La struttura: cenni sulle tecniche di costruzione nei vari materiali

Istituto Statale di Istruzione Superiore
"A. Omodeo" - Mortara

06/02/2019

Ing. Giovanni Moncalieri

Costruzioni del Passato

600 a.C.

La scienza delle costruzioni ed il suo sviluppo storico

I due sentieri:

-" il primo sentiero, percorso da uomini che intendevano risolvere nella equilibrata armonia delle leggi geometriche governanti l'ordine classico ogni genere di problemi che caratterizzavano il costruire, attraversava per intero i secoli dell'Umanesimo e del Rinascimento e si misura con monumenti di impareggiabile qualità

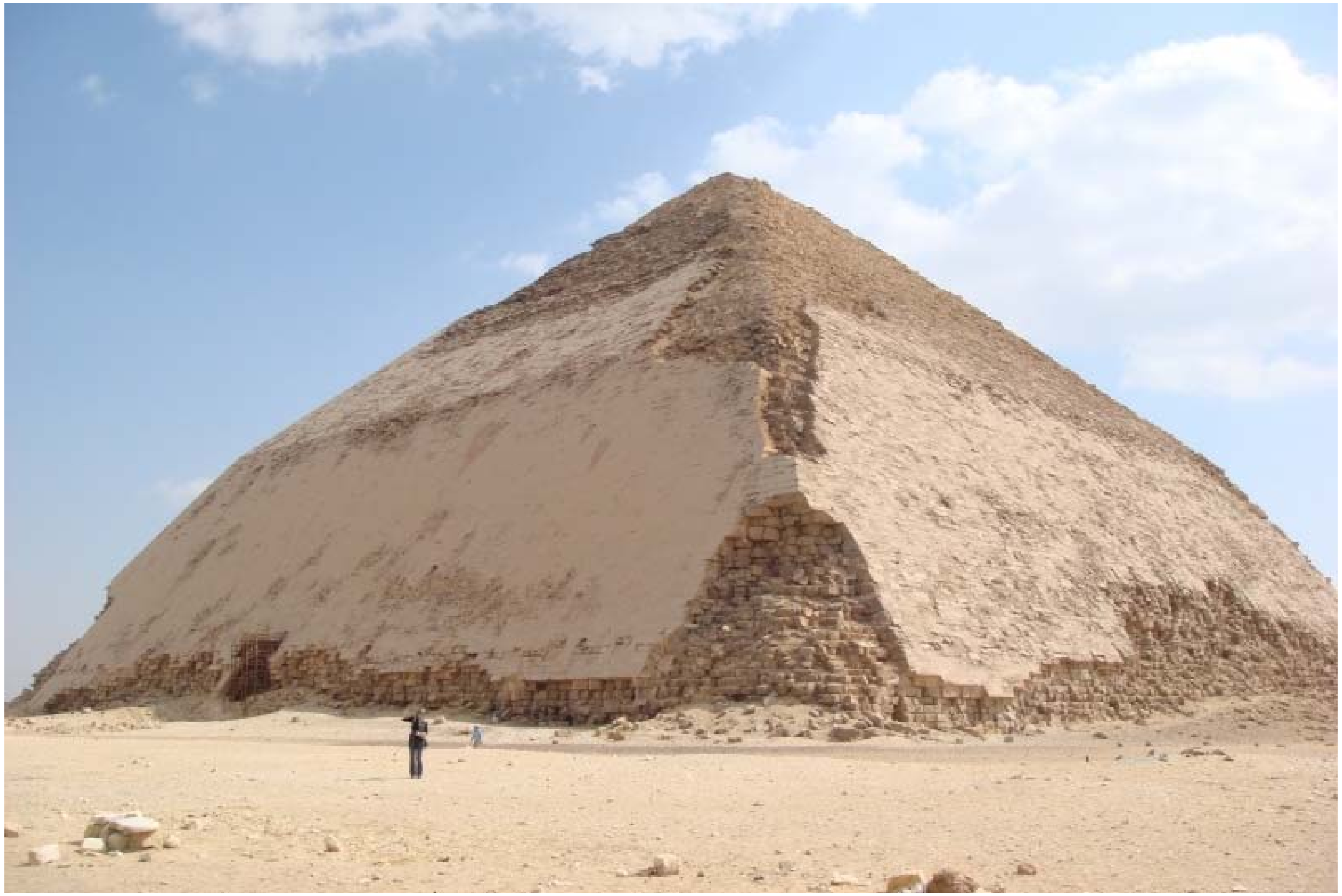
- Con Leonardo e poi con Galileo, si apre il Secondo sentiero, la direzione del quale è ancora in parte avvolta nella nebbia, ma dove la bussola è lo strumento matematico

E. Benvenuto – La scienza delle Costruzioni ed il suo sviluppo storico

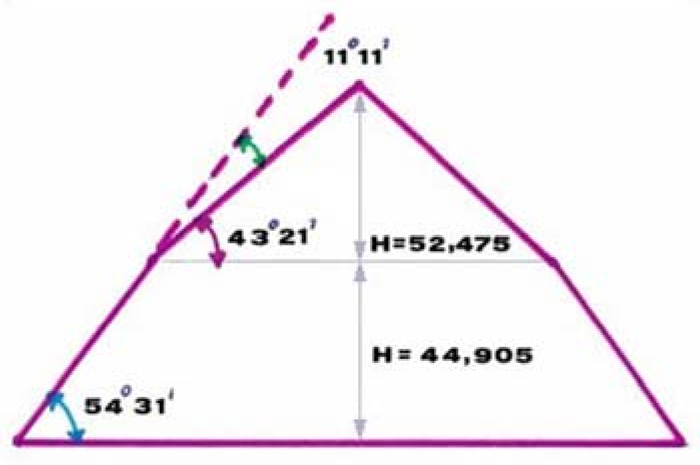
Le grandi Piramidi

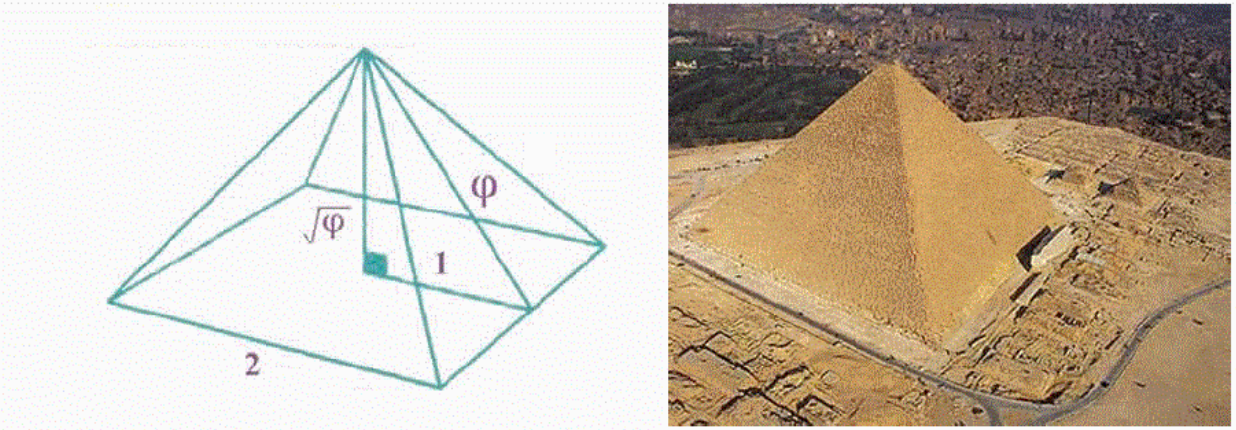
2000 a. C.

La Piramide di Snefru



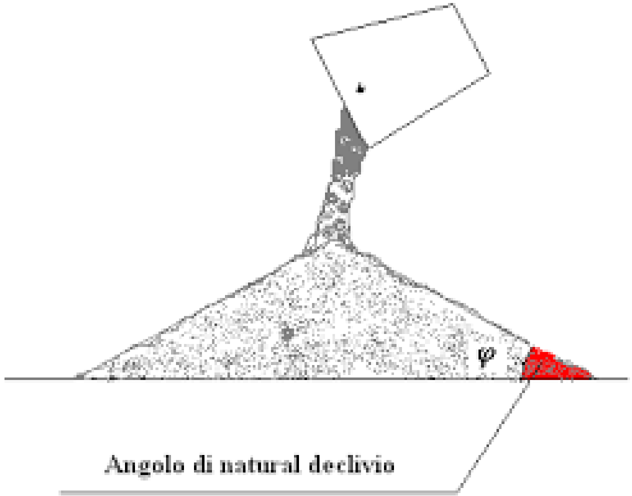
La Piramide di Snefru

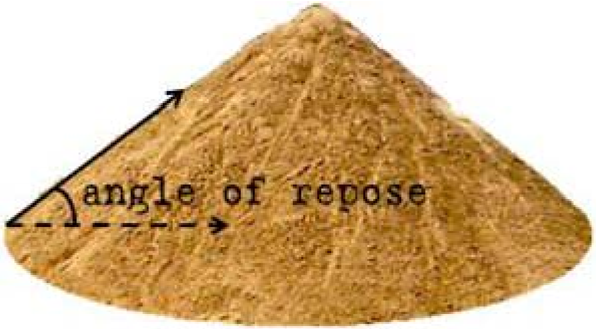

11°11'
43°21'
H=52,475
H= 44,905
54°31'



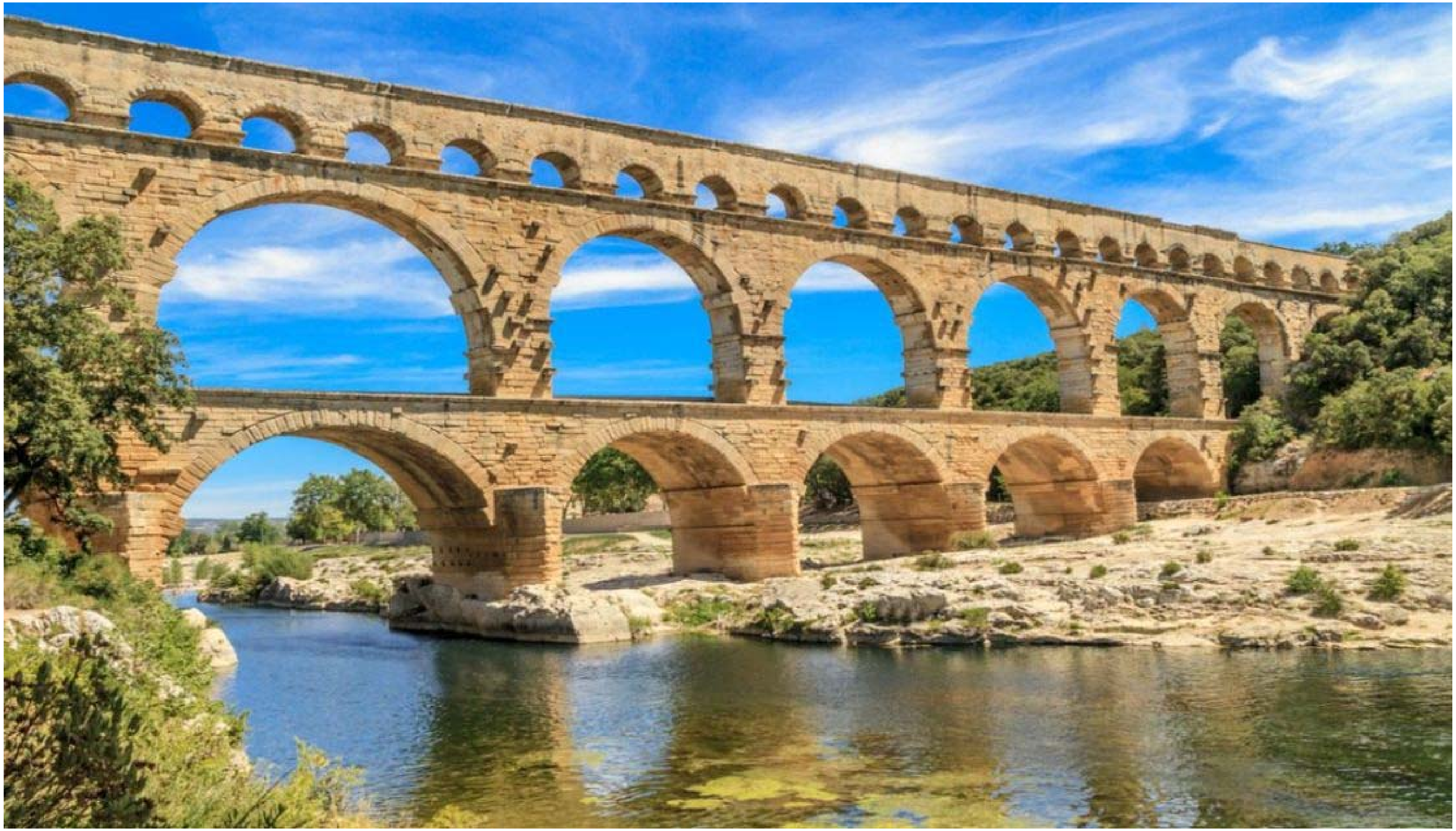
L'altezza della faccia laterale è ottenuta dalla semi-base moltiplicata per il ***numero aureo****, mentre per ottenere l'altezza la semi-base si deve moltiplicare per la radice di questo.*

Angolo di Attrito


φ
Angolo di natural declivio

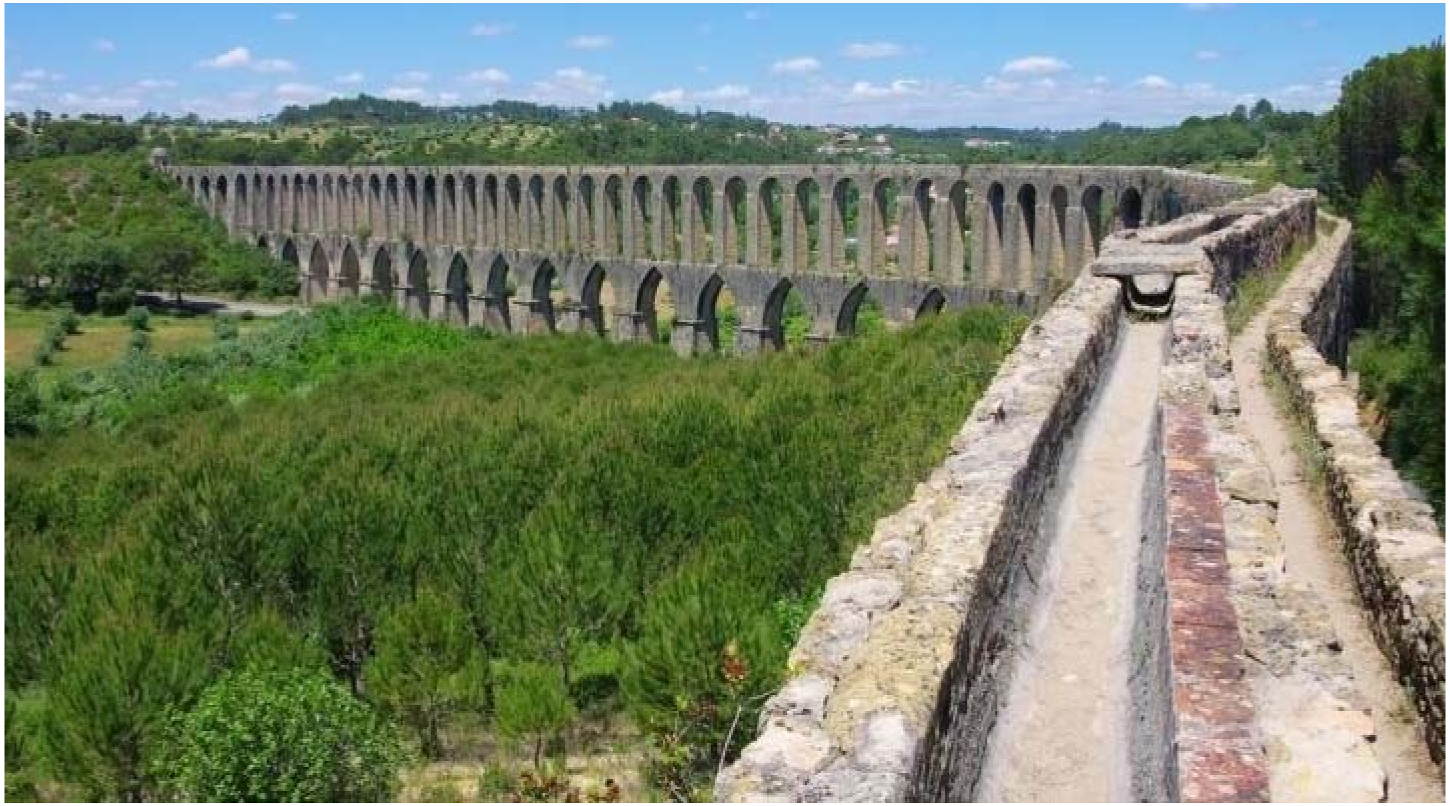

angle of repose

Acquedotto Romano

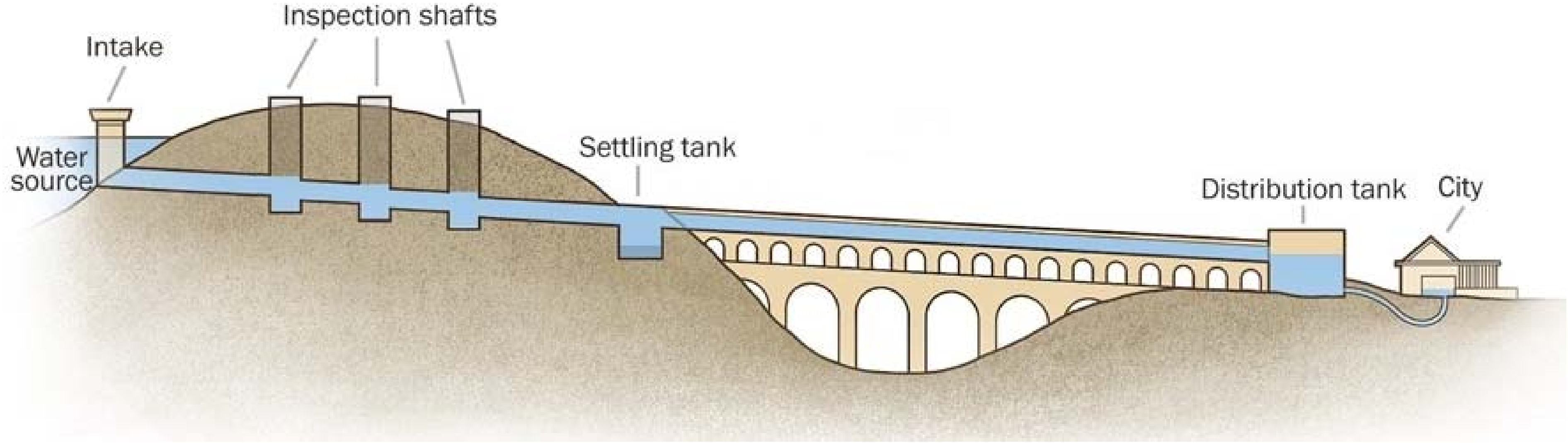


17 a. C.

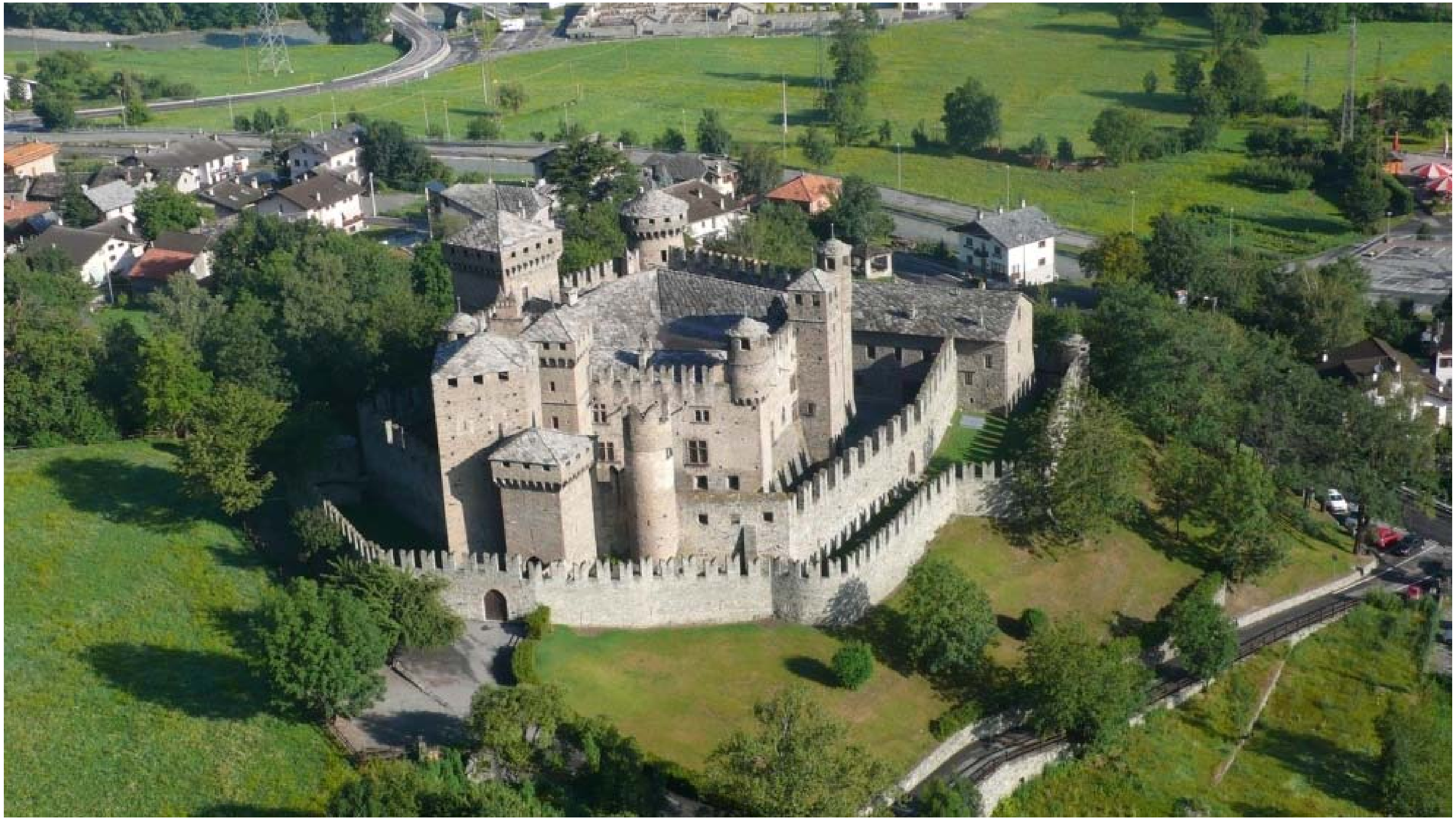
Acquedotto Romano



Acquedotto Romano

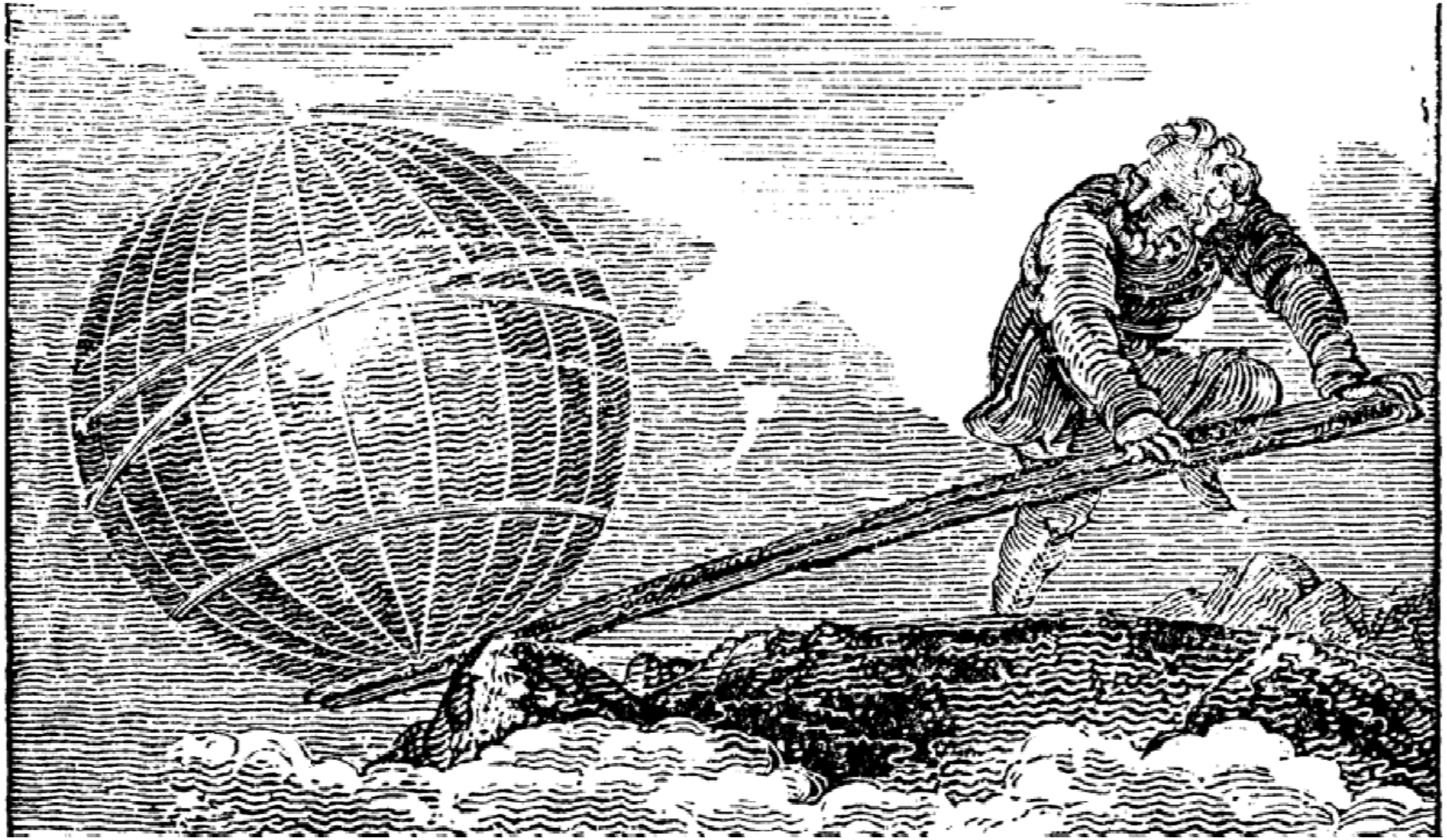

Inspection shafts
Intake
Water source
Settling tank
Distribution tank
City

Castello Medievale di Fenis



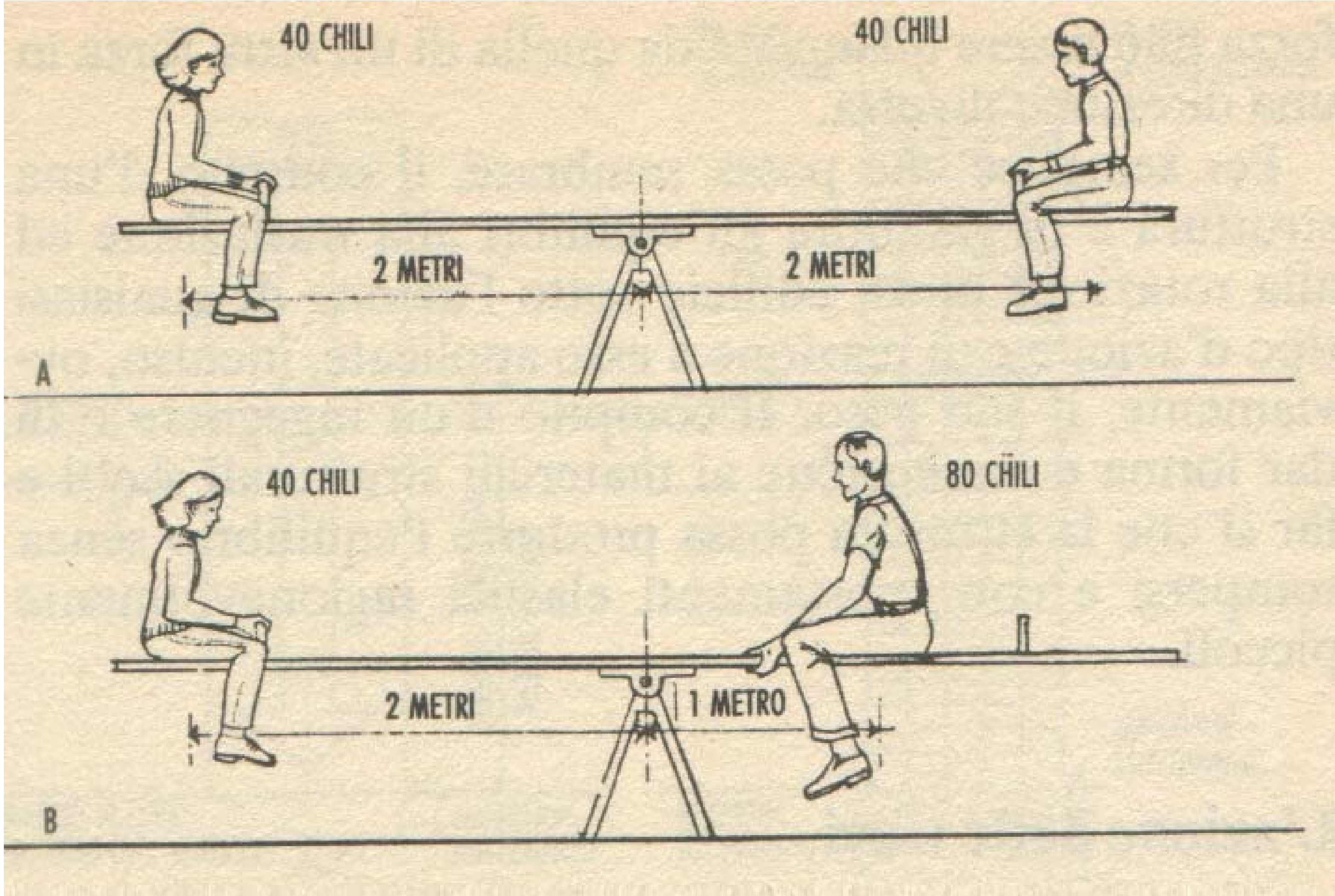
1350

Archimede

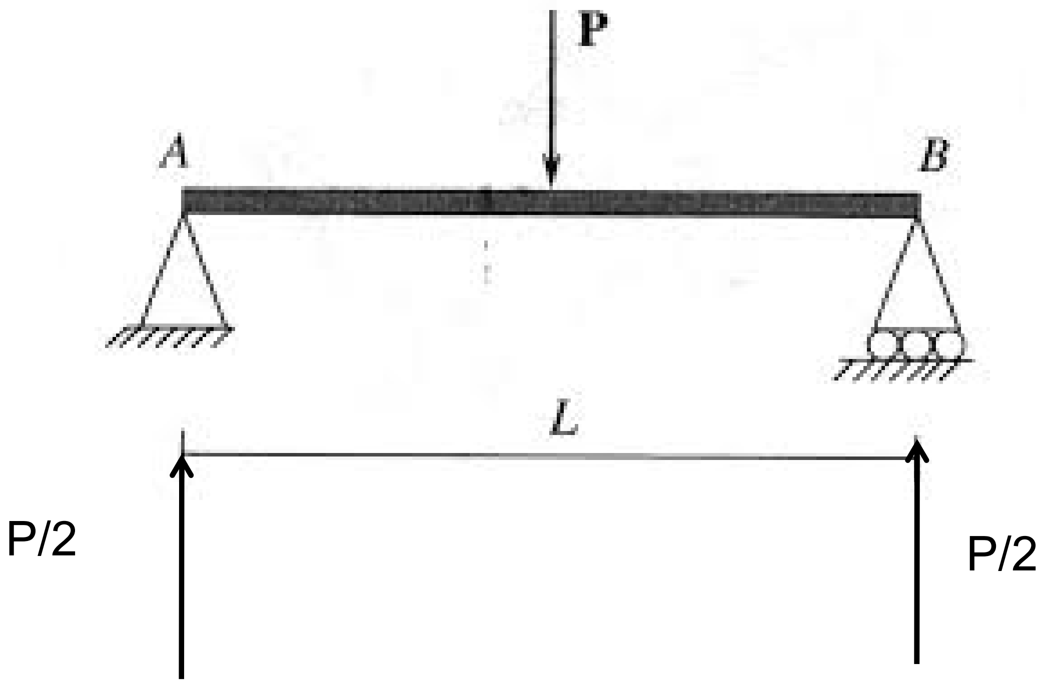


212 a.C.

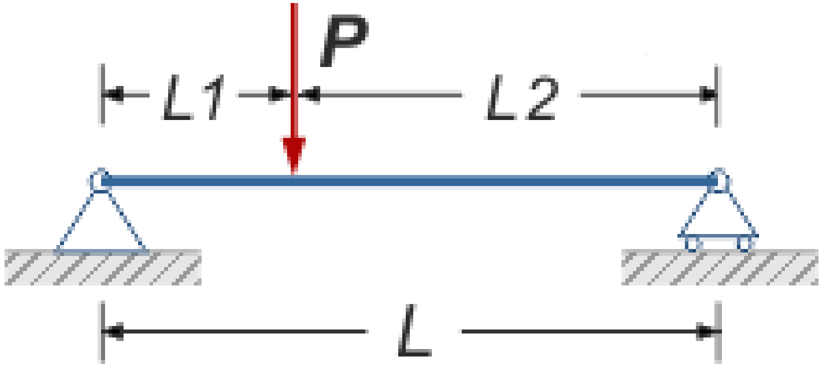
Principio della Leva


40 CHILI
40 CHILI
2 METRI
2 METRI
A
40 CHILI
80 CHILI
2 METRI
1 METRO
B

Trave su due appoggi

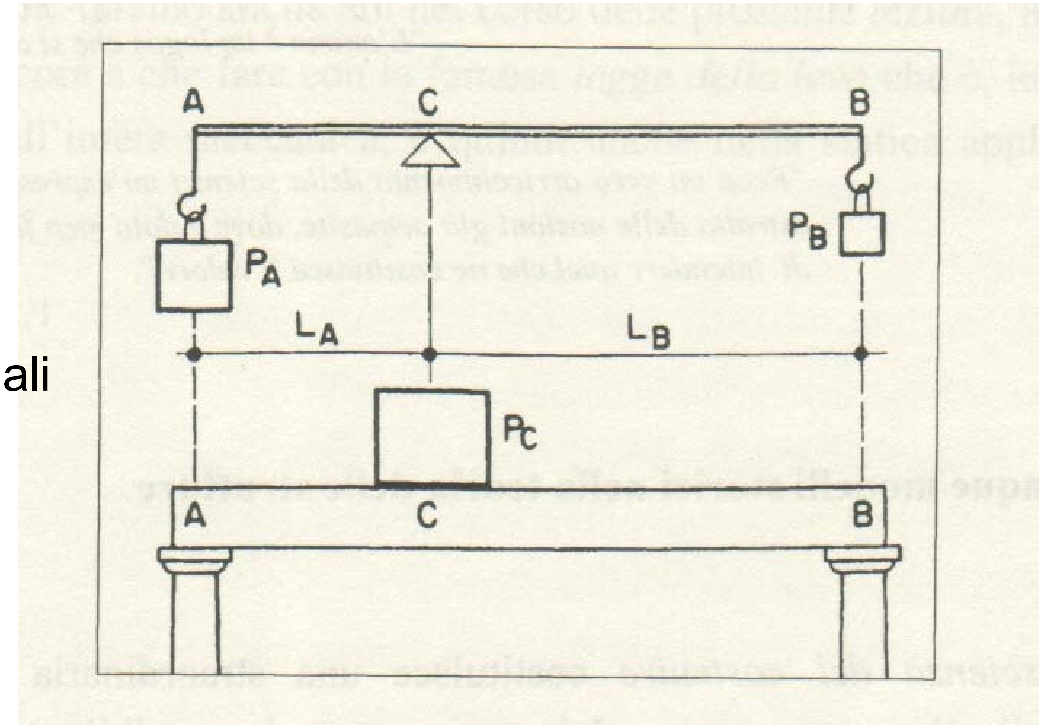


Trave su due appoggi

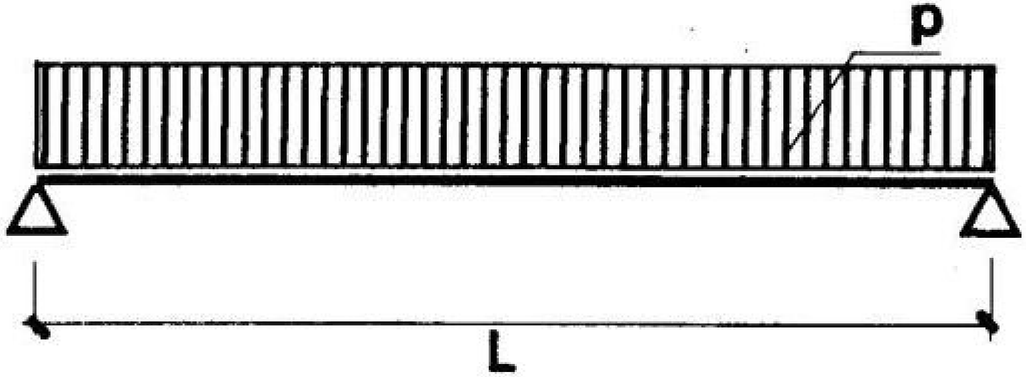


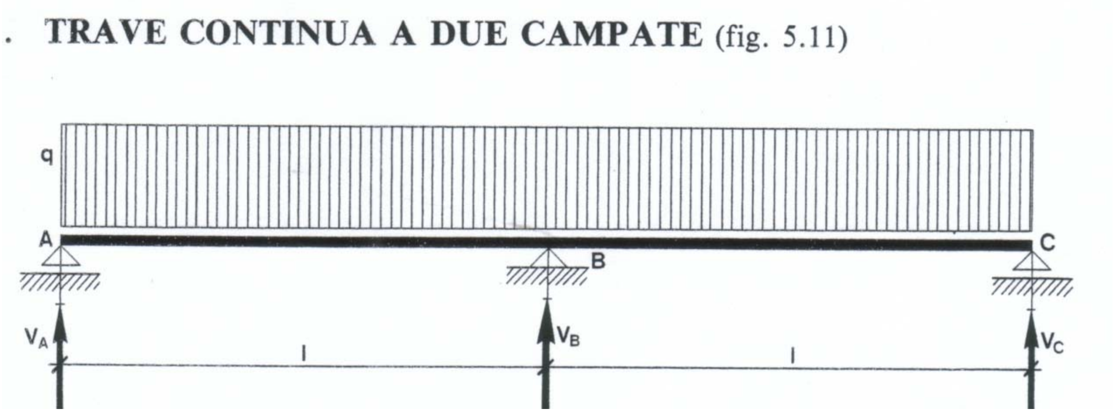
$$\begin{cases}\sum \bar{F} = 0 \\ \sum M = 0\end{cases}$$

Equazioni cardinali
Della statica


A
C
B
P_B
P_A
L_A
L_B
P_C
A
C
B

Trave su due appoggi con carico distribuito

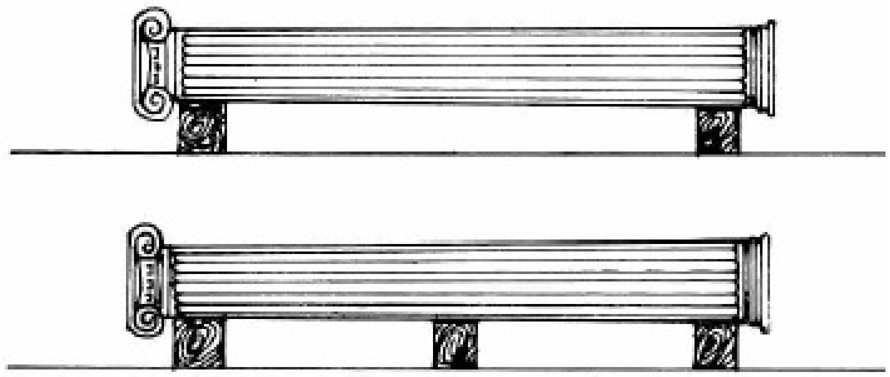


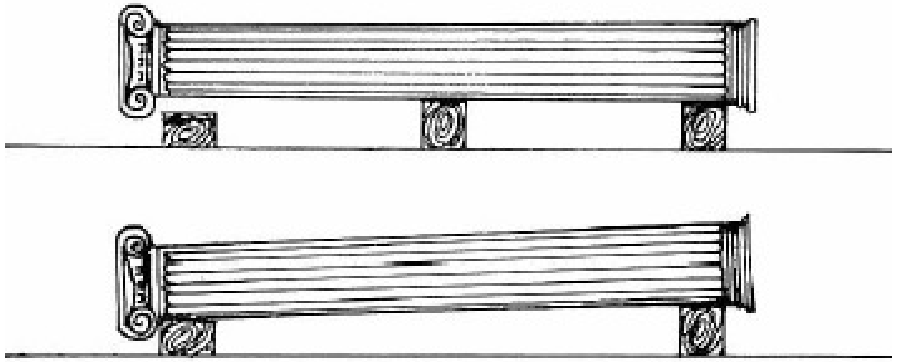


. TRAVE CONTINUA A DUE CAMPATE (fig. 5.11)

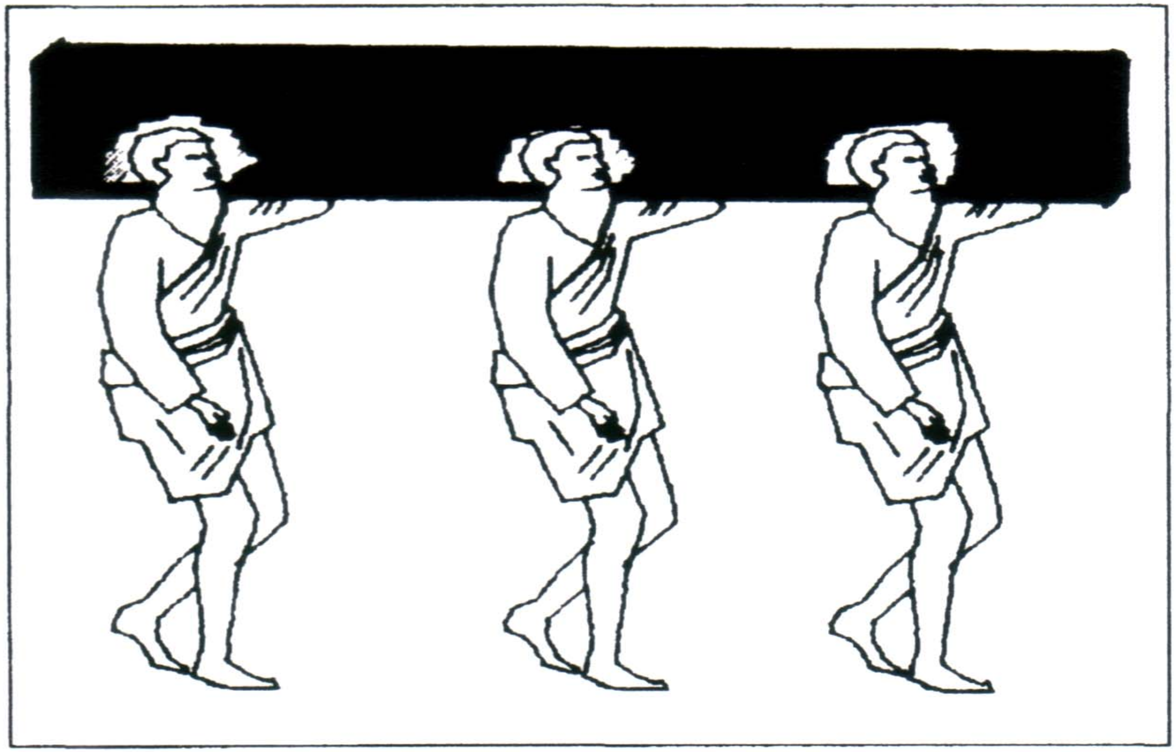
q
A
B
C
V_A
V_B
V_C
l
l

Trave su tre appoggi



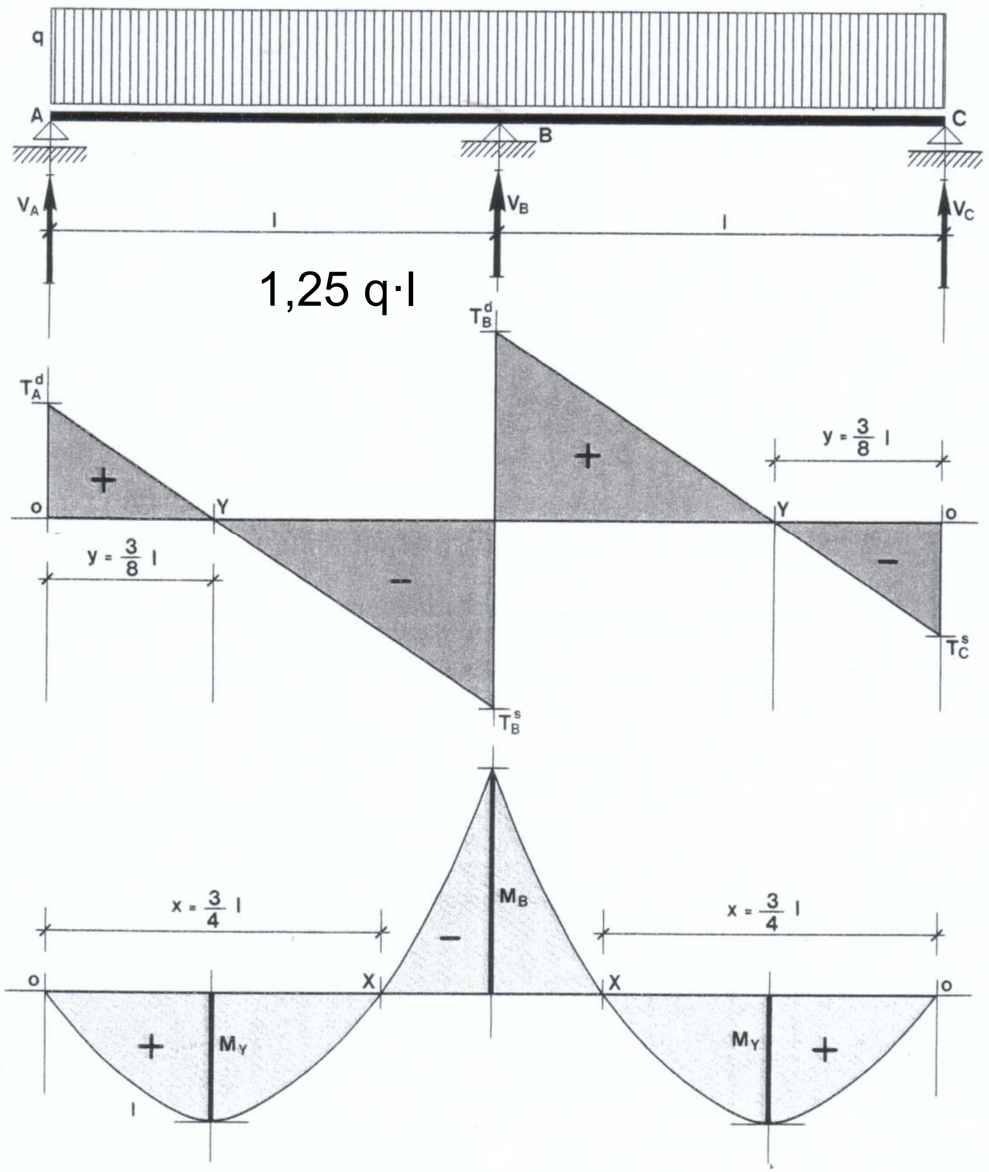


Ricerca della soluzione



Il problema dei tre portatori

. TRAVE CONTINUA A DUE CAMPATE (fig. 5.11)

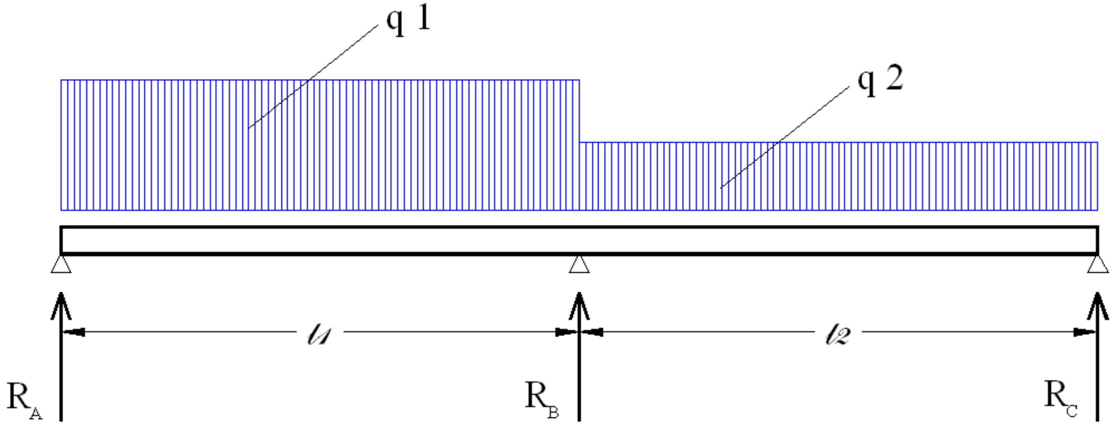

q
A
B
C
V_A
V_B
V_C
l
l
T_A^d
T_B^d
+
o
Y
$y = \frac{3}{8}$ l
−
+
Y
o
$y = \frac{3}{8}$ l
−
T_B^s
T_C^s
$x = \frac{3}{4}$ l
M_B
−
$x = \frac{3}{4}$ l
o
X
X
o
+
M_Y
M_Y
+
l

0,375 q·l

1,25 q·l

0,375 q·l

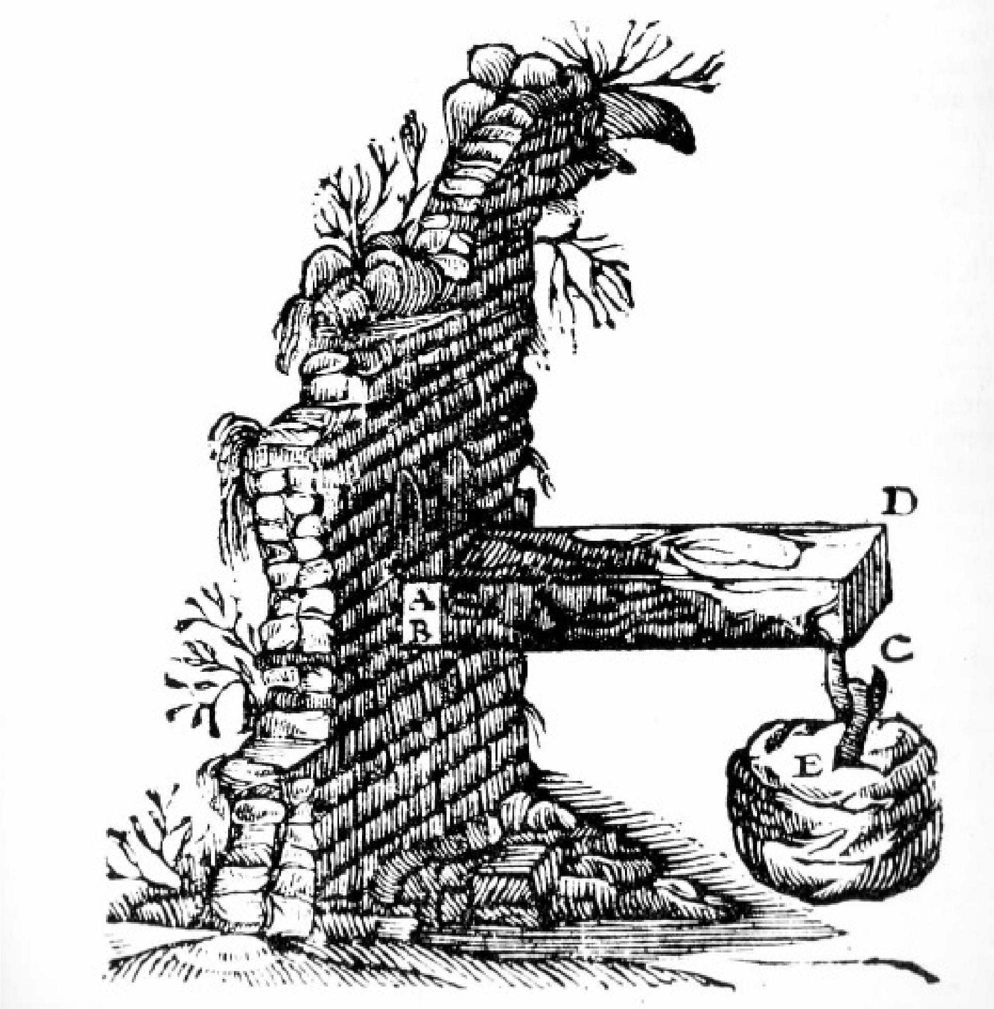
Calcolo delle travi continue - Clapeyron 1857


q 1
q 2
l1
l2
R_A
R_B
R_C

$$M_A \cdot l_1 + 2M_B \cdot (l_1 + l_2) + M_C \cdot l_2 = \frac{1}{4} \cdot \left(q_1 \cdot l_1^3 + q_2 \cdot l_2^3\right)$$

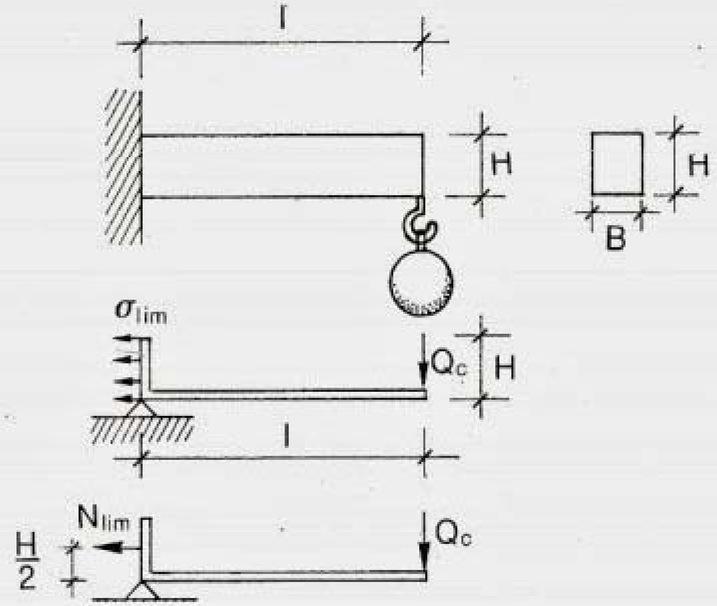
Equazione dei tre Momenti

Galileo 1638



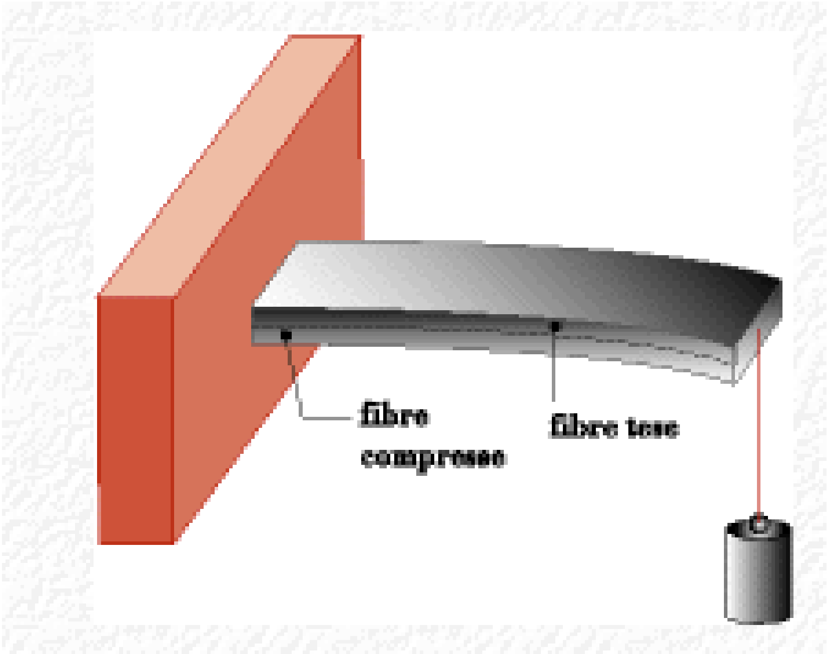
Discorsi intorno a due nuove scienze

Galileo 1638

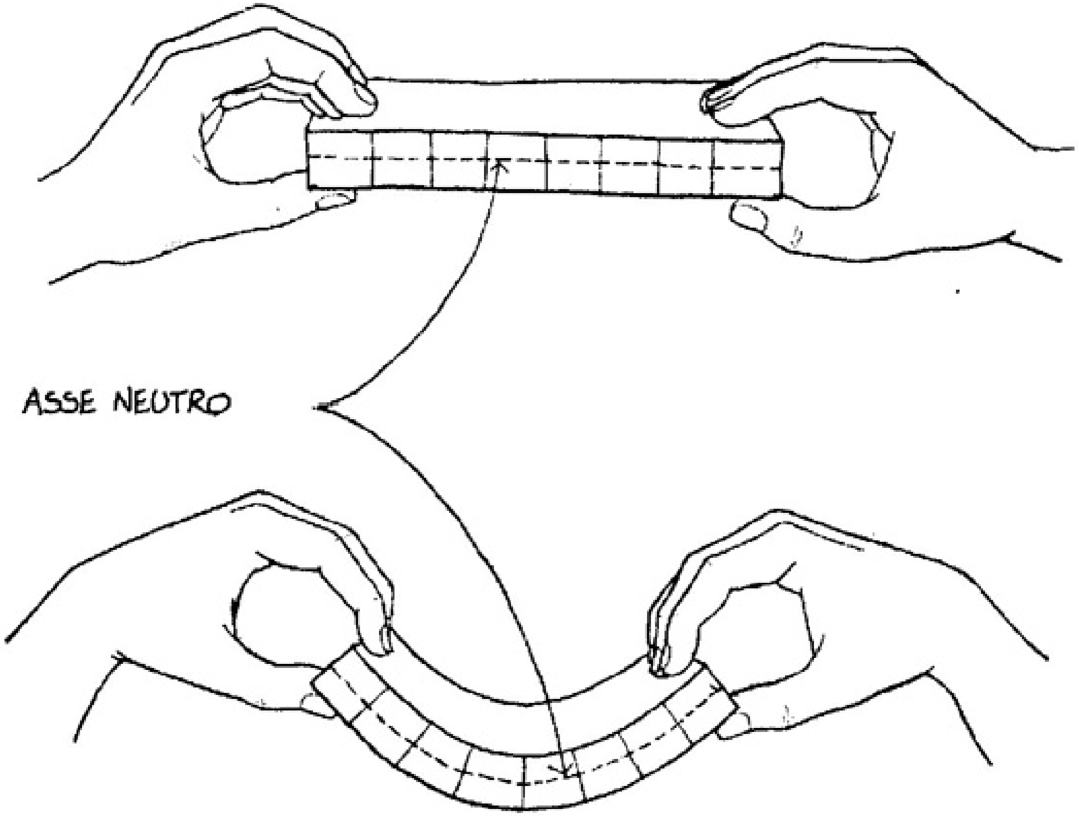

l
H
H
B
σlim
Qc
H
l
Nlim
Qc
H/2

Q_c = carico limite

M_{lim} = momento resistente limite

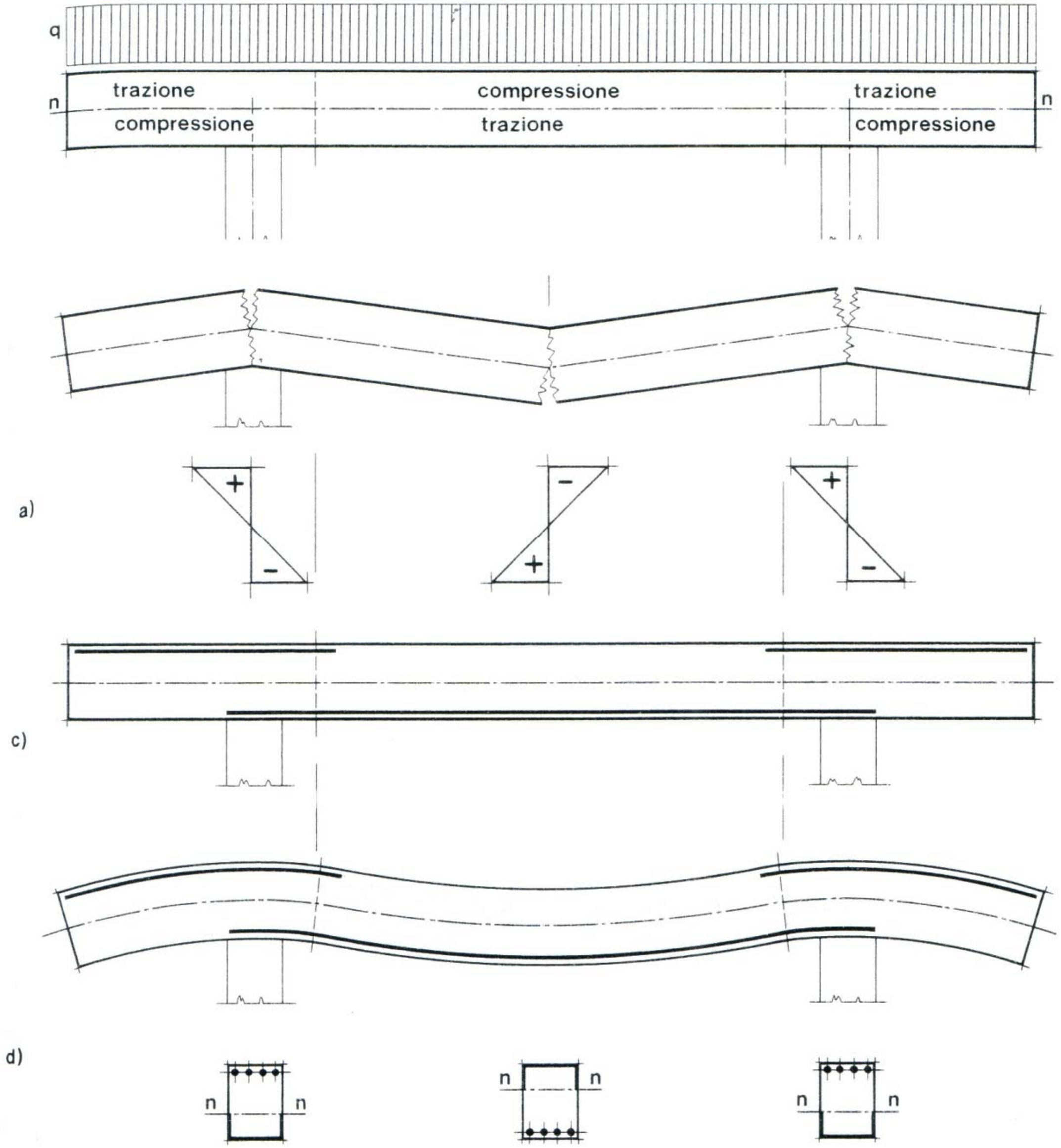

fibre compresse
fibre tese

Asse neutro


ASSE NEUTRO

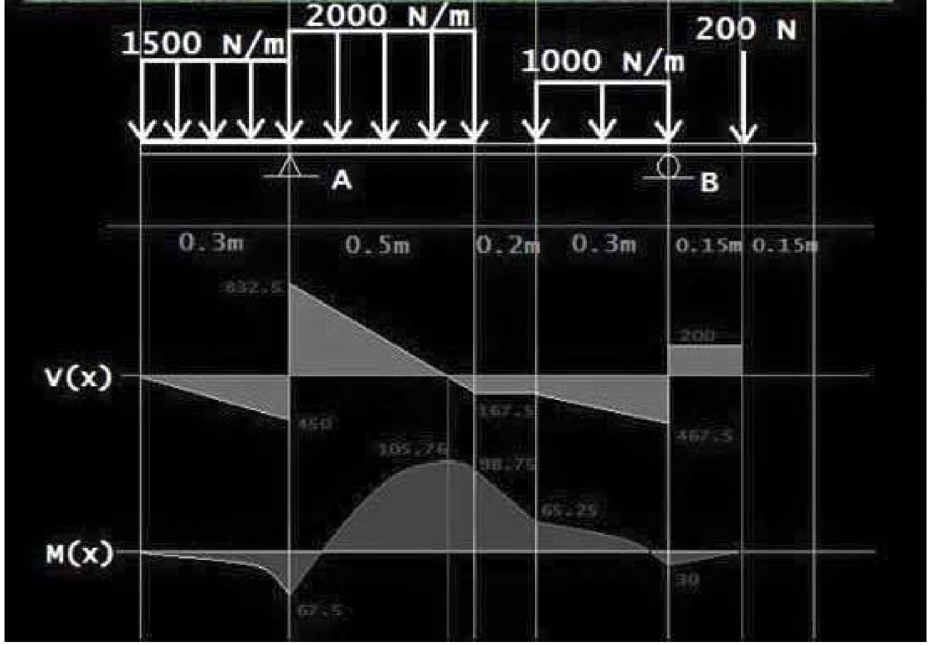
3.3

Tensioni e deformazione trave su due appoggi

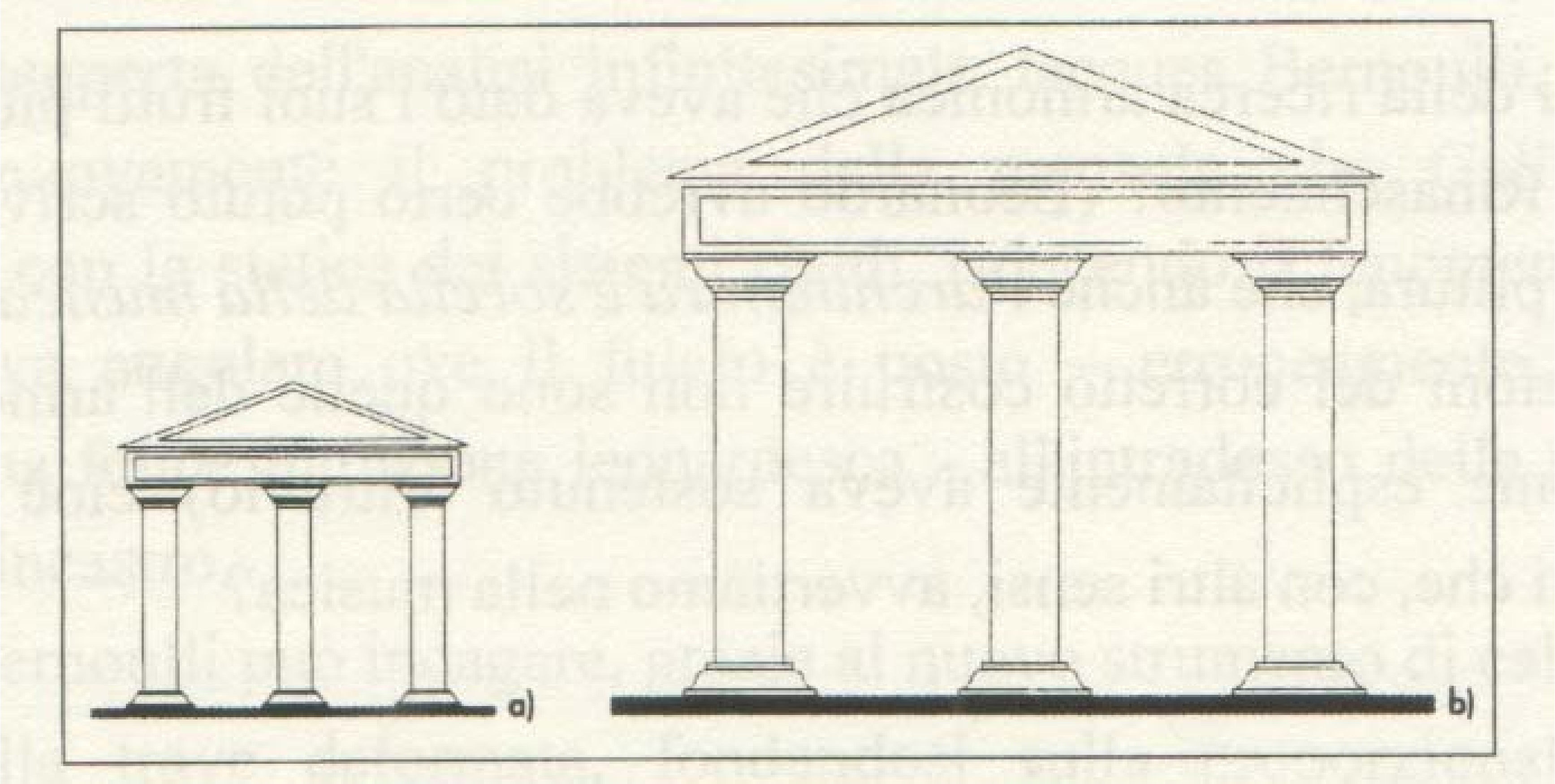

q
n
trazione
compressione
compressione
trazione
trazione
compressione
n
a)
+
−
−
+
+
−
c)
d)
n
n
n
n
n
n

Realtà

Modello
Matematico


1500 N/m
2000 N/m
1000 N/m
200 N
A
B
0.3m
0.5m
0.2m
0.3m
0.15m
0.15m
V(x)
832.5
450
167.5
200
467.5
M(x)
105.76
98.75
65.25
67.5
30

Problema dimensionale



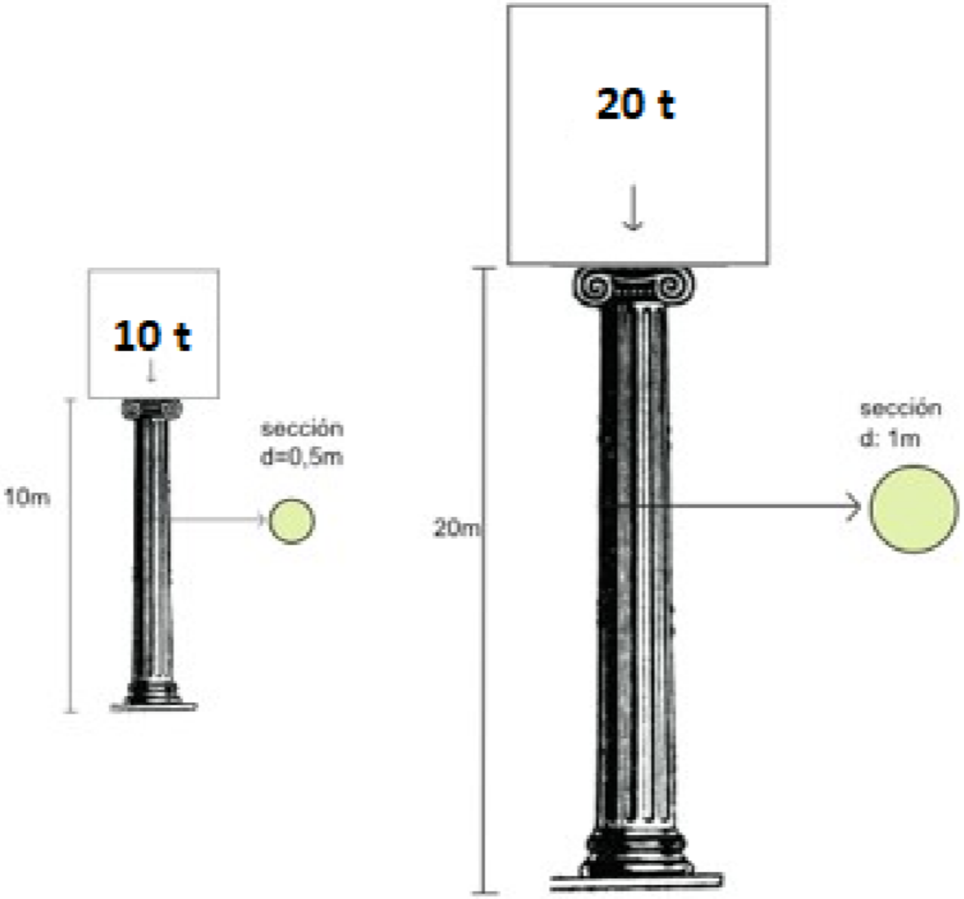
Galileo 1635

"…Un ponte sia potente a sostenere 1000 libbre. Cercasi ora se un altro ponte del medesimo legname, ma in tutti i suoi membri accresciuto in quadrupla proporzione, sarà potente a reggere il peso di 4000 libbre

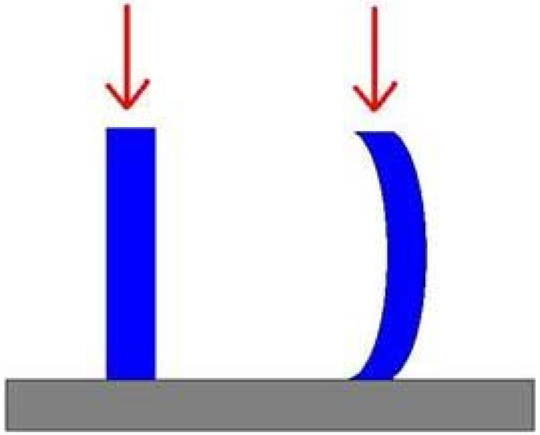
Dove io dico di no: e talmente dico di no che potrebbe anco accadere che anche per il solo peso proprio lo fiaccasse"

Da Discorsi intorno a due nuove scienze

Problema dimensionale


10 t
10m
sección
d=0,5m
20 t
20m
sección
d: 1m

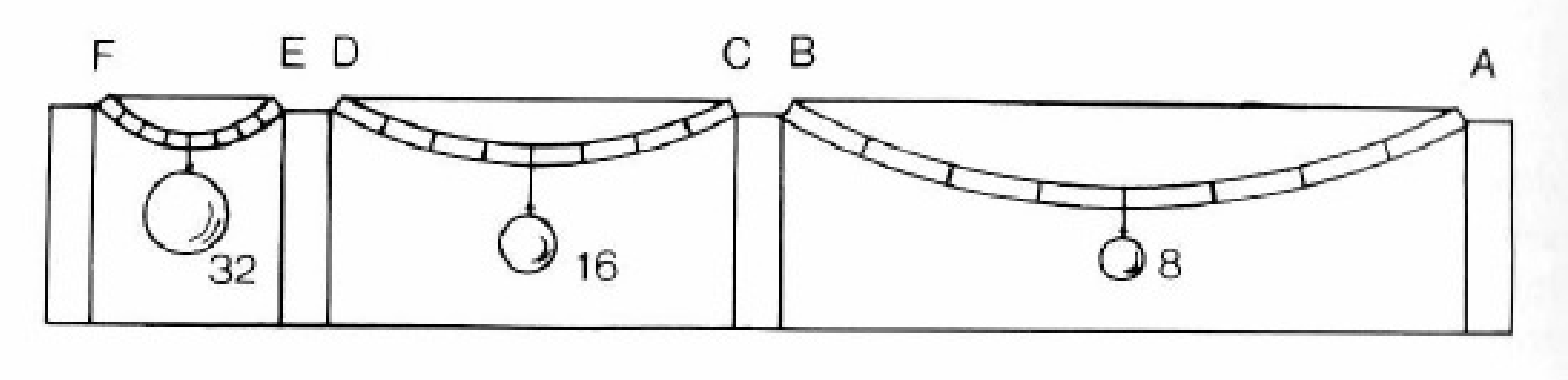
Carico di punta



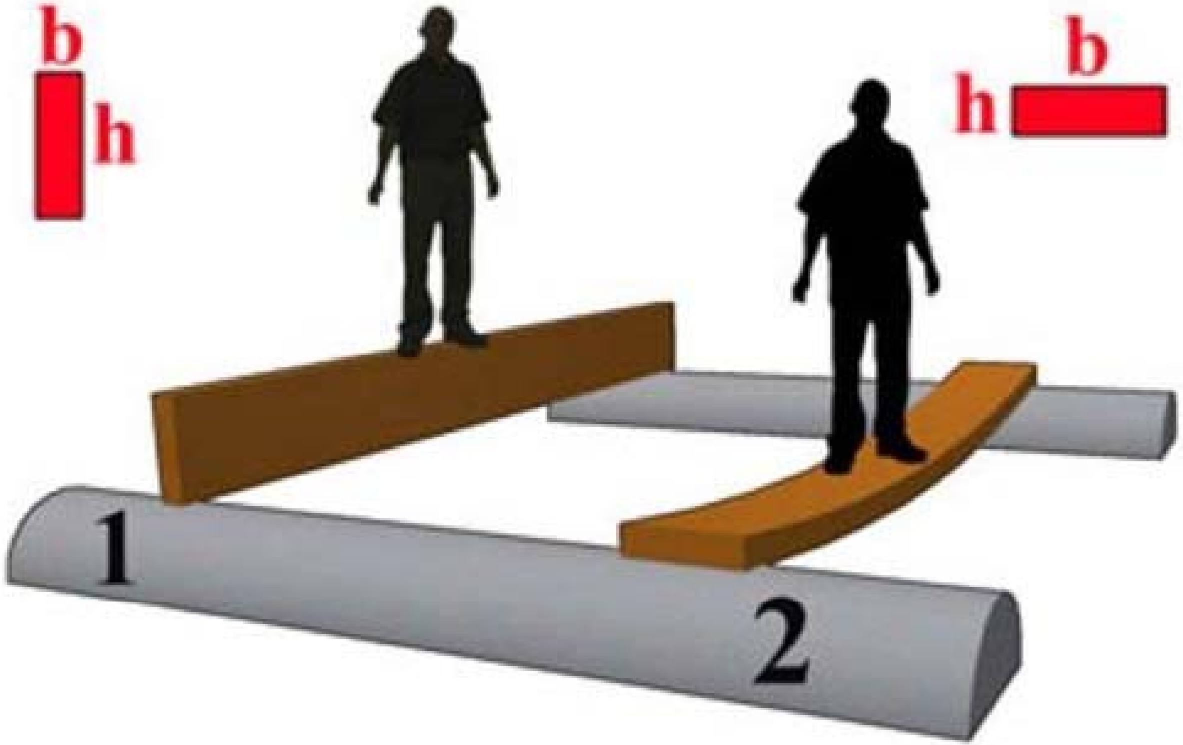
Le invenzioni delle macchine in piccolo, in grande poi non sussistono (Galileo)

3Bmeteo
MARINA MILITARE

Proporzioni di scala

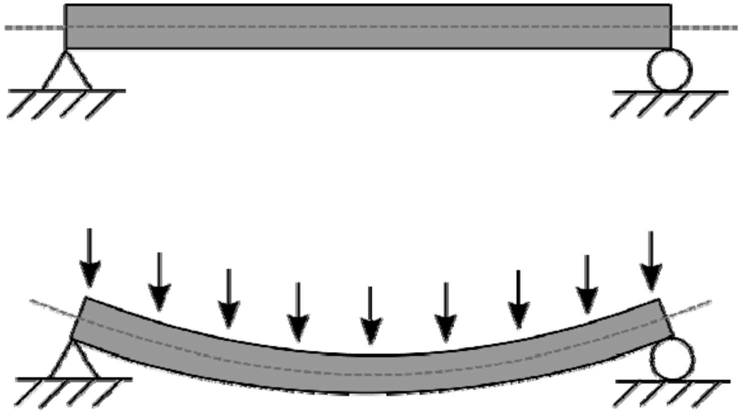

F
E
D
C
B
A
32
16
8

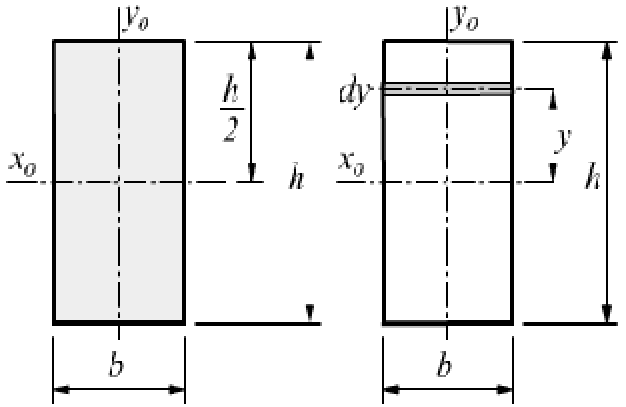
Resistenza a Flessione

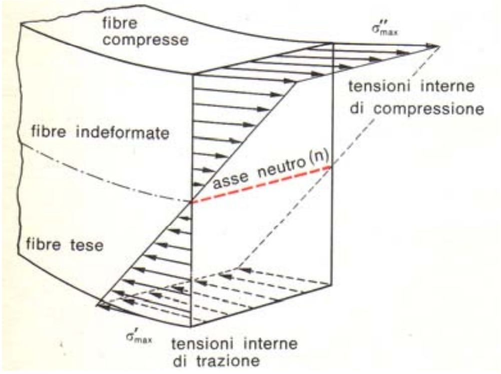


Area e Momento di inerzia

A=B·H




y_o
$\frac{h}{2}$
x_o
h
b
y_o
dy
y
x_o
h
b

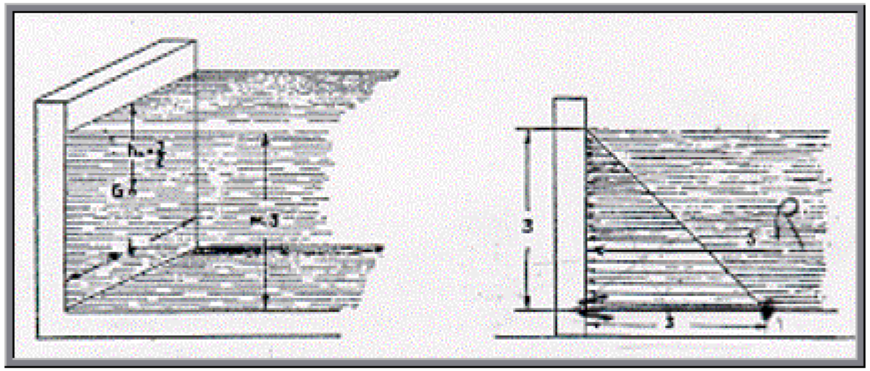

fibre compresse
σ''_{max}
tensioni interne di compressione
fibre indeformate
asse neutro (n)
fibre tese
σ'_{max}
tensioni interne di trazione

$$\sigma = \frac{M \cdot y}{J}$$

Dove $$J = \frac{b \cdot h^3}{12}$$

Formula di Navier 1800 circa

Non linearità delle azioni

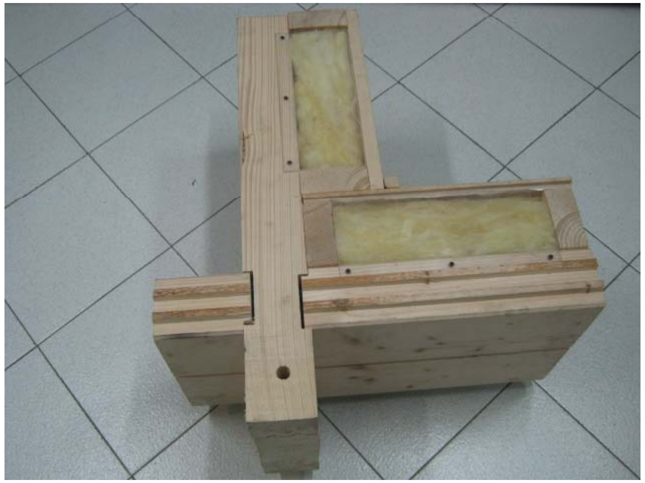


$h = 2m$ $$S = \frac{1}{2} \cdot h^2 \cdot \gamma = 0{,}5 \cdot 4 \cdot 1000 = 2000\ daN$$

$h = 4m$ $$S = \frac{1}{2} \cdot h^2 \cdot \gamma = 0{,}5 \cdot 16 \cdot 1000 = 8000\ daN$$

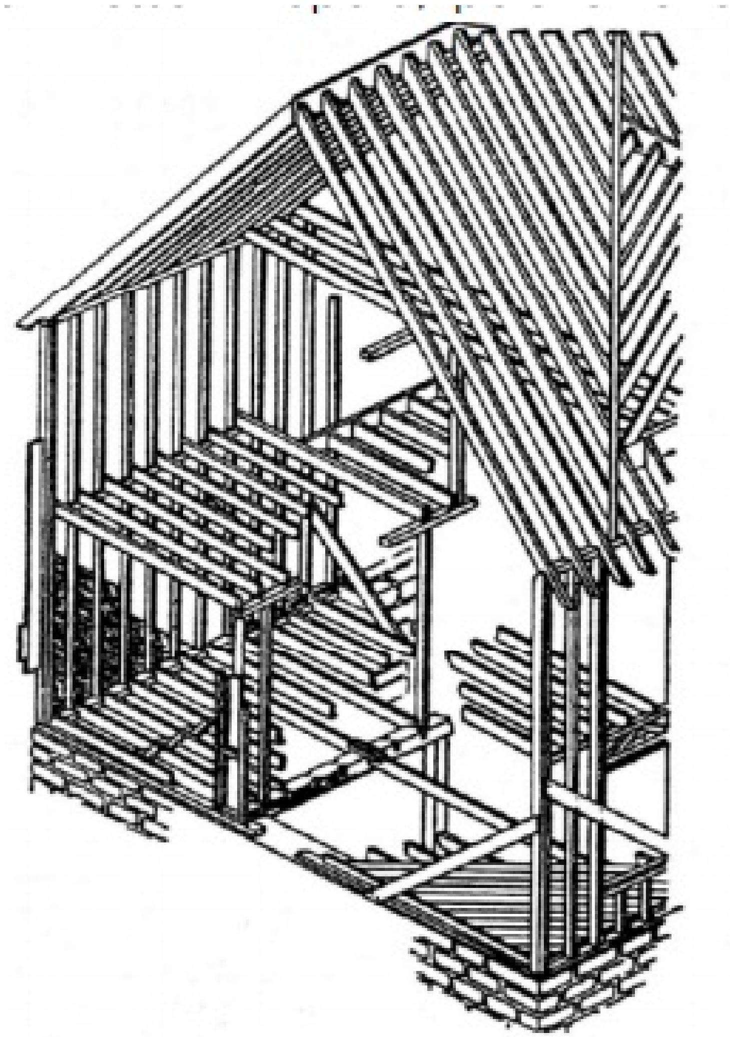
Strutture in legno

Block House



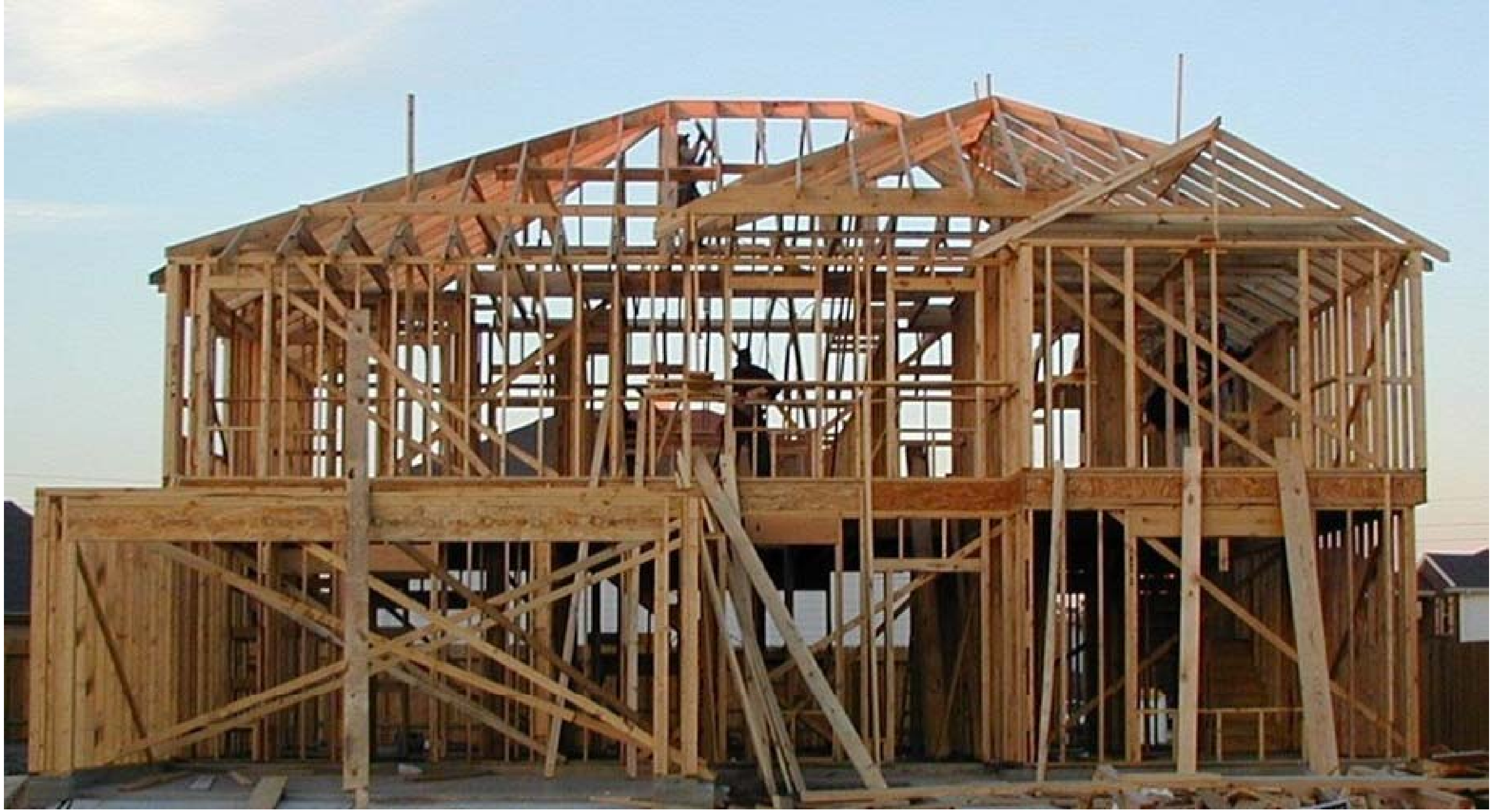
Strutture in legno

Balloon Frame



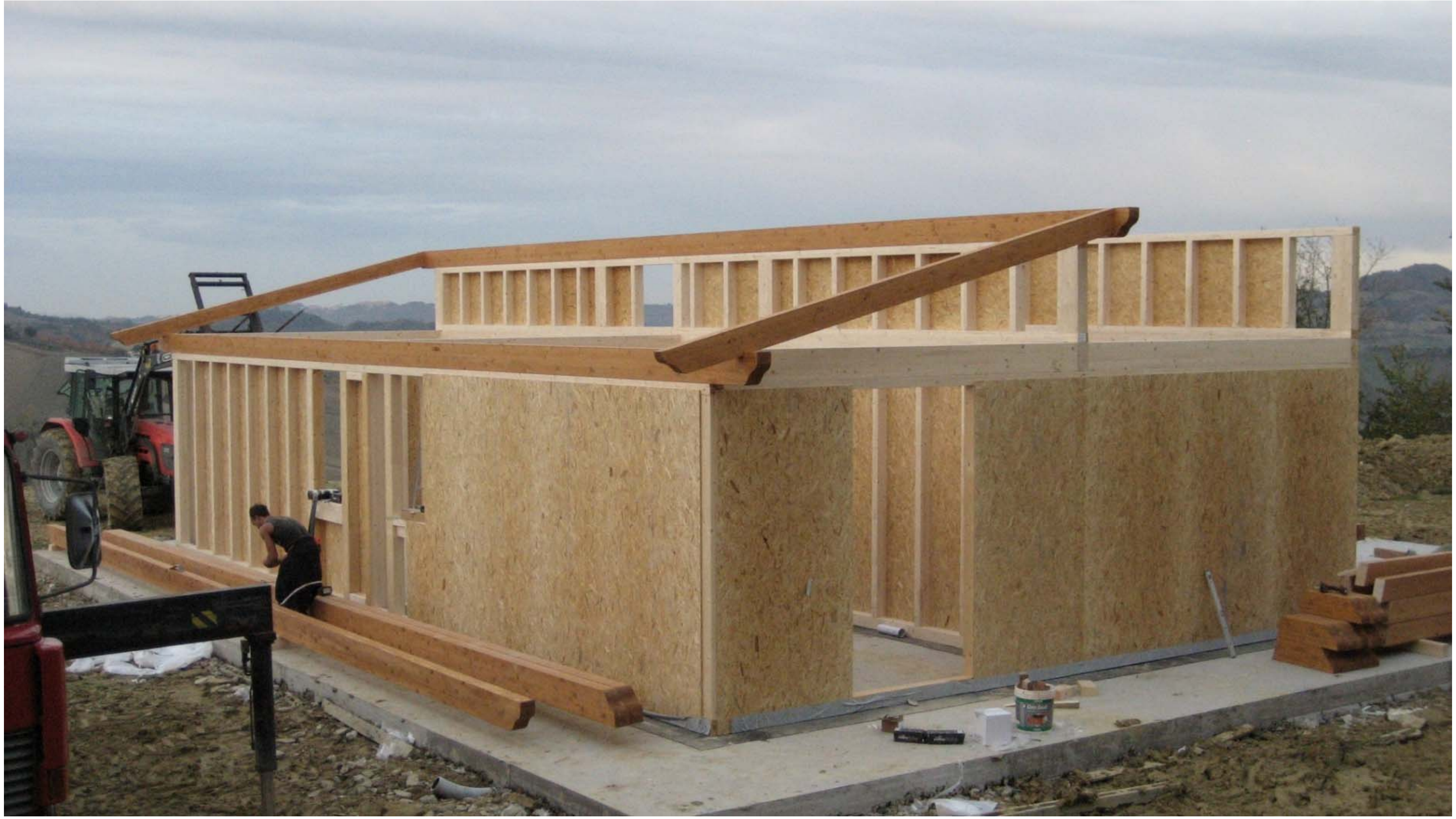
Strutture in legno

Balloon Frame



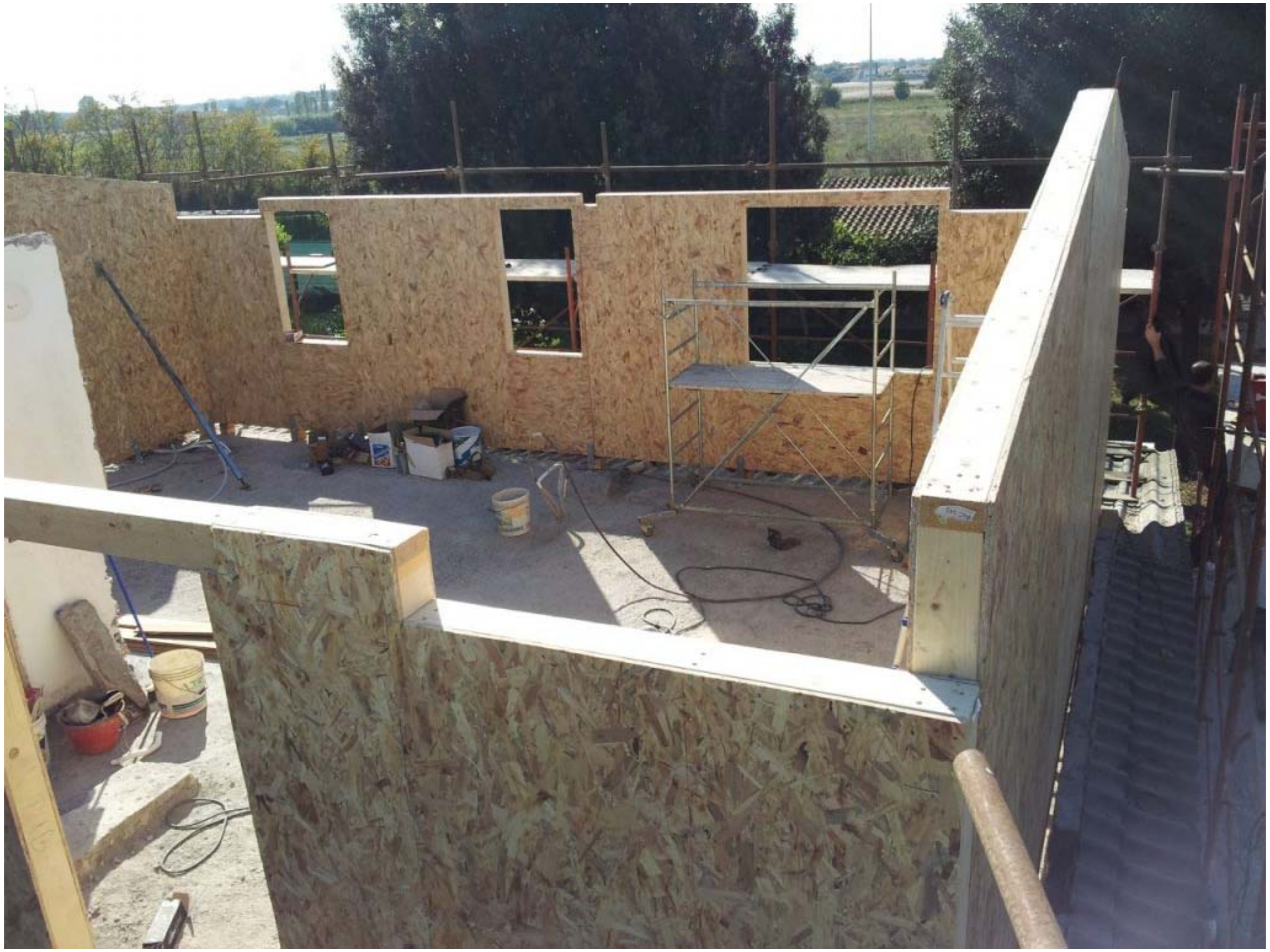
Strutture in legno

Platform Frame

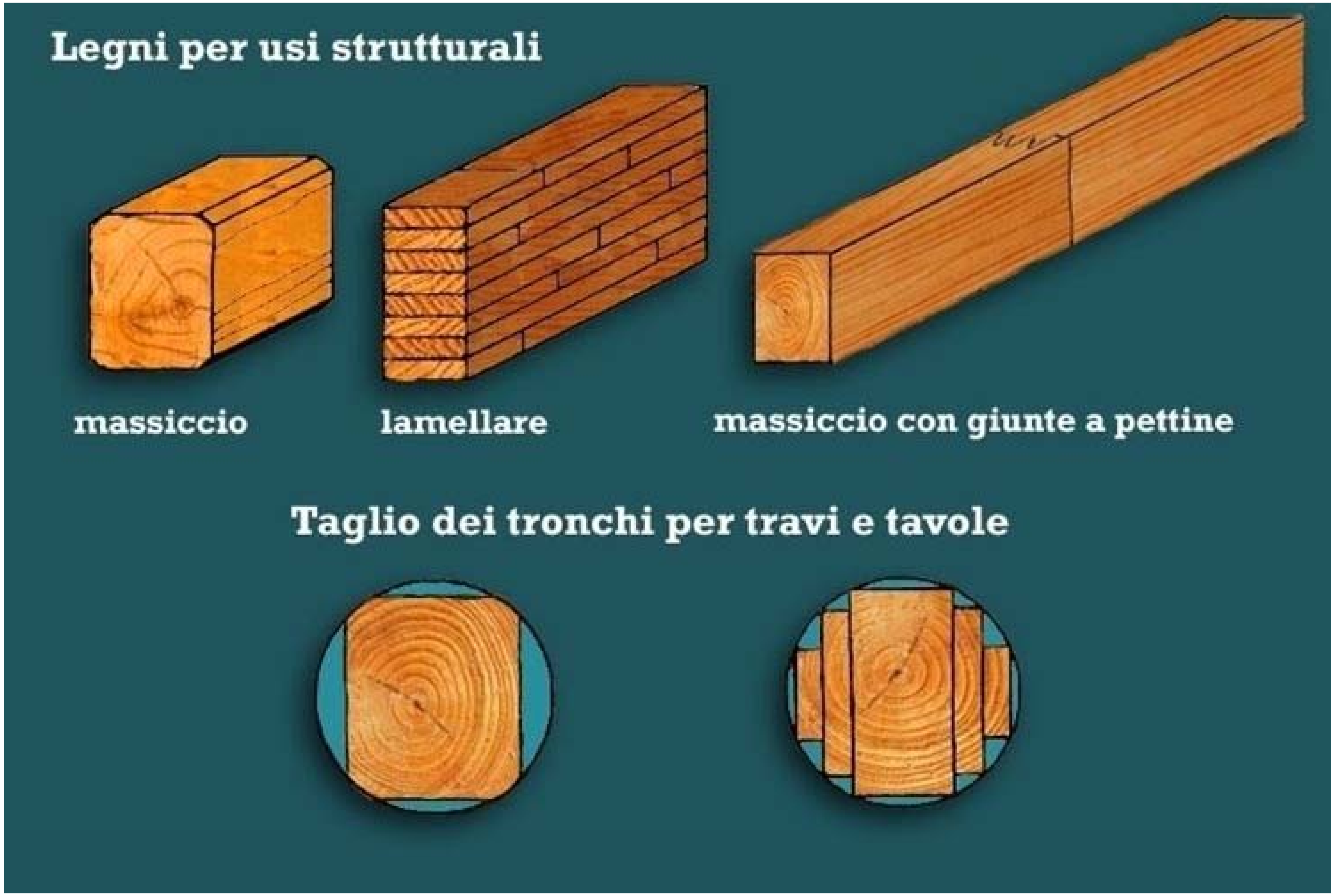


Strutture in legno

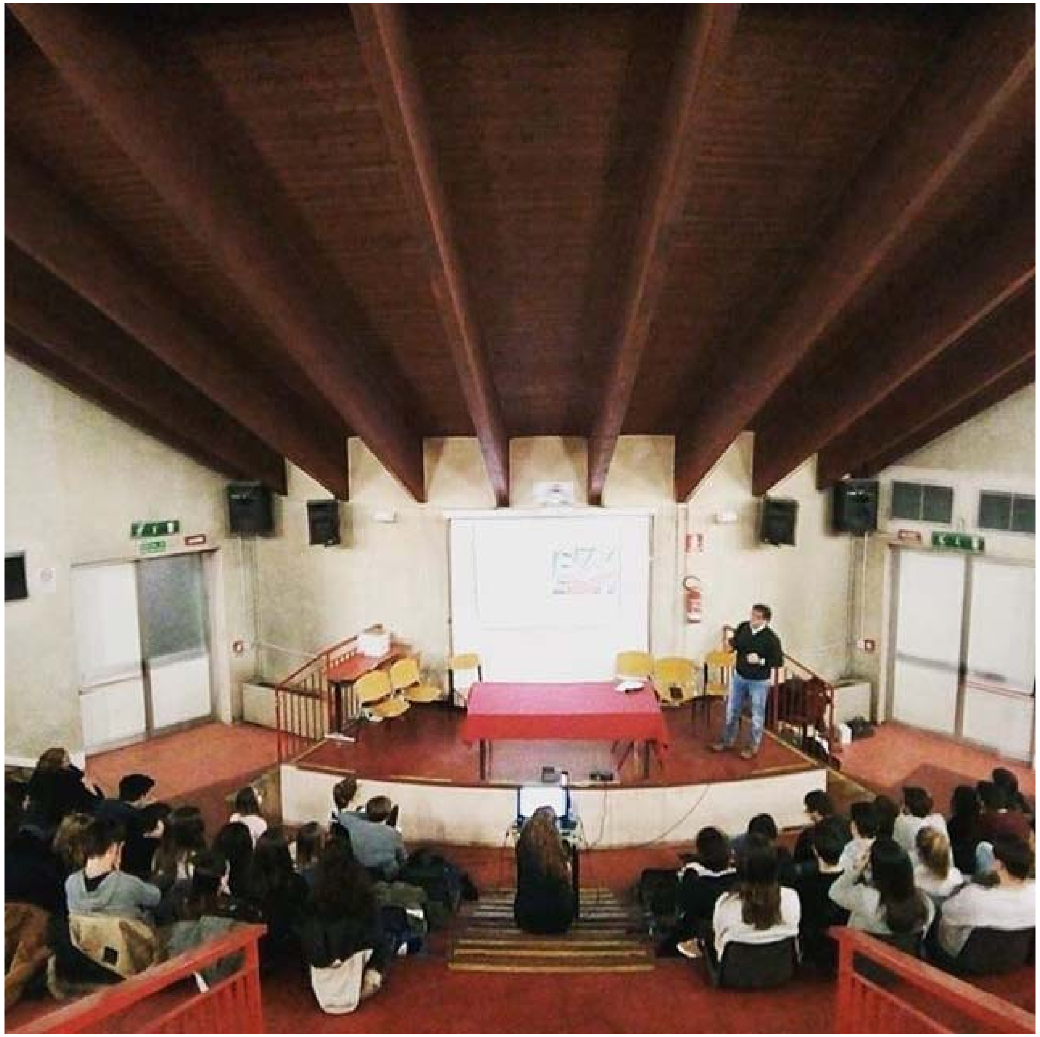
Platform Frame



Legno Lamellare


Legni per usi strutturali
massiccio
lamellare
massiccio con giunte a pettine
Taglio dei tronchi per travi e tavole

Legno Lamellare



Strutture in legno

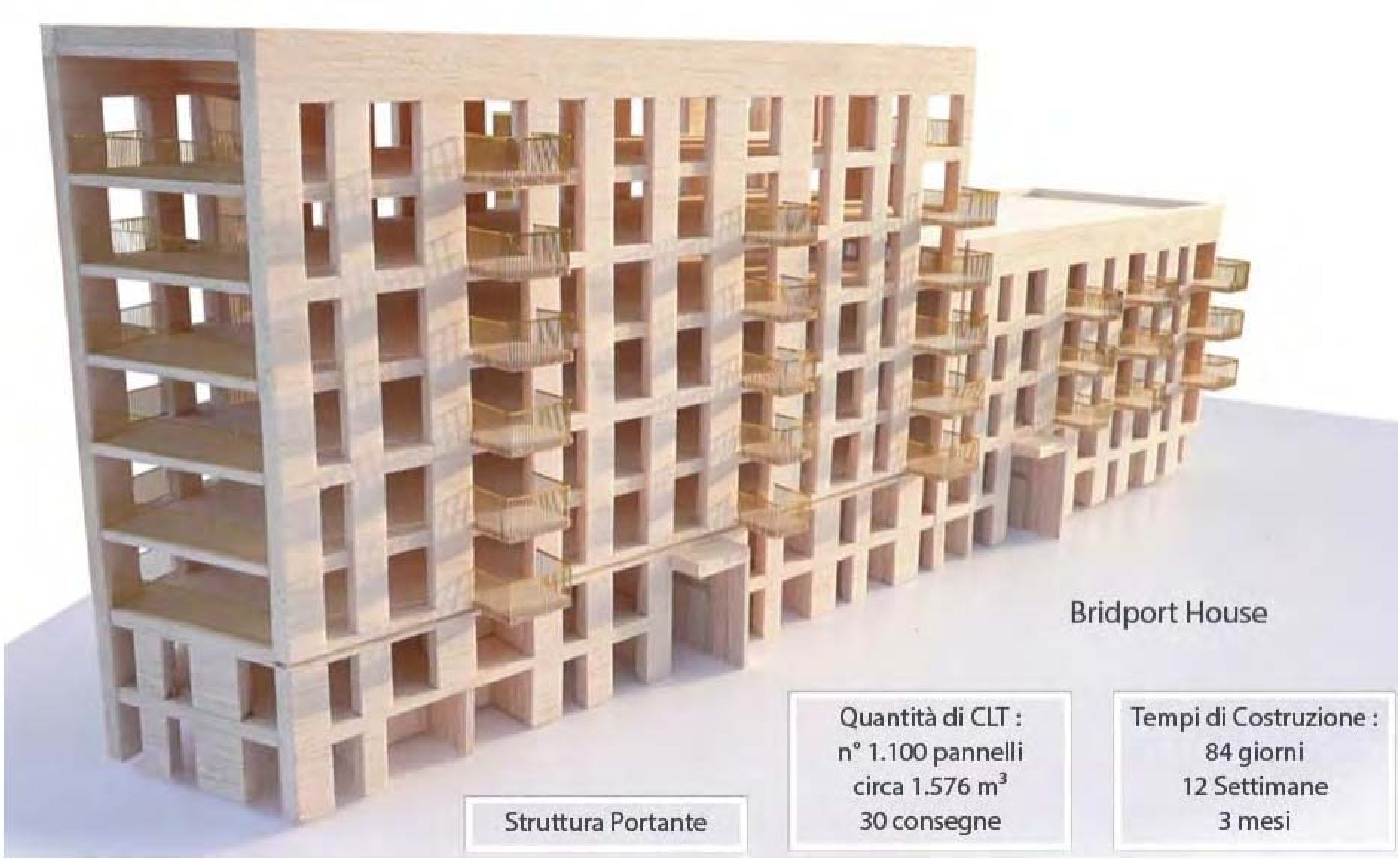
Xlam

Strutture in legno

Xlam

Strutture in legno

Xlam



Bridport House

Struttura Portante

Quantità di CLT :
n° 1.100 pannelli
circa 1.576 m^3
30 consegne

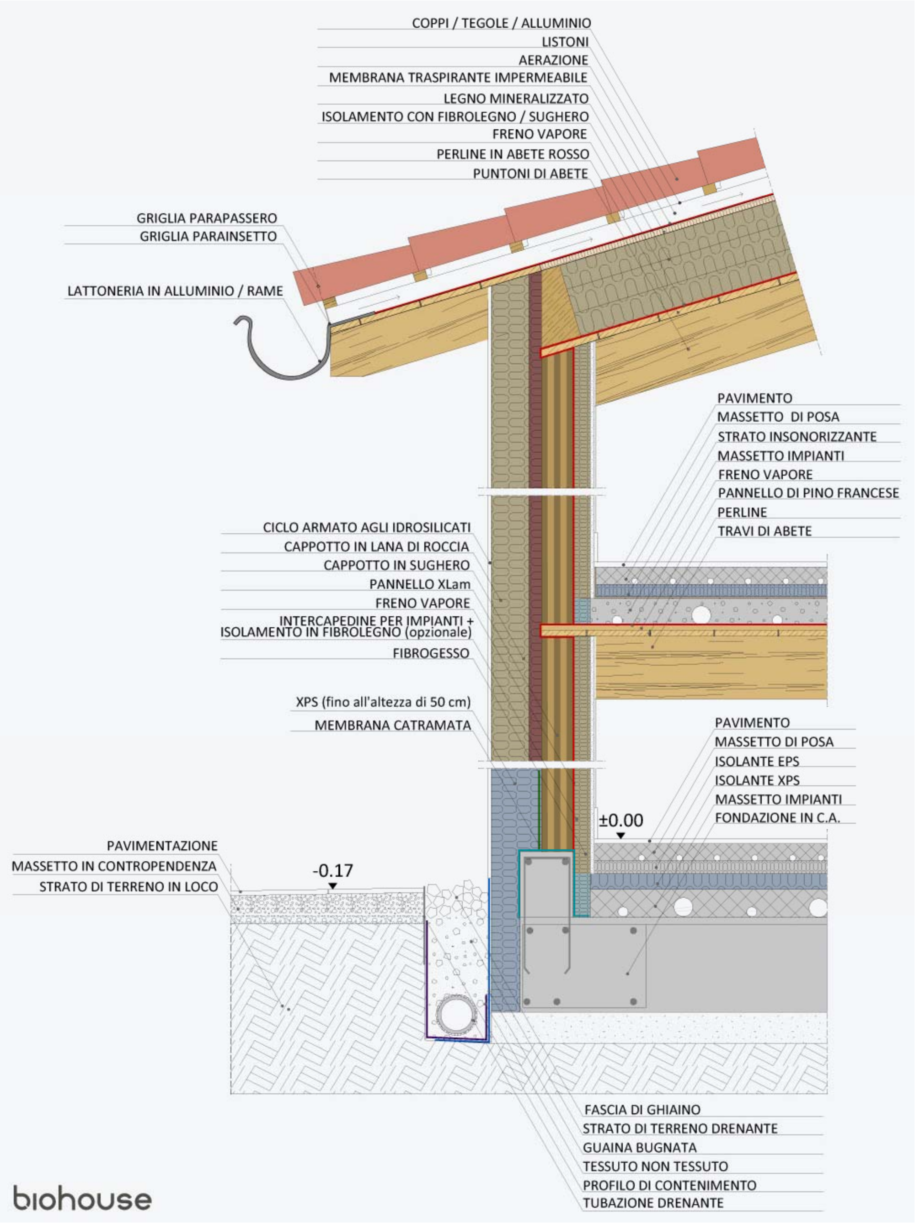
Tempi di Costruzione :
84 giorni
12 Settimane
3 mesi

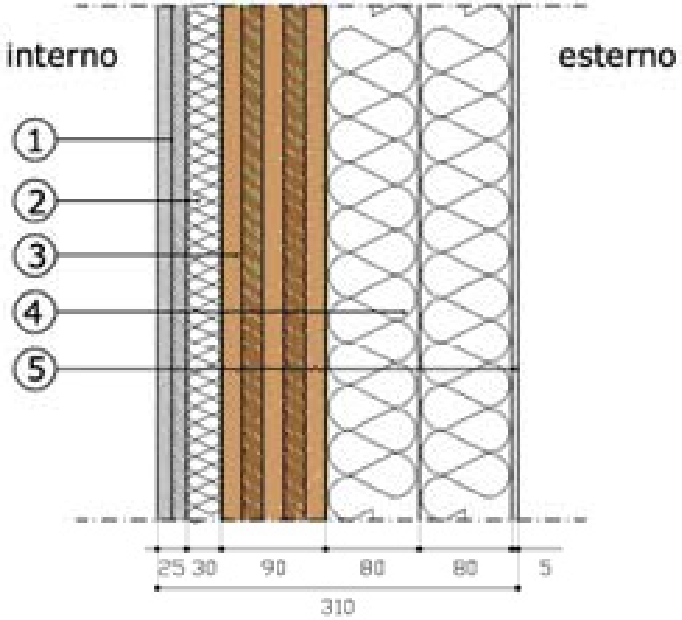
Strutture in legno

Xlam

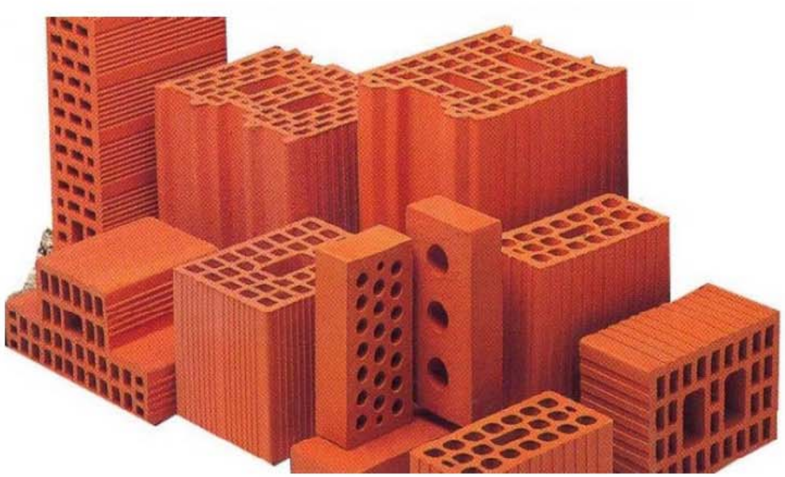
Bridport House
Edifcio Multipiano Residenziale
Architettura : Karakusevic Carson Architects
Luogo dell'intervento : Londra, UK
Anno di costruzione : 2010 - 2011
8 Piani 41 - Appartamenti

Xlam – Particolari costruttivi

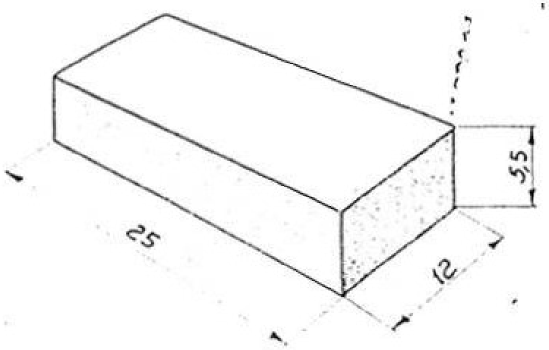

COPPI / TEGOLE / ALLUMINIO
LISTONI
AERAZIONE
MEMBRANA TRASPIRANTE IMPERMEABILE
LEGNO MINERALIZZATO
ISOLAMENTO CON FIBROLEGNO / SUGHERO
FRENO VAPORE
PERLINE IN ABETE ROSSO
PUNTONI DI ABETE
GRIGLIA PARAPASSERO
GRIGLIA PARAINSETTO
LATTONERIA IN ALLUMINIO / RAME
PAVIMENTO
MASSETTO DI POSA
STRATO INSONORIZZANTE
MASSETTO IMPIANTI
FRENO VAPORE
PANNELLO DI PINO FRANCESE
PERLINE
TRAVI DI ABETE
CICLO ARMATO AGLI IDROSILICATI
CAPPOTTO IN LANA DI ROCCIA
CAPPOTTO IN SUGHERO
PANNELLO XLam
FRENO VAPORE
INTERCAPEDINE PER IMPIANTI +
ISOLAMENTO IN FIBROLEGNO (opzionale)
FIBROGESSO
XPS (fino all'altezza di 50 cm)
MEMBRANA CATRAMATA
PAVIMENTO
MASSETTO DI POSA
ISOLANTE EPS
ISOLANTE XPS
MASSETTO IMPIANTI
FONDAZIONE IN C.A.
±0.00
PAVIMENTAZIONE
MASSETTO IN CONTROPENDENZA
STRATO DI TERRENO IN LOCO
-0.17
FASCIA DI GHIAINO
STRATO DI TERRENO DRENANTE
GUAINA BUGNATA
TESSUTO NON TESSUTO
PROFILO DI CONTENIMENTO
TUBAZIONE DRENANTE
biohouse

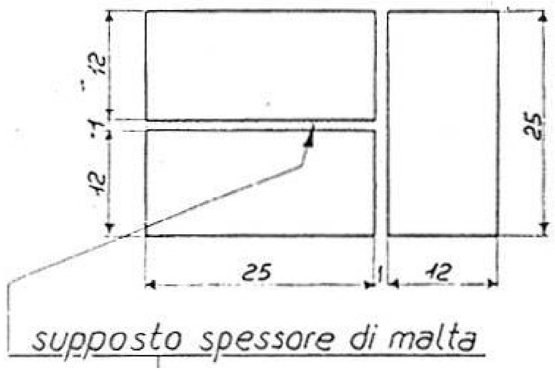

interno
esterno
1
2
3
4
5
25
30
90
80
80
5
310

Edifici in muratura Portante

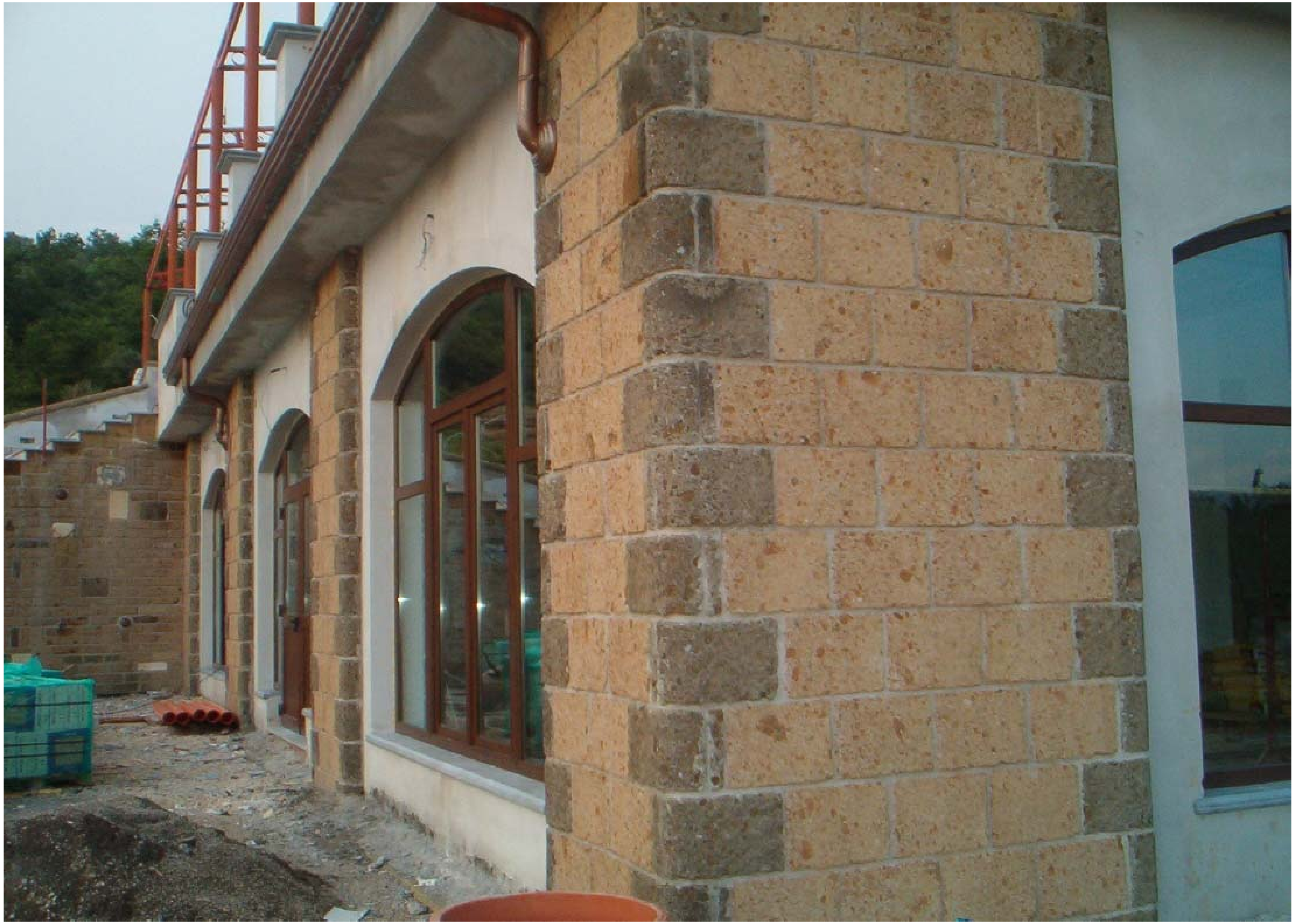


Edifici in muratura di mattoni pieni

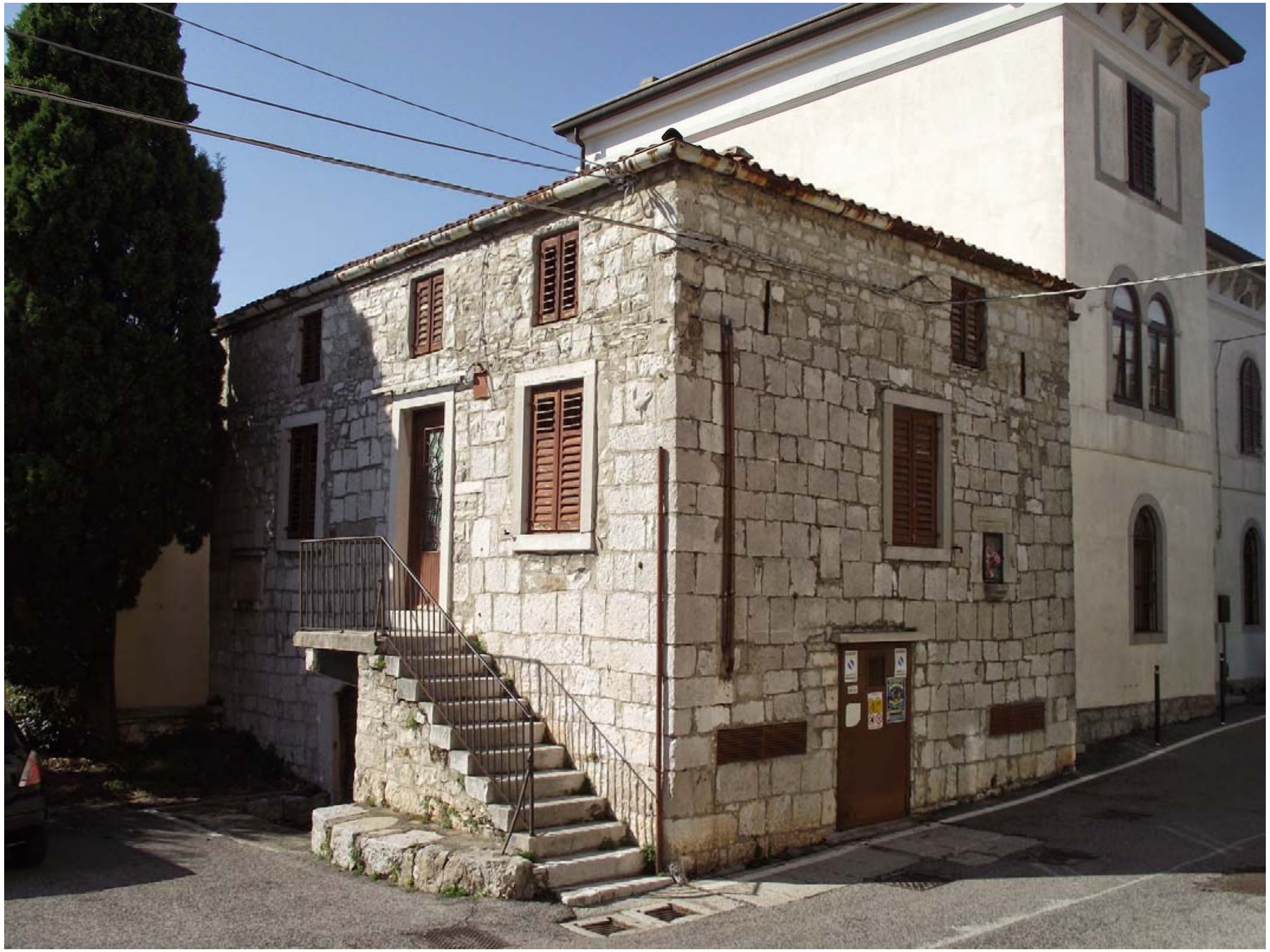



12
1
12
25
25
1
12
supposto spessore di malta

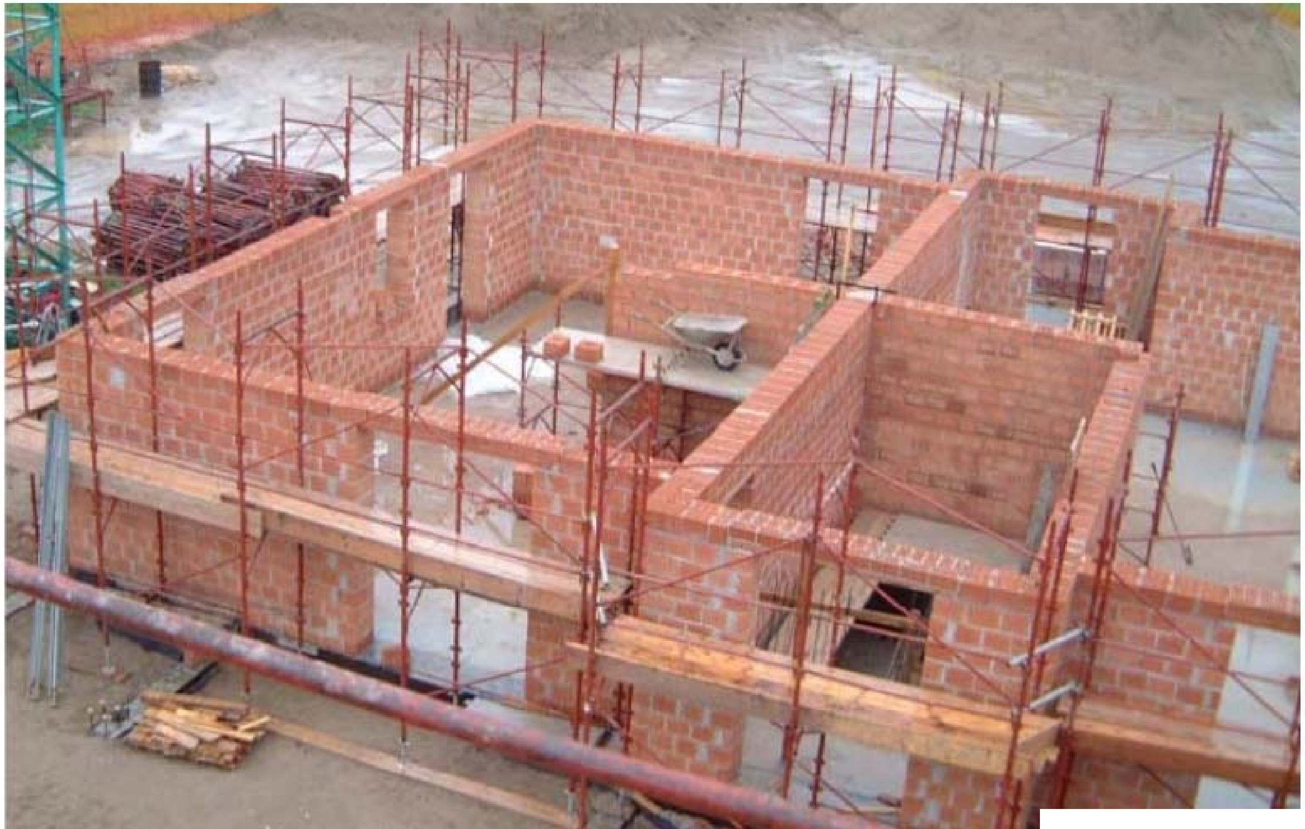
Edifici in muratura blocchi di tufo

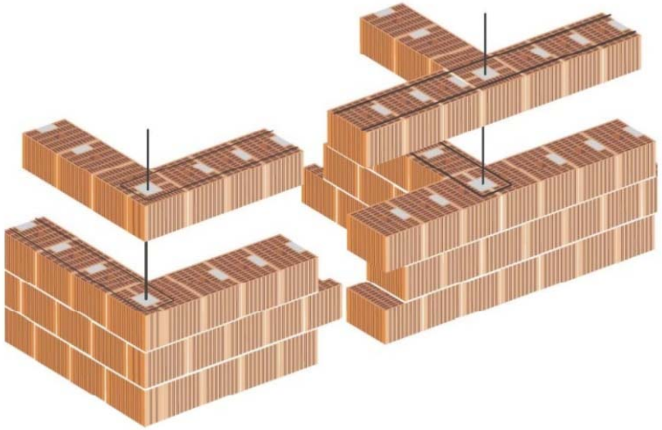


Edificio in muratura di pietrame

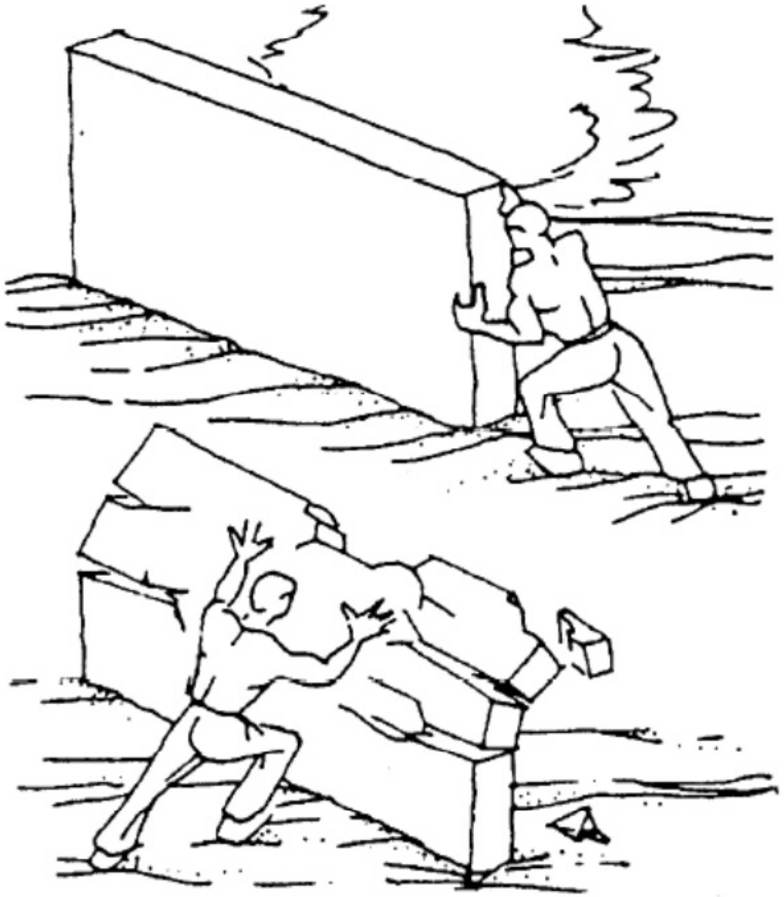


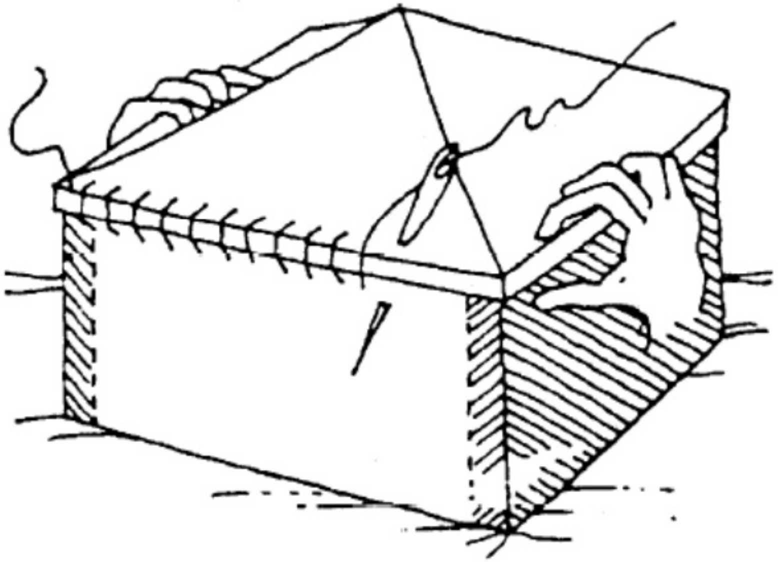
Edificio in muratura di laterizio





Principio statico della muratura





Edifici in muratura Portante

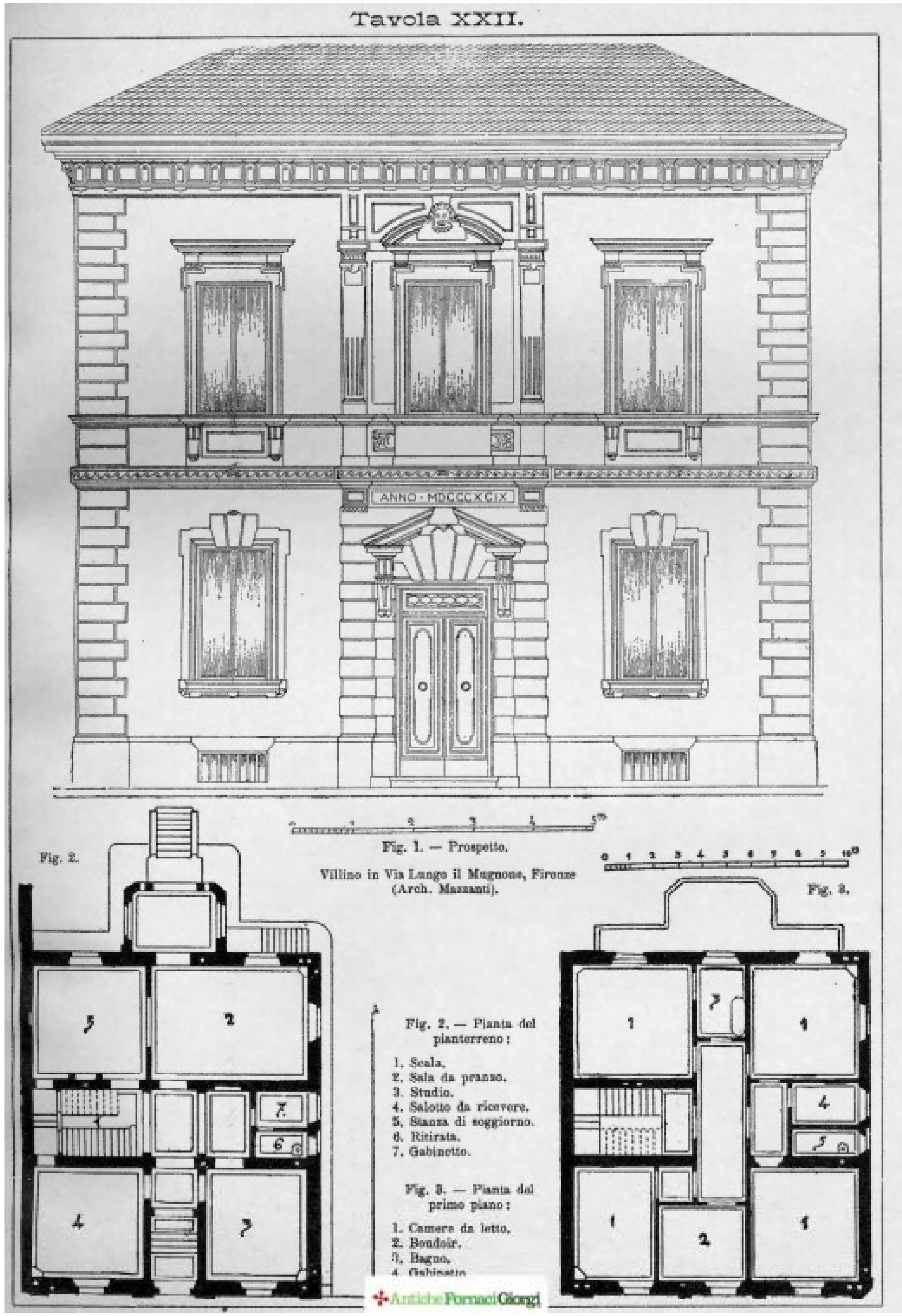

ANNO · MDCCCXCIX
0 1 2 3 4 5m
0 1 2 3 4 5 6 7 8 9 10m
Fig. 2.
Fig. 3.
5
2
1
7
6
4
7
1
1
7
1
4
5
1
2
1

Tavola XXII.

Fig. 1. — Prospetto.

Villino in Via Lungo il Mugnone, Firenze
(Arch. Mazzanti).

Fig. 2. — Pianta del pianterreno:

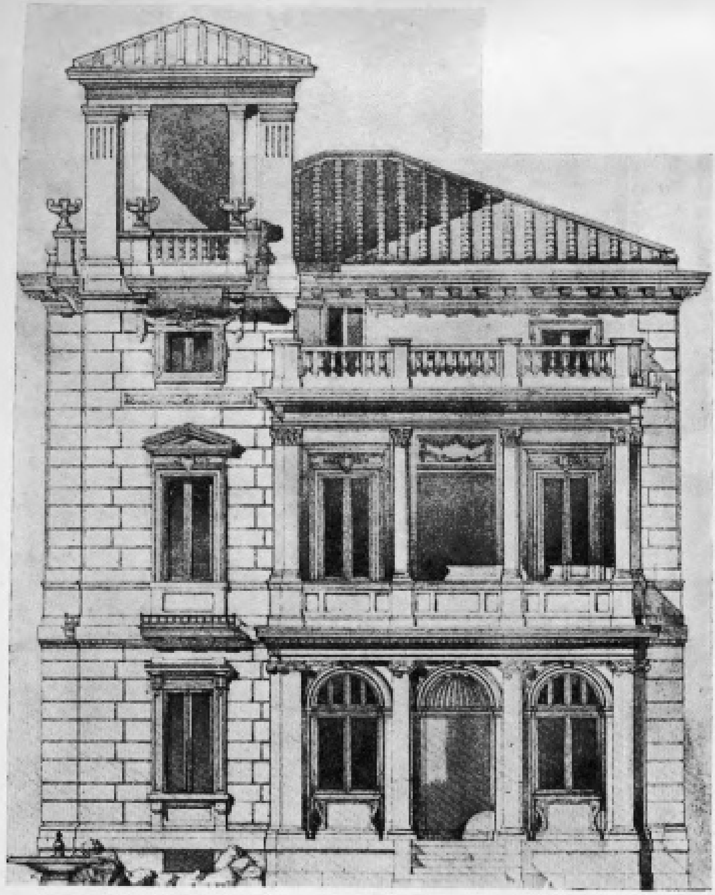
1. Scala.
2. Sala da pranzo.
3. Studio.
4. Salotto da ricevere.
5. Stanza di soggiorno.
6. Ritirata.
7. Gabinetto.

Fig. 3. — Pianta del primo piano:

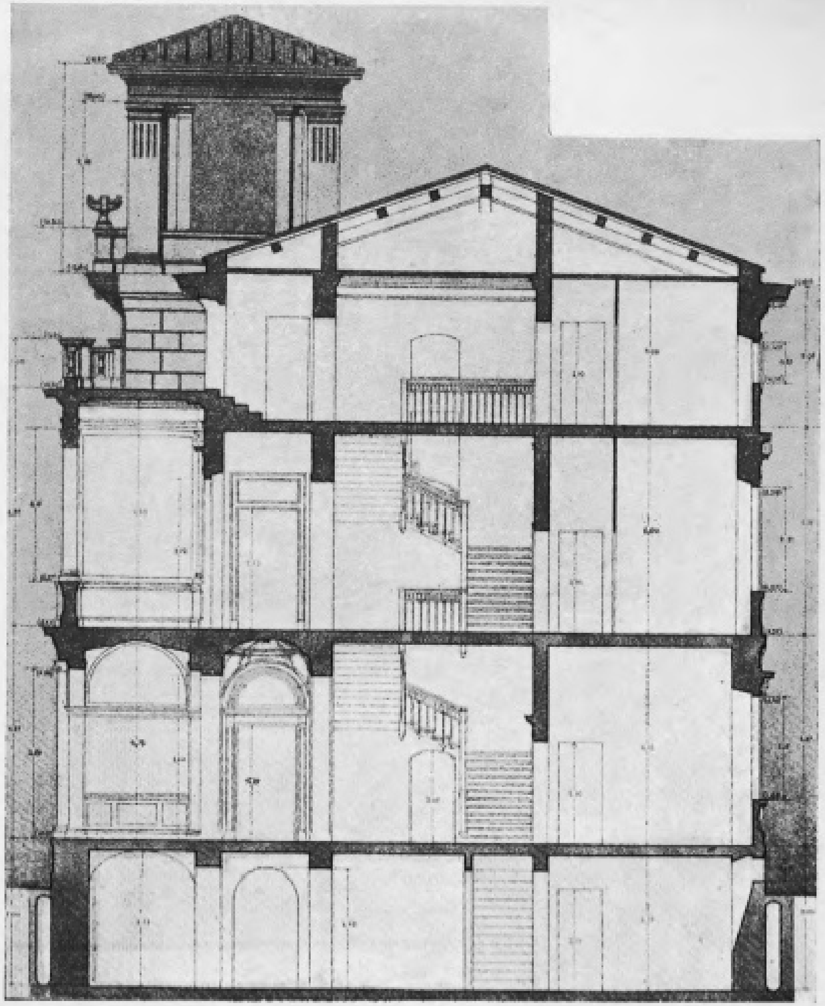
1. Camere da letto.
2. Boudoir.
3. Bagno.
4. Gabinetto.

AnticheFornaciGiorgi

Tavola XXXI.



Prospetto principale (Scala 1. 140).

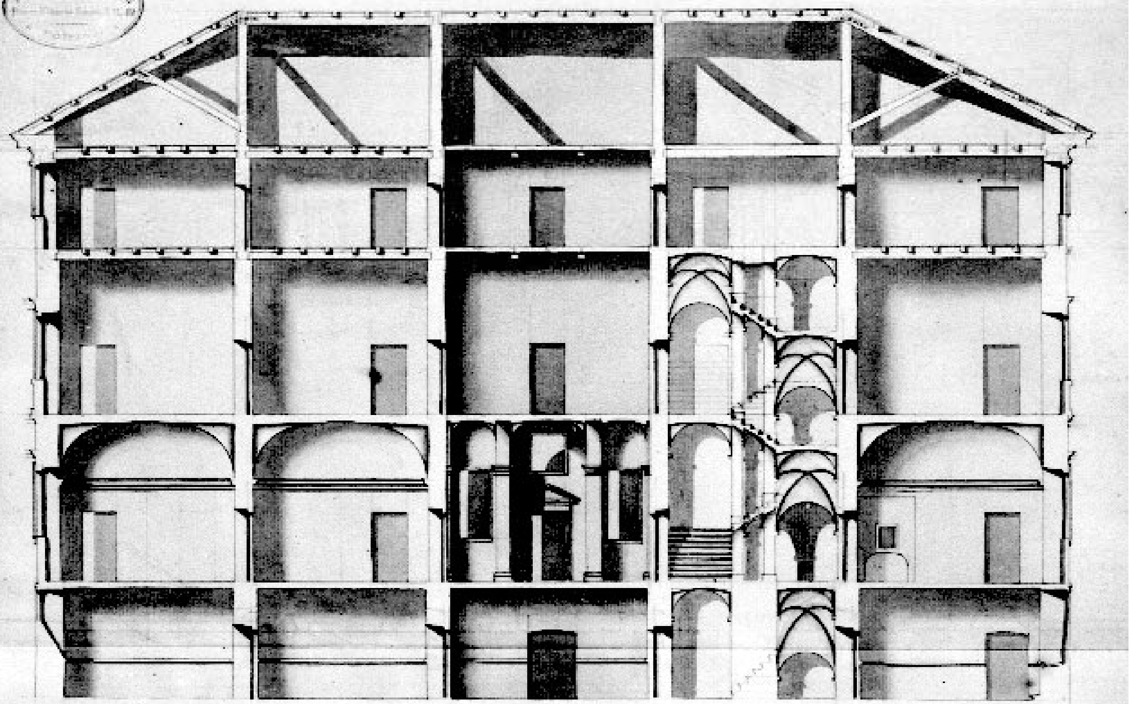


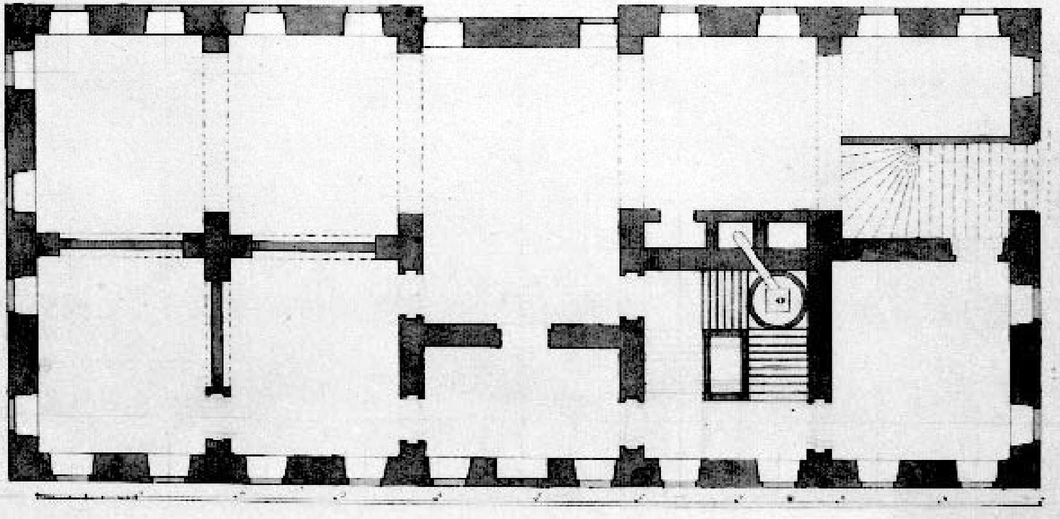
Sezione trasversale A B.

PROGETTO DI UN VILLINO

AnticheFornaciGiorgi

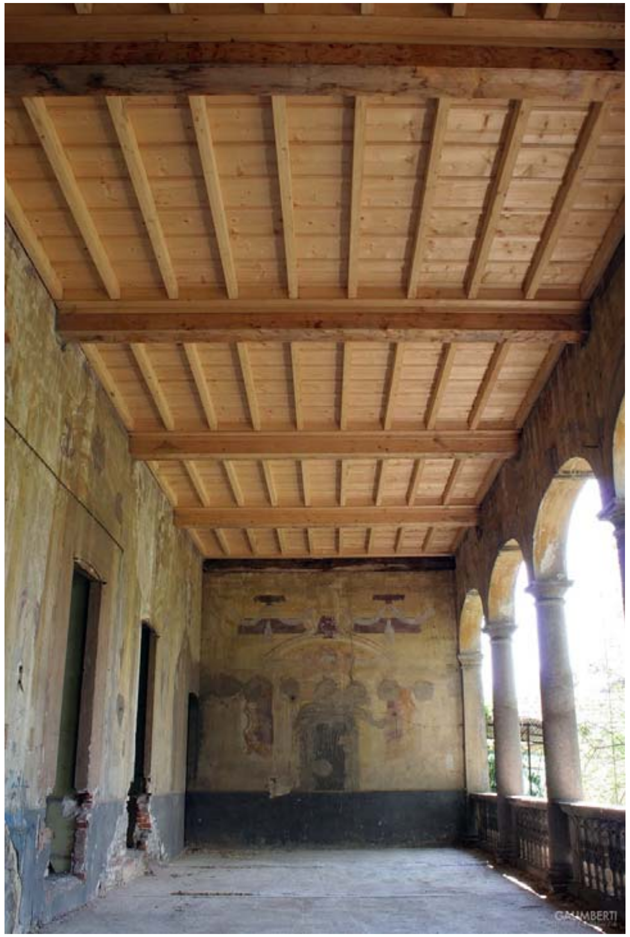
Proprietà artistica.





Solai

Solai in legno


GALIMBERTI

Solai a volte

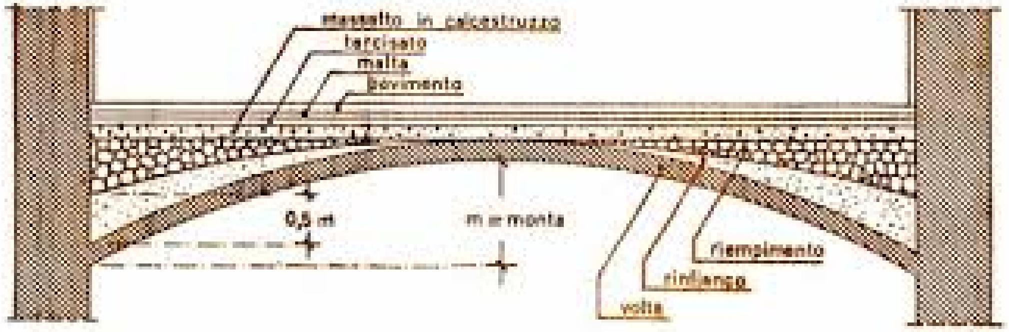
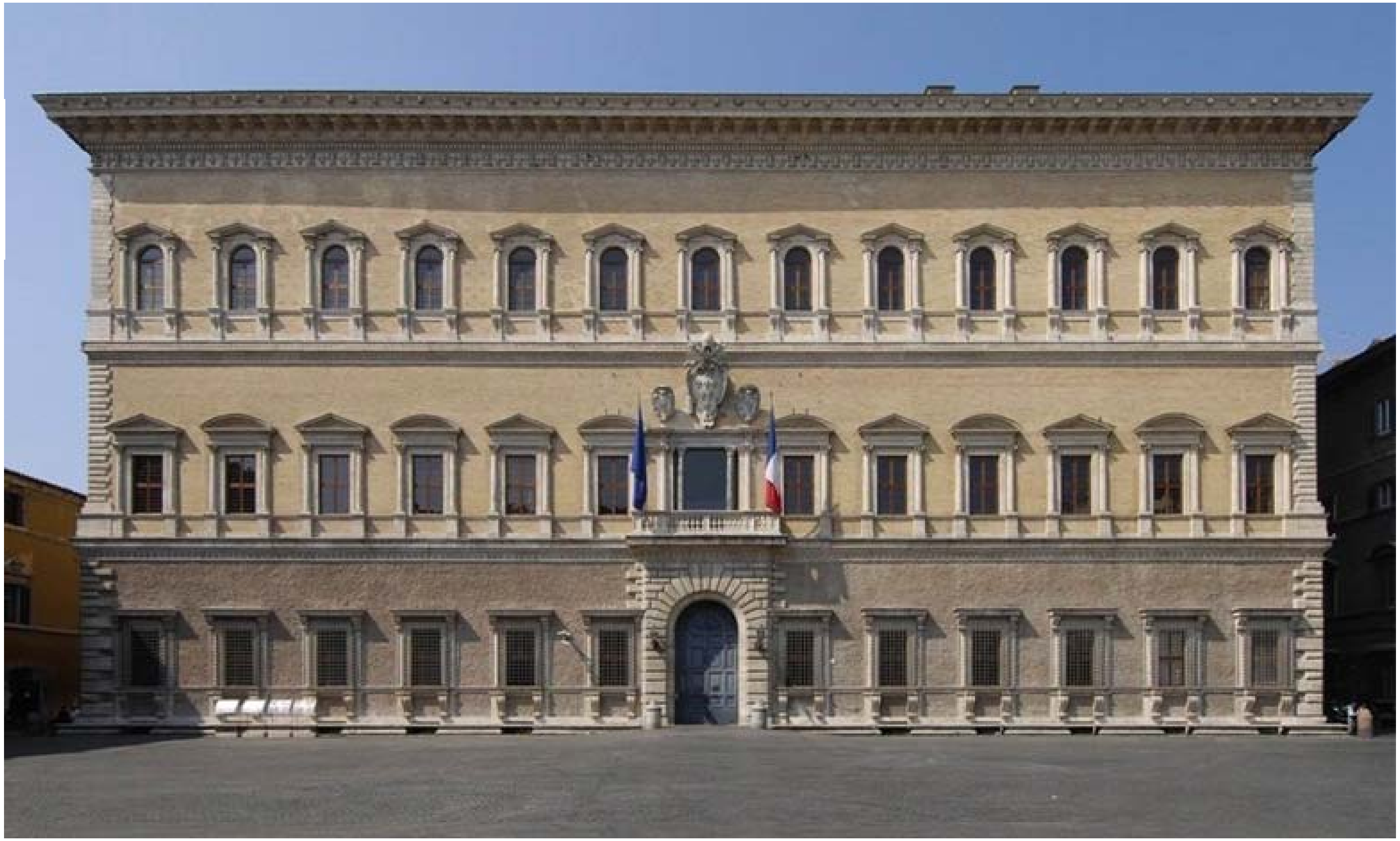

massetto in calcestruzzo
tarcisato
malta
pavimento
0,5 m
m = monta
riempimento
rinfianco
volta

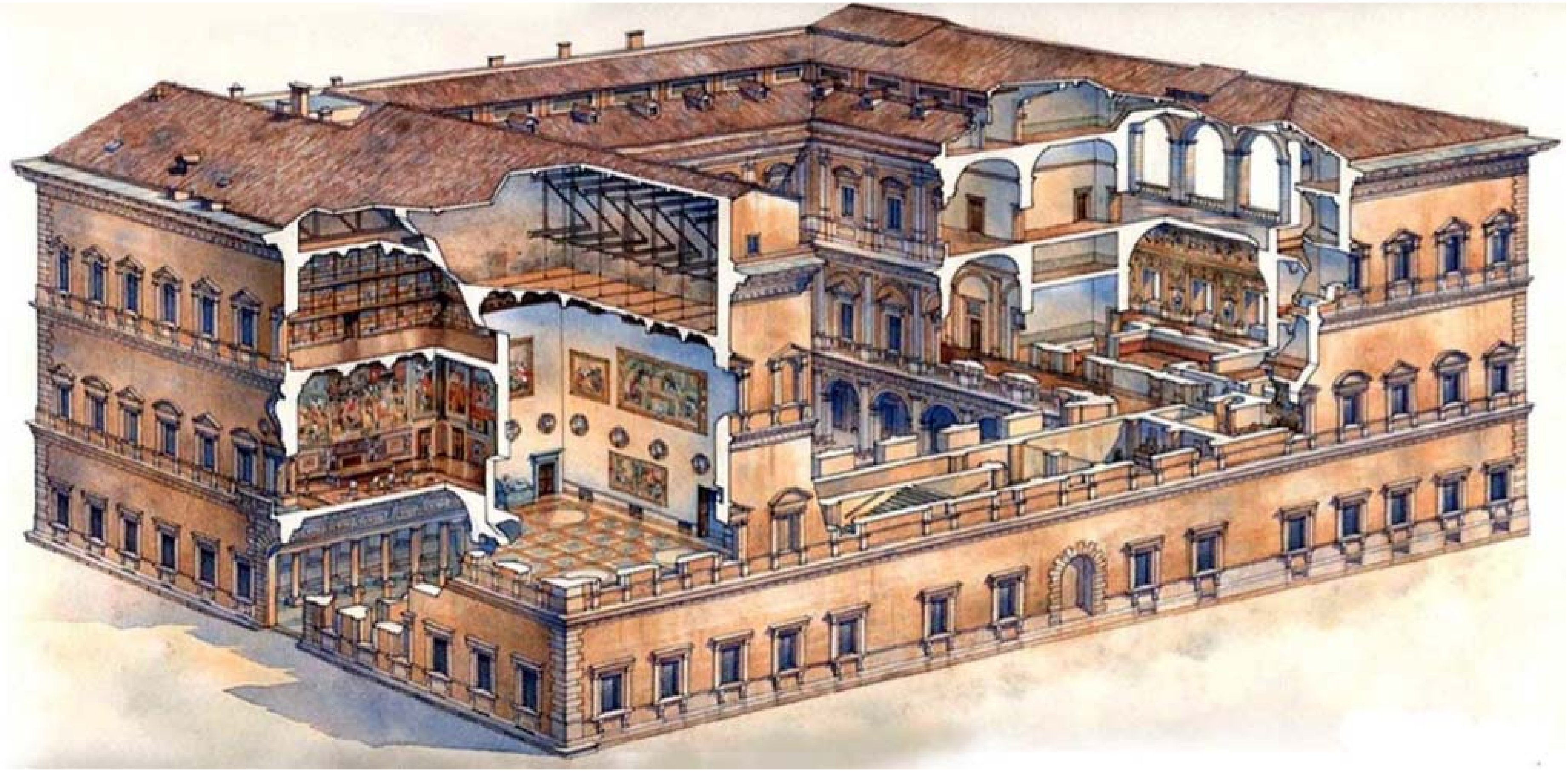
Immagine scaricata da: travelitalia.com

Palazzo Farnese

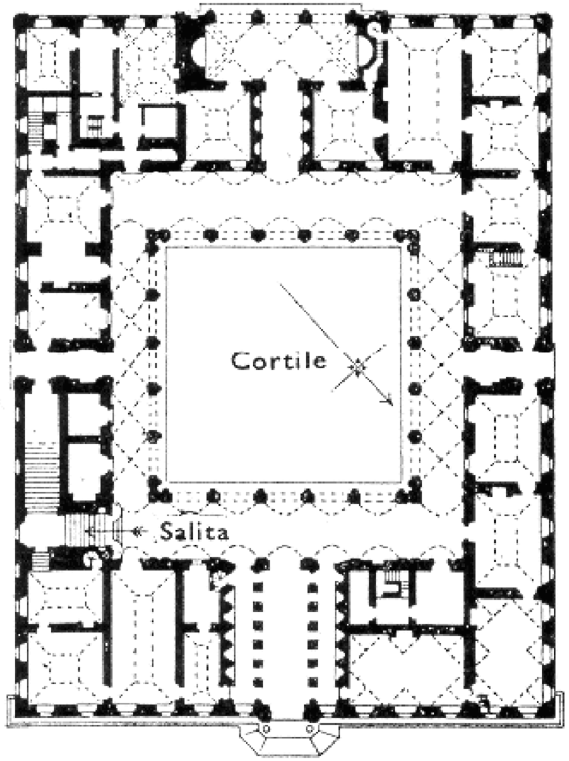


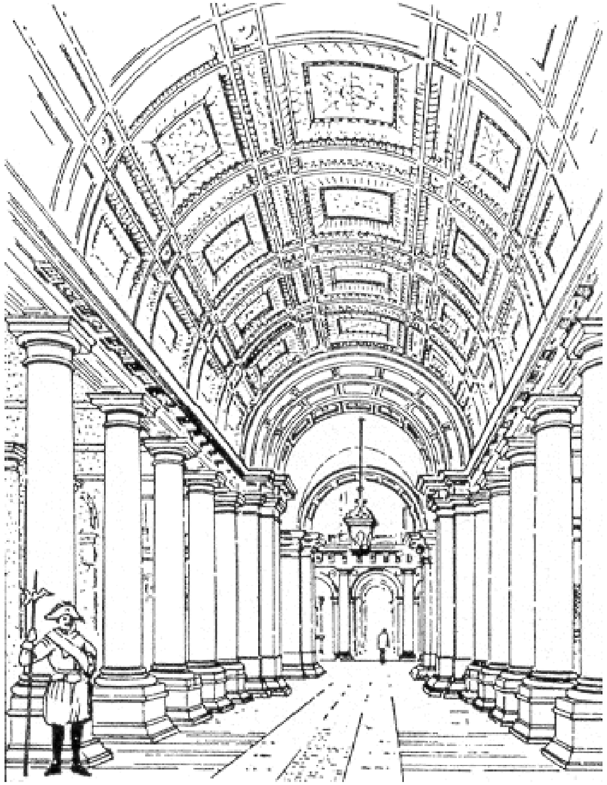
1550 circa

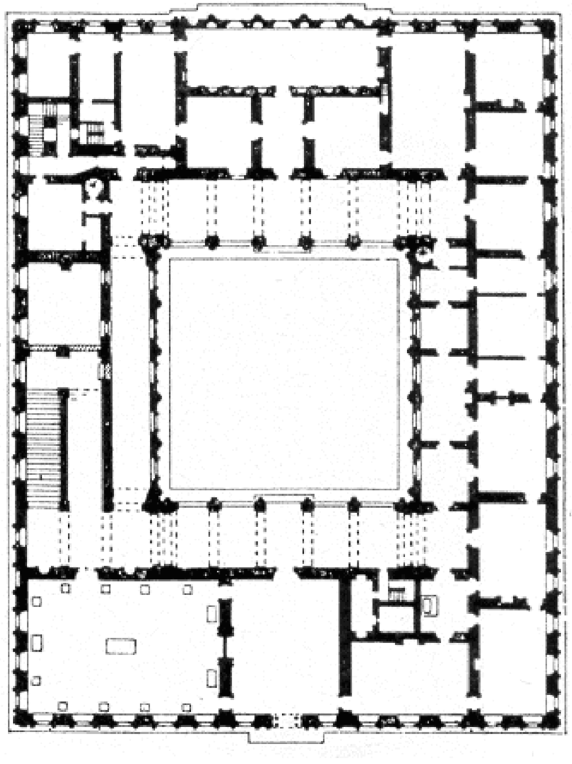
Palazzo Farnese



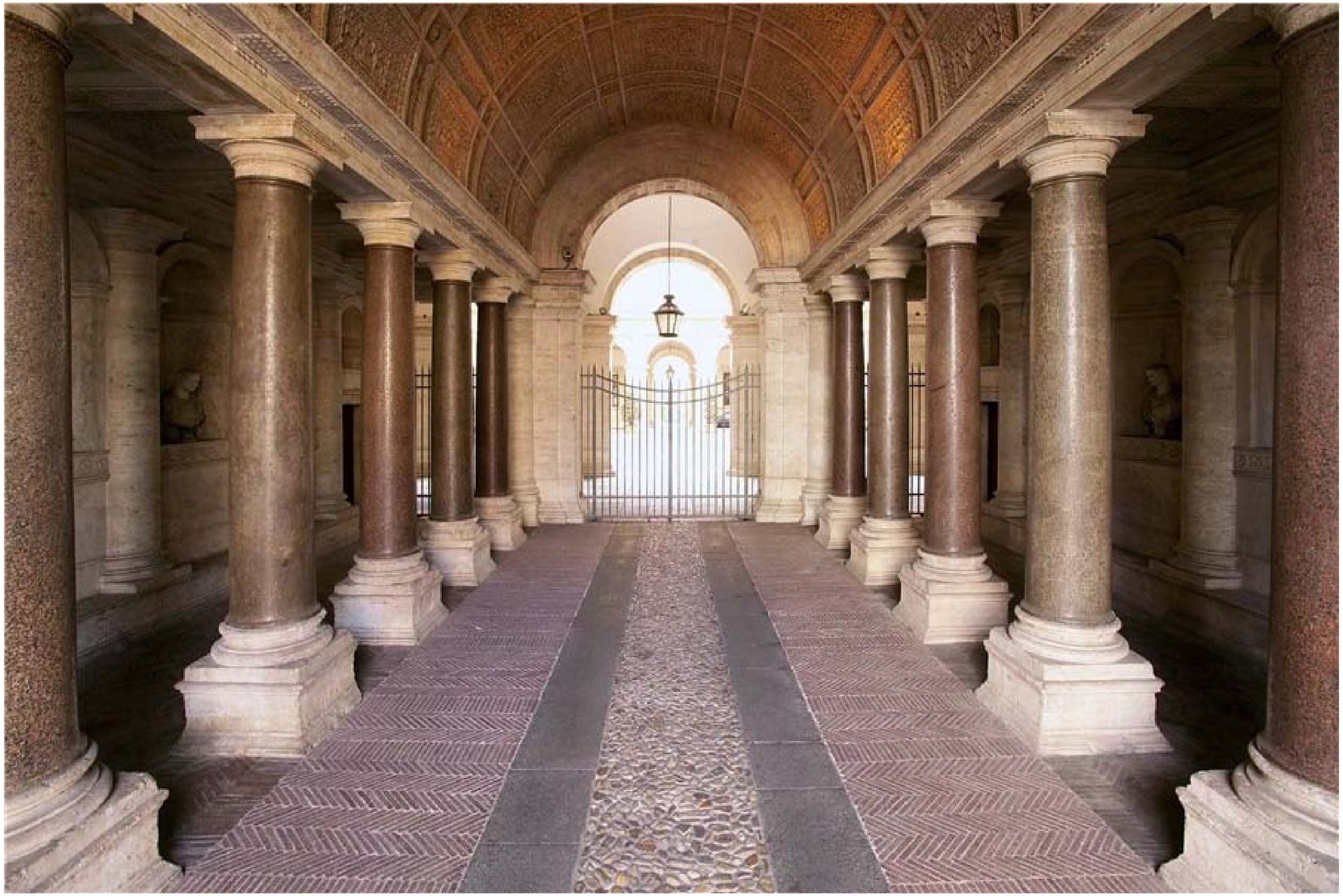
Palazzo Farnese


Cortile
Salita





Palazzo Farnese



Criteri per il dimensionamento delle murature

Muratura di
Pietrame

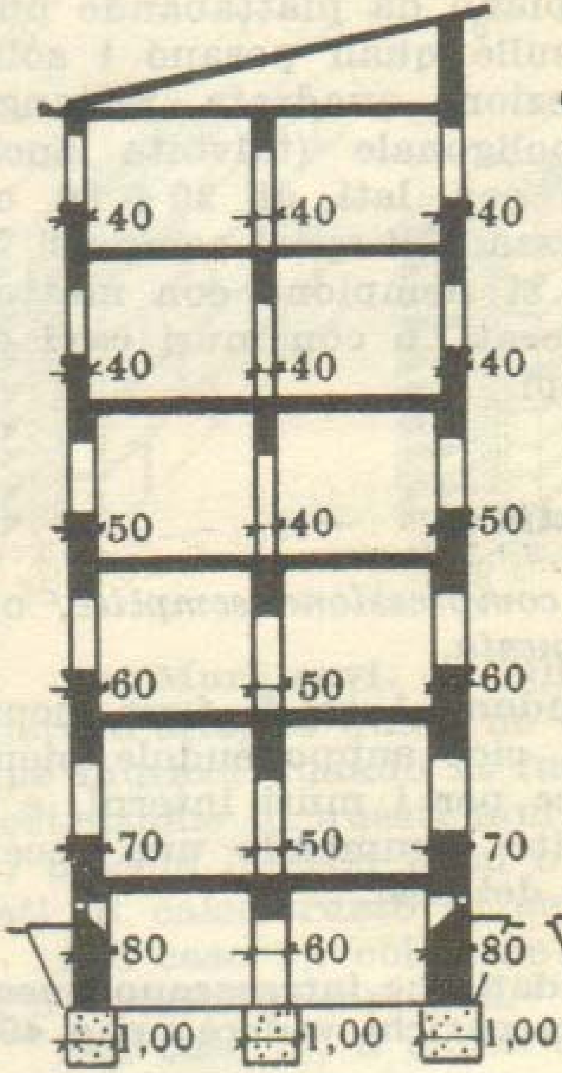

40
40
40
40
40
40
50
40
50
60
50
60
70
50
70
80
60
80
1,00
1,00
1,00

Fig. 7

Muratura di
Mattoni

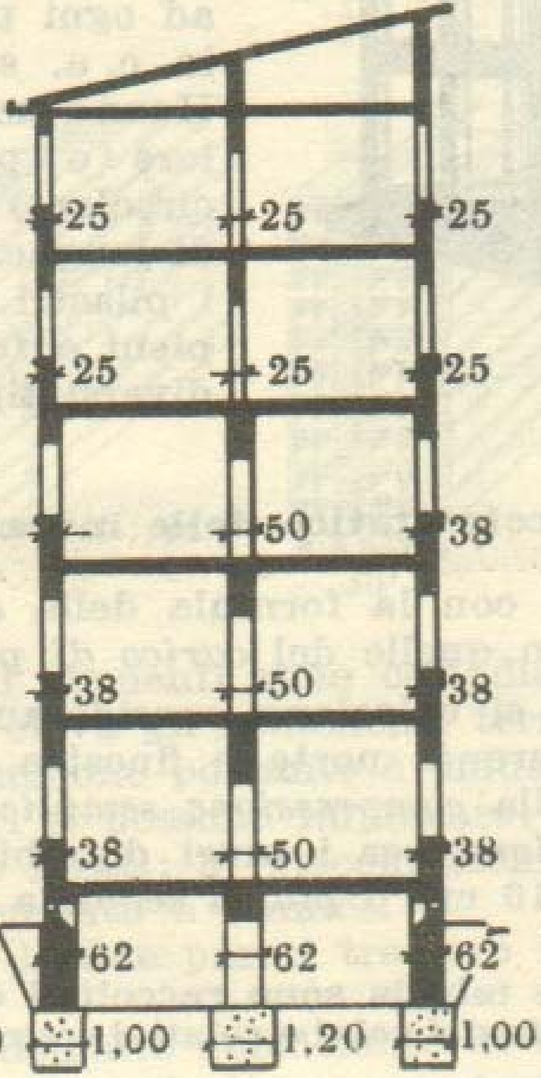

25
25
25
25
25
25
50
38
38
50
38
38
50
38
62
62
62
1,00
1,20
1,00

Fig. 8

Breymann -1850

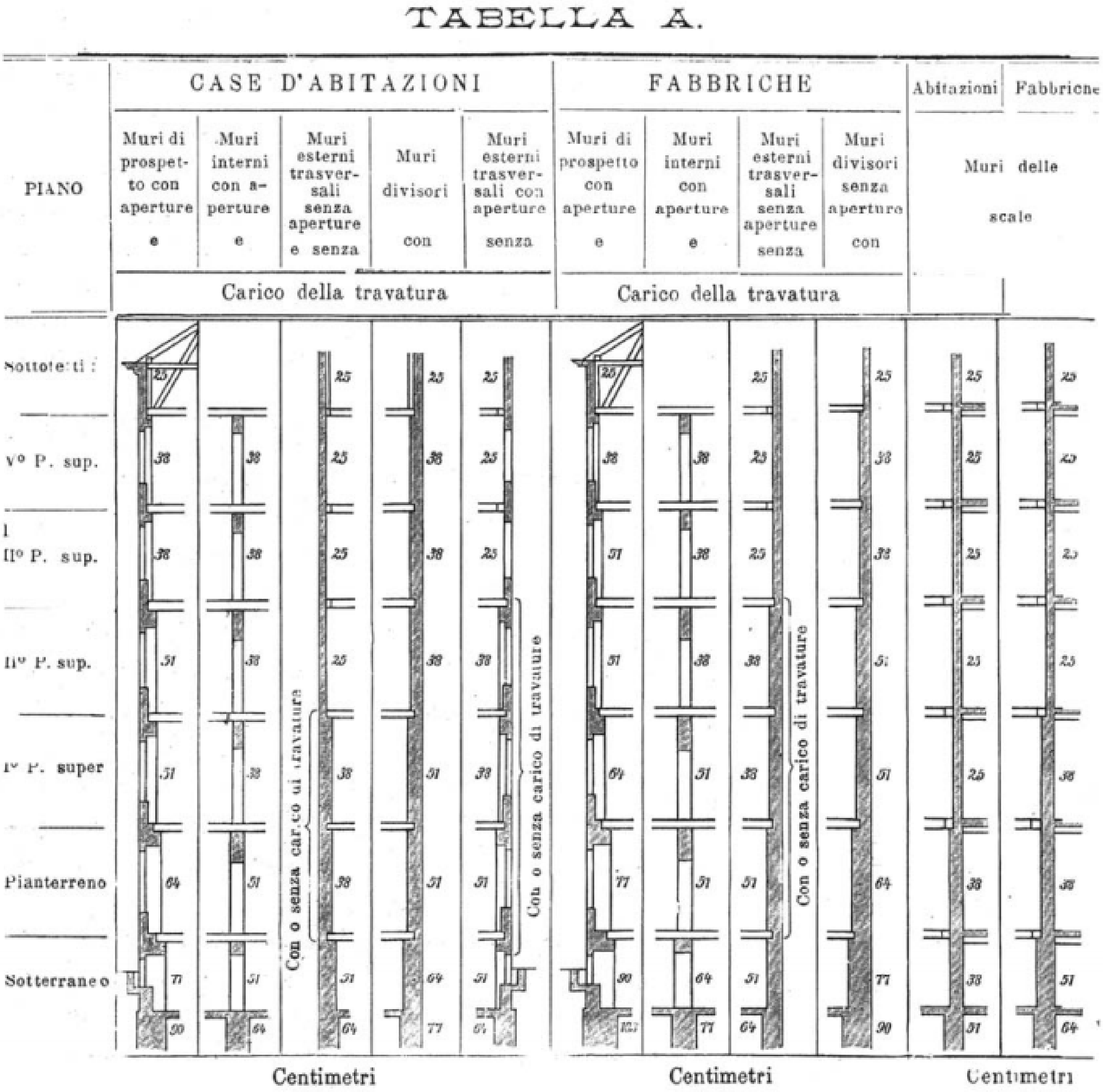


TABELLA A.

PIANO	CASE D'ABITAZIONI					FABBRICHE				Abitazioni	Fabbriche
	Muri di prospetto con aperture e	Muri interni con aperture e	Muri esterni trasversali senza aperture e senza	Muri divisori con	Muri esterni trasversali con aperture senza	Muri di prospetto con aperture e	Muri interni con aperture e	Muri esterni trasversali senza aperture senza	Muri divisori senza aperture con	Muri delle scale	
	Carico della travatura					Carico della travatura					
Sottotetti	25		25	25	25	25		25	25	25	25
V° P. sup.	38	38	25	38	25	38	38	25	38	25	25
III° P. sup.	38	38	25	38	25	51	38	25	38	25	25
II° P. sup.	51	38	25	38	38	51	38	38	51	25	25
I° P. super	51	38	38	51	38	64	51	38	51	25	38
Pianterreno	64	51	38	51	51	77	51	51	64	38	38
Sotterraneo	71	51	51	64	51	90	64	51	77	38	51
	90	64	64	77	64	103	77	64	90	51	64

Con o senza carico di travatura

Con o senza carico di travature

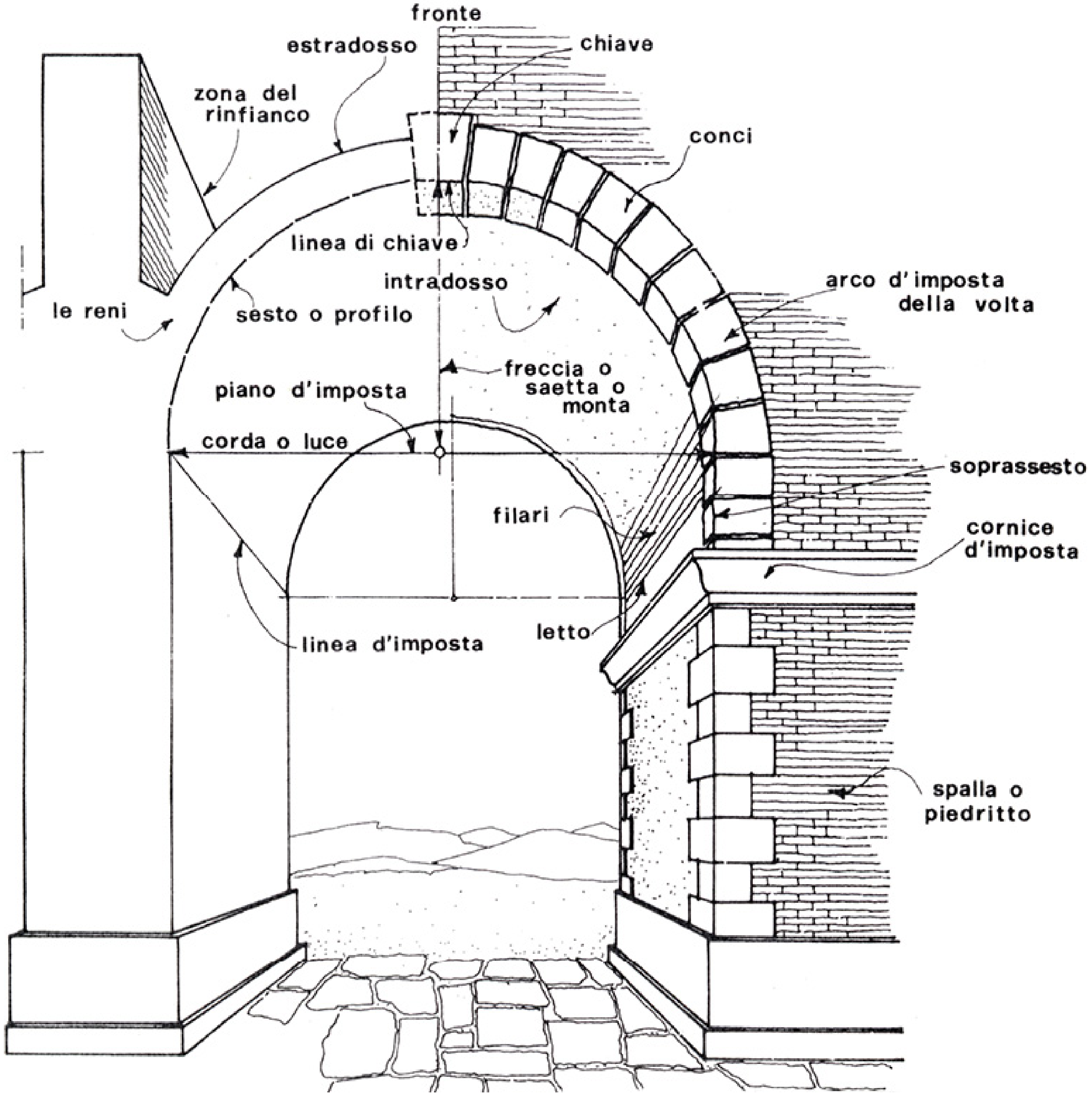
Con o senza carico di travature

Centimetri

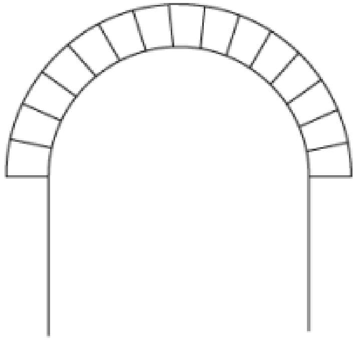
Centimetri

Centimetri

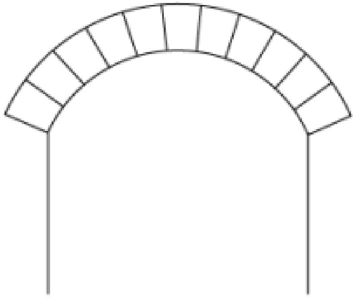
Archi e Volte


fronte
estradosso
chiave
zona del rinfianco
conci
linea di chiave
intradosso
arco d'imposta della volta
le reni
sesto o profilo
freccia o saetta o monta
piano d'imposta
corda o luce
soprassesto
filari
cornice d'imposta
letto
linea d'imposta
spalla o piedritto

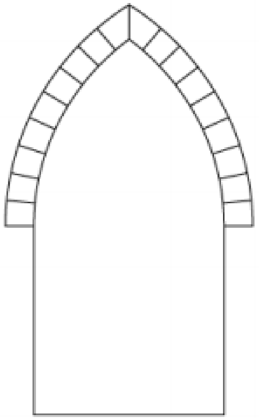
Tipologie di Archi



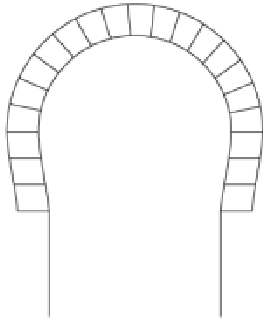
A TUTTO SESTO



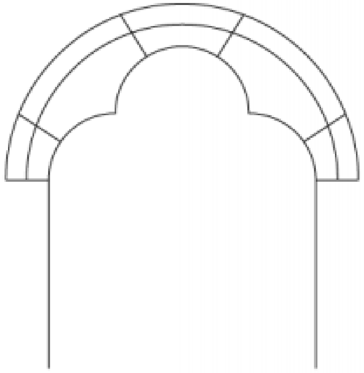
A SESTEO RIBASSATO



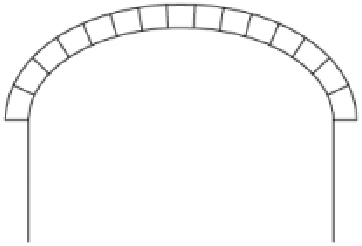
A SESTO ACUTO



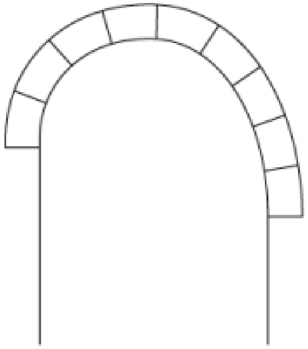
A SESTO RIALZATO



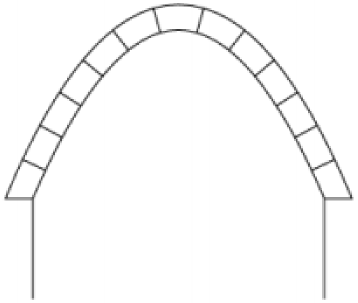
TRILOBATO



ELLITTICO



RAMPANTE



PARABOLICO

Centine per la realizzazione di Archi e Volte

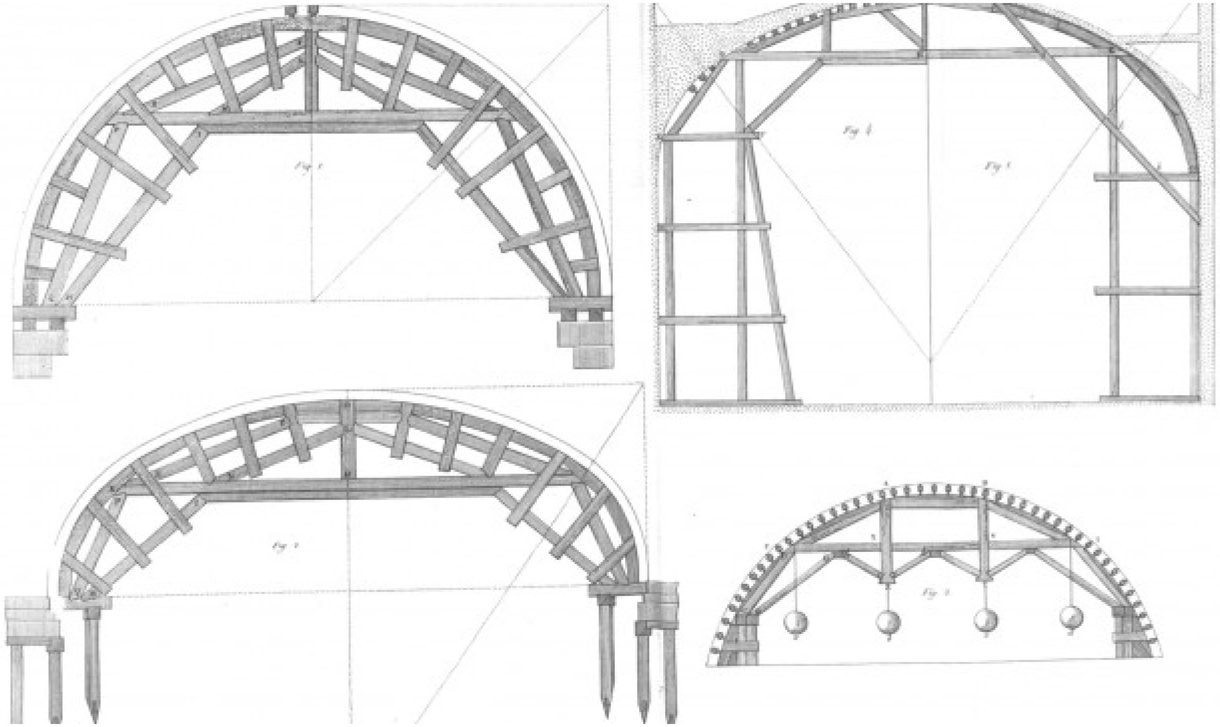
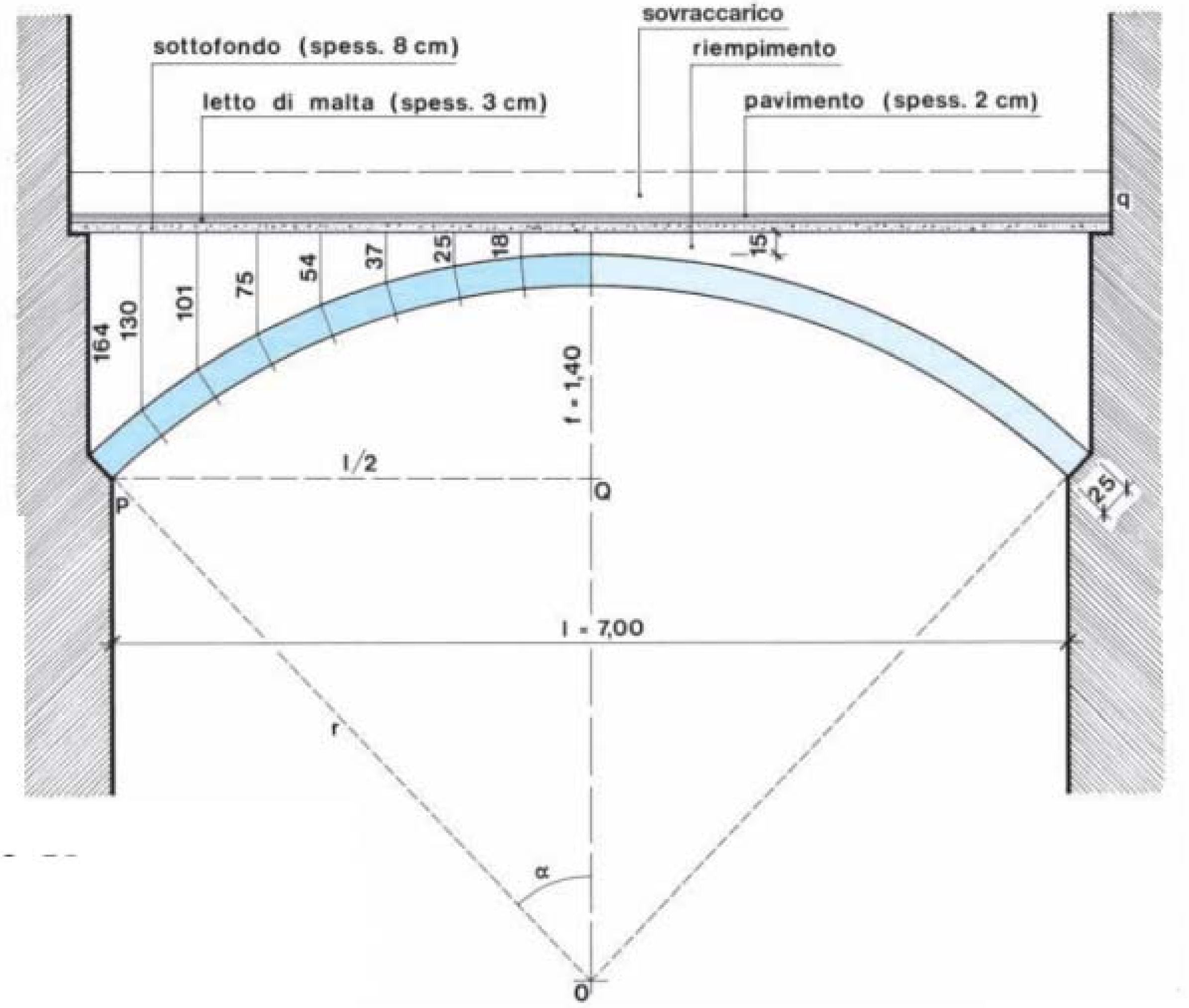
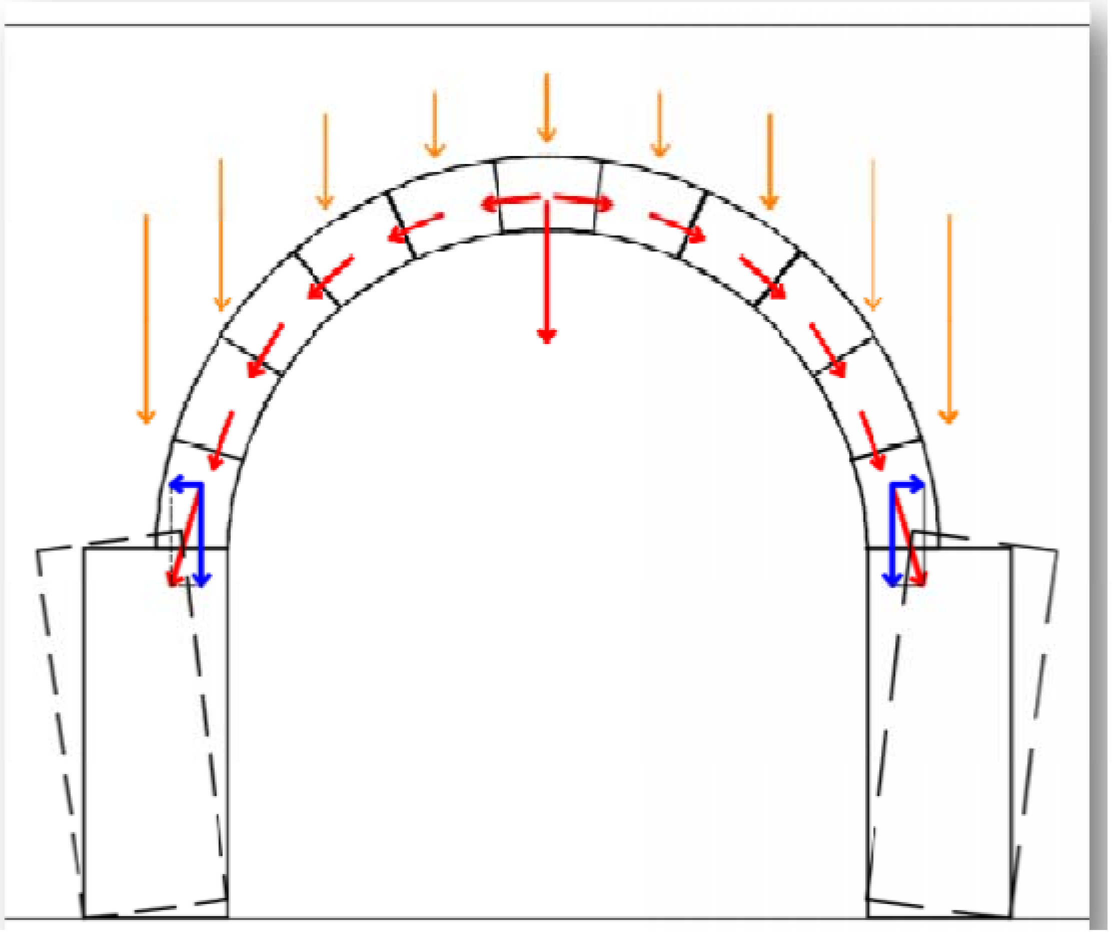


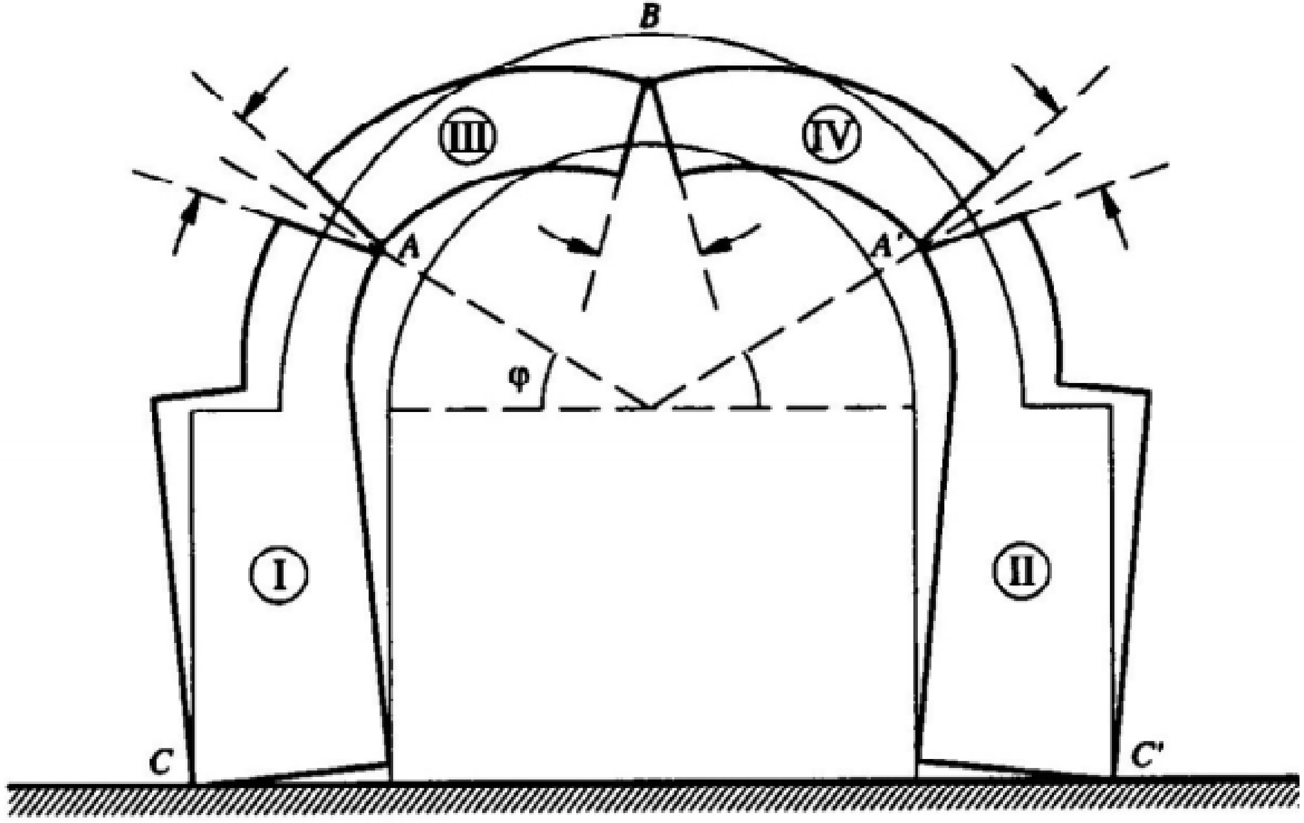
Fig. 1
Fig. 4
Fig. 5
Fig. 2
Fig. 3


sovraccarico
sottofondo (spess. 8 cm)
riempimento
letto di malta (spess. 3 cm)
pavimento (spess. 2 cm)
q
15
164
130
101
75
54
37
25
18
f = 1,40
l/2
P
Q
25
l = 7,00
r
α
O

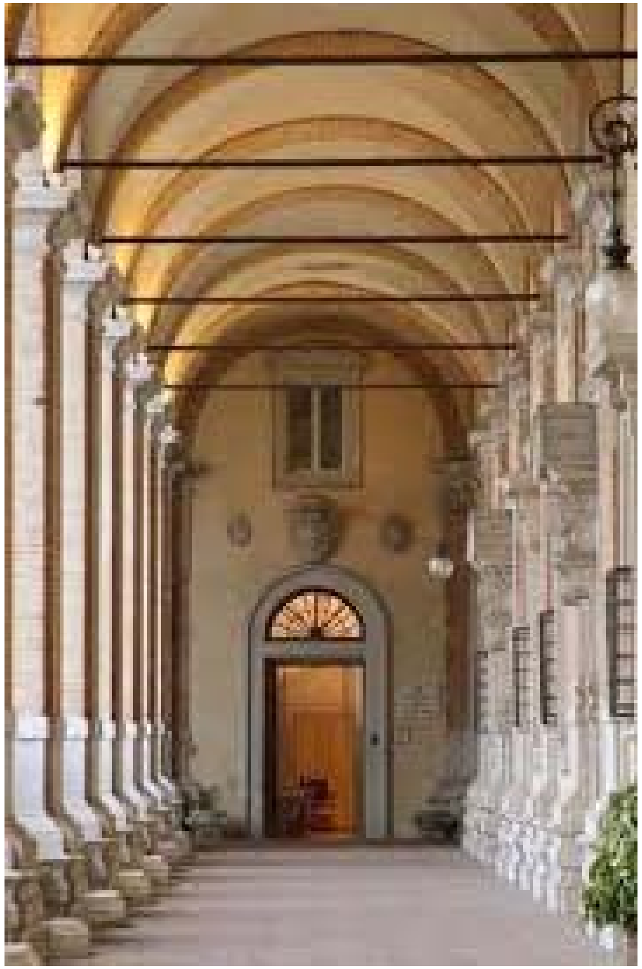
Funzionamento

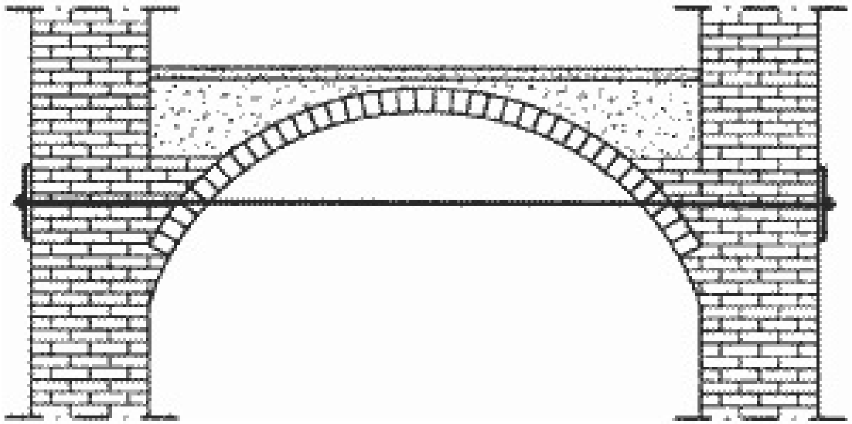


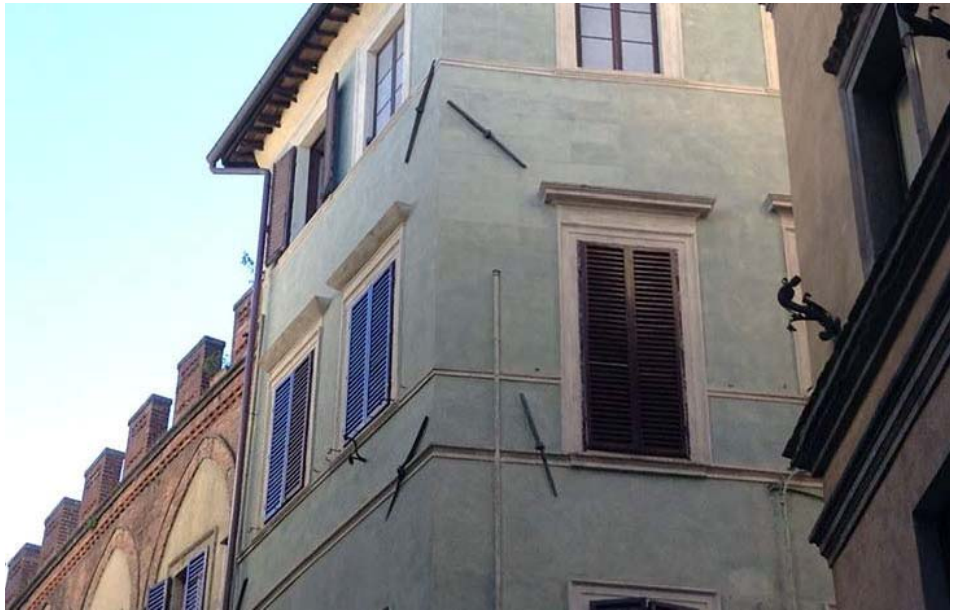
Meccanismo di rottura Arco a tutto sesto


B
III
IV
A
A'
φ
I
II
C
C'

Tiranti







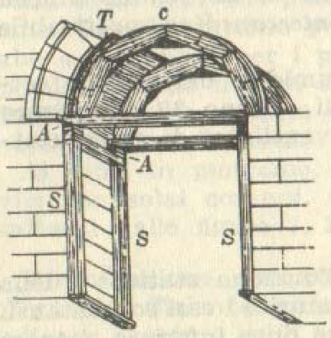


Fig. 16

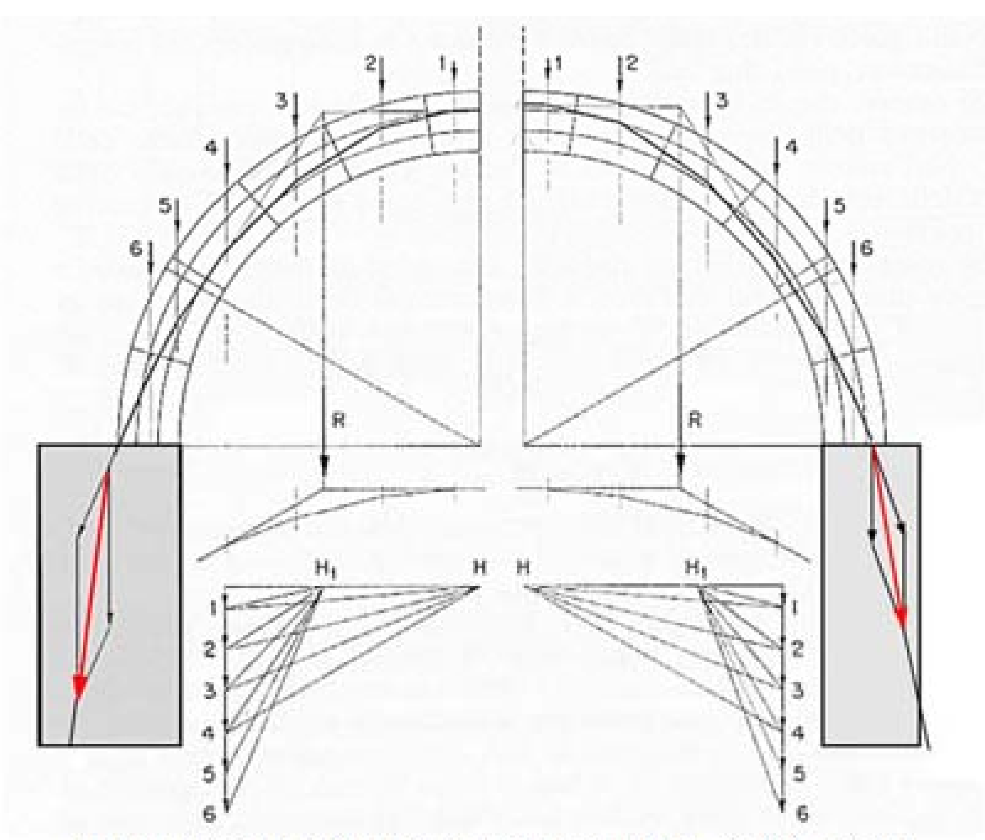
Le piattabande di piccola luce sono formate di un solo elemento di pietra. La luce massima delle piattabande formate di molti elementi (mattoni o pietre) è di circa 6 metri. Oggidì si preferisce sostituirle con travi di ferro a doppio T oppure con travi di c. a.

La costruzione degli archi e delle piattabande avviene dopo preparata una conveniente armatura di sostegno, normalmente in legname e formata (fig. 16) di una impalcatura di sostegno *S*, di due centine *c*, di un *tamburo T* e di apparecchi di disarmo *A*.

TAB. III. — Spessore degli archi e delle piattabande.

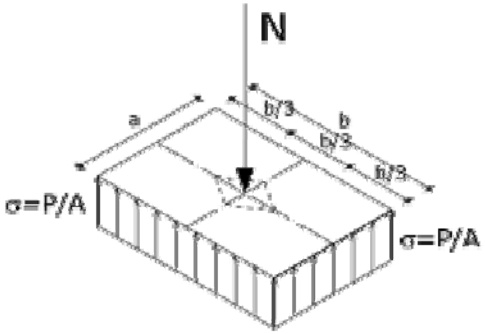
Luce m	Spessore in teste			
	Archi		Piattabande	
	a pieno centro	a centro ribassato	senza arco di scarico	con arco di scarico
fino a 1	2	2	2	—
1,00 ÷ 1,50	2	3	3	3
1,50 ÷ 2,00	3	3	4	3
2,00 ÷ 3,00	3	4	5	4
3,00 ÷ 4,00	4	4	6	4
4,00 ÷ 5,00	4	5	7	5
5,00 ÷ 6,00	5	6	—	—
6,00 ÷ 8,00	6	7	—	—

Statica dell’arco Tracciamento curva delle pressioni

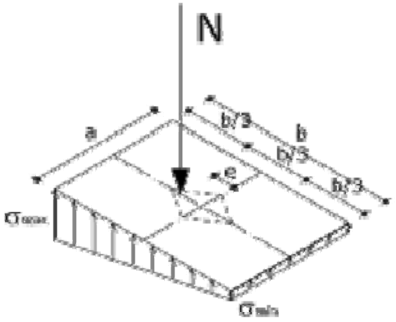

1
2
3
4
5
6
R
R
H
H
H_1
H_1

Tracciamento del poligono funicolare dei carichi (A. Giuffrè, 19923)

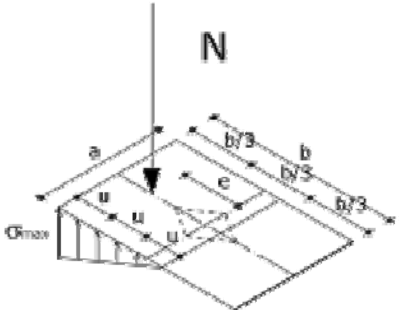
e=0


N
a
b/3
b
b/3
b/3
σ=P/A
σ=P/A

e≤b/6

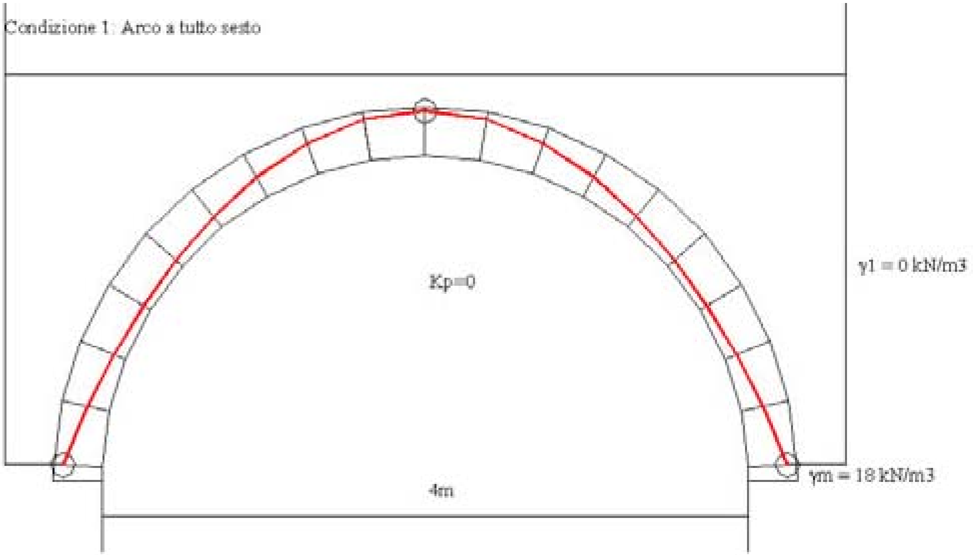

N
a
b/3
b
b/3
b/3
e
σmax
σmin

b/6≤e≤b/2

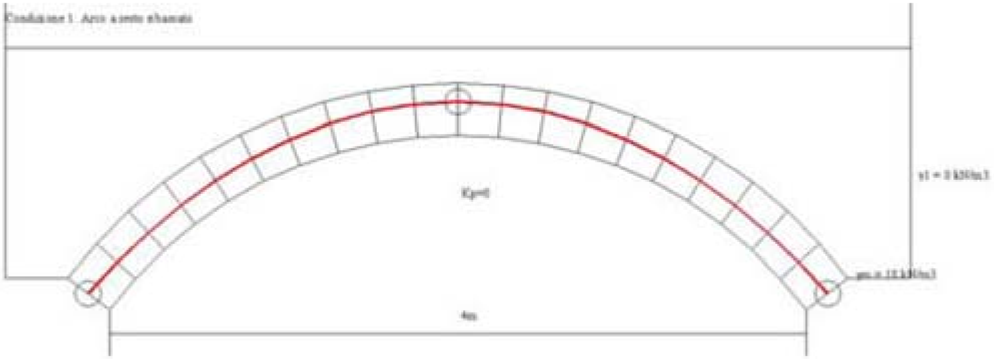

N
a
b/3
b
b/3
b/3
e
u
u
u
σmax

Metodo del Mèry 1840

Arco a tutto sesto

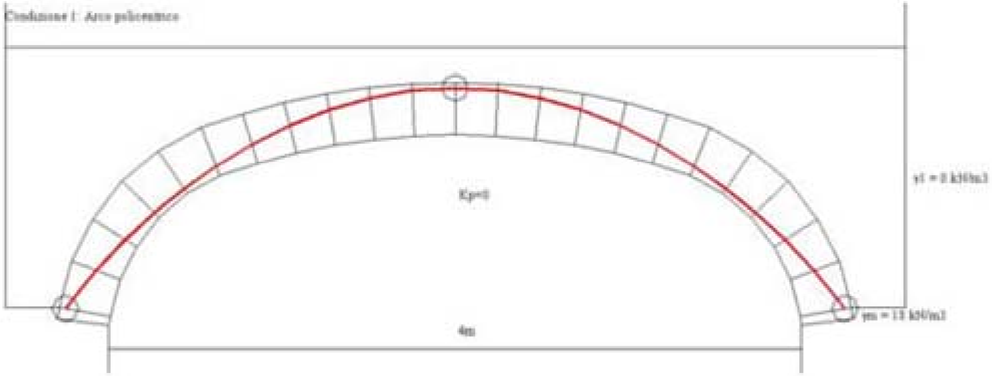

Condizione 1: Arco a tutto sesto
γ1 = 0 kN/m3
Kp=0
γm = 18 kN/m3
4m

Arco ribassato

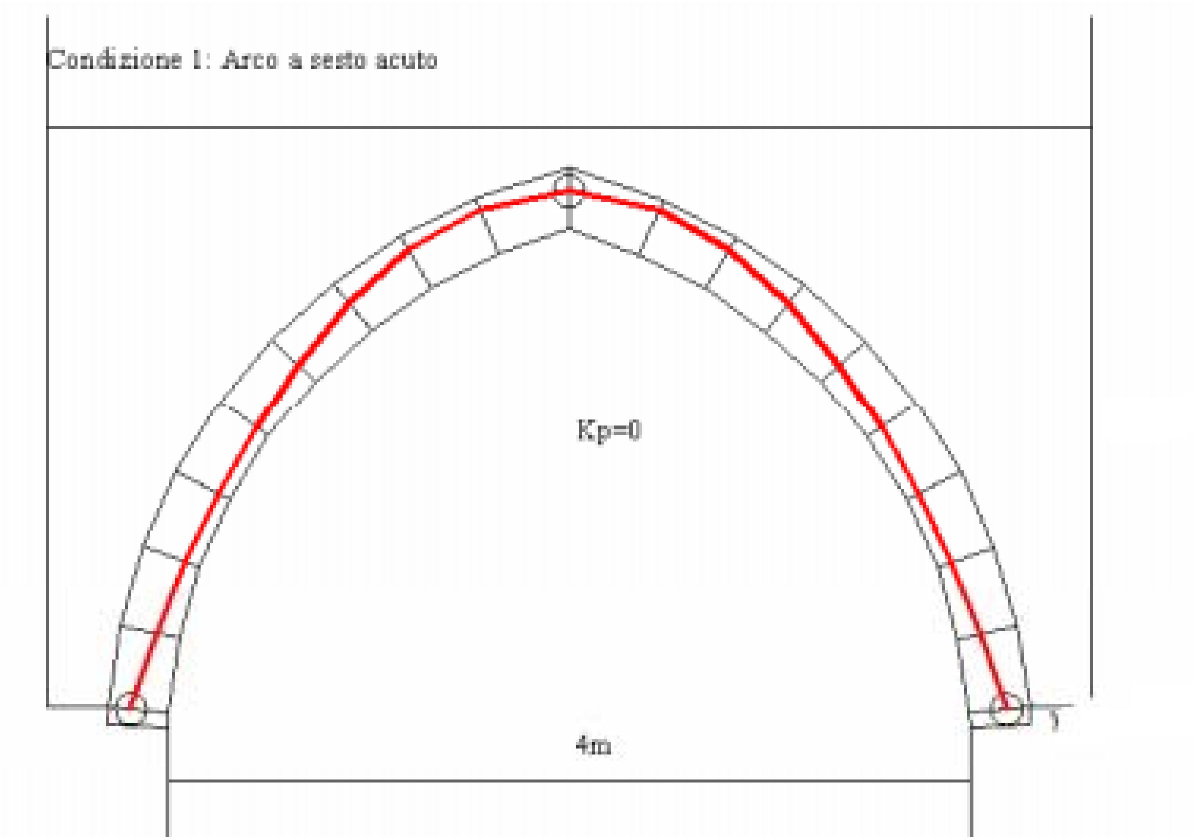

Condizione 1: Arco a sesto ribassato
γ1 = 0 kN/m3
Kp=0
γm = 18 kN/m3
4m

Arco Policentrico

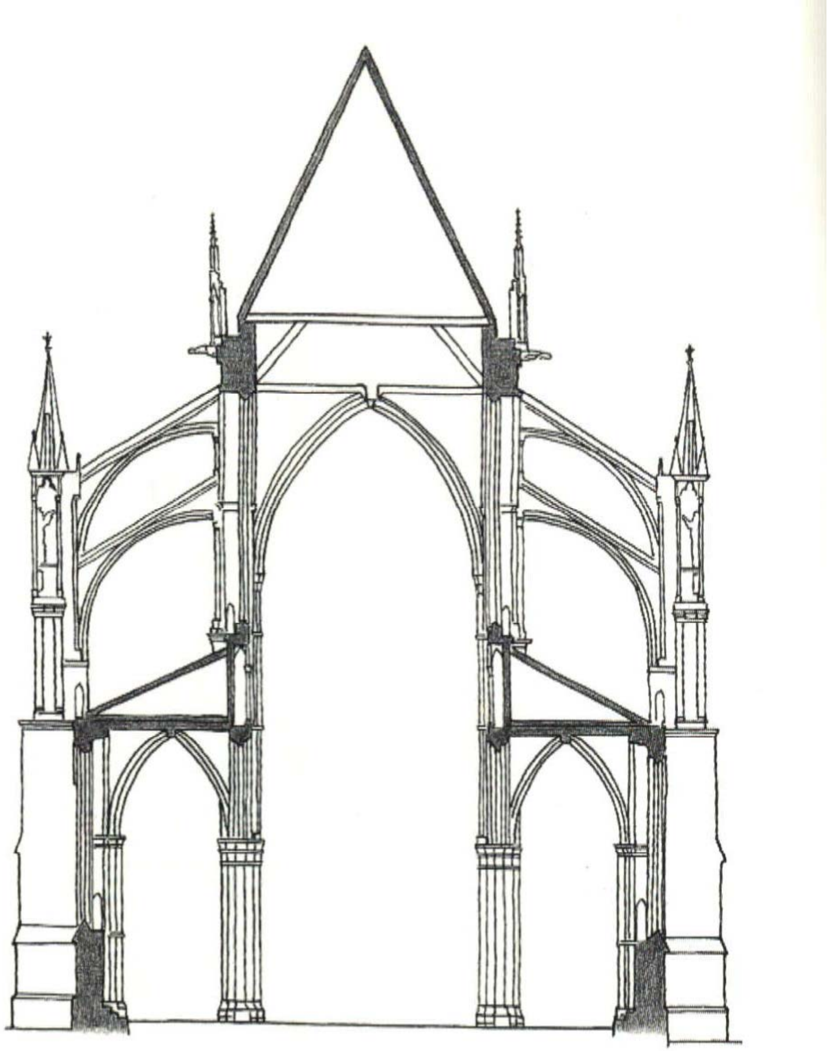
Arco policentrico


Condizione 1: Arco policentrico
γt = 0 kN/m3
Kp=0
γm = 13 kN/m3
4m

Arco a sesto acuto

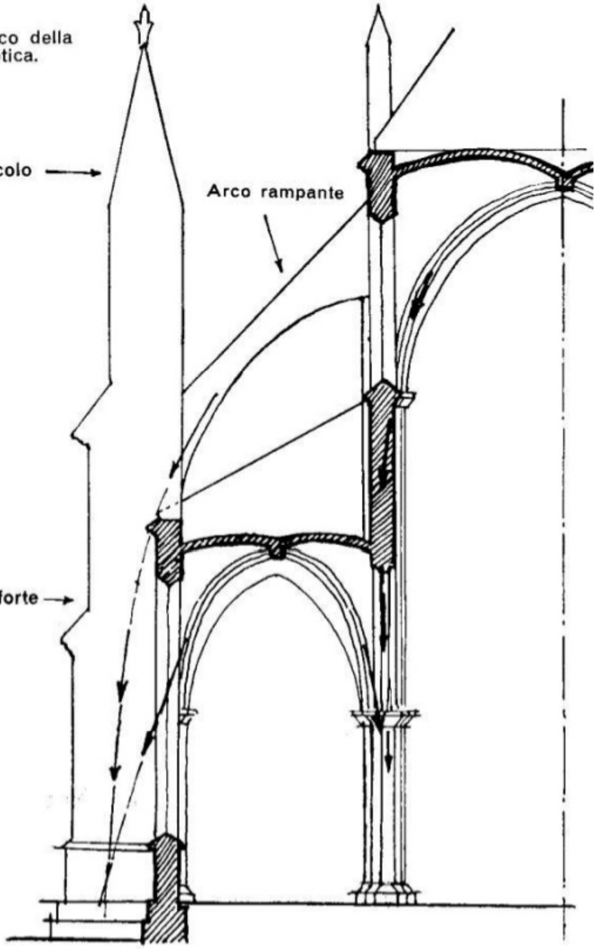

Condizione 1: Arco a sesto acuto
Kp=0
4m

Per eliminare il tirante

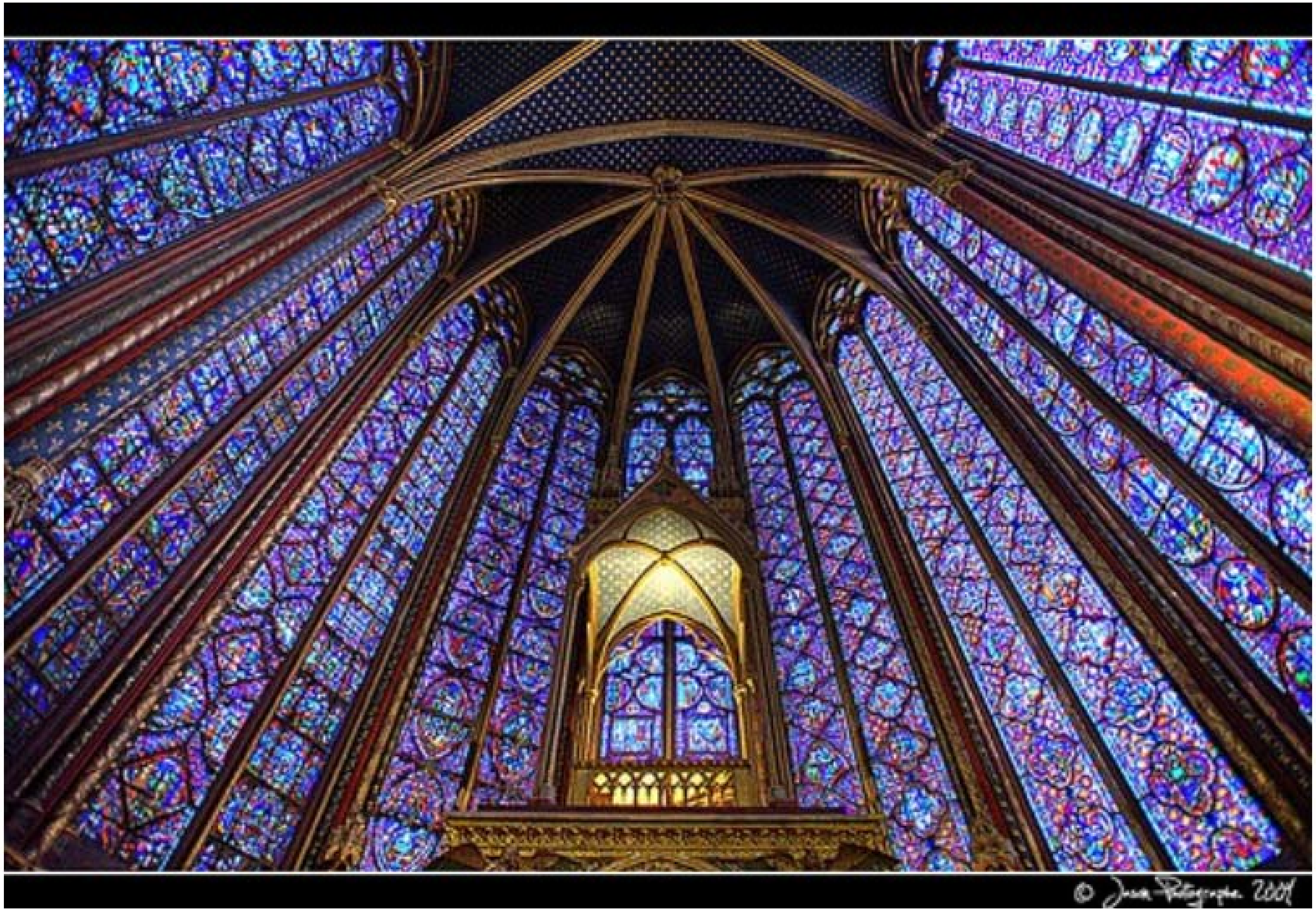


Basilica con contrafforti (sezione)

Equilibrio statico della
costruzione gotica.

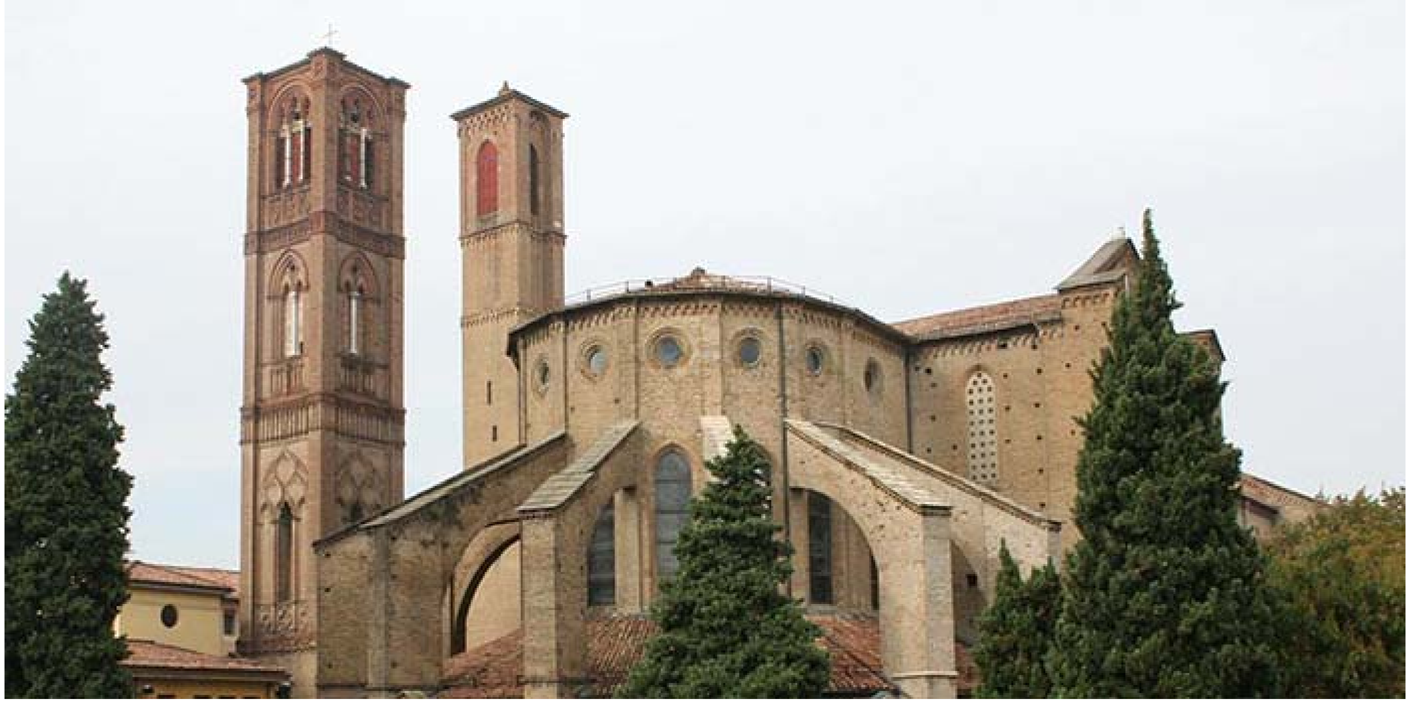

Pinnacolo
Arco rampante
Contrafforte

Le vetrate gotiche

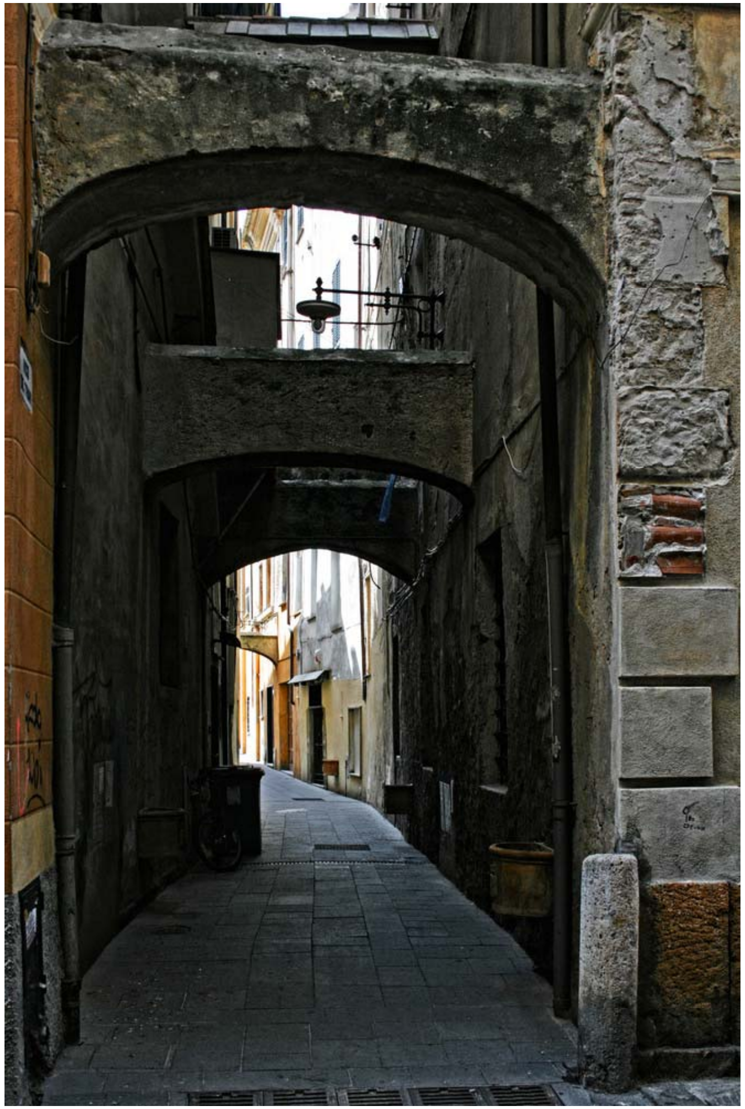


Parigi, Sainte Chapelle, 1243-1248. Cappella superiore

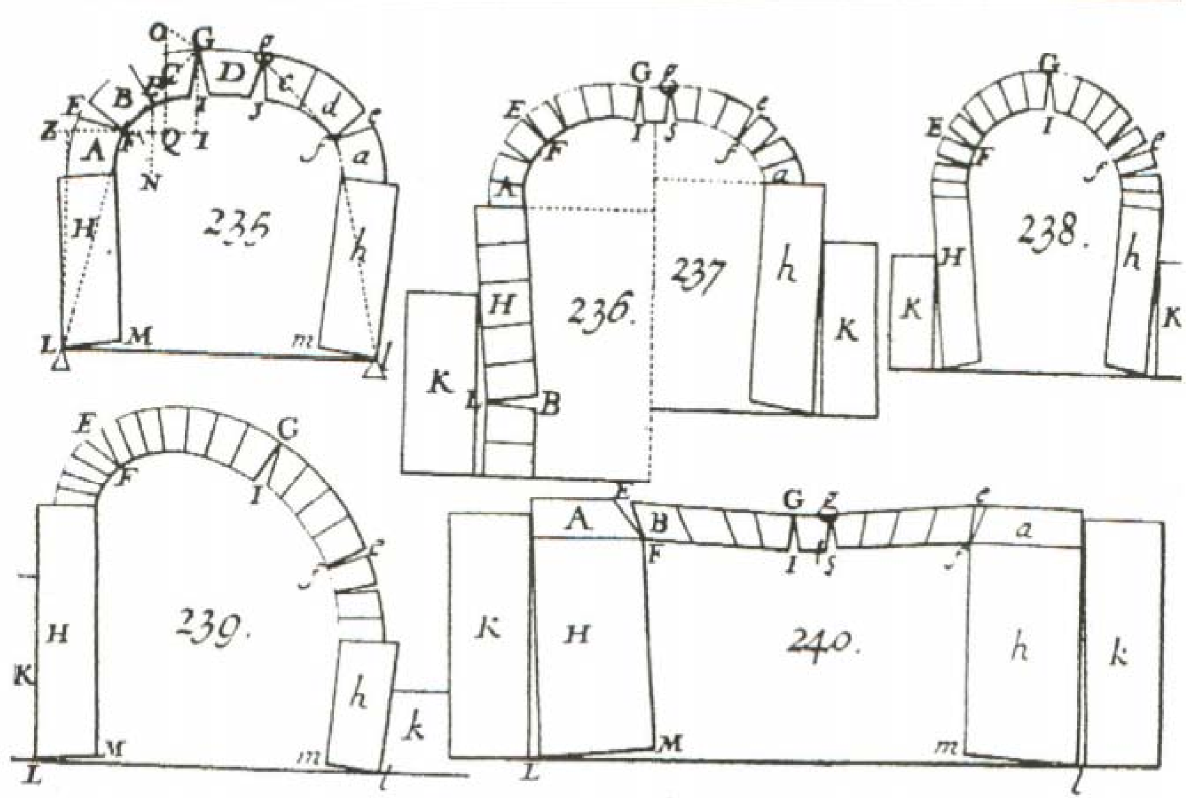
Contrafforti

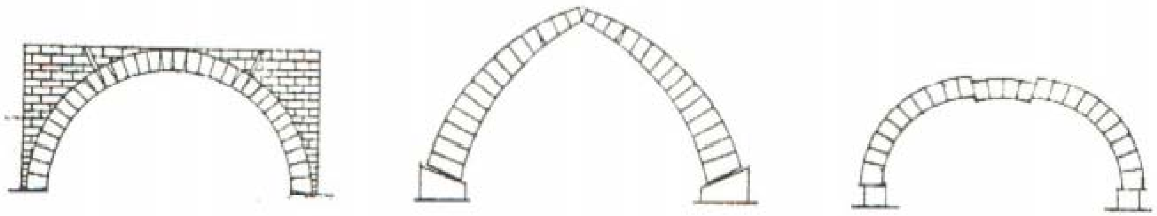


archi di controspinta

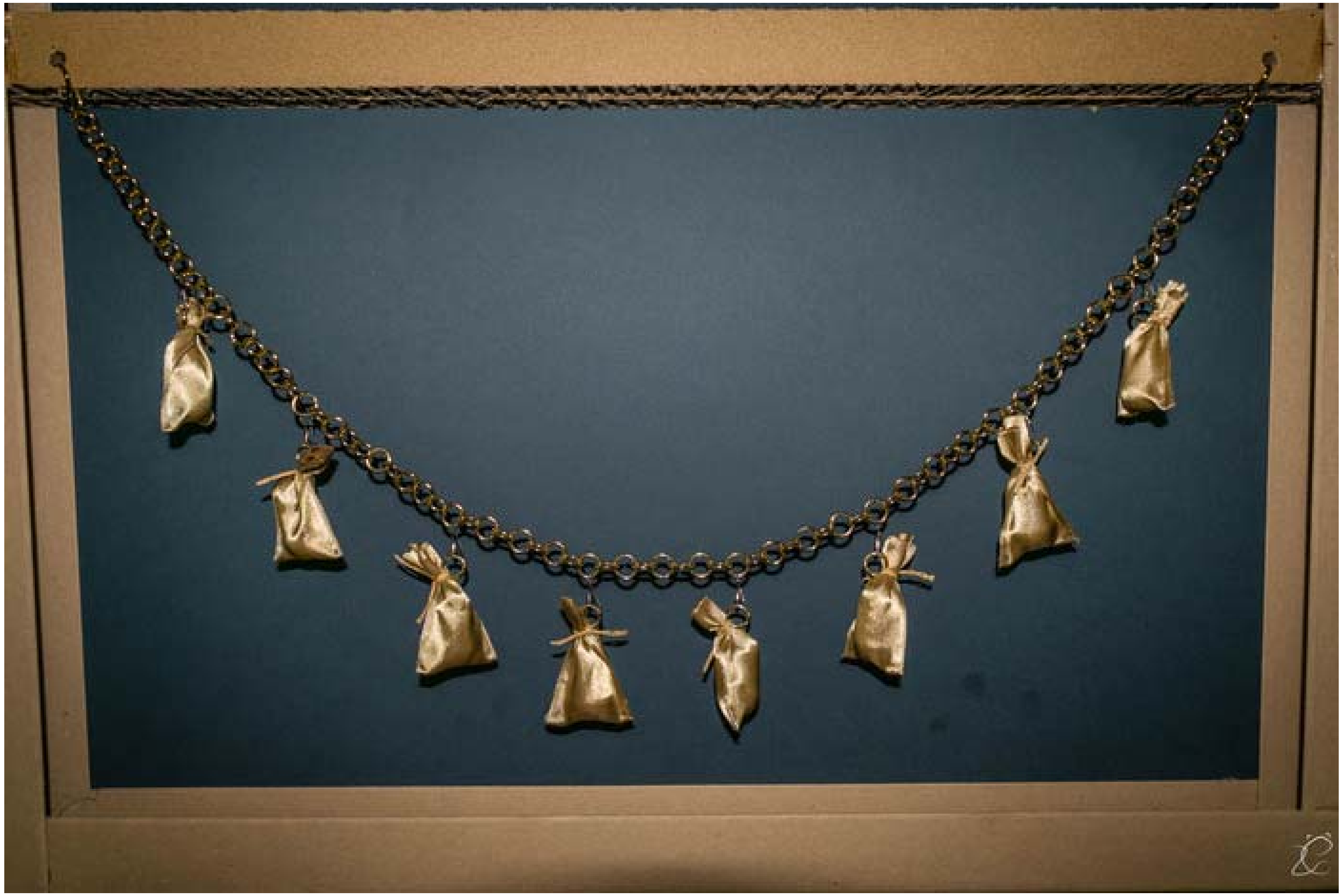


La forma ideale dell’arco

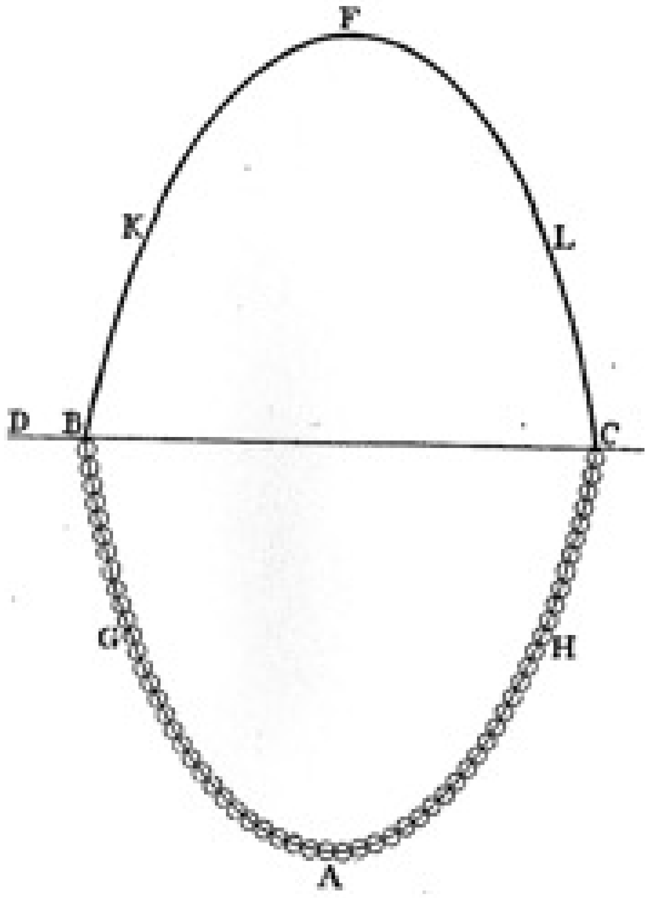

235
236.
237
238.
239.
240.
O
G
g
C
D
c
d
e
E
B
Z
A
F
Q
I
S
N
a
f
H
h
K
k
L
M
m
l



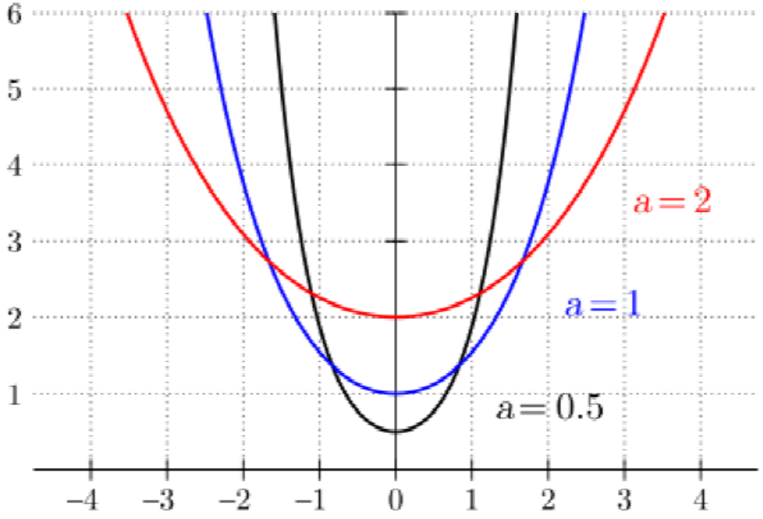
La forma ideale dell’arco



La forma ideale dell’arco
La Catenaria rovesciata



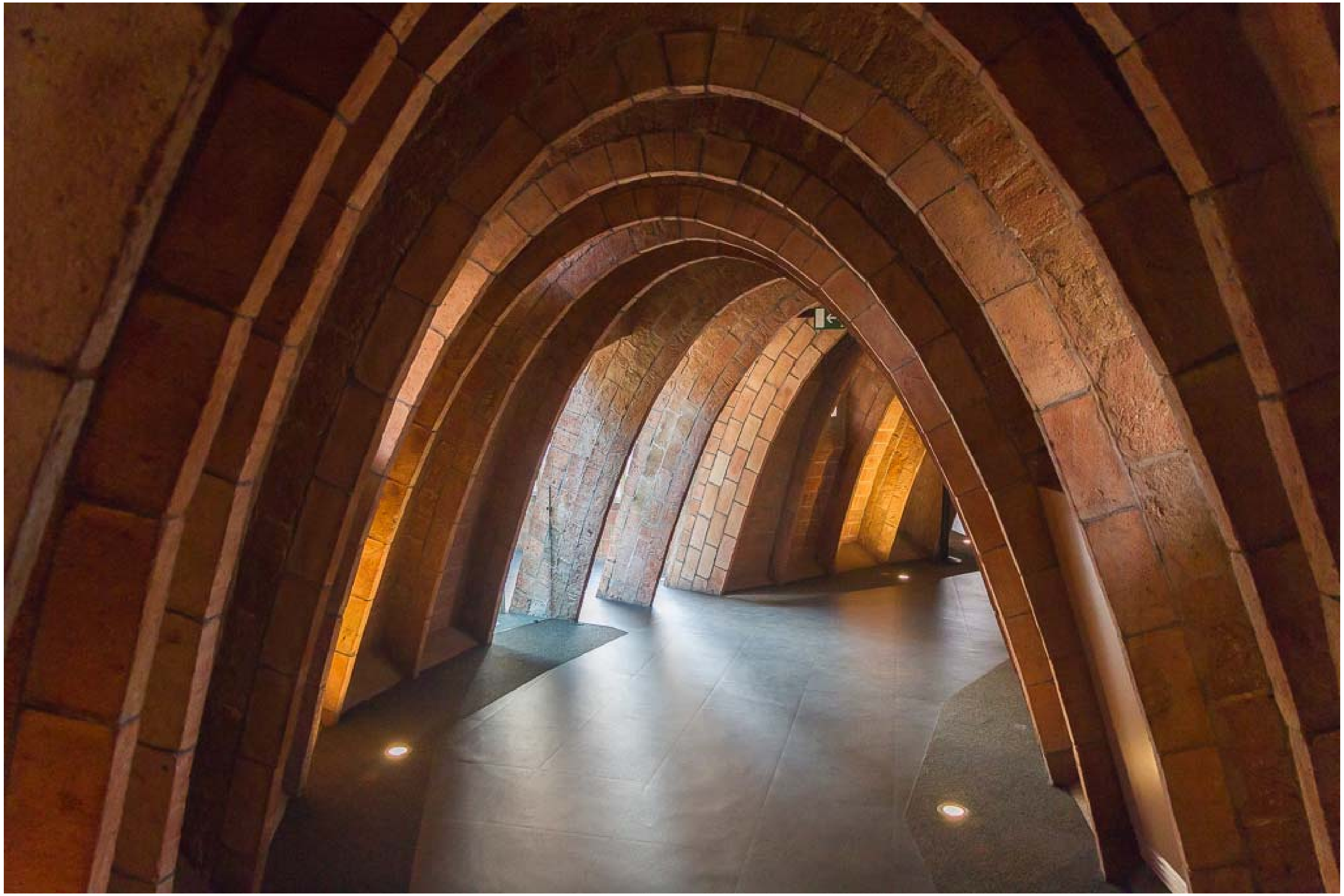
Equazione matematica della catenaria



$$y = a \cdot \cosh\left(\frac{x}{a}\right) = \frac{a}{2}\left(e^{x/a} + e^{-x/a}\right)$$

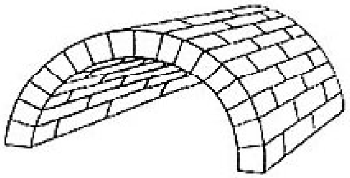
$$y = \cosh(x)$$

Arco Catenario

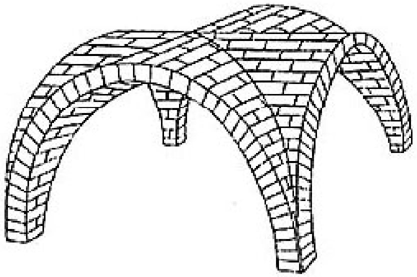


La Pedrera Barcellona – Gaudì 1912

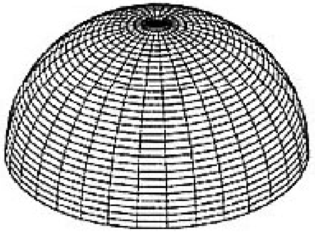
Volte e Cupole



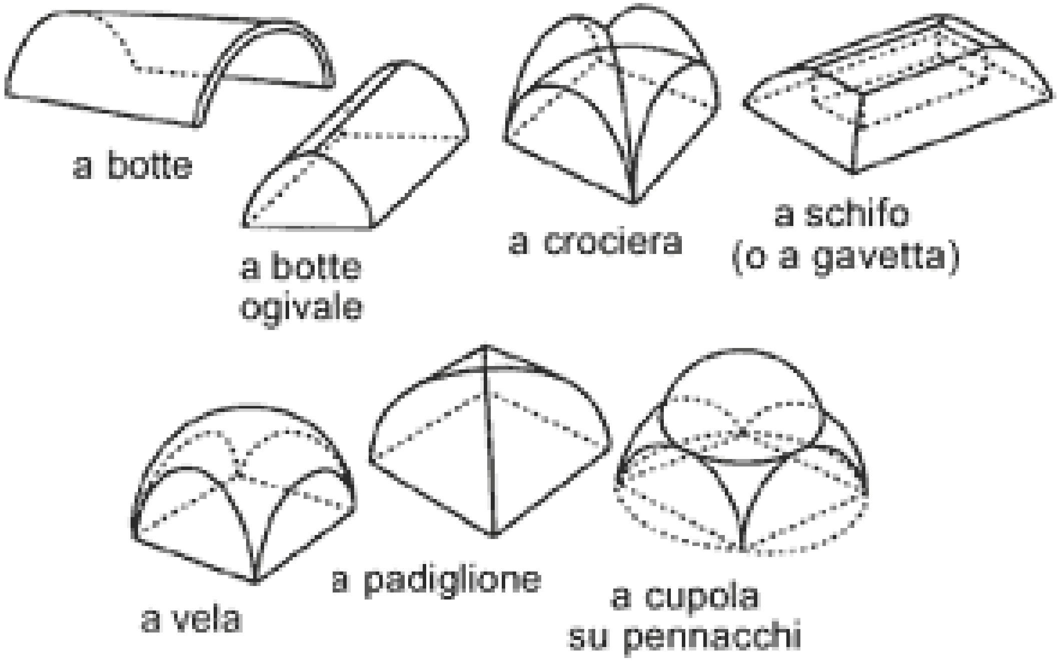
volta a botte



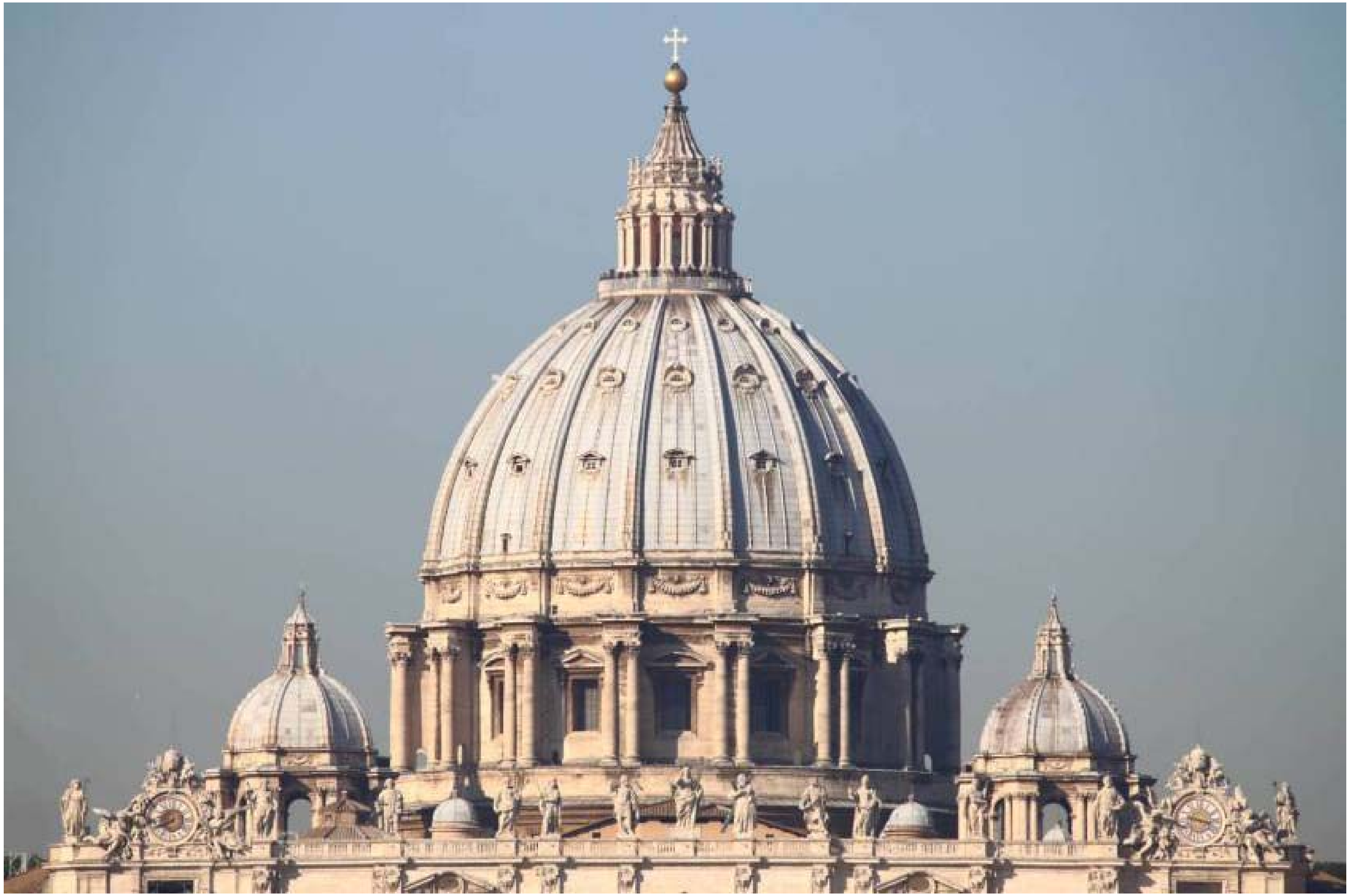
volta a crociera



cupola

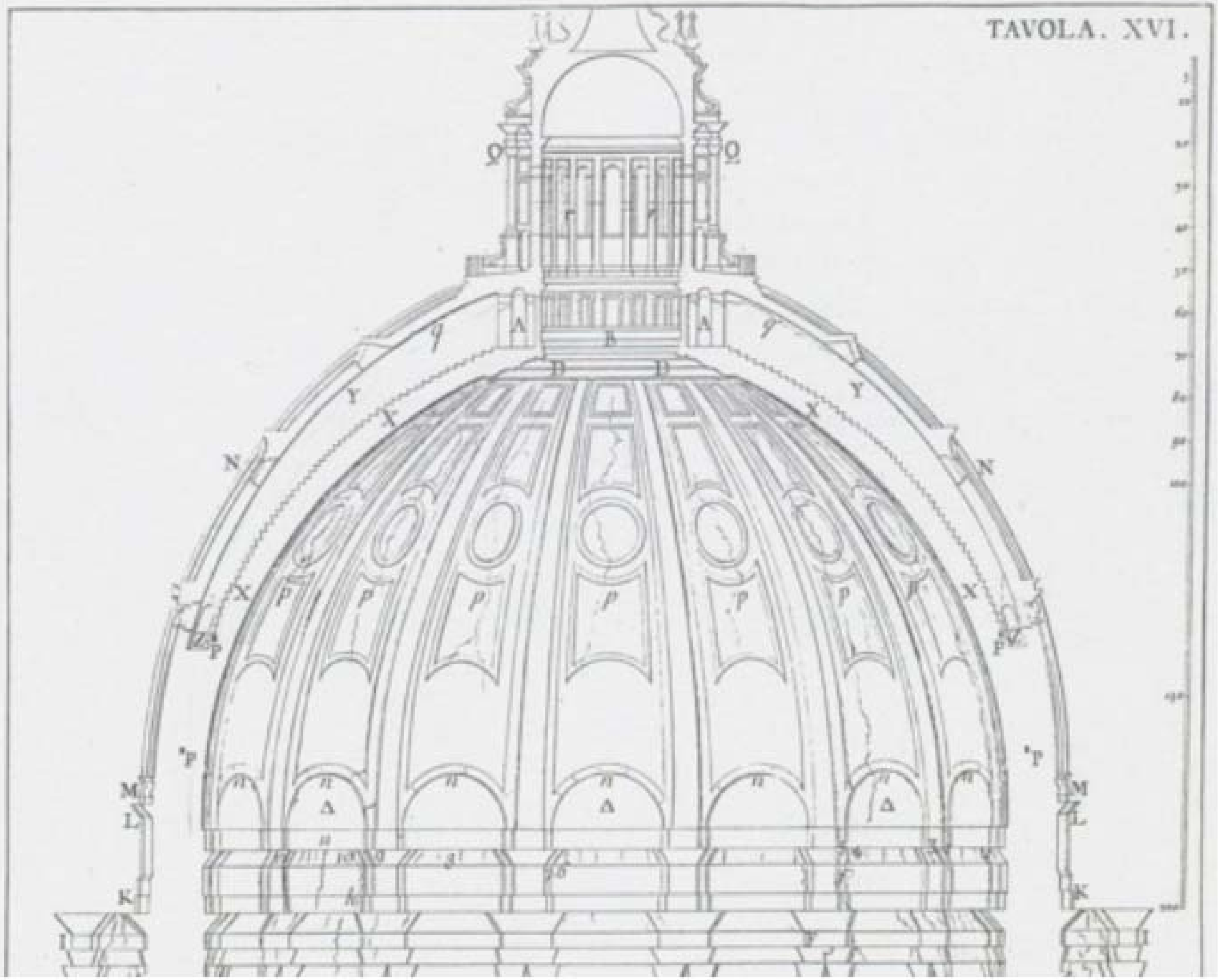

a botte
a botte ogivale
a crociera
a schifo (o a gavetta)
a vela
a padiglione
a cupola su pennacchi

Le cupole

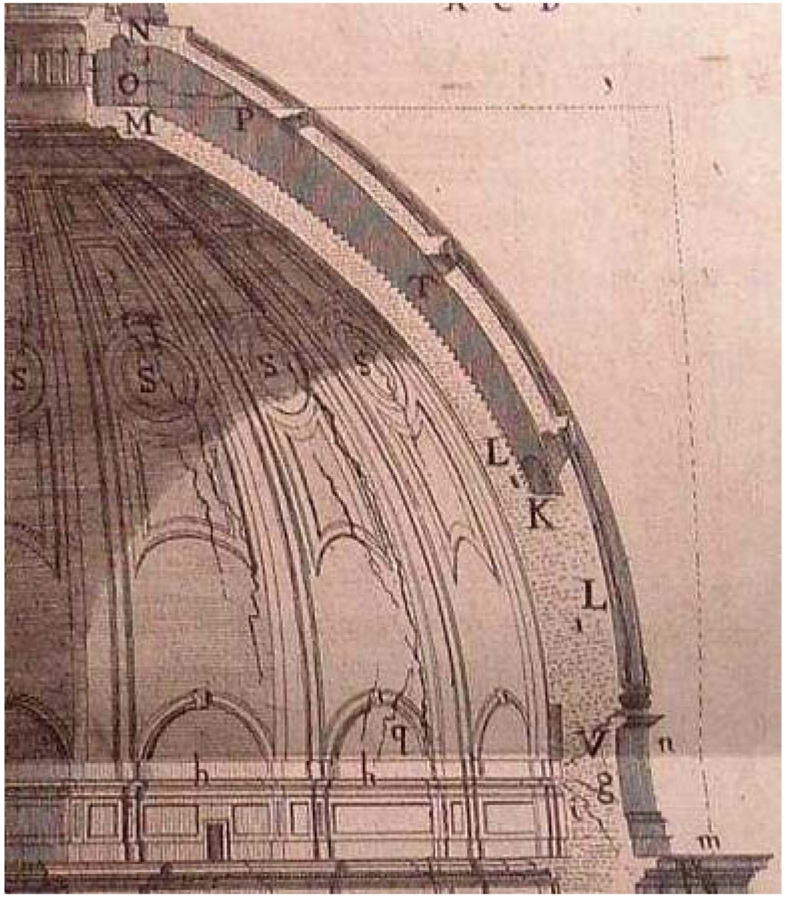


San Pietro Roma 1500/1600

TAVOLA. XVI.


Q
Q
A
B
A
D
D
q
q
Y
Y
X
X
N
N
X
X
p
p
p
p
p
p
p
p
Z
P
P
Z
P
P
M
M
L
L
n
n
n
n
n
n
n
Δ
Δ
Δ
K
K
I
I

Quadro fessurativo del 1603


N
O
M
P
T
S
S
S
S
L
K
L
V
n
g
h
h
q
m

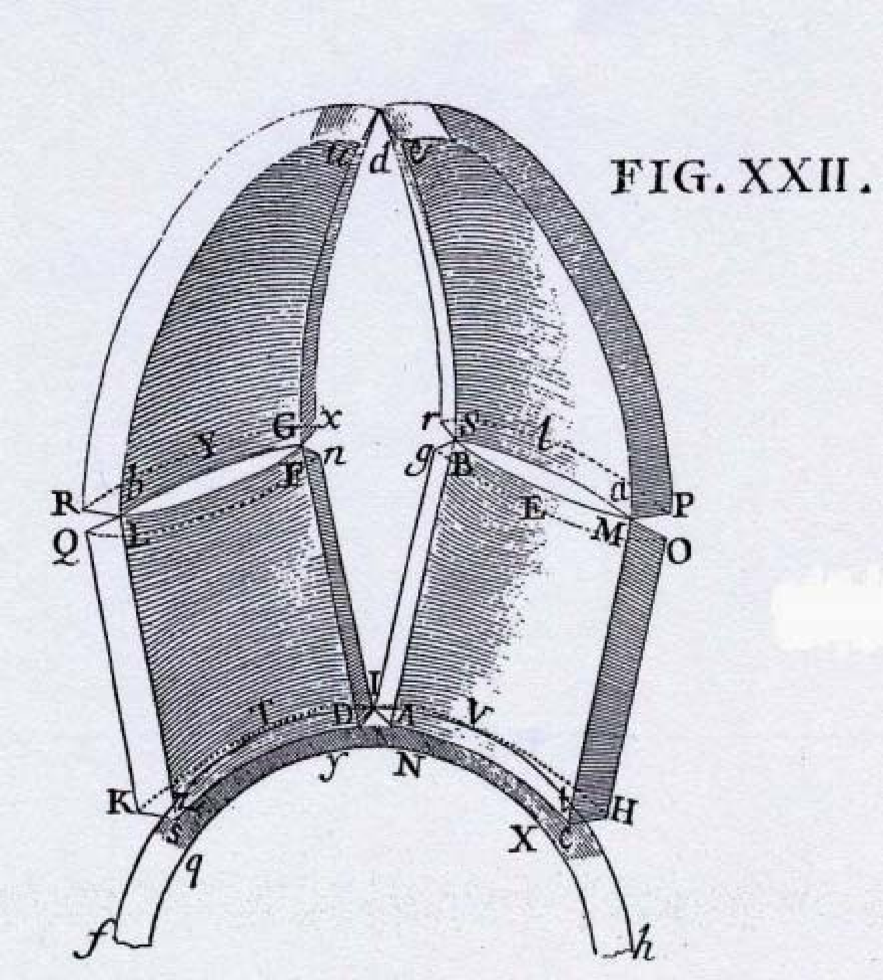


FIG. XXII.
u
d
V
G
x
n
F
Y
r
S
t
g
B
E
a
R
b
P
Q
L
M
O
I
T
D
A
V
y
N
K
s
H
X
c
q
f
h

Giovanni Poleni 1743

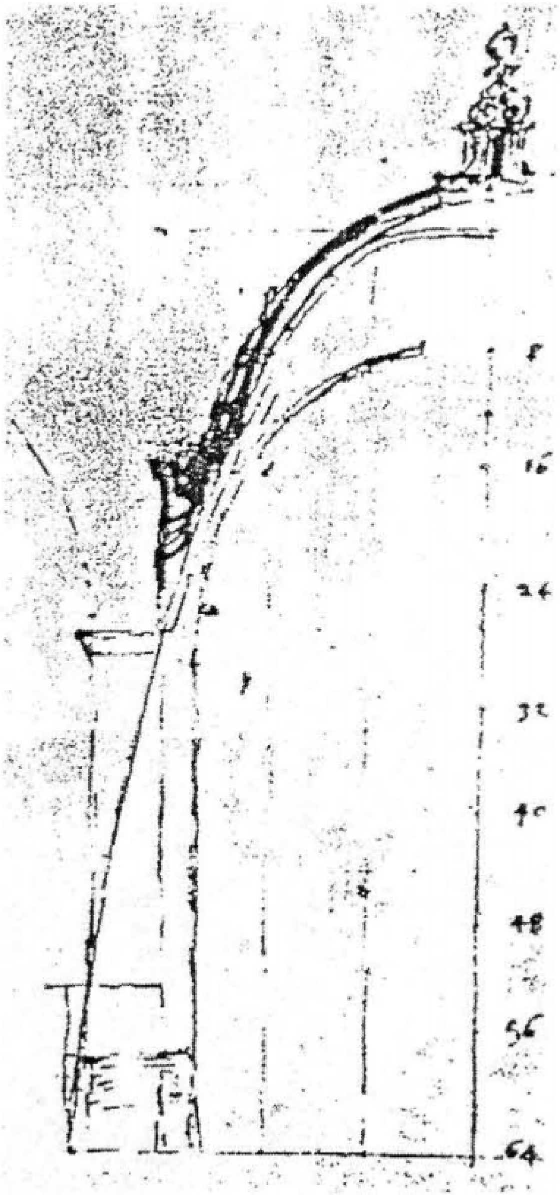

8
16
24
32
40
48
56
64

FIG. XIV.

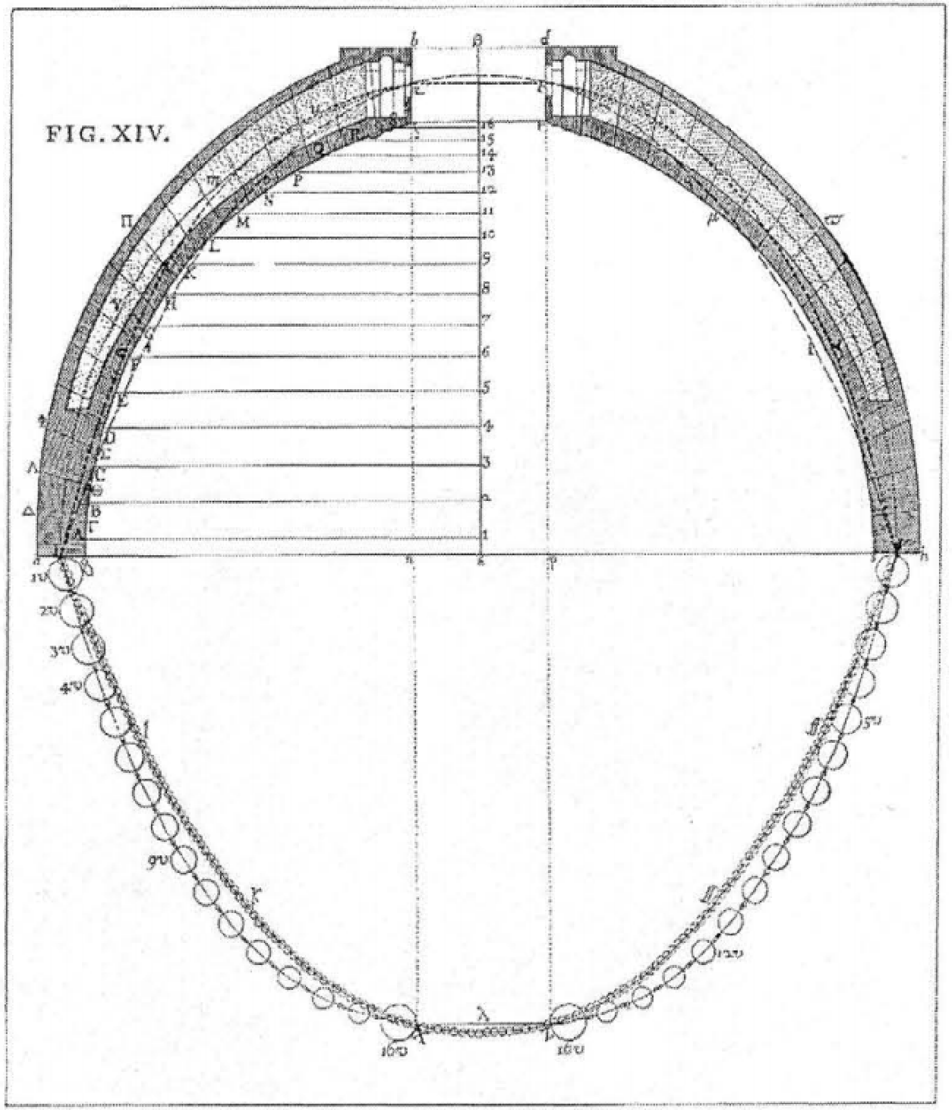
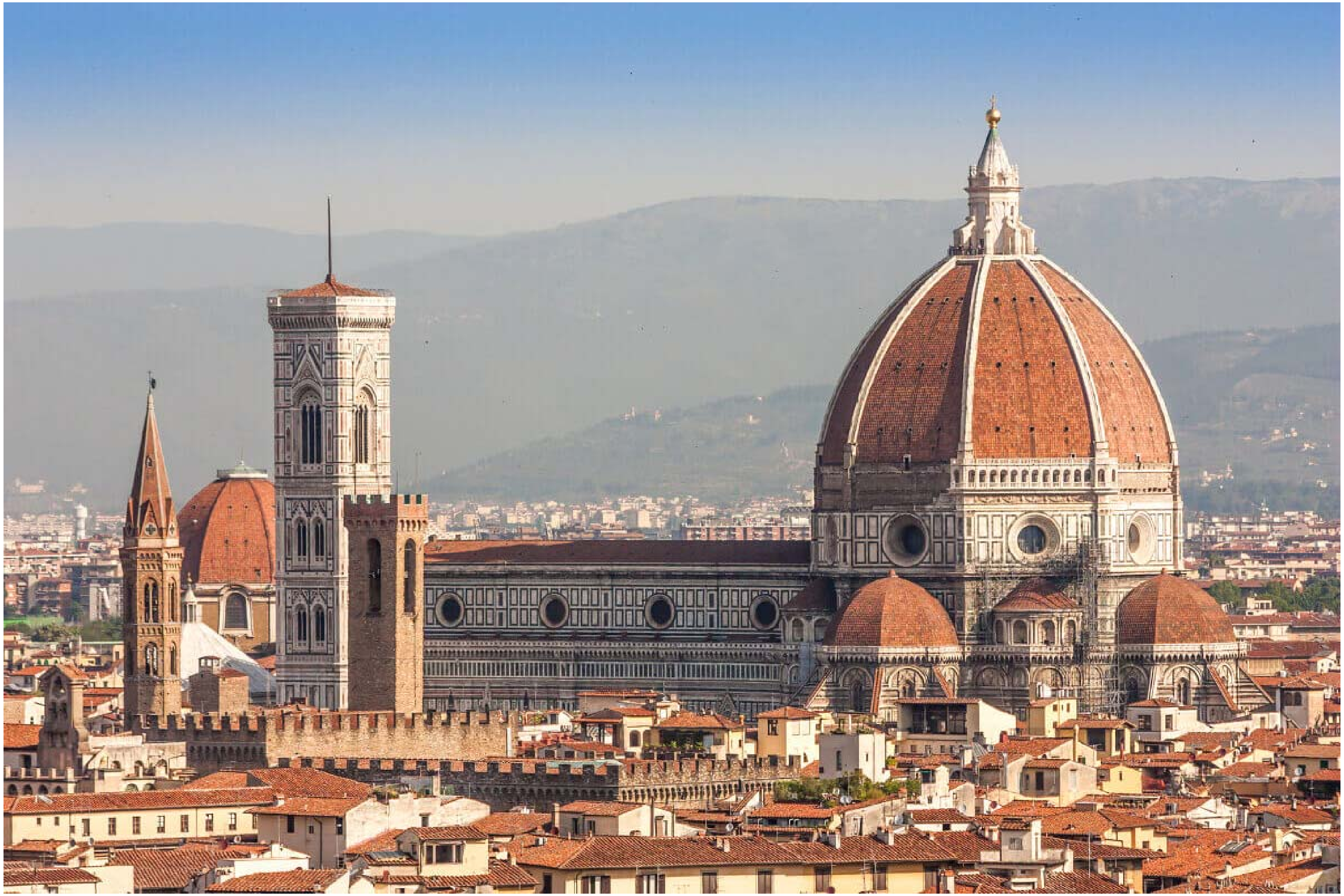

b
β
d
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Π
M
N
P
L
K
H
Λ
Δ
μ
ϖ
1υ
2υ
3υ
4υ
5υ
9υ
12υ
16υ
λ

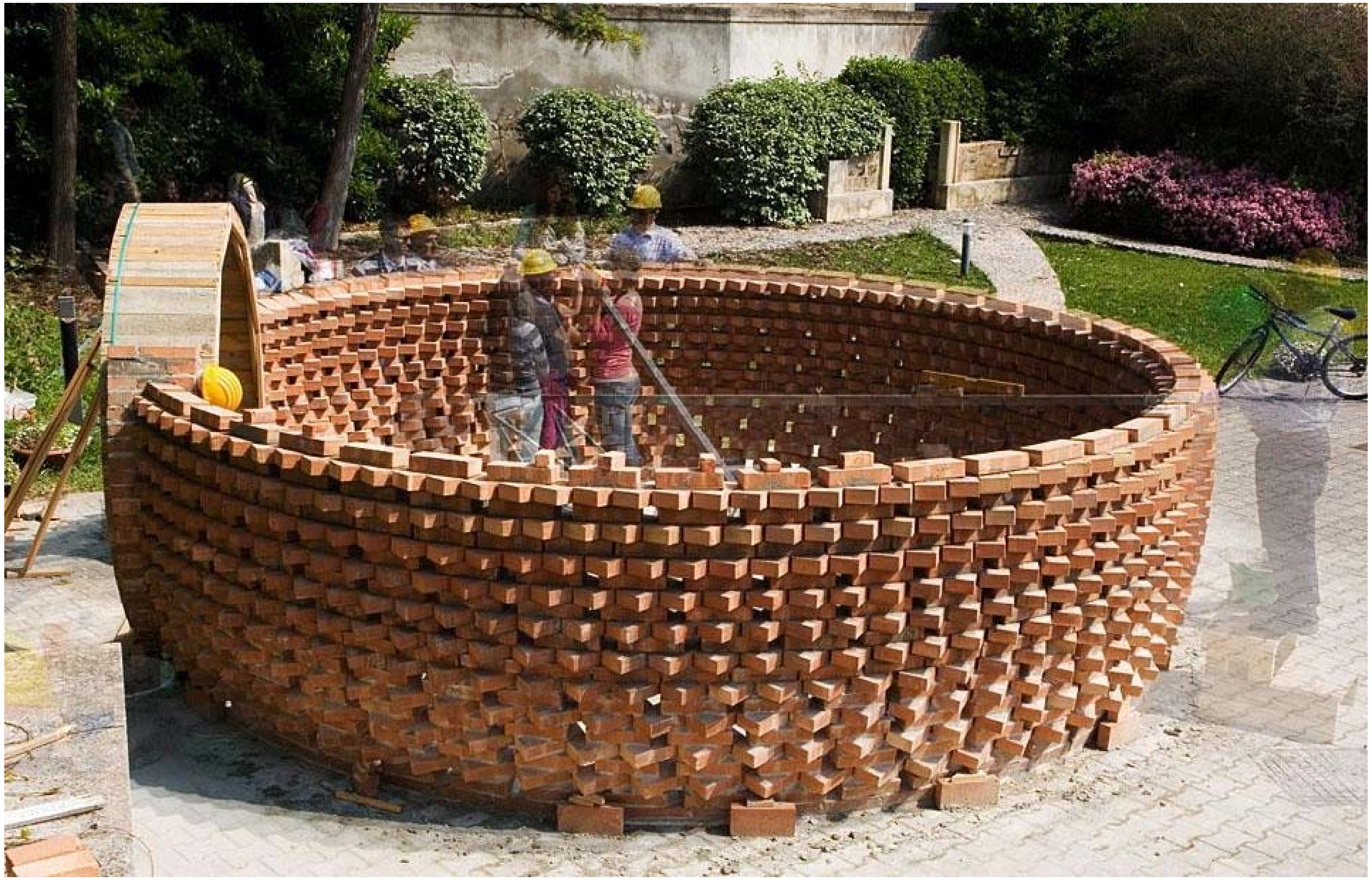
Figure 17. Poleni's hanging model constructed to check the stability of St.

Cupola del Brunelleschi Santa Maria del Fiore Firenze

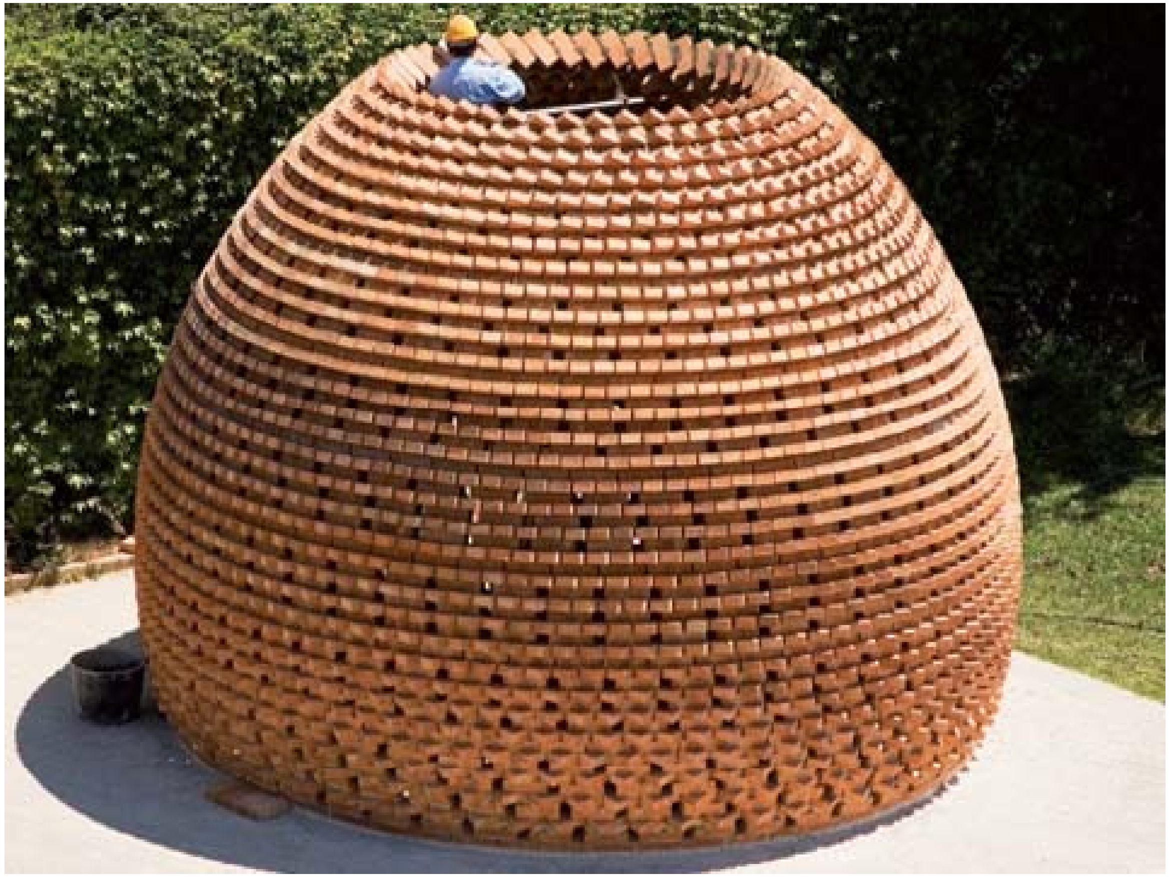


1450 circa

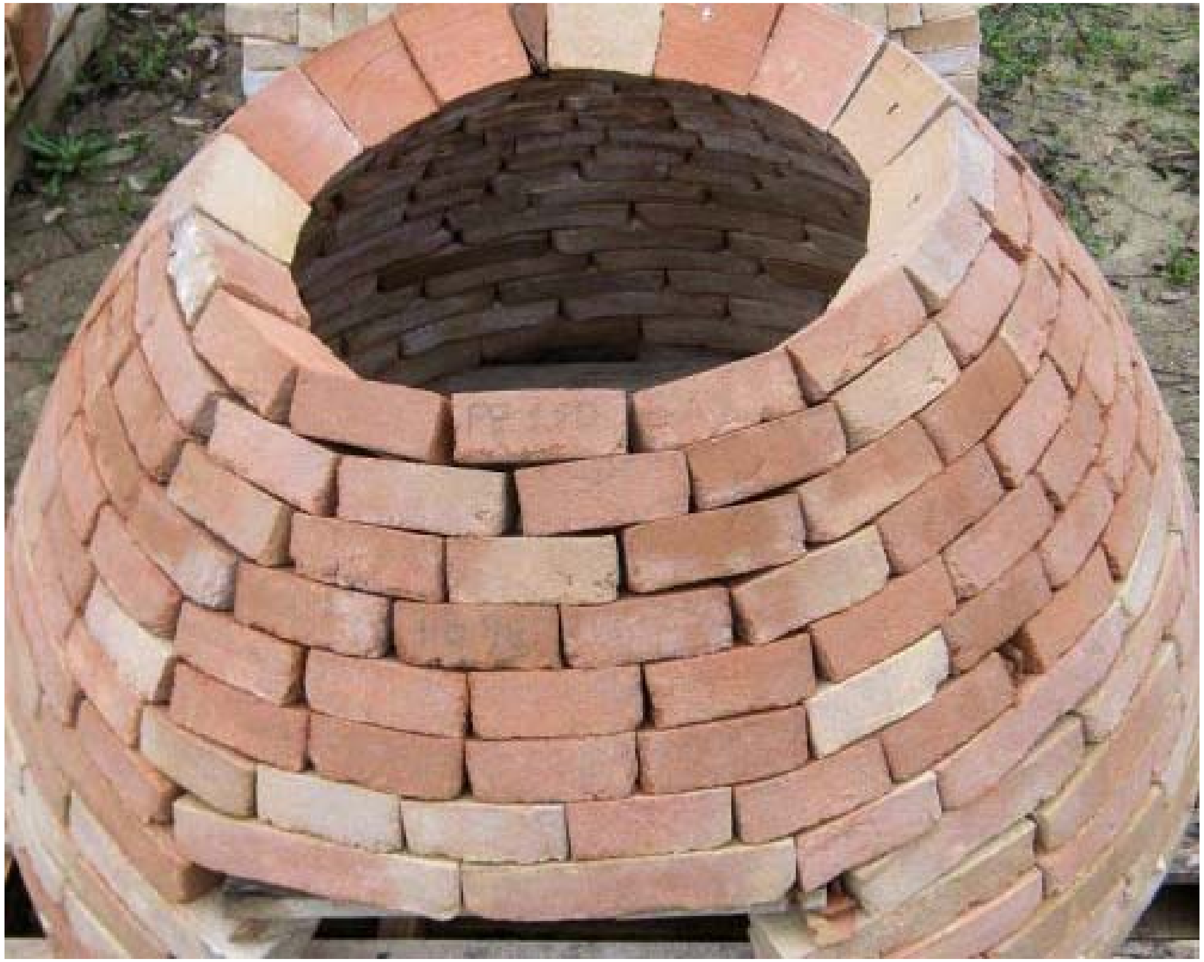
Schema costruttivo cupola del Brunelleschi



Schema costruttivo cupola del Brunelleschi

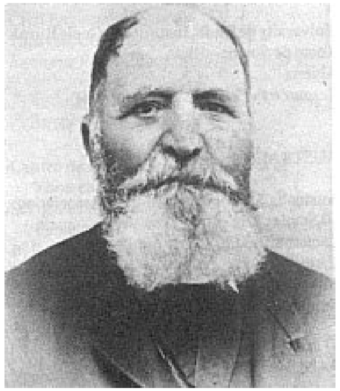


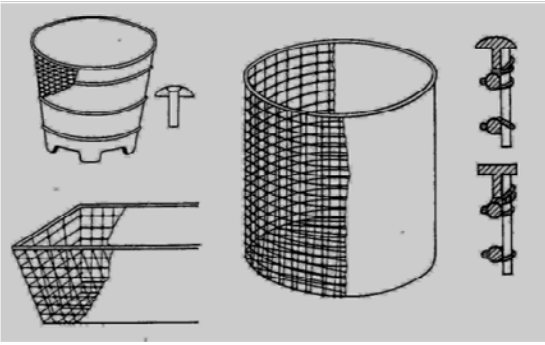
Schema costruttivo cupola del Brunelleschi



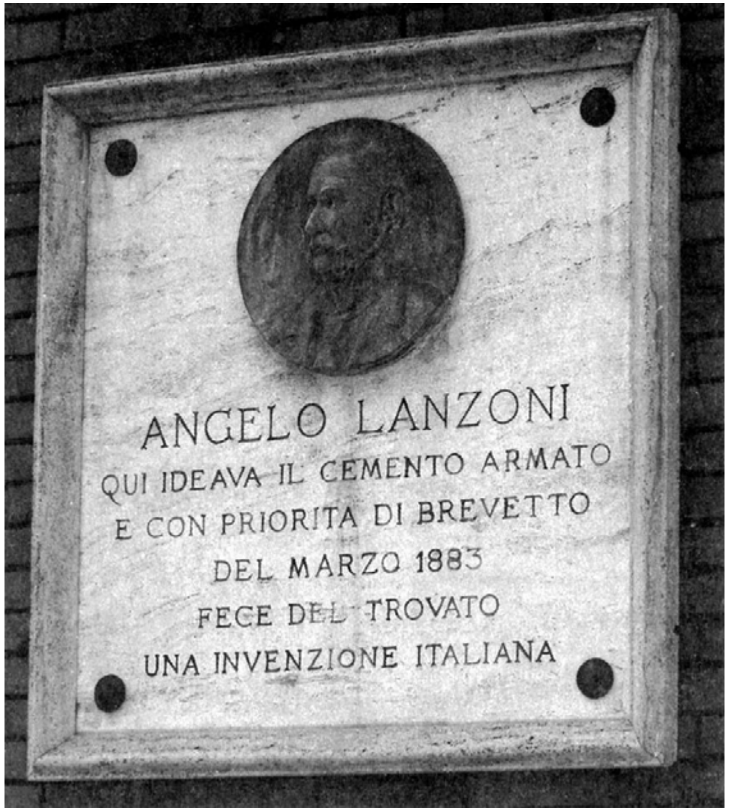
Calcestruzzo Armato

Joseph Monier
1867

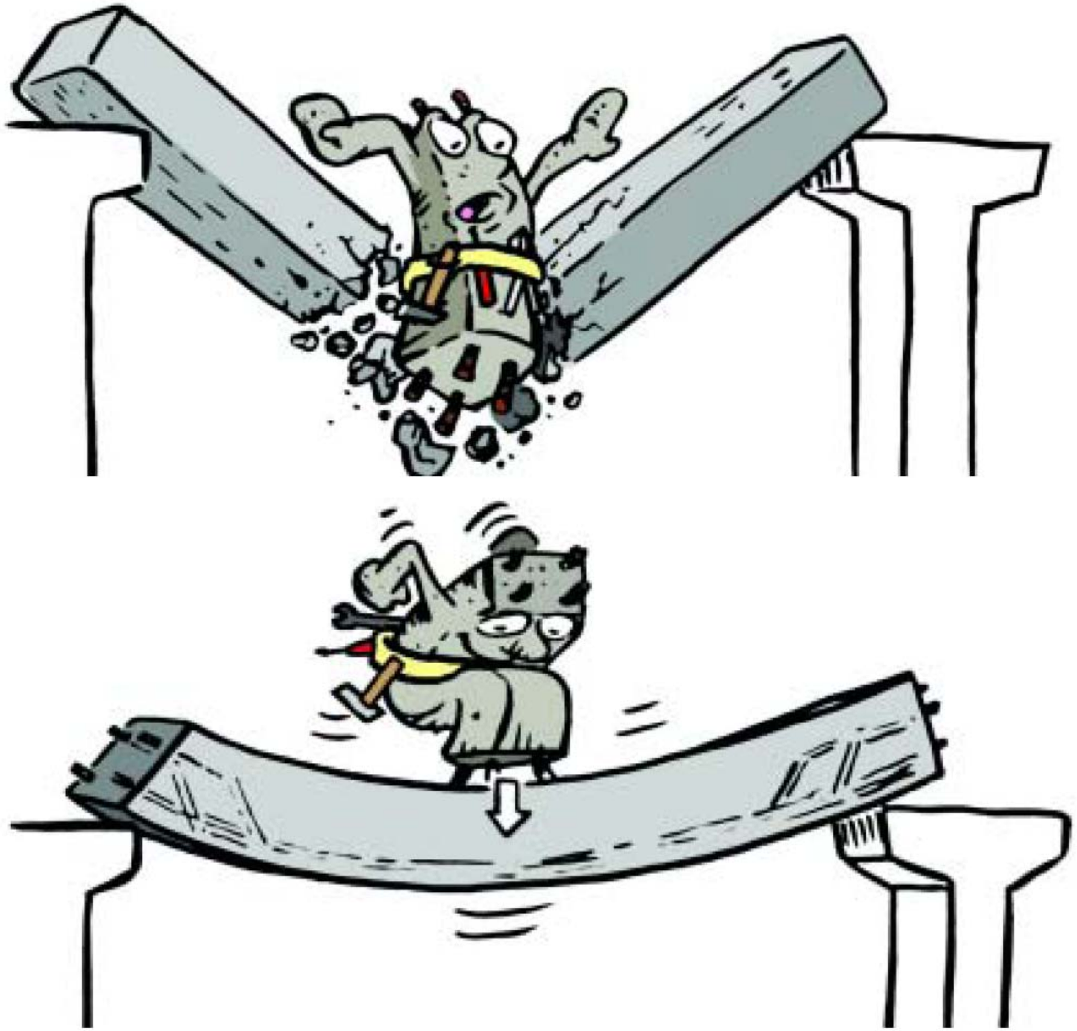




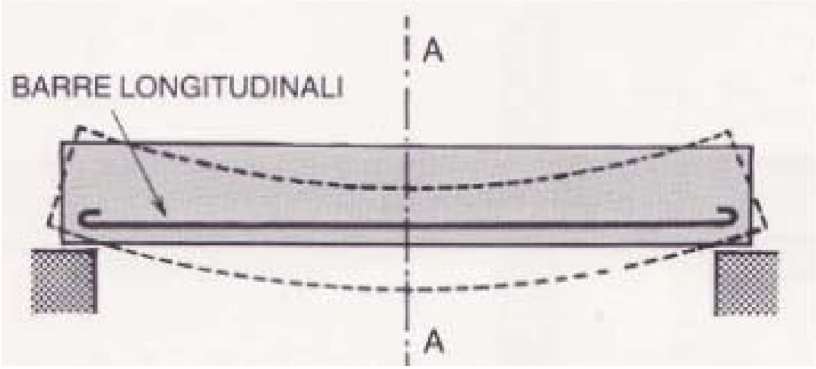
Ing. Angelo Lanzoni
1883 Pavia

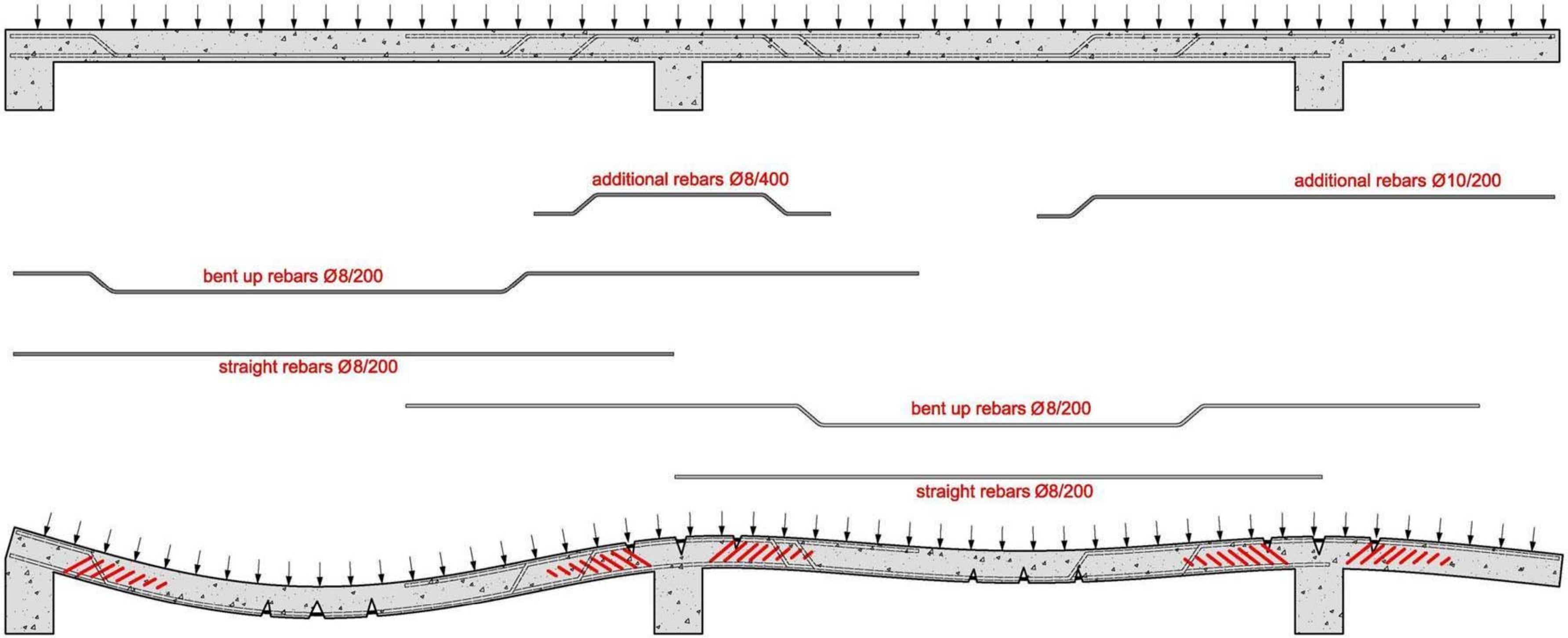

ANGELO LANZONI
QUI IDEAVA IL CEMENTO ARMATO
E CON PRIORITA DI BREVETTO
DEL MARZO 1883
FECE DEL TROVATO
UNA INVENZIONE ITALIANA

Cemento Armato: principio di funzionamento

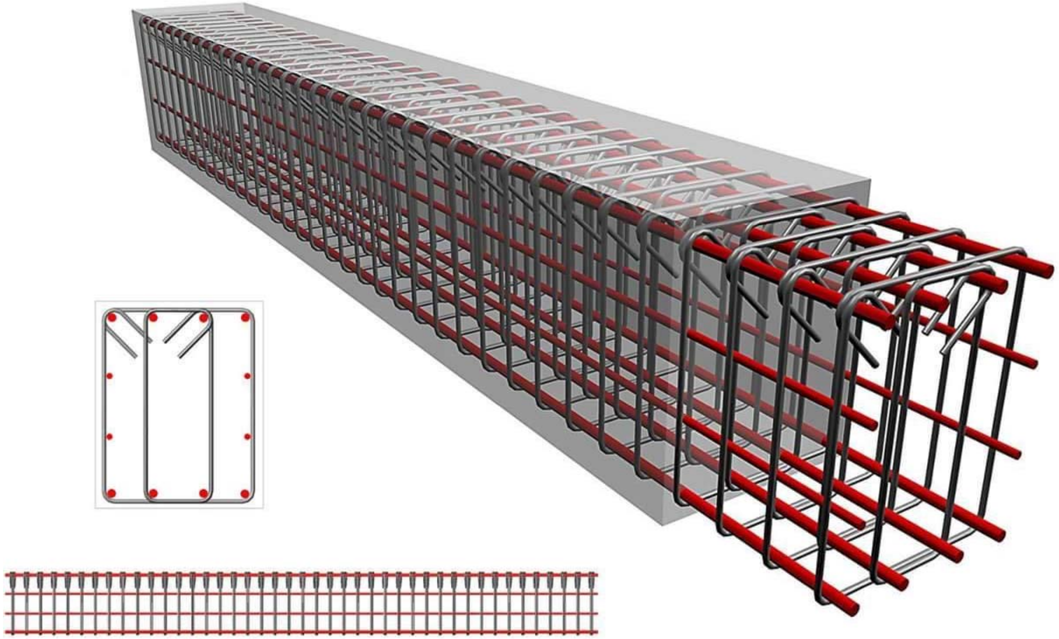


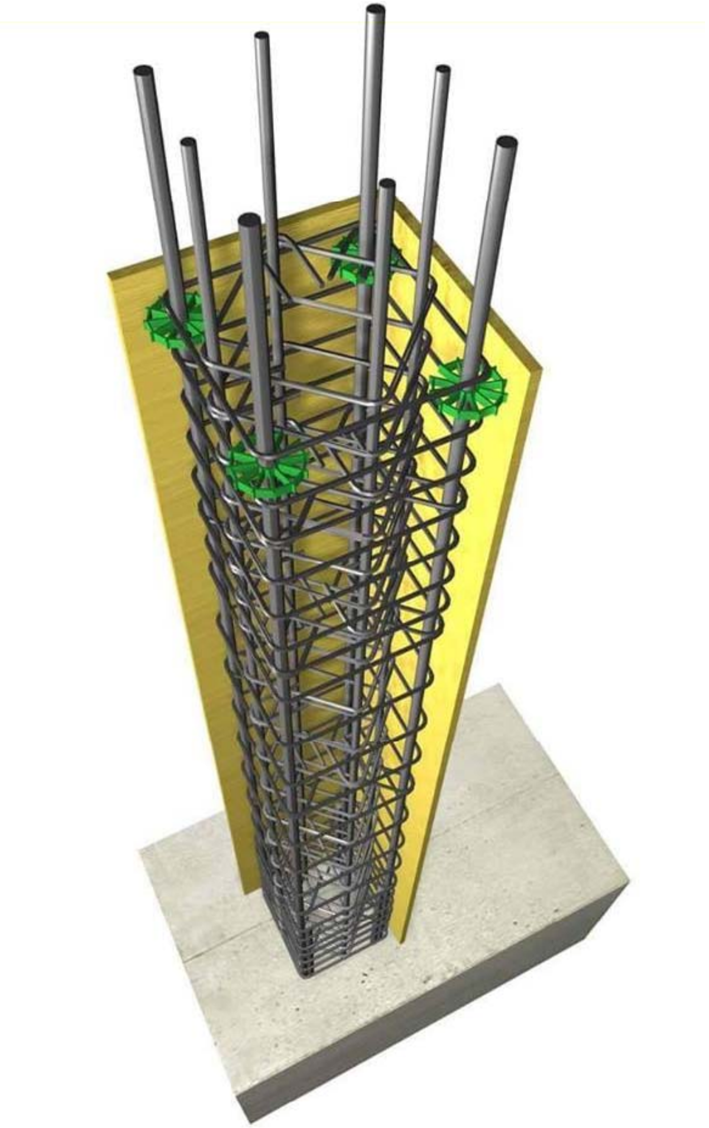
Cemento Armato principio di funzionamento


A
BARRE LONGITUDINALI
A

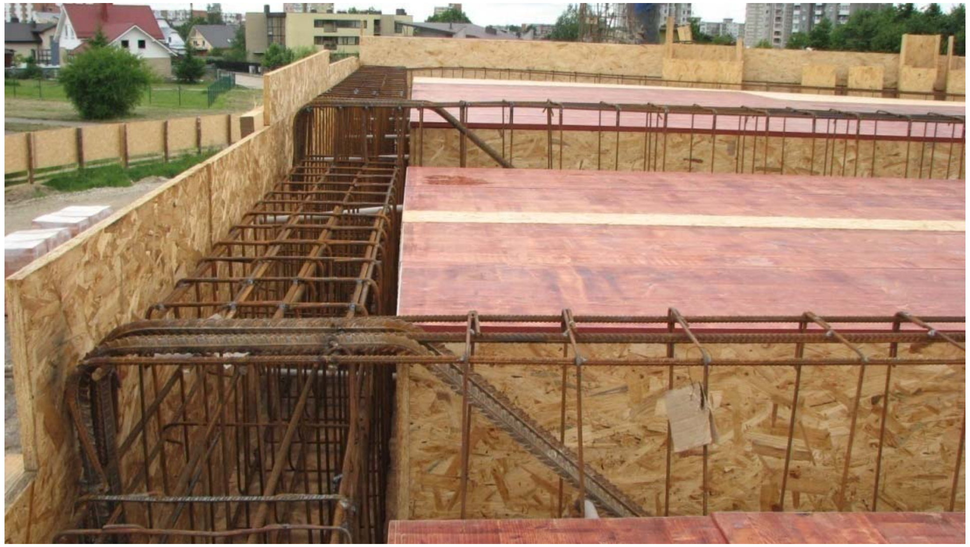

additional rebars Ø8/400
additional rebars Ø10/200
bent up rebars Ø8/200
straight rebars Ø8/200
bent up rebars Ø8/200
straight rebars Ø8/200

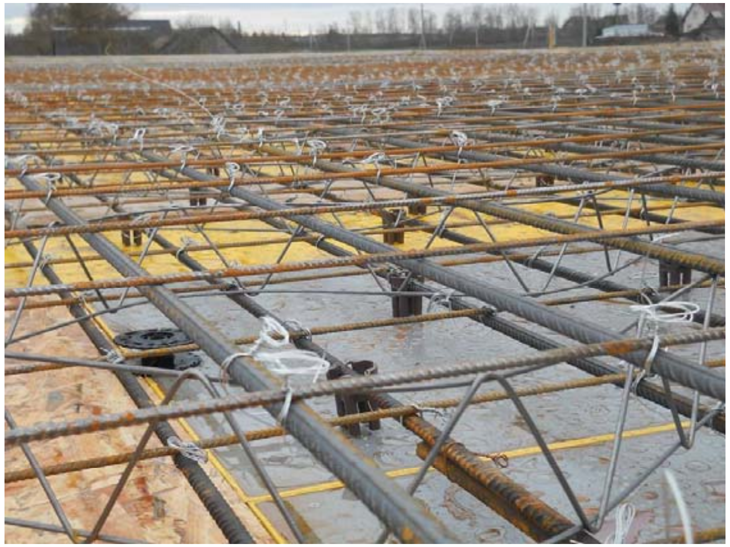
Armatura nel Calcestruzzo Armato

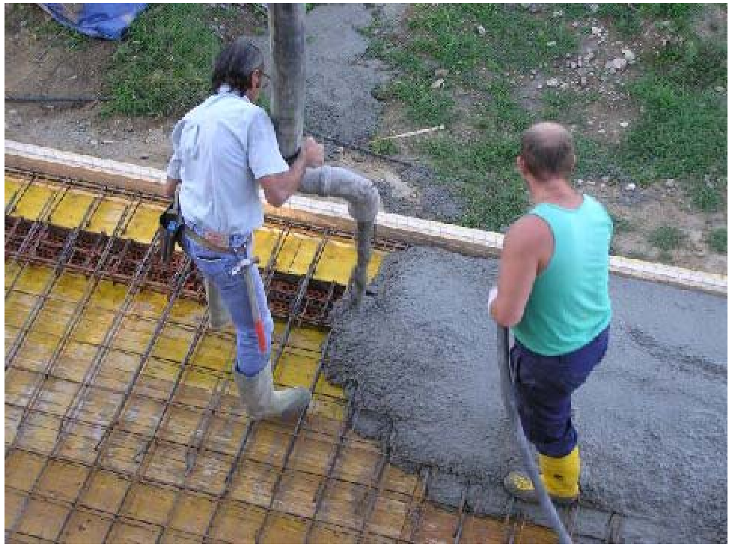




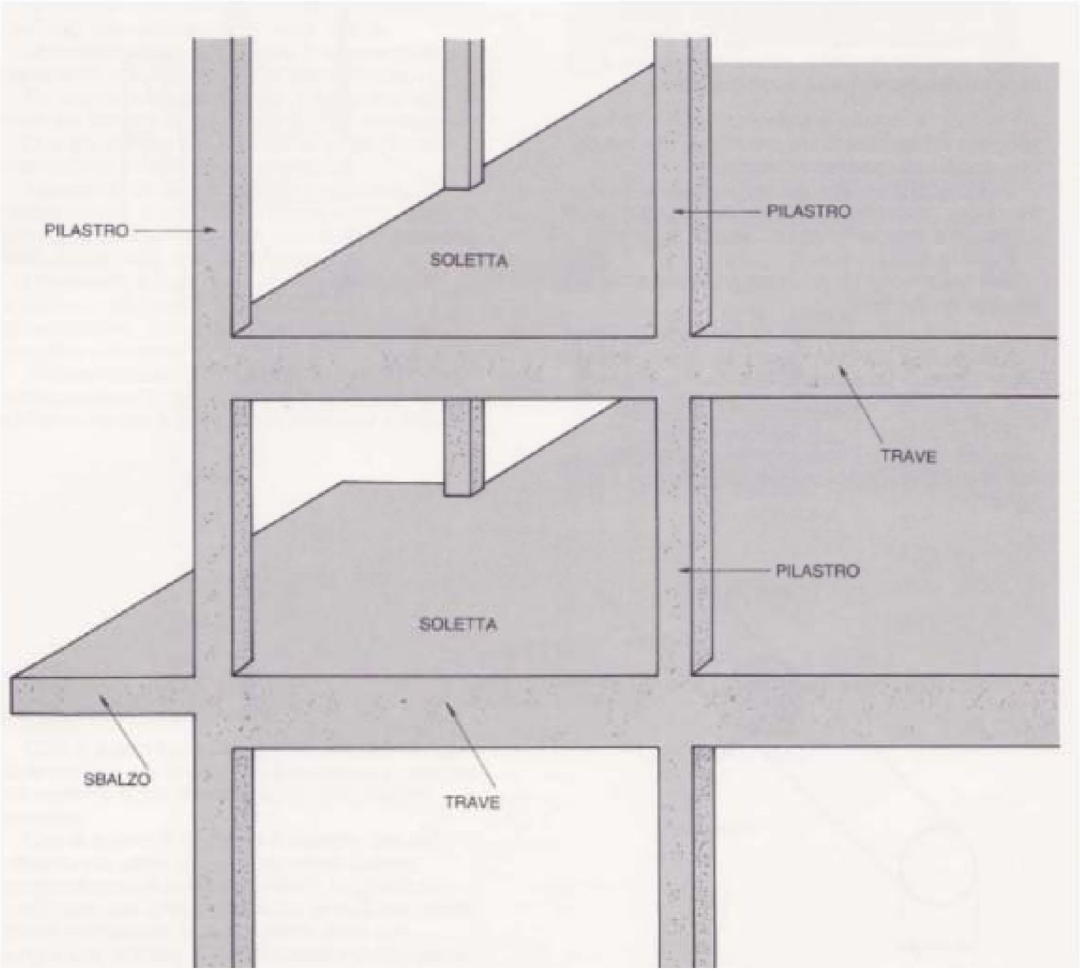
Calcestruzzo Armato







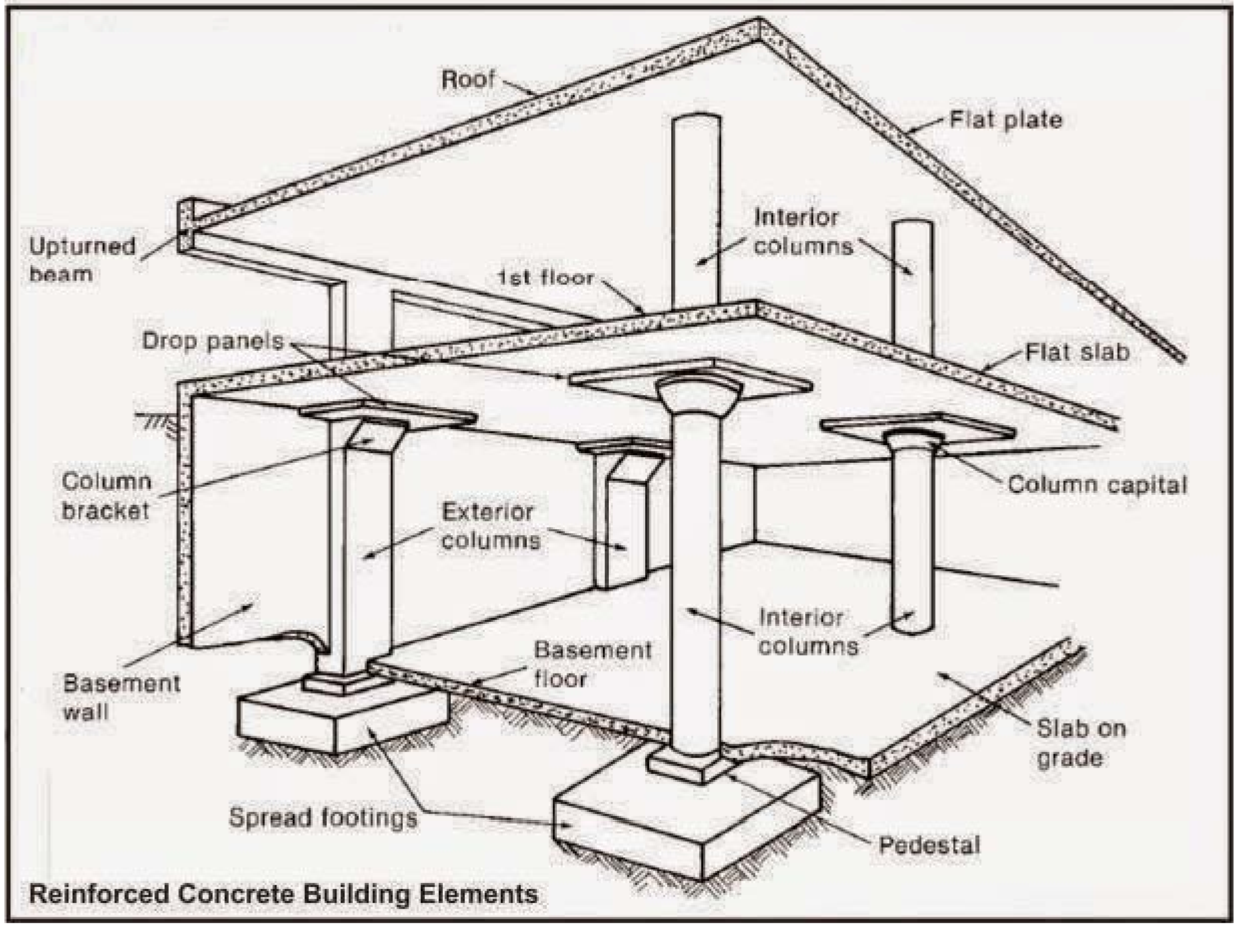
Elementi strutturali in calcestruzzo armato


PILASTRO
SOLETTA
PILASTRO
TRAVE
PILASTRO
SOLETTA
SBALZO
TRAVE

Le parti che normalmente formano una struttura in cemento armato sono:

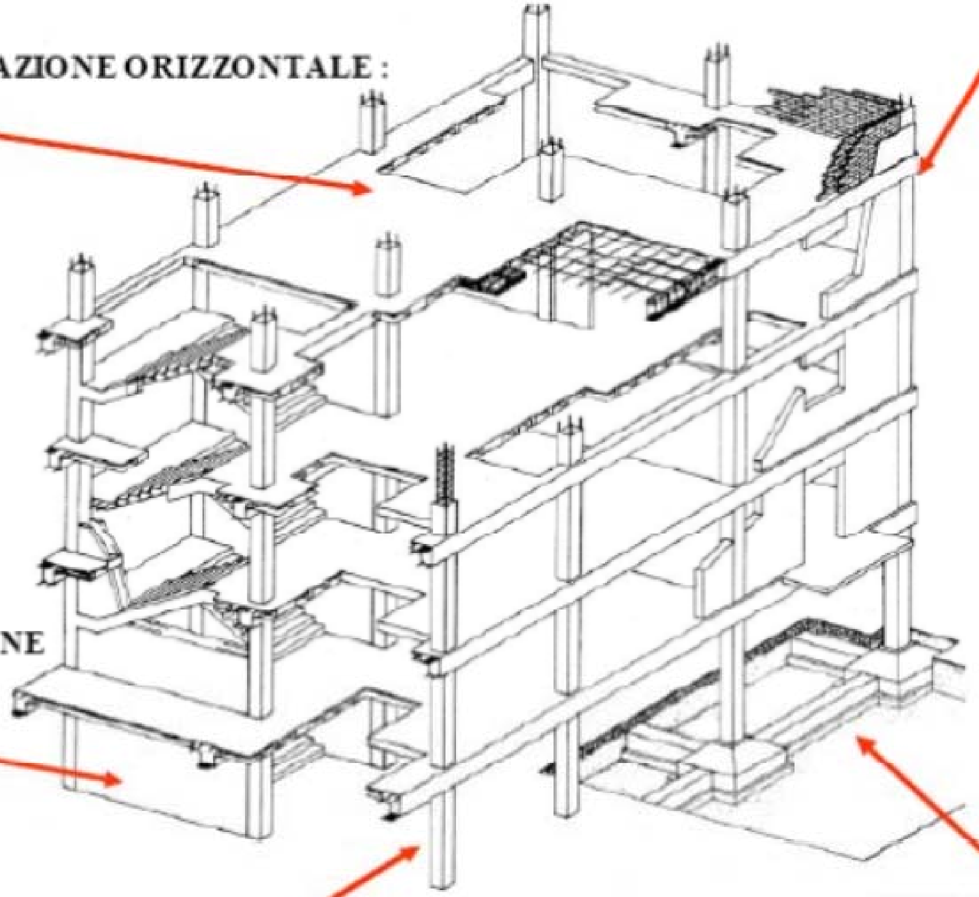
- gli elementi verticali, o pilastri
- gli elementi orizzontali, travi
- gli elementi orizzontali con grande sviluppo di superficie, solette
- gli elementi di collegamento verticale, scale

Strutture in cemento armato


Roof
Flat plate
Upturned beam
Interior columns
1st floor
Drop panels
Flat slab
Column capital
Column bracket
Exterior columns
Interior columns
Basement floor
Basement wall
Slab on grade
Spread footings
Pedestal

Reinforced Concrete Building Elements

Strutture in cemento armato



STRUTTURE IN ELEVAZIONE ORIZZONTALE :
TRAVI

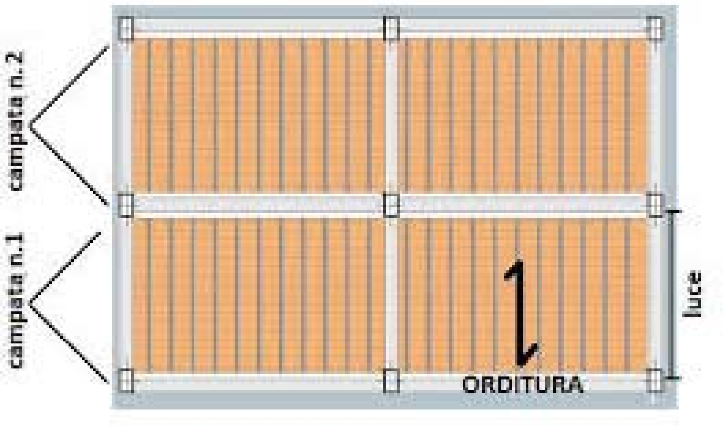
STRUTTURE IN ELEVAZIONE ORIZZONTALE :
SOLAI

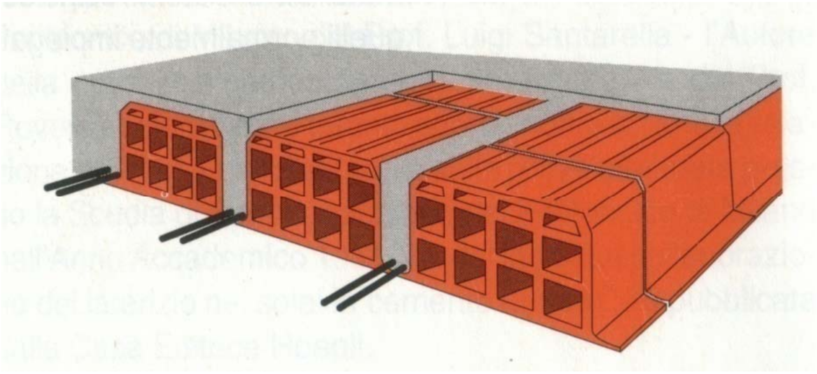
STRUTTURE IN ELEVAZIONE
VERTICALE :
SETTI

STRUTTURE DI
FONDAZIONE

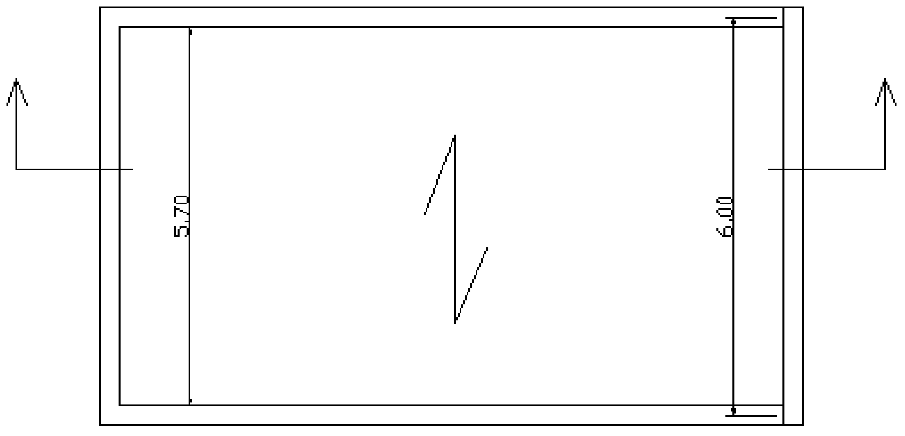
STRUTTURE IN ELEVAZIONE VERTICALE :
PILASTRI

Solai in calcestruzzo armato
Spessore 1/25 della Luce

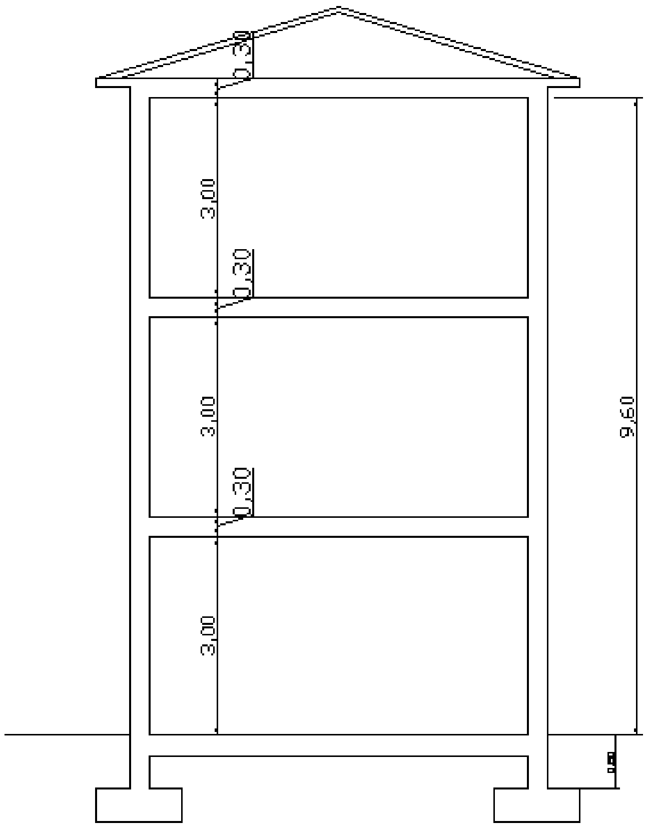

campata n.2
campata n.1
luce
ORDITURA



ESEMPIO: solaio



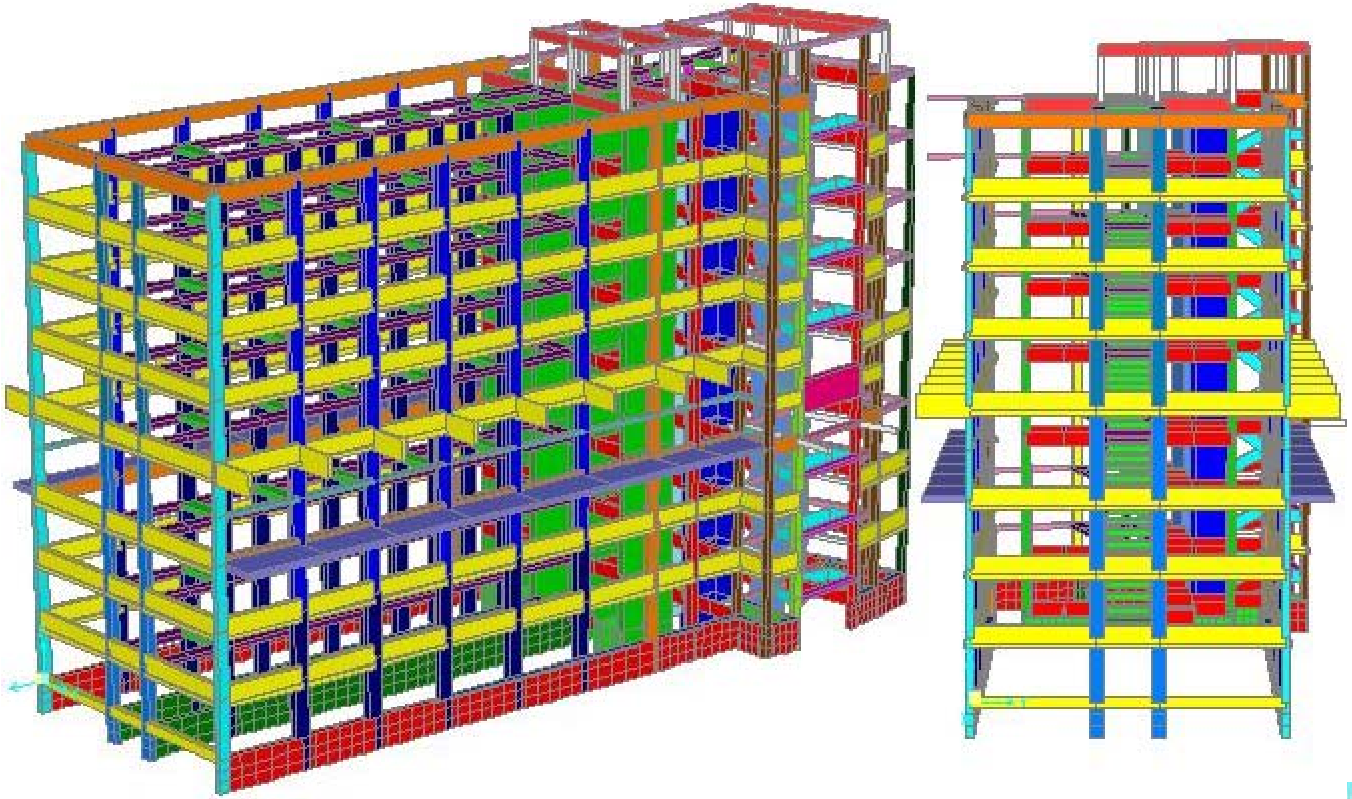
Pianta


0,30
3,00
0,30
3,00
0,30
3,00
9,60

Sezione

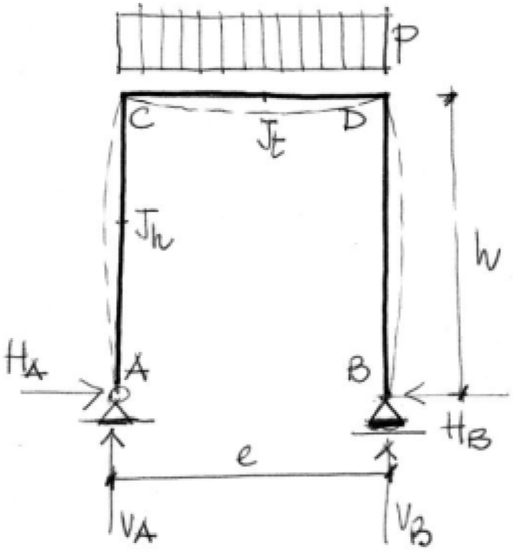
Carichi permanenti: totale gk ***600 daN/m2***
Carichi accidentali: totale qk ***200 daN/m2***

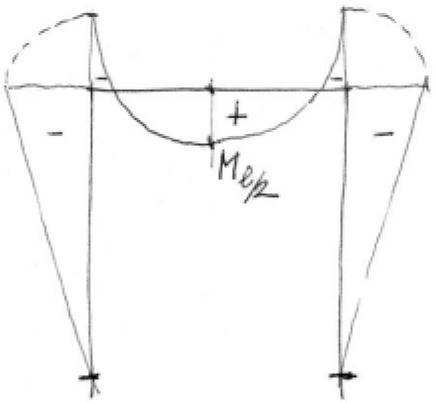
Struttura a Telaio



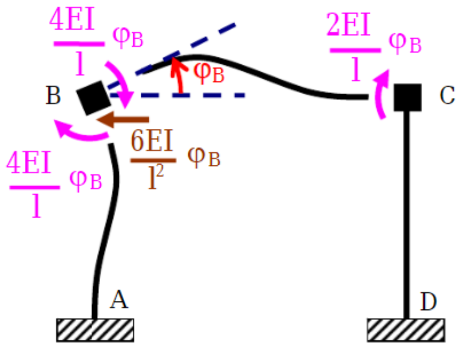
UnderstandConstruction.com

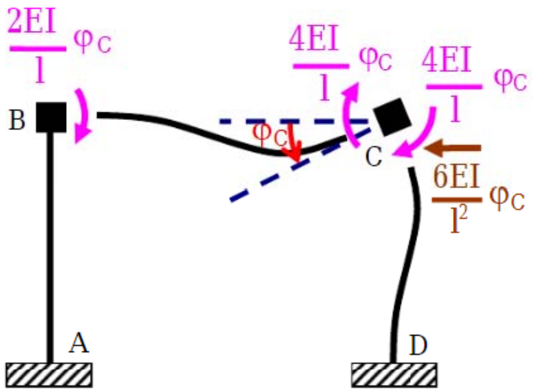
Risoluzione dei telai

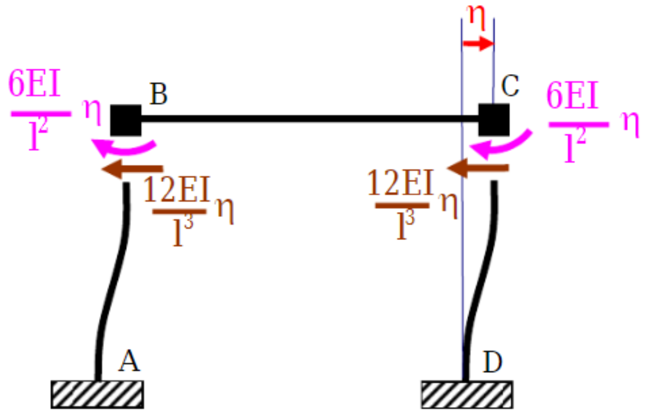

P
C
J_t
D
J_h
h
H_A
A
B
H_B
e
V_A
V_B



Metodo degli spostamenti


4EI/l φB
φB
2EI/l φB
B
C
4EI/l φB
6EI/l² φB
A
D


2EI/l φC
4EI/l φC
4EI/l φC
B
φC
C
6EI/l² φC
A
D

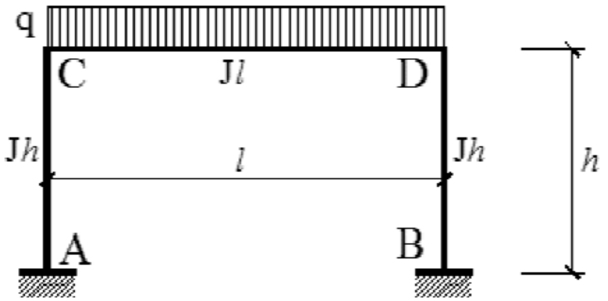

η
6EI/l² η
B
C
6EI/l² η
12EI/l³ η
12EI/l³ η
A
D

Metodo degli spostamenti

$$4\frac{EI}{l}\varphi_B + 4\frac{EI}{l}\varphi_B + 2\frac{EI}{l}\varphi_C + 6\frac{EI}{l^2}\eta - m = 0$$

$$2\frac{EI}{l}\varphi_B + 4\frac{EI}{l}\varphi_C + 4\frac{EI}{l}\varphi_C + 6\frac{EI}{l^2}\eta = 0$$

$$6\frac{EI}{l^2}\varphi_B + 6\frac{EI}{l^2}\varphi_C + 12\frac{EI}{l^3}\eta + 12\frac{EI}{l^3}\eta = 0$$

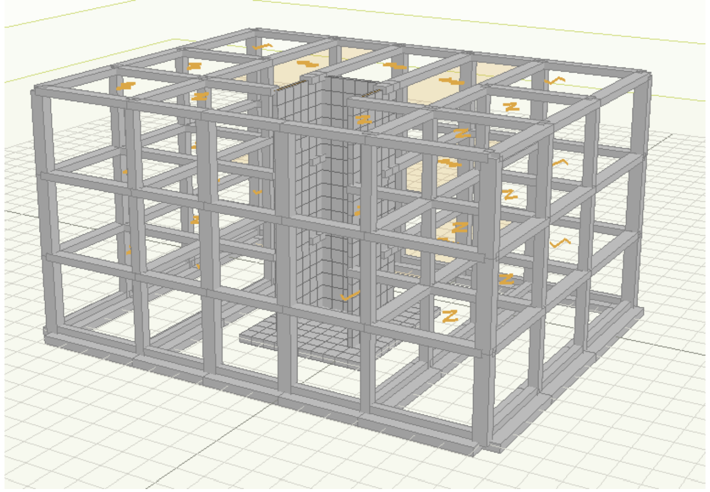


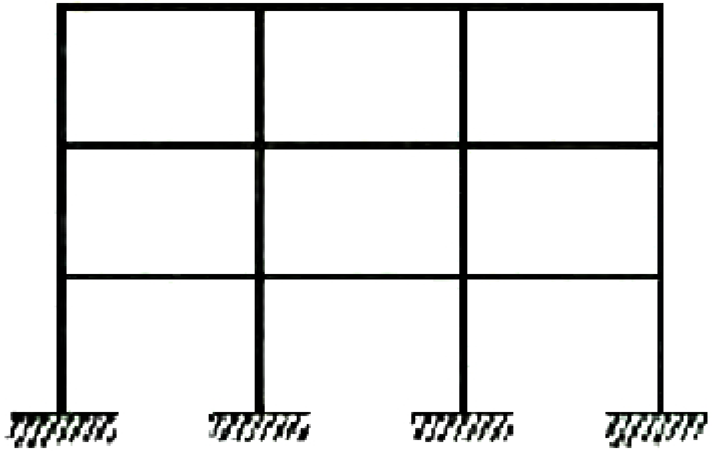
$$\varphi_B = +\frac{13}{84}\frac{ml}{EI}$$

$$\varphi_C = -\frac{1}{84}\frac{ml}{EI}$$

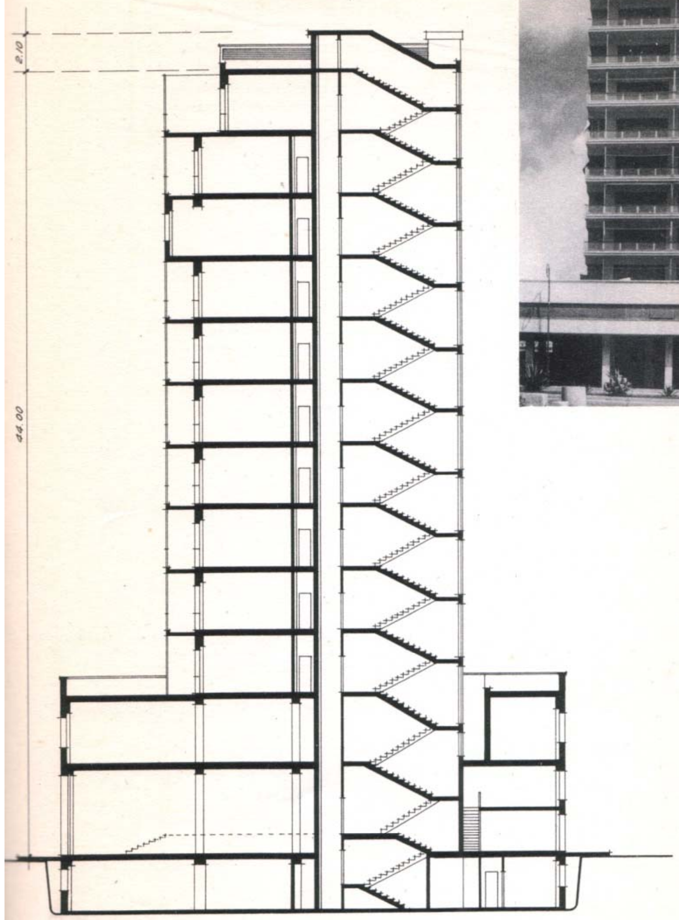
$$\eta = -\frac{3}{84}\frac{ml^2}{EI}$$

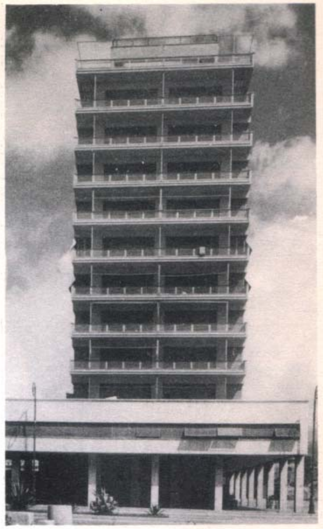
Strutture a telaio





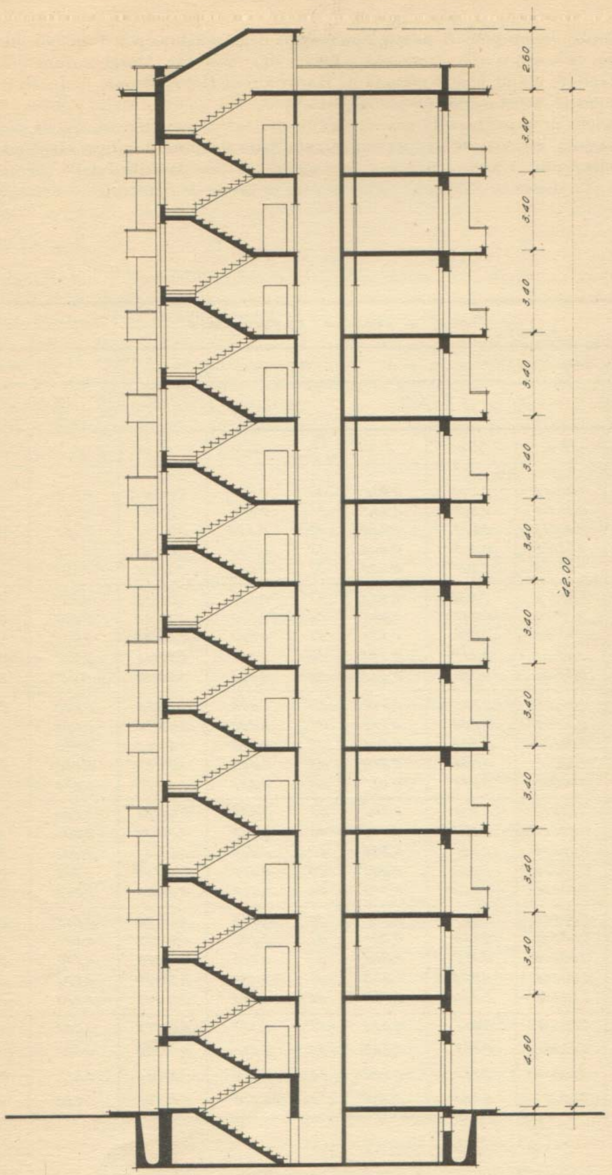
Esempio edificio in c.a.





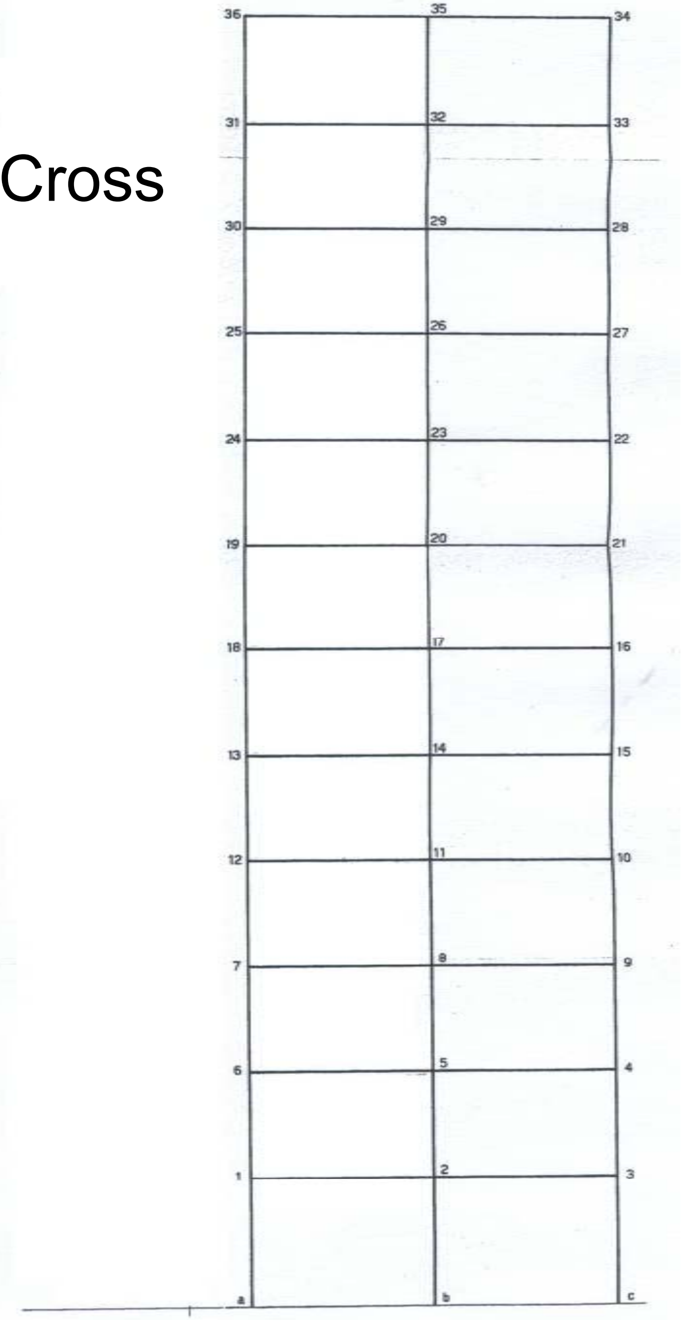
“il Grattacielo in Cemento Armato”
1953

Esempio edificio in c.a.

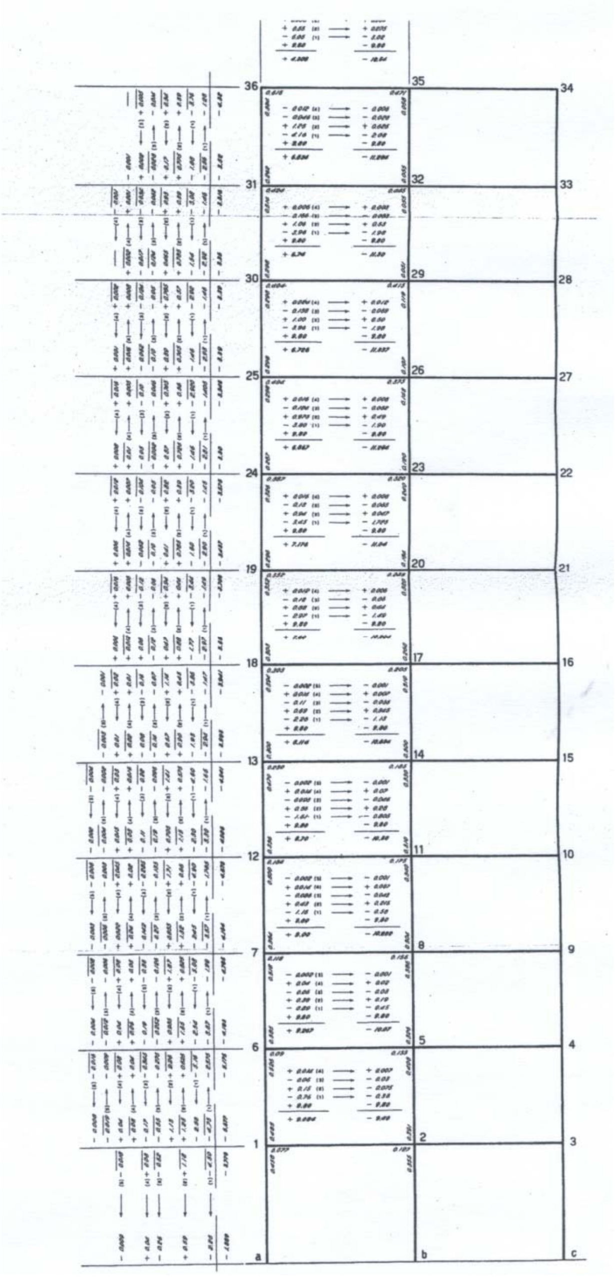

2.60
3.40
3.40
3.40
3.40
3.40
3.40
3.40
3.40
3.40
3.40
3.40
4.60
42.00

Metodo di Hardy Cross

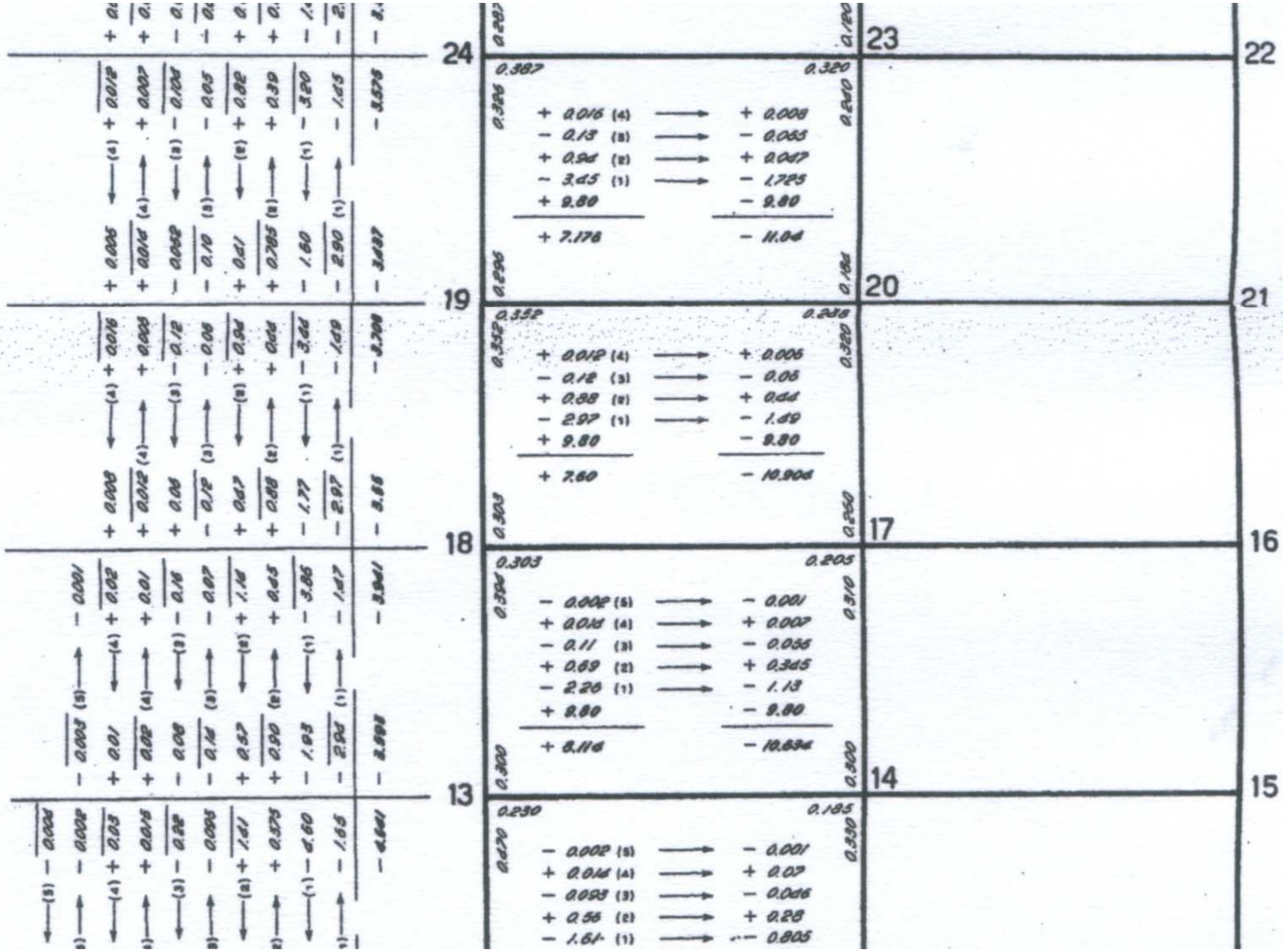
Metodo per successive
approssimazioni


36
35
34
31
32
33
30
29
28
25
26
27
24
23
22
19
20
21
18
17
16
13
14
15
12
11
10
7
8
9
6
5
4
1
2
3
a
b
c

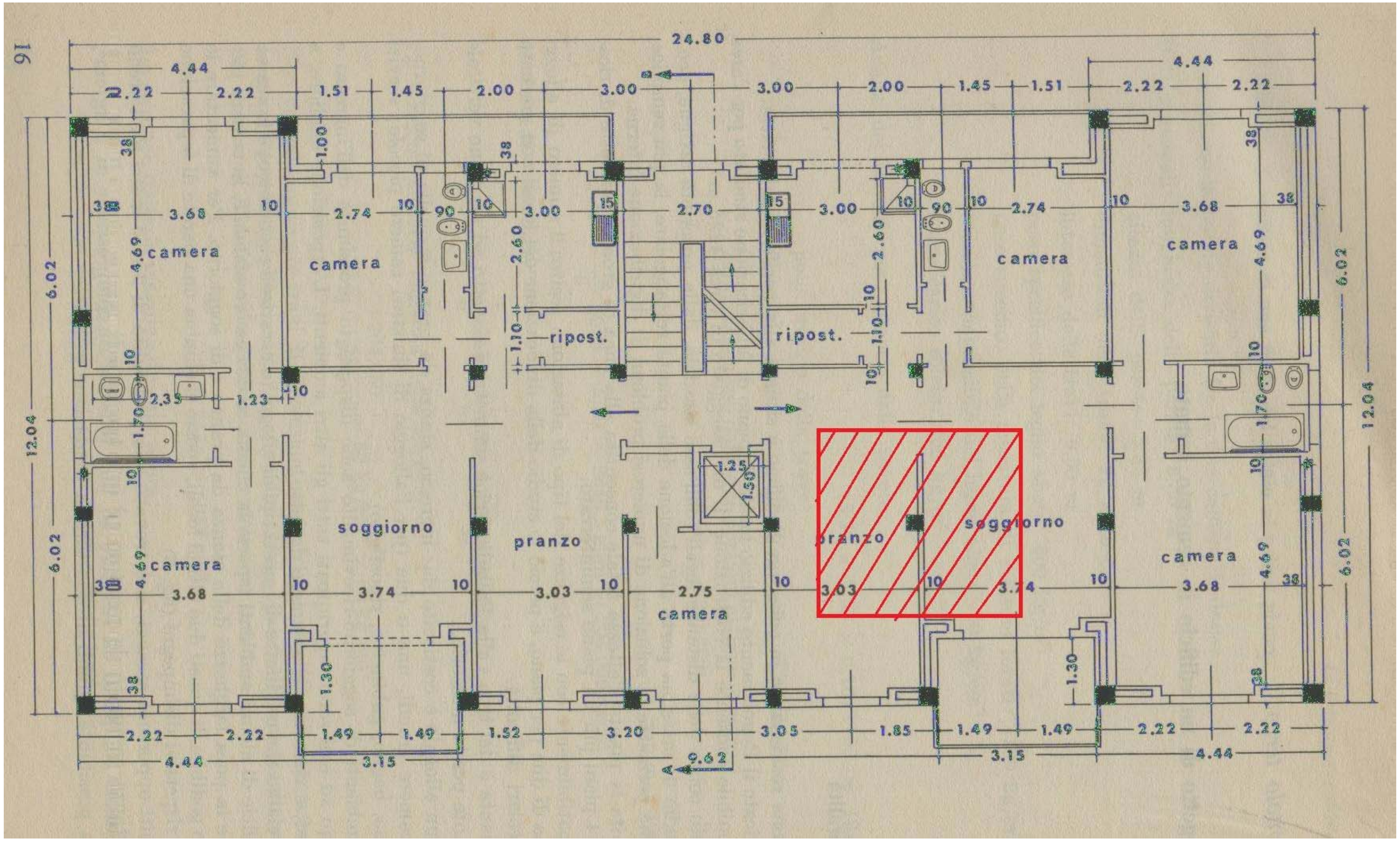
Metodo di Hardy - Cross


36
35
34
31
32
33
30
29
28
25
26
27
24
23
22
19
20
21
18
17
16
13
14
15
12
11
10
7
8
9
6
5
4
1
2
3
a
b
c

“il Grattacielo in Cemento Armato”
1953


24
23
22
19
20
21
18
17
16
13
14
15
0.387
0.320
0.396
0.240
+ 0.016 (4) → + 0.008
− 0.13 (3) → − 0.065
+ 0.94 (2) → + 0.047
− 3.45 (1) → − 1.725
+ 9.80
− 9.80
+ 7.176
− 11.04
0.296
0.184
0.352
0.236
0.352
0.320
+ 0.012 (4) → + 0.006
− 0.12 (3) → − 0.06
+ 0.88 (2) → + 0.44
− 2.97 (1) → − 1.49
+ 9.80
− 9.80
+ 7.60
− 10.904
0.303
0.250
0.303
0.205
0.304
0.310
− 0.002 (5) → − 0.001
+ 0.014 (4) → + 0.007
− 0.11 (3) → − 0.055
+ 0.69 (2) → + 0.345
− 2.26 (1) → − 1.13
+ 9.80
− 9.80
+ 8.114
− 10.634
0.300
0.300
0.230
0.185
0.470
0.330
− 0.002 (5) → − 0.001
+ 0.014 (4) → + 0.07
− 0.093 (3) → − 0.046
+ 0.56 (2) → + 0.28
− 1.61 (1) → − 0.805
− 3.576
− 3.437
− 3.708
− 3.56
− 3.941
− 3.598
− 4.641

Esempio area di carico pilastri


16
24.80
4.44
2.22
2.22
1.51
1.45
2.00
3.00
3.00
2.00
1.45
1.51
2.22
2.22
4.44
38
1.00
38
38
3.68
10
2.74
10
90
10
3.00
15
2.70
15
3.00
10
90
10
2.74
10
3.68
38
camera
4.69
camera
2.60
2.60
camera
camera
4.69
6.02
6.02
ripost.
1.10
ripost.
1.10
10
10
10
10
2.35
1.23
10
1.70
1.70
12.04
12.04
10
1.25
1.50
10
soggiorno
pranzo
pranzo
soggiorno
camera
camera
4.69
4.69
6.02
6.02
38
3.68
10
3.74
10
3.03
10
2.75
10
3.03
10
3.74
10
3.68
38
camera
1.30
1.30
38
38
2.22
2.22
1.49
1.49
1.52
3.20
3.05
1.85
1.49
1.49
2.22
2.22
4.44
3.15
9.62
3.15
4.44

CALCOLO DEI PILASTRI.

Numerati i pilastri dal n° 1 al n° 18 (essendo simmetrico lo schema), determiniamo per ognuno la superficie portante. Tenuto conto che il sovraccarico non può essere contemporaneo e completo in tutti i piani, agli effetti del calcolo dei pilastri, si può considerare che il carico medio dovuto ad un solaio sia Kg/mq. 600 in luogo dei 700 Kg/mq. già riscontrato al Pg. 22 (Normalmente si ammette una diminuzione progressiva del sovraccarico a partire dal solaio di copertura, fino al 50% del valore al piano più basso).

Pilastri 1 e 3.

Superficie di solaio sopportata: ml. 2,50 × 3,60 = mq. 9,00
Carico per ogni piano:
Peso trasmesso dal solaio, completo di tramezze ecc.:
mq. 9,00 × kg/mq. 600 = ... kg. 5.400.—
Muratura a camera d'aria:
ml. 5,20 × 3,00 × 350 kg/mq. × 4/5 (per i vuoti delle finestre) = » 4.360.— 350
Peso medio del pilastro e delle travi:
4,80 × 0,20 × 0,40 × 2.500 kg/mc = ... » 960.— 250
3,40 × 0,50 × 0,40 × 2.500 kg/mc = ... » 1.680.— 1600
Totale kg. 12.400.—

1° Ordine:
Assumiamo:

$$P = 12 \times 12400 = 148800 \text{ Kg.}$$

$$F_c = \text{cm. } 80 \times \text{cm. } 40 = 3200 \text{ cmq.} \qquad F_f = 22{,}72 \text{ cmq.} (4 \phi 18 + 4 \phi 20)$$

$$F_f / F_c = 0{,}72\% \qquad F_i = 3200 + 227 = 3427 \text{ cmq.}$$

$$\sigma_c = \frac{148800}{3427} = \sim 45{,}5 \text{ Kg/cmq.}$$

2° Ordine:
Assumiamo:

$$P = 11 \times 12400 = 156400 \text{ Kg.}$$

$$F_c = \text{cm. } 70 \times \text{cm. } 40 = 2800 \text{ cmq.} \qquad F_f = 20{,}32 \text{ cmq.} (8 \phi 18)$$

$$F_i = 2800 + 203 = 3003 \text{ cmq.}$$

$$\sigma_c = \frac{136400}{3003} = \sim 45{,}40 \text{ Kg/cmq.}$$

3° Ordine:
Assumiamo:

$$P = 10 \times 12400 = 124000 \text{ Kg.}$$

$$F_c = \text{cm. } 60 \times \text{cm. } 40 = 2400 \text{ cmq.} \qquad F_f = 18{,}20 \text{ cmq.} (4 \phi 16 + 4 \phi 18)$$

$$F_i = 2400 + 182 = 2582 \text{ cmq.}$$

$$\sigma_c = \frac{124000}{2582} = \sim 47{,}90 \text{ Kg/cmq.}$$

4° Ordine:
Assumiamo:

$$P = 9 \times 12400 = 111600 \text{ Kg.}$$

$$F_c = \text{cm. } 55 \times \text{cm. } 40 = 2200 \text{ cmq.} \qquad F_f = 16{,}08 \text{ cmq.} (4 \phi 16)$$

$$F_i = 2200 + 160 = 2360 \text{ cmq.}$$

$$\sigma_c = \frac{111600}{2360} = \sim 47{,}30 \text{ Kg/cmq.}$$

5° Ordine:
Assumiamo:

$$P = 8 \times 12400 = 99200 \text{ Kg.}$$

$$F_c = \text{cm. } 50 \times \text{cm. } 40 = 2000 \text{ cmq.} \qquad F_f = 14{,}20 \text{ cmq.} (4 \phi 14 + 4 \phi 16)$$

$$F_i = 2000 + 140 = 2140 \text{ cmq.}$$

$$\sigma_c = \frac{99200}{2140} = \sim 47{,}00 \text{ Kg/cmq.}$$

6° Ordine:
Assumiamo:

$$P = 7 \times 12400 = 86800 \text{ Kg.}$$

$$F_c = \text{cm. } 45 \times \text{cm. } 40 = 1800 \text{ cmq.} \qquad F_f = 12{,}32 \text{ cmq.} (8 \phi 14)$$

$$F_i = 1800 + 123 = 1923 \text{ cmq.}$$

$$\sigma_c = \frac{86800}{1923} = \sim 45{,}00 \text{ Kg/cmq.}$$

7° Ordine: $P = 6 \times 12400 = 74400$ Kg.

Assumiamo: $F_c = \text{cm.}\,40 \times \text{cm.}\,40 = 1600$ cmq. $F_f = 10{,}68$ cmq. $(4\,\phi\,14 + 4\,\phi\,12)$

$F_i = 1600 + 107 = 1707$ cmq.

$$\sigma_c = \frac{74400}{1707} = \sim 43{,}4 \text{ Kg/cmq.}$$

8° Ordine: $P = 5 \times 12400 = 62000$ Kg.

Assumiamo: $F_c = \text{cm.}\,35 \times \text{cm.}\,35 = 1225$ cmq. $F_f = 10{,}68$ cmq.

$F_i = 1225 + 107 = 1332$ cmq.

$$\sigma_c = \frac{62000}{1332} = \sim 46{,}5 \text{ Kg/cmq.}$$

9° Ordine: $P = 4 \times 12400 = 49600$ Kg.

Assumiamo: $F_c = \text{cm.}\,30 \times \text{cm.}\,35 = 1050$ cmq. $F_f = 8{,}44$ cmq. $(4\,\phi\,14 + 2\,\phi\,12)$

$F_i = 1050 + 84 = 1134$ cmq.

$$\sigma_c = \frac{49600}{1134} = \sim 43{,}50 \text{ Kg/cmq.}$$

10° Ordine: $P = 3 \times 12400 = 37200$ Kg.

Assumiamo: $F_c = \text{cm.}\,30 \times \text{cm.}\,30 = 900$ cmq. $F_f = 6{,}16$ cmq. $(4\,\phi\,14)$

$F_i = 900 + 61 = 961$ cmq.

$$\sigma_c = \frac{37200}{961} = \sim 38{,}90 \text{ Kg./cmq.}$$

11° Ordine: $P = 2 \times 12400 = 24800$ Kg.

Assumiamo: $F_c = \text{cm.}\,30 \times \text{cm.}\,30 = 900$ cmq. $F_f = 4{,}52$ cmq. $(4\,\phi\,12)$

$F_i = 900 + 45 = 945$ cmq.

$$\sigma_c = \frac{24800}{945} = \sim 26{,}20 \text{ Kg/cmq.}$$

12° Ordine: $P = 12400$ Kg.

Assumiamo: $F_c = \text{cm.}\,25 \times \text{cm.}\,25 = 625$ cmq. $F_f = 3{,}14$ cmq. $(4\,\phi\,10)$

$F_i = 625 + 31 = 656$ cmq.

$$\sigma_c = \frac{12400}{656} = \sim 19{,}00 \text{ Kg/cmq.}$$

Pilastro 2.

Area sopportata: ml. 2,50 × 6,40 = mq. 16,00

Carico per ogni piano:

Peso trasmesso dal solaio completo di tramezze ecc.:
mq. 16,00 × kg/mq. 600 = ... kg. 9.600.—
Muratura a camera d'aria:
m. 5,90 × 3,00 × kg/mq. 350 × 4/5 = ... » 4.950.—
Peso medio del pilastro e delle travi:
5,90 × 0,20 × 0,40 × 2.500 = ... » 1.180.—
0,40 × 0,50 × 3,40 × 2.500 kg/mc. = ... » 1.700.—
Totale kg. 17.430.—

1° Ordine: $P = 12 \times 17400 = 208800$ Kg.

Assumiamo: $F_c = \text{cm.}\,60 \times \text{cm.}\,75 = 4500$ cmq. $F_f = 30{,}40$ $(8\,\phi\,22)$

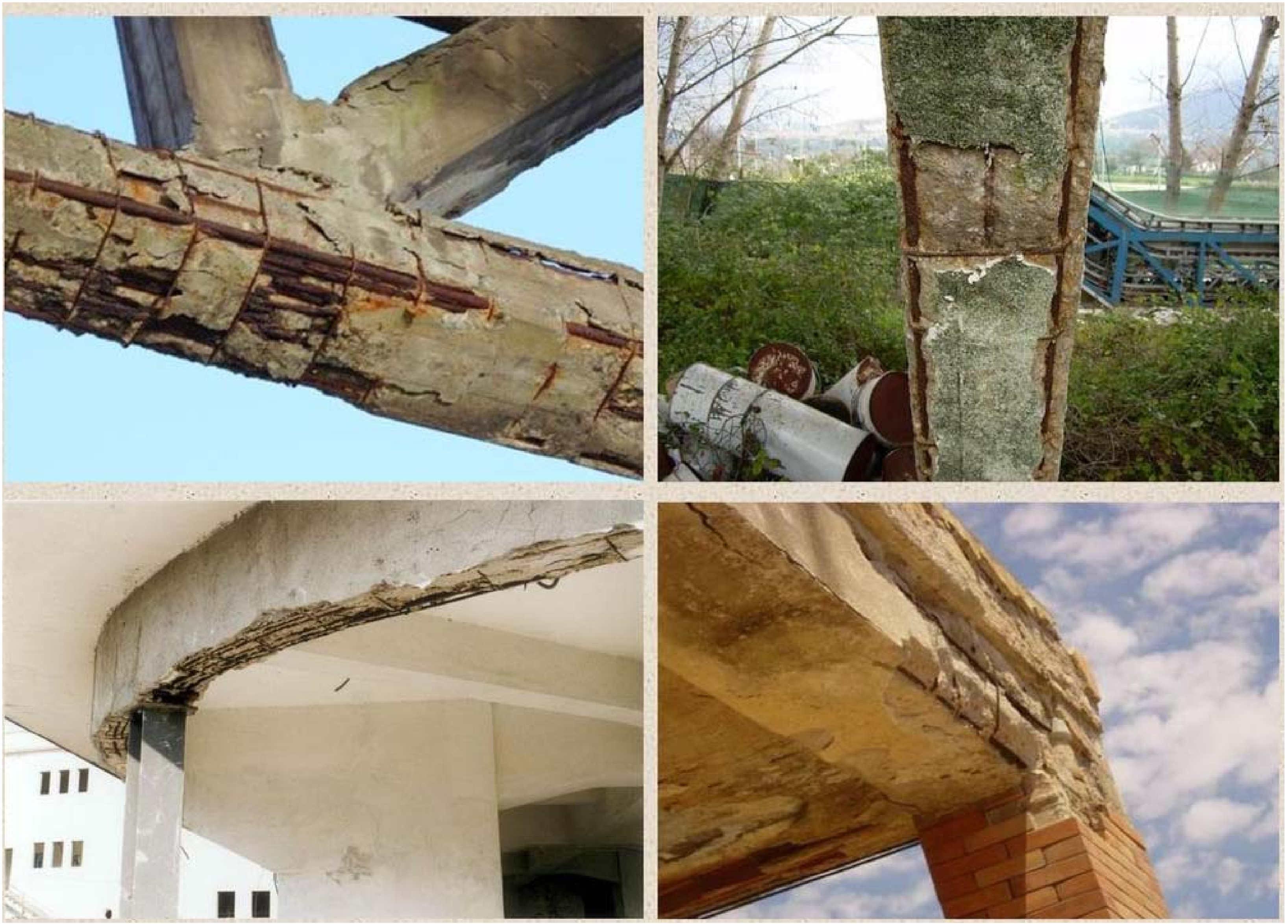
$F_i = 4804$ cmq. $\sigma_c = \dfrac{208800}{4804} = \sim 43{,}40$ Kg/cmq.

2° Ordine: $P = 11 \times 17400 = 191400$ Kg.

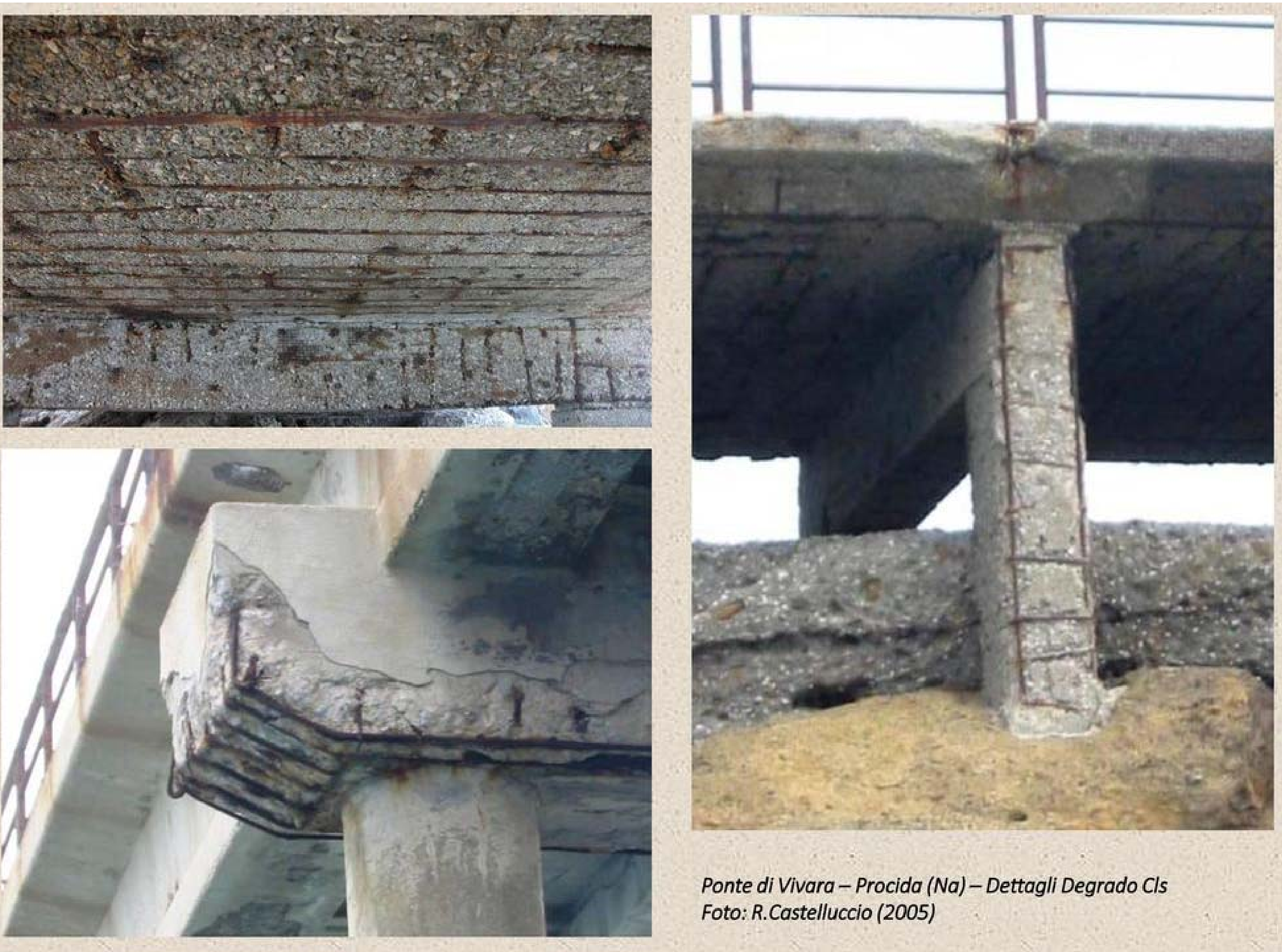
Assumiamo: $F_c = \text{cm.}\,60 \times \text{cm.}\,70 = 4200$ cmq. $F_f = 30{,}40$ cmq.

$F_i = 4504$ cmq. $\sigma_c = \dfrac{191400}{4504} = \sim 42{,}50$ Kg/cmq.

Degrado del cemento armato

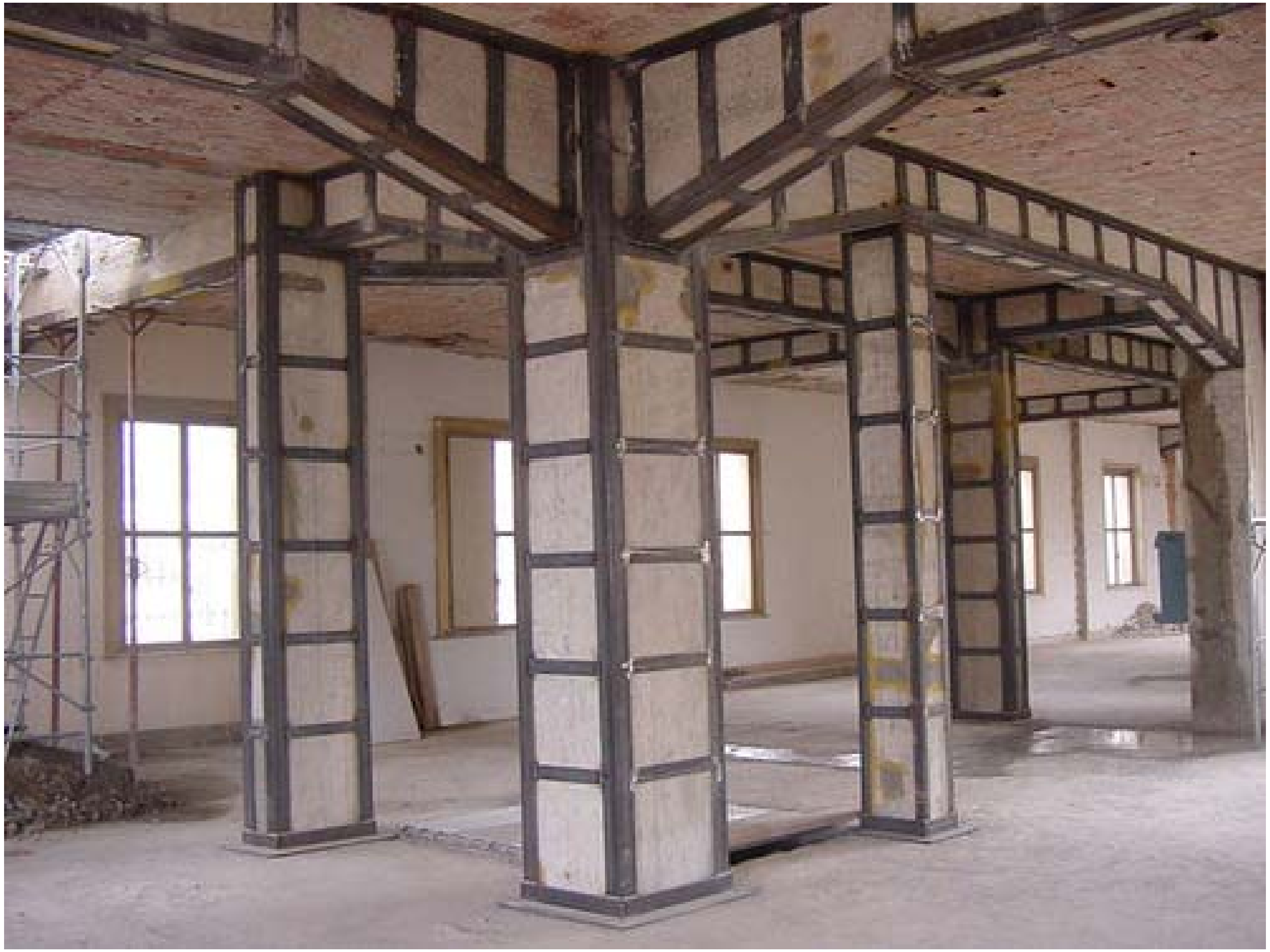


Degrado del cemento armato

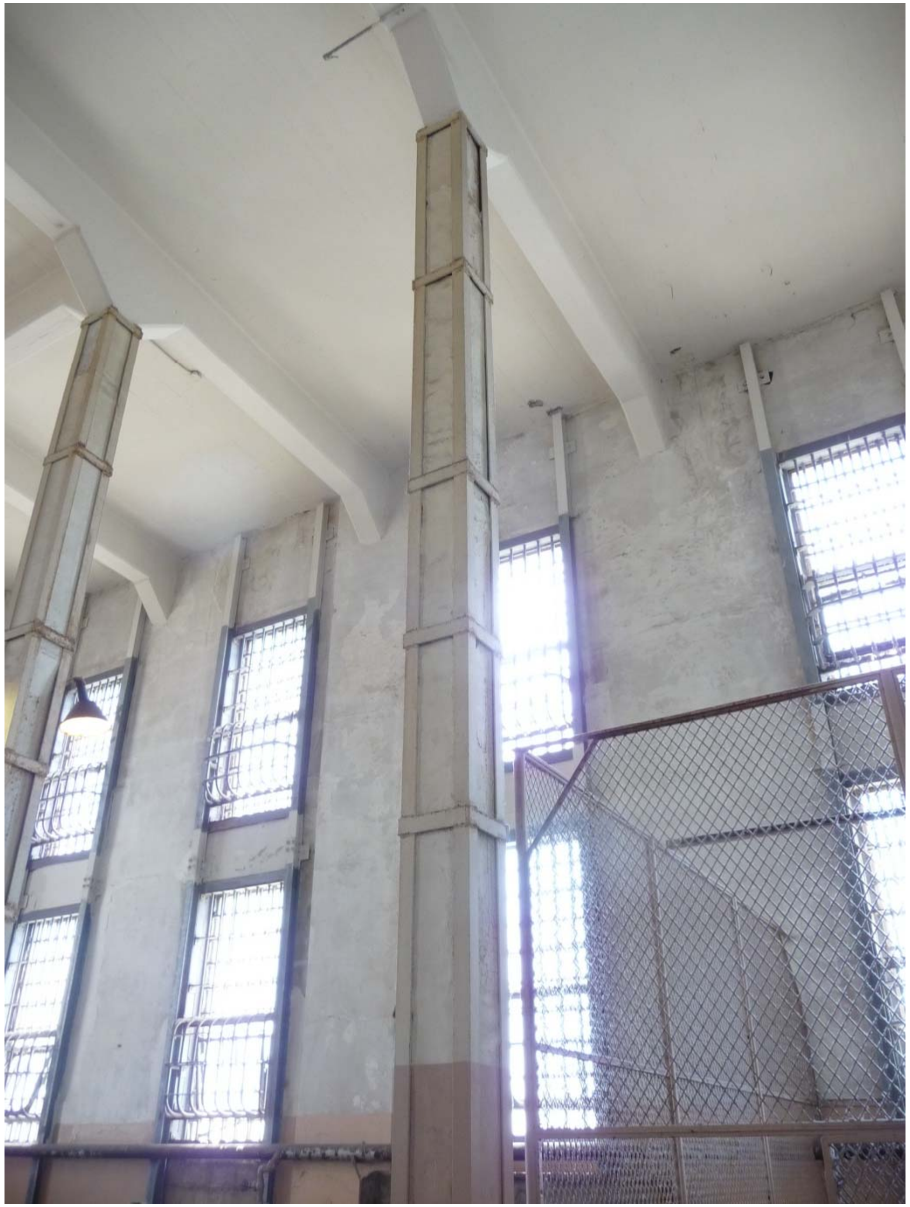


Ponte di Vivara – Procida (Na) – Dettagli Degrado Cls
Foto: R.Castelluccio (2005)

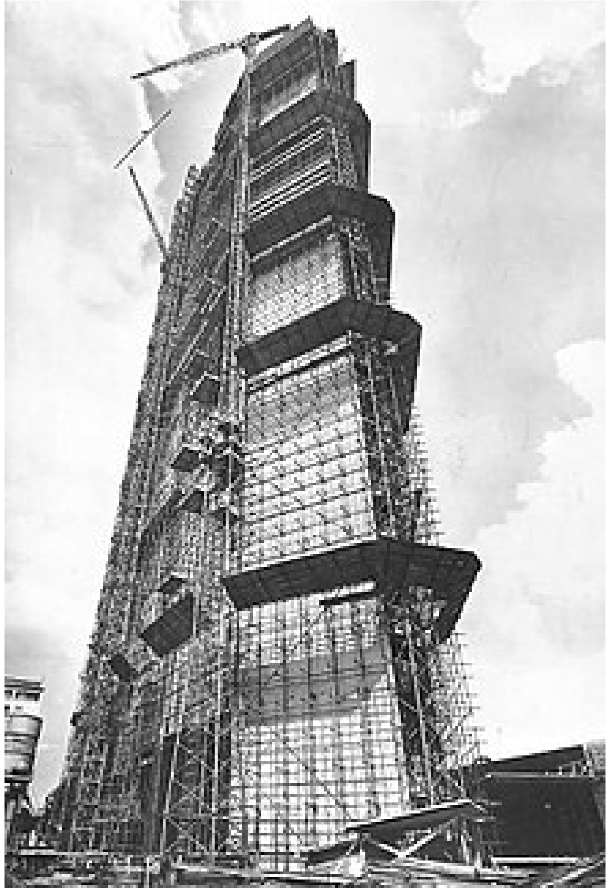
Rinforzo strutture in c.a.



Rinforzo strutture in c.a.



Grattacielo Pirelli
Gio’ Ponti – Pierluigi Nervi
1961
Altezza 127 metri

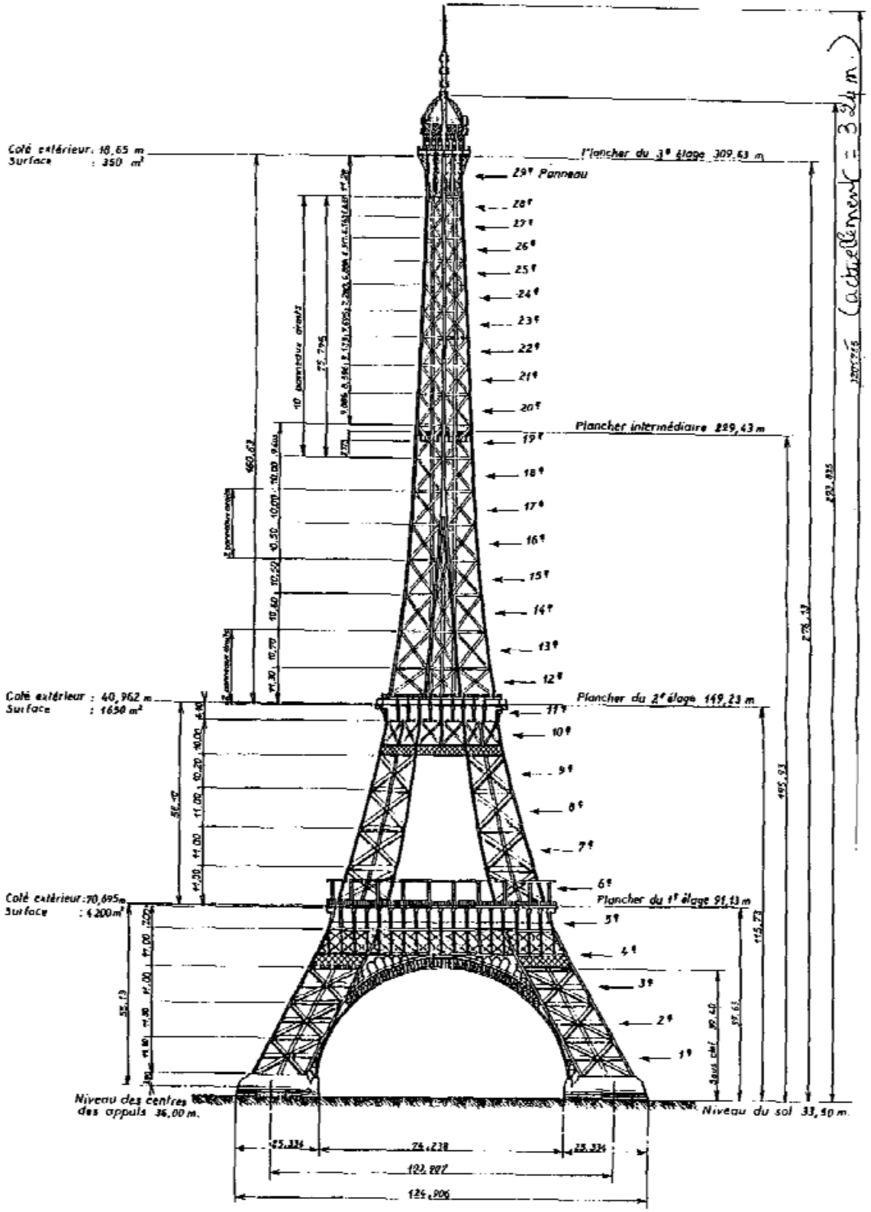


Strutture in Acciaio

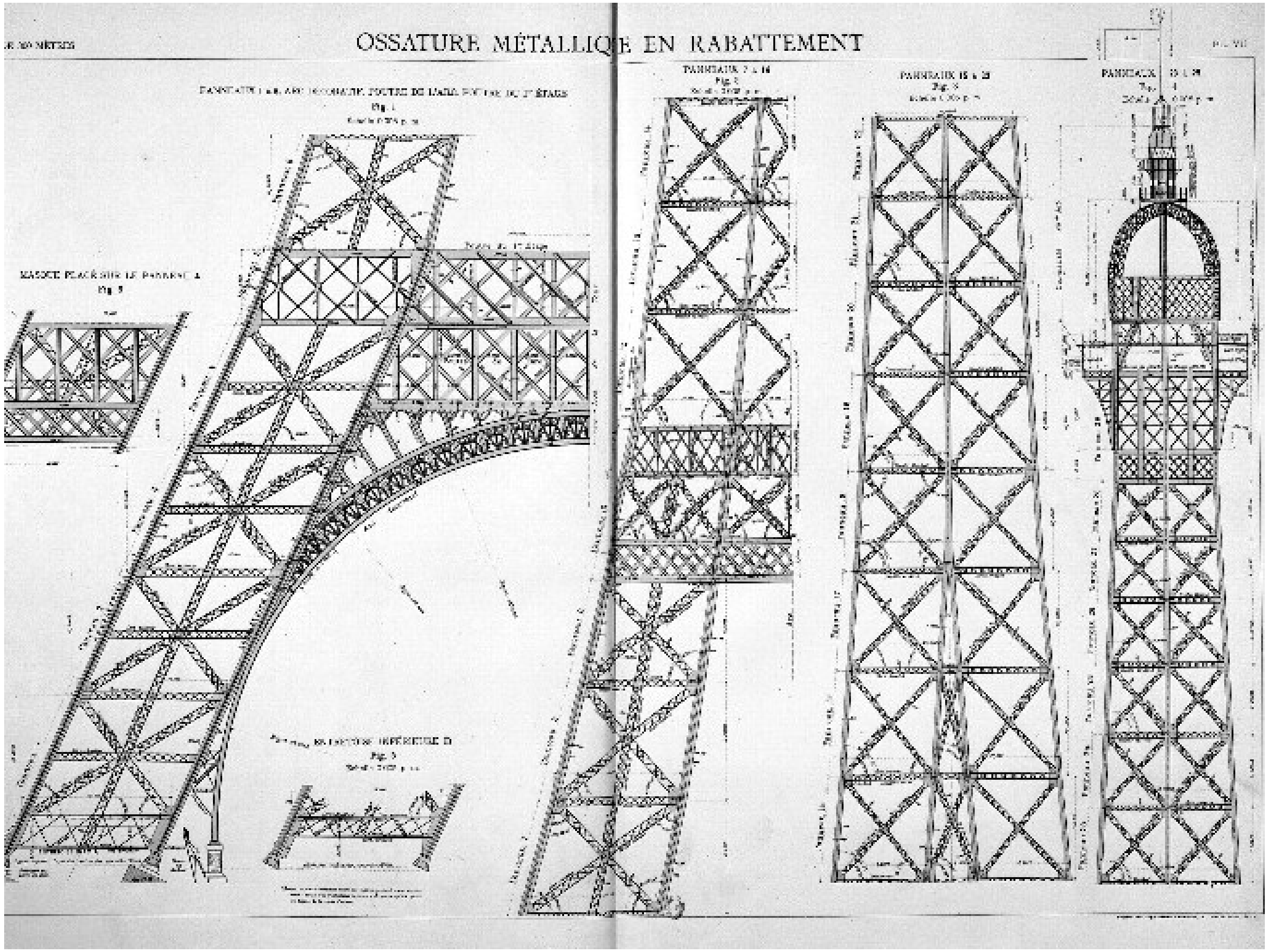
1932 **Rockefeller Center New York**
Lunch atop a skyscraper

La torre Eiffel 1889

ÉCHELLE : 1/1000

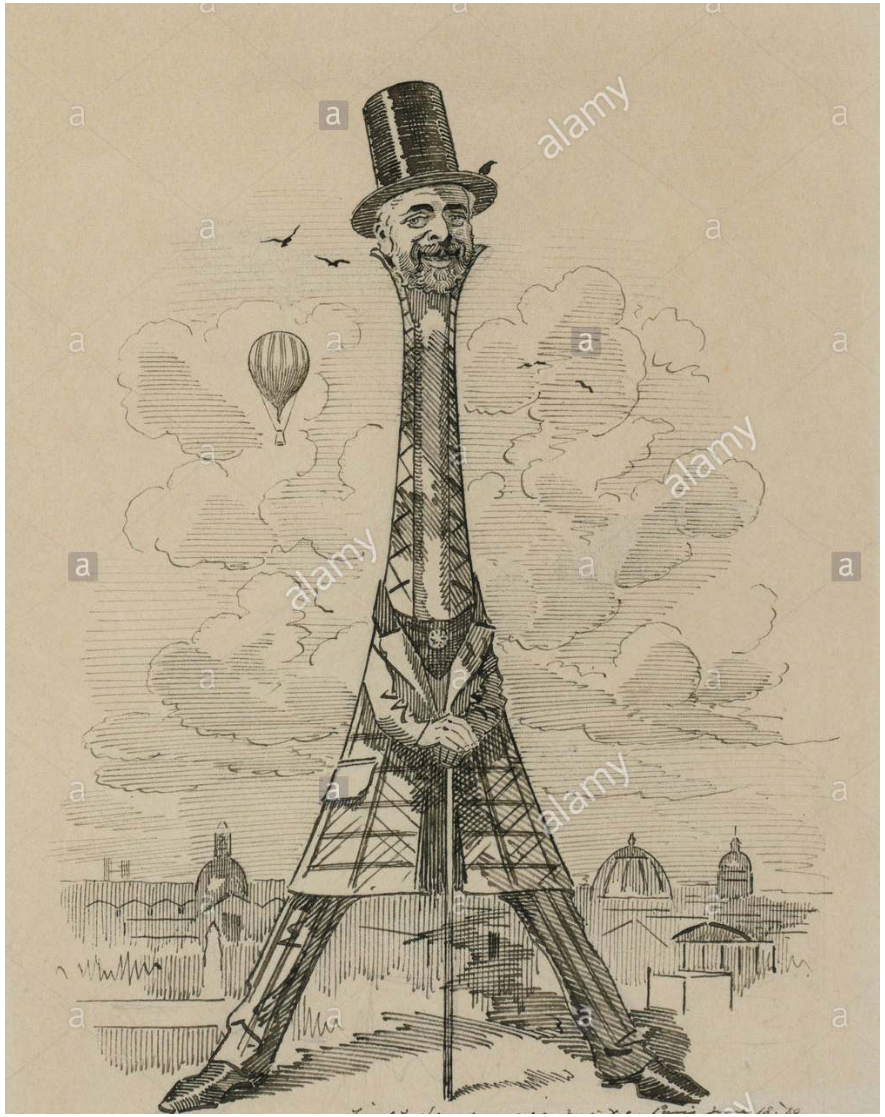

Coté extérieur : 18,65 m
Surface : 350 m²
Coté extérieur : 40,962 m
Surface : 1650 m²
Coté extérieur : 70,695 m
Surface : 4 200 m²
Niveau des centres des appuis 36,00 m.
Plancher du 3e étage 309,63 m
29e Panneau
28e
27e
26e
25e
24e
23e
22e
21e
20e
Plancher intermédiaire 229,43 m
19e
18e
17e
16e
15e
14e
13e
12e
Plancher du 2e étage 149,23 m
11e
10e
9e
8e
7e
6e
Plancher du 1er étage 91,13 m
5e
4e
3e
2e
1e
Niveau du sol 33,50 m.
10 panneaux droits
75,795
160,62
9,40
10,00
10,50
10,52
10,60
10,70
11,30
2 panneaux droits
57,40
58,10
4,50
10,00
10,20
11,00
11,00
11,30
55,13
7,20
11,00
11,00
11,30
11,80
3,80
Sous clef 39,40
57,63
115,73
195,93
276,13
292,835
300,725
25,334
74,238
103,907
124,906
(actuellement = 324 m.)

La torre Eiffel 1889


OSSATURE MÉTALLIQUE EN RABATTEMENT
PANNEAUX 7 à 14
Fig. 2
PANNEAUX 15 à 23
Fig. 3
PANNEAUX
Fig. 4
Fig. 1
Fig. 5

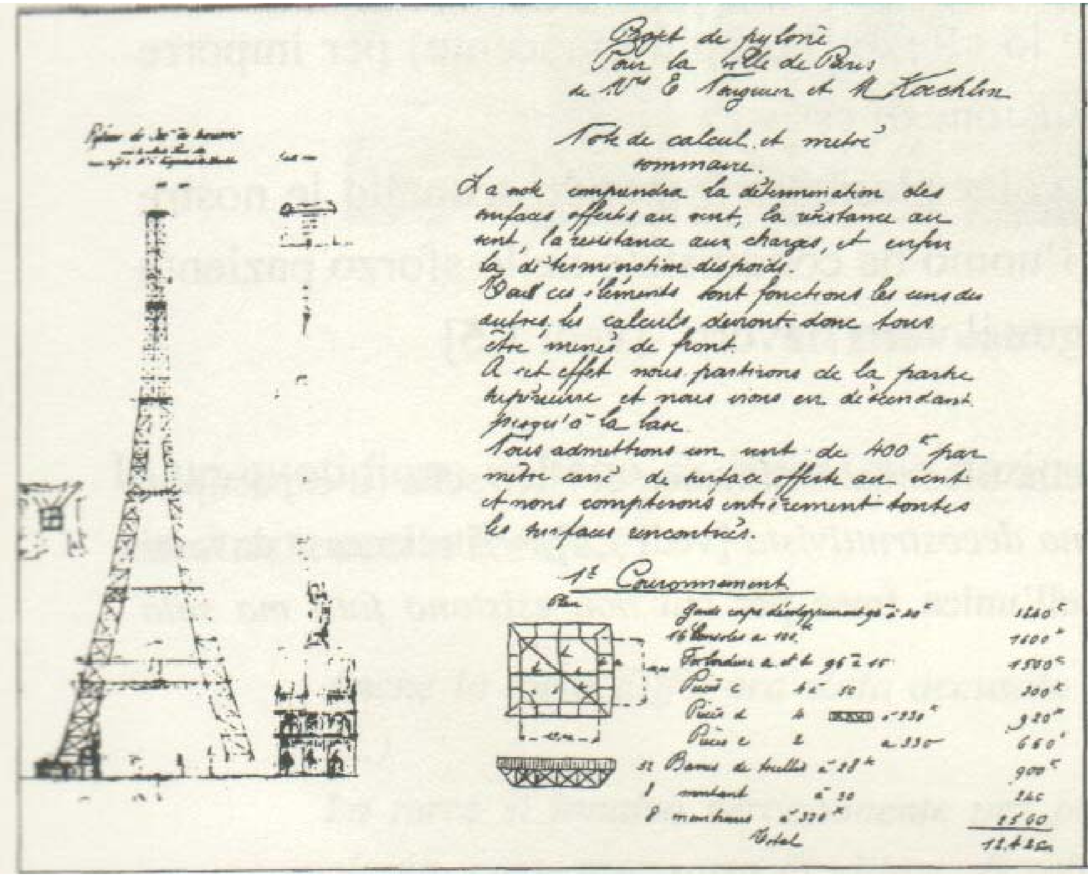
1333 mq di disegni

La torre Eiffel 1889



Vento di progetto: 238 km/h
Altezza: 300 m

La torre Eiffel 1889



Projet de pylône
Pour la ville de Paris
de Mrs E. Nouguier et M. Koechlin

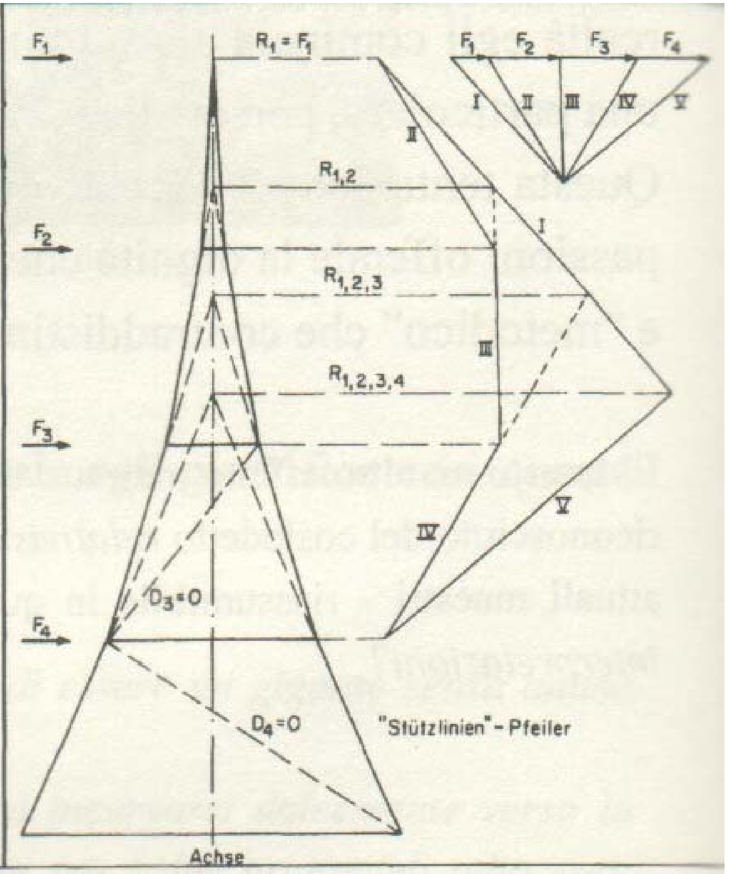
Note de calcul et métré
sommaire.

La note comprendra la détermination des surfaces offertes au vent, la résistance au vent, la résistance aux charges, et enfin la détermination des poids.
Tous ces éléments sont fonctions les uns des autres, les calculs devront donc tous être menés de front.
A cet effet nous partirons de la partie supérieure et nous irons en descendant jusqu'à la base.
Nous admettrons un vent de 400k par mètre carré de surface offerte au vent et nous compterons entièrement toutes les surfaces rencontrées.

1e Couronnement

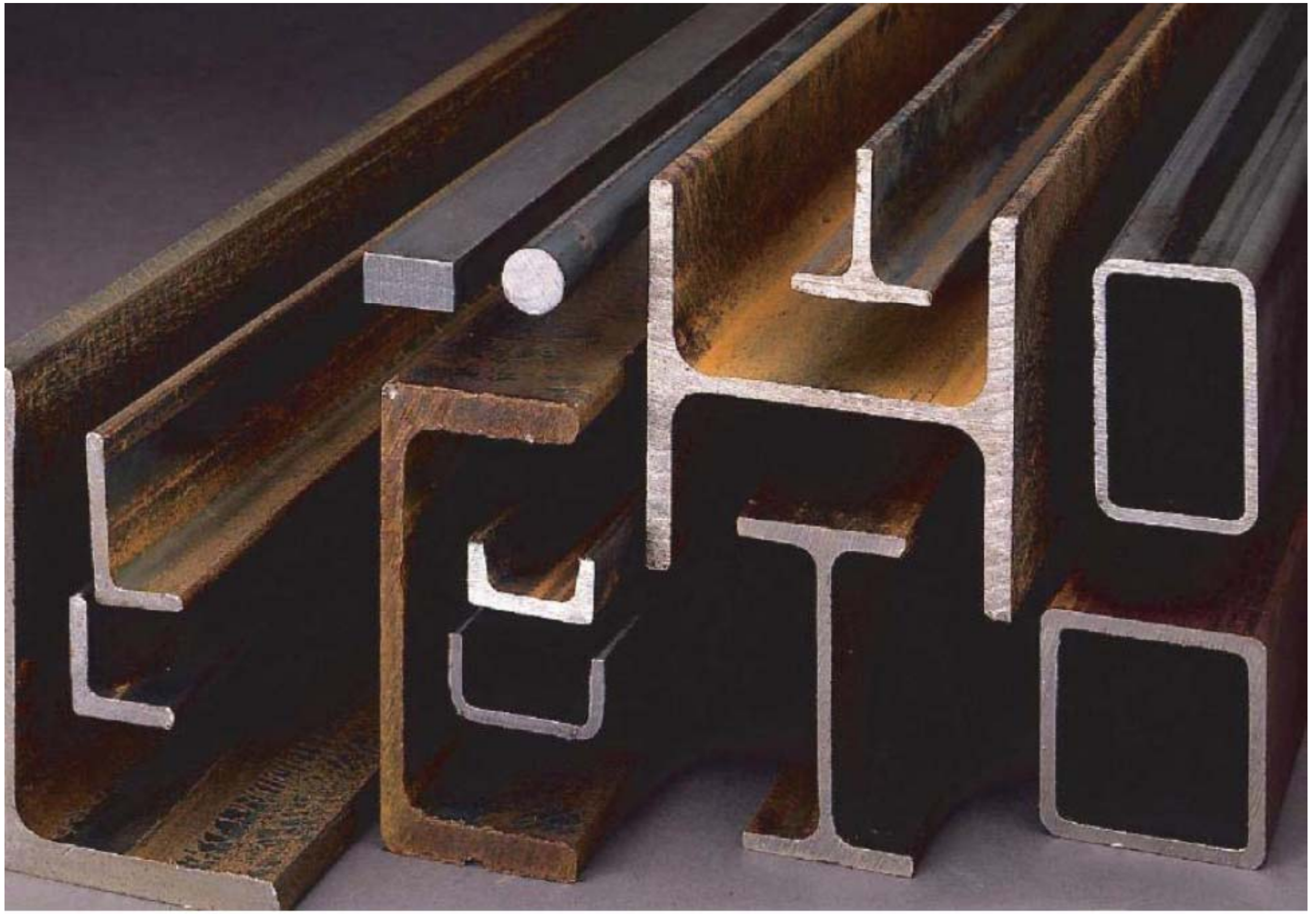
Plan

Grande coupe développement 92 à 20k	1840k
16 Consoles à 100k	1600k
Poutres a et b 96 à 15	1500k
Pièces c 4 à 50	200k
Pièces d 4 à 230k	920k
Pièces e 2 à 330	660k
32 Barres de treillis à 28k	900k
1 montant à 30	240
8 montants à 320k	2560
Total	12,420

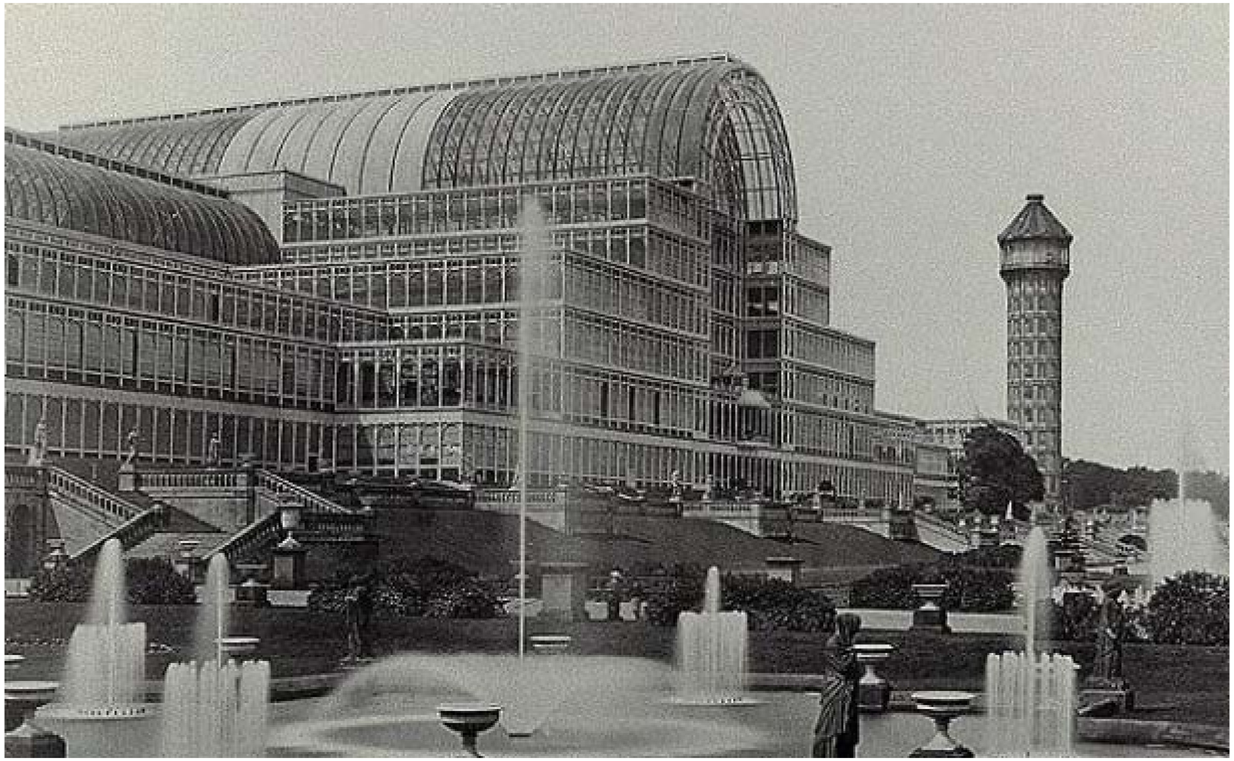


Calcolo di Maurice Koechlin

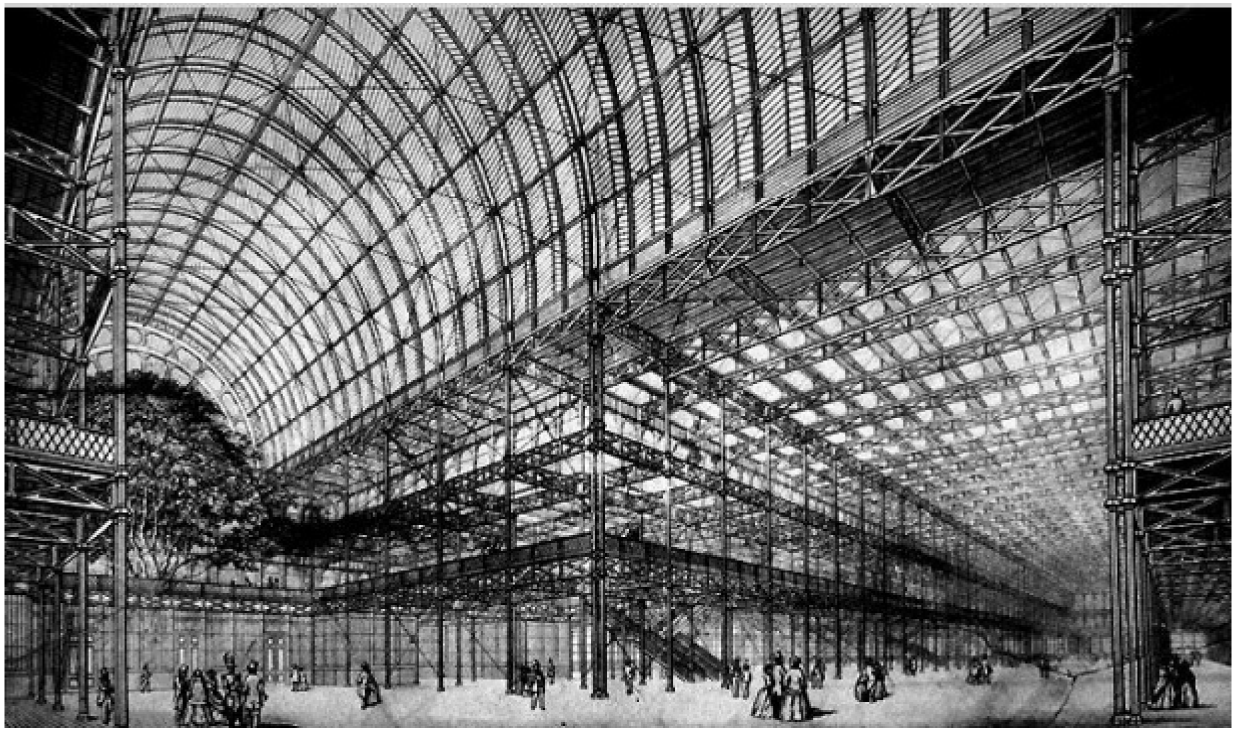
Le strutture in acciaio



Crystal Palace Londra 1851

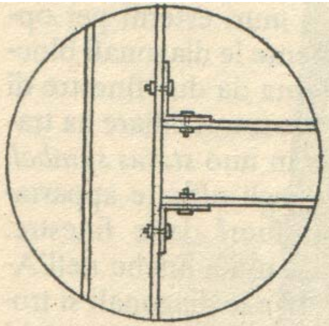


Crystal Palace 1851
Primi calcoli a telaio

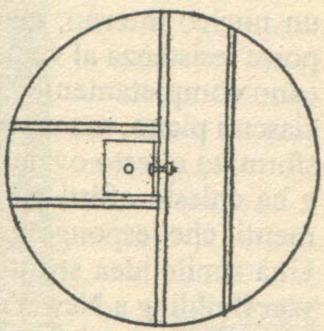


Struttura a telaio

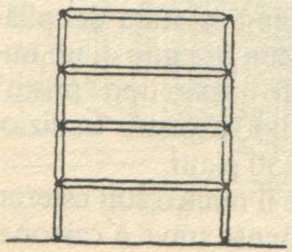
Telai e controventature

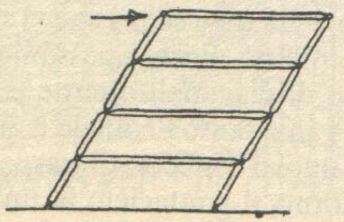


7.6 GIUNTO RIGIDO, RESISTENTE A FLESSIONE E TAGLIO



7.7 GIUNTO INCERNIERATO, RESISTENTE SOLO A TAGLIO





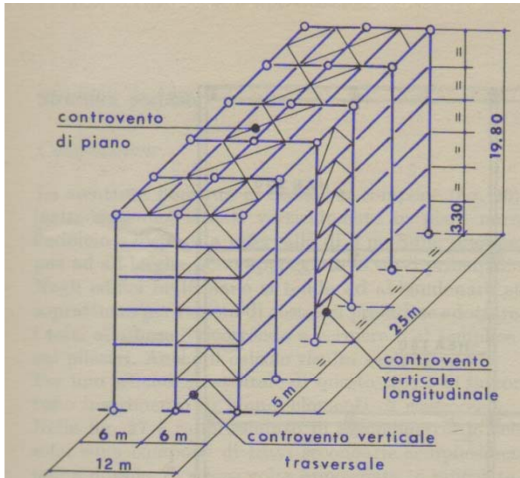

controvento
di piano
19.80
3.30
25 m
5 m
controvento
verticale
longitudinale
6 m
6 m
12 m
controvento verticale
trasversale

Fig. 27. Assonometria dello schema statico.

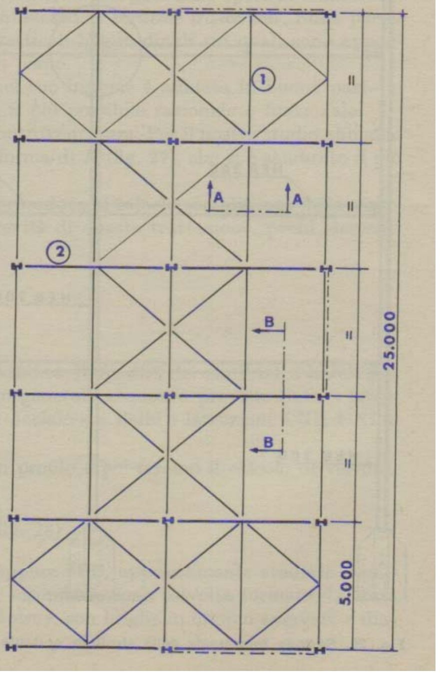
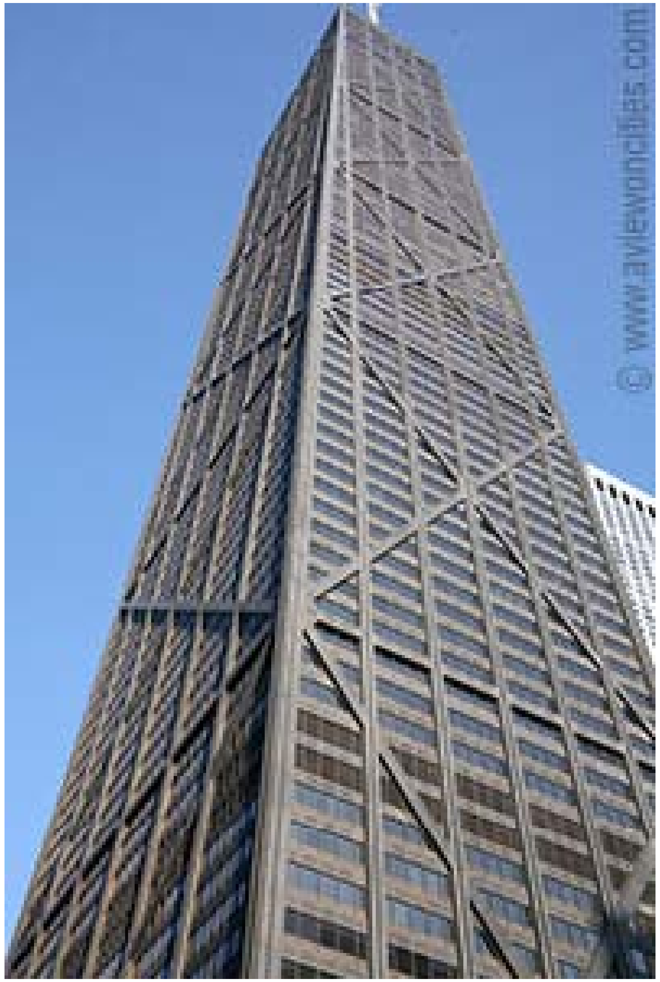

①
②
A
A
B
B
25.000
5.000

Fig. 28. Orditura piano tipo.

John Hancock Building Chicago 1969



© www.aviewoncities.com

Esempio edificio in Acciaio

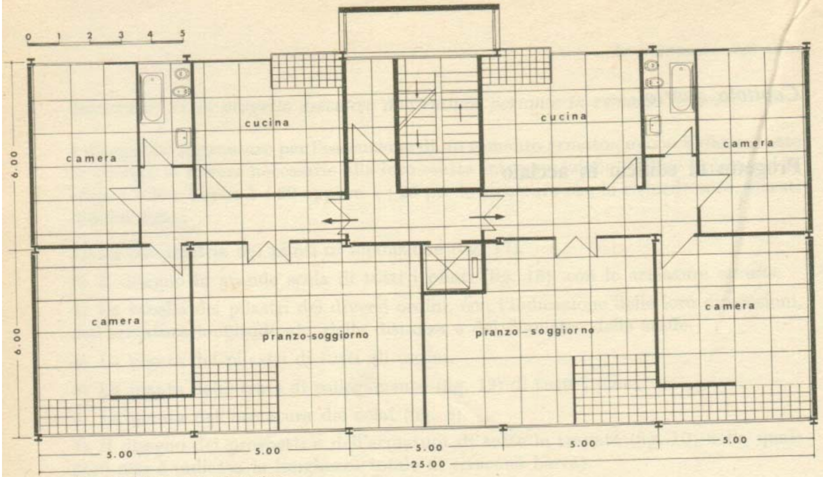
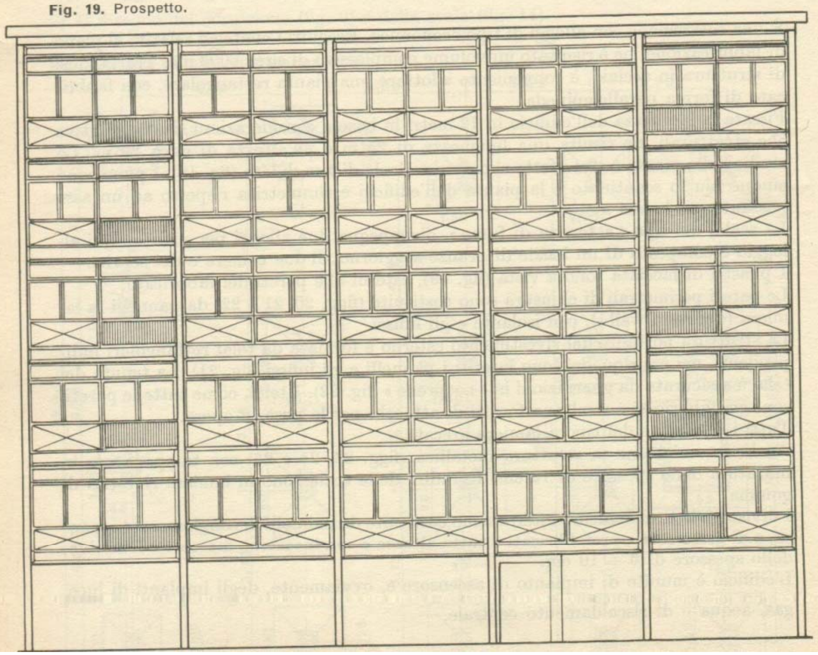
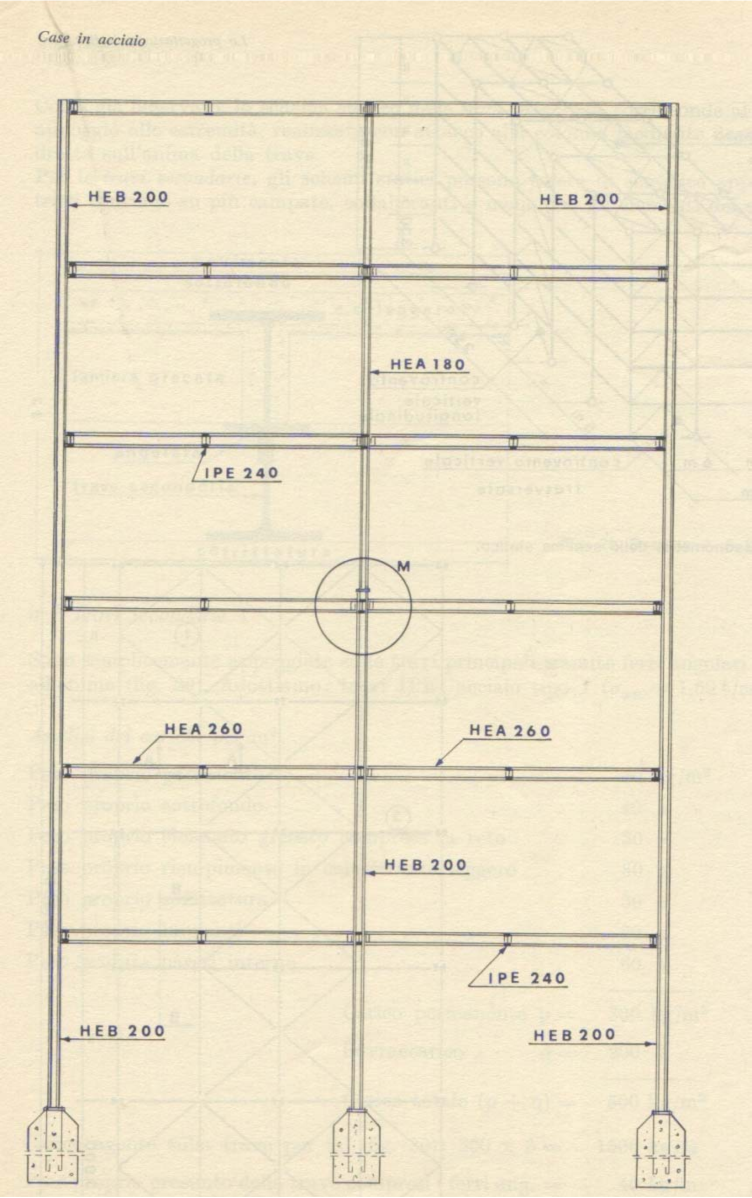

0 1 2 3 4 5
cucina
cucina
camera
camera
camera
camera
pranzo-soggiorno
pranzo – soggiorno
6.00
6.00
5.00
5.00
5.00
5.00
5.00
25.00

Fig. 18. Pianta del piano tipo.

Fig. 19. Prospetto.



Esempio edificio in Acciaio


Case in acciaio
HEB 200
HEB 200
HEA 180
IPE 240
M
HEA 260
HEA 260
HEB 200
IPE 240
HEB 200
HEB 200

L’essenza della struttura: I ponti

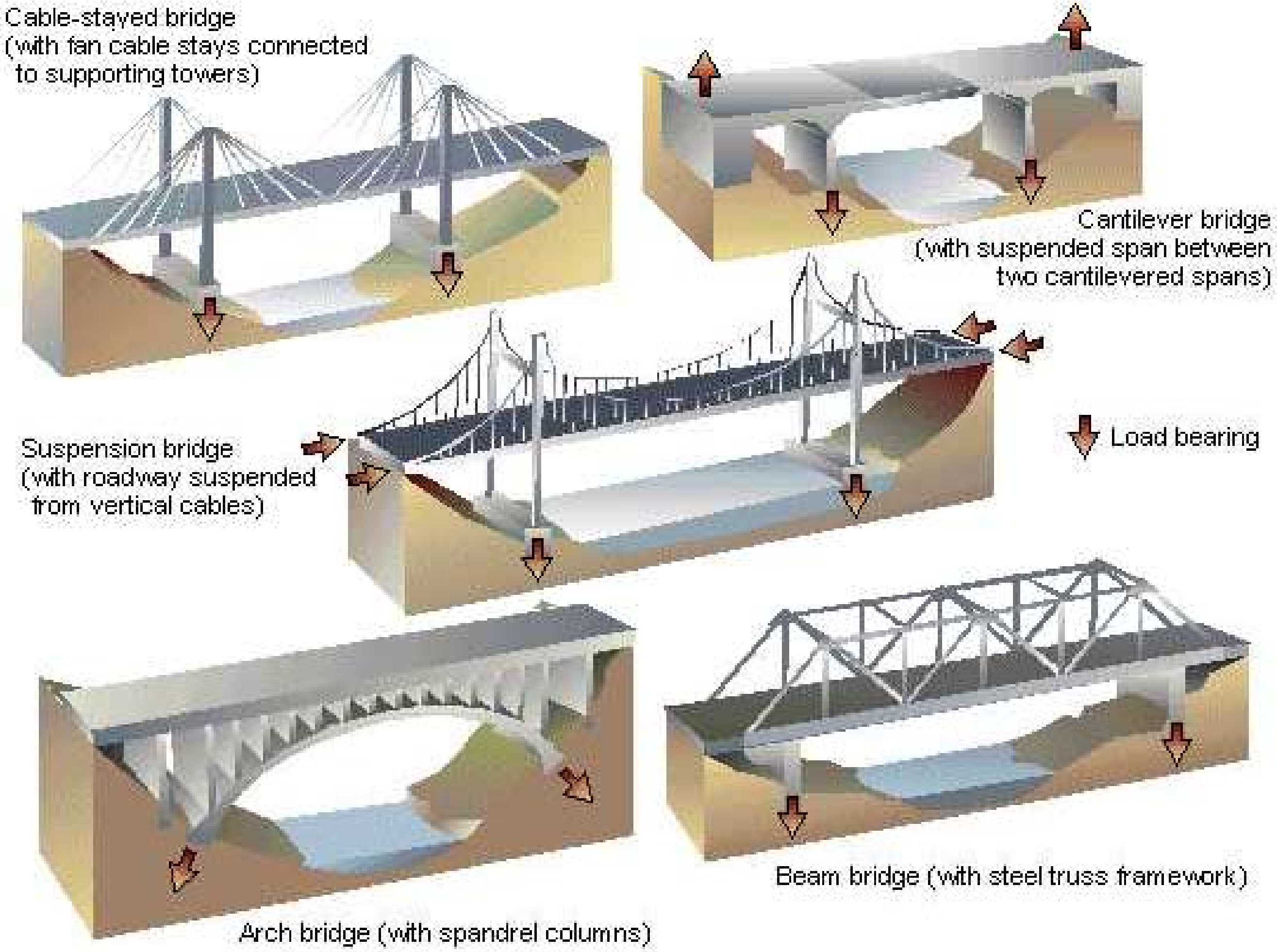
Definizioni

Ponte:costruzione, che unisce le rive opposte di un fiume, di un canale, ecc.,

Cavalcavia: ponte che passa sopra una strada, negli incroci con altra strada o con una ferrovia

Viadotto: costruzione permette ad una strada o a una ferrovia di superare una valle o una depressione di terreno.

L’essenza della struttura: I ponti


Cable-stayed bridge
(with fan cable stays connected to supporting towers)
Cantilever bridge
(with suspended span between two cantilevered spans)
Suspension bridge
(with roadway suspended from vertical cables)
Load bearing
Arch bridge (with spandrel columns)
Beam bridge (with steel truss framework)

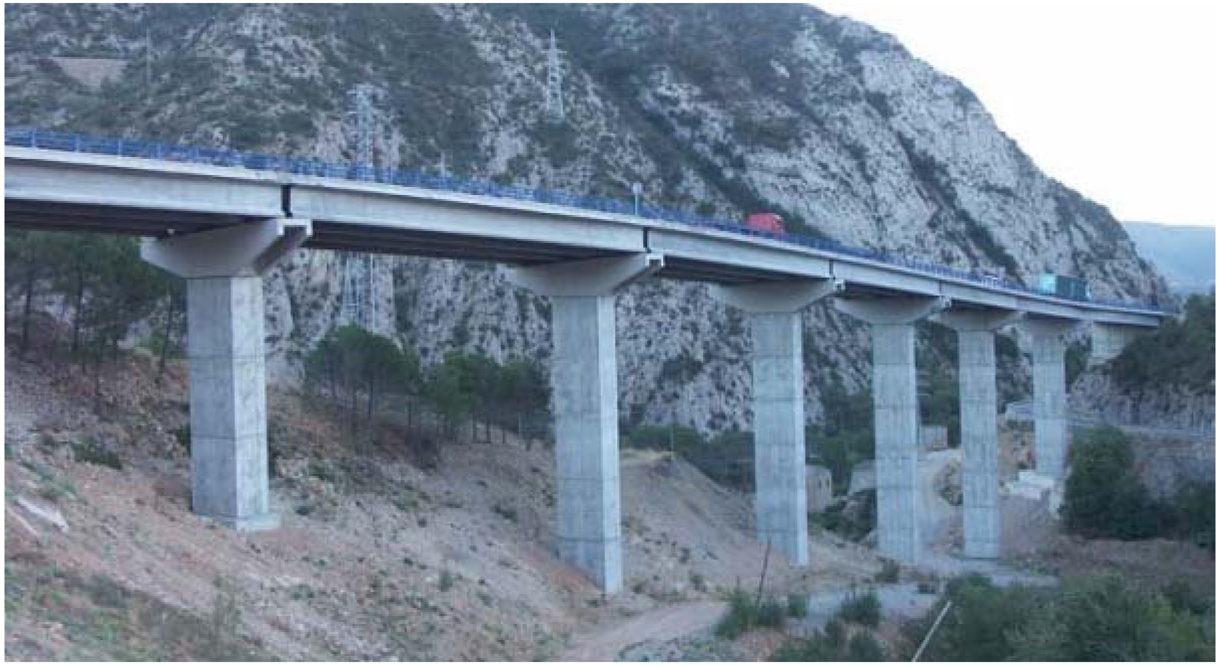
Several types of bridges
Microsoft Corporation

Ponti

Classificazione in base allo schema statico

Ponti a Travata
Ponti ad Arco
Ponti a Travata reticolare
Ponti Strallati
Ponti Sospesi

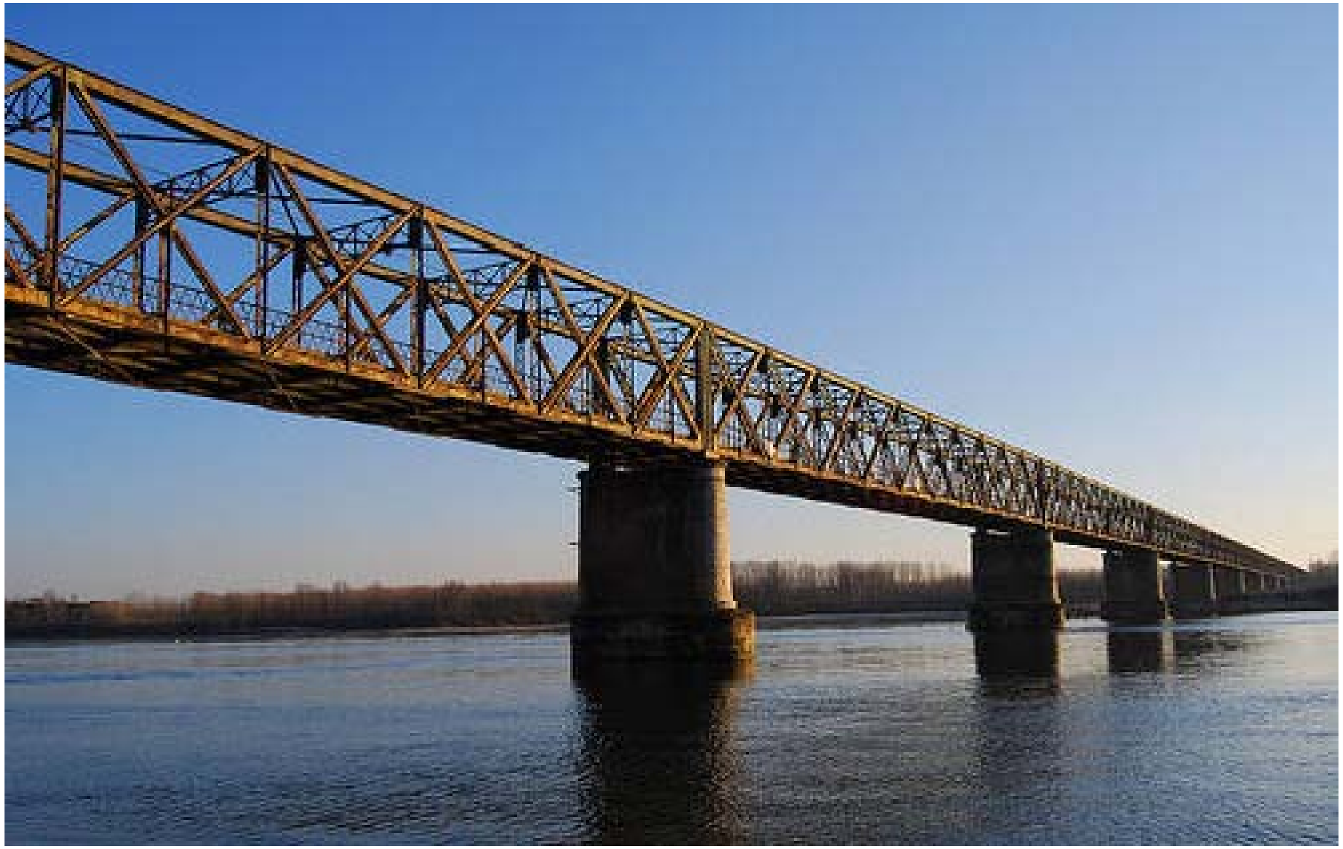
Ponti a travata



Ponti ad Arco

Ponti Maillart

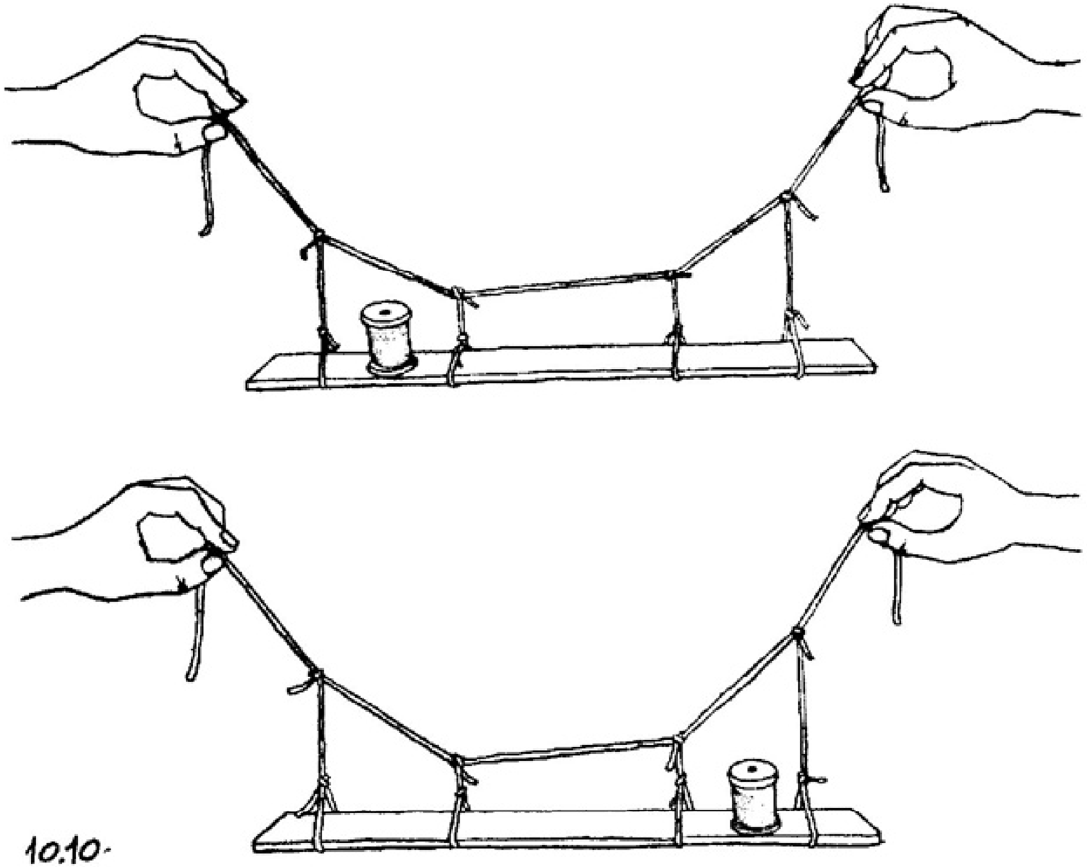
Ponti a travata reticolare



Ponte della Becca Pavia 1912

Ponti Strallati

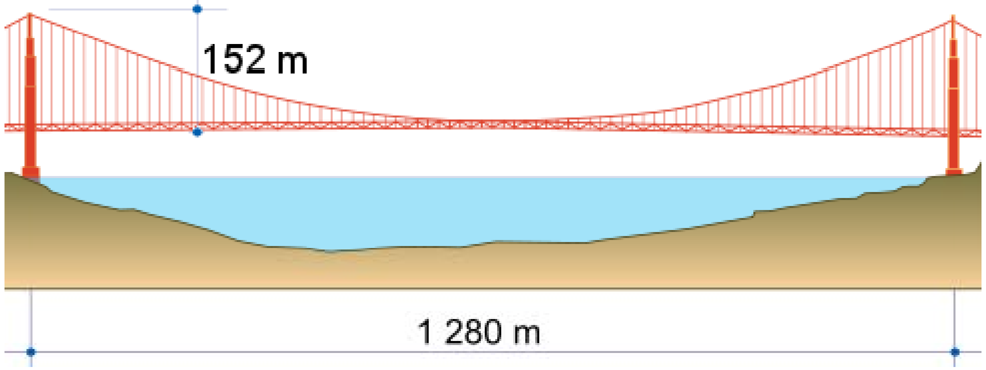
Ponti sospesi



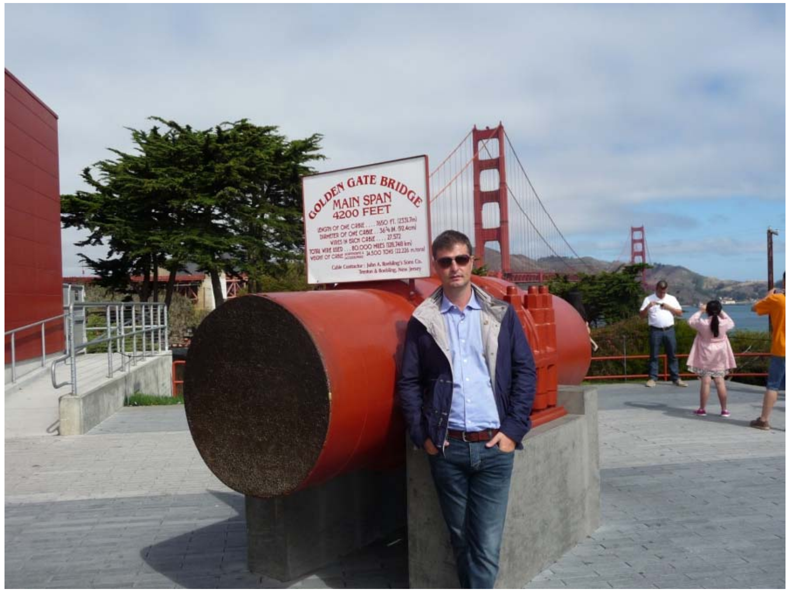
10.10-

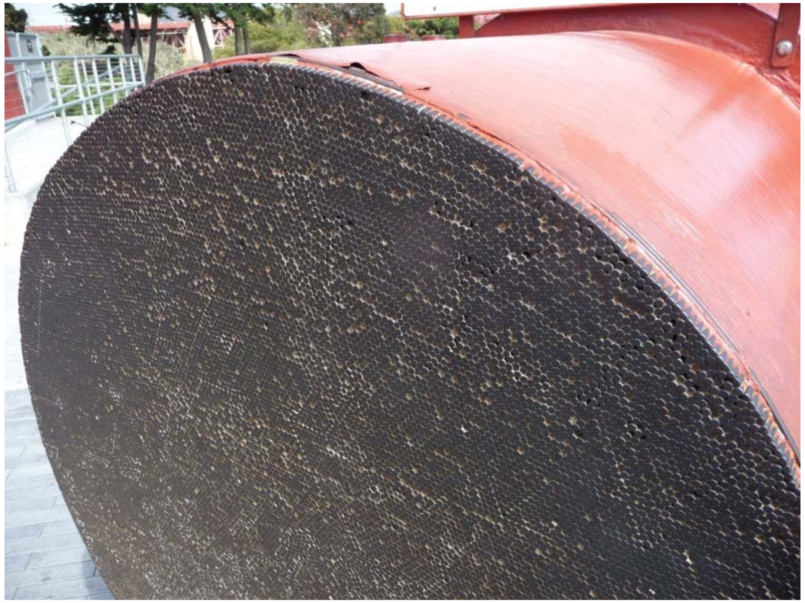
Ponti sospesi

Golden Gate Bridge – San Francisco 1937



Cavo di sospensione

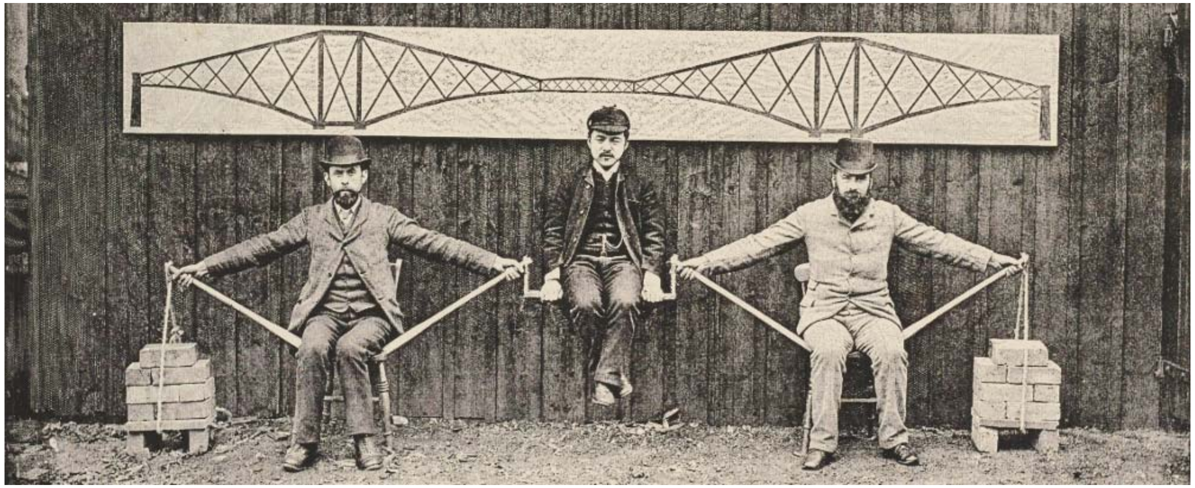

GOLDEN GATE BRIDGE
MAIN SPAN
4200 FEET
LENGTH OF ONE CABLE .... 7650 FT. (2331.7m)
DIAMETER OF ONE CABLE .. 36 3/8 IN. (92.4cm)
WIRES IN EACH CABLE 27,572
TOTAL WIRE USED ... 80,000 MILES (128,748 km)
WEIGHT OF CABLE ... 24,500 TONS (22,226 m.tons)
Cable Contractor : John A. Roebling's Sons Co.
Trenton & Roebling, New Jersey



Ponte di Brooklyn – NY 463m - 1883

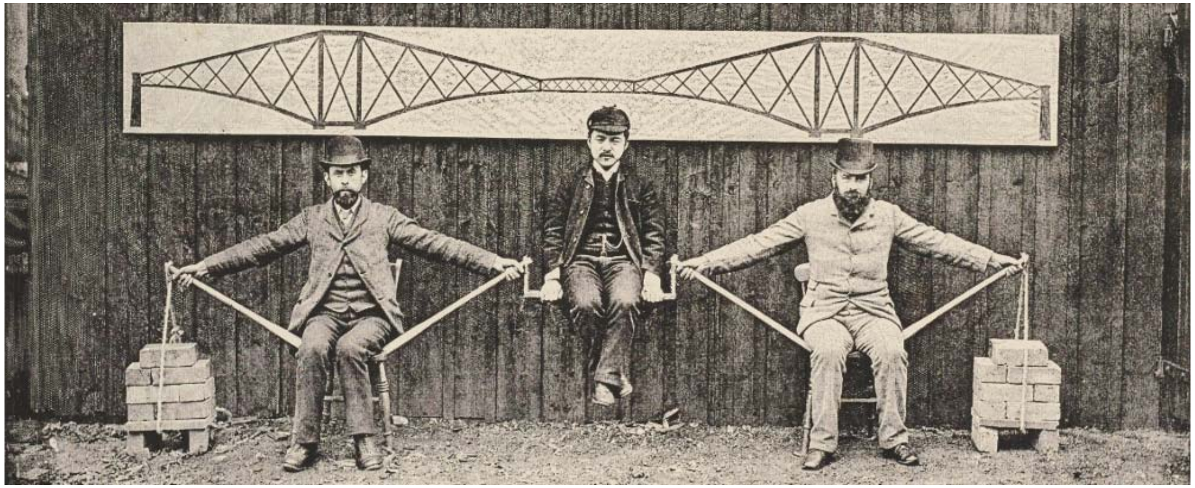
Forth Bridge - Scozia 1890

Sistema della trave Gerber

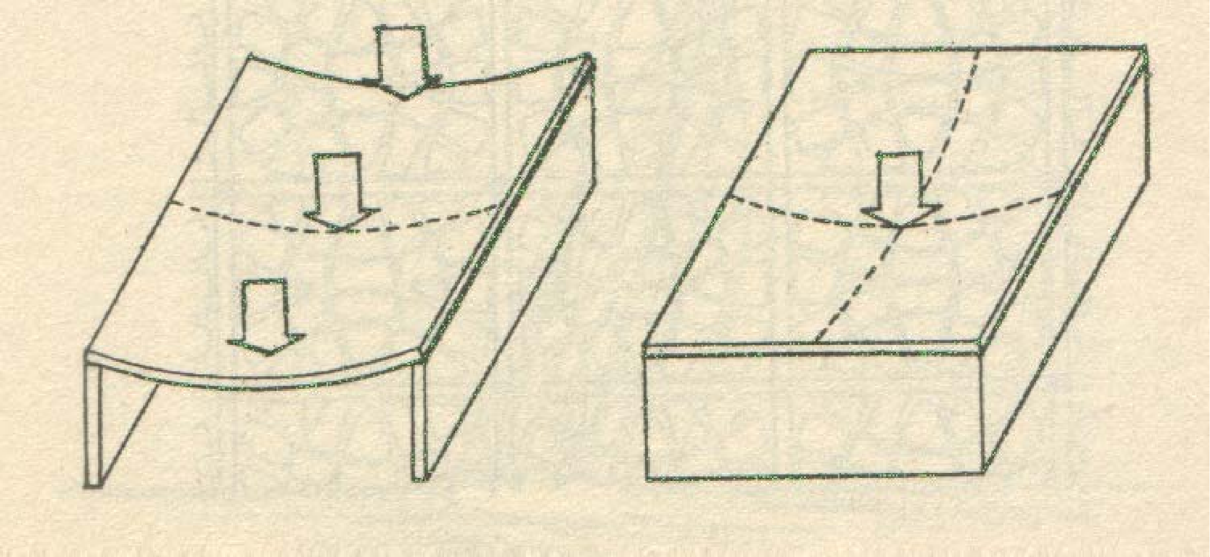


Forth Bridge - Scozia 1890

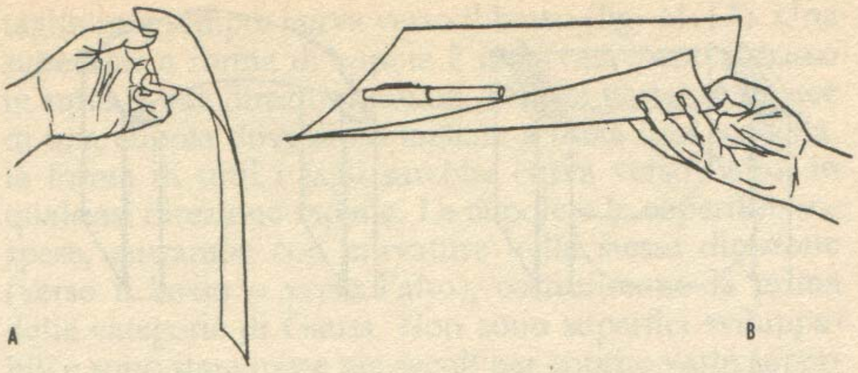
Ponte Morandi 1967



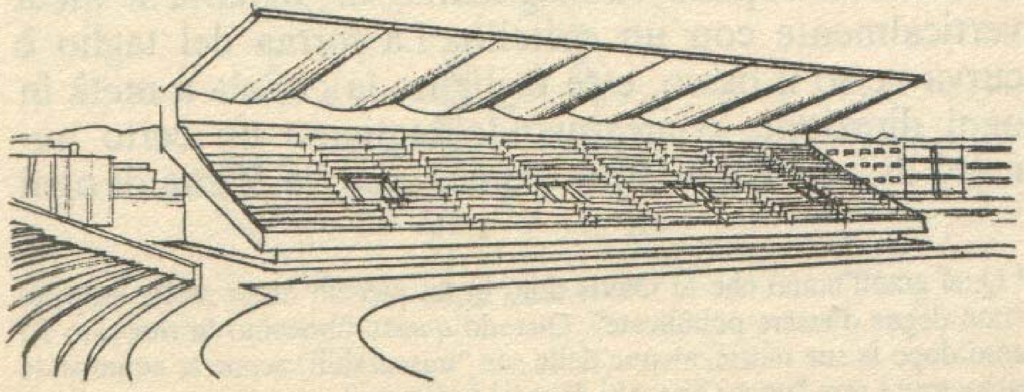
Resistenza per forma



Resistenza per forma

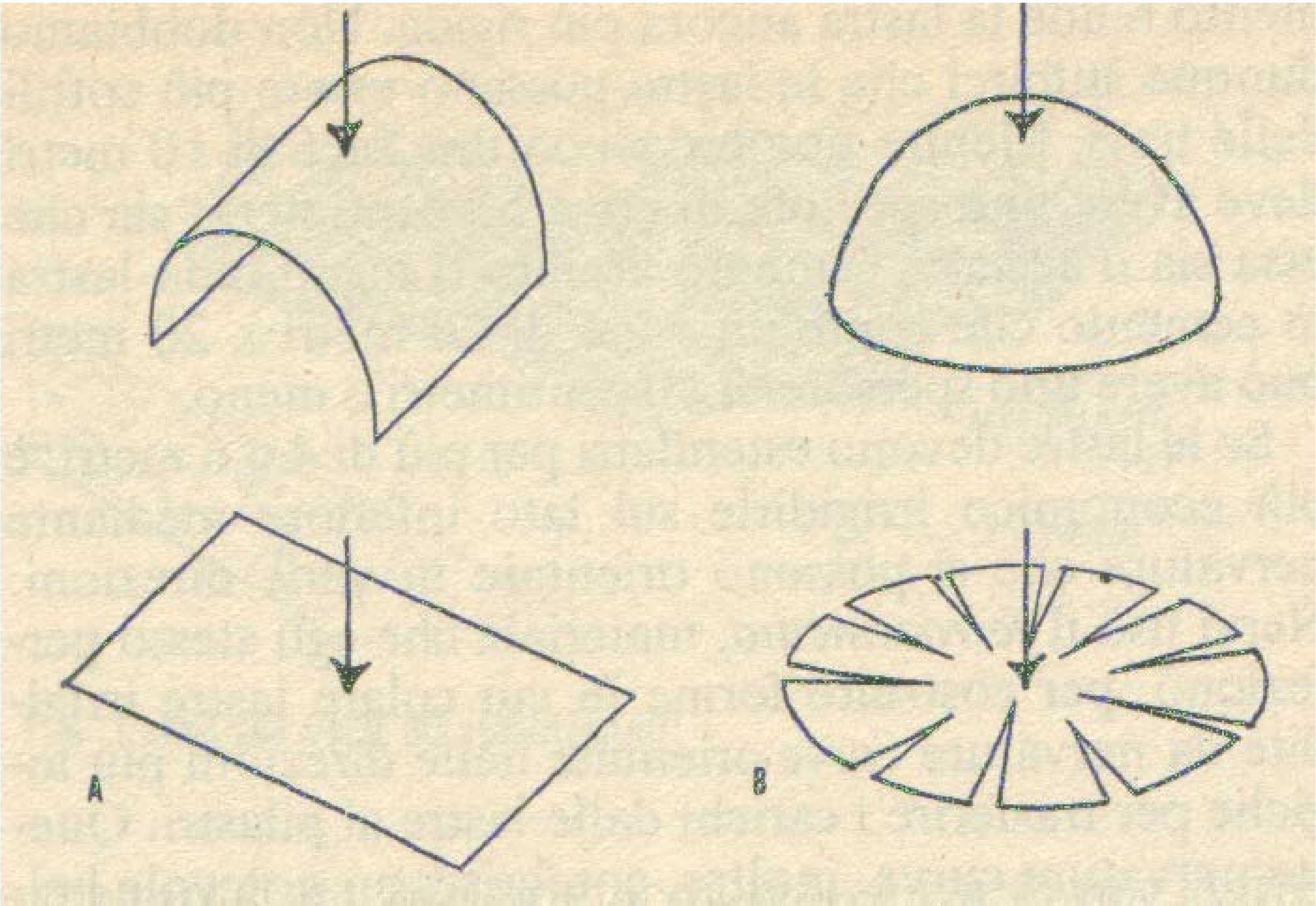


11.10 FOGLIO DI CARTA IRRIGIDITO DALLA CURVATURA



11.11 COPERTURA DELLA GRADONATA D'UNO STADIO

Resistenza per forma

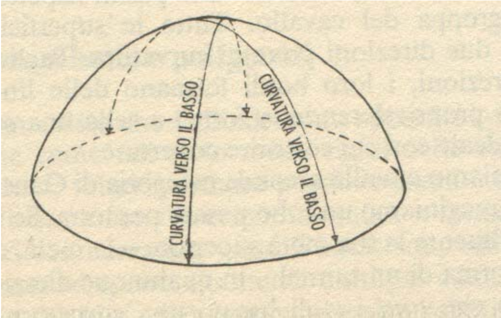


Superfici Sviluppabili

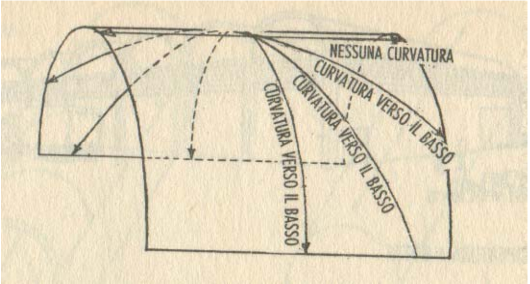
Superfici Non Sviluppabili

Superfici curve elementari

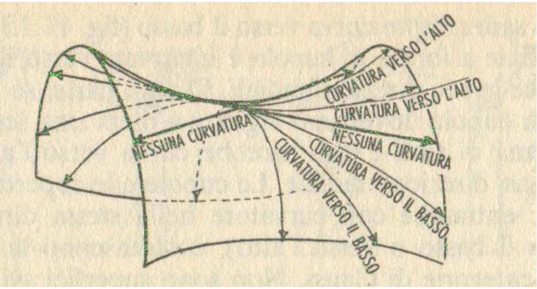
Cupola


CURVATURA VERSO IL BASSO
CURVATURA VERSO IL BASSO

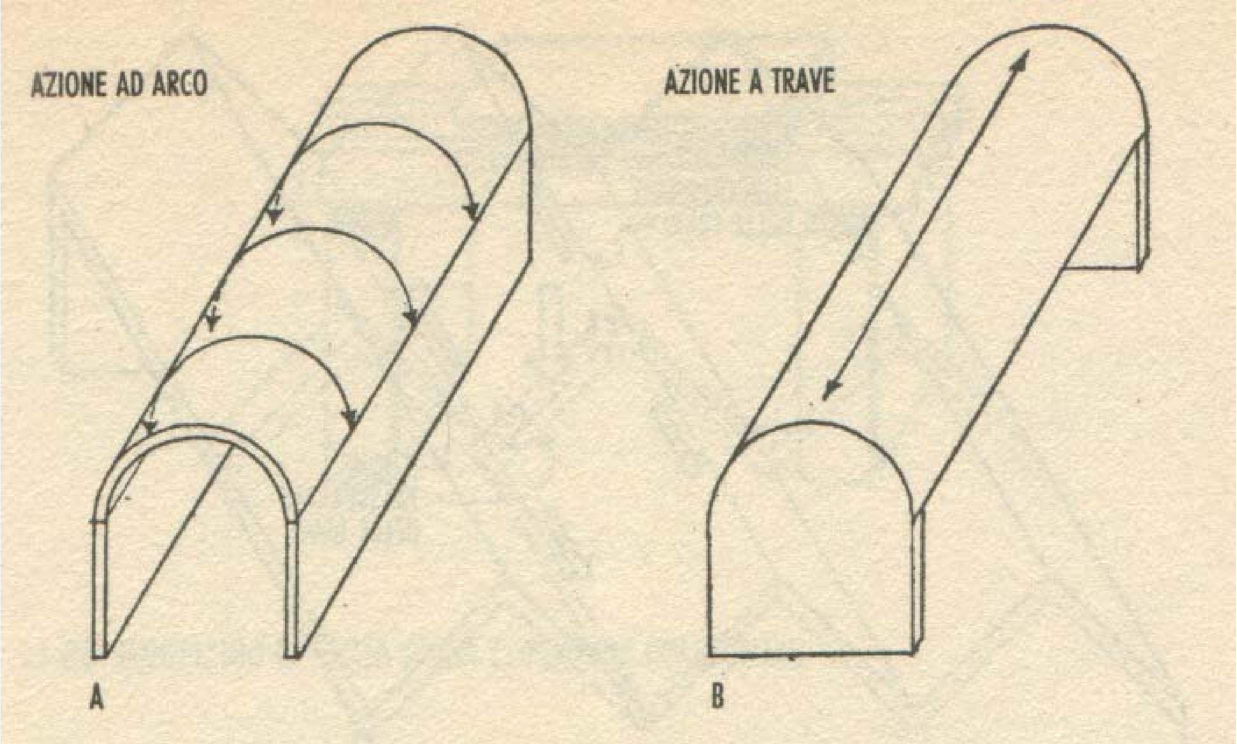
Cilindro


NESSUNA CURVATURA
CURVATURA VERSO IL BASSO
CURVATURA VERSO IL BASSO

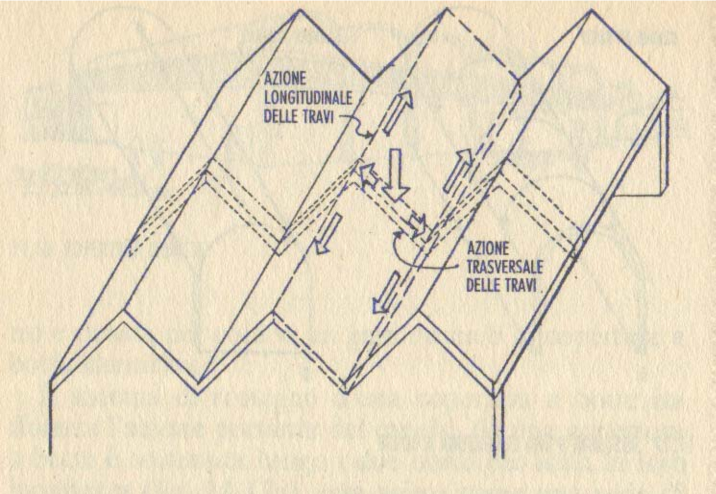
Sella


CURVATURA VERSO L'ALTO
CURVATURA VERSO L'ALTO
NESSUNA CURVATURA
NESSUNA CURVATURA
CURVATURA VERSO IL BASSO
CURVATURA VERSO IL BASSO

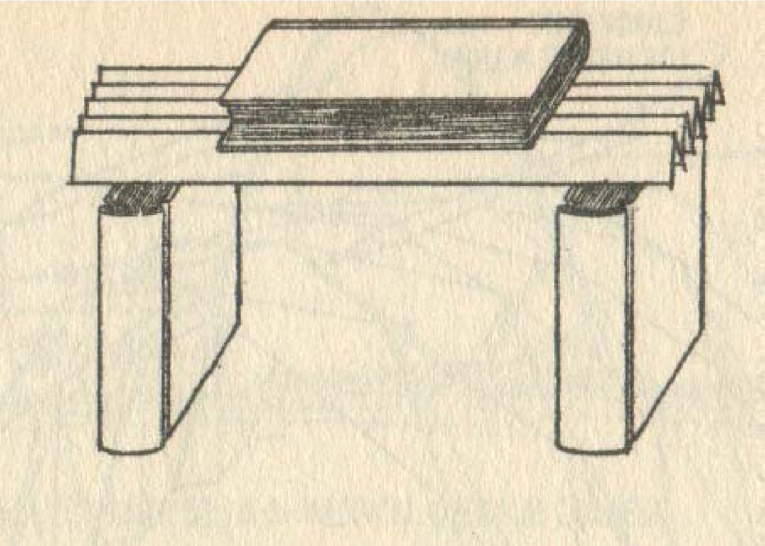
Cilindro


AZIONE AD ARCO
AZIONE A TRAVE
A
B

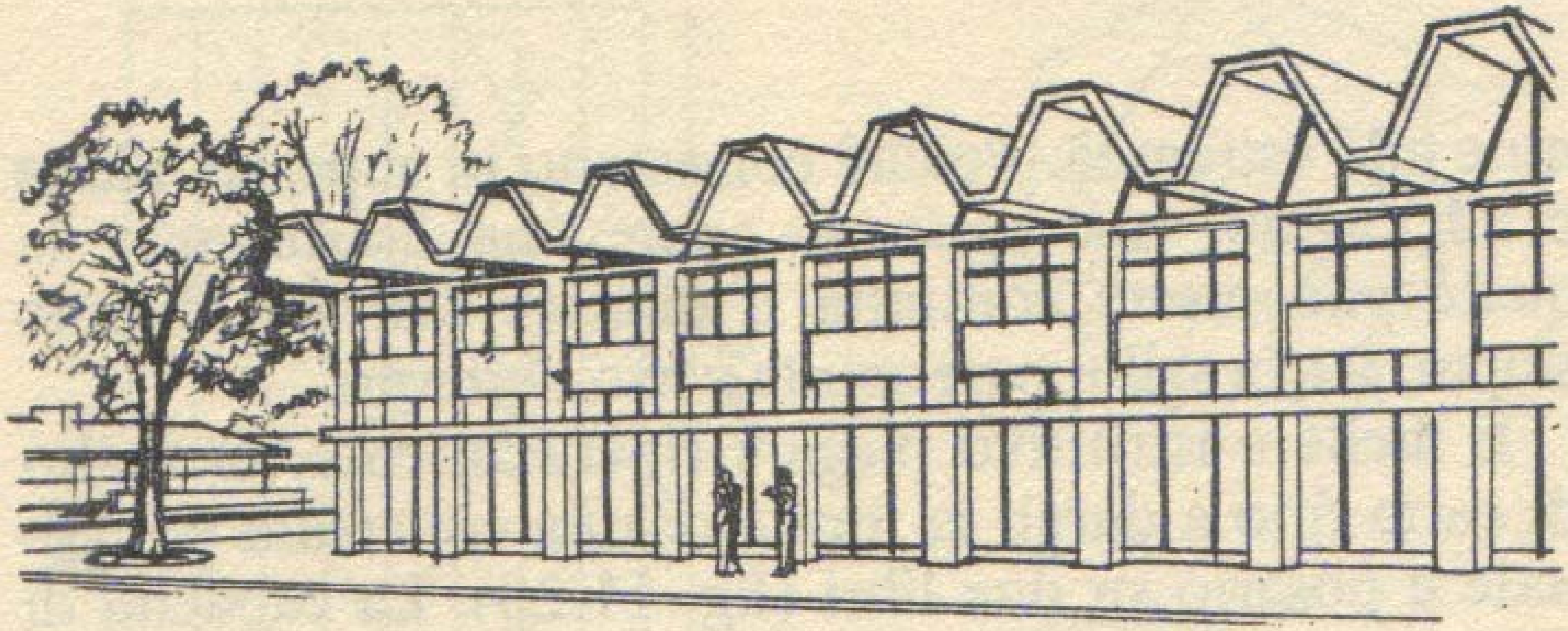
Poligonale


AZIONE LONGITUDINALE DELLE TRAVI
AZIONE TRASVERSALE DELLE TRAVI

11.19 PERCORSI DEI CARICHI IN UNA COPERTURA POLIGONALE

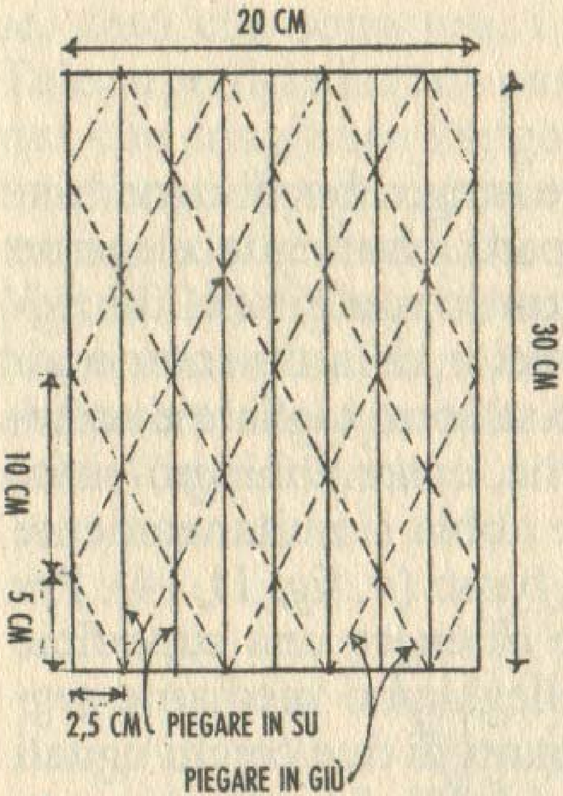


Poligonale

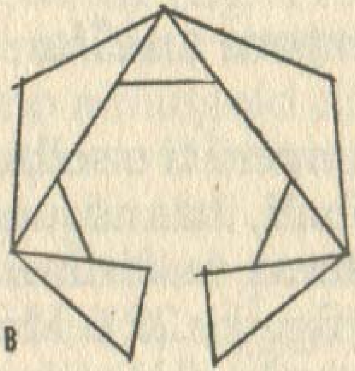


11.18 COPERTURA POLIGONALE

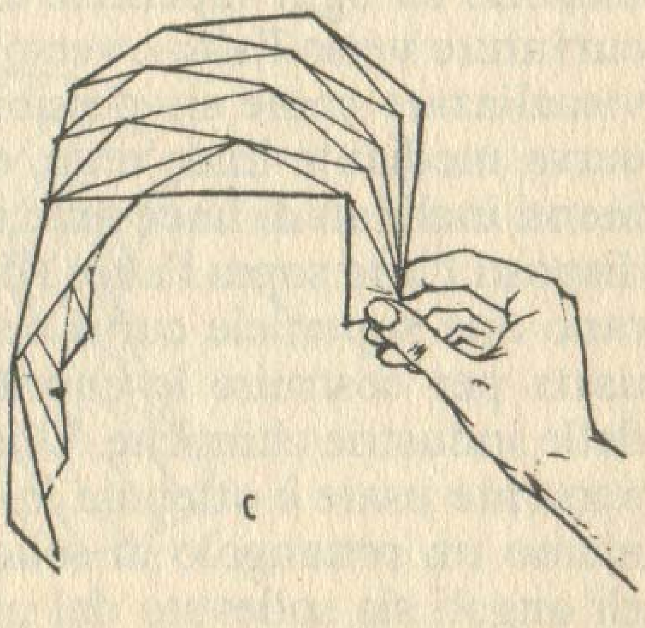
Unione dei due: Botte poligonale


20 CM
30 CM
10 CM
5 CM
2,5 CM
PIEGARE IN SU
PIEGARE IN GIÙ

A

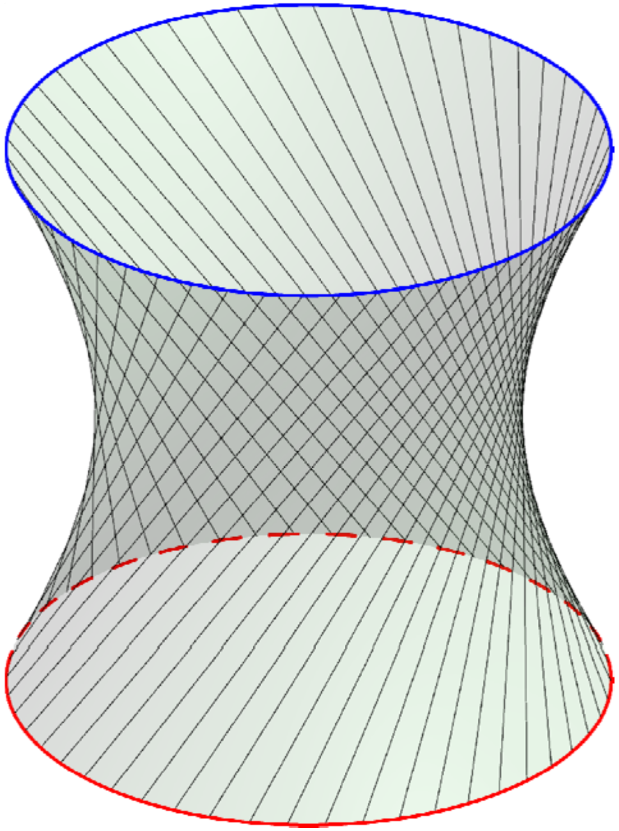


B

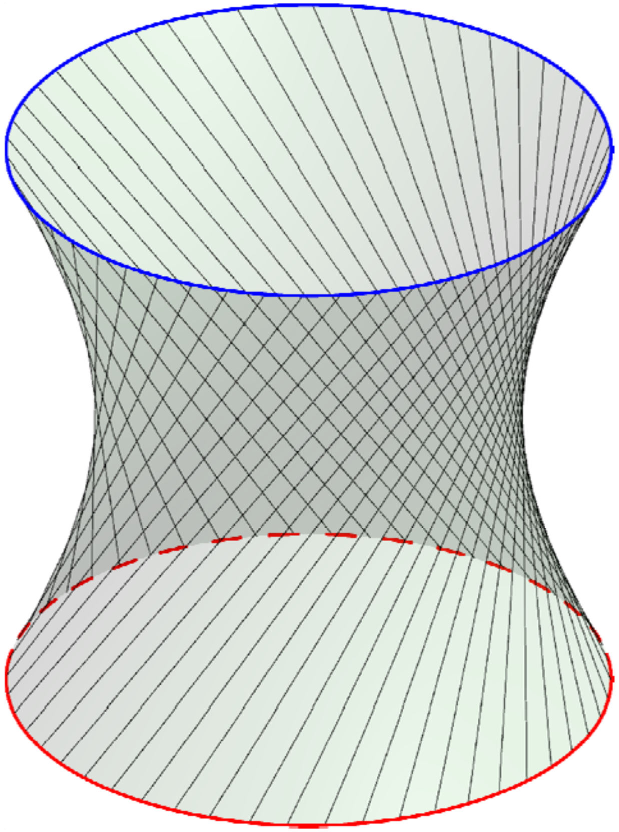


C

Iperboloide di rotazione

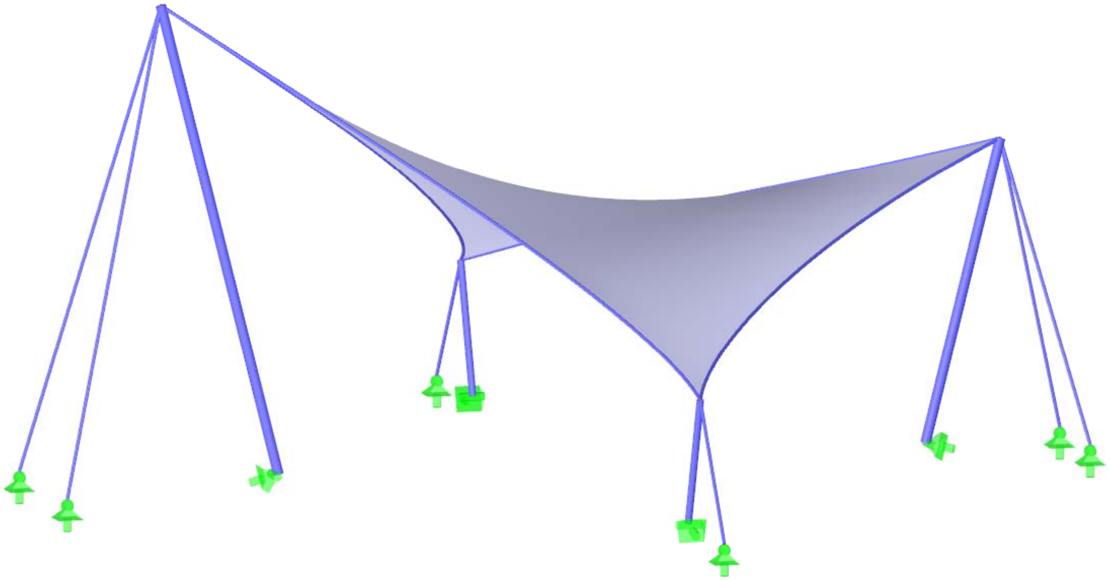


Iperboloide di rotazione



Torri Shukhov

Paraboloide iperbolico



Sella

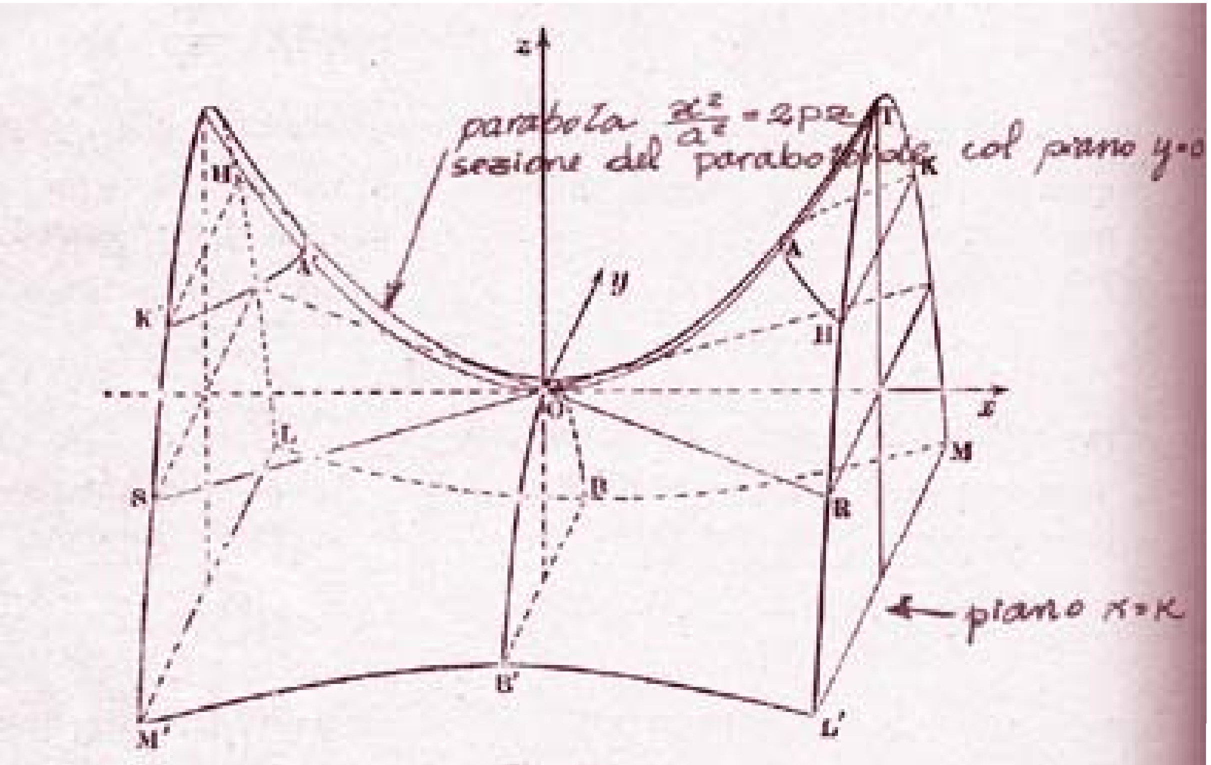
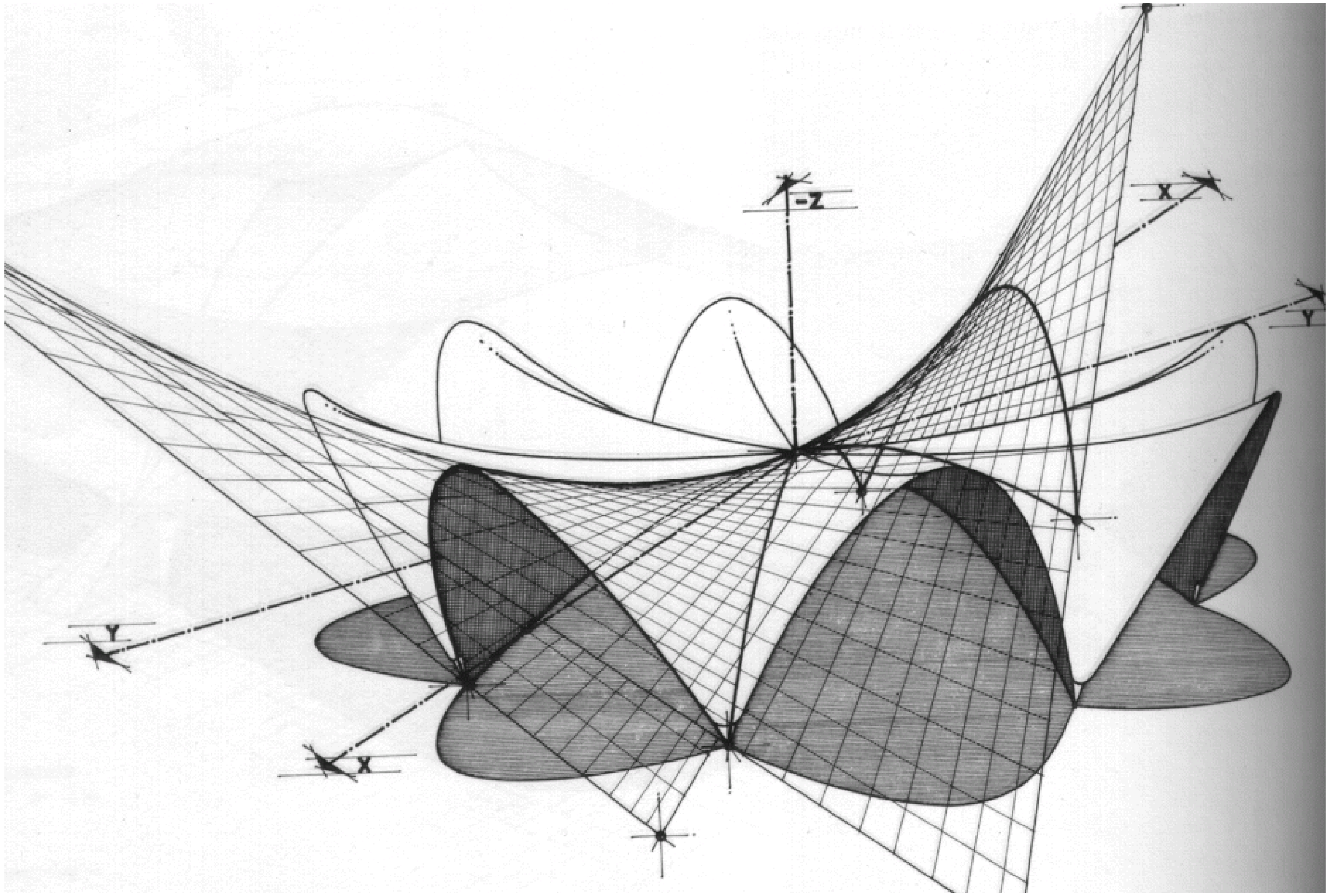

z
parabola $\frac{x^2}{a^2} = 2pz$
sezione del paraboloide col piano y=0
T
K
H
A
K'
H
A
y
O
x
L
M
S
B
R
piano x=K
B'
M'
L'

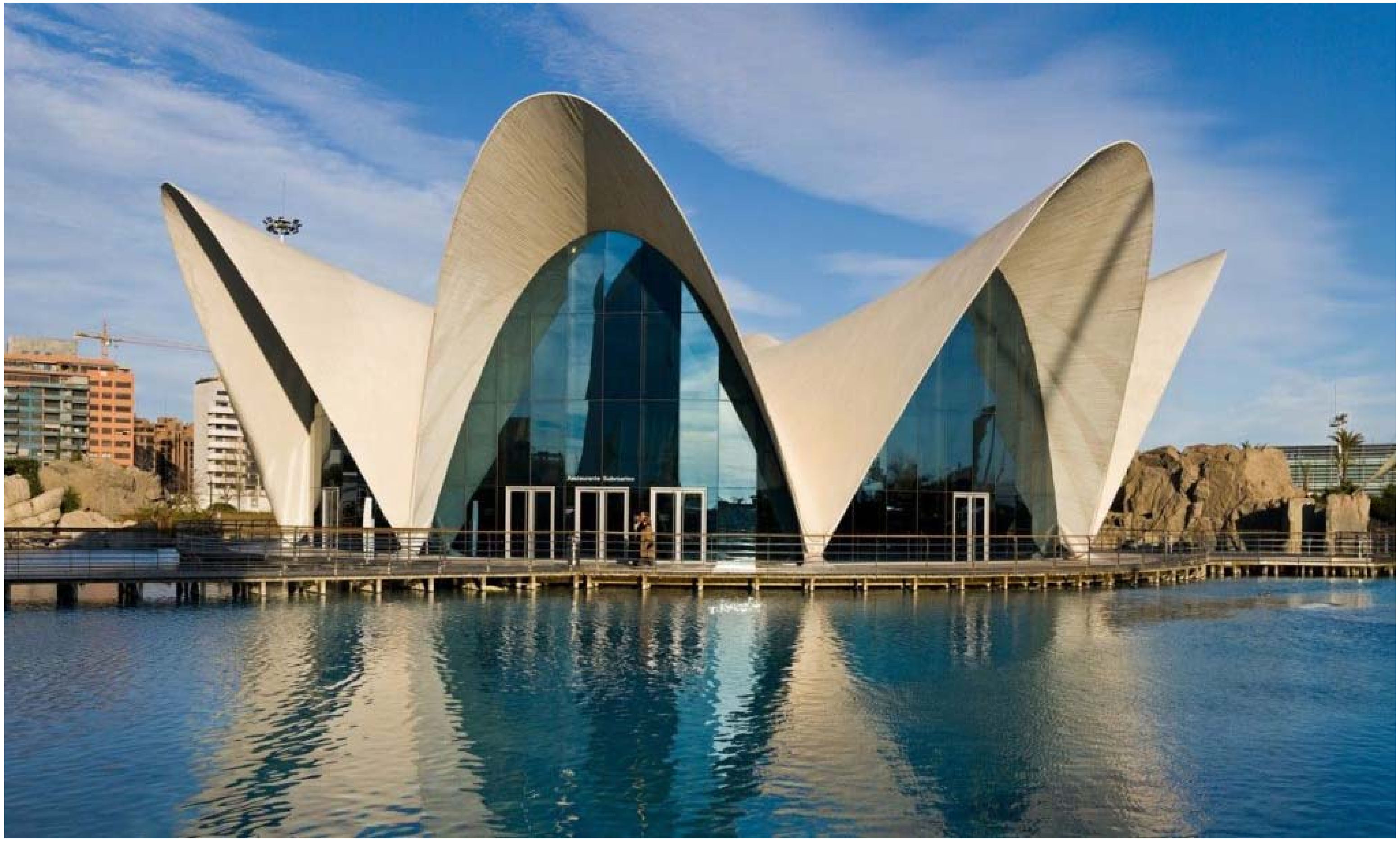
Fig. 302

equazione cartesiana: $$z = \frac{x^2}{a^2} - \frac{y^2}{b^2}.$$

Sella

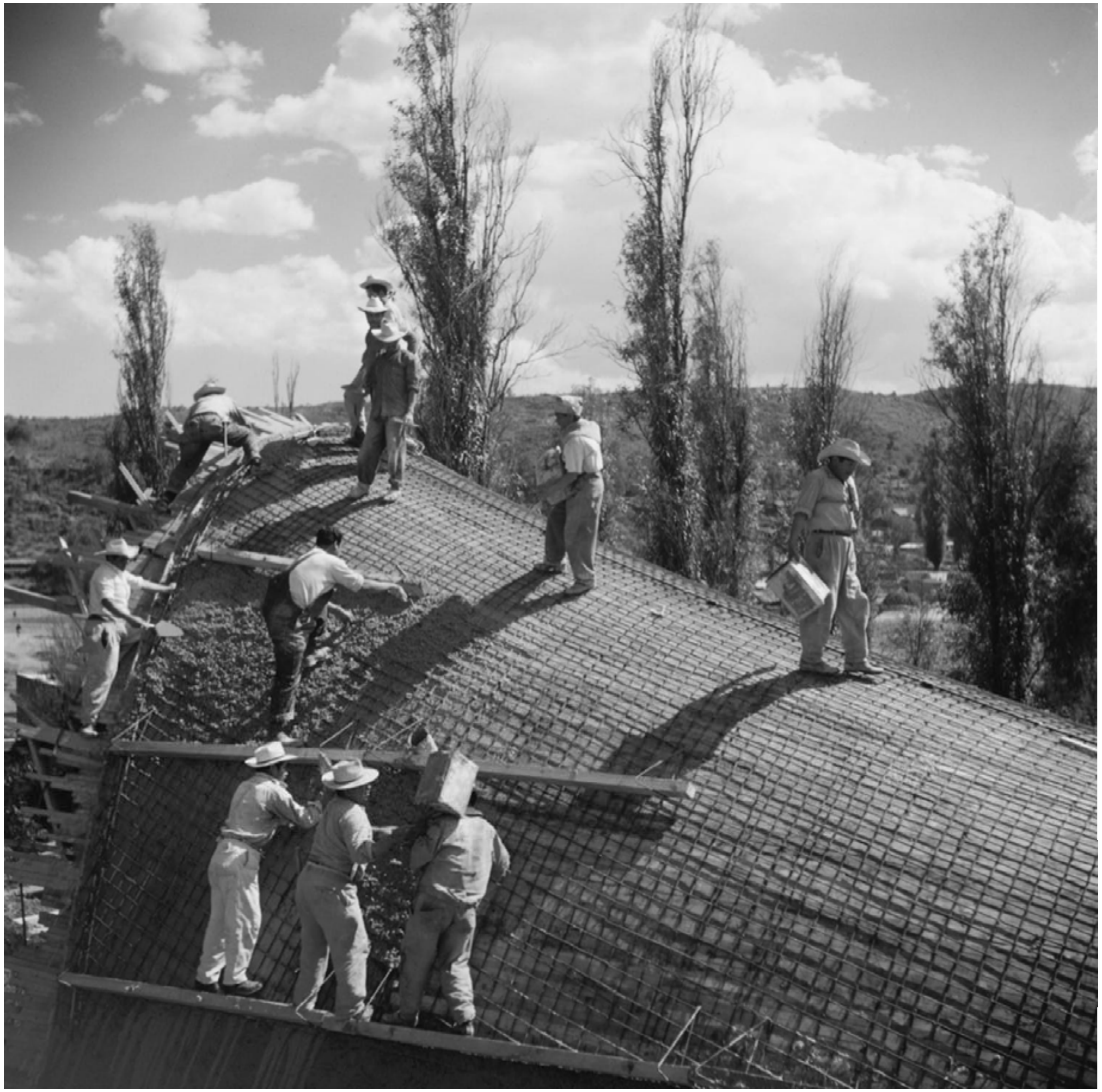


Fèlix Candela

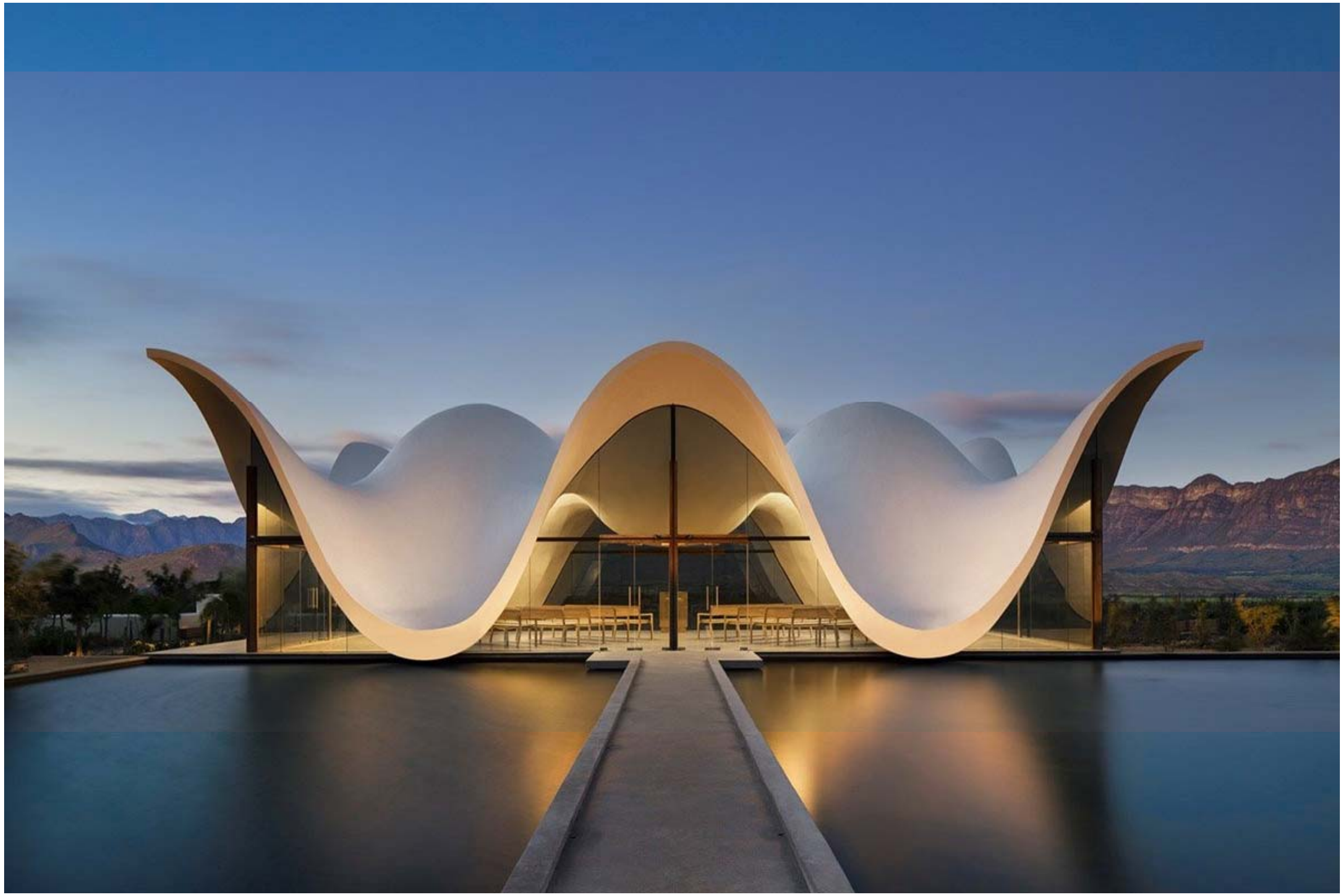


Parco oceanografico di Valencia

Fèlix Candela - fasi del getto di un guscio

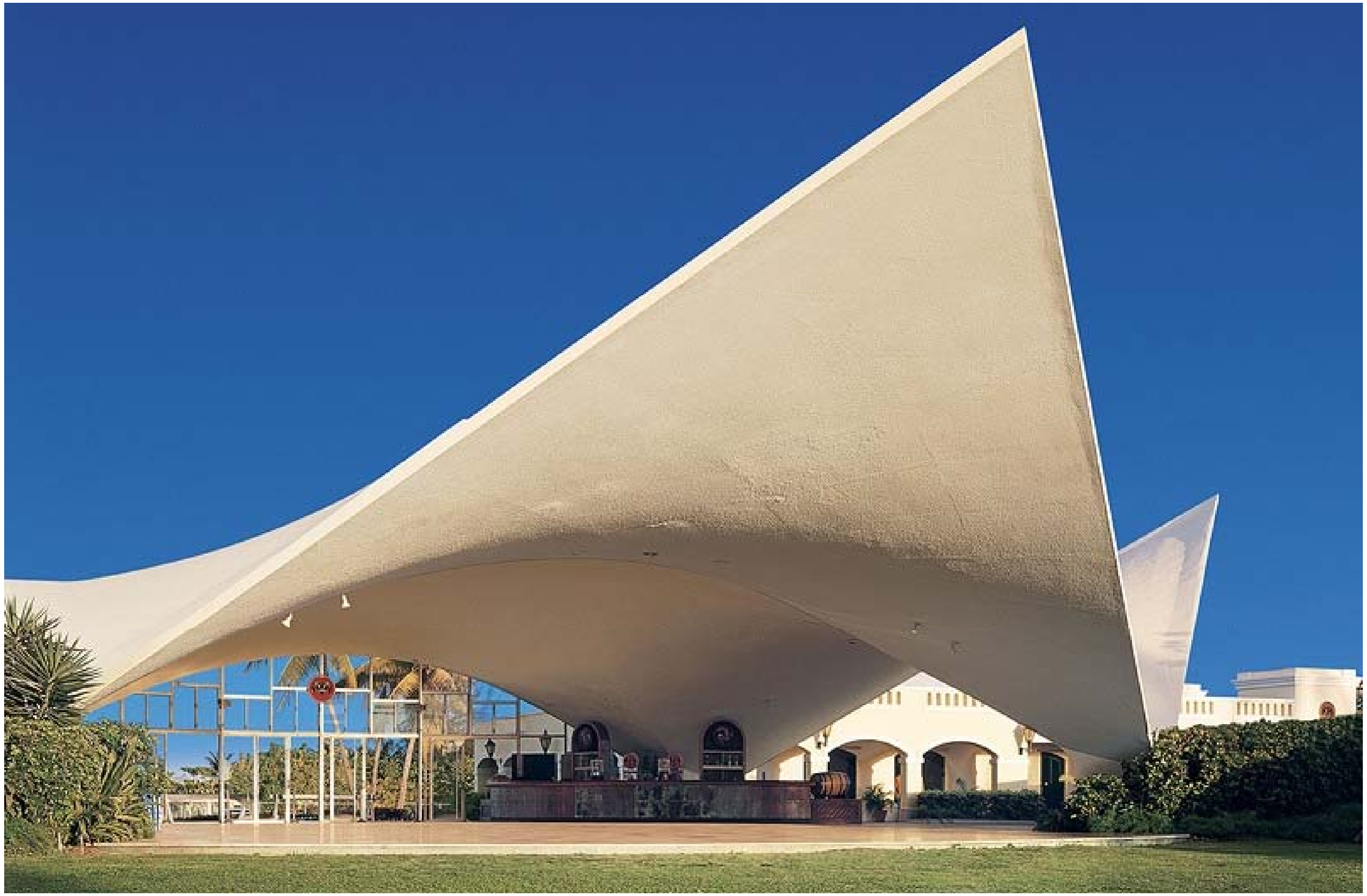


Fèlix Candela



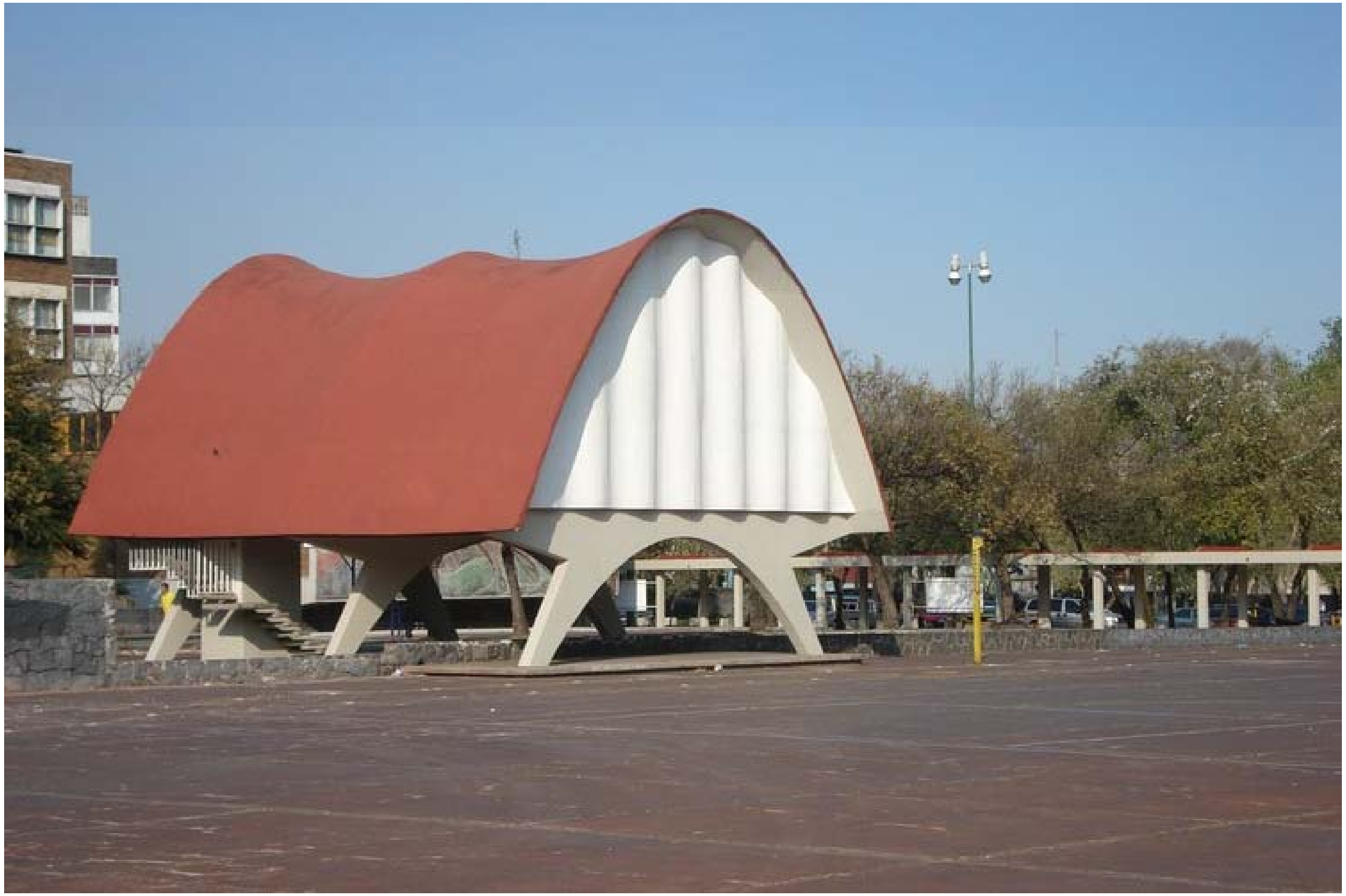
Chiesa in Sudafrica

Fèlix Candela



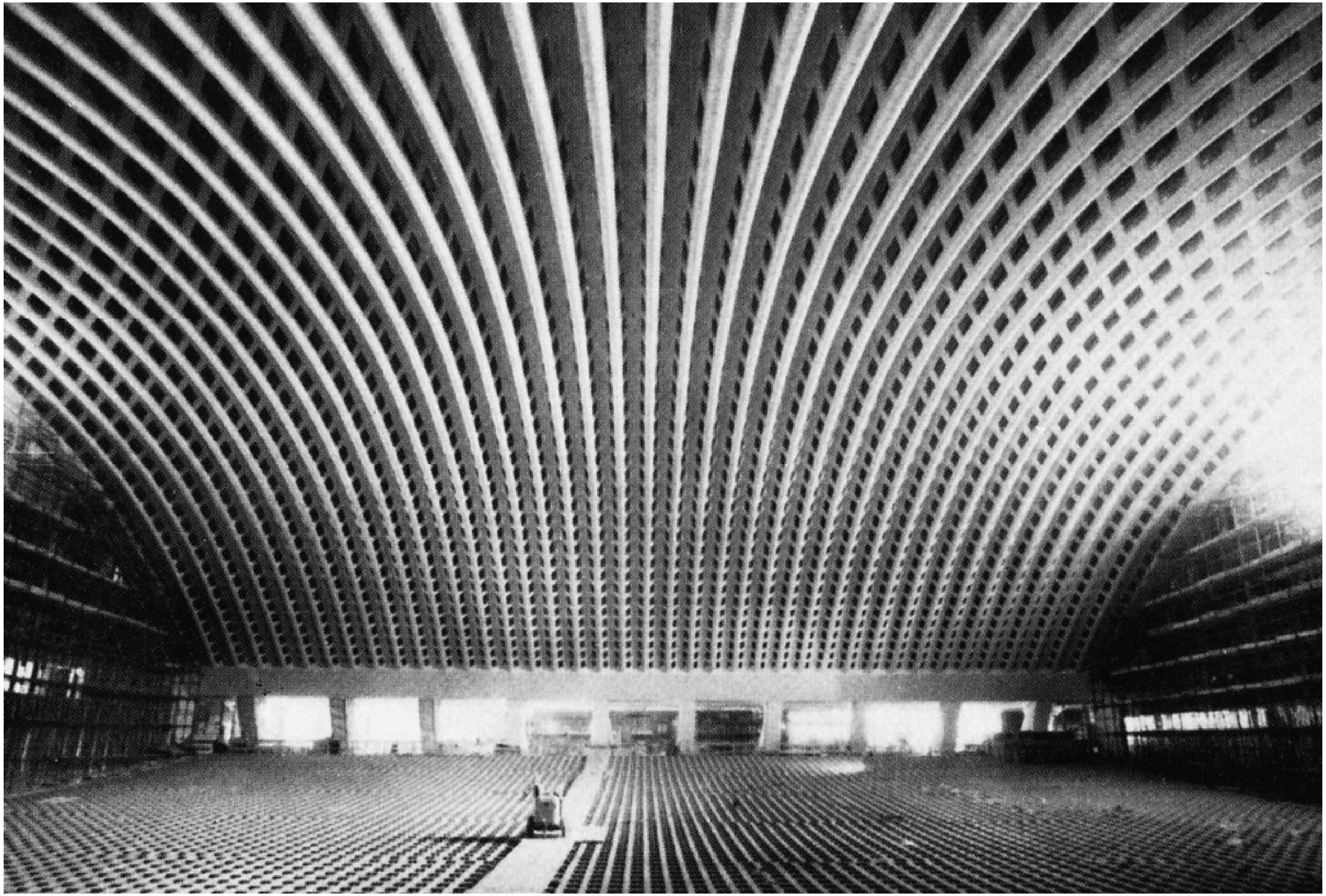
Bacardi Visitor center – Porto Rico

Fèlix Candela



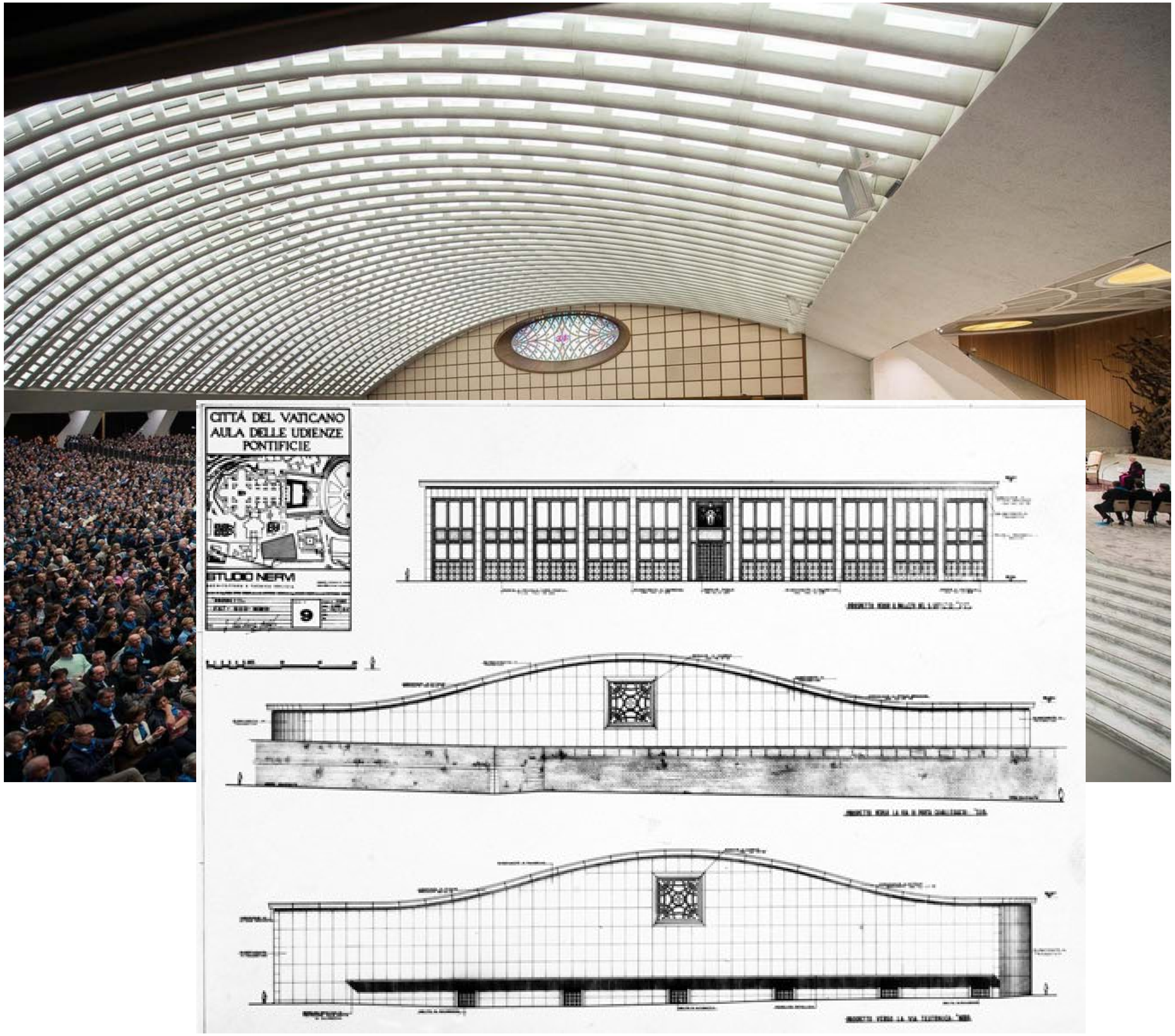
Cosmic Rays Laboratory – Città del Messico

Pierluigi Nervi



Aula delle Udienze Pontifice – Vaticano 1971

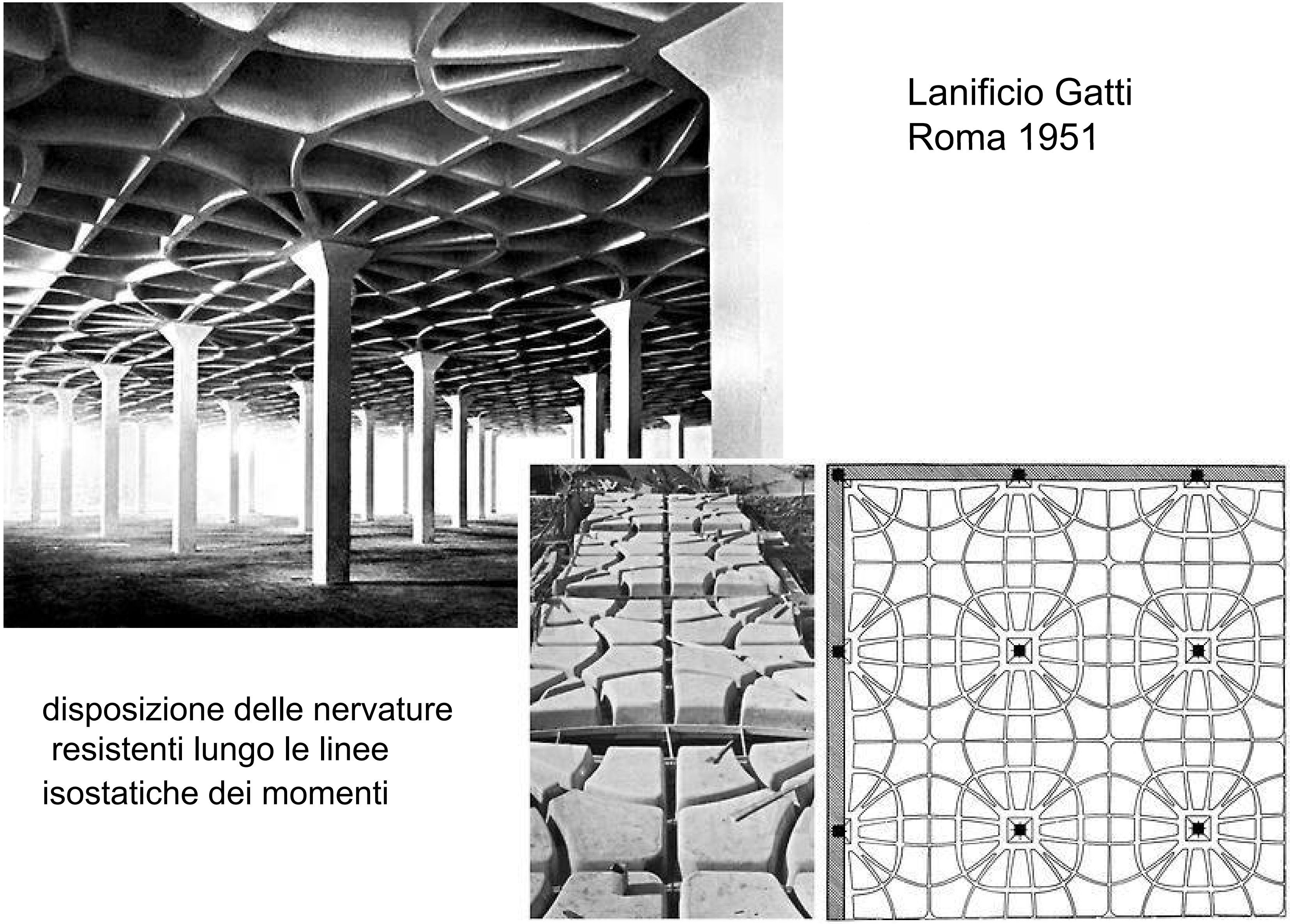
Pierluigi Nervi


CITTÁ DEL VATICANO
AULA DELLE UDIENZE
PONTIFICIE
STUDIO NERVI
9

Aula delle Udienze Pontificie – Vaticano 1971

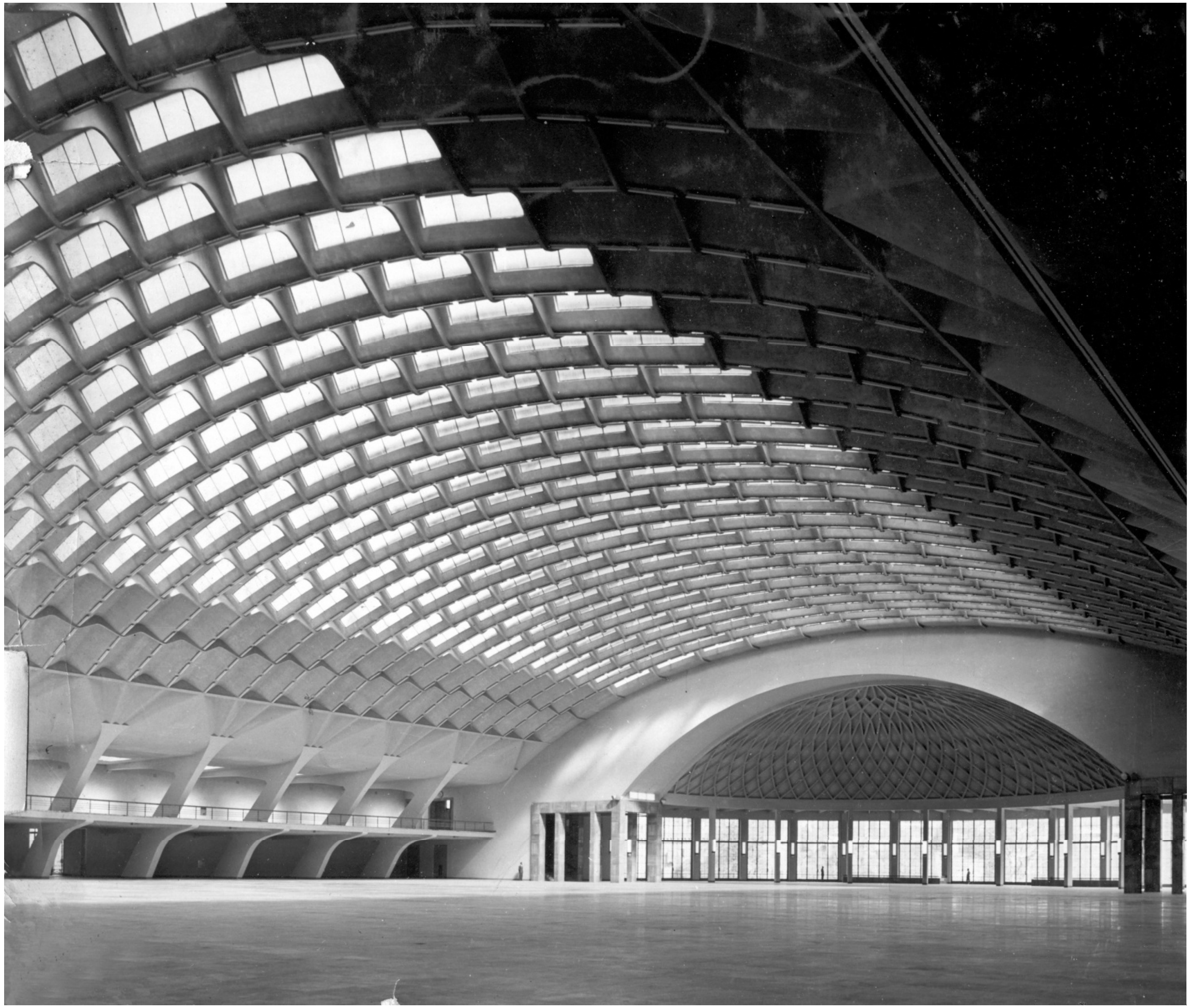
Pierluigi Nervi

Lanificio Gatti
Roma 1951



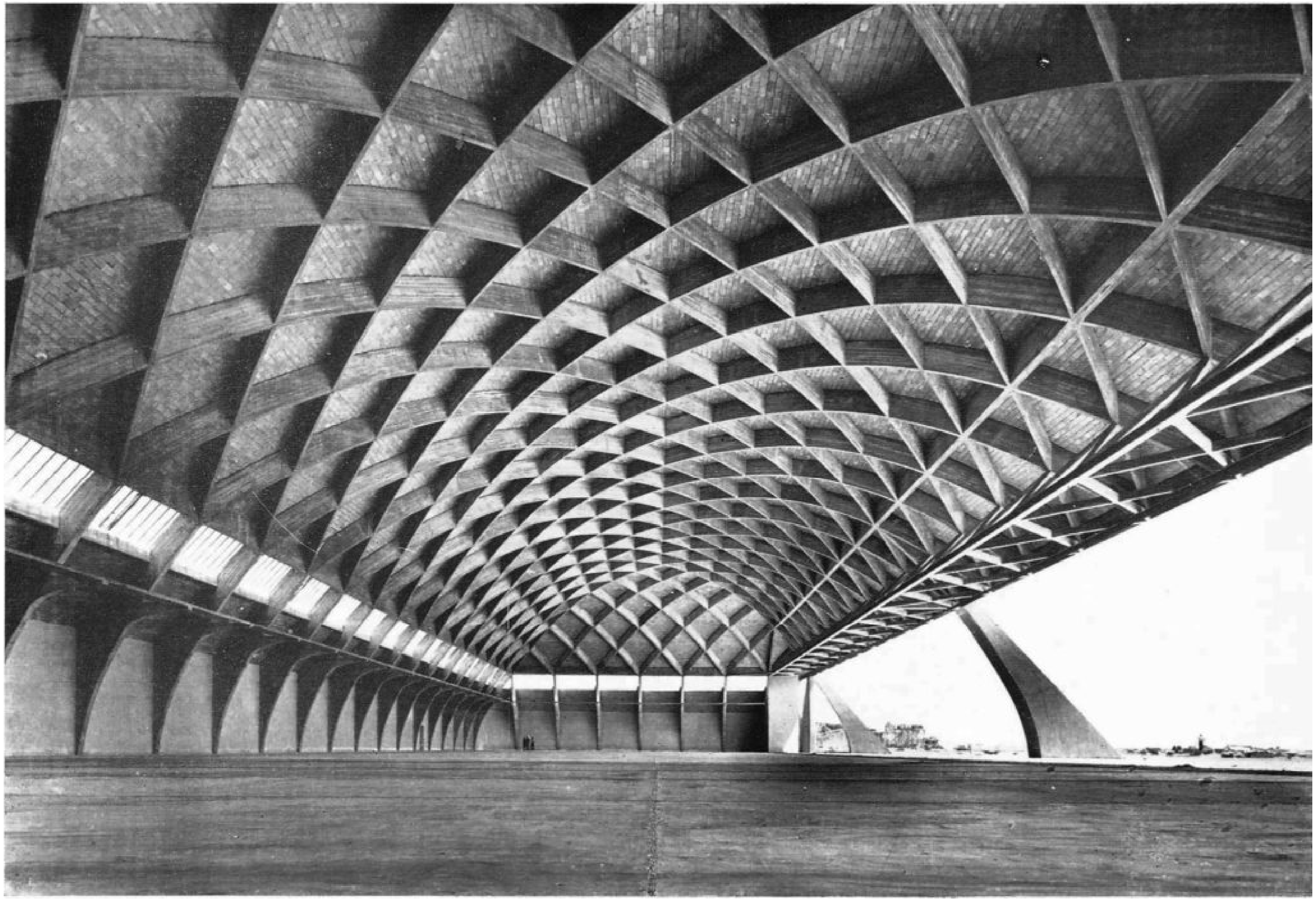
disposizione delle nervature
resistenti lungo le linee
isostatiche dei momenti

Pierluigi Nervi



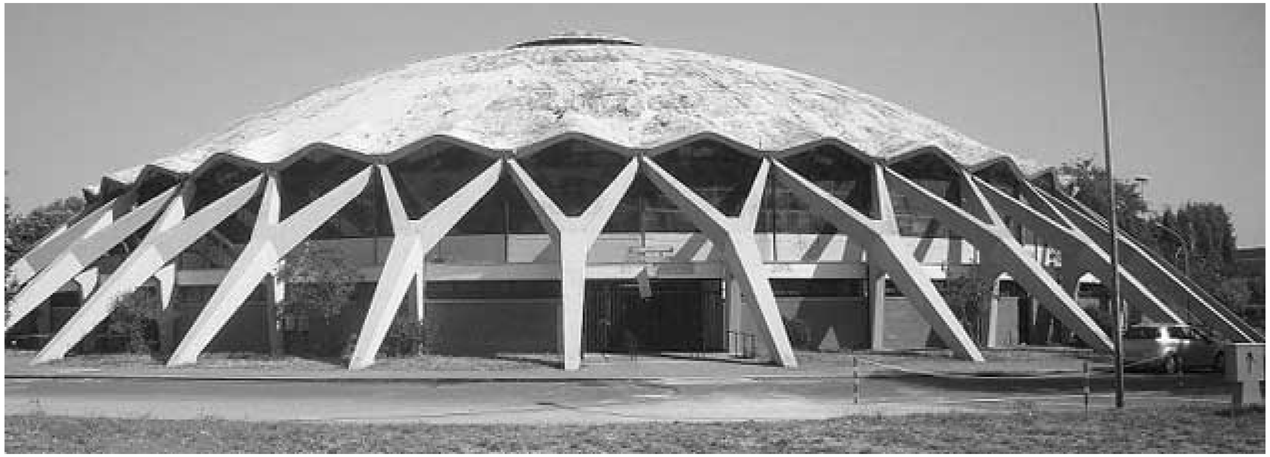
Salone per Torino Esposizioni 1949

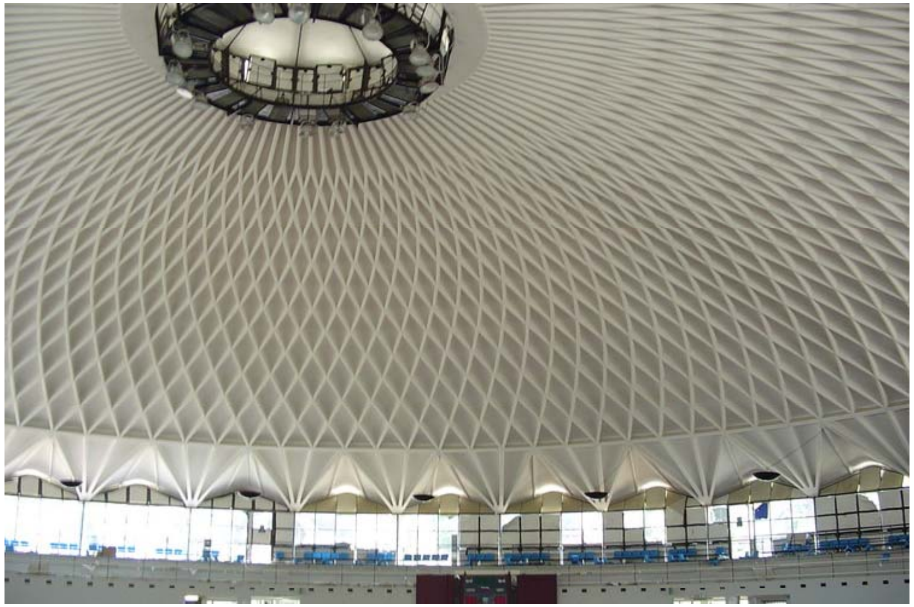
Pierluigi Nervi



Hangar - Orvieto 1935

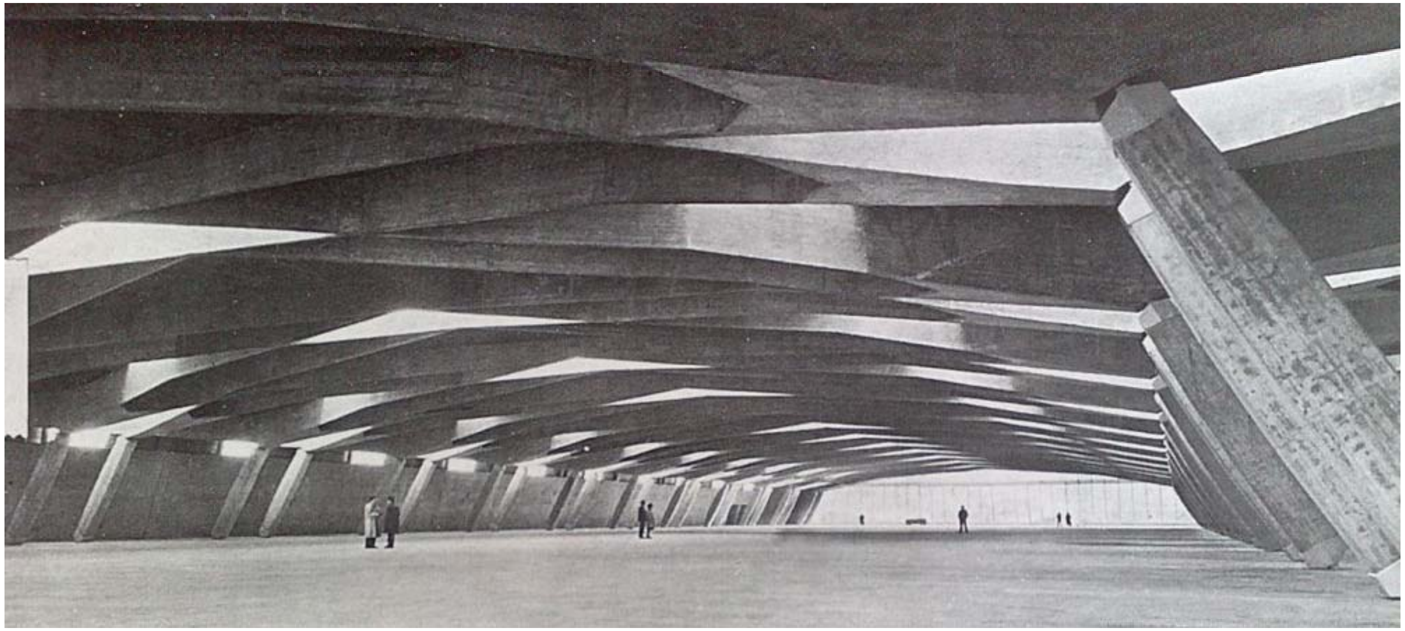
Pierluigi Nervi

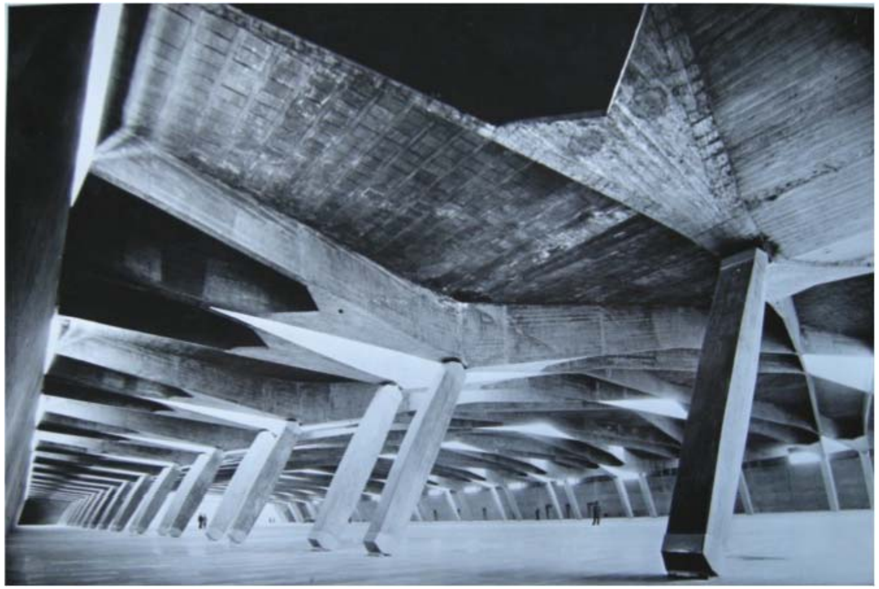




Palazzetto dello Sport – Roma 1957

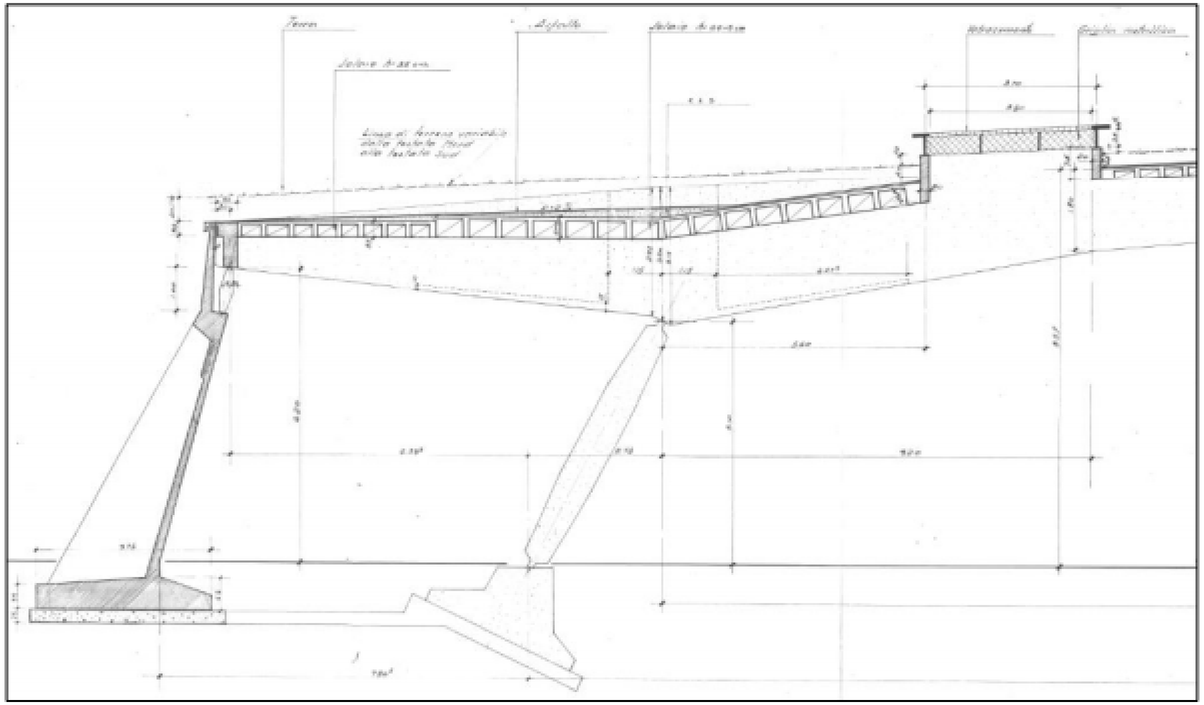
Riccardo Morandi





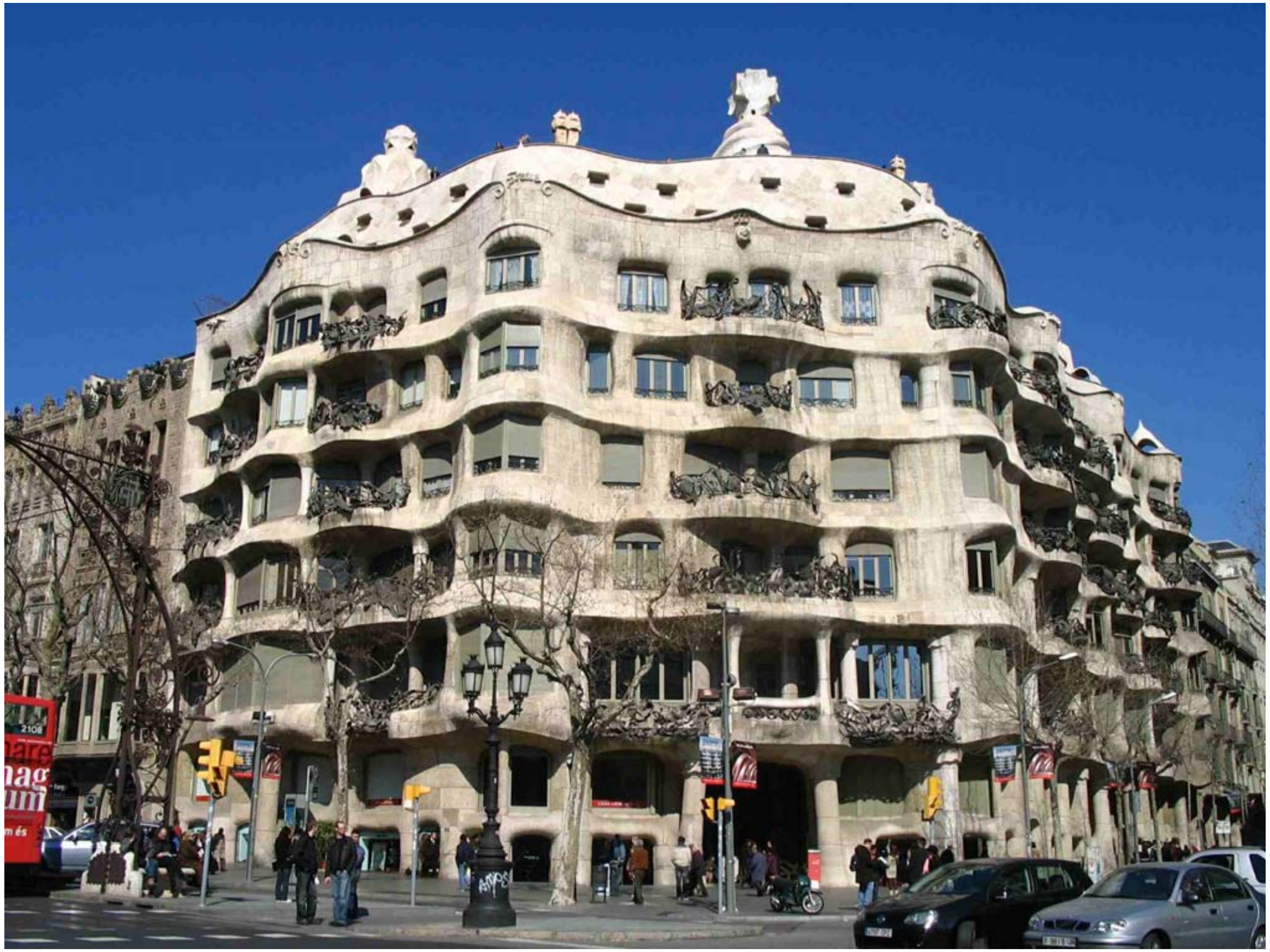
Padiglione sotterraneo Esposizione Internazionale – Torino 1961

Riccardo Morandi


Terra
Asfalto
Vetrocemento
Griglia metallica

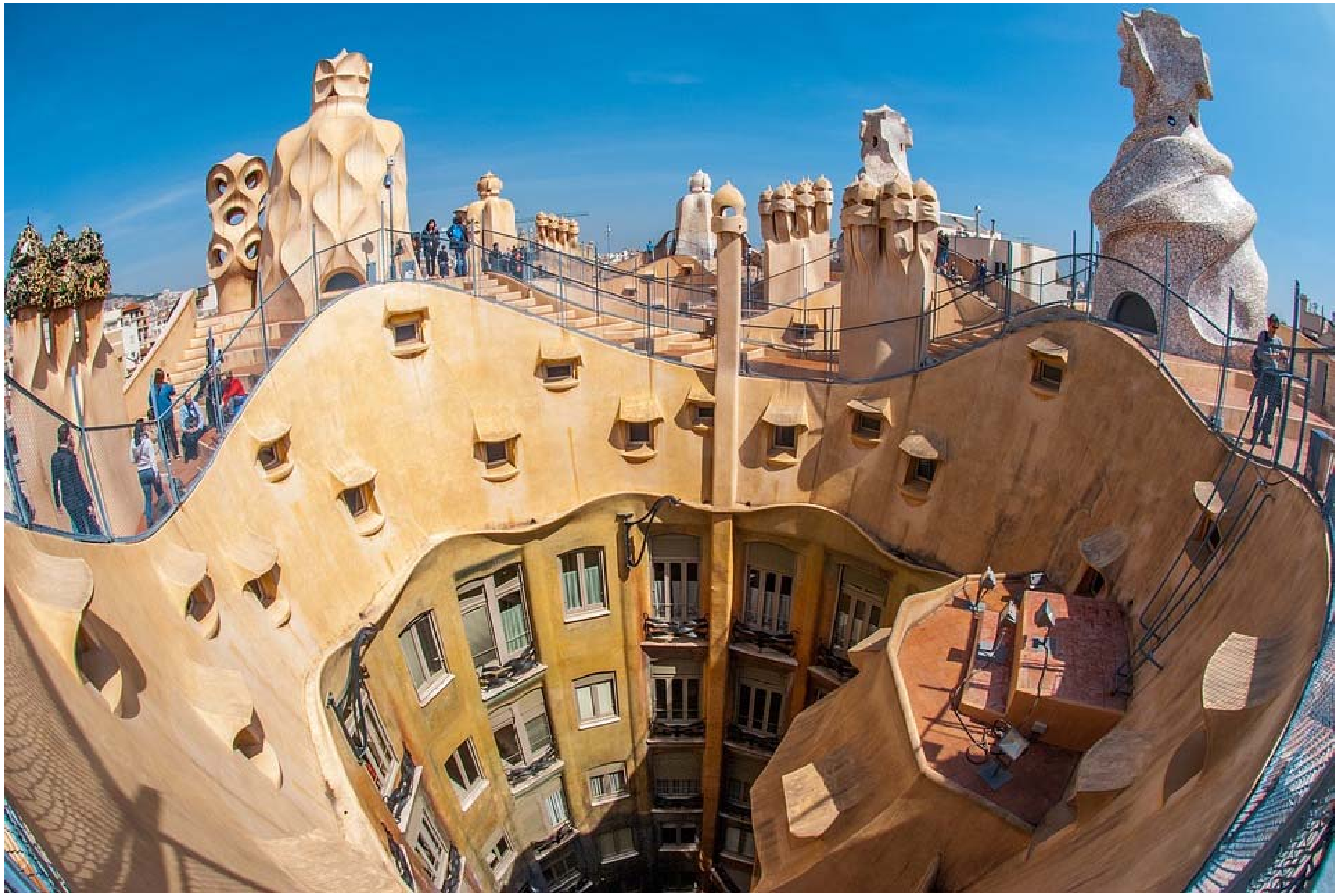
Padiglione sotterraneo Esposizione Internazionale – Torino 1961

Gaudì



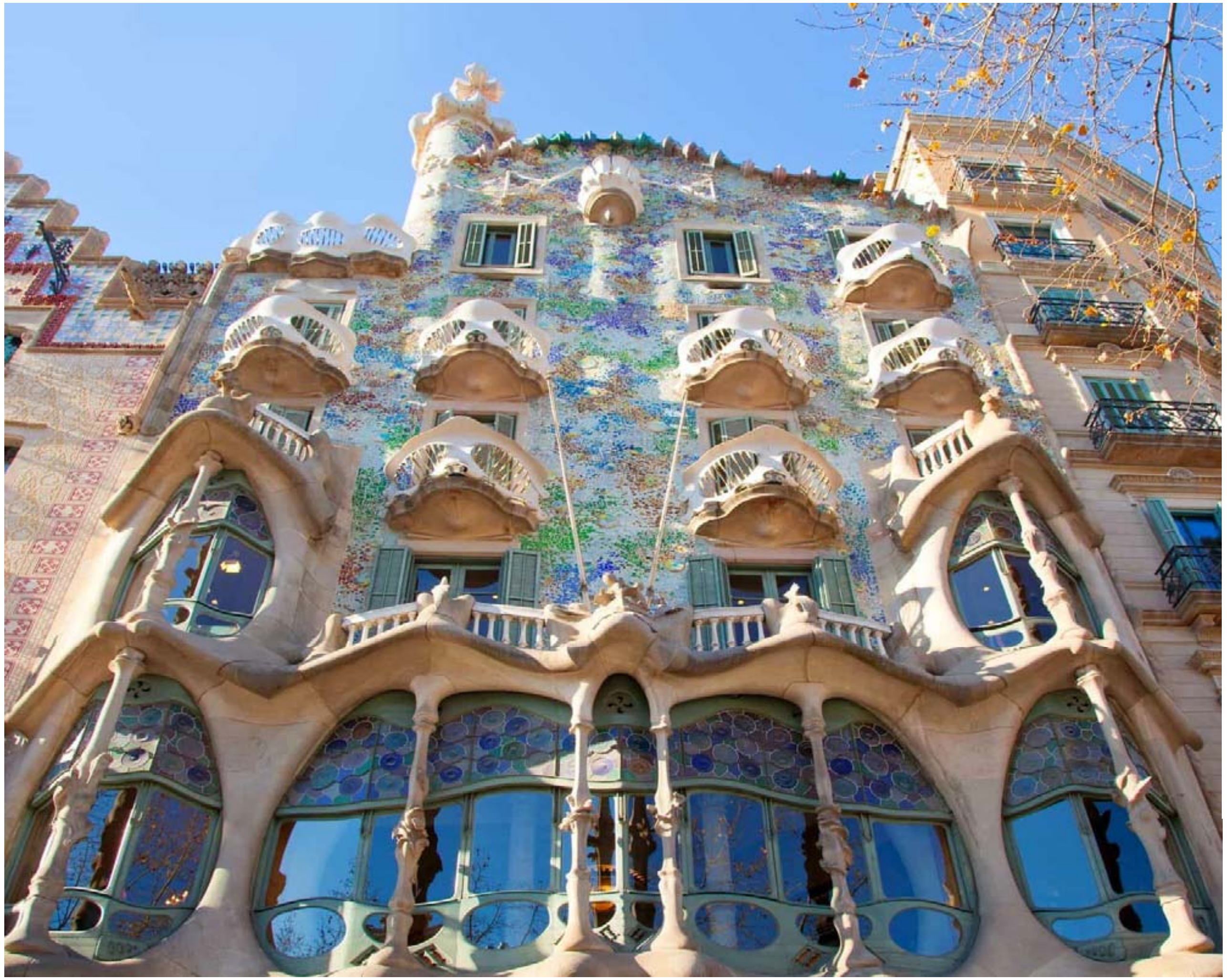
Casa Milà – La Pedrera 1912

Gaudì



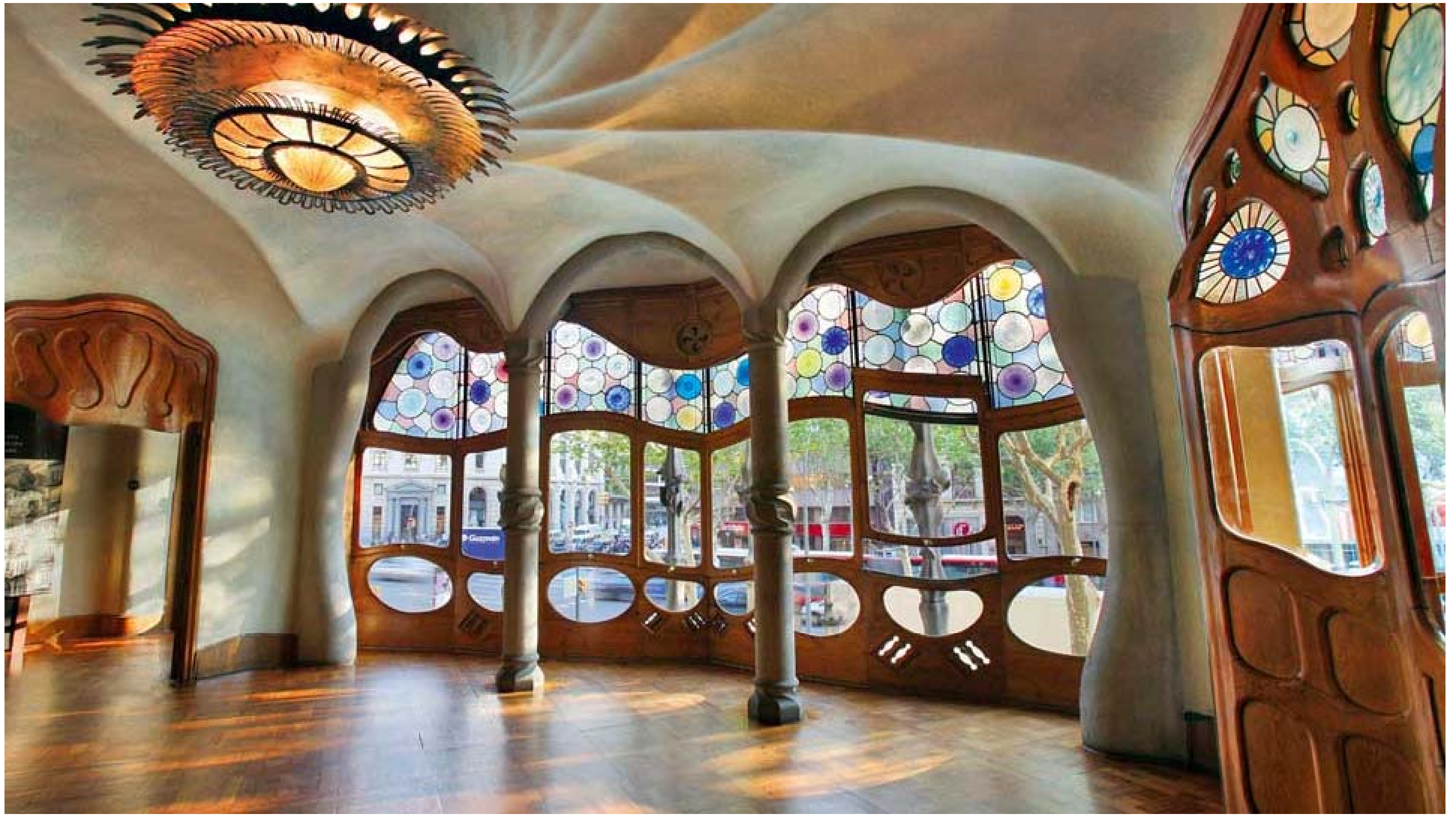
Casa Milà – La Pedrera 1912

Gaudì



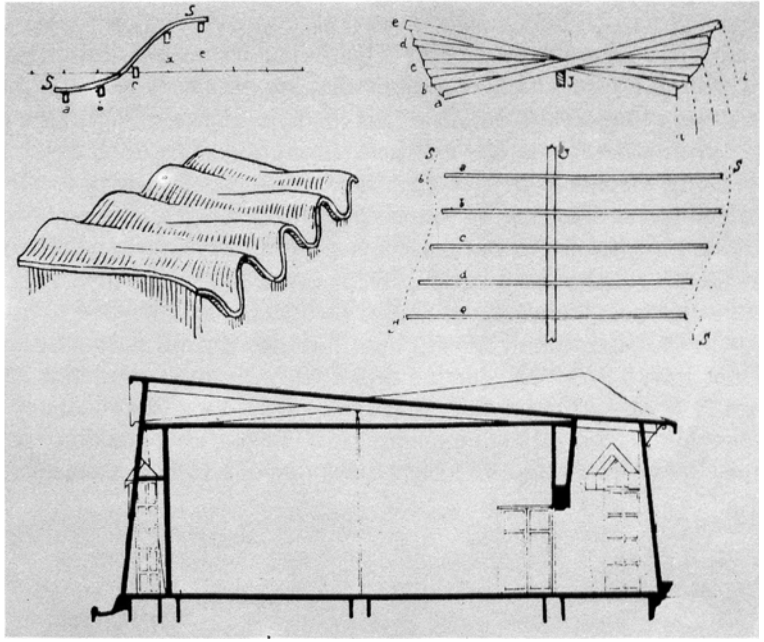
Casa Batllò – Barcellona 1907

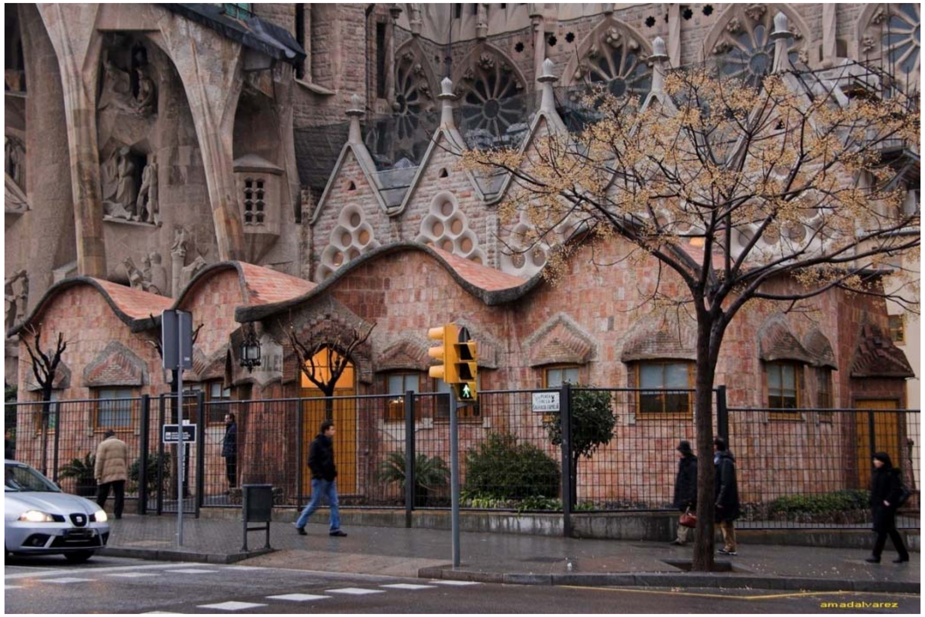
Gaudì

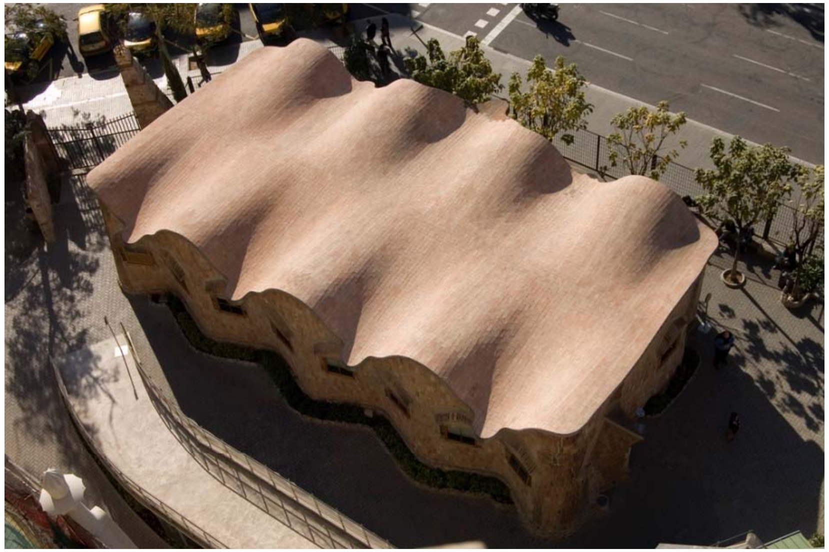


Casa Batllò – Barcellona 1907

Gaudì

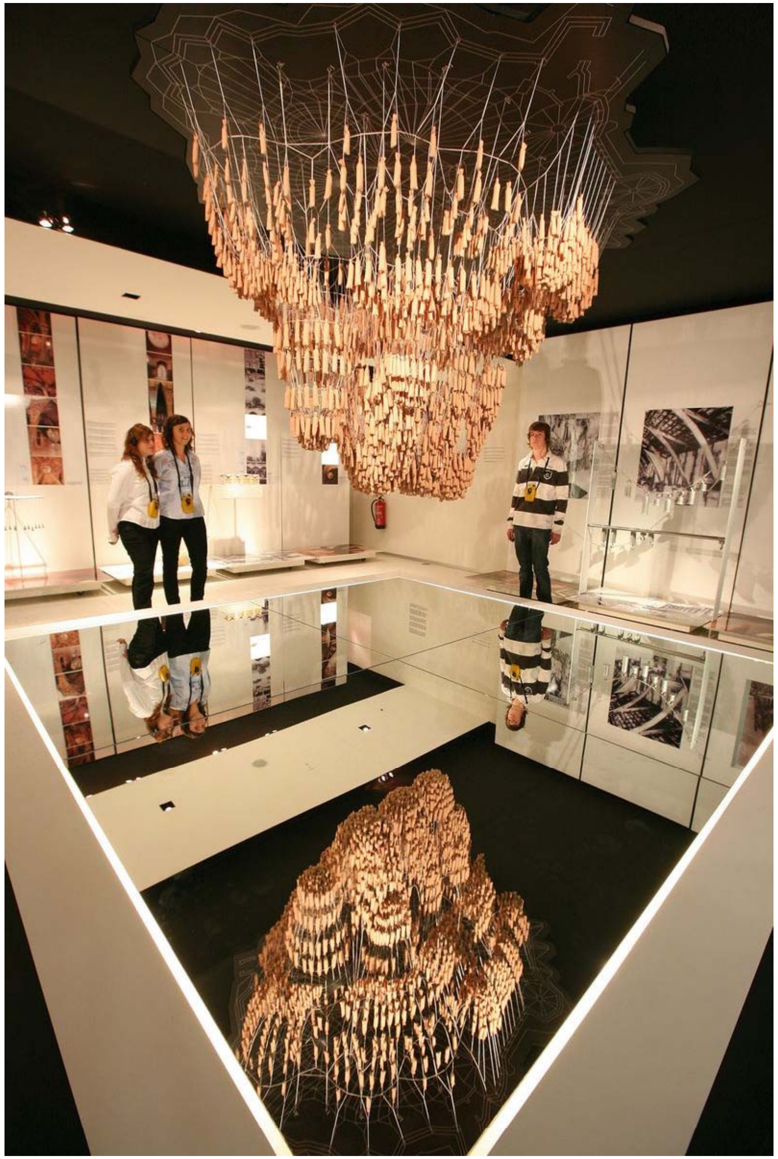


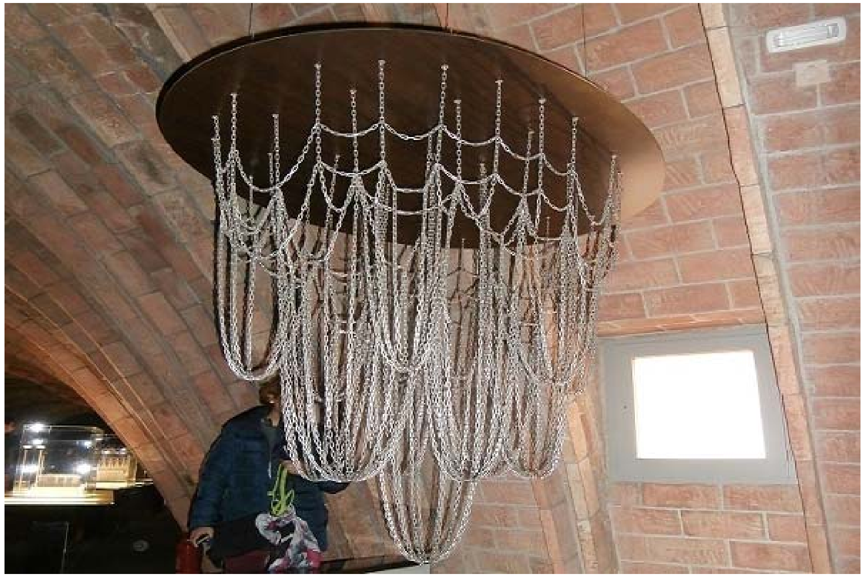

amadalvarez

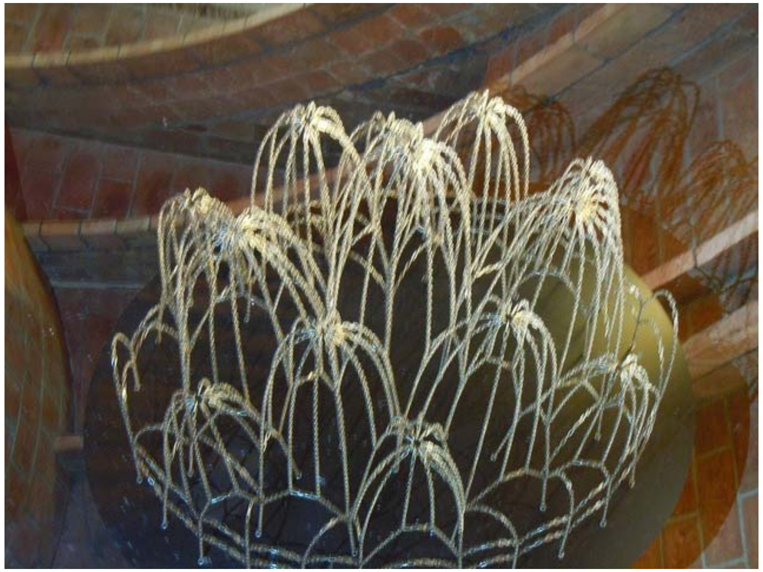


Escoles de Gaudì – Barcellona

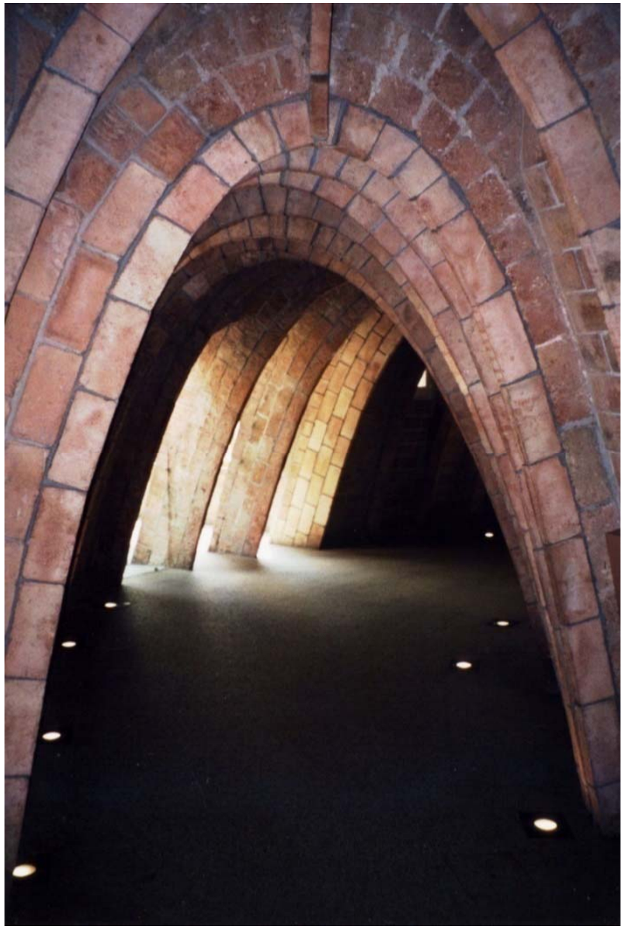
Gaudì – studio delle catenarie

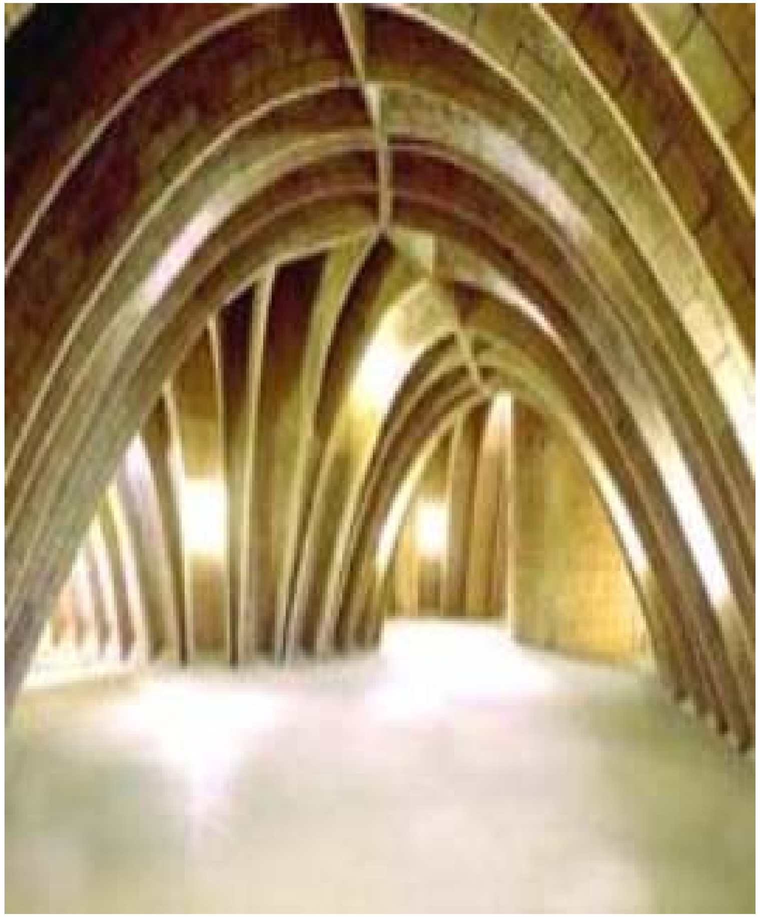




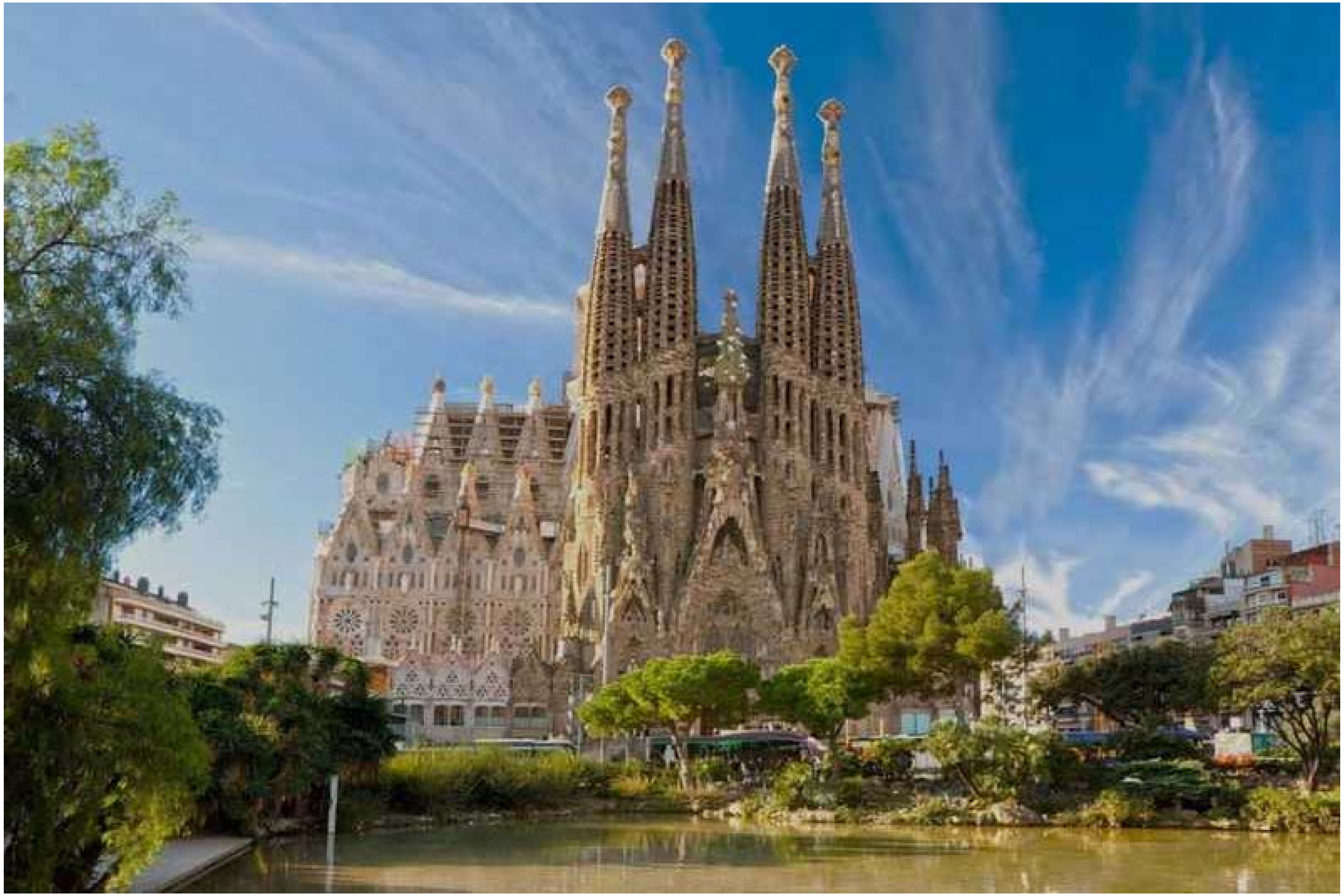


Gaudì – studio delle catenarie



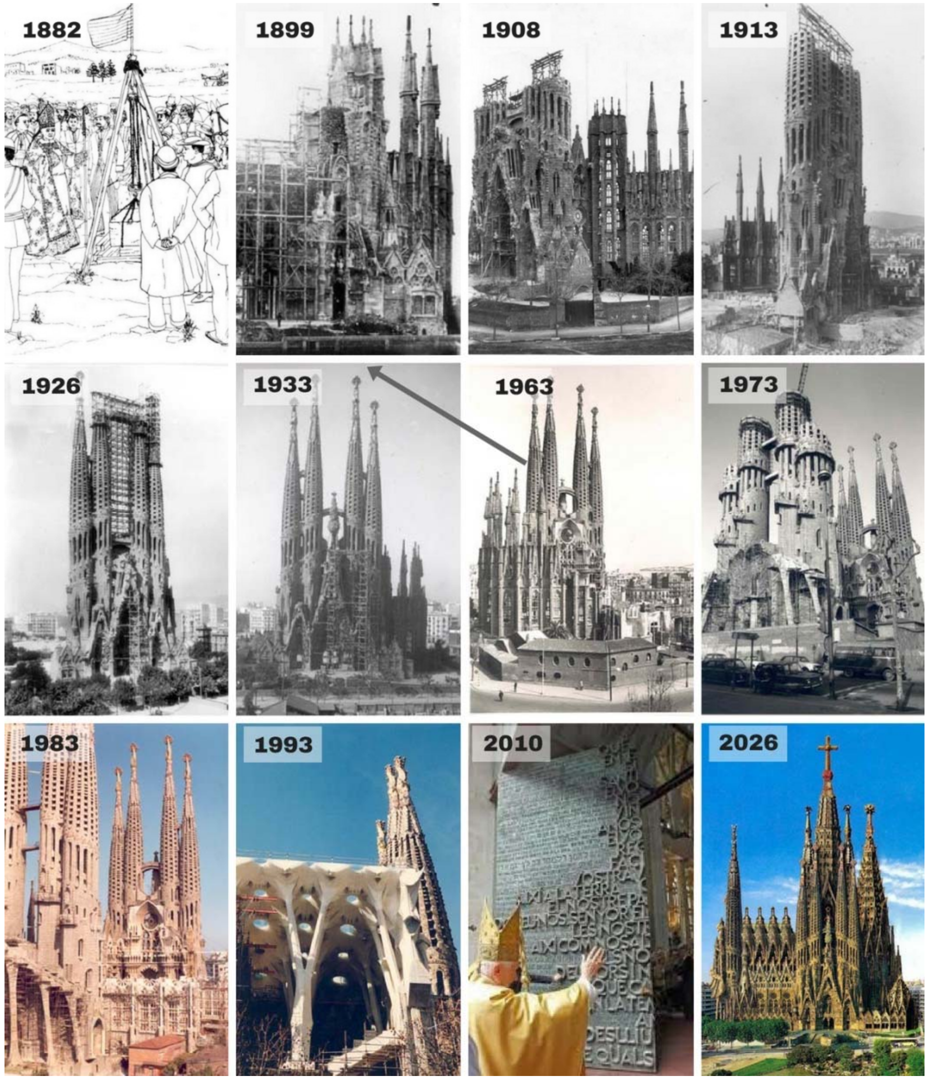


Gaudì



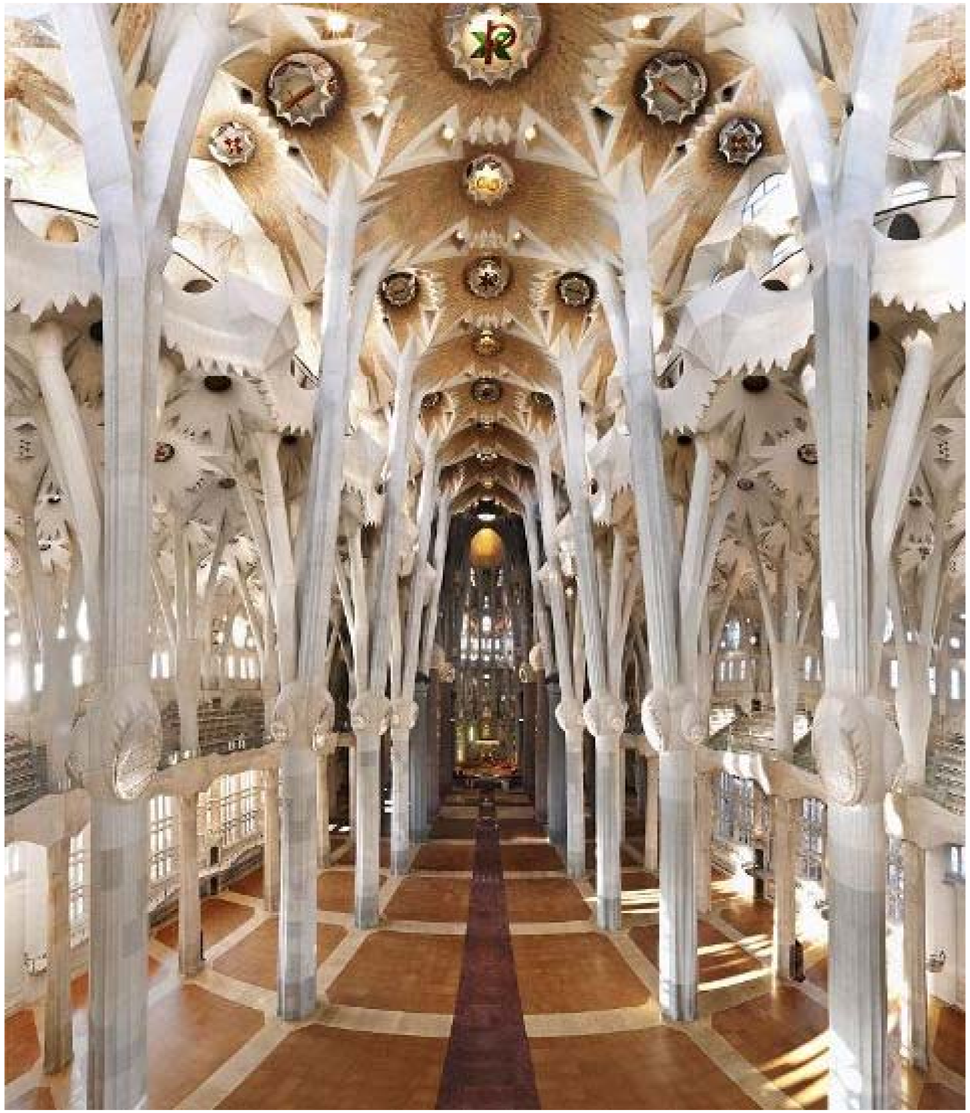
Sagrada Familia

Gaudì


1882
1899
1908
1913
1926
1933
1963
1973
1983
1993
2010
2026

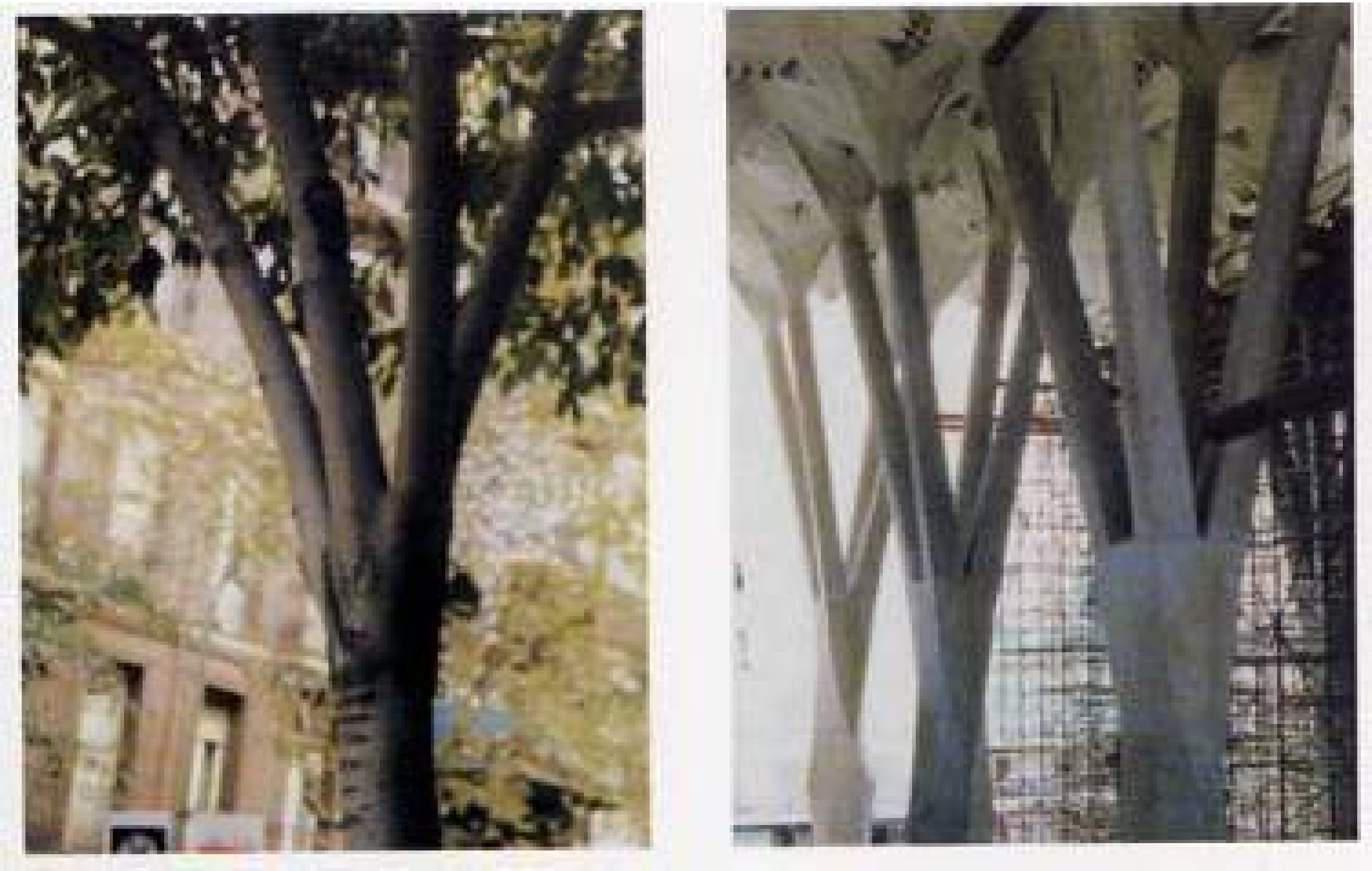
Sagrada Familia– Barcellona

Gaudì



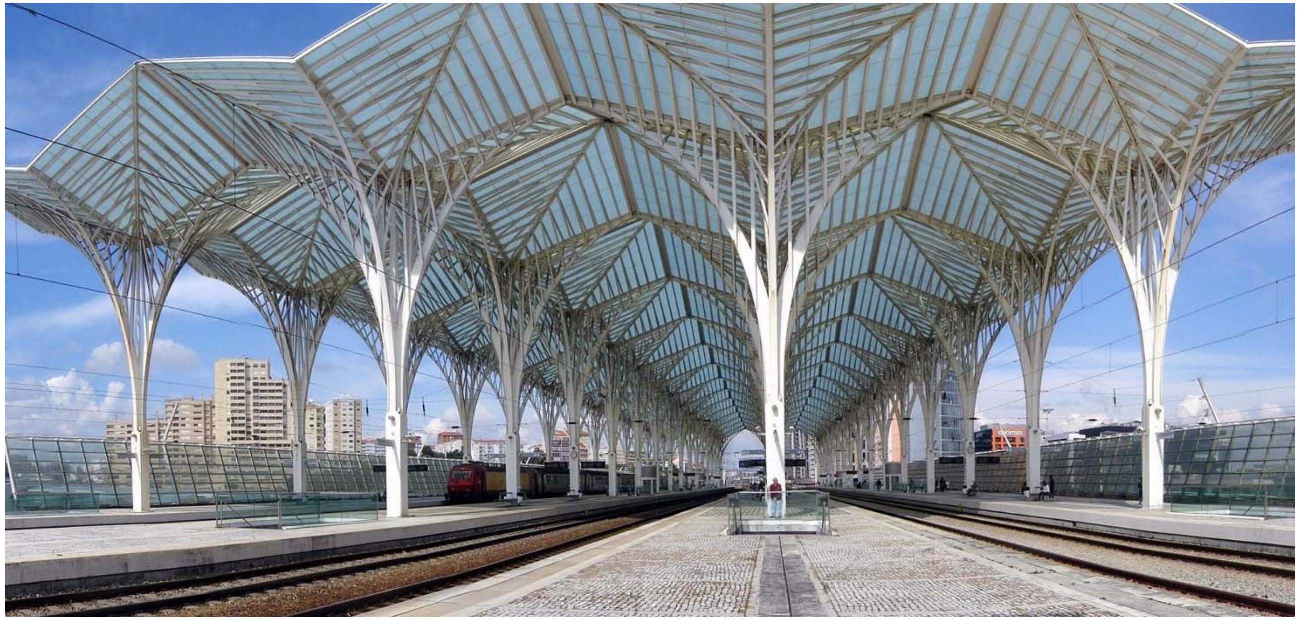
Sagrada Familia– Barcellona

Gaudì



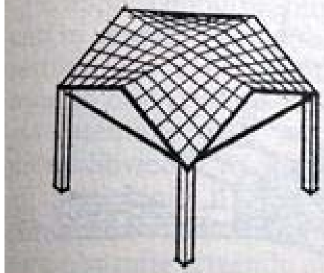
Sagrada Familia– Barcellona

Santiago Calatrava

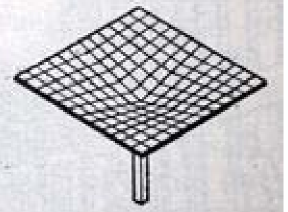


Stazione di Lisbona 1998

Ombrello Hypar (Paraboloide iperbolico)

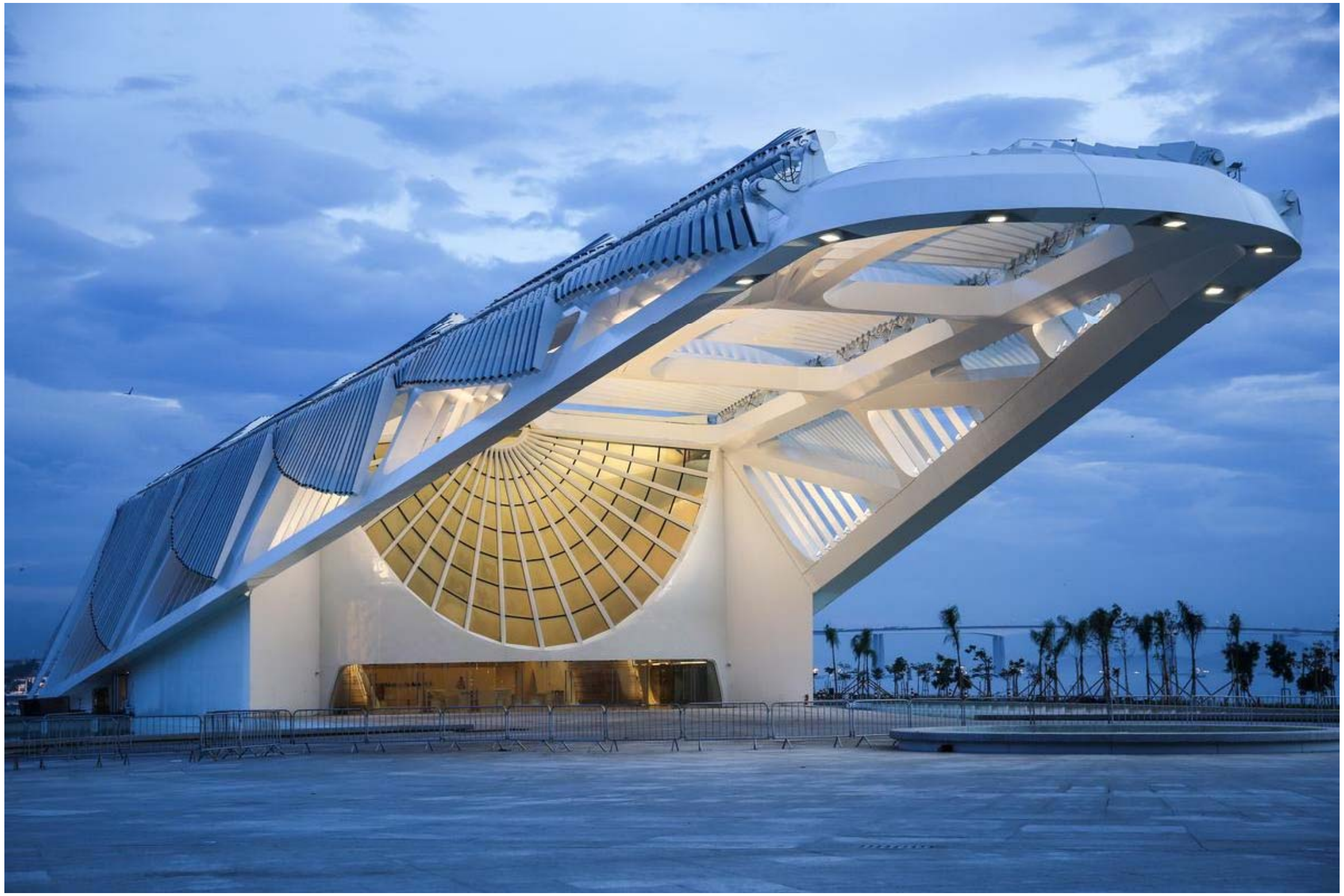


11.28A COPERTURA HYPAR



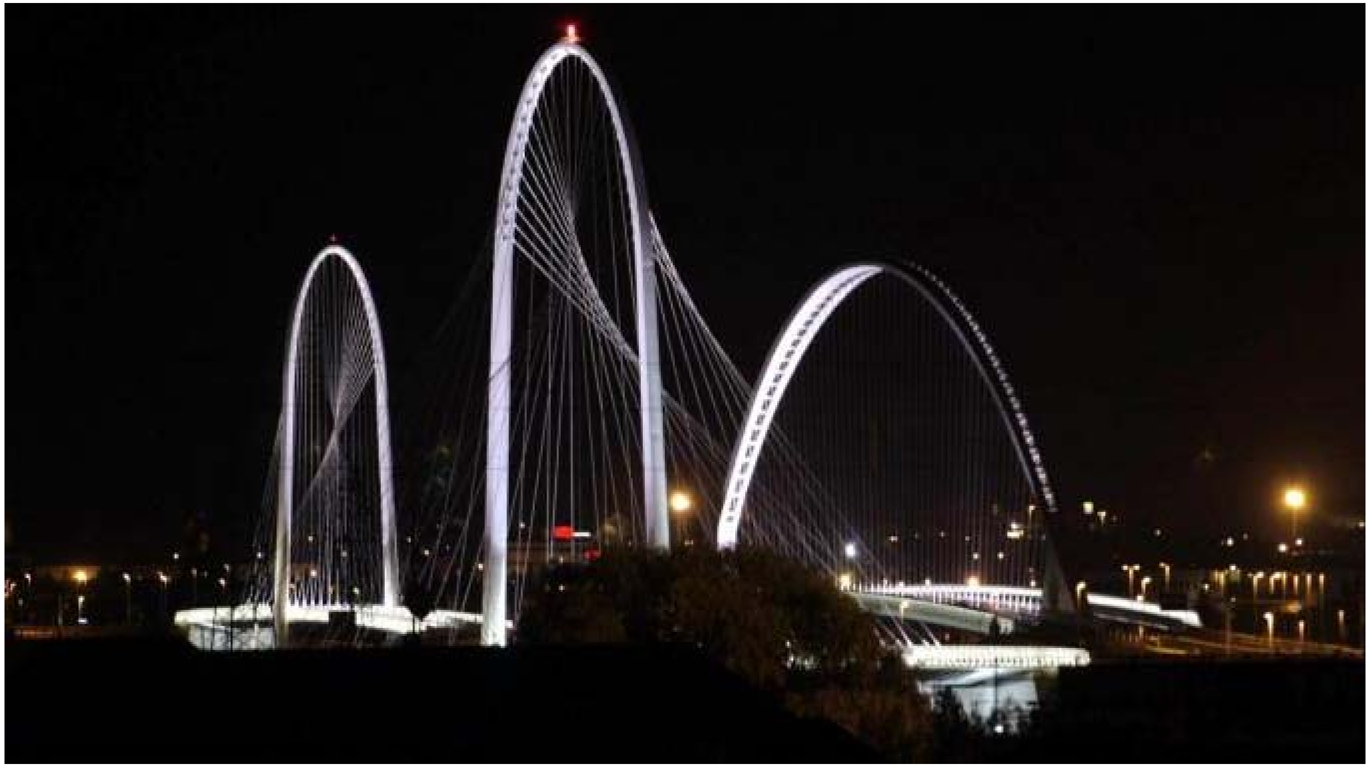
11.28B OMBRELLO HYPAR

Santiago Calatrava



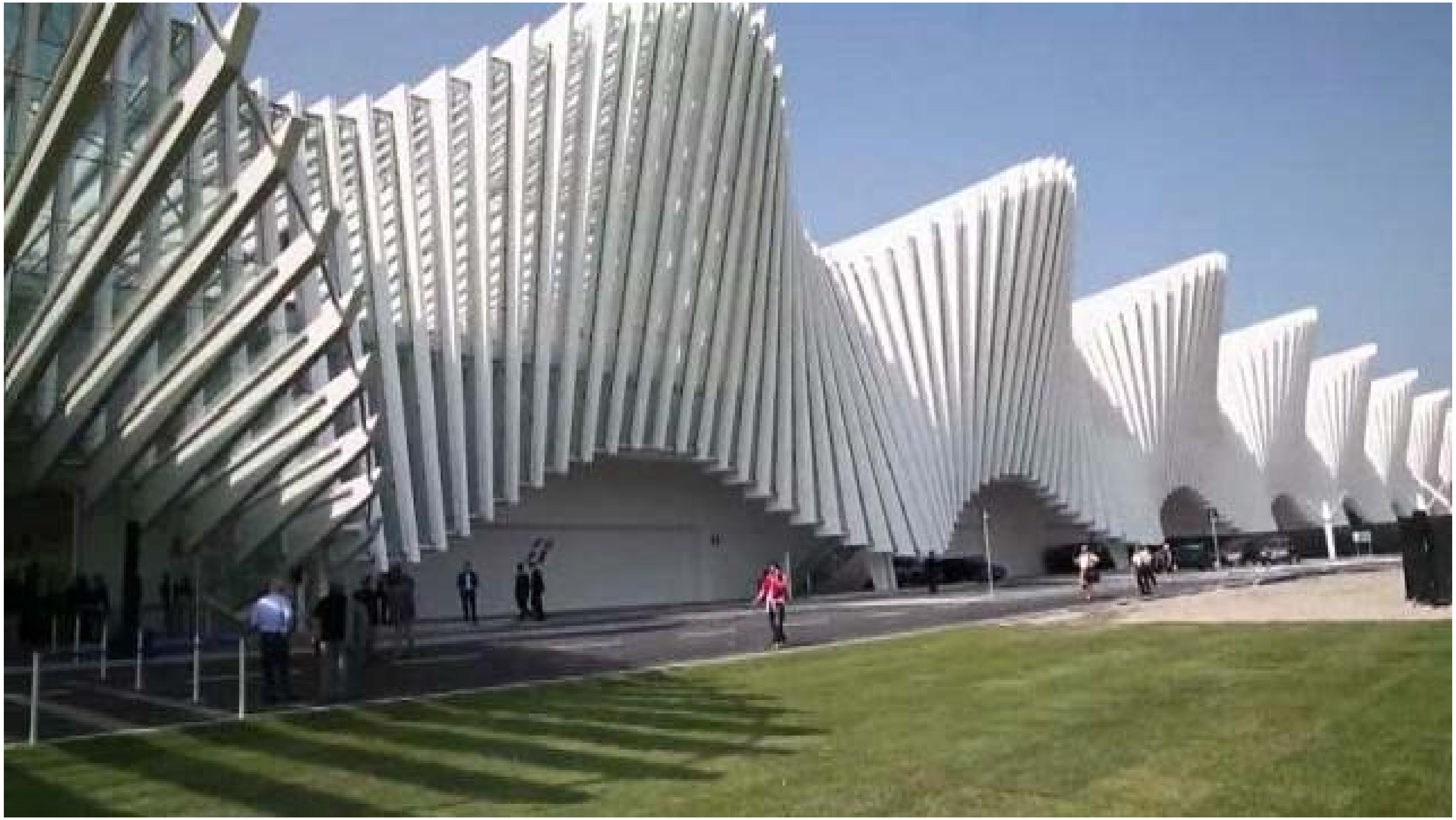
Museum of tomorrow Rio 2015

Santiago Calatrava



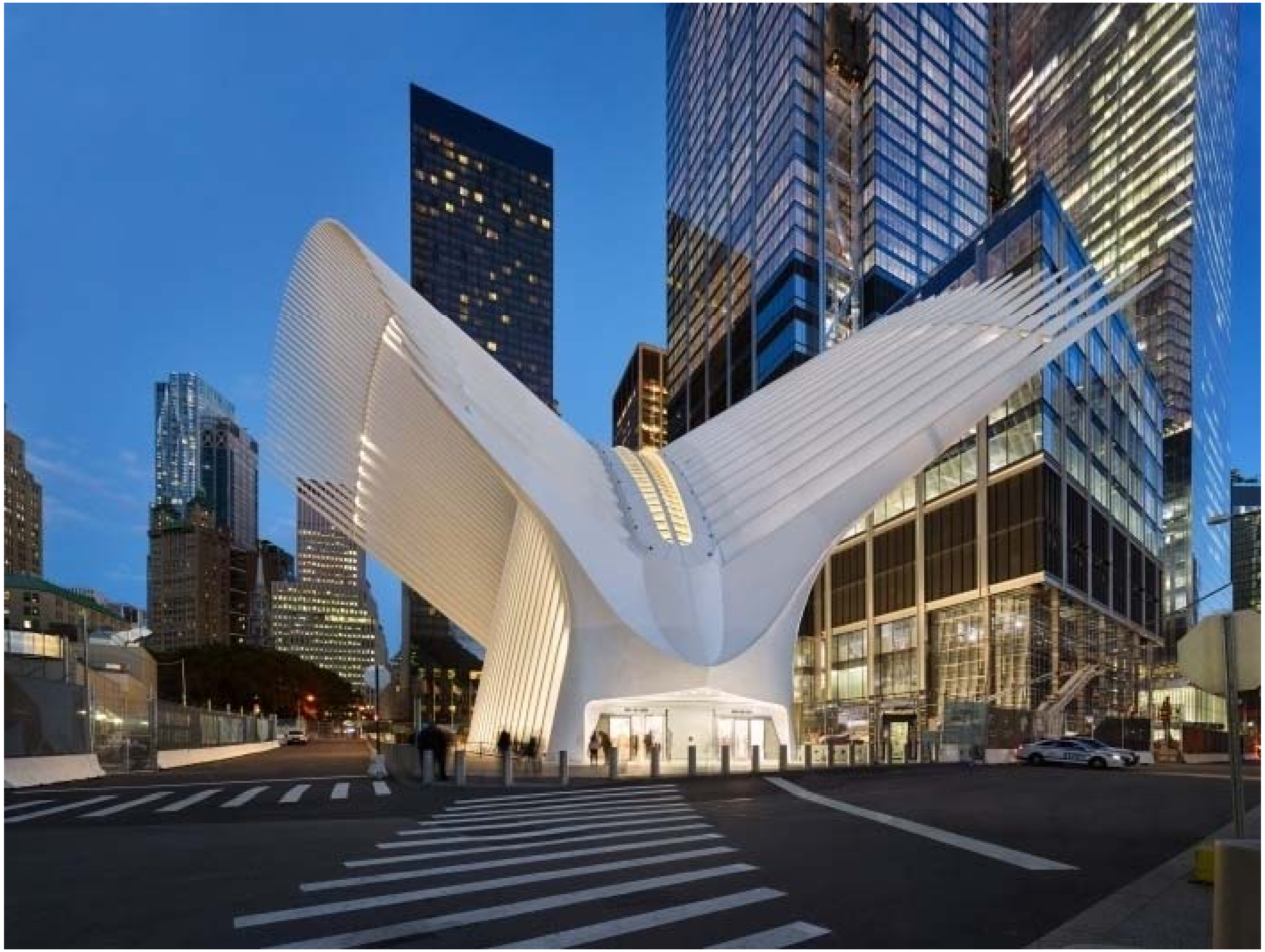
Ponti in Emilia Romagna 2007

Santiago Calatrava



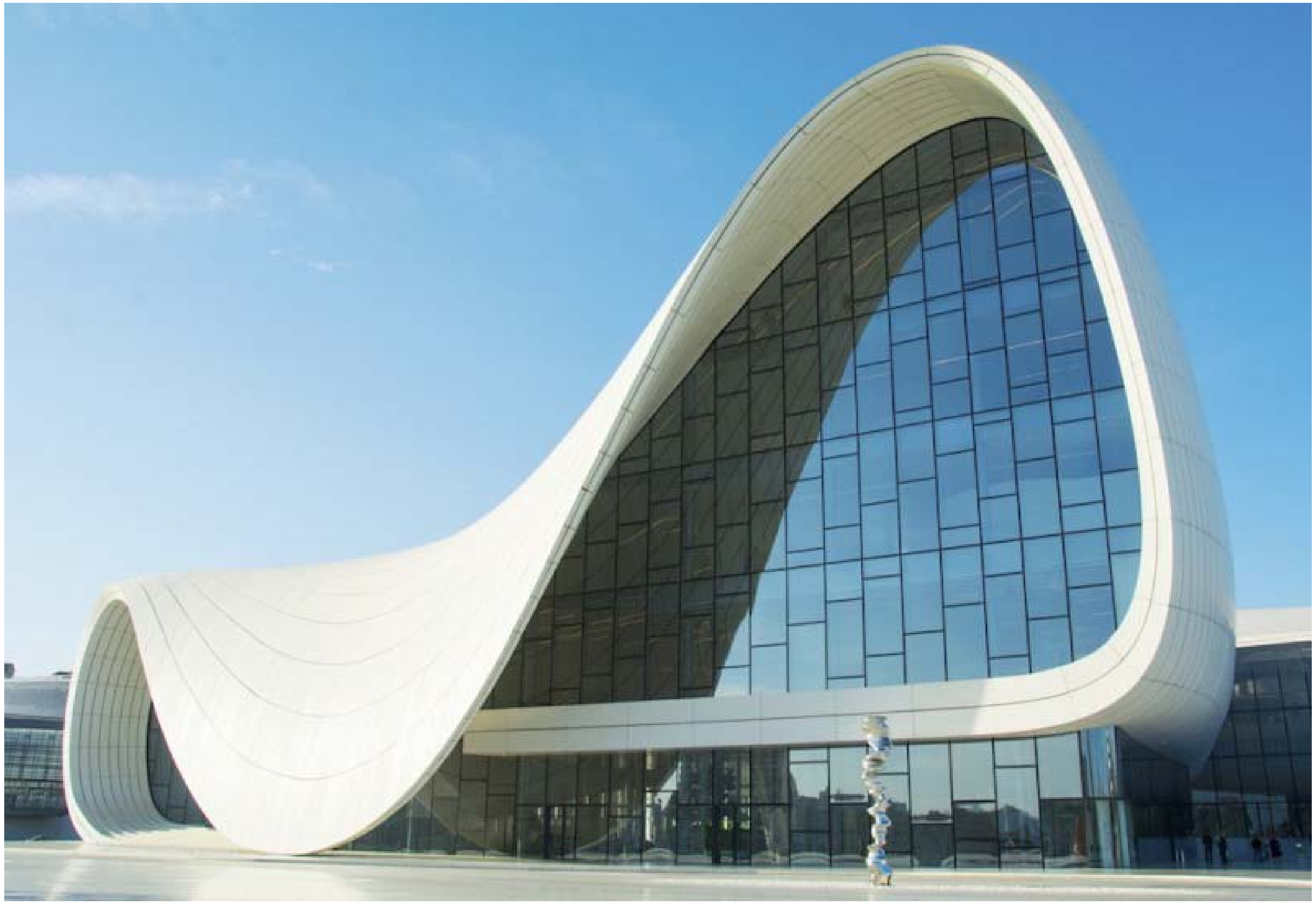
Stazione di Reggio Emilia 2013

Santiago Calatrava



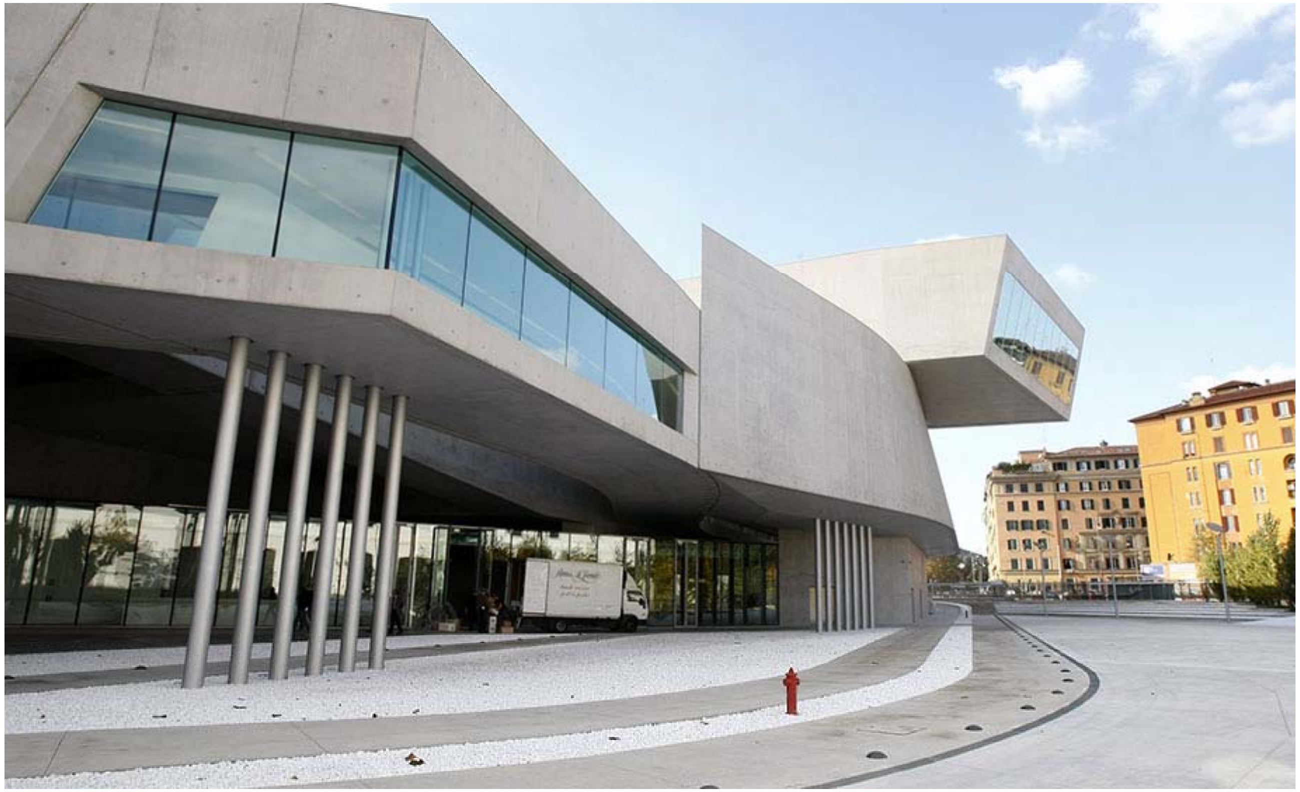
World Trade Center Oculus 2016

Zaha Hadid



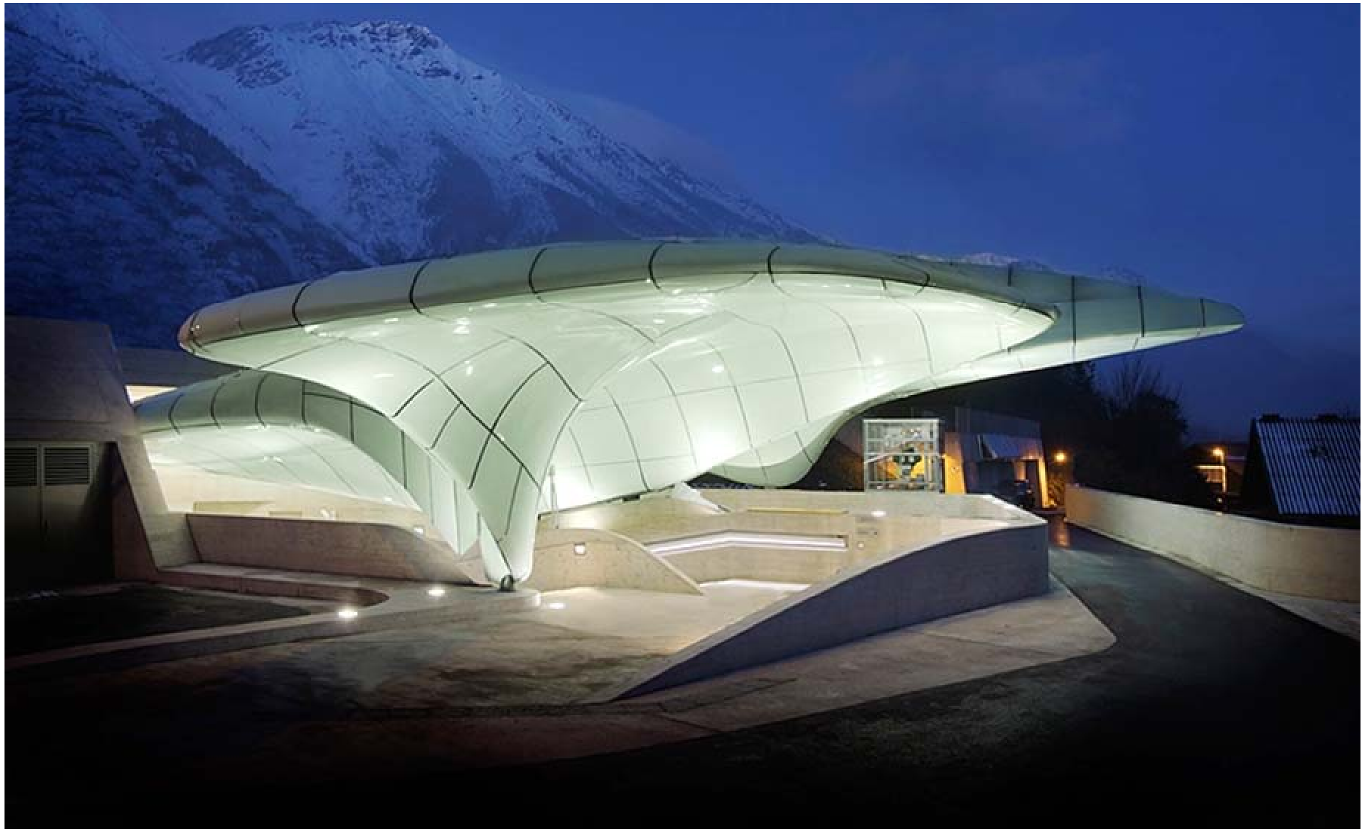
Centro Culturale Heydar Aliyev, Baku, Azerbaigian

Zaha Hadid



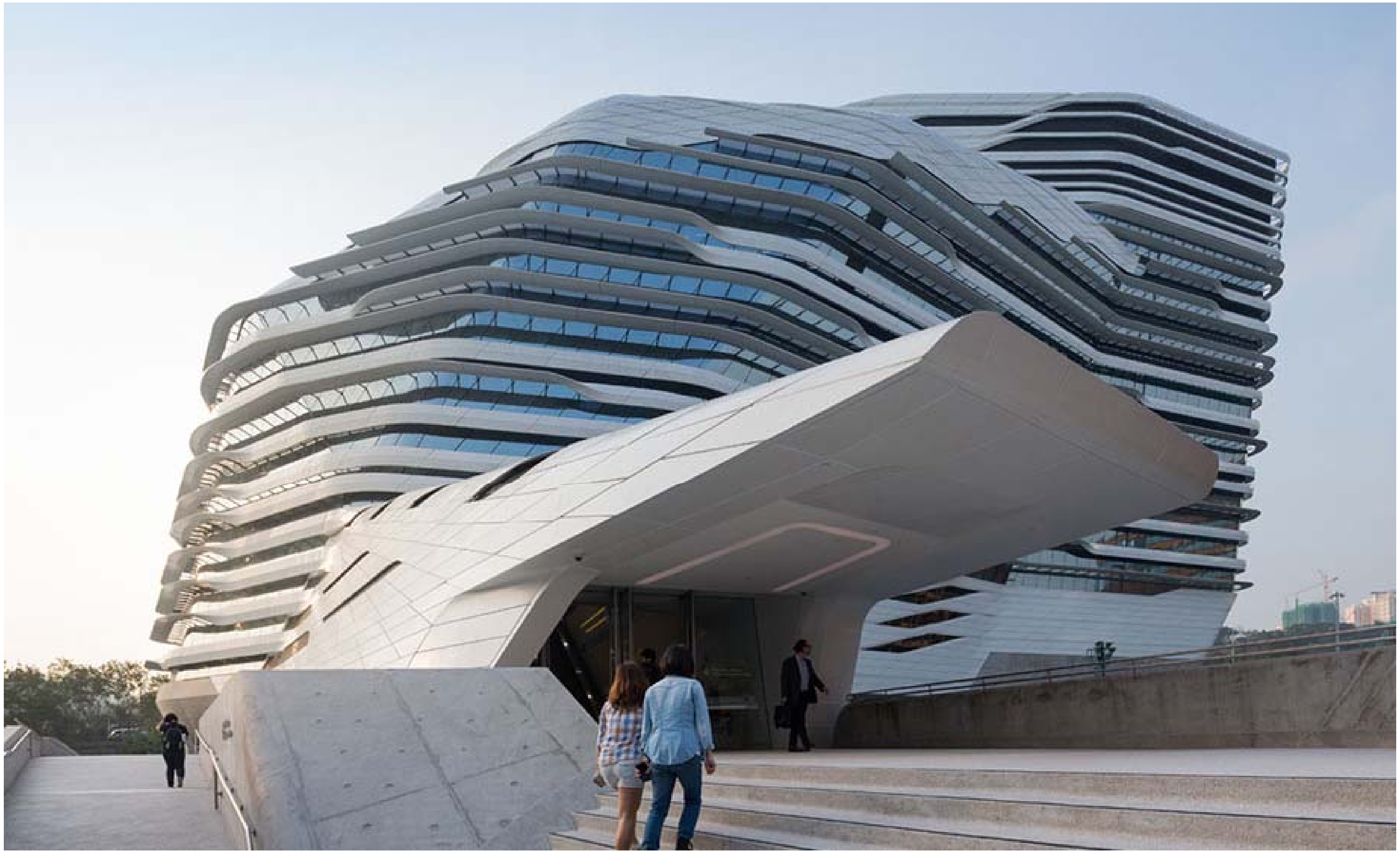
Museo MAXXI - Roma

Zaha Hadid



Nordpark Railway Stations, Innsbruck

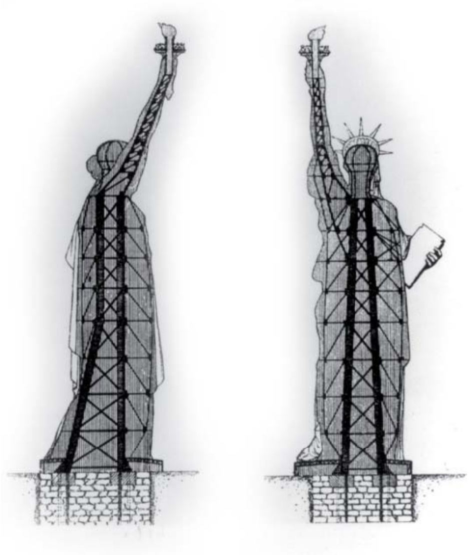
Zaha Hadid



Jockey Club Innovation Tower, Hong Kong

L’architetto non deve credersi un libero creatore di forme, ma scopritore e costruttore delle sole forme razionali.

In altre arti la libertà di linguaggio è molto più ampia, L’uomo è portato facilmente ad utilizzare false analogie, ad esempio non si può paragonare un pittore all’architetto; il pittore praticamente ha una libertà sconfinata di dipingere ciò che vuole, l’architetto no.
Già per uno scultore, tutta questa libertà non c’è: per fare una statua di 3 metri lo scultore può fare ciò che vuole ma la statua della libertà riesce a sostenersi perché al suo interno c’è una struttura portante in ferro (progettata da Eiffel).



Grazie per l'attenzione

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