A New Approach to Reservoir Control

The author proposes a technique of reservoir control based on the phenomena of wave propagation. He believes that the adoption of this technique would enable storage-type hydro-electric plants to be operated more efficiently

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NOW that water turbines have achieved a very high degree of efficiency it might be thought that the practical limit of development has been reached. However, we should ask ourselves whether everything has been done to ensure the maximum possible efficiency of a hydro-electric development as a whole. Some hydro-electric schemes are operating far below their full hydraulic capacity, and in some cases the utilisation is as low as 50%. Are all aspects of hydro-electric projects tackled with the same degree of care as the generating plant? Should not greater efforts be made to attain the maximum possible reservoir efficiency?

Reservoir operation, so far from being a simple matter, can be most complicated; that is why it should be considered from various points of view. Reservoir operation is mainly based on watershed characteristics and on rainfall and run-off statistics, but it should always be kept in mind that reservoir control is a problem of hydrodynamics, a problem of space and time, and movement from stream to reservoir and reservoir to stream.

Water Waves

In hydrodynamics, only the motion of particles is considered and described by the equations of motion. But as these particles are of the same physical nature, their indistinguishability poses a very difficult problem. Therefore, we shall attempt to consider the motion of a system of particles subjected to gravity, friction and inertia in a given environment, as a water wave. Flood waves and every rise of stage in a flowing river may be considered to be water waves.

Thus, in general, all hydrological phenomena may be considered from the point of view of motion, not only the motion of particles, but also the motion of a system of particles under well-determined gravitational and environmental conditions. The propagation of flood waves may be analysed, their shape, deformation and mutual interaction being dependent upon their relative positions in space.

Hence, it would seem that hydrological research should no longer confine itself to the pure description of hydraulic phenomena, and that more efforts should be made to direct it towards the study of the dynamic structure of the hydrological process. The scope of research is now limited by a descriptive and purely statistical approach. Modern telemetric instruments make it possible to study whole sections of natural channels much more effectively than with hydraulic laboratory models.

By studying the dynamic structure of the hydrological process it is possible to explain some very complex phenomena and, in particular, it may be possible to arrive at a better understanding of forecasting problems; but such studies should not be based solely on classical hydrodynamics as represented by the theory of motion due to Lagrange and Euler. In the Lagrangian form of the differential equation, one has in mind a direct description of the motion of each particle of the fluid as a function of time. In the Eulerian form of the equation, attention

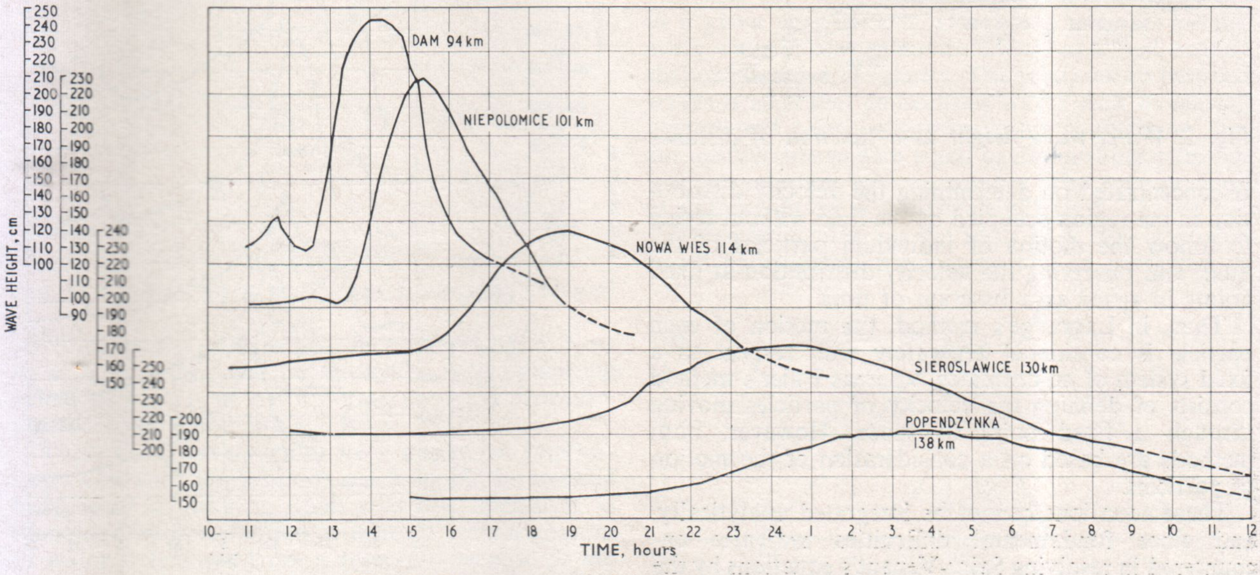

Fig. 1. Water-wave height as a function of time

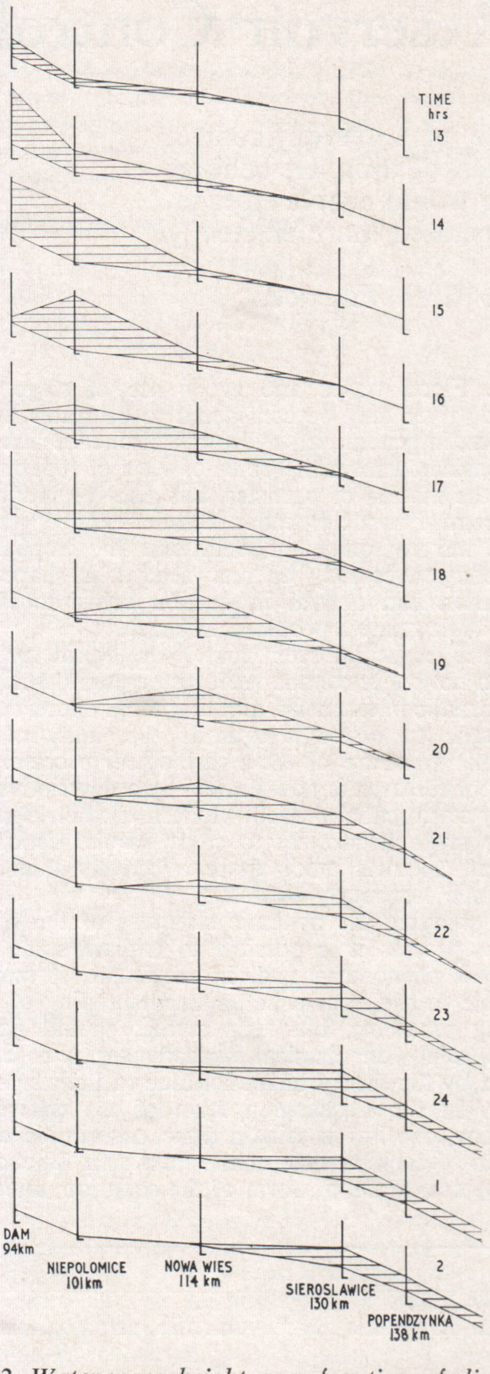

Fig. 2. Water-wave height as a function of distance

is concentrated on determining the velocity distribution in the region occupied by the fluid without trying to follow the motion of individual particles of the fluid, and observing the velocity distribution at fixed points in space as a function of time.

Thus, in Lagrange's method, the motion of each particle is considered separately with respect to a fixed system of co-ordinates, whereas Euler's method consists of defining the velocity of particles moving through a fixed point in space. However, both methods are based on a consideration of the motion of particles.

These equations cannot be integrated analytically, and since fundamental difficulties are also encountered in resolving Saint-Venant's equations by the method of finite differences in the case of discontinuous flow, another approach to these problems must be sought. A new opportunity of solving this difficult problem may be afforded by observing the motion of an entire mass of fluid, or, in other words, by studying the motion of a system of particles constituting a water wave, and by seeking to determine how its shape changes in time and in space. Following Louis de Broglie[1] we must try to find "a picture of the particle in which it appears as the centre of an extended wave phenomenon involving the particle in an intimate way."

However, the aim of this article is not to discuss the relationship between flood waves and the general problems of wave mechanics, nor is it intended to cover the mathematical analysis of these problems. Its purpose is to present some of the author's own observations of flood waves and to draw practical conclusions from them which can be used for another approach to hydrological phenomena and, in particular, to reservoir control.

Every rise of water level in an open channel should be regarded as a water wave. Such fluctuations should not be regarded merely as hydrological characteristics, but their movement, shape and distortion, and their mutual relationships should be studied like any mass in motion which is subjected to internal and external forces.

In the case of a rectangular open channel with constant width b, such observations may be represented by a function of space and time $h=f(t, x)$, where h is the depth, t time, and x the distance measured along the bottom of the channel. For every value of t, definite values of h can be obtained from observation at selected points x along the x-axis. The function may be presented alternatively as $h=f(t)$ (Fig. 1) or $h=f(x)$ (Fig. 2). Figs. 1 and 2 represent an artificial water wave released from a low-head hydro-electric plant and observed at five points downstream of the plant. Similar flood-wave data are obtainable from hydrographs at successive gauging sta-

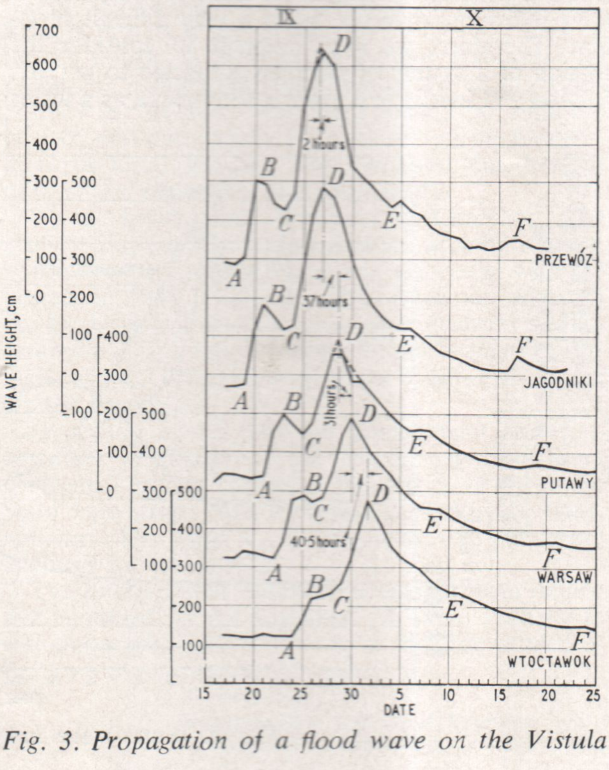

Fig. 3. Propagation of a flood wave on the Vistula

tions on a river. Fig. 3 illustrates the propagation of a flood wave on the Vistula over a distance of about 600 km with initial and terminal catchment areas of 8,300 km² and 170,000 km² respectively.

Detailed analysis of such hydrographs shows that a given flood wave will retain essentially the same shape, even over distances of several hundred kilometres (Fig. 3). The transformation of waves as they pass along a channel under the effect of changing environment is governed by gravity, friction and inertia. Such changes are represented equally well by $h_x = f(t)$ or $h_t = f(x)$, and may be analysed in Figs. 1, 2, and 3. The changes over an interval $x_2 - x_1 = \Delta x =$ constant may also be analysed as shown in Fig. 4.

The study of a large number of natural waves has clearly shown that every wave has a well-determined velocity dependent upon its height. The greater the height h of a wave, the greater will be its velocity v. Considering two consecutive waves h_1 and h_2, we have $v_1 < v_2$ when $h_1 < h_2$. Thus a large wave may overtake a smaller preceding wave, in which case the two waves will form one larger wave. On the other hand, when $h_1 \geqq h_2$, then $v_1 \geqq v_2$, and the waves will not interfere with each other. The two waves will proceed separately and their change in shape will be the same as for single or solitary waves. As a result of this, a solitary wave may be considered as being a wave formed by the conjunction of a number of superim-

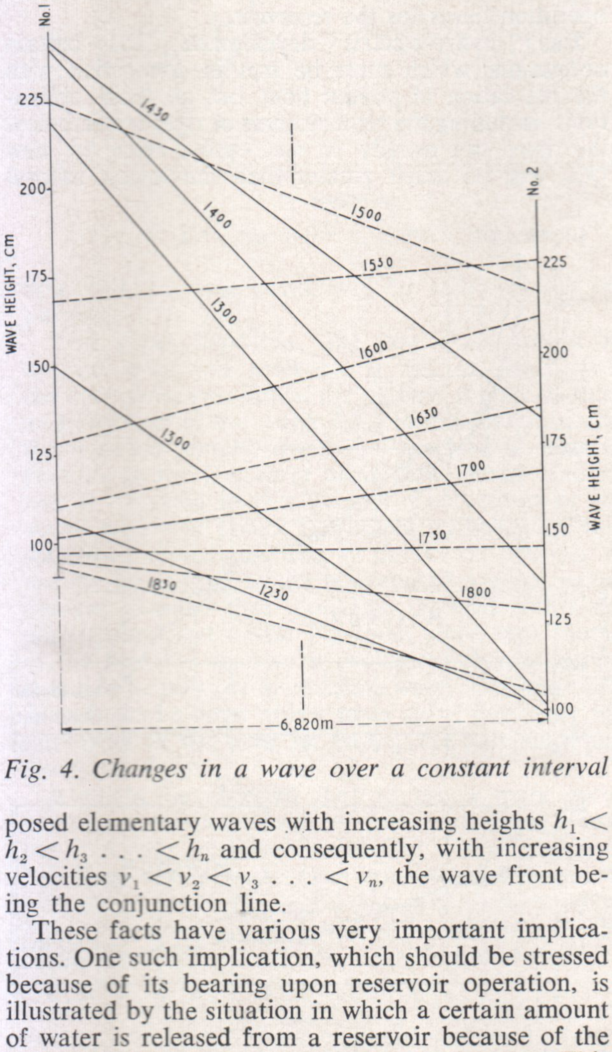

Fig. 4. Changes in a wave over a constant interval

posed elementary waves with increasing heights $h_1 < h_2 < h_3 \ldots < h_n$ and consequently, with increasing velocities $v_1 < v_2 < v_3 \ldots < v_n$, the wave front being the conjunction line.

These facts have various very important implications. One such implication, which should be stressed because of its bearing upon reservoir operation, is illustrated by the situation in which a certain amount of water is released from a reservoir because of the expected arrival of a dangerous flood wave. This

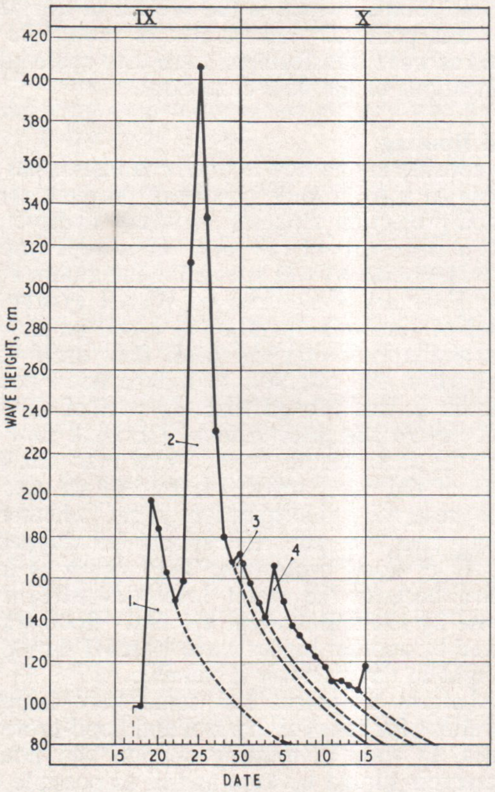

Fig. 5. Superposition of natural waves

results in a small wave followed by a bigger one, and somewhere farther downstream the two waves may merge with apparently inexplicable and disastrous results. In a given environment, such as a section of channel, a given mass of water released in a fixed interval of time will always create an identical water wave. Modifications of the cross section, such as when the shifting of silt deposits changes the flow conditions, will alter the shape of the wave in space and time. Any structure in a channel which alters the flow conditions will cause a significant change in the wave form.

Briefly, we can summarise this by saying that the shape of a wave, which is a moving system of particles, depends on the internal and external forces in a given environment. Depending upon its height, every wave in a given environment has a well-defined shape, volume, and celerity. The relationship between waves is such that consecutive waves with increasing heights and celerities proceed to a conjunction, but consecutive waves with equal or diminishing celerities will not interfere with each other. The superposition of natural waves is illustrated in Fig. 5.

These principles are the basis of a new approach to all hydrological phenomena and in particular to

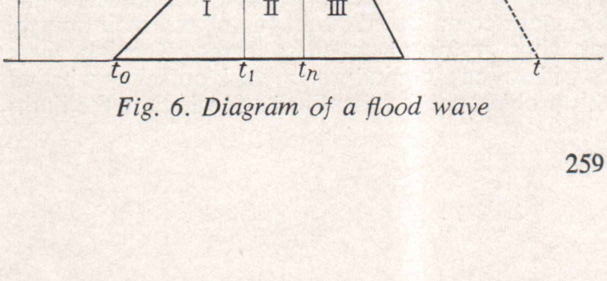

Fig. 6. Diagram of a flood wave

reservoir control, which is the subject of this article. From this point of view reservoirs should be considered as wave transformers, since they cause general modifications of the flow conditions.

Flood Routing

By considering motion in open channels it has been possible to solve a very important problem relating to flood-protection schemes. In many hydro-electric plants a large part of the reservoir is kept empty to cope with floods. As was stressed in the author's letter in the February 1962 issue of WATER POWER, this method of reservoir operation is based on the erroneous assumption that the peak flow itself has to be caught in the reservoir to reduce flood flow. In this letter it was proved that the part of the flood caught *before the peak* reduces the peak flow with almost the same result. In the event that part I and part II of the flood wave (see Fig. 6) are not held in the reservoir, the question arises whether the receding part, i.e. part III, should be trapped for purposes of flood protection. The answer is in the negative because the water level downstream will decrease by itself. But when a solitary flood wave is followed by another higher wave, part III should also be caught in the reservoir if possible.

The above statement has some very important implications for reservoir control and flood-protection schemes. In spite of existing opinion and practice, the reservoir should be kept as full as possible from the very beginning of a flood, particularly during a large flood, and should be kept at the maximum level for the duration of the flood. Thus, as far as flood protection is concerned, reservoir control becomes to some extent independent of the inflow Q_1. All that should be done is to fill the reservoir as much as possible, and this applies to any size of reservoir. From this point of view, it is unnecessary, and not at all advisable, to draw down the water surface ahead of an expected flood or to keep an empty reserve below the full reservoir level. By operating as closely as possible to the upper limit, higher heads are maintained and best reservoir efficiency and optimum flood protection is secured. As will be shown later, this approach may lead to the complete automatisation of reservoir control.

The Key Diagram

On this basis let us consider a hydro-electric development with large storage giving complete regulation, that is, whatever the inflow Q_1, the outflow Q_2 from the reservoir will always be Q_m, the long-period mean flow for the section of the river. Breaking down water requirements according to daily or weekly peak-load demand, is excluded from these considerations. This is a minor problem when a tail-water pond is available.

In such a case, where the mean flow Q_m is always available and therefore known beforehand, forecasting the amount of water for reservoir operation is no longer required, but is limited to the determination of available head. Complete regulation, however, is seldom, if ever, attained (see author's paper to the ninth Congress of the IAHR). In most hydro-electric developments only partial regulation is achieved and the more complicated problems of reservoir control are thus restricted to these cases. Keeping these considerations in mind we may ask ourselves whether this problem can be simplified and what are the limits of operation of such a reservoir.

Let us consider a large reservoir characterised by partial regulation. The degree of regulation achieved can be determined before the structure is built, and the new minimum, which is often called the guaranteed minimum flow, can be computed with a high level of probability and sometimes even with certainty. This value depends on the capacity of the reservoir and, in general, ranges from 40 to 75% of the average flow over a long period.

In the same way, the upper limit on the long-period mass curve, i.e., the new maximum monthly flow, may be fixed. This new value of the monthly maximum flow also depends on the capacity of the reservoir and is in general 30% of the unregulated monthly maximum flow. These two values, often called the hydrological characteristics of the plant, should be adhered to as strictly as possible in the operation of the reservoir. The question may be asked whether it is permissible to reduce flows below the new calculated Q_{min}. Often a minimum flow has to be maintained below the dam in the interests of water supply, avoiding stream pollution or fish conservation. Usually this is the lowest natural flow that occurred before the dam was built. This practice seems to be correct from the point of view of water conservation but it loses sight of the fact that valuable space is thus lost for future incoming flow and that it becomes impossible to apply the most suitable and efficient operation limits for the reservoir.

Each hydro-electric development has certain obligations which must be met in connection with the regulation of stream flow, but all these obligations, including the requirements of river traffic below the dam, can usually be met easily, since the new Q_{min} will be much greater than the lowest natural

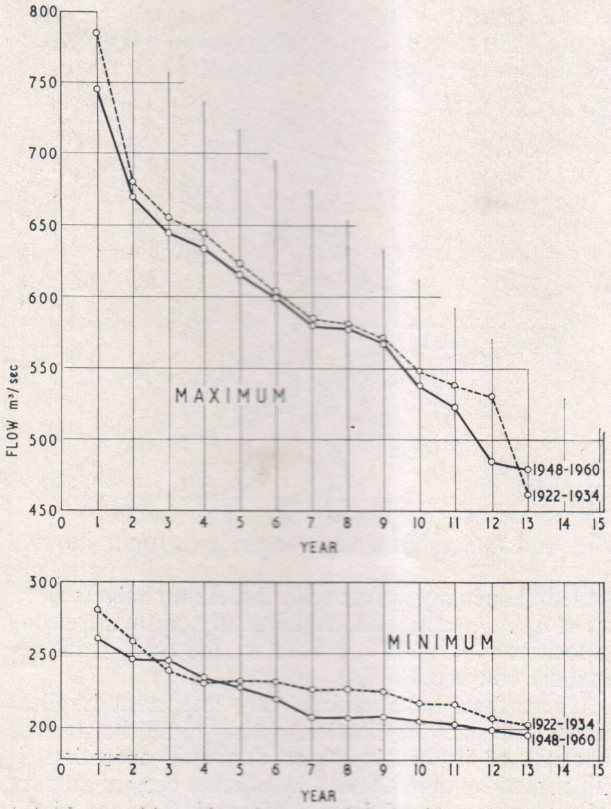

Fig. 7. Comparison between two long-term periods

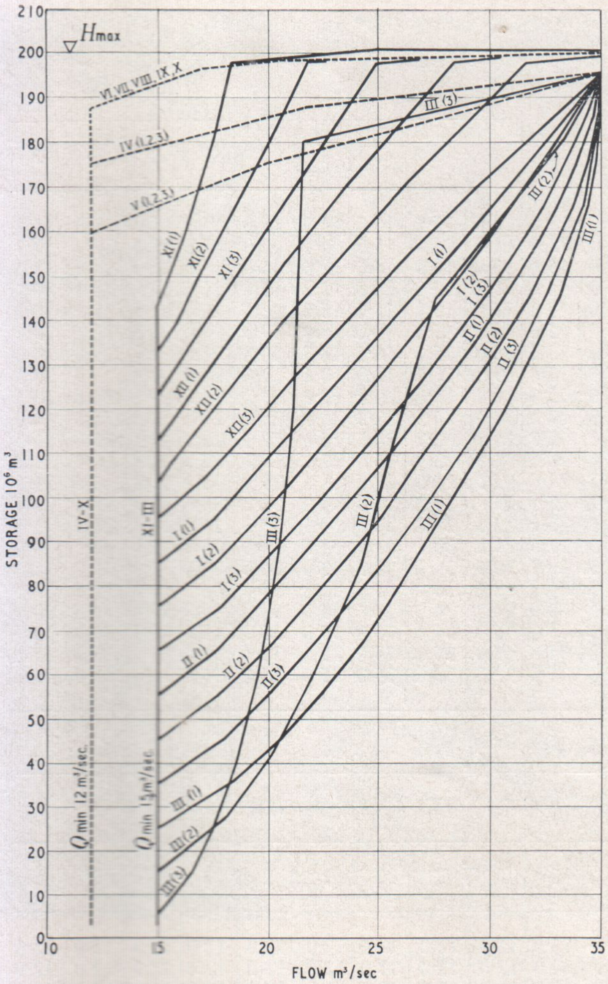

Fig. 8. Key reservoir operation diagram

flow and, in most cases, will be sufficient for the purpose.

The upper limit, i.e., the new Q_{max} should be applied when the reservoir is full or nearly full. Between these two limits, the outflow should be fixed as a function of the water level in the reservoir, and the reservoir should always be operated as near as possible to the long-term mean flow. When a reservoir is operated in this manner, fluctuations of the water level will reflect the actual hydrological and watershed conditions, and it should be possible to use the water level in the reservoir as a guide to the amount of water that should be released within the above-mentioned limits. However, since each month of the year has its own hydrological characteristics, it should be possible to correlate water level and time. This might be done with a mass curve in exactly the same way as was used for finding the maximum and minimum flows.

It might be objected that to use a mass curve assumes a full knowledge of future incoming flows; but it should be remembered that if one compares two long-term periods, each consisting of the same number of years, the hydrological characteristics of the two periods will be found to be substantially the same. Thus, the assumption is justified, and reservoir operation on this basis can be precise.

Many cases of the above have been tested by the author, and in Fig. 7 an example is given in which decreasing values of the maximum and minimum stages at a gauging station with a catchment area of 168,000 km² have been plotted for the years 1922–1934 and 1948–1960. Such comparisons may be made for every watershed where the stream is not artificially modified by engineering structures and where it has a fairly stable cross section.

From these considerations it is evident that a new cycle for a given catchment area is a repetition of the same hydrological phenomena; it involves waves with different conjunctions and different superpositions, but it is the role of a reservoir to even out such differences. It does not matter whether the exact cycle is known or not since it can be justifiably argued that the hydrological characteristics corresponding to two long periods are very similar. For this reason a reservoir operation scheme based on a long-period mass curve may be successfully applied to everyday plant operation on condition that it is followed as strictly as possible between the assigned limits of operation. This will ensure the full use of storage and maximum head. The gain resulting from a higher average head will certainly more than offset any small loss of spill, and it can be shown that by operating in this manner maximum reservoir efficiencies are obtained.

The author has worked out what may be called a key reservoir operation diagram (Fig. 8) for a section of a mountainous river having the following natural hydrological characteristics:

Catchment area ... ... $A = 1{,}100$ km²
Long-period average flow $Q_m = 21$ m³/sec
Maximum momentary flow $Q_{max} = 1{,}200$ m³/sec
Minimum flow ... ... $Q_{min} = 4$ m³/sec

The characteristics for a long period are given in Table I. The reservoir was assumed to have a useful capacity of 200 million m³, and careful analysis was applied to prepare a reservoir operation schedule on the basis of a long-period mass curve plotted from ten-day flows. It should be remembered that a mass curve plotted from ten-day flows is correct only at the beginning and end of each ten-day interval, since the variation of flow during the interval is not taken into consideration.

A key reservoir operation diagram was thus obtained (Fig. 8 and Table II) which indicates exactly what flow Q_2 should be released from the reservoir

TABLE I

MONTHLY SUMMER FLOW (M³/SEC)

Month	IV	V	VI	VII	VIII	IX
Maximum ...	45·6	59·8	69·0	98·0	57·0	63·5
Minimum ...	6·9	11·2	14·2	9·2	7·8	6·7
Average	26·9	27·9	25·9	31·2	24·8	18·8

MONTHLY WINTER FLOW (M³/SEC)

Month	X	XI	XII	I	II	III
Maximum ...	41·3	34·2	20·2	22·4	23·1	46·9
Minimum ...	4·8	4·2	4·7	4·0	4·1	4·3
Average	16·4	15·4	11·0	9·9	13·1	22·3

TABLE II.—$Q_2 = m^3/sec$

Month	X			XI			XII			I			II			III		
Ten-days	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
For 10 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·8
For 20 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	16·3	17·1
For 30 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	16·8	18·6	18·0
For 40 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	16·6	19·5	20·0	18·7
For 50 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	16·5	19·1	21·5	21·2	19·2
For 60 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	16·4	19·9	21·0	23·2	22·2	19·6
For 70 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	16·5	18·7	20·8	22·8	24·7	23·0	20·0
For 80 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	15·0	16·4	18·9	20·6	22·6	24·5	26·1	23·9	20·3
For 90 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	15·0	16·4	18·6	20·7	22·3	24·2	26·0	27·3	24·6	20·6
For 100 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	15·0	16·2	18·6	20·5	22·5	24·0	25·8	27·5	28·5	25·0	20·8
For 110 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	15·0	16·1	18·3	20·3	22·2	24·3	25·6	27·1	28·9	29·7	25·6	21·0
For 120 million m^3	12·0	12·0	12·0	15·0	15·0	15·0	16·1	17·6	20·0	22·1	23·9	26·0	27·1	28·6	30·0	30·8	26·1	21·2
For 130 million m^3	12·0	12·0	12·0	15·0	15·0	16·0	17·6	19·2	21·8	23·9	25·6	27·6	28·6	29·9	31·1	31·8	26·8	21·3
For 140 million m^3	12·0	12·0	12·0	15·0	15·9	17·3	19·0	20·9	23·7	25·7	27·2	29·0	30·0	31·2	32·1	32·7	27·4	21·4
For 150 million m^3	12·0	12·0	12·0	15·6	17·0	18·7	20·5	22·7	25·6	27·5	28·8	30·5	31·3	32·3	33·0	33·5	28·5	21·4
For 160 million m^3	12·0	12·0	12·0	16·3	18·0	20·0	22·0	24·5	27·5	29·2	30·2	31·7	32·5	33·2	33·8	34·1	30·1	21·4
For 170 million m^3	12·0	12·0	12·0	17·0	19·1	21·3	23·6	26·3	29·5	30·9	31·7	32·9	33·4	34·0	34·4	34·6	31·7	21·6
For 180 million m^3	12·0	12·0	12·0	17·6	20·1	22·7	25·3	28·2	31·7	32·6	33·0	33·8	34·2	34·6	34·7	34·9	33·3	21·6
For 190 million m^3	13·6	13·6	13·6	18·0	21·1	24·0	27·1	30·2	33·9	34·2	34·3	34·6	34·8	34·9	34·9	35·0	34·5	30·5
For 195 million m^3	16·6	16·6	16·6	18·3	21·6	24·6	28·0	31·2	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0
For 198 million m^3	20·0	20·0	20·0	20·0	23·0	26·7	29·5	33·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0
For 199 million m^3	30·0	30·0	30·0	30·0	30·0	30·0	33·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0	35·0

Month	IV			V			VI			VII			VIII			IX		
Ten-days	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
For 150 million m^3	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0
For 160 million m^3	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0
For 170 million m^3	12·0	12·0	12·0	17·5	17·5	17·5	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0
For 180 million m^3	16·3	16·3	16·3	24·2	24·2	24·2	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0	12·0
For 190 million m^3	26·7	26·7	26·7	31·6	31·6	31·6	13·6	13·6	13·6	13·6	13·6	13·6	13·6	13·6	13·6	13·6	13·6	13·6
For 195 million m^3	35·0	35·0	35·0	35·0	35·0	35·0	16·6	16·6	16·6	16·6	16·6	16·6	16·6	16·6	16·6	16·6	16·6	16·6
For 198 million m^3	35·0	35·0	35·0	35·0	35·0	35·0	20·0	20·0	20·0	20·0	20·0	20·0	20·0	20·0	20·0	20·0	20·0	20·0
For 199 million m^3	35·0	35·0	35·0	35·0	35·0	35·0	30·0	30·0	30·0	30·0	30·0	30·0	30·0	30·0	30·0	30·0	30·0	30·0

when only the water level or the amount of water in the reservoir is known. This diagram was checked and verified against another long-period mass curve. A detailed analysis of the diagram will show its adaptability to the complete automatisation of reservoir control.

As was mentioned earlier, the key reservoir operation diagram should be followed as closely as possible, but this does not mean that within periods of a day, a week, or even more, it may not be modified to suit the demand for power or to secure the optimum correlation with steam plant, always under the condition that over this period the total release shall be in accordance with the diagram.

In this respect the key reservoir operation diagram is entirely different from the so-called rule curves which summarise detailed engineering analyses and thus serve merely as guides.

Forecasting

Paradoxical as it might seem, it would appear that there is no further need for forecasting in connection with reservoir control, and in certain respects this is true. It will not be necessary to draw down reservoir storage to leave room for flood water or to leave any margin below the full reservoir level for the same purpose. The reservoir can be filled without endangering the dam by operating in accordance with the key diagram and, in particular, when a flood occurs it should be kept full as long as the flood persists.

Storage should not be based on maintaining specific reservoir levels at particular times of the year, as is the conventional practice, but on maintaining a pattern of flow in accordance with the levels that are found to obtain in any particular period. By the use of long-term mass curves this pattern can be determined with a high degree of probability.

The provision of reservoir-control techniques along these lines may effect a remarkable improvement in the efficiency of a hydro-electric plant.

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