

Ild. Influence of concreting and dilatation joints

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II d

Influence of Concreting and Dilatation Joints.

Einfluß von Betonierungs- und Bewegungsfugen.

Influence des reprises de bétonnage et des joints de dilatation.

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II d 1

Concreting and Dilatation Joints in Engineering Structures.

Betonierungs- und Bewegungsfugen in Ingenieurbauten.

Joints de bétonnage et de dilatation
dans les constructions du Génie civil.

Dr. techn. F. Baravalle,
Ingenieur am Stadtbauamt Wien.

The new knowledge obtained within recent years in reinforced concrete engineering by research work both scientific and practical has made it possible to increase concrete stresses without reducing the safety of the supporting frames. In spite of the increased stresses on concrete, the safety with regard to concrete strength is often higher than formerly, the reason being that

- 1) the quality of cement is continuously being improved,
- 2) the important experience gained in tests concerning the granulation of the aggregate can be utilised more and more in building practice,
- 3) adequate attention can be paid to the water-cement ratio and the air-content of fresh concrete in structures,
- 4) it is possible to use reinforcement steels without a definite yield point,
- 5) investigations of the theoretical fundamental for determining the actually effective stresses have made considerable progress
- 6) the knowledge of the development of internal strains in reinforced concrete supporting structures could be considerably increased by comprehensive series of tests within the last few years.

But together will an increase in admissible stresses on concrete many more questions and tasks arise in the construction of supporting structures. The dimensions of the supporting structure determined for a certain load, type of stress or type of construction do not correspond in like manner in the case of slender or squat, high or low cross sections. If the sectional dimensions are reduced, the influences of

- 1) bending,
- 2) oscillations of the supporting structure,
- 3) buckling,
- 4) chemical, mechanical and thermic attacks are increased, while the internal tensions are often reduced by influences of temperature and by the concrete shrinking.

The investigation of internal stresses, of the plasticity of concrete and the oscillations of the supporting structure in the case of impulse loads¹ and one-million-fold charges of load² within the most various limits of stresses has hitherto advanced so far that at least an approach to a determination of these influences is always possible by computation. Similarly, the internal stresses arising in consequence of variations of temperature and shrinkage of concrete can be computed; however, the dimension-ratio of the tensional influences varies in structural parts of different slenderness, and, therefore, the rules of construction hitherto in standard practice in modern reinforced concrete engineering will in many ways have to be changed.

The following juxtaposition of the building construction of two water reservoirs of the Vienna Municipal Water Supply is to show the influence of new experience in reinforced concrete engineering on the shape and construction of supporting structures.

The Water Reservoir at Krapfenwaldl.

This reservoir was built in the years 1923 to 1926 and has an area of 27.7 m by 39.4 m, a mean height of 5.58 m and a capacity of 4900 m³. A 30 cm continuous middle well divides the reservoir into two independent reservoir chambers. The arrangement of the supporting structure (beamless ceiling) is shown in ground elevation and in its main sections in Fig. 1—2. The dimensioning of the reinforced concrete cross sections was based on the "regulations on the construction of supporting structures of reinforced concrete or concrete in the case of high buildings", 1921. The maximum stresses arise in the reservoir floor slab and are computed for condition II (tension zone in concrete imagined to be entirely neglected), with

$$n = \frac{E_e}{E_b} = 15 \text{ at } \sigma_b = 60 \text{ kg/cm}^2 \quad \sigma_e = 1200 \text{ kg/cm}^2$$

The cube-strength required in accordance with these stresses (cube 20 by 20 by 20 cm) was $W_{b,28} = 170 \text{ kg/cm}^2$. The maximum stress in the reservoir columns, not considering variations in temperature, amounts to 38 kg/cm² when the floor is loaded in panels, and 25 kg/cm² with the floor slab under full load all over.

In order to eliminate shrinkage tensions and tensions due to unequal settling of the structure, the arrangement of construction (expansion) joints was recommended by Hofrat Prof. Dr. Ing. R. Saliger. These joints divide the entire supporting structure of the reservoir foundation without concrete base, wells and floor slabs into 6 individual structural parts of a maximum length of 13.7 m each (Fig. 1 and 3).

¹ Testing reinforced concrete beams under impulse load, by Prof. Dr. Ing. R. Saliger, Vienna Polytechnical Institute. — Test Report being prepared.

² Duration tests of reinforced concrete beams with different steel-reinforcements, by Prof. Dr. Ing. R. Saliger, Vienna Polytechnical Institute. — Reports on tests carried out by the Oest. Eisenbetonausschuß, vol. 15, Vienna 1935, Oe. I. A. V.

³ The Water Reservoir Krapfenwaldl of the Vienna Water Supply, by Prof. Dr. Ing. R. Saliger, Polytechnical Institute, Vienna. Beton und Eisen 1926, volume 13, Oest. Bauzeitung 1926, issue 17).

Arbeitsfuge 1m breit
 joint de bétonnage 1m de largeur
 temporary construction joint, 1m wide

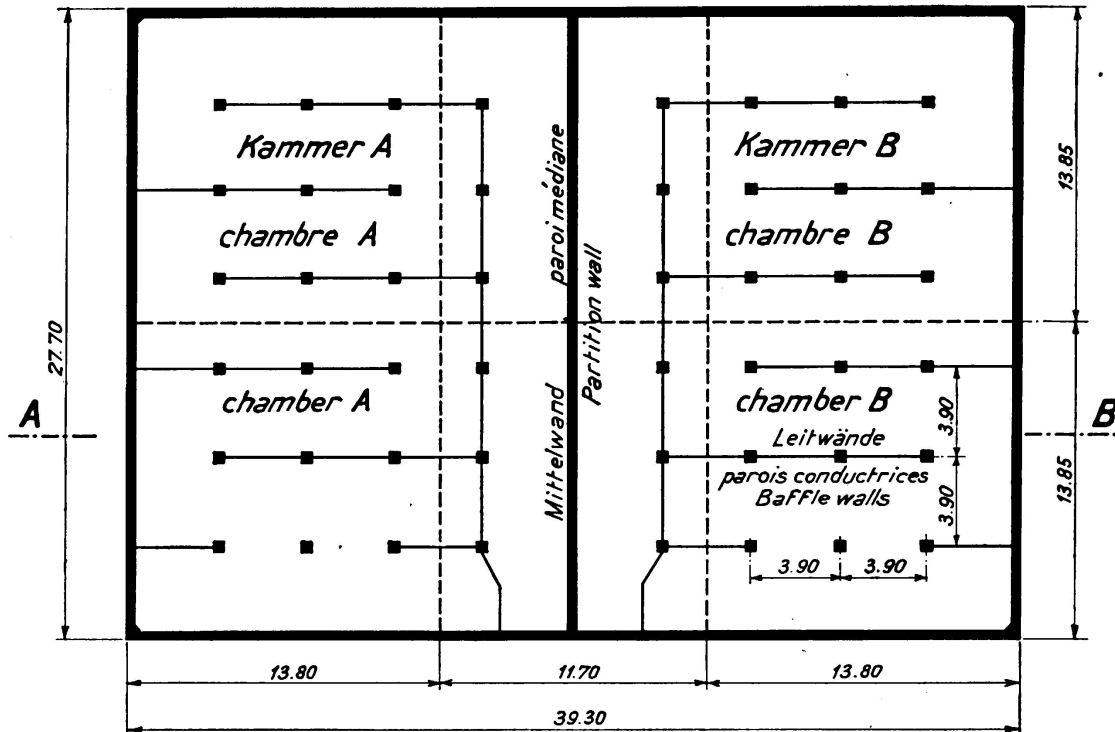


Fig. 1.

Water reservoir Krapfenwaldl. Plan.

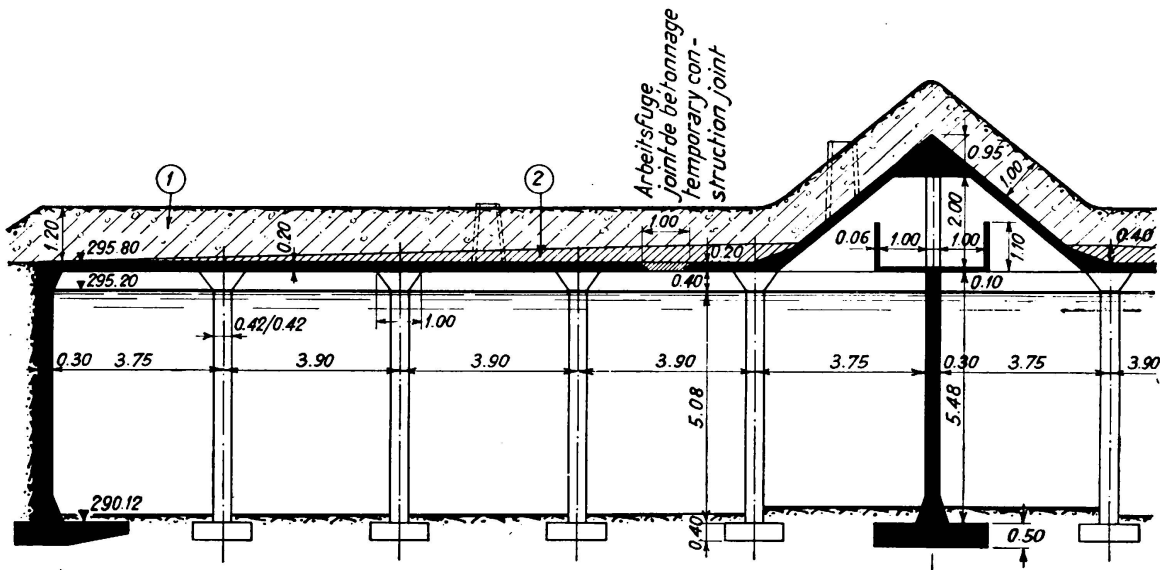


Fig. 2.

Water reservoir Krapfenwaldl. Section A—B.

- 1 Earth filling
- 2 Lean concrete.

These working joints between the individually constructed component parts, into which the steel reinforcement of the floor slab, and the distribution bars in the wall respectively, enter at least 70 cm on either side — but not continuously, in order to prevent unfavourable pre-stressing in steel and concrete, were kept open for a period as long as possible. The width of the joints is given

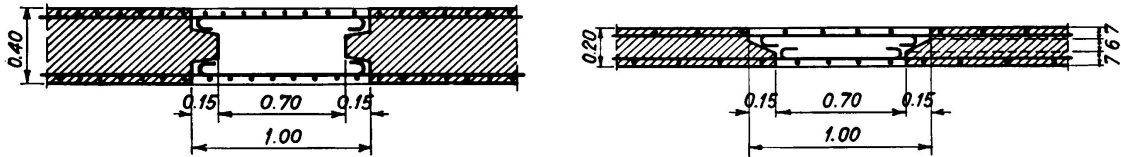


Fig. 3.

Water reservoir Krapfenwaldl. Work joint in wall and in roofslab.

by the length of the necessary overlapping of the reinforcements. Their arrangement in the wall and floor slab is shown in Fig. 3. The concrete used for filling the construction joints was of the same composition as that of the adjoining supporting structure.

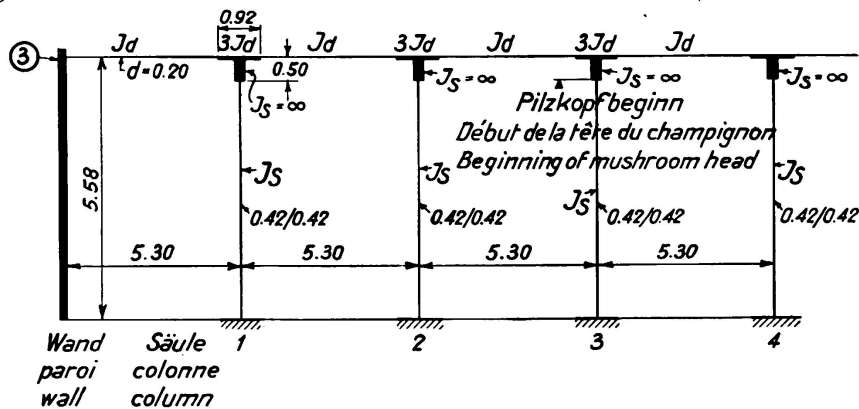


Fig. 4.

Water reservoir Krapfenwaldl.

3 Restraint; assumed rigidity of wall ∞ , M_o , M_R , M_u .

By arranging these short-period, purposeful construction joints, the harmful influences of shrinkage which is strongest in new concrete could be avoided and, as the completed parts were poured in previous to the filling of these joints,

the greater portion of the influence of the supporting structure settlements was limited to the individual parts of the building. By means of these construction joints it was possible to carry out the reservoir without permanent joints. When reckoning, according to the method of the substituting frame, of the fixed-point theory, the bending moments and stresses arising in the structure at variations of temperature of $t = 10^{\circ} \text{C}$, the values in table I will be obtained.

The suppositions for the computation of the frame and the course of moments are shown in Fig. 4.

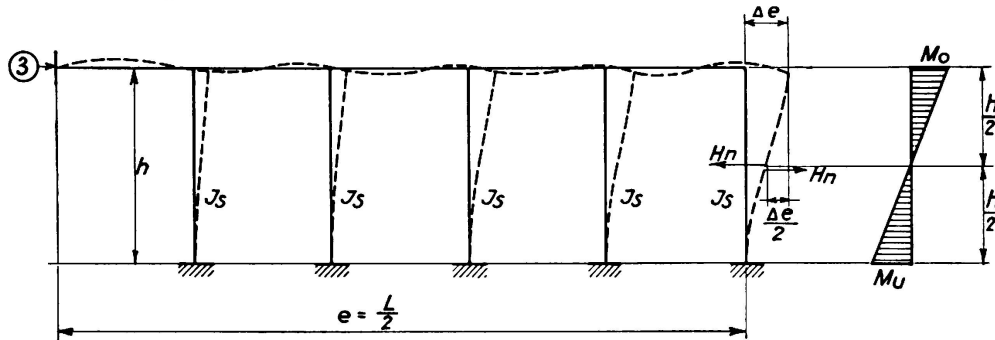


Fig. 5.

Water reservoir Krapfenwaldl.

3 Restraint; assumed rigidity of wall ∞ , M_0 , M_R , M_U .

This considerable influence of head effects is explained by the size of the structure and the comparatively insignificant slenderness of the square columns at a mean height of $h = 5.58 \text{ m}$ and a thickness of $d = 42 \text{ cm}$, the ratio of slenderness of the columns is $\frac{h}{d} = \frac{558}{42} = 13.3$.

From the consideration of the deformation of a frame irrespective of the bending of the beams, the forces and stresses arising in the columns (imagined to be rigidly held at the bottom) by prolonging the beam, with the same slenderness of pillars, can be computed according to Fig. 5.

$$\frac{H_n \cdot \left(\frac{h}{2}\right)^3}{3 \cdot E \cdot J_s} = \frac{H_n \cdot h}{2} \frac{\left(\frac{h}{2}\right)^2}{3 E \cdot J_s} = \frac{\Delta e}{2} = \frac{w \cdot \Delta t \cdot L}{4}$$

M being $= \frac{H_n \cdot h}{2}$, according to condition I, $E_{bz} = E_{lul} = 210\,000 \text{ kg/cm}^2 = E$, and not taking into account the reinforcements

$$M = \frac{H_n \cdot h}{2} = \frac{2 J_s}{d} \cdot \sigma_b \text{ and } \frac{2 J_s}{d} \cdot \sigma_b = \frac{w \cdot \Delta t \cdot L}{4} \frac{3 E J_s}{\left(\frac{h}{2}\right)^2},$$

$$\text{hence } \sigma_b = \frac{d \cdot w \cdot \Delta t \cdot L}{4 \cdot 2 \cdot J_s} \frac{3 E J_s}{\left(\frac{h}{2}\right)^2} = \frac{3}{2 h^2} \cdot d \cdot w \cdot \Delta t \cdot E \cdot L$$

For $h = 5.58 \text{ m}$, $d = 0.42 \text{ m}$, $L = 2 \cdot 4 \cdot 3.90 \text{ m} = 31.20 \text{ m}$,
 $\Delta t = 10^{\circ} \text{C}$, $w = 0.00001$, $E = 210\,000 \text{ kg/cm}^2$,

$$\sigma_b = \frac{3}{2 \cdot 5.58^2} \cdot 42 \cdot 0.00001 \cdot 10 \cdot 210\,000 \cdot 3120 = 13.2 \text{ kg/cm}^2,$$

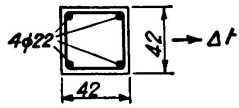
as opposed to,
 $\sigma_b = 13 \text{ kg/cm}^2$ according to the exact computation of the frame (Fig. 4, table I).

Table I.

Water reservoir Krapfenwaldl.

Bending moments and stresses arising in the columns due to a change in temperature of $\Delta t = 10^\circ \text{C}$ (computed acc. to condition II: concrete tension zone imagined to be entirely eliminated and

$$n = \frac{E_e}{E_b} = 15).$$

Distance from external wall latter assumed to be in fixed position		column 1 3,90	2 7,80	3 11,70	4 15,60 m
Bending moment in the axis of the structure	M_0	0,480	0,968	1,514	1,703
Bending moment at the transition between mushroom and slab	M_R	0,398	0,800	1,250	1,410
Dimensions of cross section and column reinforcement at the transition between mushroom and slab		 <p style="text-align: right;"> $d = 42 \text{ cm}$ $h = 39 \text{ cm}$ $x_{II} = 10,62 \text{ cm}$ $F_e = F'_e = 7,60 \text{ cm}^2$ $J_{II} = 115356 \text{ cm}^4$ </p>			
Compression stress in concrete due to M_R		3,7	7,4	11,5	13,0 kg/cm^2
Tensile stress in steel due to M_R		147	296	462	520 kg/cm^2

The values of the stresses arising in such structures due to variations in temperature can therefore be determined approximately by means of this formula.

*The Water Reservoir at Lainz*⁴.

This reservoir, which is at present being built in the game preserves near Vienna and, to the author's knowledge, represents the largest covered drinking water reservoir in the world, has an area of 25.000 m², a mean height of 6.85 m and a capacity of 144 000 m³. A central partition wall 35 cm in thickness

⁴ The building of the water reservoir in the game Preserves at Lainz by Oberstadtbaurat Dr. Ing. R. Tillmann, Zeitschrift des Oest. Ing. und Arch. Vereines 1936, issues 21/22. The projected design carried out was based on the design by Hofrat Prof. Dr. Ing. R. Saliger for the water reservoir Krapfenwaldl. The building operations were superintended by Oberstadtbaurat Dr. Ing. R. Tillmann and Baurat Ing. Zaubek of Municipal Dept. 27 (Vienna Municipal Waterworks). The following contractors were entrusted with the construction (avoised by Prof. Dr. Ing. R. Saliger and Dr. Ing. F. Pongratz). Cooperative work in building the water reservoir at Lainz: Allgemeine Baugesellschaft A. Porr; Universale Redlich & Berger, Bau A.-G.; Bauunternehmung H. Rella & Co.; Bauunternehmung F. Swittalek's Witwe; Bauunternehmung G. & W. Gröger. Zeitschrift des Oest. Ingenieur- und Architekten-Vereins 1936, issue 21/22.

divides the reservoir into two independent chambers A and B. The arrangement of the structure mushroom construction in plan and in section is shown in Fig. 6 and 11. The construction of the reservoir was based on the 4th edition (already completed in draft) of Austrian Reinforced Concrete Standards B 2302. Besides its importance as a structure of water-supply engineering, the building deserves special attention because of its type of construction.

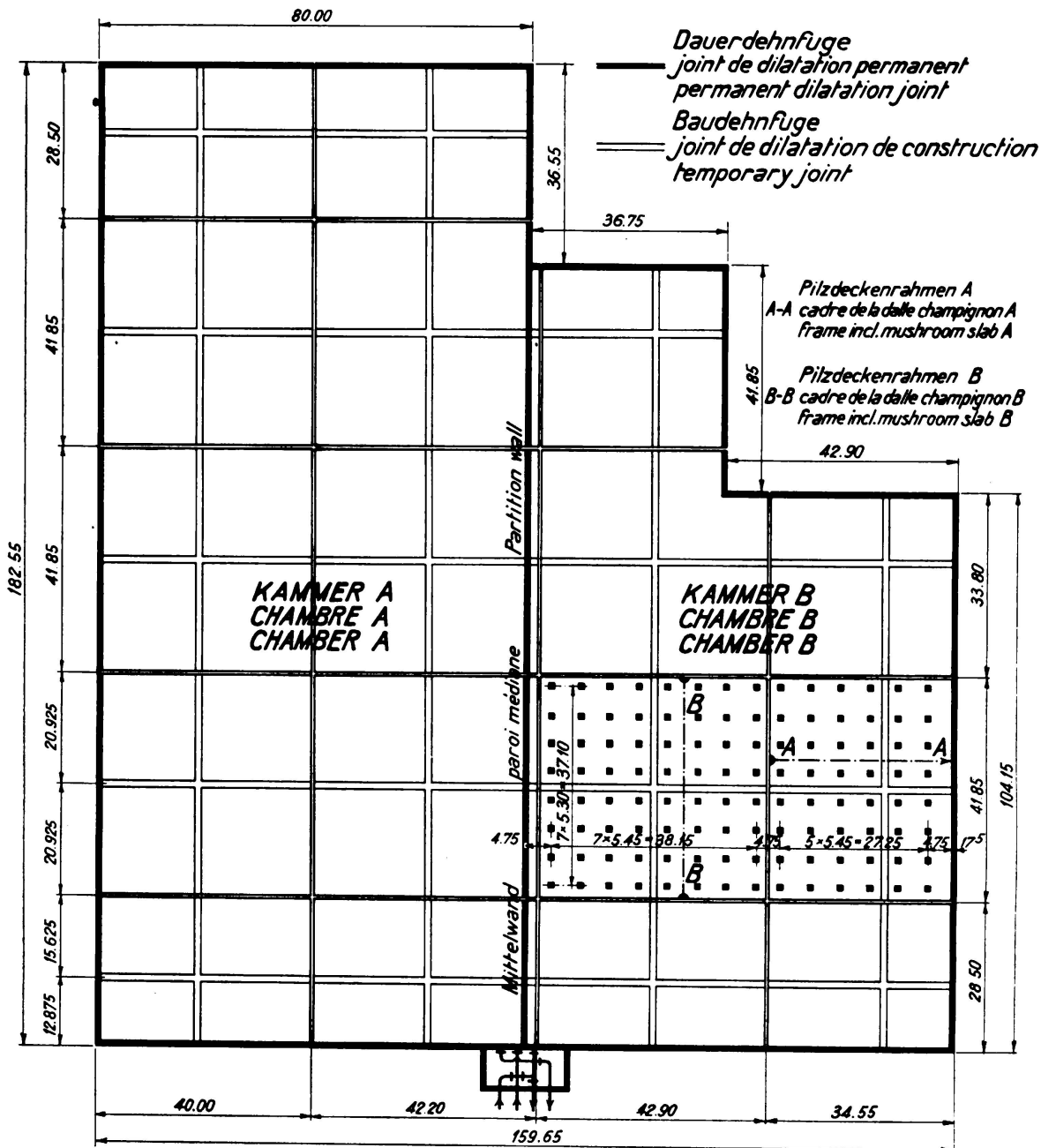


Fig. 6.

Water reservoir Zoo. Garden, Lainz. Plan.

The permissible compressive stress in concrete, computed for condition II (tension zone in concrete imagined to be entirely neglected), with n being

$\frac{E_e}{E_b} = 15$, amounts to

$$\sigma_{bd \text{ perm.}} = \frac{Wb \ 28}{3},$$

but, as a maximum

for the reservoir walls ($d = 35 \text{ cm} < 40 \text{ cm}$)	80 kg/cm ²
for the floor slab ($d = 45 \text{ cm} > 40 \text{ cm}$)	100 kg/cm ²
for the roof slab ($d = 25 \text{ cm} < 40 \text{ cm}$)	
for loading arrangement I	80 kg/cm ²
for loading arrangement II, $80 \cdot 1,20 =$	96 kg/cm ²
for the columns ($d = 45 \text{ cm} > 40 \text{ cm}$)	
for loading arrangement I	100 kg/cm ²
for loading arrangement II, $100 \cdot 1,20 =$	120 kg/cm ²

The permissible stress in steel, computed for condition II (tension zone in concrete imagined to be entirely neglected), with n being $= \frac{E_e}{E_b} = 15$, amounts to:

loading arrangement I, for steel St. 37,11	1200 kg/cm ²
for Isteg steel $1500 \cdot 1,10 =$	1800 kg/cm ²
loading arrangement II, for steel 37,11: $1200 \cdot 1,25 =$	1500 kg/cm ²
for Isteg steel $1500 \cdot 1,10 =$	2000 kg/cm ²

The following cases of load were taken into account for the calculation of stability of the external walls.

- 1) Reservoir empty, full earth pressure, roof slab with alternate bays loaded without earth cover and earth cover plus live load
- 2) Reservoir full, half earth pressure, roof slab, alternate bays without earth cover and earth cover plus live load.

For the calculation of the roof slab and columns, consideration is made loading condition I for the most unfavourable effect of

dead weight plus earth cover
 earth pressure
 water pressure
 live load (snow load)
 $\Delta t = \pm 10^{\circ} \text{ C}$

loading condition II for the most unfavourable effect of

dead weight plus half the weight of the earth cover
 earth pressure
 water pressure
 live load (half weight of earth cover plus snow load)
 $\Delta t = \pm 10^{\circ} \text{ C}$

With regard to the water flowing through the reservoir in a maximum period of 17 hours, an earth-deposit of 50 cm thickness is sufficient for the insulation of the reservoir.

A design of the structure without expansion joints as in the case of the Krapfenwaldl reservoir was not possible owing to the great dimensions of the reservoir (about 25 000 m² area).

The computation of the stresses in the above-mentioned approximate method (Fig. 5) gives for

$$\begin{aligned} h &= 6,85 \text{ m} & w &= 0,00001 \\ d &= 0,45 \text{ m} \\ \Delta t &= 10^0 \text{ C} & E &= 210\,000 \text{ kg/cm}^2 \\ L \text{ being} &= 182,55 \text{ m}, & \sigma_b &= \frac{3}{2h^2} \cdot d \cdot w \cdot \Delta t \cdot E \cdot L \\ &= \frac{3}{2 \cdot 685^2} \cdot 42 \cdot 0,00001 \cdot 10 \cdot 210\,000 \cdot 18\,255 = 52 \text{ kg/cm}^2, \end{aligned}$$

and, $L \text{ being} = 159,65 \text{ m} \dots \sigma_b = 45 \text{ kg/cm}^2$ respectively and shows that the arrangement of permanent expansion joints is absolutely necessary. However, the number of permanent expansion joints required could be considerably reduced by arranging at the same time construction joints similar to those in the building of the Krapfenwaldl reservoir, which included a reduction of building costs as well as in the future costs of maintenance.

The arrangement of construction joints and permanent expansion joints chosen can be seen in Fig. 6. The maximum distance between the latter is about 40 m, with a corresponding stress of concrete, according to computation by approximation for $\Delta t = 10^0 \text{ C}$, of

$$\sigma_b = 11 \text{ kg cm}^2.$$

In the construction joints, the individual parts of the building are absolutely separated as far as the concrete sub-base, and the steel reinforcements of the ceiling as well as the distribution bars in the wall pass into the joints at an adequate distance on either side, but they are not continuous, so that unfavourable pre-tensions in steel and concrete are avoided.

The type of construction joints in the wall and in the floor slab is shown in Fig. 7.

As in the case of the Krapfenwaldl reservoir, the construction joints were kept open until the earth deposit on the roof slab and at the side of the wall are completed, the structure meanwhile being carefully shored-up.

The construction joints should be kept open for at least 6 months in order to allow all injurious influence (shrinkage, unequal settling of the separate parts of the structure etc.) so that they may have their full effect before the closing of the joints takes place.

The concrete used for the joints should be of the same composition as that of the adjoining structure.

Semicircular work-shafts are provided at the construction joints of the external walls (Fig. 7). They are built in dry masonry and will also be accessible after completion of the structure so that there is always a possibility for control.

The type of permanent expansion joints together with the insulation provided can be seen in Fig. 8.

Rectagonal inspection shafts are arranged at the permanent expansion joints of the external walls and drainage provided underneath them (Fig. 8) in order to prevent moisture from penetrating into the soil. As the adjacent clay swells and finally softens entirely when in contact with water, the quality of soil had also to be considered when arranging the expansion joints.

By dividing the structure of the reservoir by means of permanent expansion joints, two principal kinds of frames will result:

Frame A, a 5 to 7-panel frame with 5 to 7 columns (42 cm square section), a solid wall (either external or internal wall) and a mushroom roof slab (Fig. 6).

Frame B, a 7-panel frame with 8 columns (42 cm square section) and a mushroom roof slab (Fig. 6).

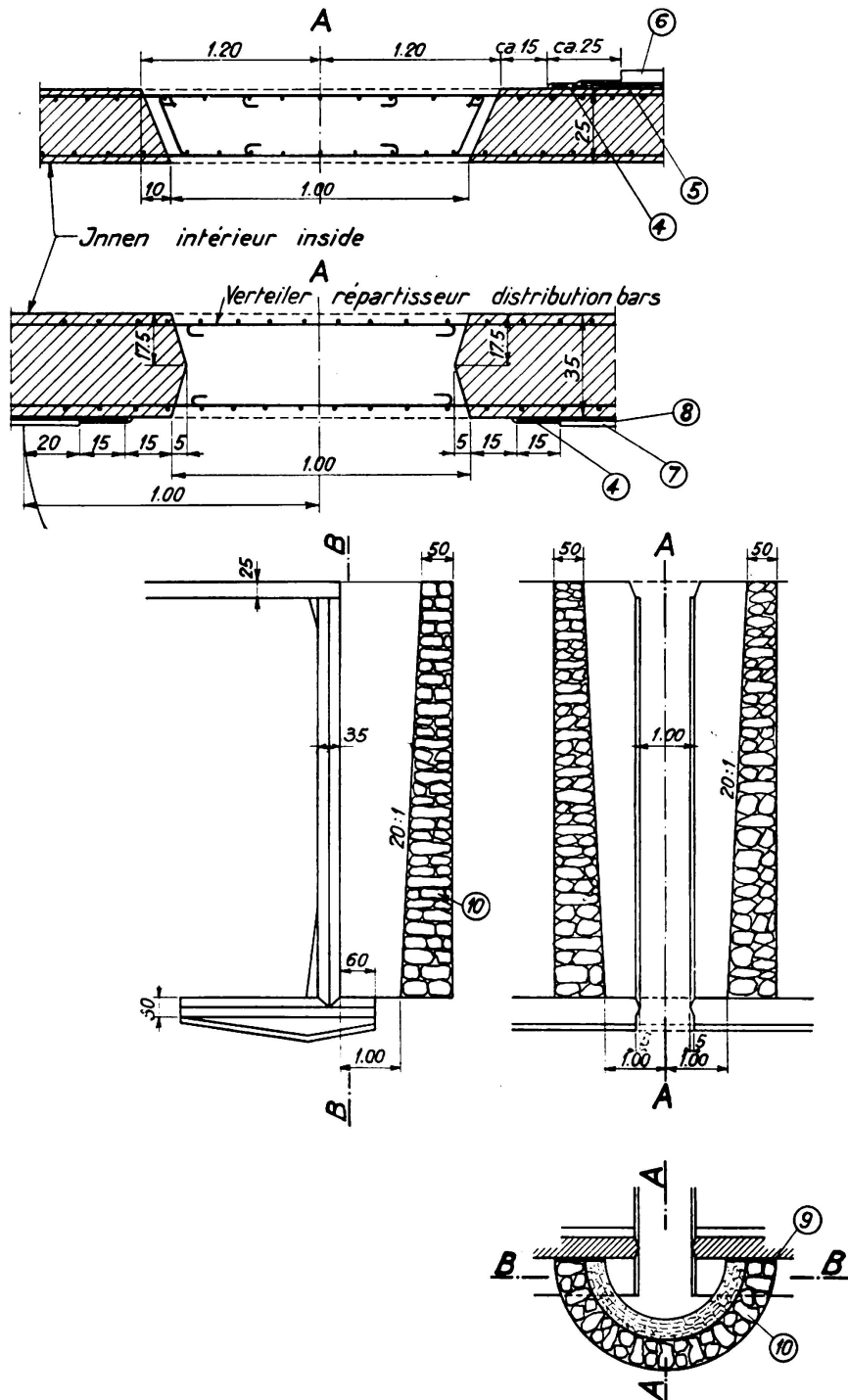


Fig. 7.

Water reservoir
Zoo. Garden, Lainz.

- 4) Tarred paper stuck on
- 5) Horizontal insulation (2 ply tarred paper)
- 6) Protective concrete 4 cm thick
- 7) Surface rendering
- 8) Vertical insulation
- 9) Two-ply tarred paper
- 10) Dry masonry.

Each of these frames was calculated by means of the fixed-point method as a mushroom slab-frame with columns rigidly fixed at the bottom. The width of a whole panel was introduced as the width of the frame under consideration.

The distribution of the calculated bending-moments over the panel and girder strips was made according to the Austrian Regulations on Reinforced Concrete.

The bending moments and stresses arising in the supporting frames A and B in consequence of variation in the temperature of $\Delta t = 10^{\circ} \text{C}$ are stated in

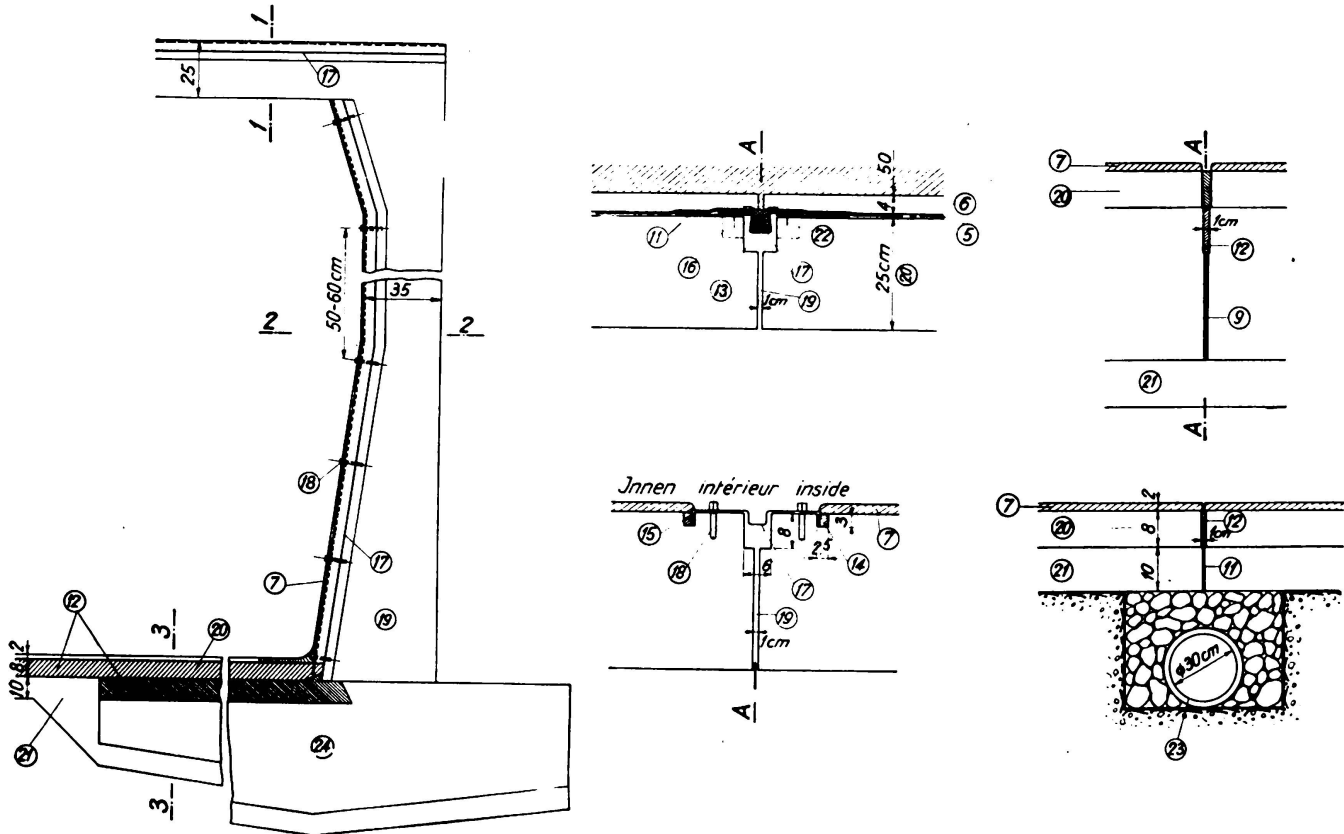


Fig. 8.

Water reservoir Zoo. Garden, Lainz.

- | | |
|--------------------------------|------------------------------------|
| 11) Tarred paper | 18) Copper screw |
| 12) Asphalt caulking | 19) Joint |
| 13) Asphaltic putty | 20) Reinforced concrete slab |
| 14) Grouted with cement mortar | 21) Base concrete |
| 15) Tarred hemp rope | 22) Nail soldered on top |
| 16) Wooden plug | 23) Clay drain pipe |
| 17) Copper sheet | 24) Joint with tarred paper layer. |

tables II and III as well as in Fig. 9, together with the fundamental assumptions for the calculations.

A comparison with regard to the sensitiveness of the structure of the Lainz Reservoir and that of the Krapfenwaldl Reservoir is given by the stresses caused by change of temperature $\Delta t = 10^{\circ} \text{C}$ at a certain distance from the wall, which is considered to be in a fixed position:

Table II.

Water Reservoir at Lainz.

Bending moments and stresses arising in the columns of frame A owing to a change in temperature of $\Delta t = 10^\circ \text{C}$. (computed acc. to condition II: concrete tension zone imagined to be entirely

eliminated and $n = \frac{E_s}{E_b} = 15$)

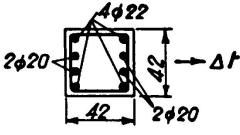
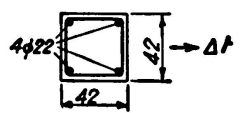
Distance from external wall latter assumed to be in fixed position		column 1	2	3	4	5	4	
		4,75	10,20	15,65	21,10	26,55	32,00 m	
Bending moment in the axis of the structure	M_o	0,440	0,937	1,450	1,912	2,561	2,518 tm	
Bending moment at the transition between mushroom and slab	M_R	0,365	0,776	1,210	1,595	2,130	2,080 tm	
Dimensions of cross section and column reinforcement at the transition between mushroom and slab							$d = 42 \text{ cm}$ $h = 39 \text{ cm}$ $x_{II} = 12,77 \text{ cm}$ $J_{II} = 192784 \text{ cm}^4$	
Compression stress in concrete due to M_R		2,4	5,0	8,0	10,5	14,1	13,8 kg/cm ²	
Tensile stress in steel due to M_R		75	157	248	326	435	426 kg/cm ²	

Table III.

Water Reservoir at Lainz.

Bending moments and stresses arising in the columns of frame B owing to a change in temperature of $\Delta t = 10^\circ \text{C}$ (computed acc. to condition II: concrete tension zone imagined to be

entirely eliminated and $n = \frac{E_s}{E_b} = 15$).

Distance from external wall latter assumed to be in fixed position		column 1	2	3	4	
		2,65	7,95	13,25	18,55 m	
Bending moment in the axis of the structure	M_o	0,252	0,717	1,296	1,470 tm	
Bending moment at the transition between mushroom and slab	M_R	0,210	0,595	1,080	1,215 tm	
Dimension of cross section and column reinforcement at the transition between mushroom and slab					$d = 42 \text{ cm}$ $h = 39 \text{ cm}$ $x_{II} = 10,62 \text{ cm}$ $J_{II} = 115356 \text{ cm}^4$	
Compression stress in concrete due to M_R		1,9	5,5	10,0	11,2 kg/cm ²	
Tensile stress in steel due to M_R		77	220	397	446 kg/cm ²	

Water Reservoir Krapfenwaldl (Fig. 4):

$$\begin{aligned} \text{for } \frac{L}{2} &= 15,60 \text{ m according to Table I} \\ \text{is } M_R &= 1,410 \text{ tm} \\ \sigma_b &= 13,0 \text{ kg/cm}^2 \text{ and} \\ \sigma_e &= 520 \text{ kg/cm}^2. \end{aligned}$$

Water Reservoir Lainz (Fig. 9, frame A):

$$\begin{aligned} \text{for } \frac{L}{2} &= 15,65 \text{ m according to Table II} \\ M_R &= 1,210 \text{ tm} \end{aligned}$$

the dimension of the columns and reinforcements being the same as at the Krapfenwaldl Reservoir, so that

$$\begin{aligned} \sigma_b &= 11,1 \text{ kg/cm}^2 \text{ and} \\ \sigma_e &= 446 \text{ kg/cm}^2 \end{aligned}$$

and with the reinforcement actually carried out, the dimension of the columns being the same,

$$\begin{aligned} \sigma_b &= 8 \text{ kg/cm}^2 \\ \sigma_e &= 248 \text{ kg/cm}^2. \end{aligned}$$

The possibility of a more slender construction, therefore, comprises the possibility of increasing the distance of the expansion joints, within certain limits. This influence, however, must not be overestimated without exact theoretical proof.

In the following, the influences beginning at point 1—6 of the above mentioned progress in improving the quality of concrete and reinforcement steel, and the exact theoretical knowledge with regard to the danger of cracking and the spacing of expansion joints will be discussed.

Besides the one advantage of increasing the stresses by modern design of concrete with an ensuing reduction of cross section dimensions, this method of making concrete includes two more:

- a) The danger of cracking is considerably reduced by the increased uniformity of the concrete, as weak spots (honeycombing and the like) are reduced to a minimum or do not occur at all.
- b) By the modern vibrating method, the quantity of cement and the amount of water can be kept essentially lower and, besides, the air content of the new concrete is reduced.

For the construction of the Krapfenwaldl reservoir, wet concrete of usual pit-broken stone and 350 kg cement per 1 m³ of finished concrete was mixed and placed according to standard practice.

In the case of the Lainz reservoir, the aggregate is dosed according to weights and in different mesh. The quantity of cement was fixed at 240—270 kg per 1 m³ of finished concrete, this stipulation being based on thorough previous tests and taking into account the mode of concreting by means of vibrating-apparatus. By making concrete batches of high uniformity and low cement content the influence of shrinkage stresses is very much reduced. Accordingly,

the expansion joints in structures of such type can be spaced wider apart or the safety against cracking is increased by retaining the hitherto customary distance between the joints.

There is a further possibility of development in the employment of steel of a high yield point, or of steel without a definite yield point, briefly termed high-grade steels.

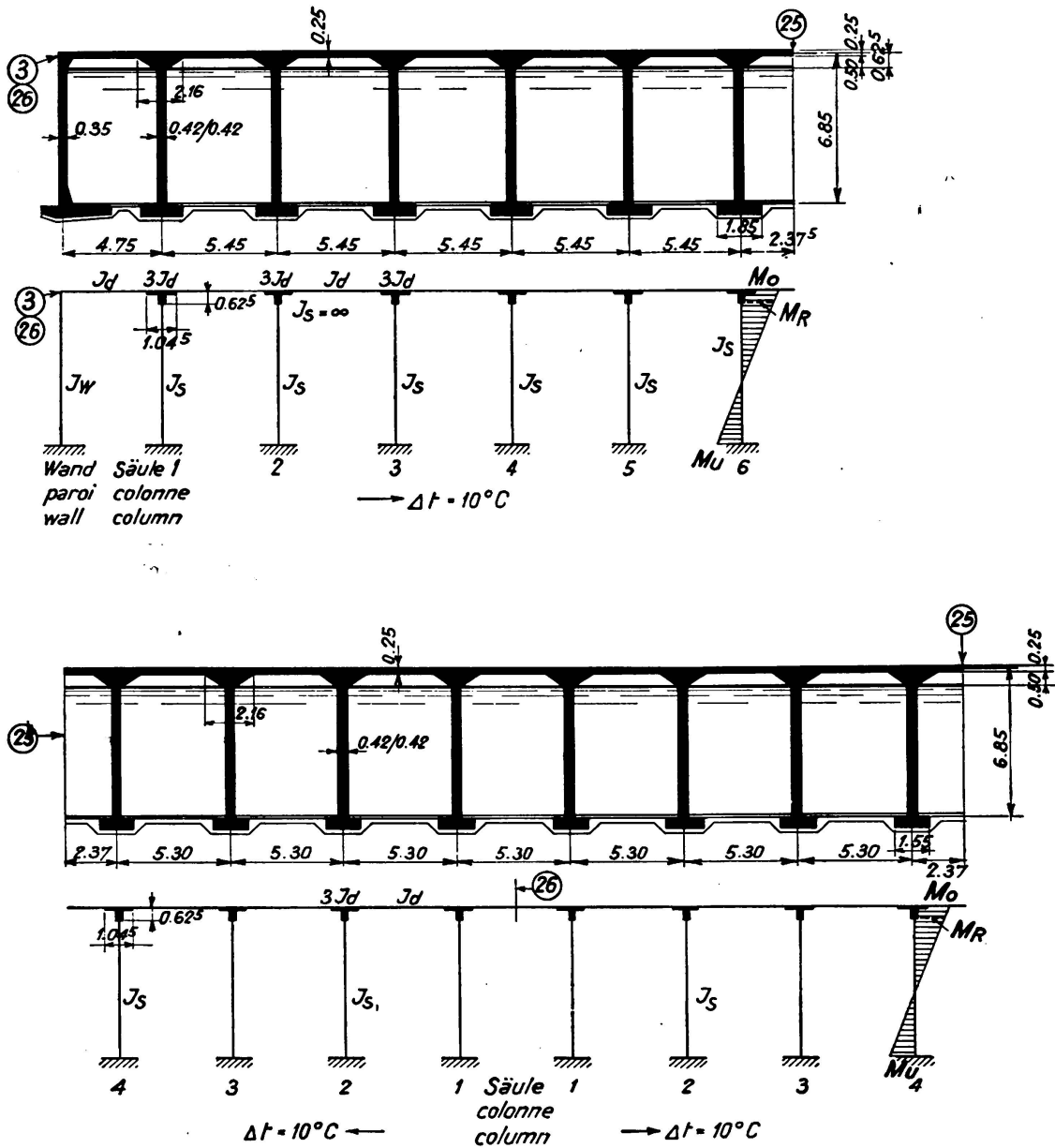


Fig. 9.

Water reservoir Zoo. Garden, Lainz.

25) Permanent joint

26) Restraint for temperature changes M_o , M_R , M_u .

Thus a concentration of increased expansion in steel and, in connection with this, the early formation of cracks and fissures are avoided.

The cracking depth was determined by a number of tests with beams of dif-

ferent steel reinforcements (carried out by Prof. Dr. Ing. *R. Saliger*, Vienna)¹ by Dr. Ing. *Jonas Kuodis* by means of thin liquid solutions of colour (4% nigrosine colour solution in alcohol absolutus) and the results described in his theses⁵.

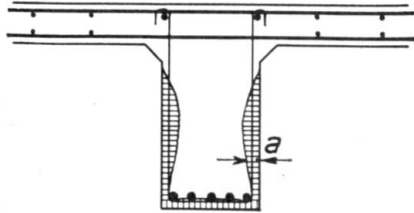


Fig. 10.

Depth of cracks a in reinforced concrete beams.

According to inferences, the crack arising at stationary load-test do not pan right through the whole beam section, but are distributed as in Fig. 10. Fig. 11 shows the result of the colouring-test on beam 54a by Prof. Dr. Ing. *R. Saliger* at crack 5. The colour was allowed to take effect till the colour-absorbing capacity of the crack hed ceased.



Fig. 11.

Practical determination of depth of crack for reinforced concrete beams by colouring,

New directions for the dimensioning of the structures as well as for their construction are indicated by taking into account the actual course of the cracks inside the cross section of reinforced concrete, which is still an object of more exact research work by detailed and thorough series of tests based on the present examination.

A further reduction of the danger of cracking can be achieved by increasing

⁵ not yet published.

the tensile strength and ductility of the concrete. All progress along this line will necessarily involve progress in construction and shape.

A building in which the arrangement of prearranged construction joints proved most successful during execution is the new large market hall at Ploesti (Fig. 12)⁶.

Besides the task of eliminating shrinkage stresses the construction joints served the special purpose of taking tensile forces off the roof slab, for the cupola slab with its transom girders, the roof slab beams running all the way round as well as roof slabs themselves would act as a tie for the arched supporting structures of the cupola if no special measures were taken.

The free movability of permanent expansion joints is the fundamental condition for their intended effect. This is best ensured if the supporting structure is divided by continuous joints into two independent parts.

The utmost care must be taken in the construction of movable bearings in supporting structures with suspension girders.

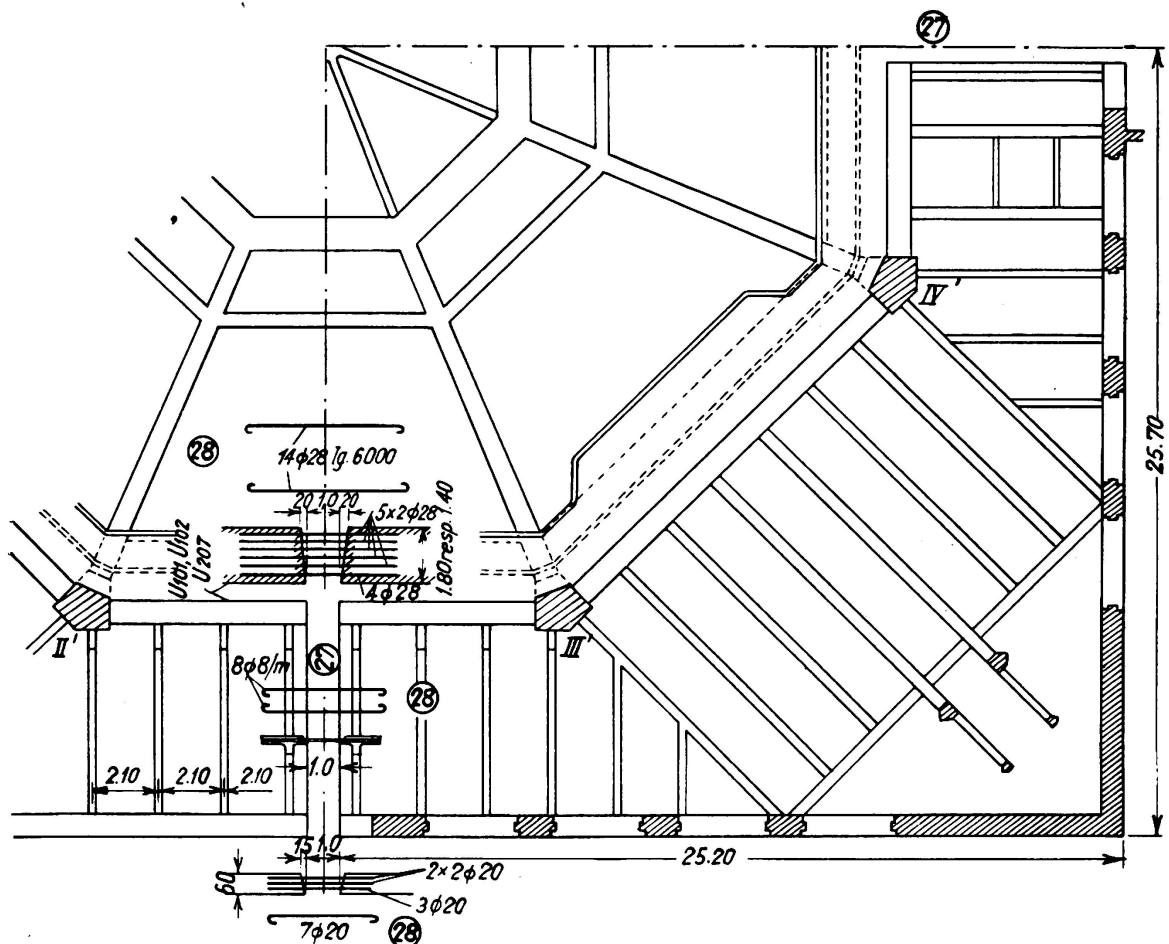


Fig. 12.

Hall Market in Ploesti

27) Joint 1 m wide 28) Additional bars.

⁶ The New Market Hall at Ploesti by Prof. Dr. Ing. R. Saliger and Dr. Ing. F. Baravalle, *Der Bauingenieur* 1933, issue 21/22.

Warning is here given for steel sliding-bearings both in structural engineering and bridge building. The sliding friction of steel against steel does not only amount to 44 % of the bearing pressure⁷ in the case of dry, even surfaces, but this value is nearly always considerably raised by rusted surfaces, thus making the efficient working of such bearings questionable. The consequence is cracking in the brackets or in the structure proper.

Wherever movable bearings are required, proper roller-bearings should be carried out and every provision made that the cleaning and possible renewal of the rollers can easily be effected by lifting the structure. At any rate, the friction of 0,2—0,6 % of the bearing pressure must be taken into account heretoo, and their reception in the different parts of the structure be properly taken care of.

The friction and yield of the soil at retaining and revetment walls, as well as at strip-foundations or slab foundations, exercise a great influence on the movability and, therefore, also on the spacing of expansion joints. By inserting sufficiently thick layers of sand below and beside the concrete masses a greater distance between expansion joints can be effected, but this type of construction is permissible only with such kinds of soil where a permanent existence of this layer of sand is ensured. Tests on the dependence of the distance between expansion joints upon the kind of soil and its practical influence on the latter would be highly important, especially for present-day large-scale road-building schemes.

With all theoretical and practical investigations, it is absolutely indispensable to determine the duration of shrinkage processes and variations in temperature.

Ing. *Karl Fischer* (Building Dept. of the Municipality of Vienna) determined the course of temperature in a concrete wall 1 m in thickness, immediately after concreting (table IV). 1 m³ of concrete contained 470 kg of quick-hardening Portland cement. This shows that the highest point of heat in concrete is already reached after a few hours by means of the process of setting and hardening, in the case of supporting structures dimensioned according to standard practice, and that the temperature of the concrete then gradually approaches the outside temperature. Similarly to the influence of the duration of the temperature course in new concrete, the duration of subsequent alterations of temperature in the supporting structure itself must be observed. The influences of slow changes of temperature and those of quick changes as well as their intensity, together with the frequency of their occurrence, are decisive for the selection of the distance between expansion joints.

It is only by an intelligent arrangements of joints — be their construction or permanent ones — and by taking into account all effective forces, that subsequent damages can be avoided. The knowledge gained in both theoretical and practical concrete research should show the way for the arrangement and construction of the joints. However, structures between the designed construction and permanent joints should be concreted without any interruption all at once, even if operation in several shifts be necessary for this purpose.

⁷ Zahlen für Jedermann, by Dr. *Hermann von Baravalle*, Frankh'sche Verlagsbuchhandlung, Stuttgart.

Table IV.
Temperatures in a 1 m Concrete Wall with 470 kg of Rapid-hardening Portland cement per 1 m³ of concrete.

Time of readings		Time of setting and hardening	Measuring Position 1	Measuring Position 2	Outside temperature
day	hour				
17th Oct. 1936	17 h'd	Stop of Concreting	—	—	
18th Oct.	9 h'el		+ 21° C	+ 26° C	+ 12° C
	12 h'el		+ 40°	+ 44°	
	15 h'el	1st day	+ 45°	+ 47°	
19th Oct.	9 h'el		+ 40°	+ 39°	+ 9°
	13.30 h'el		+ 37°	+ 38°	
	15 h'el	2nd day	+ 37°	+ 36°	
	22 h'el		+ 35°	+ 34°	
20th Oct.	14 h'el	3rd day	+ 28°	+ 28°	
21st Oct.	8 h'el		+ 18°	+ 18°	+ 8°
	14 h'el		+ 20°	+ 20°	
	19 h'el	4th day	+ 24°	+ 23°	
22nd Oct.	8 h'el	5th day	+ 15°	—	+ 4°
23rd Oct.	9 h'el	6th day	+ 15°	—	+ 5°
24th Oct.	10 h'el	7th day	+ 15°	—	+ 5°

Summary.

The juxtaposition of the Water Reservoir Krapfenwaldl built by the Municipality of Vienna in the years 1923—1926 and the largest covered Drinking Water Reservoir in the world now in course of construction at Lainz, the Game Preserves near Vienna, shows the influence of modern design in concrete on the division of the structure. By a purposeful mixing of high-quality concrete the sectional dimensions can be reduced and owing to the resulting degree of slenderness of the structural parts, the permanent expansion joints spaced wider apart with the same degree of safety. Furthermore, the uniformity of concrete as to its composition and strength reduces the danger of cracking in the supporting structures. By employing highgrade steels of high yield point, or of steels without any definite yield point, the concentration of more intense expansions in steel and, along with these, the early formation of wide cracks will be avoided.

The temporal course of warming up in consequence of the setting and hardening processes as well as that of the subsequent effects of outside temperature are of special importance for arrangement of expansion joints.

The best way of dividing the masses in modern structural engineering is the division carried out in building the above-mentioned large-scale water reservoirs and many other structures following a suggestion by Hofrat Professor Dr. Ing. R. Saliger:

- 1) Temporary construction joints, which are only kept open during building operations for at least several weeks and then concreted, thus no longer marring the uniform total effect of the structure.
- 2) Permanent expansion joints, lastingly dividing the individual component parts of the supporting structure and giving the latter the required working power.

The temporary construction joints protecting the supporting structure from the influences of the first and greatest shrinkage and the first and usually greatest settling, their provision may allow the permanent expansion joints to be spaced wider apart than otherwise, thus reducing cost of construction and subsequent maintenance work.

When dividing a supporting structure necessitating the movability of the individual members of the building by means of movable bearings, only roller bearings should be provided owing to the high friction value between the bearing plates (44 % of the bearing pressure in the case of dry, smooth steel surfaces). The effective frictional force must be taken into account as being 0,2—0,6 % of the bearing pressure when dimensioning the individual parts of the supporting structure.

The influence of soil friction and soil yield on the distance between joints in revetment and retaining walls as well as in foundation strips and slab foundations forms a problem in itself.

The consideration of the spacing of joints in various buildings, the computations made and practical ideas presented are to be the dues for type, arrangement and construction of expansion joints in modern concrete engineering.

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