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Application of steel in hydraulic construction.

Anwendung des Stahles im Wasserbau.

Application de l'acier dans la construction hydraulique.

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Use of Steel in Hydraulic Structures, Fixed Plants.

Anwendung des Stahles im Wasserbau, feste Anlagen.

Application de l'acier en construction hydraulique, installations fixes.

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To deal with the theme "Steel in hydraulic structures" at an international congress in connection with bridge building and structural engineering may at first seem rather strange. But when one considers that steel as a building material plays an important part in the foundations of bridges and other structures, it is comprehensible that the Congress should deal also with this subject. In the first place there are the fixed structures mentioned in my report, which stand in direct relationship to bridges and other structures, i. e. steel sheet piling as a means now universally adopted for constructing the foundations of piers, etc., and then there are also the steel piles next described, as well as other types of foundations. But since the experience made with supporting structures of steel is considerably greater and more interesting than the observations made on piers, such supporting structures will often be referred to in what follows. But it may be mentioned that the results of observations and experience with such structures may at once be applied to piers and other foundation work made of steel.

1) The physical and chemical behaviour of steel as a building material.

Since wood and reinforced concrete do not in all cases possess the physical and chemical resistance required by building materials when used in structures in water, it was only with hesitation that the adoption of steel for such work was started about 30 years ago in Germany. Its use has been extending more and more since 1920. It was considered that the introduction of steel would diminish to a great degree the attacks to which building material was exposed, and that a material had been introduced into hydraulic structures which would prove more suitable than any other in this respect. However, the extent to which steel is also liable to physical and chemical attacks in air, in alternating air and water, in water itself, and in the ground, was not overlooked.

Now that the experience of nearly a generation is available, it is time to compare the theoretical considerations and the experience gained in research stations with the observations made on structures executed, and thereby draw conclusions concering the suitability of adopting this material, and as to how it can be improved. The difficulty in the stipulated requirements consists, also for steel, in the material having in most cases to pass through four different zones, each of which has its own particular physical and chemical characteristics tending to attack the material, and further that different kinds of steel behave differently according to the composition of the soil and of the water. But it is demanded that the building material should be as far as possible equally resistant in all the zones (air, alternating air and water, water and soil). Observations, however, show that this aim has not been attained up to the present. It appears to be as yet fundmentally impossible to get one and the same material to be absolutely suitable for all the different conditions in the four zones.

In the air, it is principally oxygen which acts on the material, but in some cases there are also industrial fumes and atmospheric influences. The attacks on the building material are mainly take the form of corrosion.

With alternating air and water, there is more severe corrosion because of the considerably greater amount of moisture and the corrosive substances present in the free water. By means of alternating wetting and drying and by wave action, the oxygen is communicated to the material to a greater degree and the chemical reaction is thereby greatly intensified. In addition there are mechanical damage and stressing, through traffic, ice, etc., etc.

The difference between alternating wetting and drying in sea water and in fresh water should be noted. In sea water this causes more extensive destruction, since it takes place in continuous short alternations (tides); the wave action is also more intense, and the salt contained in the sea water has an unfavourable effect. In hot regions, the warming of the water and of the material plays a further important part.

It ought to be expected that the destruction of the material under water would be less. But this is in general only the case down from a certain depth below the water line downwards. In fact, the first 1—2 m below the alternating air and water zone often displays even more severe destruction than in the part above; and here again this action is more intense in hot regions than in temperate ones.

In the soil the durability of building material depends on the corrosive properties of the soil and sub-soil water. There are kinds of soil in which the material is practically not attacked at all. It may also happen, however, that destruction occurs which it would hardly have been possible to determine beforehand, since there is seldom any opportunity of withdrawing sheet piling again after any length of time.

2) The causes of damage to steel and their obviation.

Mechanical damage will also occur to steel structural work driven into the ground, such as piles and sheet piling, if the cross-sections are not chosen properly in relation to the density of the soil and to the desired depth to which the material has to be rammed. In such cases damage occurs at the top and bottom of these structural parts, and this may lead to the continuity of the wall being destroyed. Whilst the parts of the piles and sheet piling underground remain invisible to the eye, the damage at the top may be restified by cutting away with the burner. But this remedy can be repeated only up to a certain limit, otherwise the required depth of ramming cannot be obtained. The part played by the ramming in causing damage to the steel in structures in earth and water may be considerably more extensive than the damage caused under ultimate service conditions, if the fundamental principle mentioned above is not observed.

Under the heading of mechanical damage comes wear in consequence of the motion of the water. Steel structures erected in stationary water will have a much longer life than those in running water, where the current carries mud and floating matter along with it. For example, damage will soon be caused by the action of quartz sand. Cases are known to us, where the most exposed parts of a steel sheet piling have been cut through at ground level within seven years. Abrasion by sand is certainly the principle cause of mechanical damage to the steel; on the other hand, the stressing caused by ice and wave action must be considered as slight.

Wear and tear from shipping traffic, from being knocked by vessels when arriving and departing, depends to a very great degree on the amount of deflection caused in the structure in the service condition and on the protection afforded by fenders, etc. No case is known, where such damaged has been so great as to necessitate renewal of the structure or of any of its parts. Nevertheless the stressing thus caused should be clearly recognised beforehand, and the design arranged in such a way that softer materials, for example wood, and bundles of cane or faggots, are adopted as fenders to absorb any great forces which might cause damage.

Damage from weather (rain and great differences in temperature) come less into consideration in our regions.

The durability of steel structures is impaired much more by chemical action, especially by corrosion in the separate zones. The experiences hitherto made in Germany differ so much from each other, that it has not yet been found possible to reduce all phenomena to one common denominator, although the question of corrosion has been very fully treated at meetings and in publications. It is therefore unnecessary for me to go further into this matter; I would, however, point out that, because of corrosion, it is to-day still impossible to foretell the life of a steel structure with certainty, since the figures with regard to corrosion which have occasionally been made available by different makers show very great variations. That is not surprising, since the structures have been observed under very different conditions, and the amount of corrosion depends to a very great degree on the composition of the water, air and soil, and also on the temperature. There are districts in Germany which, for this reason, cause quick destruction of the steel, whilst in other districts the life of the steel is many times longer, although there are possibly no differences in the composition of the water which are at all visible or can be chemically determined.

I may in this connection refer to the experience made in connection with continuous steel walls, i. e. that they are less liable to corrosion than separate posts standing in the water. For example, at a place on the North Sea coast, destruction of the steel in such posts took place within 20 years, whilst a wall near them, of the same cross-section, though it showed somewhat severe corrosion, was not damaged to such an extent as to render it unserviceable.

Undoubtedly the composition of the steel is of essential importance, but to what extent has not yet been fully explained.

Based on investigations made by various authorities, the *lowest* limit of the life of steel poles and steel sheet piling subjected to corrosion, under normal circumstances, is estimated as being at least 60 years, a figure which must be considered as amply sufficient in very respect in comparison with the economical service life of such structures. I may, however, also remark that the upper estimated limit of life extends to several hundred years, a figure which of course cannot be scientifically proved.

Tests mode on a structure for shipping traffic at the mouth of the Weser showed that the extent of corroding of the sheet piling, eight years after erection, was 0.2 mm.

One type of destruction which must not be regarded too lightly is electrolysis. A preliminary condition for causing electrolytic action in steel structures is the use; in the presence of moisture, of materials of different composition, in certain cases even the use of different kinds of steel, and not only different metals, may suffice to set up electrolysis. A great deal of work will have been done in future in this connection in order to guard against damage.

As already mentioned in the introduction to this paper, it is possible to give only general particulars with regard to the frequency with which damage occurs. However, the seat of the trouble will generally be found either in the zone subjected to alternating air and water or a few metres below it. The most common and the most dangerous phenomena leading to damage are corrosion and defects caused by the work of ramming.

Sources of greater damage may easily be deduced from what has already been said. It has up to the present not been found possible to control them sufficiently by adopting any particular precautions against them. They must first of all be accepted as inevitable, and the endeavours of the engineer must be directed to trying to reduce the damage by other methods.

Of significance here is first of all the presence or absence of rolling mill scale. This scale is an unavoidable drawback of the rolling process, since it has no longer any reliable connection with the steel beneath it. It therefore tends to flake off easily as soon as a wedge enters between it and the material. The scale can be removed by sand blasting. Nevertheless, in hydraulic engineering, one is in most cases content to leave the scale on the material.

The precautions which can be taken against the damages mentioned above comprise a particular composition for the steel, external painting, metallic coatings, encasing in concrete, and appropriate choice of external shapes. The most preferable method of all is to keep to the course hitherto pursued, i. e. improvement of the composition of the material. Even if the addition of copper has not everywhere fulfilled the expectations based on laboratory tests, it is a fact that in many cases a properly proportioned admixture has made the steel more resistant; this is an advantage which has proved extremely favourable at least when ramming. The experience made with high-grade steels in actual service certainly does not point to the absolute suitability of these admixtures in every case.

The experience made in Germany and also in England is far from being uniform. Whilst it was found in England that ordinary steel corroded most severely when fully immersed in very salt water and least severely when fully immersed in brackish water, chromium steel corrodes easier above water than under it. On the whole, however, the latter steel proved to be superior. At another place, on the other hand, very severe local corrosion was found in chromium-nickel steel, and experience showed that the addition of copper led to no improvement.

The conclusion which can be drawn from the experience made in England tends to show that carbon steel is apparently superior in the air, whilst wrought iron has proved to have a greater resistance in alternating water and air and also under water. It is rather surprising that an addition of copper gives greater resistance against corrosion in the air and in fresh water, but not against corrosion in alternating water and air, or under water in the sea. The same may be stated with regard to chromium steel and nickel steel.

The effects of electrolysis when different kinds of steel are brought into contact in sea water are rather interesting. The phenomena which occur protect St 37 at the expense of wrought iron, and protect chromium or nickel steel at the expense of carbon steel. The material of higher grade is consequently protected at the expense of the material of simpler composition.

External painting to protect steel structures[1] can really be adopted only for parts exposed to the air. For parts in the ground, which have to be pressed in by force, painting should not be adopted, since it will certainly become damaged to a certain extent during ramming operations. It should also be clearly understood that an external coat of paint requires continual maintenance. The expenses in connection with this work are to be considered in connection with the economical service life of the structure. There is not a single kind of paint at present on the market which will give complete protection unless it is also continually attended to and renewed. If a paint manufacturer asserts that it is possible to bring a coat of paint undamaged into the subsoil, even if the part in question is rammed through sharp sand, this can be regarded only as a purely theoretical assumption.

On the other hand, metallic coatings (zinc or lead) are extremely valuable and prevent the occurrance of wasting through corrosion. But since these coatings are very expensive, they come into question only for certain parts, such as ladders, mooring bitts and rings, corner protecting irons, etc., but not for sheet piling or piles.

Also giving the steel piles and sheet piling a light casing of concrete does not offer any durable protection, since, as is well known, it is difficult to get concrete to adhere to large iron surfaces.

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[1] Cf. also the remarks of Ministerialrat *Burkowitz.*

The external shape of the walls may contribute greatly towards diminishing the effect of wave action, currents, and abrasion by sand. Profiles have therefore been designed which ensure the surface remaining as smooth as possible, in order to prevent the formation of eddies and thereby the development of friction surfaces.

The question now arises, to what extent the above-mentioned precautions for reducing damage to the material allow existing steel sections to be set at full value in the calculations, i. e. to what extent it is justifiable when designing the work to base calculations on the existing dimensions of the steel parts. With regard to this, no uniform rule can be given. It will not always be possible to do without allowances for corrosion of the structure, especially if experience has shown that steel is liable to be severely attacked in the district in question. In Germany such districts hitherto found are happily only at isolated places, particularly on the coast. In all other cases there should, in my opinion, be no reason why the steel used as a structural material should be calculated with reduced stresses, in order thereby to give the steel an apparently longer life. I would even go so far as to describe this as an unnecessary waste of material.

3) Development in the adoption of steel in hydraulic engineering.

When it is considered that thirty years ago steel was principally used in Germany only for certain equipment of navigation service structures, such as bollards, ladders, mooring bitts and rings, edge protectors, anchors, etc., etc., it is remarkable that to-day it has been adopted for preventing the movement of earth by means of sheet piling to such an extent that more steel than wood or reinforced concrete is now used for such structures. The steel production for hydraulic engineering purposes in Germany is now estimated at more than 200,000 tons per annum in round figures. The figures for the output of a large German steel works give an idea of the increasing use of steel in hydraulic engineering (based on value in Marks):

1910: 100	1925: 196	1928: 1183	1932: 373
1915: 130	1927: 855	1930: 790	1934: 1150

During the last five years, steel piles particularly have been slowly but surely becoming more widely adopted and, in my opinion, their use will still continue to extend and to decrease the use of the two other materials, wood and reinforced concrete, unless reinforced concrete should also strike out in new directions for this purpose.

4) The possibilities of adopting steel.

The bulk of the steel used under present-day conditions in foundation work and hydraulic engineering is to be found in sheet piling structures, which are still being more and more adopted, since they are employed not only for finished structures, but also for temporary constructional work, for example for supporting foundation pits. They come into question not only with pile foundations, but also with solid foundation work (sinking with compressed air and well foundations).

The advantages of sheet piling lie in its economy and convenience, and in the speed with which it can be erected. The ramming makes a saving in excavation work and water retention. On the other hand, special demands are certainly made on steel for sheet piling, and they may be briefly defined as high values for yield point and elongation, high resistance to notching tests, great resistance to corrosion and to abrasion. These values vary with respect to each other in the different kinds of steel on the market, but can be developed in relation to each other in one and the same kind of steel. The various manufacturers of sheet piling supply special steels which may be described as all essentially similiar, and certainly as equally good.

These high-tensile steels will, however, be adopted only in exceptional cases; in general it will be found sufficient to use standard structural steel. When the latter is used, the profiles will be of greater cross-section than when high-tensile steels with greater permissible stresses are used; consequently the sheet piling structure will have less tendency to bend. Further, the greater liability to corrosion will in many cases be partly compensated, since with the greater quantity of steel in the structure a wastage of 1—2 mm will weaken the section less in percentage than would be the case with the thin-walled sections made of high-tensile steel. In addition, a greater moment of inertia may be of advantage in long sheet-piling walls when ramming, since the stresses which occur when forcing it into the ground, especially forces tending to cause buckling and vibration, are then diminished. Often it will be found impossible to avoid using high-tensile steels, particularly if the material is to be subjected to indentation and also to great abrasion during ramming operations.

From what has been stated in the second section of this paper, it can be seen that the final word has not yet been spoken concerning the suitable composition of high-tensile steels. In addition, from the point of view of economy, it is not possible to give any definite, general ruling as to when normal steel or high-tensile steel is preferable.

The demands made on sheet piling are the following:

High section modulus with little weight, i. e. a high coefficient of quality. Little deflection, conditioned by large depth of cross-sections. Staunchness of the wall, which will be attained essentially by the form and position of the locking joints. Good guidance for the locking joint, and little tipping or twisting of the wall during ramming.

When the different sections used for sheet piling are compared, it will be found that by far the greater number are of the single-lock single-wall type. The single-lock two-wall type (box section) and the two-lock two-wall type (Peiner section) come into question only for structures of fairly large size, and these are becoming more and more common in consequence of the increasing traffic demands made during recent years.

Advantages of the corrugated section as compared with the two-wall sections are that it is more easily possible to obtain a good junction of the structural parts, and that it is easier to ram, since there is less soil to be displaced and obstacles may be rammed round to a certain extent. In structures such as coffer dams, which require to be very well strengthened and stiffened inwardly, an endeavour will be made to manage as far as possible with corrugated sections, unless it es desired to use single-wall coffer dams, which are statically more exactly calculable, or even the double-lock two-wall section which is certainly of advantage in such cases.

In recent years trials have also been made with welded sections of many different shapes which, with the high development of welding technique in Germany, are to be considered as just as good as the rolled sections from the point of view of construction and service. Whether they will become commonly used will depend on their economy. It is left to the steel makers to strike out in new directions in this respect.

Besides the demands made on the material and on the section there are finally the demands on the finished wall. The erection of hydraulic structure by sheet piling methods is limited by the fact that these allow only plain flat walls to be constructed, and then only if certain unavoidable deviations from the intended line are permitted. With this method of working, only walls can be provided in which small irregularities play no important part. Further, since the wall is quite flat from top to bottom and the later provision of any recessing is almost impossible, the structure will always be of a rough block-like character. In many cases a compromise may be found possible, by cutting the sheet piling through above the water line and adding a reinforced concrete construction, the shape of which can be of more elaborate design. Nevertheless, the sheet piling method of construction cannot be considered at all for structures which are intended to make any display of elaborate work externally.

As already mentioned, the extensive use of steel sheet piling in Germany shows that it has now been adopted for constructional work of many different kinds. There is no need to refer to them here individually. It should be mentioned that, based on past experience in executing structures, there is to-day no longer any difficulty in driving sheet piling walls, 30—35 m long, into the ground undamaged, if the requisite precautions are taken. With the Peiner sheet piling we have thicknesses of section which are theoretically sufficient for banks of any free height, unless it is preferred to adopt the proved design of single-lock sheet piling walls, for example steel coffer dams.

Trials made with transverse welded sheet piling in order to simplify transport to site, have also shown that these walls behave well when being rammed. Reference may also be made to the practice, first introduced by me, of strengthening single-lock sheet piling by welding-on a flat iron strip where the greatest moment of resistance is required. Eight years' service has not yet shown any drawbacks.

From the sheet piling methods of construction, in consequence of the necessity of supporting vertical loads through the sheet piling, the adoption of steel for piles has also developed, after their great serviceability for this purpose had been determined from loading tests made on single-lock and two-lock sheet piling. The demands made on the steel for piles are the same as those made on steel for sheet piling. The section of the steel pile must conform to the following requirements:

In piles brought into the ground not by ramming but by screwing, the section must be of such a shape that it provides the necessary supporting resistance corresponding to the required carrying capacity of the pile, and the piles must also be as little as possible liable to damage from stones or other obstacles encountered underground. This leads to certain requirements regarding the width and pitch of the screw thread. This type of steel pile has scarcely been adopted at all in Germany, since its success would be very doubtful with the conditions found in our soil. On the other hand it is to be more frequently found in out-of-the-way districts, for instance in the tropics, since it is difficult there to acquire modern building equipment.

In contrast to the screwed piles, the rammed piles have only been adopted in Germany in recent years. In this respect we are still at the beginning of a development, for the number of such piles used up to the present is still small. The sections of the driven piles must show great friction between the pile and the soil, but without thereby causing the resistance to ramming to become insuperably great. Further, the cross-section of the pile must be amply large, so that the carrying capacity may not be limited by the permissible stressing of the material. The section must have a moment of inertia sufficiently high and as far as possible the same in all directions, so that the necessary factor of safety exists both when the pile is being rammed and when it is in service. In order that the pile may, at least approximately, take up its intended position in the soil, the section must be sufficiently resistant to prevent bending underground during ramming operations.

With regard to the design of steel piles, two different procedures have been adopted. In one an endeavour is made to obtain the required carrying capacity of the pile not so much by the surface friction of soil on steel, as by a great point resistance, but then, because of the large circular or rectangular cross-section, considerable quantities of earth must be displaced, unless resort is had to "washing" the pile in, in spite of all the drawbacks this method entails.

In the other method an endeavour is made to obtain the carrying capacity in the first place through the friction of soil on soil, in that, when ramming, the soil is artificially pressed so firmly in between the flanges of the I-shaped pile, that this pressure is greater than the friction of soil on steel. These piles have the advantage that they are comparatively narrow in the lower part and displace the soil only to a small extent.

The first kind of pile is of tubular or of box-shaped section; they are rammed with or without earth filling or with artificial points. However, since the soil forces its way into the section after a short time, it is of no great importance for the resistance to ramming whether the pile is fitted below with a concrete plug or not. Fitting such a plug may therefore be described as superfluous and uneconomical.

The whole earth filling of the pile is only washed out afterwards and replaced by concrete, provided the water is of an exceptionally corrosive character. If the steel wastes away later, a concrete pile remains; if found advisable, iron can also be previously embedded in the concrete.

The open sections consist of T-beams without bulb (flat iron tips), which possess a comparatively low carrying capacity. For this reason, the double T-beams have been provided with a bulb which, when of the right length and in the right position, can greatly increase the carrying capacity of the piles in compression or in tension.

More suitable than double T-beams is the use of broad-flanged sections with or without bulb. If the Peiner sheet piling is used, piles are obtained by ramming separate units together, and within these piles the provision of bulbs is rendered superfluous, since the soil very quickly presses in between the separate flanges. In this case an intermediate construction between closed and open sections is obtained.

Further open sections are U or Z sheet piling, which can be rammed as separate units or with several set together. In this case a bulb has hitherto not been provided, since these cross-sections are used mostly when the vertical stressing is only a partial stressing, and the principal forces occur horizontally.

Welded beams have not yet made their appearance as steel piles. But there seems to be an extensive field open for their adoption here, since they are capable of being suited to the special requirements of the rammed pile much better than rolled sections.

The carrying capacity hitherto determined by tests on various kinds of steel piles is very high in comparison with wooden piles, and almost the same as for reinforced concrete piles. In this connection no great difference could be found between open and closed sections. The figures obtained from experience with compression piles vary between 80 and 120 tons, with a settling of 2—8 mm at the maximum for bulb section piles. Loading of Peiner double sheet piling in the wall gave a carrying capacity of about 300—350 tons with a settling of 15—20 mm. There is very little information available concerning tests on the resistance of tension piles, so that no conclusions can be formed from them regarding the tensile strength of steel piles. In Bremen 70 to 80 tons was found, with a permanent rise of 3 to 4 mm.

The adoption of steel piles to any great extent has hitherto been confined to the strengthening of quay walls in Hamburg and Bremen.

Experience has however shown that open piers should not be constructed with steel piles, since they offer too great surfaces for attack by corrosion. They should therefore be enclosed by sheet piling and surrounded by earth.

For a long time steel jackets for reinforced concrete piles have been used when the reinforced concrete pile must pass through ground which is liable to attack the material, and also in cases of special constructions, such as tube and press piles, where the tube is again withdrawn. No special demands are here made on the steel. There are no great innovations to be mentioned in this connection.

Steel is also adopted instead of reinforced concrete and masonry in well shafts when water and soil layers liable to attack the material have to be passed through when sinking. Nevertheless, the execution of well shafts in steel is very much rarer than execution in concrete.

In the same way steel caissons are not so widely used as those of reinforced concrete, since steel with a concrete filling only seldom proves sufficiently economical for this method of construction. It will be adopted when, in consequence of unfavourable building ground, the building materials are to be subjected to very great stressing, which cannot be ascertained in advance. In service the steel caissons show no drawbacks, as is proved by the large works carried out by German firms at home and abroad.

Finally, mention should be made of certain parts of the equipment of traffic structures in waterways, to which I have previously referred briefly. These are principally bollards, ladders, mooring bitts and rings, corner protectors, etc.

The chief requirement of the steel here is a very great resistance to abrasion. Since the quantity of steel coming into question is always small, the composition of the material may be chosen of much higher quality than is possible in the case of the great quantities of steel for the main parts of the structures.

With corner protectors, care must always be taken that it is possible to obtain really effective attachement to the masonry or concrete, since, in consequence of the shrinkage of the concrete and fairly great expansion, a space will be formed between the steel and the concrete.

5) Possibilities of development.

As will have already been seen from the production figures given in section 3. the use of steel in foundation work and hydraulic engineering has progressed with extraordinary rapidity during the last 15 years. This is doubtless to be attributed to the fact that the makers have been doing everything possible to improve the material and to develop the shapes of the structural members, in order to make them suitable resistant to the various stresses.

We must, however, clearly understand that in steel we do not possess a material which can eliminate all the drawbacks of wood and reinforced concrete. Steel is particularly subject to corrosion, and this still limits the possibility of adopting it.

It will be easier to get an increase in the moment of resistance of sheet piling and piles once the rolling mills have made considerably more progress towards producing larger sections, and welding technique has given us the possibility of adopting shapes which can suit requirements in any particular case.

The obtaining of a steel which will fulfil the stipulated requirements more efficiently is the case at the present day, can only become possible by close collaboration between physicists, chemists, steel makers, owners of rolling mills. mathematicians, designers and builders, whereby the experience gained with works already executed will play a very important part.

Summary.

The paper gives a compilation of experiences gathered with the application of steel for underground and hydraulic structures. The physical and chemical behaviour of steel is explained, if subjected to the influences of air, water, and alternating changes between water and air. The means of keeping these deleterious effects as small as possible are mentioned, as well as the life timo of steel structures. The counteracting measures to reduce destruction are: the composition of steels, painting, metal covers, encasing in concrete, and the shape of structural parts. A short description of the development of steel production for the use in hydraulic structures is given, in particular steel sheet piling. The claims put in steel sheet piling are discussed. Steel can also be used for piles, wells, caissons, and equipment such as bollards, ladders etc.

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Steel Pressure Pipe for the Hydro-electric Plant "La Bissorte".

Le tuyau d'acier de l'usine hydro-électrique de "La Bissorte".

Stahldruckrohr des Kraftwerkes „La Bissorte".

J. Bouchayer,
Administrateur-Délégué des Etablissements Bouchayer et Viallet, Grenoble.

Steel tubing has quite recently played a prominent part in the arrangement of the hydro-electric power station at "La Bissorte", whose delivery pipe carrying the water to the turbines is made entirely of steel from its very starting point (that is, from the source of supply) and this delivery pipe is one of the outstanding modern pipe lines, not only on account of the height of the head under which it operates and which exceeds 1,000 metres, but also on account of the energy developed, which works out at 100,000 h. p.

It would therefore seem opportune to submit a few facts about the installation of this piping, which is remarkable both for the originality and for the boldness of its design.

Description of the Pipe Line.

The entire installation is of metal, its length being 3,037 metres, no less than 3,800 tons of steel was used for its construction. It starts at the Barrage, or dam, at an altitude of 2,028 and finishes at an altitude of 936. The level of the dam is 2,082; if account be taken of the hydraulic recoil produced by the turbines, the piping of the lower portion is subjected to the very heavy pressure of 132 kg per sq. centimetre.

Diameters have been calculated for a volume of 7.5 cubic metres per second.

The pipe line consists of two main parts:

One part with a small gradient over a length of 1,080 metres with a diameter of 1.8 m, is installed entirely in a passage cut in the rock; the tubes are electrically welded and equipped from the very start with all the necessary cocks, valves and safety devices.

The other part has a steep gradient over a length of 1,957 metres; some of the pipes along this portion are water gas welded, their diameter being 1.4 m, and the remainder are armoured. The diameters of these latter are 1.4 and 1.3 m. and the remainder are armoured. The diameters of these latter are 1,4 and 1,3 m.

These also are fitted from the start with valves and safety appliances.

These two main parts are united in an equalising chimney shaft with a diameter of 2.5 m and height of 70 metres situated in a vertical pit cut out of the rock.

The lower end of the pipe line, which before reaching the power station crosses the Mont Cenis railway line from France to Italy, is terminated by a collector fitted with three branch pipes, each of which feeds a 34,700 h. p. turbine.

The erection of the pipe line was exceedingly complicated owing to the very steep gradients of the slopes and the individual weight of each piece of tube. Some of the latter actually weigh as much as 15 tons.

The power of pressure piping is expressed by the product HD^2 (H being the height of the head, D the mean characteristic diameter).

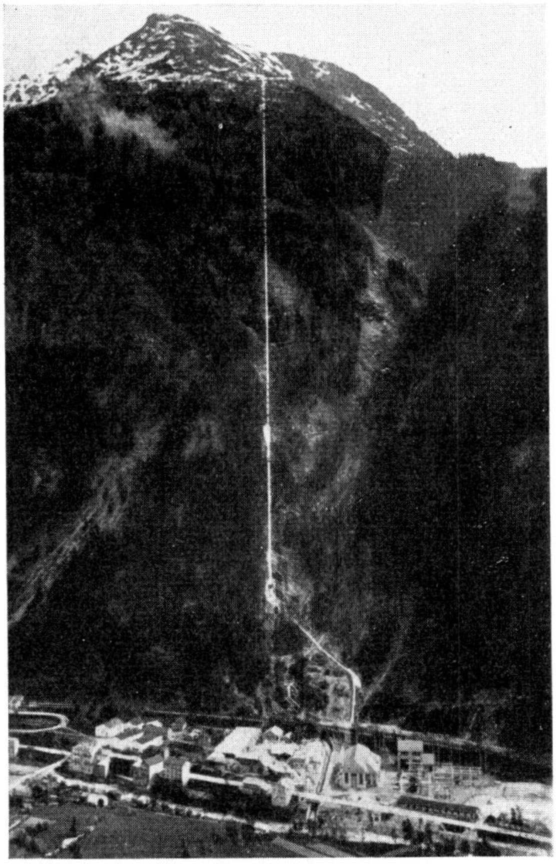

Fig. 1.
"La Bissorte" Pressure Pipeline. General view.

The pipe line of "La Bissorte" is remarkable from this point of view and sets up a record with $HD^2 = 2360$.

Its dimensions were calculated to resist an atmospheric vacuum and a static pressure corresponding to the maximum level of water in the reservoir plus either a 15 per cent. linear overpressure due to hydraulic recoil when closing the turbines, or to overpressure resulting from hydraulic recoil caused by oscillations which would produce a uniform overload of 55 m, admitting for each the maximum value of the two overloads in question.

The fatigue strengths under these maximum pressures would be the following:

8 kg/cm^2 for the metal plate,

24 kg/mm^2 for the spiral reinforcement,

2 kg/cm^2 for the pressure of the masonry work which acts as an anchor block.

The sheet metal used for the pipes was Siemens-Martin extra mild steel with a 35 kg/mm^2 tensile strength and 30 per cent. longitudinal pull respectively. The reinforcement to armour the tubes is made of treated special Siemens Martin steel having minimum coefficients of 90 kg/mm^2 tensile strength, 60 kg/mm^2 limit of elasticity and 8 per cent. longitudinal pull.

Low gradient section.

This part includes:

Anchor piece of pipe with inside diameter 1.8 m from the outlet gallery (altitude 2028.86) to the sluice chamber (altitude 2028.78) with a gradient of 0.001 m per metre, made of electrically welded tubes with walls 12 mm thick, length of each section of piping 9 metres, these assembled on the site by socket joints and butt welded with the electric arc.

The tubes in the passage or gallery are entirely concreted in order to form an anchoring plug and to be absolutely water tight. After concreting, pressure injections were carried out through the wall of the sheet metal along the whole of the concrete plug.

A pipe line of 1.8 m inside diameter and 1000 metres long, leading from the sluice chamber (altitude 2028.78) to the spherical valve fitted at the point where the steel slope (altitude 2015.36) starts with a gradient of 0.0134 m per metre, erected entirely inside a gallery large enough to ensure the passage of inspectors and attendants and access to the regulating flap.

This pipe line of electrically welded tubes 8 and 9 mm thick and in sectional lengths of 9 metres, was assembled on the site of erection by sleeve joints and rivets.

Reinforcement hoops, at intervals of 3 metres, reduce the effects of fluctuating atmospheric pressure.

Two intermediate anchoring blocks which keep the bends in position, ensure strength which is further increased by pedestals of masonry work with steel saddles. The pedestals or supports are placed at equal distances of 9 metres.

There are no expansion joints in the pipe line, altough the bends are anchored in position. The reason for this is that temperature fluctuations within the gallery are but very slight.

Equalizing shaft.

This has a diameter of 2.5 m and branches just above the point where the low gradient section meets the steep gradient portion.

The equalizing chamber includes a horizontal part placed inside a gallery 62 metres long and another 65 metres high situated in a vertical shaft.

It is made of arc welded tubes with walls 8 to 15 mm thick, the section lengths are each 6 m, they were assembled in situ, rivets being used for the horizontal part and welding for the vertical part. Each tube of the horizontal portion rests on a masonry pillar and has a steel saddle. The tubes in the vertical part are surrounded by concrete which fills in the space between metal and rock.

Steep gradient section.

This part includes:

Tubes welded by water gas — inside diameter 1.4 m length 522 metres up to level 1,704 metres, passing in turn through the horizontal passage or gallery, then through the gallery inclined at a gradient of 0.849 m per metre over a length of 229 metres and then reaching the open air.

This section, which has no expansion joints, is made of water gas welded tubes 11 to 39 mm thick; they were assembled on the site of erection by socket jointing and riveting.

The bends are joined to the straight pieces of piping by tongued flanges and bolts.

The blocks of masonry are spaced along the profile of the water conduct in such a way as to leave the bends free to play their part in equalizing the longitudinal stresses due to temperature fluctuations.

In the spaces between the blocks, the pipe line rests on masonry pillars situated at intervals of 12 metres and fitted with steel saddles.

A section of armoured tubes with an inside diameter of 1.4 m, length 152 metres up to an altitude of 1,632 metres.

This section is erected in the open air.

It is made of plates 12 mm thick, spiral reinforcing 60×22 to 60×26; the tubes were assembled in situ by riveted sleeve joints and by special sliding joints, one for every three tubes.

All the bends are held down on masonry blocks and connected to the straight pipes by sliding joints.

The straight parts are also fixed to the intermediate blocks at intervals of 19 metres; in these spaces the pipe line rests on pillars 6.40 metres apart and with steel saddles. Between each block that is, at equal distances of 19 metres free expansion of the pipe line is secured by sliding joints which do the work of expansion joints of short dilatation range.

A section of armoured tubes, inside diameter 1.3 m, with riveted sleeve joints, length of tubing 827 metres up to a height of 1,120. This section passes first of all into the passage and then out into the open.

It is made of plates 12 to 20 mm thick and with reinforced spiral casing 60×24 to 80×48. The length of the tubes, the method of assemblage and the bolting appliances are identical with those of the preceding section.

A section of armoured tubes, inside diameter 1.3 m, fitted with sliding joints, length of pipe line 451 metres, leads to the collector (altitude 936.70 m) erected partly:

— in the open, partly

— on a metal footbridge (crossing the "Bissorte" torrent) and partly:

— in the interior of a metal-plated passage under the railway line.

It is made of tubes 22 and 24 mm thick with spiral reinforcement 90×48 to 100×54. The tubes are assembled by sliding joints.

Just as in the preceding sections, this part of the pipe line also has its bends clamped and the straight parts are fixed at intervals of 19 metres, free expansion between the clamped parts being ensured by sliding joints. Furthermore, the pipe line rests on pillars with steel saddles; the distance between the pillars is 6.4 m.

A collector with inside diameter 1.3 m — 1.1 m — 0.8 m horizontal, length 38 metres including three branch pipes for connecting the line to the turbines.

It is entirely encased (except for the bifurcated parts) in a block of masonry which forms part of the foundations of the power station. The tubes are armoured, thickness of the metal being 35 mm; they are assembled by collars and the branch pipes are of cast steel.

Method of construction.

The construction included mainly the manufacture of tubes of certain lengths and of three different types:

Electric arc welded tubes,

Water gas welded tubes,

Armoured tubes.

The raw material was submitted first of all to the Steel Works for approval by a special Checking Department placed under the direct supervision of the Superintendent of the work. On arrival at the workshops they were checked up by their casting and series numbers and thereupon sorted and arranged in lots according to their kind and size.

Index cards referring to their manufacture were then filled in, mentioning dimensions of the various parts and their reference marks. All the manufacturing operations were given in full on these cards, so that by consulting them a strict control could be kept on the whole course of manufacture of any tube.

Electric arc welded tubes.

The manufacture of these tubes was carried out according to the usual sequence: planing, bevelling, bending, welding, hydraulic testing, machining and painting.

In view of the comparatively light thickness of the metal, the bending was effected while the metal was in a cold state and by means of a vertical hydraulic press. Coated electrodes were used for the electric arc welding and melted into V-bevelled joints after which heat treatment was applied to the apex of the V. Longitudinal welds were automatically machined, while the transversal welds were hand made.

Water gas welded tubes.

The operations of planing, bevelling and bending were carried out as in the preceding case. The gas for the water gas welds was obtained by pouring water vapour on incandescent coke. This gas, having a high content of hydrogen and carbon monoxide, has reducing properties and when it burns has a deoxidising effect. When welding the ends of the plates these are made to overlap each other and no deposit metal is applied. The ends of the metal which overlap are brought to a white welding heat by the reducing flame of the blowpipes heated by water gas and air. After this they are hammered.

These welds lead to deformation of the tube, induce high internal stresses and overheat the metal. In order to eliminate these stresses and improve the metal in the welding zone, the shape of the tube which has just been made is corrected by annealing at a temperature of 950⁰ (the temperature being controlled by a pyrometer).

Advantage is taken of the annealing operation to place the red hot tube in a special re-shaping machine. This operation is effected very rapidly because it must be terminated before the temperature of the tube drops below 500⁰. After

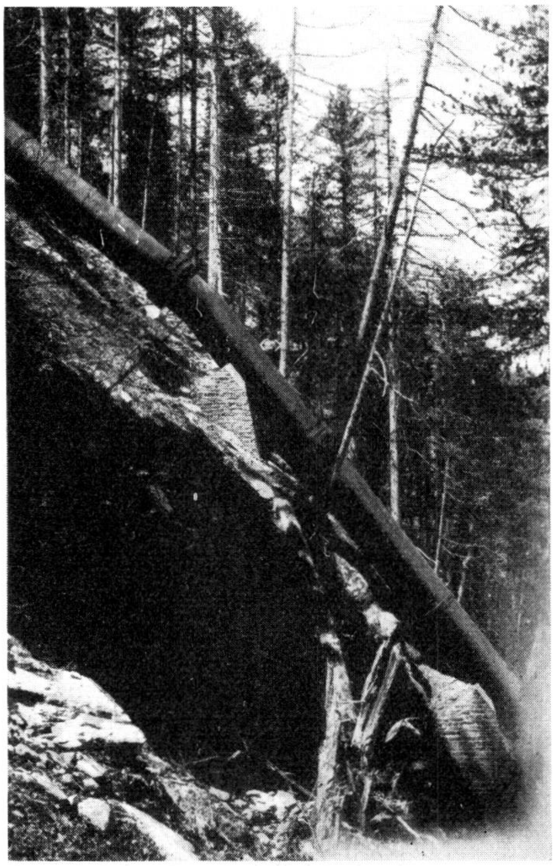

Fig. 2.
Water-gas welded portion öf pipeline at level 1810.

the operations of annealing and re-shaping, flanging machine is used and while the metal is still hot the ends of the tubes are widened to facilitate their assemblage.

The ends of the tubes are then polished on a lathe, after which follow hydraulic tests, machining and painting.

With regard to the pipe line at "La Bissorte", the longitudinal welds were carried out in a special automatic machine suitable for tubes with a diameter of 3 m and a length of 6.50 m, and in which hammering can be done by a compressed air rammer. The transversal welds were executed by hand hammering and the shift of workers included on occasion three operators for hammering the heavy plates.

Annealing was effected in a large furnace fed by gazogene and of such dimensions that tubes of 3 m diameter and 6.50 m length could be dealt with.

Temperature was checked permanently by a recording pyrometer kept in the foreman's office.

Four-cylinder bending machines were used for re-shaping or curving. These were arranged so as to allow of the welds being rolled as soon as operations started.

Armoured tubes.

As this is a novel system not applied before 1925, it might not be out of place to describe the process, which is one that is commonly practised by armament manufacturers when making heavy guns.

The operation when dealing with tubes is briefly the following:

Assuming the tube to be of appropriate steel the contents of which are known, with local thicknesses calculated in such a way that after the reinforcement, the

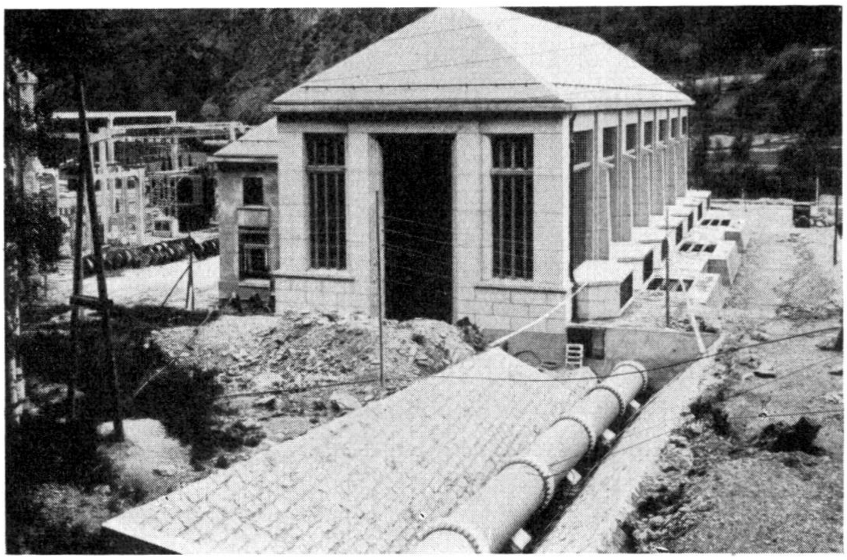

Fig. 3.
Pressure Pipeline on arrival at powerhouse.

cannon will be made of one single piece of metal (without addition of outer spiral reinforcing).

In its original state, the tube could not comply with the service conditions laid down for the muzzle designed.

The resistance necessary to fulfil its aim not being attained, it is secured by self-reinforcement and the increased strength is exactly determined and checked by the operation itself.

Assuming further that a powerful hydraulic press is available and that it communicates with the interior of the tube, the two extremities are then closed down by an absolutely watertight joint.

The liquid under pressure is poured into this tube while the pressure is gradually increased up to a point at which it slightly exceeds the figure corresponding to the limit of elasticity of the tube, remaining however, definitely below breaking point.

The pressure is maintained for a certain period at this degree and then gradually dropped until it corresponds with that of atmospheric pressure.

The tube which is expanded beyond its limit of elasticity maintains distortion permanently, while the component metal becomes harder and denser.

This kind of hydraulic expansion induces a tensile state in the outside layers of the tube while the internal layers are subjected to compression.

This complicated phenomenon is called self-reinforcement because it gives the impression of the tube having acquired increased strength without any outer force intervening.

There is no need to dwell further on this technical aspect of artillery manufacture nor to go into the manifold uses to which this new process has been put.

We will merely bear in mind two main points:

1) The fact that in the case of the tube under consideration, it has been permanently distorted without reaching breaking point and under a known pressure which exceeds its original limit of elasticity and it is thus enabled to withstand any further renewal of this same pressure without any permanent deformation being produced. It has thus acquired a limit of elasticity which is higher than that which it possessed in the first instance.

This therefore represents a change of mechanical properties.

For instance, self-reinforcement, carried out under precisely determined conditions, will be found in the case of a tube thus hardened to have increased its limit of elasticity by one-third and its tensile resistance by 8 per cent.

As a matter of fact, by means of this self-reinforcement, a steel tube of new metal can be manufactured to dimensions which can be definitely calculated and so comply with certain service requirements.

2) The application is of particular interest in that the pressure produced by the desired hardening represents simultaneously a proof of resistance which is both very great and very exact, since it corresponds to the service pressure affected by a multiplier which, ceasing to be theoretical, has taken on a definite value. Manufacture and control are thus carried out by a single process and the final product offers absolutely scientific guarantees of resistance and lightness while the process itself is a remarkably inexpensive one.

The service conditions which have to be complied with by pressure pipe lines are obviously not identical with those set up for tubes used in the manufacture of heavy guns.

But what has been said above about self-reinforcement of these guns explains the application of a process of the same kind to pressure piping.

The aim is the same: namely, that of securing the best combination, in the circumstances, of resistance and ligthness together with controlled scientific guarantees.

A self-reinforced tube required for pressure piping consists of:

— — — a tube, generally of welded steel plate, which forms the inside wall, the thickness of the latter being a fraction of that necessary for an ordinary tube possessing the same resistance, and:

— — — reinforcement rings equally spaced along the outside surface of the wall.

These rings are of rectangular section, rolled, weldless, made of steel treated for robustness, and having a limit of elasticity which is considerably higher than that of the tube sheet metal, this being an essential condition when reinforcing by this process.

The next point is to see how the manufacturing process will endow the wall of the tube and the rings with a combination of stresses which will provide a solution to the problem of resistance combined with lightness.

The outside diameter of the wall of the tube is slightly less than the inside diameter of the rings; these can thus be placed at appropriate distances when the metal is cold.

A strong hydraulic press is brought to bear on the two ends and these are closed so as to be absolutely watertight.

Pressure is increased progressively until it is twice the running pressure (static pressure plus overpressure) required when the tube will be in commission.

This maximum pressure is called "pressure of reinforcement", and once it is reached it is maintained for a period of a minute and then allowed to drop to running pressure. The tube and rings are then hammered, any control or other measures which may be necessary follow on, and the wall is examined to see whether it has behaved satisfactorily during the period of deformation. Pressure is then increased up to the rate of reinforcing pressure, and this is maintained for at least five minutes.

The operation of reinforcement and the pressure tests can then be considered as terminated.

Il would be well to add a few observations to this brief exposition in order to emphasise the usefulness of this manufacturing process:

1) The progressively increasing action causes the wall of the tube to expand and it then begins to adhere to the rings.

Pressure goes on rising, the tube tends to expand the rings and the metal is then found to be in a state of tension.

After a definite drop of pressure, the wall of the tube which has exceeded its original limit of elasticity becomes permanently deformed; it acquires a new limit of elasticity, higher than the previous one and is strengthened by the self-reinforcement. Meanwhile, the rings, owing to the great tensile strength of their component metal, have undergone elastic deformation only and set up a tightening of the outside wall analogous to that which would be produced by the contraction of rings fitted to the tube when in a hot state.

2) The experiment proves that, where strength while in commission is the same, a tube which was self-reinforced in this way would weigh half what an ordinary welded tube weighs.

3) The process offers a great advantage in that the metal can be watched in its various stages of manufacture; the strains it undergoes can be followed with the help of suitable apparatus, and action taken if necessary in order to adjust the pressure of the reinforcement required so as to obtain the best nominal conditions.

4) Finally, the manufacturing process supplies all the tests required to check the results obtained.

When the operation has been concluded, the self-reinforced tube is a guarantee of strength equal to at least twice the working pressure aimed at because this guaranteed pressure has made its production possible.

The pressure tests made after erection will thus be required only for controlling the leakproof properties of the joints assembling the various tubes.

5) As a matter of fact the self-reinforced tubes might be compared to weldless tubes, that is, they are very robust.

Consequently, their use for pressure piping has proved to be very great when their calibre exceeds that of unwelded tubes, and also in cases where ordinary welded tubes are not strong enough or require such thickness that they would be eliminated by considerations of weight or manufacturing problems.

The main operation of manufacture of the self-reinforced tubes is thus seen to be that of strengthening, a process which at the same time constitutes a hydraulic test at a pressure which is at least double that of the maximum working pressure (static pressure plus overpressure).

The result is that the most important plant for the construction of self-reinforced tubes is a hydraulic testing press which can deal with large dimensions when pressure piping such as that of "La Bissorte" has to be manufactured.

A very important point on which we must insist concerns the control of the stresses in the material which can be effected thanks to the method of manufacturing self-reinforcing tubes.

Before applying pressure, a certain number of tensometers are placed on the rings, these tensometers being distributed uniformly along the length of the tube which is to be reinforced.

While the process of reinforcing ensues, the first thing to do is to record the pressure when bringing the wall into contact with the rings, then the tension of the rings under increasing pressure up to the pressure of reinforcement, then under decreasing pressure to the working pressure, then again under the reinforcing pressure and under the operating or working pressure and finally, with the pressure at nil, so as to control the degree of tightening of the rings on the wall.

While these operations are proceeding, the pressure of reinforcement calculated as necessary to carry out the nominal tensions can be corrected in good time.

This control, effected during the process of manufacture or when systematic experiments are being carried out, has shown that the real stresses correspond with the nominal ones very satisfactorily and it proves that a very wide margin exists between the limit of elasticity of the reinforcement rings and their maximum tension as a result of the reinforcement pressure.

The wall of self-reinforced tubes is generally made of extra mild steel having the following coefficients:

Tensile strength	$\geqslant$ 34 kg/mm^2
Limit of elasticity	$\geqslant$ 19 kg/mm^2
Longitudinal pull	$\geqslant$ 30 %
Mesnager notching action . .	$\geqslant$ 7 kg/cm^2
Permissible stress	8 kg/mm^2

or of special very ductile steel with the following coefficients:

Tensile strength	$\geqslant$ 54 kg/mm^2
Limit of elasticity	$\geqslant$ 36 kg/mm^2
Longitudinal pull	$\geqslant$ 20 %
Mesnager notching action . .	$\geqslant$ 7 kg/cm^2
Permissible stress	12 kg/mm^2.

In the first of these cases, the reinforcement rings were made of treated steel with the following mechanical properties:

Tensile strength	$\geqslant$ 90 kg/mm^2
Limit of elasticity	$\geqslant$ 60 kg/mm^2
Longitudinal pull	$\geqslant$ 8 %
Mesnager notching action . .	$\geqslant$ 4 kg/cm^2
Permissible stress	24 kg/mm^2

and in the second case they were made of treated special steel having the following properties:

Tensile strength	$\geqslant$ 115 kg/mm^2
Limit of elasticity	$\geqslant$ 95 kg/mm^2
Longitudinal pull	$\geqslant$ 6 %
Mesnager notching action . .	$\geqslant$ 5 kg/cm^2
Permissible stress	36 kg/mm^2.

The average fatigue strength of a self-reinforced tube calculated in proportion to the total section of the wall and the reinforcement ring is 16 kg/mm^2 in the first case, and 24 kg/mm^2 in the second. This means that a tube made in this way has a weight which is half that of an ordinary tube whose resistance coefficients would be 8 and 12 kg/mm^2 respectively.

The advantage of using self-reinforcing tubes is not limited solely to pressure piping with a very high head of water, for experience has shown that it is an economic proposition to use them from the time when maximum working pressure reaches 320 metres, but that their limit of utilisation is far wider in certain specific cases and also when the question of safety is of very special importance.

Besides the saving made on the initial cost when self-reinforced tubes are used and the great security they offer, these self-reinforced tubes have another advantage for those who operate hydro-electric works.

This appliance offers, in fact, the advantage of reducing considerably the hydraulic recoil owing to the large diametral expansion of the tubes under the effect of internal pressure consequent upon the degree of fatigue strength that can be admitted for the metal of which it is made.

As a matter of fact the hydraulic recoil depends on the speed or velocity of wave propagation and diminishes with the value of the latter.

The velocity in the case of steel tubes has the following coefficients:

$$a = \frac{9.900}{\sqrt{48.3 + 0.5\frac{D}{e}}} \quad \text{now: } \frac{D}{e} = \frac{2R}{p} \quad \text{therefore: } a = \frac{9.900}{\sqrt{48.3 + \frac{R}{p}}}$$

taking for instance:

— internal pressure: . . . $p = 1000$ m of water or 1 kg/mm^2
— degree of fatigue strength: $R = 8\ kg/mm^2$ for ordinary tube,
$R = 16\ kg/mm^2$ for self-reinforced tube.

In the first case: a = 1320 metres and:
in the second case: a = 1230 metres,
which confirms what has just been stated.

The self-reinforced tubes which constitute the larger part of the pressure piping of "La Bissorte" were constructed of gas welded tubes. In order to

Fig. 4.
Armoured portion of entering inclined gallery at level 1220.

obtain great homogeneity at the time of self-reinforcing, the precaution was taken of making each tube with sheet metal belonging to the same casting.

While self-reinforcing proceeded the rings and welds were hammered vigorously.

On the other hand, for a certain number of tubes, measurements on wall and rings were effected from a pressure starting at nil to the reinforcement pressure and inversely by aid of Huggenberger tensometers. By this means it was possible to verify that the real and nominal resistances were in line.

At the time of reinforcing, strong calibration plates were fitted to the ends of the tubes and thus perfectly calibrated elements were obtained and the machining could be done along lines of repetition work without final touching up of the plates of the couplers fixed to the ends of the tubes.

The plates at the ends of a certain number of tubes were welded to these with the electric arc, and additional pressure tests were carried out to ensure the leakproof properties and soundness of these welds.

Painting in the workshops.

Before despatching the tubes they were brushed, the slag was removed and they were then tarred inside and out while in a hot state.

Before tarring was effected, the tubes were heated with powerful gas blowpipes at a temperature of about 80^0, and then plunged into a large tank of tar previously raised to the same temperature. When taken out of the tank they were found after the tar had dripped off them to be impregnated with a protective coat of tar which adhered very closely to the tube.

Inspection.

The inspection of the manufactured tubes was carried out on the one hand by the Contractor's Permanent Checking Department, which was responsible for effecting this control during manufacture by cutting off sample test pieces, checking the welds, applying hydraulic tests and verifying the dimensions, while, on the other hand, intermittent checking was effected by another Department under the Superintendent of the work.

Each tube was stamped and a report drawn up on the results of the inspection; particular attention was paid to the hydraulic pressure.

Hydraulic test.

The hydraulic test which is the main operation in the manufacture of self-reinforced tubes is the most important operation in the revision and inspection of pressure piping, since it determines the minimum coefficient of practical safety.

All the tubes of the pressure piping at "La Bissorte" have been tested for double the maximum pressure (static pressure plus overpressure) that they have to withstand when in commission.

To that end a "hydraulic test pressure" of 3 500 tons[1] capable of testing tubes up to 3 m diameter and 13 metres in length was used.

We have already explained how the hydraulic test was carried out at the same time as the self-reinforcing operation for the self-reinforced tubes.

When testing electric arc welded tubes or water gas welded tubes, the pressure was raised in the first place to double the maximum working pressure and kept there for a minute, then lowered to the working pressure at which the welds were thoroughly hammered; after that the testing pressure was maintained for at least five minutes.

The finished tubes were then taken to the site by motor lorry and in such quantities as were required for erection.

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[1] The load of 3500 tons corresponds to the hydraulic thrust on the press rams in the course of tests made with that portion of tubes below 264 kg/cm^2.

Erecting the tubes.

The first and most important of the problems to be solved in order to carry out the work of erection satisfactorily was that of transporting the tubes to the final site; here the weight of the various parts, the uneven ground over which they had to be transported and the very steep gradient of the slope, to which must be added the presence of the bends in a horizontal position and the inclined galleries presented great complications.

The following was the solution selected:

Transport by overhead cable of all the elements for the region lying between levels 1030 m and 2015 m, and in addition to this special plant was provided for lowering the tubes for each gallery in this zone.

Transport on an inclined plane of the elements in the region situated between levels 945 m and 1030 m.

No special difficulty was encountered when erecting the section situated just below that just mentioned, because the slope was a gradual one and access therefore easier.

The programme depended on the following factors:

- the transporting capacity of the overhead cable;
- need for carrying out simultaneously the laying of the piping and the civil engineering work;
- impossibility of increasing the shifts on account of risks of accident.

The work had to be divided into two sections.

The first included that carried out in the region between 1030 m and 1650 m and the section between the power house (inclusive of the collector) to the third anchorage block.

The second section was not so heavy, and it was here that the work was terminated by joining the pressure piping on the low gradient to the three turbines.

The erection work was started at four places, but work preceeded simultaneously on two out of four at most; when work was checked or there was a breakdown on one section, operations were at once continued on one of the others. This method proved necessary in particular whenever a gallery was entered or working shifts and masons had to relieve one another.

From the time erecting was started an initial experiment was made with hydraulic pressure on one of the first sections laid and rivets made on the site capable of meeting a normal working pressure of 100 kg/cm^2 were tested for leakage.

A second test, this time more exhaustive, was made after all the self-reinforced tubes (the collector included) had been erected, in other words, the whole section from the power station (level 936.7 m) to level 1715 m.

These tests aimed in the first place at verifying that the joints were leakproof and then of controlling the strength of the pressure piping and the anchorage blocks when subjected to the maximum pressures which the piping would have to stand in the long run.

The pipe line was first filled by little streams of water from above; pressure was increased by means of an electropump set up in the power station; this delivered the water direct to the collector. Pressure gauges were placed at the two ends of the section under test.

On the first day pressure reached 115 kg/cm² in the collector, then on the second day 124 kg and on the third 132 kg. At that time the pressure registered by the manometer placed on the upper part was 54 kg/cm².

As compared with the static pressure the over-pressure induced in the section under test was 15% in the collector and 50% in the upper part.

The test, which was a very severe one, in particular for the upper parts of the section tested, gave rise to no observations. Thanks to the tests it was possible to ascertain that the joints and rivets were all perfectly leakproof and that the clamping blocks, when subjected to very high stresses, stood absolutely firm.

As soon as erection had been completed the piping was cleaned and then given a coat of bituminous paint.

The inside paint which had been passed at the Works but damaged during erection was touched up; those parts which were exposed to the air were given a coat of paint and two coats were given to the parts inside the passage.

The lower part in the neighbourhood of the power station was furthermore given an additional coat of aluminium paint.

In October 1934 the whole plant was ready to be put in commission and the hydro-electric works started working in May 1935, from which time it has given ample proof of the beneficial effects of its well regulated power.

Summary.

The plant at the Bissorte Falls, which consumes a volume of 7.5 m^3 under a head of 1150 metres, develops energy to the extent of 100,000 h. p.

It consists of one single turbine pipe line having a diameter of 1.3 m at the base and 1.8 in the upper part.

3,800 tons of steel were used to construct this turbine pipe line, which has a total length of 3.037 metres. It feeds three turbines of 34,700 h p. each.

When work is normal the tubes of the lower part are subjected to a maximum pressure of 132 kg/cm².

Thanks to the introduction of self-reinforced tubes into the design, this pipe line has been constructed in such a way that it offers perfect guarantees from the point of view of design and execution, together with very great advantages from the economic point of view.

The self-reinforced tubes consist of cylindrical tubing and reinforcement rings, these latter have an internal diameter which is slightly greater than the diameter of the tube in question.

In order to ensure the rings being a tight fit on the tubes, the process applied is not the hot one as that would eliminate the possibility of using treated high tensile steel for the rings, but a process carried out while metal is in the cold state, namely, by extending the wall obtained by applying a certain pressure, called "reinforcement pressure", to the inside of the tube.

The advantages of this process of self-reinforcement are many:

1. The permanent deformation to which the tube is subjected does not deprive the latter of any of its good characteristics; on the contrary, it endows it with new qualities inherent in self-reinforcement.
2. The self-reinforcing pressure required so that the tube may become self-reinforced constitutes at the same time a very severe resistance test. By one and the same operation the tube is manufactured and controlled at a pressure which is at least twice its maximum working pressure.
3. Controll of fatigue strength of the metal can easily be effected by help of appropriate appliances.
4. From the point of view of saving, the reduced thickness of the wall and the use of unwelded reinforcement rings made of steel treated to resist high pressures allows of reducing the amount of metal required, that is, of weight, very considerably.
5. With this type of tube the hydraulic recoil is greater reduced.

A large number of pipelines have been constructed with self-reinforced tubes, among them being the following:

Les Sept-Laux	(1050 metres)	
Le Lac Mort	(625 metres)	
Le Vintrou	(276 metres)	diameter of 2.00 m
Escaldès	(492 metres)	
Eylie	(1040 metres).	

La Bissorte was put in commission in May 1935 and has confirmed the excellence of this new technique applied to turbine pipe lines and devised by M. Georges Ferrand, Delegate-Member of the Board of the Dauphiné Company for Investigation and Erection, at Grenoble.

VIIb 3

Use of Steel in Hydraulic Structures, Movable Plants.

Anwendung des Stahles im Wasserbau, bewegliche Anlagen.

Application de l'acier en construction hydraulique, installations mobiles.

Ministerialrat K. Burkowitz,
Reichs- und Preußisches Verkehrsministerium, Berlin.

The Material "Steel".

While in general I may refer to the contribution of Prof. Dr.-Ing. *Agatz*, I should like personally to say something with regard to the special section "movable plants".

Movable plants are more exposed to all external influences than the "fixed plants" are. The water, which often flows past them with considerable speed and force, the alternations between wet and dry state, cold and heat, and the adverse effects of external forces — these are all factors which in many cases, if not the majority, put a bigger strain on the material than is the case with fixed plants. In many cases the calculations made on a purely static basis are not sufficient to take account of dynamic forces, and a great deal of experience — much of it unsatisfactory — will be necessary before it is possible to evolve new methods of calculation.

As regards strength conditions, there are plenty of goods grades of steel (from ordinary Structural Steel 37, Steel 48 and Steel "Si" to Steel 52) available for meeting the various requirements; but it is not always the steel of higher-strength which is the better for the particular conditions involved, when corrosion, tendency to vibration, machinability, etc. have be allowed for as well. "Steel" is an excellent material for the moving parts of hydraulic plants, but its high elasticity, and the deformation capacity which this involves, call for special consideration when the parts are being designed and machined. Riveted joints in structural steel parts were only regarded as a makeshift in movable hydraulic structures, until the engineer had learnt how to do things better. Welding is now being adopted, enabling the material to be placed just where it is required, besides preventing the weakening of cross-sections by rivet-holes and the accumulation of materials at inaccessible places. Welding also meets the requirements of impermeability much better. It is only to be hoped that rolling mill practice will soon follow the requirements of welding technology, so as to avoid having to weld with rolled sections which were really designed for riveting. Fortunately a start is being made in this direction.

Of the many enemies of steel besides rust, we may merely mention the balanids in marine areas. Rusting is combated by painting (see Agatz's paper), but sufficient experience has not been gained in this connection to enable definite and generally applicable specifications to be given for the under-water painting of structural steel parts. It is true that certain "general rules" are followed, but these leave considerable latitude for further research and experience. This particular subject has been dealt with in greatest detail by Mr. *Wedler*,[1] Government Adviser.

The balanids penetrate the film of paint and expose the steel underneath to the destructive effects of sea-water. Even poisonpaints have been unsuccessful in getting the upper hand of these creatures. An effective means appears to have been discovered recently for combating the balanids. This is a cement-milk type of paint which forms a hard, vitreous ground-film on the iron Dunker & Co., Hamburg). It has been used for the Holtenau sliding lockgates of the Kaiser-Wilhelm Canal. Opinions differ at present as to the value or otherwise of a red-lead ground coating for steel structures in water. The paints which seem to have proved most satisfactory for under-water steel structures are those with a bituminous base, applied hot in a fairly thick coating.

Nature of the novable Plants.

The big majority of the movable steel structures or structural parts are used for barring or giving access to the water in definite channels; they are "seals" or "valves" such as are used in a smaller but similar form in engineering.

Throttle valves are there for the purpose of throttling the flow of water, and are often used as emergency stops in pipe-lines. But they can only be used in their limiting positions of "open" and "closed", because in all intermediate positions they result in unfavourable conditions of flow in the pipe. Even when wide open they constrict the cross-section of flow, and are so much exposed to the flow that they usually require special protection.

Sluice Valves, resembling ordinary stop valves, are constructed and used up to considerable dimensions. When open, they leave the cross-section of the pipe quite free, but they are very difficult to move under high and full water-pressure. When partly open, the flow conditions at the edges of the sluice-valve are extremely unfavourable, and there is a risk of cavitation.

Cylindrical Type Valves are frequently used and preferred. There are two usual types: (1) the simple form, consisting of long cylinders open at the top, which are raised vertically and seal with their bottom edge (a type frequently adopted on locks of the water-storage basins); (2) the closed form, recently developed by the Krupp-Gruson-Werk, in which the cylindrical sliding portion is drawn into a bell-shaped cowl, closed at the top and suspended from a traverse in the valve shaft (Fig. 1). The closed design prevents air being drawn in as well and causing trouble farther down the line (sluice at Fürstenberg-on-the-Oder).

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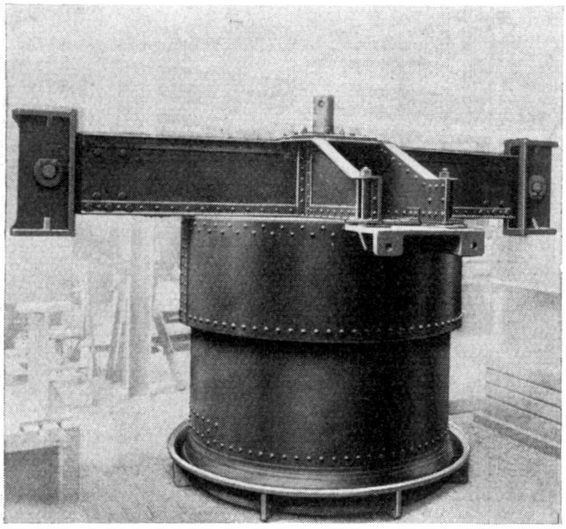
[1] Wedle: Unterwasseranstriche für Stahlbauteile im Wasserbau, besonders von Schleusen und Wehren. Bautechnik, No. 17 (1934), p. 232.

A typical cylindrical valve of unusual dimensions (made of cast steel) is that incorporated in the bottom outles of the Ottmachau dam (Fig. 2, taken from ZDVI., N° 31 (1935), p. 858), which was constructed by the Vereinigte Oberschlesische Hüttenwerke, Donnersmarckhütte, Hindenburg (Upper Silesia) to the designs of Government Adviser Mr. Chop of Breslau. These valves (six in all) have to carry off 500 m³ of water per sec, at a head of 12.5 m. Special deflection of the water is said to considerably destroy the energy of the flowing water in the valve without fear of cavitation. Exhaustive model tests preceded the manufacture of these valves and have since been confirmed by practical performance. These valves are also successfully used for fine regu-



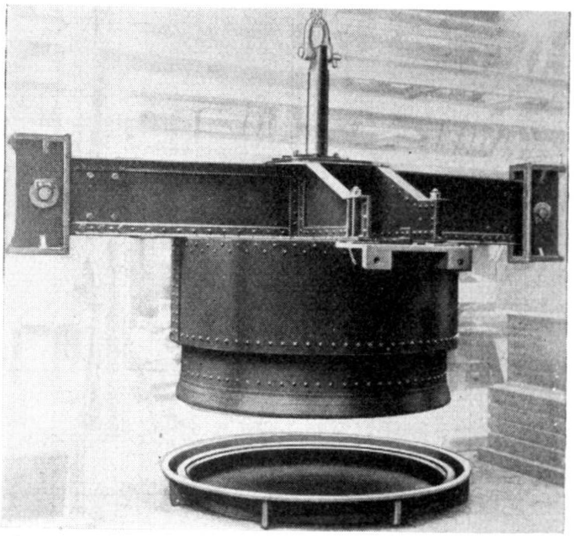
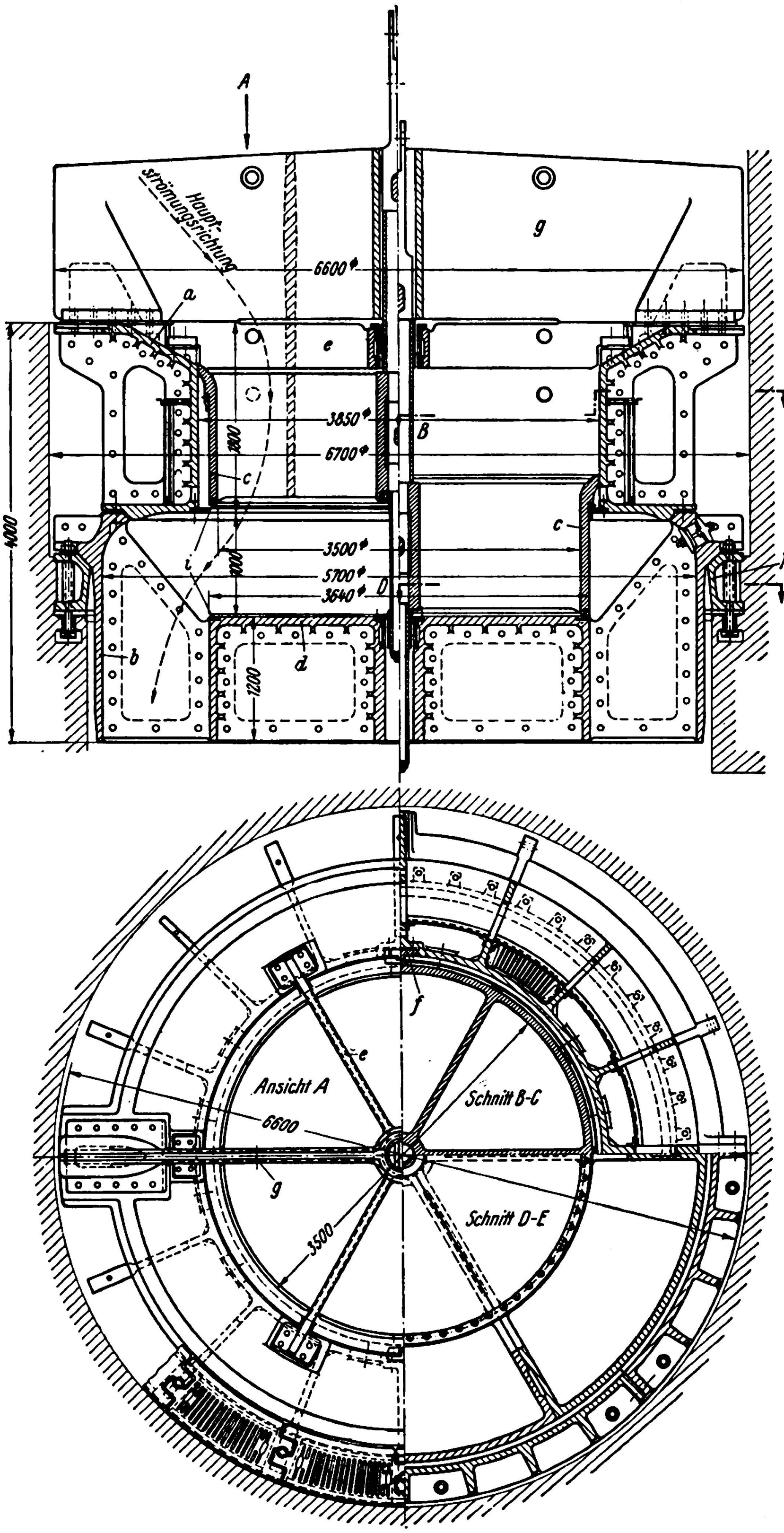


Fig. 1.
Closed type cylinder valve of Friedr. Krupp Grusonwerk A.-G., Magdeburg-Buckau, (patented).

lation in the intermediate positions between "open" and "shut". Fig. 3 gives en idea of the size of these valves, which the makers found it a very big problem to cast and machine.

Larner-Johnson Valves are called for where the valves have to be arranged horizontally instead of vertically. They are circular slide valves with a horizontal stem, and can be built to seal in both directions. At the same time, the pressure of water can be extensively utilised for releasing the moving portion of the valve, so that very little power is required for opening and closing the valve. The valve can even be made to close automatically against the pressure of the water. Two-way annular valves of the Maschinenfabrik Gebr. Ardelt (Eberswalde) have been fitted in the form of compensating valves beetween the two shaft locks of the twin-lock at Fürstenberg-on-the-Oder. The Krupp-Grusonwerke of Magdeburg manufacture bottom discharge valves of the Larner-Johnson type in which the head pressure itself is utilised for opening and closing the valve (Fig. 4, from ZDVI. N° 22 (1934)). Only the small needle valve *i*, requiring a sligth amount of power, need be operated to make the water-pressure in the chambers a, b, d available for opening or closing. Such valves have been fitted, inter alia, at the Sösetal Dam and at the Oder im the Harz, in the latter



a Inlet-funnel of the upper valve casing.
b Outlet in the lower valve casing.
c Valve cylinder.
d Valve floor (impact floor).
e Guides in star-arrangement.
f Guides for valve-body.
g Water divider.
h Seating ring for valve cylinder.
i Metal-packing with rubber gasket.

Fig. 2.

Bottom outlet-valve for the Storage Basin Ottmachau-Upper Silesia.
(Sectional elevation left half: Valve open, right half: Valve closed.)
Makers: Vereinigte Oberschlesische Hüttenwerke, Werk Donnersmarckhütte in Hindenburg/O.-S

case for a maximum volume of 30 m^3/sec. at a head of 55 m, with an inside diameter of 1.27 m.

Like the Larner-Johnson Valves, the Drum Sluice Valves can also be fitted into horizontal conduits. Fig. 5 shows their make-up and how they operate. When open, the slide valve completely frees the cross-section of the pipe, and adapts itself perfectly to the curvature of the pipe-wall. When closed, it forms a kind of mitre gate against the flow of the water, but can seal against both direction of flow. The end settings are satisfactory, but in the interme-

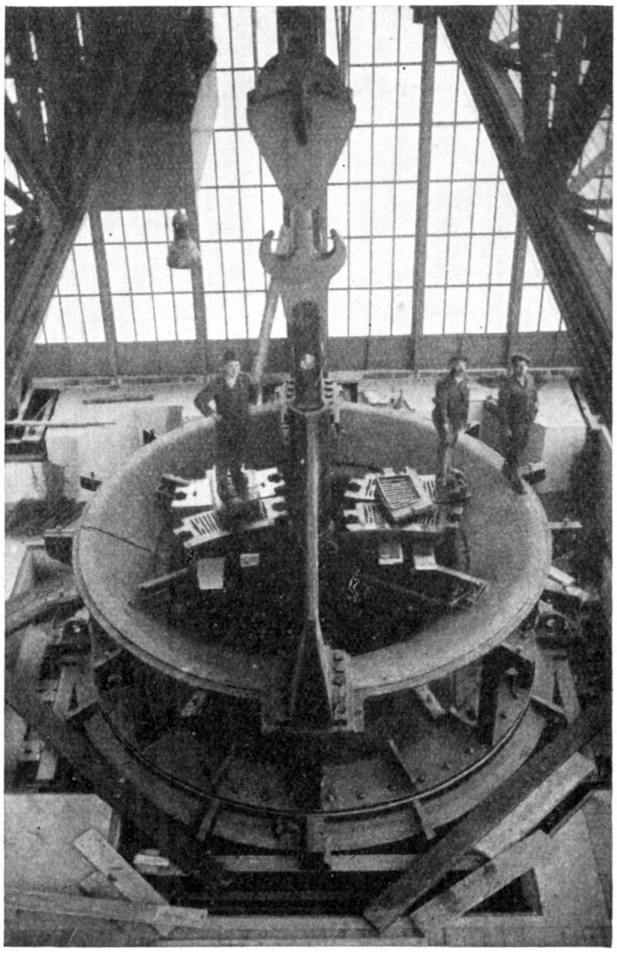


Fig. 3.
Cylinder valve as Fig. 2 during erection.

diate positions cavities form which may lead to heavy knocking effects by water (Fürstenberg/Oder Lock). It is therefore inadvisable to keep these valves in the intermediate positions for any length of time.

Sliding Sluice Valves are among the oldest of sealing devices on locks, dykes, weirs, etc. They are extremely simple and cheap, and also sufficiently tight, but they set up considerable resistance to motion when the dimensions are too large or the water-pressures too high. In the latter case, they are replaced by Roller Sluices. The arrangements for guiding and sealing must be separated, the former being taken care of by supporting wheels on tracks, and the latter by special arrangements. The sluice gate usually seals at the bottom and also at the top, when necessary — by abutment against a packing strip which may be of timber, or of machined steel or other metal. A slightly elastic packing (say, rubber, or springs) is generally used for the top, since hard surfaces in two planes cannot ensure a watertight joint in the long run. The

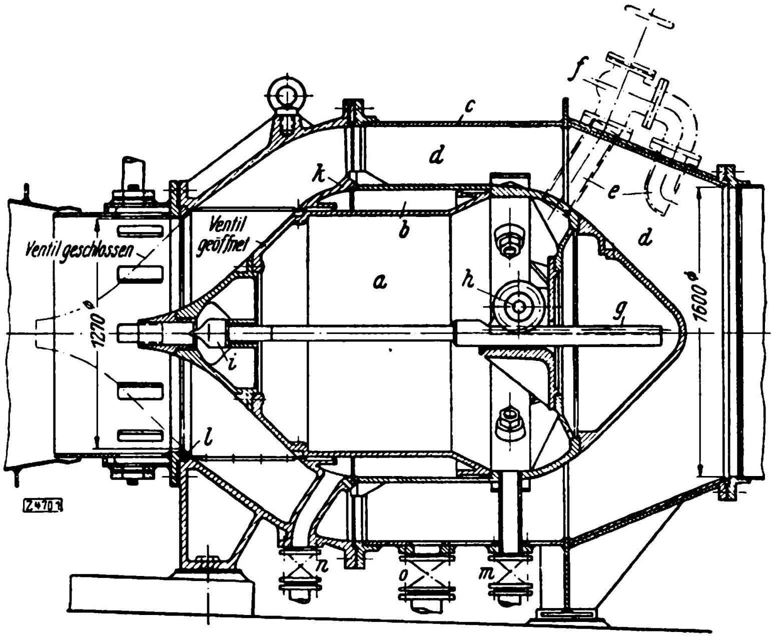
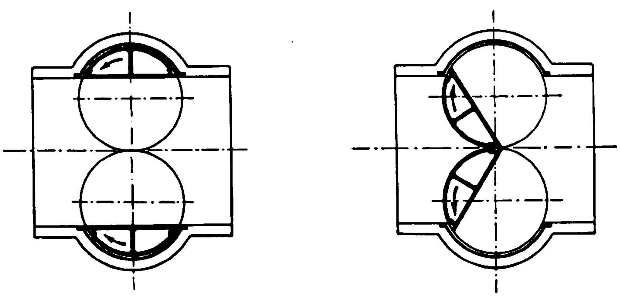


Fig. 4.
Bottom outlet valve Larner-Johnson system, type Krupp-Grusonwerk-Magdeburg.

side packing unsually consists of spring-strips, aided by the pressure of the water. It is best to make the side packings wedge-shaped in front, so that the packings .can travel easily into their final positions. Tapered packings must be prevented from jamening or seizing by providing for a certain amount of



Valve open Valve closed
Fig. 5.
Drum-valve, general arrangement.

"give" in the packing strips. Rolling Wedge Sluice Gates are now being manufactured up to considerable dimensions. They have the big advantage of being very accessible, and if necessary can be lifted out bodily. A large Wedge Sluice Gate of the Krupp-Grusonwerk Company 'is shown in Fig. 6 (Fürstenberg-on-Oder Lock; down stream end 7.2 m² discharge section, head 15.8 m).

Locks provide the most frequent and natural incentive for the use of shutters of the most varied types. Originally, mitre leaf gates were probably used almost exclusively for sealing off lock-chambers. They can scarcely be surpassed for simplicity and reliability, and for this reason are still used up to very large dimensions. They almost lead the field for inland waterway locks. But they are only safe when movements of the crowns are excluded, and they become inconvenient when the resisting pressure becomes too high, or the heigt/width ratio of the leaves of the gate is too unfavourable. Generally speaking, mitre leaf lock gates should never be used in regions subject to mine subsidences (see Lift Gates). One inconvenience of the mitre leaf lock gates is that a separate drive is required for each leaf, thus involving mechanical equipment on either side. Drop gates avoid this drawback, for they can be operated from one side provided they are rigid when rotated and do not have to be moved against water pressures that are too high. Their weight can be sufficiently compensated for by the buoyancy of the water. Drop gates may therefore be satisfactorily instal-

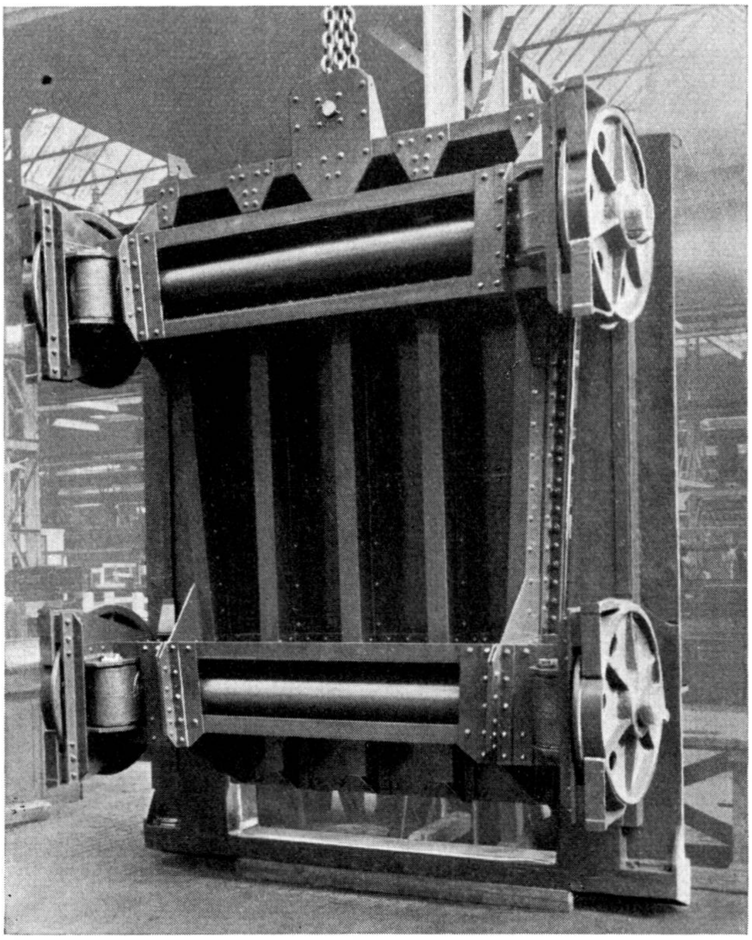


Fig. 6.

Twin sluice gate for Fürstenberg a. O. Down-stream head, tapered wedge sluice gate made by Krupp-Gruson Werk.

led in the upstream end of locks with elevated sills (see Fig. 7, Drop Gate of the Fürstenberg Lock, upstream end; in the background, a cylindrical valve). A remarkable feature of the illustration is that the lower bearings of the drop gate, which would otherwise be difficult of access, are supported against a spring thrust rod, so that the bearing can gape when foreign matter sticks in it. These bearings are also supported on vertical suspension ties, thus enabling the gate to be floated rigth up and the bearings inspected at the surface of the water.

In those regions of Germany subject to mine subsidences, Lift Gates are preferred to other types, as these are able to follow the pronounced displace-

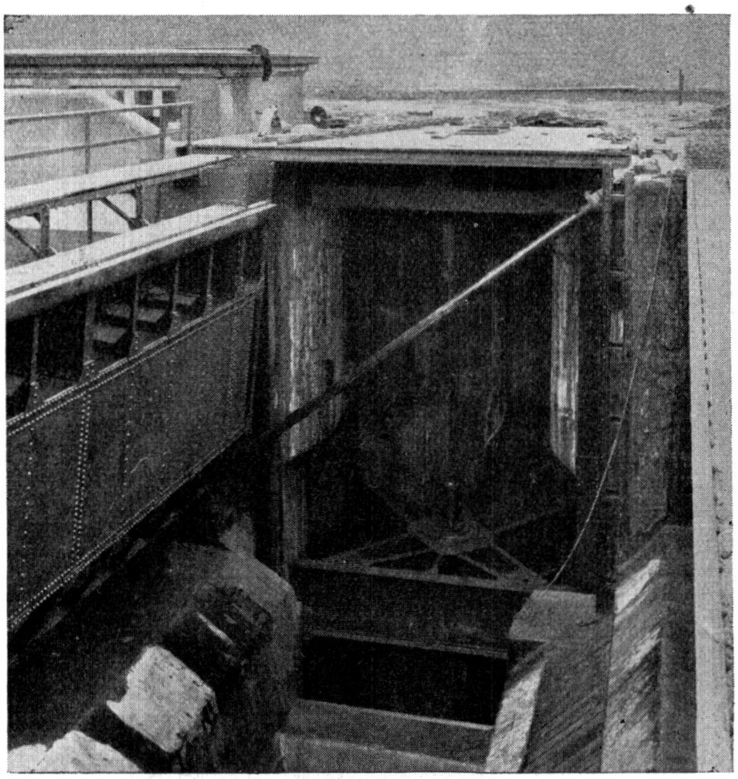


Fig. 7.
Twin shaft-sluice Fürstenberg a. O.
Up-stream head cylindrical bell-shaped valve made by Krupp-Grusonwerk.

ments of the abutments. The locks on the Weser-Datteln Canal are equipped in this way. Lift gates are also the type to use for the pumping and trough connections on ships' hoisting gear (Henrichenburg, Niederfinow), at the down stream end of shaft locks, and everywhere that sufficient height can be provided for raising the gates, and lifting stages do not cause inconvenience. The lift type of gate has the big advantage of accessibility, with the drawback of high cost; it is probably the most expensive of all lock gates. One of the newest lift gates for lock is shown in Fig. 8. The plant was constructed in 1934 for the Herbrum Lock of the Dortmund-Ems Canal.

In some respects, the lift gates are nothing more than a large roller sluice. The idea occured to engineers to utilise the gate itself as a sluice, at least from the moment when the gate has been partially released from the load of its headwater. The question then arose: "By-passes or not?"[2 3]

With the increase in the lengths of the chambers and the heads, engineers realised that no further progress could be achieved with the original arrangement of sluices in the lock gates, as the vessels in the lock would be too much disturbed when the volume of water necessary for locking them economically was thrown against them from the gate. By-passes were therefore devised, and were regarded as indispensible for long and deep locks, especially for towing locks, although the by-passes with their shutters and the piercing of the walls of the chambers were anything but simple and desirable. In regions subject to

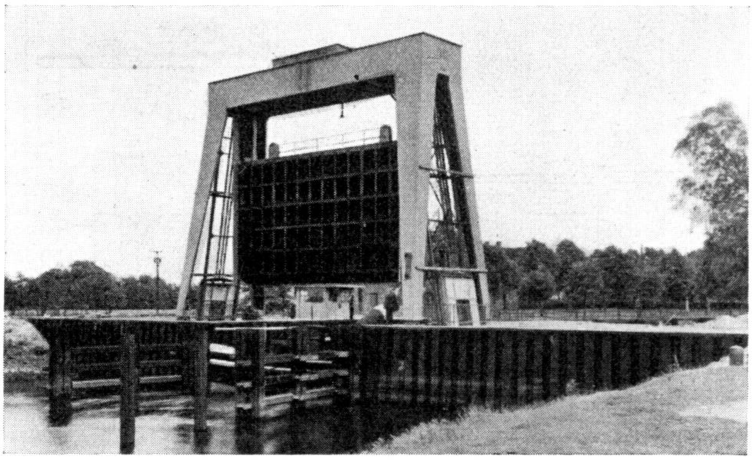


Fig. 8.
Sluice at Herbrum of the Dortmund-Ems-Canal,
Liftgate, with hinged sprocket-bar drive, built by M.A.N.

mine subsidences, in particular, the weakening of the head and chamber walls was very undesirable. Exhaustive model tests showed that by-passes can very well be dispensed with provided the water be guided and retarded properly.

It is true that the heavy lift gates cannot be raised against the weight of water, and the mitre-leaf gates less so. However, sluices are incorporated in the gates (reverting to a certain extent to the original design of an earlier age), and the water is filled or discharged through the gates. But care is taken to ensure that the rushing water is not directed against the ships, but is deflected several times so that it loses its force and cannot endanger the craft. The aim is to get, behind the gate, a drop in level whose highest point comes just behind the gate, so that ships lying in the lock-chamber only experience a moderate current always running in the same direction. Segment-shaped sluices are very suitable for fitting in lock-gates, as they are easy to move and provide a satisfactory outlet. Fig. 9 shows a nonbypassed shutter of this kind formed by a mitre-leaf gate with segmented sluice, and installed at the "Hirschhorn" Lock. The breakwater beam is heavily reinforced with iron, and illustrates one

[2] Regierungsbaurat Dr. Ing. *Burkhardt* about model tests with locks without circulation in "Die Bautechnik" (1927), No. 3.

[3] The same just there, No. 31, about observations and experiences at the Double lock (without circulation) Ladenburg of the Neckar Canal.

advantageous use of steel in hydraulic construction work. It seems as if the non-bypassed type of lock will be the rule in future, since previous operating results have proved satisfactory.

For sea locks of very large dimensions, especially those located in the tidal region, where the gate must be able to shut off the water in both directions, mitre-leaf gates are often unsuitable, being replaced by Sliding Gates, which are very little effected by waves. In large modern sliding gates, the front end runs on a bottom carriage, from which it may be lifted, and the rear end

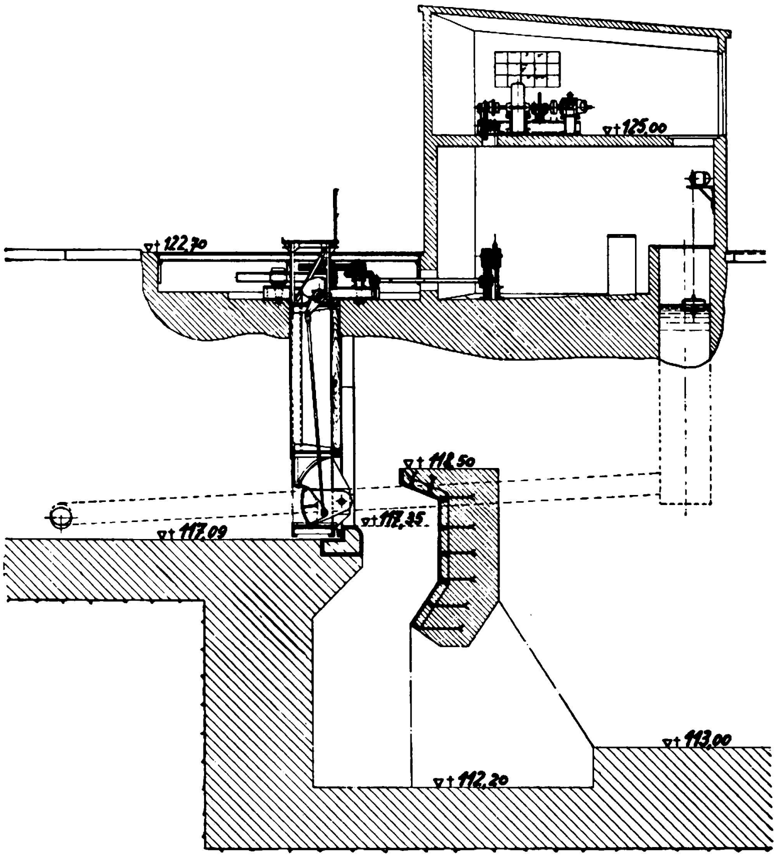


Fig. 9.

Sluice at Hirschhorn of the Neckar Canal, up-stream head, mitre-gate with segmental sluices. No by-passes. Energy of water destroyed by braking chamber.

on a top carriage (Fig. 10, showing the sliding gate at Bremerhaven; chamber 372 m long and 50 m wide). Among the sliding gates should be included the Floating Caisson Gates, which are used specially for closing off dry docks. They have to be balanced by floating caissons so that they can be floated into the gate recess and lowered in this on to the sill. They form the transition from hydraulic construction work to shipbuilding.

An attempt is being made to fill a lock-chamber without bypasses, probably by overflow over a gate which can be lowered [upper gate of the Sersno (Upper Silesia) lock], now under construction. For this purpose, the segment gate acting against the head water is the most suitable type, as it is raised from its sill by the pressure of the water. To obviate unnecessary slip under load, the raising and lowering movements will be separate from the pressure movement closing, and such closing pressure will only be applied when the gate is stationary. The overfall conditions have been satisfactorily settled by preliminary tests, but actual experience must show whether this type of gate is efficient in other respects.

What lock-gates are on a small scale, Safety Gates are on a big scale. Their function is to prevent higher reaches of the canal wasting water if a dam bursts, etc., and they also belong in the vicinity of the upper retaining shutters of ships' hoists. Very wide gates of the lift type are usually employed, as they must block or free the entire cross-sectional area of the canal. They must be capable of being lowered at any time without much delay, often by remote

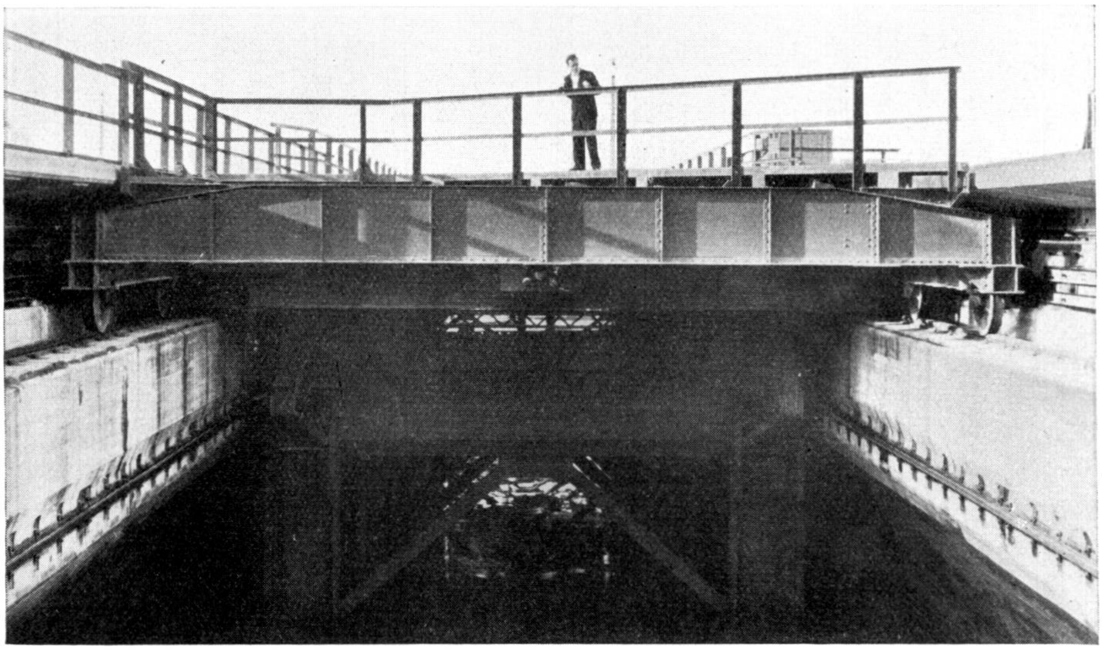


Fig. 10.
Sliding gate at Bremerhaven, upper carriage, built by M.A.N.

release operated from a point of observation, whereas more time can be allowed for raising them again. One of the newest safety lift gates is the one constructed for Duisburg-Meiderich for 11.5 m lift and a lifting force of 100 tons (Fig. 11).

Weirs, so far as they are movable, must not only keep up the water-level, but also control it as desired. The old-fashioned needle weir, which ist still used, only partially fulfils this requirement, and with a certain amount of risk for the attendant. The needle supports and weir pedestals are nevetheless remarkable as exemplifying the use of steel in hydraulic construction works of an earlier period. The more modern weirs are made almost throughout of steel, except for the masonry or concrete of the body.

Even Sluice Weirs, which initially were often made of timber on an iron framing, are tending more and more to become purely steel structures, with the result that the spans attainable have increased to 40 m, and the heights of damming to 12.5 m. For smaller weirs and low heigths of damming, it was formerly the practice to regulate the flow of water by raising the lower edge of the sluice-gate; but it was soon found preferable to operate the fine regulation by lowering the top edge, i. e., making the sluice in at least two parts, and dropping a lower top-portion down behind (in the direction of the water flow) a stiffened plate wall of the lower leaf. The M.A.N. Company have developed very suitable types with a common track for the lower and upper sluices. In cases of need, the upper sluice can be made so high that fairly large volumes of water can be carried off above the weir instead of below it. By placing on the lower sluice a folding flap in place of a top portion, the Drop Gates

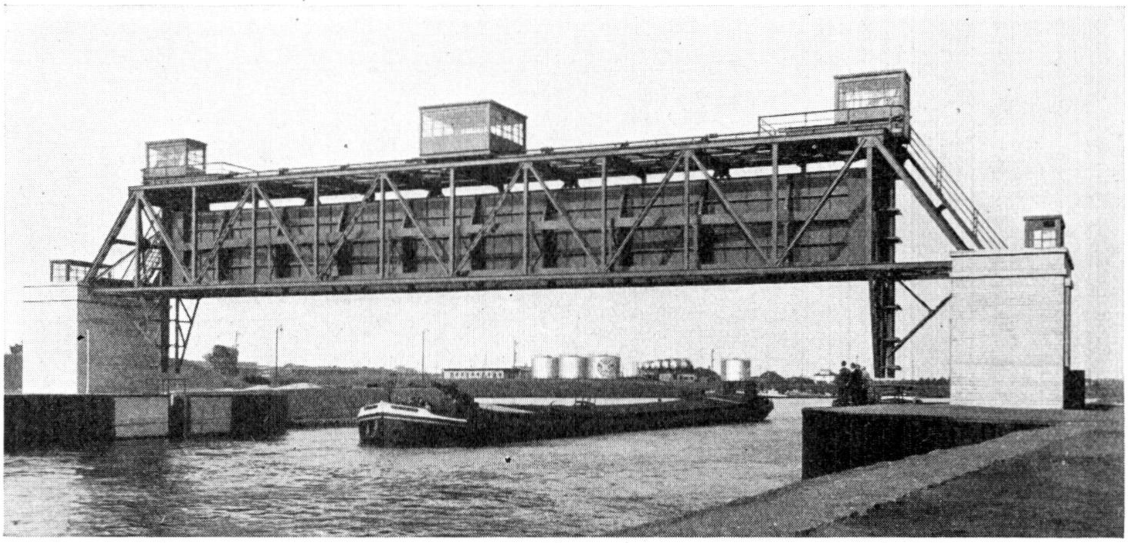


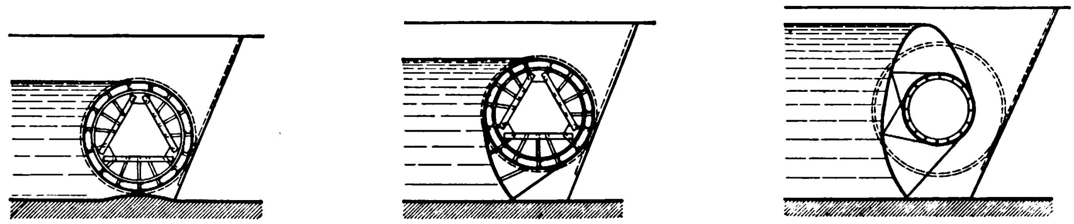
Fig. 11.
Safety lift-gate at Duisburg-Meiderich, built 1935 by M.A.N.

resulted, in which one operating gear usually folds back the flap first and then raises the entire sluice.

Drop Weirs have been built on similar lines to the drop gates for locks. They are lowered by the pressure of the water and erected against it. To enable these to be operated with power from one side only, the flap had to have a fixed (non-turning) axis and be made non-rotating itself. This led to the development of the fish-bellied type of flap, an example of which is found at the discharge weir on the equalising reservoir of the Bleiloch barrage.

A feature of the drop weir is that the flap must be rigidly pivoted to the sill of the weir, and can only operate by overflow. The flap has to be moved over completely when it is desired to release precipitates which have accumulated in front of it. For this reason, the drop weir pure and simple will only be suitable in specific cases.

The Roller Weir has a much wider scope and has been extensively adopted. It is very strong and insensitive, can bridge wide spans, and, being rigid, can be operated with a single drive, whereas sluice-gate weirs must always be operated from both sides. Ice and rubble do not affect it, and the overflow conditions are satisfactory without additional aid. The M.A.N. Co., in particular, have spent more than 30 years developing this type of weir. When conditions are suitable, the diameter of the roller is made the same as the height dammed, and the water allowed to flow over the roller. For small spans but greater heigths of damming, the diameter of the roller would be excessive. In this case a separate apron is itted in front of the suitably dimensioned roller, or the roller extended downwards by a kind of bill (Fig. 12, taken from an M.A.N. prospectus). The bill or apron comes up against the roller free from deposits of any kind when the roller rolls down. The rollers are raised on the track (having a marked upward incline) usually with Gall chains (sprockets), racks on the guide rails ensuring that the roller runs up evenly at both ends. It is a remarkable fact that, of all the types of weirs, the roller is probably least affected by ice. Hitherto, roller weirs in Germany have not had to be



Roller weir of cylindrical section for relatively small depth of water in comparison to the length of the roller.

Roller weir with beak-shaped attachment for a deeper depth of water in relation to the length of the roller.

Roller weir with articulated damming shield for deep water compared with the length of the roller.

Fig. 12.

Three basic forms of roller-weirs of the M.A.N.

heated to prevent their freezing up. This has only been necessary in northern countries.

Like sluice weirs, roller weirs can also be provided with flaps (fish bellied form) when special conditions, say, getting ice away over the roller, make it necessary. These flaps are usually operated at the same time as the roller by the winding gear for the latter. Larger weirs are now usually divided up so that one opening with a flap roller (or sometimes a drop roller) is placed between two openings with standard rollers.

Steel is also used in movable plants of hydraulic construction works such as dredgers, scouring apparatus, tugs, barges, etc., but it is beyond the scope of this paper to discuss these further. Nor will further mention be made of pipelines for scouring plants, siphon discharge plants, or hydro-electric plants, as these are no longer counted as movable hydraulic construction works.

On the other hand, it might be well to deal briefly with operating gear for hydraulic construction works. Every mobile engineering job requires gear to operate it — to make it move in the direction desired and overcome all obstacles. Man power is usually insufficient to perform the lifting or shifting operations required. In the big majority of cases electricity can and must be employed, as it is now procurable nearly everywhere. As to whether it is direct or alternating current does not matter much for the present purpose. Only where large and heavy masses have to be reliably controlled will direct current

Ward-Leonard controlled circuits be preferable. Our large electrical firms have also developed satisfactory methods for enabling drives to be operated electrically in the same direction from different points, where there are difficulties in carrying a mechanical shaft through for the same purpose.

The operating gears consist practically throughout of "steel", including, of course, cast steel and, cast iron. All the main supporting parts like roller steel. ropes, chains, etc. are "steel", and therefore come into the category of steel parts for hydraulic construction works. The "link racks" developed by the M.A.N. Co. constitute a remarkable component of modern times, since they combine the advantages of Gall chains with those of rigid racks. They stand up equally well to tensile and compressive stresses, and have the added advantage of ensuring frictionless guiding at the driving pinion. They can also be successfully used for the largest, heavy-duty operating gears, such as those on the sliding gates at Bremerhaven.

Special Phenomena.

The high strength of steel combined with its high elasticity makes steel structures into units capable of vibrating, and each units has its own individual frequency. If regularly occurring impulses initiative vibrations in such structures, they will vibrate, and these vibrations may be dangerous when the inciting impulses keep step with the individual frequency of the structure (resonance). These vibrations may attain such dimensions that they eventually lead to fatigue fractures. If the vibrating units are made up of parts of different types, each capable of vibrating at different frequencies, stresses may occur at the connecting members (bolts, rivets, straps, etc.) which many times exceed the figures found by static tests. The parts particularly endangered are connections which have to be capable of standing up to considerable work of extension under vibrational deformation, but are unable to do so. Cases have been known where long and thin bolts have held up, whereas short and thick ones have broken in a short time.

Large-span sluices on weirs for instance, are prone to such vibrations. They are put under tension by the pressure of the water and set in vibration by its flow, pretty much the same as a violin string is vibrated when the bow is drawn across it. Dangerous vibrations of this kind have been observed at the weirs at Oldau and Marklendorf. In the overflow weir, the flaps, and in the underflow weir the entire body of the weir started to vibrate seriously, and the vibrations were always most pronounced for definite heigths of overflow and gap-openings. The weirs mentioned, with sluices of 15 m wide and 3.70 m high, vibrated at between 10 and 25 cm opening of gap, but most pronouncedly at 15 cm (above 25 cm, everything was quiet again!). Rivets were sheared, and cracks occurred in the main girders. Fine measurements showed that the damming wall vibrated at a different rate to the lattice supporting structure on the downstream side, and this must have set up considerable shearing stresses. The vibrations and their attendant dangers were gradually eliminated by making the lower dam beam of special shape (varying the section every metre). The effect of this was to disarrange the jets — previously uniform and closed — of the water rushing through the gap, and to deprive them of the possibility of initiating a particular type of vibration in the body of the weir. Model tests for further elucidating the problem of vibrations at weirs are in hand. This should open up a new and promising field of research.

Vibrations may, under certain circumstances, also be set up in valves, pipelines and the like and endanger the material. It would be a good thing if experience in this connection could be interchanged between the various contries.

Many parts of movable structures are sensitive to frost in a high degree. Sluice weirs may freeze up, lift gates become immovable before shipping has to lie up for the winter. One effectual but expensive way of combating this trouble would be to heat the sensitive parts by electricity (theoretically, 1 kilowatt-hour of energy yields only 860 calories!), following the practice adopted at the Weser Weir at Döverden or the ship's elevator at Niederfinow.

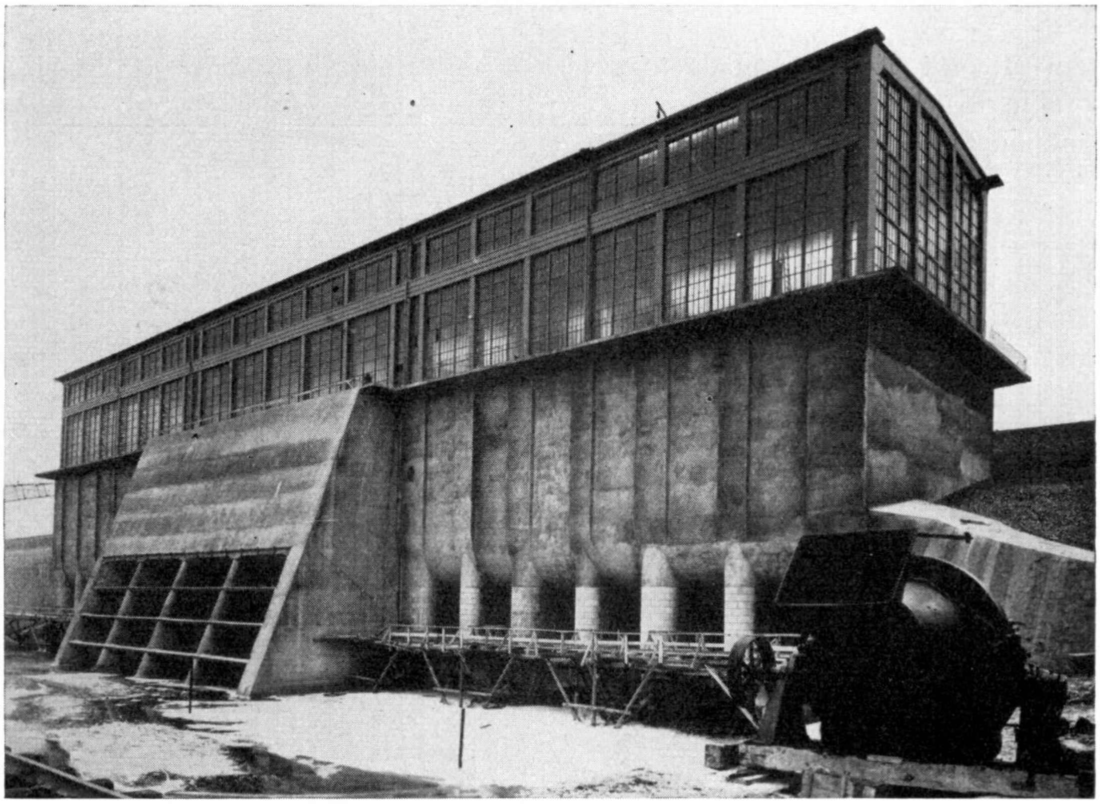


Fig. 13.
The steel-framed hall over the bottom outlet of the hydro-electric plant in the barrage of Ottmachau.

Other brief References.

Among the "movable" structures in water construction works should be included the ships elevators, of which the Henrichenburg and the Niederfinow plants are already working in Germany; they are probably the largest steel constructional works in hydraulic engineering in Germany, but they will not be dealt with here, as the ships' lifting plant at Niederfinow is discussed in a separate paper.

The Operating station above the bottom outlet and power plant at the Ottmachau reservoir dam is worthy of mention. Here a steel frame building has been erected and glazed over a large area (Fig. 13).

Brief mention only need be made of the fact that Pumping Stations and Power Plants use steel extensively in their construction, but they cannot be included among the "movable" structures of hydraulic engineering.

Summary.

An attempt has been made to give a brief review of the use of "steel" in movable hydraulic construction works, illustrated by examples of more recent plants in Germany.

VIIb4

Welded Weirs and Sluice Gates in Belgium.

Geschweißte Wehre und Schleusentore in Belgien.

Barrages et portes d'écluses soudés en Belgique.

A. Spoliansky,
Ingénieur des Constructions Civiles et Electricien A. I. Lg.

Foreword.

The construction of the Albert Canal, and the great operations for improving Belgian waterways, have necessitated the building of numerous weirs and locks during the last five years.

Nearly all of the steel structures of these plants have been welded.

In a series of articles, some of which have been published in this country, we have shown the remarkable progress that has been made since the erection of the first welded bridge in Belgium — the "Pont de Lanaye" on the Albert Canal, in 1931. Welding had not only become firmly established, but had also become the object of general attention.

In the construction of fixed framework, welding has proved itself a particularly economical and convenient method; it should be so with greater reason in all cases of movable structures, such as swing-bridges, movable bridges, railway wagons, etc., on account of the resulting decrease in weight and, in the case of hydraulic structures, by reason of the water-tight joints assured by its use.

If, even in the present state of the art of welding, and from the standpoint of the sole criterion, the total cost price, competition can still be regarded as possible between riveting and welding in certain operations — it is beyond doubt, that for many structures, and apart from the question of cost, welding can come to the fore by reason of its intrinsic properties.

Thus, in connection with weirs and lock gates:

1) The lightness of the metallic framework will admit of an appreciable reduction in the mechanical appliances and of the working expenses, whilst fully assuring great rigidity of the structure.

2) Perfect water-tightness can be easily and economically obtained.

3) The facility of maintenance, characteristic of all welded structures, will ensure greater durability.

Now, rigidity, lightness, water-tightness, ease of maintenance, constitute the principal qualities which well-designed lock-gates should possess.

It is, therefore, not surprising that after the first welded lock-gates or weirs had been constructed — on the initiative of a single constructor — the Belgian Bridges and Roads Department began to specify welding.

Structural Designs.

A lock-gate is only another type of bridge decking with a plated surface carried on girder work.

The thickness of the plates is generally the minimum compatible with the task to be performed. Nevertheless, it is possible to keep down to these minima by the ease with which stiffeners can be arranged, and there is nothing to prevent building the gates with plates of one thickness throughout.

Moreover, the monolithic character of the structure allows the skin plating to be employed to a certain extent as an integral part of the intermediate ribs and for reducing their weight, just as in the case of ribbed slabs in reinforced concrete.

The intermediate beams, the main cross girders and the verticals, can be built up of sections or of welded girders. Both methods have been adopted, according to the circumstances, in the construction of Belgian lock-gates.

The monolithic character of the welded structure is in itself sufficient to ensure inflexibility; nevertheless, cross bracings of the St. Andrew's cross type have generally been provided.

Apart from these few special features, the essential attributes of a good design remain the same both for riveted and welded work.

The main, if not the only, difficulty attaching to the construction of a welded lock-gate is the temperature deformation set up by the asymmetry of the constituent parts (a single skin plate, for example).

The method of performing the welding should be specially studied and every means adopted to avoid deformations, with a view to preventing serious miscalculations.

A brief description will now be given of some of the works recently carried out in Belgium.

The Marcinelle Lock.

This great undertaking of correcting the Sambre at Charleroi, so as to prevent calamitous floods and at the same time to facilitate navigation, was put up for tender in June, 1931, on the basis of the general scheme of M. Caulier, chief engineer of the Bridges and Roads Department. The metal portion formed the subject of a competition and the welding scheme adopted was that of the Societé Métallurgique d'Enghien, St. Eloi.

A complete review of the work is outside the scope of this paper, which will be confined to a description of the essentially metallic portions.

Lock-Gates: — The gates of the lock, whose chamber is about 130 m long, are of the single leaf type, moving in a plane at right angles to the axis of the lock chamber. Each gate (Fig. 1) is suspended by steel cables from a carriage which runs on a foot-bridge supported by concrete columns. The suspension of the gates from the foot-bridges is designed in such a way that the leaves can be easily lifted clear of the water by means of tackles.

The opening of each leaf is effected by the pull exerted on the gate by the metal cable connected with a winch with straight gear-teeth and mounted directly on the lock wall for the up-stream gate, and on the shore wall for the down-stream gate.

The closing of the gate is similarly effected by the pull exerted on the gate by the metal cable connected to the same winch. The hinged sluices are operated by winches at the gangway level of the gates.

Down-stream gate: — Width of gate 12 m 90,
Height of gate 7 m 10,
Thickness of gate 0 m 70.

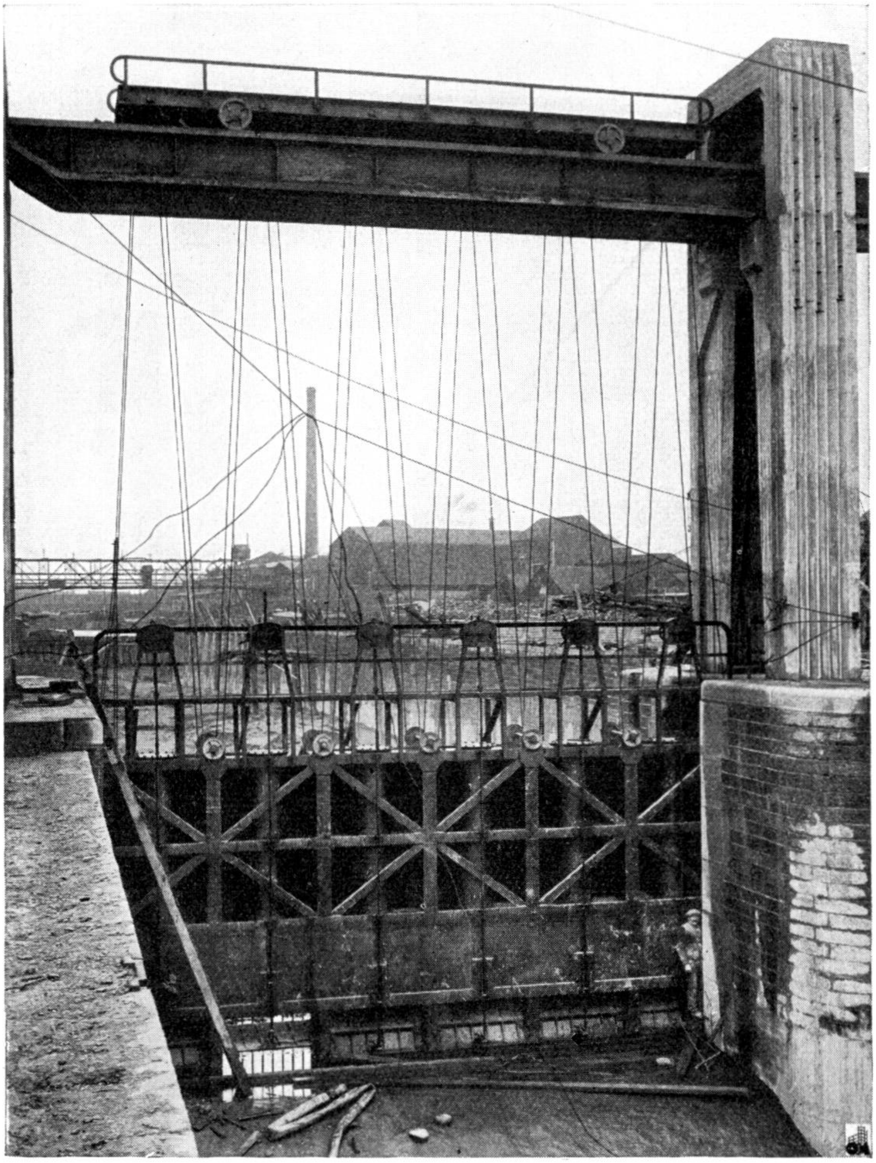


Fig. 1.
One of the lock gates at Marcinelle Lock.

The gate is calculated to meet the two following conditions: —

1) Full pressure on the up stream side with no water on the down stream side: stress 12 kg/mm^2.
2) Under normal conditions, with a difference of level of 2 m 35 and a stress of 10 kg/mm^2. The weight of a riveted leaf was 25,384 kg, The weight of a welded leaf was 20,234 kg, i. e. a saving of about 20 % in weight.

The gates consist of a double skin of 10 mm plate, thus providing buoyancy for balancing the gate. Water-tight inspection ducts are provided to facilitate maintenance of the interior.

The gate is built up of six cross girders having a web 750 mm by 10 mm with welded flange plates of varying width and thickness. The two vertical end frames have exactly the same scantling. The intermediate verticals have a web 750 mm by 10 mm, the flanges being formed by the skin plating.

A certain number of stiffeners have also been provided in the form ol light standard sections.

On account of the double skin, the plating extends only as far as the cross girders and is welded to the latter.

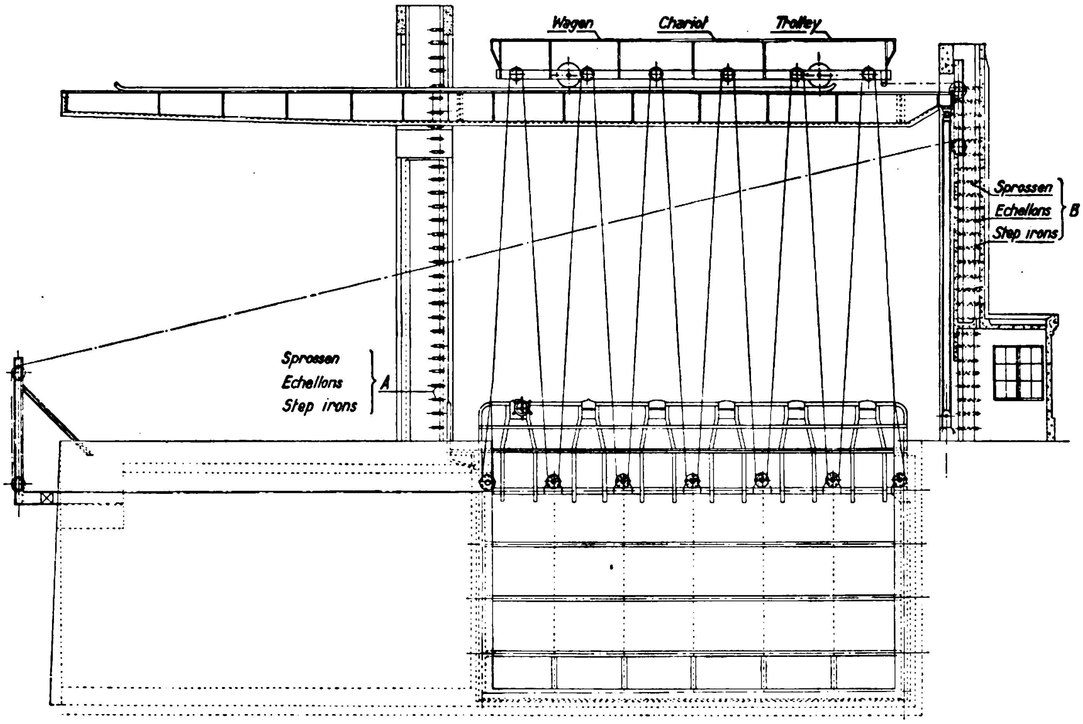


Fig. 2.
Marcinelle Lock, Upstream gate, general arrangement.

A monolithic system is obtained (Fig. 2) which is perfectly rigid; cross bracings having, however, been nevertheless provided.

The gate was delivered (the gangway and the two end verticals being detached) in two sections, each comprising three cross girders. These two pieces were joined up at site by welding on the central skin plating, the end verticals and gangway being then welded.

Up-stream gate:— Width 9 m 40,
Height 4 m 55.

As in the down-stream gate, and for the same reason, a double skin is provided, the construction being exactly the same, but comprising five cross girders. The up-stream gate weighs 15780 kg welded, as compared with 20200 kg riveted, thus giving a similar saving of about 22%.

L o c k. The actual lock comprises a sluice of sufficient dimensions to justify the use of the Stoney type.

The sluice (Fig. 3) is composed of a vertical steel wall supported on two main horizontal girders of the Vierendeel type. This type was chosen, not with a view to obtaining an illusory saving in weight as compared with a framed girder with parallel chords, but in order to have the heavy ironwork, rather than small parts, in the water. At the up-stream side, the main girders serve to carry the points supporting the skin plating (Fig. 4), and are cross-braced on the up-stream face. These two principal girders are joined to the vertical end girders supporting the trains of rollers.

Each sluice presents a recess 10 m 39 wide forming an overflow, which can be controlled by means of a baffle plate movable about a horizontal axis.

The "Stoney" rollers are carried on roller tracks of steel secured in guide grooves provided in the masonry.

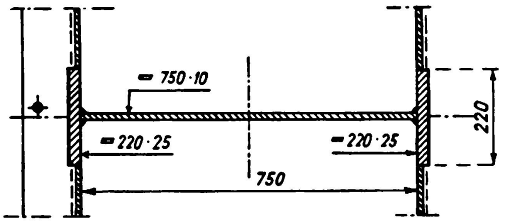


Fig. 3.
Marcinelle Lock
Detail of cross girder and connection of plates to flange.

The lateral water-tightness is obtained by upright steel bars, covered with rubber. These bars bear against the skin plating of the sluice and on castings fixed in the grooves. The water-tightness between the sluice and the sluice valve is obtained by means of chrome-leather joints.

The sluice wall is suspended by cables to the operating winches located on a concrete service foot-bridge and balanced by cast-iron counterweights housed in the columns.

The dimensions of the sluice prevented delivery in a fully assembled condition. The two end verticals forming the framework of the sluice valve had to be sent to the site dismantled and to be welded during erection. The great difficulty in erection was that the play between members had to be reduced to the minimum compatible with good water-tightness and the working of the sluice valve.

The width of the sluice is 13 m.

The height of the sluice, less the adjustable fittings, is 2 m 68.

The maximum height with fittings in position, is 4 m 60.

The total weight in riveted structural work would have been 43000 kg. The weight of the welded work is 35864 kg, i. e. a saving of 16.8%.

S l u i c e V a l v e: — The sluice valve also comprises two main horizontal girders to which is welded the 10 mm plating with stiffeners of plates and standard light sections. These girders are supported by two end verticals, by means of which the structure can be lifted.

On account of its special design (Fig. 5) and the arrangement of parts which are entirely asymmetrical, very careful precautions were needed to avoid any kind of warping and thus to maintain the axes of rotation in a rectilinear plane and obtain perfect water-tightness combined with efficient performance.

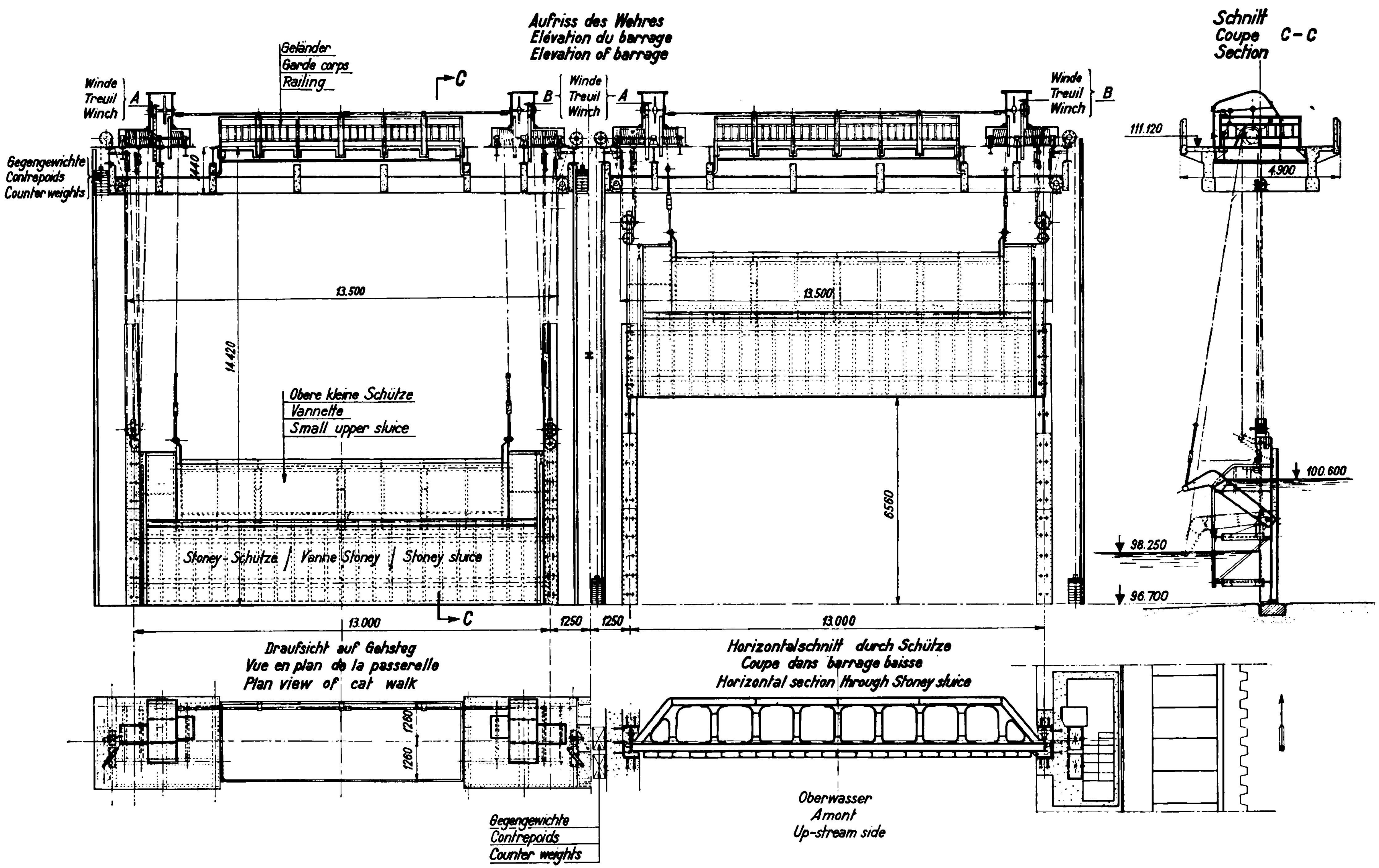


Fig. 4.

Marcinelle, General view of Lock

The welded sluice valve weighs 10 tons, as compared with 11 tons for riveted work.

Sundry Details: — Many interesting examples could be cited in which welding has resulted in great simplification in carrying out the work, or economy of material.

It was quite by chance that the first welding executed in lock work should be the Marcinelle weir, in which the variety of shapes and of members was very great. Being afraid of unduly prolonging the present paper, we will merely cite

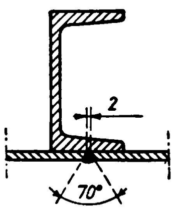


Fig. 5.
Stoney Sluice, connection between plating and channel sections.

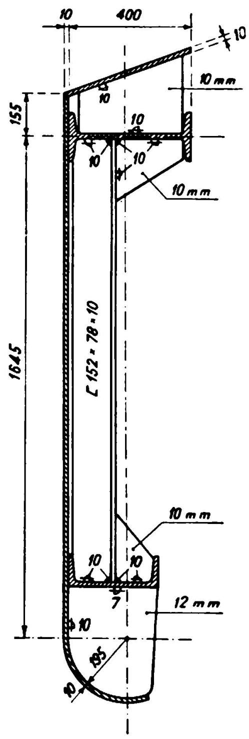


Fig. 6.
Marcinelle Lock
Small uper sluice

an example of replacing a piece of cast-steel — a suspension device (Fig. 6) — by welding.

The Wyneghem Lock (1933—1934).

The Wyneghem Lock is one of the six locks in the rise between Antwerp and Liége, and is in duplicate, so as to be independent of any damage and in respect of repairs. Each lock chamber is 136 m long with a breadth of 16 m, and represents a difference of level of 5 m 70.

The lock is constructed on new principles with regard to the filling and emptying of the lock chamber: discharge channels in the lock wall have been dispensed with and the water for filling and emptying flows through sluices constructed of cast steel segments provided in the gates themselves, two being contained in each leaf, and each presenting an opening 2 m 20 by 800 mm.

The lateral water-tightness of these sluices is obtained by means of flexible, rustless steel plates, covered with bronze sectors and moving over cast-steel members fixed on the walls of the gate openings. At the bottom, these sluices rest on plates (also of bronze) and the water-tightness of the upper part is attained by means of rubber joints pressed against a cast-steel guide by means of flexible, rustless steel plates.

The kinetic energy of the water is absorbed in the vortex chamber (faced with cast steel) built in the up-stream head, in such a manner that refilling is effected without affecting the steadiness of the boat in the lock. Similarly, chambers for absorbing energy are provided at the down-stream head to check the flow of the outlet water.

The lock, designed by Mr. A. Braeckman, Chief Engineer of the Bridges and Roads Department, is the result of careful tests on models which enabled the

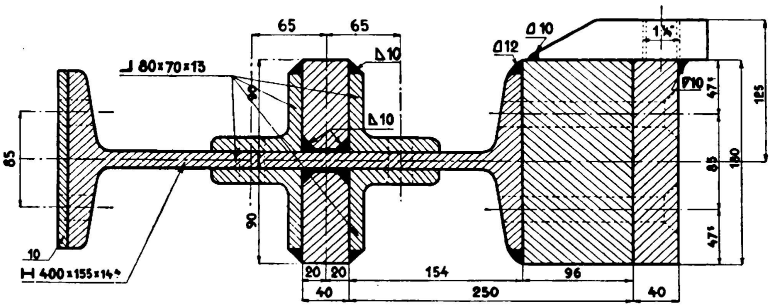


Fig. 7.

Marcinelle Lock Details of suspension for Stoney sluice

shape of the chambers, the width of the sluices, and the method of working to be decided upon.

Intercommunication between the two lock chambers is provided by means of segmental sluices similar to those of the gates. They are formed in the central wall and coupled in pairs, each pair serving to empty one of the lock chambers into the other. In this manner either of the lock chambers serves as a reserve basin for the other in case of shortage in the water supply.

The only difference between the intercommunicating sluices and the gate sluices is in the design of the lateral water-tight joints, which had to be so arranged as to obtain water-tightness in both directions. Actually, in the case of the gates, the pressure on the sluices is always exerted from the up-stream towards the down-stream direction, whilst in the case of the central wall, it is exerted on the one side or the other, according to whether the water in the right or left hand lock chamber is at the level of the down-stream, or at that of the up-stream.

The whole of the operating machinery is placed below the platform level, for which reason the general appearance of this very important work has a character of its own.

The gates are of the arched type.

Down-stream Gate:— Width of one leaf 8 m 839,
Rise of arch 3 m 00,
Total height 9 m 95.

The gate comprises seven equidistant transverse beams resting on the two end verticals. To avoid deflection of the vertical members, the transverse beams have separate cast-steel supports against the jamb. These transversals are composed of a web 890 mm by 10 mm and welded flanges of variable width and thicknesses. The single 10 mm and 11 mm plating is fixed to the flanges of the cross girders by two fillet welds, thus assuring good water-tightness (Fig. 7). Verticals are provided, composed of a 940 mm by 8 mm web and two flanges, one of which is a flat 120 mm by 8 mm member, and the other is formed by the skin plating. The end verticals have a web 568 mm by 10 mm and flanges 280 mm by 10 mm.

The plating is also stiffened, horizontally, by light sections.

The weight of a welded leaf is 23.5 tons; in riveted work it would have been 27 tons. Thus a saving of about 13% has been obtained.

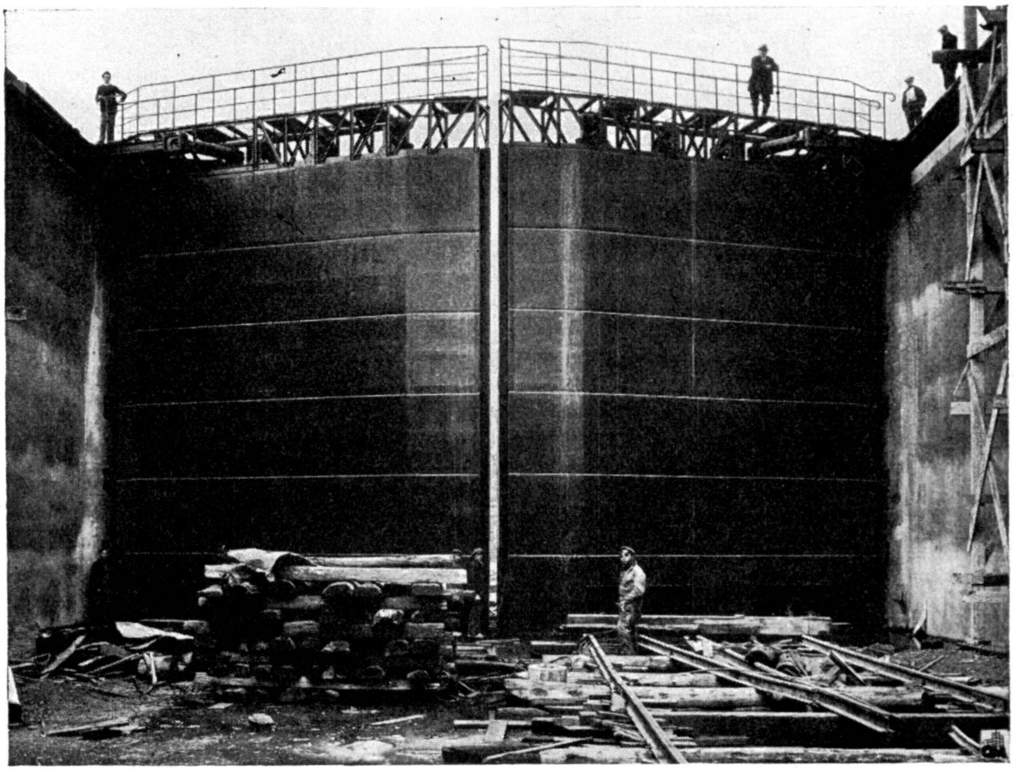


Fig. 8.
Downstream gate of Wyneghem Lock, seen from upstream.

Notwithstandig the high rigidity of the whole, a cross bracing, in the form of a St. Andrew's cross, has been provided.

The Up-stream Gate (Fig. 8), having a theoretical height of 5 m 75, is of similar construction to that of the down-stream gate. There are four cross girders having a web 868 mm by 10 mm; the end verticals have a web 568 mm by 10 mm with flanges of varying sizes; the intermediate verticals have a web 840 mm by 8 mm, and the skin is a single 10 mm plate, the stiffeners being flats.

The Hérenthals Lock.

The Hérenthals Lock is situated at the junction of the Hérenthals wharf basin with the Albert Canal. This junction is designed for boats of 600 tons.

The lock, which has a fall of 7 m 30, has a lock chamber 55 m long and 7 m 50 wide. In the lower part of the wall is a wide longitudinal channel with three filling openings in the chamber. In its upper part, the wall contains a small independent channel serving to feed the down-stream reach. The filling is effected by cylindrical sluices, all on the up-stream side, and by the rolling sluices on the down-stream side. The cylindrical sluices are of great utility, but their dimensions become excessive in the case of down-stream sluices of a lock with a high fall.

The lock is the work of Mr. M. A. Bijls, Chief Engineer and Director of the Bridges and Roads Department.

The arched gates are of the pointed-arch type.

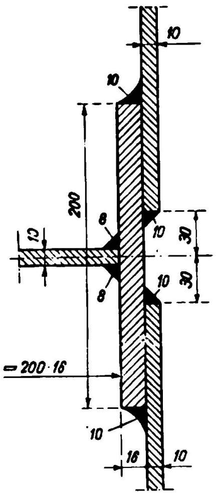


Fig. 9.
Wyneghem Lock
Details of welding for plate connections.

Down-stream Gate:— Width of a leaf 4 m 12,
Rise of arch 1 m 65,
Total height 10 m 975,
Single skin plating of 10 mm plate.

The gate consists of three verticals, on which rest the four rows of the transverse beams, and the verticals are of DIN 65 sections. The stiffeners are of standard light sections and the diagonal bracings in the form of St. Andrew's cross, are of channel section 200 PN.

The gate, of particularly strong and economical construction, weighs 18412 kg.

The Up-stream Gate has a height of 3 m 525 and a single skin plating 10 mm thick. The verticals and transverse beams are of section DIE 45. The construction is identical with that of the down-stream gate. Its weight is 3,980 kg.

The cylindrical sluices and the roller sluices are also of welded work throughout. The cylindrical sluices are composed of a cylinder of 10 mm plate, to the lower part of which is welded the forged steel valve, which rests on a cast-steel seating embedded in the concrete. The cylinder, strongly stiffened inside by diagonals and cross members of rolled sections, is guided in the pit by means of rollers.

The roller sluices, of trapezoidal form, are built up of "Grey" sections with single skin plating. They run on vertical roller tracks fixed in the concrete, and provide water-tight jointing by bearing against fitting strips in cast-steel frames fixed in the masonry.

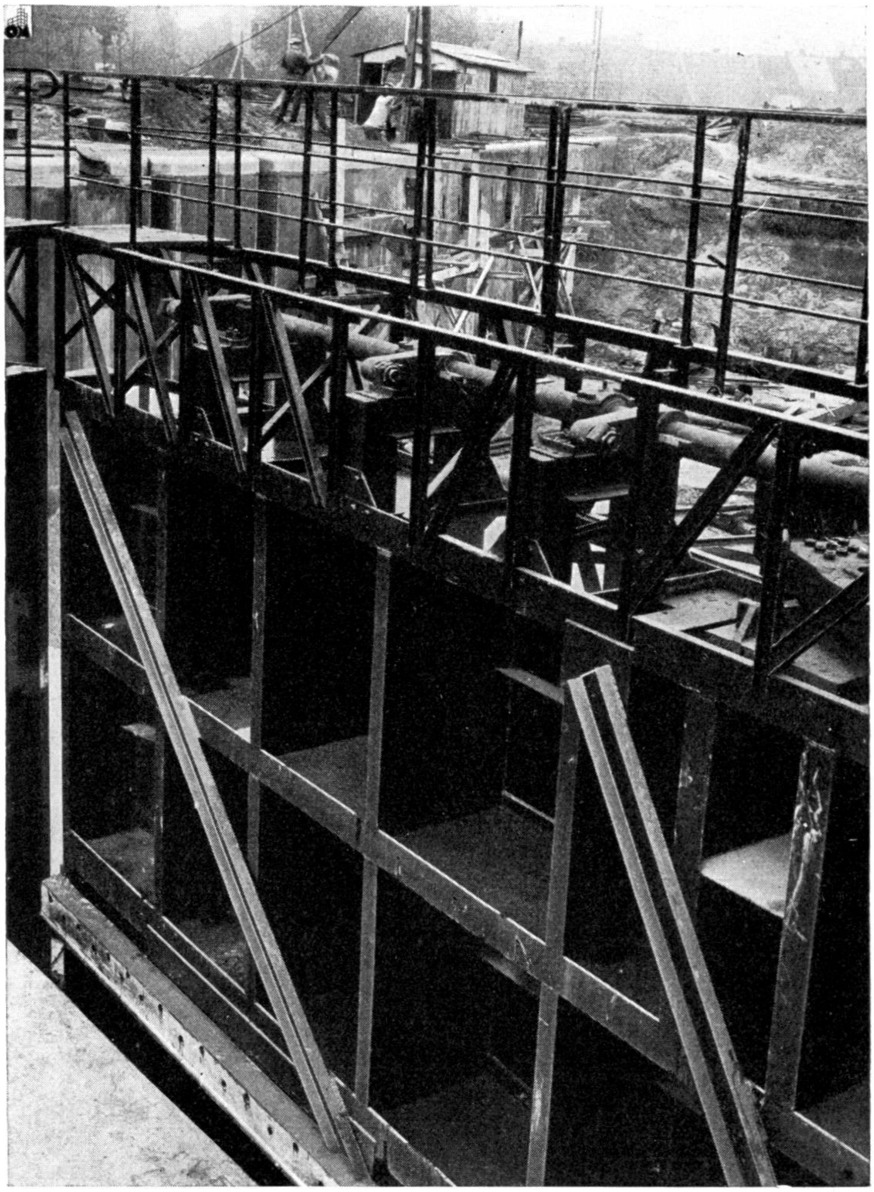


Fig. 10.
Upstream leaf of Wyneghem Lock.

The operating gears of the gates and sluices are electro-mechanical, of the rack and pinion type.

The Nèthe Lock.

The Nèthe Lock is the work of M. Claudot, Engineer-in-chief. This work, situated on the River Nèthe, provides a connection with the Albert Canal and is normally designed for boats of 600 tons. It will, nevertheless, allow the passage of boats of 1350 tons.

The lock comprises a lock chamber about 82 m long and 12 m 50 wide, for a fall of 5 m.

In general arrangement, this lock is of the Wyneghem type, that is, with cast-steel sluice segments in the gates. The gates are of the pointed-arch type, like those at Hérenthals. However, their design is not so simple, the space taken up by the sluices having necessitated increased thickness in the lower portion of the gates (Fig. 9).

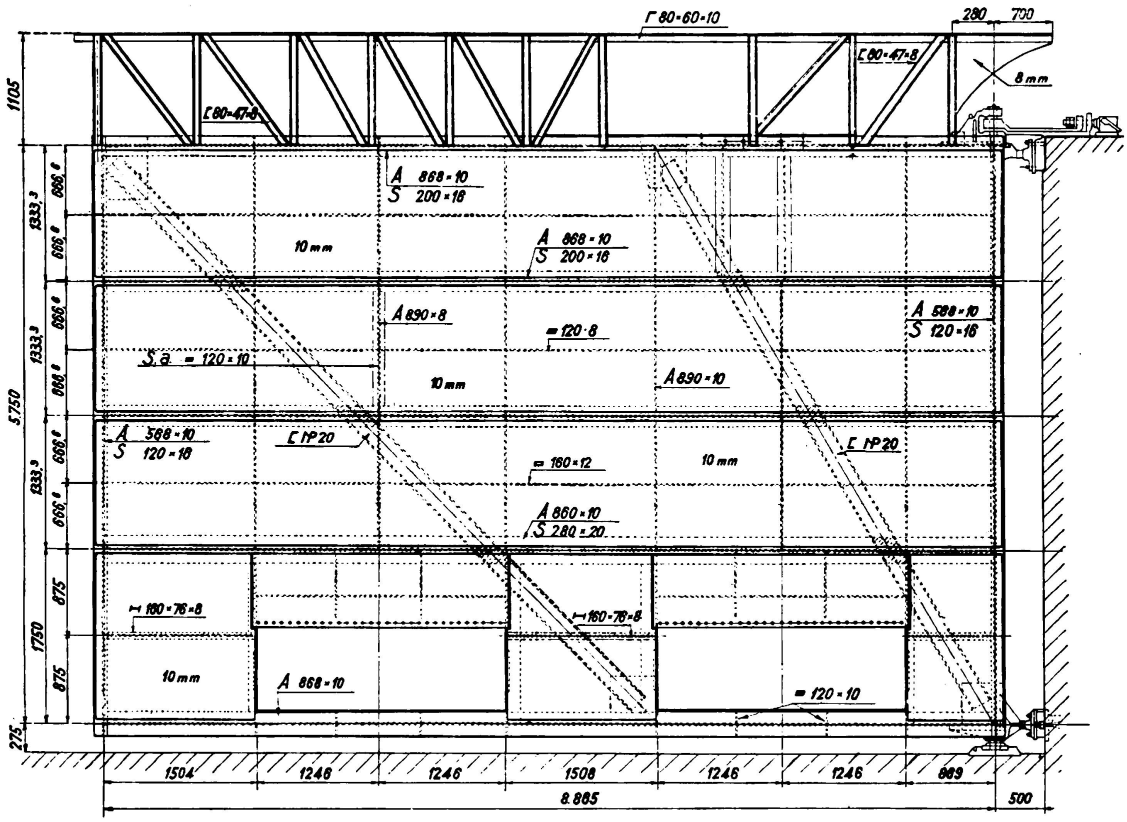


Fig. 11.

Wyneghem Lock, General arrangement of upstream gate.

The principal characteristics of the gates are the following: —

D o w n - s t r e a m G a t e : — Width of a leaf 5 m 67,
Rise of arch 1 m 90,
Theoretical height 9 m 49.

The single skin plating is of 10 and 10,5 mm plate. The framing and the central verticals are of DIL 50 joist. The transverse beams, except the bottom one, are of PN 500 pattern; the stiffeners of PN 120, 200, and 280; the St. Andrew's cross bracing is of channel section PN 240. The weight of a leaf is 19,524 kg.

T h e U p - s t r e a m G a t e, of similar construction to the down-stream gate, has a height of 5 m 365 and weighs 12 tons.

The method of making the sluices water-tight adopted at Wyneghem has been completely modified at Hérenthals.

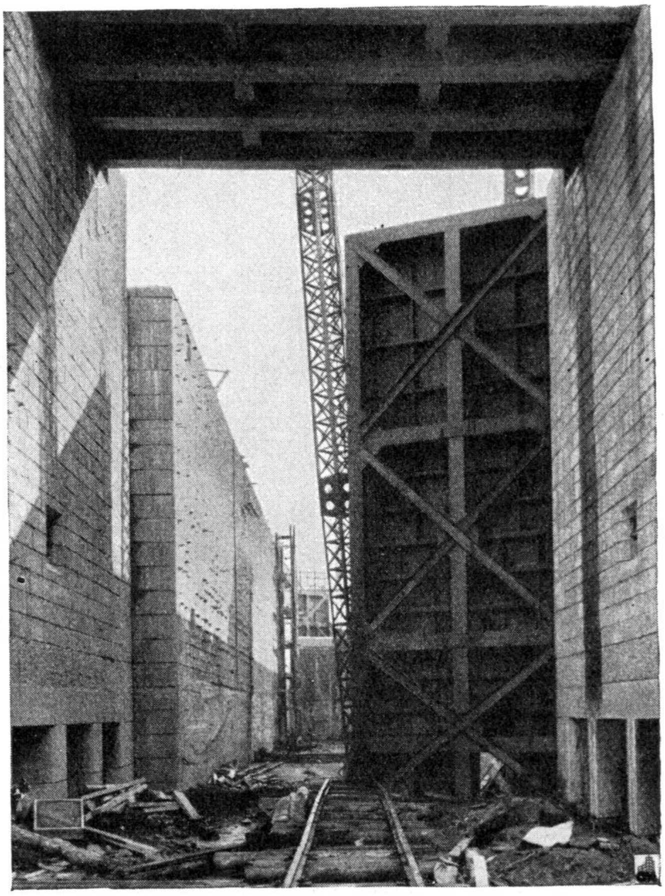


Fig. 12.
Erection of downstream gate of Herenthals Lock.

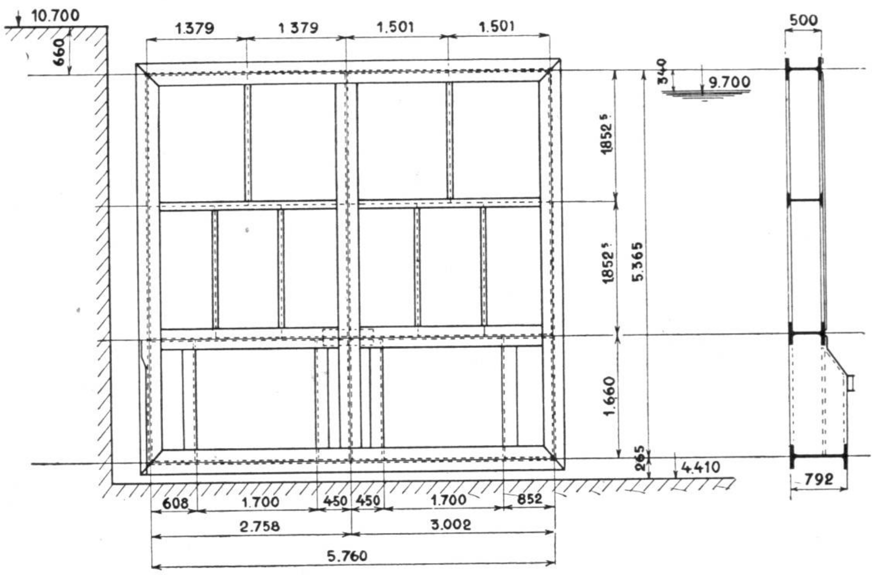


Fig. 13.
Nèthe Lock.

The rustless steel plates with bronze sectors have been discarded, and replaced, on the side and upper walls, by a one-piece rubber joint. For the lower part, the bronze plate has been retained. The rubber joint presses tightly on cast-steel

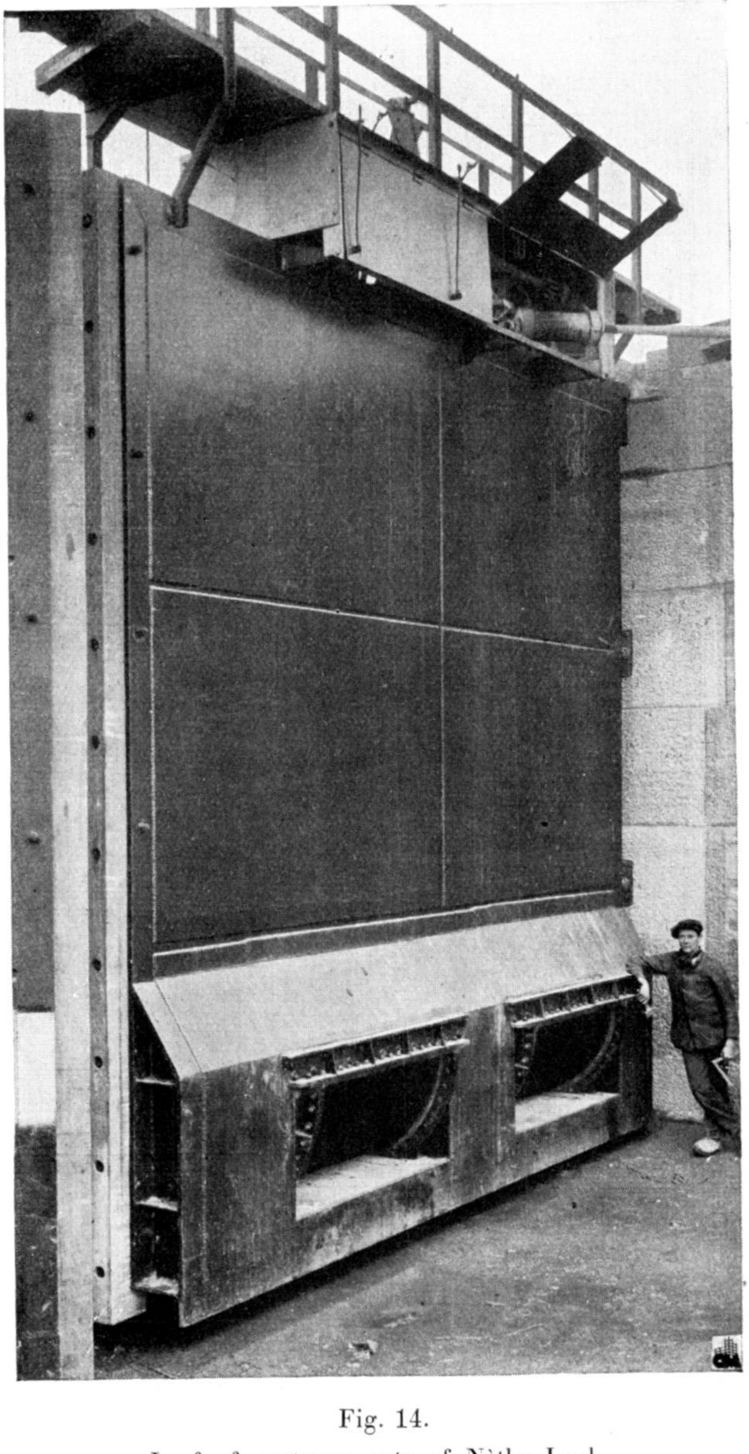


Fig. 14.
Leaf of upstream gate of Nèthe Lock.

guides fixed on the gate, and by means of a hollow round bar adapted to flatten out under the pressure of water, thus assuring perfect water-tightness.

The operating gears of the gate are of the type known as "Panama". The sluice segments are actuated from the foot-bridge of the gate by means of a rod and crosshead. The two sluices of each leaf are operated simultaneously. The gears are actually worked by hand, but are designed for subsequent electrification.

Summary.

In the course of the last five years, several lock-gates and sluices have been constructed in Belgium, partly on the new Albert Canal, partly for carrying out extensive works for the correction of waterways.

Most of these works were carried out with the aid of welding, which has brought about: —

1) A considerable saving in weight.
2) Perfect water-tightness.
3) Facility of maintenance.

As examples of these constructions, a description has been given of: —

1) The Marcinelle Lock, on the Sambre at Charleroi, in which the sluice is of the Stoney type and the lock-gates have a single lifting leaf.
2) The Wyneghem Lock with arched gates:
3) The Hérenthals Lock with arched gates.
4) The Nèthe Lock with arched gates.

Belgian constructors have completely solved the problem of welding for lock-gates.

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The Steel Structures of the Hydro-Electric Plant at Wettingen.

Die Stahlkonstruktionen des Limmatwerkes Wettingen.

Les constructions métalliques de l'usine hydro-électrique de Wettingen.

P. Sturzenegger,
Direktor der Eisenbau-Gesellschaft, Zürich.

1. Constructional Details.

The Limmat Power Station at Wettingen, 20 km below the point where the river leaves the Lake of Zurich, supplies electric power for the City of Zurich. It is one of the latest constructions of its type in Switzerland and was completed in 1933. The fall of the River Limmat is utilised by means of artificial damming, a solid barrage being combined with the actual power station in the river itself. As shown in Figs. 1 and 2, movable sluice gates are constructed in and above the solid barrage to cope with flood water. The barrage is of the articulated type, having interior hollows and four sluices. The latter are each 11 m wide, with intermediate piers 5 m in width having double-closing bottom outlets. These outlets, which close to a permissible leakage of 50 1/sec., are each provided upstream with a sliding sluice of 2.8 m and downstream with a segmental sluice of 2.5 m clear height. The maximum statical water pressure at sill level is equivalent to a head of 19.5 m. At its crown the barrage is provided with 4 automatic overflow traps, 2.5 m high, which serve for the fine regulation of surface level. Under normal working conditions the turbines and overflow traps together can cope with the average quantity of water brought down, so that it is only very seldom that the bottom sluices must be opened. The upstream sliding sluices form the closing device proper, while the rear segmental sluices serve to regulate the effective flow. Consequently, the sliding sluices operate permanently either entirely closed or fully open. In the latter case they are raised at least 1 m above the upper edge of the opening, so that they are not affected by hydro-dynamic influences.

The two bottom sluices are operated by seperate lifting gear. The mechanism for the sliding sluices is situated in the control cabins on the top of the barrage; that for the segmental sluices in the chambers inside it. The operating gear for the segmental sluices is designed for a full one-sided water pressure of 19.5 m, that of the sliding sluices is designed to operate at a one-sided water-head pressure of only 5 m. The reduction of the static pressure of 19.5 m head of water acting upon the closed sliding sluices to the pressure of 5 m at which the sliding sluices are designed to operate, in effected by pressure release pipes leading to the chamber between the two sluices. The valves of these pipes are situated in the same hollow chamber as the lifting gear of the segmental

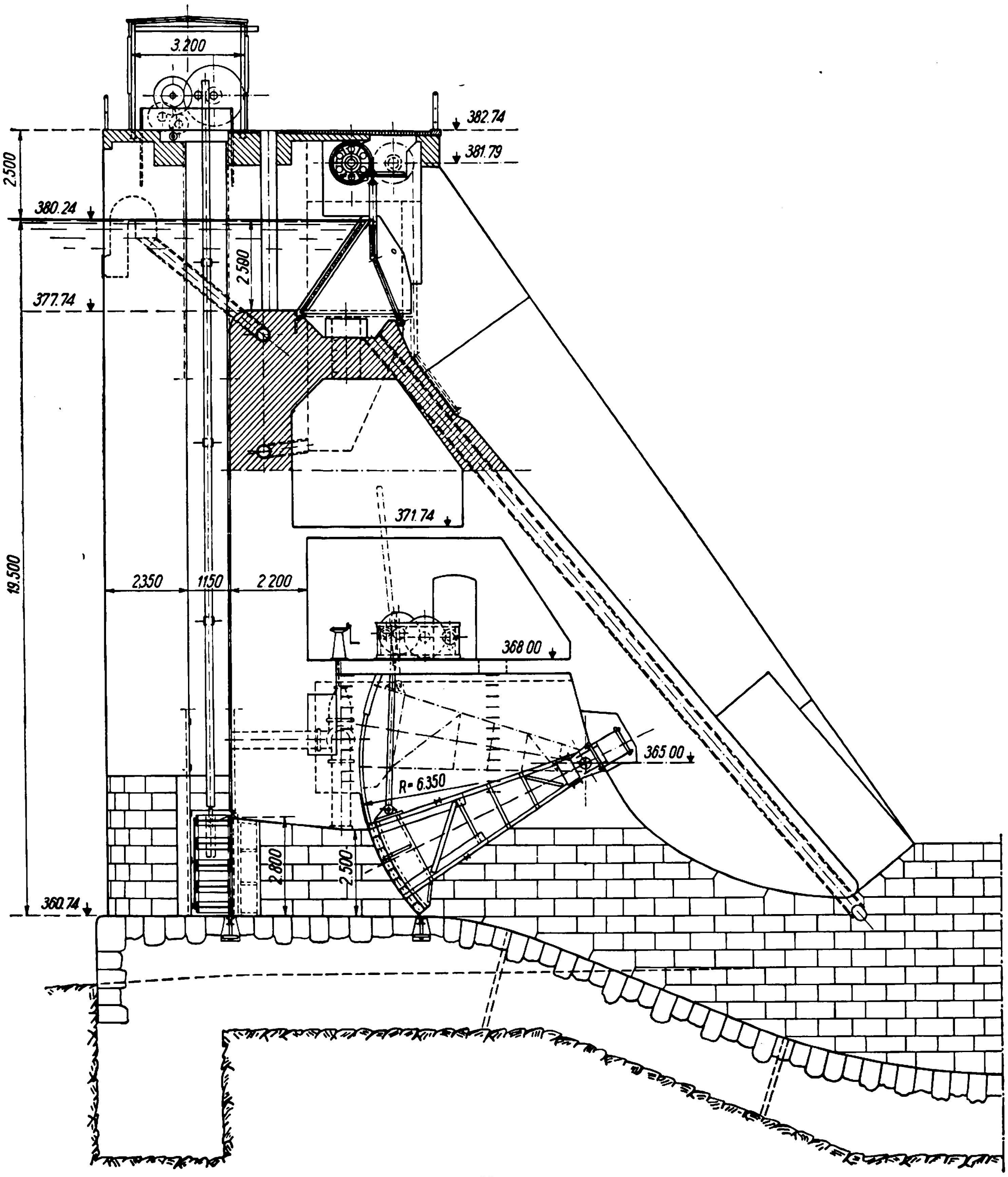

Fig. 1.
The Weir in cross section.

sluices. Water level in this chamber is constantly controlled by means of a Piezometer while the lifting gears are being operated. Complete equalisation of pressure was avoided so that the sliding sluices cannot lift away from their closures and permit the entrance of small floating particles between sluice and closure frame. Lifting gear constructed to operate at 5 m overpressure has to be comparatively sturdy. The lifting speed of the sliding sluices with electrical drive amounts to 0.2 m per minute, and with emergency drive (4 men per sluice) 0.55 m an hour. The segmental sluices, which have consequently to be operated under the full one-sided water pressure of 19.5 m, are equipped with extremely powerful lifting gear with lifting speeds of 0.5 m per minute when electrically driven and 0.7 m per hour with emergency drive (2 men per sluice).

The segmental sluices as well as the downstream faces of the closed sliding sluices are accessible via shafts leading from the middle deck of the barrage, so that the functioning of all movable parts, and above all the watertight closures can be inspected at any time. The overflow traps on the top of the barrage serving to effect more delicate surface regulations can effect automatic adjustments of + 2 cm and — 0 cm. We shall now proceed to examine the sluice valves of this rather unusual structure, which reveal some new and interesting departures in the construction of watertight closures.

2. *Watertight valves of the bottom outlets.*

The sliding sluices, of which Fig. 3 shows cross section, suspension, and support in the grooves, protrude on both sides into the separating piers, and in these grooves are situated the devices for supporting them and effecting a watertight closure. The body of the sluice, 11.4 m wide, consists of five rolled broad-flange girders which on the upstream side carry a skin of steel plating

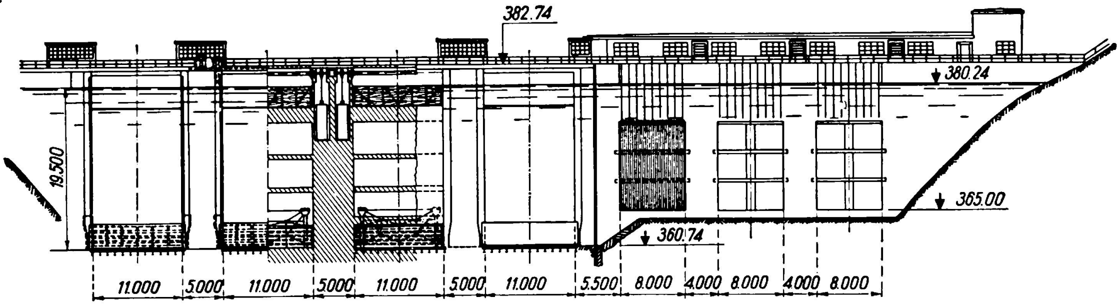

Fig. 2.
View and longitudinal section of Weir.

12.1 mm thick and which are connected by vertical solid webbed braces and by the lateral girders fitting into the grooves, to form a rigid unit. The flange of the bottom girder is situated 12 cm above the lower edge of the sluice gate — suffliciently high to obviate blocking of this opening from upstream and the consequent underpressure that would interfere with the movement of the sluice gate. Both top and bottom girders are additionally stiffened by ribs welded on between the braces on account of the direct vertical water load. Both girders fitting into the grooves transmit the water pressure to the lateral guides of the grooves by means of interchangeable bronze edging affixed to the sluice gates. These slide on the smoothly machined steel lining of the grooves, which can also be interchanged. The upper watertight closure is similarly designed. The closure at the sill of the sluice comprises a smooth steel edging affixed to the gate; the effective breadth of its edge is 25 mm, which contacts with the steel-reinforced sill as seen in Fig. 4. In order to obtain close grouting of the sill, the necessary holes were bored and cement mortar pressed in. These holes were then sealed with threaded plugs. All the other borings carried out for bolting were also subsequently filled up with lead, so that a smooth contact surface was given to the sill. By placing the upper girder in contact with the top closure deformation of this girder was obviated and a really watertight closure obtained. The lower main girders, on the other hand, are gradually deformed by the high water pressure, so that at the sill it was necessary to provide additonal closure in the form of an interchangeable hollow rod, as shown in Fig. 4. This, with its loose seating and ability to deform, presses tightly against the bottom edge of the sluice gate and the body of the sill, thus forming a closure which practical experience has proved to be perfectly watertight.

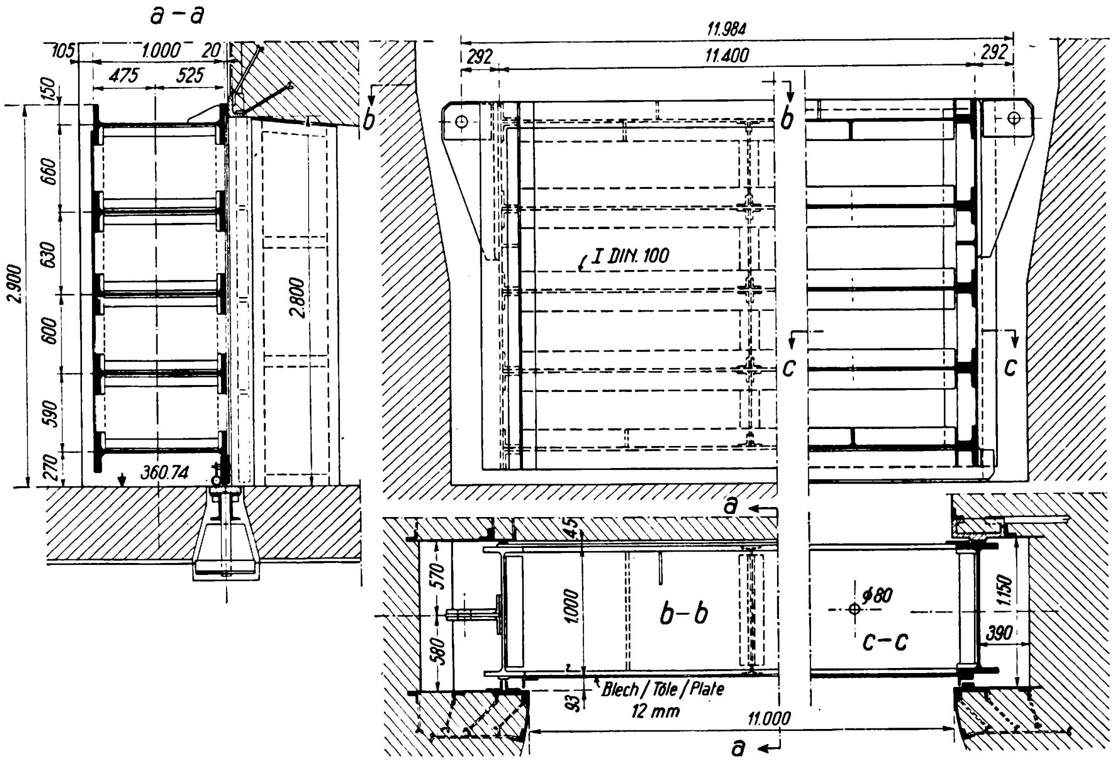

Fig. 3.
Sliding sluice.

The sluice gate is suspended in the gravity plane by means of two racks working in brackets situated entirely in the grooves and therefore out of reach of the force of the water rushing through the openings. The racks are divided into sections and can be removed when the sluice gate is completely drawn up for revision. The groove linings consist of cast steel rails in the region where they contact with the closed sluice gate, while their upper portion right to the top is of [-rails of lighter quality, overlaid with smooth steel plating, for here the gate moves under approximate equalisation of pressure. The upstream groove rails are similarly designed: here the sliding elements comprise the groove lining and the angle of the sluice gate edge. These angles, which recede slightly towards the grooves, also prevent floating objects from being drawn into the grooves. In the vicinity of the grooves the piers are reinforced with 12 mm thick interchangeable steel plates, while the interior of the grooves is lined with granite.

The sliding sluices are dimensioned for a static pressure of a 20 m head of water, in accordance with the Swiss Federal Regulations for Steel Structures.

The material used was Steel 37 with a yield stress limit of minimum 0.6 of the tensile strength; in the rails and watertight closures bronze, cast steel and special castings were employed. The lifting forces were calculated on the weight of the sluice gates minus upthrust, the weight of the racks, the water load on the upper closure edges (2 cm wide at 5 m overpressure), the weight of water on the sill closure (width 2.5 cm, overpressure 19.5 m), the frictional resistances based on a coefficient of friction of 0.35, giving a lifting force of 100 t in each opening. The closing power of the sluice gates was calculated on the frictional resistances as above, minus the dead weight of the gate and racks, less upthrust. In this manner a closing power of 30 tons was obtained for each opening, and the racks are designed to withstand a buckling force of this amount.

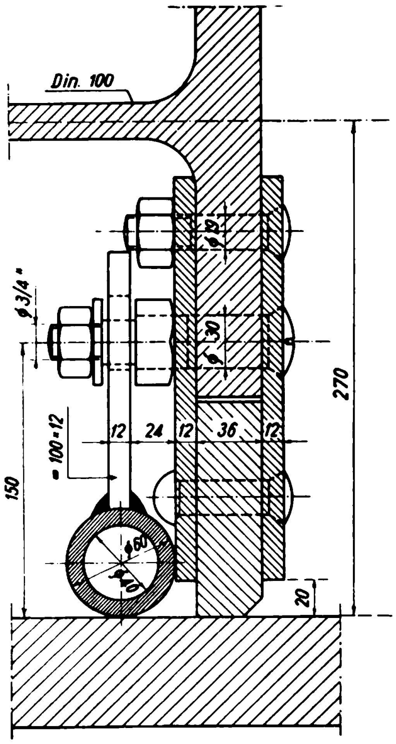

Fig. 4.
Watertight closure between gate and sill of sliding sluice

3. Regulation sluices of the bottom outlets.

The regulation sluices of the bottom outlets are designed as segmental sluices. They give an upstream closure at the same level and on the same constructional principle as the sliding sluices. They are pivoted at level 365.0. As shown in Fig. 5, these segmental sluices are constructed of two two-hinged girders with tie carried by the king pin bearing. The two pivoting walls, forming component parts of the two-hinged girders, together with the intermediate internal points of intersection of the latter carry transverse plate girders, which in turn carry the curved steel plating via a system of longitudinal girders. The tie member releases the king pin bearing from horizontal thrust. The two external pivoted walls were placed one meter back from aperture of the outlet to render the sluices and their closures easy of access. In consideration of the amount of water passing through under the pressure of a 19.5 m head, the segmental sluice was calculated on hydraulic dynamic efforts. All the bolts or nuts that have to be loosened or replaced are made of bronze. The sluice gate is suspended from the bracket-like projections of the transverse girders in the plane of the pivoted walls (Fig. 1).

The closure of the lower edge of the gate with the sill surface is effected by means of accurately machined cast steel edging fitted into the gate. The closure

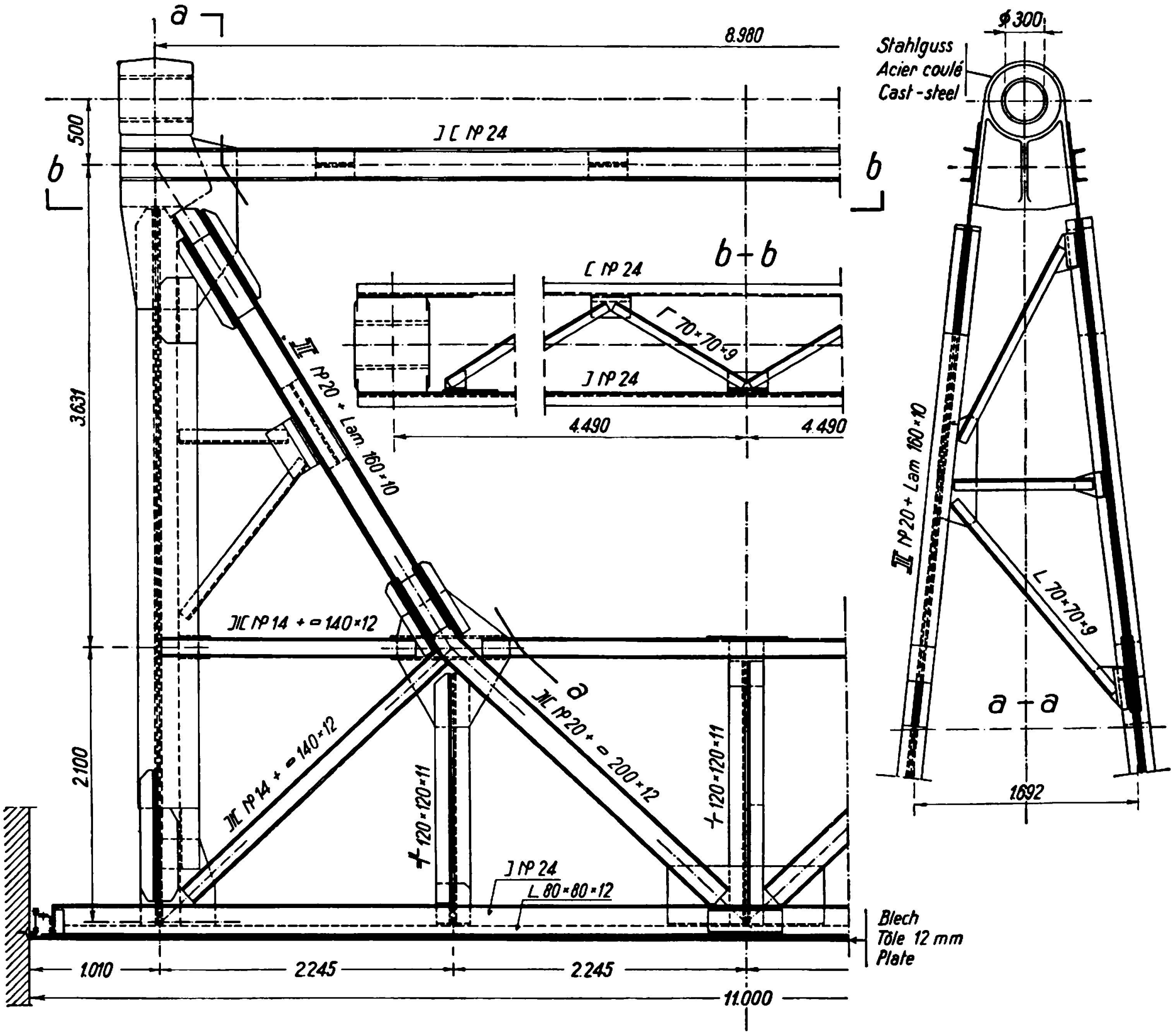

Fig. 5.
Steelwork of segmental sluice.

surfaces are restricted to 40 mm, in order to minimise any water pressure from below that might offer resistance to the closing of the sluice. As the total water pressure on this knife-edge just before the gate closes is entirely converted into speed, this pressure may be calculated as nil.

The lateral closures (Fig. 6) consist of spring steel plating of high strength. The thickness is 3 mm, and they are movably fastened to the binding angles of the gate. Their free ends are provided with angle reinforcement, which in turn carries the surface closure. In order to ensure closer fitting with the closure guides cemented into the walls of the piers, this edging is made in

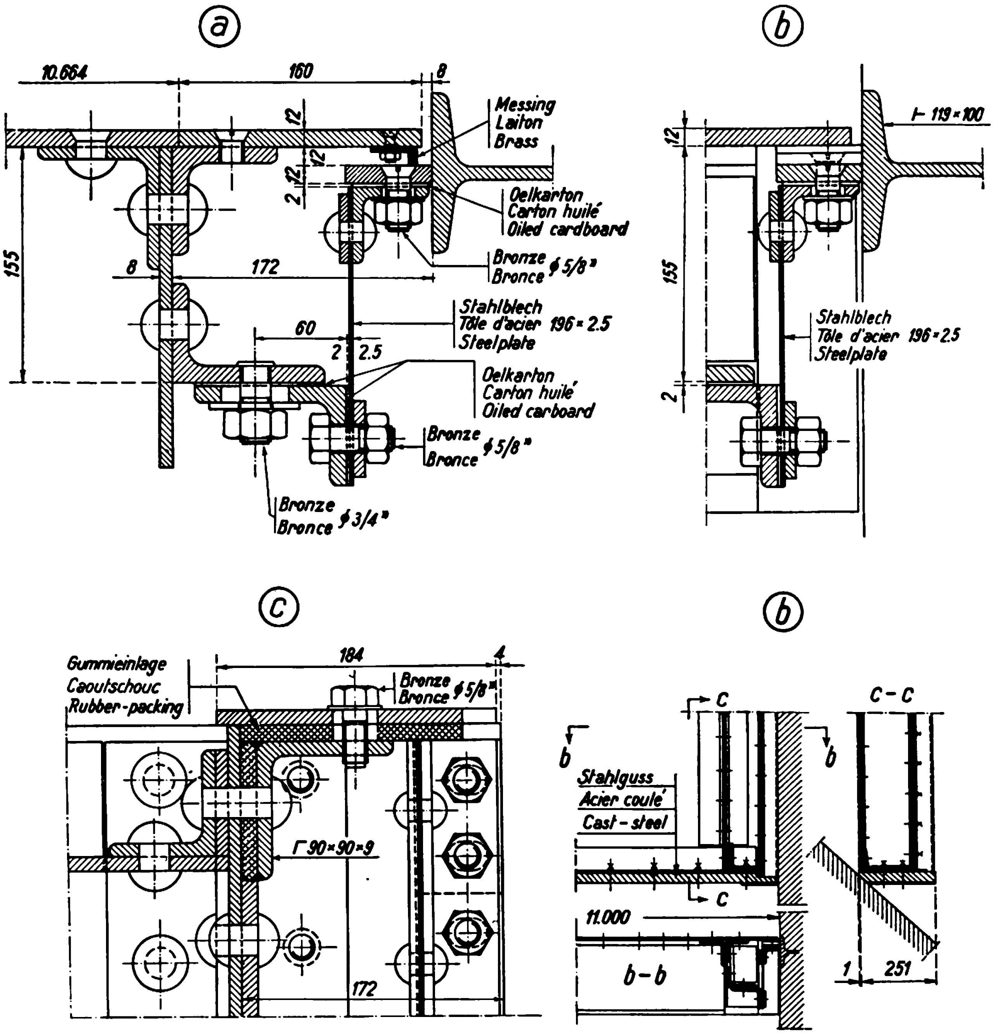

Fig. 6.

Closure of segmental sluice, shown in detail.

a) Normal lateral closure.
b) Transition from lateral to sill closure.
c) Transition from lateral to bottom edge closure (on sluice gate).

60 cm lengths and accurately machined. It fits into correspondingly planed guides in the walls of the piers. These fixed guides in the pier walls are continued upwards to a height prescribed by the necessity of resisting the water pressure on the lateral walls. The lateral closure edging is pressed up stream by the pressure of the water against the guides, the head of water in the lower part of the lateral closures entering the space between spring steel and the outermost cross girder over a length of about 10 cm. In order to prevent the water above from escaping unrestrictedly, the space referred to is closed with a compressible rubber plate and superimposed steel plating (Fig. 6). To prevent escape of the water under head pressure in the space between the outermost girder and the steel plating of the closure at the sides — over the top closure — angled brass edging is inserted between the steel plating of the gate and the fixed closure edging. The top closure (Fig. 7) consists of a tube fitting into the slot under pressure of the water. The steel casting forming the

guide for the tube is fastened to a continuous anchor iron at the head of the aperture. The top closure is so high that it comes above the upper edge of the aperture and is not affected by the rushing of the water. The entry of the water from upstream is effected by openings arranged in the upstream face of this casting and protected from impurities by copper wire gauze. In addition, the tube is also held in position by brass springs. This arrangement ensures that it is drawn surely in the direction of the fall of potential of the water and into the fissure it is designed to close. When testing the segmental sluice with these carefully constructed closures the permissible leakage of 50 l/sec. was therefore not reached.

By means of the two-hinged girders and their partial action as pivoted walls the water pressure on the segmental sluice, is transmitted to two king pin bearings situated 9 m apart and with a bearing capacity of 300 tons (Fig. 5).

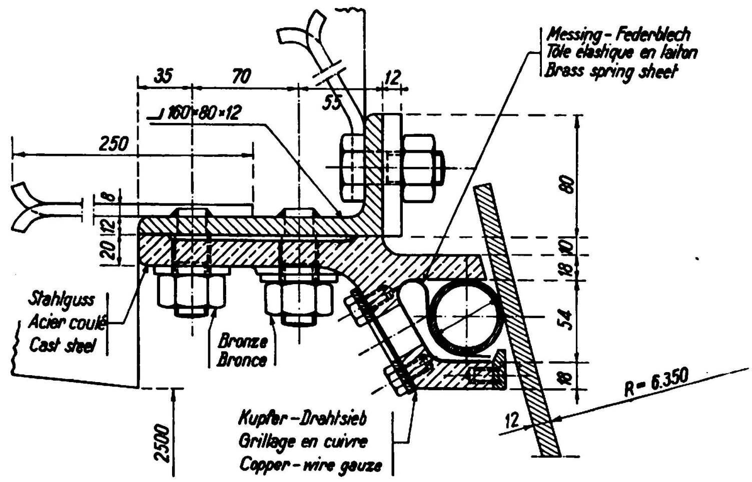

Fig. 7.
Closure at bottom edge of segmental sluice.

These cast steel bodies, with pin hinges of SM Steel secured against turning, are set in bronze bushes of 20 mm thickness and provided with Tecalemite-Grease pressure lubrication. The specific pressure in the bearings for the slow movement involved is maximum 165 kg/cm^2, a figure which was based on many years' experience.

The segmental sluice was assembled and bolted on scaffolding, after which the bearings were placed in their final position and grouted. Only then was the sluice gate riveted and, simultaneously, the lifting gear was erected so that the gate could subsequently be operated. When the segmental sluice had been installed and made movable, the closures were inserted in the following order: bottom, side and top. In order to fit the lateral closures accurately, their guides anchored in the masonry were first screwed on to the sluice gate and only grouted after their positions were accurately determined. The top closure was also temporarily affixed to the gate with its cast steel fittings, the gate lifted and after its passage every part examined for accurate fit. Only then were the fixed elements of the top closure grouted.

When calculating the steelwork of the structure the Swiss Federal Regulations were adhered to. The lifting forces were determined from the turning moments composed of dead weight, and friction at the bearings, top and side closures, these aggregate turning moments being equivalent to the lifting force multiplied by the leverage of the rack. To this was added 25% as a precautionary margin for resistances, giving a lifting force of 30 tons per opening. The amount of force required for closing the gates was ascertained from the turning moment consisting of dead weight, from which the turning moments of bearing friction opposed to the action of closing, the friction at top and side closures and the water pressure from below, acting with 20% of its full value on the sill closure

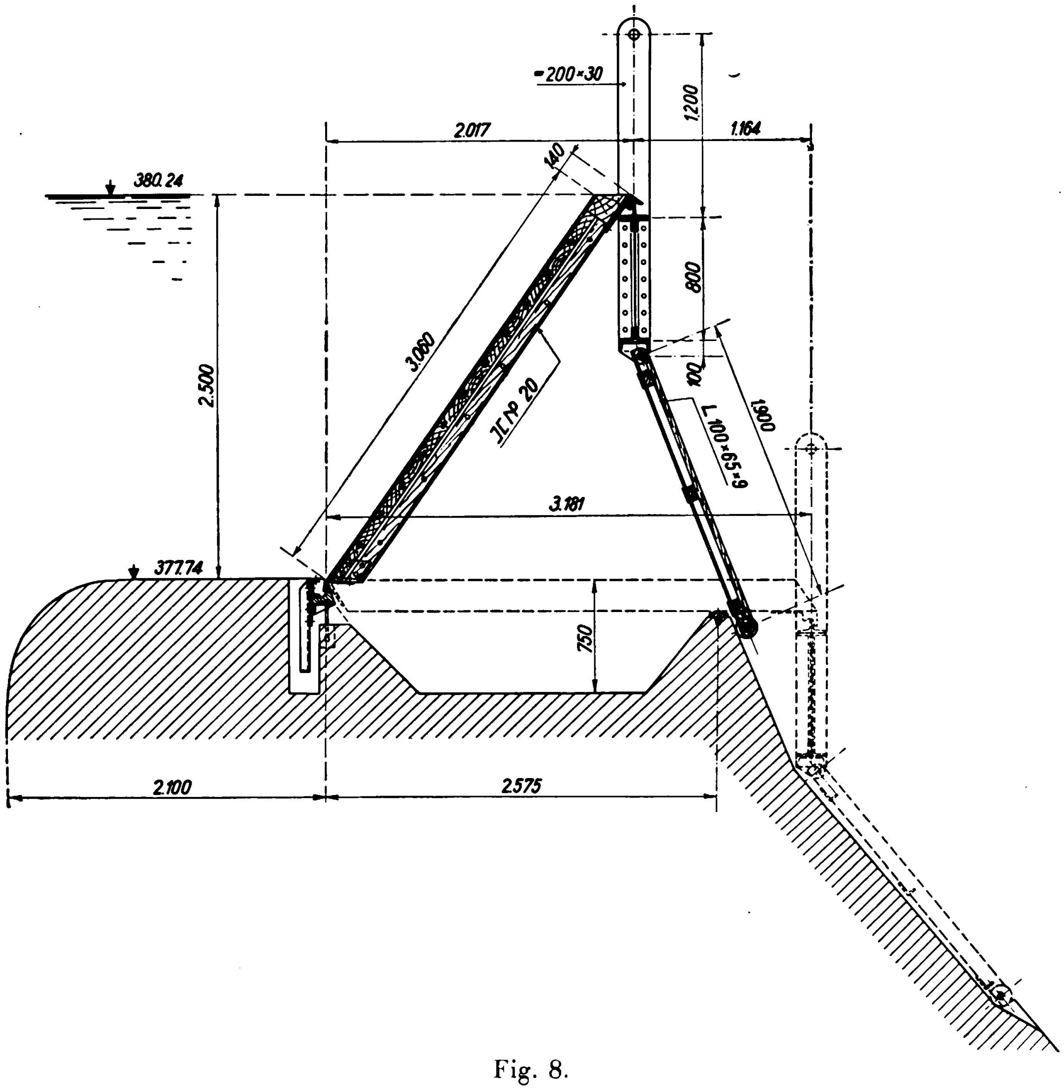

Fig. 8.

Section through overflow trap.

were deducted. By assuming equality with the closing force multiplied by the leverage of the rack, a downward thrust of 4 tons was obtained. Although the rack thus remained subject to tension, it was designed to withstand buckling. The top closure is calculated for a water pressure of 17.0 m head, the spring steel plates of the lateral closures, with a width of 16 cm, for a mean head pressure of 18.5 m. The coefficients of friction are assumed as 0.40 for the top closure, 0.30 for the planed lateral closures, the latter value also taken for pin friction under negligence of the lubricating effects.

4. Automatic overflow traps for fine surface regulation.

The overflow traps, System Hubert & Lutz, Zurich, are self-acting for hydrostatic pressure on the traps. The traps rest on knife-edge bearings situated on the top-sill of the barrage. At the top they are also supported by knife edges on a girder, the ends of the latter being affixed to straps attached to Gall-chains which are rolled up on chain drums. The drums are carried on a tube passing across the openings. The counterweights are housed in shafts situated in the piers of the barrage. When the traps move the rollers roll forward on rails anchored in the side walls, slipping being prevented by a rack guide. The

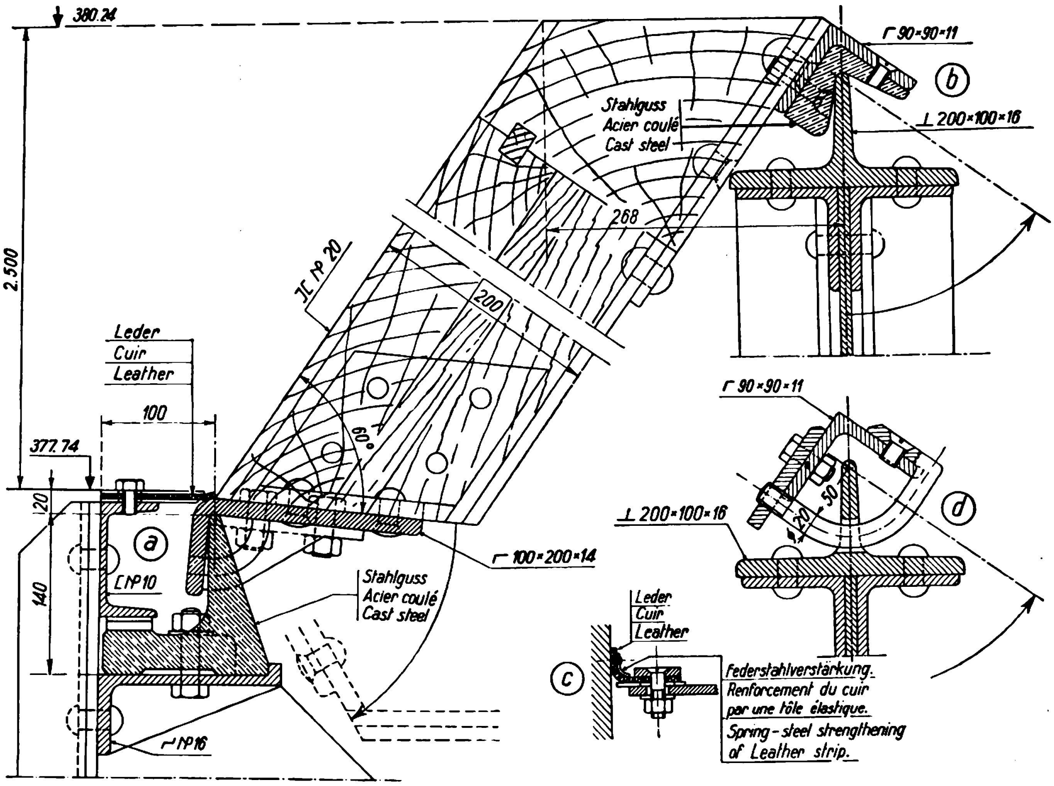

Fig. 9.

Overflow trap closure, shown in detail.
a) Lower knife-edge bearing.
b) Upper knife-edge bearing.
c) Lateral closure against pier.
d) Safety arrangement against lifting.

torsion strength of the rollers ensures a perfectly uniform movement of the trap, even if it has to operate under unequally distributed loading. In the construction illustrated in Fig. 8 the girders on the upstream side of the trap are overlaid with wood which is constantly kept either wet or at least under easy flow of water, so that it cannot dry out or crack. On the lower side of the supporting full-web girder is suspended a protecting apron composed of a steel skeleton covered with planking; its lower end is free to descend on rollers on the downstream face of the barrage crown. Thus a space isolated from the temperature of the outside air is formed below each trap, so that even at very low temperatures there is no danger of the trap bearings becoming iced.

This has been proved by experience. If desired, this chamber can be heated by warm air fed from the machine room.

The knife-edge bearings of the trap on the sill of the barrage and on the supporting girder (Fig. 9) reduce undesirable frictional resistances to a minimum. The closure of the trap along its pivot axis and at the sides is rendered almost completely watertight with the aid of leather strips reinforced with steel plating. The leakage is collected in a chute passing below the trap and taken off at the sides by pipes, so that the downstream face of the crown of the barrage is kept dry when the trap is closed. The side walls of each aperture, which contain the counterweight shafts, are covered with removable steel plating.

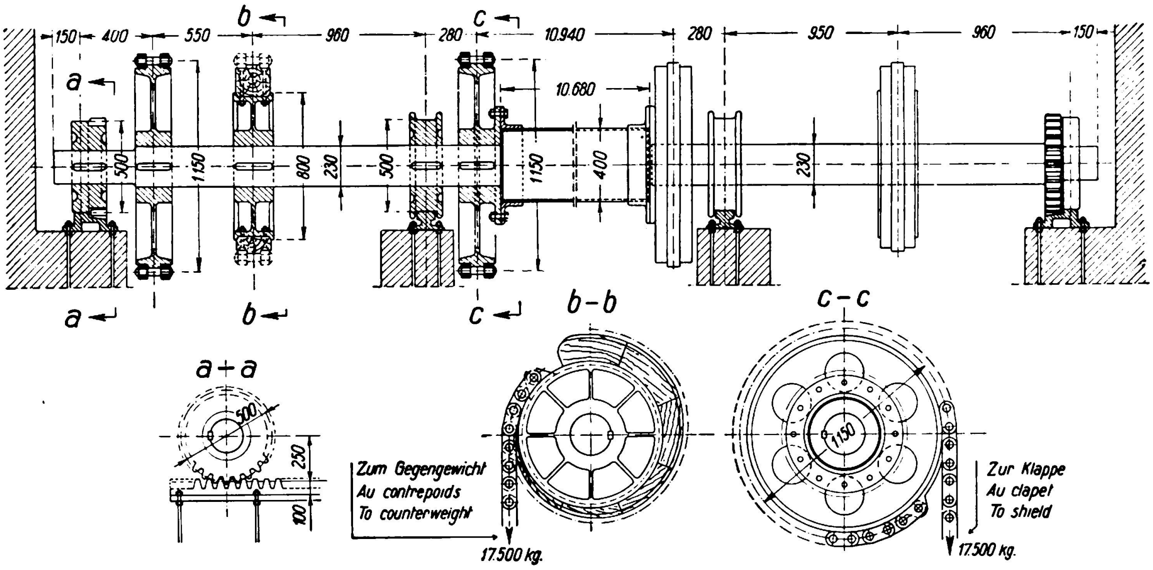

Fig. 10.

Upper bearing and counterweights of overflow trap.

They, as well as the air in the counterweight shafts, are electrically heated when the outside temperature is low, so that the danger of the closures and counterweights freezing up is obviated.

The traps are suspended at the sides by means of chain drums connected together by a steel axle high enough to clear the movable action of the trap; these drums roll on horizontal rails when the trap moves, as shown in Figs. 1 and 10. The cam-wheels regulating the action of the counterweights are mounted on the same axle. These cam-wheels are so calculated that the counterweights maintain equilibrium with the water pressure and the dead weight of the trap in any position of the latter so long as the surface of the water is at the normal level of 380.24. If the water level rises, the pressure on the trap increases and the latter is depressed; on the surface of the water sinking below the normal level, the trap is raised again by the action of the counterweights. The latter were designed somewhat heavier than was calculated, an auxiliary hydraulic force being utilised by connecting the counterweight shafts by means of tubes; the shafts now act as intercommunicating containers, so that by altering the quantity of water in them various degrees of upthrust can be imparted to the counterweights themselves. When the surface of the water rises an overflow feeds water to the shafts, increasing the upthrust on the counterweights, neutralising the overweight of the latter and allowing the trap to be depressed. This movement is again checked when the counterweights emerge from the water contained in the shafts, i. e. unless the surface of the water upstream continues to rise and further overflow is fed to the shafts. Vice versa, the trap cannot return to its erect position before the water in the shafts has been drained off by small overflows. Thus the trap moves smoothly and without the slightest jerkiness.

The two counterweights of each trap are so dimensioned that when they are submerged to a depth of 50 cm the traps are fully depressed, the overflow now feeding a quantity of water to the shafts corresponding to a clear overflow height of 2 cm.

The traps can also be depressed by outside agency, a valve situated below the sill of the trap allowing water to be fed to the counterweight shafts; the action of this artificial raising of the water level in the shafts is the same as if the water had entered normally through the overflow. Furthermore, the traps cope with sudden exigencies, as when a turbine is shut off and the flow of water downstream would otherwise become liable to sudden interruption. The automatic action of the traps not being quick enough in this case, especially if the level of the water is some what low vertically adjustable overflow funnels are arranged in the shafts to permit of the traps being depressed within two minutes of the inlet valve being opened. Thus 40 m^3/sec. of water can be allowed to escape.

Summary.

The paper describes a modern barrage in the damm shutting of the flow of the river contains four doubly-sealed bottom or scouring sluices, the upstream closure having the form of a sliding sluice gate, the downstream closure that of a regulating segmental sluice. The fine regulation of the surface is effected by automatic traps placed on the upper sill of the barrage; these can regulate the surface of the water to an extent of $+$ 2 cm and at the same time cope with sudden exigencies, such as when a turbine is shut off and additional quantities of water have to be let through. The bearings of the sluice gates and particularly their closures and hydraulic or electro-mechancial motive power are of improved design and action, so that the total leakage of the whole barrage is no more than 50 l/sec.