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WOOD-STAVE PIPES

BY

STIG REGNELL

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Acknowledgements

This treatise is based on the thesis for the degree of »teknologie licentiat», i. e. Lic. Eng. submitted by the Author to the Royal Institute of Technology, Stockholm, in February, 1947.

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I. Preface

Wood pipes have probably been employed for conveying water from time immemorial, but they seem not to have been put to any important uses until the rise of large cities. The water supply system in London is the most renowned installation of this kind. The development of this system began on a small scale as early as in the 13th century, and at that time the pipes were made of lead. In the 17th century, pipes made of hollow trunks of trees began to come into use, and after that the water supply system gradually increased to a considerable size. For a period of time, one of the private water supply companies in London, the New River Company, operated wood pipe water mains exceeding 640 km in total length, see Fig. 1.

Fig. 1. An old water main in London. The construction of these mains was begun in the 17th century. The pipes were made of bored tree trunks. Several pipes were connected in parallel in order to increase the carrying capacity of the main. (Reproduced from a publication of the Metropolitan Water Board entitled »The Water Supply of London» (67).)

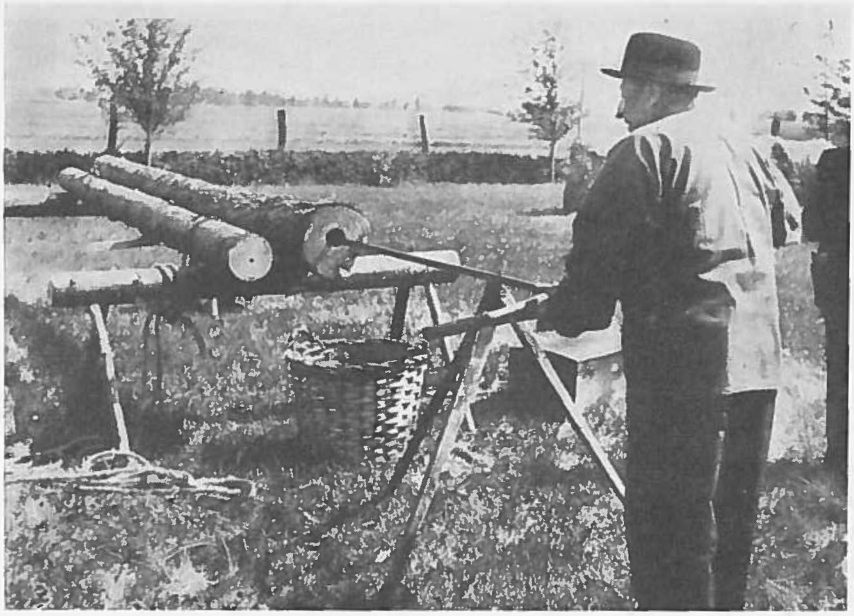

Fig. 2. Boring of pine trunk pipes for water mains in Sweden, Province of Västergötland, in 1945. The same method of manufacture was used as early as at the beginning of the 19th century, when wood pipes of this type were common in this part of the country. (This photograph has been published in the Västra Sveriges Lantmannatidning).

In Sweden, the most famous installation of this type was the water supply system in Gothenburg (30). A water main between Kallebäck and Gothenburg was built of wood in 1787, and was inaugurated by King Gustaf III. Towards the end of the 1860ies, the Gotenburg water supply system had been developed to an extent that it fed 15 public hydrants and 15 service mains connected to private estates. It is surprising that this old type of wood pipes should have continued to exist in our time. In Germany, where the first recorded use of bored trunks of trees as pipes dates back to the 13th century, the old methods were put in practice again during the Second World War (5). The reason was presumably a shortage of cast iron pipes. In Sweden, too, bored wood pipes are still manufactured, although, of course, on a small scale and frequently for special purposes, e. g. for conveying acids in the wood pulp industry. Fig. 2 shows that primitive tools are employed in the manufacture of these pipes.

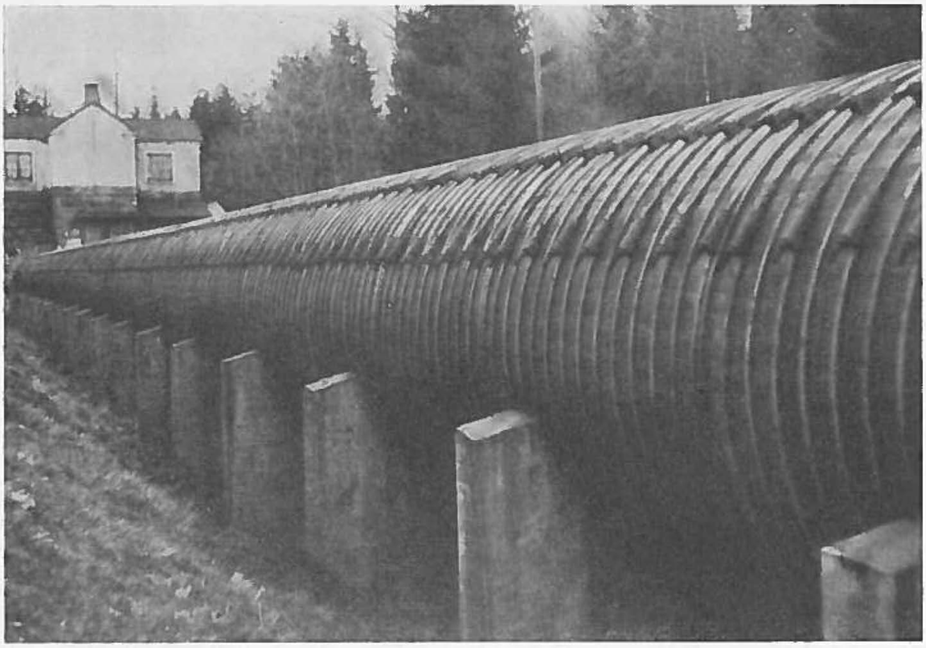

Fig. 3 b.

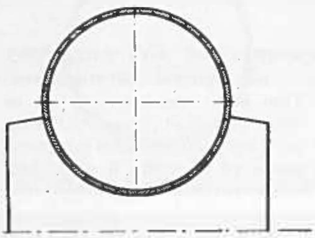

Fig. 3 a.

Fig. 3. Wood-stave pipe carried on concrete cradles. a. General arrangement. b. Pipe at Rydö. Diameter 2.5 m. Distance between supports 3.3 m.

Nowadays, owing to the requirements in respect of higher resistance to the internal water pressure, and in view of the need for greater carrying capacity, i. e. larger diameters, this old type of wood pipe has been almost completely superseded by wood-stave pipes[1]) which began to come into use in some North-American towns in the 1880ies[2]). As early as in 1910, wood-stave pipe systems were used in Denver,

[1]) A wood-stave pipe is made up of separate staves which are held together by steel bands so as to form a circular pipe.

[2]) It is reported that a wood-stave pipe has been constructed in Finland by the Tervakoski Company in 1860 (88).

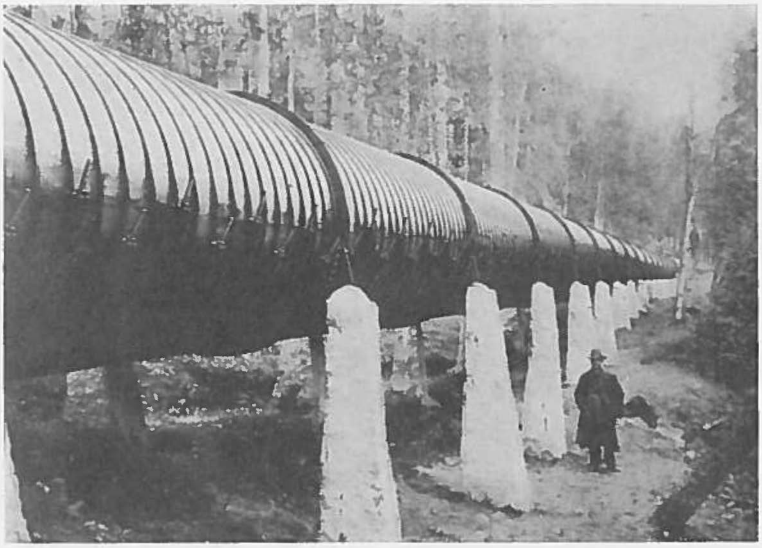

Fig. 4 b.

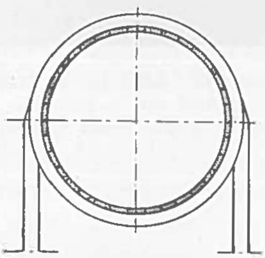

Fig. 4 a.

Fig. 4. Wood-stave pipe provided with ring stiffeners at the supports. a. General arrangement. b. Pipe at Lindesnäs. This pipe was constructed in 1921, and was at that time the first pipe with ring stiffeners. Diameter 1.7 m. Distance between supports 4.0 m. Ring stiffeners made of 40 × 80 × 6 mm steel angles at each supports. Supports made of natural stone.

Colo., and in Tacoma, Wash., totalling more than 160 km and 60 km respectively. Since that time, development has made rapid strides, and wood-stave pipes are now employed for various uses. In sanitary water engineering, use is mostly made of machine-banded pipes of relatively small diameter, whereas the wood-stave pipes used in water power engineering are armoured with special steel bands, and are made in diameters of up to 5 m.

Although wood-stave pipes have been built and generally used for so many years, their operating characteristics have long been unascertained in several important respects. Furthermore, in spite of the fact that numerous methods of design have been suggested, it has not been possible to draw up reliable rules for designing wood-

Fig. 5 b.

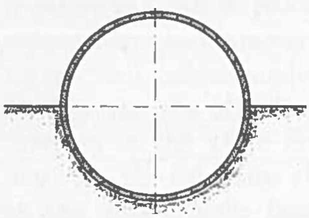

Fig. 5 a.

Fig. 5. Wood-stave pipe carried on a bed of broken stone. a. General arrangement. b. Two pipes at the Sikfors Hydro-Electric Station, which have been in operation 30 years. Diameter 3.2 m. Cf. Fig. 13.

stave pipes so as to take into account the moments and the forces due to swelling of the wood.

In order to collect experience regarding the operating characteristics of wood-stave pipes, the Swedish Water Power Association has circulated a questionnaire to most users of these pipes in Sweden in the autumn of 1941. The answers to this questionnaire have supplied a good deal of valuable information on decay and freezing of wood-stave pipes. The results of this investigation have recently been published (78).

The design of wood-stave pipes should take into account the mechanical strength, the operating conditions, and the economic factors. The strength calculations include the general design of the pipe, in which distinction is made between pipes carried on supports and pipes laid on beds (see Figs. 3 to 5), and the design of some individual components, such as bands, shoes, staves, etc. With respect to operating conditions, the design takes into consideration such factors as decay of wood, freezing, loss of head, etc. Finally, with respect to economic factors, the design comprises the determination of the most economic diameter and construction of the pipe.

All these factors are dealt with to some extent in this paper, but special attention is given to strength. In the last chapter, a description is given of the various types of wood-stave pipes used in Sweden at the present time.

II. Strength

A. Introduction

A wood-stave pipe shall be designed so as to afford adequate safety against failure on account of the loads and forces to which it is submitted.

The stresses due to the *internal pressure* above atmospheric are independent of the method of support of the pipe, whereas the stresses due to the *weights* of the pipe shell and the water vary according to whether the pipe is carried on supports or on a bed. The stresses due to the weights are of importance in comparison with the stresses caused by the internal pressure in those cases where the internal pressure is small in relation to the diameter of the pipe, and in those cases where the diameter of the pipe is large. Consequently, according to the well-known model law, if a given pipe were linearly enlarged in all parts in a given ratio, the stresses at homologous points would increase in the same ratio, i. e. the enlargement factor. Generally, however, the thickness of the pipe wall does not increase in proportion to the enlargement factor, and remains relatively small when the diameter of the pipe is large.

In normal cases, wood-stave pipes are carried on supports. This applies to 90 per cent of the wood-stave pipes used in Sweden. When the diameter of the pipe is very large, it is difficult to design the supports so that they should be satisfactory from an economic point of view, since the distance between the supports must be relatively small in order that the stresses and deformations should not become too great. For this reason, large pipes are often supported on beddings which are usually made of gravel or coarse broken stone reaching to half the height of the pipe. Since embedded pipes are supported continuously, the stress problem is reduced to a plane at right angles to the longitudinal axis of the pipe. In this case, the moment acting on the wall and the deformation (distorting the cross section of the pipe from a circle into an oval) are of importance in the design of the pipe.

Compressive stresses in the staves are caused by tightening of the bands and by swelling of the wood. In some cases, where the bands are weak and the thickness of the staves is great, the force due to swelling of the wood can become predominant in comparison with the other forces. The external forces due to the reactions at the supports must also be taken into account.

This chapter deals with the general design of pipes carried on supports and embedded pipes, and with the design of some of their components. The stresses due to swelling of the wood have been made the subject of a special investigation (see p. 47). Since the stresses in embedded pipes are most difficult to find by calculation, these stresses have been determined by theoretical calculations and by measurements made on four pipes which are among the largest used in Sweden.

B. Loads and Forces

A wood-stave pipe is acted upon by the following forces:

1. *Internal pressure* above atmospheric due to the liquid flowing in the pipe. This pressure is taken to be equal to the pressure of the liquid at the crown of the pipe (if the surface of the liquid is on a level with the crown of the pipe, i. e. if there is no head over the pipe, the internal pressure exerted on the pipe is zero).
2. *External pressure* due to reactions at the supports and to the load caused by the earth covering the pipe.
3. *Forces due to weights.*
   a. Weight of the liquid.
   b. Weight of the pipe shell.
4. *Compressive stresses in the staves.*

The *internal pressure* causes only normal forces in the wall of the pipe, and these forces are taken by the bands.

The stresses due to the *external pressure* are dependent on the method of support. When the pipes are covered with earth, these stresses are dependent on the properties of the covering material and on the method of bedding.

The *weights* cause stresses in the pipe, which are difficult to calculate. The effect of the weight of the pipe shell is relatively insignificant in comparison with the effect of the weight of the liquid. For embedded pipes, according to FEDERHOFER (21), the ratio of the moments in the wall of the pipe due to the weight of the wall and the weight of the liquid is $\frac{2\,G}{\gamma\, r}$. For an ordinary wood-stave pipe, this ratio does not exceed 1 to 10, and the weight of the pipe may therefore generally be disregarded. The stresses due to the weight of the liquid vary according to whether the pipe is carried on supports or embedded, and are of great importance in the design. A special type of stresses due to the weight of the liquid is produced when the pipe is not completely filled.

The *compressive stresses* in the staves are caused by tightening of the bands and by swelling of the wood.

In an embedded pipe, no moments occur in the vertical plane passing through the longitudinal axis of the pipe, and it is therefore only the moments in the plane at right angles to this axis that are to be considered.

C. General

1. Normal Forces

The normal forces acting in the transverse plane are taken by the steel bands. The internal pressure gives rise to the normal force $+\gamma\, Hr$. The compressive stress in the staves produces the normal force $+\, pd$. These normal forces are independent of whether the pipe is carried on supports or embedded, and are equal at all points on the circumference of the pipe.

The weights of the liquid and the pipe shell cause normal forces which are dependent on whether the pipe is carried on supports or embedded, and which are not equal at different points on the circumference of the pipe. The effect of the weight of the pipe shell is insignificant, and can generally be disregarded.

2. Moments

The moments which occur in a wood-stave pipe act either in the vertical plane passing through the longitudinal axis of the tube or in a plane at right angles to this axis.

If the pipe is carried on supports, both these moments are of importance. The moments acting in the transverse plane can be eliminated by using an appropriate method of support, cradles or ring stiffeners. In that case, the supports must be designed for this purpose.

D. Wood-Stave Pipes Carried on Supports

1. Introduction

The design of pipes carried on supports had been dealt with by several authors on the assumption that the wall of the pipe is homogeneous and that its deformations are elastic. As early as in 1880, Bach (2) deduced some formulae for calculating the stresses in a thick-walled pipe acting in a plane at right angles to the longitudinal axis of the pipe. These formulae seem to be mostly applicable to concrete pipes. The stresses occurring in thin-walled pipes under various conditions of loading have been dealt with by Forchheimer (23) and by Fröhlich (28). Subsequently, Karlsson (46, 47) evolved a method of carrying the pipes by means of ring stiffeners fitted at each support. This method was based on the theories advanced by Fröhlich. According to Hökerberg (44), the moments acting on these ring stiffeners can be reduced if the stiffeners are supported at two points which are located slightly outside the pipe shell instead of being situated right at the shell. The calculation of stresses in the vertical plane passing through the longitudinal axis of the pipe seems to have been of little interest. However, Thoma (93) has deduced a relatively simply formula for computing these stresses on the assumption that the pipe is homogeneous. When the internal water pressure in the pipe becomes so low that the water level sinks below the crown of the pipe, we are confronted with an intricate stress problem. The solution of this problem has been dealt with from a mathematical point of view by Westerberg (96), Samsioe (81), Anzelius and Faxén (1).

Strictly speaking, most of the methods of design mentioned in the introduction to this section are applicable only to pipes having homogeneous walls, e. g. iron or concrete pipes. A wood-stave pipe cannot be regarded as homogeneous, and hence we revert to the old question of whether the separate staves can transmit shearing forces by friction. As long as the staves do not move in relation to one another, the pipe shell can be considered to be homogeneous in the calculation of stresses. On this assumption, the load-carrying capacity of the pipe is determined by the maximum shearing stress. The wall of the pipe is generally so thin that it is assumed not to be able to withstand any moments, and the stresses can be calculated from THOMA's formula (93). This method of stress calculation will be referred to as the »membrane method» in what follows.

When the force due to friction between the staves is exceeded, the weight of the pipe must be carried jointly by the separate staves.[1]) On some occasions, the ability of the staves to take care of shearing stresses seems to be momentarily eliminated. This is proved by the fact that water is sometimes ejected from the joints between the staves when the pipe is subjected to water hammer. On such occasions, the pipe acts as a regular pile of staves. The method of stress calculation described in what follows, which is based on this assumption, will be termed the »stave pile method».

2. Membrane Method

If the pipe is continuously supported at several points, and is provided with cradles or ring stiffeners at the supports, we obtain the following normal forces and stresses by means of the membrane method.

1. *Weight of Water.*

$$N_\varphi = \gamma r^2 (1 + \cos\varphi) \tag{1}$$

$$\sigma_z = \frac{\gamma}{2d}\left(\frac{L^2}{12} - z^2\right)\cos\varphi \tag{2}$$

$$\tau = \gamma \frac{rz}{d}\sin\varphi \tag{3}$$

2. *Weight of Pipe Shell.*

$$N_\varphi = G r \cos\varphi \tag{4}$$

$$\sigma_z = \frac{G}{rd}\left(\frac{L^2}{12} - z^2\right)\cos\varphi \tag{5}$$

$$\tau = \frac{2Gz}{d}\sin\varphi \tag{6}$$

In the above formulae, σ_z denotes the tensile stress in the wood staves in a direction parallel to the longitudinal axis of the pipe.

[1]) See SAMSIOE (80).

The most heavily stressed cross section is located at a support where the moment reaches a maximum. This value is approximate since Poisson's ratio has been put equal to zero.

As the shearing stress cannot exceed that value which corresponds to the stress due to friction between two adjacent staves, this stress determines the choice of the thickness of staves and the distance between the supports.

It follows from Eqs. (3) and (6) that the maximum shearing stress occurs at the supports, and is given by

$$\tau_m = \frac{\gamma\, r\, L}{2\, d}\left(1 + \frac{2\, G}{\gamma\, r}\right) \tag{7}$$

If f is the coefficient of friction between two staves moving in parallel to the wood fibres, and if p is the compressive stress in the surface of contact between staves, then we have $\tau_m = fp$. If we neglect the weight of the pipe shell expressed by the term $\frac{2\, G}{\gamma\, r}$, we obtain the maximum distance between the supports

$$L_m = \frac{2\, d\, f}{\gamma\, r}\, p \tag{8}$$

At a low estimate, the coefficient of friction may be taken to be 1/4, and the maximum distance between the supports is

$$L_m = \frac{d\, p}{2\, \gamma\, r} \tag{9}$$

The compressive stress in the surface of contact between the staves can be caused by the force due to tightening of the bands or by the force due to swelling of the wood, or by the simultaneous action of both these forces. As will be shown on p. 47, the force due to swelling of the wood can be about 10 kg per cm². However, stress measurements indicate that the swelling force in an old wood-stave pipe is very small. The maximum distances between the supports given in Table 1 and Fig. 6 for various values of the diameter of pipe and the thickness of staves have been calculated from the above formula on the assumption that the swelling force is 10 kg per cm².

TABLE 1. *Maximum Distance between Supports, in m for Pipes Varying in Diameter and in Thickness of Staves. Membrane Method.*

The maximum distance has been calculated on the assumption that it is determined by the maximum shearing stress. No safety factor is included in the values of the maximum distance given in this table.

Diameter of Pipe m	Maximum Distance between Pipe Supports, in m — Thickness of Staves, in mm: 50	75	100
1	5.0	7.5	—
2	2.5	3.8	5.0
3	1.7	2.5	3.3
4	—	1.9	2.5
5	—	—	2.0

3. Stave Pile Method

The stave pile method is based on the following assumptions. The pipe is supposed to be continuously supported at several points and to be provided with cradles or ring stiffeners at the supports. Furthermore, it is assumed that the staves can transmit only normal forces, but no frictional forces. Finally, it is assumed that the pipe is distorted in a vertical plane only, and that the deflections of all staves in the same vertical cross section are equal. In other words, this means that the cross section of the pipe remains unchanged during deformation. Then the maximum stress in the separate staves will occur in those staves which stand on their edge, i. e. in the staves forming the sides of the pipe. According to the detailed calculations given in Appendix A, we obtain

1. *Weight of Water.*

$$N_\varphi = \gamma r^2 \left(1 + \frac{b^2 + 2\,d^2}{b^2 + d^2} \cos\varphi\right) \tag{10}$$

$$\sigma = \frac{\gamma\, r\, L^2}{2\, b\, d} \cdot \frac{1}{1 + \dfrac{d^2}{b^2}} \tag{11}$$

$$\tau_s = \frac{3\,\gamma\, r\, L}{4\, d} \cdot \frac{1}{1 + \dfrac{d^2}{b^2}} \tag{12}$$

2. *Weight of Pipe Shell.*

$$N_\varphi = G\, r \frac{b^2 - d^2}{b^2 + d^2} \cos\varphi \qquad (13)$$

$$\sigma = \frac{G\, L^2}{b\, d} \cdot \frac{1}{1 + \dfrac{d^2}{b^2}} \qquad (14)$$

$$\tau_s = \frac{3\, G\, L}{2\, d} \cdot \frac{1}{1 + \dfrac{d^2}{b^2}} \qquad (15)$$

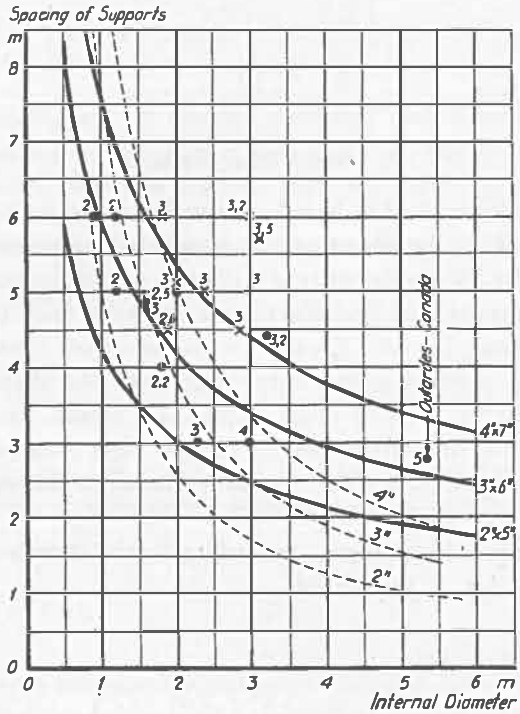

Fig. 6. Variation in spacing of supports with internal diameter, thickness of staves, and width of staves for wood-stave pipes carried on cradles or in ring stiffeners. The full-line curves refer to the distance between the supports calculated from the formula suggested in this paper (stave pile method). This formula results in a factor of safety of about 3. The dash-line curves refer to the maximum distance between the supports computed by means of the membrane method. No factor of safety is included.

Furthermore, the diagram shows empirical values taken from Table 3. The values represented by dots refer to those pipes which have proved to be fully satisfactory in operation. The values represented by crosses refer to those pipes which had to be reinforced, or which have been heavily deformed. The figures at each dot or cross indicate the thickness of staves in inches after planing.

If we neglect the effect of the weight of the pipe shell, we can deduce from Eq. (11) a relatively simple expression for the maximum allowable distance between the supports

$$L_s = \sqrt{\frac{4\,\sigma\,b\,d}{\gamma\,D}\left(1 + \frac{d^2}{b^2}\right)} \tag{16}$$

For a pipe filled with water, if we insert the maximum allowable tensile stress in bending of 60 kg per cm², we obtain

$$L_c = 100\sqrt{\frac{24\,b_c\,d_c}{D_c}\left(1 + \frac{d_c^2}{b_c^2}\right)} \tag{17}$$

where all lengths are expressed in centimetres.

The weight of the pipe shell can approximately be taken into account by assuming that D_c denotes the external diameter of the pipe.

It is seen from this formula that the maximum distance between the supports is also dependent on the width of staves. On the assumption that the width of staves is normal,[1]) this formula gives the distances between the supports represented in Table 2 and Fig. 6.

TABLE 2. *Maximum Distance between Supports, in m for Pipes Varying in Diameter and in Thickness of Staves. Stave Pile Method.*

This table is applicable to the widths of staves normally used in practice on the assumption that the maximum distance between the supports is determined by the tensile stress in bending in the staves. This stress has been taken to be 600 tons per m². This value corresponds in round numbers to a safety factor of 3 referred to the ultimate strength.

Diameter of Pipe	Maximum Distance between Pipe Supports, in m		
	Thickness of Staves, in mm		
m	50	75	100
1	4.3	5.9	—
2	3.0	4.2	5.4
3	2.5	3.4	4.4
4	—	3.0	3.8
5	—	—	3.4

[1]) Normal relations between the thickness and the width of staves are e. g. 2 in. × 5 in., 3 in. × 6 in., and 4 in. × 7 in.

TABLE 3. *Some Swedish Wood-Stave Pipes Carried on Cradles, Which Have Been Distorted into Oval Shape or Had to Be Reinforced by Means of Intermediate Supports, and Pipes Having Unusually Large Distance between Supports. Comparison of Calculated and Actual Distances between Supports.*

Pipe No.	Locality	Thickness of Staves (mm)	Diameter of Pipe (m)	Actual Distance between Supports (m)	Distance between Supports Calculated by Means of		Remarks
					Stave Pile Method (m)	Membrane Method (m)	
1	Knon	75	3.2	4.4	3.3	2.4	—
2	Gideåbruk	89	3.0	5.7	4.0	2.8	Provided with intermediate supports. Pipe demolished in 1947.
3	Nain	75	3.0	5.0	3.4	2.5	—
4	Ludvika	100	3.0	3.0	4.4	3.4	—
5	Semla	80	3.0	6.0	3.5	2.7	Pipe distorted into oval shape.
6	Billsta II	75	2.8	4.5	3.5	2.7	Provided with intermediate supports.
7	Fågelfors	75	2.4	5.0	3.8	3.1	Provided with intermediate supports.
8	Loforsen	75	2.3	3.0	3.9	3.3	—
9	Flögfors	75	2.0	5.1	4.2	3.8	Pipe distorted into oval shape.
10	Gammelkroppa	62	2.0	5.0	3.6	3.1	—
11	Hamrånge	56	1.8	4.0	3.4	3.1	—
12	Billsta I	65	1.8	4.5	4.0	3.6	Provided with intermediate supports.
13	Svarteström	75	1.8	6.0	4.4	4.2	Probably provided with intermediate supports.
14	Gopa	75	1.6	4.8	4.7	4.7	Provided with additional steel bands.
15	Strömmen	50	1.2	6.0	3.9	4.2	—
16	Kvarnedsforsen	50	1.2	5.0	3.9	4.2	—
17	Gisslefors	75	0.9	8.0	6.3	8,3	—
18	Närsidan	50	0.9	6.0	4.5	5,6	—

4. Summary

On condition that friction between the staves is not eliminated, and that the compressive stress in the surface of contact between the staves is sufficiently high, the use of the membrane method may be considered justifiable. As has been pointed out in the above, the frictional force can be eliminated on account of water hammer. Some investigations, which will be described further on, show that the compressive stress (the stress due to swelling of the wood) diminishes in course of time, and can decrease to 2 or 3 kg per cm^2 in 30 years old pipes. Consequently, the membrane method is not generally applicable, and the stave pile method is therefore to be recommended for the design of pipes carried on supports.

Both these methods of calculation have been applied to some wood-stave pipes existing in Sweden. Table 3 and Fig. 6 give the characteristics of some Swedish pipes carried on supports whose spacing is unusually large in comparison with the distances in general use. It is seen that some of these pipes have been distorted in some way or other. In particular, the deformations have occurred in pipes exceeding 1.8 m in diameter, where the distance between the supports is 15 to 70 per cent greater than that calculated by means of the stave pile method and 25 to 100 per cent greater than that computed by means of the membrane method. For pipes less than 1.8 m. in diameter, the actual distances between the supports did not exceed those calculated by means of these two methods by more than 50 and 40 per cent respectively. No drawbacks due to this large spacing of the supports have been reported.[1]) It seems that the stave pile method can be applied to pipes of any ordinary size, but when the diameter of the pipe is large, the safety factor obtained by means of this method may perhaps be inadequate with respect to deformations. On the other hand, the membrane method, when applied to large pipes, gives a distance between the supports which is smaller than the spacing of supports generally used for such pipes. The Norwegian rules for wood-stave pipes give a formula for computing the distance between the supports, which is stated to be completely empirical. The distance between the supports calculated from this

[1]) This circumstance may possibly indicate that the use of the membrane method is to a certain extent justifiable when the diameter of the pipe is small.

formula[1]) is slightly smaller than that obtained by means of the stave pile method, but the structure of these two formulae is fundamentally similar. Therefore, Norwegian experience also seems to confirm the correctness of the stave pile method.

Since the greatest stresses occur in the staves standing on the narrow edge, it is necessary to make sure that these staves, in particular, are of high quality and that they are as long as possible. The joints between the staves on the sides of the pipe should be located at those points where they are subjected to the smallest moment, that is to say at a distance of about $0.2\,L$ from the supports.

Incorrect choice of the distance between the supports can give rise to considerable difficulties, as will be shown in the following description of that section of the *Gideåbruk wood-stave pipe* which is carried on supports. This pipe, which was built in 1913, is 3.0 m in diameter, and the staves are 3½ in. in thickness. In the autumn of 1944, it was found that this pipe had undergone deformation, and was leaking profusely. In the beginning, the distance between the supports had been 5.7 m (see pipe No. 2 in Table 3). This distance was soon found to be too large, and the pipe was reinforced with horizontal iron stays extending through the whole of the pipe. However, these stays had to be removed a short time afterwards because the flow of water produced large vibrations in the stays. After that, vertical wooden poles were placed on either side of the pipe (see Fig. 7), and these poles were held together by means of steel braces at the top and at the bottom, outside the pipe shell. Nevertheless, this device proved to be of little help, and intermediate supports consisting of wooden trestles or concrete blocks had to be installed between the old supports, so that the distance between the supports was 2.85 m. According to the above-mentioned formula, the spacing of the supports should not exceed 4.0 m. Subsequently (in 1947), the pipe section carried on supports was pulled down, and a new pipe, 3.5 m in diameter, was installed on a bed made of broken stone.

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[1]) According to the Norwegian formula, the distance between the supports shall not exceed $L_{max} = C_1 \sqrt{\frac{t\,(t+b)}{D}}$, where t and b are expressed in inches and D in metres. The constant C_1 is 1.0 for straight pipes and 0.8 for curved pipes.

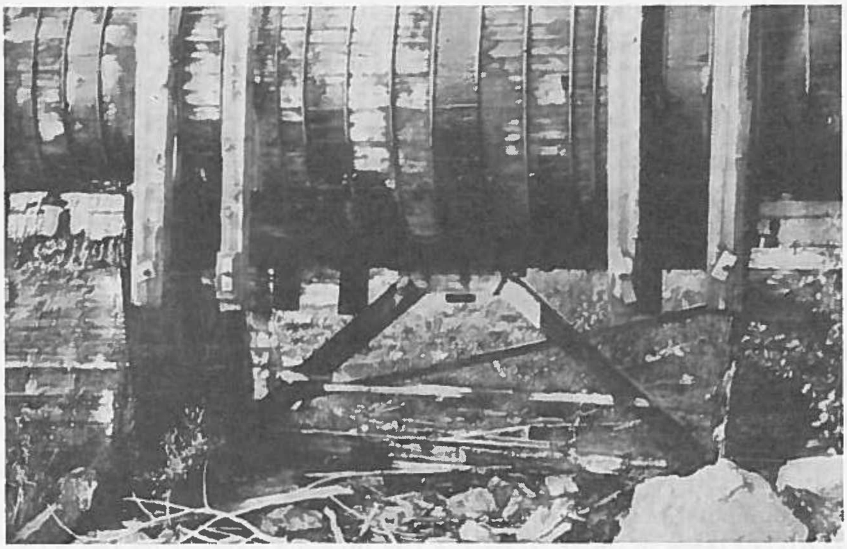

Fig. 7. Wood-stave pipe at the Gideåbruk Hydro-Electric Station. As the distance between the supports was found to be too great, the pipe had to be reinforced, first by means of struts between the supports, and then by means of wooden poles placed in pairs on either side of the pipe and held together by means of steel braces at the top and at the bottom. The distance between the supports was originally 5.7 m. The diameter of the pipe is 3 m. In 1947, the section of the pipe carried on supports was pulled down, and was replaced by a new pipe laid on a bed made of broken stone.

5. Moments Acting on Ring Stiffeners

At the present time, the supports used for wood-stave pipes consist either of cradles or of ring stiffeners. Cradles are generally designed so as to ensure adequate strength, particularly if they are made of concrete, and the pipe retains its original circular shape. It is very important to provide a sufficiently large surface of contact between the cradle and the pipe, so that the reactions at the supports are distributed over a large area, otherwise the wall of the pipe is liable to be caved in and distorted. For pipes from 0.5 to 3 m in diameter, the Norwegian rules (71) recommend that the arc of contact between the pipe and the support should not be less than 42 to 47 per cent of the circumference.

The use of ring stiffeners for supporting wood-stave pipes has been suggested by Karlsson. For the first time, this method of support was employed for the wood-stave pipe at Lindesnäs (see

Fig. 4 b). According to KARLSSON (46, 47), the design of the ring stiffeners is based on the moment

$$M_1 = 0.015\, PR \tag{18}$$

Later, HÖKERBERG (44) has shown that, if the points of support are moved outward a distance of 0.04 R, the moment is reduced to

$$M_2 = 0.010\, PR \tag{19}$$

These moments have been calculated on the assumption that the pipe acts as a homogeneous shell. The stave pile method gives slightly different values of the moments. The latter moments are probably smaller, and the use of the formulae given in the above affords additional safety.

In general, however, the design of the pipe is not determined by the moments because the width of the supporting ring must be relatively large in order to reduce the pressure exerted on the wood-staves to an allowable value.

6. Conclusions Regarding Wood-Stave Pipes Carried on Supports

It follows from the above that the steel bands for wood-stave pipes should be designed, so as to withstand the maximum normal force acting at the bottom of the pipe ($\varphi = 0$)

$$N_o = \gamma\, H\, r + p\, d + \left(2 + \frac{d^2}{b^2 + d^2}\right)\gamma\, r^2 + \frac{b^2 - d^2}{b^2 + d^2}\, G\, r \tag{20}$$

The distance between the supports should not exceed the value given below

$$L_s = \sqrt{\frac{4\, \sigma\, b\, d}{\gamma\, D}\left(1 + \frac{d^2}{b^2}\right)} \tag{16}$$

Eq. (16) corresponds to a safety factor of about 3 referred to the ultimate tensile strength in bending of the most heavily stressed stave.

If the pipe is carried on cradles, the surface of contact between the pipe and the cradle should be sufficiently large. If the pipe is carried in ring stiffeners, these rings should be designed so as to withstand the moments given by Eqs. (18) and (19).

E. Embedded Wood-Stave Pipes

1. Introduction

Large wood-stave pipes carried on supports are often subjected to considerable stresses. In order to reduce the stresses, such pipes are laid on continuous beds, usually made of stone, which reach to half the height of the pipe. Since embedded pipes are supported continuously, the moments and the normal forces can only occur in a plane at right angles to the longitudinal axis of the pipe. If the diameter of the pipe is small and the water pressure is high, the design of an embedded wood-stave pipe is relatively simple since, practically speaking, the wall of the pipe is submitted to normal forces alone. On the other hand, if the diameter of the pipe is large and the water pressure is low, the design presents far more intricate problems. The fundamental design formulae have been deduced by Boussinesq and Forchheimer (24).

2. Forchheimer's Method of Design

The differential equation of a ring deduced by Boussinesq is

$$\eta + \frac{d^2 \eta}{d \varphi^2} = \frac{r^2 M}{E I} \tag{21}$$

Taking this equation as a starting-point, Forchheimer has calculated the forces and the moments acting on the ring, for various

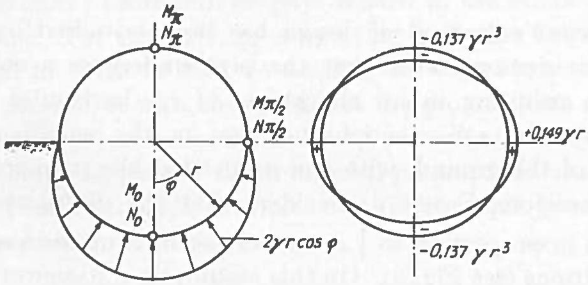

Fig. 8. Soil reaction and moment distribution in an embedded pipe determined by Forchheimer on the assumption that the pipe is filled with water, but is not subjected to any internal pressure.

methods of support, on the assumption that the pipe is filled with water, but is not subjected to any internal pressure above atmospheric. The weight of the pipe shell has been neglected. The most interesting case is the pipe embedded in earth to half its height (see Fig. 8). In this case, FORCHHEIMER has supposed that the soil reaction is directed at right angles to the surface of the pipe, and varies according to a $\cos \varphi$ law from a maximum value right under the pipe to zero at the surface of the ground. On this assumption, the external forces are symmetrical with respect to both the vertical axis and the horizontal axis passing through the centre of the pipe, and the mathematical treatment of this problem is relatively simple. Using the notations given in Fig. 8, we obtain the moment at the point φ,

$$M_\varphi = \frac{-\gamma r^3}{2}\left(\frac{4}{\pi} - \varphi \sin \varphi - \cos \varphi\right)^{1)},$$

and for the characteristic points

$$M_\pi = -0.137\,\gamma r^3 \qquad (22)$$

$$M_{\pi/2} = +0.149\,\gamma r^3 \qquad (23)$$

$$M_o = -0.137\,\gamma r^3 \qquad (24)$$

$$N_\pi = 0.500\,\gamma r^2 \qquad (25)$$

$$N_{\pi/2} = 0.215\,\gamma r^2 \qquad (26)$$

$$N_o = 0.500\,\gamma r^2 \qquad (27)$$

3. SAMSIOE's Method of Design

FORCHHEIMER's method of design has been criticised by SAMSIOE (79) who has demonstrated that the pipe undergoes a considerable deformation resulting in an elongation of its horizontal diameter. Consequently, the soil reaction increases in the neighbourhood of the surface of the ground, with the result that the moments become smaller. Therefore, SAMSIOE considers that the distribution of the soil reaction in proportion to $\sqrt{\cos \varphi}$ corresponds more closely to the actual conditions (see Fig. 9). On this assumption, SAMSIOE calculates the moments and the normal forces as follows.

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1) The moment is reckoned as positive if it tends to reduce the radius of curvature.

$$M_{\pi} = -0.103\,\gamma\,r^3 \tag{28}$$
$$M_{\pi/2} = +0.103\,\gamma\,r^3 \tag{29}$$
$$N_{\pi} = 0.417\,\gamma\,r^2 \tag{30}$$
$$N_{\pi/2} = 0.215\,\gamma\cdot r^2 \tag{31}$$

The action of the external forces produces a greater or smaller deformation of the pipe, and the moments are therefore reduced. The deformation of the pipe is expressed by the coefficient of distortion $\varepsilon = \frac{D_h - D_v}{D}$. For the value of $\varepsilon = 0.12$, which corresponds e. g. to a deflection of 18 cm at the crown of a pipe 3 m in diameter, the following moments and normal forces are obtained according to SAMSIOE[1])

$$M_{\pi} = +0.080\,\gamma\,r^3 \tag{32}$$
$$M_{\pi/2} = -0.085\,\gamma\,r^3 \tag{33}$$
$$N_{\pi} = 0.370\,\gamma\,r^2 \tag{34}$$
$$N_{\pi/2} = 0.215\,\gamma\,r^2 \tag{35}$$

4. FEDERHOFER's Method of Design

It follows from the above that the deformation of the pipe gives rise to a considerable reduction of the moments. The deformation of the pipe has also been taken into account by FEDERHOFER (21), who has used FORCHHEIMER's equations as a starting-point, but has arrived at a result which is slightly different from that obtained by earlier investigators.

Furthermore, FEDERHOFER pays regard to the effect of the weight of the pipe. For example, he shows that in a pipe 1.6 m in diameter embedded in earth to half its height, the edge stress in the wall of the pipe on a level with the surface of the ground (this example deals with a steel pipe) increases from 394 kg per cm^2 at an internal pressure of 20 m to 827 kg per cm^2 at an internal pressure of 0 m. The application of FORCHHEIMER's formula to this example gives a stress of 352 kg per cm^2.

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[1]) On account of the assumption referred to in the above, the forces acting on the pipe are *not symmetrical* with respect to the horizontal axis passing through the centre of the pipe, contrary to the supposition made in FORCHHEIMER's method of design.

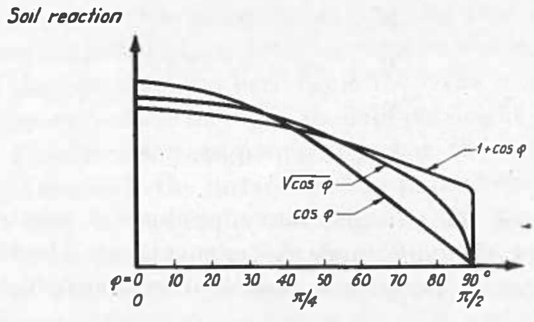

Fig. 9. Distribution of soil reaction at right angles to wall of pipe (φ is the angle at the centre of the pipe).

a. Distribution assumed to be proportional to $\cos\varphi$ (FORCHHEIMER).

b. Distribution assumed to be proportional to $\sqrt{\cos\varphi}$ (SAMSIOE).

c. Distribution assumed to be proportional to $1 + \cos\varphi$ (REGNELL).

5. Method of Design Advanced by the Author

It may be questioned whether the actual soil reaction close to the surface of the ground is not greater than the value computed on the assumption that the soil reaction is proportional to $\cos\varphi$ or $\sqrt{\cos\varphi}$. A relatively greater soil reaction close to the surface of the ground is obtained when the soil reaction is taken to be proportional to $1 + \cos\varphi$. The distribution of soil reaction in accordance with these three assumptions is shown in Fig. 9. If we disregard the weight of the pipe shell, the last-mentioned assumption gives the following moments, normal forces, and shearing forces for a circular pipe

$$M_\pi = -0.080\,\gamma\,r^3 \tag{36}$$

$$M_{\pi/2} = +0.065\,\gamma\,r^3 \tag{37}$$

$$M_o = -0.040\,\gamma\,r^3 \tag{38}$$

$$M_{\max} = +0.079\,\gamma\,r^3 \quad \text{(this point is located at a distance of } 12^\circ \text{ above the horizontal diameter)} \tag{39}$$

$$N_\pi = 0.360\,\gamma\,r^2 \tag{40}$$

$$N_{\pi/2} = 0.215\,\gamma\,r^2 \tag{41}$$

$$N_o = 0.320\,\gamma\,r^2 \tag{42}$$

$$V_{\pi/2} = 0.140\,\gamma\,r^2 \tag{43}$$

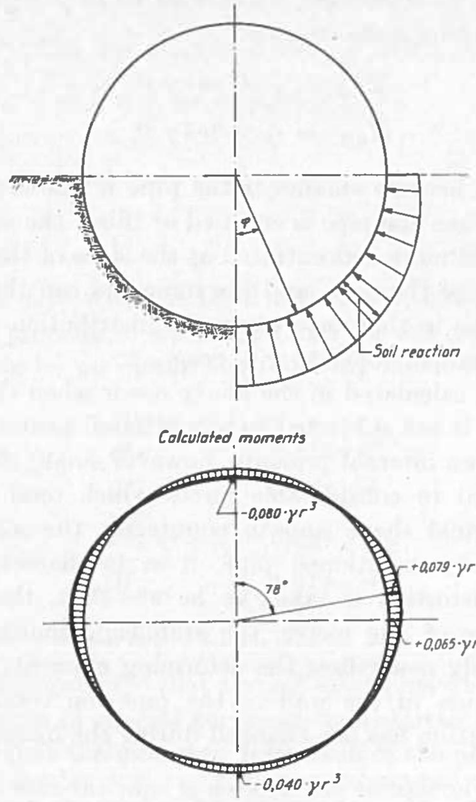

Fig. 10. Moment distribution in an embedded wood-stave pipe determined on the assumption that the soil reaction is proportional to $1 + \cos\varphi$. The pipe is supposed to be completely filled with water, but not to be subjected to any internal pressure.

In this case, the load is not symmetrical with respect to both the vertical axis and the horizontal axis passing through the centre of the pipe. Therefore, the maximum moment occurs slightly above the surface of the ground, and not on a level with this surface, see Fig. 10. The moments at the upper edge and at the surface of the ground are 58 and 44 per cent respectively of the values computed by means of FORCHHEIMER's method, or 78 and 63 per cent respectively of those calculated by means of SAMSIOE's method. Of course, the moments are smaller in this case too, if the pipe is distorted into an oval shape. If we take into account the deformation of the pipe, and if the coefficient of distortion is assumed to be $\varepsilon = 0.12$, we obtain the following approximate moments

$$M_{\pi} = -0.058\,\gamma\,r^3 \tag{44}$$

$$M_{\pi/2} = +0.052\,\gamma\,r^3 \tag{45}$$

The moments become smaller if the pipe is allowed to undergo a deformation. When the pipe is emptied or filled, the soil reaction can happen to be still more concentrated at the sides of the pipe owing to the deformation of the pipe, and the moments can therefore become even smaller than in that case where the distribution of soil reaction is taken to be proportional to $1 + \cos\varphi$.

The moments calculated in the above occur when the pipe is filled with water, but is not subjected to any internal pressure. If the pipe is submitted to an internal pressure, however small, then a deformed pipe is subjected to considerable forces which tend to restore the pipe to its original shape and to counteract the moments. If we consider the above-mentioned pipe, 3 m in diameter, and if the coefficient of distortion is taken to be $\varepsilon = 0.12$, then, even at an internal pressure of one metre, the stabilising moment is so great that it completely neutralises the deforming moment, that is to say, no moment occurs in the wall of the pipe, on condition that the amount of distortion has not changed during the increase in pressure.

6. Summary of Methods of Design for Embedded Wood-Stave Pipes

It is of course difficult to determine the actual distribution of soil reaction. The pressure distribution is dependent on the magnitude of the internal pressure in the pipe. When the internal pressure is high, FORCHHEIMER's assumption that the distribution of soil reaction is proportional to $\cos\varphi$ is probably most closely in agreement with reality. On the other hand, when the pipe is filled to the crown, it is probable that pressure distribution is approximately proportional to $1 + \cos\varphi$. FORCHHEIMER's values of the normal forces should therefore be used in the design of pipes subjected to internal pressure.

a. *Normal Forces.*

For a circular wood-stave pipe embedded in stone to half its height, we obtain the following normal forces which must be taken by the bands

$$N_{\pi} \quad = H r + p d + 0.50\, \gamma r^2 \qquad (46)$$

$$N_{\pi/2} = H r + p d + 0.215\, \gamma r^2 - 0.5\, \pi r G \qquad (47)$$

$$N_o \quad = H r + p d + 0.50\, \gamma r^2 \qquad (48)$$

The stress in the band is $\sigma_B = N \dfrac{l}{F}$.

b. *Moments.*

For a circular pipe which is filled with water, but is not subjected to any internal pressure, if we assume that the soil reaction is proportional to $1 + \cos \varphi$, we obtain the following moments in the wall of the pipe

$$M_{\pi} \quad = - 0.080\, \gamma r^3 \qquad (49)$$

$$M_{\pi/2} = + 0.065\, \gamma r^3 \qquad (50)$$

$$M_o \quad = - 0.040\, \gamma r^3 \qquad (51)$$

$$M_{\max} = + 0.079\, \gamma r^3 \qquad (52)$$

c. *Design of Wood-Stave Pipes for Operation under Normal Conditions.*

It has been demonstrated that a wood-stave pipe which is embedded to half its height is in general permanently distorted into oval shape, with the result that the moments in the wall of the pipe computed on the assumption that the pipe is circular are completely neutralised by the moments which are caused by the effect of the normal force due to deformation, even if the internal pressure is very slight. Therefore, in designing a pipe for normal conditions of operation, it is generally not necessary to take into account any moments whatever. The steel bands should be designed so as to be able to withstand the normal forces given by Eqs. (46) to (48).

d. *Design of Wood-Stave Pipes for Operation under Emptying and Filling Conditions.*

If the diameter of the pipe is relatively small in comparison with the thickness of the staves, the moments which occur when the pipe is being emptied and filled do not affect the design in view of the stiffness of the walls and the smallness of these moments.

[1]) The last term is an approximation of the influence of the shell weight. It is of no importance when designing the pipe. It has been included in calculating the tensions of the tensions of the tested pipes.

When the diameter of the pipe becomes greater, the moments increase as the third power of the diameter, whereas the stiffness of the walls changes very little. For this reason, the moments are of greater importance when the diameter of the pipe is large. A pipe which is submitted to a relatively low internal pressure in normal operation is usually so distorted that the moments in the wall are small while the pipe is being emptied or filled. A pipe which is submitted to a relatively high internal pressure in normal operation is closer to the circular shape, and is therefore subjected to greater moments when emptied or filled. According to the measurements on four large embedded pipes ($D = 3.0$ to 3.4 m) which are described in what follows, the normal force acting on the bands decreases continuously as the water pressure becomes smaller. Both at high and at low pressures, the observed values of the normal force acting on the bands are in good agreement with the values calculated from Eqs. (46) to (48) on the assumption that the moments are neglected. This shows that the normal force in the bands due to the moments in the wall is of no importance in the design of the pipe. Of course, the deformation of the pipe causes a high edge stress in the bands. However, this stress does not affect the design, as may be inferred from the following discussion. Since the moments occur only occasionally and at a low internal pressure, it ought to be possible to allow a high stress, on condition that adequate safety is afforded against failure.

e. *Discussion.*

The moments in the wall of the pipe shall be sustained by the wood-staves in combination with the steel bands. A certain initial reaction moment is produced by the force due to swelling of the wood and by the tensile forces in the bands due to this force. A moment which exceeds this reaction moment causes a higher average tensile stress in the bands and a corresponding compressive stress in the wood staves and gives rise to bending of the bands and the wood wall of the pipe. A calculation made on the assumption that the bending of the bands is perfectly elastic shows that considerable stresses are set up in the bands, and Samsioe has demonstrated in an example that, for a given pipe of large diameter, these stresses exceed the yield point stress of ordinary steel. On account of bending of the bands, the pipe will undergo deformation with the result that a condition of equilibrium will be reached, in which the initial moment due to the water pressure is neutralised by the reaction moment which is produced during the deformation of the pipe by the effect of the normal force and by the moment due to the combined action of the bands and the staves. Since the pipe is emptied and filled on comparatively rare occasions, the allowable stresses on such occasions may be higher than under normal conditions. Even if the edge stresses in the bands may approach to the yield point stress when the water level is at the crown of the pipe, this fact in itself seems to be of secondary importance. The reason is that the safety providing against failure should be referred to the maximum average tensile stress in the bands, and not to the maximum edge stress. The edge stress can reach high values, but this stress results in bending of the bands and in deformation of the pipe. As the deformation is restricted on account of the reaction moments due to the normal force, the edge stress is also limited. Since a pipe is emptied and filled at considerable intervals, the bands are not liable to fatigue. Moreover, the stress in the staves may probably exceed the maximum swelling stress or the yield point stress obtained under a static load for a short time, without causing any notable permanent deformations.

It is obvious that unduly large deformations of the wall of the pipe cannot be allowed for other reasons. When the crown of the pipe is deflected, the curvature of the pipe at the crown becomes increasingly flatter, and if the pipe is large, the wall of the pipe is exposed to the risk of buckling since the staves at the crown may tumble down. (See p. 54). This risk is particularly great if the water in the pipe is liable to surge when the pipe is emptied and filled.

F. Stress Measurements

1. Method of Testing

As has been shown in the above, several formulae are available for the design of wood-stave pipes. In order to determine the stresses in the bands, measurements have been made on four of the largest embedded wood-stave pipes existing in Sweden, viz., the pipes at the hydro-electric stations Sikfors on the river Piteälv, Harreselsfors on the river Umeälv, Gideåbruk on the river Gideälv, and Äggfors on the river Indalsälven. All these pipes have been subjected to a more or less detailed examination according as the method of testing used for these measurements proved to be applicable. In connection with these tests, a questionnaire has been circulated in order to collect information on operating experience, which is dealt with in Chapter III »Operating Conditions».

Since the stress in the steel bands is a product of the strain and the modulus of elasticity, the general principle of the measurements consisted in determining the strain by observations made on the pipes and in determining the modulus of elasticity of the respective steel bands by means of laboratory tests. For the determination of the modulus of elasticity, the pipe owners have kindly supplied specimens of steel bands taken from the same lots which had been used in the construction of the pipes. For the Äggfors pipe, no steel bands were left over, and the modulus of elasticity had therefore to be estimated.

The strain measurements were made by means of Huggenberger tensometers which were attached to several bands at various points on the circumference of the pipe (see Figs. 11 and 12). The water pressure was measured by means of a mercury manometer and by

Fig. 11. View of test set-up for strain measurements made on the wood-stave pipe at the Sikfors Hydro-Electric Station. The tensometers were attached to the bands at several points on the circumference of the pipe. The water pressure was measured by means of a mercury manometer which communicated with the water in the pipe through a rubber hose. At low pressures, the water level in the pipe was indicated by a glass tube gauge.

Fig. 12. Strain measurements on the wood-stave pipe at the Äggfors Hydro-Electric Station. Tensometers attached on the upper side of the pipe.

direct observation of the water level in a glass tube. After taking the tensometer readings, the shoes of the bands were loosened until the bands were entirely relieved from tension, and readings were taken again. Moreover, several readings were taken while loosening the shoes, in order to check the measurements. This precaution proved to be justifiable since the greatest error in the measurements was probably made while loosening the shoes. The reason was that the positions of the tensometers were easily disturbed when the band was lightly struck, e. g. with a wrench. In order to prevent the bands from twisting, they were held up by means of pipe tongs. Furthermore in some test or tests, the tensometer deflections were observed on all pipes at full pressure and while the pipe was empty.

The stresses in the steel bands were calculated from the tensometer deflections and the moduli of elasticity. The results are graphically represented in polar diagrams in Appendices B to E. These diagrams also show the stresses computed from Eqs. (46) to (48), in which the compressive stress in the staves has not been taken into account.

The tests made at Sikfors, Harrselsfors, Gideåbruk, and Äggfors are described in the following sections.

2. Tests Made at Sikfors

The Sikfors Hydro-Electric Station is situated on the river Piteälv, and utilises a head of 14.5 to 16.6 m at a normal discharge of 50 m^3 per sec. This plant was erected in 1912 and reconstructed in 1924. At the present time, it is equipped with two identical Francis generating units having a turbine rating of 2 500 kW each. From an intake located on the left bank of the river, the water flows through two wood-stave pipes, 470 m in length, to a surge tower which is situated quite close to the power station. The head over the crown of the pipes varies in operation from about 5 m at the intake to about 9 m at the surge tower. The water is taken from a pool which is covered with ice in the winter. Normally, the velocity of the water is 3.1 m per sec, and the discharge is 25 m^3 per sec per pipe. Originally, the pipes were circular, 3.2 m in inside diameter, and were embedded in broken stone reaching to half their height. In the tests made in September 1944, the horizontal diameter was found to be 3.36 m in the upper section of the left pipe (head of 4.87 m over the crown of the pipe) and 3.31 m in the lower section of this pipe (head of 8.12 m). Fig. 13 shows one of the pipes in process of construction in 1912. A view of the pipes in their present condition is shown in Fig. 5 b. The walls are made of continuous 3 in. wood staves which are stated not to be provided with tongue-and-grove joints— The staves are held together by bands made of steel rods 31 mm in dia-

Fig. 13. One of the two wood-stave pipes at the Sikfors Hydro-Electric Station in process of construction in 1913. The diameter of the pipe is 3.2 m. Cf. Fig. 5.

meter. The bands are spaced 60 cm in the upper section of the pipe and 35 cm in the lower section. Each band is tightened by means of an eccentric shoe made of open hearth steel. The shoes are fitted at the crown of the pipe, and are slightly displaced in relation to one another.

The first stress test (test No. 1) was made on a band in the upper section of the left pipe. Huggenberger tensometers were attached to both sides of the band at intervals between the surface of the ground and the crown of the pipe. The water pressure was measured by means of a mercury manometer. After taking the tensometer readings, the shoe was loosened by means of a wrench, while holding the band by means of pipe tongs. When the band was almost completely loosened, the pipe began to leak profusely, and the test had to be interrupted for fear that the pipe should fail. Tensometer readings were taken again. The modulus of elasticity was obtained from a tension test made on a band which was taken from the same lot as those used in the construction of the pipe. The difference between the stresses in the band under normal operating conditions and after loosening the band was calculated from the modulus of elasticity and the tensometer readings. The stress was also computed from Eqs. (46) to (48) without taking into account any no-load stresses,[1]) e. g. the stress caused by swelling of the wood. As may be seen from Appendix B, the agreement between the observed and the calculated stresses is fairly close. For the cross section of the pipe at Sikfors, a no-load stress of 1 kg per cm^2 corresponds to a stress of 58 kg per cm^2 in the band. Since it was not certain that the band was completely relieved from stresses after loosening, this test cannot give any conclusive evidence of the occurrence of swelling stresses. On the contrary, it is to be noted that the observed stresses at two points are slightly lower than the calculated values. Furthermore, in view of the slow movements of the tensometers during the last stage of loosening, the stress in the band may be assumed to have decreased to zero. The estimated no-load stress in the staves can scarcely be higher than 1 kg per cm.^{2}

The tests Nos. 2 and 3 were made on a band in the lower sections of the left pipe while it was being emptied and filled. The tenso-

[1]) The term »no-load stress» is used in what follows to designate that compressive stress which occurs in the staves when the pipe does not carry any load, i. e. when it is empty. The no-load stress may be due either to swelling of the wood or to tightening of the bands.

Fig. 14. The large wood-stave pipe at the Harrselsfors Hydro-Electric Station, 3.0 m in diameter. In view of the operating experience with the two older, smaller pipes (visible on the right), this pipe has been completely enclosed by plank walls in order to protect it from freezing.

meters were fitted around the pipe at approximately the same intervals as in the test No. 1. Readings were taken at an internal pressure of 9.9 m, and then the pipe was slowly emptied, while observing the water pressure and the tensometer deflections, until the water level has sunk 35 cm below the crown of the pipe (this distance is reckoned from the inside of the staves at the crown). At this low pressure, the tensometer readings varied but slightly as the pressure was still further reduced, and therefore it was not deemed necessary to empty the pipe still more, in order to obtain the values corresponding to the empty pipe. The tensometer were restored to zero, and the pressure was increased again, while observing the tensometer deflections.

As has already been pointed out, the stresses in all tests were calculated without taking into account any moments or any no-load stresses, e. g. the stress due to swelling of the wood. It is seen from Appendix B that the observed and the calculated stresses are in fairly close agreement in all tests. Hence it follows that the no-load stress was negligible.

3. Tests Made at Harrselsfors

The Harrselsfors Hydro-Electric Station, which is owned by the Vännäs Power Co., is situated on the river Umeälv, and utilises a head of 14.8 m at a normal discharge of 37 m^3 per sec. This station, which was built in 1916 to 1917, and was extended in 1923 to 1924 and in 1942 to 1943, is equipped with two Francis generating units having a turbine rating of 770 kW each, a Francis unit rated at 1 100 kW, and a Francis unit rated at 1 840 kW. The three former units are connected to two wood-stave pipes, 2.0 m in diameter and 280 cm in length. The 1 840 kW unit, which was installed in 1942 to 1943, is connected to a wood-stave pipe, 3.0 m in diameter and 302 m in length. The water flows from a concrete intake on the right bank of the river (the dam does not run across the whole width of the river) through these wood-stave pipes to the surge towers (see fig. 14). The older part of the plant has not always operated satisfactorily. For instance, one of the wood-stave pipes failed in 1944 (cf. p. 66). The stress measurements were made on the new wood-stave pipe. The characteristics of this pipe are given in what follows.

The pipe was built in 1942. It has an inside diameter of 3.0 m, and is embedded in broken stone reaching to about half its height. The wall of the pipe is made of continuous 3 in. staves which are stated not to be provided with tongue-and-groove joints. The pipe is equipped with bands made of steel rods, 22 mm in diameter, spaced 15 to 25 cm. Each band is tightened by means of two centric shoes which are fitted right above the surface of the ground on each side of the pipe. The head over the crown of the pipe varies from 0.5 to 7.0 m. Under normal conditions, the velocity of the water is 2.2 m per sec, and the discharge is 15.5 m^3 per sec.

Since it was in general very easy to loosen the shoes, nine tests were made on this pipe, and tensometer readings were taken before and after loosening the bands (tests Nos. 1 to 9). In the tests Nos. 6 to 7, the tensometer deflections were also observed before loosening the shoes both at full pressure and while the pipe was empty. The stresses were calculated from the modulus of elasticity which was obtained from laboratory tests by means of Eqs. (46) to (48) on the assumption that the no-load stress in the staves was zero. The stresses are shown in the polar diagrams in Appendix C. The results of the tests can be summarised as follows.

Test No. 1.

The observed stresses in the bands are approximately equal to the calculated values. The greatest difference (about 100 kg per cm²) was observed at the crown of the pipe. If this difference were caused by the no-load stress alone, it would be equal to 2.5 kg per cm².

Test No. 2.

The observed stresses are approximately equal to the calculated values. Hence it follows that the no-load stress was negligible.

Test No. 3.

The observed stresses are higher than the calculated values. The difference between them corresponds to a no-load stress of 4.4 kg per cm².

Test No. 4.

The observed stresses are considerably higher than the calculated values. The difference between them corresponds to a no-load stress of 8.0 kg per cm².

Test No. 5.

At the crown of the pipe, the observed stresses agree with the calculated values, whereas the observed stresses on the sides are considerably higher than those obtained from the calculations. However, the dispersion of the measurements is great, and no great importance should therefore be attached to this test. The mean value of the stresses observed on the sides of the pipe exceeds the calculated value by 335 kg per cm². This difference corresponds to a no-load stress of 6.8 kg per cm².

Test No. 6.

This test is of special interest since the stresses were measured at full pressure, on the empty pipe, and after loosening the bands. It is seen from the diagrams that the observed stresses are in very close agreement with the calculated values. The observed values were obtained from the difference between the tensometer readings taken on the empty pipe and at full pressure. The difference between the tensometer readings taken at full pressure and after loosening the bands gives a slightly higher stress. This difference corresponds to an actual no-load stress in the staves which varied from 1.2 to 5.8 kg per cm^2 and amounted to 3.0 kg per cm^2 on the average.

Test No. 7.

This test was made in the same manner as the test No. 6. The agreement between the calculated and the observed stresses is very close. The no-load stress is lower than in the test No. 6. On the average, this stress is 2.3 kg. per cm^2.

Test No. 8.

The agreement between the calculated and the observed stresses is good for one side of the pipe, that is to say, the no-load stress is equal to zero. On the other side, some disturbance has probably occurred when the band was loosened since two values are unusually high, whereas the stresses at the crown of the pipe are low.

Test No. 9.

The agreement between the calculated and the observed stresses is close at the crown of the pipe and near the surface of the ground, but the stresses are remarkably high at some intermediate points. The mean value of the no-load stress for all test points is 4.3 kg per cm^2.

4. Tests Made at Gideåbruk

The Gideåbruk Hydro-Electric Station, which is owned by the Gideå and Husums Co., is situated on the river Gideälv, and utilises a head of 11.5 to 13.8 m at a normal discharge of 22 m^3 per sec. The station was built in 1913, and is equipped with three Francis generating units having a turbine rating of 1 900 kW at a net head of 11 m each. The water flows from a concrete regulation dam through a canal, 165 m in length, to a concrete intake, and then passes through a wood-stave pipe, 480 m in length, to a surge tower which is situated quite close to the station.

The characteristics of the wood-stave pipe are as follows. The pipe was originally circular, and its inside diameter was 3.0 m. The upper section (190 m in length) of the pipe is carried on supports, whereas the lower section is embedded in gravel to half its height. The horizontal diameter of the embedded section of the pipe was measured during the tests, and was found to vary from 3.22 to 3.16 m (the

Fig. 15. Embedded section of the wood-stave pipe at the Gideåbruk Hydro-Electric Station, 3.2 m in diameter. This pipe has been in operation more than 30 years. The upper surface of the pipe is slightly damaged by decay. The shoes are of the eccentric type.

head over the crown of the pipe was about 2.6 m). The wall of the pipe is made of continuous wood staves, 89 mm in thickness. The pipe is provided with round steel bands, 25 mm in diameter, spaced 39 cm (on the greater part of the embedded section of the pipe). The cast iron shoes of an eccentric type are fitted at the crown of the pipe. The head over the crown of the pipe varies in operation from 2 m at the intake to 2.6 m at the surge tower. The normal velocity of the water is 3.1 m per sec. Cf. Fig. 15.

The shoes were very hard to loosen because they were rusty, and it was therefore difficult to obtain any reliable values from the stress measurements. Nevertheless, the shoes at the crown of the pipe were loosened in the tests Nos. 1 to 13. The tests Nos. 14 and 15 were made while emptying and filling the pipe, but the values obtained from these tests are not fully reliable since it was raining during the measurements. As the head was about the same in all tests, the stresses should also be approximately equal. On the other hand, as will be seen from Appendix D, the results show considerable. differences between the tests. These differences are most clearly visible in the polar diagram in Appendix D. On the average, however, the observed stresses seem to agree with the calculated values. This is also the case in the tests made while emptying and filling the pipe. According to the well-known theory of swelling stresses (see p. 47), all stresses should have been equal, or should at least show a smaller dispersion, and should have been much higher than the calculated values. As has been pointed out in the above, these measurements are relatively unreliable, but their number is so great that the mean value of the stresses ought nevertheless to give a fairly good result. In the Author's opinion, the great dispersion is largely due to the fact that the bands are tightened at regular intervals, and are therefore subjected to an initial stress of varying magnitude. In general, no swelling stress seems to be present.

5. Tests Made at Äggfors

The Äggfors Hydro-Electric Station is owned by the Krångede Co., but has been leased to the Äggfors Co. The station is situated on the river Indalsälven, and utilises a head of 13 to 13.5 m at a normal discharge of 60 m^3 per sec. The station was built in 1914 to 1915, and is mostly used for supplying power to wood pulp mill grinders. The generating plant comprises five Francis turbines directly coupled to grinders and having a total rating of 5 000 kW, and three Francis turbines coupled to electric generators and totalling 590 kW. The water passes through a canal, 180 m in length, to an intake and then flows through two wood-stave pipes, 115 m in length. These pipes unite in a single steel pipe which bridges over the left arm of the river, and is directly connected to the turbine installed in the pulp mill building, see Fig. 16. (The station was shut down in 1949).

The two wood-stave pipes were originally circular, with an inside diameter of 3.4 m, and were embedded in gravel and broken stone to half their height. The horizontal diameter measured in the tests was found to be 3.53 m. The walls of the pipe are made of continuous wood staves, 4 in. thick, held together by round steel bands, 37 mm in diameter, which are spaced 37 to 54 cm. The bands are tightened by means of eccentric cast iron shoes. The head over the crown of the pipe varies in operation from 3.2 m at the intake to 7.4 m at the connection to the steel pipe. The normal velocity of the water is 3.7 m per sec.

Fig. 16. Two wood-stave pipes at the Äggfors Hydro-Electric Station after 35 years of operation. On account of the presence of shrubbery in the neighbourhood of the pipe and because of careless maintenance, the pipe has been damaged by decay, particularly in the longitudinal joints between the staves (cf. the two pipes at Sikfors which are equally old, see Fig. 5).

The bands were difficult to loosen, and the pipes were in a bad condition. For these reasons, the stress measurements were made only while emptying and filling the pipes. The test No. 1 was carried out on the right pipe. The tensometer readings were taken when the pipe was in operation and after the pipe had been emptied. The difference in stress was calculated from the difference between the tensometer deflections. The test No. 2 was made in a similar manner. The tensometer readings were taken when the pipe was empty, when it was submitted to an internal pressure, and when it was empty again. The results of the tests are graphically represented in Appendix E. Since the bands were not loosened, it was not possible to measure the no load stress, if any. The diagram shows that the observed stresses are in fairly close agreement with the calculated values.

6. Summary

On account of numerous sources of errors, the results of the tests described above are not quite satisfactory. The Huggenberger tensometers, which were used in the tests, are difficult to handle in general, and in these tests in particular, since they had to be made under field conditions. The shoes were often affected with rust to such an extent that great difficulties were experienced in loosening the nuts, and there was always the risk of disturbing the tensometers. Consequently, there can be errors in the individual tensometer readings. On the whole, however, a reliable estimate of the results can be formed. The results of the tests can be summarised as follows.

In some tests, the tensometer readings were taken both at full water pressure and when the pipe was empty. From these tests we can obtain those stresses in the bands which are caused by the external forces alone (the water pressure and the weight), that is to say, the stresses which are not influenced by the no-load compressive stresses (due to swelling of the wood and to tightening of the bands), if any. In all tests carried out in this manner (2 tests at Sikfors, 2 tests at Harrselsfors, 2 tests at Gideåbruk, and 2 tests at Äggfors), the stresses calculated from Eqs. (46) to (48) are in close agreement with the observed values, as may be seen from the detailed description of the tests in the above and from Appendices B to E.

The actual stresses in the bands were measured by loosening the shoes (21 tests). This was possible in the tests made on two pipes only. In the tests made at Sikfors, the shoes stuck fast so hard that it was not possible to make more than one tests. At Harrselsfors, on the other hand, it was very easy to loosen the bands. Loosening of the bands was facilitated by the fact that each band was provided with a shoe on either side of the pipe, close to the surface of the ground, in contradistinction to the bands on the other test sites. At Gideåbruk, it proved possible to loosen 13 bands, although it cost much trouble, whereas the shoes at Äggfors were so rusty that the bands were not to be moved. The results of these tests show considerable variations of the stress in the bands, but these variations are probably in part due to errors in measurements. The difference between the observed and the calculated values has been expressed in terms of the corresponding no-load stress. According to the tests made at Sikfors, the no-load stress should not exceed 1 kg per cm^2, whereas the no-load stress at Harrselsfors varied from 0 to 8 kg per cm^2, and the mean value of this stress for 9 tests was 4 kg per cm². The tests made at Gideåbruk were interesting in that the water pressure was nearly unchanged in all 13 tests, whereas the stress in the bands was found to vary to a fairly large extent. Nevertheless, the mean value of the observed stresses agrees with the calculated value. In general, the no-load stress seems to be insignificant, and its estimated value does not exceed 4 kg per cm² under normal conditions.

The tests do not afford any information on the cause of the no-load stress (swelling of the wood or tightening of the bands), but considering the large dispersion of the test results, there is every reason to believe that tightening of the bands has contributed to the no-load stress more than swelling of the wood. According to the results obtained from earlier tests (see p. 47), the no-load stress should be constant, and should amount to about 10 kg per cm², irrespective of any initial stresses due to tightening of the bands by hand.

The following conclusions can be drawn from the tests described in the above.

1. The observed stresses in the bands caused by the internal water pressure and by the weight of the pipe and the water are in close agreement with the stresses calculated from Eqs. (46) to (48).

2. The no-load stress in the wood staves (caused either by tightening of the bands or by swelling of the wood) is subject to variations within wide limits, probably owing to differences in band tension. The greatest no-load stress was observed on the most recently constructed pipe, at Harrselsfors, and was found to be 5.8 kg per cm². On the average, the no-load stress varies from 2 to 3 kg per cm². The swelling stress is of course still smaller, and is probably quite negligible.

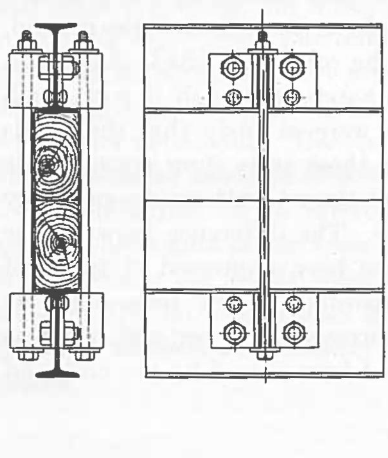

Fig. 17. Test set-up used by Samsioe in 1915 for measuring the force due to swelling of the wood.

G. Swelling Stresses

It follows from the above that the swelling characteristics of the wood are of primary importance in the design of wood-stave pipes. Swelling of the wood has the negative consequence that the swelling stress must be taken into account in the design of the bands, and the positive consequence that the pipe is kept watertight without any special devices. Both the swelling stress in itself and its effect on the design of the pipe are of interest, and will therefore be touched upon in what follows.

1. Earlier Tests

So far as is known to the Author, the first measurements of the force due to swelling of wood were made in 1915 by Samsioe (79). He used a test set-up which essentially comprised two I beams which were connected by means of two round iron rods (see Fig. 17). Wood stave in an air-dry condition were inserted between the beams, and the nuts on the tie rods were tightened until a definite initial stress was produced in the wood, as desired. After that, the whole test set-up was immersed in water. In the measurements of the swelling stress, the beams were pressed together in a hydraulic press until it was possible to loosen the nuts on the tie rods. The compressive force required for this purpose was considered to correspond to the swelling force. On the basis of 3 tests, Samsioe has found that the swelling stress in pine at right angles to the fibres after immersion in water for 2 to 3 months was about 10 kg per cm², irrespective of the initial stress. The tests were made at the initial stresses of 0, 15, and 20 kg per cm².

In 1931, a series of tests was carried out by the Vattenbyggnadsbyrån (VBB), Consulting Engineers, Stockholm, in order to investigate free swelling of wood and to determine the magnitude of the swelling stress. The Author has been given an opportunity to acquaint himself with the results of these tests. The method of testing was the same as in the earlier tests referred to in the above, but the test specimens were cube-shaped, and arrangements were made for preventing their lateral expansion in one, two, or all directions. The tests comprised 3 series, and 3 cubes without any initial stress were used in each series. When lateral expansion was prevented in one direction at right angles to the fibres, a swelling stress of 8 kg per cm² was observed after the test specimens had been immersed in water for 5 months. When lateral expansion was prevented in two transverse directions, the observed swelling stresses were 19 kg per cm² in one direction and 17 kg per cm² in the other. When lateral expansion was prevented in all directions the observed swelling stresses were 8 kg per cm² in the longitudinal direction and 18 kg per cm² in the transverse directions.

In all tests mentioned in the above, the swelling stresses were measured by means of fundamentally similar methods, namely, the wood specimen was pressed together between two opposite beams until it was possible to loosen the nuts on the ties holding the beams together. However, this method of testing seems to involve considerable errors in measurement since the test specimen is subjected to deformation every time the measurements are made. Some measurements indicate that the deformations varied between 0.5 and 0.9 mm. In order to produce these deformations alone, a compressive stress of 1.7 to 3.1 kg per cm² in the wood is required, if the modulus of elasticity is taken to be 1 000 kg per cm².

The Norwegian rules for the design of wood-stave pipes assume that the swelling stress is 7 to 10 kg per cm². This figure is based on the above-mentioned tests made by SAMSIOE and on some tests carried out in Norway. The Norwegian swelling stress measurements were made on pipes 1 m in diameter. For pine, spruce, and impregnated pine, the observed swelling stresses were 8.3, 6.5, and 7.6 kg per cm² respectively.

2. New Tests

In the stress measurements on old wood-stave pipes referred to in the above, the observed swelling stress was on the average only 2 to 3 kg per cm². This seems to indicate that the swelling stress decreases in course of time. In order to provide further information on the magnitude of the swelling stress and its variation with time, the Author has started some tests which are still carried on. By using a specially designed test set-up (see Figs. 18 and 19), the Author has reduced the deformations during the stress measurements to a negligible amount, which corresponds to a stress not exceeding 0.1 kg per cm². It is seen from Figs. 18 and 19 that the stress is measured by means of a spring-type pressure gauge. The load is applied to the gauge by tightening a nut every time a reading is taken. The displa-

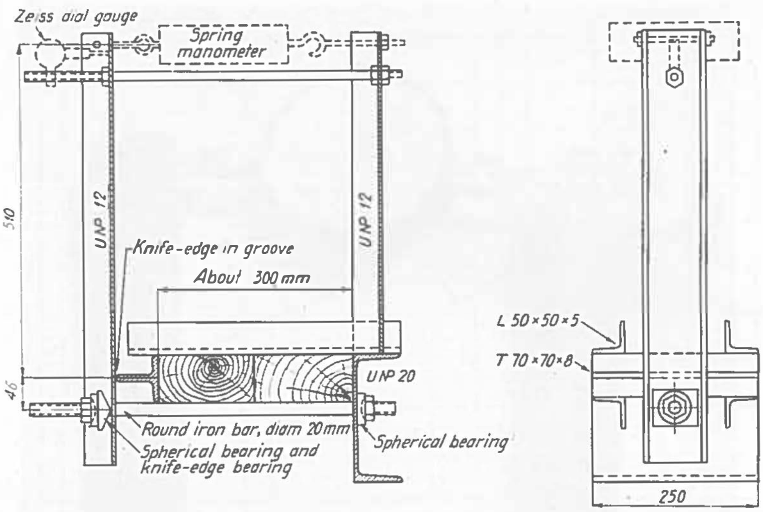

Fig. 18. Test set-up used by REGNELL for measuring the force due to swelling of the wood. Owing to improved control of the deformations of wood, this test set-up gives more reliable results than the type shown in Fig. 17. In principle, these two test set-ups are similar.

cement of the lever with respect to the end stay is measured by means of a dial gauge. It is assumed that the load has been completely transmitted to the pressure gauge when the deflection of the dial gauge begins to change. The lever is provided with a special spherical bearing and a knife-edge fulcrum. Since the length of the lever is known, the stress in the wood can be calculated. Two tests have been carried out up to now. The wood specimens used in both these tests were made of air-dry pine which was free from knots and had a moisture content of about 15 per cent. In the test No. 1, the initial stress was zero when the test specimen was immersed in water. In the test No. 2, the initial stress was 6.5 kg per cm². The test results (see Fig. 20) show that the swelling stress in the test No. 1 reached a maximum of 9.4 kg per cm² in 50 days, whereas the swelling stress in the test No. 2 increased to a maximum of 10.9 kg per cm² in 14 days. After that, the swelling stress gradually decreased in both test specimens. In the test No. 1, the swelling stress was 7.8 kg per cm² after 7 months, while in the test No. 2, where the specimen was subjected to an initial stress, the swelling stress was about 8 kg per cm² after 6 months. It may be approximately inferred from Fig. 20

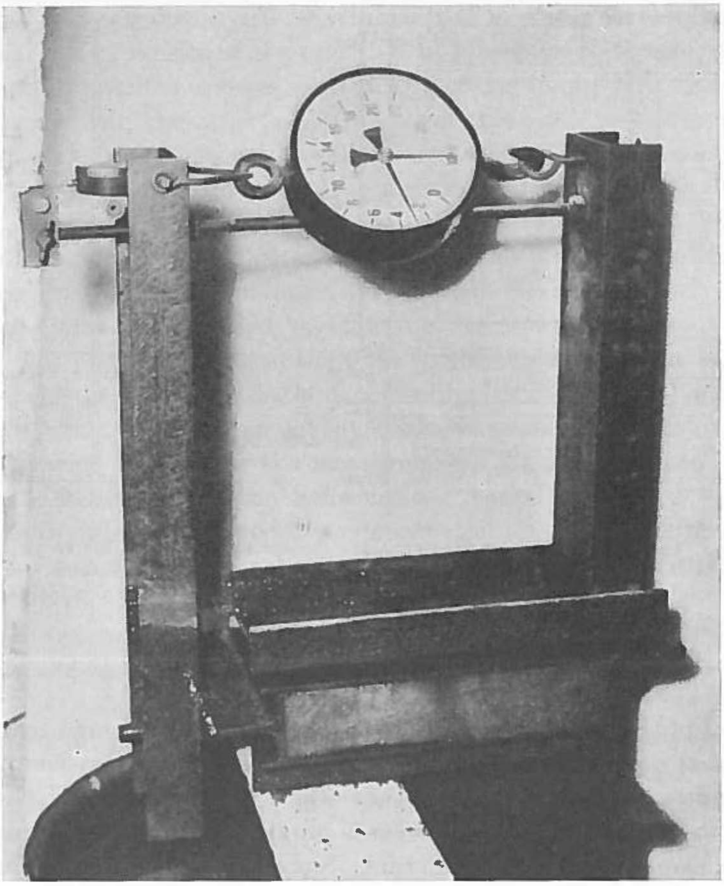

Fig. 19. View of the test-up shown in Fig. 18 during the tests (removed from the water tank).

that the decrease of the swelling stress to a figure as low as 2 to 3 kg per cm² would probably take several years.

3. Conclusions Regarding Swelling Stresses

In order that a wood-stave pipe shall be watertight, and shall be held together as a whole, it is necessary that the staves should be subjected to a certain compressive stress which can be caused either by swelling of the wood or by tightening of the bands. The results of the tests made on old pipes indicate that the compressive stress in

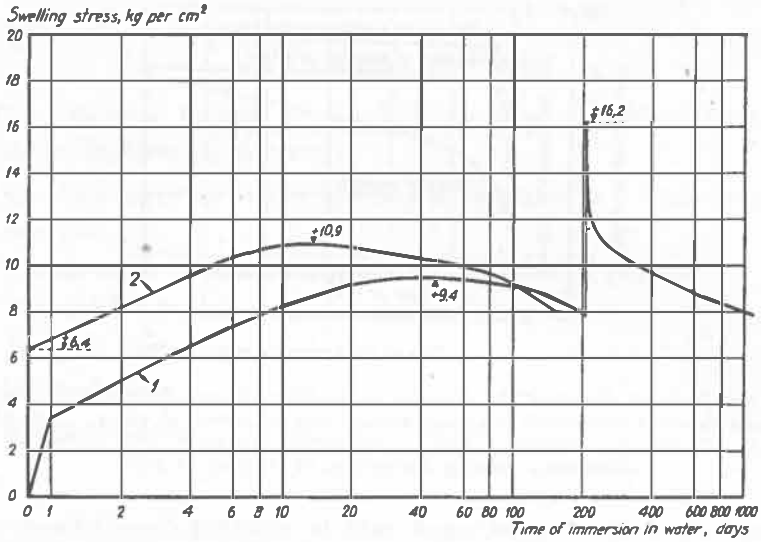

Fig. 20. Stress due to swelling of pine samples tested in water. Preliminary results of tests which are not yet completed. On the whole, the method of testing reproduces the conditions in a wood-stave pipe made of staves 70 mm in thickness and provided with round bands, 20 mm in diameter, spaced 250 mm. The timber samples used for the two tests in process of execution were taken from the same stave which was made of common air-dry pine. The sample No. 1 was immersed in water without tightening the nuts on the tie rods (that is to say, the initial stress in the wood was zero). The sample No. 2 was immersed in water after tightening the tie rods, and the initial stress was 6.4 kg per cm². After some 200 days, the nuts of the rods of the sample No. 1 were tightened, with the result that the compressive stress in the wood increased to 16,2 kg per cm². Two years later the compressive stress decreased to 7,8 kg per cm².

the surface of contact between the staves may be low (2 to 3 kg per cm²), without giving rise to leakage. Therefore, a swelling stress of 10 kg per cm² seems to be unnecessarily high, so far as water-tightness is concerned, and it appears justifiable to take appropriate measures in order to reduce the swelling stress. It has been usual of old to adjust the band tension during the construction of wood-stave pipes so as to render the pipe watertight and to prevent the swelling stress from becoming too high with respect to the bands. Fig. 21 shows that the expansion of unrestrained pine specimens at right angles to the fibres during swelling approximately reaches the maximum value of 4 to 5 per cent after only 3 to 5 days. For a pipe of, say, 2 m in diameter, this expansion corresponds to an elongation of 30 cm on the circumference. The expansion due to swelling is so

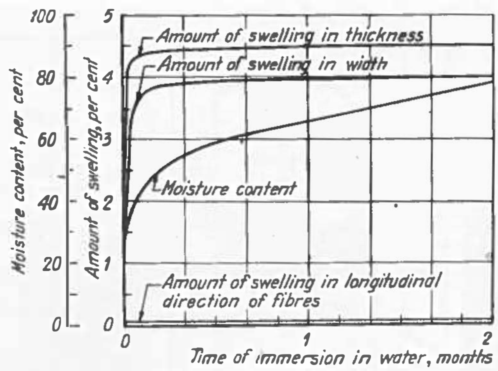

Fig. 21. Relation between the amount of swelling of unrestrained pine samples and the moisture content of originally air-dry pine immersed in water. Tests made by the Vattenbyggnadsbyrån (VBB), Consulting Engineers, Stockholm, at the Government Testing Institution of Sweden in 1932.

considerable that it is difficult to reduce the swelling stress in large pipes to the desired degree only by loosening the shoes. Therefore, it seems natural to reduce the swelling stress by using staves which have a certain initial moisture content and hence are slightly swollen from the outset, unless this is not possible for practical reasons. Even a moisture content of 30 per cent, as against the usual figure of 15 to 20 per cent, will probably be sufficient to produce a substantial reduction of the swelling stress. Consequently, this problem deserves further investigation, especially as its solution may be expected to lead to a relatively large saving in band material.

The tests and measurements described in the above show that the swelling stress in originally air-dry pine staves reaches a maximum of about 10 kg per cm² after immersion in water for not more than one month, and that the swelling stress considerably decreases in course of time. The magnitude of the swelling stress seems to be largely dependent on the initial moisture content of the wood prior to immersion in water. Therefore, it ought to be possible to reduce the swelling stress if the initial moisture content of the staves used for the construction of the pipe is increased to a certain degree.

H. Thickness of Staves

The following factors are to be considered in determining the requisite thickness of staves:

1. The load-carrying ability of the pipe regarded as a beam between supports.
2. The strength of the staves submitted to internal pressure.
3. The risk of buckling due to external pressure.
4. The risk of leakage.
5. Internal wear.
6. External decay.

1. Load-Carrying Ability of Pipe Regarded as a Beam between Supports

According to the stave pile method proposed in the above, see Eq. (16), the load-carrying ability of the pipe regarded as a beam between supports should be taken into account in determining the thickness of staves.

2. Strength of Staves Submitted to Internal Pressure

If we imagine the staves to be continuously supported on the bands and to be submitted to an internal pressure, a bending stress will be produced in the staves, and this stress should not exceed the allowable value. The stickness of staves can be written

$$d = 0.707\, l \sqrt{\frac{H_i}{\sigma}} \tag{53}$$

If the allowable stress in the staves is taken to be 600 tons per m^2 we obtain

$$d = 0.029\, l \sqrt{H_i} \tag{54}$$

The value of the internal pressure to be inserted in this formula is its maximum value which also comprises a certain allowance for surges.

3. Risk of Buckling Due to External Pressure

When the pipe is subjected to an external pressure which can be caused by partial vacuum in the interior of the pipe or by the earth covering, the pipe may be caved in or buckled. In order that the pipe shall be able to withstand any external pressure at all, the wall of the pipe must possess a certain stiffness. When a wood-stave pipe is exposed to buckling, which usually manifests itself in the collapse of the pipe at the crown, the stiffness of the walls is mostly determined by the thickness of the staves, since the bands prevent buckling in the outward direction only. We can write

$$H_{\text{crit}} = \xi_1 \cdot E \left(\frac{d}{\varrho}\right)^3 \tag{55}$$

where ξ_1 is constant for a given method of support. We assume that the radius of curvature of the pipe wall has increased to the value ϱ on account of permanent deformation. If we suppose that the pipe has been distorted into an elliptic shape and if we denote the coefficient of distortion by ε, we obtain approximately

$$\varrho = \frac{D}{2}\left(1 + \frac{3}{2}\,\varepsilon\right) \tag{56}$$

and consequently

$$H_{\text{crit}} = \xi_2 \frac{E}{\left(1 + \frac{3}{2}\,\varepsilon\right)^3} \left(\frac{d}{D}\right)^3 \tag{57}$$

Hence it follows that a small thickness of staves, a large diameter, and a deformation of the cross section of the pipe are unfavourable since they facilitate buckling of the wall. It is of interest to study the variation in the critical pressure with the dimensions of the pipe and with the amount of distortion. If we assume that the coefficient of distortion is maximum 0.2 for a large pipe, 4 m in diameter, and that this coefficient varies in proportion to the diameter, we obtain the values of the thickness of staves shown in Fig. 22. This diagram represents circular pipes and deformed pipes. If a pipe carried on supports is correctly constructed, it is fixed in a circular shape at

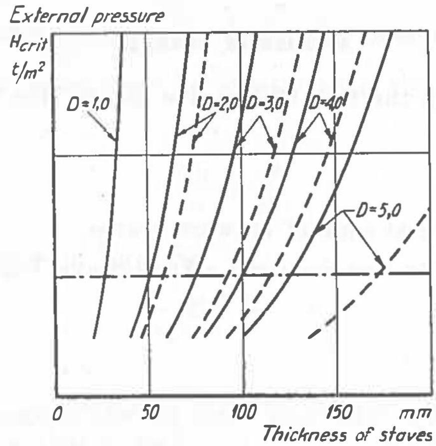

Fig. 22. Variation in the external pressure due to buckling of wood-stave pipes with the diameter of the pipe and the thickness of staves. The full-line curves refer to circular pipes. The dash-line curves refer to permanently deformed pipes. The diameters of these pipes in their initial, circular shape are indicated in the diagram. The horizontal dash-dot line represents the minimum value of the relative external pressure to be used in the design of pipes.

the supports, and the possibility of deformation between the supports is slight. Therefore, the curves for circular pipes ($\varepsilon = 0$) can generally be used for pipes carried on supports.

Embedded pipes, on the other hand, are in general liable to deformation, and the thickness of the staves used for pipes of this type must therefore be greater. Fig. 22 shows a horizontal line which represents a maximum of H_{crit} which has been found in practice to be suitable under normal conditions in view of the possibility of partial vacuum in the pipe.

The surges occurring when a pipe is emptied are often likely to produce dangerous vacuum. The pipes should therefore be emptied slowly and very carefully. If there is reason to fear that the vacuum in the pipe can become considerable, arrangements should be made for air supply by means of stand pipes or vacuum valves.

At the Sikfors Hydro-Electric Station ($D = 3.2$ m, $d = 0.075$ m), the crown of the pipe has collapsed on two occasions while emptying the pipe. After a vacuum valve had been fitted at the most dangerous point, such accidents have not happened again. One of the most known examples of pipes which have failed while being emptied is the pipe at Nymboida, Australia ($D = 1$ m, $d = 0.020$ m).

4. Risk of Leakage

Leakage through the wall of the pipe can be due to the following causes:

1. Percolation.
2. Insufficient lateral contact between staves.
3. Leaky end joints (see Section I »Wood-Stave Joints»).

a. *Percolation.*

About 1920, some American rules for the design of wood-stave pipes stipulated that in those cases where the internal pressure exceeded 24 m, the annual rings in the staves should be at right angles to the direction of pressure in order that the pipe should be watertight (59). Subsequent watertightness tests made by SEITZ (86) have shown that wood staves about 50 mm in thickness are practically watertight at pressures of up to 70 to 120 m, even if the annual rings are parallel to the direction of pressure. Knots and cracks are detrimental to watertightness.

b. *Lateral Contact between Staves.*

The lateral joints between the staves are rendered tight by the compressive stress which is always present in the surface of contact between the staves. This compressive stress is caused partly by tightening of the bands and partly by swelling of the wood. In America, it is considered that the compressive stress between the staves should be at least 1.5 times the internal water pressure or at least 8.7 kg per cm².

The water pressure causes the stave wall to bulge between the bands, with the result that cracks are formed between the staves. These cracks ought to be closed owing to the elasticity of the staves at right angles to the fibres. According to the theory of strength of materials, the deflection in the middle between the bands is

$$\eta_s = \frac{H_i\, l^4}{384\, E_{11}\, I} \tag{58}$$

The relative elongation in the tangential direction is $\frac{\eta_s}{r}$. As the resilience of the wood shall be greater than this value, we obtain the condition

$$\frac{\eta_s}{r} \leqq \frac{p}{E_{\perp}}$$

and hence, in virtue of Eq. (58),

$$p \geqq \frac{E_{\perp}}{E_{11}} \frac{H_i}{r} \frac{l^4}{32\, d^3} \tag{59}$$

Consequently, the compressive stress between the staves shall be at least as high as that given by Eq. (59) in order to prevent leakage between the staves. According to SAMSIOE (79), $E_{\perp} = 10\,000$ tons per m² under a static load. In the direction parallel to the fibres, the value of E may be assumed to be much higher, approximately $E_{11} = 1\,000\,000$ tons per m². Thus, the ratio of these moduli of elasticity is of great importance. When the load is applied suddenly, i. e. in the event of water hammer, the value of $E_{\perp}$ will probably be considerably higher. The value of the ratio $E_{\perp}/E_{11} = 1/10$ may be suggested as appropriate.

c. *Amount of Leakage.*

Leakage in a wood-stave pipe is dependent on many factors. Nevertheless, it may be of interest to know what amount of leakage may generally be expected to occur in a wood-stave pipe. Some data on leakage are tabulated below.

Source (notice)	Leakage litres per hour per m²	Remarks
E. N. R. (17)	0.25	Assumed
LEDOUX (54)	0.62	Under favourable conditions
CAMPBELL (9)	0.06	Observed
	0.33	»
MAIER (62)	0.61	Observed after 1 day
	0.25	Observed after 133 days.
MAURY (65)	0.45	Normal
SEITZ (87)	0.80	Favourable
SPIES (88)	0.59	Based on German experiences

The above table shows that the opinions on this subject are rather divergent. However, it seems that the value of 0.6 may be regarded as an appropriate mean value. In Sweden, the amount of leakage may be assumed to be slightly greater. For instance, in the tests made on the wood-stave pipe at the Stockvik Works, which was built in 1944 ($D = 0.75$ m, $L = 5\,407$ m, $H = 55$ m), the leakage was found to be 1.2 litres per hour per m^2, and the leakage in the water supply main at Boden ($D = 0.25$ m, $H = 60$ m) was unusually high, 2.3 litres per hour per m^2. On the basis of these figures, 1.0 litres per hour per m^2 may be taken as an appropriate standard value of maximum allowable leakage.

When a pipe is covered with earth, there is the risk of deformations which can give rise to leakage if the pipe is incorrectly constructed. Serious secondary damage can be caused by the water oozing through the leaks due to deformations. After that, the water leaking out of the pipe can give rise to serious erosion of the bedding material (98, 99). According to an American view, drains should therefore be provided at right angles to the pipe, so as to limit erosion of bedding material.

5. Internal Wear

Internal wear of wood-stave pipes is generally very slight. If the water flowing in the pipe is pure, there is practically no internal wear, and velocities of the water not exceeding 12 m per sec are not regarded as dangerous (7). According to Swedish experiences (78), considerable internal wear is extremely rare. One wood-stave pipe has been completely worn out in 22 years, and another pipe had to be lined with 25 mm thick staves on the inside. In some other pipes, the observed amount of wear varied from 3 to 20 mm. In the latter pipes, however, the velocity of the water did not exceed 2.4 m per sec. Waste water discharged from sulphite pulp mills seems to have a very unfavourable influence on wood-stave pipes. In the Norwegian rules for wood-stave pipes (71), it is stated that velocities of water flow up to 2.5 m per sec have occurred in Norwegian installations, without causing any appreciable internal wear. Consequently, it seems that internal wear does not require any increase of the thickness of staves, apart from exceptional cases.

6. External Decay

If the pipe is properly constructed, external decay appears to the rather insignificant, as will be seen in Chapter III. Therefore, it seems that no increase of the thickness of staves is required in order to allow for external decay.

I. Wood-Stave Joints

According to the method of jointing the staves, it is usual to distinguish between three types of wood-stave pipes, viz., collar type pipes, continuous stave pipes, and group stave pipes.

1. Collar Type Pipes

Collar type pipes are built up of abutting pipe units which are made of staves of equal length without any joints. The joints between the abutting ends of the pipe units are sealed by means of short wood or iron collars which are fitted on the outside of the pipe so as to cover the joints. The collars are usually made of wood and provided with steel bands, just as the pipe itself (see Fig. 23). For small pipes, up to 0.5 m in diameter, use is sometimes made of a still simpler type of joint by providing the abutting ends of the staves with tenon and mortise joints (see Fig. 24). The mortised end is often provided with additional bands. The collar type of pipe had been prevalent until about 1910, but was gradually superseded in water power

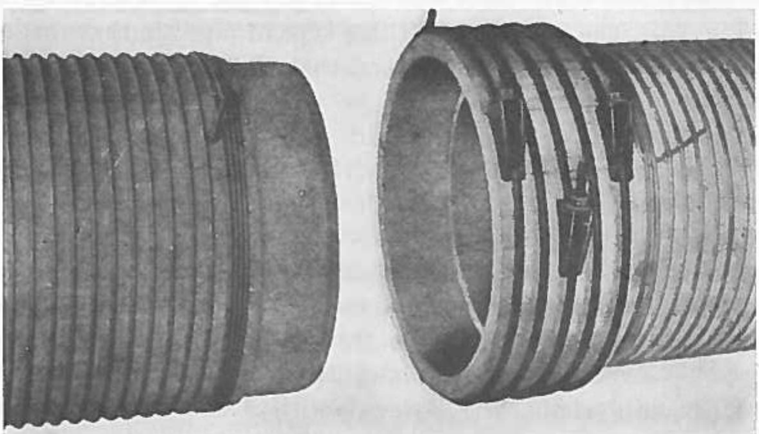

Fig. 23. Wood-stave pipe with individual banded wood collars. This pipe is machine-banded, and the collars are fitted with bands and shoes of the ordinary type.

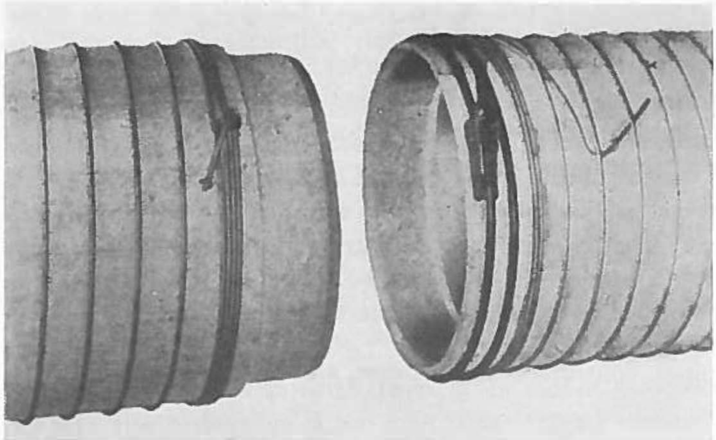

Fig. 24. Wood-stave pipe with reinforced inserted joints. This type of pipe is used at low pressures and for small diameters.

engineering by continuous stave pipes or by group stave pipes. In sanitary engineering, use is still made of collar type pipes for small diameters. These pipes are usually manufactured in factories and consist of pipe units of great length banded with wire and completely ready for assembly on the site. This type of pipe is known as machine-banded pipe.

2. Continuous Stave Pipes

In continuous stave pipes, the staves are laid so as to break joints (see Fig. 25). The advantages of this type of pipe are that the length of timber is utilised to the full, and that the pipe can easily be laid in curves, and hence can better be adapted to the profile of the ground. Special devices, which are described in what follows, are generally required for sealing the joints between the abutting ends of the staves (end joints). The continuous stave pipe was first developed in the United States of America, and came into use in Europe in the early 1920 ies. This type of pipe is still commonly employed. Additional bands are sometimes required in order to prevent the ends of the staves from being thrust outwards (see Fig. 26). To obviate this drawback, it is often required that all end joints should be covered with bands or iron plates.[1])

[1]) According to the Norwegian rules (71), the distance from the bands to the end joints should not exceed 10 cm.

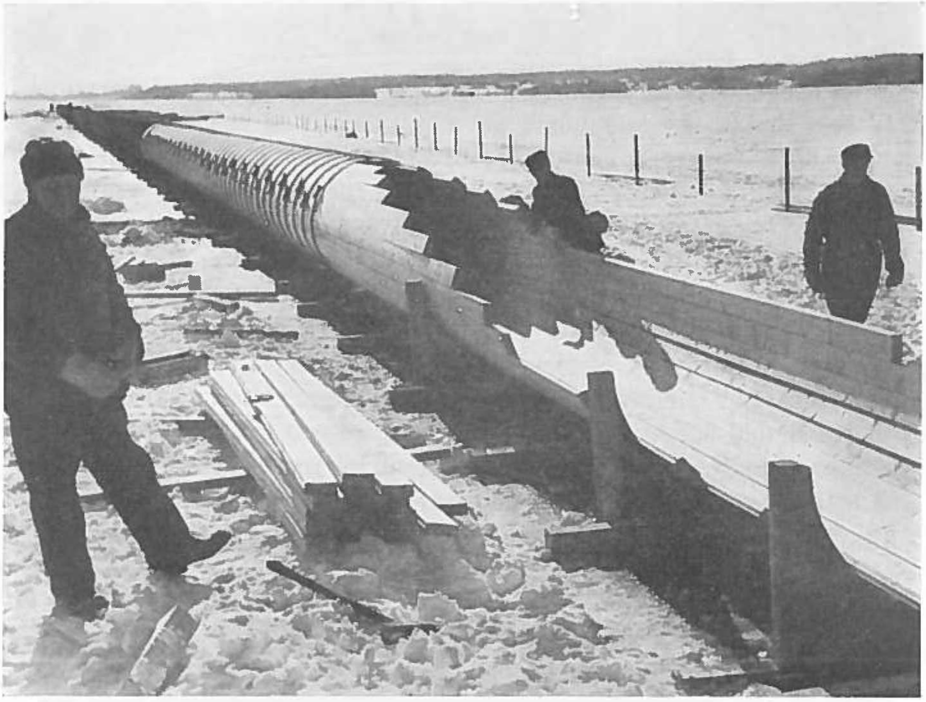

Fig. 25. Continuous stave pipe at Karlskoga in process of construction in 1945. Diameter 1.2 m. Length 500 m.

Under such conditions, use is made of »group stave pipes», which are an intermediary type between collar joint pipes and continuous stave pipes.

3. Group Stave Pipes

Group stave pipes comprise two groups of staves of equal length. Every second stave belongs to the same group. Every stave in one group is displaced by a given stave length with respect to the staves in the other group. Consequently, every second stave is jointed in the same cross section of the pipe. The end joints are covered with a flat steel band which surrounds the pipe and is held in place by another steel band (see Figs. 46 and 49). This device supports the ends of the staves, and hence increases the tightness of the end joints. Group stave pipes have become increasingly common in Sweden and Norway during the past few years.

4. End Joints

As the amount of swelling of the wood in the longitudinal direction is very small, the end joints are not as tight as the lateral joints between the staves. In continuous stave pipes, the end joints are not covered with collars or flat bands, and the staves have a tendency to bulge outwards, with the result that water can leak out through the joints. If the end joints of a pipe become leaky, they can be covered with iron plates on the outside of the pipe (see Fig. 5 b). Nowadays, the end joints are always provided with iron plates inserted in the ends of the staves, and these plates are generally sufficient for effective prevention of leakage. However, the iron plates should not be too large, otherwise ice can easily be formed in winter, and the staves can be damaged by freezing. According to American practice, the iron plates are made of 1 1/2 in. by 1/8 in. (38 mm by 3 mm) galvanised hoop iron, and are inserted in slots

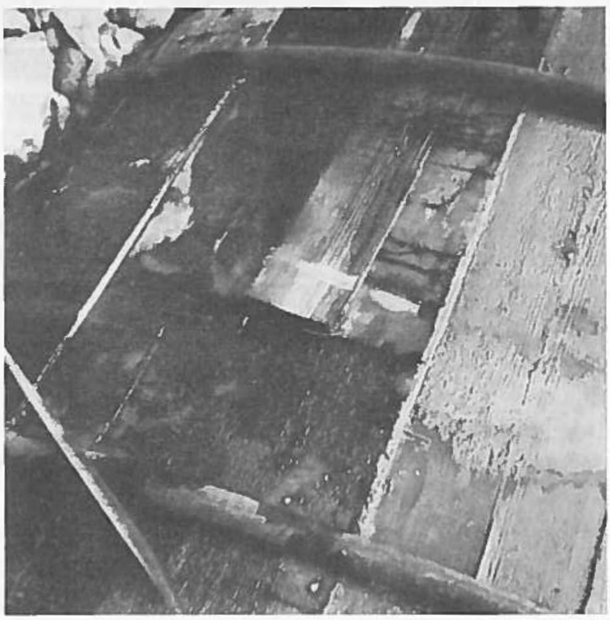

Fig. 26. Example showing a warped stave in a plain joint in one of the pipes at the Sikfors Hydro-Electric Station. The cause of warping is the large distance between the bands and probably also the absence of metal plates inserted in the joints. The damage caused to the pipe was slight. The joints were effectively repaired by means of iron plates and additional bands. See Fig. 5b, left tube.

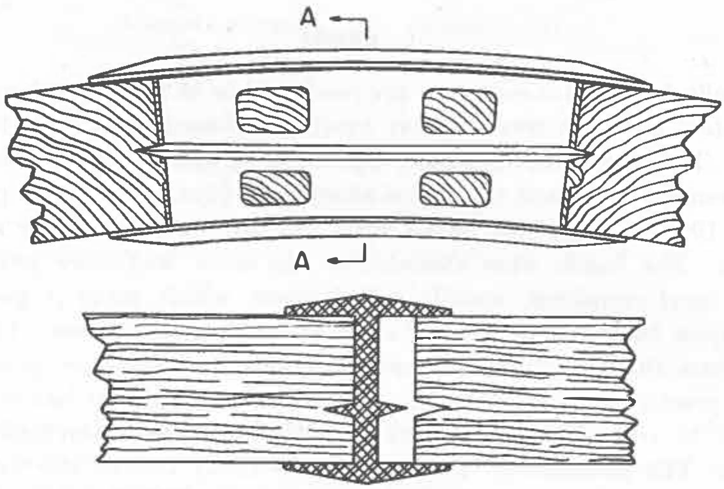

Fig. 27. Connection between abutting ends of staves, using iron butt joint (National Tank and Pipe Company, U. S. A.).

sawn in the ends of the staves (see Figs. 46 and 48). When the bands are tightened, the lateral joints between the staves are also rendered watertight by the plates. Sometimes use is made of other, more effective joint seals of cast iron and sheet iron which cover the ends of the staves both on the inside and on the outside (see Fig. 27). In one of the largest wood-stave pipes in the United States, the old joints fitted with ordinary iron plates are being successively improved by covering the abutting ends of the staves with plates both on the inside and on the outside. Each pair of plates is connected by two bolts passing through the joint. Effective sealing is ensured by means of a rubber gasket placed between the plates and the staves.[1])

[1]) The Author owes this information to the Utah Power and Light Co. The wood-stave pipe in question is installed at the Grace Hydro Plant, and is 3.4 m in diameter and 6 700 m in length. The head varies from 6.4 m at the intake to 38 m at the surge tower which is situated at the lower end of the pipe.

J. Bands

Bands for wood-stave pipes are made either of round steel rods or flat steel bars. In Sweden, flat bands were used, as a rule, before 1910. The bands were generally tightened by driving wooden wedges between the band and the staves around the pipe. During the period from 1910 to 1920, flat bands were gradually superseded by round bands. The bands were threaded at the ends, and were provided with band couplings, usually called shoes, which made it possible to adjust the tension of the bands better than heretofore. During the years 1920 to 1938, the use of flat bands for wood-stave pipes has been practically abandoned. In the past few years, flat bands have begun to come into use again, but they were now provided with shoes. The reversion to flat bands was chiefly due to the wish to reduce the pressure per unit surface between the bands and the staves. Furthermore, it was considered necessary to obviate the risk of local damage to the staves at the edges of flat bands, and the bands are therefore bevelled at the edges on that side which is in contact with the staves.

Since the staves are liable to decay if they are damaged, it is not advisable to use too high contact pressures. The compressive stress between the bands and the staves is

$$p_s = \frac{S_\varphi \, l}{r \, \Phi} \tag{60}$$

According to a view held in the United States (11), this compressive stress should not exceed the allowable stress for wet wood. In calculating this stress, the width of the surface of contact Φ is taken to be equal to half the diameter of the round band or the width of the flat band. For wet redwood, an allowable stress of 50 to 40 kg per cm² is considered to be suitable when the diameter of the round band is 3/8 to 7/8 in.

According to the Norwegian rules (71), the allowable stress for pine and spruce should not exceed 24 kg per cm² in the case of thin flat bands (used for thread diameters below 5/8 in.) and 18 kg per cm² in the case of thicker bands. For round bands, the following relation between the diameter of the band and the diameter of the pipe is given in the Norwegian rules.

Diameter of Band inches	Diameter of Pipe m
1/2	Not less than 0.5
5/8	» » » 0.9
3/4	» » » 1.3
7/8	» » » 1.8
1	» » » 2.4

K. Shoes

There are two types of shoes, viz., eccentric and centric.

The advantage of the eccentric shoes over the centric type, which is used at the present time, is that the former apply tension to both ends of the band. Therefore, eccentric shoes are sometimes employed on small wood-stave pipes at low pressures (see Fig. 49). In general,

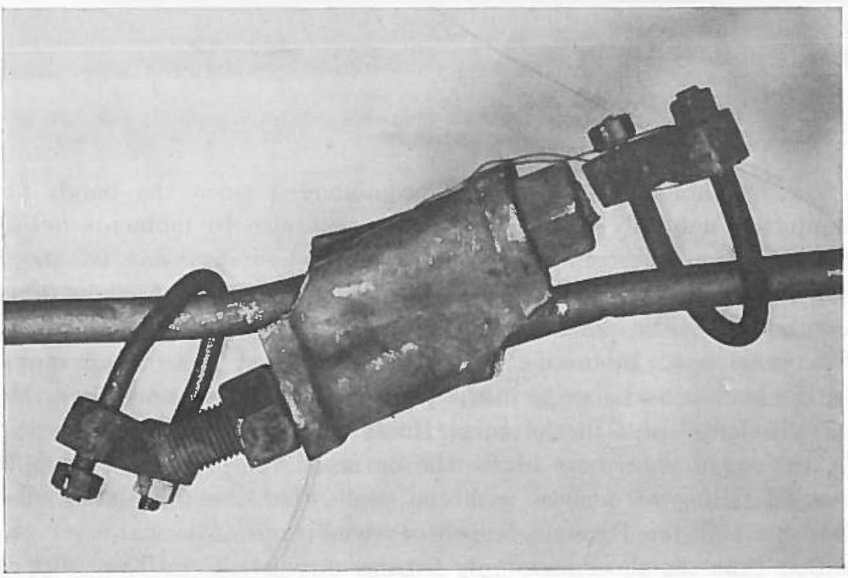

Fig. 28. Eccentric shoe shown after tension test in testing machine. This type of shoe is used on the pipe at the Sikfors Hydro-Electric Station. When the bands are tightened, the corners of the shoes often split off, and the shoes had therefore to be reinforced with U-shaped staples. When the shoes were tested without staples, the corners split off at a tensile force of 17 tons. When the shoes were fitted with staples, they proved able to withstand a tensile force of 30 tons, but the shoes were considerably deformed.

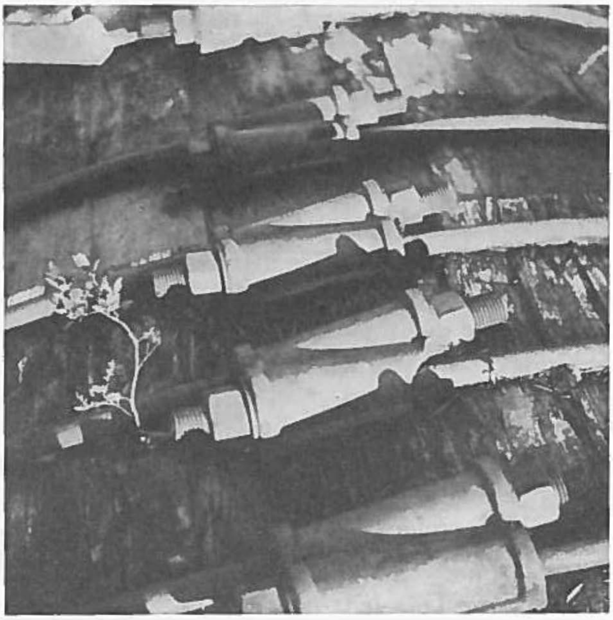

Fig. 29. Shoes used on the wood-stave pipe at the Äggfors Hydro-Electric Station. These shoes are of the eccentric type. The corners are split off at a few points.

eccentric shoes are not to be recommended since the bands are subjected not only to tensile stresses, but also to moments acting in the plane of the pipe.

At the Harrselsfors Hydro-Electric Station, eccentric shoes have caused trouble in connection with failure of the wood-stave pipe a few years ago. In brief, it occurred as follows. A sudden removal of the load caused a surge in the pipe. This surge was so violent that the discharge pipe in the surge tower was crushed, and the water in the surge tower rose above the normal level. The high pressure caused failure of a shoe, with the result that the pipe burst open like a zip fastener over a length of about 5 m. Afterwards, it was found that the shoes have only become untwisted, and have slipped off the bands one by one. This pipe failure caused great damage to the hydro-electric station where the water rose 1.5 m above the floor level of the turbine room, and injured the alternator and other equipment. The wood-stave pipe at the Sikfors Hydro-Electric Station is also equipped with eccentric shoes. When the bands are

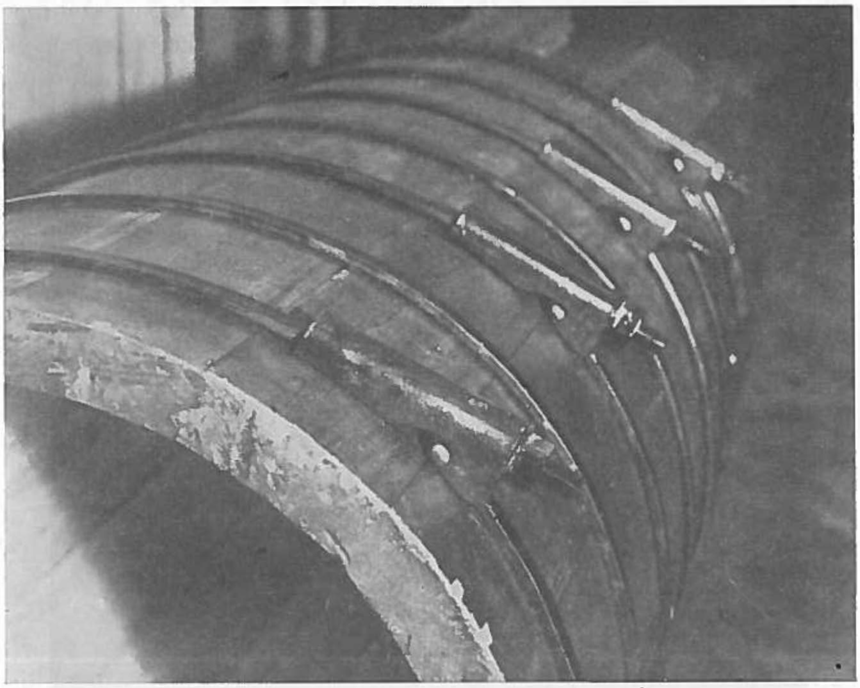

Fig. 30. Centric sheet steel shoes manufactured by the Boxholms AB, Boxholm, Sweden.

tightened, the corners of the shoes often split off, and the shoes must be reinforced with U-shaped staples. When the bands and shoes fitted with staples were subjected to a tension test in a testing machine, it was found that the shoes were able to withstand a load of nearly 30 tons (diameter of band 32 mm, tensile stress 3 800 kg per cm^2) without any damage other than the deformation of the staples. When the staples were removed, the shoes were able to withstand only slightly more than 17 tons, and the corners were split off (see Fig. 28). Consequently, owing to the low strength of the shoes (in their original condition, i. e. without staples), the maximum possible stress in the bands was equal to no more than half the ultimate strength of the bands themselves.

Centric shoes were introduced in Europe about 1920, following the example of the United States. In the beginning, the shoes were made of cast steel, but nowadays they are often made of bent sheet steel[1]) (see Figs 30 and 47). One end of the band is forged into a *T* shape, and can be hooked up in the shoe, while the other end is threaded, so that the band can be tightened by means of a nut.

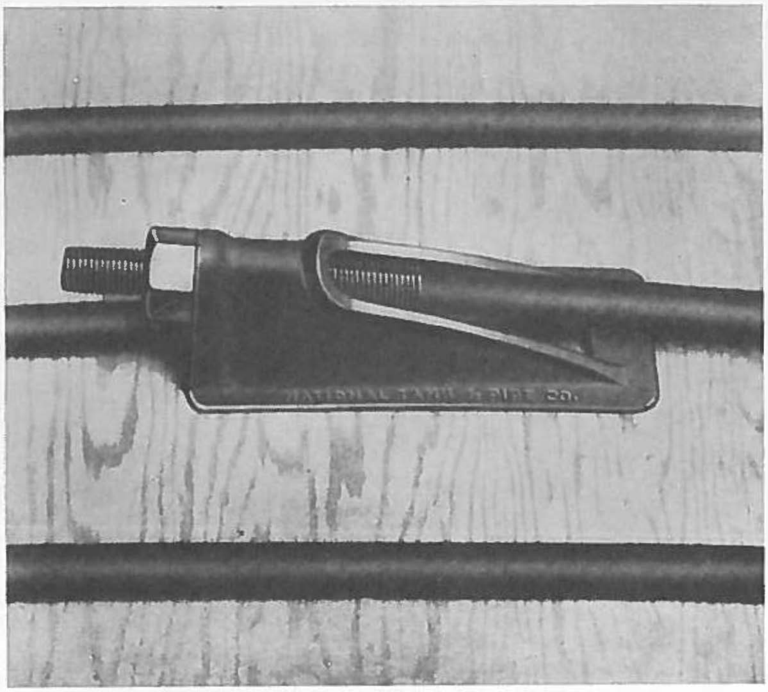

Fig. 31. Centric cast iron shoe manufactured by the National Tank and Pipe Company, U. S. A.

[1]) The Author has been informed that this type of shoe has been designed by the Vattenbyggnadsbyrån (VBB), Consulting Engineers, Stockholm, and by the Boxholms AB, Boxholm.

III. Operating Conditions

In the design of wood-stave pipes, regard must be paid not only to their strength, but also to operating conditions, e. g. decay, freezing, and losses of head.

A. Decay

1. Previous Experience

The length of life of a wood-stave pipe is generally determined by decay of wood, and it is therefore very important to examine the influence of this factor. The first data on decay of wood-stave pipes were collected in the United States as early as in 1913. According to TIFFANY (94), the joints of wood collar type wood-stave pipes are very easily exposed to decay because the collars are not thoroughly saturated with moisture. The use of sheet iron collars is therefore preferable. In particular, those pipes which are in operation during a part of the year should be protected by a first-rate asphalt coating which must also cover the bands and the collars at the joints. If the asphalt coating is inadequate, the pipe can decay very rapidly. However, it is to be noted that the climate conditions in the United States are entirely different from those in Sweden. During the warm part of the year (the season of irrigation) from about April 1st to November 1st, the climate in the United States is very dry nearly desert-like, while the winter is relatively cold, and the amount of precipitation, mostly rain, varies from 100 to 150 mm. The pipes are generally operated during the dry season only, whereas they are empty in the winter. As most American pipes are covered with earth, they retain moisture quite well during the winter, and are thus kept from decay. The length of life of such pipes, which are in operation during the season of irrigation only, is estimated at about 20 years. In a contribution to the discussion on the paper by TIFFANY, CHANDLER

(10) agrees with the above-mentioned conclusions, and gives an account of further experience concerning a pipe which had been completely covered with earth at the beginning, but was subsequently laid bare in some places by the wind. After 5 years of operation, the bared parts of the pipe and that part which was covered with moist earth were found to be thoroughly sound, whereas another part covered with dry earth had decayed to a considerable extent. After that, the whole pipe was laid on supports, and this was regarded as a very important improvement, not only in view of reduced decay, but also because it became easier to inspect the pipe and to stop up leaks, if required.

A few years later, HENNY (34) published a comprehensive report on operating experience regarding some 80 wood-stave pipes in the United States. As far as decay is concerned, HENNY gives the following figures for the life of pipes.

Non-Impregnated Pipes.

Pine, non-covered pipes	12 to 20 years
Pine, pipes covered with watertight earth	20 years
Pine, pipes covered with loose earth	4 to 7 years

Impregnated Pipes (generally with asphalt or tar).

Pine, pipes covered with watertight earth	25 years
Pine, pipes covered with loose earth	15 to 20 years

It is stated that the life of non-impregnated wood-stave pipes made of American redwood and embedded in watertight earth or in sand can exceed 25 years. HENNY arrives at the conclusion that a non-covered pipe has a longer life than a pipe covered with loose earth, and that a pipe covered with watertight earth has a longer life than a non-covered pipe. A low water pressure has an unfavourable influence on the length of life, particularly if the pipe is made of pine staves. The use of wood-stave pipes is not considered to be suitable if the pipe is to be empty during comparatively long periods of time. If the pipes are to be impregnated, this should be done thoroughly, and the penetration of tar and asphalt should be at least 1/16 in.

According to PARTRIDGE (74), the life of wood-stave pipes is dependent on the following factors:

1. Type of wood.
2. Quality of timber.
3. Method of drying.
4. Impregnation.
5. Diameter and spacing of bands.
6. Design, construction, and workmanship.

In view of the varying requirements in the above-mentioned respects, PARTRIDGE proposes that wood-stave pipes should be divided into three standard classes denoted by A, B, and C. In Class A, for instance, the pipes shall be made of redwood only, and their life is estimated at about 25 years, or more. In Class B, use can also be made of impregnated spruce, and the life is taken to be at least 10 years.

A subsequent investigation of 196 wood-stave pipes in the United States, which was carried out by the U. S. Reclamation Service (69), arrived on the whole at the same conclusions as those referred to in the above. Furthermore, the results of this investigation indicate a relation between decay and the age of the pipe. In the group of pipes less than 5 years old, 4.6 per cent of the total number of pipes in this group have decayed, while the corresponding figure in the group 6 to 10 years was 15.3 per cent, and in the above 11 years, 24 per cent.

As has already been pointed out, American experience is not directly applicable to European conditions since the climates are quite different. In a paper published in 1921, LUDIN (59) remarks that the figures on the length of life of wood-stave pipes given in the above-mentioned American sources appear to be too small, considering some special American and German experiences. According to German experience, the life of a correctly designed and constructed wood-stave pipe, which is permanently filled with water under a head of at least 6 m, may be expected to exceed 20 years. In Austria, a few years after the construction of wood-stave pipes in that country had been started in imitation of American models, it was realised that the life of these pipes could be much longer. This view was also based on American experience with wood-stave pipes which have become 50 to 60 and, in exceptional cases, even 100 years old (55). At about the same time, a far more optimistic opinion on the life of wood-stave pipes was expressed in a paper published in the United States (97). The author of this paper, BYRON WHITE, states that there are many examples of non-impregnated pipes which have lasted 20 or 30 years and longer, but on the other hand, there are also examples of pipes of similar types which began to decay after a few years. White considers that the wood-stave pipes operated at low water pressures should preferably be completely non-covered, and should be provided with some kind of external protective coating, in order to prolong their life as much as possible.

In the United States, small thickness of staves is considered to be an advantage so far as decay is concerned, since the timber is thoroughly saturated with moisture, even if the pressure is low. In order to reduce the risk of decay and to achieve a saving in material at the same time, the thickness of staves in Germany has in some cases been reduced to 25 and 30 mm, although the internal pressure in the former case was as high as 80 m, and the pipe in the latter case was covered with earth to a height of 5 m above the crown (38). If the timber used for the staves is of good quality, a thickness of 18 to 20 mm is regarded as adequate in order to ensure the watertightness of the pipe wall. In addition to impregnation with asphalt and tar or creosote, which is commonly employed in the United States, particularly for machine-banded pipes, use has been made in Germany of impregnation with a cyanogen compound, especially for collar type pipes. It is considered that the life of wood-stave pipes can reach about 35 years under favourable conditions, and at least 25 years under other conditions. When the pipe is to be embedded in earth, it is particularly important to make sure that the earth does not contain any humus or plant roots. In order to protect the pipe from surface water, which may contain putrefactive bacteria, the pipe should be surrounded by a filtering layer of earth, and the thickness of this layer above the crown of the pipe should not be less than 0.6 to 0.8 m.

So far as is known to the Author, no summary of results obtained from the operation of wood-stave pipes in Norway has been published. However, some stipulations concerning staves are included in the Norwegian rules for wood-stave pipes issued in 1942. For instance, it is stated that the pipe may be made either of pine staves or of spruce staves, but not of both at the same time. The staves shall be sawn from the lower part of the trunk, and shall not consist of loose, rapidly grown timber. In respect of planing it is stipulated that the heartwood side should be turned inwards, otherwise the staves tend to warp so that the concave side faces outwards. The rules do not contain any data on the life of wood-stave pipes, but the Author has been informed that the life of these pipes is nowadays considered in Norway to be approximately equal to that of steel pipes, that is to say from 40 to 50 years.

2. Recent Swedish Experience

In Sweden, a considerable number of wood-stave pipes are in use, but no operating results have been collected and published until quite recently (78). This publication is based on data concerning 213 pipes. A weak point in this investigation, just as in the previously mentioned American report published in 1921, is the fact that the degree of decay was estimated by the respective pipe owners or members of local staff. Accordingly, these estimates are of course greatly dependent on the qualifications of the persons in question and on their choice of terms. On the whole, however, the correctness of the final results can be relied upon, considering that the material collected for this investigation was very extensive.

Information as to decay has been furnished on 213 pipes, and it was found that 55 per cent of the pipes have decayed in a greater or smaller degree. Exceptionally heavy decay has been reported in

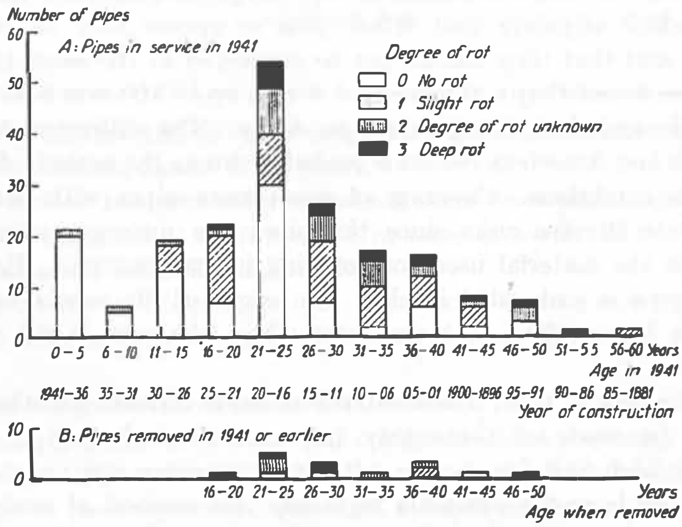

Fig. 32. Number of decayed wood-stave pipes classified according to age and degree of decay.

the case of 9 per cent of the total number of pipes. In general, the pipes have not been subjected to any impregnation whatever. Fig. 32 shows a classification of the pipes according to their age and degree of decay. It is seen from this diagram that the percentage of pipes damaged by decay increases at a particularly high rate after the first 15 to 20 years of service, and finally reaches 90 to 100 per cent when the pipes are 45 years old or still older. On the other hand, the percentage of old pipes which have undergone exceptionally heavy decay is not very high. In the interval from 25 to 50 years, only 10 to 15 per cent of the pipes have decayed to a considerable extent. Moreover, these figures should be accepted with great reserve since the collected data include only those pipes of a definite age, e. g. 30 years, which are still in operation, but these data do not indicate the total number of pipes built 30 years ago. A classification of the pipes removed in 1941 or earlier is given in the bottom diagram of Fig. 32, and this diagram can be regarded as fairly complete.

It follows from the above that the life of wood-stave pipes in Sweden, in so far as it is determined by decay, generally reaches 40 to 45 years, but does not exceed 45 years in normal cases. It is to be noted that the pipes in Sweden are usually not impregnated, contrary to American practice, but most of these pipes are filled with water the whole year. A comparison between pine and spruce staves has confirmed the above-mentioned view expressed in the Norwegian rules which stipulate that either pine or spruce may be used for staves, and that they should not be employed at the same time. It has been found that a thickness of staves up to 100 mm is not unfavourable and does not conduce to decay. The difference between Swedish and American results is probably due to the entirely different climatic conditions. Covering of wood-stave pipes with earth has proved to involve risks since the pipes can undergo considerable decay if the material used for covering is inappropriate. However, if the pipe is embedded in clay, the length of life can be expected to even longer than in those cases where the pipe is not covered at all.

At the present time, it is often considered in Sweden that the staves should be made of thoroughly impregnated timber, e. g. treated with Boliden Salt (an arsenic salt) under pressure and vacuum. No experience is so far available regarding this method of timber preservation, especially the influence of the salt on the bands and shoes, but there are reasons to believe that the life of the pipes which are

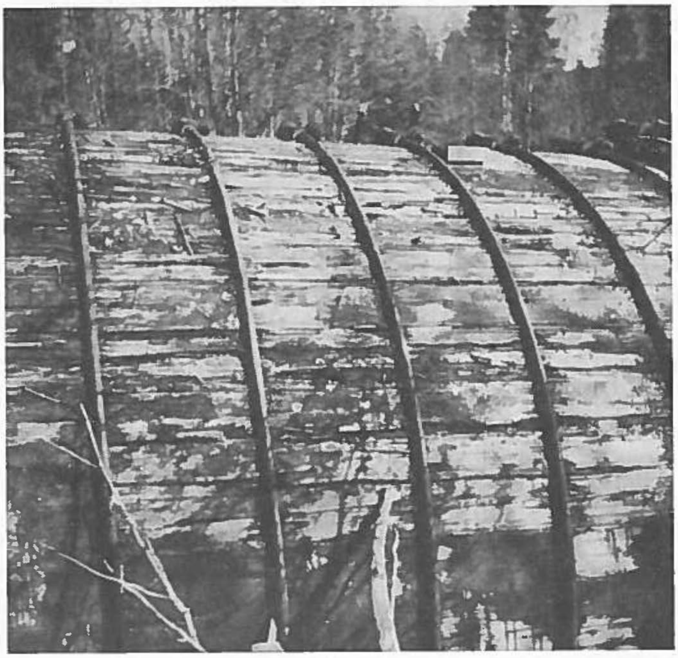

Fig. 33. Decayed part of the wood-stave pipe at the Äggfors Hydro-Electric Station. This pipe is about 35 years old. The staves are decayed under the bands to a depth not exceeding one inch. In this case, decay is probably due to two causes, viz., first, leaves, and the like, accumulate in the corners between the bands and the staves, and second, the contact pressure between the bands and the staves is unusually high.

made of staves impregnated by means of this method, and are correctly constructed in other respects, will be even longer than the above-mentioned figure of 45 years.

The Author has personally examined some wood-stave pipes which are to be regarded as characteristic in some measure. This examination has given the following results.

Äggfors Hydro-Electric Station. Two wood-stave pipes embedded in gravel to half their height. Diameter 3.4 m. Thickness of staves 4 in.

Head 3.2 to 7.4 m. Built in 1914 to 1915.

These pipes are in a very bad condition (1944), as may be seen from Figs. 16, 29, and 33. The staves are more or less generally decayed in the longitudinal joints to a depth of about one inch. So are the wooden collars connecting both ends of the wood-stave pipes to the steel pipes. Large leaks occur in both pipes. Nevertheless, the remaining internal part of the pipe wall, which is 4 in. thick, is sound, and continued use of the pipes is therefore not endangered. The pipes are provided with round bands, 37 mm in diameter, spaced 37 to 54 cm. According to American practice, the diameter of the bands in such a case should not exceed 7/8 in., and the distance between the bands should not be greater than 20 cm. As may be seen from Fig. 33, the bands have pierced the wood, apparently cut through the fibres, and given rise to decay. In the Author's opinion, it is possible that the decay of these pipes was primarily initiated by leaves, and the like, which have accumulated in the corners between the bands and the staves, and have caused rot, and hence destruction of the fibres. The pipes are surrounded by abundant vegetation, and are covered by leaves, mosse, and dirt. Moreover, the pipes are carried on a gravel, bed, and it is therefore remarkable that they have proved to last as long as 30 years.

When the Mörsil Power Station was built in 1949, both these pipes were pulled down. It was found that the degree of decay was less than had been expected. Decay was observed only in the surface of contact between the bands and the staves as well as in some longitudinal joints, but the depth of the decayed layer did not exceed about one inch.

Sikfors Hydro-Electric Station. Two wood-stave pipes embedded in broken stone to half their height. Diameter 3.2 m. Thickness of staves 3 in. Head 4.9 to 8.1 m. Built in 1912.

These pipes are in a very good condition, in spite of the fact that they are as old as 32 years (see Fig. 5 b). It is interesting to compare these pipes with the pipes at Äggfors, which are in a bad state. The design data and the ages of these pipes are nearly equal. The diameter and the spacing of the bands are also of the same order of magnitude. At Sikfors, the bands are 31 mm in diameter, and the distance between them is 35 to 60 cm. A few isolated staves have decayed to a negligible degree. The steel bands have not pierced the wood, and no decay comparable to that of the Äggfors pipes has occurred in this case. This confirms the assumption that the bands on the Äggfors pipes have cut through the fibres indirectly on account of decay. The reason why the pipes at Sikfors are in such a good condition, as compared with those at Äggfors, is probably to be found in the careful maintenance of the former. The vegetation in the neigbourhood of the pipes is regularly cut down within a distance of several tens of metres on both sides of the pipes, the bed is permanently kept free from plants and mould, and the bed material is occasionally washed. In contrast with the pipes at Äggfors, the Sikfors pipes exemplify the great importance of appropriate upkeep in preventing decay of wood-stave pipes.

Gideåbruk Hydro-Electric Station. The pipe is partly carried on supports and partly embedded to half its height. Diameter 3.0 m. Thickness of staves 3 1/2 in. Head 2.6 m. Built in 1913.

That section of the pipe which is embedded is in a relatively good condition, whereas the part carried on supports is in such a bad state that a complete reconstruction of the pipe is contemplated at the present time.[1]) The bad state of the pipe seems to be due to the fact that the pipe has been deformed because the distance between the supports has previously been too great (see Fig. 7). Decayed spots have been observed here and there at the crown of the pipe, but the depth of the decayed layer is very small (see Fig. 15).

Harrselsfors Hydro-Electric Station. This pipe, 3 m in diameter, was built as recently as in 1942, and is naturally in an excellent condition. The two older pipes are of greater interest in this connection. One of these pipes (2.0 m in diameter, head 10 m, built in 1916) has been covered with marshy earth in the 1920ies in order to provide heat insulation. In a few years, the pipe had decayed to such an extent that it proved necessary to remove the earth covering and to repair the pipe. At that time, the decayed layer of timber was 1 to 2 cm thick. This experience confirms the remarks made in the above concerning the harmfulness of covering wood-stave pipes with dirty earth.

Karlsnäs Hydro-Electric Station.[2]) Collar type pipe carried on supports. Diameter 2.20 m. Thickness of staves 3 in. Head 5.7 m. Built in 1899. The staves are slightly caved in at some supports. Wood specimens were taken from the wall by means of a core drill at several points along the pipe. Most staves were found to be perfectly sound. Decayed spots reaching to a depth of 1 cm were observed only in some joints between adjacent staves. Those staves which have been caved in were not decayed. On the other hand, the wedges

[1]) That part of the pipe which had been carried on supports was replaced in the autumn of 1947 by a new pipe, 3.5 m in diameter, which was embedded in broken stone.

[2]) This station is situated on the river Ronnebyån, and is owned by the Kockums Iron Works, Kallinge. The wood-stave pipe was examined by the Author in June, 1947.

under the bands and collars were decayed in several places. The fact that this pipe has decayed so little may be assumed to be largely due to careful maintenance. The pipe is coated with a mixture of asphalt and tar every year. At the same time, the wedges and the timber in the collar joints are overhauled and replaced, if required.

3. Conclusions Regarding Decay

Under Swedish conditions, the following conclusions can be drawn from operating experience regarding decay:

1. *Type of Wood.* Pine or spruce staves can be used. However, these two types of wood should not be employed in the same pipe.
2. *Quality of Timber.* It does not seem to be necessary to stipulate the use of heartwood. On the other hand, it is advisable to make sure that the timber is not too loose and does not originate from trees which have grown too rapidly. The timber shall be free from knots passing through the whole thickness of the staves and from edge defects. It may possibly be recommendable to stipulate that the staves shall be cut from the lower part of the trunk, in accordance with the Norwegian rules.
3. *Impregnation.* Under Swedish climatic conditions, no impregnation is generally required for preservation of timber. Impregnation with aspalt and tar to a depth of 1/16 in., as practised in the United States, is of course effective, but expensive. Creosote has proved to be a good preservative. Complete impregnation, e. g. with creosote or Boliden Salt (an arsenic salt), in vacuum and under pressure is perhaps a good method.
4. *Design and Construction.* In general, the pipes should be carried on supports, and should be entirely non-covered. If the pipe is to be covered with earth, only a material that is free from humus and is relatively watertight, preferably clay, should be used, at least in the immediate vicinity of the pipe. Earth beds reaching to half the height of the pipe should be avoided, but are nevertheless probably most suitable for large pipes from an economic point of view. In that case, it seems that the bed should be made of thoroughly drained, clean stone. The rule that vegetation, which gives rise to contamination of the pipe and the bed material, should be cut down in the neighbourhood of the pipe, is applicable to all wood-stave pipes.

B. Freezing[1])

A wood-stave pipe is less exposed to freezing than a steel pipe of equal diameter and length, if located in the same place. This has been found in many cases both in Sweden and in other countries. In one case, a 15 cm thick layer of ice has been observed in a steel pipe, whereas no ice coating whatever was present in the wood-stave pipe connected to it. The danger of freezing is often the decisive factor in determining whether non-covered pipes can be used at all. Steel pipes require often heat insulation in the form of earth covering, while this insulation can frequently be completely dispensed with, or its thickness can be reduced, in the case of wood-stave pipes since their walls are thicker and have a higher heat-insulating value. For instance, a 0.7 m thick earth covering has proved adequate for insulating a wood-stave pipe, 9 300 m in length, used for water supply to a town in Germany, whereas the calculated thickness of the earth covering for a steel pipe was 1.5 m (83).

Freezing is dependent on many factors, and it is desirable to elucidate their effects. To begin with, freezing should therefore be considered from a theoretical point of view.

1. Theory of Freezing Process

a. *Heat Transmission by Conduction and Convection.*

The process of freezing is in a very high degree influenced by the temperature of the water at the intake and the temperature of the air. When the weather is cold, the water flowing in the pipe is continuously cooled, and its temperature can become so low that an ice layer is formed on the inside of the pipe. The heat transmitted from the water through the pipe shell is $k \cdot (t - t_y)$, where $t =$ the temperature of the water, $t_y =$ the temperature of the air, and $k =$ the coefficient of heat transfer through the wall of the pipe. The difference in temperature is mostly due to external, non-controllable circumstances. On the other hand, the coefficient of heat transfer can be varied within fairly wide limits by various practical measures.

The total thermal resistance to heat transfer through the wall of the pipe is equal to the sum of the thermal resistance of the wall and the thermal resistances of the films on the outside and on the inside

[1]) This chapter is largely based on the investigation published by the Author in «Experience Regarding Wood-Stave Pipes» (78).

of the wall. Saturation of the wood with water and freezing have a very unfavourable effect on the thermal resistance. The thermal resistance of saturated timber is 40 per cent, and that of saturated and frozen timber (at — 15° C) is 20 per cent, of the thermal resistance of timber in an air-dry condition.

The thermal resistances of the films on the outside and on the inside of the wall are dependent on the velocities of flow of the air and the water respectively. The external film resistance varies from 0.20 m² · °C · hour per kcal in the absence of wind to, say, 0.04 m² · °C · hour per kcal at a wind velocity of 5 m per sec. The internal film resistance is so small that its influence on the heat transfer can be disregarded. On the other hand, this resistance practically determines how and where an ice layer is formed in the pipe. The internal film resistance is about 0.001 m² · °C · hour per kcal at a velocity of water flow of 2.0 m per sec, and 0.004 m² · °C · hour per kcal at a velocity of 0.2 m per sec.

b. *Freezing of Pipes.*

Freezing of pipes can be divided into three distinct phases, which are subjected to mathematical treatment in Appendix *F*. The first phase is the cooling of the water, the second phase is characterised by the presence of a thin ice layer, and the third phase consists in the formation of a thick ice layer. The temperature drop Δt of the water passing through the pipe is

(a) directly proportional to the difference $(t - t_y)$ between the temperature of the water and the temperature of the air,

(b) directly proportional to the external area of the pipe $L \cdot \pi \cdot D$,

(c) inversely proportional to the thermal resistance m per unit surface,

(d) inversely proportional to the rate of water flow Q. Consequently, we have

$$\Delta t = \text{const.} \frac{(t - t_y) \cdot L \cdot \pi \cdot D}{m \cdot Q} \tag{61}$$

In order to reduce the rate of cooling when the length of the pipe and the rate of water flow are given, the diameter of the pipe D should therefore be as small as possible. However, a definite economical value of the diameter is required in view of the losses of head, and the choice of the diameter may consequently necessitate a compromise.

If the velocity of water flow v is inserted in the expression for the temperature drop instead of the rate of water flow Q, we obtain

$$\Delta t = \text{const.} \frac{(t - t_y) \cdot 4 \cdot L}{m \cdot v \cdot D} \tag{62}$$

If we put $v \cdot \frac{D}{L} = v_s$, which we call the »specific velocity of water flow», we get

$$\Delta t = \text{const.} \frac{(t - t_y) \cdot 4}{m \cdot v_s} \tag{63}$$

The specific velocity of water flow can be defined as that velocity which produces in an imaginary unit pipe, 1 m in diameter and 1 m in length, the same temperature drop as in the actual pipe at a given difference in temperature and a given thermal resistance of the wall. This concept is used in what follows for the sake of simplification.

According to the theory of cooling, the temperature in the pipe decreases along the pipe in conformity with a practically linear law. If the temperature at the intake is low, the temperature of the water in the pipe gradually approaches zero. It is interesting to note, however, that water begins to turn into ice at a temperature which is higher than the freezing point. This is due to the presence of a sudden bend in the temperature gradient line at the boundary surface between the water and the wall of the pipe. A similar, but greater bend occurs at the boundary surface between the wall and the air.

That temperature at which water only just begins to turn into ice will be termed »the limit temperature» in what follows. The limit temperature for various pipes, velocities of water flow, and weather conditions can readily be determined by means of Fig. 34, in which t_l-t_y, i. e. the ratio of the limit temperature to the temperature of the air is plotted as a function of the velocity of water flow for varying thickness of the wall in the presence and in the absence of wind. The curves shown in Fig. 34 have been calculated from the relations deduced in Appendix *F*.

The ice layer formed on the inside of the pipe acts to a certain extent as heat insulation. The thickness of the ice layer continues to increase until equilibrium conditions are attained. Freezing of water liberates a considerable quantity of heat, and therefore takes

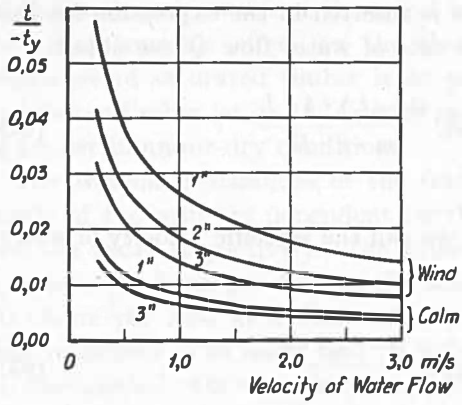

Fig. 34. Temperature ratio $-\frac{t}{t_y}$ at the beginning of freezing plotted against velocity of water flow, in metres per second, for varying thickness of pipe wall, in inches, and various wind conditions. The curves apply to a pipe diameter of 3 m. The values of the temperature ratio corresponding to a pipe diameter of 1 m amount to 70 per cent of those given in the diagram.

a relatively long time. In most cases, the ice layer does not become very thick. In order to examine this aspect of the problem, a detailed calculation has been made for a definite pipe. The results of this calculation are represented in Fig. 35. It is found that the temperature in the pipe decreases linearly to the limit point of 0.47° C. After that, the temperature curve asymptotically approaches zero as the distance from the intake becomes greater. Downstream of the limit point, the thickness of the ice layer increases at a relatively high rate, and reaches a value which is nearly constant over a long stretch in the lower section of the pipe. Within this stretch, the increase in thickness of the ice layer is constant, and is equal to 4.5 cm per day. Right downstream of the limit point, at a distance of 900 m from the intake, the increase in thickness of the ice layer is very small. In this part of the pipe, the thickness of the ice layer relatively quickly approaches a value which characterises the state reached after an infinite time. The temperature of the water in a given cross section of the pipe is almost constant in the course of the whole process of freezing.

c. *Heat Transmission by Radiation and Evaporation.*

The above calculations have been carried out on the assumption that heat transmission through a wall, in this case the wall of the pipe, is dependent only on the temperatures on both sides of the wall as well as on the total thermal resistance of the wall and the respective films. This assumption is commonly used in heat transfer calculations in building construction, but it is not quite in agreement with

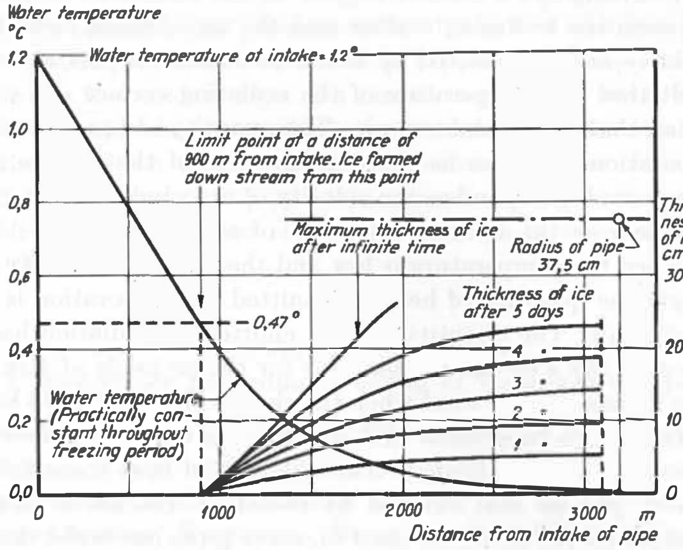

Fig. 35. Formation of ice in a wood-stave pipe. This diagram shows the results obtained by calculating the thickness of the ice layer and the temperature of the water along a wood-stave pipe at various instants after the beginning of freezing. The diagram shows that ice is formed only downstream of a definite point located at a distance of 900 m from the intake, and that the thickness of the ice layer increases at a relatively uniform rate throughout the greater part of the freezing section of the pipe. These results agree with many Swedish and foreign experiences. The water temperature is practically independent of the thickness of the ice layer, at least in this case. The calculations were made on the following assumptions, viz., air temperature —20° C, water temperature at intake +1.2° C, windy weather, and a constant water flow of 0.175 m³ per second. The pipe was taken to be of the same type as the pipe at Höljebro. If a pipe is so long that the water is cooled down close to the freezing-point, and ice is formed in a comparatively long portion of the pipe, it does not matter whether the pipe is still longer: the thickness of the ice in the pipe at any given moment will at all events be approximately constant beyond the frozen portion.

reality since the heat losses on the outside of the pipe are influenced by radiation and evaporation, although evaporation is generally of secondary importance. As a rule, convection on the outside of the pipe increases as radiation becomes smaller, and vice versa. Consequently, the sum of the quantities of heat transmitted by convection and by radiation remains roughly equal to the value obtained from the approximate calculation carried out in the preceding section, at least in the presence of wind. Heat radiation from the outside of the pipe is mostly dependent on the degree of cloudiness, but is also affected, although to a smaller degree, by the difference in temperature between the radiating surface and the air. During clear nights, the quantity of heat emitted by radiation reaches a maximum, with the result that the temperature of the radiating surface can even be lower than that of the ambient air. The quantity of heat transmitted by evaporation increases as the temperatures of the surface and of the air become higher, and as the velocity of the wind becomes greater, but decreases as the relative humidity of the air increases. In the winter, when the temperature is low and the relative humidity of the air is high, the quantity of heat transmitted by evaporation is therefore very small. The quantity of heat emitted by radiation has been computed at 158 kcal per m^2 per hour for a pipe made of 1 in. thick staves in the absence of wind when the sky is clear, and at 26 kcal per m^2 per hour for a pipe made of 4 in. thick staves in the presence of wind when the sky is clouded. The quantity of heat transmitted by convection, just as that emitted by radiation, reaches a maximum (260 kcal per m^2 per hour) for the 1 in. stave pipe, but under diametrically opposite weather conditions, i. e. in the presence of wind when the sky is cloudy. For the 4 in. stave pipe, the quantity of heat transmitted by convection decreases to 17 kcal per m^2 per hour in the absence of wind when the sky is clear. The quantity of heat transmitted by evaporation from the surface of a pipe which is frozen in the winter is insignificant, and may be taken not to exceed 4 per cent of the total amount of heat given off by the pipe. This figure is applicable on the assumption that the relative humidity of the air is 80 per cent, which corresponds to ordinary winter conditions. When the air is drier, the effect of evaporation is of course greater, but can nevertheless scarcely be supposed to be of any importance so long as the temperature of the air is low. Hence it follows that the quantity of heat removed from a 4 in. stave pipe in the absence of wind when the sky is clear is about the same as in the presence of wind when the sky is clouded. For the 1 in. stave pipe, on the other hand, the quantity of heat given off in the presence of wind when the sky is clouded is 43 per cent greater than in the absence of wind when the sky is clear, and about 13 per cent greater than the heat flow which is obtained from the ordinary approximate calculation which does not take into account the heat transmitted by radiation and evaporation. For a 4 in. stave pipe, the results of the ordinary simplified calculation are roughly in agreement with those of the exact calculation.

It follows from the above that the most dangerous case is met with in the simultaneous presence of wind and cold weather, but this case is not frequent. In Sweden, it is reported that the velocity of wind on the five coldest days of a month is generally lower than the monthly average. In any case, clear nights do not often occur at the same time as both windy and cold weather. As has already been mentioned, freezing of a pipe takes several days, and it is scarcely probable that these severe conditions continue to exist during such a long time.

2. Experience Regarding Freezing of Wood-Stave Pipes

Reference to experience concerning freezing of wood-stave pipes and to rules for designing these pipes with due regard to freezing are very scanty in the litterature (18, 50, 51, 52, 56, 59). In his recent publication »Experience Regarding Wood-Stave Pipes» (78), the Author has compared the theoretical conclusions in this respect with practical experience concerning 200 wood-stave pipes used in Sweden. Only the most important final results of this investigation are examined in what follows. Out of the total number of pipes covered by this investigation, 27 per cent have frozen to a greater or smaller extent. This shows that freezing is an important problem under Swedish climatic conditions.

In the statistical treatment of the material collected for the above-mentioned investigation, regard has been paid to the local variations in the temperature of the water and the air by dividing the pipes into groups according to favourable and unfavourable intake conditions and according to »northern» and »southern» climatic conditions.[1] No consideration has been given to the variations in the thermal resistance due to differences in thickness of staves, seeing that this thickness generally varied within very narrow limits, from 60 to 80 mm. The investigation has shown that the specific velocity of water flow v_s is a factor of paramount importance in determining whether a pipe is liable to freezing. The material collected for the previously mentioned investigation has been used as a basis for plotting the

[1]) The pipes situated in that region of Sweden where the mean temperature in January is lower than —5° C are reckoned in the »Northern Climate Group», whereas all other pipes are included in the »Southern Climate Group».

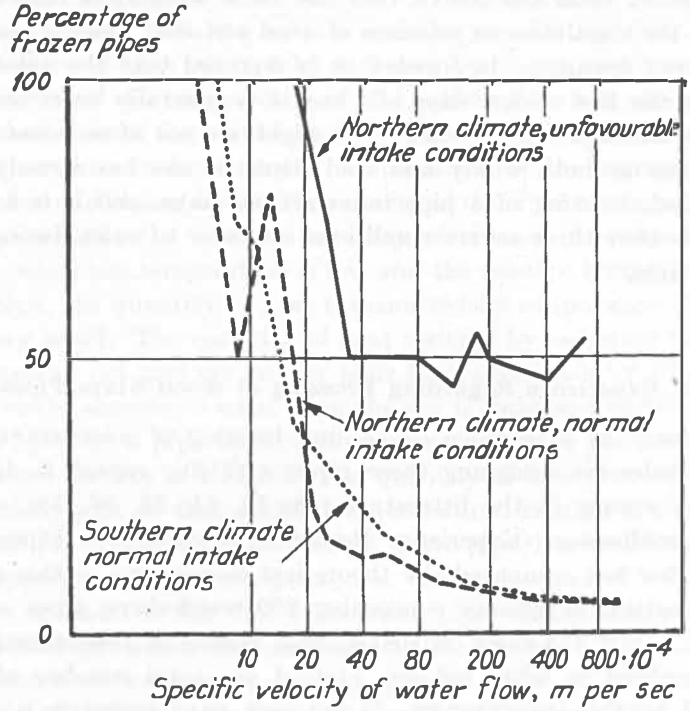

Fig. 36. Percentage of frozen wood-stave pipes as a function of specific velocity of water flow. This diagram is based on the material collected by the Swedish Water Power Association for an earlier investigation (78). The pipes are divided into three groups according to intake conditions and climatic conditions. In each group, the number of frozen pipes is expressed in per cent of the total number of pipes in which the specific velocity of water flow is less than a certain definite value. For instance, in the group »Southern Climate», 22 per cent of the total number of pipes in which the specific velocity of water flow is less than 0.004 m per sec have frozen in operation.

diagram shown in Fig. 36 which represents the percentage of heavily frozen pipes in relation to the total number of pipes in which the specific velocity of water flow is less than a certain definite value. This diagram has the character of a probability diagram showing the probability of freezing at various specific velocities of water flow. It is seen from Fig. 36 that the specific velocity of water flow should be higher than about 0.003 m per sec. under normal intake conditions.[1]) Under unfavourable intake conditions, e. g. when the intake is situated close to an open stretch of rapids, there can be a risk of freezing even if the velocity of the water is high. In such cases, it is necessary to ensure that the water upstream of the intake can calm down in a pond which is covered with ice in the winter. Under very severe conditions, a considerable improvement can be achieved by regulating the bed profile of the watercourse upstream of the intake. Of course, it is also possible to increase the thermal resistance of the wall of the pipe. This should primarily be done by providing shelter from wind, e. g. by building a plank wall which completely surrounds the pipe. This view is confirmed by experiences concerning the pipes at Shawinigan,[2]) which are enclosed in a concrete shell, and at Harrselsfors, where no ice layer has been formed in the pipes since they were completely surrounded by a plank wall.

The formation of an ice layer provides additional safety against clogging of the pipe by ice, first, because the ice constitutes a heat-insulating layer which progressively increases in thickness so long as the water continues to freeze, and second, because freezing involves a transformation of energy which is so considerable that it usually takes a long time until the ice layer reaches any notable thickness. In general, the thickness of the ice layer increases 25 to 50 mm per day.

These conclusions, which have been drawn from the diagram in Fig. 35 deduced by theoretical methods, are fully confirmed by experience relating to a pipe at Everett in the United States and

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[1]) This value is applicable to non-insulated pipes made of staves about 75 mm (3 in.) in thickness. If the heat insulation of the pipe is substantially improved by special measures, e. g. by building a plank wall around the pipe, the conditions become more favourable. In that case, the velocity of water flow may be lower than the value given in the above. The velocity of the water may be reduced to

$$v = \frac{0.13}{m} \frac{L}{D} v_s \qquad (64)$$

where m is the total thermal resistance (in m² °C hour per kcal) of the wall of the pipe and the heat insulation. In calculating this velocity, it is to be noted that the thermal resistance of wet timber is about 40 per cent of the resistance of air-dry timber.

[2]) The two steel pipes at Shawinigan, 4.25 m in diameter and 180 m in length, are heat-insulated by means of a thin concrete shell at a distance of 40 to 50 cm from the pipes. Experience shows that an ice layer, 30 to 40 cm in thickness, would have formed on the inside of the pipes if they had been operated at the full rate of water flow in the winter without this insulation (52).

to the pipes at Höljebro and Skutskär in Sweden. Swedish experience indicates that the risk of freezing can be reduced by locating the intake at a considerable depth below the water level since the water at a comparatively great depth is warmer, and is less mixed with ice crystals (66, 73).

3. Summary Regarding Freezing of Wood-Stave Pipes

It follows from the above that it is often very difficult to estimate the risk of freezing of wood-stave pipes. Under unfavourable intake conditions, wood-stave pipes have frozen even when they were appropriately constructed in other respects. Favourable intake conditions, i. e. a high temperature of the water at the intake, can be secured in several ways, e. g. by damming up the watercourse upstream of the intake, so as to obtain smooth flow of water. It is vary important to ensure that the watercourse is covered with ice in the winter as soon as possible. In particular, the stretch of the watercourse immediately upstream of the intake should flow smoothly, and should be covered with ice. Furthermore, as the water in the surface layers cools more quickly than in the deeper layers, it is advantageous to place the intake at some depth below the water surface, so as to obtain a higher water temperature in the winter.

TABLE 4. *Relation between Minimum Velocity of Water Flow and Diameter of Pipe.*

Diameter of Pipe	Minimum Velocity of Water Flow, m per sec, at Normal Rate of Water Flow Length of Pipe, m			
m	200	500	1 000	2 000
1.0	0.6	1.5	3.0	6.0
2.0	0.3	0.8	1.5	3.0
3.0	0.2	0.5	1.0	2.0
4.0	0.2	0.4	0.8	1.5

Under normal intake conditions in Sweden, the specific velocity of water flow in non-insulated wood-stave pipes of normal wall thickness should not be less than 0.003 m per sec. This implies that the velocity of water flow should not be smaller than the values given in Table 4 for various diameters and lengths of pipe.

The factors and practical measures which counteract freezing of wood-stave pipes can be summarised as follows.

1. Under normal intake conditions in Swedish climate, the specific velocity of water flow in wood-stave pipes should be higher than 0.003 m per sec.

2. Unfavourable intake conditions should be avoided or improved. The water should preferably be drawn from a pond which is covered with ice in the winter. Immediately upstream of the intake, there should be no long stretches of rapids which are open in the winter. An improvement of intake conditions can also be achieved by regulation of the watercourse.

3. If open stretches of flowing water cannot be avoided, the intake should be placed at a depth of at least a few metres below the surface of the water. In that case, it is to be expected that the thickness of the ice layer in the pipe will be reduced since the concentration of ice particles in deep water is less than at the surface.

4. If an old pipe is to be reconstructed and provided with heat insulation, it is in the first place advisable to equip the pipe with a shelter protecting it from wind. The shelter should completely enclose the pipe, and should be built at such a distance from the pipe as to permit inspection. If the empirical values given in the above indicate that a new pipe to be constructed may be expected to freeze to a considerable extent unless it is covered, the most radical solution is probably to cover the pipe with earth.

C. Losses of Head

1. Introduction

In the design of wood-stave pipes, it is necessary to take into account the losses of head, e. g. losses in gates, entrance losses, frictional losses, losses in bends and losses due to sudden expansion and contraction, and sometimes also losses of velocity head, e. g. at the inlet to the surge tower.

The loss of pressure head due to friction is dependent on the dimensions of the pipe and the roughness of the internal surface of the pipe. The coefficient of roughness of wood-stave pipes is general lower than that of iron, steel, and concrete pipes. According to Scobey (85), the carrying capacity of a wood-stave pipe is usually 15 per cent greater than that of a 10 years old cast iron pipe or a new riveted sheet steel pipe, and 25 per cent greater than that of a 20 years old cast iron pipe or a 10 years old riveted sheet steel pipe, the dimensions and the head being equal. Consequently, at a given rate of water flow, the frictional loss of head in a steel pipe, 10 and 20 years old, is 30 and 60 per cent greater, respectively, than in a wood-stave pipe of equal dimensions. Furthermore, it is important to note that the frictional loss of head in a steel pipe considerably increases in the course of time. It is reported that the carrying capacity of cast iron pipes, 0.75 m in diameter, decreases to about 60 per cent after 40 years of service. At a given rate of water flow, this implies that the frictional loss of head increases nearly three times. In wood-stave pipes, on the other hand, the loss of head due to friction is generally independent of the age of the pipe.

2. Formulae for Calculation of Frictional Loss of Head

Several formulae are available for the calculation of the frictional loss of head. The most common of these formulae are given below.

1. Chézy.

$$v = C \sqrt{RI} \qquad (65)$$

According to GANGUILLET and KUTTER,

$$C = \frac{\frac{1}{n} + 23 + \frac{0.00155}{I}}{1 + \left(23 + \frac{0.00155}{I}\right)\frac{n}{\sqrt{R}}}$$

where n is the coefficient of roughness of the pipe, I is the frictional loss of head, and R is the hydraulic radius. For wood-stave pipes, according to HORTON (39), n varies from 0.010 to 0.013.

2. HAZEN and WILLIAMS.

$$v = c\, 0.85\, R^{0.63}\, I^{0.54} \qquad (66)$$

For wood-stave pipes, $c = 120$ is regarded as an appropriate value (31).

3. SCOBEY.

$$v = 122\, R^{0.65}\, I^{0.556} \qquad (67)$$

4. MANNING.

$$v = M\, R^{2/3}\, I^{1/2} \qquad (68)$$

where $M = \frac{1}{n}$ (n = KUTTER's coefficient given in the above).

In spite of its intricate structure, CHÉZY's formula, with the value of C calculated according to GANGUILLET and KUTTER, is very common in the United States, and its use is facilitated by alignment charts. As regards wood-stave pipes, HAZEN and WILLIAM's formula is based on inadequate and irregular test data (the value of c varied from 111 to 152), whereas SCOBEY's formula is based on very comprehensive tests, and seems, therefore, to be particularly suited to wood-stave pipes. The greatest advantage of MANNING's formula (63) is the fact that it can be used for calculating the frictional loss of head both in pipes and in open conduits, but the coefficient M is slightly dependent on the velocity of water flow and on the hydraulic radius. HELLSTRÖM (33) and FORCHHEIMER (25) have pointed out that the exponent of I is greater than 0.50 as in MANNING's formula. As a general rule, this exponent varies inversely as the degree of roughness. SCOBEY's formula appears to be most reliable, but it seems that MANNING's formula is nevertheless to be preferred on account of its simple structure.

The valuable results obtained by SCOBEY ought to be applicable to MANNING's formula if the value of the constant M is appropriately chosen. If we compare SCOBEY's final formula with MANNING's equation, it can readily be demonstrated that

$$M = 84 \frac{v^{0.101}}{d^{0.082}} \tag{69}$$

Consequently, according to SCOBEY, when the velocity of water flow is 1 m per sec and the diameter of the pipe is 1 m, the corresponding value of M is 84. Moreover, it follows that M is approximately constant, so long as the value of v/D is invariable. This is illustrated by Table 5.

TABLE 5. *Values of M Resulting in the Same Frictional Loss of Head as* SCOBEY's *Formula When Inserted in* MANNING's *Formula.*

v/D	M	Per Cent
0.5	80	94
1.0	84	100
2.0	91	107
3 0	94	112

When the velocity of water flow (in m per sec) in a pipe is numerically equal to the diameter of the pipe (in m), that is to say when $v/D = 1$, SCOBEY's values correspond to $M = 84$ in MANNING's formula.

For HAZEN and WILLIAMS' formula, it can similarly be shown that the corresponding value of M also increases as the ratio of v/D becomes greater. For $v/D = 1$, $c = 120$ corresponds to $M = 81$.

Concerning MANNING's formula, it is furthermore important to note that, according to American practice, KUTTER's coefficient can be used directly since $M = \frac{1}{n}$. Therefore, the limit values of n given by HORTON (0.013 and 0.010) correspond to $M = 77$ and $M = 100$, according to the character of the internal surface of the pipe.

It follows from the above that the value of M is included in the interval from 80 to 90 under normal operating conditions, according to the unanimous opinion of SCOBEY, HAZEN and WILLIAMS, and HORTON (KUTTER's coefficient).

3. Observed Frictional Loss of Head

In connection with the stress measurements described in Chapter II, the Author has obtained some data on the frictional loss of head observed in the wood-stave pipes at Sikfors and Gideåbruk. These data are briefly summarised in what follows.

Sikfors.

The frictional loss of head was determined by levelling the water surface at two points, viz., close to the intake and at the surge tower. The distance between these points was 441 m. At the velocity of water flow of 3 m per sec, the observed frictional loss of head was 0.81 m corresponding to $M = 81$ (v/D was about 0.94). The water level in the surge tower has been observed by the staff of the hydro-electric station at various loads. The observed values have been reduced so as to take into account the velocity head and the entrance loss. It was found that the value of M varied from 78 at $v = 1.4$ m per sec. ($v/D = 0.45$) to 85 at $v = 2.8$ m per sec ($v/D = 0.9$).

Gideåbruk.

The Author has examined the results obtained from some measurements of the frictional loss of head which were conducted by Mr. E. RAMVALL, C. E. These measurements were made by levelling the water surface at both ends of the pipe. The distance between these points was 480 m. When the velocity of water flow v was 2 m per sec ($v/D = 0.7$), M was 73, and at $v = 3.1$ m ($v/D = 1.0$), M was 75. Thus, the values of M were slightly lower than at Sikfors, probably because the pipe was partly in a very bad condition.

4. Conclusions Regarding Loss of Head

On the whole, the results of the measurements made at Sikfors confirm the correctness of the values of the coefficient M in MANNING's formula given above. On the other hand, the value of M obtained from the measurements at Gideåbruk was about 10 per cent lower. This indicates that the value of M can be slightly below normal when the pipe is in a bad condition from a hydraulic point of view.

From the above we can conclude that the value of M varies within the approximate limits from 70 to 90 under normal conditions (v/D between 0.5 and 2.0). On the basis of this conclusion, Table 6 gives the probable values of M in relation to the ratio of the velocity of water flow to the diameter of the pipe and the condition of the interior of the pipe from a hydraulic point of view.

TABLE 6. MANNING'S *Coefficient M for Wood-Stave Pipes.*

v/D Per Sec.	MANNING'S Coefficient M Hydraulic Condition of Pipe		
	Bad	Normal	Good
0.5	71	76	81
1.0	75	80	85
2.0	81	87	92
3.0	84	90	95

IV. Economic Factors

A. General

Wood-stave pipes are generally more economical than sheet steel, cast iron, and concrete pipes.

The advantages of wood-stave pipes are:

1. Low initial cost of wood-stave pipe.
2. Low cost of transport.
3. Simple and rapid assembly.
4. Easy adaptation to the profile of the ground. Curved sections of continuous stave pipes can be built without using any special devices.
5. The interior of wood-stave pipes is not liable to attack by adulterants, such as carbonic acid or other acids. In fact, these pipes are specially used for conveying acids.
6. Small frictional loss of head, and hence greater carrying capacity.
7. If correctly constructed, great length of life; practically as great as that of sheet steel and cast iron pipes.
8. Greater safety against freezing.
9. Insensitivity to temperature stresses.

The drawbacks of wood-stave pipes are:

1. Sensitivity to external pressure caused by earth covering or due to surges.
2. Greater leakage than in other pipes.
3. Risk of decay.
4. Cannot be operated when half-filled (considering the danger of decay).
5. The earth used for covering embedded pipes should be relatively watertight.

Most of the advantages and drawbacks enumerated above have already been dealt with in this book. In what follows, we shall therefore confine ourselves to some considerations concerning the economic factors to be taken into account in the design of wood-stave pipes.

Wood-stave pipes are naturally most economical at low water pressures, whereas iron and steel pipes are most advantageous when the pressure exceeds a certain definite limit value. This limit is of course dependent on the prices of iron and steel, but it increases as timber becomes cheaper in comparison with iron. The cost of transport is also of importance. In 1924, the economical limit pressure was about 60 to 70 m (4, 62). This figure was applicable to pipes from 0.5 to 1.0 in diameter. For larger diameters, the limit of head is lower.[1]) In general, a prefabricated wood-stave pipe ready for assembly is considerably cheaper than an iron or steel pipe (about 50 to 75 per cent of the cost of the latter). If the wood-stave pipe is completely finished, this difference is smaller.

In the design of wood-stave pipes, the diameter of the pipe should be determined so as to reduce the annual cost to a minimum, while taking into account the loss of head. This diameter, which is termed »the most economical diameter», is generally larger for wood-stave pipes than for iron and steel pipes on account of the lower cost of the wood-stave pipes. The most economical diameter can be calculated as follows.

B. Calculation of Most Economical Diameter

If the initial cost of a wood-stave pipe is assumed to be a function of the diameter, the most economical diameter[2]) can be determined by purely mathematical methods. The problem is simplified if we assume that the initial cost is directly proportional to the diameter. Experience shows that this assumption is approximately in agreement with the actual conditions, at least so long as the head is not too high.

The annual cost k, in per cent, is taken to include depreciation, interest, and maintenance charges. The maintenance charge may be supposed to be 1 per cent.

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[1]) The maximum head that is practically possible is much higher. According to WHITE (97), this limit of head is about 75 m for a pipe 3.6 m in diameter, and is of course higher for pipes of smaller diameter.

[2]) The length of the pipe is assumed to be so great that the entrance losses, etc., are negligible in comparison with the frictional loss of head, and that the cost of the intake, the surge tower, etc., is relatively independent of the diameter or is small in comparison with the cost of the pipe.

a. The annual cost per metre of pipe is

$$\frac{k}{100}\,a\,D \tag{70}$$

b. If the loss of head is calculated from MANNING's formula, the loss of power per metrė is

$$8\,Q\,I = \frac{82.4\;Q^3}{M^2\;D^{16/3}} = \text{const.}\cdot Q^3\;\text{kW per m} \tag{71}$$

If the rate of water flow is constant, it is directly inserted in the above formula. On the other hand, if the rate of water flow is variable, and is given by the values

Q_1 during the period of time T_1,

Q_2 during the period of time T_2, etc.,

and if the total time of operation is $T_1 + T_2 + \ldots = T_{tot}$, then the loss of power is

$$\text{const.}\;\frac{Q_1^3\,T_1 + Q_2^3\,T_2 + \ldots.}{T_{tot}}$$

The following calculation is facilitated if we use a constant rate of water flow Q_m which gives the same loss of power on the average during the period T_{tot}. We obtain

$$Q_m = \sqrt[3]{\frac{Q_1^3\,T_1 + Q_2^3\,T_2 + \ldots.}{T_{tot}}} \tag{72}$$

If the average price of power[1]) is ω Swedish kronor per kW per year, the annual cost of the loss of power is

$$\omega\,\frac{82{,}4\;Q_m^3}{M^2\;D^{16/3}}\;\text{Swedish kronor per m} \tag{73}$$

[1]) Special regard must sometimes be paid to the seasonal variations in the price of power.

7

Consequently, the total annual cost A is the sum of the costs given by Eqs. (70) and (73)

$$A = \frac{k}{100}\, a\, D + \omega\, \frac{82.4\, Q_m^3}{M^2\, D^{16/3}} \tag{74}$$

Now the diameter D is to be determined so that A should be a minimum, that is to say,

$$\frac{d\,A}{d\,D} = 0$$

By solving the above equation, we obtain the most economical diameter D in metres

$$D = \left(\frac{44\,000\,\omega}{k\,a\,M^2}\right)^{3/19} Q_m^{9/19} \tag{75}$$

From Eq. (71) we can also compute the corresponding velocity of water flow, i. e. the most economical velocity v in m per sec

$$v = \left(\frac{k\,a\,M^2}{21\,300\,\omega}\right)^{1/3} D^{1/9} \tag{76}$$

By inserting the mean value $M = 80$ in conformity with the foregoing chapter, we obtain

$$D = \left(6,9\,\frac{\omega}{k\,a}\right)^{3/10} Q_m^{9/19} \tag{77}$$

$$v = \left(0.3\,\frac{k\,a}{\omega}\right)^{1/3} D^{1/9} \tag{78}$$

Since the values of D are included within the range from 0.25 m to 5.0 m, the term $D^{1/9}$ does not vary considerably from unity. For design purposes, we can put approximately

$$v_{\text{appr}} = \sqrt[3]{0.3\,\frac{k\,a}{\omega}} \tag{79}$$

At a given rate of water flow, this velocity corresponds to a definite diameter of the pipe. If this diameter is known, the value of the velocity can be adjusted, if desired, through multiplication by the factor $D^{1/9}$ which is tabulated below.[1]) For instance, if the diameter of the pipe is 4 m, the approximate value of the velocity of water flow should be increased by 17 per cent.

C. Summary Regarding Economic Factors

It follows from the above that the most economical velocity of water flow is dependent on the annual cost k expressed in per cent of the initial cost of pipe,[2]) the factor a in the expression for the initial cost of pipe $a \cdot D$ Swedish kronor per metre, and the price of power ω Swedish kronor per kW per year, but it is only slightly influenced by the diameter of the pipe. The value of a increases as the pressure head becomes greater. At the present time, the magnitude of a is of the order of 100 to 150 Swedish kronor per metre length of pipe per metre of diameter at low heads, but should be increased as the head becomes greater. The increase in a, expressed in per cent, is approximately equal to the increase in head, expressed in metres. If we assume that a is 150 and that the price of power is 50 Swedish kronor per kW per year, we obtain

$$v_{\text{appr}} = \sqrt[3]{0.9\, k} \tag{80}$$

If the pipe will be in use 40 years and if the rate of interest is 4 per cent, we get $k = 6.1$ and $v_{\text{appr}} = 1.77$ m per sec. Hence we

[1]) The factor $D^{1/9}$ varies with the diameter as follows

D in m	0.25	0.5	1	2	3	4	5	6
$D^{1/9}$	0.86	0.92	1.00	1.08	1.13	1.17	1.20	1.22

[2]) The annual cost k, inclusive of 1 per cent maintenance charge, can be obtained from the following table.

Rate of Interest Per Cent	Annual Cost in Per Cent of Initial Cost Numbers of Years of Use of Pipe					
	10	20	30	40	50	60
3	12.7	7.7	6.1	5.3	4.9	4.6
4	13.3	8.4	6.8	6.1	5.7	5.4
5	14.0	9.0	7.5	6.8	6.5	6.3

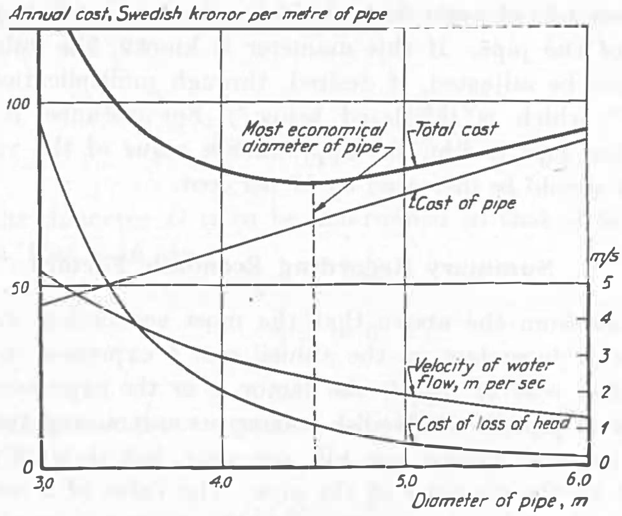


Fig. 37. Variation in total cost per metre of wood-stave pipe, i. e. initial cost plus loss of head expressed in terms of cost of power, with diameter of pipe at rate of water flow of 38 m³ per sec at specified prices of power and pipe materials. The lowest total cost is obtained when the diameter of the pipe is 4.5 m.

obtain the following relation between the most economical velocity of water flow and the diameter of the pipe

Diameter, m	0.5	1	2	3	4	5
Velocity, m per sec	1.65	1.77	1.91	2.00	2.07	2.12

On the other hand, if we have to *design* a pipe for a given rate of water flow, say 20 m³ per sec, the procedure is as follows.

$v_{\mathrm{appr}} = 1.77$ m per sec, as given above.

Cross-sectional area $= \dfrac{20}{1.77} = 11.3$ m². Hence $D_{\mathrm{appr}} = 3.8$ m.

From the table in Footnote 2, p. 99, we obtain $D_{\mathrm{appr}}^{1.9} = 1.16$, and therefore $v = 1.16 \cdot 1.77 = 2.06$. This velocity corresponds to $D = 3.5$ m.

Fig. 37 gives an example showing the variation in total cost with the diameter of the pipe. It is seen from this diagram that the total cost curve is comparatively flat in the neighbourhood of the minimum. Consequently, small errors in the determination of the diameter have no appreciable effect on the total cost.

The calculations carried out in this section are based on some simplified assumptions regarding the price of power, MANNING's coefficient M ($= 80$), and the cost of pipe. The price of power may of course be slightly higher or lower, but the value of 50 Swedish kronor per kW per year seems to be an appropriate average for large power companies at the present time. Small companies often sell power at a higher price. As has been shown on p. 94, MANNING's coefficient may vary from 70 to 90. In other words, the possible variations in the most economical velocity of water flow calculated on the assumption that M is equal to 80 do not exceed 10 per cent. After determining the most economical diameter, it is of course necessary to check the correctness of the assumed cost of pipe. For this purpose, all those items of the initial cost which vary with the diameter must be calculated in detail. However, the method of design outlined in this section may be considered to be fully satisfactory for long pipes on the assumptions made in the above. The result indicates the interesting circumstance that the most economical velocity of water flow in wood-stave pipes is more or less constant, irrespective of the diameter of the pipe.

After having made this economic calculation, it is furthermore necessary to take into consideration other factors influencing the design, such as the risk of freezing, which may sometimes call for an increase of the velocity of water flow, and the surge conditions and internal wear, which determine the maximum allowable velocity.

V. Modern Wood-Stave Pipes

In order to demonstrate the possibilities of wood-stave pipes at the present time and to illustrate the types used by various manufacturers, a description is given in this chapter of the following pipes.

A. Pressure pipe at the Dammfallet Hydro-Electric Station.
B. Pressure pipe at the Furudal Hydro-Electric Station.
C. Pressure pipe at the Kalltorp Hydro-Electric Station.
D. Water supply main from Tisaren to the Kvarntorp Shale Oil Plant.
E. Water supply main from Nolby to the Stockvik Works.
F. Water supply main of the Oskarström Sulphite Mills.
G. Water supply main of the Älvenäs Cellulose Wool Factory.
H. Pressure pipe at the Torsby Hydro-Electric Station.
I. Group stave pipe with covered end joints.
J. Shoes, bands, and end joints manufactured by the Boxholm Co., Boxholm, Sweden, and the Skandinavisk Traerör A/S, Oslo, Norway.
K. Wood-stave pipe manufactured by the AB Armerade Trärör, Gothenburg, Sweden.

A. Pressure Pipe at the Dammfallet Hydro-Electric Station

Manufacturer . . .	Boxholms AB.
Owner	Svenska Kullagerfabriken (SKF), Hofors.
Built in	1942 (Replaced an older pipe built in 1894).
Principal data . . .	Length 650 m. Internal diameter 2.0 m. Head 4 to 34.5 m, including an allowance of 50 per cent for surges. Non-covered, carried on supports, distance from centre to centre 3.2 m and 2.9 m. Group stave type. Velocity of water flow 2.4 m per sec.

Fig. 38.

Staves . . . . Pine heartwood staves, 2 1/2 in. by 6 in. (63 mm by 150 mm), with triangular tongue-and-groove joints. Heartwood side facing outwards. End joints provided with copper plates, 30 mm by 3 mm, and covered with 2 mm thick tar board and flat iron hoops, 90 mm by 2.5 mm, held together by 2 bands.

Bands . . . . Two-part special rolled section bands, 24 mm by 12 mm. Distance from centre to centre 81 to 290 mm. Steel grade St 37. One part of the band is upset and threaded (B. S. W. 7/8 in.) over a length of 130 mm at both ends. The other part of the band is upset at both ends so as to form heads. The bands are coated with red lead paint.

Shoes . . . . Centric type. Made of 9.5 mm thick hot-bent sheet steel. Length 140 mm. Rust-proofed by tar treatment.

Supports . . . . . . Concrete supports. Distance from centre to centre 3.2 m in the straight section and 2.9 m in the curved section.

B. Pressure Pipe at the Furudal Hydro-Electric Station

Manufacturer . . .	Boxholms AB.
Owner	Korsnäs Sågverks AB.
Built in	1943.
Principal data . . .	Length 1 670 m. Internal diameter 2.4 m. Head 14 m. (Static head). The greater part of the pipe, 1 640 m in length, is laid on a bed and covered with earth. The remaining section of the pipe is non-covered and carried on supports. Group stave type. Velocity of water flow 1.9 m per sec.
Staves	Pine staves of good quality, 2 1/2 in. by 7 in. (63 mm by 175 mm), with triangular tongue-and-groove joints. Heartwood side facing outwards. Impregnated on the outside with a mixture of tar and creosote. End joints provided with copper plates, 26 mm by 2 mm, and covered with 2 mm thick tar board and flat iron hoops, 90 mm by 2.5 mm, held together by 2 bands.
Bands	Round bands, 7/8 in. in diameter. Distance from centre to centre 250 mm. Steel grade St 44. The bands are upset so as to form a head at the one end, and are threaded at the other end.
Shoes	Centric type. Made of 9.5 mm thick hot-bent sheet steel. Length 250 mm. Rust-proofed by tar treatment.
Supports	Concrete supports. Distance from centre to centre 3.15 m. Thickness 260 mm. The arc of contact between the pipe and the supports is equal to 40 per cent of the circumference of the pipe.

Fig. 39 a.

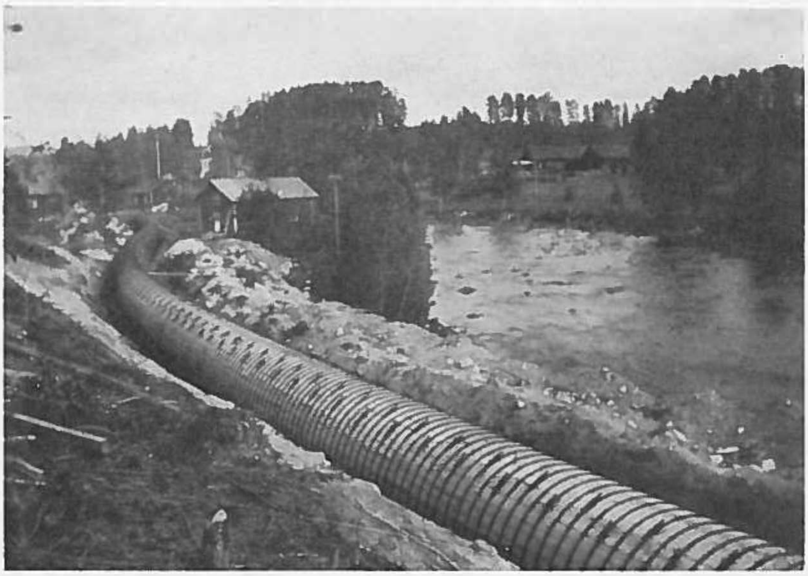


Fig. 39 b.

C. Pressure Pipe at the Kalltorp Hydro-Electric Station

Manufacturer . . .	AB Armerade Trärör.
Owner	Sjuntorps AB, Sjuntorp.
Built in	1944 to 1945.
Principal data . . .	Length 150 m. Internal diameter 1.0 m. Head 5 to 18 m. Non-covered, carried on supports, distance from centre to centre 3.5 m. Continuous stave type.
Staves	Pine staves, 2 1/2 in. by 6 in. (63 mm by 150 mm), with triangular tongue-and-groove joints. The timber is impregnated with Boliden Salt (an arsenic salt). End joints provided with galvanised iron plates, 25 mm by 2 mm.
Bands	Flat bands, cross-sectional area 2 cm^2, distance from centre to centre 143 mm. One end of the band is upset and threaded (B. S. W. 5/8 in.). The other end is welded to the shoe. Coated with anti-corrosive paint.
Shoes	Centric type. Made of 8 mm thick sheet steel. One end of the band is inserted in the shoe and fillet-welded.
Supports	Concrete supports. Distance from centre to centre 3.5 m.

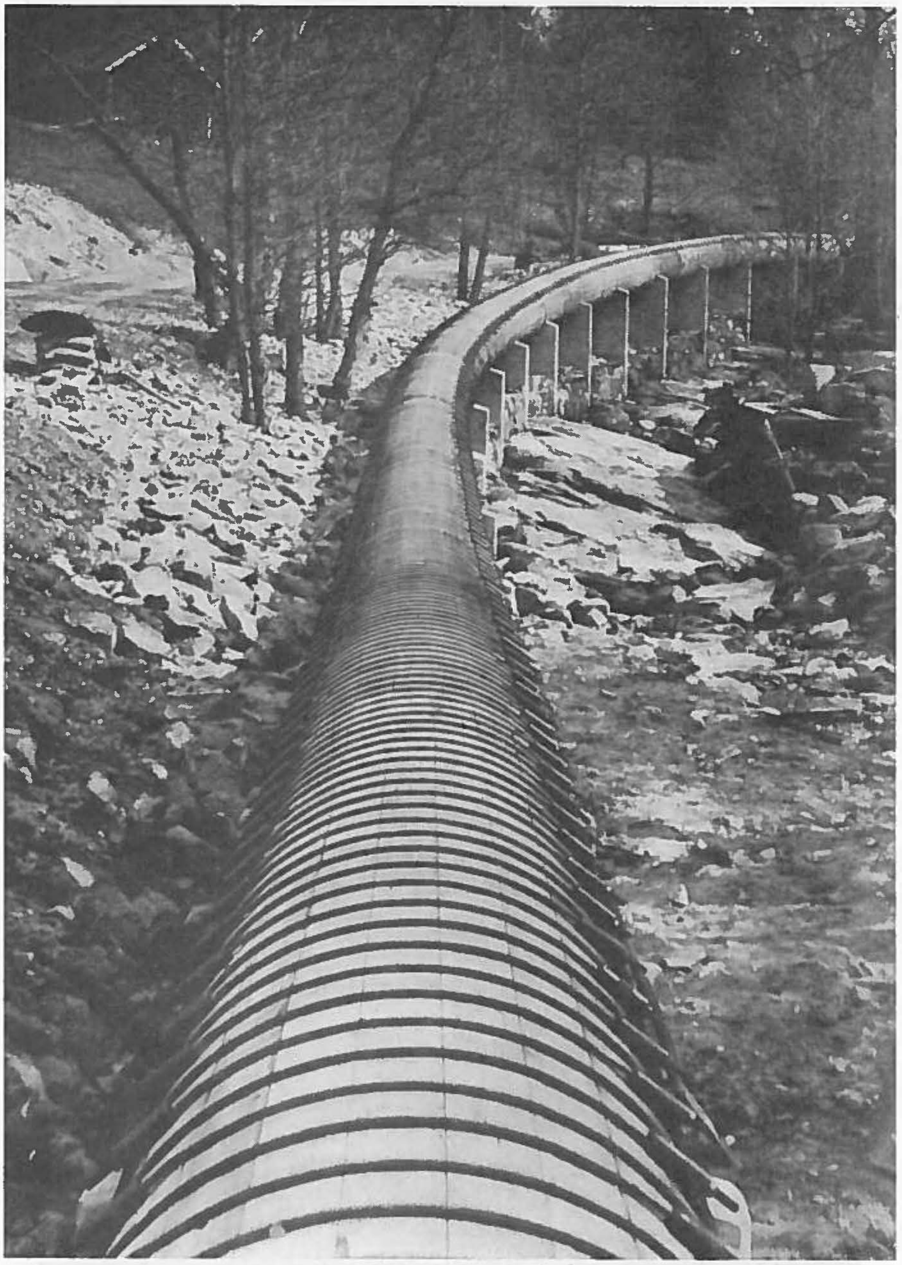


Fig. 40.

D. Water Supply Main from Tisaren to the Kvarntorp Shale Oil Plant

Manufacturer . . .	Boxholms AB.
Owner	Svenska Skifferolje AB.
Built in	1946.
Principal data . . .	Length 11 940 m. Internal diameter 0.6 m. Maximum head 40 m. One part of the pipe, 9 440 m in length, is non-covered and carried on supports, distance from centre to centre 4.4 m. The other part, 2 500 m in length, is laid on a bed and covered with earth. Group stave type. Velocity of water flow 1.25 m per sec.
Staves	Pine heartwood staves, 2 in. by 4 1/2 in. and 2 in. by 5 in. (50 mm by 113 mm and 50 mm by 125 mm), with triangular tongue-and-groove joints. Heartwood side facing outwards. End joints provided with galvanised sheet iron plates, 30 mm by 3 mm, and covered with 2 mm thick tar board and flat iron hoops, 75 mm by 2 mm, held together by 2 bands.
Bands	Special rolled section bands, 16 mm by 8.25 mm. Distance from centre to centre 59 to 250 mm. Standard steel St 44. One end of the band is upset and threaded (B. S. W. 5/8 in.) over a length of 110 mm. The other end is upset so as to form a head. Rust-proofed by tar treatment.
Shoes	Centric type. Made of 7 mm thick hot-bent sheet steel. Length 130 mm. Rust-proofed by tar treatment.
Supports	Iron supports on concrete bases. Distance from centre to centre 4.4 m. Thickness 150 mm. The arc of contact between the pipe and the supports is equal to 40 per cent of the circumference of the pipe.

Fig. 41 a.

Fig. 41 b.

E. Water Supply Main from Nolby to the Stockvik Works

Manufacturer . . .	Boxholms AB.
Owner	Stockholms Superfosfat Fabriks AB.
Built in	1944.
Principal data . . .	Length 5 407 m. Internal diameter 0.75 m. Head 55 m. Carried on supports, distance from centre to centre 4 m, and covered with earth. Group stave type. Velocity of water flow 1.1 m per sec.
Staves	Pine heartwood staves, 2 in. by 4 1/2 in. (50 mm by 113 mm) and 2 in. by 5 in. (50 mm by 125 mm), with triangular tongue-and-groove joints. Heartwood side facing outwards. End joints provided with stainless steel plates, 26 mm by 2 mm, and covered with 2 mm thick tar board and flat iron hoops, 80 mm by 2 mm, held together by 2 bands.
Bands	Special rolled section bands, 20 mm by 10.5 mm. Distance from centre to centre 87 to 238 mm. Standard steel St 44. One end of the band is upset and threaded (B. S. W. 3/4 in.) over a length of 120 mm. The other end is upset so as to form a head. Rust-proofed by tar treatment.
Shoes	Centric type. Made of 11 mm thick hot-bend sheet steel. Length 150 mm. Rust-proofed by tar treatment.
Supports	Concrete supports. Distance from centre to centre 4 m. Thickness 160 mm. The arc of contact between the pipe and the supports is equal to 40 per cent of the circumference of the pipe.

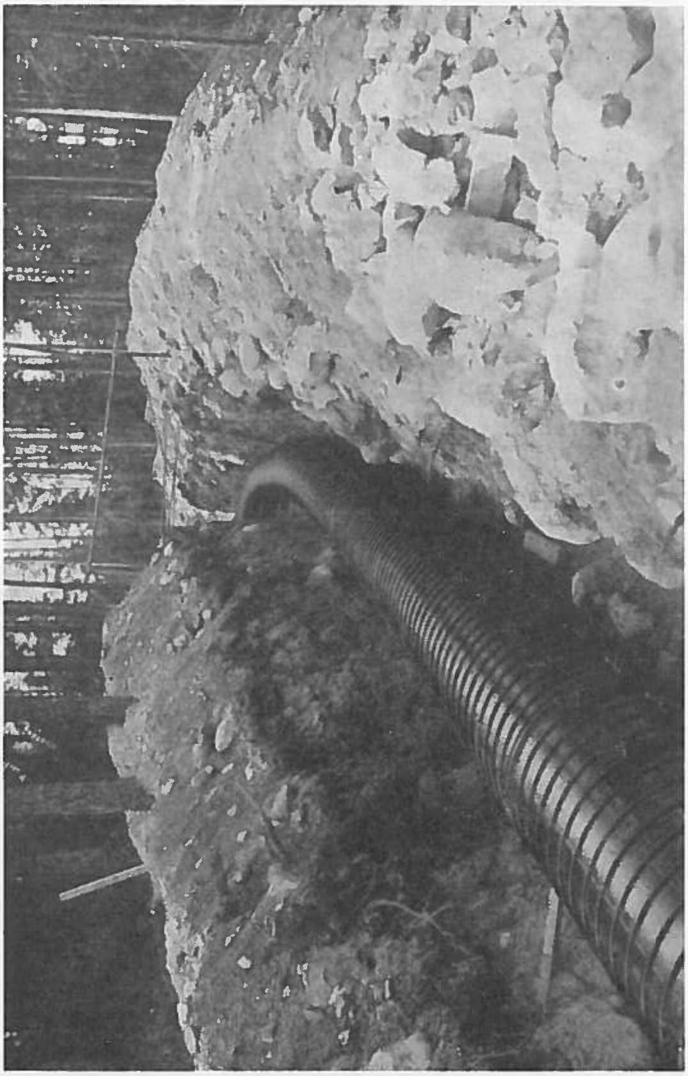


Fig. 42.

F. Water Supply Main of the Oskarström Sulphite Mills

Manufacturer . . .	AB Armerade Trärör.
Owner	Oskarström Sulphite Mills AB.
Built in	1934.
Principal data . . .	Length 280 m. Internal diameter 0.9 m. Head 4 m. Non-covered, carried on supports, distance from centre to centre 1 m. Continuous stave pipe.
Staves	Pine staves, 2 1/2 in. by 6 in. (63 mm by 150 mm), with triangular tongue-and-groove joints. End joints provided with galvanised iron plates, 25 mm by 2 mm. Coated with Inertol on the outside.
Bands	Round bands, 5/8 in. in diameter. Distance from centre to centre 300 mm. Both ends are threaded. Rust-proofed by tar treatment.
Shoes	Eccentric type. Made of cast iron. Rust-proofed by tar treatment.
Supports	Wood supports carried on steel beams. Distance from centre to centre 1 m.

Fig. 43.

G. Water Supply Main of the Älvenäs Cellulose Wool Factory

Manufacturer . . . AB Armerade Trärör.
Owner . . . . AB Cellull.
Built in . . . . 1942 and 1943.
Principal data . . . Length 320 m. Internal diameter 0.6 m. Head 20 m. This pipe is immersed in water. Continuous stave type.
Staves . . . . Pine staves, 2 in. by 5 in. (50 mm by 125 mm), with triangular tongue-and-groove joints. End joints provided with galvanised iron plates, 25 mm by 2 mm.
Bands . . . . Round bands, 5/8 in. in diameter, distance from centre to centre 150 mm. Both ends are threaded. Rust-proofed by tar treatment.
Shoes . . . . Eccentric type. Made of cast iron. Rust-proofed by tar treatment.

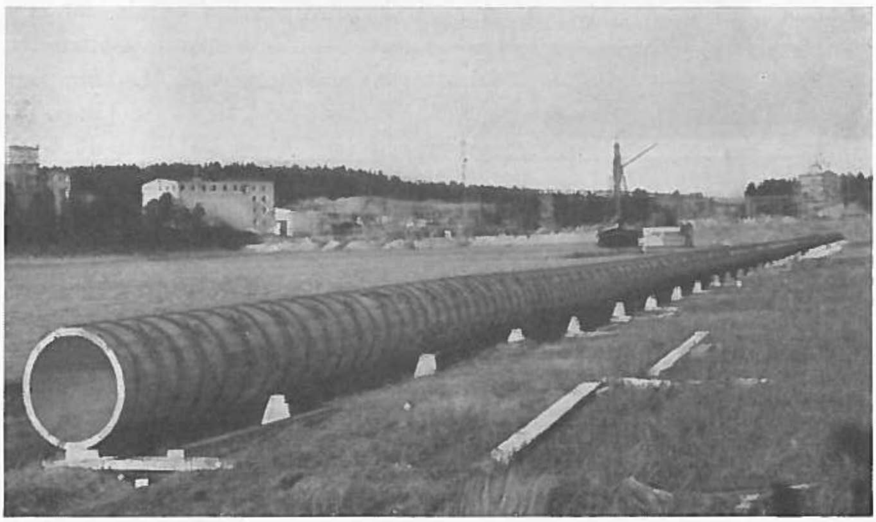


Fig. 44.

H. Pressure Pipe at the Torsby Hydro-Electric Station

Manufacturer . . .	Boxholms AB.
Owner	Billeruds AB.
Built in	1940.
Principal data . . .	Length 106 m. Internal diameter 2.0 m. Head 7.5 to 15 m, including an allowance of 50 per cent for surges. Non-covered, carried on supports, distance from centre to centre 3.4 m. Group stave type. Velocity of water flow 2.0 m per sec.
Staves	Pine heartwood staves, 2 1/2 in. by 5 1/2 in. (63 mm by 138 mm), with triangular tongue-and-groove joints. Heartwood side facing outwards. Impregnated on the outside with a mixture of tar and Carbolineum (a carbolic acid product). End joints provided with copper plates, 30 mm by 3 mm, and covered with tar board, 2 mm in thickness, and flat iron hoops, 80 mm by 2.5 mm, held together by 2 bands.
Bands	Two-part special rolled section bands, 20 mm by 10.5 mm. Distance from centre to centre 100 mm. Steel grade St 44. One part of the band is upset and threaded (B. S. W. 3/4 in.) over a length of 100 mm at both ends. The other part of the band is upset at both ends so as to form heads. The bands are coated with red lead paint.
Shoes	Centric type. Made of hot-bent 8 mm thick sheet steel. Length 125 mm. Rust-proofed by tar treatment. Concrete supports. Distance from centre to centre 3.4 m.

Fig. 45.

I. Group Stave Pipe with Covered End Joints

Manufacturer . . .	Boxholms AB. Fig. 46 shows a pipe sample presented by the manufacturer to the Institution of Hydraulic Engineering at the Royal Institute of Technology, Stockholm. This pipe sample is used for instruction.
Principal data . . .	Internal diameter 0.6 m.
Group joints	Length of joint 1.1 m. Each group consists of 9 staves.
Staves	Pine heartwood staves, 42 mm by 115 mm, with triangular tongue-and-groove joints. End joints provided with copper plates, 30 mm by 3 mm.
Bands	Special rolled section bands, 16 mm by 8 mm. One end of the band is upset and threaded (B. S. W. 5/8 in.). The other end is upset so as to form a head. Rust-proofed by treatment.
Shoes	Centric type. Made of hot-bent sheet steel. Length 130 mm.

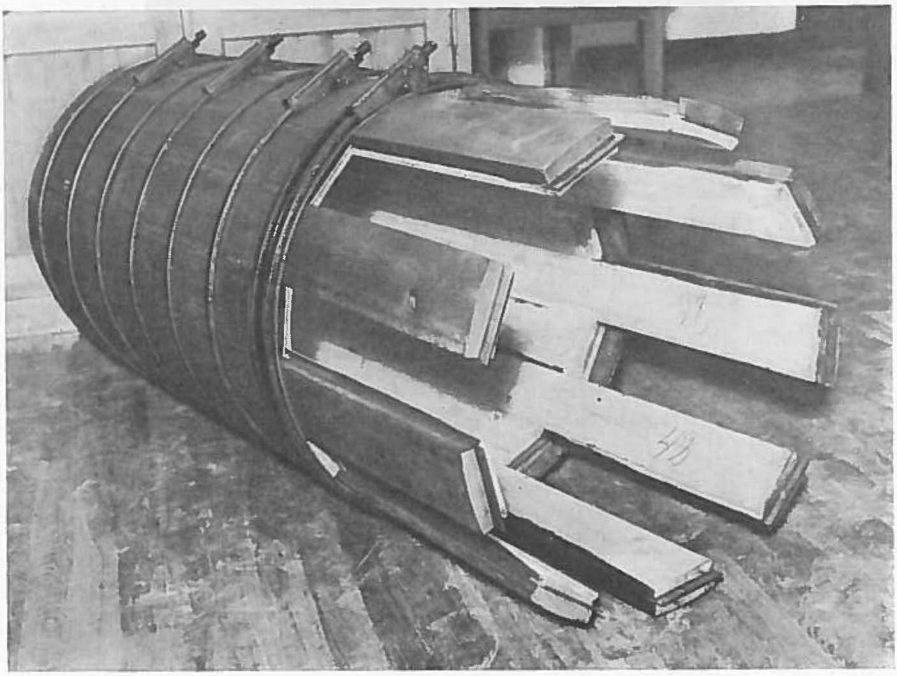


Fig. 46.

J. Shoes, Bands, and End Joints Manufactured by the Boxholms AB, Boxholm, Sweden, and the Skandinavisk Traerör A/S, Oslo, Norway

Shoe.

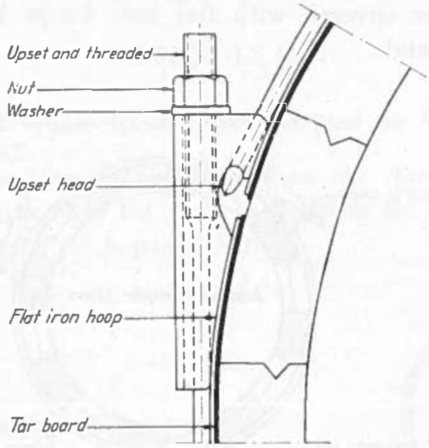


Fig. 47.

End joint with flat iron hoop and special rolled section band.

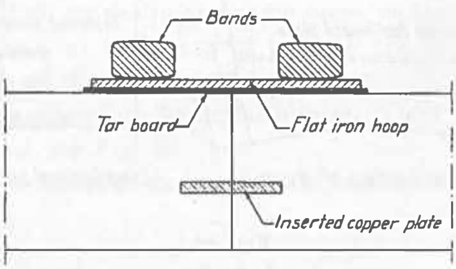


Fig. 48.

K. Wood-Stave Pipe Manufactured by the AB Armerade Trärör, Gothenburg, Sweden

This example shows a wood-stave pipe for water supply systems.

The pipe is supplied in prefabricated units from 7 to 9 m in length. The pipe units are fitted together by means of group stave joints. All end joints are covered with flat iron hoops held together by ordinary round bands.

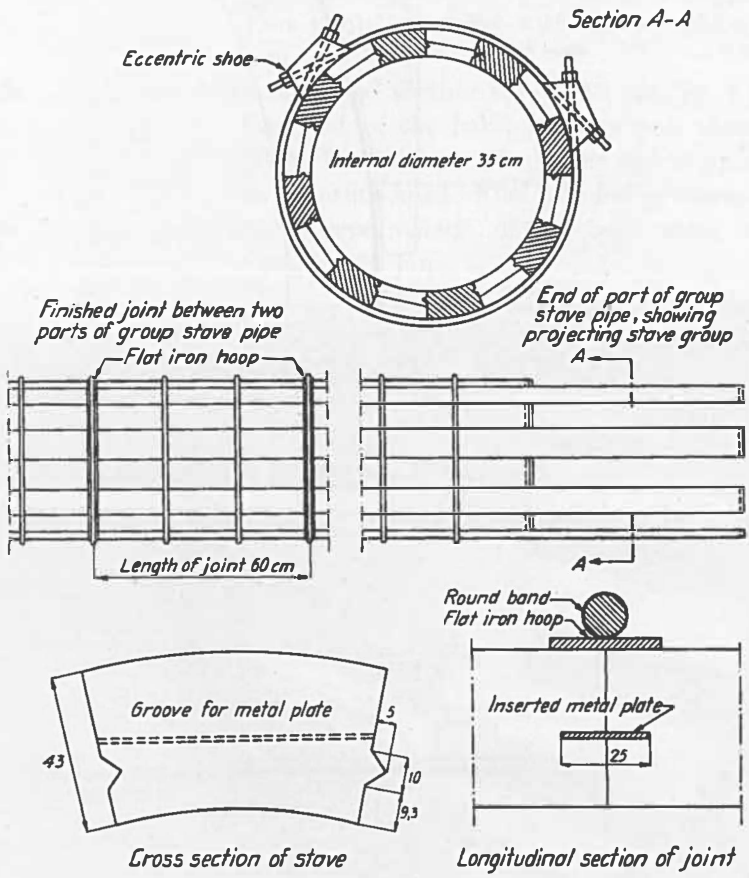


Fig. 49.

VI. Appendices

APPENDIX A

Design of Wood-Stave Pipes Carried on Supports

Application of the Stave Pile Method on the Assumption that the Load-Carrying Capacity of the Pipe is Equal to the Sum of the Load-Carrying Capacities of the Separate Staves.

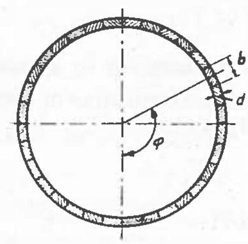


Fig. 50. Schematic cross section of pipe showing notations used in Appendix A.

We make the following assumptions.

The pipe is carried on supports. The distances from centre to centre of supports are equal. The pipe is acted upon by the weight of the water and the weight of the pipe shell only. The number of staves in a cross section of the pipe is denoted by n. No frictional forces act between the staves. The pipe is deformed in a vertical plane, and the deflections of all staves in the same cross section are equal. The staves are symmetrical with respect to both the horizontal and the vertical diameter of the pipe.

All staves are assumed to be equal in size. Their breadth is b and their thickness is d, see Fig. 50.

The principal moment of inertia of a separate stave is

$$I_{\max} = \frac{db^3}{12} \text{ och } I_{\min} = \frac{bd^3}{12}$$

For a stave at an angle φ with respect to the vertical diameter of the pipe, the moment of inertia is

$$I = I_{max} \sin^2 \varphi + I_{min} \cos^2 \varphi$$

or

$$I = I_{max} \left(\sin^2 \varphi + \frac{d^2}{b^2} \cos^2 \varphi \right)$$

The moment of inertia of the whole cross section of the pipe is obtained by summing up the moments of inertia of n staves

$$I_{total} = I_{max} \sum_1^n \left(\sin^2 \varphi_n + \frac{d^2}{b^2} \cos^2 \varphi_n \right)$$

This expression becomes

$$I_{total} = \frac{1}{2} n \, I_{max} \left(1 + \frac{d^2}{b^2} \right)$$

or

$$I_{total} = \frac{1}{2} n \left(I_{max} + I_{min} \right) \tag{81}$$

The polar moment of inertia of a separate stave is

$$I_{pol} = I_{max} + I_{min}$$

and hence

$$I_{total} = \frac{1}{2} n \, I_{pol}$$

Maximum Bending Stress in Staves.

Since it has been assumed that the deformations of all staves in a cross section of the pipe are equal, it follows from the differential equation of a beam that the ratio M/I in a given cross section of the pipe is constant. Consequently, the moment acting on the whole cross section is distributed among the separate staves in proportion to their respective moments of inertia. The load-carrying ability of the whole cross section is determined by the stress in the most heavily strained staves, that is to say in those staves which have the greatest moment of inertia or, in other words, in the staves standing on the narrow edge, i. e. the staves on the sides of the pipe. If the moment acting on the cross section as a whole is denoted by M_{total}, the moment acting on the most heavily stressed stave is

$$M = \frac{I_{max}}{I_{total}} M_{total}$$

If we disregard the fact that the staves are jointed, we can assume that the moment distribution in the pipe as a whole corresponds to that in a continuously supported beam. In that case, the maximum moment, which occurs at the supports, can be written

$$M_{total} = \frac{\gamma \pi D^2 L_s^2}{48}$$

By inserting the moment of inertia and the moment of resistance of the most heavily stressed stave, we obtain the maximum tensile stress in bending

$$\sigma = \frac{\gamma D L_s^2 b^2}{48 I_{pol}} \tag{82}$$

and hence the maximum distance between the supports

$$L_s = \sqrt{\frac{4 \sigma bd}{\gamma D}\left(1 + \frac{d^2}{b^2}\right)} \tag{83}$$

Since the ultimate tensile strength of wet timber is relatively low (it can be as low as 50 per cent of the strength in an air-dry state), the allowable stress for pine and spruce staves should not exceed 60 kg per cm². In that case, the factor of safety is about 3. This factor of safety makes an allowance for the fact that the staves are jointed, and that the structure is therefore slightly weakened. For $\sigma = 60$ kg per cm², we obtain, with a sufficient degree of accuracy, an appropriate value of the distance between the supports

$$L_c = 100 \sqrt{\frac{24 b_c d_c}{D_c}\left(1 + \frac{d_c^2}{b_c^2}\right)} \tag{84}$$

All lengths are expressed in centimetres.

If the pipe is inclined at an angle α with respect to the horizontal plane, the distance between the supports measured along the slope is $L_c/\sqrt{\cos\alpha}$ according to SCHWERIN (84).

Normal Forces in Transverse Plane.[1])

In the preceding section it has been assumed that the pipe is deformed in the vertical plane only. Consequently, the horizontal displacement is zero.

1. *Weight of Water.*

The deflection of the pipe can be written

$$\delta = k_1 \frac{\gamma \pi r^2}{I_{\text{total}}} \tag{85}$$

where k_1 is a constant which is dependent on the distance between the supports, etc.

The radial displacement of a separate stave is

$$\eta = k_1 b \frac{q - \frac{N}{r}}{I_{\min}} = \delta \cos\varphi \tag{86}$$

$$q - \frac{N}{r} = \gamma \pi r^2 \frac{I_{\min}}{b I_{\text{total}}} \cos\varphi$$

By inserting the values of $I_{\min}$ and I_{total}, we obtain

$$q - \frac{N}{r} = \gamma r \frac{d^2}{b^2 + d^2} \cos\varphi$$

The water pressure is

$$q = \gamma r (1 + \cos\varphi) \tag{87}$$

$$\because N = \gamma r^2 \left[1 + \frac{b^2 + 2 d^2}{b^2 + d^2} \cos\varphi \right] \tag{88}$$

---

[1]) The calculations in this section were made in collaboration with K. J. SUNDQUIST, C. E.

Hence we obtain the normal forces due to the weight of the water

$$N_{\pi} = -\gamma r^2 \frac{d^2}{d^2 + b^2} \tag{89}$$

$$N_{\pi/2} = \gamma r^2 \tag{90}$$

$$N_o = \gamma r^2 \left[2 + \frac{d^2}{d^2 + b^2} \right] \tag{91}$$

If $b \gg d$, we get

$$N \cong \gamma r^2 (1 + \cos \varphi) \tag{92}$$

2. *Weight of Pipe Shell.*

For the weight of the pipe shell, we obtain analogously

$$G \cos \varphi - \frac{N}{r} = 2 G \frac{d^2}{b^2 + d^2} \cos \varphi$$

$$\because N = G r \frac{b^2 - d^2}{b^2 + d^2} \cos \varphi \tag{93}$$

If $b \gg d$, we get

$$N \cong G r \cos \varphi \tag{94}$$

It is of interest to note that the normal forces calculated from Eqs. (92) and (94) on the assumption that $b \gg d$ agree with those computed by THOMA (93) on the basis of the membrane theory, that is to say, on the assumption that the wall of the pipe cannot withstand any moments.

APPENDIX B

Stress Measurements Made at Sikfors

The following notations are used in the diagrams.

Observed stresses in the bands.

× Actual stress.

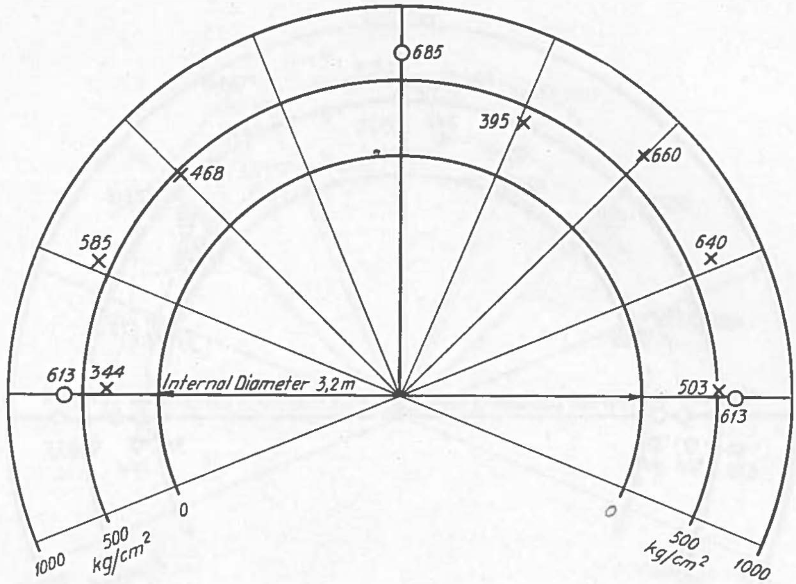
▽ Difference between stresses in the bands observed when the pipe was subjected to a certain pressure and after the pipe was emptied (i. e. not including the no-load stress).

Calculated stresses in the bands.

○ Stress calculated on the assumption that the compressive stress p in the surface of the contact between the staves is zero.

The figures at the points marked with the above symbols express the stress in kg per cm².

When readings taken at different times and different pressures are reproduced in the same diagram, the points at which the stresses were observed at the same time are joined by dash lines.



SIKFORS.

Test No. 1.

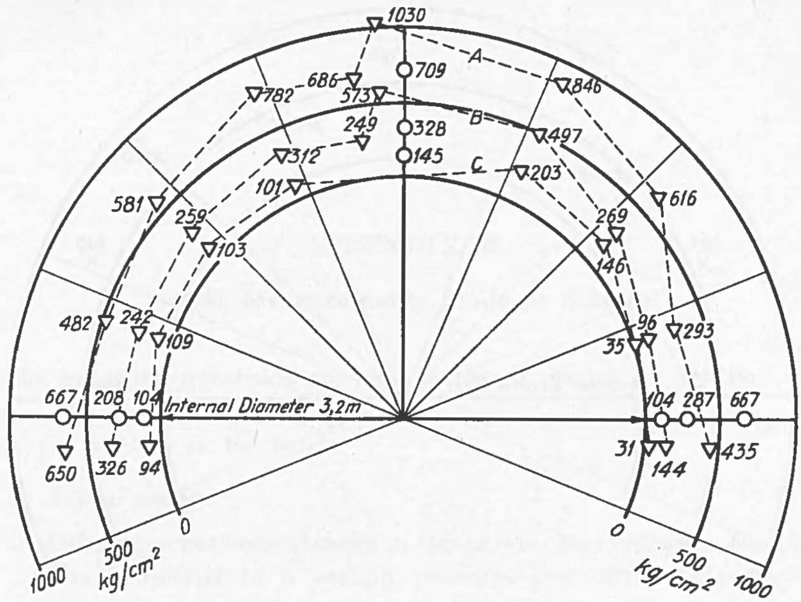
Head 4.87 m.

Distance from centre to centre of bands 60.5 cm.

The surface of the ground was on a level with the horizontal diameter of the pipe.

Internal diameter of the pipe 3.2 m.

NOTE. The test was interrupted after the band had been loosened as much as possible. The pipe leaked profusely. It is probably that the band was not completely relieved from stresses after loosening, and the actual stresses may therefore be slightly higher than the observed values.



SIKFORS.

Test No. 2.

Varying heads, see below.

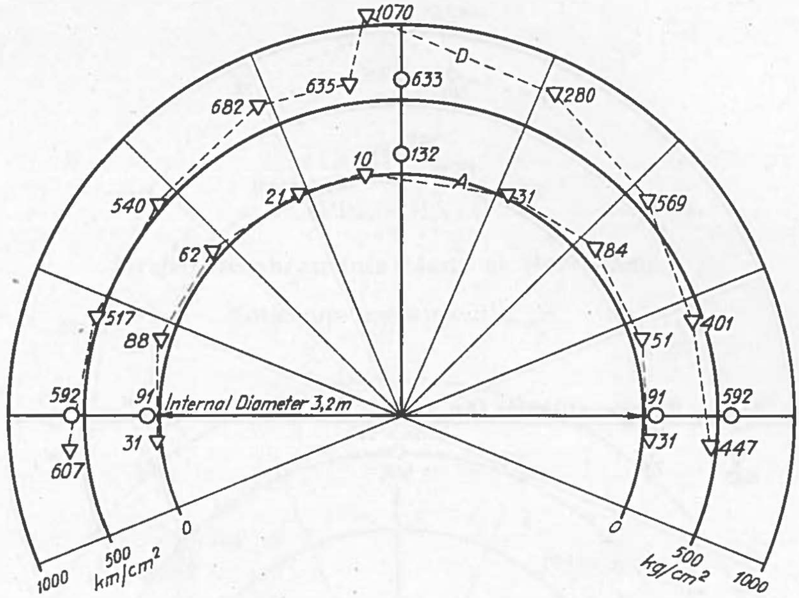
Distance from centre to centre of bands 35.5 cm.

The surface of the ground was 0.2 m below the horizontal diameter of the pipe.

Internal diameter of pipe 3.2 m.

The readings were taken at the following heads A 9.19 m,
B 3.84 m,
C 1.5 to 1 m.

NOTE. The stresses observed at the crown of the pipe were remarkably high. This seems to be due to the fact that the tensometers were attached close to the shoe and on both sides of the band. The shoe was of the eccentric type. Therefore, it produced bending stresses in the band, and these stresses exerted a particularly strong influence on the nearest tensometer.



SIKFORS.

Test No. 3, A and D.

Head 1.07 and 8.12 m.

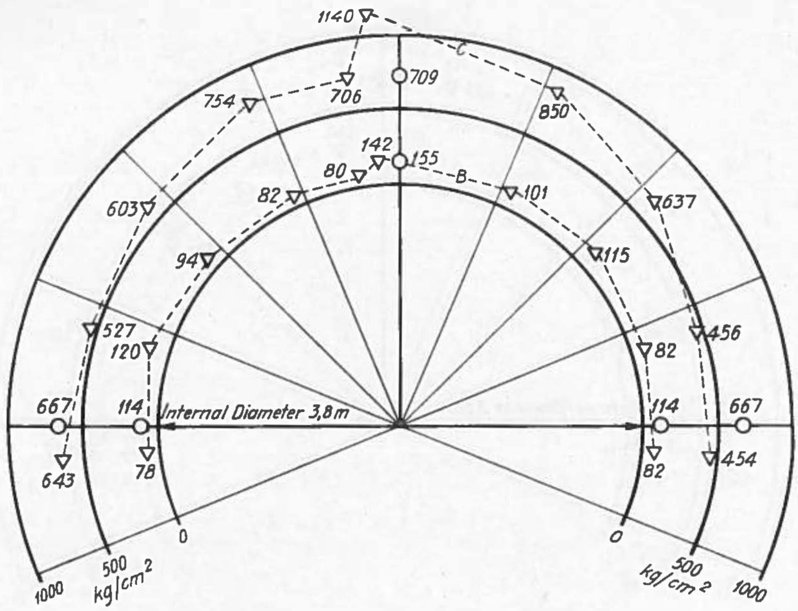
Distance from centre to centre of bands 35.5 cm.

The surface of the ground was 0.2 m below the horizontal diameter of the pipe.

Internal diameter of pipe 3.2 m.

The readings were taken at the following heads *A* 1.07 m,
D 8.12 m.

Regarding the stresses observed at the crown of the pipe, see Test No. 2, Note.



SIKFORS.

Test No. 3, *B and C.*

Head 9.19 and 1.3 to 1.5 m.

Distance from centre to centre of bands 35.5 cm.

The surface of the ground was 0.2 m below the horizontal diameter of the pipe.

Internal diameter of pipe 3.2 m.

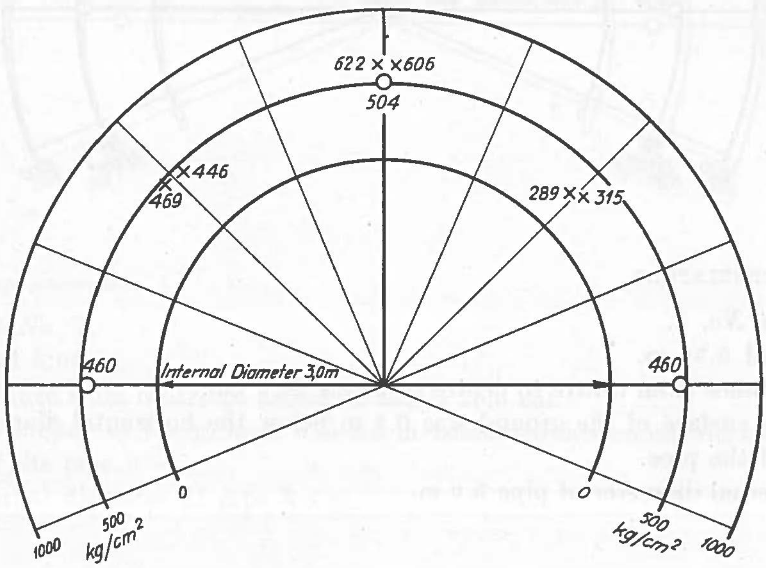
The readings ware taken at the following heads B 1.3 to 1.5 m.
C 9.19

Regarding the stresses observed at the crown of the pipe, see Test No. 2, Note.

APPENDIX C

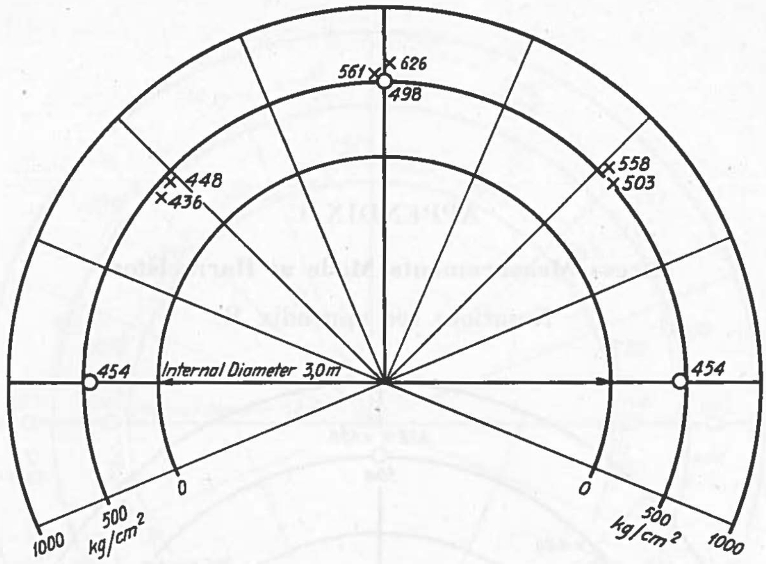
Stress Measurements Made at Harrselsfors

Notations see appendix B.



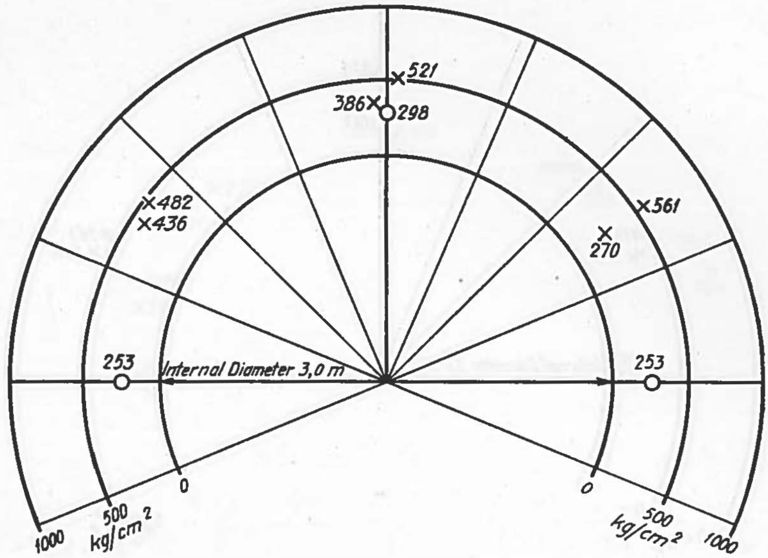
HARRSELSFORS.

Test No. 1.
Head 5.78 m.
Distance from centre to centre of bands 19.5 cm.
The surface of the ground was 0.3 m below the horizontal diameter of the pipe.
Internal diameter of pipe 3.0 m.



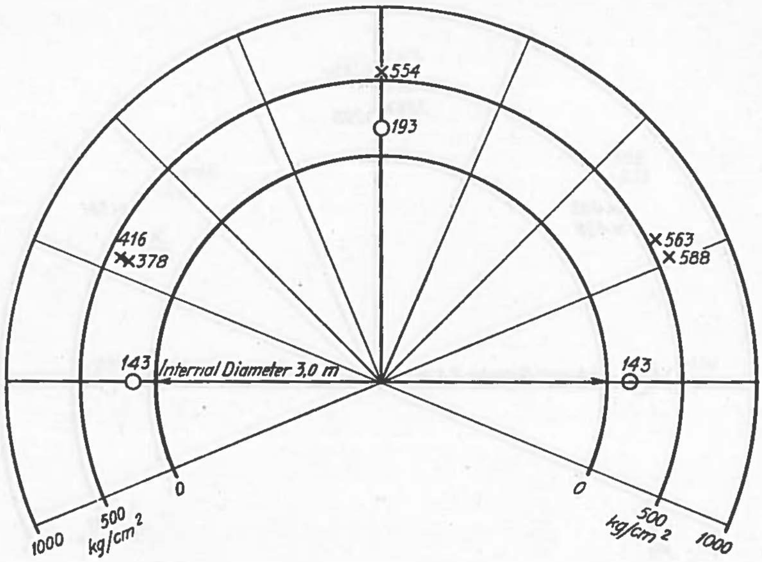
HARRSELSFORS.

Test No. 2.
Head 5.50 m.
Distance from centre to centre of bands 20.2 cm.
The surface of the ground was 0.3 m below the horizontal diameter of the pipe.
Internal diameter of pipe 3.0 m.



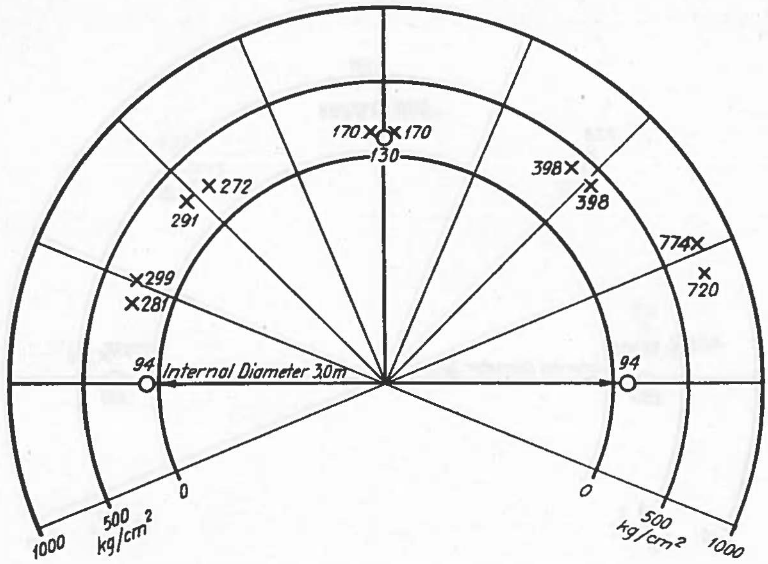
HARRSELSFORS.

Test No. 3.
Head 3.02 m.
Distance from centre to centre of bands 20.0 cm.
The surface of the ground was 0.2 m below the horizontal diameter of the pipe.
Internal diameter of pipe 3.0 m.



HARRSELSFORS.

Test No. 4.
Head 1.44 m.
Distance from centre to centre of bands 22.3 cm.
The surface of the ground was 0.2 m below the horizontal diameter of the pipe.
Internal diameter of pipe 3.0 m.



HARRSELSFORS.

Test No. 5.

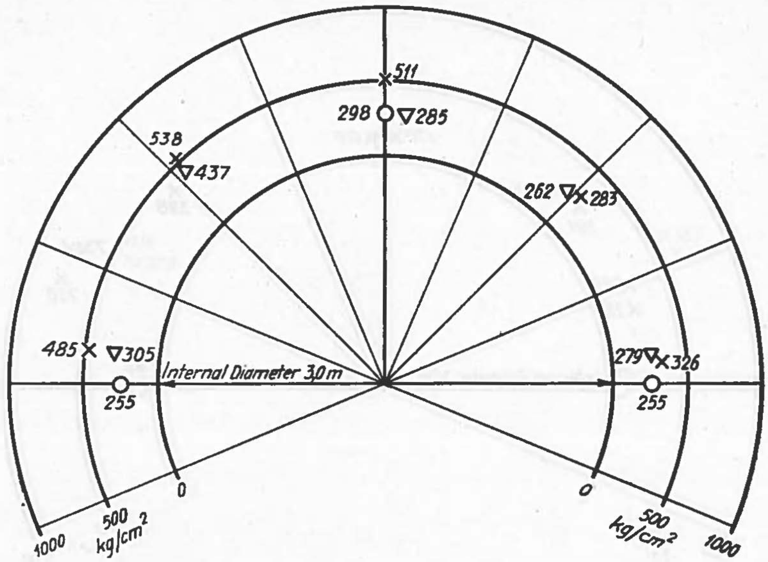
Head 0.57 m.

Distance from centre to centre of bands 25.0 cm.

The surface of the ground was 0.1 m below the horizontal diameter of the pipe.

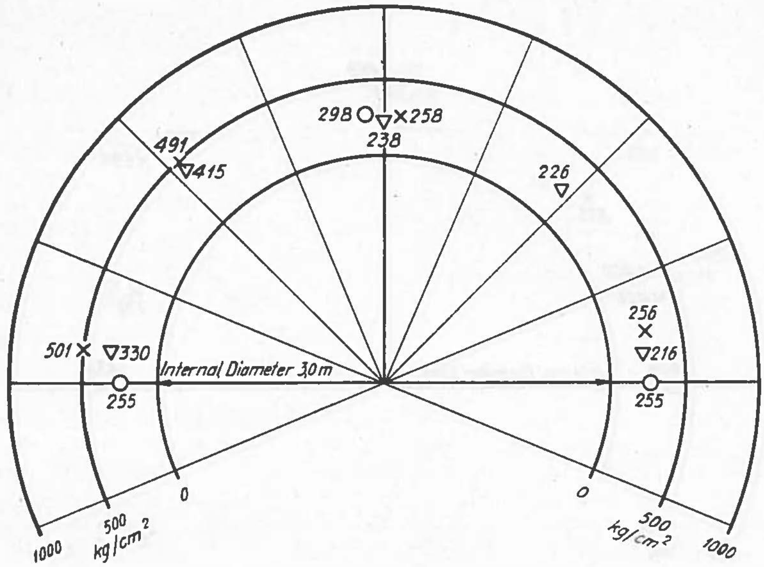
Internal diameter of pipe 3.0 m.

NOTE. When the band was tightened again after the test, the tensometers returned to approximately the same positions as at the beginning of the test.



HARRSELSFORS.

Test No. 6.
Head 3.03 m.
Distance from centre of bands 20.0 cm.
The surface of the ground was 0.2 m below the horizontal diameter of the pipe.
Internal diameter of pipe 3.0 m.



HARRSELSFORS.

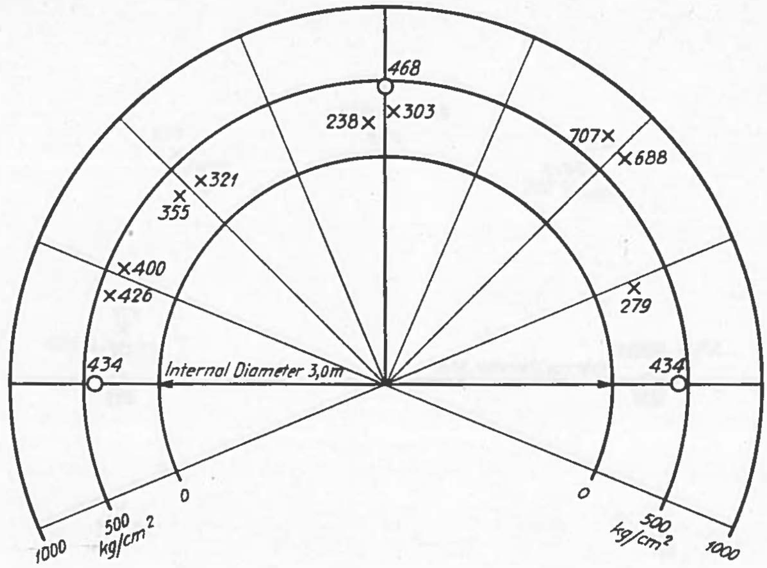
Test No. 7.

Head 3.03 m.

Distance from centre to centre of bands 20.0 cm.

The surface of the ground was 0.2 m below the horizontal diameter of the pipe.

Internal diameter of pipe 3.0 m.



HARRSELSFORS.

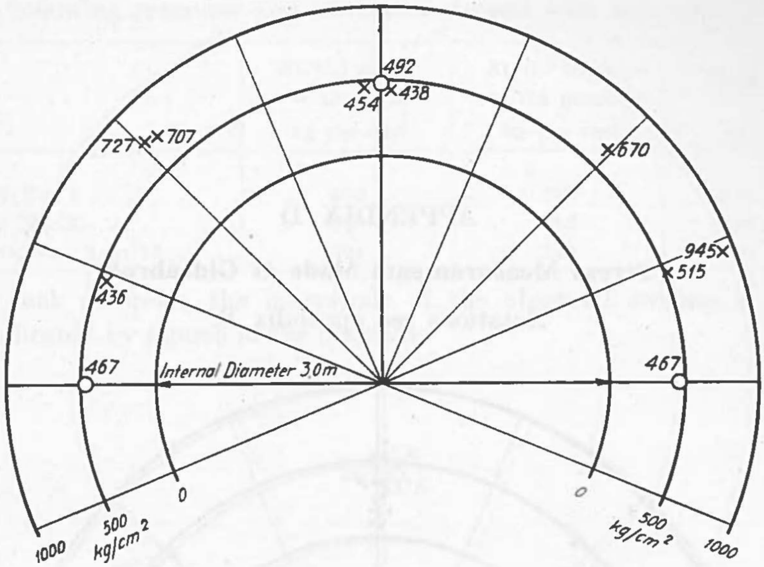
Test No. 8.

Head 6.99 m.

Distance from centre to centre of bands 15.3 cm.

The surface of the ground was 0.3 m below the horizontal diameter of the pipe.

Internal diameter of pipe 3.0 m.



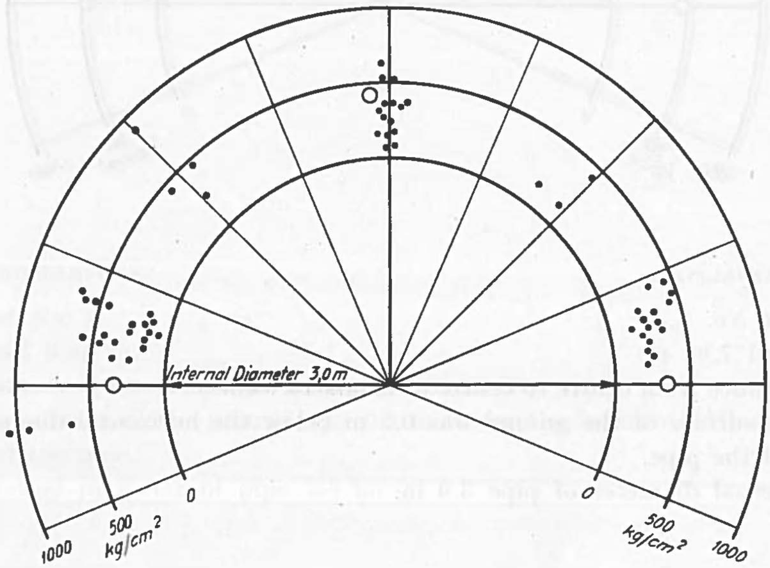
HARRSELFORS.

Test No. 9.
Head 7.03 m.
Distance from centre to centre of bands 16.0 cm.
The surface of the ground was 0.3 m below the horizontal diameter of the pipe.
Internal diameter of pipe 3.0 m.

APPENDIX D

Stress Measurements Made at Gideåbruk

Notations see appendix B.



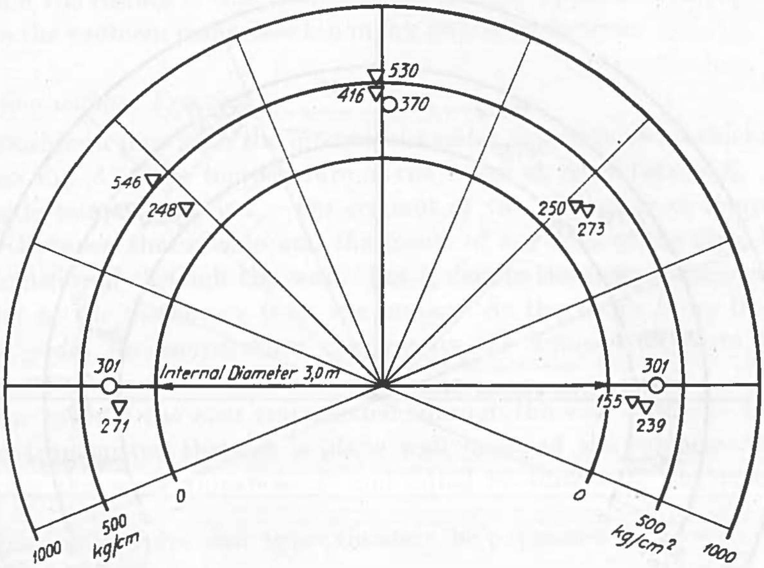
GIDEÅBRUK.

Tests Nos. 1 *to* 13.
Head 2.62 to 2.68 m.
Distance from centre to centre of bands 38.5 to 39.0 cm.
The surface of the ground was 0.1 m below the horizontal diameter of the pipe.
Internal diameter of pipe 3.0 m.

The following pressures and calculated stresses were obtained.

	At the crown of the pipe kg per cm²	At the surface of the ground kg per cm²	Head m
Test No. 1	405	338	2.68
Test No. 2	410	342	2.68
Tests Nos. 3 to 13	404	335	2.63

For lack of space, the magnitude of the observed stresses is not indicated by figures in the diagrams.



GIDEÅBRUK.

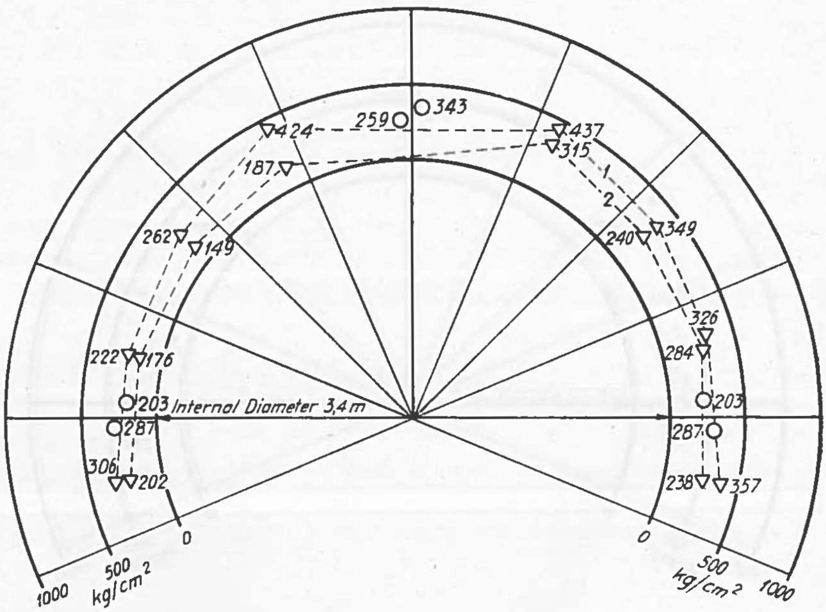
Tests Nos. 14 *and* 15.
Head 2.35 m.
Distance from centre to centre of bands 39.0 cm.
The surface of the ground was 0.1 m below the horizontal diameter of pipe.
Internal diameter of pipe 3.0 m.

NOTE. Since it was raining during these two tests, it is probable that the tensometers were slightly displaced, and the observed stress values are therefore uncertain.

APPENDIX E

Stress Measurements Made at Äggfors

Notations see appendix B.



ÄGGFORS.

Tests Nos. 1 *and* 2.
Head 3.26 and 2.25 m.
Distance from centre to centre of bands 53.0 cm.
The surface of the ground was 0.2 m below the horizontal diameter of the pipe.
Internal diameter of pipe 3.4 m.
Test No. 1, head 3.26 m.
Test No. 2, head 2.25 m.

APPENDIX F

Theoretical Study of Freezing of Wood-Stave Pipes

In order to elucidate the problem of freezing of wood-stave pipes, it is very useful to make a purely theoretical study of this problem, even if the results of this study are not directly applicable in practice, since the problem comprises too many unknown factors.

Cooling without Freezing.

Consider a pipe with the internal diameter D and the wall thickness d, see Fig. 51. The temperature of the water at the intake is t_o. The outside temperature is t_y. On account of the difference in temperature between the outside and the inside of the wall of the pipe, heat is transferred through the wall. Let t_x denote the temperature of the water at the distance x from the intake. As the water flows from x to $x + dx$, its temperature changes by the amount dt_x from t_x to $t_x + dt_x$.

The ratio of the heat transmitted through the wall of a pipe to the heat transmitted through a plane wall made of the same material, having the same thickness d, and equal in surface to the internal surface of the pipe can approximately be expressed by $f = 1 + \frac{d}{D}$.

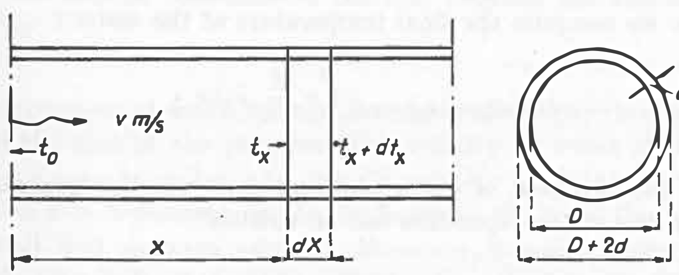


Fig. 51. Schematic longitudinal section and cross section of pipe showing notations used in Appendix F.

If the coefficient of heat transmission through this plane wall is denoted by K kcal per m² per °C per hour (this coefficient includes the heat transmission by conduction through the timber wall as well as the heat transfer from the water to the timber wall and from the timber wall to the air), and if the thermal resistance of the wall is $m = \frac{1}{K}$, then the thermal resistance of the pipe is $\frac{m}{f}$.

During the time when a water particle moves the distance dx, the quantity of heat W_1, kcal, passes through the wall of the pipe. The velocity of water flow is v m per sec.

$$W_1 = \frac{f}{m} \cdot (t_y - t_x) \cdot \pi \cdot D \cdot dx \cdot \frac{dx}{v} \cdot \frac{1}{3\,600} \text{ kcal} \tag{95}$$

During the same time, the heat contained in the water particle changes by the amount

$$W_2 = dt_x \cdot \pi \cdot \frac{D^2}{4} \cdot dx \cdot 1\,000 \text{ kcal} \tag{96}$$

The quantity of heat W_2 which is given off by the water passes through the wall of the pipe into the air. Therefore, $W_1 = W_2$. By solving this equation, and by inserting the correlated values $x = 0$ and $t_x = t_o$ for the upstream end of the pipe and $x = L$ and $t_x = t$ for the downstream end of the pipe, we obtain the relation

$$-\frac{f}{m} = 9 \cdot 10^5 \cdot \frac{v \cdot D}{L} \cdot \ln \frac{t - t_y}{t_o - t_y} \tag{97}$$

Hence we compute the final temperature of the water t

$$t = (t_o - t_y) \cdot e^{-\frac{L}{v \cdot D} \cdot \frac{f}{m} \cdot \frac{1}{9 \cdot 10^5}} + t_y \tag{98}$$

Since the exponent of e is a small number, which is generally less than 0.1, the above expression can be written

$$t = (t_o - t_y) \cdot \left(1 - \frac{L}{v \cdot D} \cdot \frac{f}{m} \cdot \frac{1}{9 \cdot 10^5}\right) + t_y$$

or

$$t = t_o - (t_o - t_y) \cdot \frac{L}{v \cdot D} \cdot \frac{f}{m} \cdot \frac{1}{9 \cdot 10^5} =$$

$$= t_o - \frac{t_o - t_y}{\left(\frac{v \cdot D}{L}\right) \cdot \left(\frac{m}{f} \cdot 9 \cdot 10^5\right)} = t_o - \frac{t_o - t_y}{v_s \cdot a} \tag{99}$$

It follows from this equation that the difference between the initial temperature of the water and its temperature at the distance L, metres, from the intake is directly proportional to the difference between the initial water temperature t_o and the outside temperature t_y. Furthermore, the drop in water temperature is proportional to the product $v_s \cdot a$, where

$$v_s = v \cdot \frac{D}{L}$$

$$a = \frac{m}{f} \cdot 9 \cdot 10^5 = \frac{(m_i + m_v + m_y)}{f} \cdot 9 \cdot 10^5$$

In the above expression,

m_v = the thermal resistance of the timber wall = d/λ, where λ is the thermal conductivity and d is the thickness of the wall,

m_i = the thermal resistance of the film between the water and the wall,

m_y = the thermal resistance of the film between the wall and the air.

The expression v_s, which has the dimension of velocity, is dependent only on the size of the pipe and the velocity of water flow. The quantity v_s can be called »the specific velocity of water flow». The expression a is dependent on the thickness of the wall, the material of the wall, and the wind velocity. Moreover, it is also dependent — although, as a rule, to a slight extent — on the velocity of water flow, which influences m_i.

Formation of Thin Ice Layer Inside Wood-Stave Pipes.

An ice layer can form on the inside of a wood-stave pipe, even if the temperature of the water is above zero. The reason is the temperature difference across the water film on the inside of the pipe. Owing to this temperature difference, the temperature of the wall may be below zero, while the temperature of the water is above zero. On the outside of the pipe, there is a similar temperature difference across the air film. The temperature gradients are given by the difference between the air temperature t_y and the temperature t_s on the outside of the pipe wall, and between the temperature t_v on the inside of the wall and the temperature t of the water. Using the thermal resistances m_y, m_v, and m_i, we obtain

$$W = \frac{t - t_v}{m_i} = \frac{t_v - t_s}{m_v} = \frac{t_s - t_y}{m_y} = \frac{t - t_y}{m_i + m_v + m_y} \tag{100}$$

No ice forms on the inside surface of the wall if $t_v > 0°$, or if

$$t > - t_y \cdot \frac{m_i}{m_v + m_y} \tag{101}$$

If the internal thermal film resistance m_i is high, the water temperature must be several degrees above zero in order to prevent freezing. If the internal surface of the pipe is smooth, m_i is relatively small, and the risk of freezing is therefore slight. Consequently, a smooth internal surface of the pipe is favourable not only from a hydraulic point of view, but also because it reduces the danger of freezing.

When the water flows along the wall of pipe, the internal thermal film resistance is dependent on the degree of roughness of the wall, the velocity of water flow, the diameter of the pipe, and the temperature, see Fig. 52, but the variation in film resistance with the temperature is disregarded in this diagram, since this variation is very slight.

Fig. 52 shows that m_i varies within the approximate limits from 0.001 at the velocity of water flow of 2 m per sec to 0.004 at the velocity of 0.2 m per sec. The thermal film resistance m_y on the outside of the pipe varies from 0.04 at a high wind velocity to 0.20 in the absence of wind. For frozen timber at a temperature of — 15° C, λ is 0.8 (it is slightly lower at a higher temperature). Therefore, for a wall thickness of, say, 50 mm, $m_v = 0.06$. Fig. 34 represents the limit value of the ratio $t/ - t_y$ at the beginning of freezing in accor-

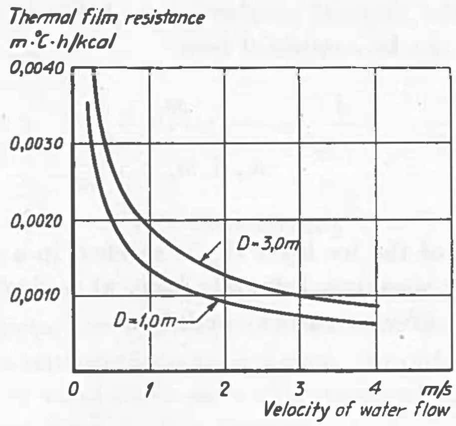


Fig. 52. Thermal film resistance $m_i = \frac{1}{K_i}$ between water and pipe wall calculated from SOENNECKEN's formula $K_i = \frac{735}{D^{0.3}} \cdot v^{0.6} (1 + 0.014\, t)$ kcal per m² per °C per hour. This formula has been deduced for rough metal walls, but it may probably be applied to ice-covered walls without involving any serious errors. It is to be noted, however, that the value of K_i for smooth walls is much lower. The diagram shows the variation in thermal film resistance with the velocity of water flow for the water temperature $t = 0$ and pipe diameter $D = 1.0$ m and 3.0 m.

dance with Eq. (101) as a function of the velocity of water flow v for varying thickness of the pipe wall d in the presence and in the absence of wind. For a pipe 3 m in diameter, made of 2 in. thick staves, at a velocity of water flow of 2 m per sec in the presence of wind, we obtain $t/ - t_y = 0.012$. For $t_y = -30°$ C, we get $t = 0.36°$ C. If v is 0.20 m per sec, we get $t/ - t_y = 0.040$, and hence, for $t_y = -30°$ C, $t = +1.20°$ C. For a wood-stave pipe 1 m in diameter, we obtain values which amount to 70 per cent of the above on account of the variation in the thermal film resistance between the water and the wall with the diameter of the pipe, cf. Fig. 52.

Formation of Thick Ice Layer Inside Wood-Stave Pipes.

The ice coating formed on the inside of a wood-stave pipe acts as a heat-insulating layer, with the result that freezing gradually ceases as the thickness of the ice layer approaches a definite value. If

$t_v = 0$, and if the thermal conductivity of the ice is taken to be 1.55, this value can be computed from

$$\frac{t}{-t_y} = \frac{m_i}{m_v + m_y + \frac{d_{ice}}{1{,}55}} \, . \tag{102}$$

The thickness of the ice layer d_{ice} is reached in a pipe after a time which is, strictly speaking, infinitely long, at a given temperature of the water under given weather conditions.

VII. **Summary**

After a brief historical review, this treatise deals with the strength of wood-stave pipes, the operating conditions of these pipes, and the economic factors entering into their design. In order to demonstrate the possibilities of wood-stave pipes at the present time, a description is given of several pipes used in practice.

The strength problems are treated in Chapter II. The Author advances some design rules which are deduced from a new theory of wood-stave pipes carried on supports. The design of wood-stave pipes is discussed on the basis of stress measurements made in actual operation on pipes which were embedded to half their height. The results of tests carried on without interruption during more than three years are used to show that the swelling stress in a wood-stave pipe increases at a relatively high rate, and then slowly decreases in the course of time. Suggestions are made for the determination of the dimensions of staves and the selection of the type of joints, bands, and shoes.

Chapter III deals with decay, freezing, and losses of head. Some conclusions as to the life of wood-stave pipes and the appropriate measures for preventing decay are drawn from practical experience regarding decay of pipes in various countries. The section on freezing of wood-stave pipes is mostly based on a theory of freezing published by the Author in a previous paper. This theory has been verified by observations made on several frozen pipes. The Author suggests that Manning's formula should be used for calculating the losses of head, and proposes certain definite values of the coefficient M which are dependent on the character of the internal surface of the pipe and on the ratio of the velocity of water flow to the diameter of the pipe.

The advantages and drawbacks of wood-stave pipes are enumerated in Chapter IV, which also gives examples of economically designed pipes. Finally, several wood-stave pipes and their accessories made by various manufacturers are described in Chapter V.

VIII. References

The references in this list are arranged in the alphabetical order according to the names of the authors. In those cases where the name of the author is not indicated, the references are classified according to the name of the review or the publisher.

The following abbreviations of titles of periodical publications are used in this list.

A. S. C. E. Trans.	=	American Society of Civil Engineers, Transactions.
E. u. M.	=	Elektrotechnik und Maschinenbau.
E. N. R.	=	Engineering News-Record.
I. V. A. handl.	=	Ingeniörsvetenskapsakademiens handlingar.
Schw. Bauz.	=	Schweizerische Bauzeitung.
SVKF publ.	=	Svenska vattenkraftföreningens publikationer.
Tekn. T. V.	=	Teknisk Tidskrift, väg- och vattenbyggnadskonst.
U. S. Dep. Agr. Bull.	=	U. S. Department og Agriculture, Bulletin.
W. u. W.	=	Wasserkraft und Wasserwirtschaft.
Z. V. D. I.	=	Zeitschrift des Vereins Deutscher Ingenieure.
Z. ges. Turb.	=	Zeitschrift für das gesamte Turbinenwesen.
Z. Math. Mech.	=	Zeitschrift für angewandte Mathematik und Mechanik.
Z. Ö. I. A. V.	=	Zeitschrift des Österreichischen Ingenieur- und Architekten-Vereines.

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L = Distance between supports, m.
L_c = Distance between supports calculated by means of stave pile method, cm.
L_s = Distance between supports calculated by means of stave pile method, m.
L_m = Distance between supports calculated by means of membrane method, m.
l = Distance between bands, m.
M = Coefficient in Manning's formula for friction loss of head.
M = Moment, ton-m.
M_1, M_2 = Moment acting on ring stiffener, ton-m.
M_o = Moment acting in transverse plane at given point at bottom of pipe, ton-m per m.
$M_{\pi/2}$ = Moment acting in transverse plane at given point on level with longitudinal axis of pipe, ton-m per m.
M_{π} = Moment acting in transverse plane at given point at crown of pipe, ton-m per m.
M_{total} = Total moment in cross section, ton-m.
m = Total thermal resistance of wall of pipe, m² °C hour per kcal.
m_i = Internal thermal film resistance (between liquid and wall), m² °C hour per kcal.
m_y = External thermal film resistance (between wall and air), m² °C hour per kcal.
m_v = Thermal resistance of wall, m² °C hour per kcal.
N = Normal force, tons per m.
N_{φ} = Normal force acting in transverse plane, tons per m.
N_o = Normal force acting in transverse plane at given point at bottom of pipe, tons per m.
$N_{\pi/2}$ = Normal force acting in transverse plane at given point on level with longitudinal axis of pipe, tons per m.
N_{π} = Normal force acting in transverse plane at given point at crown of pipe, tons per m.
n = Coefficient of roughness in Ganguillet's and Kutter's formula for value of C.
n = Number of staves in cross section of pipe.
P = Total vertical reaction at supports, tons.
p = Compressive stress in surface of contact between two adjacent staves, tons per m².
p_s = Compressive stress between the bands and the staves, tons per m².
Q = Rate of water flow, m³ per sec.
Q_m = Average rate of water flow during given period of time, m³ per sec.
q = Internal pressure, tons per m².
R = Radius of ring stiffener, m.
R = Hydraulic radius, for circular cross section $R = D/4$, m.
r = Internal radius of pipe $= D/2$, m.
S = Static moment, m³.
S_{φ} = Normal force acting on bands, ton per m.
T_1, T_2, etc. = Partial periods of time, sec.
T_{tot} = Total time of operation, sec.
t = Temperature of water, °C.
t_o = Temperature of water at intake of pipe, °C.

t_x	=	Temperature of water at distance x from intake of pipe, °C.
t_y	=	Temperature of air on outside of pipe, °C.
t_s	=	Temperature of wall on outside, °C.
t_v	=	Temperature of wall on inside, °C.
$V_{\pi/2}$	=	Shearing force acting on wall of pipe in horizontal plane through longitudinal axis of pipe at right angles to this axis, tons.
v	=	Average velocity of water flow, m per sec.
v_{appr}	=	Approximate value of v, m per sec.
v_s	=	Specific velocity of water flow, m per sec.
W_1	=	Quantity of heat, kcal.
W_2	=	Quantity of heat, kcal.
x	=	Distance from intake to given point of pipe, m.
z	=	Distance from centre between two supports to given point, m.
α	=	Factor in expression $\alpha D =$ initial cost per metre of length of pipe.
γ	=	Specific gravity, tons per m^3.
δ	=	Vertical deflection of the pipe, m.
ε	=	Coefficient of distortion $= \dfrac{D_h - D_v}{D}$.
φ	=	Angle in plane at right angles to longitudinal axis of pipe. The initial line of the angle passes through the lowest point of the pipe.
Φ	=	Width of band, m.
η	=	Displacement of given point of pipe wall in radial direction, m.
η_s	=	Deflection of stave at centre, m.
λ	=	Thermal conductivity, kcal per m per °C per hour
ω	=	Price of power, Swedish kronor per kW per year.
ϱ	=	Internal radivs of pipe, m.
σ	=	Maximum tensile stress in bending in the most heavily stressed stave, tons per m^2.
σ_B	=	Tensile stress in bands, tons per m^2.
σ_z	=	Tensile stress in staves parallel to longitudinal axis of pipe, tons per m^2.
τ	=	Shearing stress in the wall of pipe, tons per m^2.
τ_m	=	Maximum shearing stress in the wall of pipe calculated from the membrane method, tons per m^2.
τ_s	=	Maximum shearing stress in the wall of pipe calculated from the stave pile method, tons per m^2.
ξ_1, ξ_2	=	Constants.

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