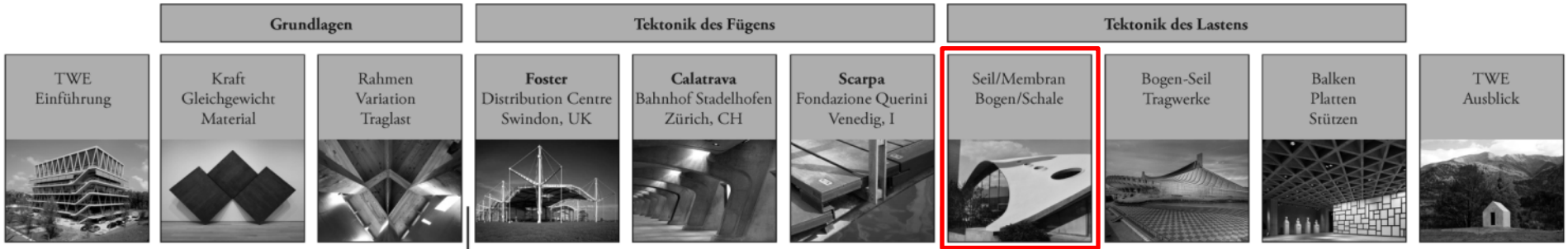
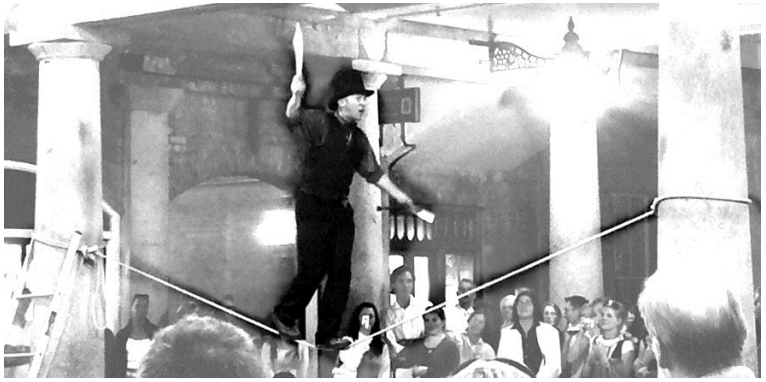
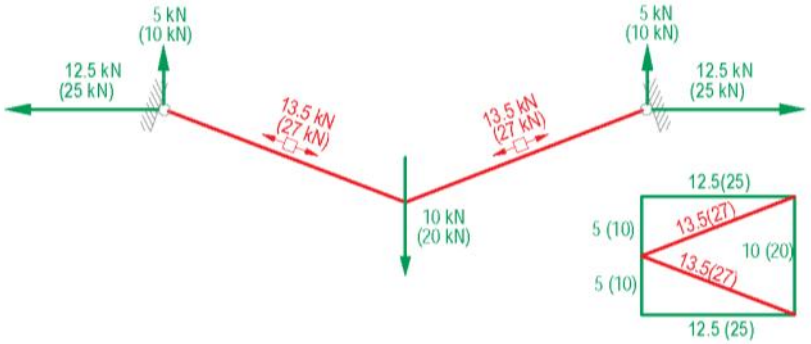
Tragwerksentwurf III

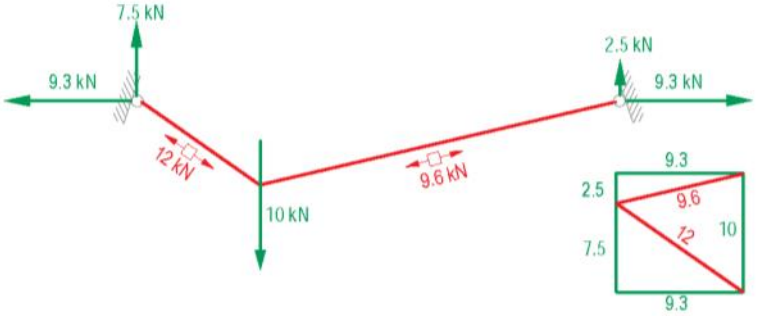


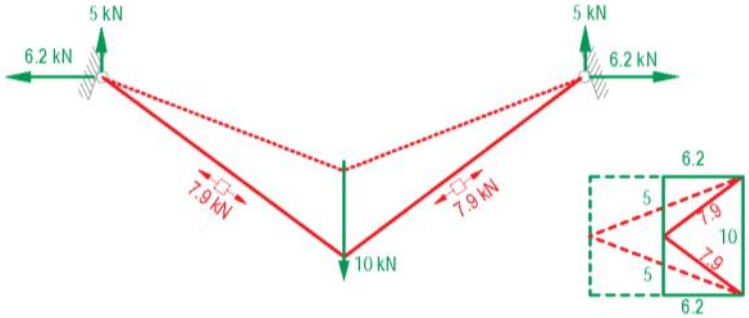
Symposium
Die Autorität der Tragstruktur in der Architektur
Freitag, 5.10.2012

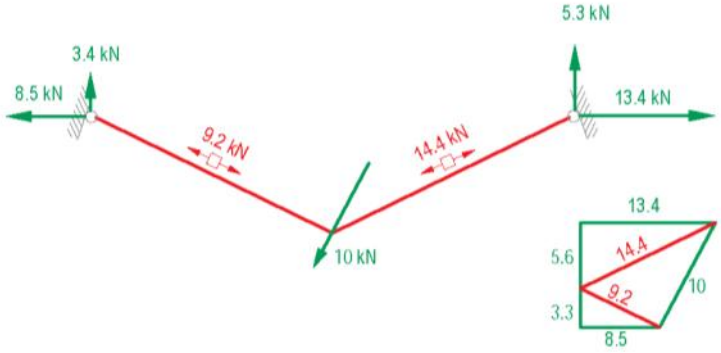
Seminarwoche

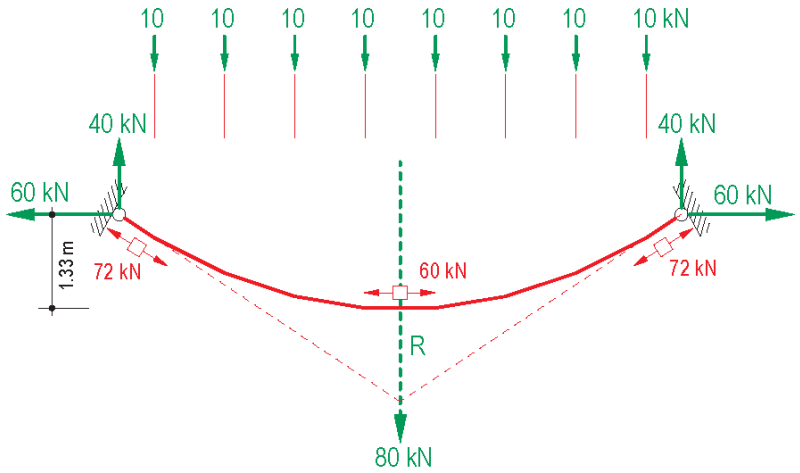


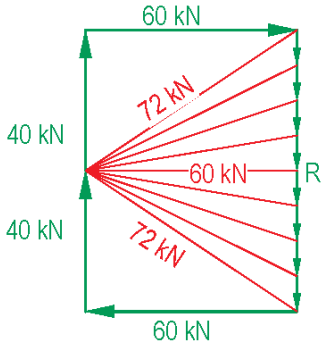

5 kN
(10 kN)
12.5 kN
(25 kN)
13.5 kN
(27 kN)
13.5 kN
(27 kN)
5 kN
(10 kN)
12.5 kN
(25 kN)
10 kN
(20 kN)
12.5(25)
5 (10)
13.5(27)
10 (20)
13.5(27)
5 (10)
12.5 (25)

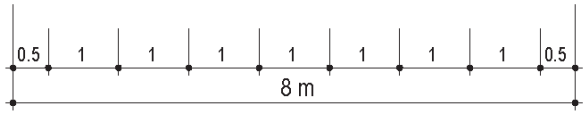

7.5 kN
9.3 kN
12 kN
9.6 kN
2.5 kN
9.3 kN
10 kN
9.3
2.5
9.6
12
10
7.5
9.3

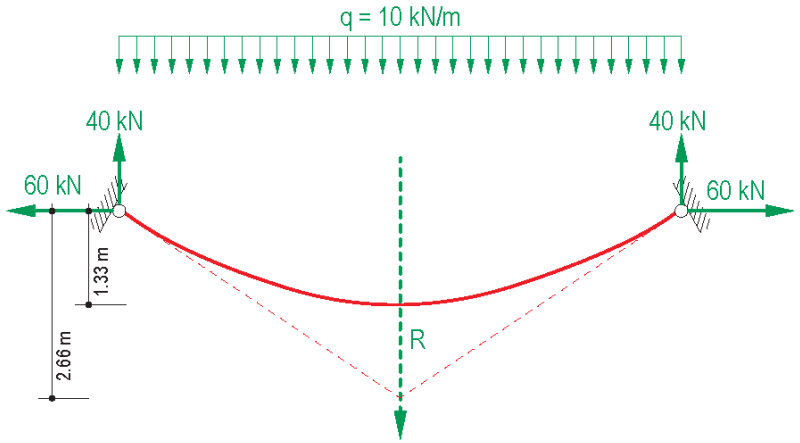

5 kN
6.2 kN
7.9 kN
7.9 kN
5 kN
6.2 kN
10 kN
6.2
5
7.9
10
7.9
5
6.2

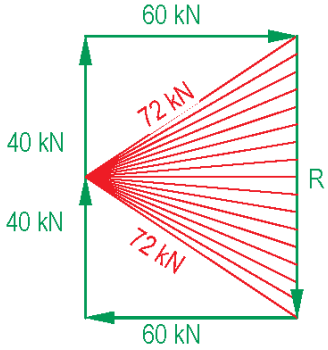

3.4 kN
8.5 kN
9.2 kN
14.4 kN
5.3 kN
13.4 kN
10 kN
13.4
5.6
14.4
9.2
10
3.3
8.5


10
10
10
10
10
10
10
10 kN
40 kN
40 kN
60 kN
60 kN
1.33 m
72 kN
72 kN
60 kN
R
80 kN


60 kN
72 kN
40 kN
60 kN
R
40 kN
72 kN
60 kN


0.5
1
1
1
1
1
1
1
0.5
8 m

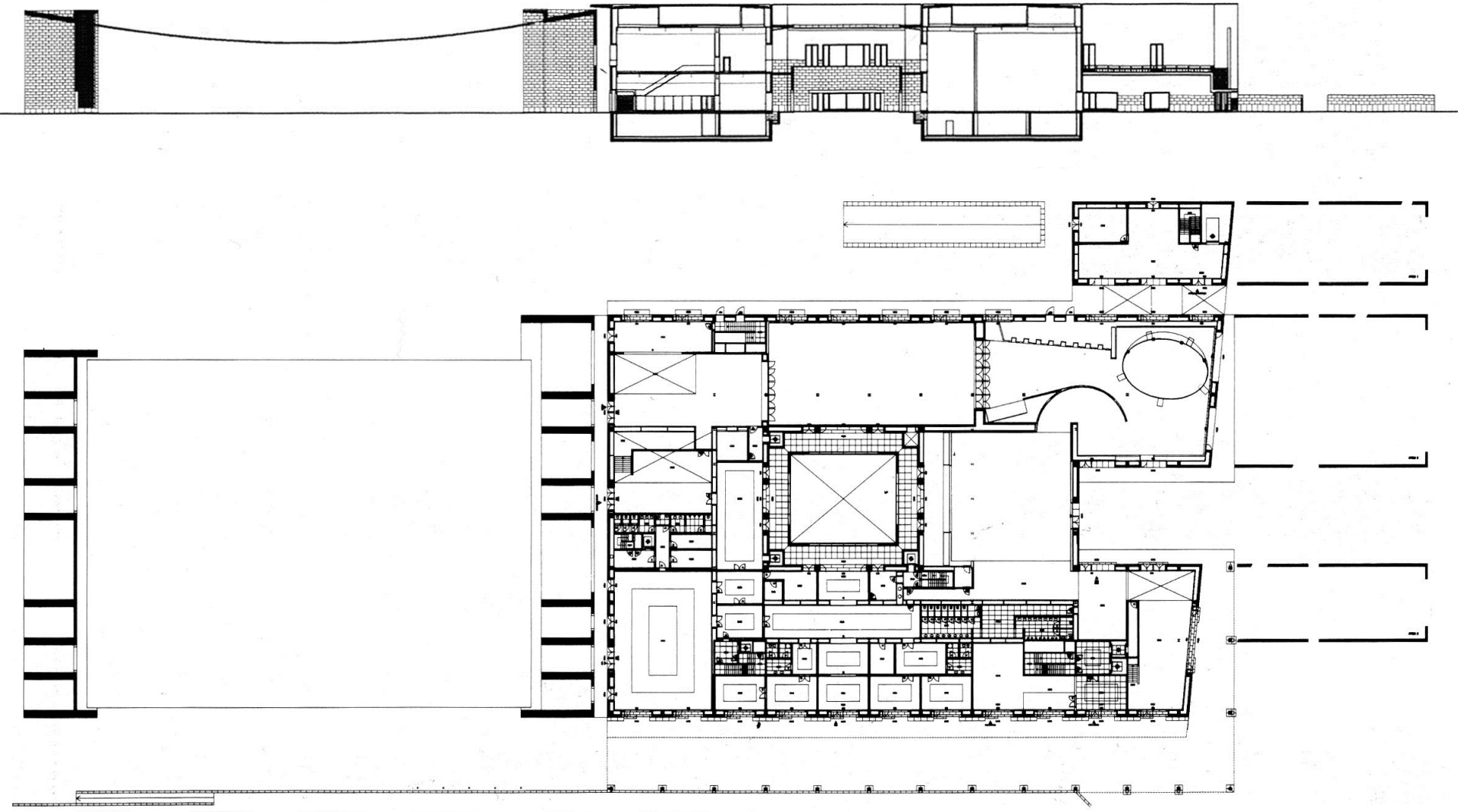

q = 10 kN/m
40 kN
40 kN
60 kN
60 kN
1.33 m
2.66 m
R


60 kN
72 kN
40 kN
R
40 kN
72 kN
60 kN

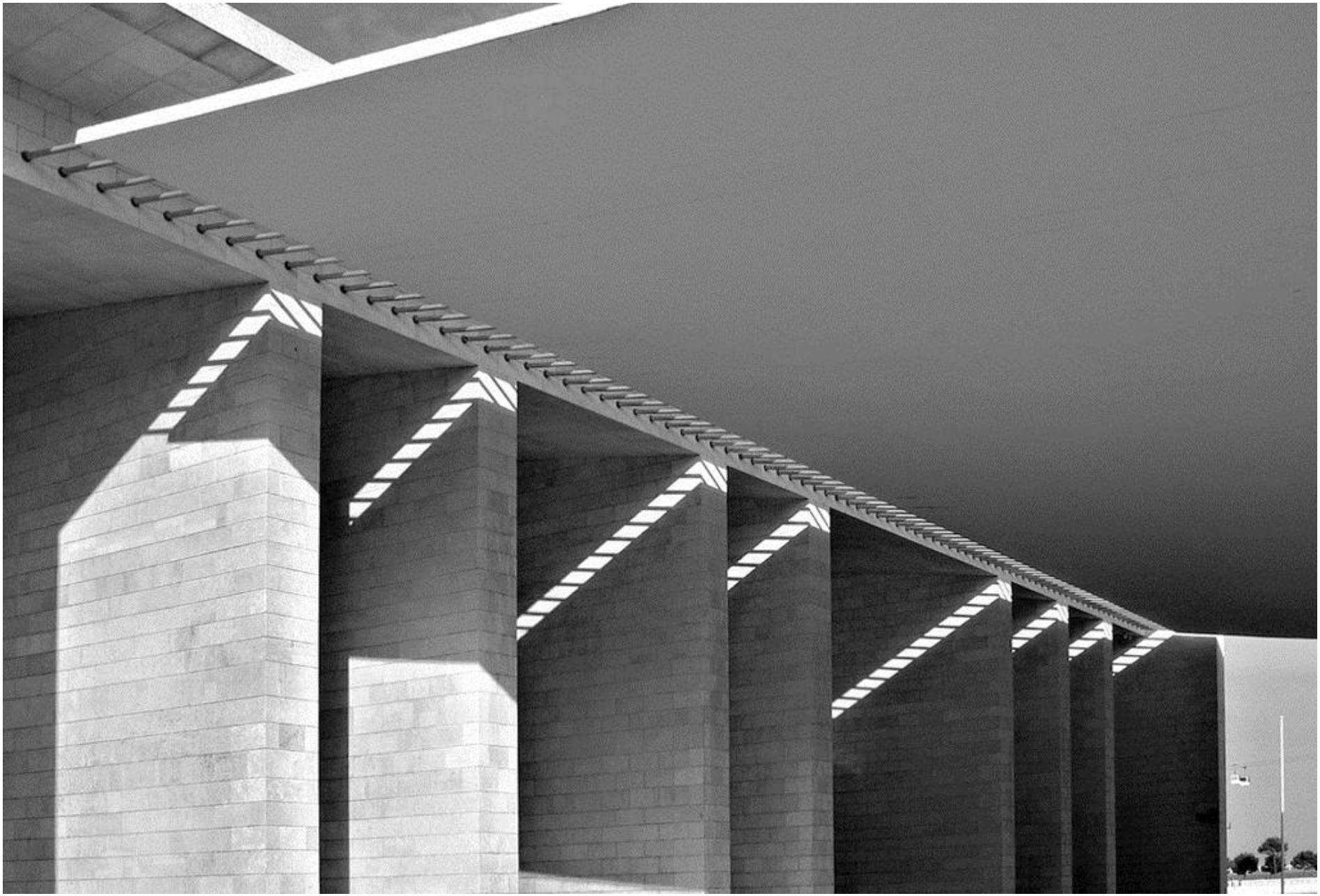
Alvaro Siza

Pavillon World EXPO
Lissabon, Portugal, 1998

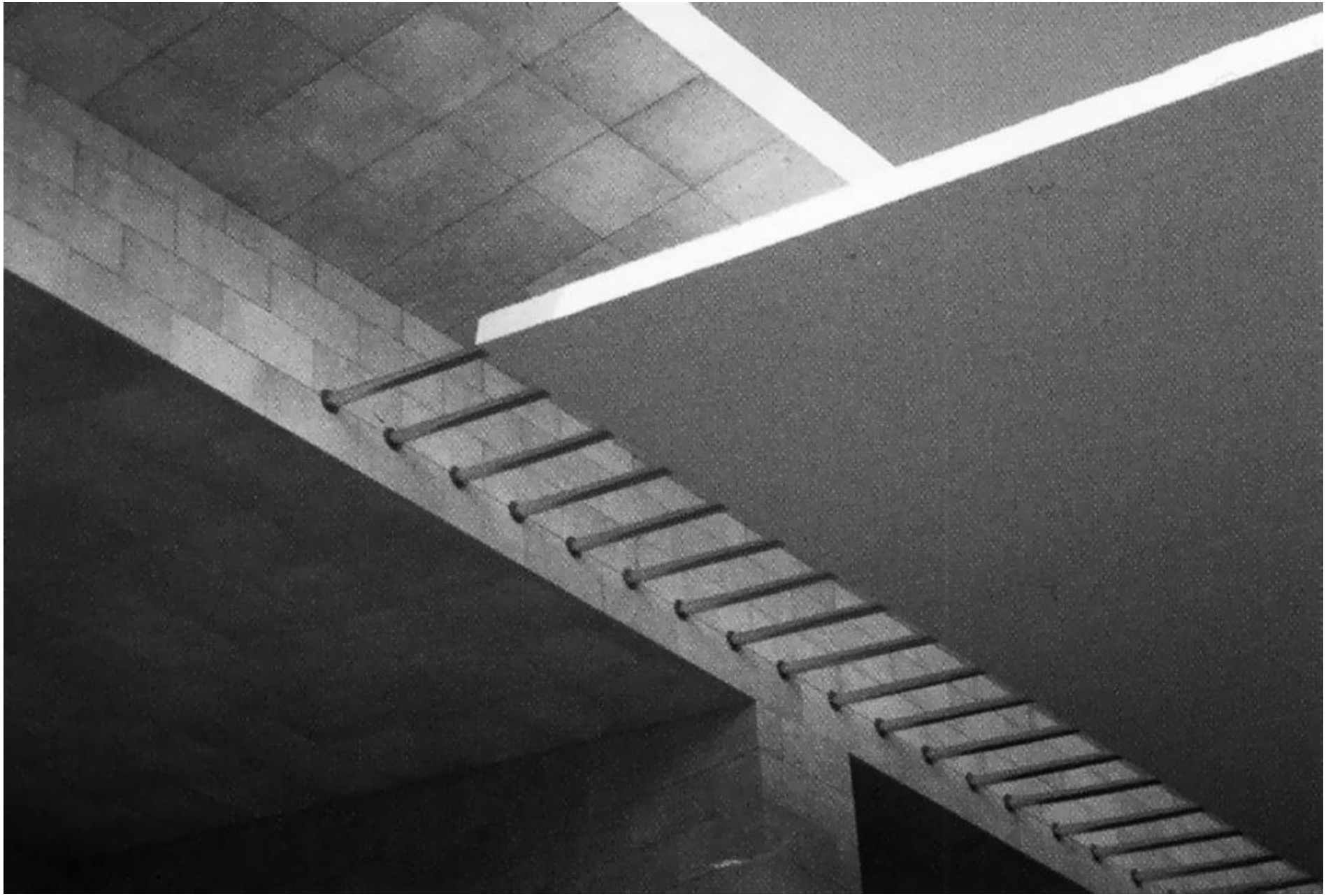
Ansicht vom Wasser



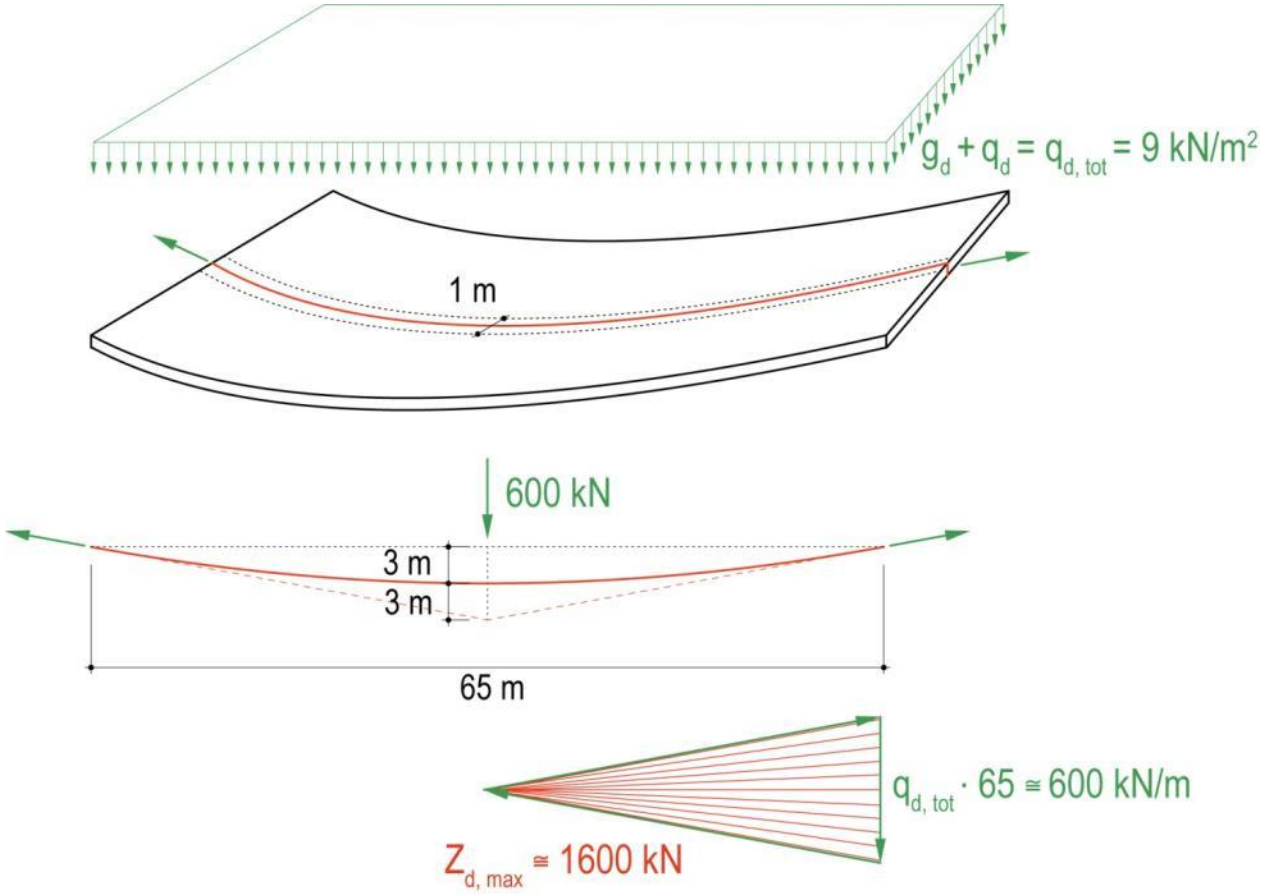
Grundriss und Längsschnitt



Verankerung der Membran in den Widerlagerkörpern I



Verankerung der Membran in den Widerlagerkörpern II



Eigenlasten: $g_d = 1.35 \cdot 0.2\ m^2 \cdot 25\ kN/m^3 \cong 7.0\ kN/m^2$

Nutzlasten: $q_d = 1.5 \cdot 1.3 \cong 2.0\ kN/m^2$

Totale Last: $q_{d,tot} \cong 9.0\ kN/m^2$

$Z_d = Z_{d,max} = 1'600\ kN.$

Ermittlung der Zugkraft in der Membran - Rechnung bezogen auf einen Meterstreifen mit einer Betonquerschnittsfläche von 0,2m²

schlaff bewehrte Membran
als Zugelement

Betonstahl	B500A	B500B	B450C
f_{yd} [N/mm^2]	435	435	≥ 390
ε_R	0.020	0.045	0.065

$$f_{yd} = \frac{f_y}{\gamma_M}$$

$$\gamma_M = 1.15$$

$$E = 210'000\ N/mm^2$$

Durch-messer [mm]	Querschnittsfläche A_s in mm^2 bei einer Anzahl Stäbe von									
	1	2	3	4	5	6	7	8	9	10
8	50	101	151	201	251	302	352	402	452	503
10	79	157	236	314	393	471	550	628	707	785
12	113	226	339	452	565	679	792	905	1018	1131
14	154	308	462	616	770	924	1078	1232	1385	1539
16	201	402	603	804	1005	1206	1407	1608	1810	2011
18	254	509	763	1018	1272	1527	1781	2036	2290	2545
20	314	628	942	1257	1571	1885	2199	2513	2827	3142
22	380	760	1140	1521	1901	2281	2661	3041	3421	3801
26	531	1062	1593	2124	2655	3186	3717	4247	4778	5309
30	707	1414	2121	2827	3534	4241	4948	5655	6362	7069

Eigenschaften von Betonstahl

Betonsorten und charakteristische Werte der Festigkeit

Betonsorte										
C	12/15	16/20	20/25	25/30	30/37	35/45	40/50	45/55	50/60	.../...
LC	12/13	16/18	20/22	25/28	30/33	35/38	40/44	45/50	50/55	.../...
f_{ck} [N/mm²]	charakteristischer Wert der Zylinderdruckfestigkeit									
C/LC	12	16	20	25	30	35	40	45	50	> 50
$f_{ck\,cube}$ [N/mm²]	charakteristischer Wert der Würfeldruckfestigkeit									
C	15	20	25	30	37	45	50	55	60	> 60
LC	13	18	22	28	33	38	44	50	55	> 55
f_{cm} [N/mm²]	Mittelwert der Zylinderdruckfestigkeit									
C/LC	20	24	28	33	38	43	48	53	58	f_{ck} + 8
f_{ctm} [N/mm²]	Mittelwert der Betonzugfestigkeit									
C	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	> 4,1

Bemessungswerte für Normalbeton

Beton	C 12/15	C 16/20	C 20/25	C 25/30	C 30/37	C 35/45	C 40/50	C 45/55	C 50/60	C .../...
f_{cd} [N/mm²]	8,0	10,5	13,5	16,5	20,0	22,0	24,0	26,0	28,0	Gl. (2)

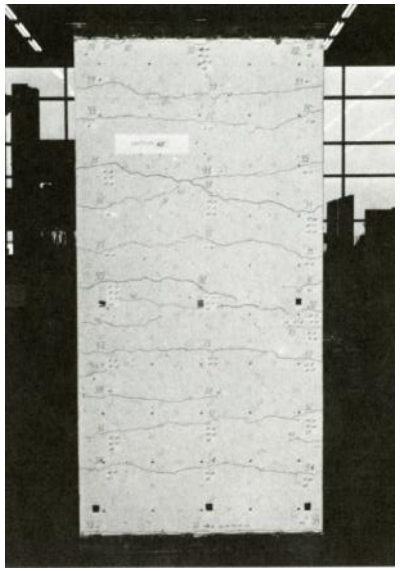
Der Mittelwert des Elastizitätsmoduls E_{cm} von Normalbeton kann mit folgender Gleichung abgeschätzt werden:

$E_{cm} = k_E \sqrt[3]{f_{cm}}$ $\quad$ E_{cm} in N/mm²

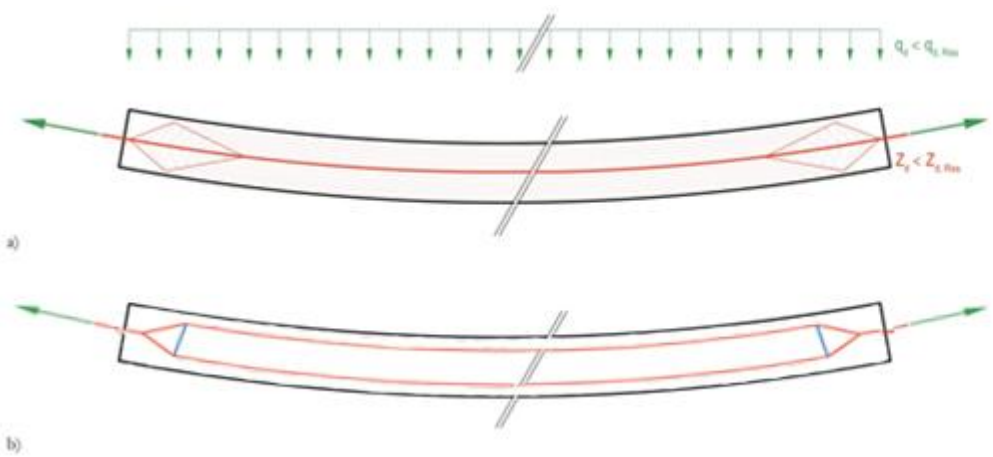
Der Beiwert k_E liegt für Gesteinskörnungen aus Alluvialkies zwischen 10 000 und 12 000, für solche aus gebrochenem Kalk zwischen 8000 und 10 000 und für glimmerhaltiges Gestein zwischen 6000 und 8000. Je nach Betonzusammensetzung können bedeutende Abweichungen von diesen Werten auftreten; gegebenenfalls ist E_{cm} durch Versuche zu bestimmen.

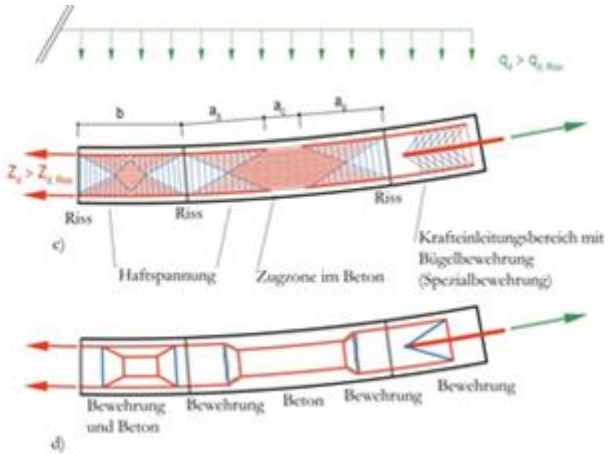
$$f_{cd} = \frac{f_c}{\gamma_M}$$

$$\gamma_M = 1.5$$



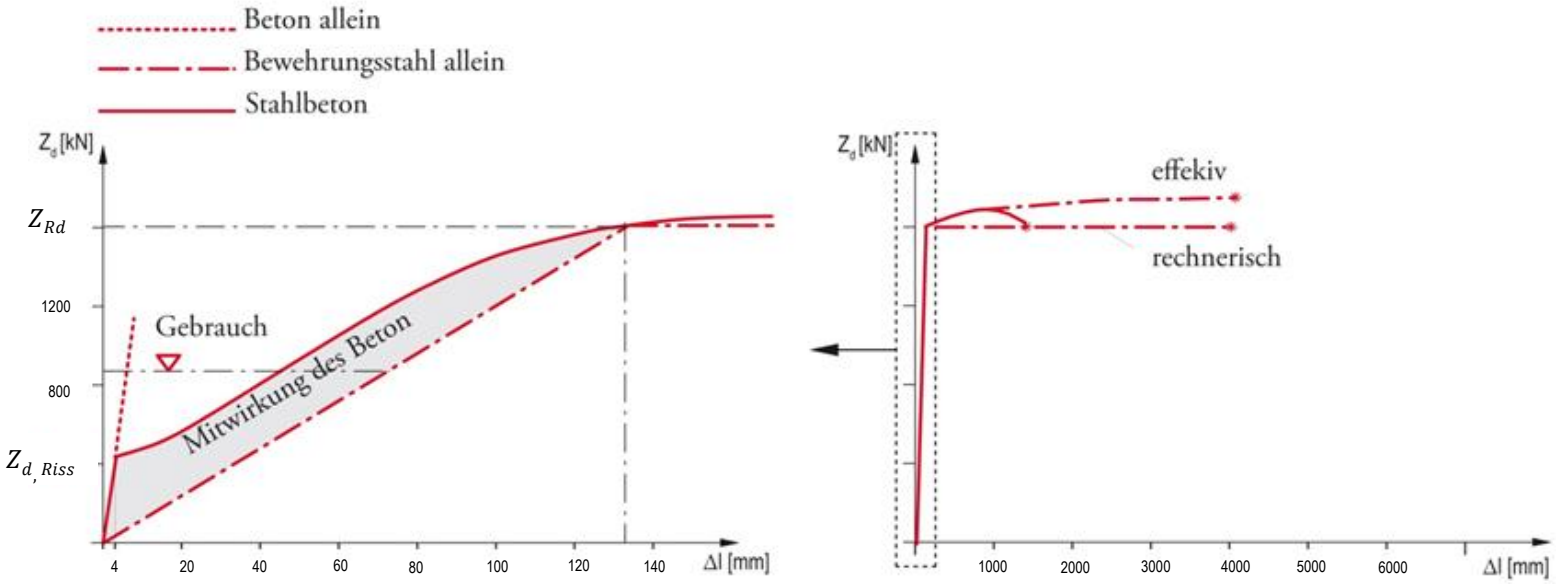
Eigenschaften von Beton





$$A_{sL,\,erf} = \frac{Z_d}{f_{yd}} \qquad \Delta l_y = \frac{Z_y \cdot l}{E_s \cdot A_s}$$

Tragverhalten des schlaff bewehrten Zugelements im ungerissenen Zustand: Spannungsfeld a) und Resultierende b) und im gerissenen Zustand: Spannungsfeld c) und Resultierende d)



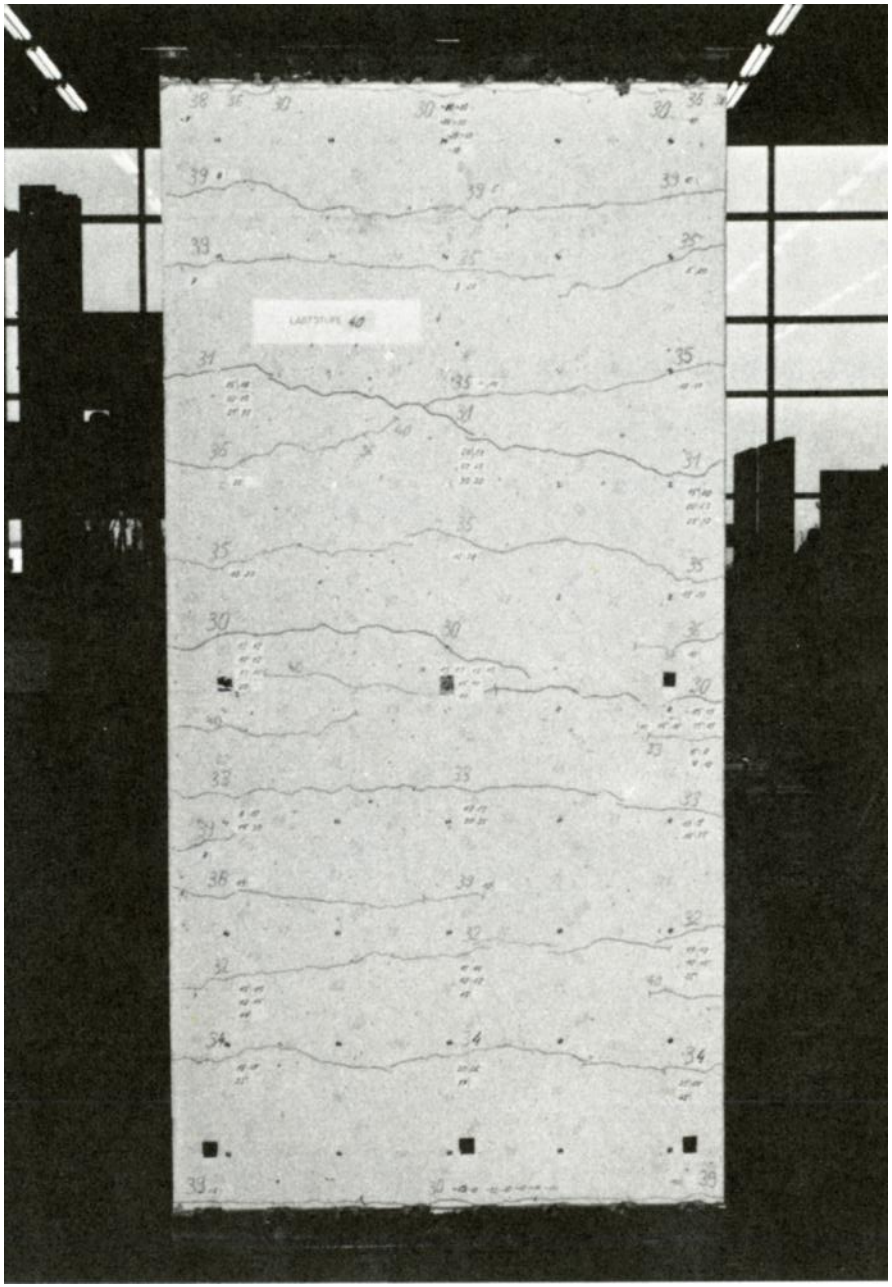
$$Z_{d,\,Riss} \cong A_c \cdot f_{ctm} / \gamma_M$$

$$\Delta l_{Riss} = \frac{Z_{d,\,Riss} \cdot l}{E_c \cdot A_c}$$

$$Z_{Rd} = \frac{A_s \,.\, f_y}{\gamma_M} \quad = A_s \cdot f_{yd}$$

Last-Verformungsverhalten der schlaff bewehrten Zugstrebe bis zum Bruch

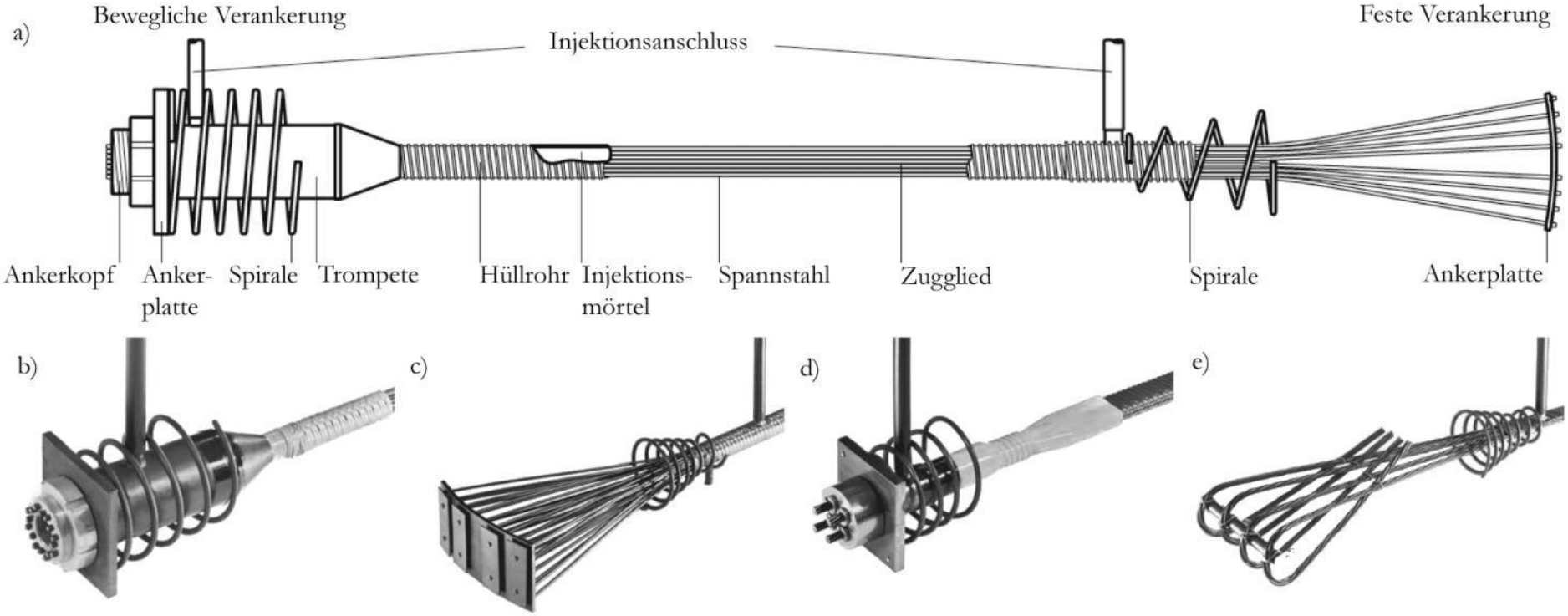
Schlaffe Bewehrung einer Stahlbetondecke



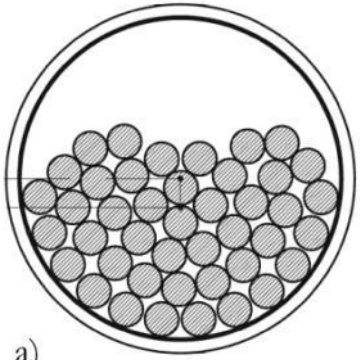
Gerissene Decke

Nachbehandlung der Oberfläche der Membran mittels Abdecken des jungen Betons

vorgespannte Membran
als Zugelement



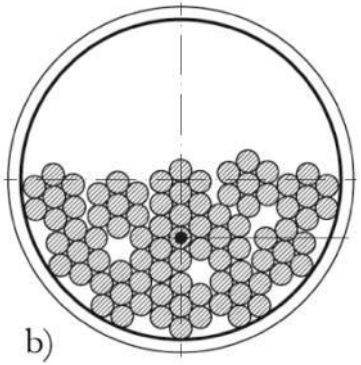
Spannkabel a), Vorspannung mit Drähten: fester Ankerkopf b) und beweglicher Ankerkopf c), Vorspannung mit Litzen: fester Ankerkopf d) und beweglicher Ankerkopf e)



a)

Kaltgezogener, runder Spannstahl mit glatter Oberfläche

Nenndurchmesser	ϕ	7 mm
Querschnittsfläche	A_p	38.48 mm^2
Zugfestigkeit	f_{pt}	1670 N/mm^2
Fliessgrenze	f_{py}	1440 N/mm^2



b)

Spannstahllitzen, verdrillt aus 7 Drähten

Nenndurchmesser	ϕ	15.7 mm
Querschnittsfläche	A_p	150 mm^2
Zugfestigkeit	f_{pt}	1770 N/mm^2
Fliessgrenze	f_{py}	1520 N/mm^2

$$f_{pyd} = \frac{f_{py}}{\gamma_M} \qquad \gamma_M = 1.15 \qquad E = 210'000\ N/mm^2$$

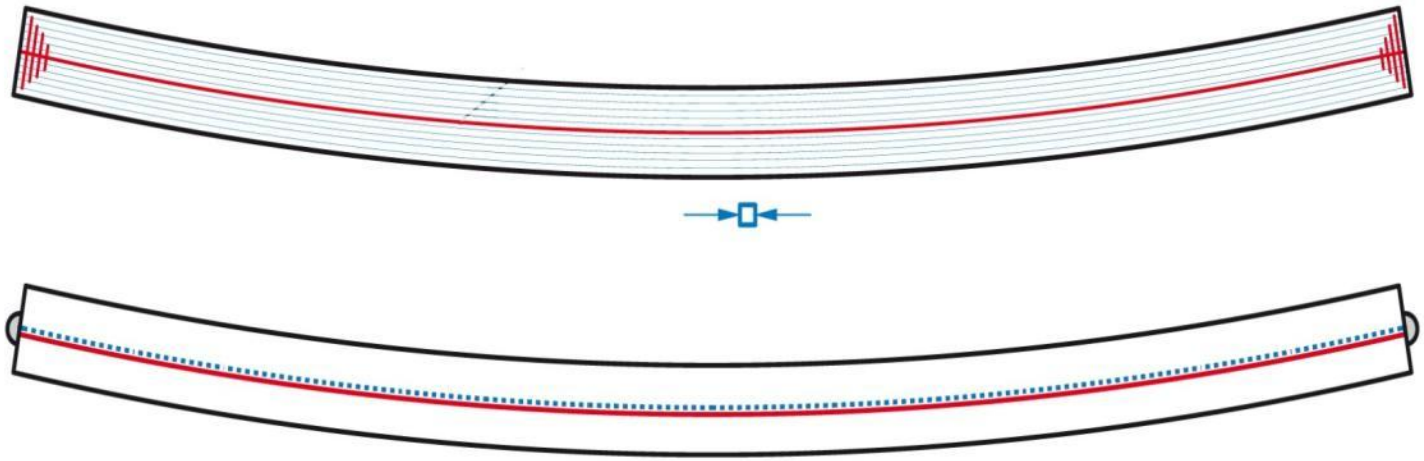
Spanndrähte und -litzen, Drähte im Hüllrohr a), Litzen im Hüllrohr b)

a)

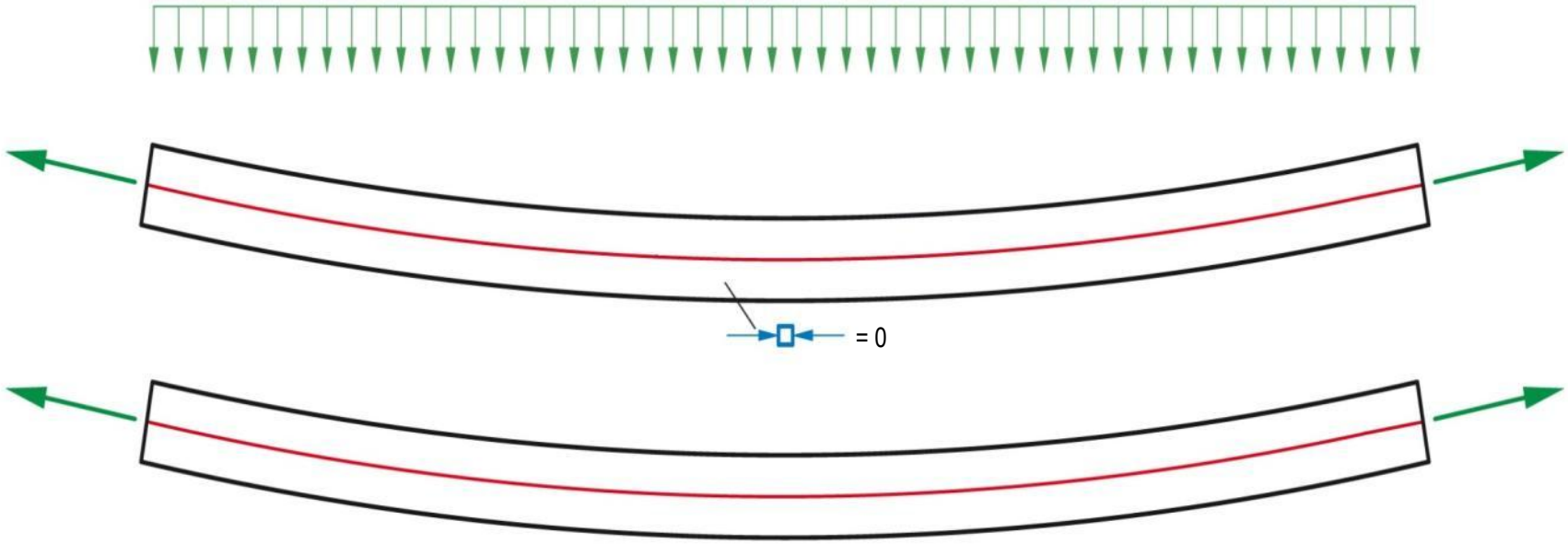
b)

c)

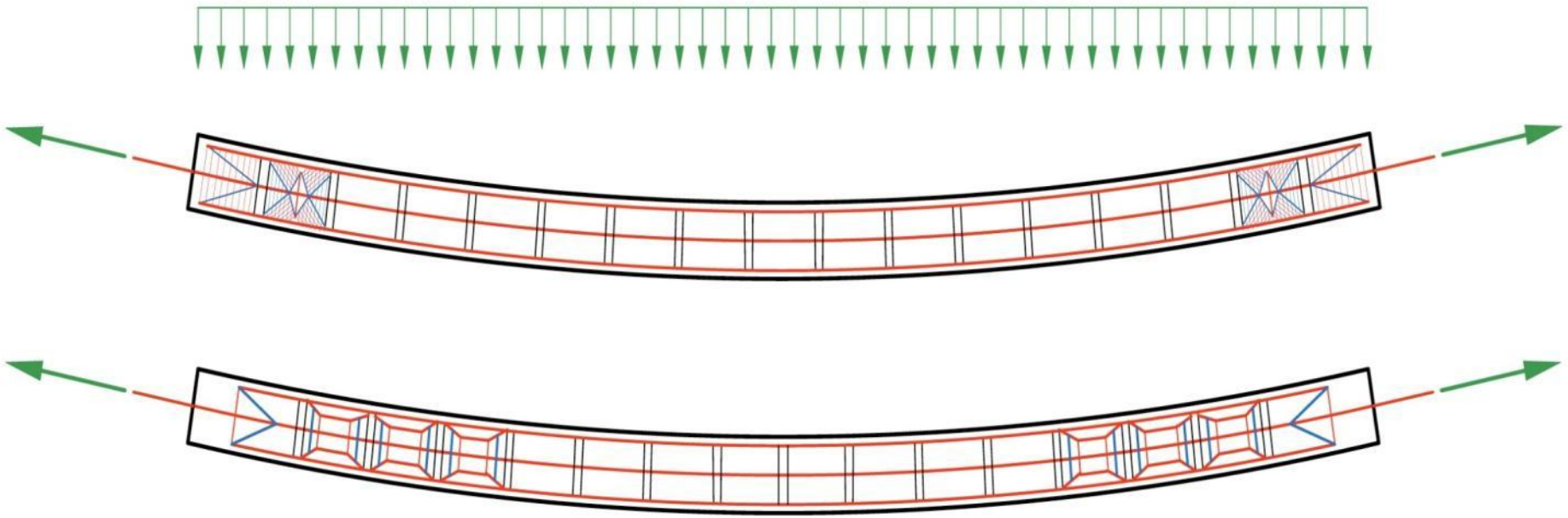
Spanndrähte und Litzen, Einzeldraht a), Einzellitze b) und Spannvorgang mittels hydraulischer Presse c)



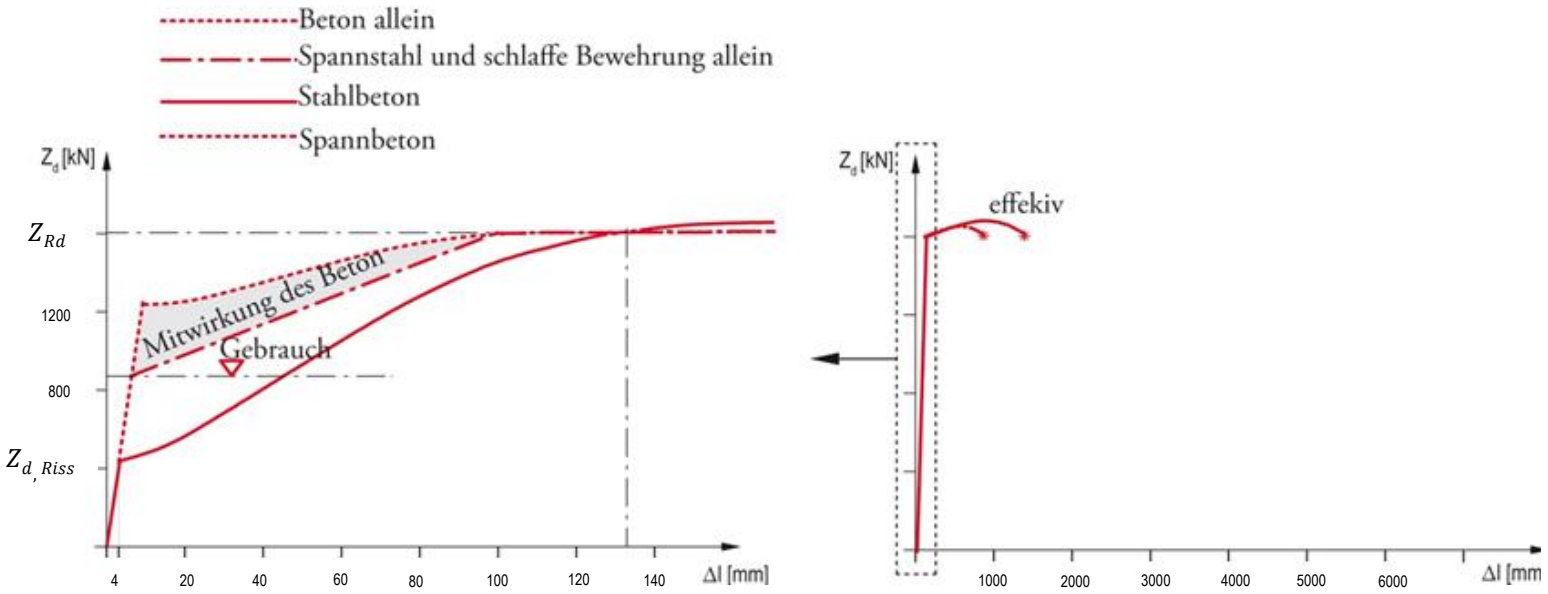
Tragverhalten des Vorgespannten Zugelements:Tragelement nach dem Spannen: Spannungsfeld und Resultierende



Innere Zugkraft = Vorspannkraft: Spannungsfeld und Resultierende



Traglast: Spannungsfeld und Resultierende



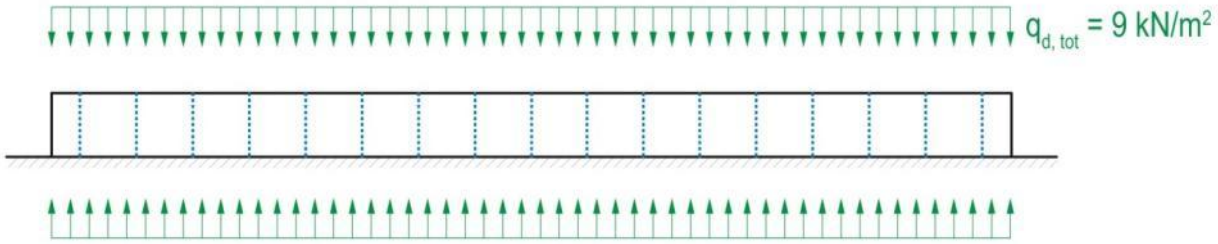
$$Z_{Rd} = A_p \cdot f_{pyd} + A_s \cdot f_{yd}$$

Last-Verformungsverhalten der vorgespannten Zugstrebe bis zum Bruch

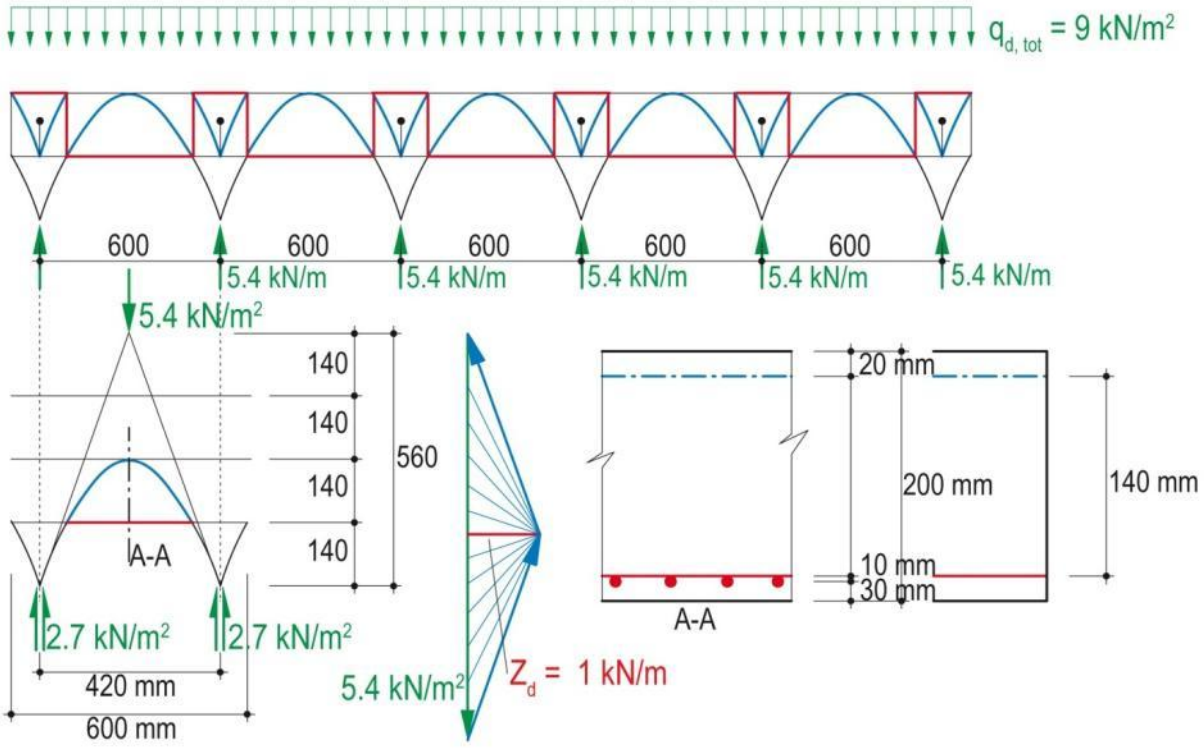
Membran mit Auflagerbauten

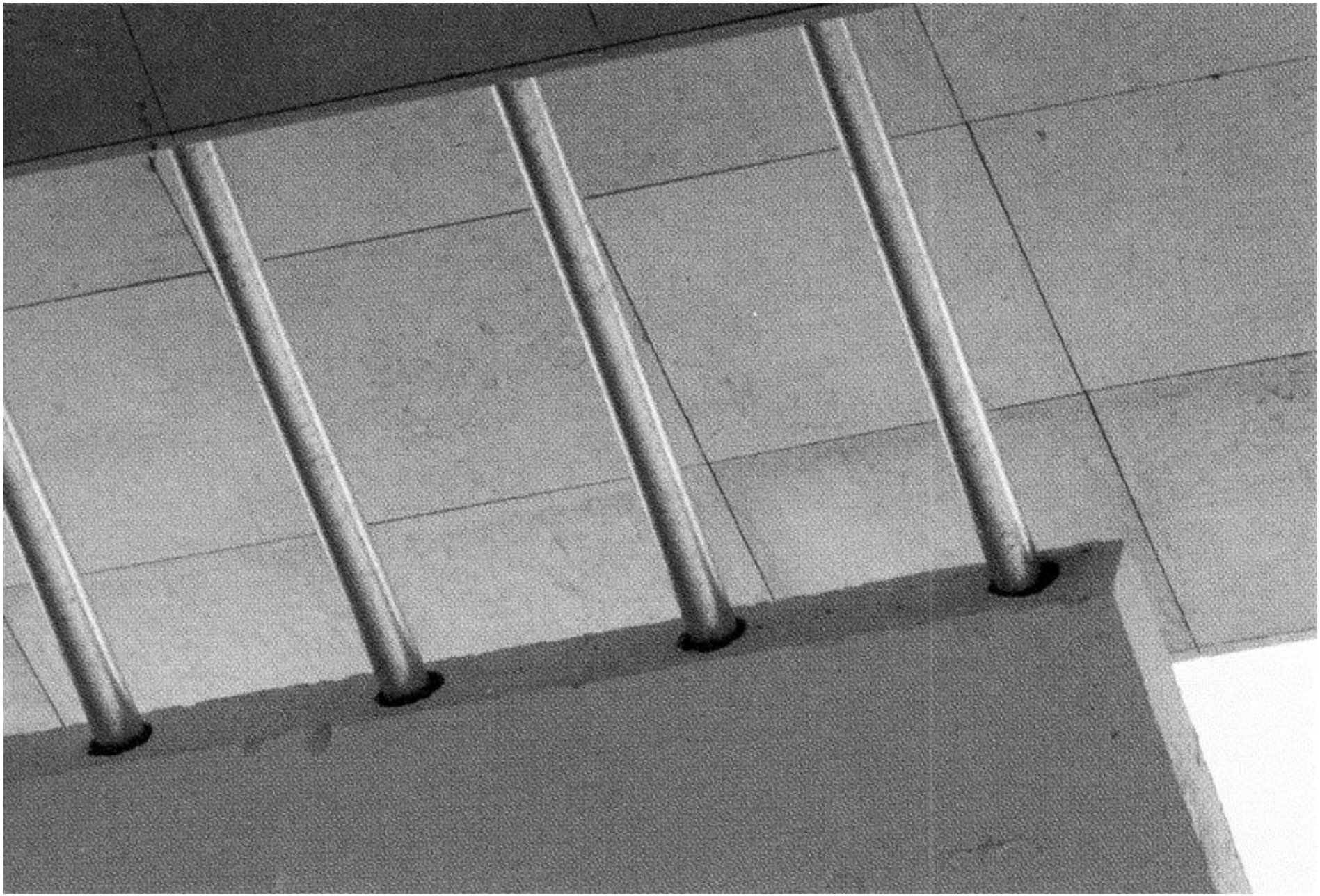
Bürogebäude in Canberra, Australien, 1974, Arch.: Harry Seidler

ausgeführte Lösung:
schlaff bewehrte Membran
auf Stahlseilen aufgelegt



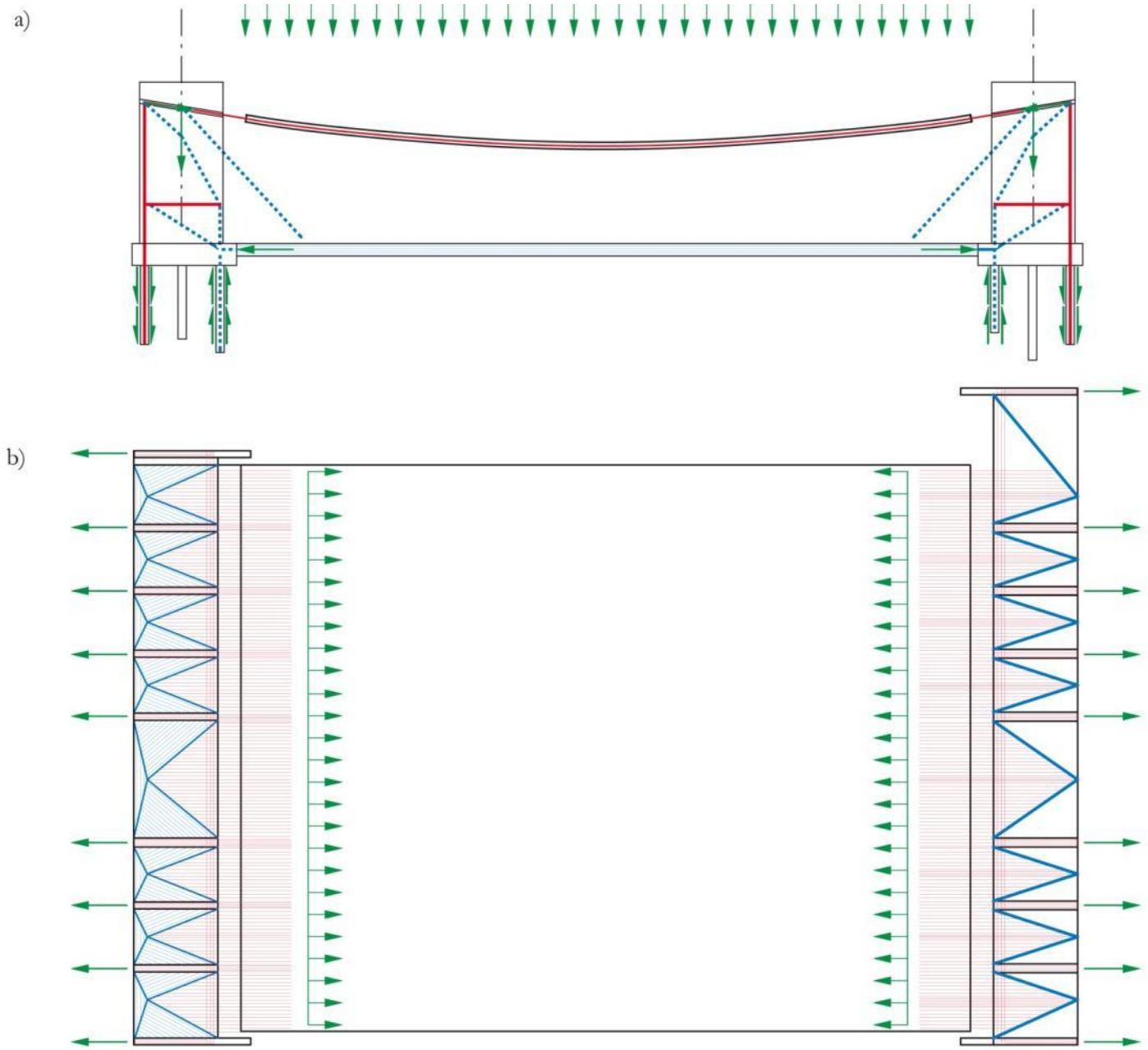
$$A_{s,\,min} = A_c \cdot \frac{f_{ctm}}{f_y}$$


$q_{d,\,tot}$ = 9 kN/m²
600
600
600
600
600
5.4 kN/m
5.4 kN/m
5.4 kN/m
5.4 kN/m
5.4 kN/m
5.4 kN/m²
140
140
140
140
560
A-A
2.7 kN/m²
2.7 kN/m²
420 mm
600 mm
5.4 kN/m²
Z_d = 1 kN/m
20 mm
200 mm
10 mm
30 mm
140 mm
A-A



Kabel des Baldachins

Auflagerkörper, beansprucht durch die grossen Kabelkräfte



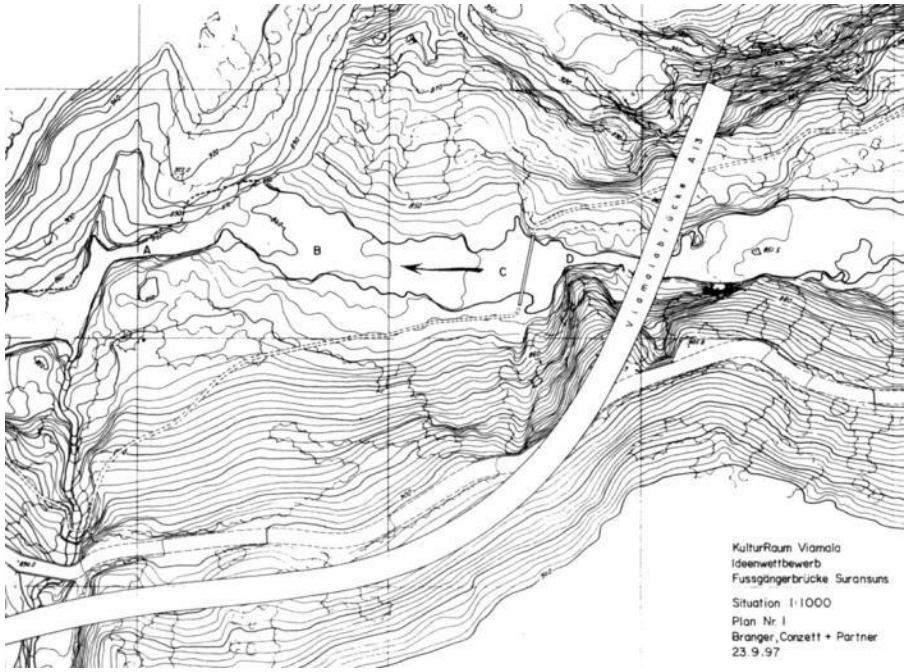
Innere Kräfte in den Auflagerkörpern, Tragwirkung der Querschotten a), Tragwirkung der Dachscheibe b)

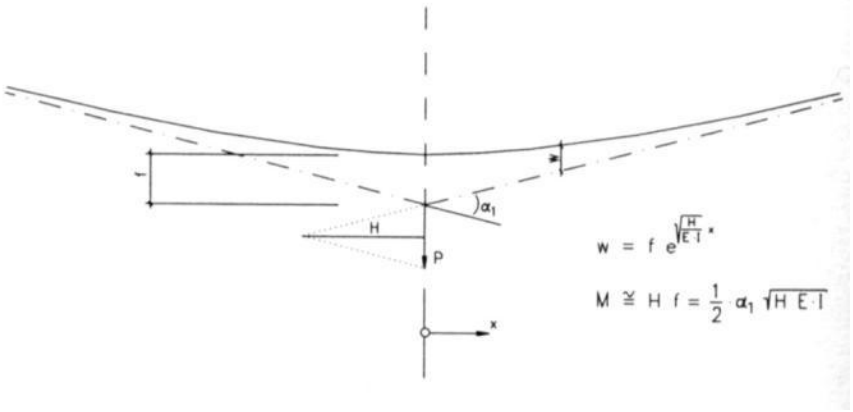
Pavillon vom Wasser

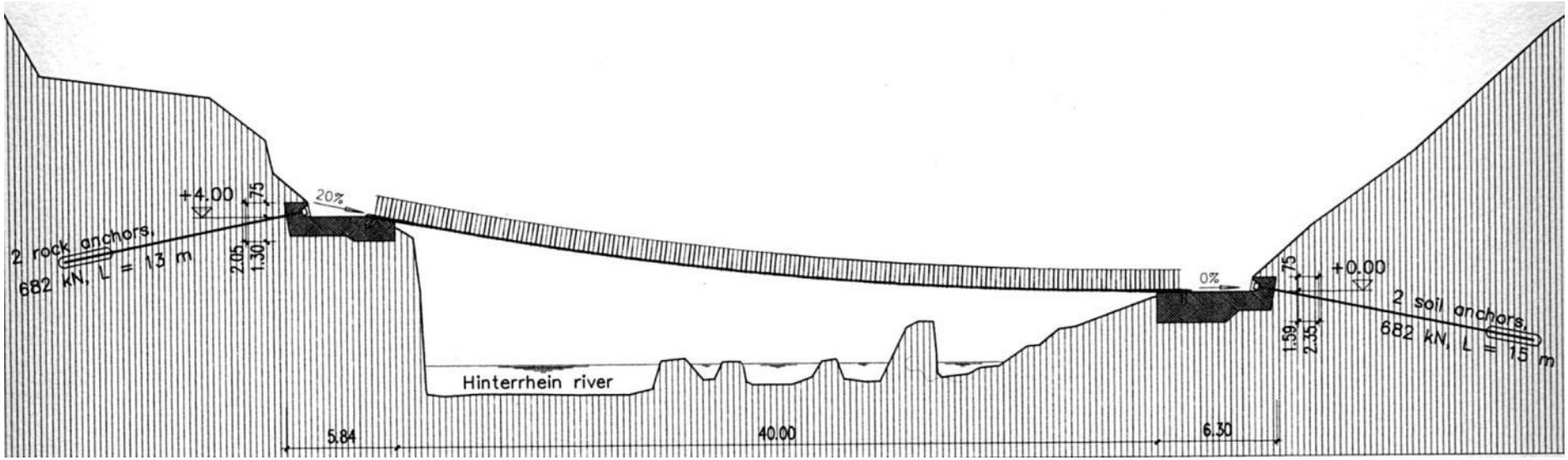
Jürg Conzett

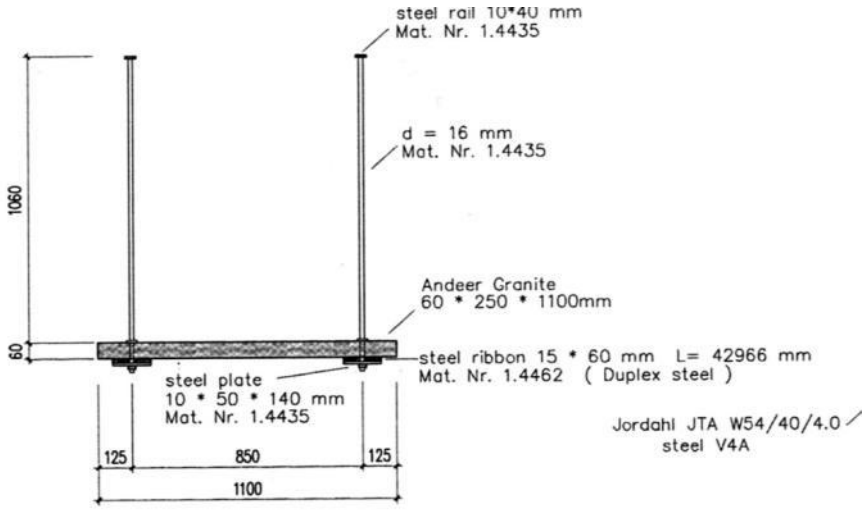
Pùnt da Suransuns

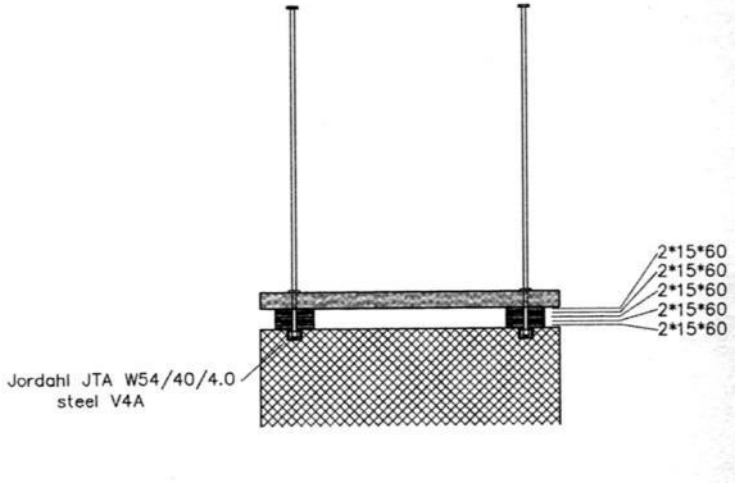
Viamala, CH, 1997-1999

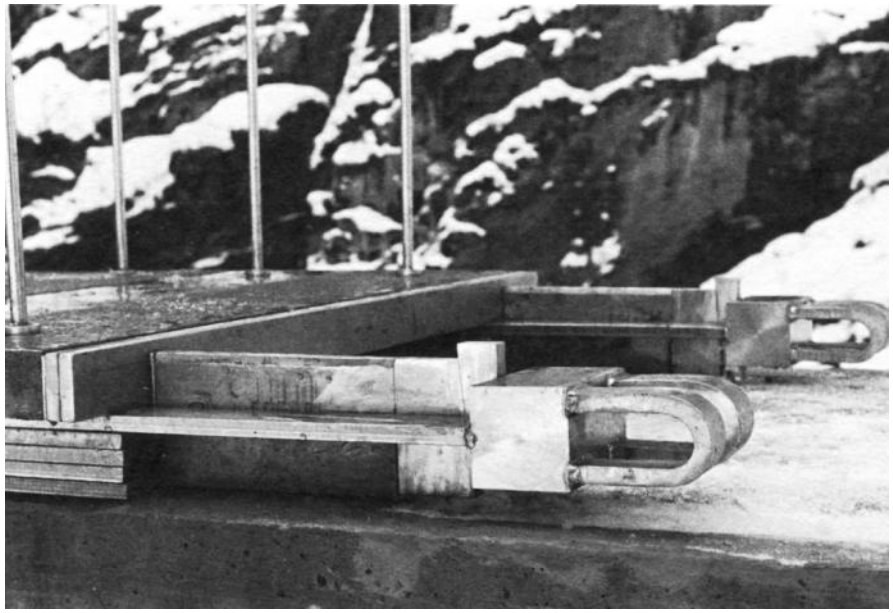

KulturRaum Viamala
Ideenwettbewerb
Fussgängerbrücke Suransuns
Situation 1:1000
Plan Nr. 1
Branger, Conzett + Partner
23.9.97

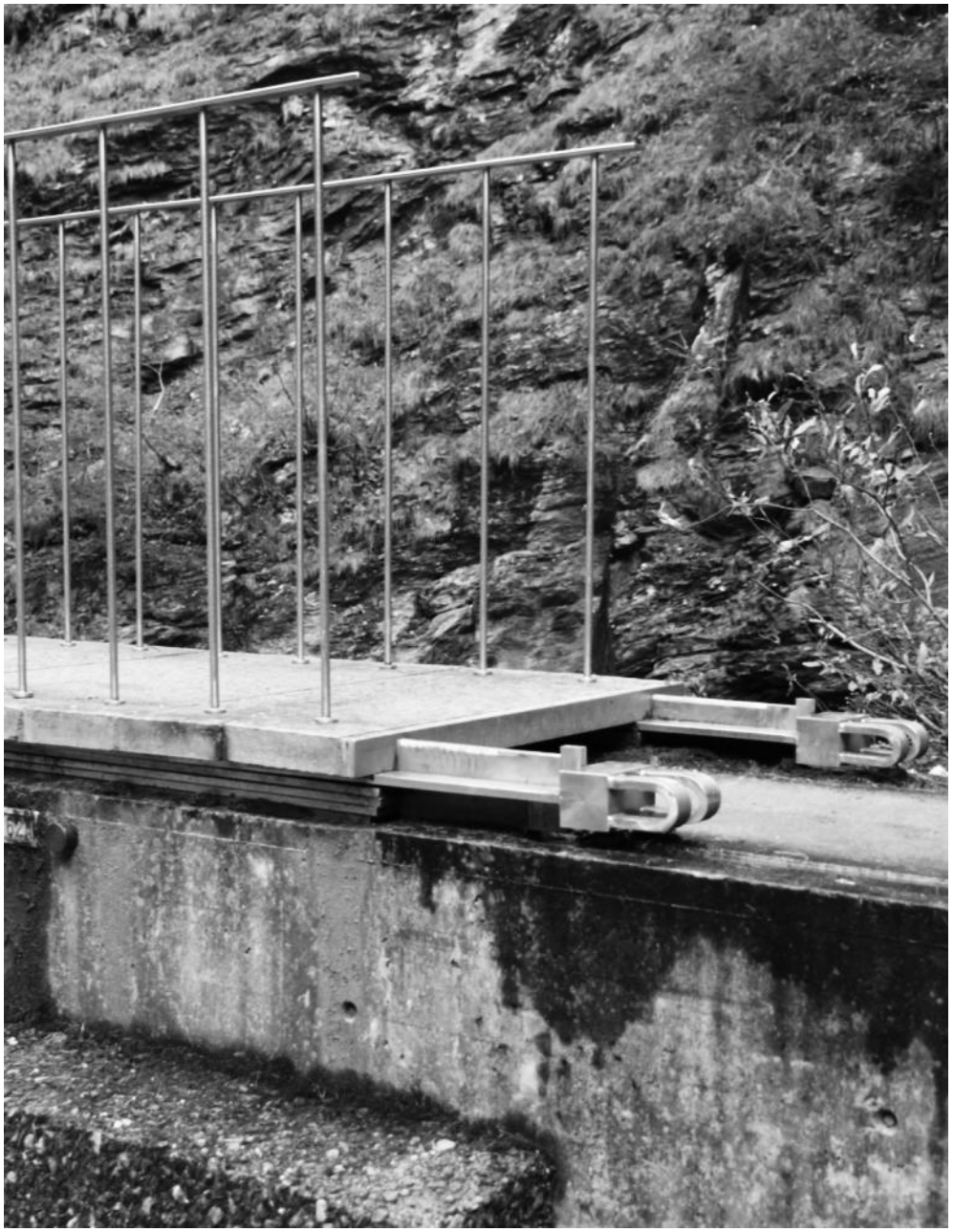

H
P
x
$w = f\, e^{\sqrt{\frac{H}{EI}}\,x}$
$M \cong H\, f = \frac{1}{2}\,\alpha_1 \sqrt{H\,E\,I}$

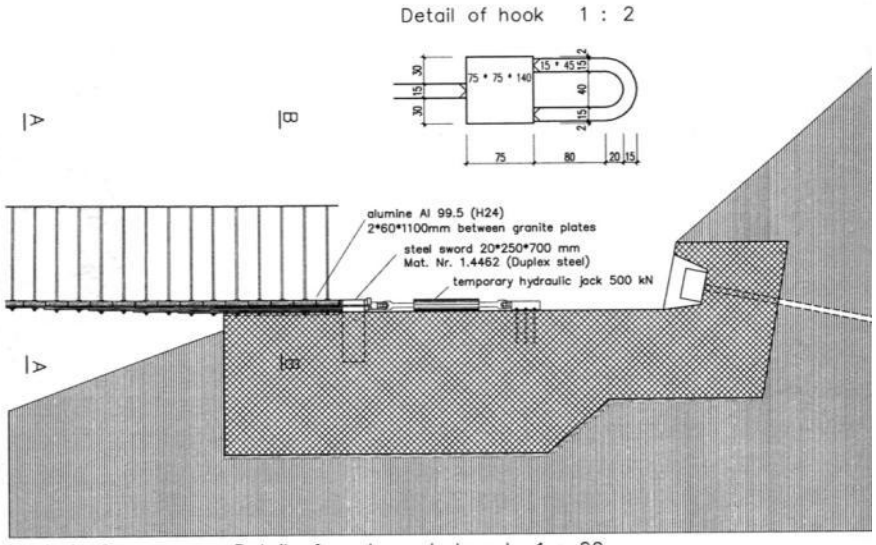

+4.00
75
20%
2 rock anchors,
682 kN, L = 13 m
2.05
1.30
0%
+0.00
75
2 soil anchors,
682 kN, L = 15 m
1.59
2.35
Hinterrhein river
5.84
40.00
6.30


steel rail 10*40 mm
Mat. Nr. 1.4435
d = 16 mm
Mat. Nr. 1.4435
1060
Andeer Granite
60 * 250 * 1100mm
60
steel ribbon 15 * 60 mm L= 42966 mm
Mat. Nr. 1.4462 (Duplex steel)
steel plate
10 * 50 * 140 mm
Mat. Nr. 1.4435
Jordahl JTA W54/40/4.0
steel V4A
125
850
125
1100


2*15*60
2*15*60
2*15*60
2*15*60
2*15*60
Jordahl JTA W54/40/4.0
steel V4A

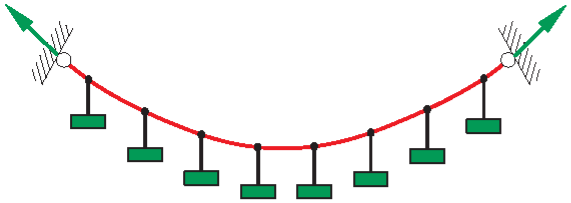


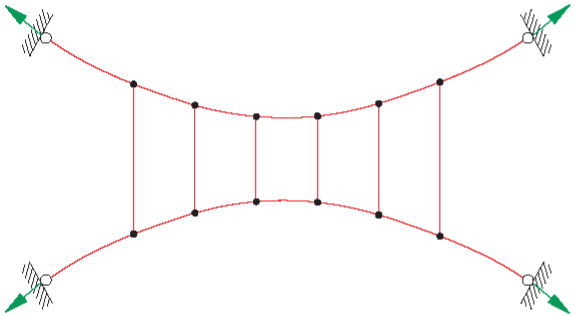


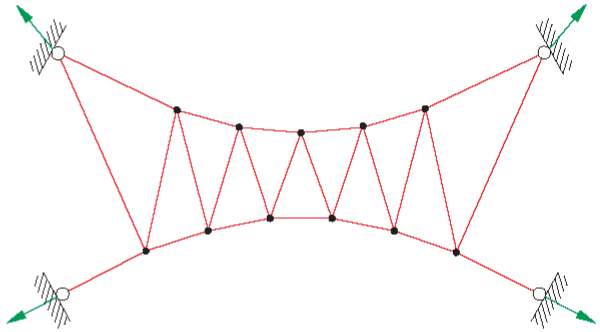


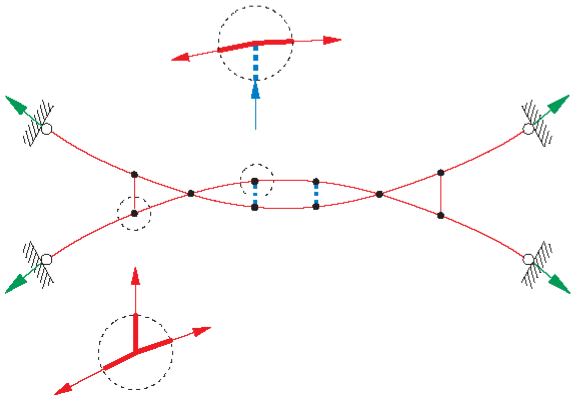
Pünt da Suransuns Detail of western abutment 1 : 20

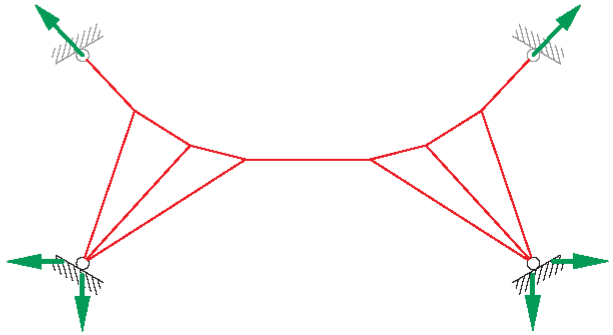
Stabilisierung von Seiltragwerken

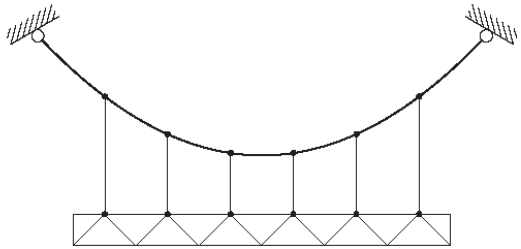


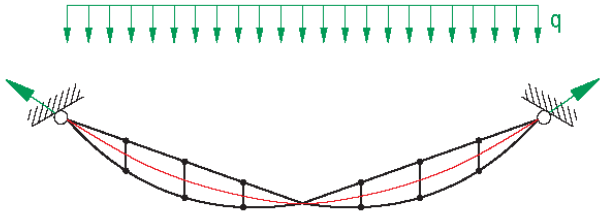








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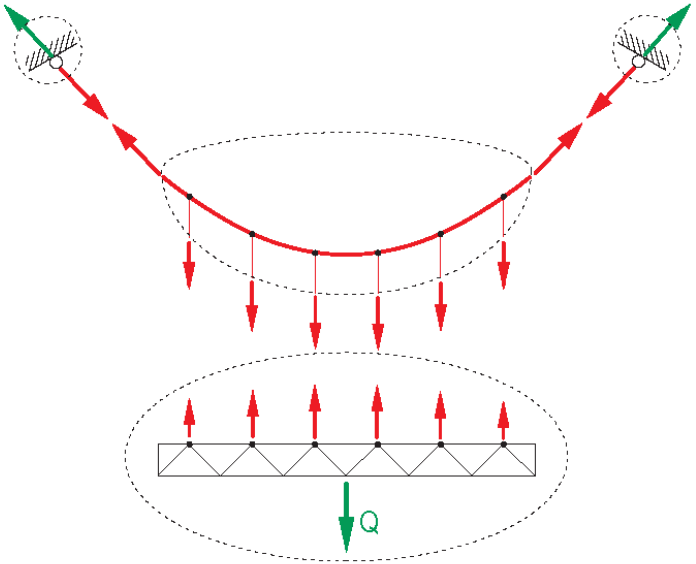
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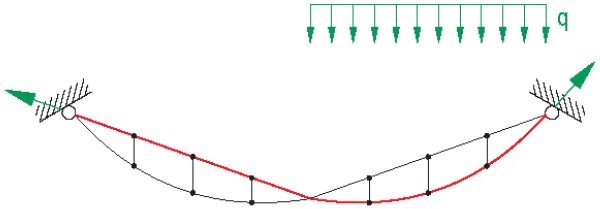
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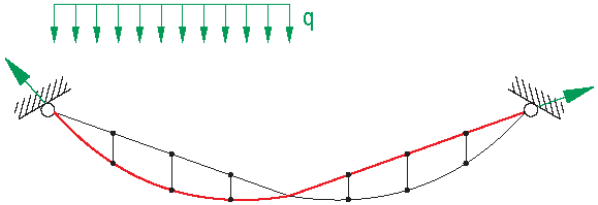
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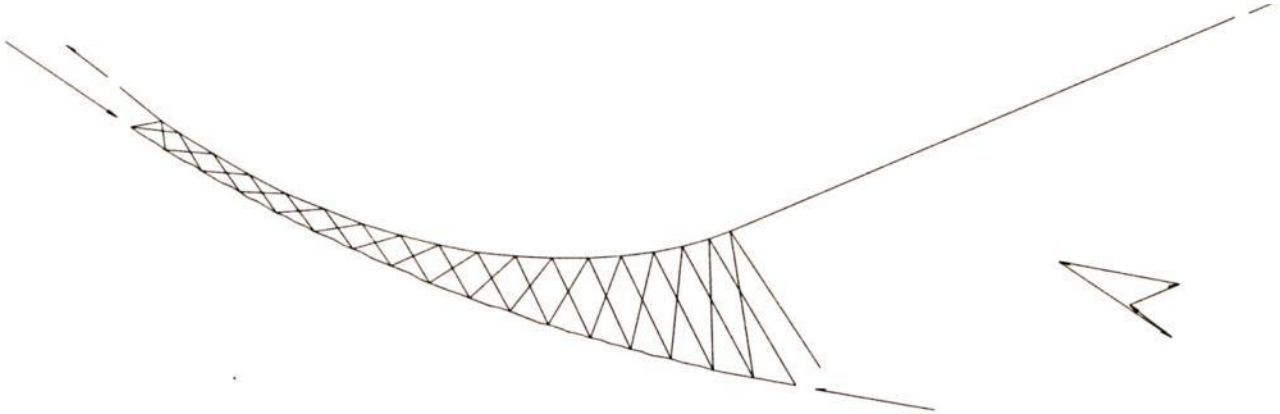
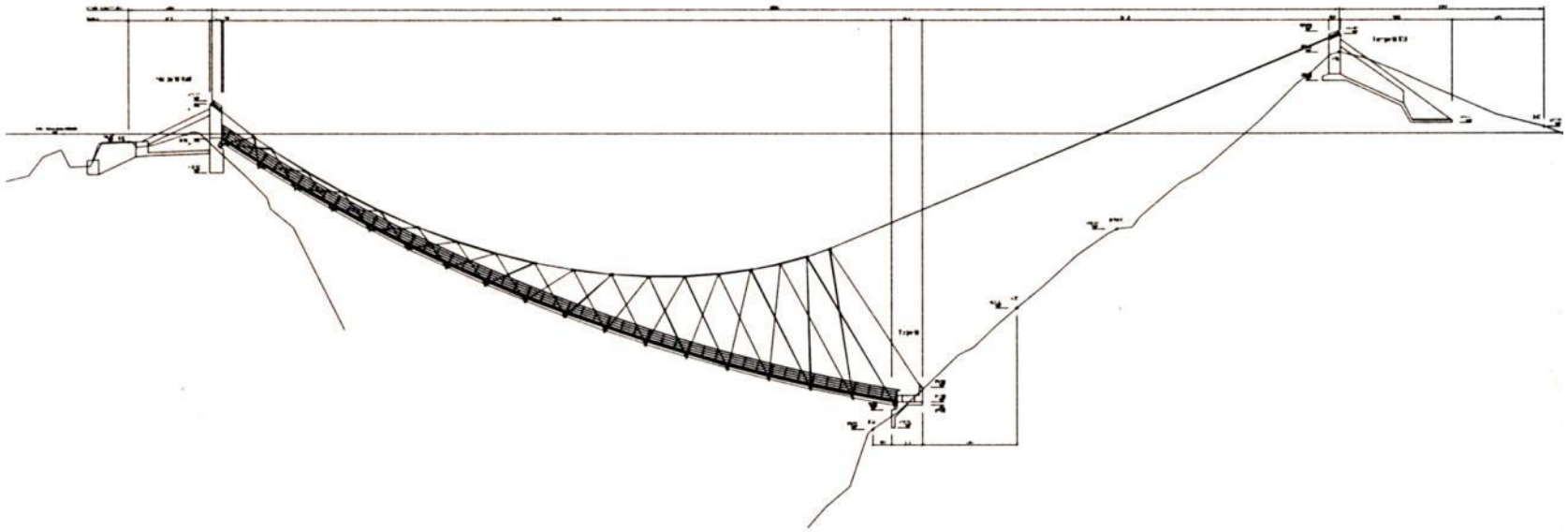
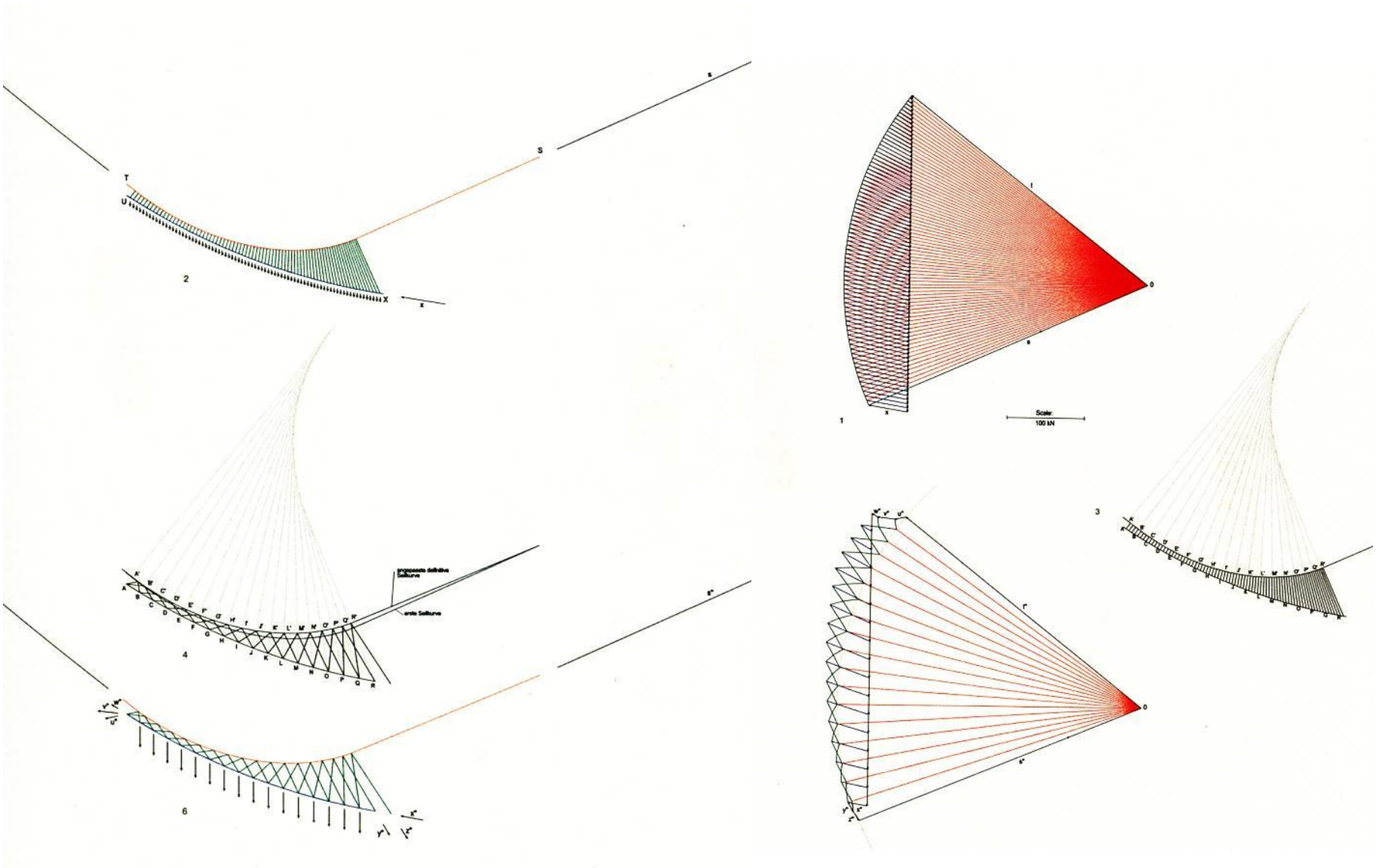
Jürg Conzett

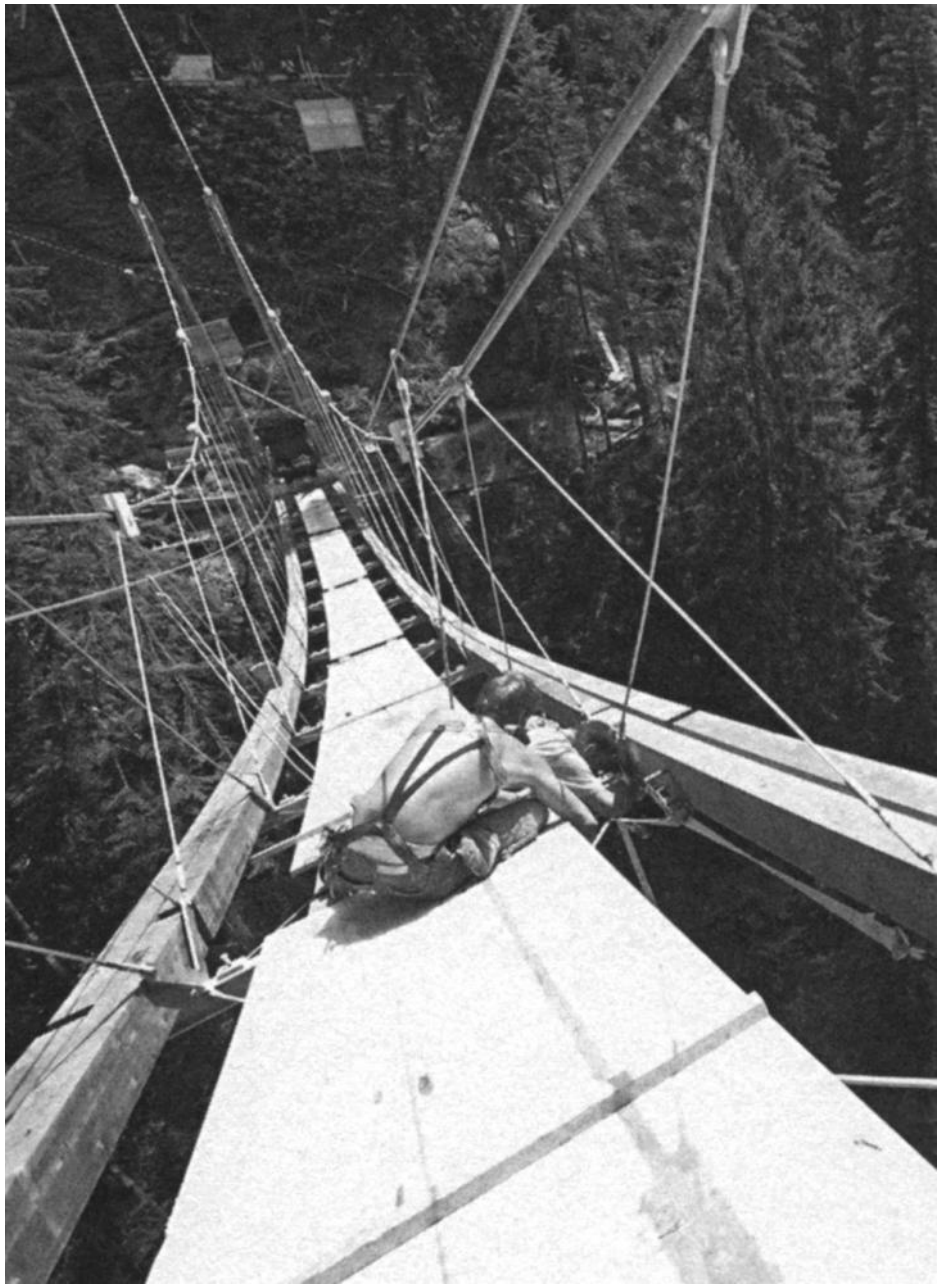
Second Traversina Bridge

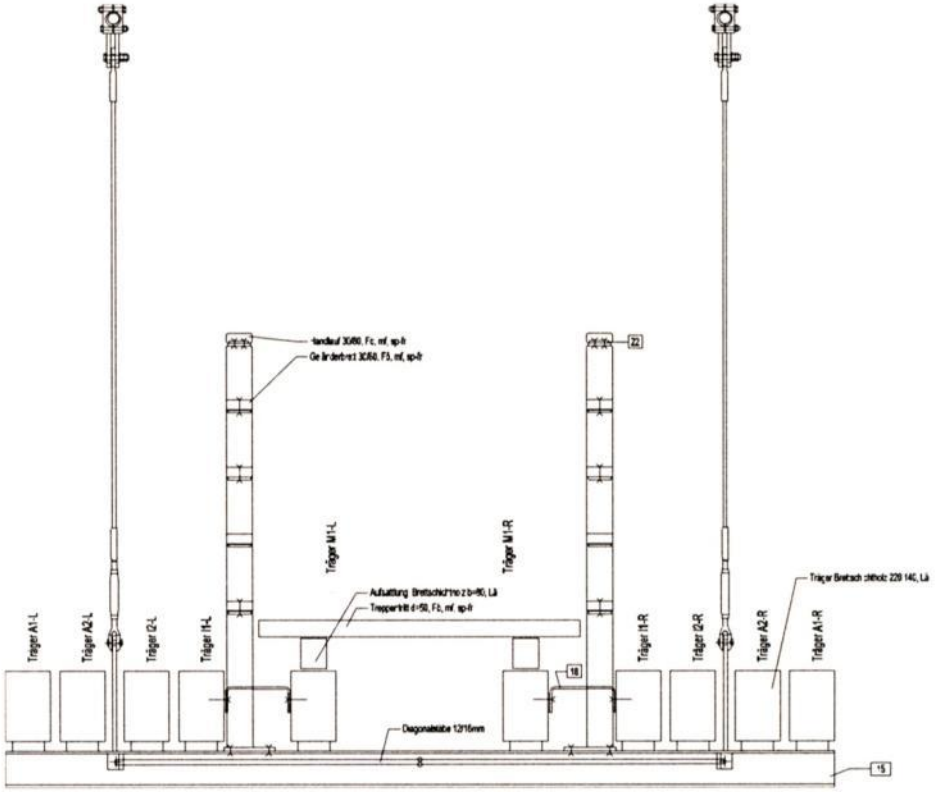
Graubunden, CH, 1995-2005



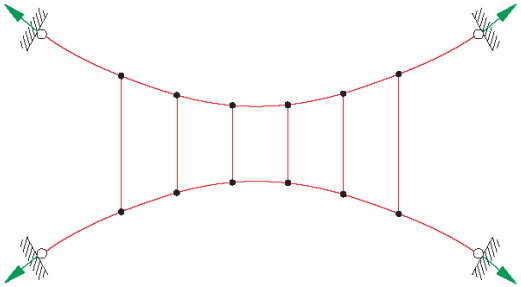


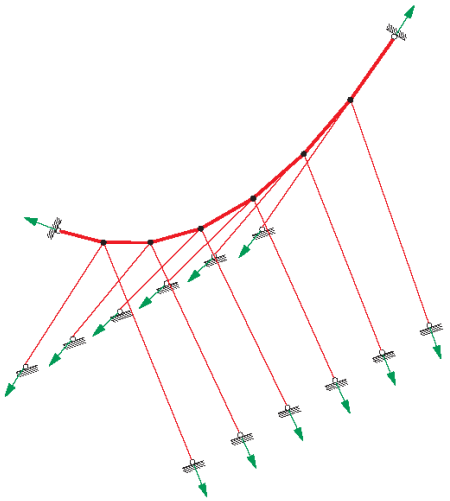

2
4
6
1
3
Scale:
100 kN

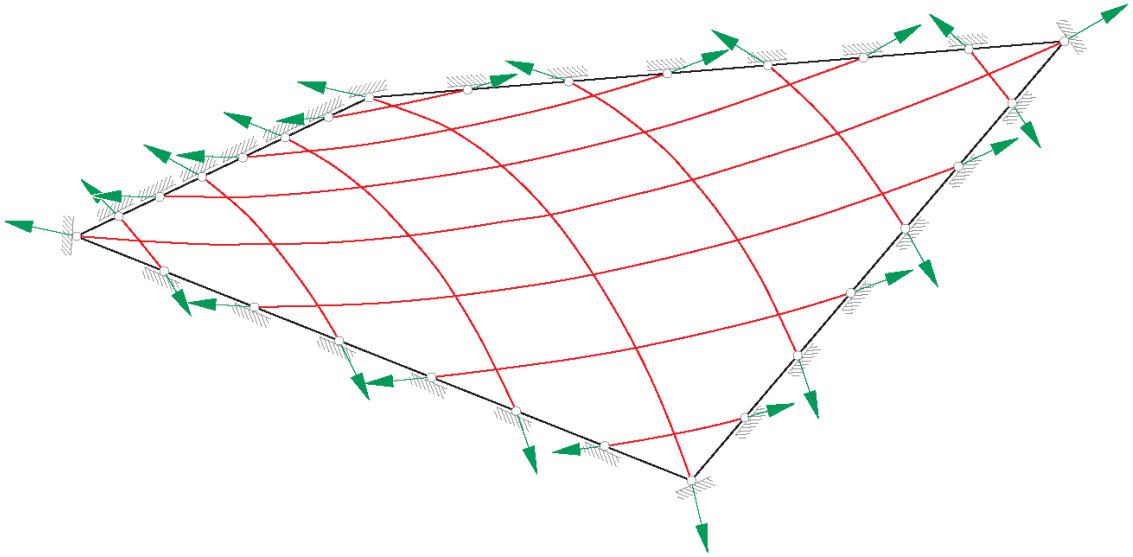



Träger A1-L
Träger A2-L
Träger I2-L
Träger I1-L
Träger M1-L
Träger M1-R
Träger I1-R
Träger I2-R
Träger A2-R
Träger A1-R
Diagonalstäbe 12/16mm

vom Seil zur
doppelt gekrümmten Membran

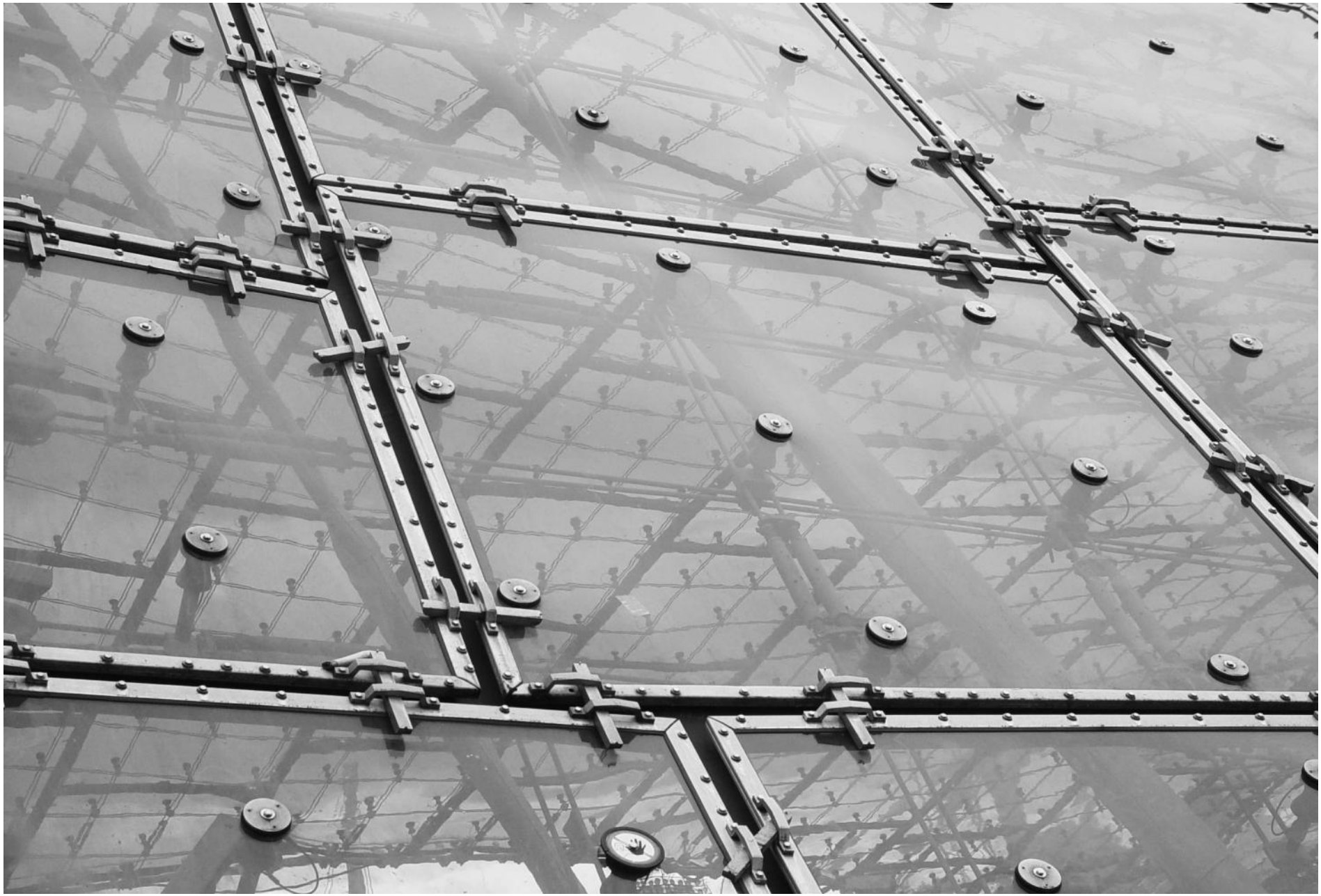






Günter Behnisch und Frei Otto

Olympiastadion

München, Deutschland, 1972

PHB