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# **HYDRAULIC DESIGN FEATURES OF THE CANADIAN RIVER AQUEDUCT**

by

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## HYDRAULIC DESIGN OF CANADIAN RIVER AQUEDUCT

The 320-mile Canadian River Aqueduct is the Bureau of Reclamation's longest pipeline system. It delivers municipal water from Lake Meredith at Sanford Dam to 11 Texas cities. In the Northern System, water is raised approximately 720 ft in 4 lifts to a regulating reservoir at Amarillo. Detailed cost comparisons dictated pipeline sizes of 72 and 78 in. Surge and water hammer problems were solved by use of open surge tanks and careful timing of valve and pump operation. Automatic controls at each of the pumping plants were designed to reduce surge problems and coordinate the electric power demand with local power supply. Pipe sizes from 96 to 54 in carry the gravity flow through the Central System. The hydraulic gradient was made parallel to the ground surface, resulting in low head pipe and the most economical pipe sizes. "Pipe check" and "pipe stand" control structures were placed at strategic points along the line, practically eliminating pressure transients and surges. A computer program using elastic water column theory evaluated the design and location of these controls and closely confirmed laboratory test results. For the Southern System below Lubbock, a thin-wall type of pipe with embedded steel cylinder and cement mortar lining and coating was used. Flow is by gravity in a closed system with the static head being reduced in a series of steps by pressure-reducing stations.

DESCRIPTORS-- \*aqueducts/ \*pipelines/ hydraulic structures/ \*pumping plants/ reservoirs/ forebays/ cost comparisons/ operation and maintenance/ \*surges/ surge tanks/ valves/ \*water hammer/ \*automatic control/ air traps/ timing circuits/ standpipes/ electric power demand/ hydraulic gradients/ computer programming/ check structures/ water pressures/ design data/ pumps/ flow control/ laboratory tests

IDENTIFIERS-- Canadian Riv Aqueduct, Tex/ \*hydraulic design/ Canadian River Proj, Tex/ water column separation/ Texas

## HYDRAULIC DESIGN FEATURES OF THE CANADIAN RIVER AQUEDUCT

The Canadian River Aqueduct is a series of pipelines totaling about 320 miles in length. The aqueduct, a principal feature of the Bureau of Reclamation's Canadian River Project in northwestern Texas, is the Bureau's longest pipeline system. It serves the cities of Amarillo, Borger, Pampa, Lubbock, Plainview, Brownfield, Levelland, Slaton, Tahoka, O'Donnell, and Lamesa with water from the Canadian River, which is stored in Lake Meredith, the reservoir impounded by the project's Sanford Dam near Sanford, Texas. Many types of pipe are used, largely of reinforced concrete. The aqueduct also has many types of hydraulic structures, including pumping plants and the large regulating reservoirs at Amarillo and Lubbock.

For the purposes of this paper, the aqueduct is divided into three main divisions, as the basic operations of each are distinctly different from a hydraulic standpoint. These divisions are designated Northern, Central, and Southern Systems, as shown in Figure 1. The Northern part extends from Sanford Dam to Amarillo and includes the East Aqueduct serving Borger and Pampa. The Central part extends from Amarillo to Lubbock, and the Southern System extends south and southwest of Lubbock.

### Northern System

As it is simpler to treat each division separately, I will begin with the Northern System. The main part of this system is the pipeline bringing water from the dam to a regulating reservoir

at Amarillo. To accomplish this, under normal operation, the water must be raised approximately 720 feet. This is done in four lifts by that many pumping plants. For reference, these plants, starting at the dam, are consecutively numbered 1, 2, 3, and 4. See Figure 2. Plants No. 2, 3, and 4 are of equal capacity, 160 cfs. Plant No. 1, which must provide the extra amount required for the East Aqueduct serving Borger and Pampa, has a capacity of 183 cfs.

Each plant, except No. 1, is preceded immediately by forebays that are small concrete-lined reservoirs. Plant No. 1 is located at the dam and draws directly from Lake Meredith. The forebay for Plant No. 2, in addition to supplying East Aqueduct, also serves as afterbay for Plant No. 1. A similar arrangement follows for the other plants with the regulating reservoir at Amarillo serving as afterbay for Plant No. 4. The pipelines between plants vary in length from 1.9 to 14.3 miles with internal diameters 72 and 78 inches. The 72-inch sizes are used in regions of higher heads. These sizes were determined by economic considerations, including first costs, interest on the investment, and pumping costs.

Concerning cost studies, a graphical method introduced by Julian Hinds in the Engineering News-Record, January 28, 1937, and March 25, 1937, was used for the preliminary economic studies. These proved quite accurate. When the layouts of the pipelines were fairly well established, detailed cost comparisons of various pipe sizes were made. These confirmed the preliminary results by graphical methods in every instance.

As studies progressed, it became evident that it would be feasible to make all pump units at Plants No. 2, 3, and 4 of equal size. Since this was considered desirable, especially from maintenance standpoint, the final locations of pumping plants were slightly adjusted to complement this consideration.

The next step was to determine control of operations and most economical ways of dealing with water-hammer and surge problems. Operations of controls greatly influence transient pressures and surges, which in turn can be minimized by judicious use of valves, surge tanks, air chambers, etc. The schematic drawing shown in Figure 3 illustrates the operation of the Northern System. We found this drawing invaluable for coordination of the five branches involved in this design.

The best means of achieving economy in pipe costs usually dictates types of controls, valves, timing, size of pumps, etc. This drawing serves as a working scheme for controls as we work out our pipe design, and it also serves as design data for the other branches involved. Each branch then contributes to developing the final schematic which will provide the basis of the designers' operating criteria.

Now in the type of system shown here, it is apparent that a control valve must be used on the discharge side of each pump to keep the pipes full. These should be automatically controlled to close individually when their attendant pump stops and to open when the pump starts. The timing must be adjustable so as to open

or close at prescribed rates. The time of closure will greatly affect water hammer and this in turn depends on the pump characteristics, in particular the inertia of the pump and motor and the pump's specific speed. In the Northern System, hydraulically operated square-bottom gate valves were selected that open and close at a uniform rate of speed. In some cases it is advantageous to specify variable rates of opening or closing. Sometimes check valves are used where sudden closure is permissible, as in the East Aqueduct and Southwest Aqueduct.

Beyond the control valve, the pipeline must be protected against excessive transient pressures, particularly those that could be brought about by water column separation. This consideration is usually met most positively and economically by simple open surge tanks located at carefully selected points in the line. In some instances, open surge tanks are not feasible because topographic conditions would require an excessively high tank. A one-way surge tank may then be used in conjunction with an open tank. A one-way surge tank provides water so as to prevent water column separation but does not protect against high-pressure transients. Another disadvantage of one-way surge tanks is that they must depend upon the operation of valves; therefore, the valves must be given the very best continuous maintenance to insure their operation at all times. For safety, the Bureau has always provided duplicate valves in parallel. Figure 4 gives a comparison of resulting pressure waves and design gradients using open and one-way surge tanks.

Air chambers often serve effectively where an open surge tank cannot be built economically. These also require careful maintenance, and in addition to valve problems, a supply of compressed air must be maintained.

Surge suppressors and pressure relief or pressure reducing valves are to be used when all else is not feasible. We have always found that these valves respond too slowly to give instantaneous relief and are uneconomical to use when resulting pressure transients are considered in the pipeline. In fact, where possibilities of water column separation exist, and the profile of the pipeline is such that the points of separation cannot be pinpointed, we cannot predict the magnitude of the pressure transients. It must be borne in mind that in long, large diameter pipelines, such as I am now speaking of, a slight increase in design head greatly affects the cost of the pipeline. This is quite different from small lines where even nominal standard pipe usually provides strengths far in excess of design heads and pressure transients often are not critical.

As mentioned before, timing of the controls is most critical and therefore limitations must be imposed on timing. The frequency at which motors are started and stopped affects their life. Sudden changes in power load must be kept within the capabilities of the power system to respond. Our pumps are controlled by float switches in forebays and afterbays which start or stop pumps as water surfaces rise or fall. You can visualize that the capacity of the forebays

controls the timing of operation of pumps; or vice versa, the capacity of the control reservoirs are determined by timing.

The Amarillo Regulating Reservoir, mentioned earlier, is different from the other afterbays. The reservoir was placed at the end of the Northern Branch where the flow of the aqueduct changes from pumped to gravity flow. It is also near a point where the capacity of the aqueduct changes considerably because of deliveries made to Amarillo.

We originally considered this reservoir to have a smaller capacity. However, after considering the scheme that the power company proposed for supplying economical power, the capacity was increased to 750 acre-feet; 250 acre-feet are for emergency use for Amarillo, and the remaining 500 acre-feet are to provide flexibility in the operation of pumps. Some shutdown time of the pumps is necessary because the power company is not required to furnish power during periods of peak load or when generating economies are affected. The power company therefore is allowed to control pump operation. This is done by remote manual operation of Plant No. 2. All other pumps at the other plants are automatically controlled.

The East Aqueduct, a part of the Northern System, takes water from Forebay No. 2 and delivers it to Borger and Pampa. Its operation involves features common to the Main Aqueduct just discussed and features that will be discussed later under the Southern System.



### Central System

Proceeding south along the aqueduct, the Central System of Canadian River Aqueduct will occupy our attention. As shown on the condensed profile, Figure 5, the aqueduct proceeds by gravity flow to Lubbock where another regulating reservoir receives the flow. This stretch is 115 miles long. Only one small delivery is made before reaching Lubbock, and that is at Plainview. The maximum capacity of the Central System is 92 cfs.

As you can see, pipe sizes range from 96 to 54 inches with the predominant size 66 inches. The hydraulic gradient has been made parallel to the general ground surface, which results in low head pipe and the most economical pipe sizes. You will notice also that the design gradient is the same as the hydraulic gradient, which means that controls are such as to practically eliminate pressure transients and surges. This is accomplished by controlling the flow at the upstream end. Such a system of controls would present no problems if the flow were always steady, but it must be remembered that the entire aqueduct must operate on a demand basis. The demand is not as immediate as in a city distribution system; nevertheless, it must meet requirements.

For example let us consider two extremes of meeting demand: If the flow in the pipeline from Amarillo to Lubbock were controlled at the upper end and the pipe allowed to drain whenever it was shut down, it would take many days to refill the line and bring it up to

full flow. On the other hand, if control of the line is at the lower end, at Lubbock, flow could be started immediately, but the design head for the pipe would be the static head established by the water elevation in Amarillo Reservoir.

Obviously, neither of the above schemes are the solution to the problem. The first one is too slow and the second one is too expensive. The increased design head would have cost the project some 2-1/2 million dollars extra.

The solution used is to control the flow at the upper end but to make provisions to keep the pipe full at all times. To do this, a series of structures were placed at strategic points along the line so as to establish a series of pools, as shown in Figure 5. These structures are not all the same because, as we shall show later, some had to perform other functions as well as establish a pool elevation.

The simplest structure is the one shown in Figure 6. We have called it a pipe check. This structure gives the smallest hydraulic losses, and is least expensive to construct. However, there is practically no room for storage of water, which would limit the use of the structure.

Combined with the pipe check, we used the structure shown in Figure 7. For convenience, we designate this a pipe stand. It also holds the water upstream in the line at a certain pool elevation, but in addition it has storage capacity. The size of the tanks was varied as required by hydraulic considerations.

At all the pipe check and pipe stand structures along the line, a vent was placed immediately downstream as shown. It is imperative that most of the air be removed from the system to prevent accumulation of air pockets; otherwise an analysis to determine surges would be nearly impossible.

It may appear that we are talking about the most simple type water conveyance system, or simply, "turn the water on whenever you want it and let it run." Many years ago, however, the Bureau had an unfortunate experience with this very type of system for an irrigation project. A series of check structures were used, and it was found that under certain operating conditions, the surges at the various checks came into resonance resulting in surges that literally blew the lid off structures and discharged water high in the air. This phenomenon is difficult to explain in a short discussion, but if one runs through a simple graphical solution of the surge problem, it becomes clear.

The problem resolves itself basically into a solution of Newton's law; force is equal to mass times acceleration. In an individual reach of pipe, the accelerating force is established by the difference in heads at the structures at each end of the reach. You can visualize, however, that when several such reaches are placed in series, the surges that are taking place in one affect those in another and the problem becomes compounded.

Figure 8 shows a graphical solution for a hypothetical problem consisting of three reaches. The solution is based on rigid water

column theory, which is accurate enough when surging alone is being considered. Surges at the three structures are plotted against flows, but time is also involved. You will notice that surging at the first structure is relatively mild or the water rises in this structure at a fairly uniform rate. However, at points farther down the line, surging progressively increases and becomes unmanageable at Pipe Check No. 4. A more detailed drawing of this particular solution is shown in Figure 8A. This demonstrates the complexity of a graphical solution for a rather simple system.

Our first study of the Central System surging was done graphically. There are 21 reaches, and needless to say, it was very difficult and extremely time consuming. It did show that we had problems even in the one and only case considered, and that there was a definite need to study many other conditions that may be expected to arise in ordinary operation of the project. Therefore, since considerable money was involved, it was decided to develop a computer program for surge studies. It was also decided that some experimental verification of the program should be attempted. All this was carried out, but it became evident that in order to check experimental results, considerable refinement would be required in the computer program. Therefore, a very precise program was needed, using elastic water column theory. We decided also to develop a very flexible program.

In Volume 218, Part 1, 1963, of the Transactions of the ASCE, a paper was published entitled "Water Hammer Analysis Including

Fluid Friction" by Victor L. Streeter and Clinton Lai. This paper outlines a theoretical analysis of water hammer with the inclusion of hydraulic friction losses by a set of nonlinear, partial differential equations. These equations are transformed into characteristic equations that are then organized into difference equations, suitable for high-speed digital computers of fairly large storage capacity.

The program has been written for the Bureau's H-800 Honeywell computer (32,000 words), and laboratory tests have verified the theoretical analysis. Figure 9 shows the laboratory setup, and Figure 10 shows a comparison of test results with theoretical results. Surging is plotted against time for the last two structures in the laboratory setup.

Our studies show, that by using some pipe stand structures interspersed with pipe check structures and a prescribed rate of operation of the control valve at the upper end, the overall operation of the Central System becomes virtually problem free.

The pipe stand structures, by virtue of their storage capacity, have the effect of placing small reservoirs at various points along the pipeline, which tend to break the system up into many smaller systems. Now one could proceed along this line and arrive at a solution to the problem by using large pipe stand structures or even regulating reservoirs spaced periodically with pipe check structures between, and analyze the separate reaches individually by graphical methods. However, we were able to keep the size of

structures down and obtain considerable savings in the cost of this system of the aqueduct.

There probably are an infinite number of modifications of the Central System that could be used, but this is most economical. The pipe is minimum size, minimum head, and no valves whatever are required below the control valve near Amarillo. It takes several hours to increase flow from zero to full capacity, but this is rapid enough considering the large regulating reservoir at Lubbock.

Southern System

There were many factors that influenced selection of type of hydraulic system used south of Lubbock. Nearly all considerations of economy and operation lead to the same conclusion; namely, that this should be a pressure system.

You may know that the City of Lubbock has entered into agreement with all the other cities served by the Canadian River Aqueduct south of Lubbock to treat their water. Each city will probably wish to take their delivered water almost directly into the mains. Therefore, the entire Southern System will function best as a demand system, with each city operating independently of the demands of the others. Any other type of system would be very difficult to operate.

Pipe sizes were to be small as compared with the Northern and Central Systems; furthermore, we were persuaded that a thin wall type of pipe with embedded steel cylinder and cement mortar lining and coating would be economical in the area. This was because coarse aggregate for thick wall pipe is not available in the near

vicinity. Minimum designs for this type of pipe are good for rather high heads, and therefore design pressures would not be as critical as in the other two systems. This factor made it economically feasible to use some form of pressure system which would satisfy most requirements.

On Figure 1 you will notice that the main aqueduct proceeds due south and a branch called the Southwest Aqueduct takes off just north of Lubbock. The main aqueduct flows by gravity, whereas the first part of the Southwest Aqueduct requires pumping. Four pumping plants are required all together with open surge tanks and forebays. The forebays are concrete tanks unlike the open concrete-lined reservoirs on the Northern System, but aside from such differences, the two pumping systems are very much alike. Therefore, I will not talk any more about the Southwest Aqueduct, but will devote the remainder of my discussion to the Main Aqueduct.

As mentioned before, flow is by gravity in a closed system, but the full static head is not imposed on the entire pipeline. The static head is reduced in a series of steps by means of structures that essentially amount to pressure-reducing stations, as shown in Figure 11.

The locations of the pressure-reducing stations were first determined by turnouts. For hydraulic reasons regarding surges and pressure transients, it is desirable to have a regulating tank at all such points. Fortunately, placing regulating tanks at these points could be used to advantage for pressure regulation, and the

spacing was good as regards the resulting static heads on the various reaches of pipe. That is, the maximum heads in all the various reaches of pipe were fairly equal. Furthermore, each reach was in the head range where the nominal strength of pipe would generally not be exceeded.

I believe a little more detail about the pressure-reducing stations would be of interest. Figure 12 shows the controls at regulating Tank No. 6, the one at the turnout for the South Lubbock Lateral. The three valves shown are installed in parallel but act in series; that is, they open and close at different times, but one, two, or all three may be open at the same time. Each valve is controlled by a float in the regulating tank. As shown, one 16-inch valve begins to open when the water surface drops below elevation 3200.6. As the water continues to drop, the valve opens wider and becomes fully open after the water surface has dropped 1 foot. The next 16-inch valve begins to open at this point, and the process continues, if the water surface continues to drop, until all three valves are open.

The valves open or close through a limited range of water level variation in the tank. Each valve will remain at a constant position if the water surface in the tank remains constant. Therefore, if the demand downstream from the regulating tank remains constant, the valves will adjust themselves to maintain an equal amount of steady flow into the tank. This valve feature is described as a modulating feature. The three valves, two 16-inch and one 20-inch, are the



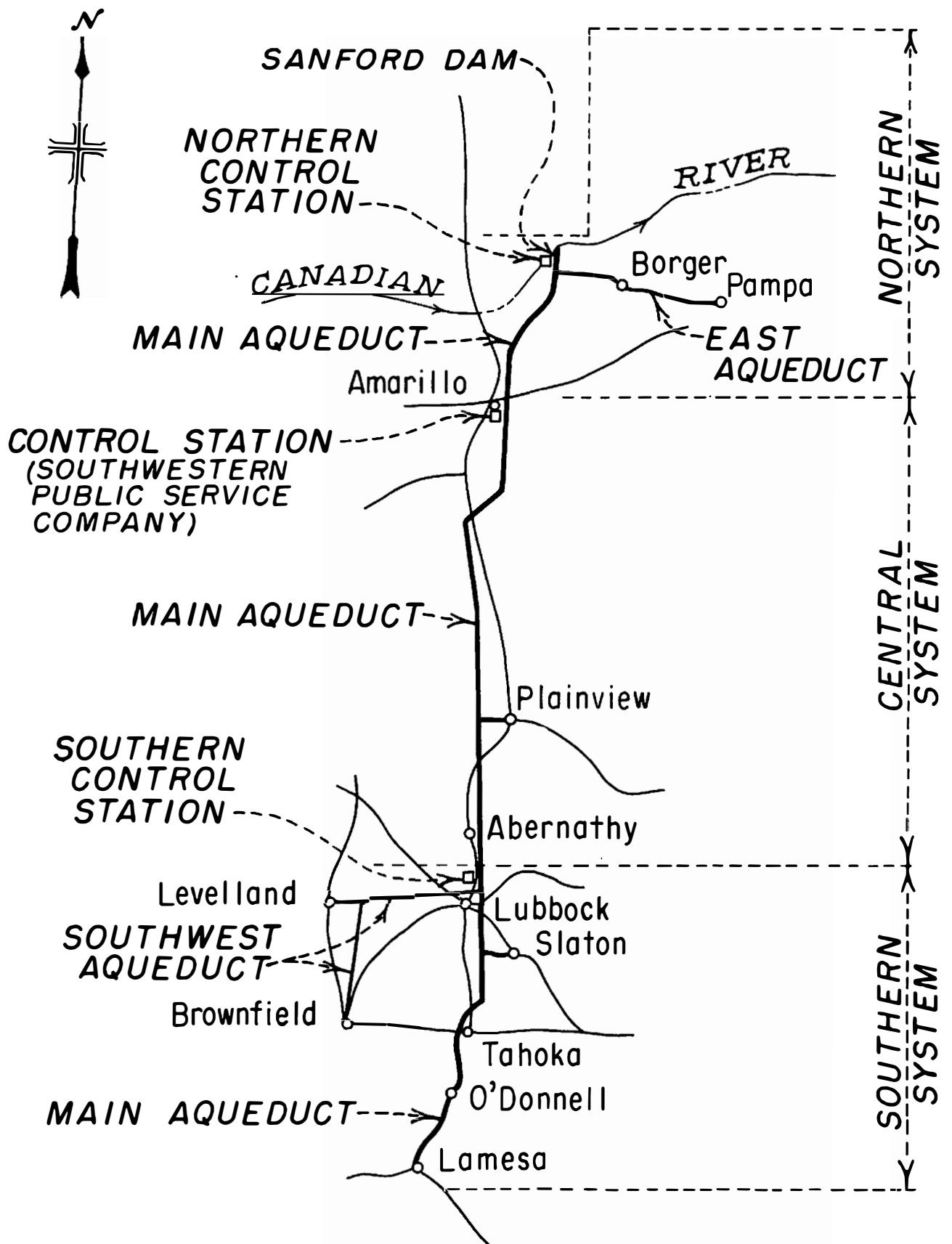
main controls, and each is equipped with a rate-of-flow controller which is set to limit the maximum flow so that the total flow from the three valves is limited to a predetermined amount. This is to insure that no part of the system may take more than their allotted share of water and possibly rob another part.

The valves themselves are hydraulically operated, pneumatically controlled butterfly valves. In some cases, where the difference in head across the valves is great and cavitation may result when the valves are operating in a nearly closed position, air is introduced through the valve on the downstream side. This has proven very effective in reducing cavitation. There are different schemes for introducing the air; sometimes through the valve disc and sometimes through the body of the valve around the perimeter just downstream from the disc-seating area. The latter method seems to be most effective, especially for smaller valves.

You will notice a fourth large 30-inch butterfly valve just upstream from the three controlling modulating valves. This large valve is fully opened or fully closed by float switches in surge Tank No. 10 upstream. When the water at the surge tank drops to a predetermined low, the valve closes so as to prevent draining any part of the pipeline upstream. The other valves shown are merely shutoff valves to be used for servicing the control valves.

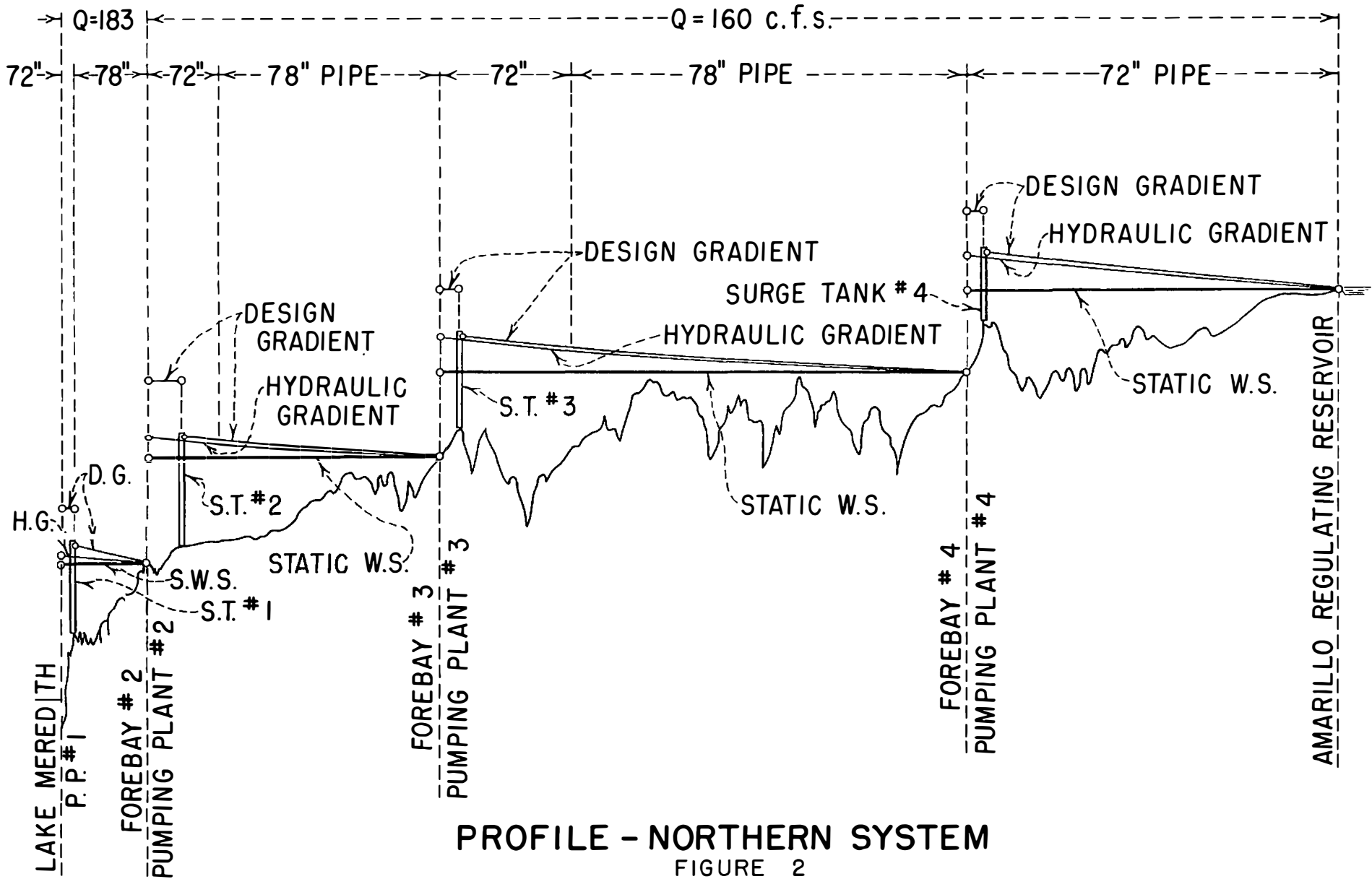
At each delivery point along the main aqueduct, a pressure-reducing station has been set up featuring the type of control just described.

This briefly covers the hydraulic features of the Canadian River Aqueduct. I am sure you would find it interesting to visit the project, now under construction, to see completed parts and view the work in progress. You will find personnel of the Bureau of Reclamation very cooperative and eager to help you.



# MAP OF MAJOR SYSTEMS

FIGURE I



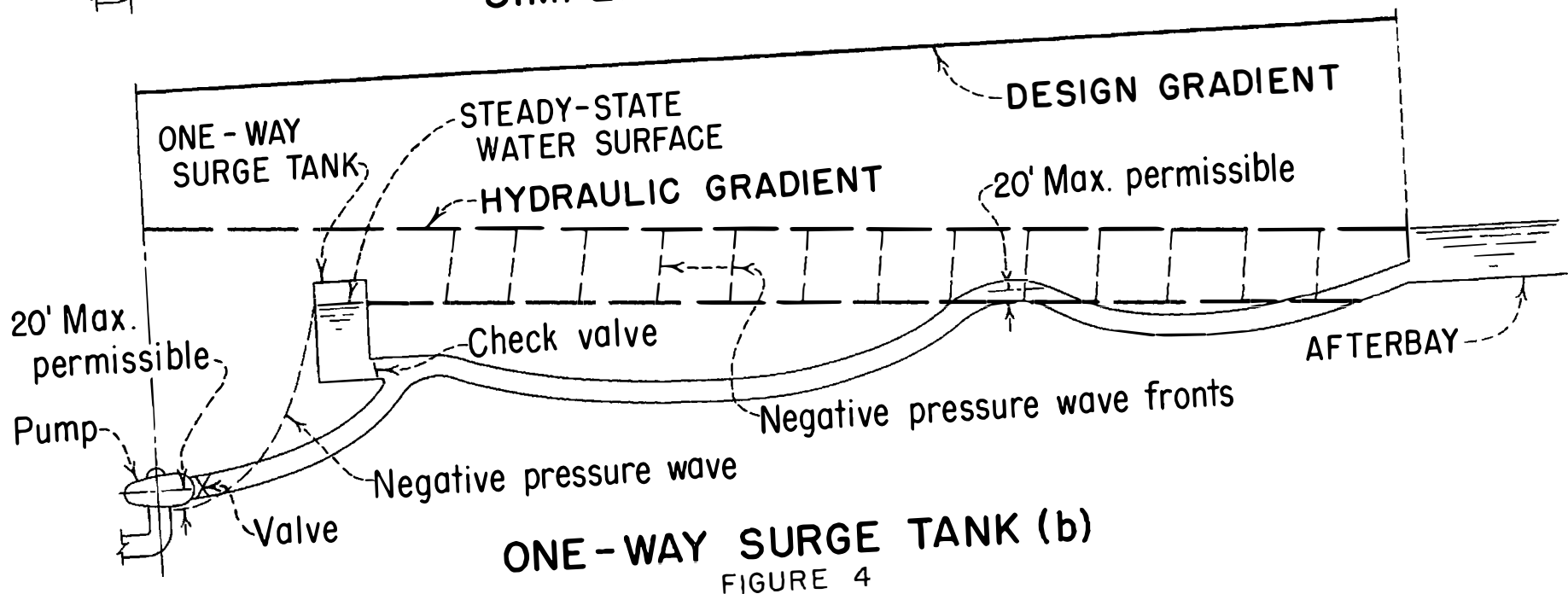
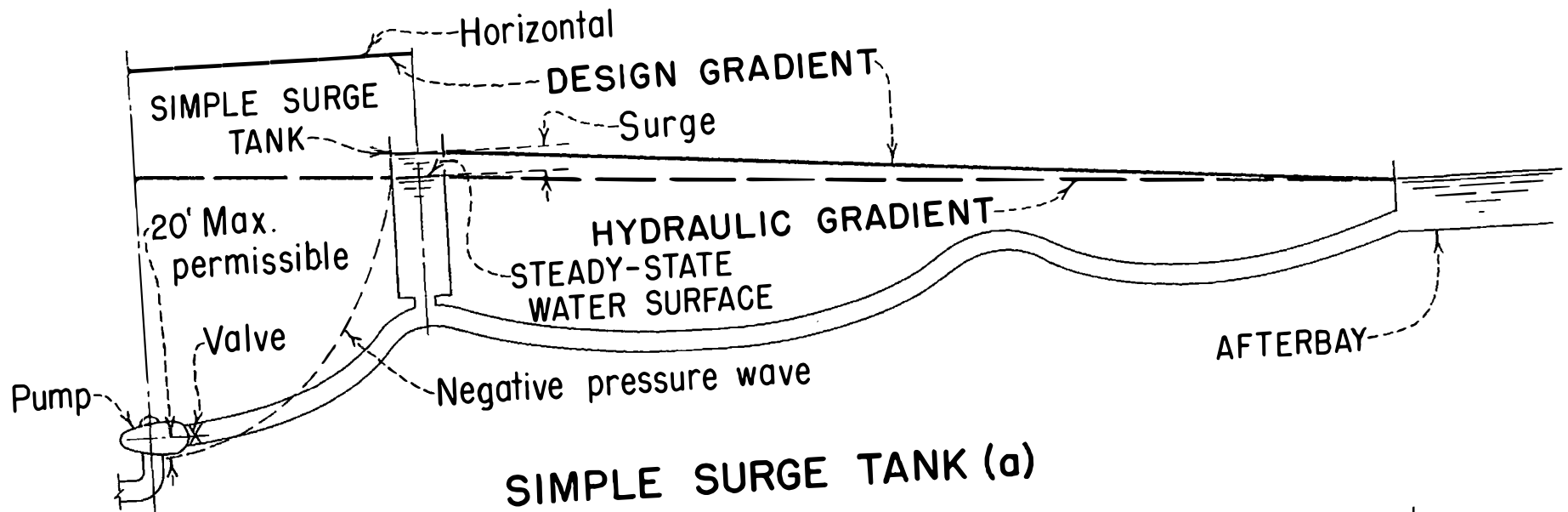
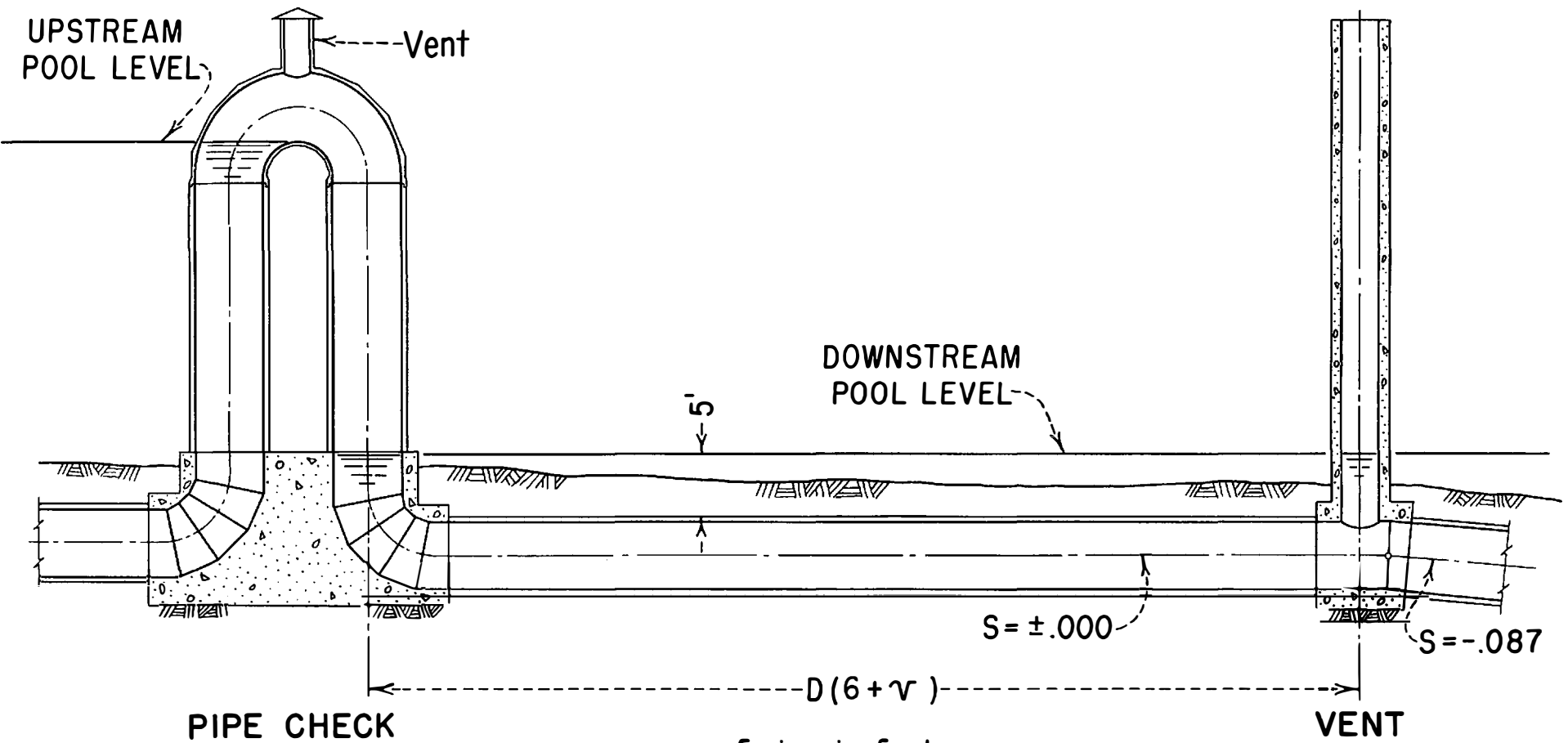


FIGURE 4

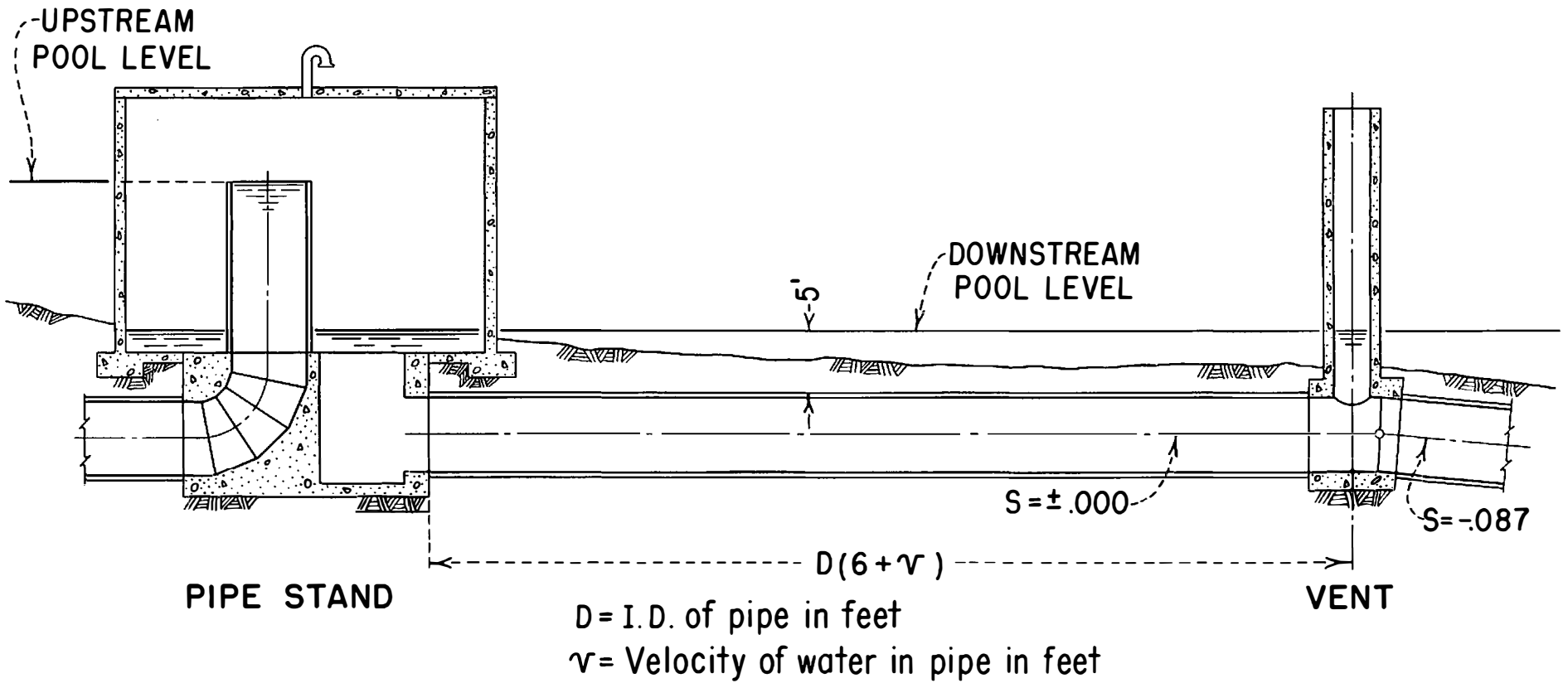




$D$  = I.D. of pipe in feet  
 $v$  = Velocity of water in pipe in feet

## PIPE CHECK AND VENT

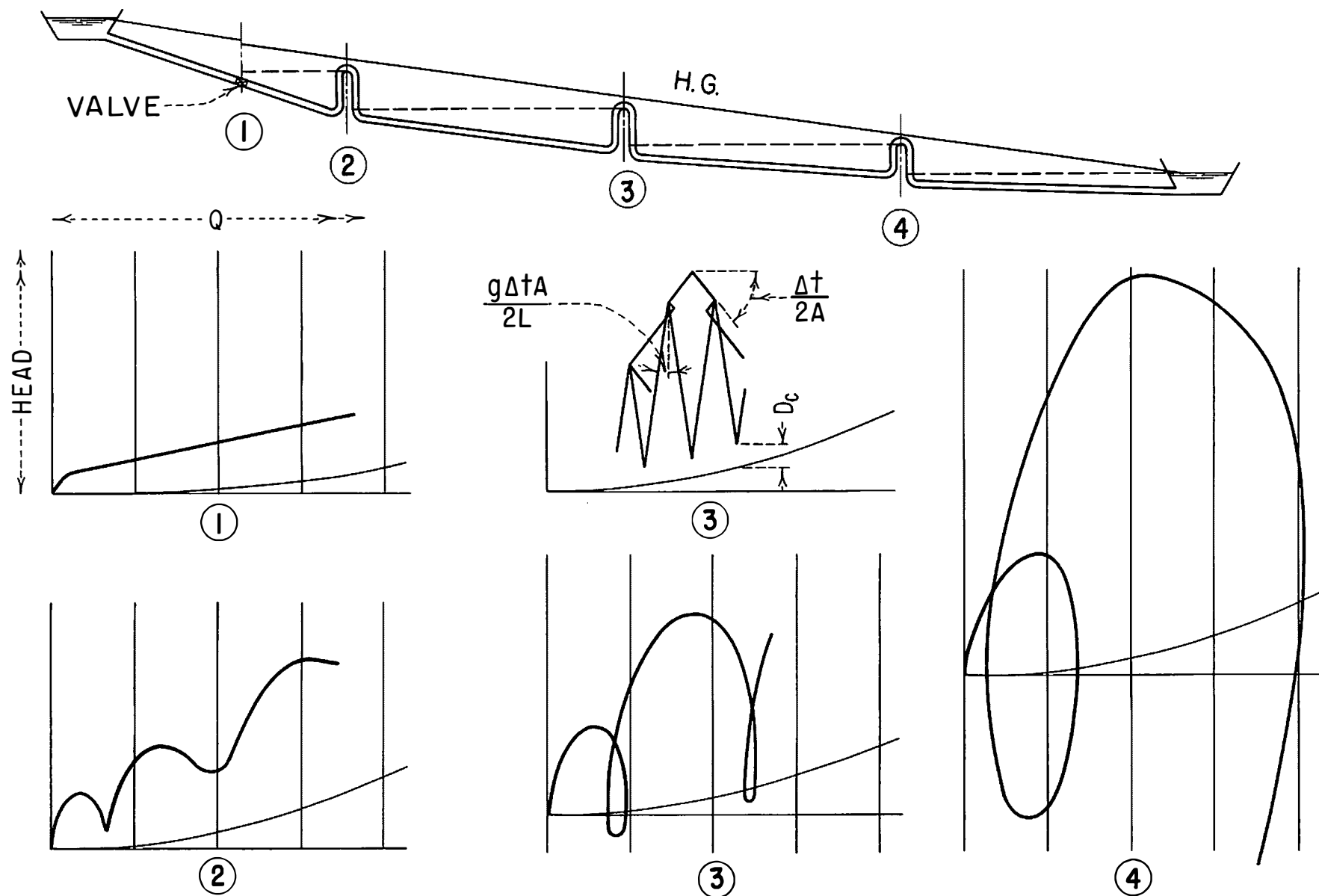
FIGURE 6



## PIPE STAND AND VENT

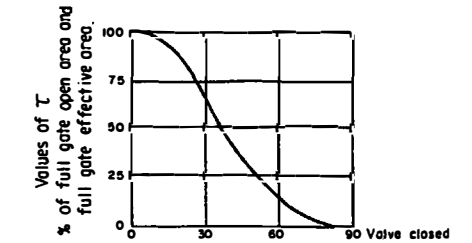
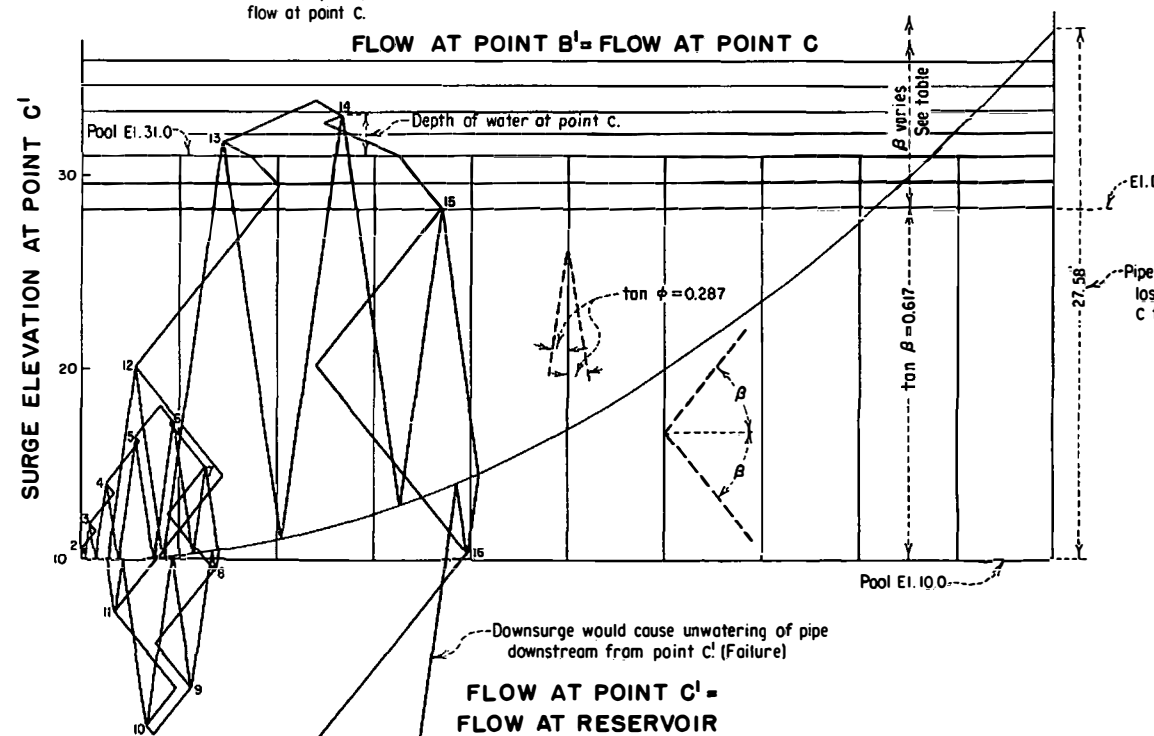
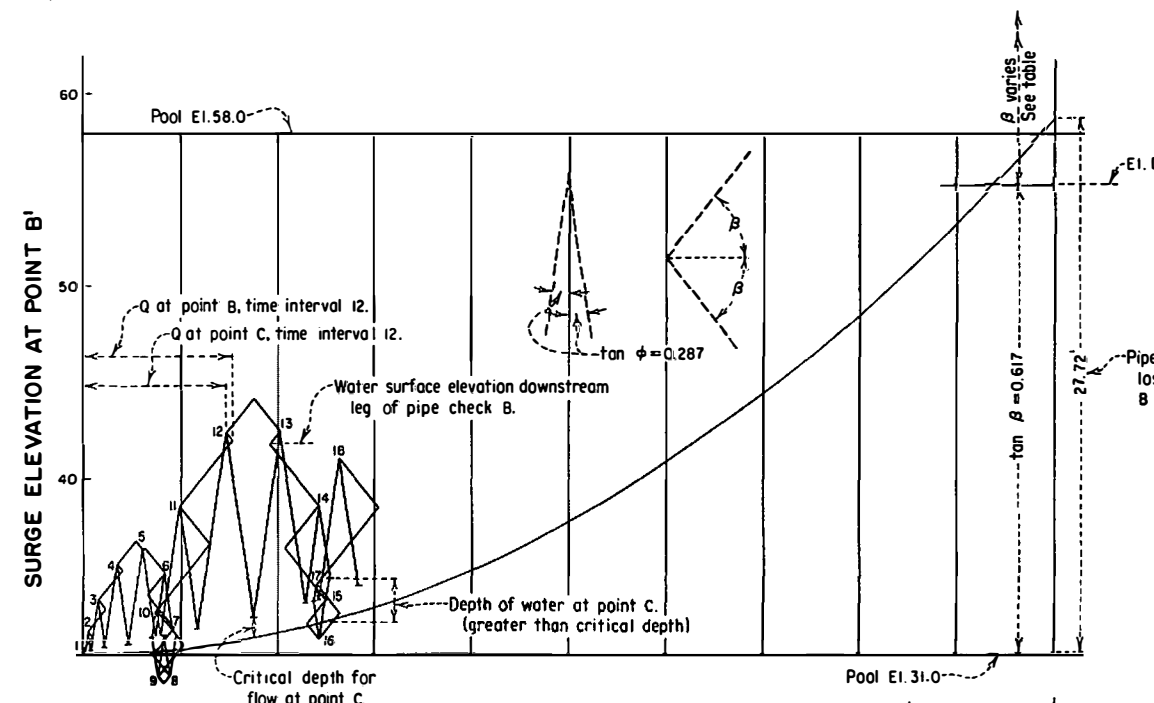
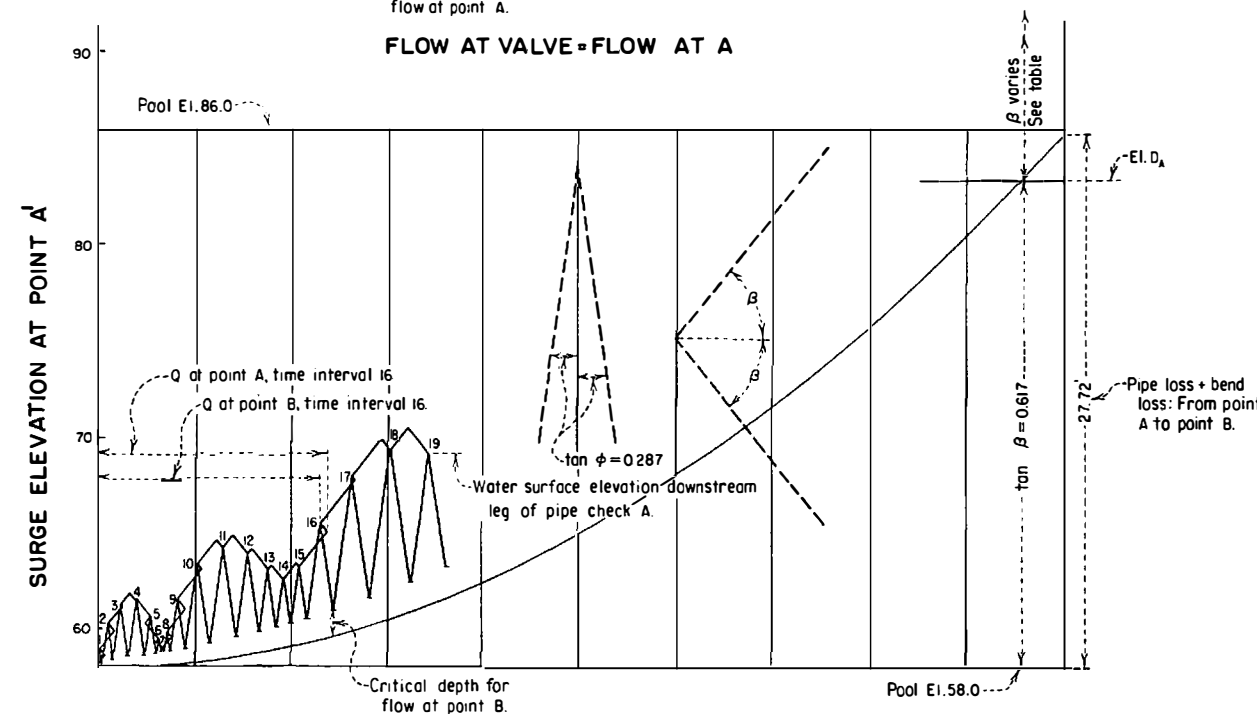
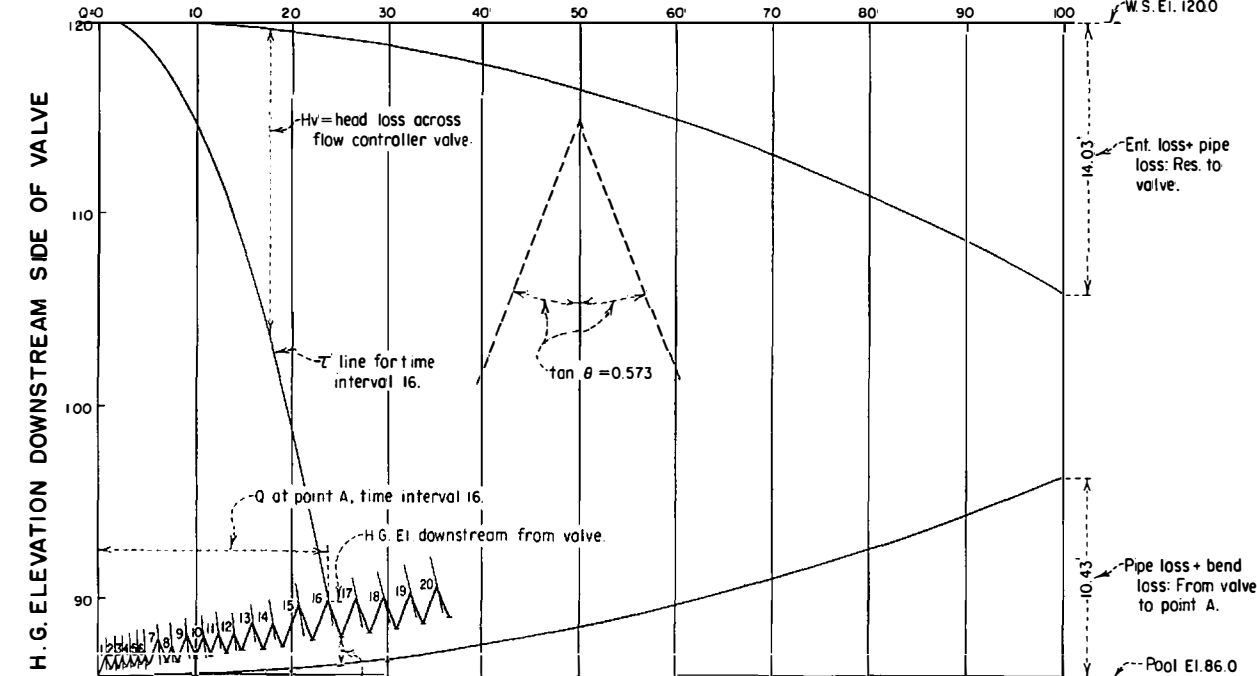
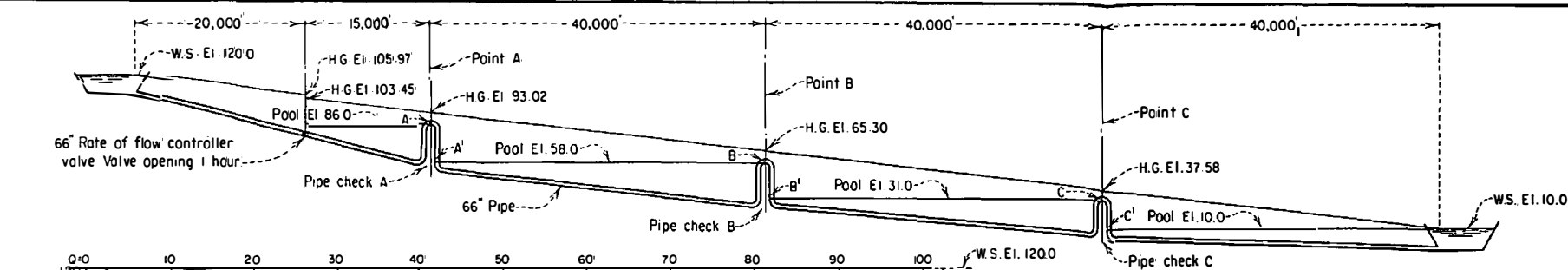
FIGURE 7





# GRAPHICAL SURGE ANALYSIS

FIGURE 8



FLOW CHARACTERISTIC CHART FOR BUTTERFLY VALVE

#### HYDRAULIC PROPERTIES OF PIPE LINE

Q = 100 c. f. s.  
A = 23.76 sq. ft.  
V = 4.21 ft. / sec.  
S = .000686

Values of  $\tau$  obtained from flow characteristic chart for butterfly valve.

$$H_v = \frac{Q^2}{2g\tau^2 A_o^2}$$

Q = Quantity of water  
g = 32.16 ft. / sec.  
A\_o = Area of fully opened valve

$$\tan \theta = g\Delta t A / 2L$$

$\Delta t$  = 30 sec. interval assumed  
A = Area of pipe  
L = Length of pipe

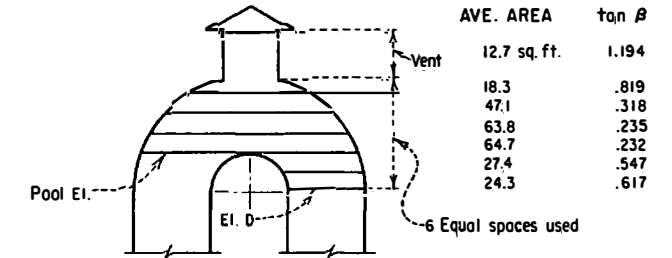
$$= 32.16 \times 30 \times 23.76 / 2 \times 20000 = .573$$

$$\tan \phi = g\Delta t A / 2L = 32.16 \times 30 \times 23.76 / 2 \times 40000 = .287$$

$$\tan \beta = \Delta t / 2A_i \quad A_i = \text{Area of vertical pipe in check structure}$$

$$= 30 / 2 \times 23.76 = .631$$

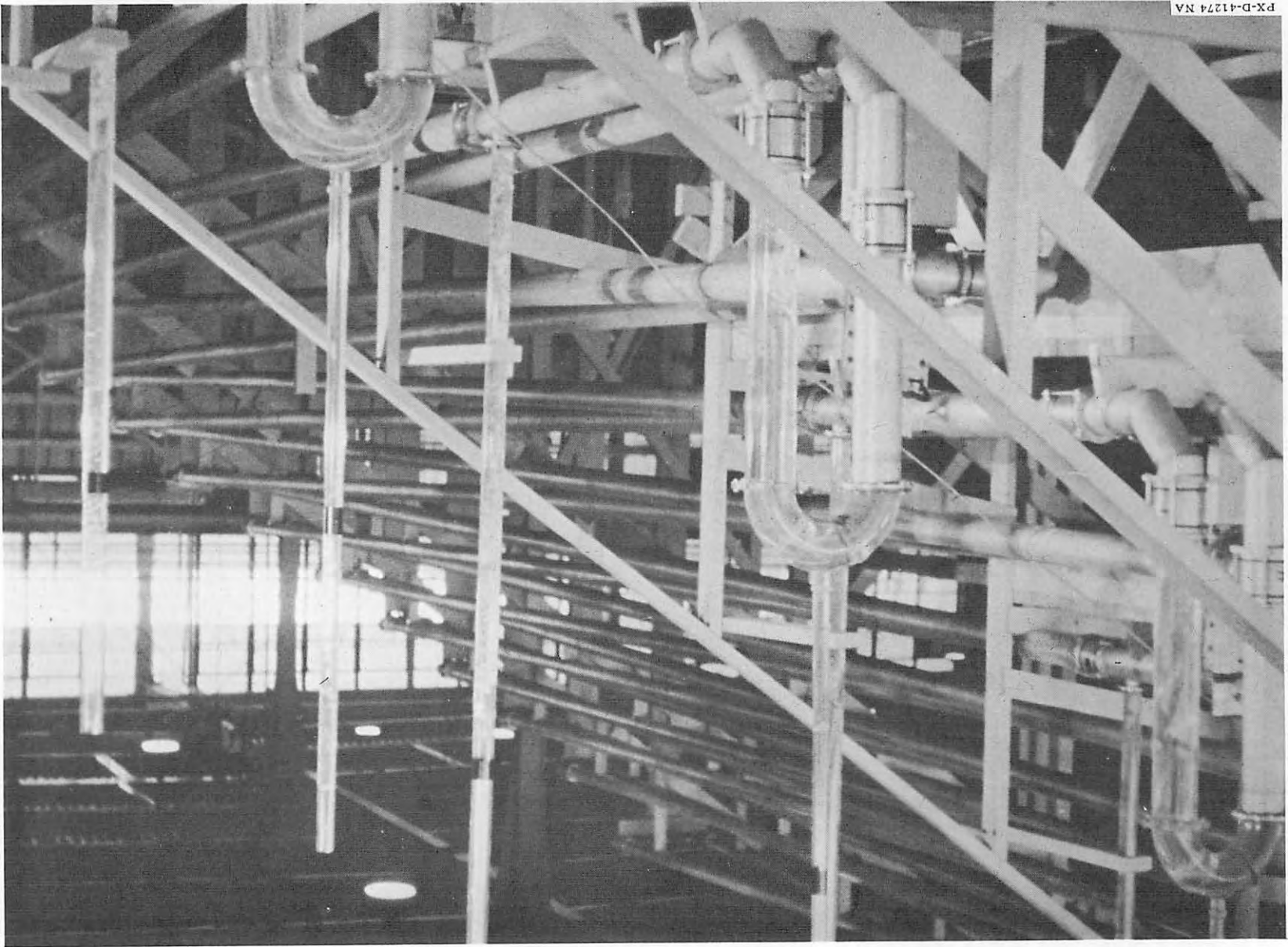
$\beta$  above El. D

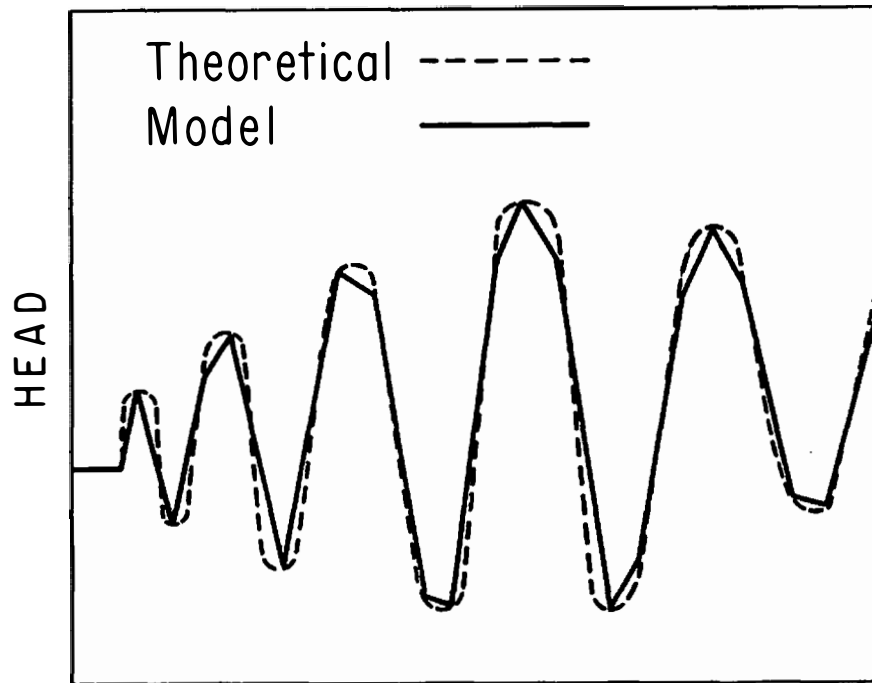


GRAPHICAL SURGE ANALYSIS

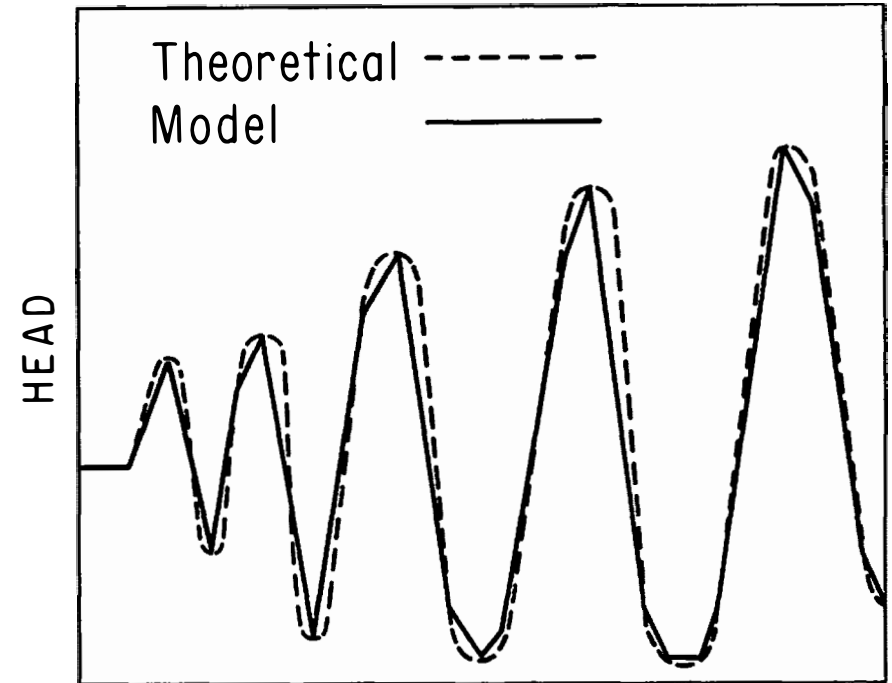
FIGURE 8A

FIGURE 9





TIME  
CHECK NO. 5

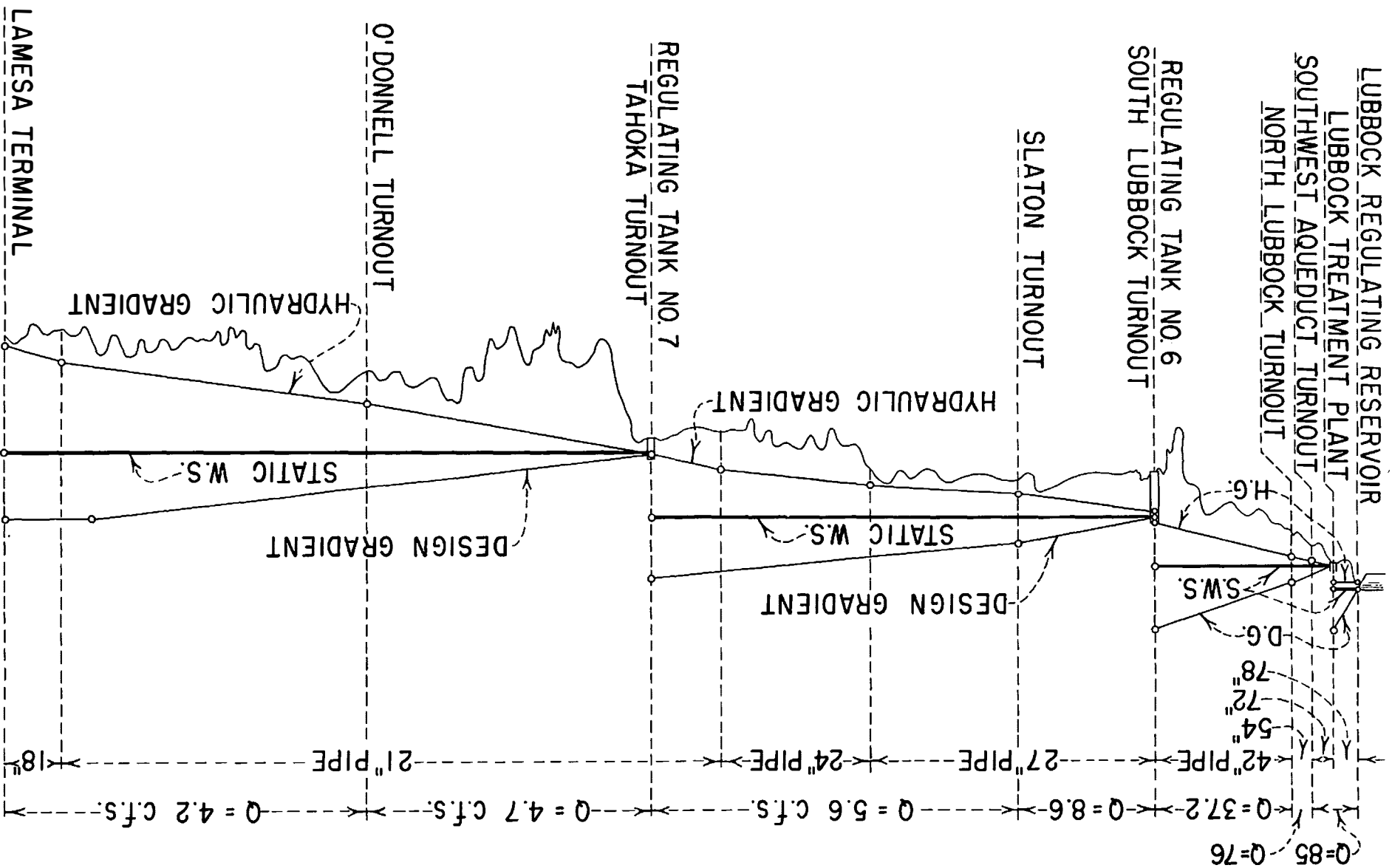


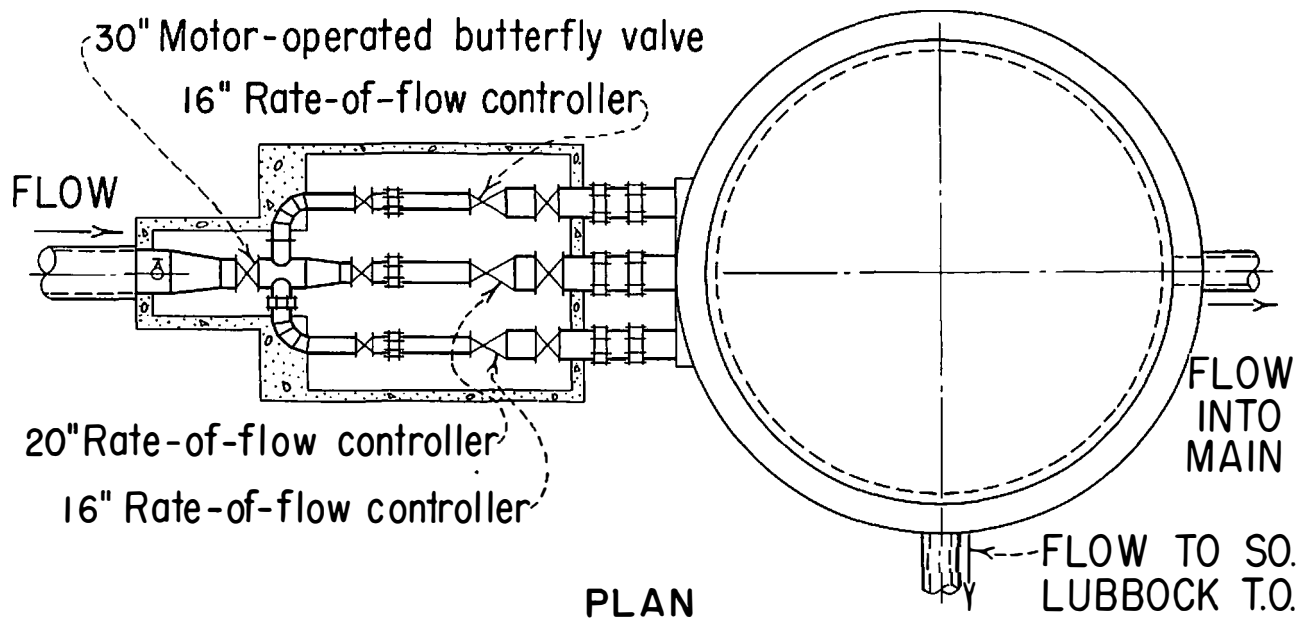
TIME  
CHECK NO. 6

## COMPARISON OF THEORETICAL AND MODEL RESULTS

FIGURE 10

PROFILE - SOUTHERN SYSTEM  
FIGURE II



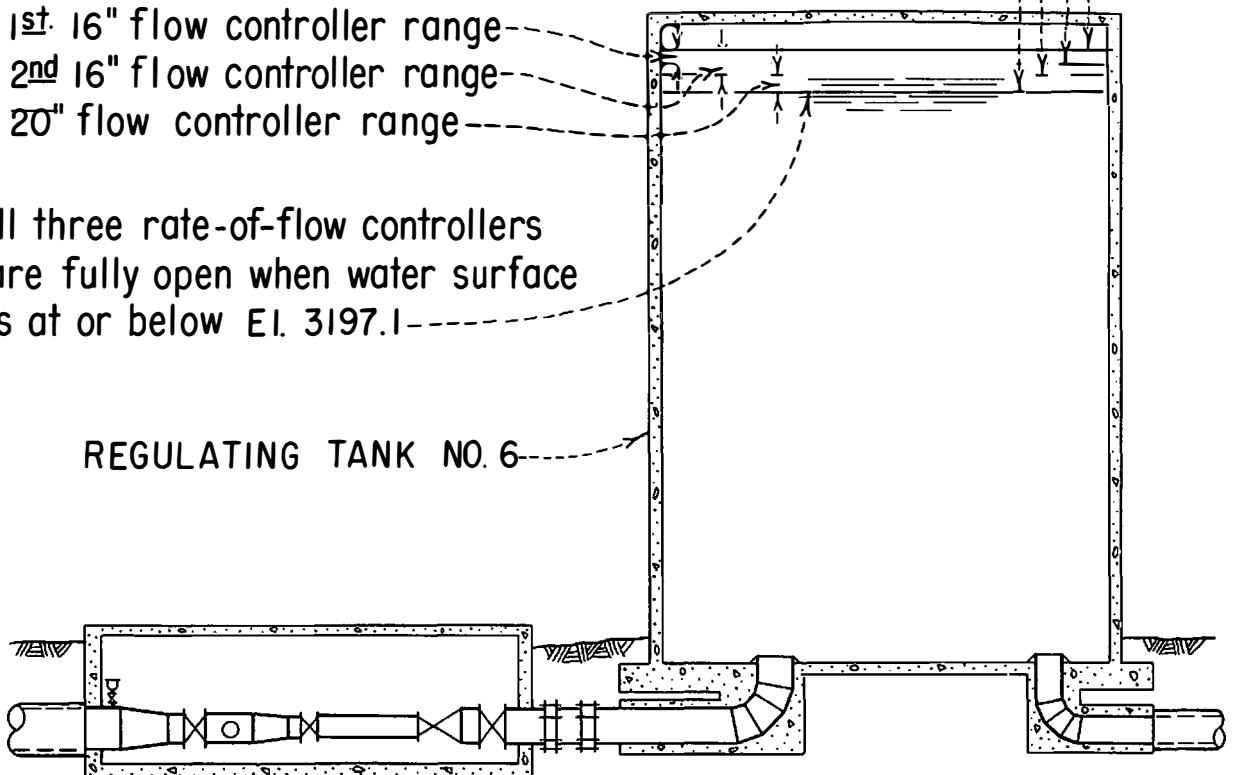


### FLOAT MODULATION RANGES

- 1<sup>st</sup> 16" flow controller range
- 2<sup>nd</sup> 16" flow controller range
- 20" flow controller range

All three rate-of-flow controllers are fully open when water surface is at or below El. 3197.1

El. 3200.6  
El. 3199.6  
El. 3198.6  
El. 3197.1



### ELEVATION PRESSURE REDUCING STATION AT REGULATING TANK NO. 6

FIGURE 12



