100

" " 1-9 1650 x 4,45/2

= 3650

p.p. 0,30 x 0,40 x 4,20 x 2500

= 1250

10000 Kg

Pilastri 2-7-29-34

trave 2-1

= 5100

" " 2-3 3050 x 3,55/2

= 5400

p.p.

= 1250

11750 Kg

Pilastri 3-6-30-33

trave 2-3

= 5400

" " 3-4 3950 x 2,00/2

= 3950

p.p.

= 1250

10600 Kg

Pilastri 4-5-31-32

trave 4-3

= 3950

" " 4-5 1200 x 3,00/2

= 1800

" " 4-12 2400 x 4,45/2

= 5350

p.p.

= 1250

12350 Kg

Pilastri 9-16-36-43

trave 9-1

= 3650

" " 9-20 1650 x 4,90/2

= 4050

" " 9-10 3700 x 3,35/2

= 6200

p.p.

= 1250

15150 Kg

Pilastri 10-15-37-42

trave 10-9

= 6200

" " 10-11 3700 x 3,55/2

= 6550

p.p.

= 1250

14000 Kg

Pilastri 11-14-38-41

trave 11-10	= 6550
" " 11-12 3700 x 2,00/2	= 3700
p.p.	= 1250
	11500 Kg

Pilastri 12-13-39-40

trave 12-11	= 3700
" " 12-4	= 5350
" " 12-17 2400 x 1,80/2	= 2150
p.p.	= 1250
	12450 Kg

Pilastri 17-18-44-45

trave 17-12	= 2150
" " 17-19 1200 x 1,60/2	= 950
aliquota solaio 6,50 x 2,40 x 0,60/2	= 450
p.p. 0,825 x 0,20 x 4,20 x 2500	= 1750
	5300 Kg

Pilastri 19-46

trave 19-17	= 950
" " 19-18	= 950
" " 19-23 1200 x 1,50/2	= 900
" " 19-24	= 900
p.p. 1,90 x 0,20 x 4,20 x 2500	= 4000
	7700 Kg

Pilastri 20-27-47-54

trave 20-9	= 4050
" " 20-21 4050 x 2,65/2	= 5350
p.p.	= 1250
	10650 Kg

Pilastri 21-26-48-53

trave 21-20

= 5350

" " 21-22 4050 x 4,10/2

= 8300

p.p.

= 1250

14900 Kg

Pilastri 22-25-49-52

trave 22-21

= 8300

" " 22-23 4050 x 2,70/2

= 5450

p.p.

= 1250

15000 Kg

Pilastri 23-24-50-51

trave 23-22

= 5450

" " 23-19

= 900

p.p.

= 1250

7600 Kg

5-3-0 - Verifica ed armatura pilastri -

5-3-1 Carichi al piede dei pilastri

Pilastri	Cop. IV	Cop. III	Cop. II	Cop. I	Cop. portico fondazione
1,8,28,35	8600	17200	25800	34400	44400
2,7,29,34	12100	24200	36300	48400	60150
3,6,30,33	9450	18900	28350	37800	48400
4,5,31,32	11700	23400	35100	46800	59150
9,16,36,43	13050	26100	39150	52200	67350
10,15,37,42	12450	24900	37350	49800	63800
11,14,38,41	10200	20400	30600	40800	52300
12,13,39,40	11750	23500	35250	47000	59450
17,18,44,45	4800	9600	14400	19200	24500
19,46	6600	13200	19800	26400	34100
20,27,47,54	9350	18700	28050	37400	48050
21,26,48,53	13600	27200	40800	54400	69300
22,25,49,52	13650	27300	40950	54600	69600
23,24,50,51	6850	13700	20550	27400	35000

5-3-2

Sezione ed armatura pilastri fondazione e portico

N°	P tn	A' c cmq	Af cmq	Armatura	Sezione cm x cm	A' ci cmq	σ c Kg/cmq
1,8,28,35	44	900	7,2	4Ø14 + 4Ø12	30 x 40	1307	33
2,7,29,34	60	1250	10	4Ø14 + 6Ø12	30 x 50	1629	36
3,6,30,33	48	1000	8	4Ø14 + 4Ø12	30 x 40	1307	36
4,5,31,32	59	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	45
9,16,36,43	67	1350	10,8	4Ø14 + 6Ø12	30 x 50	1629	41
10,15,37,42	64	1300	10,4	4Ø14 + 6Ø12	30 x 50	1629	39
11,14,38,41	52	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	39
12,13,39,40	59	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	45
17,18,44,45	24	500	4	14 Ø 10	20 x 82 ²	1759	13
19,46	34	700	5,6	24 Ø 10	20 x 190	3988	<10
20,27,47,54	48	1000	8	4Ø14 + 4Ø12	30 x 40	1307	36
21,26,48,53	69	1400	11,2	4Ø14 + 6Ø12	30 x 50	1629	42
22,25,49,52	69	1400	11,2	4Ø14 + 6Ø12	30 x 50	1629	42
23,24,50,51	35	700	5,6	4Ø14 + 4Ø12	30 x 40	1307	26

Staffe Ø 8^m / 15^m

5-3-3 Sezione ed armatura pilastri piano I e II

N°	P tn	A' c cmq	A f cmq	Armatura	Sezione cm x cm	A ci cmq	σ c Kg/cmq
1,8,28,35	34	700	5,6	8 Ø 12	30 x 30	990	34
2,7,29,34	48	1000	8	4Ø14 + 4Ø12	30 x 40	1307	36
3,6,30,33	38	800	6,4	8 Ø 12	30 x 30	990	38
4,5,31,32	46	950	7,5	8 Ø 12	30 x 30	990	45
9,16,36,43	52	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	39
10,15,37,42	50	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	38
11,14,38,41	41	850	6,8	8 Ø 12	30 x 30	990	41
12,13,39,40	47	1000	8	8 Ø 12	30 x 30	990	46
17,18,44,45	19	400	3,2	14 Ø 8	20 x 82 ²	1759	<10
19,46	26	550	4,4	24 Ø 8	20 x 190	3988	<10
20,27,47,54	37	750	6	8 Ø 12	30 x 30	990	37
21,26,48,53	54	1100	8,8	4Ø14 + 4Ø12	30 x 40	1307	41
22,25,49,52	54	1100	8,8	4Ø14 + 4Ø12	30 x 40	1307	41
23,24,50,51	27	550	4,4	8 Ø 12	30 x 30	990	27

Staffe Ø 6" / 15"

5-3-4 Sezione ed armatura pilastri piano III e IV

N°	P	A' c	A f	Armatura	Sezione
	tn	cmq	cmq		cm x cm
1,8,28,35	17	350	2,8	8 Ø 10	30 x 30
2,7,29,34	24	500	4,0	4Ø12 + 4Ø10	30 x 30
3,6,30,30	19	400	3,2	8 Ø 10	30 x 30
4,5,31,32	23	450	3,6	8 Ø 10	30 x 30
9,16,36,43	26	550	4,4	4Ø12 + 4Ø10	30 x 30
10,15,37,42	25	500	4	4Ø12 + 4Ø10	30 x 30
11,14,38,41	20	400	3,2	8 Ø 10	30 x 30
12,13,39,40	23	450	3,6	8 Ø 10	30 x 30
17,18,44,45	10	200	1,6	14 Ø 8	20 x 82 ²
19,46	13	250	2	24 Ø 8	20 x 190
20,27,47,54	19	400	3,2	8 Ø 10	30 x 30
21,26,48,53	27	550	4,4	4Ø12 + 4Ø10	30 x 30
22,25,49,52	27	550	4,4	4Ø12 + 4Ø10	30 x 30
23,24,50,51	14	300	2,4	8 Ø 10	30 x 30

Staffe Ø 6" / 15"

5-4-0 Fondazioni

5-4-1 -Analisi di carico sul terreno-

Pilastri 1432000

travi 205000

1637000 Kg

5-4-2 -Tensioni sul terreno-

$$\sigma_t = \frac{1637000}{1220900} = 1,34 \text{ Kg/cm}^2$$

1220900

5-4-3 Travi 1-2-3-4-5-6-7-8; 9-10-11-12-13-14-15-16; 20-21-22-23-24-25
26-27-28-29-30-31-32-33-34-35; 36-37-38-39-40-41-42-43; 47-48-49-50
51-52-53-54

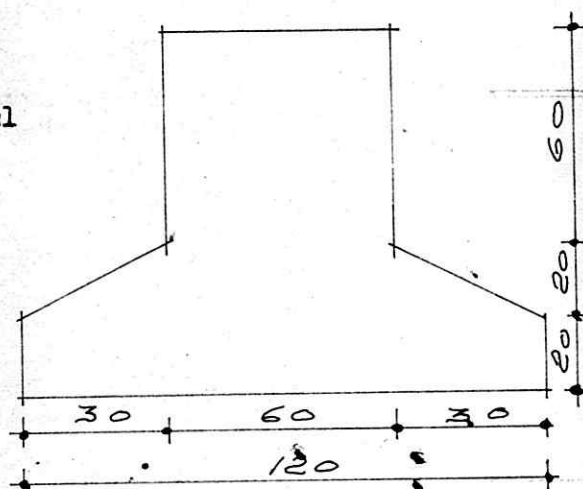
Carico sulla trave

$$q = \frac{1432000}{1220900} \times 100 \times 120 = 14050 \text{ Kg/ml}$$

1220900

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 14050 \times 3,80^2 = 16900 \text{ Kgm} \quad H = 100'' \quad b = 60''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 33 \text{ Kg/cm}^2 \quad A_f = 13,2 \text{ cm}^2$$

$$M_{camp} = 16900 \text{ Kgm} \quad H = 100'' \quad b = 120''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 23 \text{ Kg/cm}^2 \quad A_f = 13,2 \text{ cm}^2$$

5-4-4 Travi 1-9-20; 8-16-27; 28-36-47; 35-43-54

Carico sulla trave

$$q = \frac{1432000}{1220900} \times 60 \times 100 = 7050 \text{ Kg/ml}$$

1220900

-Caratteristiche geometriche-

Vedi disegno

-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 7050 \times 4,90^2 = 14100 \text{ Kgm}$$

$$H = 100'' \quad b = 30''$$

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

$$\sigma_c = 44 \text{ Kg/cm}^2$$

$$A_f = 10,9 \text{ cm}^2$$

$$M_{camp} = 14100 \text{ Kgm}$$

$$H = 100'' \quad b = 60''$$

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

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

$$A_f = 10,9 \text{ cm}^2$$

5-4-5

Travi 2-10-21; 3-11-22; 4-12-17-19-23; 5-13-18-19-24; 6-14-25;
7-15-26-29-37-48; 30-38-49; 31-39-44-46-50; 32-40-45-46-51; 33-41
52; 34-42-53

Carico sulla trave

$$q = \frac{1432000}{100} \times 100 \times 90 = 10550 \text{ Kg/ml}$$

$$1220900$$

-Caratteristiche geometriche-

Vedi disegno

-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 10550 \times 4,90^2 = 21100 \text{ Kgm} \quad H = 100'' \quad b = 30''$$

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

$$\sigma_c = 56 \text{ Kg/cm}^2$$

$$A_f = 17 \text{ cm}^2$$

$$M_{camp} = 21100 \text{ Kgm}$$

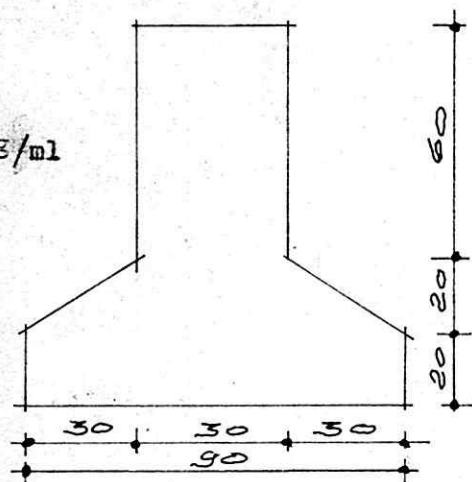
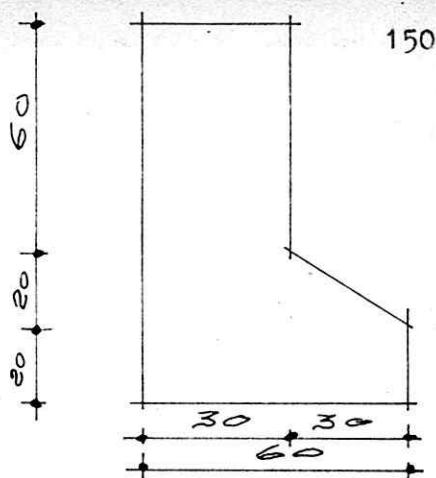
$$H = 100''$$

$$b = 90''$$

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

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

$$A_f = 15,5 \text{ cm}^2$$



5-5-0 - Vano Scala -

5-5-1 Calcolo del gradino

Analisi di carico

$$\text{p.p.} \quad \frac{0,235 + 0,07}{2} \times 0,30 \times 1,00 \times 2500 = 115$$

sovracc. perm. (intonaco, grado

$$\text{sottogradio e massetto}) = 150 \times 0,30 = 45$$

$$\text{sovracc. accid.} = 400 \times 0,30 = 120$$

280 Kg/ ml

$$M = \frac{1}{2} \times 280 \times 1,20^2 = 200 \text{ Kgm} \quad H = \frac{0,235 \times 0,07}{2} = 0,15 \quad b = 30 \text{ cm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 35 \text{ Kg/cm}^2 \quad A_f = 1,10 \text{ cm}^2$$

5-5-2 Calcolo dei pianerottoliSolaio in latero-cemento $H = 16'' + 4'' \quad i = 50''$

Analisi di carico

$$\text{p.p.} = 140$$

$$\text{soletta} \quad 0,04 \times 1,00 \times 1,00 \times 2500 = 100$$

$$\text{sovracc. accid. (intonaco, massetto, pavimento)} = 110$$

$$\text{sovracc. perm.} = 400$$

750 Kg/cm²

$$M_{inc} = \frac{1}{12} \times 750 \times 2,70^2 = 455 \text{ Kgm} \quad H = 20'' \quad b = 100''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 22 \text{ Kg/cm}^2 \quad A_f = 0,96 \text{ cm}^2 / 50''$$

$$M_{camp} = \frac{1}{14} \times 750 \times 2,70^2 = 390 \text{ Kgm} \quad H = 20'' \quad b = 100''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 20 \text{ Kg/cm}^2 \quad A_f = 0,82 \text{ cm}^2 / 50''$$

5-5-3 Calcolo delle travi a ginocchio

Analisi di carico

Rampa 850 x 1,20

= 1100

tompagno

= 400

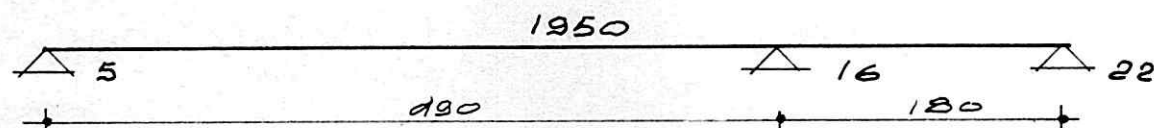
p.p. 0,30 x 0,60 x 1,00 x 2500

= 450

1950 Kg/ml

Verifica ed armatura a flessione

Schema di calcolo



$$M_5 = M_{5-16} = \frac{1}{14} \cdot 1950 \times 4,90^2 = 3350 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 35 \text{ Kg/cm}^2 \quad A_f = 4,4 \text{ cm}^2$$

$$M_{16} = \frac{1}{10} \cdot 1950 \times 4,90^2 = 4700 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 43 \text{ Kg/cm}^2 \quad A_f = 6,3 \text{ cm}^2$$

$$M_{16-22} = \frac{1}{8} \cdot 1950 \times 1,80^2 - 4700 = -3900 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 39 \text{ Kg/cm}^2 \quad A_f = 5,3 \text{ cm}^2$$

$$M_{22} = \frac{1}{12} \cdot 1950 \times 1,80^2 = 550 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c < 15 \text{ Kg/cm}^2 \quad A_f = 1,5 \text{ cm}^2$$

Verifica a torsione della trave 5-16

$$m_i = \frac{1}{2} \cdot 850 \times 1,20^2 = 612 \text{ Kgm} \quad m_t = 612 + 1020 \times 0,15 = 765$$

$$t_i = 850 \times 1,20 = 1020 \text{ Kgm} \quad M_t = 765 \times 4,90 = 1875 \text{ Kgm}$$

$$\gamma = 3 + \frac{2,6}{0,45 + \frac{60}{30}} = 4,061$$

$$\tau = \frac{4,061 \times 187500}{60 \times 30^2} = 14,1 \text{ Kg/cm}^2$$

- Armatura a torsione -

$$F = 55 \times 25 = 1375 \text{ cm}^2$$

$$V = (55 + 25) \times 2 = 160 \text{ cm}$$

$$\Sigma f_l = \frac{187500}{2 \times 1400 \times 1375} = 160 = 7,8 \text{ cm}^2$$

$$\Sigma f_s = \frac{187500}{2 \times 1400 \times 1375} = 0,0487 \text{ cm}^2/\text{cm}$$

5-5-4 Calcolo della trave al pianerottolo di riposo

$$M = \frac{1}{12} 1200 \times 3,00^2 = 900 \text{ Kgm} \quad H = 40'' \quad b = 20''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 34 \text{ Kg/cm}^2 \quad A_f = 1,8 \text{ cm}^2$$

5-6-0

BALCONI

$$q = 800 \text{ Kg/mq} \quad H = 20 + 5 \text{ cm} \quad b = 100 \text{ cm}$$

$$M = \frac{1}{2} 800 \times 1,05^2 = 440 \text{ Kgm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 17 \text{ Kg/cm}^2 \quad A_f = 1,6 \text{ cm}^2/\text{ml}$$

$$i = 50 \text{ cm} \quad A_f = 0,8 \text{ cm}^2$$

$$M = \frac{1}{2} 800 \times 1,15^2 = 530 \text{ Kgm}$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 18 \text{ Kg/cm}^2 \quad A_f = 1,7 \text{ cm}^2/\text{ml}$$

$$i = 50 \text{ cm} \quad A_f = 0,85 \text{ cm}^2/50$$

5-7-0 - SOLAI -

5-7-1 - SOLAIO COPERTURA PIANO TERRA -Carico per nervatura $650 \times 0,30 = 325 \text{ Kg/ml}$ $\sigma_f = 1400 \text{ Kg/cmq}$ $b = 500$ luce netta $3,90 \times 1,05 = 4,10\text{m.}$

incastro	$M = \frac{1}{12} 325 \times 4,10^2 = 455 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 24 \text{Kg/cmq}$	$A_f = 1,47 \text{cmq}$
		$H=18\text{cm}$	$\sigma_c = 32 \text{Kg/cmq}$	$A_f = 1,93 \text{cmq}$

campata	$M = \frac{1}{14} 325 \times 4,10^2 = 390 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 22 \text{Kg/cmq}$	$A_f = 1,25 \text{cmq}$
		$H=18\text{cm}$	$\sigma_c = 30 \text{Kg/cmq}$	$A_f = 1,68 \text{cmq}$

luce netta $3,63 \times 1,05 = 3,80 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 3,80^2 = 390 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 22 \text{Kg/cmq}$	$A_f = 1,25 \text{cmq}$
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campata	$M = \frac{1}{14} 325 \times 3,80^2 = 335 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 20 \text{Kg/cmq}$	$A_f = 1,05 \text{cmq}$
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luce netta $2,40 \times 1,05 = 2,50 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 2,50^2 = 170 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c < 15 \text{Kg/cmq}$	$A_f = 0,8 \text{cmq}$
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campata	$M = \frac{1}{14} 325 \times 2,50^2 = 145 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c < 15 \text{Kg/cmq}$	$A_f = 0,8 \text{cmq}$
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luce netta $1,00\text{m.}$

$M = \frac{1}{2} 325 \times 1,00^2 \times 425 \times 1,00 = 585 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 28 \text{Kg/cmq}$	$A_f = 1,92 \text{cmq}$
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5-7-2

- SOLAI PIANO TIPO-luce netta $3,90 \times 1,05 = 4,10\text{m.}$

incastro	$M = \frac{1}{12} 325 \times 4,10^2 = 455 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 24 \text{Kg/cmq}$	$A_f = 1,47 \text{cmq}$
		$H=18\text{cm}$	$\sigma_c = 32 \text{Kg/cmq}$	$A_f = 1,93 \text{cmq}$

campata	$M = \frac{1}{14} 325 \times 4,10^2 = 390 \text{Kgm}$	$H=25\text{cm}$	$\sigma_c = 22 \text{Kg/cmq}$	$A_f = 1,25 \text{cmq}$
		$H=18\text{cm}$	$\sigma_c = 30 \text{Kg/cmq}$	$A_f = 1,68 \text{cmq}$

luce netta 3,63 x 1,05 = 3,80m.

incastro $M = \frac{1}{12} 325 \times 3,80^2 = 390 \text{Kgm}$ $H = 25 \text{cm}$ $\sigma_c = 22 \text{Kg/cmq}$ $A_f = 1,25 \text{cmq}$

campata $M = \frac{1}{14} 325 \times 3,80^2 = 335 \text{Kgm}$ $H = 25 \text{cm}$ $\sigma_c = 20 \text{Kg/cmq}$ $A_f = 1,05 \text{cmq}$

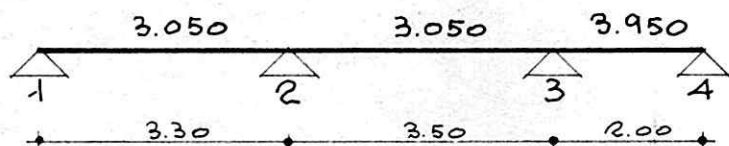
luce netta 2,40 x 1,05 = 250 m.

incastro $M = \frac{1}{12} 325 \times 250^2 = 170 \text{Kgm}$ $H = 25 \text{cm}$ $\sigma_c < 15 \text{Kg/cmq}$ $A_f = 0,8 \text{cmq}$

campata $M = \frac{1}{14} 325 \times 250^2 = 145 \text{Kgm}$ $H = 25 \text{cm}$ $\sigma_c < 15 \text{Kg/cmq}$ $A_f = 0,8 \text{cmq}$

5-8-0

5-8-1 Travi 1-2-3-4; 5-6-7-8;
28-29-30-31; 32-33-34-35



- Armatura a flessione -

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

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

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

Sezione 1; 1-2

$$M = \frac{1}{14} 3050 \times 3,30^2 = 2400 \text{ Kgm}$$

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

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

Sezione 2; 3

$$M = \frac{1}{12} 3050 \times 3,50^2 = 3100 \text{ Kgm}$$

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

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

Sezione 2-3

$$M = \frac{1}{16} 3050 \times 3,50^2 = 2350 \text{ Kgm}$$

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

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

Sezione 3-4; 4

$$M = \frac{1}{14} 3950 \times 2^2 = 1100 \text{ Kgm}$$

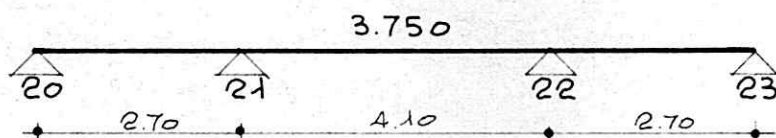
$$\sigma_c < 15 \text{ Kg/cm}^2$$

$$A_f = 1,5 \text{ cm}^2$$

$$\text{Taglio massimo } T = 5350 \text{ Kg}$$

$$\tau_{\max} = \frac{5350}{25 \times 0,9 \times 93} = 2,5 \text{ Kg/cm}^2$$

5-8-2 Travi 20-21-22-23; 24-25-26-27;
47-48-49-50; 51-52-53-54;



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = \frac{25}{20} \text{ cm} \quad b = 100 \text{ cm}$$

Sezione 20, 20-21, 22-23, 23

$$M = \frac{1}{14} 3750 \times 2,70^2 = 1900 \text{ Kgm} \quad \sigma_c = 37 \text{ Kg/cm}^2 \quad A_f = 6,3 \text{ cm}^2$$

Sezione 21, 22

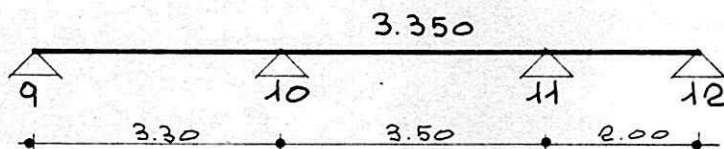
$$M = \frac{1}{12} 3750 \times 4,10^2 = 5200 \text{ Kgm} \quad \begin{array}{l} H = 25 \text{ cm} \quad \sigma = 67 \text{ Kg/cm}^2 \\ H = 20 \text{ " } \quad \sigma_c = 93 \text{ " " } \end{array} \quad \begin{array}{l} A_f = 18 \text{ cm}^2 \\ A_f = 24 \text{ " } \end{array}$$

Sezione 21-22

$$M = \frac{1}{16} 3750 \times 4,10^2 = 3900 \text{ Kgm} \quad \begin{array}{l} H = 25 \text{ cm} \quad \sigma = 57 \text{ Kg/cm}^2 \\ H = 20 \text{ " } \quad \sigma_c = 72 \text{ " " } \end{array} \quad \begin{array}{l} A_f = 13,5 \text{ cm}^2 \\ A_f = 16,6 \text{ " } \end{array}$$

$$\text{Taglio massimo } T = 7700 \text{ Kg} \quad \tau_{\max} = \frac{7700}{23 \times 0,9 \times 100} = 3,7 \text{ Kg/cm}^2$$

5-8-3 Travi 9-10-11-12, 13-14-15-16;
36-37-38-39, 40-41-42-43;

**- Armatura a flessione -**

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 90 \text{ cm}$$

Sezione 9, 9-10

$$M = \frac{1}{14} 3350 \times 3,30^2 = 2600 \text{ Kgm} \quad \sigma_c = 47 \text{ Kg/cm}^2 \quad A_f = 8,8 \text{ cm}^2$$

Sezione 10, 11

$$M = \frac{1}{12} 3350 \times 3,50^2 = 3400 \text{ Kgm} \quad \sigma_c = 55 \text{ Kg/cm}^2 \quad A_f = 11,5 \text{ cm}^2$$

Sezione 10-11

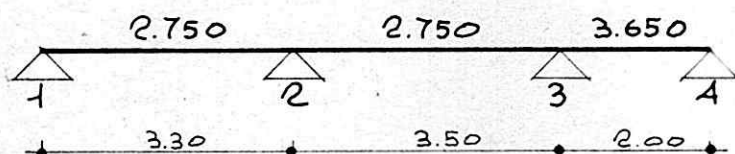
$$M = \frac{1}{16} 3350 \times 3,50^2 = 2550 \text{ Kgm} \quad \sigma_c = 47 \text{ Kg/cm}^2 \quad A_f = 8,8 \text{ cm}^2$$

Sezione 11-12; 12

$$M = \frac{1}{14} 3350 \times 2,00^2 = 950 \text{ Kgm} \quad \sigma_c = 27 \text{ Kg/cm}^2 \quad A_f = 3,2 \text{ cm}^2$$

$$\text{Taglio massimo } T = 5850 \text{ kg} \quad \tau_{\max} = \frac{5850}{23 \times 0,9 \times 90} = 3,1 \text{ Kg/cm}^2$$

5-8-4 Travi 1-2-3-4; 5-6-7-8; In variante al piano tipo rispetto al-
28-29-30-31; 32-33-34-35; la copertura piano terra.



- Armatura a flessione -

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad H = 25 \text{ cm} \quad b = 70 \text{ cm}$$

Sezione 1; 1-2

$$M = \frac{1}{14} 2750 \times 3,30^2 = 2150 \text{ Kgm} \quad \sigma_c = 49 \text{ Kg/cm}^2 \quad A_f = 7,7 \text{ cm}^2$$

Sezione 2; 3

$$M = \frac{1}{12} 2750 \times 3,50^2 = 2800 \text{ Kgm} \quad \sigma_c = 58 \text{ Kg/cm}^2 \quad A_f = 9,7 \text{ cm}^2$$

Sezione 2-3

$$M = \frac{1}{16} 2750 \times 3,50^2 = 2100 \text{ Kgm} \quad \sigma_c = 49 \text{ Kg/cm}^2 \quad A_f = 7,7 \text{ cm}^2$$

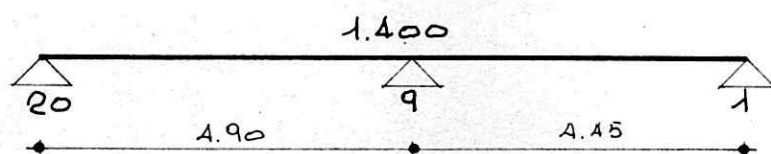
Sezione 3-4; 4

$$M = \frac{1}{14} 2750 \times 3,50^2 = 2400 \text{ Kgm} \quad \sigma_c = 52 \text{ Kg/cm}^2 \quad A_f = 8,2 \text{ cm}^2$$

Taglio massimo $T = 4800 \text{ kg}$

$$\tau_{\max} = \frac{4800}{23 \times 0,9 \times 70} = 3,3 \text{ Kg/cm}^2$$

5-8-5 Travi 1-9-20; 8-16-27;
28-36-47; 33-43-54;



- Armatura a flessione -

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

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

$$b = 50 \text{ cm}$$

Sezione 20; 20-9

$$M = \frac{1}{14} 1400 \times 4,90^2 = 2400 \text{ Kgm}$$

$$\sigma_c = 64 \text{ Kg/cm}^2$$

$$A_f = 8,4 \text{ cm}^2$$

Sezione 9

$$M = \frac{1}{10} 1400 \times 4,90^2 = 3350 \text{ Kgm}$$

$$\sigma_c = 79 \text{ Kg/cm}^2$$

$$A_f = 11,8 \text{ cm}^2$$

Sezione 9-1; 1

$$M = \frac{1}{14} 1400 \times 4,45^2 = 2000 \text{ Kgm}$$

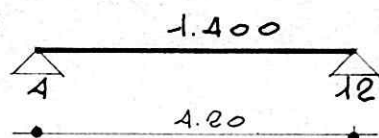
$$\sigma_c = 57 \text{ Kg/cm}^2$$

$$A_f = 6,8 \text{ cm}^2$$

Taglio massimo $T = 3450 \text{ Kg}$

$$\tau_{\max} = \frac{3450}{23 \times 0,9 \times 50} = 3,3 \text{ Kg/cm}^2$$

5-8-6 Travi 4-12; 5-13;
31-39; 32-40;



- Armatura a flessione -

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

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

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

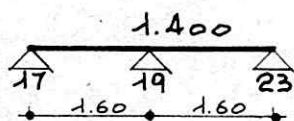
Sezione 15; 15-24; 24

$$M = \frac{1}{12} 1400 \times 4,20^2 = 2050 \text{ Kgm}$$

$$\sigma_c = 80 \text{ Kg/cm}^2$$

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

5-8-7 Travi 17-19-23; 44-46-50;
18-19-24; 45-46-51;

**- Armatura a flessione -**

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

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

$$b = 20 \text{ cm}$$

Sezione 4; 4-9; 9

$$M = \frac{1}{12} 1400 \times 1,60^2 = 300 \text{ Kgm}$$

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

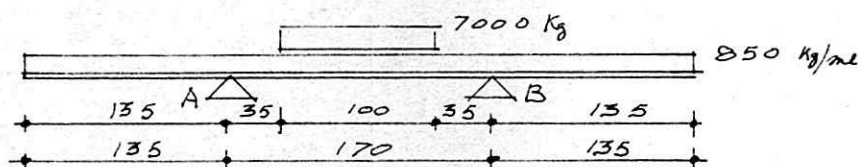
$$A_f = 1,5 \text{ cm}^2$$

5-9-0 - Calcolo della soletta ascensore -

5-9-1 Analisi di carico

p.p.	$0,25 \times 1,00 \times 1,00 \times 2500$	= 625
sovracc. perm.		= 75
sovracc. accid.		= 150
		850 Kg/mq
macchinario		= 6000
basamento		= 1000
		7000 Kg

5-9-2 Condizioni di carico



$$M_A = M_B = 770 \text{ Kgm}$$

$$M_{AB} = \frac{1}{8} 850 \times 1,70^2 + \frac{1}{8} 7000 \times 1,00 \times 1,70 \left(1 + 2 \frac{0,35}{1,70}\right) - 770 = 1630 \text{ Kgm}$$

Armando simmetricamente verifichiamo il momento massimo

$$M = 1630 \text{ Kgm} \quad H = 25 \text{ cm} \quad h = 23 \text{ cm} \quad h' = 2 \text{ cm} \quad b = 100 \text{ cm}$$

$$A_f = A'_f = 10,78 \text{ cmq} \quad (1 + 1 \phi 14/15) \quad A_f^x = 21,56 \text{ cmq} \quad h^x = 12,5 \text{ cm}$$

$$\gamma = \frac{A_f}{A_f^x} = 1 \quad x = 5,47 \text{ cm}$$

$$\sigma_c = 19 \text{ Kg/cmq} \quad \sigma_f = 608 \text{ Kg/cmq}$$

5-9-3 Coefficienti di sicurezza

nel calcestruzzo $\xi_c = \frac{180}{19} = 9,4$

nel ferro $\xi_f = \frac{4200}{608} = 6,9$

5-9-4

Travi portasolette

Analisi di carico

Travi 28-23

soletta $850 \times 1,50/2 \times 7000/2$ = 4150sbalzo $850 \times 1,25$ = 1050

tompagno = 800

p.p. $0,20 \times 0,60 \times 1,00 \times 2500$ = 500

6500 Kg/ml

Travi 23-21

soletta $850 \times 1,50/2$ = 650sbalzo $850 \times 1,25$ = 1050

tompagno = 800

p.p. = 400

2900 Kg/ml

Armatura a flessione

 $M_{\max} = 2100 \text{ Kgm}$ $H = 60 \text{ cm}$ $b = 20 \text{ cm}$ $\sigma_f = 1400 \text{ Kg/cmq}$ $\sigma_c = 34 \text{ Kg/cmq}$ $A_f = 2,85 \text{ cmq}$