

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

agg

N°

agg

agg

RELAZIONE DI CALCOLO
PRIMA 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

glu 3eq

Il lotto comprende 4 edifici per un totale di 11 vani scala. L'edificio A, composto da due vani scala, si articola in piano terra a portico, 5 piani abitazione, un piano per cantinole e volumi tecnici; l'edificio A bis è del tutto simile al precedente; l'edificio B, composto da 5 vani scala, si articola in piano terreno a portico, 4 piani abitazione ed un piano per cantinole e volumi tecnici; l'edificio C, composto da due vani scala, si articola in piano terra, parte a portico e parte destinato a cantinole, 4 piani abitazione e volumi tecnici.

Le strutture sono del tipo tradizionale in c.a. costituite da pilastri, travi a spessore di solaio o di calata, solai in latero-cemento, sporti in latero-cemento, scale in c.a. a sbalzo da travi a ginocchio, murature perimetrali a doppia foderà.

I solai di calpestio e di copertura sono previsti dell'altezza di cm. 25, compresa la soletta collaborante di cm. 5, con interasse di cm. 50, mentre per le zone ribassate dei servizi l'altezza di viene di cm. 18.

Le fondazioni saranno del tipo diretto a travi rovesce, e si assume come tensione del terreno $\sigma_t = 1,5 \text{ Kg/cm}^2$.

Si è previsto l'uso di calcestruzzo classe R 250 dosato a 3 ql/mc di cemento tipo 425 (ex 730), ferro tondo omogeneo A_q 42-50.

Le sollecitazioni massime nel calcestruzzo armato sono:

- 70 Kg/cm². per le strutture sollecitate a flessione e pressoflessione.
- 45 Kg/cm². per le strutture sollecitate a pressione semplice.

Nelle sezioni inflesse ove le sollecitazioni sono maggiori di quelle predette, tenendo conto della doppia armatura e della riduzione a filo pilastri del momento, esse si riducono nei limiti suddetti. I calcoli sono stati eseguiti coll'ausilio del regolo calcolatore.

- ANALISI GENERALE DEI CARICHI -**S O L A I**

Solaio tipo in latero-cemento $H = 20'' + 5''$
interasse = 50 cm.

p.p.		= 160
soletta	$0,05 \times 1,00 \times 1,00 \times 2500$	= 125
sovracc. perm. (intonaco, massetto, pavimenti, aliquota tramezzi)		= 65
sovracc. accid.		= 200
		650 Kg/mq

M U R A T U R E

Muratura di compagno a doppia fodera, con fodera esterna in mattoni forati da 13" e fodera interna in tufelle da 10"

- fodera esterna	$105 \times 3,05$	= 330
- fodera interna	$0,10 \times 1,00 \times 3,05 \times 1700$	= 520
		850 Kg/ml.

B A L C O N I

A sbalzo in latero-cemento	$H = 20'' + 5''$	$i = 50''$
p.p.		= 160
soletta		= 125
sovracc. perm. (intonaco, massetto, masso pendio, pavimento)		= 115
sovracc. accid.		= 400
		800 Kg/mq

S C A L E

Scale in c.a. a sbalzo da trave a ginocchio

p.p. $\frac{20 + 4 \times 1,00 \times 1,00 \times 2500}{2}$ = 300sovracc. perm. (intonaco, massetto,
grado e sottogrado.) = 150

sovracc. accid. = 400

850 Kg/mqSOVRACCARICHI TRAVI PIATTE

sovracc. perm. = 165

sovracc. accid. = 200

365 Kg/mq.

1-0-0 EDIFICIO TIPO A - A bis.

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

1-1-1 COPERTURA

Trave S-12-14-15-16-17-19-S; S-45-47-48-49-50-52-S

Solaio 650 x 270 = 1750

" " 650 x 270 = 1750

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

4100 Kg/ml

Trave 21-22; 54-55

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 21-29; 22-29; 54-61; 55-62

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

1-1-2 COPERTURA PIANO TIPO

Trave 1-2-3-4; 7-8-9-10; 34-35-36-37;

- 40-41-42-43 ;

Solaio 650 x 3,85/2 = 1250

Sovracc. 365 x 0,70 = 250

Tompagno = 850

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

2800 Kg/ml

Trave 4-5; 6-7; 37-38; 39-40;	
Solaio 650 x 3,85/2	= 1250
Sbalzo 800 x 1,15	= 900
Sovracc. 365 x 0,70	= 250
Tompagno	= 850
p.p. 0,70 x 0,25 x 1,00 x 2500	= 450
	3700 Kg/ml

Trave 11-12-13-14-15-; 16-17-18-19-20;	
44-45-46-47-48; 49-50-51-52-53;	
Solaio 650 x 3,85/2	= 1250
Solaio 650 x 4,15/2	= 1350
Sovracc. 365 x 0,90	= 350
p.p. 0,90 x 0,25 x 1,00 x 2500	= 550
	3500 Kg/ml

Trave 24-25-26-27-; 30-31-32-33;	
57-58-59-60; 63-64-65-66;	
Solaio 650 x 4,15/2	= 1350
Sovracc. 365 x 0,70	= 250
Tompagno	= 850
p.p. 0,70 x 0,25 x 1,00 x 2500	= 450
	2900 Kg/ml

Trave 27-28; 29-30; 60-61; 62-63	
Solaio 650 x 4,15/2	= 1350
Sbalzo 800 x 1,05	= 850
Sovracc. 365 x 0,70	= 250
Tompagno	= 850
p.p. 0,70 x 0,25 x 1,00 x 2500	= 450
	3750 Kg/ml

Trave 5-6; 38-39; 21-23; 22-23; 54-56; 55-56;
23-28; 23-29; 56-61; 56-62;

Tompagno

p.p. 0,30 x 0,40 x 1,00 x 2500

= 850

= 350

1200 Kg/ml

Trave 5-15+21; 6-16-22;

38-48-54; 39-49-55

Rampa 850 x 1,30

Tompagno

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

= 1100

= 850

= 450

2400 Kg/ml

Trave 1-11-24; 10-20-30; 34-44-57;

43-53-66

Tompagno

Sovracc. 365 x 0,50

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

= 850

= 200

= 350

1400 Kg/ml

1-2-0

- Analisi di carico sui pilastri -

1-2-1 COPERTURA

Pilastri 12; 19; 45; 52;

Sbalzo 4100 x 2,10 = 8600

Trave 12-14 4100 x 5,55/2 = 11350

p.p. 0,30 x 0,30 x 2,70 x 2500 = 600

20550 Kg

Pilastri 14; 17; 47; 50;

Trave 14-12; = 11350

" " 14-15 4100 x 2,75/2 = 5650

p.p. = 600

17600 Kg

Pilastri 15; 16; 48; 49

Trave 15-14 = 5650

" " 15-16 4100 x 3,00/2 = 5150

p.p. = 600

12400 Kg

Pilastri 21; 22; 54; 55

Trave 21-22 950 x 1,50/2 = 750

" " 21-28 1350 x 3,35/2 = 2250

p.p. 0,90 x 0,20 x 2,70 x 2500 = 1200

4200 Kg

Pilastri 21; 29; 61; 62;

Trave 21-28 = 2250

p.p. = 650

2900 Kg

1-2-2 Copertura piano tipo

Pilastri 1; 10; 34; 43;

Trave 1-2 2800 x 4,00/2

= 5600

" " 1-11 1400 x 4,65/2

= 3250

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

= 950

9800 Kg

Pilastri 2; 9; 35; 42;

Trave 1-2

= 5600

" " 2-3 2800 x 3,60/2

= 5050

p.p.

= 950

11600 Kg

Pilastri 3; 8; 36; 41

Trave 3-2

= 5050

" " 3-4 2800 x 3,55/2

= 5000

p.p.

= 950

11000 Kg

Pilastri 4; 7; 37; 40

Trave 4-3

= 5000

" " 4-5 3700 x 2,20/2

= 4050

p.p.

= 950

10000 Kg

Pilastri 5; 6; 38; 39;

Trave 5-4

= 4050

" " 5-6 1200 x 2,00/2

= 1200

" " 5-15 2400 x 4,65/2

= 5600

p.p.

= 950

11800 Kg

Pilastrì 11; 20; 44; 53;

Trave 11-1

" " 11-24 1400 x 4,95/2

" " 11-12 3500 x 4,00/2

p.p.

= 3250
= 3450
= 7000
= 950

14650 Kg

Pilastrì 12; 19; 45; 52;

Trave 12-11

" " 12-13 3500 x 3,60/2

p.p.

= 7000
= 6300
= 950

14250 Kg

Pilastrì 13; 18; 46; 51;

Trave 13-12

" " 13-14 3500 x 2,90/2

p.p.

= 6300
= 5050
= 950

12300 Kg

Pilastrì 14; 17; 47; 50

Trave 14-13

" " 14-15 3500 x 2,75/2

p.p.

= 5050
= 9600
= 950

15600 Kg

Pilastrì 15; 16; 48; 49

Trave 15-14

" " 15-5

" " 15-21 2400 x 1,65/2

p.p.

= 9600
= 5600
= 2000
= 950

18150 Kg

Pilastri 21; 22; 54; 55

Trave 21-15

" " 21-23 $1200 \times 1,60/2$ aliquota solaio $\frac{2,65 \times 650 \times 0,70}{2}$ p.p. $0,20 \times 0,90 \times 3,05 \times 2500$

= 2000

= 950

= 600

= 1450

5000 Kg

Pilastri 23; 56

Trave 23-21

" " 23-22

" " 23-28 $1200 \times 1,90/2$ " " 28-29 $1200 \times 1,90/2$ p.p. $0,20 \times 1,90 \times 3,05 \times 2500$

= 950

= 950

= 1150

= 1150

= 2900

7100 Kg

Pilastri 24; 33; 57; 66;

Trave 24-11

" " 24-25 $2900 \times 4,00/2$

p.p.

= 3450

= 5800

= 950

10200 Kg

Pilastri 25; 32; 58; 65

Trave 25-24

" " 25-26 $2900 \times 2,95/2$

p.p.

= 5800

= 4300

= 950

11050 Kg

Pilastri 26; 31; 59; 64

Trave 26-25

" " 26-27 $2900 \times 3,50/2$

p.p.

= 4300

= 5050

= 950

10300 Kg

Pilastri 27; 30; 60; 63

Trave 27-26

" " 27-28 3750 x 3,40/2

p.p.

= 5050

= 6400

= 950

12400 Kg

Pilastri 28; 29; 61; 62

Trave 28-27

" " 28-23

p.p.

= 6400

= 1150

= 950

8500 Kg

1-3-0 - Verifica ed armatura pilastri -

1-3-1 Carichi al piede dei pilastri

Pilastri	Copertura	Cop. V	Cop. IV	Cop. III	Cop. II	Cop. I	Cop. portico fondazione
1,10,34,43,		9800	19600	29400	39200	49000	58800
2,9,35,42,		11600	23200	34800	46400	58000	69600
3,8,36,41		11000	22000	33000	44000	55000	65000
4,7,37,40		10000	20000	30000	40000	50000	60000
5,6,38,39		11800	23600	35400	47200	59000	70800
11,20,44,53		14600	29300	43900	58600	73200	87900
12,19,45,52	20550	37600	51850	66100	80350	94600	108850
13,18,46,51		12300	24600	36900	49200	61500	73800
14,17,47,50	17600	33200	48800	64400	80000	95600	111200
15,16,48,49	12400	35500	53600	71800	89900	108100	126200
21,22,54,55	4200	9200	14200	19200	24200	29200	34200
23,56		7100	14200	21300	28400	35500	42600
24,33,57,66		10200	20400	30600	40800	51000	61200
25,32,58,65		11000	22000	33000	44000	55000	65000
26,31,59,64		10300	20600	30900	41200	51500	61800
27,30,60,63		12400	24800	37200	49600	62000	74400
28,29,61,62	3000	11500	20000	28500	37000	45500	54000

1-3-2 Sezione ed armatura pilastri fondazione e portico

N°	P tn	A' c cmq	A f cmq	Armatura	Sezione cm x cm	A c i cmq	d c Kg/cmq
1,10,34,43	59	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	45
2,9,35,42	70	1450	11,6	4Ø14 + 6Ø12	30 x 50	1629	43
3,8,36,41	65	1350	10,8	4Ø14 + 6Ø12	30 x 50	1629	40
4,7,37,40	60	1250	10,0	4Ø14 + 4Ø12	30 x 40	1307	45
5,6,38,39	71	1450	11,6	4Ø14 + 6Ø12	30 x 50	1629	43
II,20,44,53	88	1800	14,4	10 Ø 14	30 x 60	1954	45
12,19,45,52	109	2250	18	12 Ø 14	40 x 60	2584	42
13,18,46,51	74	1500	12	4Ø14 + 6Ø12	30 x 50	1629	45
14,17,47,50	111	2300	18,4	12 Ø 14	40 x 60	2584	43
15,16,48,49	126	2600	20,8	14 Ø 14	40 x 70	3015	41
21,22,54,55	34	700	5,6	14 Ø 10	20 x 90	1910	17
23,56	43	900	7,2	24 Ø 10	20 x 190	4037	10
24,33,57,66	61	1250	10	4Ø14 + 4Ø12	30 x 40	1307	45
25,32,58,65	66	1350	10,8	4Ø14 + 6Ø12	30 x 50	1629	40
26,31,59,64	62	1300	10,4	4Ø14 + 6Ø12	30 x 50	1629	38
27,30,60,63	75	1550	12,4	4Ø14 + 6Ø12	30 x 50	1629	45
28,29,61,62	54	1100	8,8	4Ø14 + 4Ø12	30 x 40	1307	41

Staffe Ø 8^m /15^m

1-3-3 Sezione ed armatura pilastri piano I e II

N°	P tn	A' c cmq	Af cmq	Armatura	Sezione cm x cm	A ci cmq	σ c Kg/cmq
1,10,34,43	49	1000	8	8 Ø 12	30 x 40	1290	38
2,9,35,42	58	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	44
3,8,36,41	55	1150	9,2	4Ø14 + 4Ø12	30 x 40	1307	42
4,7,37,40	50	1050	8,4	8 Ø 12	30 x 40	1290	38
5,6,38,39	59	1250	10	4Ø14 + 4Ø12	30 x 40	1307	45
11,20,44,53	73	1500	12	4Ø14 + 6Ø12	30 x 50	1629	44
12,19,45,52	95	1950	15,6	10 Ø 14	40 x 50	2154	44
13,18,46,51	62	1300	10,4	4Ø14 + 4Ø12	30 x 40	1307	45
14,17,47,50	96	2000	16	10 Ø 14	40 x 50	2150	44
15,14,48,49	108	2250	18	12 Ø 14	40 x 60	2585	41
21,22,54,55	29	600	4,8	14 Ø 10	20 x 90	1910	<10
23,56	36	750	6	24 Ø 10	20 x 90	4037	<10
24,33,57,66	51	1050	8,4	8 Ø 12	30 x 40	1290	39
25,32,58,65	55	1150	9,2	4Ø14 + 4Ø12	30 x 40	1307	42
26,31,59,64	52	1100	8,8	4Ø14 + 4Ø12	30 x 40	1307	39
27,30,60,63	62	1300	10,4	4Ø14 + 4Ø12	30 x 40	1307	45
28,29,61,62	46	950	7,6	8 Ø 12	30 x 40	1290	35

Staffe Ø 6" / 15"

1-3-4 Sezione ed armatura pilastri piano III e IV

N°	P tn	A' c cmq	Af cmq	Armatura	Sezione cm x cm
1,10,34,43	30	650	5,2	4Ø12 + 4Ø10	30 x 30
2,9,35,42	35	750	6	4Ø12 + 4Ø10	30 x 30
3,8,36,41	33	700	5,6	4Ø12 + 4Ø10	30 x 30
4,37,40	30	650	5,2	4Ø12 + 4Ø10	30 x 30
5,6,38,39	36	750	6	4Ø12 + 4Ø10	30 x 30
11,20,44,53	44	900	7,2	8 Ø 12	30 x 40
12,19,45,52	66	1350	10,8	4Ø14 + 6Ø12	30 x 50
13,18,46,51	37	800	6,4	4Ø12 + 4Ø10	30 x 30
14,17,47,50	65	1350	10,8	4Ø14 + 6Ø12	30 x 50
15,16,48,49	72	1500	12	4Ø14 + 6Ø12	30 x 50
21,22,54,55	19	400	3,2	14 Ø 10	20 x 90
23,56	21	450	3,6	24 Ø 10	20 x 190
24,33,57,66	3I	650	5,2	4Ø12 + 4Ø10	30 x 30
25,32,58,65	33	700	5,6	4Ø12 + 4Ø10	30 x 30
26,31,59,64	31	650	5,6	4Ø12 + 4Ø10	30 x 30
27,30,60,63	37	750	6	4Ø12 + 4Ø10	30 x 30
28,29,61,62	29	600	4,8	4Ø12 + 4Ø10	30 x 30

Staffe Ø 6" / 15"

1-3-5 Sezione ed armatura pilastri piano V e Vol. tecnici

N°	P tn	A' c cmq	Af cmq	Armatura	Sezione cm x cm
1,10,34,43	10	200	1,6	8 Ø 10	30 x 30
2,9,35,42	12	250	2	8 Ø 10	30 x 30
3,8,36,41	11	250	2	8 Ø 10	30 x 30
4,7,37,40	10	200	1,6	8 Ø 10	30 x 30
5,6,38,39	12	250	2	8 Ø 10	30 x 30
11,20,44,53	15	300	2,4	8 Ø 10	30 x 30
12,19,45,52	38	800	6,4	4Ø12+4Ø10	30 x 30
13,18,46,51	12	250	2	8 Ø 10	30 x 30
14,17,47,50	33	700	5,6	4Ø12 + 4Ø10	30 x 30
15,16,48,49	36	750	6	4Ø12 + 4Ø10	30 x 30
21,22,54,55	9	200	1,6	14 Ø 8	20 x 90
23,56	7	150	1,2	24 Ø 8	20 x 190
24,33,57,66	10	200	1,6	8 Ø 10	30 x 30
25,32,58,65	10	200	1,6	8 Ø 10	30 x 30
26,31,59,64	10	200	1,6	8 Ø 10	30 x 30
27,30,60,63	12	250	2	8 Ø 10	30 x 30
28,29,61,62	12	250	2	8 Ø 10	30 x 30

Staffe Ø 6" /15"

1-4-0 Fondazioni

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

pilastrini 2453400

trave 313200

2766600 Kg

1-4-2 -Tensione sul terreno-

$$\sigma_t = \frac{2766600}{1936800} = 1,43 \text{ Kg/cm}^2$$

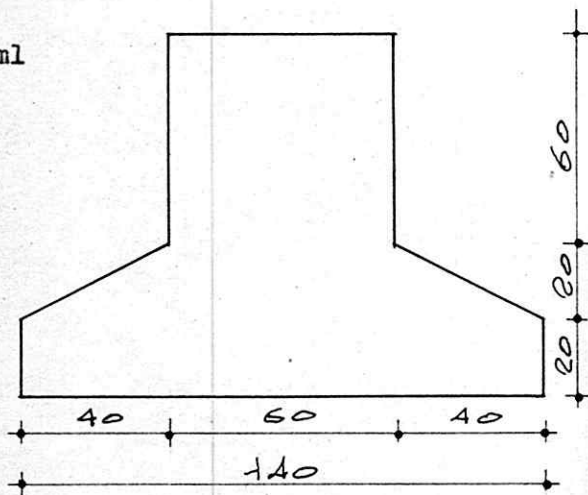
1-4-3 Travi 1-2-3-4-5-6-7-8-9-10; 24-25-26-27-28-29-30-31-32-33-34-35-36
37-38-39-40-41-42-43; 57-58-59-60-61-62-63-64-65-66

Carico sulla trave

$$q = \frac{2453400}{1936800} \times 100 \times 140 = 17750 \text{ Kg/ml}$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 17750 \times 3,85^2 = 21900 \text{ Kgm} \quad H = 100'' \quad b = 60''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 38 \text{ Kg/cm}^2 \quad A_f = 17,3 \text{ cm}^2$$

$$M_{camp} = 21900 \text{ Kgm} \quad H = 100'' \quad b = 140''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 24 \text{ Kg/cm}^2 \quad A_f = 17 \text{ cm}^2$$

-Armatura trasversale-

$$M = \frac{1}{2} 17750 \times 0,40^2 = 1450 \text{ Kgm} \quad H = 40'' \quad b = 100''$$

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

1-4-4 Travi 11-12-13-14-15-16-17-18-19-20; 44-45-46-47-48-49-50-51-52-53

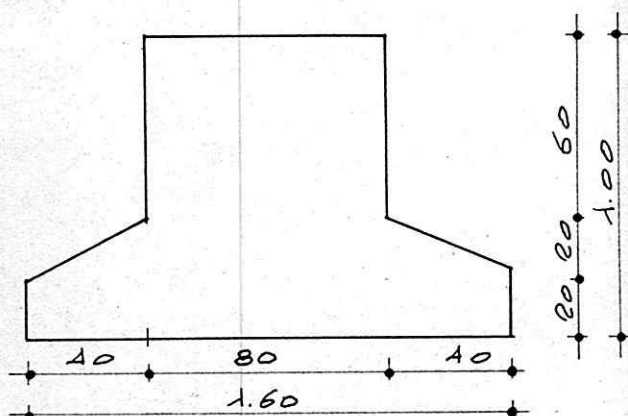
Carico sulla trave

$$q = \frac{2453400}{1936800} \times 100 \times 160 = 20250 \text{ Kg/ml}$$

1936800

Caratteristiche geometriche

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 20250 \times 3,85^2 = 25000 \text{ Kgm} \quad H = 100'' \quad b = 80''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 35 \text{ Kg/cm}^2 \quad A_f = 19,7 \text{ cm}^2$$

$$M_{camp} = 25000 \text{ Kgm} \quad H = 100'' \quad b = 160''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 24 \text{ Kg/cm}^2 \quad A_f = 19,4 \text{ cm}^2$$

-Armatura trasversale-

$$M = \frac{1}{2} 20250 \times 0,40^2 = 1600 \quad b = 100'' \quad H = 40''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 20 \text{ Kg/cm}^2 \quad A_f = 3,2 \text{ cm}^2$$

1-4-5 Travi 1-11-24; 10-20-33; 34-44-57; 43-53-66

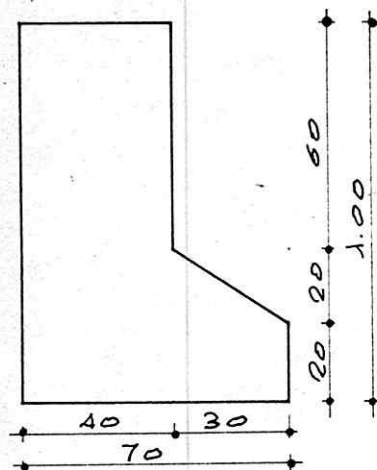
Carico sulla trave

$$q = \frac{2453400}{1936800} \times 100 \times 70 = 8850 \text{ Kgm}$$

1936800

Caratteristiche geometriche

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 8850 \times 4,95^2 = 18050 \text{ Kgm}$$

$$H = 100'' \quad b = 40''$$

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

$$\sigma_c = 43 \text{ Kg/cm}^2$$

$$A_f = 14,2 \text{ cm}^2$$

$$M_{camp} = 18050 \text{ Kgm}$$

$$H = 100''$$

$$b = 70''$$

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

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

$$A_f = 13,9 \text{ cm}^2$$

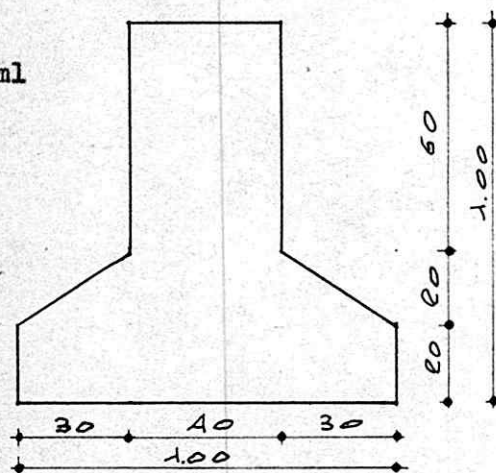
1-4-6 Travi 2-12-25; 3-13-26; 4-14-27; 5-15-21-23-28; 6-16-22-23-29+7-17-30;
8-18-31; 9-19-32; 35-45-58; 36-46-59; 37-47-60; 38-48-54- 56-61; 39-49
55-56-62; 40-50-63-41-51-64; 42-52-65

Carico sulla trave

$$q = \frac{2453400}{1936800} \times 100 \times 100 = 12650 \text{ Kg/ml}$$

Caratteristiche geometriche

Vedi disegno

**-Armatura a flessione-**

$$M_{inc} = \frac{1}{12} 12650 \times 4,95^2 = 25850 \text{ Kgm}$$

$$H = 100'' \quad b = 40''$$

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

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

$$A_f = 20,4 \text{ cm}^2$$

$$M_{camp} = 25850 \text{ Kgm}$$

$$H = 100''$$

$$b = 100''$$

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

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

$$A_f = 20 \text{ cm}^2$$

1-5-0 - Vano Scala -

1-5-1 Calcolo del gradino

Analisi di carico

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

$$\text{sovracc. perm. (intonaco, grado sottogrado e massetto)} = 150 \times 0,30 = 45$$

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

280 Kg/ml

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

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

1-5-2 Calcolo dei pianerottoliSolaio in latero-cemento $H = 16'' + 4''$ $i = 50''$

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/cmq

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

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

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

$$\sigma_f = 1400 \text{ Kg/cmq} \quad \sigma = 20 \text{ Kg/cmq} \quad A_f = 0,82 \text{ cmq} / 50''$$

1-5-3 Calcolo delle travi a ginocchio

Analisi di carico

Rampa 850 x 1,20

tompagno

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

= 1100

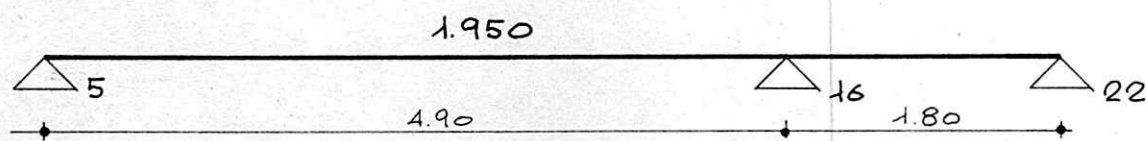
= 400

= 450

1950 Kg/ml

Verifica ed armatura a flessione

Schema di calcolo



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

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

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

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

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

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

$$M_{22} = \frac{1}{12} 1950 \times 1,80^2 = 550 \text{ Kgm} \quad H = 50'' \quad b = 30''$$

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

Verifica a torsione della trave 5-16

$$m_i = \frac{1}{2} 850 \times 1,20^2 = 612 \text{ Kgm}$$

$$m_t = 612 + 1020 \times 0,15 = 765 \text{ Kgm}$$

$$t_i = 850 \times 1,20 = 1020 \text{ Kgm}$$

$$M_t = 765 \times \frac{4,90}{2} = 1875 \text{ Kgm}$$

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

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

-Armatura a torsione-

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

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

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

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

1-5-4 Calcolo della trave al pianerottolo di riposo

$$M = \frac{1}{12} 1200 \times 3,00^2 = 900 \text{ Kgm}$$

$$H = 40''$$

$$b = 20''$$

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

$$\sigma_c = 34 \text{ Kg/cm}^2$$

$$A_f = 1,8 \text{ cm}^2$$

1-6-0 BALCONI

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

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

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

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

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

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

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

1-7-0

SOLAI 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 $3,68 \times 1,05 = 3,85 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 3,85^2 = 400 \text{ Kgm}$	$H = 25 \text{ cm}$	$\sigma_c = 23 \text{ Kg/cm}^2$	$A_f = 1,32 \text{ cm}^2$
----------	--	---------------------	---------------------------------	---------------------------

campata	$M = \frac{1}{14} 325 \times 3,85^2 = 345 \text{ Kgm}$	$H = 25 \text{ cm}$	$\sigma_c = 21 \text{ Kg/cm}^2$	$A_f = 1,12 \text{ cm}^2$
---------	--	---------------------	---------------------------------	---------------------------

luce netta $4,28 \times 1,05 = 4,50 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 4,50^2 = 550 \text{ Kgm}$	$H = 25 \text{ cm}$	$\sigma_c = 27 \text{ Kg/cm}^2$	$A_f = 1,8 \text{ cm}^2$
----------	--	---------------------	---------------------------------	--------------------------

	$H = 18 \text{ cm}$	$\sigma_c = 36 \text{ Kg/cm}^2$	$A_f = 2,35 \text{ cm}^2$
--	---------------------	---------------------------------	---------------------------

campata	$M = \frac{1}{14} 325 \times 4,50^2 = 470 \text{ Kgm}$	$H = 25 \text{ cm}$	$\sigma_c = 25 \text{ Kg/cm}^2$	$A_f = 1,55 \text{ cm}^2$
---------	--	---------------------	---------------------------------	---------------------------

	$H = 18 \text{ cm}$	$\sigma_c = 33 \text{ Kg/cm}^2$	$A_f = 2,0 \text{ cm}^2$
--	---------------------	---------------------------------	--------------------------

luce netta $2,83 \times 1,05 = 3,00 \text{ m.}$

incastro	$M = \frac{1}{12} 325 \times 3,00^2 = 250 \text{ Kgm}$	$H = 25 \text{ cm}$	$\sigma_c = 18 \text{ Kg/cm}^2$	$A_f = 0,85 \text{ cm}^2$
----------	--	---------------------	---------------------------------	---------------------------

campata	$M = \frac{1}{14} 325 \times 3,00^2 = 200 \text{ Kgm}$	$H = 25 \text{ cm}$	$\sigma_c < 15 \text{ Kg/cm}^2$	$A_f = 0,70 \text{ cm}^2$
---------	--	---------------------	---------------------------------	---------------------------

1-8-0 - Calcolo a flessione delle travi -

1-8-1 Si effettua il Cross per il telaio 1-2-3-4-5-6-7-8-9-10-34-35-36-37-38-39-40-41-42-43

Lo schema di carico e le grandezze geometriche sono riportate in disegno.

Calcolo dei momenti di inerzia

Pilastri 1-10; 4-7; 10-11; 9-12; 12-19; 8-13; 13-18; 7-14; 14-17; 6-15; 15-16

$$J = 90000 \text{ cm}^4$$

Pilastri 2-9; 3-8; 5-6

$$J = 112500 \text{ cm}^4$$

Pilastri 20-21; 21-30; 30-31; 19-22; 22-29; 29-32; 18-23; 23-28; 28-33; 17-24; 24-27; 27-34; 16-25; 25-26; 26-35

$$J = 67500 \text{ cm}^4$$

Travi

$$J = 91145 \text{ cm}^4$$

1-8-2 - Calcolo dei fattori di rigidità -

$$K_{65} = K_{83} = K_{92} = 4 E x 300$$

$$K_{74} = K_{104} = 4 E x 240$$

$$K_{615} = K_{1516} = K_{714} = K_{1417} = K_{813} = K_{1318} = K_{912} = K_{1219} = K_{1011} = K_{1120} = 4 E x 273$$

$$K_{1225} = K_{2526} = K_{2635} = K_{1724} = K_{2427} = K_{2734} = K_{1823} = K_{2328} = K_{2833} = K_{1922} = K_{2229} = K_{2932} = K_{2021} = K_{2130} = K_{3031} = 4 E x 205$$

$$K_{67} = K_{1514} = K_{1617} = K_{2524} = K_{2627} = K_{3534} = 4 E x 414$$

$$K_{78} = K_{1413} = K_{1718} = K_{2423} = K_{2728} = K_{3430} = 4 E x 268$$

$$K_{89} = K_{1312} = K_{1819} = K_{2322} = K_{2829} = K_{3332} = 4 E x 253$$

$$K_{910} = K_{1211} = K_{1920} = K_{2221} = K_{2930} = K_{3231} = 4 E \times 237$$

1-8-3 - Calcolo dei fattori di ripartizione-

Nodo 6

$$6-5 = \underline{0,30} \quad 6-7 = \underline{0,42} \quad 6-15 = \underline{0,28}$$

Nodo 7

$$7-4 = \underline{0,20} \quad 7-6 = \underline{0,35} \quad 7-14 = \underline{0,23} \quad 7-8 = \underline{0,22}$$

Nodo 8

$$8-3 = \underline{0,27} \quad 8-7 = \underline{0,24} \quad 8-13 = \underline{0,26} \quad 8-9 = \underline{0,23}$$

Nodo 9

$$9-2 = \underline{0,28} \quad 9-8 = \underline{0,24} \quad 9-12 = \underline{0,26} \quad 9-10 = \underline{0,22}$$

Nodo 10

$$10-1 = \underline{0,32} \quad 10-9 = \underline{0,31} \quad 10-11 = \underline{0,37}$$

Nodo 11

$$11-10 = 11-20 = \underline{0,35} \quad 11-12 = \underline{0,30}$$

Nodo 12

$$12-9 = 12-19 = \underline{0,26} \quad 12-11 = \underline{0,23} \quad 12-13 = \underline{0,25}$$

Nodo 13

$$13-8 = 13-19 = \underline{0,26} \quad 13-12 = \underline{0,23} \quad 13-14 = \underline{0,24}$$

Nodo 14

$$14-7 = 14-17 = \underline{0,22} \quad 14-13 = \underline{0,21} \quad 14-15 = \underline{0,35}$$

Nodo 15

$$15-6 = 15-16 = \underline{0,28} \quad 15-14 = \underline{0,44}$$

Nodo 16

$$16-15 = \underline{0,31} \quad 16-25 = \underline{0,23} \quad 16-17 = \underline{0,46}$$

Nodo 17

$$17-14 = \underline{0,23} \quad 17-16 = \underline{0,36} \quad 17-24 = \underline{0,18} \quad 17-18 = \underline{0,23}$$

Nodo 18

$$18-13 = \underline{0,27} \quad 18-17 = \underline{0,27} \quad 18-23 = \underline{0,21} \quad 18-19 = \underline{0,25}$$

Nodo 19

$$19-18 = \underline{0,26} \quad 19-22 = \underline{0,22} \quad 19-20 = \underline{0,24} \quad 19-12 = \underline{0,28}$$

Nodo 20

$$20-19 = \underline{0,33} \quad 20-21 = \underline{0,29} \quad 20-11 = \underline{0,31}$$

Nodo 21-30

$$21-20 = 21-30 = 30-21 = 30-31 = \underline{0,32} \quad 21-22 = 30-29 = 0,36$$

Nodo 22-29

$$22-19-22-29-29-32 = \underline{0,23} \quad 22-21-29-30 = \underline{0,26} \quad 22-23-29-28 = \underline{0,28}$$

Nodo 23-28

$$23-18; 23-28; 28-33 = \underline{0,22} \quad 23-22-28-29 = \underline{0,27} \quad 23-24-28-27 = \underline{0,29}$$

Nodo 24-27

$$24-17; 24-27; 27-34 = \underline{0,18} \quad 24-23-27-28 = \underline{0,25} \quad 24-25-27-26 = \underline{0,39}$$

Nodo 25-26

$$25-16; 25-26; 26-35 = \underline{0,25} \quad 25-24-26-27 = \underline{0,50}$$

Nodo 31

$$31-30 = 0,46 \quad 31-32 = 0,54$$

Nodo 32

$$32-29 = \underline{0,30} \quad 32-31 = \underline{0,34} \quad 32-33 = \underline{0,36}$$

Nodo 33

$$33-32 = \underline{0,35} \quad 33-28 = \underline{0,28} \quad 33-34 = \underline{0,37}$$

Nodo 34

$$34-33 = \underline{0,30} \quad 34-27 = \underline{0,23} \quad 34-35 = \underline{0,37}$$

Nodo 35

$$35-26 = \underline{0,33} \quad 35-34 = \underline{0,67}$$

1-7-3 - Momenti d'incastro perfetto -

Travi da 2,20 m.

$$M = \frac{1}{12} 3700 \times 2,20^2 = 1500 \text{ Kgm}$$

Travi da 3,40 m.

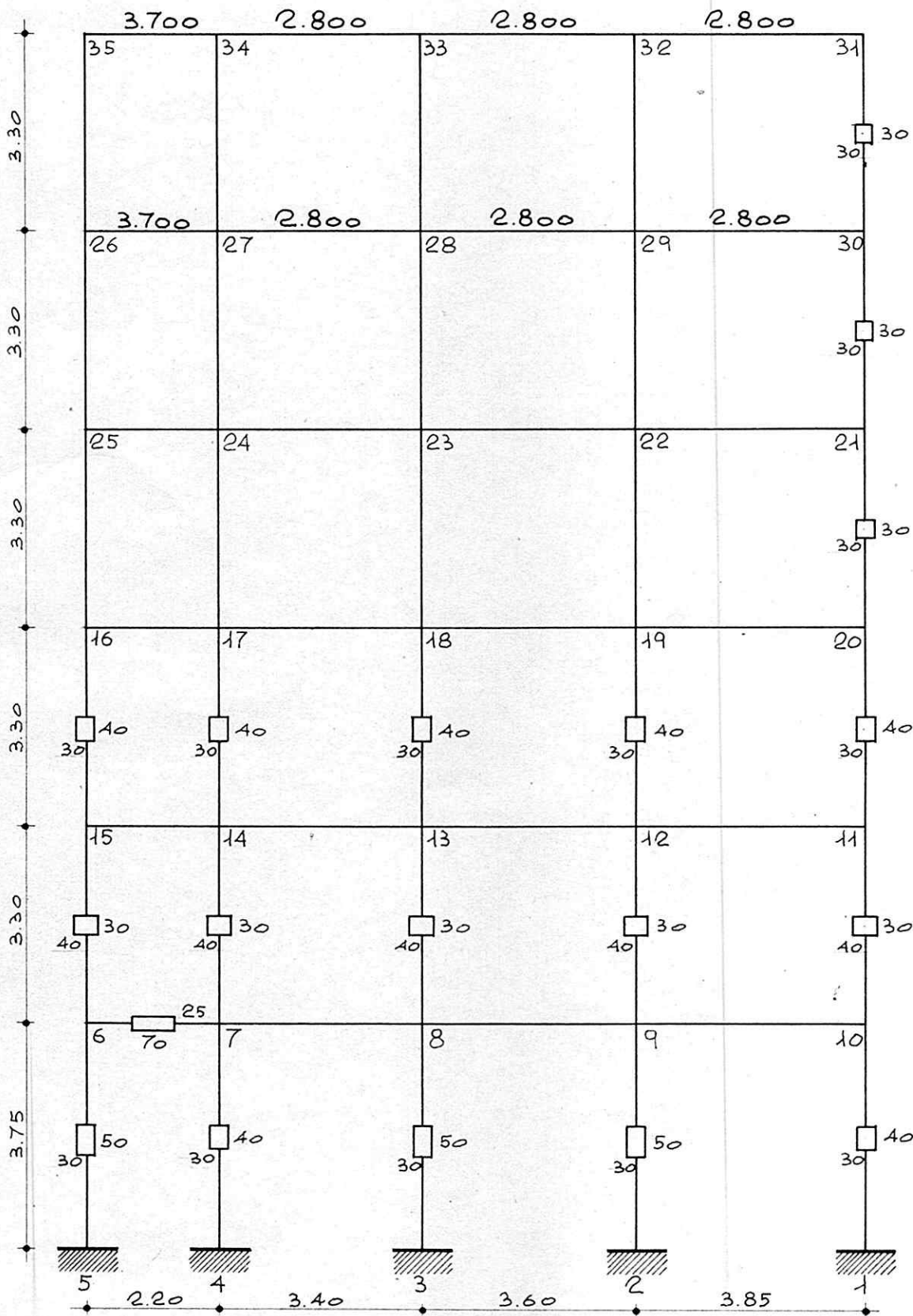
$$M = \frac{1}{12} 2800 \times 3,40^2 = 2700 \text{ Kgm}$$

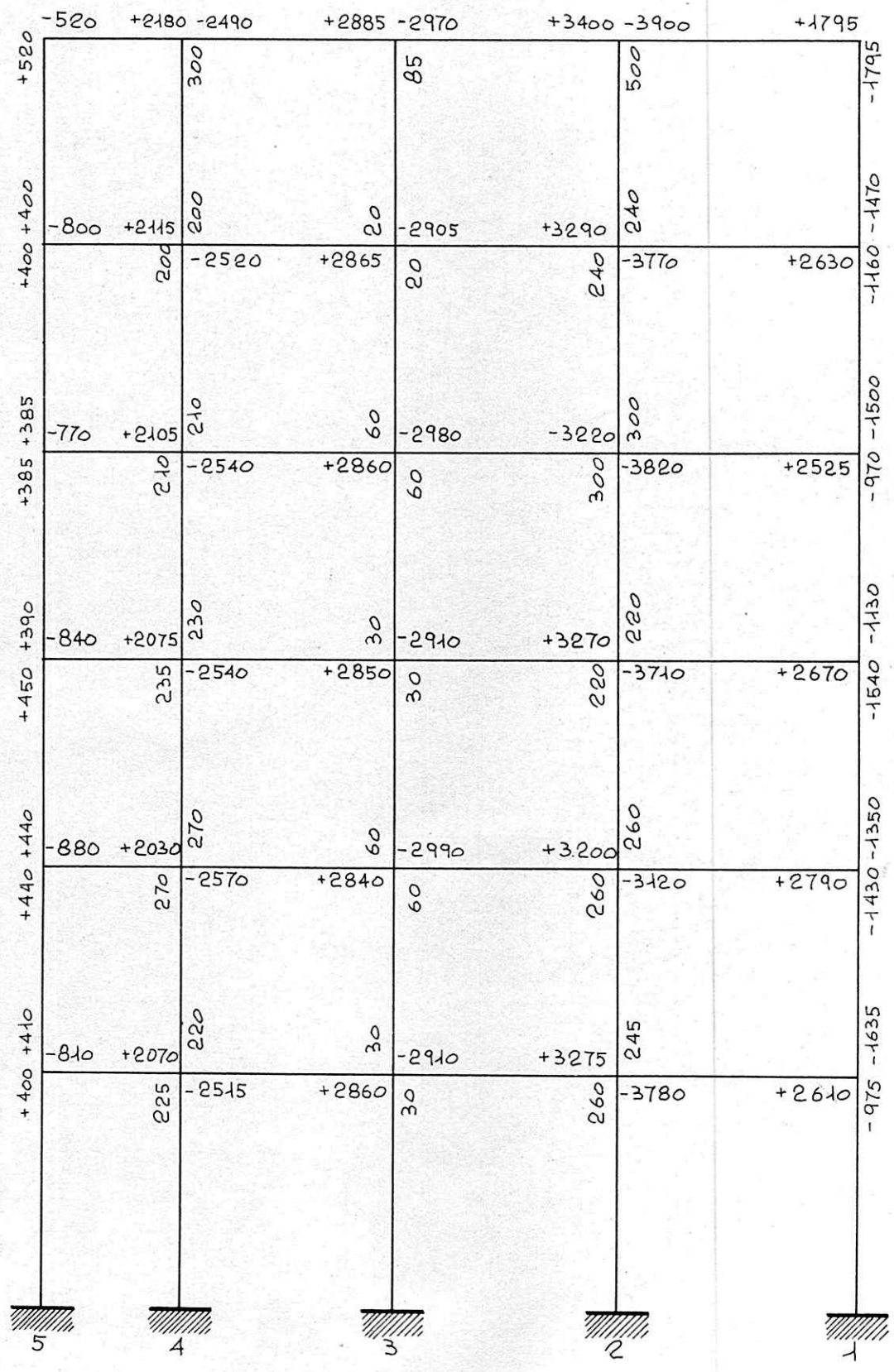
Travi da 3,60 m.

$$M = \frac{1}{12} 2800 \times 3,60^2 = 3050 \text{ Kgm}$$

Travi da 3,85 m.

$$M = \frac{1}{12} 2800 \times 3,85^2 = 3450 \text{ Kgm}$$





1-8-4 Verifica ed armatura travi

Travi 1-2 ai piani tipo

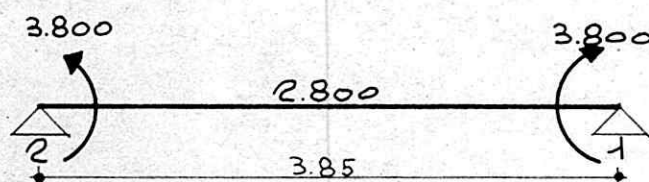
$R_2 = 5730 \text{ Kg}$

$R_1 = 5050 \text{ Kg}$

$x = 2,05 \text{ m. da 2}$

$M_{\max} = 2060 \text{ Kgm}$

$\tau_{\max} = 3,9 \text{ Kg/cm}^2$



Verifiche ed armatura

$H = 25 \text{ cm} \quad b = 70 \text{ cm}$

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

Sezione 1 $M = 2500 \text{ Kgm}$

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

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

Sezione 1-2 $M = 2050 \text{ Kgm}$

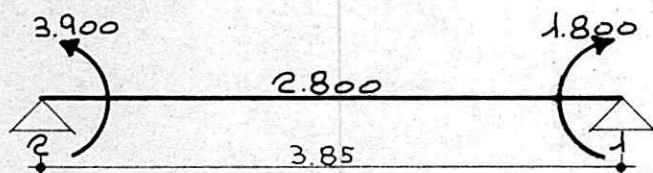
$\sigma_c = 47 \text{ Kg/cm}^2$

$A_f = 6,9 \text{ cm}^2$

Sezione 2 $M = 3800 \text{ Kgm}$

$\sigma_c = 69 \text{ Kg/cm}^2$

$A_f = 13,1 \text{ cm}^2$

1-8-5 Travi 1-2 alla copertura

$R_2 = 5950 \text{ Kg}$

$R_1 = 4850 \text{ Kg}$

$x = 2,15 \text{ m.}$

$M_{\max} = 2450 \text{ Kgm}$

$\tau_{\max} = 4,1 \text{ Kg/cm}^2$

Verifiche ed armatura

$H = 25 \text{ cm} \quad b = 70 \text{ cm}$

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

Sezione 1 $M = 1800 \text{ Kgm}$

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

$A_f = 6,1 \text{ cm}^2$

Sezione 1-2 $M = 2450 \text{ Kgm}$

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

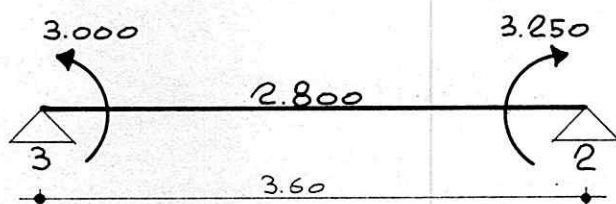
$A_f = 8,3 \text{ cm}^2$

Sezione 2 $M = 3900 \text{ Kgm}$

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

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

1-8-6 Travi 2-3



$$R_3 = 4950 \text{ Kg}$$

$$R_2 = 5150 \text{ Kg}$$

$$x = 1,75 \text{ m.}$$

$$M_{\max} = 1400 \text{ Kgm}$$

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

Verifiche ed armatura $H = 25 \text{ cm}$ $b = 70 \text{ cm}$ Sezione 2 $M = 3250 \text{ Kgm}$ $\sigma_c = 63 \text{ Kg/cm}^2$ Sezione 2-3 $M = 1400 \text{ Kgm}$ $\sigma_c = 43 \text{ Kg/cm}^2$ Sezione 3 $M = 3000 \text{ Kgm}$ $\sigma_c = 60 \text{ Kg/cm}^2$

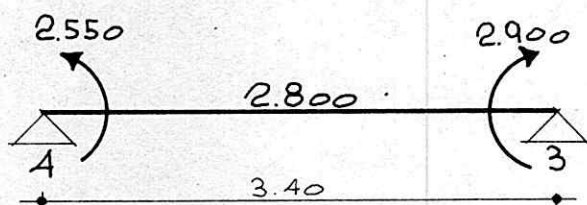
$$\sigma_f = 1400 \text{ Kg/cm}^2$$

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

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

$$A_f = 10,4 \text{ cm}^2$$

1-8-7 Trave 3-4



$$R_4 = 4700 \text{ Kg}$$

$$R_3 = 4800 \text{ Kg}$$

$$x = 1,65 \text{ m.}$$

$$M_{\max} = 1400 \text{ Kgm}$$

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

Verifiche ed armatura $H = 25 \text{ cm}$ $b = 70 \text{ cm}$ Sezione 3 $M = 2900 \text{ Kgm}$ $\sigma_c = 59 \text{ Kg/cm}^2$ Sezione 4-3 $M = 1400 \text{ Kgm}$ $\sigma_c = 43 \text{ Kg/cm}^2$ Sezione 4 $M = 2550 \text{ Kgm}$ $\sigma_c = 55 \text{ Kg/cm}^2$

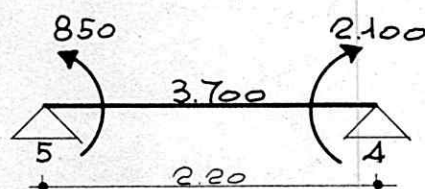
$$\sigma_f = 1400 \text{ Kg/cm}^2$$

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

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

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

1-8-8 Travi 4-5



$$R_5 = 3500 \text{ Kg}$$

$$R_4 = 4650 \text{ Kg}$$

$$x = 0,95 \text{ m.}$$

$$M_{\max} = 800 \text{ Kgm}$$

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

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

Sezione 4 $M = 2100 \text{ Kg}$ $\sigma_c = 49 \text{ Kg/cm}^2$ $A_f = 7,2 \text{ cm}^2$

Sezione 4-5 $M = 800 \text{ Kgm}$ $\sigma_c = 28 \text{ Kg/cm}^2$ $A_f = 2,8 \text{ cm}^2$

Sezione 5 $M = 850 \text{ Kgm}$ $\sigma_c = 29 \text{ Kg/cm}^2$ $A_f = 2,9 \text{ cm}^2$

1-8-9 Verifica in testa del pilastro 1

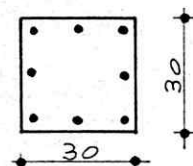
$R = 9000 \text{ Kg}$ $M = 1800 \text{ Kgm}$ $A_c = 30 \times 30$ $A_f = 8 \text{ } \varnothing 10$

$a = 30 \text{ cm}$; $b = 30 \text{ cm}$; $h = 28 \text{ cm}$; $h' = 20 \text{ cm}$

$A_f = A'_f = 2,36 \text{ cm}^2$ $e = 20 \text{ cm}$

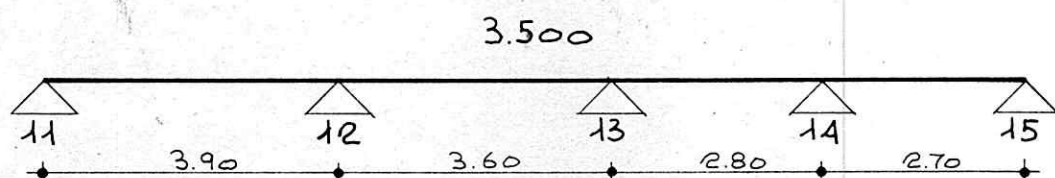
$d = 33 \text{ cm}$ $d' = 7 \text{ cm}$ ricaviamo la posizione dell'asse neutro

$x = 10,1 \text{ cm}$ $\sigma_c = 69 \text{ Kg/cm}^2$



Esaminati i risultati del Cross e le relative verifiche sulle travi si adottano dei coefficienti nella soluzione delle altre strutture.

1-8-10 Travi 11-12-13-14-15; 16-17-18-19-20; 44-45-46-47-48; 49-50-51-52-53



- Armatura a flessione -

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

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

$$b = 90 \text{ cm}$$

Sezione 11; 11-12

$$M = \frac{1}{14} 3500 \times 3,90^2 = 3800 \text{ Kgm} \quad \sigma_c = 60 \text{ Kg/cm}^2 \quad A_f = 13,2 \text{ cm}^2$$

Sezione 12

$$M = \frac{1}{10} 3500 \times 3,90^2 = 5300 \text{ Kgm} \quad \sigma_c = 73 \text{ Kg/cm}^2 \quad A_f = 18,5 \text{ cm}^2$$

Sezione 12-13

$$M = \frac{1}{16} 3500 \times 3,60^2 = 2800 \text{ Kgm} \quad \sigma_c = 50 \text{ Kg/cm}^2 \quad A_f = 9,6 \text{ cm}^2$$

Sezione 13

$$M = \frac{1}{12} 3500 \times 3,60^2 = 3800 \text{ Kgm} \quad \sigma_c = 60 \text{ Kg/cm}^2 \quad A_f = 13,2 \text{ cm}^2$$

Sezione 13-14

$$M = \frac{1}{16} 3500 \times 2,80^2 = 1700 \text{ Kgm} \quad \sigma_c = 37 \text{ Kg/cm}^2 \quad A_f = 5,8 \text{ cm}^2$$

Sezione 14

$$M = \frac{1}{10} 3500 \times 2,70^2 = 2550 \text{ Kgm} \quad \sigma_c = 47 \text{ Kg/cm}^2 \quad A_f = 8,7 \text{ cm}^2$$

Sezione 14-15; 15

$$M = \frac{1}{14} 3500 \times 2,70^2 = 1800 \text{ Kgm}$$

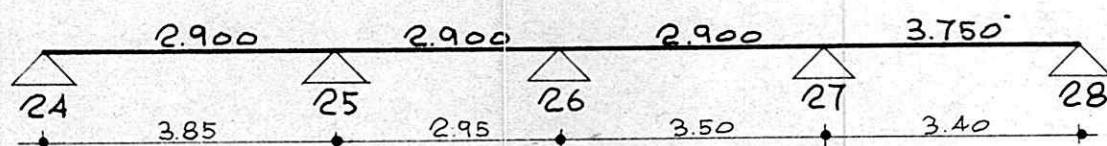
$$\sigma_c = 38 \text{ Kg/cm}^2$$

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

$$\text{Taglio massimo } T = 6800 \text{ Kg}$$

$$\tau_{\max} = \frac{6800}{90 \times 0,9 \times 23} = 3,6 \text{ Kg/cm}^2$$

1-8-11 Travi 24-25-26-27-28; 29-30-31-32-33
57-58-59-60-61; 62-63-64-65-66



- Armatura a flessione -

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

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

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

Sezione 24; 24-25;

$$M = \frac{1}{14} 2900 \times 3,85^2 = 3050 \text{ Kgm}$$

$$\sigma_c = 61 \text{ Kg/cm}^2$$

$$A_f = 10,6 \text{ cm}^2$$

Sezione 25

$$M = \frac{1}{10} 2900 \times 3,85^2 = 4300 \text{ Kgm}$$

$$\sigma_c = 76 \text{ kg/cm}^2$$

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

Sezione 25-26

$$M = \frac{1}{16} 2900 \times 2,95^2 = 1550 \text{ kgm}$$

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

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

Sezione 26

$$M = \frac{1}{12} 2900 \times 3,50^2 = 2950 \text{ Kgm}$$

$$\sigma_c = 60 \text{ Kg/cm}^2$$

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

Sezione 26-27

$$M = \frac{1}{16} 2900 \times 3,50^2 = 2200 \text{ Kgm}$$

$$\sigma_c = 50 \text{ Kg/cm}^2$$

$$A_f = 7,5 \text{ cm}^2$$

Sezione 27

$$M = \frac{1}{10} 3750 \times 3,40^2 = 4300 \text{ Kgm}$$

$$\sigma_c = 76 \text{ Kg/cm}^2$$

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

Sezione 27-28; 28

$$M = \frac{1}{14} 3750 \times 3,40^2 = 3100 \text{ Kgm}$$

$$\sigma_c = 61 \text{ Kg/cm}^2$$

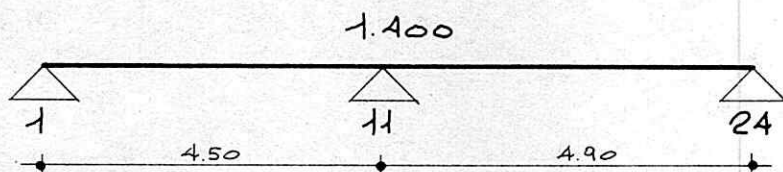
$$A_f = 10,7 \text{ cm}^2$$

Taglio massimo $T = 6300 \text{ Kg}$

$$\tau_{\max} = \frac{6300}{70 \times 0,9 \times 23}$$

$$= 4,3 \text{ Kg/cm}^2$$

1-8-12 Travi 1-11-24; 10-20-33; 34-44-57; 43-53-66



- Armatura a flessione -

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

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

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

Sezione 1; 1-11

$$M = \frac{1}{14} 1400 \times 4,50^2 = 2050 \text{ Kgm}$$

$$\sigma_c = 59 \text{ Kg/cm}^2$$

$$A_f = 7,1 \text{ cm}^2$$

Sezione 11

$$M = \frac{1}{12} 1400 \times 4,90^2 = 2800 \text{ Kgm}$$

$$\sigma_c = 71 \text{ Kg/cm}^2$$

$$A_f = 9,8 \text{ cm}^2$$

Sezione 11-24; 24

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

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

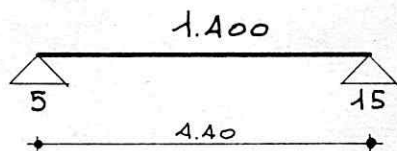
$$A_f = 8,5 \text{ cm}^2$$

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

$$\tau_{\max} = \frac{3450}{50 \times 0,9 \times 23}$$

$$= 3,3 \text{ Kg/cm}^2$$

1-8-13 Travi 5-15; 6-16; 38-48; 39-49

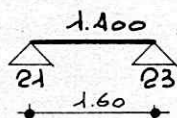


- Armatura a flessione -

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

Sezione 5; 5-15; 15

$$M = \frac{1}{12} 1400 \times 4,40^2 = 2250 \text{ Kgm} \quad \sigma_c = 62 \text{ Kg/cm}^2 \quad A_f = 7,8 \text{ cm}^2$$

1-8-14 Travi 21-23; 21-24; 23-28; 23-29;
54-56; 55-56; 56-61; 56-62

Sezione 21; 21-23; 23

$$H = 25 \text{ cm} \quad b = 20 \text{ cm}$$

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

1-9-0 - Calcolo della soletta ascensore -

1-9-1 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

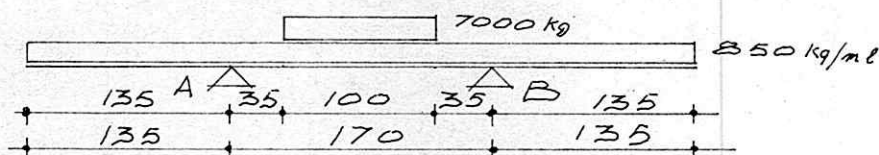
= 6000

basamento

= 1000

7000 Kg

1-9-2 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

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

$$A_f = A'_f = 10,78 \text{ cmq} \quad (1 + 1014/15) \quad A_f^x = 21,56 \text{ cmq} \quad h^x = 12,5 \text{ cm}$$

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

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

1-9-3 Coefficienti di sicurezza

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

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

1-9-4 - Travi portasolette -

Analisi di carico

travi 28-23

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

tompagno

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

= 4150

= 1050

= 800

= 500

6500 Kg/ml

Travi 23-21

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

tompagno

p.p.

= 650

= 1050

= 800

= 400

2900 Kg/ml

Armatura a flessione

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

1-10-0 ~ Copertura Volumi tecnici ~

1-10-1 -Solai-

Carico per nervatura $q = 650 \times 0,50 = 325 \text{ Kg/ml}$ $\sigma_f = 1400 \text{ Kg/cm}^2$ $H = 25 \text{ cm}$ $b = 50 \text{ cm}$

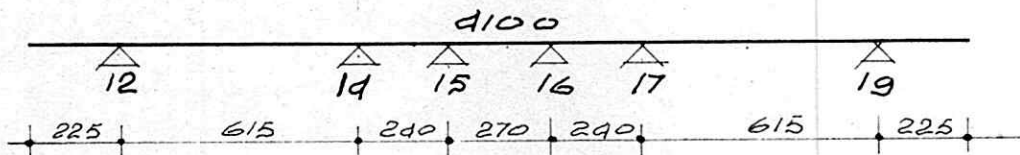
luce netta 2,70 m.

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

luce netta 1,50 m.

 $M = \frac{1}{2} 325 \times 1,50^2 = 360 \text{ Kgm}$ $\sigma_c = 21 \text{ Kg/cm}^2$ $A_f = 1,15 \text{ cm}^2$ luce netta = $2,70 \times 1,05 = 2,85 \text{ m.}$ $M = \frac{1}{12} 325 \times 2,83^2 = 200 \text{ Kgm}$ $\sigma_c < 15 \text{ Kg/cm}^2$ $A_f = 1 \text{ cm}^2$

1-10-2 Travi S-12-14-15-16-17-19-S
S-45-47-48-49-50-52-S



- Armatura a flessione -

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

Sezione 12; 14

$$M = \frac{1}{12} 4100 \times 6,15^2 = 12900 \text{ Kgm} \quad \sigma_c = 55 \text{ Kg/cm}^2 \quad A_f = 13 \text{ cm}^2$$

Sezione 12-14

$$M = \frac{1}{16} 4100 \times 6,15^2 = 9700 \text{ Kgm} \quad \sigma_c = 47 \text{ Kg/cm}^2 \quad A_f = 9,8 \text{ cm}^2$$

Sezione 14-15; 15-16;

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

Sezione 15

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

Taglio massimo $T = 12600 \text{ Kg}$

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

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

2-0-0 EDIFICIO TIPO B1

2-1-0 - Analisi di carico sulle travi -

2-1-1 COPERTURA

Trave 9-12-14-15-16-17-19-9; 9-45-47-48-49-50-52-9

Solaio 650 x 270 = 1750

" " 650 x 270 = 1750

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

4100 Kg/ml

Trave 21-22; 54-55

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 21-29; 22-29; 54-61; 55-62

Solaio 650 x 1,30/2 = 490

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

2-1-2 COPERTURA PIANO TIPO

Trave 1-2-3-4; 7-8-9-10; 34-35-36-37

40-41-42-43;

Solaio 650 x 3,85/2 = 1250

Sovracc. 365 x 0,70 = 250

Tompagno = 850

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

2800 Kg/ml

Trave 4-5; 6-7; 37-38; 39-40

Solaio 650 x 3,85/2

Sbalzo 800 x 1,15

Sovracc. 365 x 0,70

Tompagno

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

= 1250

= 900

= 250

= 850

= 450

3700 Kg/ml

Trave 11-12-13-14-15; 16-17-18-19-20;

44-45-46-47-48; 49-50-51-52-53

Solaio 650 x 3,85/2

Solaio 650 x 4,15/2

Sovracc. 365 x 0,90

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

= 1250

= 1350

= 350

= 550

3500 Kg/ml

Trave 24-25-26-27; 30-31-32-33;

57-58-59-60; 63-64-65-66

Solaio 650 x 4,15/2

Sovracc. 365 x 0,70

Tompagno

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

= 1350

= 250

= 850

= 450

2900 Kg/ml

Trave 27-28; 29-30; 60-61; 62-63;

Solaio 650 x 4,15/2

Sbalzo 800 x 1,05

Sovracc. 365 x 0,70

Tompagno

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

= 1350

= 850

= 250

= 850

= 450

3750 Kg/ml

Trave 5-6; 38-39; 21-23; 22-23; 54-56; 55-56;
23-28; 23-29; 56-61; 56-62

Tompagno

p.p. 0,30 x 0,40 x 1,00 x 2500

= 850

= 350

1200 Kg/ml

Trave 5-15-21; 6-16-22;

38-48-54; 39-49-55

Rampa 850 x 1,30

Tompagno

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

= 1100

= 850

= 450

2400 Kg/ml

Trave 1-11-24; 10-20-30; 34-44-57;

43-53-66

Tompagno

Sovracc. 365 x 0,50

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

= 850

= 200

= 350

1400 Kg/ml

2-2-0

- Analisi di carico sui pilastri -

2-2-1

COPERTURA

Pilastri 12; 19; 45; 52;

Sbalzo 4100 x 2,10

= 8600

Trave 12-14 4100 x 5,55/2

= 11350

p.p. 0,30 x 0,30 x 2,70 x 2500

= 600

20550 Kg

Pilastri 14; 17; 47; 50

Trave 14-12

= 11350

" " 14-15 4100 x 2,75/2

= 5650

p.p.

= 600

17600 Kg

Pilastri 15; 16; 48; 49

Trave 15-14

= 5650

" " 15-16 4100 x 3,00/2

= 6150

p.p.

= 600

12400 Kg

Pilastri 21; 22; 54; 55

Trave 21-22 950 x 1,50/2

= 750

" " 21-28 1350 x 3,35/2

= 2250

p.p. 0,90 x 0,20 x 2,70 x 2500

= 1200

4200 Kg

Pilastri 21; 29; 61; 62

Trave 21-28

= 2250

p.p.

= 650

2900 Kg

2-2-2 Copertura piano tipo

Pilastri 1; 10; 34; 43

Trave 1-2 2800 x 4,00/2

= 5600

" " 1-11 1400 x 4,65/2

= 3250

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

= 950

9800 Kg

Pilastri 2; 9; 35; 42

Trave 1-2

= 5600

" " 2-3 2800 x 3,60/2

= 5050

p. p.

= 950

11600 Kg

Pilastri 3; 8; 36; 41

Trave 3-2

= 5050

" " 3-4 2800 x 3,55/2

= 5000

p.p.

= 950

11000 Kg

Pilastri 4; 7; 37; 40

Trave 4-3

= 5000

" " 4-5 3700 x 2,20/2

= 4050

p.p.

= 950

10000 Kg

Pilastri 5; 6; 38; 39;

Trave 5-4

= 4050

" " 5-6 1200 x 2,00/2

= 1200

" " 5-15 2400 x 4,65/2

= 5600

p.p.

= 950

11800 Kg

Pilastri 11; 20; 44; 53

Trave 11-1

" " 11-24 1400 x 4,95/2

" " 11-12 3500 x 4,00/2

p.p.

= 3250

= 3450

= 7000

= 950

14650 Kg

Pilastri 12; 19; 45; 52

Trave 12-11

" " 12-13 3500 x 3,60/2

p.p.

= 7000

= 6300

= 950

14250 Kg

Pilastri 13; 18; 46; 51

Trave 13-12

" " 13-14 3500 x 2,90/2

p.p.

= 6300

= 5050

= 950

12300 Kg

Pilastri 14; 17; 47; 50

Trave 14-13

" " 14-15 3500 x 2,75/2

p.p.

= 5050

= 9600

= 950

15600 Kg

Pilastri 15; 16; 48; 49

Trave 15-14

" " 15-5

" " 15-21 2400 x 1,65/2

p.p.

= 9600

= 5600

= 2000

= 950

18150 Kg

Pilastri 21; 22; 54; 55

Trave 21-15

" " 21-23 $1200 > 1,60/2$ aliquota solaio $\frac{2,65 \times 650 \times 0,70}{2}$

2

p.p. $0,20 \times 0,90 \times 3,05 > 2500$

= 2000

= 950

= 600

= 1450

5000 Kg

Pilastri 23; 56

Trave 23-21

" " 23-22

" " 23-28 $1200 \times 1,90/2$ " " 28-29 $1200 \times 1,90/2$ p.p. $0,20 \times 1,90 \times 3,05 \times 2500$

= 950

= 950

= 1150

= 1150

= 2900

7100 Kg

Pilastri 24; 33; 57; 66

Trave 24-11

" " 24-25 $2900 \times 4,00/2$

p.p.

= 3450

= 5800

= 950

10200 Kg

Pilastri 25; 32; 58; 65

Trave 25-24

" " 25-26 $2900 \times 2,95/2$

p.p.

= 5800

= 4300

= 950

11050 Kg

Pilastri 26; 31; 59; 64

Trave 26-25

" " 26-27 $2900 \times 3,50/2$

p.p.

= 4300

= 5050

= 950

10300 Kg

Pilastrini 27; 30; 60; 63

Trave 27-26

" " 27-28 3750 x 3,40/2

p.p.

= 5050

= 6400

= 950

12400 Kg

Pilastrini 28; 29; 61; 62

Trave 28-27

" " 28-23

p.p.

= 6400

= 1150

= 950

8500 Kg

2-3-0 - Verifica ed armatura pilastri -

2-3-1 Carichi al piede dei pilastri

Pilastri	Copertura	Cop. IV	Cop. III	Cop. II	Cop. I	Cop. portico fondazione
1,10,34,43		9800	19600	29400	39200	49000
2,9,35,42		11600	23200	34800	46400	58000
3,8,36,41		11000	22000	33000	44000	55000
4,7,37,40		10000	20000	30000	40000	50000
5,6,38,39		11800	23600	35400	47200	59000
11,20,44,53		14650	29300	43950	58600	73000
12,19,45,52	20550	37600	51850	66100	80350	95000
13,18,46,51		12300	24600	36900	49200	62000
14,17,47,50	17600	33200	48800	64400	80000	96000
15,16,48,49	12400	35500	53650	71800	89950	108000
21,22,54,55	4200	9200	14200	19200	24200	29000
23,56		7100	14200	21300	28400	36000
24,33,57,66		10200	20400	30600	40800	51000
25,32,58,65		11050	22100	33150	44200	55000
26,31,59,64		10300	20600	30900	41200	52000
27,30,60,63		12400	24800	37200	49600	62000
28,29,61,62	3000	11500	20000	28500	37000*	46000

2-3-2 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
1,10,34,43	49	1000	8	4Ø14 + 4Ø12	30 x 40	1307	38
2,9,35,42	58	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	44
3,8,36,41	55	1150	9,2	4Ø14 + 4Ø12	30 x 40	1307	42
4,7,37,40	50	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	38
5,6,38,39	59	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	45
11,20,44,53	73	1500	12	4Ø14 + 6Ø12	30 x 50	1629	44
12,19,45,52	95	1950	15,6	10 Ø 14	40 x 50	2154	44
13,18,46,51	62	1300	10,4	4Ø14 + 4Ø12	30 x 40	1307	45
14,17,47,50	96	2000	16	10 Ø 14	40 x 50	2154	44
15,16,48,49	108	2250	18	12 Ø 14	40 x 60	2585	41
21,22,54,55	29	600	4,8	14 Ø 10	20 x 90	1910	<10
23,56	36	750	6	24 Ø 10	20 x 190	4037	<10
24,33,57,66	51	1050	8,4	4Ø14 + 1Ø12	30 x 40	1307	38
25,32,58,65	55	1150	9,2	4Ø14 + 4Ø12	30 x 40	1307	42
26,31,59,64	52	1050	8,4	4Ø14 + 4Ø12	30 x 40	1307	39
27,30,60,63	62	1300	10,4	4Ø14 + 4Ø12	30 x 40	1307	45
28,29,61,62	46	950	7,6	4Ø14 + 4Ø12	30 x 40	1307	35

Staffe Ø 8" / 15"

2-3-3 Sezione ed armatura pilastri piano I e II

N°	P tn	A' c cmq	Af cmq	Armatura	Sezione cm x cm	A ci cmq	σ c Kg/cmq
1,10,34,43	39	800	6,4	8 Ø 12	30 x 30	990	39
2,9,35,42	46	950	7,6	8 Ø 12	30 x 30	990	45
3,8,36,41	44	900	7,2	8 Ø 12	30 x 30	990	44
4,7,37,40	40	850	6,8	8 Ø 12	30 x 30	990	40
5,6,38,39	47	950	7,6	8 Ø 12	30 x 30	990	45
11,20,44,53	59	1200	9,6	4Ø14 + 4Ø12	30 x 40	1307	45
12,19,45,52	80	1650	13,2	4Ø14 + 6Ø12	40 x 50	2129	37
13,18,46,51	40	850	6,8	8 Ø 12	30 x 30	990	40
14,17,47,50	80	1650	13,2	4Ø14 + 6Ø12	40 x 50	2129	37
15,16,48,49	90	1850	14,8	10 Ø 14	40 x 50	2154	41
21,22,54,55	24	500	4,0	14 Ø 10	20 x 90	1910	<10
23,56	28	550	4,4	24 Ø 10	20 x 190	4037	<0
24,33,57,66	41	850	6,8	8 Ø 12	30 x 30	990	40
25,32,58,65	44	900	7,2	8 Ø 12	30 x 30	990	45
26,31,59,64	41	850	6,8	8 Ø 12	30 x 30	990	40
27,30,60,63	50	1000	8,4	8 Ø 12	30 x 30	990	45
28,29,61,62	37	750	6	8 Ø 12	30 x 30	990	37

Staffe Ø 6" / 15"

2-3-4 Sezione ed armatura pilastri piano III e IV

N°	P tn	A' c cmq	Af cmq	Armatura	Sezione cm x cm
1,10,34,43	20	400	3,2	8 Ø 10	30 x 30
2,9,35,42	23	450	3,6	8 Ø 10	30 x 30
3,8,36,41	22	450	3,6	8 Ø 10	30 x 30
4,7,37,40	20	400	3,2	8 Ø 10	30 x 30
5,6,38,39	24	500	4,0	8 Ø 10	30 x 30
11,20,44,53	29	600	4,8	4Ø12 + 4Ø10	30 x 30
12,19,45,52	52	1100	8,8	8 Ø 12	30 x 40
13,18,46,51	25	500	4,0	8 Ø 10	30 x 30
14,17,47,50	49	1000	8,0	8 Ø 12	30 x 40
15,16,48,49	54	1100	8,8	8 Ø 12	30 x 40
21,22,54,55	14	300	2,4	14 Ø 8	20 x 90
23,56	14	300	2,4	24 Ø 8	20 x 19
24,33,57,66	20	400	3,2	8 Ø 10	30 x 30
25,32,58,65	22	450	3,6	8 Ø 10	30 x 30
26,31,59,64	21	450	3,6	8 Ø 10	30 x 30
27,30,60,63	25	500	4,0	8 Ø 10	30 x 30
28,29,61,62	20	400	3,2	8 Ø 10	30 x 30

Staffe Ø 6" / 15"

2-3-5 Sezione ed armatura pilastri piano Volumi -tecnici

N°	P tn	A' c cmq	Armatura	Sezione cm x cm
12,19,45,52	21	450	8 Ø 10	30 x 30
14,17,47,50	18	400	8 Ø 10	30 x 30
15,16,48,49	12	250	8 Ø 10	30 x 30
21,22,54,55	4		14 Ø 8	20 x 90
28,29,61,62	3		8 Ø 10	30 x 30

Staffe Ø 6" /15"

2-4-0 Fondazioni

2-4-1 -Analisi di carico sul terreno-

pilastri. 2036000

trave 276900

2312900 Kg

2-4-2 -Tensione sul terreno-

$$\sigma_t = \frac{2312900}{1764600} = 1,31 \text{ Kg/cm}^2$$

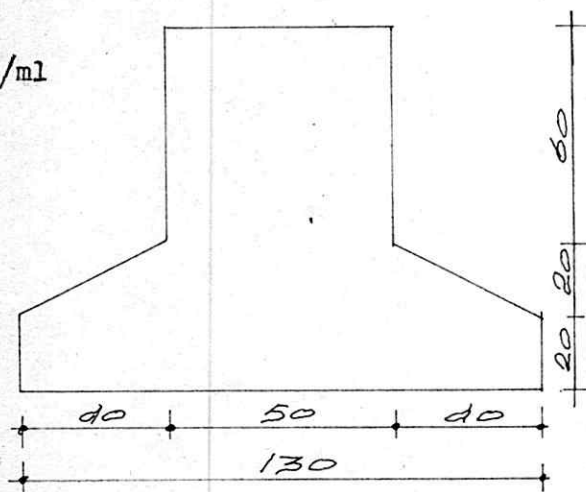
2-4-3 Travi 1-2-3-4-5-6-7-8-9-10; 24-25-26-27-28-29-30-31-32-33; 34-35-36-37
38-39-40-41-42-43; 57-58-59-60-61-62-63-64-65-66

Carico sulla trave

$$q = \frac{2036000}{1764600} \times 100 \times 130 = 15000 \text{ Kg/ml}$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 15000 \times 3,85^2 = 18500 \text{ Kgm}$$

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

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

$$\sigma_c = 39 \text{ Kg/cm}^2$$

$$A_f = 14,7 \text{ cm}^2$$

$$M_{camp} = 18500 \text{ Kgm}$$

$$H = 100''$$

$$b = 130''$$

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

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

$$A_f = 14,0 \text{ cm}^2$$

-Armatura trasversale-

$$M = \frac{1}{2} 15000 \times 0,40^2 = 1200$$

$$b = 100''$$

$$H = 40''$$

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

$$\sigma_c = 16 \text{ Kg/cm}^2$$

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

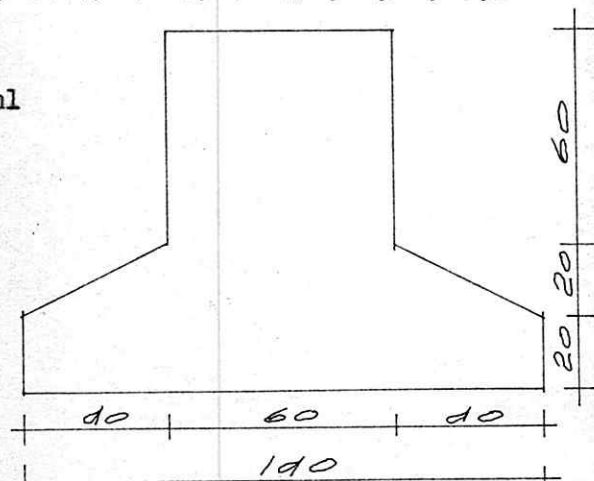
2-4-4 Travi 11-12-13-14-15-16-17-18-19-20; 44-45-46-47-48-49-50-51-52-53

Carico sulla trave

$$q = \frac{2036000}{1764600} \times 100 \times 140 = 16150 \text{ Kg/ml}$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 16150 \times 9,85^2 = 20000 \text{ Kgm}$$

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

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 37 \text{ Kg/cm}^2 \quad A_f = 16 \text{ cm}^2$$

$$M_{camp} = 20000 \text{ Kgm} \quad H = 100'' \quad b = 40''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 23 \text{ Kg/cm}^2 \quad A_f = 15,6 \text{ cm}^2$$

-Armatura trasversale-

$$M = \frac{1}{2} 6150 \times 0,40^2 = 1300 \text{ Kgm} \quad H = 40'' \quad b = 100''$$

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

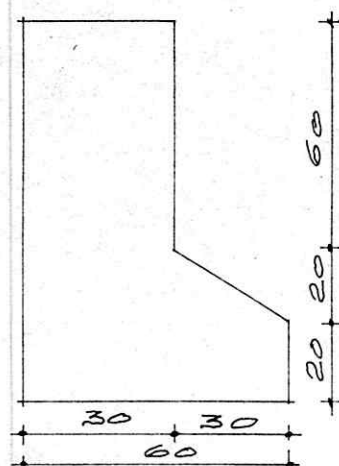
2-4-5 Travi 1-11-24; 10-20-33; 34-44-57; 43-53-66

Carico sulla trave

$$q = \frac{2036000}{1764600} \times 100 \times 60 = 7000 \text{ Kg/ml}$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 7000 \times 4,95^2 = 14300 \text{ Kgm}$$

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

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 45 \text{ Kg/cm}^2 \quad A_f = 11,4 \text{ cm}^2$$

$$M_{camp} = 14300 \text{ Kgm} \quad H = 100'' \quad b = 60''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 30 \text{ Kg/cm}^2 \quad A_f = 11,2 \text{ cm}^2$$

2-4-6 Travi 2-12-25; 3-13-26; 4-14-27; 5-15-21-23-28; 6-16-22-23-29-7-17-30;
8-18-31; 9-19-32; 35-45-58; 36-46-59; 37-47-60; 38-48-54-56-61; 39-49-
55-56-62; 40-50-63-41-51-64; 42-52-65

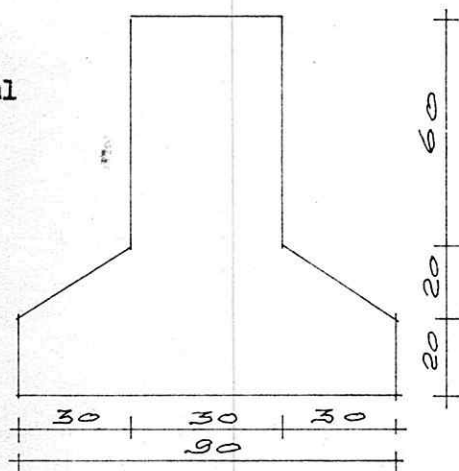
Carico sulla trave

$$q = \frac{2036000}{100} \times 100 \times 90 = 10350 \text{ Kg/ml}$$

$$1764600$$

-Caratteristiche geometriche-

Vedi disegno



-Armatura a flessione-

$$M_{inc} = \frac{1}{12} 10350 \times 4,95^2 = 21150 \text{ Kgm} \quad H = 100'' \quad b = 30''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 56 \text{ Kg/cm}^2 \quad A_f = 17,0 \text{ cm}^2$$

$$M_{camp} = 21150 \text{ Kgm} \quad H = 100'' \quad b = 90''$$

$$\sigma_f = 1400 \text{ Kg/cm}^2 \quad \sigma_c = 30 \text{ Kg/cm}^2 \quad A_f = 16,7 \text{ cm}^2$$

2-5-0 - Vano scala -

2-5-1 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

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

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

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

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

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

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

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

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

2-5-3 Calcolo delle travi a ginocchio

Analisi di carico

Rampa 850 x 1,20

= 1100

tompagno

= 400

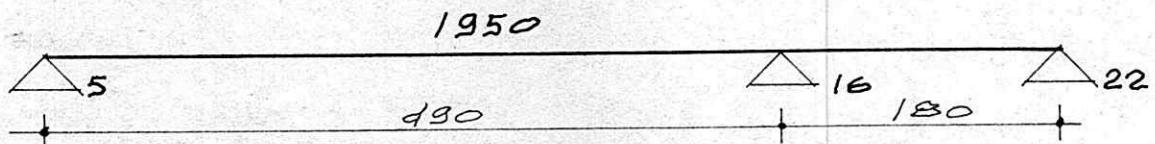
p.p. 0,30 x 0,60 x 1,00 x 2500

= 450

1950 Kg/ml

Verifica ed armatura a flessione

Schema di calcolo



$$M_5 = M_{15-16} = \frac{1}{14} 1950 \times 4,90^2 = 3350 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

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

$$M_{15} = \frac{1}{10} 1950 \times 4,90^2 = 4700 \text{ Kgm} \quad H = 60'' \quad b = 30''$$

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

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

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

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

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

Verifica a torsione della trave 5-16

$$m_i = \frac{1}{2} 850 \times 1,20^2 = 612 \text{ Kgm}$$

$$m_t = 612 + 1020 \times 0,15 = 765 \text{ Kgm}$$

$$t_i = 850 \times 1,20 = 1020 \text{ Kgm}$$

$$M_t = 765 \times \frac{4,90}{2} = 1875 \text{ Kgm}$$

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

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

-Armatura a torsione-

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

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

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

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

2-5-4 Calcolo della trave al pianerottolo di riposo

$$M = \frac{1}{12} 1200 \times 3,00^2 = 900 \text{ Kg} \cdot \text{m}$$

$$H = 40''$$

$$b = 20''$$

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

$$\sigma_c = 34 \text{ Kg/cm}^2$$

$$A_f = 1,8 \text{ cm}^2$$

2-6-0 BALCONI

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

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

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

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

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

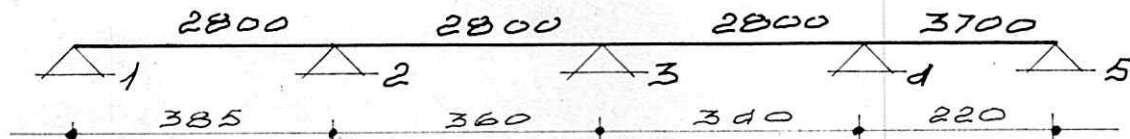
$$\sigma_f = 1400 \text{ Kg/cmq} \quad \sigma_c = 18 \text{ Kg/cmq} \quad A_f = 1,7 \text{ cmq/ml}$$

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

2-7-0 SOLAI 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 $3,68 \times 1,05 = 3,85 \text{ m.}$ incastro $M = \frac{1}{12} 325 \times 3,85^2 = 400 \text{ Kg} \cdot \text{m}$ $H = 25 \text{ cm}$ $\sigma_c = 23 \text{ Kg/cm}^2$ $A_f = 1,32 \text{ cm}^2$ campata $M = \frac{1}{14} 325 \times 3,85^2 = 345 \text{ Kg} \cdot \text{m}$ $H = 25 \text{ cm}$ $\sigma_c = 21 \text{ Kg/cm}^2$ $A_f = 1,12 \text{ cm}^2$ luce netta $4,28 \times 1,05 = 4,50 \text{ m.}$ incastro $M = \frac{1}{12} 325 \times 4,50^2 = 550 \text{ Kg} \cdot \text{m}$ $H = 25 \text{ cm}$ $\sigma_c = 27 \text{ Kg/cm}^2$ $A_f = 1,8 \text{ cm}^2$ $H = 18 \text{ cm}$ $\sigma_c = 36 \text{ Kg/cm}^2$ $A_f = 2,35 \text{ cm}^2$ campata $M = \frac{1}{14} 325 \times 4,50^2 = 470 \text{ Kg} \cdot \text{m}$ $H = 25 \text{ cm}$ $\sigma_c = 25 \text{ Kg/cm}^2$ $A_f = 1,55 \text{ cm}^2$ $H = 18 \text{ cm}$ $\sigma_c = 33 \text{ Kg/cm}^2$ $A_f = 2,0 \text{ cm}^2$ luce netta $2,83 \times 1,05 = 3,00 \text{ m.}$ incastro $M = \frac{1}{12} 325 \times 3,00^2 = 250 \text{ Kg} \cdot \text{m}$ $H = 25 \text{ cm}$ $\sigma_c = 18 \text{ Kg/cm}^2$ $A_f = 0,85 \text{ cm}^2$ campata $M = \frac{1}{14} 325 \times 3,00^2 = 200 \text{ Kg} \cdot \text{m}$ $H = 25 \text{ cm}$ $\sigma_c = < 15 \text{ Kg/cm}^2$ $A_f = 0,70 \text{ cm}^2$

2-8-0 -Calcolo a flessione della trave -

2-8-1 Travi 1-2-3-4-5; 6-7-8-9-10;
34-35-36-37-38; 39-40-41-42-43

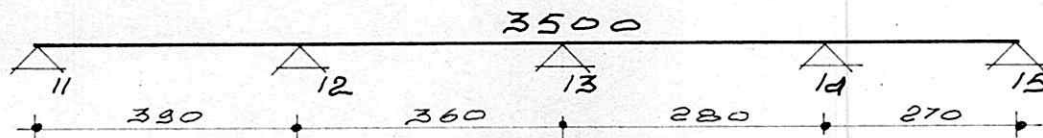


- Armatura a flessione -

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

Vedi Cross effettuato per il fabbricato A - Abis.

2-8-2 Travi 11-12-13-14-15; 16-17-18-19-20;
44-45-46-47-48; 49-50-51-52-53



- Armatura a flessione -

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

Sezione 11; 11-12

$$M = \frac{1}{14} 3500 \times 3,90^2 = 3800 \text{ Kgm} \quad \sigma_c = 60 \text{ Kg/cm}^2 \quad A_f = 13,2 \text{ cm}^2$$

Sezione 12

$$M = \frac{1}{10} 3500 \times 3,90^2 = 5300 \text{ Kgm} \quad \sigma_c = 73 \text{ Kg/cm}^2 \quad A_f = 18,5 \text{ cm}^2$$

Sezione 12-13

$$M = \frac{1}{16} 3500 \times 3,60^2 = 2800 \text{ Kgm.} \quad \sigma_c = 50 \text{ Kg/cm}^2 \quad A_f = 9,6 \text{ cm}^2$$

Sezione 13

$$M = \frac{1}{12} 3500 \times 3,60^2 = 3800 \text{ Kgm} \quad \sigma_c = 60 \text{ Kg/cm}^2 \quad A_f = 13,2 \text{ cm}^2$$

Sezione 13-14

$$M = \frac{1}{16} 3500 \times 2,80^2 = 1700 \text{ Kgm} \quad \sigma_c = 37 \text{ Kg/cm}^2 \quad A_f = 5,8 \text{ cm}^2$$

Sezione 14

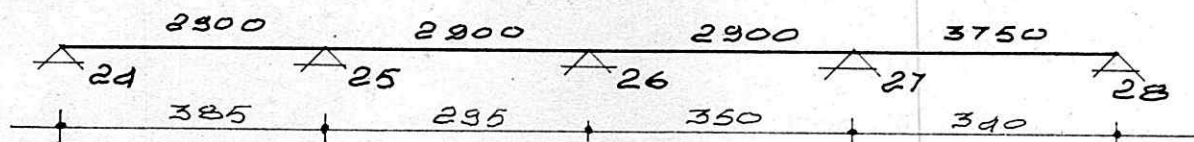
$$M = \frac{1}{10} 3500 \times 2,70^2 = 2550 \text{ Kgm} \quad \sigma_c = 47 \text{ Kg/cm}^2 \quad A_f = 8,7 \text{ cm}^2$$

Sezione 14-15; 15

$$M = \frac{1}{14} 3500 \times 2,70^2 = 1800 \text{ Kgm} \quad \sigma_c = 38 \text{ Kg/cm}^2 \quad A_f = 6,0 \text{ cm}^2$$

$$\text{Taglio massimo } T = 6800 \text{ Kg} \quad \tau_{\max} = \frac{6800}{90 \times 0,9 \times 23} = 3,6 \text{ Kg/cm}^2$$

2-8-3 Travi 24-25-26-27-28; 29-30-31-32-33;
57-58-59-60-61; 62-63-64-65-66



- Armatura a flessione -

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

Sezione 24; 24-25

$$M = \frac{1}{14} 2900 \times 3,85^2 = 3050 \text{ Kgm} \quad \sigma_c = 61 \text{ Kg/cm}^2 \quad A_f = 10,6 \text{ cm}^2$$

Sezione 25

$$M = \frac{1}{10} 2900 \times 3,85^2 = 4300 \text{ Kgm} \quad \sigma_c = 76 \text{ Kg/cm}^2 \quad A_f = 15,4 \text{ cm}^2$$

Sezione 25-26

$$M = \frac{1}{16} 2900 \times 2,95^2 = 1550 \text{ Kgm}$$

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

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

Sezione 26

$$M = \frac{1}{12} 2900 \times 3,50^2 = 2950 \text{ Kgm}$$

$$\sigma_c = 60 \text{ Kg/cm}^2$$

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

Sezione 26 - 27

$$M = \frac{1}{16} 2900 \times 3,50^2 = 2200 \text{ Kgm}$$

$$\sigma_c = 50 \text{ Kg/cm}^2$$

$$A_f = 7,5 \text{ cm}^2$$

Sezione 27

$$M = \frac{1}{10} 3750 \times 3,40^2 = 4300 \text{ Kgm}$$

$$\sigma_c = 76 \text{ Kg/cm}^2$$

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

Sezione 27-28; 28

$$M = \frac{1}{14} 3750 \times 3,40^2 = 3100 \text{ Kgm}$$

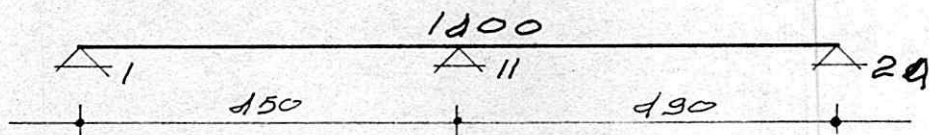
$$\sigma_c = 61 \text{ Kg/cm}^2$$

$$A_f = 10,7 \text{ cm}^2$$

Taglio massimo $T = 6300 \text{ Kg}$

$$\tau_{\max} = \frac{6300}{\quad} = 4,3 \text{ Kg/cm}^2$$

2-8-4 Travi 1-11-24; 10-20-33; 34-44-57; 43-53-66



- Armatura a flessione -
 $\sigma_f = 1400 \text{ Kg/cm}^2$ $H_f = 25 \text{ cm}$ $b = 50 \text{ cm}$

Sezione 1; 1-11

$$M = \frac{1}{14} 1400 \times 4,50^2 = 2050 \text{ Kgm}$$

$$\sigma_c = 59 \text{ Kg/cm}^2$$

$$A_f = 7,1 \text{ cm}^2$$

Sezione 11

$$M = \frac{1}{12} 1400 \times 4,90^2 = 2800 \text{ Kgm}$$

$$\sigma_c = 71 \text{ Kg/cm}^2$$

$$A_f = 9,8 \text{ cm}^2$$

Sezione 11-24; 24

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

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

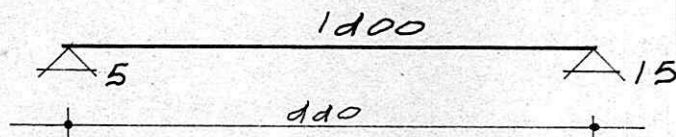
$$A_f = 8,5 \text{ cm}^2$$

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

$$\tau_{\max} = \frac{3450}{50 \times 0,9 \times 23}$$

$$= 3,3 \text{ Kg/cm}^2$$

2-8-5 Travi 5-15; 6-16; 38-48; 39-49



- Armatura a flessione -

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

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

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

Sezione 5; 5-15; 15

$$M = \frac{1}{12} 1400 \times 4,40^2 = 2250 \text{ Kgm}$$

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

$$A_f = 7,8 \text{ cm}^2$$

2-8-6 Travi 21-23; 21-24; 23-28; 23-29;
54-56; 55-56; 56-61; 56-62

Sezione 21; 21-23; 23

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

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

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

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

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

2-9-0

- Calcolo della soletta ascensore -

2-9-1 Analisi di carico

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

= 625

sovracc. perm.

= 75

sovracc. accid.

= 150

850 Kg/ml

macchinario

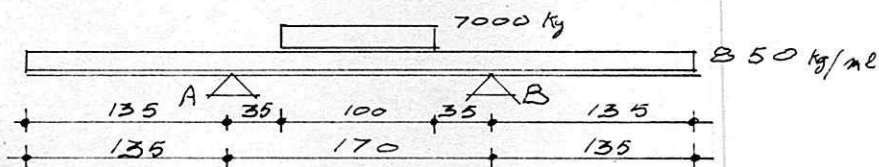
= 6000

basamento

= 1000

7000 Kg

2-9-2 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

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

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

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

$$\sigma_c = 19 \text{ Kg/cm}^2 \quad \sigma_f = 508 \text{ Kg/cm}^2$$

2-9-3 Coefficienti di sicurezza

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

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

2-9-4 - 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

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

2-10-0 - Copertura Volumi tecnici-

2-10-1 -Solai-

Carico per nervatura $q = 650 \times 0,50 = 325 \text{ Kg/ml}$ $\sigma_f = 1400 \text{ Kg/cm}^2$ $H = 25 \text{ cm}$ $b = 50 \text{ cm}$

luce netta 2,70 m.

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

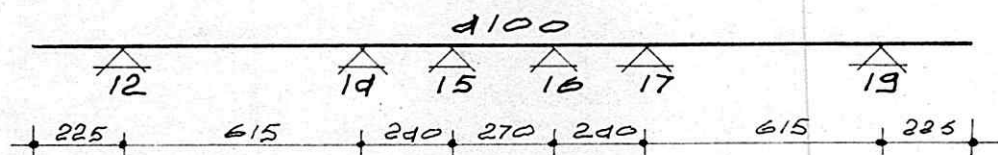
luce netta 1,50 m.

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

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

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

2-10-2 Travi S-12-14-15-16-17-19-S
S-45-47-48-49-50-52-S



- Armatura a flessione -

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

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

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

Sezione 12, 14

$$M = \frac{1}{12} 4100 \times 6,15^2 = 12900 \text{ Kgm}$$

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

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

Sezione 12-14

$$M = \frac{1}{16} 4100 \times 6,15^2 = 9700 \text{ Kgm}$$

$$\sigma_c = 47 \text{ Kg/cm}^2$$

$$A_f = 9,8 \text{ cm}^2$$

Sezione 14-15, 15-16

$$M = \frac{1}{18} 4100 \times 2,70^2 = 1650 \text{ Kgm}$$

$$\sigma_c = 17 \text{ Kg/cm}^2$$

$$A_f = 1,6 \text{ cm}^2$$

Sezione 15

$$M = \frac{1}{12} 4100 \times 2,70^2 = 2500 \text{ Kgm}$$

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

$$A_f = 2,45 \text{ cm}^2$$

$$\text{Taglio massimo } T = 12600 \text{ Kg}$$

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

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