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Technical note no. 40

217 90WA

Republic of Yemen / Kingdom of the Netherlands

Rada Integrated Rural Development Project

Water reservoirs for the water supply schemes in Al Bayda Province

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Annex A - General description for the construction of masonry water reservoirs

Annex B - General description for the construction of elevated reinforced concrete water reservoirs

Annex C - Standard drawings for masonry reservoirs

MR-01 Masonry water reservoir 30 m^3

MR-02 Masonry water reservoir 50 m^3

MR-03 Masonry water reservoir 75 m^3

MR-04 Masonry water reservoir 100 m^3

MR-05 Masonry water reservoir 250 m^3

MR-06 Steel manhole cover

Annex D - Standard drawings for elevated, reinforced concrete water reservoirs

ER-01 Elevated water reservoir 50 m^3

ER-02 Elevated water reservoir 15 m^3

ER-03 Steel manhole cover

SYMBOLS

Aa	cross sectional area of main tension reinforcement.
Ab	gross cross sectional area of concrete (i.e. including cross sectional area of steel)
B 17.5	designation for quality (strenght class) of concrete
b	width of concrete cross section
c	concrete cover
Ea	modulus of elasticity of reinforcing steel, plain or deformed
e	eccentricity of a force
F	concentrated load
fa	design value for tensile strenght of reinforcing steel
h	effective depth of concrete cross section
ht	overall depth of concrete cross section
L	clear span of a beam or slab, clear height of a column
l	effective span of a beam, slab or column
lx	least effective span of a slab (width)
ly	largest effective span of a slab (length)
M	bending moment
Md	bending moment occurring at desing values of loads (design value of M)
N	normal force
Nd	normal force occurring at design values of loads (design value of N)
q	load per unit lenght of area (general)
Rd	bearing reaction occurring at design values of loads (design value of R)
T	shear force
Td	shear force occurring at design values of loads (design value of T)
j	coefficient associated with the limit state considered
f	normal stress
ft	shear stress occurring in censequence of a shear force Td
wo	geometric percentage of reinforcement of rectangular cross section, referred to effective depth = 100 Aa/bh

1. INTRODUCTION

Since 1977 the Engineering Section, previously named the Water Section, has been involved in the improvement of the domestic rural water supply in Rada district and later on in the whole Al Bayda province. In 1979 the RIRDP started to implement complete water supply schemes.
An important and expensive component of a water supply scheme is the water reservoir.
In 1982 the Engineering Section made drawings and specifications for standard masonry reservoirs and for elevated reinforced concrete reservoirs.
The standard volumes of the masonry reservoirs are 30, 50 , 75, 100 and 250 m^3.
The standard volumes of the elevated reinforced concrete reservoirs are 15 and 50 m^3.
At the end of 1989 about 70 masonry reservoirs and 20 elevated reinforced concrete reservoirs had been built.

Two Technical Notes (no. 15 in 1984 and no. 21 in 1986) have been issued, dealing with the construction of water supply schemes. Especially Technical Note no. 21 gives a complete review of all procedures and steps to be taken during implementation.

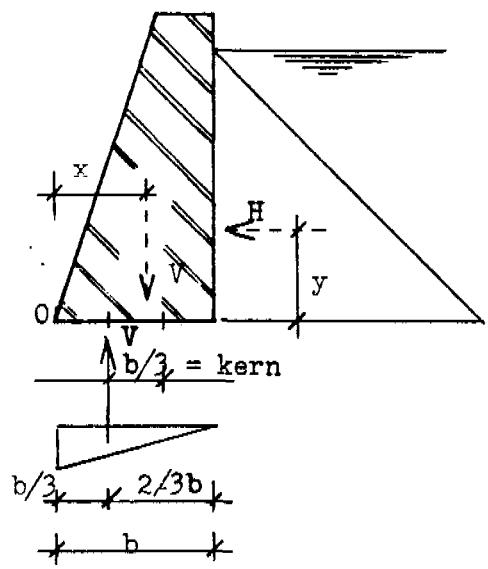
This Technical Note no. 40 focuses mainly on the structural design of the water reservoirs. Because of leakages found in some of the constructed reservoirs the drawings made in 1982 have been examined. New calculations in accordance with the latest Dutch standards have been carried out.
Earthquake has been taken into account. If necessary the drawings of the reservoirs are revised.

Detailed descriptions of all calculatons required to obtain technically sound and cost effective structures are presented in the following chapters.

Annexes A and B provide general recommendations for the construction of water reservoirs; in Annexes C and D the revised standard drawings for a number of different reservoir types are presented.

2. DESIGN CRITERIA FOR MASONRY RESERVOIRS

Dutch standards have been applied. For masonry: "TBG 1972 - steen". The permissible tensile stress of masonry is very low. It depends on a solid construction and close supervision. It is common practice to design a watertight masonry structure on a no tension basis. The masonry consists of granite blocks with a weight of 28 kN/m3. The walls will be calculated as gravity walls.
If the resultant of the horizontal and the vertical force falls within the kern of the structure, a distance of "b/6" on both sides of the centerline of the wall with width "b", no tensile force will occur.



$Mo = V.x - H.y$

$Mo/V \geq 1/3.b$

$Mo \geq 1/3.b.\ V$

$V.x - H.y \geq 1/3\ b.V$

$V.(x - 1/3\ b) \geq H.y$

0 = toe of wall
V = vertical force by the weight of the wall
H = horizontal force by the waterpressure

If $V.(x - 1/3\ b) < H.y$ then the tensile stress is

$f = V/b\ (-1 + (1/2.b - Mo/V).6/b)$

The joints between the blocks have to be filled with mortar all around, hollow spaces are not acceptable. The reservoirs will be founded on solid rock. The bottom of the reservoir is a reinforced concrete slab on an elastic foundation with fixed edges. That means that the soil under the bottom slab has to be compacted very well. To eliminate the effect of settlements of the soil the slab will usually be designed as a slab that is supported along all sides. For watertight structures the minimum thickness of reinforced concrete is 200 mm.
The concrete cover should be 25 mm.
For more design criteria for concrete structures see chapter 3.

3. DESIGN CRITERIA FOR ELEVATED REINFORCED CONCRETE RESERVOIRS

Originally the Dutch concrete specifications of 1961 (GBV) have been used for the design.
To date more stringent rules for the design of concrete water retaining structures are applicable; for reinforced concrete: VB 1974/84.
The water reservoir has to be watertight.
According to the Dutch standards VB 74/84 the minimum thickness of walls and bottom slab should be 200 mm.
The walls have to have a minimum concrete cover to the outside reinforcement of 30 mm. The bottom slab has to have a minimum cover of 25 mm.
In Yemen it is normal to use mild steel while the quality of the concrete is not high. Therefore concrete structures will be designed for mild steel FeB 220, tensile strength is 220 N/mm^2, and a quality of concrete B 17.5. For columns, however, high yield steel FeB 400 will be used!
The minimum reinforcement for watertight structures is 0.2% of the concrete area.
The walls will be considered as slabs with three fixed edges. The bottom slab will be considered as a slab with four fixed edges.
The design of the water reservoir has to be very simple. Complicated formwork is unacceptable.
Complicated formwork of reinforced concrete will affect the quality of the structure negatively, because it becomes difficult to compact the concrete properly.
The foundation of the elevated r.c. reservoirs consists of reinforced concrete columns on footings founded on solid rock.
The framework of columns and beams will be calculated taking the vertical load from the water compartment, horizontal wind forces and earthquake forces into account.

Technical data:

- concrete quality B 17.5
- allowable ultimate shear stress 0.55 N/mm^2
- allowable ultimate punching shear stress 1.10 N/mm^2
- the characteristic cube strength is 17.5 N/mm^2, which means: the <u>average</u> cube strength of concrete test cubes made on the site should reach a value of at least 27.0 N/mm^2.

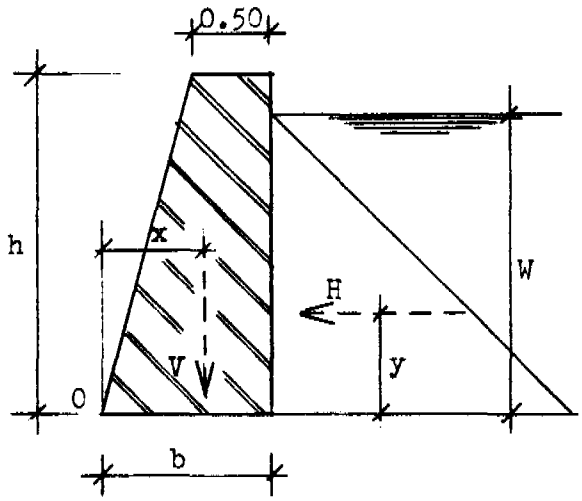
4. COMPARISON OF THE EXISTING STANDARD DESIGN OF MASONRY WATER RESERVOIRS AND R.C. ELEVATED WATER RESERVOIRS WITH THE DESIGN CRITERIA NOW APPLICABLE

4.1 Masonry reservoirs

Structural calculations of the existing masonry reservoirs are not available anymore.

4.1.1 **Examination of the walls**

a. Volume of the reservoir 30 m^3 and 50 m^3.



h = 2.75 m
W = 2.25 m
b = 1.00 m

Vertical forces (own weight)

	V(kN)	x (m)	Moment about 0 (kNm)
0.5 x 2.75 x 28 =	38.50	0.75	28.875
1/2 x 0.5 x 2.75 x 28 =	19.25	0.333	6.417
	57.75	0.61	35.292

Horizontal forces (water pressure)

H = 1/2 x 22.50 x 2.25 = 25.31 kN

y = 2.25/3 = 0.75 m

V (x - 1/3 b) = 57.75 (0,61 - 1.00/3) = 15.98 kNm

H.y = 25.31 x 0.75 = 18.98 kNm

$V\ (x - 1/3b) < H.y$

In the wall occurs unacceptable tensile stress.

The tensile stress is

$f = V/b\ (- 1 + (1/2\ b - Mo/V)6\ /b) = 57.75/1\ (- 1 + 6\ (0.5 +$

$- (35.292 - 18.89)/57.75)) = \underline{17.628\ kN/m^2}$

b. Volume of the reservoir 75 m^3

$h = 2.95$ m

$W = 2.45$ m

$b = 1.00$ m

Vertical forces:

$V = 0.5 \times 2.950 \times 28 + 1/2 \times 0.5 \times 2.95 \times 28 =$

$= 41.30 + 20.65 = \underline{61.95\ kN}$

Moment about 0:

$Mov = 41.30 \times 0.75 + 20.65 \times 0.333 = \underline{37.85\ kNm}$

$x = 37.85/61.95 = \underline{0.61\ m}$

Horizontal forces:

$H = 1/2 \times 24.5 \times 2.45 = \underline{30\ kN}$

Moment about 0:

$Moh = 30 \times 2.45/3 = \underline{24.51\ kNm} = H.y$

$V(x - 1/3b) = 61.95\ (0.61 - 1.00/3) = \underline{17.14\ kNm} < H.y$

Tensile stress:

$f = 61.95/1.00\ (-1 + 6\ (0.5 - (37.85 - 24.51)/61.951)) = \underline{43.86\ kN/m^2}$

is unacceptable.

c. Volume of the reservoir 100 m^3

$h = 3.21$ m

$W = 2.75$ m

$b = 1.00$ m

Vertical forces:

$V = 0.5 \times 3.21 \times 28 + 1/2 \times 0.5 \times 3.21 \times 28 = 44.94 + 22.47 =$

$= \underline{67.41\ kN}$

Moment about 0:

$Mov = 44.94 \times 0.75 + 22.47 \times 0.333 = \underline{41.18\ kNm}$

$x = 41.18/67.41 = \underline{0.61\ m}$

Horizontal forces:

H = 1/2 x 27.50 x 2.75 = 37.81 kN

Moment about 0:

Moh = 37.81 x 2.75/3 = 34.66 kNm = H.y

V(x - 1/3b) = 67.41 (0.61 - 1.00/3) = 18.65 kNm < H.y

Tensile stress:

f = 67.41/1.00 (- 1 + 6 (0.5 - (41.18 - 34.66)/67.41)) = 95.70 kN/m^2

d. Volume of the reservoir 250 m^3

h = 3.38 m

W = 2.88 m

b = 1.60 m

Vertical forces:

V = 0.5 x 3.38 x 28 + 1/2 x 1.10 x 3.38 x 28 = 47.32 + 52.05 =

= 99.37 kN

Moment about 0:

Mov = 47.32 x 1.35 + 52.05 x 0.733 = 102.05 kNm

x = 102.05/99.37 = 1.03 m

Horizontal forces:

H = 1/2 x 28.8 x 2.88 = 41.47 kN

Moment about 0:

Moh = 41.47 x 2.88/3 = 39.81 kNm = H.y

V(x - 1/3 b) = 99.37(1.03 - 1.6/3) = 49.35 kNm > H.y

In the wall occurs no tensile stress.

4.1.2 Examination of the bottom slab

All reservoirs have a bottom slab of reinforced concrete with a thickness of 150 mm.

The reinforcement consists of a single cross net at the bottom of the slab.

For the 250 m^3 reservoir this cross net consists of bars of ϕ 16 c/c 150 mm.
For all other reservoirs the cross net consists of bars ϕ 12 c/c 150 mm.
There is no reinforcement for negative bending moments along the edges. The thickness of the slab is less than 200 mm which is a minimum for watertight structures.

4.1.3 **Conclusions**

The bottom slab of all reservoirs has a thickness of 150 mm which is not sufficient to guarantee watertightness and there is no reinforcement for negative bending moments along the edges.

The masonry walls of the 250 m^3 reservoirs have a thickness of 1.6 m which is sufficient.
The masonry walls of the other reservoirs have a thickness of 1.0 m which is not sufficient to prevent tensile stress.
Modification of the design is recommended, to fulfill the requirements of the now applicable standards.

4.2 Elevated reinforced concrete reservoirs

Only preliminary structural calculations are available. Concerning the foundation and the frames there are no remarks.

4.2.1. **Examination of the elevated water reservoir**

There are two standard reservoirs, 50 m^3 and 15 m^3. Both reservoirs have walls and a bottom slab with a thickness of 150 mm.
The bottom slab is supported by concrete beams in both directions. Those beams together with the bottom slab, the walls and the columns of the frames form one complete structure without joints.

The walls and bottom slab of the 15 m^3 reservoir have a reinforcement consisting of a double cross net ϕ 12 c/c 125 mm. For the 50 m^3 reservoir it is ϕ 8 c/c 120 mm. For watertight reinforced concrete structures a thickness of 150 mm for walls and bottom slab is not sufficient according to the new applicable standards. According to the Dutch standards now in force this has to be 200 mm. The form work of the bottom slab with beams is rather complicated. This means that also the reinforcement is complicated and that it is difficult to pour and compact the concrete properly.

4.2.2 **Conclusion**

For a contractor it is very difficult to construct a watertight water reservoir without cracks if he has to execute it according to the existing standard designs.
Modification of the design is recommended.

5. REVIEW OF THE DESIGN AND STRUCTURAL CALCULATIONS FOR MASONRY RESERVOIRS

5.1 General cross section

See figure 2, chapter 10

Main demensions:

See figure 4-1 to 4-3, chapter 10

Reservoir (m^3)	Lx (m)	Ly (m)	W (m)	h (m)
30	3.50	4.20	2.10	2.35
50	4.50	5.20	2.20	2.45
75	4.50	6.70	2.55	2.80
100	5.00	7.20	2.85	3.10
250*	8.70	10.50	3.00	3.25

* The 250 m^3 reservoirs have a load bearing wall with a width of 1.00 m in the middle of the span.

5.2 Top slabs

See figure 4-4 and 4-5, chapter 10

Concrete: B 17.5

Reinforcement: mild steel

Concrete cover to the reinforcement: 20 mm

Thickness = t1 > 1/35 lx + 20 mm (lx in millimeters)

Dead load = 24 t1 kN/m^2 (t1 in meters)

Live load = 0.5 kN/m^2

q = dead load + live load

The possitive moment per unit length in middle section parallel to the long edge (ly) is Mvx.

The possitive moment per unit length in middle section parallel to the short edge (lx) is Mvy.

See table E8 of chapter 8

ly/lx	1.14	1.18	1.28	1.40	2.53
Mvx = 0.001 $q lx^2$ x	50.1	52.7	59.2	67.0	110.4
Mvy = 0.001 $q lx^2$ x	36.8	35.6	33.4	31.0	23.9

Reservoir (m^3)	ly (m)	lx (m)	ly/lx	t1 (mm)	dead load (kN/m^2)	q (kN/m^2)	Mvx (kNm)	Mvy (kNm)
30	4.70	4.00	1.18	150	3.6	4.1	3.46	2.34
50	5.70	5.00	1.14	175	4.2	4.7	5.89	4.32
75	7.20	5.00	1.44	175	4.2	4.7	8.15	3.57
100	7.70	5.50	1.40	200	4.8	5.3	10.74	4.97
250	11.00	4.35	2.53	200	4.8	5.3	11.07	2.40

Calculation of reinforcement:

wo = percentage of reinforcement

Ax = area of reinforcement in the x- direction = wo.b.hx

Ay = area of reinforcement in the y- direction = wo.b.hy

hx = t1 - 20 (cover to reinf.)- 5 (1/2 diameter of bar) = t1 - 25 mm

hy = t1 - 20 (cover to reinf.)- 10 (diameter of bar) - 5 (1/2 diameter of bar) = t1 - 35 mm

b = 1000 mm (for slabs only)

The minimum area of reinforcement =

= A min = 0.15/100 x b x t1 =

= 0.15/100 x 1000 x t1 = 1.5 t1 mm^2

Reservoirs (m^3)	Mvx (kNm)	wo (%)	Ax (mm^2)	Mvy (kNm)	wo %	Ay (mm^2)	A min. (mm^2)
30	3.46	0.17	217	2.34	0.14	159	225
50	5.89	0.21	309	4.32	0.17	242	263
75	8.15	0.29	431	3.57	0.14	200	263
100	10.74	0.28	486	4.97	0.14	236	300
250	11.07	0.29	501	2.40	0.07	113	300

Reservoirs (m^3)	Reinforcement	
	x-direction	y-direction
30	ϕ 8 c/c 200	ϕ 8 c/c 200
50	ϕ 8 c/c 150	ϕ 8 c/c 175
75	ϕ 10 c/c 175	ϕ 8 c/c 175
100	ϕ 10 c/c 150	ϕ 8 c/c 150
250	ϕ 12 c/c 175	ϕ 8 c/c 150

5.3 Masonry walls of granite blocks.

See figure 4-6, chapter 10

The load per meter on the top of the long wall by the weight of the top slab is

$$Px = (2\,Ly - Lx + 1).\frac{Lx + 1}{4(Ly + 1)}.\,24.t1$$

When the weight of the granite blocks is 28 kN/m^3, than the weight of the wall per meter is $V = 7\,b\,(1+2h) + 7h - 3.5$

The horizontal load per meter on the wall by waterpressure is $H = 5\,W^2$.

The resultant of the horizontal and the vertical forces will fall within the kern of the wall, because no tensile force is accepted. That means that the width of the wall at bottom level is

$b = 3.(Px.\,x1 + V.x2 - H.1/3W)/(Px + V)$

when $k = 14\,h$

$l = 7\,h - 3.5 + 2\,Px$

$m = 1.75 - 0.75\,Px - 5W^3 - 3.5\,h$

Than $b = \dfrac{-l + \sqrt{l^2 - 4\,km}}{2k}$ = width of the wall at bottom level

When $Q = V + Px$

than the maximum stress in the masonry wall is $f = 2Q/b$

Reservoir (m^3)	Px (kN/m)	H (kN/m)	V (kN/m)	Q (kN/m)	f (kN/m^2)	b (m)
30	4.60	22.05	53.39	57.98	114.43	1.01
50	6.43	24.20	56.70	63.13	121.13	1.04
75	7.43	32.51	72.62	80.05	130.86	1.22
100	9.13	40.61	87.34	96.47	140.65	1.37
250	10.75	45.00	94.74	105.50	146.73	1.44

5.4 Foundation on solid rock

5.4.1 Foundation under walls

See figure 4-7, chapter 10

The loading by the weight of the top slab and the wall = Q kN/m.

When the thickness of the bottom slab is t2, than the load by the bottom slab is $q2 = 24\ t2\ kN/m^2$ (t2 in meters).

The vertical load by water in the reservoir is $q3 = 10W\ kN/m^2$ (W in meters).

The rotation in point 1 is zero and also the deflection in point 1 is zero.

With these two permisses we find that

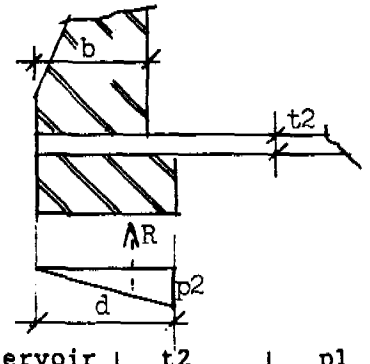
$P2 = 4\ R/b - 6\ Mr/b^2$

$p_1 = 2\ R/b - p2$

In wich $R = 1/4(q2 + q3).\ Lx + b(1/2f + q2)$

and $Mr = (1/3f + 1/2\ q2)\ b^2 - 1/48(q2 + q3)\ Lx^2$

If p1 < 0 then the width of the foundation has to be increased to d.



In that case

$d = 1.5\ (b - Mr/R)$

$p2 = 2R/d$

Reservoir (m^3)	t2 (m)	p1 (kN/m^2)	p2 (kN/m^2)	d (m)
30	0.20	36.21	132.37	b
50	0.20	5.62	182.96	b
75	0.20	28.68	167.51	b
100	0.20	29.46	181.47	b
250	0.25	0	196.36	1.96

Excluding the weight of the foundation blocks, the maximum soil pressure is 196.36 kN/m^2, wich is allowable.

5.4.2 **Foundation under bottom slab**

See figure 4-7, chapter 10

V = vertical force from bottom slab = Lx/2 (q2 + q3)

c = width of foundation

f = soil pressure excluding the weight of the foundation blocks

Reservoir (m^3)	V (kN)	c (m)	f (kN/m^2)
30	45.15	0.5	90.30
50	60.30	0.7	86.14
75	68.18	0.75	90.91
100	91.58	1.00	91.58
250	268.48	1.50	178.99

* The 250 m^3 reservoirs have a load bearing wall in the middle of the span Lx, with a width of 1 m and a height of 3.25m.
  So V = Lx/2 (q + q2 + q3) + 1 x 3.25 x 28 kN

5.5 Bottom slab

Concrete: B 17.5

Reinforcement: mild steel

Concrete cover to the reinforcement : 25 mm

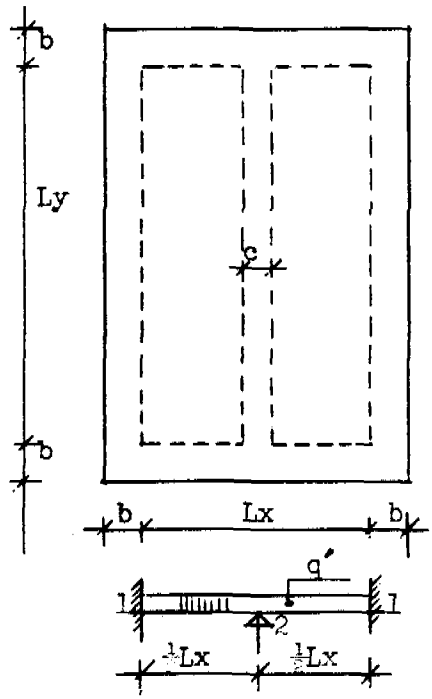
The thickness of the bottom slab has to be at least 200 mm according to the Dutch standards.

That means that the slab is stiff and will be supported along the edges. In that way the bending moments are high. Therefore it is necessary to support the slab also in the middle of the short span.

Bending moments

$M1 = M2 = -1/48.q'.Lx^2$

$M12 = 1/96.q'.Lx^2$



Thickness of slab is t2 = 200 mm.

Waterheight is W.

Uniform load $q' = 10\ W + 24t2\ kN/m^2$ (W and t2 in meters)

Reservoir (m^3)	Lx (m)	W (m)	q' (kN/m^2)	M12 (kNm)	-M1 = - M2 (kNm)
30	3.50	2.10	25.8	3.29	6.58
50	4.50	2.20	26.8	5.65	11.31
75	4.50	2.55	30.3	6.39	12.78
100	5.00	2.85	33.3	8.67	17.34
250*	8.70	3.00	36.0	28.39	56.77

* The bottom slab of the 250 m^3 reservoir has a thickness of t2 = 250 mm

Calculation of the reinforcement:

wo = percentage of reinforcement

Ax = area of reinforcement in the x - direction

Ay = 20% of Ax, is area of reinforcement in the y-direction.

The minimum area of reinforcement is A min = 0.2/100 x 1000 x t2 =

= 2 t2 mm^2 (t2 in millimeters)

Reservoir (m^3)	-M1 = - M2 (kNm)	wo (%)	Ax (mm^2)	Ay (mm^2)	+M12 (kNm)	Ax (mm^2)	Ay (mm^2)	Amin (mm^2)
30	6.58	0.18	304	61	3.29	151	31	400
50	11.31	0.31	528	106	5.65	260	52	400
75	12.78	0.35	599	120	6.39	295	59	400
100	17.34	0.48	823	165	8.67	402	81	400
250	56.77	1.02	2218	444	28.39	1055	211	500

6. REVIEW OF THE DESIGN AND STRUCTURAL CALCULATIONS FOR ELEVATED REINFORCED CONCRETE RESERVOIRS

Concrete: B 17.5

Reinforcement:

- columns high yield steel FeB 400
- walls, slabs, beams and footings mild steel FeB 220

Concrete cover:

- walls and bottom slab of the water compartment 25 mm
- columns, beams and footings 30 mm
- platform slab and top slab 15 mm

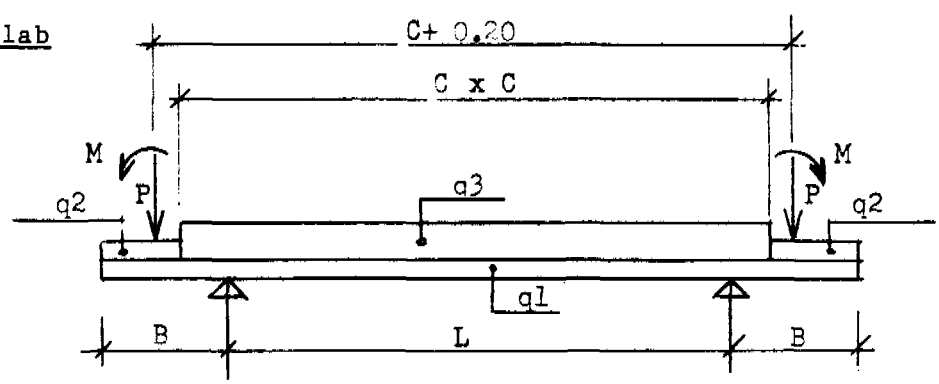
6.1 General cross section of water reservoir

See figure 3 , chapter 10

Main dimensions

Reservoir (m^3)	C (m)	H (m)	d3 (m)	L (m)	B (m)	d1 (m)	d2 (m)
15	3.04	1.75	0.30	2.70	0.67	0.20	0.15
50	4.84	2.40	0.40	4.40	0.72	0.25	0.20

6.2 Bottom slab

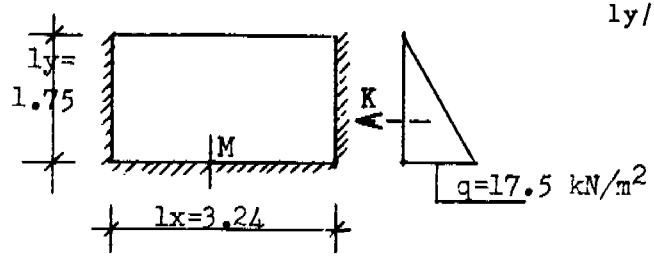


6.2.1 **Maximum negative moment in bottom slab**

a. 15 m^3 reservoir

Moment by waterpressure against wall

ly/lx = 1.75/3.24= 0.55



K is the total load on the wall by the waterpressure.

M is the bending moment in the wall at bottom level.

K = 1/2 x 17.5 x 1.75 x 3.240 = 49.61 kN

M = 84 x 49.61/1000 = 4.17 kNm/m

Weight of topslab (d2 = 150 mm)

3.44^2 x 0.15 x 24 x 1/4 x 1/3.44 = 3.10 kN/m

Weight of wall (t = 200 mm)

1.75 x 0.2 x 24 = 8,40 kN/m

P = 11,50 kN/m

Weight of bottom slab (d1 = 200mm)

q1 = 0.2 x 24 = 4.8 kN/m^2

q2 = 0.1 x 24 = 2.4 kN/m^2

Water on slab

q3 = 1.75 x 10 = 17.5 kN/m^2

The maximum negative bending moment is

- Ms = 1/2 x 4.8 x 0.67^2 + 2.4 x 0.5 x 0.42 + 17.5 x 1/2 x

x 0.17^2 + 11.5 x 0.275 + 4.17 = 9.17 kNm

The shear force by the cantilever is

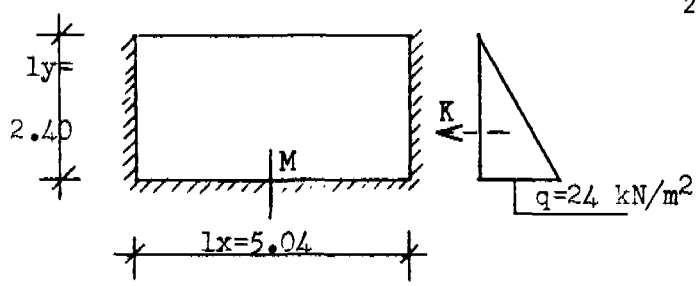
Ts = 0.67 x 4.8 + 2.4 x 0.42 + 17.5 x 0.17 + 11.5 = 18.70 kN

b. 50 m^3 reservoir

Moment by waterpressure against wall

ly/lx =

2.40/5.04 = 0.48



K = 1/2 x 24 x 5.04 x 2.4 = 145.15 kN

M = 85/1000 x 145.15 = 12.33 kNm

Weight of topslab (d2 = 200 mm)

5.24^2 x 0.2 x 24/4 x 1/5.24 = 6.29 kN/m

Weight of wall (t = 200mm)

2.40 x 02 x 24 = 11.52 kN/m

P = 17.81 kN/m

Weight of bottom slab (d1 = 250 mm)

q1 = 0.25 x 24 = 6 kN/m^2

q2 = 0.1 x 24 = 2.4 kN/m^2

Water on slab

q3 = 2.4 x 10 = 24 kN/m^2

The maximum negative moment is

- Ms = 1/2 x 6 x 0.72^2 + 2.4 x 0.5 x 0.47 + 24 x 1/2 x

x 0.22^2 + 17.81 x 0.32 + 12.3 = 20.73 kNm

The shear force by the cantilever is

Ts = 6 x 0.72 + 2.4 x 0.47 + 24 x 0.22 + 17.81 = 28.54 kN

6.2.2 **Maximum positive moment in bottom slab**

a. 15 m^3 reservoir

weight of slab (d1 = 200 mm)

0.2 x 24 = 4.8 kN/m^2 = q1

water 17.5 kN/m^2 = q3

22.3 kN/m^2 = q1 + q3

Bending moments in x and y direction:

Mx = 41/1000 x 22.3 x 2.7^2 - 9.17 = - 2.5 kNm

My = Mx

no positive moment!

Shear force:

Ts = 2.7/2 x 22.3 = 30.11 kN/m

b. 50 m^3 reservoir

weight of slab (d1 = 250 mm)

0.25 x 24 = 6 kN/m^2 = q1

water 24 kN/m^2 = q3

30 kN/m^2 = q1 + q3

Bending moments in x and y directions:

My = Mx = 41/1000 x 30 x 4.4^2 - 20.73 = 3.08 kNm

Shear force:

Ts = 4.4/2 x 30 = 66 kN/m

6.2.3 **Check on shear force**

a. 15 m^3 reservoir

Max. shear force is T max. = 30.11 kN/m

200 mm bottom slab

h = 200 - 25 - 10 - 5 = 160 mm

Max. shear stress is ft = 1.7 x 30.11 /160 = 0.32 N/mm^2 is acceptable.

b. 50 m^3 reservoir

Max. shear force is T max = 66 kN/m

250 mm bottom slab

h = 250 - 25 - 10 - 5 = 210 mm

Max. shear stress is ft = 1.7 x 66/210 = 0.54 N/mm^2 is acceptable

6.2.4 **Reinforcement**

a. 15 m^3 reservoir.

d1 = 200 mm.

Minimum area of reinforcement is A min = 400 mm^2

wo is percentage of reinforcement

- Mx = - My = 9.17 kNm.

wo = 0.28 Aa = 454 mm^2

- Mx = - My = 2.5 kNm

wo = 0.08 Aa = 400 mm^2 (minimum reinforcement A min)

b. 50 m^3 reservoir.

d1 = 250 mm

Minimum area of reinforcement is A min = 500 mm^2

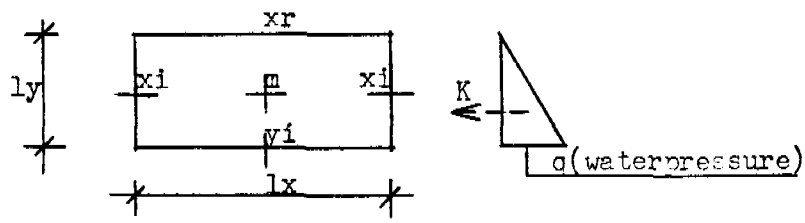
- Mx = - My = 20.73 kNm

wo = 0.38 Aa = 788 mm^2

+ Mx = + My = 3.08 kNm

wo = 0.05 Aa = 500 mm^2 (minimum reinforcement Amin)

6.3 Walls (t = 200 mm)



Reservoir (m^3)	ly (m)	lx (m)	ly/lx	q (kN/m^2)	K (kN)	M (kNm)
15	1.75	3.24	0.54	17.5	49.61	Mx =27 /1000x49.61=1.34
						My =15.5/1000x49.61=0.77
						Mix=63 /1000x49.61=3.13
						Miy=84 /1000x49.61=4.17
						Mxr=27 /1000x49.61=1.34
50	2.40	5.04	0.48	24	145.15	Mx =26/1000x145.15= 3.78
						My =14/1000x145.15= 2.04
						Mix=68/1000x145.15= 9.87
						Miy=85/1000x145.15=12.34
						Mxr=26/1000x145.15=53.78

t = 200 mm

Minimum area of reinforcement Amin = 400 mm^2

a. Reservoir 15 m^3.

The maximum moment is Mmax. = 4.17 kNm = Miy

wo = 0.11 Aa = $\underline{400\ mm^2}$ = Amin

Shear: Tmax = 1/2 x 17.5 x 1.75 = 15.31 kN/m

Shear stress: ft = (1.7 x 15.31)/160 = $\underline{0.16\ N/mm^2}$

b. Reservoir 50 m^3.

Shear: Tmax = 1/2 x 24 x 2.4 = 28.8 kN/m

Shear stress: ft = 1.7 x 28.8/160 = $\underline{0.31\ N/mm^2}$

Mx = 3.78 kNm, h = 160 mm, wo = 0.12, A = $\underline{400\ mm^2}$ = Amin

My = 2.04 kNm, h = 170 mm, wo = 0.05, A = $\underline{400\ mm^2}$ = Amin

Mix = 9.87 kNm, h = 160 mm, wo = 0.31, A = $\underline{490\ mm^2}$

Miy = 12.34 kNm, h = 170 mm, wo = 0.34, A = $\underline{578\ mm^2}$

Mxr = 3.78 kNm, h = 160 mm, wo = 0.12, A = $\underline{400\ mm^2}$ = Amin

6.4 Top slab

lx = ly = C + t (see fig. 3, chapter 10)

Reservoir (m^3)	lx = = ly (m)	d2 (m)	live load (kN/m^2)	dead load (kN/m^2)	q = total load (kN/m^2)
15	3.24	0.15	0.5	3.60	4.10
50	5.04	0.20	0.5	4.80	5.30

a. 15 m^3 reservoir

d2 = 150 mm

Minimum area of reinforcement is Amin = 300 mm^2

Maximum bending moments in x and y direction:

Mx = My = 41/1000 x 4.1 x 3.24^2 = 1.76 kNm

Percentage of reinforcement wo = 0.10

Aa = $\underline{300\ mm^2}$ = Amin

b. 50 m^3 reservoir

d2 = 200 mm

Minimum area of reinforcement is Amin = 400 mm^2

Maximum bending moments in x and y direction:

Mx = My = 41/1000 x 5.3 x 5.04^2 = 5.52 kNm

Percentage of reinforcement wo = 0.15

Aa = $\underline{400\ mm^2}$ = Amin

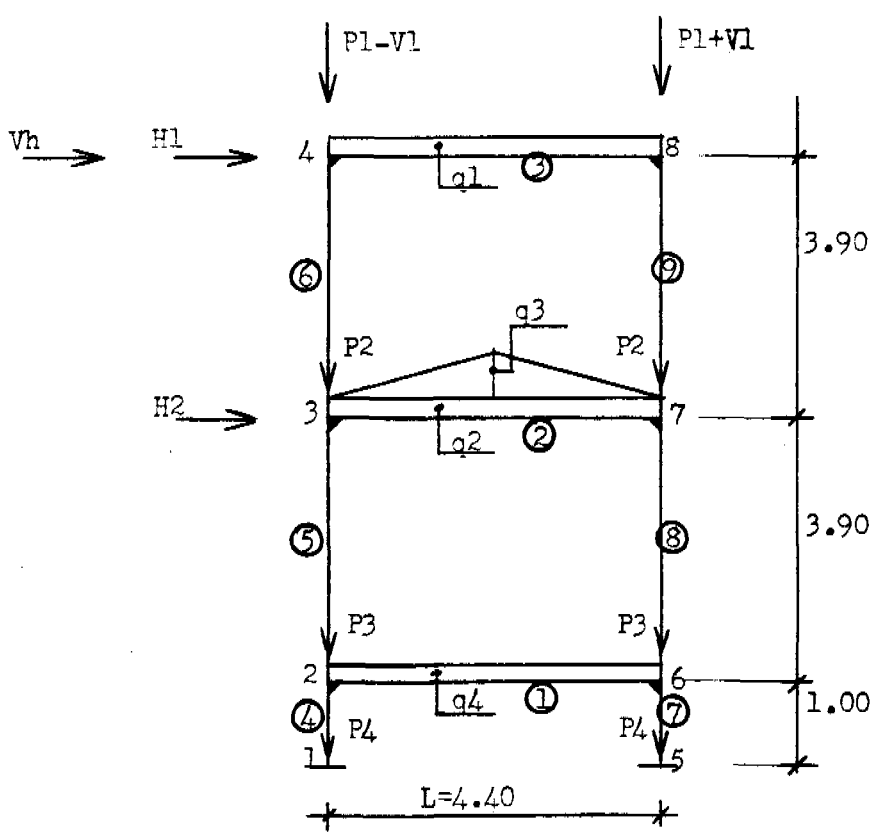
6.5 Frames of 50 m^3 reservoir

Scheme:

See figure

member	node i	node j
1	2	6
2	3	7
3	4	8
4	1	2
5	2	3
6	3	4
7	5	6
8	6	7
9	7	8

- q1, q2 and q4 are uniform loads.
- q3 is a triangular load.
- P1, P2, P3 and P4 are vertical concentrated loads in the nodes.
- H1 and H2 are horizontal concentrated wind loads.
- Vh is a horizontal concentrated earthquake load.
- V1 is the vertical concentrated load by the overturning moment of the wind load.



a. Wind load:

Wind load is 0.8 kN/m^2

Wind load factor is 1.2

	kN	kNm
5.24 x 2.61 x 1.2 x 0.8 =	13.13 x 1.58 =	20.75
5.24 x 5.24 x 0.04 x 0.8 =	0.88 x 2.88 =	2.53
2(5.24 x 2.61 x 0.04 x 0.8)=	0.88 x 1.58 =	1.38
5.84 x 0.35 x 1.2 x 0.8 =	1.97 x 0.075 =	0.15
2(5.84 x 0.35 x 0.04 x 0.8) =	0.13 x 0.075 =	--
2(5.84 x 0.3 x 0.04 x 0.8) =	0.12 x 0.27 =	0.03
	17.11	24.84
0.35 x 4.8 x 1.2 x 0.8 =	1.62	
(0.4 x 0.4 x 3.4 x 1.2 x 0.8)/2 x 4 =	1.05	
	19.78	

H1 = 19.78/2 = 9.89 kN per frame

0.5 x 4.8 x 1.2 x 0.8 =	2.31 kN
2(0.5 x 4.8 x 0.04 x 0.8) =	0.16 kN
2(4.8 x 4.8 x 0.04 x 0.8) =	1.48 kN
(0.4 x 0.4 x 3.4 x 1.2 x 0.8)/2 x 4 =	1.05 kN
	5.00 kN

H2 = 5/2 = 2.50 kN per frame:

b. Vertical load per frame:

Wind:

V1 = 24.84/4.4 x 0.5 = + 2.83 kN

V2 = - 2.83 kN

Reservoir:

top slab (5.78 x 5.78 x 0.4 - 5.28 x 5.28 x 0.2) x 24 = 186.91 kN

bottom slab (5.84 x 5.84 x 0.45 - 4.84 x 4.84 x 0.2) x

x 24= 255.90 kN

walls (5.24 x 5.24 - 4.84 x 4.84) x 2.4 x 24 = 232.25 kN

water 50 m3 500.00 kN

1175.06 kN

top beams:

(4.75 x 4.75 - 4.05 x 4.05) x 0.75 x 24 = 110.88 kN

1285.94 kN

q1 = 1285.94/(4 x 4.4) = 73.06 kN/m

middle beams:

q2 = 0.35 x 0.5 x 24 = 4.20 kN/m

middle floor slab:

q3 = 0.15 x 24 x 4.05/2 = 7.29 kN/m

ground beams:

q4 = 0.35 x 0.40 x 24 = 3.36 kN/m

Inclusive the weight of the columns:

P1 = 2.2 x 73.06 = 160.74 kN

P2 = (4.4 x 4.2 + 0.5 x 4.4 x 7.29) x 0.5 + 0.4 x 0.4 x 24 x

x 3.9 = 32.24 kN

P3 = 4.4 x 3.36 x 0.5 + 0.4 x 0.4 x 24 x 3.9 = 22.37 kN

P4 = 0.4 x 0.4 x 24 x 1 = 3.84 kN

Total vertical load (live load + dead load):

level +8.80	2 x 160.74 + 4.4 x 73.06 =	642.95 kN
level +4.90	2 x 32.24 + 4.4 x 4.2 + 2.2 x 7.29 =	99.00 kN
level +1.00	2 x 22.37 + 4.4 x 3.36 =	59.52 kN
ground level	2 x 3.84 =	7.68 kN
footings	2(0.4 x 24 x 1.9 x 1.9) =	69.31 kN
		W =878.46 kN

c. Earthquake load

According the ACI (American Concrete Institute) code the base shear force is V =

= ZKCW where Z corresponds to the American seismic zones (Z = 1 for areas of the highest seismicity),

K = 2.5 (a constant for the type of structure),

C = 0.05/(T)^1/3 = 0.08504 (for elevated reservoirs T = 0.2),

V = 1 x 2.5 x 0.08504 x 878.46 = 186.76 kN

The shear force at the top is Vh = 0.15 x 186.76 = 28.01 kN

The remainder of the shear is distributed vertically so that force Fx = (V - Vh).Vx.hx/(Vx.hx)

Vx.hx = 642.95 x 8.8 + 99 x 4.9 + 59.52 x 1 = 6202.58 kNm.

Level +8.80 Fx = (186.76 - 28.01) x 642.95 x 8.8/6202.58 =
= 144.81 kN

Level +4.90 Fx = (186.76 - 28.01) x 99 x 4.9/6202.58 =
= 12.41 kN

Level +1.00 Fx = (186.76 - 28.01) x 59.21 x 1 = 1.53 kN

So in the case of earthquake with live load and dead load:

q1 = 73.06 + 144.81/8.8 = 73.06 + 16.46 = 89.52 kN/m

q2 = 4.2 + 12.41/8.8 = 4.2 + 1.41 = 5.61 kN/m

q3 = 7.29 kN/m

q4 = 3.36 + 1.53/8.8 = 3.36 + 0.174 = 3.54 kN/m

P1 = 160.74 + 2.2 x 16.46 = 196.52 kN

P2 = 32.24 + 2.2 x 1.41 = 35.34 kN

P3 = 22.37 + 2.2 x 0.174 = 22.75 kN

P4 = 3.84 kN

Vh = 28.01 kN

The safety factor for that load case is 1.2 instead of 1.7

d. Modulus of elasticity

E is the modulus of elasticity

I is the moment of inertia

Columns:

dimension 0.40 x 0.40 m

assume that the percentage of reinforcement wo = 0.75

$E = 1/3 \times (8wo + 10) \times 10^6 = 5.33 \times 10^6$ kN/m^2

$EI = 5.33 \times 10^6 \times 1/12 \times 0.40 \times 40^3 = 11370$ kNm^2

Top beams:

dimension 0.35 x 0.75 m

assume wo = 0.5

$E = (7.5wo + 2.5) \times 10^6 = 6.25 \times 10^6$ kN/m^2

$EI = 6.25 \times 10^6 \times 1/12 \times 0.35 \times 0.75^3 = 76904$ kNm^2

Middle beams:

dimension 0.35 x 0.5 m

assume wo = 0.68

$E = (7.5wo + 2.5) \times 10^6 = 7.60 \times 10^6$ kN/m^2

$EI = 7.60 \times 10^6 \times 1/12 \times 0.35 \times 0.5^3 = 27708$ kNm^2

Ground beams:

dimension 0.35 x 0.40 m

assume wo = 1.00

$E = (7.5wo + 2.5) \times 10^6 = 10 \times 10^6$ kN/m^2

$EI = 10.00 \times 10^6 \times 1/12 \times 0.35 \times 0.4^3 = 18666$ kNm^2

e. Load case

1. dead load + live load + wind
2. dead load + live load + earthquake

f. Calculation of the member forces

See computer output at page 7 to 14, chapter 9.

g. Calculation of the second moment factor n/(n - 1)

The second moment factor n/(n-1) depends on the critical compression loads of the members.

The critical compression load according the theory of Euler is $Nk' = 3.14^2 \, EI/Lk^2$ in wich formula

E = modulus of elasticity

I = moment of inertia

Lk = buckling length

In paragraph 6.5 d the value of EI has been calculated for the several members.

The buckling length of the columns depends on the EI value and the end conditions of the columns and of the properties of the connected members to the columns.

For the members no. 4, 5 and 6 the buckling length has been calculated with a computer program.

The result is:

for member no. 4 Lk = 2.51 m

for member no. 5 Lk = 4.31 m

for member no. 6 Lk = 4.23 m

$n = Nk'/Nd'$ in wich

Nk' = critical compression load

Nd' = design value of the compression load N'

The design value Nd' = 1.7 N'

$Nk' = 3.14^2 \times (EI)/Lk^2$

columns 4 and 7: $Nk' = 3.14^2 \times 11370/(2.51^2) = 17812$ kN

columns 5 and 8: $Nk' = 3.14^2 \times 11370/(4.31^2) = 6040$ kN

columns 6 and 9: $Nk' = 3.14^2 \times 11370/(4.23^2) = 6271$ kN

Load case 1:

column	N'	Nd'	Nk'	n	n/(n - 1)
	kN	kN	kN		
4	389	662	17812	26.91	
5	364	619	6040	9.76	
6	319	543	6271	11.45	
7	412	701	17812	25.40	
8	379	643	6040	9.40	1.12
9	324	551	6271	11.38	

All member forces and displacements by the horizontal loads to be multiplied with 1.12

Load case 2:

Column	N'	Nd'	Nk'	n	n/(n - 1)
	kN	kN	kN		
4	424	509	17812	35.00	
5	412	495	6040	12.20	
6	380	456	6271	13.75	
7	536	643	17812	27.70	
8	487	585	6040	10.33	1.11
9	408	598	6271	10.49	

All member forces and displacements by the horizontal loads to be multiplied with 1.11.

h. Ground beams

Concrete cover to the reinforcement 25 mm

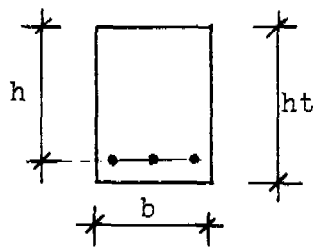
Concrete B 17.5

Reinforcement mild steel

b = 350 mm

ht = 400 mm

h = 400 - 25 - 8 - 8 = 359 mm



Load case 1

Safety factor j = 1.7

+ M = 7.34 kNm	+ Md = 12.48 kNm
- M = 16.64 kNm	- Md = 28.29 kNm
T = 12.84 kN	Td = 21.83 kN

Load case 2

Safety factor j = 1.2

+ M = 22.27 kNm	+ Md = 26.73 kNm
- M = 32.03 kNm	- Md = 38.44 kNm
T = 20.12 kN	Td = 24.15 kN

Lower reinforcement

+ Md = 26.73 kNm

wo = 0.28

Aa = 347 mm^2

Upper reinforcement

- Md = 38.44 kNm

wo = 0.40

Aa = 504 mm^2

Because of the beam stiffness (= EI) we take for lower as well as for upper reinforcement:

wo = 0.5

Aa = 630 mm^2, or 2 ϕ 12 + 2 ϕ 16

Shear force:

Td = 24.15 kN

ft = 24150/350 x 359 = 0.19 N/mm^2

gives practical ties of ϕ 8 c/c 250 mm

i. Middle beams

b = 350 mm

ht = 500 mm

h = 500 - 25 - 8 - 8 = 459 mm.

Load case 1

safety factor j = 1.7

+ M = 8.15 kNm	+ Md = 13.86 kNm
- M = 36.88 kNm	- Md = 62.69 kNm
T = 27.50 kN	Td = 46.75 kN

Load case 2

safety factor j = 1.2

+ M = 38.74 kNm	+ Md = 65.86 kNm
- M = 73.06 kNm	- Md = 87.67 kNm
T = 45.77 kN	Td = 54.92 kN

Lower reinforcement:

+ Md = 65.86 kNm

wo = 0.42

Aa = 677 mm^2, or 2 ϕ 16 + 2 ϕ 12

Upper reinforcement:

- Md = 87.67 kNm

wo = 0.57

Aa = 913 m^2, or 5 ϕ 16

Shear force:

Td = 54.92 kN

ft = 54920/350 x 459 = 0.34 N/mm^2,

gives practical ties of ϕ 8 c/c 250 mm

j. Top beams

b = 350 mm

ht = 750 mm

h = 750 - 25 - 8 - 8 = 709 mm

Load case 1

safety factor j = 1.7

+ M = 143.32 kNm + Md = 243.64 kNm

- M = 40.56 kNm - Md = 68.95 kNm

T = 161.64 kN Td = 274.79 kN

Load case 2

safety factor j = 1.2

+ M = 177.26 kNm + Md = 212.71 kNm

- M = 68.30 kNm - Md = 81.96 kNm

T = 206.75 Kn Td = 248.10 kN

Lower reinforcement:

+ Md = 243.64 kNm

wo = 0.69

Aa = $\underline{1711}$ mm^2, or 6 ϕ 20

Upper reinforcement:

- Md = 81.96 kNm

wo = 0.22

Aa = $\underline{543}$ mm^2, or 5 ϕ 12

Shear force:

Td = 248.10 kN

ft = 248100/350 x 709 = $\underline{1.00}$ N/mm^2 > 0.55 N/mm^2

Stirrups are necessary in this case.

Ast is the cross sectional area of the stirrups per meter length of beam.

Ast = 5.5 (1.00 - 0.55) 350 = 866 mm^2/m, wich gives stirrups ϕ 12 c/c 200 mm.

k. Platform

Thickness ht = 150 mm

Concrete cover 20 mm

hx = 150 - 20 - 5 = 125 mm

hy = 150 - 20 - 10 - 5 = 115 mm

Concrete B 17.5

Reinforcement: mild steel

ly = lx = 4.40 m

ly/lx = 1

Dead load 0.15 x 24 = 3.60 kN/m^2

Live load 0.50 kN/m^2

q = 4.10 kN/m^2

Mx = My = 41/100 x 4.10 x 4.4^2 = 3.25 kNm

Md = 5.53 kNm

h = 115 mm

wo = 0.19

Aa = 222 mm^2

Take a cross net ϕ 8 c/c 200 mm.

1. Columns

Concrete B 17.5

Reinforcement high yield steel!

Concrete cover 30 mm

Dimensions: 0.40 x 0.40 m

ht = 400 mm

b = 400 mm

Minimum reinforcement percentage wo = 0.75 (see d).

Member no. 4

	Load case 1	load case 2
safety factor	1.7	1.2
N' (kN)	372.08	421.24
M (kNm)	7.41	18.52
e = M/N' (mm)	19.91	43.97
Nd' (kN)	632.54	505.49
Ab (m^2)	0.16	0.16
Nd'/Ab (N/mm^2)	3.95	3.16
Nd'/Ab . e/ht	0.196	0.347
wo	0	0

Member no. 7

	Load case 1	Load case 2
safety factor	1.7	1.2
N' (kN)	420.15	527.26
M (kNm)	10.50	21.85
e = M/N' (mm)	25.00	41.44
Nd' (kN)	714.26	632.71
Ab (m^2)	0.16	0.16
Nd'/Ab (N/mm^2)	4.46	3.95
Nd'/Ab .e/ht	0.28	0.41
wo	0	0

Member no. 5

	Load case 1	load case 2
safety factor	1.7	1.2
N' (kN)	347.77	403.03
M (kNm)	12.96	30.75
e = M/N' (mm)	37.26	76.30
Nd' (kN)	591.21	483.64
Ab (m^2)	0.16	0.16
Nd'/Ab (N/mm^2)	3.70	3.02
Nd'/Ab. e/ht	0.344	0.58
wo	0	0

Member no. 8

	load case 1	load case 2
safety factor	1.7	1.2
N' (kN)	384.94	484.39
M (kNm)	15.06	32.77
e = M/N' (mm)	39.12	67.65
Nd' (kN)	654.39	581.26
Ab (m^2)	0.16	0.16
Nd'/Ab (N/mm^2)	4.09	3.63
Nd'/Ab. e/ht	0.40	0.613
wo	0	0

Member no. 6

	load case 1	load case 2
safety factor	1.7	1.2
N' (kN)	308.50	372.72
M (kNm)	16.26	10.22
e = M/N' (mm)	52.70	27.42
Nd' (kN)	536.95	453.83
Ab (m^2)	0.16	0.16
Nd'/Ab (N/mm^2)	3.36	2.836
Nd'/Ab. e/ht	0.613	0.185
wo	0	0

Member no. 9

	load case 1	load case 2
safety factor	1.7	1.2
N' (kN)	325.20	403.25
M (kNm)	40.55	68.28
e = M/N' (mm)	124.70	169.35
Nd' (kN)	552.84	483.90
Ab (m^2)	0.16	0.16
Nd'/Ab (N/mm^2)	3.46	3.02
Nd'/Ab. e/ht	1.07	1.28
wo	0.2	0.2

Because of the column stiffness (=EI) we take for all columns
wo = 0.75
That means Aa = 0.75 x 400 x 400/100 = <u>1200</u> mm^2

m. Footings

Dimensions 1.90 x 1.90 x 0.40 m.

Forces from columns:

Load case 1

N' = 424 kN

M = 10.50 kNm, e = 24.76 mm.

Load case 2

N' = 537 kN

M = 21.85 kNm, e = 40.69 mm.

Soil pressure

$$f = \frac{537}{1.9 \times 1.9} \pm \frac{21.85 \times 6}{1.9 \times 1.9^2} = 148.75 \pm 19.12 \text{ kN/m}^2$$

fmax = 167.87 kN/m^2

fmin = 129.63 kN/m^2

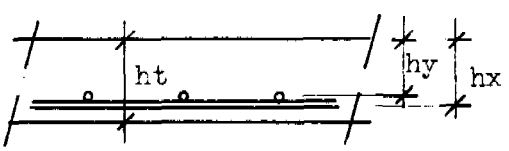
Concrete B 17.5.

Reinforcement mild steel.

Concrete cover 25 mm.

hy = 400 - 25 - 16 - 8 = 351 mm.

hx = 400 - 25 - 8 = 367 mm.



Punching shear: $f = \frac{Nd' (1 + 2 e/(h + d))}{3.14 h(h + d)} < 1.1 \text{ N/mm}^2$

Load case 1:

$$f = \frac{424000 \times 1.7 (1 + 2 \times 24.76/(367 + 509))}{3.14 \times 367 (367 + 509)} =$$

= 0.75 N/mm^2, wich is allowable.

Load case 2:

$$f = \frac{537000 \times 1.2 (1 + 2 \times 40.69/(367 + 509))}{3.14 \times 367 (367 + 509)} =$$

= 0.70 N/mm^2 which is allowable.

Bending moment:

Load case 1

- Mx = - My = 424/6.37 = 66.56 kNm

- Md = 1.7 x 66.56 = 113.16 kNm

Load case 2

- Mx = - My = 537/6.37 = 84.30 kNm

- Md = 1.2 x 84.30 = 101.16 kNm

Per meter

- Md = (113.16 + 10.5 x 1.7/2)/1.9 = 64.26 kNm

h = 351 mm

wo = 0.24

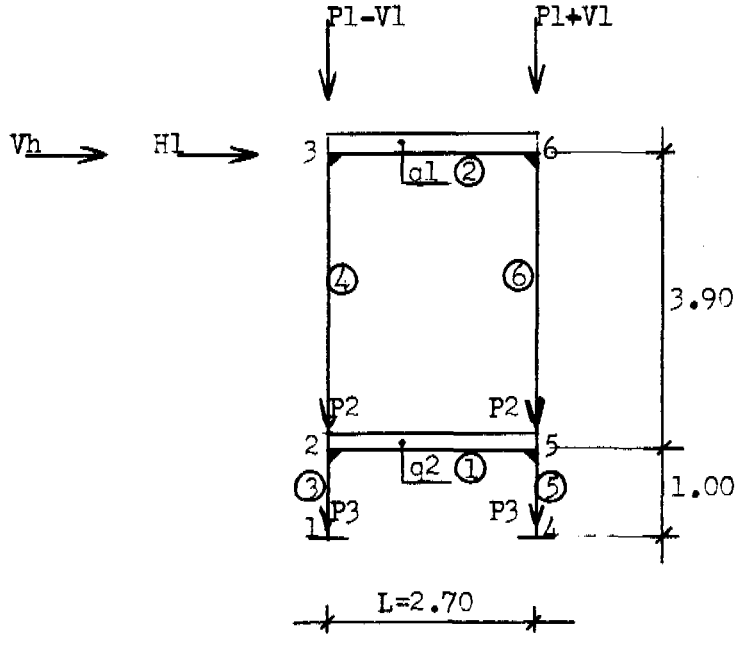
Aa = 850 mm^2, or ϕ 16 c/c 225 mm

6.6 Frame of 15 m^3 reservoir

Scheme:

See figure

Member	node i	node j
1	2	5
2	3	6
3	1	2
4	2	3
5	4	5
6	5	6



a. Wind load

Wind load is 0.8 kN/m^2

Wind load factor is 1.2

	kN	kNm
3.44 x 1.60 x 1.2 x 0.8 =	5.29 x 1.20	= 6.35
2.(3.44 x 1.60 x 0.04 x 0.8) =	0.35 x 1.20	= 0.42
3.98 x 3.98 x 0.04 x 0.8 =	0.51 x 2.30	= 1.17
3.98 x 0.30 x 1.2 x 0.8 =	1.15 x 2.15	= 2.47
2(3.98 x 0.30 x 0.04 x 0.8) =	0.08 x 2.15	= 0.17
4.04 x 0.40 x 1.2 x 0.8 =	1.55 x 0.20	= 0.31
2(4.04 x 0.40 x 0.04 x 0.8 =	0.10 x 0.20	= 0.02
	9.03	10.91

H = 9.03 + 0.3 x 3 x 1.2 x 0.8 + (0.3 x 3 x 0.04 x 0.8) 2 =

= 9.95 kN

per frame H = 9.95/2 = 4.976 kN

b. Vertical load per frame

Wind:

V1 = 10.91/2.70 x 1/2 = 2.02 kN

V2 = = - 2.02 kN

Reservoir:

	kN
Top slab (3.98 x 3.98 x 0.3 - 3.48 x 3.48 x 0.15) x x 24 =	70.45
Bottom slab (4.04 x 4.04 x 0.4 - 3.04 x 3.04 x 0.20) x x 24 =	112.33
Walls 1.75 x 24 x (3.44 x 3.44 - 3.04 x 3.04) =	108.86
Water 15 m3	150.00
	441.64
Top beams (2.95 x 2.95 - 2.45 x 2.45) x 0.5 x 24 =	32.40
	474.04

q1 = 474.04/(4 x 2.70) = 43.90 kN/m

Ground beams

q2 = 0.25 x 0.40 x 24 = 2.40 kN/m

Inclusive the weight of the columns:

P1 = 1.35 x 43.90 = 59.27 kN

P2 = 1.50 x 2.40 + 3.2 x 0.3 x 24 = 10.51 kN

P3 = 0.3 x 0.3 x 1 x 24 = 2.16 kN

Total vertical load (live load + dead load)

level + 4.90:	2 x 59.27 + 2.7 x 43.90 =	237.07 kN
level + 1.00:	2 x 10.51 + 2.7 x 2.40 =	27.50 kN
footings:	2 x 2.16 + 2(0.4 x 24 x 1.4 x 1.4) =	41.95 kN
	W =	306.52 kN

c. Earthquake load

See also frame of 50 m^3 reservoir.

V = ZKCW = 1 x 2.5 x 0.08504 x 306.52 = 65.17 kN

The shear force at the top is Vh = 0.15 x 65.17 = 9.775 kN.

Vx. hx = 237.07 x 4.9 + 27.51 x 1 = 1189.14 kNm

Vertical force:

Level +4.90

Fx = (65.17 - 9.775) x 237.07 x 4.9/1189.14 = 54.12 kN

Level +1.00

Fx = (65.17 - 9.775) x 27.50 x 1.00/1189.14 = 1.28 kN

So in the case of earthquake with dead load and live load:

q1 = 43.90 + 54.12/5.4 = 43.90 + 10.02 = 53.92 kN/m

q2 = 2.40 + 1.28/5.4 = 2.40 + 0.24 = 2.64 kN/m

P1 = 59.27 + 1.35 x 10.02 = 72.80 kN

P2 = 10.51 + 1.35 x 0.24 = 10.84 kN

P3 = 2.16 kN

Vh = 9.775 kN

The safety factor for that load case is 1.2

d. Modulus of elasticity

E = modulus of elasticity

I = moment of inertia

Columns:

dimensions 0.30 x 0.30 m

assume a percentage of reinforcement wo = 0.5

$E = 1/3 \times (8\ wo + 10) \times 10^6 = 4.66 \times 10^6\ kN/m^2$

$EI = 4.66 \times 10^6 \times 1/12 \times 0.3^4 = 3145.50\ kNm^2$

Top beams:

dimensions 0.25 x 0.50 m

assume wo = 0.8

$E = (7.5\ wo + 2.5) \times 10^6 = 8.50 \times 10^6\ kN/m^2$

$EI = 8.50 \times 10^6 \times 1/2 \times 0.25 \times 0.50^3 = 22135\ kNm^2$

Ground beams:

dimensions 0.25 x 0.40 m

assume wo = 0.5

$E = (7.5\ wo + 2.5) \times 10^6 = 6.25 \times 10^6\ kN/m^2$

$EI = 6.25 \times 10^6 \times 1/2 \times 0.25 \times 0.4^3 = 8333\ kNm^2$

e. Load cases

1. dead load + live load + wind
2. dead load + live load + earthquake

f. Calculation of the member forces

See computer output at pages 1 to 6 of chapter 9.

g. Calculation of the second moment factor n/(n-1)

For the members no 3 and 4 the buckling length has been calculated with a computer program - see also 6.5.g - The result is:

for member nr. 3 Lk = 2.27 m

for member nr. 4 Lk = 4.07 m

$Nk' = 3.14^2 \times EI/Lk^2$

Columns 4 and 6:

$Nk' = 3.14^2 \times 3145.5/4.07^2 = 1872$ kN

Columns 3 and 5:

$Nk' = 3.14^2 \times 3145.5/2.27^2 = 6018$ kN

Load case 1:

Columns 4 and 6

Max. N' = 124.22 kN

Nd' = 1.7 x 124.22 = 211.17 kN

n = 1872/211.17 = 8.864

n/(n - 1) = <u>1.127</u>

Load case 2

Columns 4 and 6

max. N' = 152.83 kN

Nd' = 1.2 x 152.83 = 183.40 kN

n = 1872./183.4 = 10.21

n/(n - 1) = <u>1.108</u>

The horizontal loads to be multiplied with 1.127 for load case 1 en with 1.108 for load case 2.

h. Ground beams

Concrete cover 25 mm)

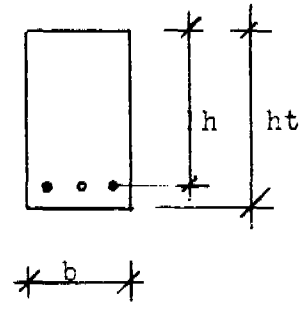
Concrete B 17.5)) for all beams

Reinforcement mild steel)

b = 250 mm

ht = 400 mm

h = 400 - 25 - 8 - 8 = 359 mm



Load case 1

Safety factor j = 1.7

+ M = 4.02 kNm	+ Md = 6.834 kNm
- M = 7.40 kNm	- Md = 12.58 kNm
T = 7.47 kN	Td = 12.70 kN

Load case 2

Safety factor j = 1.2

+ M = 9.08 kNm	+ Md = 10.90 kNm
- M = 12.97 kNm	- Md = 15.57 kNm
T = 11.73 kN	Td = 14.08 kN

max. Md = 15.57 kNm

wo = 0.22

Aa = 201 mm^2

Because of the beam stiffness (= EI) we take for lower as well as for upper reinforcement.

wo = 0.25 Aa = $\underline{225}$ mm^2, or 2 ϕ 12

Shear force

Td = 14.08 kN

ft = 14080/(250 x 359) = 0.16 N/mm^2

ties ϕ8 c/c 250 mm (practical)

i. Top beams

b = 250 mm

ht = 500 mm

h = 500 - 25 - 8 - 8 = 459 mm

Load case 1

Safety factor j = 1.7

+ M = 35.65 kNm	+ Md = 60.61 kNm
- M = 9.95 kNm	- Md = 16.92 kNm
T = 63.41 kN	Td = 107.80 kN

Load case 2

Safety factor j = 1.2

+ M = 44.00 kNm	+ Md = 52.80 kNm
- M = 16.16 kNm	- Md = 19.39 kNm
T = 80.81 kN	Td = 96.97 kN

Lower reinforcement:

+ Md = 60.61 kNm

wo = 0.55

Aa = 630 mm^2

Upper reinforcement:

- Md = 19.39 kNm

wo = 0.17

Aa = 195 mm^2

Remark: because of the stiffness (=EI) the reinforcement is more than calculated.

Lower reinforcement 3 ϕ 16

Upper reinforcement 2 ϕ 16

Shear force:

Td = 107.80 kN

ft = 107800/(250 x 459) = 0.94 N/mm^2

Stirrups are necessary in this case. Ast is the cross sectional area of the stirrups per meter lenght of beam.

Ast = 5.5 (0.94 - 0.55) 250 = 536 mm^2/m

Ties ϕ 8 c/c 175 mm

or ϕ 10 c/c 250 mm

j. Columns

Concrete B 17.5

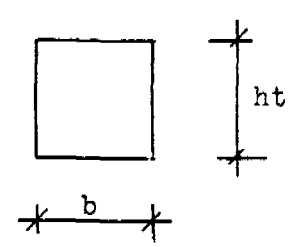
Reinforcement: <u>high yield steel!</u>

Concrete cover 30 mm

Dimensions: 0.30 x 0.30 m

ht = 300 mm

b = 300 m



Minimum reinforcement percentage wo = 0.5 (see d)

Member no. 4

	load case 1	load case 2
Safety factor	1.7	1.2
N' (kN)	112.37	137.58
M (kNm)	3.27	7.77
e = M/N' (mmm)	29.10	56.48
Nd' (kN)	191.03	165.10
Ab (m^2)	0.09	0.09
Nd'/Ab (N/mm^2)	2.12	1.83
Nd'/Ab. e/ht	0.21	0.34
wo	0	0

Member no. 6

	Load case 1	load case 2
Safety factor	1.7	1.2
N' (kN)	124.70	153.61
M (kNm)	9.95	16.16
e = M/N' (mm)	79.79	105.20
Nd' (kN)	212.00	184.33
Ab (m^2)	0.09	0.09
Nd'/Ab (N/mm^2)	2.36	2.05
Nd'/Ab. e/ht	0.63	0.72
wo	0	0

Because of the stiffness (= EI) we take for all columns wo = 0.5
That means Aa = 0.5 x 300 x 300/100 = 450 mm^2.

k. Footings

Dimensions 1.40 x 1.40 x 0.40 m

Forces from columns

Load case 1:

N' = 144.84 kN

M = 2.29 kNm e = 15.81 mm

Load case 2:

N' = 178.34 kN

M = 4.46 kNm e = 25.01 mm

Soil pressure:

$f = 178.34/1.4 \times 1.4 \pm 4.46 \times 6/1.4^3 =$

$= 90.99 \pm 9.75$ kN/m^2

Concrete B 17.5

Reinforcement: mild steel

Concrete cover 25 mm

hx = 400 - 25 - 8 = 367 mm

hy = 400 - 25 - 16 - 8 = 351 mm

Bending moment:

Load case 1

$$- Mx = - My = \frac{144.84}{6.37} = 22.74 \text{ kNm}$$

- Md = 1.7 x 22.74 = 38.65 kNm

Load case 2

$$- Mx = - My = \frac{178.34}{6.37} = 28.00 \text{ kNm}$$

- Md = 1.2 x 28.00 = 33.60 kNm

Per meter:

- Md = (38.65 + 2.29 x 1.7/2)/1.4 = 29.00 kNm

hy = 351 mm = h min

wo = 0.11

Aa = 379 mm^2

Take ϕ 12 c/c 200 mm

7. CALCULATION OF CONCRETE MIX FOR 1 M3 OF CONCRETE

DATA:

- concrete class B17.5
- gravel maximum size 31.5 mm
- cement Portland
- watertight concrete

  z = max. slump = 100 mm

For watertight concrete it is necessary to use 325 kg cement per m^3 and to use not more than 175 liter water per m^3 of concrete mix.

325 kg Portland cement = 325/3150 = 0.103 m^3
175 liter water = 175/1000 = 0.175 m^3
air 1% 0.010 m^3
0.288 m^3

rest for sand + gravel

1.00 - 0.288 = 0.712 m^3

To calculate the percentage of sand we use the formula

Pz = 10 Fz + 28 + 0.05z - 0.08 C

Fz = 3

Pz = 10 x 3 + 28 + 0.05 x 100 - 0.08 x 325= 37%

sand 37/100 x 0.712 = 0.263 m^3

gravel 63/100 x 0.712 = 0.449 m^3

sand 0.263 x 2650 = 697 kg

gravel 0.449 x 2650 = 1190 kg.

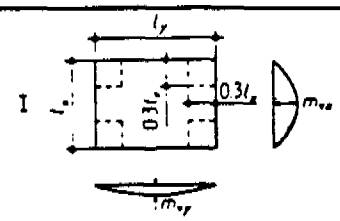
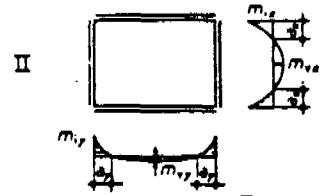
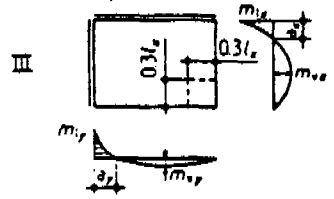
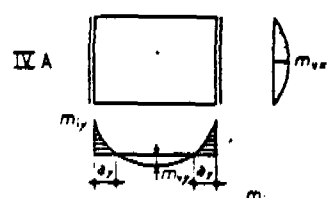
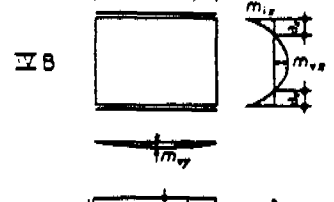
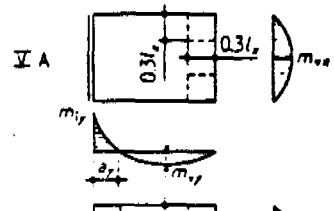
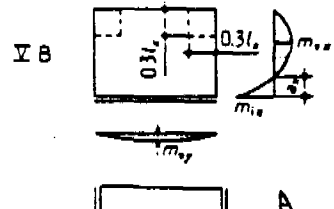
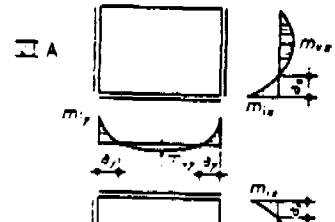
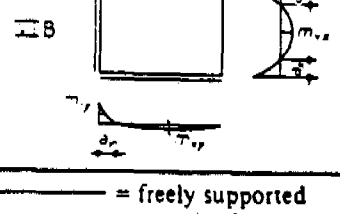
Concrete mix for 1 m^3 of concrete:

- Portland cement 325 kg
- water 175 liter
- sand 697 kg
- gravel 1190 kg

8. TABLES AND CHARTS

Collection of tables and charts from the Dutch Standards used with the calculation of the water reservoirs.

Table E 8 Critical moments per unit length in the middle strips of restrained slabs carrying uniformly distributed load

	l_y/l_x		1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0
I		$m_{vx} = 0.001\,ql_x^2 \times$	41	54	67	79	87	97	110	117
		$m_{vy} = 0.001\,ql_x^2 \times$	41	35	31	28	26	25	24	23
II		$m_{vx} = 0.001\,ql_x^2 \times$	18	26	32	36	39	41	42	43
		$m_{vy} = 0.001\,ql_x^2 \times$	18	16	12	10	10	10	10	10
		$m_{ix} = -0.001\,ql_x^2 \times$	51	63	72	78	81	82	83	83
		$m_{iy} = -0.001\,ql_x^2 \times$	51	54	55	54	54	53	51	49
		$a_x/l_x =$	0.18	0.19	0.19	0.19	0.19	0.19	0.19	0.19
		$a_y/l_y =$	0.18	0.16	0.15	0.13	0.11	0.10	0.07	0.06
III		$m_{vx} = 0.001\,ql_x^2 \times$	25	36	45	53	58	62	67	69
		$m_{vy} = 0.001\,ql_x^2 \times$	25	23	20	19	18	17	17	17
		$m_{ix} = -0.001\,ql_x^2 \times$	68	84	97	106	113	117	122	124
		$m_{iy} = -0.001\,ql_x^2 \times$	68	74	77	77	77	76	73	71
		$a_x/l_x =$	0.20	0.22	0.22	0.22	0.23	0.23	0.24	0.24
		$a_y/l_y =$	0.21	0.19	0.17	0.16	0.13	0.12	0.09	0.09
IV A		$m_{vx} = 0.001\,ql_x^2 \times$	16	28	42	56	69	80	100	112
		$m_{vy} = 0.001\,ql_x^2 \times$	29	32	32	30	27	24	20	18
		$m_{iy} = -0.001\,ql_x^2 \times$	69	85	97	105	110	112	112	112
		$a_y/l_y =$	0.19	0.19	0.17	0.17	0.16	0.15	0.12	0.11
IV B		$m_{vx} = 0.001\,ql_x^2 \times$	29	34	38	40	42	42	42	42
		$m_{vy} = 0.001\,ql_x^2 \times$	16	14	13	13	13	13	13	13
		$m_{ix} = -0.001\,ql_x^2 \times$	69	76	80	82	83	83	83	83
		$a_x/l_x =$	0.19	0.20	0.20	0.21	0.21	0.21	0.21	0.21
V A		$m_{vx} = 0.001\,ql_x^2 \times$	27	41	54	67	78	89	105	115
		$m_{vy} = 0.001\,ql_x^2 \times$	38	37	34	30	27	25	24	23
		$m_{iy} = -0.001\,ql_x^2 \times$	91	102	108	111	113	114	114	114
		$a_y/l_y =$	0.21	0.21	0.20	0.18	0.17	0.15	0.13	0.10
V B		$m_{vx} = 0.001\,ql_x^2 \times$	38	44	52	58	62	65	68	70
		$m_{vy} = 0.001\,ql_x^2 \times$	27	21	19	18	17	17	17	17
		$m_{ix} = -0.001\,ql_x^2 \times$	91	98	107	113	118	120	124	124
		$a_x/l_x =$	0.21	0.21	0.22	0.23	0.24	0.24	0.24	0.25
VI A		$m_{vx} = 0.001\,ql_x^2 \times$	18	29	39	47	54	59	66	69
		$m_{vy} = 0.001\,ql_x^2 \times$	23	23	20	17	15	14	13	13
		$m_{ix} = -0.001\,ql_x^2 \times$	54	72	88	100	108	114	121	124
		$m_{iy} = -0.001\,ql_x^2 \times$	60	69	74	76	76	76	73	71
		$a_x/l_x =$	0.19	0.21	0.23	0.23	0.23	0.24	0.24	0.24
		$a_y/l_y =$	0.19	0.18	0.17	0.15	0.14	0.14	0.10	0.08
VI B		$m_{vx} = 0.001\,ql_x^2 \times$	23	30	35	38	40	41	42	43
		$m_{vy} = 0.001\,ql_x^2 \times$	18	15	14	13	13	13	13	13
		$m_{ix} = -0.001\,ql_x^2 \times$	60	70	76	80	82	83	83	83
		$m_{iy} = -0.001\,ql_x^2$	54	55	55	54	53	53	51	49
		$a_x/l_x =$	0.18	0.19	0.19	0.19	0.20	0.21	0.21	0.21
		$a_y/l_y =$	0.19	0.18	0.14	0.13	0.13	0.10	0.08	0.07

——— = freely supported

═══ = completely restrained

m_{vx} is positive moment per unit length in middle section parallel to the long edge (l_y)

m_{vy} is positive moment per unit length in middle section parallel to the short edge (l_x)

m_{ix} is negative moment per unit length along a long edge (l_y)

m_{iy} is negative moment per unit length along a short edge (l_x)

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B 17,5

$\frac{M_u}{bh^2}$	FeB 220		FeB 400		FeB 500		k_x	k_z
	k_a	ω_0	k_a	ω_0	k_a	ω_0		
100	0,219	0,05	0,398	0,03	0,498	0,02	0,011	0,996
200	0,218	0,09	0,397	0,05	0,496	0,04	0,022	0,992
300	0,217	0,14	0,395	0,08	0,494	0,06	0,034	0,988
400	0,216	0,18	0,394	0,10	0,492	0,08	0,045	0,984
500	0,216	0,23	0,392	0,13	0,490	0,10	0,057	0,980
600	0,215	0,28	0,390	0,15	0,488	0,12	0,068	0,976
700	0,214	0,33	0,389	0,18	0,486	0,14	0,080	0,972
800	0,213	0,38	0,387	0,21	0,484	0,17	0,092	0,967
900	0,212	0,42	0,385	0,23	0,482	0,19	0,104	0,963
1000	0,211	0,47	0,384	0,26	0,479	0,21	0,116	0,959
1100	0,210	0,52	0,382	0,29	0,477	0,23	0,128	0,955
1200	0,209	0,57	0,380	0,32	0,475	0,25	0,140	0,950
1300	0,208	0,62	0,378	0,34	0,473	0,27	0,153	0,946
1400	0,207	0,68	0,377	0,37	0,471	0,30	0,165	0,941
1500	0,206	0,73	0,375	0,40	0,468	0,32	0,178	0,937
1600	0,205	0,78	0,373	0,43	0,466	0,34	0,191	0,932
1700	0,204	0,83	0,371	0,46	0,464	0,37	0,204	0,928
1800	0,203	0,89	0,369	0,49	0,462	0,39	0,217	0,923
1900	0,202	0,94	0,367	0,52	0,459	0,41	0,230	0,919
2000	0,201	0,99	0,366	0,55	0,457	0,44	0,243	0,914
2100	0,200	1,05	0,364	0,58	0,455	0,46	0,257	0,909
2200	0,199	1,11	0,362	0,61	0,452	0,49	0,270	0,904
2300	0,198	1,16	0,360	0,64	0,450	0,51	0,284	0,899
2400	0,197	1,22	0,358	0,67	0,447	0,54	0,298	0,894
2500	0,196	1,28	0,356	0,70	0,445	0,56	0,312	0,889
2600	0,195	1,34	0,354	0,74	0,442	0,59	0,327	0,884
2700	0,193	1,40	0,352	0,77	0,440	0,61	0,341	0,879
2800	0,192	1,46	0,350	0,80	0,437	0,64	0,356	0,874
2900	0,191	1,52	0,347	0,83	0,434	0,67	0,371	0,868
3000	0,190	1,58	0,345	0,87	0,432	0,70	0,386	0,863
3100	0,189	1,64	0,343	0,90	0,429	0,72	0,402	0,858
3200	0,187	1,71	0,341	0,94	0,426	0,75	0,417	0,852
3300	0,186	1,77	0,339	0,97	0,423	0,78	0,433	0,846
3400	0,185	1,84	0,336	1,01	0,420	0,81	0,449	0,841
3500	0,184	1,91	0,334	1,05	0,417	0,84	0,466	0,835
3600	0,182	1,97	0,332	1,09	0,414	0,87	0,483	0,829
3700	0,181	2,04	0,329	1,12	0,411	0,90	0,500	0,823
3800	0,180	2,11	0,327	1,16			0,517	0,817
3900	0,178	2,19	0,324	1,20			0,535	0,810
4000	0,177	2,26	0,322	1,24			0,553	0,804
4100	0,175	2,34					0,571	0,798
4200	0,174	2,41					0,590	0,791
4300	0,172	2,49					0,609	0,784
4400	0,171	2,57					0,629	0,777
4500	0,169	2,66					0,650	0,770
4600	0,168	2,74					0,670	0,762
4700	0,166	2,83					0,692	0,755

$$A_a = \frac{M_u}{k_a \cdot h} \text{ of } A_a = \omega_0 \cdot b \cdot h \cdot 10^4; \qquad x = k_x \cdot h; \qquad z = k_z \cdot h$$

M_u in kNm; b in m; h in m; A_a in mm²; $\frac{M_u}{bh^2}$ in kN/m²

FeB	$k_{x\,max}$	ω_0	$0{,}6\,k_{x\,max}$	ω_0
220	0,695	2,84	0,417	1,71
400	0,555	1,25	0,333	0,75
500	0,500	0,90	0,300	0,54

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$\frac{M_u}{bh^2}$	FeB 220		FeB 400		FeB 500		k_x	k_z
	k_a	ω_0	k_a	ω_0	k_a	ω_0		
200	0,219	0,09	0,398	0,05	0,497	0,04	0,017	0,994
400	0,217	0,18	0,395	0,10	0,494	0,08	0,035	0,988
600	0,216	0,28	0,393	0,15	0,491	0,12	0,053	0,981
800	0,214	0,37	0,390	0,21	0,487	0,16	0,071	0,975
1000	0,213	0,47	0,387	0,26	0,484	0,21	0,089	0,968
1200	0,212	0,57	0,385	0,31	0,481	0,25	0,108	0,962
1400	0,210	0,67	0,382	0,37	0,478	0,29	0,127	0,955
1600	0,209	0,77	0,379	0,42	0,474	0,34	0,146	0,948
1800	0,207	0,87	0,377	0,48	0,471	0,38	0,165	0,941
2000	0,206	0,97	0,374	0,54	0,467	0,43	0,185	0,934
2200	0,204	1,08	0,371	0,59	0,464	0,47	0,205	0,927
2400	0,202	1,19	0,368	0,65	0,460	0,52	0,225	0,920
2600	0,201	1,29	0,365	0,71	0,456	0,57	0,246	0,913
2800	0,199	1,41	0,362	0,77	0,453	0,62	0,267	0,905
3000	0,197	1,52	0,359	0,84	0,449	0,67	0,289	0,898
3200	0,196	1,63	0,356	0,90	0,445	0,72	0,311	0,890
3400	0,194	1,75	0,353	0,96	0,441	0,77	0,333	0,882
3600	0,192	1,87	~~0,350~~ 0,35	1,03	0,437	0,82	0,356	0,874
3800	0,190	2,00	0,346	1,10	0,433	0,88	0,379	0,865
4000	0,189	2,12	0,343	1,17	0,429	0,93	0,403	0,857
4200	0,187	2,25	0,339	1,24	0,424	0,99	0,428	0,848
4400	0,185	2,38	0,336	1,31	0,420	1,05	0,453	0,839
4600	0,183	2,52	0,332	1,39	0,415	1,11	0,479	0,830
4800	0,181	2,66	0,328	1,46	0,410	1,17	0,505	0,821
5000	0,178	2,80	0,324	1,54			0,533	0,811
5200	0,176	2,95	0,320	1,62			0,561	0,801
5400	0,174	3,10					0,590	0,791
5600	0,172	3,26					0,620	0,780
5800	0,169	3,43					0,652	0,769
6000	0,167	3,60					0,685	0,757

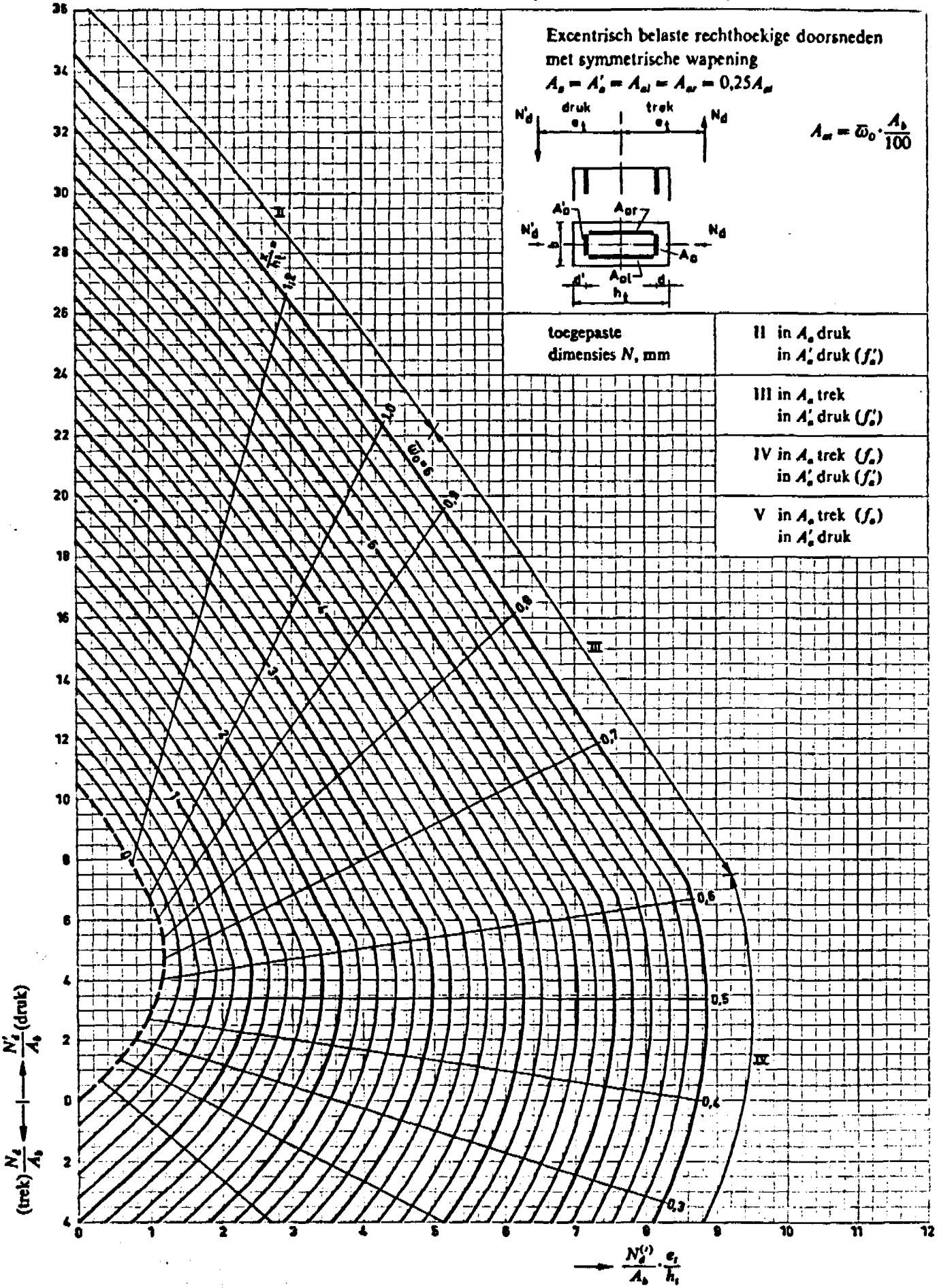
$$A_a = \frac{M_u}{k_a \cdot h} \text{ of } A_a = \omega_0 \cdot b \cdot h \cdot 10^4; \qquad x = k_x \cdot h; \qquad z = k_z \cdot h$$

M_u in kNm; b in m; h in m; A_a in mm^2; $\frac{M_u}{bh^2}$ in kN/m^2

FeB	$k_{x\,max}$	ω_0	$0{,}6\,k_{x\,max}$	ω_0
220	0,695	3,66	0,417	2,19
400	0,555	1,61	0,333	0,96
500	0,500	1,16	0,300	0,69

buiging met normaalkracht

17,5 400 0,05



buiging met normaalkracht

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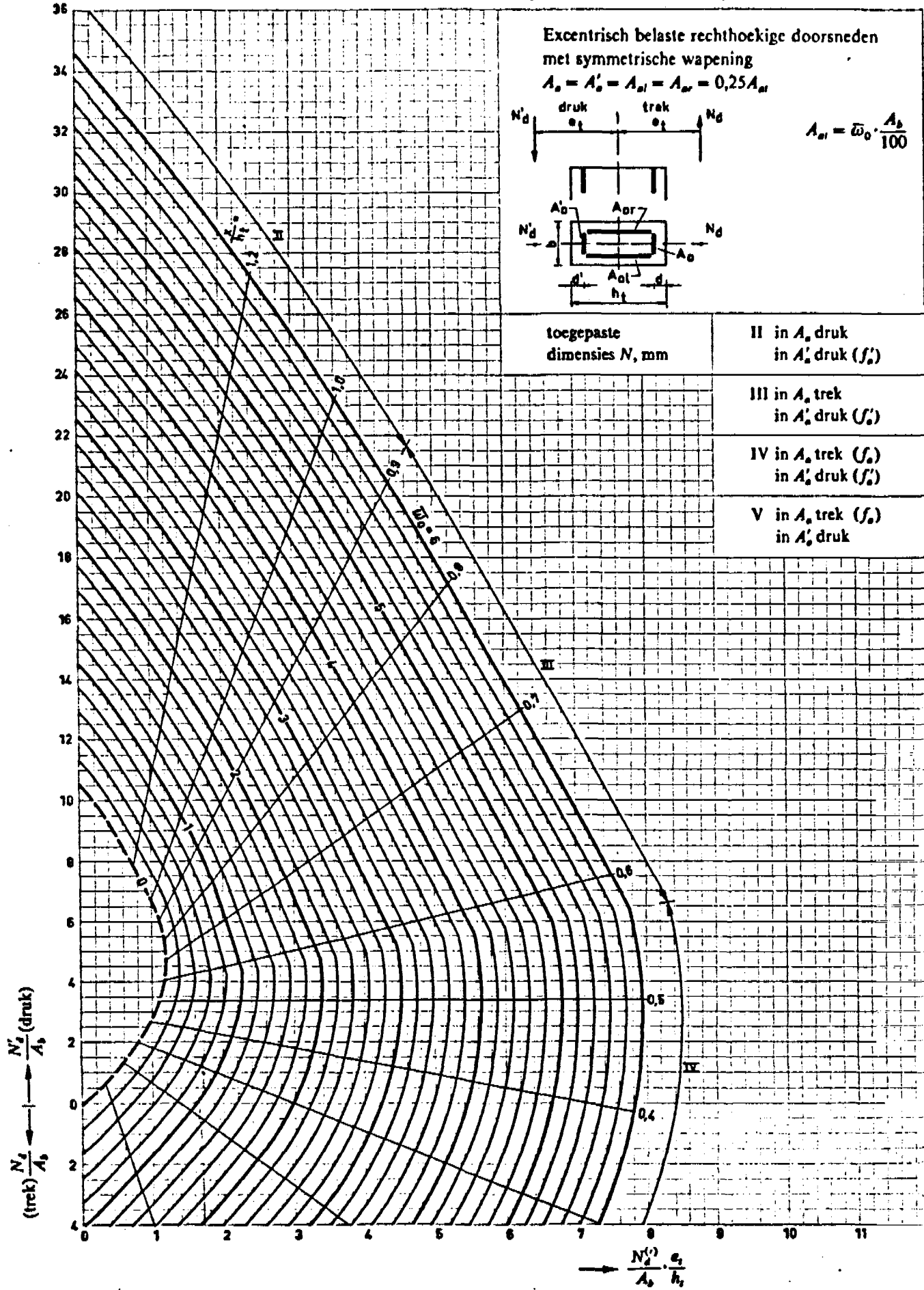

buiging met normaalkracht

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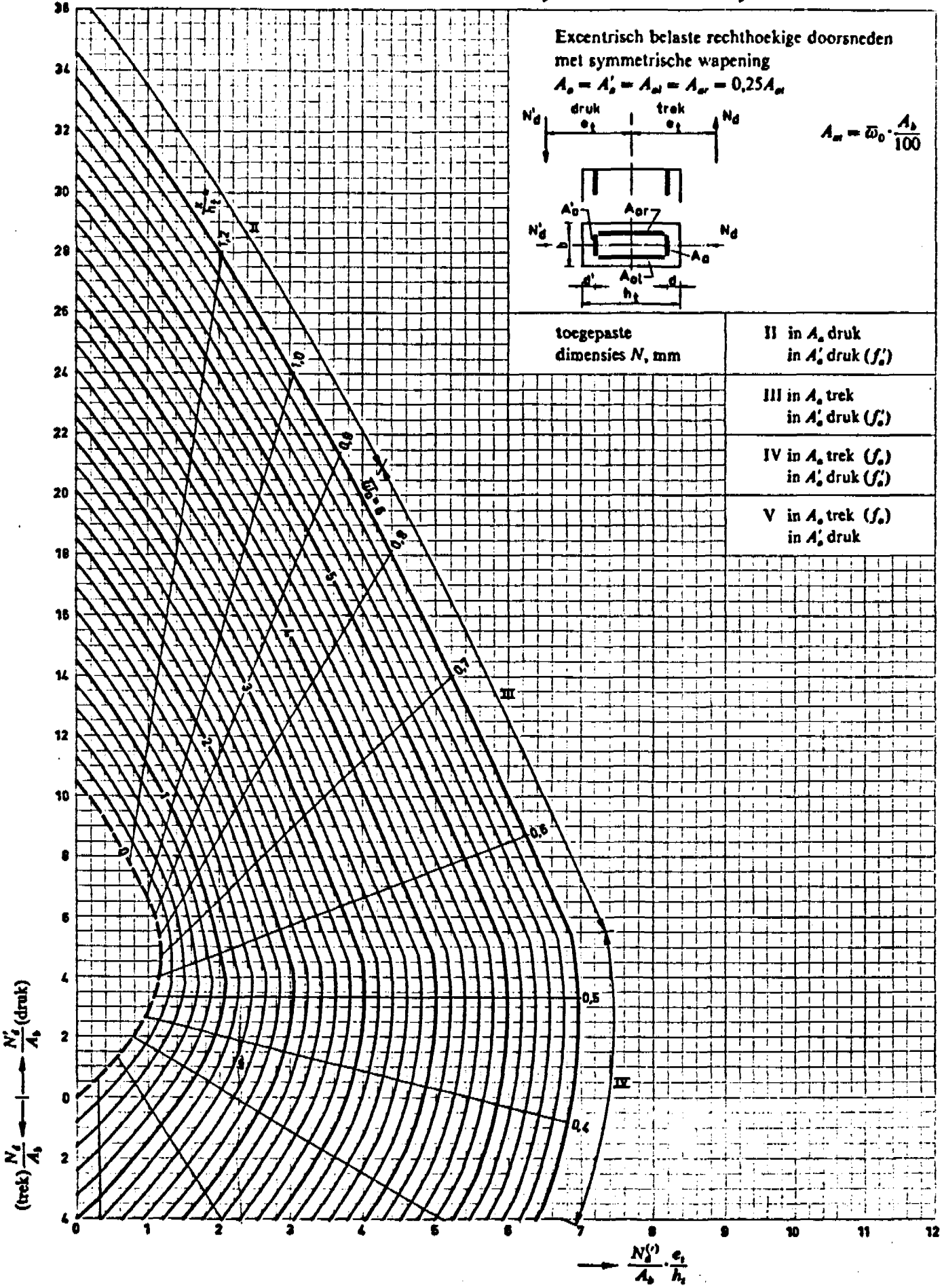

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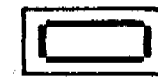

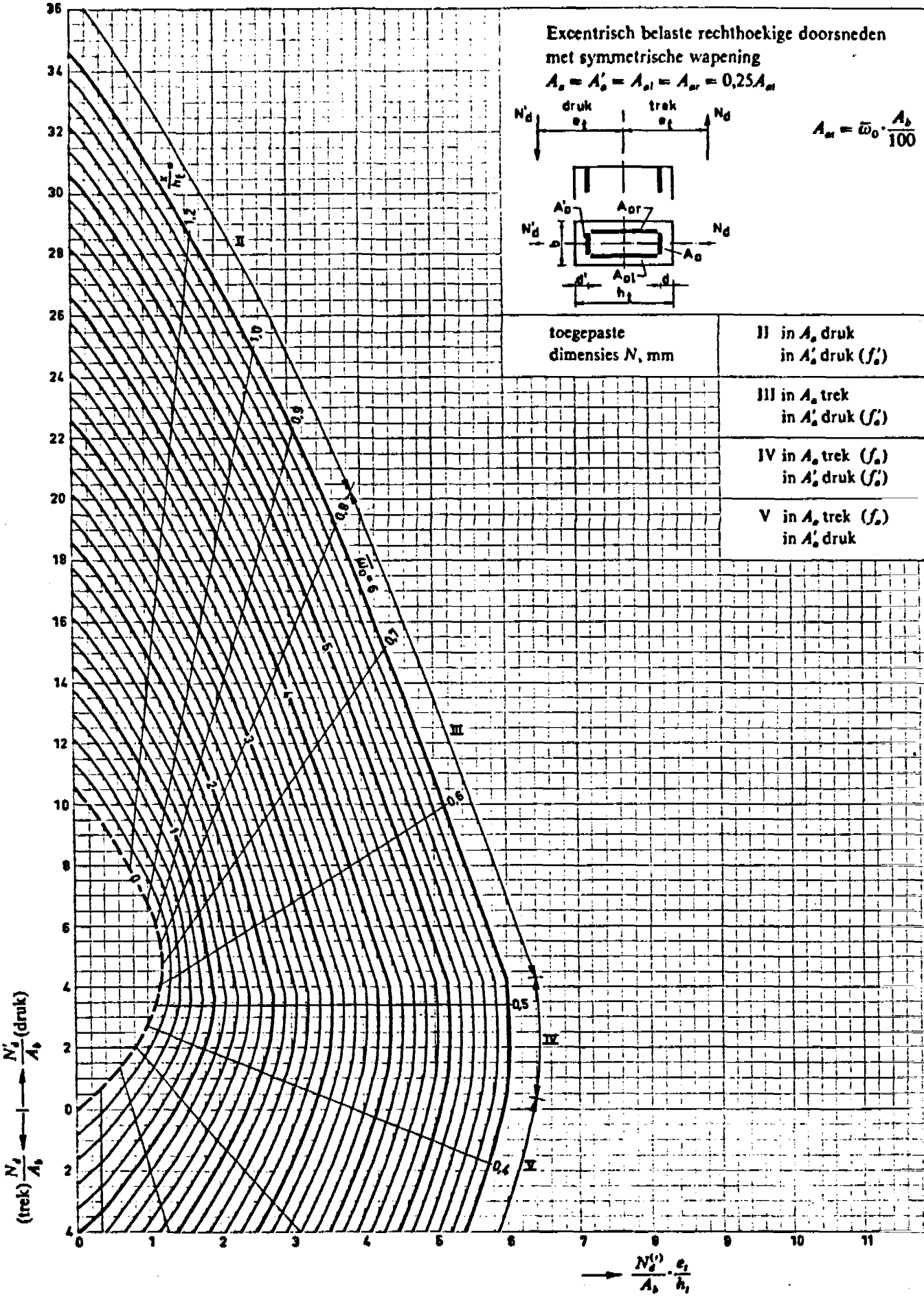

9. COMPUTER OUTPUT

Computer calculations of the frames for the 15 m^3 and 50 m^3 elevated water reservoirs.

PROJECT : ELEVATED WATER RESERVOIR, 15 M3

ONDERDEEL : FRAME
DIMENSIES : KN/M/RAD
DATUM : 90/ 5/23

KOORDINATEN

KNOOP	X	Y
1	0.0000	0.0000
2	0.0000	1.0000
3	0.0000	4.9000
4	2.7000	0.0000
5	2.7000	1.0000
6	2.7000	4.9000

MATERIALEN NORMAAL

MT	OMSCHRIJVING	OPPERVLAK	TRAAGHEID	E-MODULUS	S.M.	POIS.	FAK/BEDD
1	COLUMNS	0.9000E-01	0.6800E-03	0.4660E+07	24.00	.00	0.000E+00
2	TOP BEAMS	0.1250E+00	0.2600E-02	0.8500E+07	24.00	.00	0.000E+00
3	GROUND BEAMS	0.1000E+00	0.1330E-02	0.6250E+07	24.00	.00	0.000E+00

STAAF TOPOLOGIE

STNR	I	J	Xi	Yi	Xj	Yj	LENGTE	MT	NDM-CODE
1	2	5	0.000	1.000	2.700	1.000	2.7000	-3	NDM NDM
2	3	6	0.000	4.900	2.700	4.900	2.7000	-2	NDM NDM
3	1	2	0.000	0.000	0.000	1.000	1.0000	-1	NDM NDM
4	2	3	0.000	1.000	0.000	4.900	3.9000	-1	NDM NDM
5	4	5	2.700	0.000	2.700	1.000	1.0000	-1	NDM NDM
6	5	6	2.700	1.000	2.700	4.900	3.9000	-1	NDM NDM

OPLEGGINGEN

KN	XYR (0=LOS 1=VAST)
1	111
4	111

BELASTINGGEVAL 1

DEAD LOAD+LIVE LOAD+WIND

PROJECT : ELEVATED WATER RESERVOIR, 15 M3

KNOOPBELASTINGEN

KN	RI	KRACHT
3	1	5.61000
3	2	2.02000
6	2	-2.02000
3	2	-59.27000
6	2	-59.27000
2	2	-10.51000
5	2	-10.51000
1	2	-2.16000
4	2	-2.16000

STAAFLASTEN

STNR	TYPE NR. EN OMSCHRIJVING	Q1/FAC/P	Q2	A/ALFA	B/HT
1	1 TRAPEZIUM LOKAAL-Y	-2.4000	-2.4000	0.000000	0.000
2	1 TRAPEZIUM LOKAAL-Y	-43.9000	-43.9000	0.000000	0.000

BELASTINGGEVAL 2

DEAD LOAD+LIVE LOAD+EARTHQUAKE

KNOOPBELASTINGEN

KN	RI	KRACHT
3	2	-72.80000
6	2	-72.80000
2	2	-10.84000
5	2	-10.84000
1	2	-2.16000
4	2	-2.16000
3	1	10.83000

STAAFLASTEN

STNR	TYPE NR. EN OMSCHRIJVING	Q1/FAC/P	Q2	A/ALFA	B/HT
1	1 TRAPEZIUM LOKAAL-Y	-2.6400	-2.6400	0.000000	0.000
2	1 TRAPEZIUM LOKAAL-Y	-53.9200	-53.9200	0.000000	0.000

VERPLAATSINGEN BELASTINGSGEVALLEN

BG	KN	X-VERPL.	Y-VERPL.	ROTATIE
1	1	0.00000	0.00000	0.00000

PROJECT : ELEVATED WATER RESERVOIR, 15 M3

VERPLAATSINGEN BELASTINGSGEVALLEN

BG	KN	X-VERPL.	Y-VERPL.	ROTATIE
2	1	0.00000	0.00000	0.00000
1	2	0.00023	-0.00029	-0.00029
2	2	0.00045	-0.00034	-0.00057
1	3	0.00560	-0.00134	-0.00154
2	3	0.01072	-0.00162	-0.00198
1	4	0.00000	0.00000	0.00000
2	4	0.00000	0.00000	0.00000
1	5	0.00024	-0.00034	-0.00037
2	5	0.00046	-0.00042	-0.00068
1	6	0.00559	-0.00150	0.00119
2	6	0.01071	-0.00185	0.00137

STAAFKRACHTEN BELASTINGSGEVALLEN

BG	ST	KN/AFST	NXi/NXj	DYi/DYj	MXi/MXj	
1	1	2	-2.1882	-0.9899	-4.0199	TREK
		1.4637			0.0000	
		5	2.1882	7.4699	-7.4009	
2	1	2	-2.8288	-4.6020	-9.0815	TREK
		1.4062			-0.0001	
		5	2.8288	11.7300	-12.9666	

PROJECT : ELEVATED WATER RESERVOIR, 15 M3

STAAFKRACHTEN BELASTINGSGEVALLEN

BG	ST	KN/AFST	NXi/NXj	DYi/DYj	MXi/MXj	
1	2	3	4.4548	55.1214	-1.2382	DRUK
		1.2556			-35.8438	
		2.5335			0.0000	
		6	-4.4548	63.4086	-9.9495	
2	2	3	7.4307	64.7770	-5.4851	DRUK
		1.2014			-44.3951	
		2.4846			0.0001	
		6	-7.4307	80.8070	-16.1554	
1	3	1	121.8915	3.3432	2.5897	DRUK
		0.7746			0.0001	
		2	-121.8915	-3.3432	0.7535	
2	3	1	143.8150	6.2287	4.9217	DRUK
		0.7902			0.0000	
		2	-143.8150	-6.2287	1.3070	
1	4	2	112.3714	1.1550	3.2663	DRUK
		2.8279			0.0001	
		3	-112.3714	-1.1550	1.2382	
2	4	2	137.5770	3.3999	7.7745	DRUK
		2.2867			-0.0001	
		3	-137.5770	-3.3999	5.4851	
1	5	4	142.6785	2.2668	2.2908	DRUK
		5	-142.6785	-2.2668	-0.0240	
2	5	4	176.1770	4.6013	4.4567	DRUK
		0.9686			0.0000	
		5	-176.1770	-4.6013	0.1446	
1	6	5	124.6986	4.4550	7.4249	DRUK
		1.6666			0.0000	
		6	-124.6986	-4.4550	9.9495	
2	6	5	153.6070	7.4301	12.8220	DRUK
		1.7257			0.0000	
		6	-153.6070	-7.4301	16.1554	

PROJECT : ELEVATED WATER RESERVOIR, 15 M3

TUSSENPUNTEN: BELASTINGSGEVALLEN

BG	ST	X-POS	Y-VERPL	GRONDDR.	DWARSKR.	MOMENT	MOMENT-PLOT
1	1	0.00	-0.0003		-0.990	-4.020	******
1	1	0.27	-0.0004		-1.638	-3.665	******
1	1	0.54	-0.0004		-2.286	-3.135	*****
1	1	0.81	-0.0004		-2.934	-2.431	****
1	1	1.08	-0.0004		-3.582	-1.551	***
1	1	1.35	-0.0003		-4.230	-0.496	**
1	1	1.62	-0.0003		-4.878	0.733	**
1	1	1.89	-0.0003		-5.526	2.138	****
1	1	2.16	-0.0003		-6.174	3.717	******
1	1	2.43	-0.0003		-6.822	5.471	********
1	1	2.70	-0.0003		-7.470	7.401	**********
2	1	0.00	-0.0003		-4.602	-9.081	********
2	1	0.27	-0.0005		-5.315	-7.743	*******
2	1	0.54	-0.0005		-6.027	-6.212	******
2	1	0.81	-0.0005		-6.740	-4.488	****
2	1	1.08	-0.0005		-7.453	-2.572	***
2	1	1.35	-0.0004		-8.166	-0.463	*
2	1	1.62	-0.0003		-8.879	1.838	**
2	1	1.89	-0.0003		-9.592	4.331	****
2	1	2.16	-0.0002		-10.304	7.017	******
2	1	2.43	-0.0003		-11.017	9.896	********
2	1	2.70	-0.0004		-11.730	12.967	**********
1	2	0.00	-0.0013		55.125	-1.238	*
1	2	0.27	-0.0017		43.270	-14.521	*****
1	2	0.54	-0.0021		31.416	-24.603	********
1	2	0.81	-0.0024		19.556	-31.486	*********
1	2	1.08	-0.0026		7.703	-35.167	**********
1	2	1.35	-0.0026		-4.145	-35.648	**********
1	2	1.62	-0.0026		-15.994	-32.928	**********
1	2	1.89	-0.0024		-27.848	-27.010	********
1	2	2.16	-0.0021		-39.701	-17.890	******
1	2	2.43	-0.0018		-51.554	-5.571	***
1	2	2.70	-0.0015		-63.407	9.949	****

PROJECT : ELEVATED WATER RESERVOIR, 15 M3

TUSSENPUNTEN: BELASTINGSGEVALLEN

BG	ST	X-POS	Y-VERPL	GRONDDR.	DWARSKR.	MOMENT	MOMENT-PLOT
2	2	0.00	-0.0016		64.779	-5.485	**
2	2	0.27	-0.0021		50.220	-21.009	******
2	2	0.54	-0.0026		35.656	-32.604	********
2	2	0.81	-0.0029		21.104	-40.266	**********
2	2	1.08	-0.0031		6.537	-43.999	**********
2	2	1.35	-0.0032		-8.018	-43.800	**********
2	2	1.62	-0.0031		-22.571	-39.670	**********
2	2	1.89	-0.0029		-37.131	-31.609	********
2	2	2.16	-0.0026		-51.690	-19.619	*****
2	2	2.43	-0.0022		-66.247	-3.697	**
2	2	2.70	-0.0018		-80.806	16.155	****

REAKTIES BELASTINGSGEVALLEN

BG	KNOOP	X	Y	M	
1	1	-3.3432	124.0515	2.5897	
1	4	-2.2668	144.8385	2.2908	
		-5.6100	268.8900		SOM VAN DE REAKTIES
		5.6100	-268.8900		SOM VAN DE BELASTINGEN

REAKTIES BELASTINGSGEVALLEN

BG	KNOOP	X	Y	M	
2	1	-6.2287	145.9750	4.9217	
2	4	-4.6013	178.3370	4.4567	
		-10.8300	324.3120		SOM VAN DE REAKTIES
		10.8300	-324.3120		SOM VAN DE BELASTINGEN

PROJECT : ELEVATED WATER RESERVOIR, 50 M3

ONDERDEEL : FRAME
DIMENSIES : KN/M/RAD
DATUM : 90/ 5/23

KOORDINATEN

KNOOP	X	Y
1	0.0000	0.0000
2	0.0000	1.0000
3	0.0000	4.9000
4	0.0000	8.8000
5	4.4000	0.0000
6	4.4000	1.0000
7	4.4000	4.9000
8	4.4000	8.8000

MATERIALEN NORMAAL

MT	OMSCHRIJVING	OPPERVLAK	TRAAGHEID	E-MODULUS	S.M.	POIS.	EAK/BEDD
1	COLUMN	0.1600E+00	0.2130E-02	0.5330E+07	24.00	.00	0.000E+00
2	TOP BEAM	0.2625E+00	0.1230E-01	0.6250E+07	24.00	.00	0.000E+00
3	MIDDLE BEAM	0.1750E+00	0.3650E-02	0.7600E+07	24.00	.00	0.000E+00
4	GROUND BEAM	0.1400E+00	0.1870E-02	0.1000E+08	24.00	.00	0.000E+00

STAAF TOPOLOGIE

STNR	I	J	Xi	Yi	Xj	Yj	LENGTE	MT	NDM-CODE
1	2	6	0.000	1.000	4.400	1.000	4.4000	-4	NDM NDM
2	3	7	0.000	4.900	4.400	4.900	4.4000	-3	NDM NDM
3	4	8	0.000	8.800	4.400	8.800	4.4000	-2	NDM NDM
4	1	2	0.000	0.000	0.000	1.000	1.0000	-1	NDM NDM
5	2	3	0.000	1.000	0.000	4.900	3.9000	-1	NDM NDM
6	3	4	0.000	4.900	0.000	8.800	3.9000	-1	NDM NDM
7	5	6	4.400	0.000	4.400	1.000	1.0000	-1	NDM NDM
8	6	7	4.400	1.000	4.400	4.900	3.9000	-1	NDM NDM
9	7	8	4.400	4.900	4.400	8.800	3.9000	-1	NDM NDM

OPLEGGINGEN

KN	XYR (0=LOS 1=VAST)
1	111
5	111

PROJECT : ELEVATED WATER RESERVOIR, 50 M3

BELASTINGGEVAL 1

DEAD LOAD+LIVE LOAD+WIND

KNOOPBELASTINGEN

KN	RI	KRACHT
4	1	11.08000
4	2	2.83000
8	2	-2.83000
4	2	-160.73999
8	2	-160.73999
3	1	2.80000
3	2	-32.24000
7	2	-32.24000
2	2	-22.37000
6	2	-22.37000
1	2	-3.84000
5	2	-3.84000

STAAFLASTEN

STNR	TYPE	NR. EN OMSCHRIJVING	Q1/FAC/P	Q2	A/ALFA	B/HT
1	1	TRAPEZIUM LOKAAL-Y	-3.3600	-3.3600	0.000000	0.000
2	1	TRAPEZIUM LOKAAL-Y	-4.2000	-11.4900	0.000000	2.200
2	1	TRAPEZIUM LOKAAL-Y	-11.4900	-4.2000	2.200000	0.000
3	1	TRAPEZIUM LOKAAL-Y	-70.9600	-70.9600	0.000000	0.000

BELASTINGGEVAL 2

DEAD LOAD+LIVE LOAD+EARTH QUAKE

KNOOPBELASTINGEN

KN	RI	KRACHT
4	1	31.09000
4	2	-196.52000
8	2	-196.52000
3	2	-35.34000
7	2	-35.34000
2	2	-22.75000
6	2	-22.75000
1	2	-3.84000
5	2	-3.84000

PROJECT : ELEVATED WATER RESERVOIR, 50 M3

STAAFLASTEN

STNR	TYPE	NR. EN OMSCHRIJVING	Q1/FAC/P	Q2	A/ALFA	B/HT
1	1	TRAPEZIUM LOKAAL-Y	-3.5400	-3.5400	0.000000	0.000
2	1	TRAPEZIUM LOKAAL-Y	-5.6200	-12.9100	0.000000	2.200
2	1	TRAPEZIUM LOKAAL-Y	-12.9100	-5.6200	2.200000	0.000
3	1	TRAPEZIUM LOKAAL-Y	-87.0300	-87.0300	0.000000	0.000

VERPLAATSINGEN BELASTINGSGEVALLEN

BG	KN	X-VERPL.	Y-VERPL.	ROTATIE
1	1	0.00000	0.00000	0.00000
2	1	0.00000	0.00000	0.00000
1	2	0.00030	-0.00044	-0.00057
2	2	0.00067	-0.00049	-0.00119
1	3	0.00550	-0.00203	-0.00063
2	3	0.01264	-0.00234	-0.00155
1	4	0.00957	-0.00344	-0.00265
2	4	0.02348	-0.00404	-0.00349
1	5	0.00000	0.00000	0.00000
2	5	0.00000	0.00000	0.00000
1	6	0.00029	-0.00049	-0.00039
2	6	0.00065	-0.00062	-0.00099
1	7	0.00553	-0.00225	-0.00067
2	7	0.01268	-0.00283	-0.00164
1	8	0.00953	-0.00374	0.00228

PROJECT : ELEVATED WATER RESERVOIR, 50 M3

VERPLAATSINGEN BELASTINGSGEVALLEN

BG	KN	X-VERPL.	Y-VERPL.	ROTATIE
2	8	0.02340	-0.00468	0.00256

STAAFKRACHTEN BELASTINGSGEVALLEN

BG	ST	KN/AFST	NXi/NXj	DYi/DYj	MXi/MXj	
1	1	2	4.7773	1.9403	-7.3433	DRUK
		0.5775			-7.9035	
		2.7465			0.0001	
		6	-4.7773	12.8437	-16.6442	
2	1	2	5.2310	-4.5458	-22.2532	DRUK
		2.4870			0.0000	
		6	-5.2310	20.1218	-32.0153	
1	2	3	-9.0890	7.0248	-8.1507	TREK
		1.1505			-12.6120	
		2.7677			0.0000	
		7	9.0890	27.4932	-36.8798	
2	2	3	-12.9055	-5.0255	-38.9091	TREK
		2.5063			0.0000	
		7	12.9055	45.7915	-72.8883	
1	3	4	16.4021	150.5903	16.2590	DRUK
		0.1109			0.0000	
		2.1222			-143.5313	
		4.1335			0.0000	
		8	-16.4021	161.6337	-40.5546	
2	3	4	28.7520	176.1981	1.1048	DRUK
		0.0063			0.0000	
		2.0246			-177.2577	
		4.0429			0.0000	
		8	-28.7520	206.7339	-68.2834	
1	4	1	372.0754	1.7895	7.4088	DRUK
		2	-372.0754	-1.7895	-5.6193	
2	4	1	421.2369	10.0119	18.5160	DRUK
		2	-421.2369	-10.0119	-8.5042	
1	5	2	347.7651	6.5668	12.9626	DRUK
		1.9740			-0.0001	
		3	-347.7651	-6.5668	12.6481	

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SANITATION (IRC)

PROJECT : ELEVATED WATER RESERVOIR, 50 M3

TUSSENPUNTEN: BELASTINGSGEVALLEN

BG	ST	X-POS	Y-VERPL	GRONDDR.	DWARSKR.	MOMENT	MOMENT-PLOT
2	1	0.00	-0.0005		-4.546	-22.253	********
2	1	0.44	-0.0009		-6.104	-19.910	*******
2	1	0.88	-0.0011		-7.661	-16.882	******
2	1	1.32	-0.0011		-9.218	-13.169	*****
2	1	1.76	-0.0010		-10.776	-8.770	****
2	1	2.20	-0.0008		-12.334	-3.686	**
2	1	2.64	-0.0006		-13.891	2.084	**
2	1	3.08	-0.0004		-15.449	8.539	***
2	1	3.52	-0.0003		-17.007	15.679	******
2	1	3.96	-0.0003		-18.564	23.504	********
2	1	4.40	-0.0006		-20.122	32.015	**********
1	2	0.00	-0.0020		7.027	-8.149	***
1	2	0.44	-0.0023		4.857	-10.788	****
1	2	0.88	-0.0024		2.045	-12.330	****
1	2	1.32	-0.0025		-1.405	-12.494	****
1	2	1.76	-0.0025		-5.500	-10.999	****
1	2	2.20	-0.0024		-10.235	-7.561	***
1	2	2.64	-0.0023		-14.970	-1.993	**
1	2	3.08	-0.0022		-19.062	5.518	**
1	2	3.52	-0.0021		-22.515	14.689	*****
1	2	3.96	-0.0021		-25.325	25.237	*******
1	2	4.40	-0.0023		-27.493	36.881	**********
2	2	0.00	-0.0023		-5.024	-38.908	******
2	2	0.44	-0.0029		-7.819	-36.107	******
2	2	0.88	-0.0032		-11.255	-31.935	*****
2	2	1.32	-0.0033		-15.330	-26.109	****
2	2	1.76	-0.0031		-20.050	-18.350	***
2	2	2.20	-0.0029		-25.410	-8.372	**
2	2	2.64	-0.0026		-30.768	4.010	*
2	2	3.08	-0.0024		-35.487	18.610	***
2	2	3.52	-0.0022		-39.564	35.144	*****
2	2	3.96	-0.0023		-43.000	53.331	********
2	2	4.40	-0.0028		-45.793	72.889	**********

PROJECT : ELEVATED WATER RESERVOIR, 50 M3

STAAFKRACHTEN BELASTINGSGEVALLEN

BG	ST	KN/AFST	NXi/NXj	DYi/DYj	MXi/MXj	
2	5	2	403.0327	15.2428	30.7574	DRUK
		2.0178			0.0001	
		3	-403.0327	-15.2428	28.6896	
1	6	3	308.5002	-5.3222	-4.4974	DRUK
		0.8450			0.0000	
		4	-308.5002	5.3222	-16.2590	
2	6	3	372.7181	2.3371	10.2195	DRUK
		4	-372.7181	-2.3371	-1.1048	
1	7	5	420.1506	12.0905	10.5016	DRUK
		0.8686			-0.0001	
		6	-420.1506	-12.0905	1.5889	
2	7	5	527.2572	21.0781	21.8315	DRUK
		6	-527.2572	-21.0781	-0.7534	
1	8	6	384.9369	7.3132	15.0553	DRUK
		2.0587			0.0000	
		7	-384.9369	-7.3132	13.4660	
2	8	6	484.3854	15.8472	32.7686	DRUK
		2.0678			0.0001	
		7	-484.3854	-15.8472	29.0354	
1	9	7	325.2038	16.4022	23.4139	DRUK
		1.4275			0.0000	
		8	-325.2038	-16.4022	40.5546	
2	9	7	403.2540	28.7529	43.8529	DRUK
		1.5252			0.0000	
		8	-403.2540	-28.7529	68.2834	

TUSSENPUNTEN: BELASTINGSGEVALLEN

BG	ST	X-POS	Y-VERPL	GRONDDR.	DWARSKR.	MOMENT	MOMENT-PLOT
1	1	0.00	-0.0004		1.940	-7.343	*****
1	1	0.44	-0.0006		0.462	-7.872	******
1	1	0.88	-0.0008		-1.017	-7.750	*****
1	1	1.32	-0.0008		-2.495	-6.977	*****
1	1	1.76	-0.0008		-3.974	-5.554	****
1	1	2.20	-0.0007		-5.452	-3.481	***
1	1	2.64	-0.0006		-6.930	-0.757	**
1	1	3.08	-0.0005		-8.408	2.618	**
1	1	3.52	-0.0004		-9.887	6.643	*****
1	1	3.96	-0.0004		-11.365	11.318	*******
1	1	4.40	-0.0005		-12.844	16.644	**********

PROJECT : ELEVATED WATER RESERVOIR, 50 M3

TUSSENPUNTEN: BELASTINGSGEVALLEN

BG	ST	X-POS	Y-VERPL	GRONDDR.	DWARSKR.	MOMENT	MOMENT-PLOT
1	3	0.00-0.0034		150.599	16.266		**
1	3	0.44-0.0046		119.378	-43.128		****
1	3	0.88-0.0057		88.154	-88.785		*******
1	3	1.32-0.0065		56.929	-120.704		*********
1	3	1.76-0.0070		25.695	-138.884		**********
1	3	2.20-0.0072		-5.520	-143.320		**********
1	3	2.64-0.0070		-36.746	-134.020		**********
1	3	3.08-0.0065		-67.976	-110.983		********
1	3	3.52-0.0057		-99.197	-74.207		******
1	3	3.96-0.0048		-130.420	-23.693		***
1	3	4.40-0.0037		-161.642	40.561	****	
2	3	0.00-0.0040		176.212	1.114		*
2	3	0.44-0.0055		137.918	-67.992		*****
2	3	0.88-0.0069		99.625	-120.249		********
2	3	1.32-0.0079		61.328	-155.662		*********
2	3	1.76-0.0086		23.026	-174.215		**********
2	3	2.20-0.0088		-15.270	-175.925		**********
2	3	2.64-0.0086		-53.564	-160.782		**********
2	3	3.08-0.0079		-91.869	-128.787		********
2	3	3.52-0.0070		-130.161	-79.942		*****
2	3	3.96-0.0059		-168.460	-14.253		**
2	3	4.40-0.0047		-206.753	68.293	*****	

REAKTIES BELASTINGSGEVALLEN

BG	KNOOP	X	Y	M
1	1	-1.7895	375.9154	7.4088
1	5	-12.0905	423.9906	10.5016
		-13.8800	799.9060	SOM VAN DE REAKTIES
		13.8800	-799.9060	SOM VAN DE BELASTINGEN

REAKTIES BELASTINGSGEVALLEN

BG	KNOOP	X	Y	M
2	1	-10.0119	425.0769	18.5160
2	5	-21.0781	531.0972	21.8315

PROJECT : ELEVATED WATER RESERVOIR, 50 M3

-31.0900	956.1741	SOM VAN DE REAKTIES
31.0900	-956.1740	SOM VAN DE BELASTINGEN

10. FIGURES

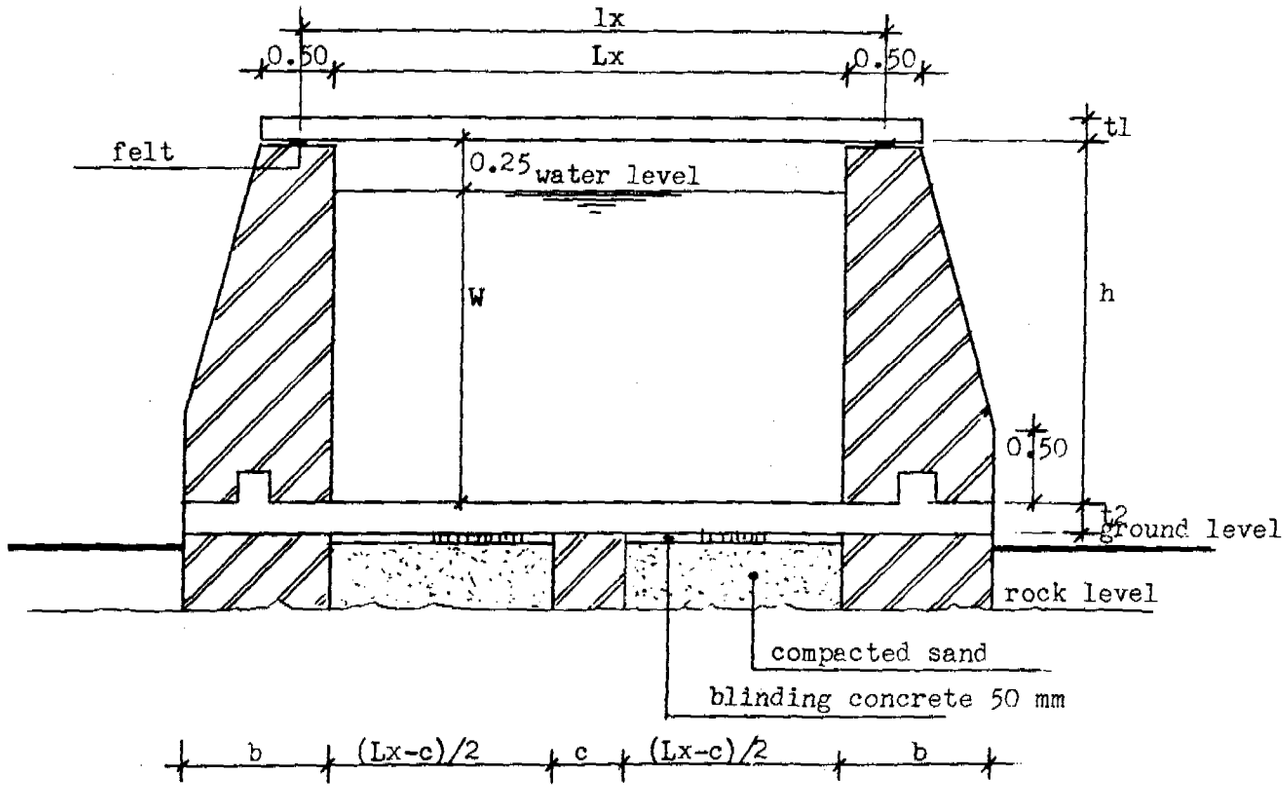

figure 2 masonry reservoir

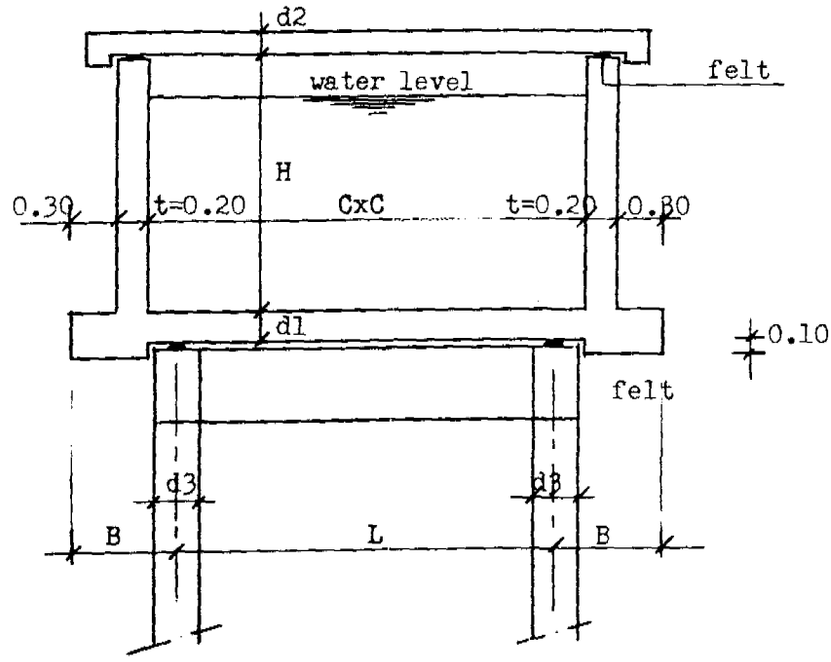

figure 3 reinforced concrete elevated reservoir

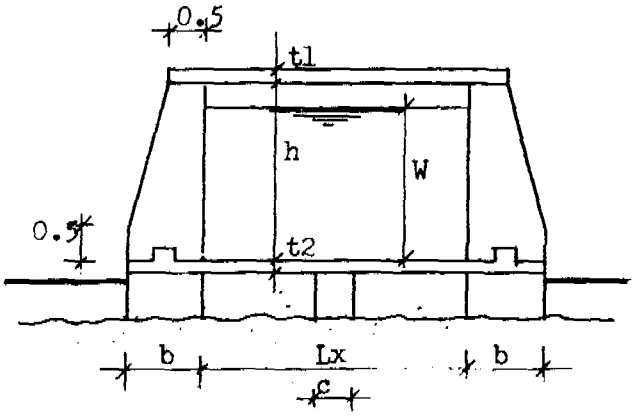

fig. 4-2 cross section A-A

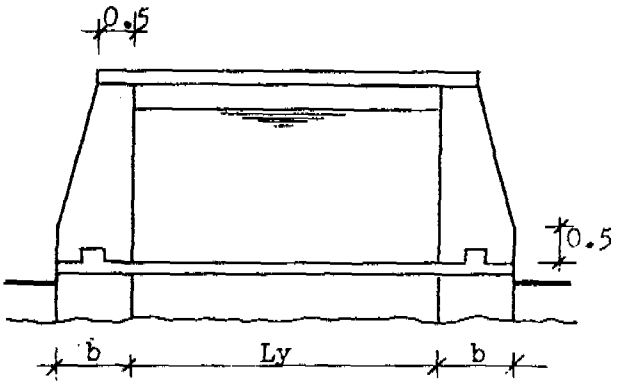

fig. 4-3 cross section B-B

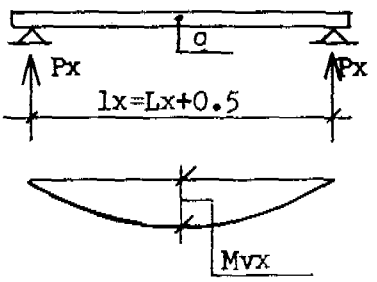

fig. 4-4

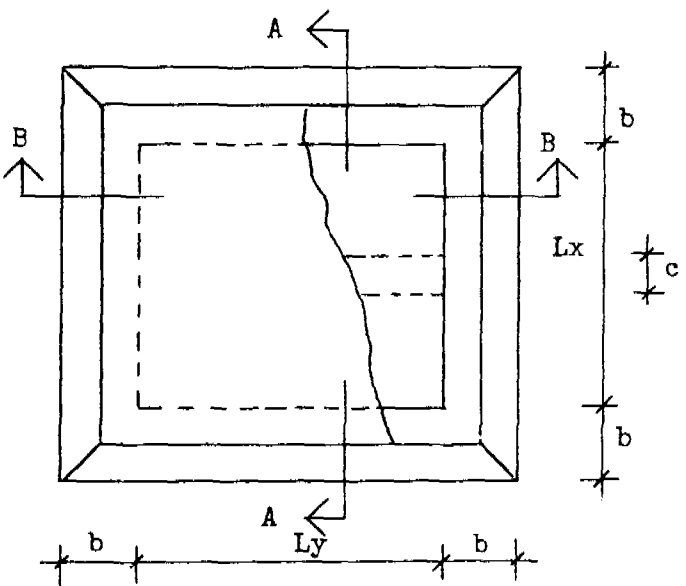

fig. 4-1 top view

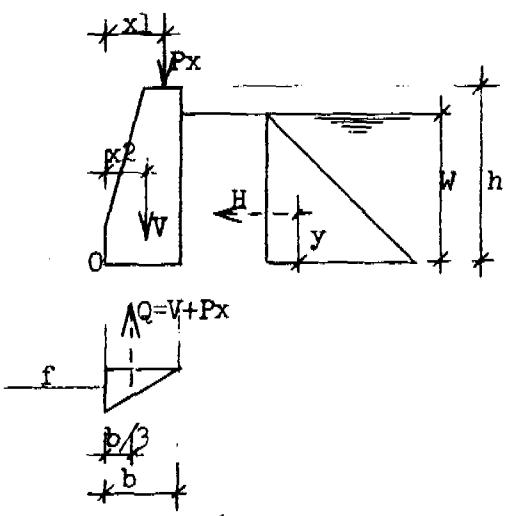

fig. 4-6 load scheme of wall

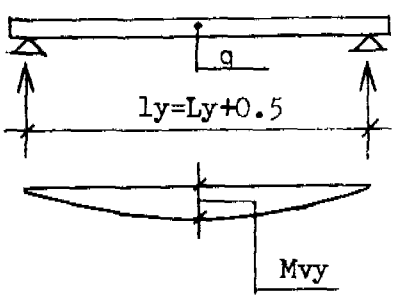

fig. 4-5

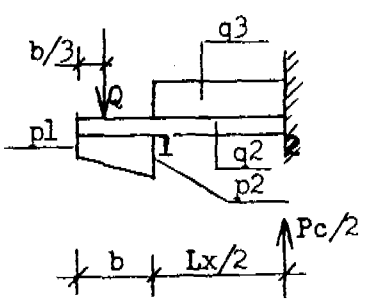

fig. 4-7 load scheme of bottom slab

figure 4 masonry reservoir

ANNEX A

GENERAL DESCRIPTION FOR THE CONSTRUCTION OF MASONRY RESERVOIRS

1. The foundation of the reservoir will, depending on site conditions, be at least 0.5 m deep.

2. In case the underground consists of solid rock the foundation will consist entirely of masonry work up to 0.2 m above the natural groundlevel.
   In case the underground consists of soft material a floorslab of mass concrete, thick 0.3 m and with the same width as the masonry work must be made. The rest of the foundation will consist of masonry work up to 0.2 m above the natural ground level.

3. The foundation will be built according to the drawings and the instructions of the supervising engineer.

4. Inside the foundation walls, soil has to be filled and well compacted up to 0.2 m under the level of the foundation walls. The top layer of 0.15 m must consist of sand and finely graded gravel.

5. A blinding layer of mass concrete, mixture 1:3:6, thickness 0.05 m, must be poured within the foundation walls so that the top of the blinding layer is level with the top of the foundation walls.

6. The bottom slab of reinforced concrete must be poured on top of the foundation walls and the blinding layer.
   The whole slab has to be cast in one day and no joints should be made.
   The use of a vibrator is obligatory and a minimum hardening period of three days should be observed before starting the masonry works.
   During at least 20 days after pouring, the slab should be kept wet.

7. The walls consist entirely of masonry work, regularly sloping on the outside so as to have a width of 0.5 m at the top.
   During bricklaying the masonry will be kept wet for a period of at least 20 days.

8. Several pipes ϕ 3" will have to be put in position through the walls and the bottom slab.
All pipes must be provided with steel collars ϕ 350 mm, properly welded on the pipes (2 wash out pipes, 2 outlets, 2 overflow pipes and one inlet pipe; see ANNEX C).

9. After sufficient hardening time of the masonry (3 days) the top slab may be cast. The slab should be put on felt strips, width 100 mm, thickness 5 mm.
For the conditions of this concrete work the same applies as under 6.

10. In the top slab 2 manhole covers of 0.6 x 0.6 m will be made. The frame and the covers must be of steel and galvanized. See AANEX C drawing nr MR-06.
Furthermore 4 ventilation pipes ϕ 3" will be placed at regular distances.

11. All pipes mentioned under 8 and 10 should be made of hot dip galvanized steel.

12. The inside of the reservoir, i.e. the walls and the bottom slab, will be plastered with cement mortar, mixture 1:3, minimum thickness on the walls 0.05 m and on the floor 0.03 m.
Plastering on the floor must be done in such a way that the floor is sloping toward the wash-out pipes.

13. All outside joints of the masonry work will be painted with bitumen and therefore must be cleaned before.

14. The mixture for 1 m^3 of all concrete works will consist of:
- minimum 325 kg Portland cement;
- maximum 175 liter clean water;
- 697 kg or 0.263 m^3 clean sand;
- 1190 kg or 0.449 m^3 gravel.

15. For all concrete works the concrete cover will be 20 mm.

16. The reinforcement will be mild steel FeB 220 and must be put according to the drawings and the instructions of the supervising engineer.

17. The strength of the concrete will be tested at a laboratory. Therefore the contractor will make 6 cubes of the concrete mixes required for the bottom slab and 6 cubes of the concrete mixes required for the top slab. After 28 days of hardening these cubes have to be tested. The minimum average cube strength should be 27 N/mm^2.
The dimensions of the cubes will be 0.2 x 0.2 x 0.2 m and have to be cast in steel moulds.

Bill of quantities for masonry reservoirs.

No.	Description	Unit	Volume of reservoir (m^3)				
			30	50	75	100	250
1	Masonry half cut stones for foundation and walls	m^3	44	58	94	120	218
2	Soil backfill	m^3	6	9	11.5	13	31
3	Mass concrete	m^3	0.7	1.0	1.3	1.5	3.5
4	Reinforced concrete bottom slab	m^3	9.24	12.28	16.30	19.30	45
5	Reinforced concrete top slab	m^3	3.5	6.0	7.5	10.0	22.5
6	Felt strips 100 x 5 mm	m	17.5	21.5	24.5	26.5	59
7	Inside plastering	m^2	51	71	95	120	282
8	Miscellaneous steel works:						
	- manhole covers 0.6x0.6 m	no	2	2	2	2	2
	- ϕ3" G.I . pipes long 1.30 m	no	7	7	7	7	7
	- ϕ3" brass valves	no	4	4	4	4	4
	- collars ϕ 350 mm	no	7	7	7	7	7
	- vent. pipes ϕ3", long 0.4 m	no	4	4	4	4	8
	- catwalk ϕ 20 mm	m	8	8	9	10	22

ANNEX B

GENERAL DESCRIPTION FOR THE CONSTRUCTION OF ELEVATED REINFORCED CONCRETE RESERVOIRS.

1. The column foundation of the reservoir will, depending on site conditions, be at last 0.5 m deep. It will consist entirely of reinforced concrete, thick 0.4 m on a layer of 0.05 m blinding (mass concrete).
2. After three days hardening, the shuttering can be placed for the columns and the lower tie beams.
   The shuttering of the columns must be absolutely upright and shored properly in such a way that topling of the shuttering cannot occur.
3. Pouring of concrete in the columns should not exceed a total height of 1.5 m per pour. Use of a vibrator is obligatory for the whole concrete structure.
4. After ten days hardening of the columns, shuttering can be placed for the rest of the frame structure.
   For the pouring of the concrete the same applies as under point 3.
5. The topbeams must be level before placing the felt strips, dimension 100 x 5 mm.
6. After a hardening time of the concrete of the top beams of at least 10 days the shuttering for the floor slab of the water reservoir can be made.
7. Directly after pouring concrete of the floor slab, steel strips 100 x 6 mm have to be put in the concrete in the center line of the walls.
   Pouring concrete of the floor slab must be done in one day and no joints should be made.
   The use of a vibrator is obligatory and an minimum hardening time of 5 days should be observed before starting the shuttering for the walls.

8. Shuttering for the walls, outside as well as inside, must consist of plywood sheets, 18 mm thick, properly shored and tightened so that bulging during the pouring of concrete cannot occur.
For the pouring of concrete the same applies as under point 3.

9. After a hardening time of the concrete of the walls of at least 8 days felt strips, dimension 100 x 5 mm, have to be put on the top of the wall and the shuttering for the top slab can be placed.
Again the shuttering should be of plywood sheets, 18 mm thick, properly supported.
The whole top slab must be poured in one day.

10. All concrete, footings, beams, columns, floors and walls should be kept wet during at least 20 days after pouring.

11. The shuttering has to be removed as follows:
- for the columns after 28 days;
- for the beams after 20 days;
- for the floors after 20 days;
- for the walls after 20 days.

12. After removal of the shuttering the concrete must be repaired with mortar, mixture 1:3, on those places where honey combing and/or the reinforcement is visible.

13. The inside and outside of the water compartment, i.e. the walls and the floor will be plastered with cement mortar, mixture 1:3, minimum thickness 30 mm.
Plastering of the bottom slab must be done in such a way that the floor is sloping towards the wash-out pipes.
Outside plastering of the walls must have a minimum thickness of 15 mm.

14. Several pipes ϕ3" will have to be put through the walls and the floor of the water compartment.
All pipes should be galvanised and be provided with steel collars ϕ 350 mm, properly welded on the pipes.

15. After completion of the reservoir a steel ladder has to be put in position in accordance with the instructions of the supervising engineer.

16. In the roof one manhole cover, 0.6 x 0.6 m, will be placed.
The frame and the cover must be of steel and galvanised.
Furthermore, 4 ventilation pipes ϕ3" will be placed.

17. The concrete cover to the reinforcement will be for:
- footings 30 mm;
- columns 30 mm;
- beams 25 mm;
- floor slabs 20 mm;
- walls 30 mm.

18. The reinforcement will consist of mild steel FeB 220 (except for the columns) and must be placed according to the drawings and the instructions of the supervising engineer.
The reinforcement of the columns will be of high yield steel FeB 400.

19. The mixture for 1 m^3 of all concrete works will consist of:
- minimum 325 kg Portland cement;
- maximum 175 litres clean water;
- 697 kg or 0.263 m^3 clean sand;
- 1190 kg or 0.449 m^3 gravel of maximum 31 mm diameter.

20. The strength of the concrete will be tested at a laboratory.
Therefore the contractor will make 6 cubes of the concrete mixes he used for each pour.
After 28 days of hardening those cubes will be tested.
The minimum average cube strength will be 27 N/mm^2.
The dimension of the cubes will be 0.2 x 0.2 x 0.2 m and will be cast in steel moulds.

Bill of quantities for elevated reinforced concrete reservoirs.

No.	Description	Unit	Volume of reservoir	
			15 m^3	50 m^3
1	mass concrete	m^3	0.5	1.0
2	reinforced concrete			
	- footings	m^3	3.5	6.0
	- columns	m^3	1.5	6.1
	- beams	m^3	2.5	10.0
	- platform	m^3	-	2.5
3	reinforced concrete of water compartment			
	- bottom slab	m^3	4.0	10.7
	- walls	m^3	5.0	9.3
	- top slab	m^3	2.92	7.8
4	felt strips 100 x 5 mm	m	24	39
5	inside plastering	m^2	43	102
	outside plastering	m^2	70	115
6	miscellaneous steelworks			
	- steel strip 100 x 6 mm	m	13	21
	- ϕ3" G.I. pipes, 0.5 m long	no	5	5
	- ϕ3" brass valves	no	3	3
	- collars ϕ 350 mm	no	5	5
	- ladder	m	7	12
	- manhole cover 0.6 x 0.6 m	no	1	1