NATIONAL ENGINEERING HANDBOOK

SECTION 16

DRAINAGE OF AGRICULTURAL LAND

CHAPTER 9. DRAINAGE OF TIDAL LANDS

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NATIONAL ENGINEERING HANDBOOK

SECTION 16

DRAINAGE OF AGRICULTURAL LAND

CHAPTER 9. DRAINAGE OF TIDAL LANDS

General

This chapter discusses the phenomenon of tides and their effect on agricultural land, and it suggests remedial measures to protect these lands. It presents a procedure for the design of gravity operated tide gates, giving consideration to tidal cycles and fluctuations, anticipated interior drainage flows, and the interior storage available. A more detailed and precise procedure for the design of tide gates is given in Technical Release No. 1, Engineering Division, SCS (1); however, the simpler procedure outlined in this handbook is considered adequate for small to medium sized drainage projects planned for the protection of agricultural lands. The charts, tables and the example presented in this chapter are based on the use of circular-metal tide gates attached to corrugated-metal pipe, as these are commonly used structural materials. For protection against corrosion from salt water, it is necessary for the corrugated-metal pipe to be asbestos bonded and asphalt coated. Design data and criteria, for other types of gates and pipe, are available in handbooks prepared by a number of commercial concerns. It is presumed that pumps, where suggested as supplemental facilities in lieu of available storage, will be selected on the basis of manufacturer's specifications.

Agricultural lands in coastal areas, located along rivers, estuaries, bays and the open sea, are subjected in varying degrees to overflow and restricted drainage caused by tidal waters. These areas may range from a few acres to thousands of acres. The extent and frequency of overflow and drainage impairment may vary widely, depending on the elevation and exposure of the sites to open tidal water. Protection from overflow usually is obtained by the enclosure of such areas with dikes. Drainage may be obtained by establishing a system of internal drains, with water discharged over the dikes by pumps, by gravity flow through gated structures, or by a combination of pumps and gated structures.

Pumps are necessary when: (a) storage for accumulating drainage water within ponding areas, ditches, and the soil profile is not available; (b) when flow through the gates is restricted over long periods by wind tides, flood flow, or inadequate outlets into open tidal waters; or (c) when construction and maintenance of foreshore channels is impractical.

A gravity-outlet ditch with a gated structure through a protecting dike, provides suitable means for removing drainage waters from most tidal areas. A gated structure consists of a box or pipe culvert through the dike with a gate placed at the tidewater end of the structure. These gates may be circular, square, or rectangular in shape. Usually they are made of cast iron and provided with single or double acting hinges at the top. Well-made gates, when properly installed, are so finely balanced that they automatically open to outflow or close against backflow at slight differences in head. Large drainage areas may require several gated structures or a battery of culverts and gates incorporated in one structure.

Action of tides

The action of tides is a complex phenomenon, some general knowledge of which is essential to the planning and construction of any works affected by them. Only a few essential facts will be discussed herein. More detailed information is available in publications of the United States Coast and Geodetic Survey, including its annual tide tables for both coasts of North and South America; and the U.S. Army Engineers Waterways Experiment Station. See references (2) and (3).

Tidal terms and definitions

The tide is the regular periodic rise and fall of the surface of the oceans. Concurrent horizontal movements of surface waters as the result of tide-producing forces, occurring either as drifts in the open ocean or as flow through entrances to tidal basins and in tidal streams are known as tidal currents. The component of the tide produced by harmonic action of tide-producing forces is referred to as the equilibrium tide. The irregular fluctuations of the tide caused by winds and variations in barometric pressure over water surfaces are referred to as the meteorologic tides.

The path of a point at a particular station which traces the water surface elevation against time through a lunar or tidal period is the tidal curve for that station. The maximum elevation or crest reached by the rising limb of the tidal cycle (flood tide) is called the high water. The maximum depression or trough reached by the falling limb (ebb tide) is called the low water. The average height of all low waters at a station over a period of time is the mean low water. Similarly, the average height of all high water at a station over a period of time is the mean high water. The difference in height between high water and low water is the tidal range. The mean tidal range is the average of differences between all high waters and all low waters. The extreme range is the maximum that has been observed.

Because of variation in density of ocean water with changes in temperature, salinity and barometric pressure, and because of differences in wind and rain from place to place, mean sea level at different tidal stations may not be on the same geodetic level surface. Mean sea level, thus, is an actual mean of sea levels determined from a long series of tidal observations taken over a number of selected points. The plane of zero reference for tidal data in the United States is established by the Coast and Geodetic Survey from which local points of reference, called datum, are established. For the Atlantic and Gulf coast the datum is mean low water. For the Pacific coast (including Hawaii and Alaska) the datum is the mean of the lower of the two daily low waters. Local points of reference for several Atlantic and Gulf coast points as given in tide tables published by the U.S. Coast and Geodetic Survey are: -4.9 at Boston, -2.3 at New York, -3.1 at Philadelphia, -0.6 at Baltimore, -1.3 at Hampton Roads, -2.7 at Charleston, -1.3 at Miami, and -0.8 at Mobile, Alabama and Galveston, Texas. Local points of reference for several Pacific coastal points are: -6.6 at Seattle, -3.0 at San Francisco, -2.8 at Los Angeles, and -2.9 at San Diego.

When using tide tables, the relation of the datum on which the tables are based to the datum on which topographic maps and bench marks being used are based, should be considered. For specific situations, tidal records of the nearest Coast and Geodetic Survey or Corps of Engineers gaging station should be consulted.

Tides may be diurnal, semi-diurnal, or mixed. Tides with but one high and low each lunar day are diurnal. Diurnal tides occur over the greater part of each month along coastal areas of the Gulf of Mexico. Semi-diurnal tides have two nearly equal high waters and two nearly equal low waters each lunar day. Such tides are common to most coastal waters of the world and are the type occurring along the Atlantic and Pacific coasts. Since the differences between the two high waters and between the two low waters on the Atlantic coast are relatively small (generally less than one foot), no distinction is made between daily high waters or between daily low waters. Mixed tides are those which have two quite unequal high waters, two unequal low waters, or both during the course of a lunar day. Mixed tides occur on both the Pacific coast of the United States and the Atlantic coast in Europe.

The highs of high waters and lows of low waters also vary from day to day during the course of a lunar month and during the course of a solar year. The highest high waters and lowest low waters during the course of a lunar cycle occur shortly after full and new moons and are known as springs. Lowest high waters and highest low waters occur shortly after the first quarters and third quarters of the moon and are known as neaps. Springs and neaps change progressively from month to month, usually being highest near the spring and autumn equinoxes and lowest near the summer and winter solstices. Along the Atlantic seaboard, the swings produced by such springs and neaps are not sufficiently large to warrant their special consideration in determining the normal tidal ranges necessary in design of drainage outlets. However, when extremes occur concurrently with storm tides, their effects are significant in considering extreme tides for dike heights.

Tidal phenomenon

Tides are the result of a number of complex forces acting upon the earth's mass. The dominant component of these forces is the equilibrium tide, which develops from the gravitational attraction between earth, moon, and sun; and the constant harmonic changes in such pulls over the surfaces of the oceans and seas as the result of the rotation of the moon in its orbit about the earth and the spinning earth in its orbit about the sun. The range of the equilibrium tides differs from place to place. Along the coastlines of the United States, such variations range from a fraction of a foot in Gulf coast waters to 40 feet in the Bay of Fundy and Alaska.

The changing effect of lunar and solar pulls during the course of their travel cancel out or augment each other progressively during the months and year. This results in the gradual tidal swings which may be as much as 2 to 10 feet in some part of the earth's waters. Such swings are comparatively small along tidal waters of the Atlantic coast. Figures 9-1 and 9-2 illustrate some characteristic tides.

Tide curves

The theoretical oscillations of tides in the open ocean and coastal waters conform to the mathematical cosine curve. See Figure 9-3 for theoretical curves for various ranges of tide. Bays and sounds affect such undulations primarily by delaying the times of high and low water occurrence. Size, shape and alignment of coastal indentations may also decrease the net tidal swings. River estuaries affect the tide in varying degree, depending upon the size of the river, its hydraulic gradient and stage of river discharge. Their most characteristic effect on the tidal curve is to prolong the ebb tide. At low stages, water may flow up rivers in conformance to the open

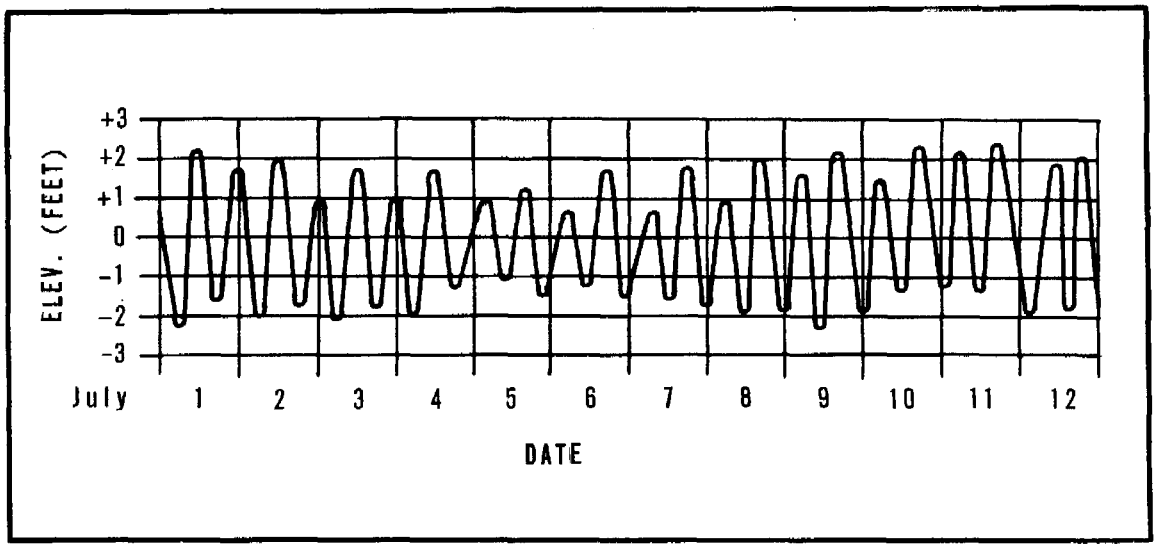


Figure 9-1, Typical ocean tide on open coastline

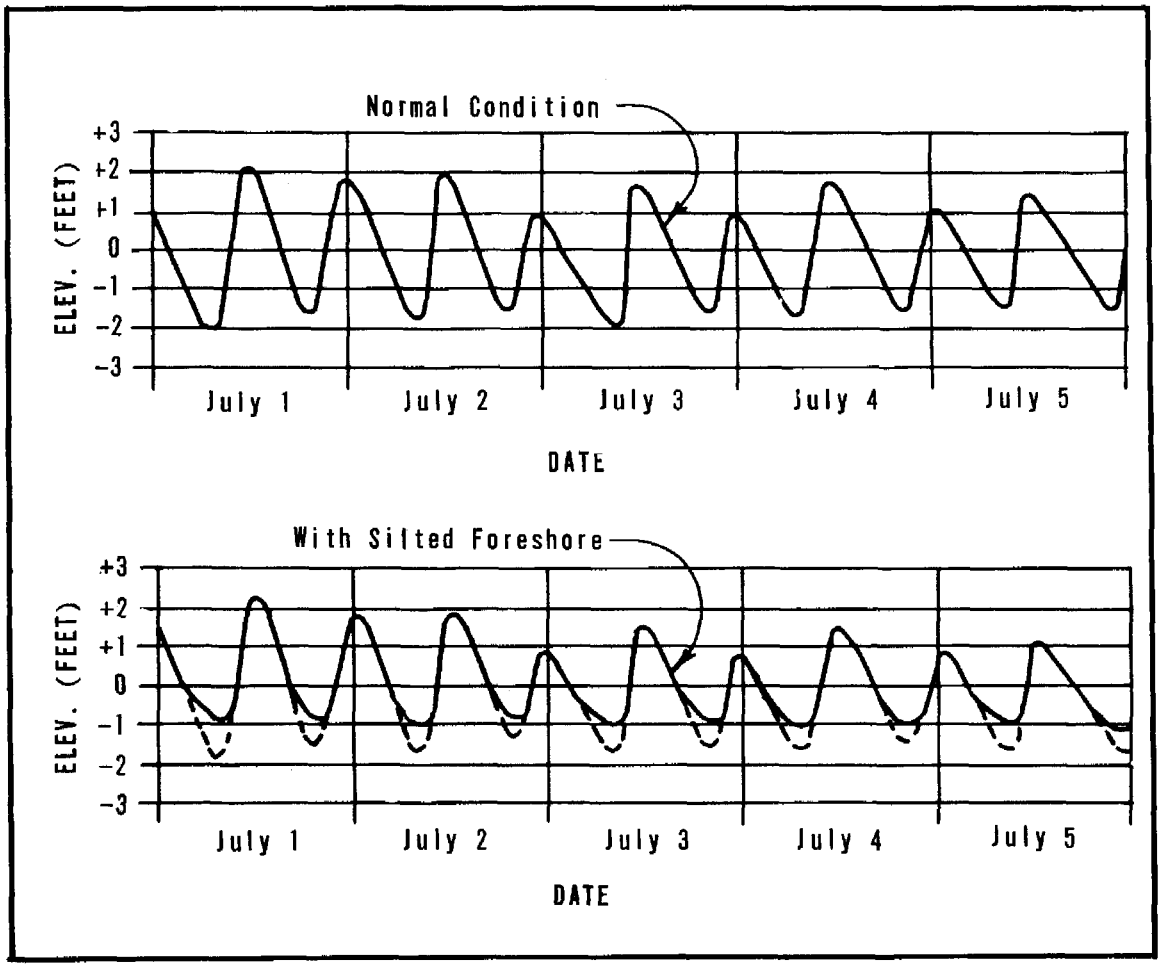


Figure 9-2, Typical estuary tides

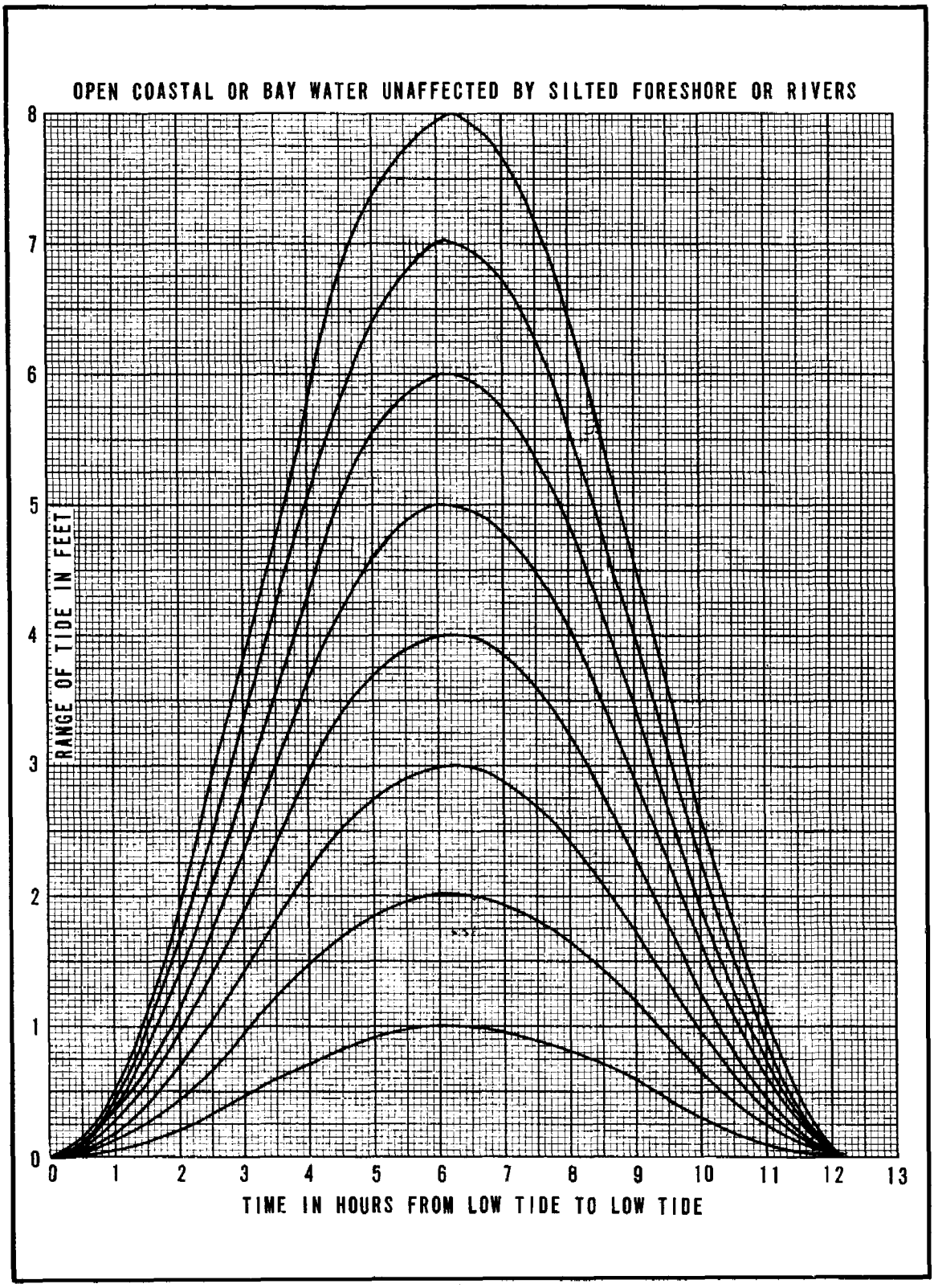


Figure 9-3, Theoretical tide curves

water tide curves. As river flow increases, tide swings become less and less. At high flood stages, effects of tides may be obliterated only a few miles upstream. Silted foreshores affect the water level oscillations by limiting the low elevations to which water might fall. The effect is on the low tide, whereas high tides tend to pour over deposits and rise to about the same elevation as if no silt deposits were present.

Determination of Local Tide Data

Tide data is best determined by direct observation at the site. Staff and automatic gages are used for this purpose. The staff gage is essentially a graduated board, set vertically at the edge of tidal water so that height can be read by an observer. Markings on the staff are usually graduated in feet and tenths for ease in reading at a distance. The automatic gage may be either of the float or bulb type that records the elevations on a reduced scale on clock driven charts.

A staff gage should be installed along with automatic gages to provide a means of checking and calibrating the automatic gages when necessary. The bulb-type gage usually is less accurate than the float-type gage; however, the bulb gage is sufficiently accurate for obtaining drainage design data and is quite desirable because of the ease with which it can be installed and removed for short-period setups at isolated sites. Gages should be referenced to accurate bench marks that are protected against damage or destruction. For convenience, gages should be set with their zeros on established local datum. Staff gage readings should be recorded over several tidal cycles. Where automatic gages are used, records should be obtained over a lunar month if possible. Readings should be taken when slight or no winds occur, or over long periods so that the effects of wind can be evaluated.

Onsite data should be correlated with the nearest local gages and records, which are often local port or harbor data. These not only provide a means of evaluating minor local effects, but make it possible to project to the site the major fluctuations caused by wind tides or river floods from long-time records of other gages. Adequacy of local records may vary, but good sources are usually available from nearby operating stations of the Corps of Engineers, Coast Guard and Coast and Geodetic Survey. Municipalities and harbor and port authorities often have complete and accurate records. Information on extreme high water can often be traced to local events recorded in old newspapers or community records or through inquiries of local residents. If the elevation of either or both low and high waters and tidal ranges are known, and effects of local distortions to the flood or ebb tide are insignificant, a reasonably accurate site curve can often be selected from the theoretical curves shown in Figure 9-3. Table 9-1 illustrates the record for an observed semi-diurnal tidal cycle obtained by an observer from watch and staff-gage readings taken at hourly intervals. This data plotted on Figure 9-4, shows the resulting tidal curve.

For reasons which will be explained later in the text, the tide data should be observed for two or three hours past the second high water stage of the first cycle. This enables plotting the elevation at which the tide gate will open in the second cycle.

Table 9-1, Field observations - tide cycle

	Time (hours & minutes)	Gage Height (feet)		Time (hours & minutes)	Gage Height (feet)
AM	6:30	3.3	PM	3:30	-0.9
	7:25	3.7		4:25	0.0
	8:30	3.4		5:32	1.5
	9:34	2.0		6:30	3.0
	10:28	0.9		7:35	3.7
	11:30	-0.4		8:30	3.6
PM	12:28	-1.5		9:25	3.0
	1:30	-1.7		10:30	1.6
	2:32	-1.5			

Site Investigations and Surveys

Tidal lands subject to impairment due to their relatively low position with respect to the ocean are subject to many complicated situations. It is not the intent of this handbook to attempt to cover all of these situations, but rather to cover the common or usual situation. This is where low lying lands in coastal areas are wet because drainage from interior land areas is temporarily blocked by high tides and does not have free outlet to the ocean except during periods of low tide. During periods of high tide, which occur twice daily in most coastal areas, drainage water from the interior land area is ponded in drainage channels and adjacent low areas, including low lying croplands. This ponded water may be fresh water, salty water or a mixture of the two, depending on the relative hydraulic gradient between the ocean and the ponded drainage water. Usually it is some mixture of the two. The basic purpose of drainage in these areas is to prevent sea water from moving into these areas during high tide and to provide for disposal of the accumulated fresh water from interior drainage during periods of low tide.

A system to accomplish this purpose is one utilizing a dike with automatic tide gate or gates incorporated in the dike. The dike is designed and constructed to prevent sea water from moving into the low areas and the tide gates are designed and installed to discharge the accumulated drainage water from the land side during periods of low tide. The success of such a system is dependent on the amount of storage available in the drainage channels, forebay area and connecting low areas on the land side of the dike. This storage area must be adequate to contain the interior drainage water accumulated during periods of high tide, and it must be below the elevation where damage to agricultural operations begins. If adequate storage is not available to meet the above requirements there are usually two alternatives. The storage area can be increased by excavating a larger drainage channel or forebay area, or pumps can be installed to remove a portion of the drain water so that the existing storage area will be adequate.

Topographic surveys

As pointed out in the previous discussion, the usual drainage plan for drainage of tidal lands involves a dike, tide gate structures and a land side

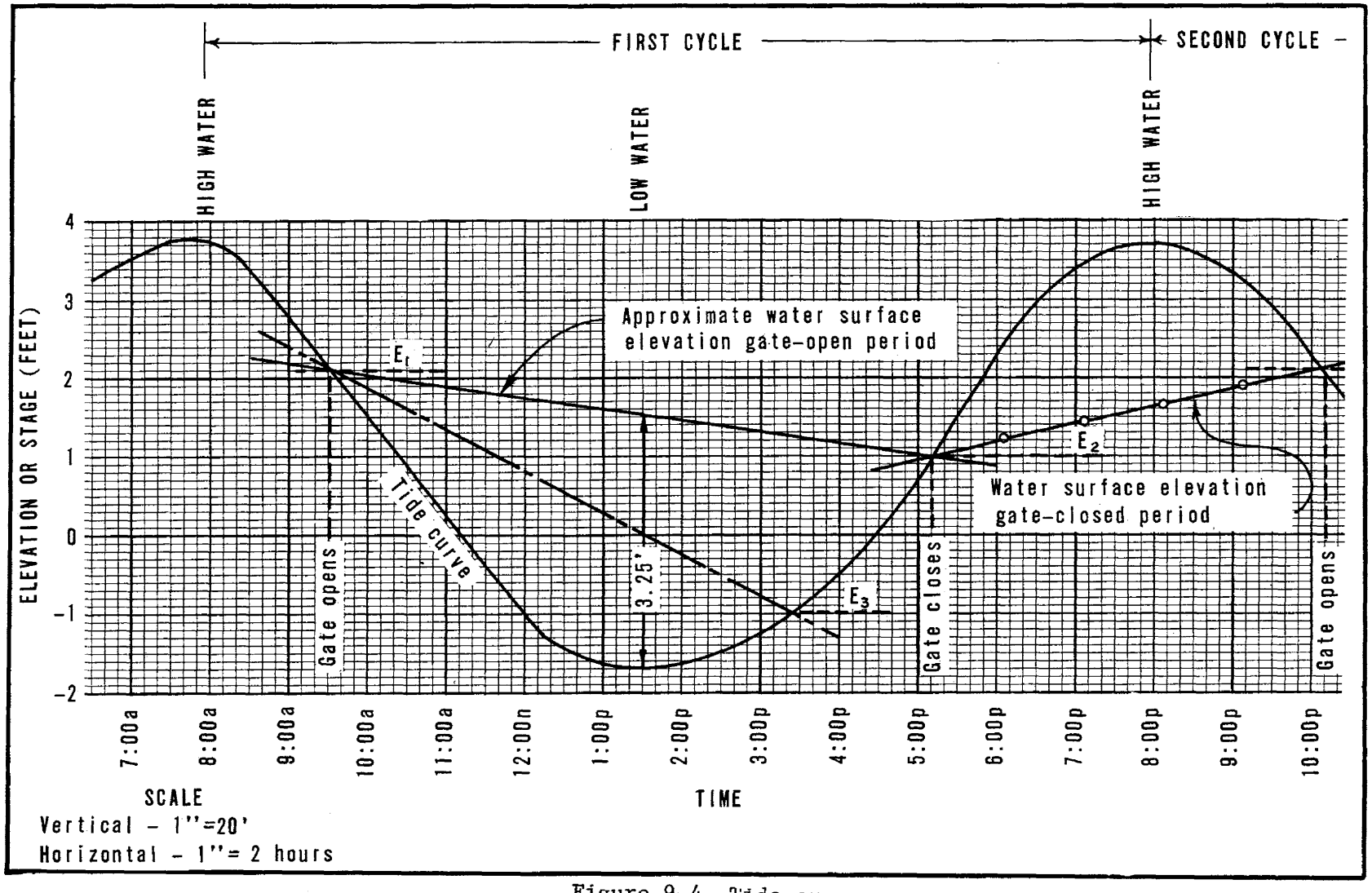


Figure 9-4, Tide curve

storage reservoir, which is the forebay area, the drainage channels and any other low areas connected with the drainage system. For design purposes it is necessary to relate the stage and volume of the storage area to the stage of the tide and to the elevation of the dike and tide gate or gates. For this reason it is necessary to have complete topography for the storage area and the site of the dike. Topographic surveys must be on the same datum as tidal observations. The topographic survey covering the storage area should be carried up to the high-water elevation at high tide or to some predetermined elevation which is the maximum permissible water level for agricultural operations.

Subsurface investigations

Dikes used in connection with the drainage of tidal lands are basically no different than dikes used for other purposes. Subsurface explorations are necessary to determine the foundation conditions and to locate suitable fill material. Dikes are classified as class I, II, or III, in accordance with the value of crops or property to be protected and the hazard to life. Refer to Table 6-1, Chapter 6, for a detailed classification of dikes. Dikes used in connection with the drainage of tidal lands usually contain tide gates and tide-gate structures. The dike and the gate structures are built as integral parts and are interdependent. If the dike should fail, there is a good chance that the gate structure would fail or be washed out also. For this reason, dikes containing gate structures are usually constructed to higher standards than dikes without gates.

Design Data

The basic information and data needed to design a drainage system for tidal lands includes the following:

1. A tide curve similar to the one shown as Figure 9-4, or one reconstructed from typical curves shown on Figure 9-3. The curve should cover one full cycle, from high water to high water, as indicated on Figure 9-4. This cycle will be referred to as the "first cycle" in this text. In addition, the curve should be continued into the "second cycle" for a period of about three hours, see Figure 9-4.

2. A stage-storage curve for the reservoir area, similar to the one shown on Figure 9-5. The vertical scale for stage or elevation must be on the same datum as the tide curve and the horizontal scale for storage should be in acre feet. The reservoir area includes the forebay, drainage channels, and any other connecting low areas that would be a part of the storage reservoir. The stage-storage curve should extend to high-water elevation at high tide, or to a previously established maximum water surface elevation in the forebay.

3. The maximum permissible water surface elevation in the forebay and reservoir area. This elevation is usually fixed by the intended land use and agronomic consideration of the crops to be grown on the land adjacent to the reservoir area. This establishes the highest level for storage of water in the forebay and also establishes the design heads through culverts and gates. As a general rule this elevation is established at about one foot below the general ground level. It is desirable to set this elevation as high as possible to obtain maximum storage.

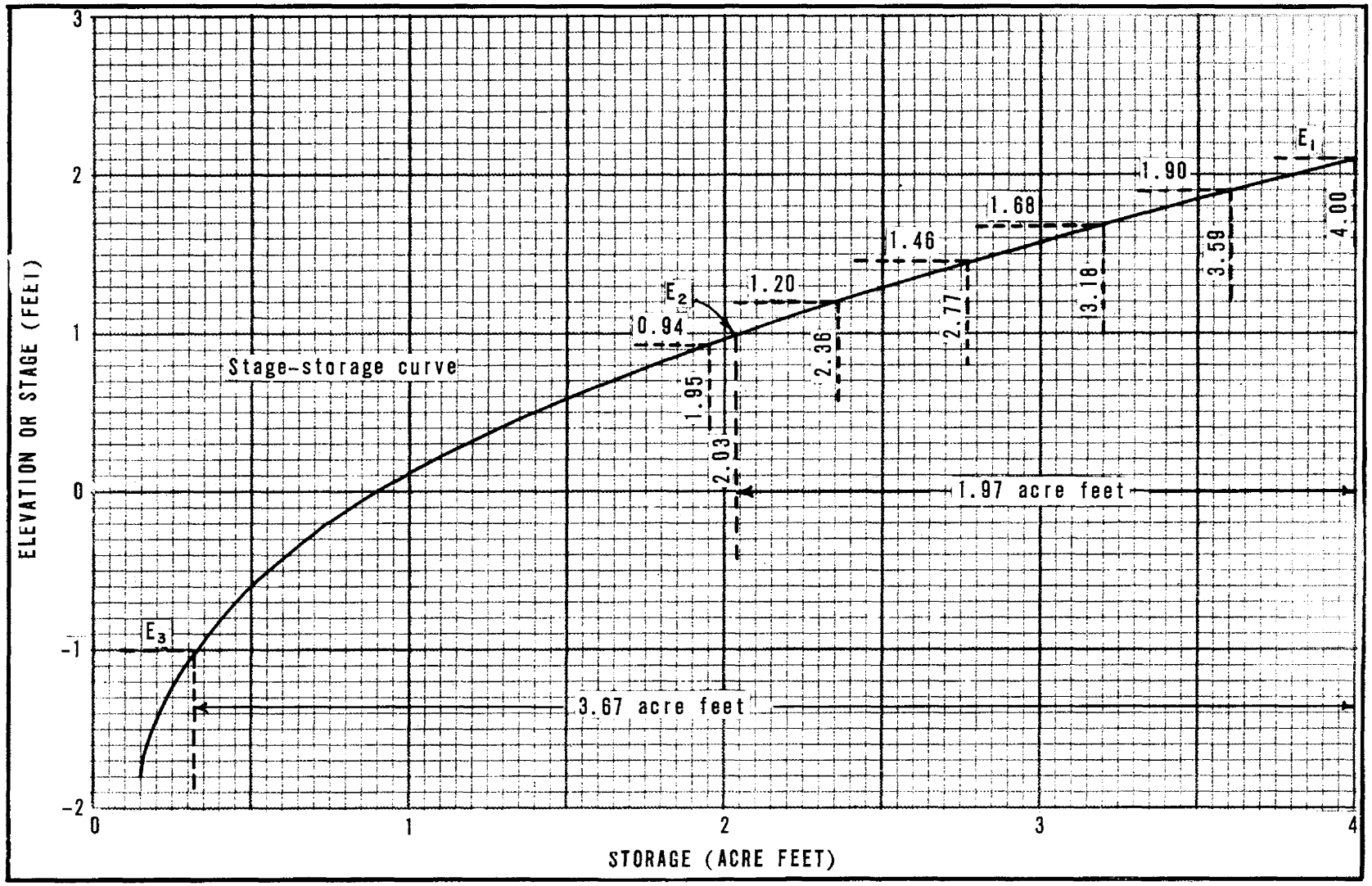


Figure 9-5, Stage-storage curve

4. The rate of drainage flow to the reservoir area. The design of the outlet system should be based on the same drainage coefficients, surface and/or subsurface, as are applicable to the adjoining non-tidal lands. Such coefficients are prescribed by local drainage guides. Usually the design for storm intensities of two year frequency is adequate for hay and pasture land, five year for rotated crops and ten to twenty year for intensive truck crops. If subsurface drains discharge into the outlet system the design flow should be increased by the amount of their accretion to the total flow.

Design Procedure and Example

The following design procedure for a tide-gate installation is a simplified graphical solution, based on the use of a tide curve and a stage-storage curve prepared for the site. The example is for a site subject to semi-diurnal tides; however, the same general procedure is applicable to diurnal tides. In view of the fact that this is a graphical method using a tide curve and a stage-storage curve prepared for the site, these curves should be prepared as accurately as possible and on a scale suitable for interpolation, consistent with the accuracy of the data plotted. The example given herein is necessarily based on the use of small scale charts suitable for inclusion in this handbook. In actual practice these charts should be drawn on a much larger scale to permit more accurate interpolation.

The following step by step explanation of this procedure, with example, is based on the assumption that a tide curve, Figure 9-4, and a stage-storage curve, Figure 9-5, have been prepared; that the drainage inflow to the system has been determined; and that the maximum water surface elevation for the forebay and reservoir area has been established. For the following example it will be assumed that the drainage inflow rate is 5.0 cfs and that the maximum permissible water surface elevation for the reservoir area is elevation + 2.10 feet.

Step 1. Plot the maximum water surface elevation + 2.10 feet on the ebb tide limbs for both cycles of the tide curve, Figure 9-4, and on the stage-storage curve, Figure 9-5. These points are noted as point E_1 on Figures 9-4 and 9-5, and are the pre-established elevation at which the tide gate will open.

Step 2. Compute the hourly volume of inflow to the storage reservoir. If the drainage inflow rate to the reservoir is 5.0 cfs, this is equivalent to a volume of 0.41 acre feet per hour. One cubic foot per second is one acre foot per 12.1 hours; therefore:

$$\text{Volume inflow per hour} = \frac{(5.0 \text{ cfs})(1.0 \text{ hour})}{12.1} = 0.41 \text{ acre feet hour}$$

Step 3. Compute the hourly volume of water in storage and the corresponding reservoir stage during the gate-closed period. Note that point E_1 in the second cycle of the tide curve is at elevation + 2.10 at 10:07 p.m. and the tide gate is just ready to open as the stage of the tide and the reservoir are the same. Referring to point E_1 on the stage-storage curve it will be noted that the storage reservoir contains 4.00 acre feet at this time. Prior to 10:07 p.m. the tide gate has been closed for several hours and the reservoir has been filling at the rate of 0.41 acre feet per hour. At 9:07 p.m. the reservoir contained 4.00 - 0.41 = 3.59 acre feet; at 8:07 p.m. it contained 3.59 - 0.41 = 3.18 acre feet etc. The water surface elevation or stage of the reservoir for any given volume of storage can be determined from the stage-storage curve. The following tabulation gives the volume of water in storage and the corresponding stage at hourly intervals during the period that the tide gate was closed:

Time	Acre feet in storage	Stage of storage reservoir
10:07 p.m.	4.00	+ 2.10
9:07 p.m.	3.59	+ 1.90
8:07 p.m.	3.18	+ 1.68
7:07 p.m.	2.77	+ 1.46
6:07 p.m.	2.36	+ 1.20
5:07 p.m.	1.95	+ 0.94

Step 4. Using the data in the above tabulation, plot the time-stage curve for the period 5:07 p.m. to 10:07 p.m. on the tide curve, Figure 9-4. Locate point E_2, the point where the tide curve and the time-stage curve intersect on the flood tide limb of the first cycle. This is the point where the tide gate will close as the stage of the tide and the storage reservoir are the same. Note that gate-closure occurs at 5:10 p.m. and the gate-closed time is 4 hours 57 minutes or 4.95 hours. The total drainage inflow during the gate-closed time is:

$$\text{Volume inflow} = \frac{(5.0 \text{ cfs})(4.95 \text{ hours})}{12.1} = 2.04 \text{ acre feet}$$

The stage at point E_2 on the tide curve is + 1.0 feet. Referring to the stage-storage curve the storage at a stage of + 1.0 is 2.03 acre feet and this subtracted from + 4.00, the storage of the full reservoir, is 1.97 acre feet. This indicates an error of 0.07 acre feet in the graphical solution which is well within the allowable limit of error.

Through the procedure described above, reservoir storage and inflow during the gate-closed period are now in balance and the time of gate-closure has been determined as 5:10 p.m. In this example the time-stage curve for the period 5:07 to 10:07 is virtually a straight line; however, it may be a flat curve in some cases, depending on the shape and configuration of the reservoir area.

Tide gate capacity and size

The previous discussion presented a method to determine the tide elevation at which the tide gate should open and close in order to balance the available forebay and reservoir storage against the drainage inflow during the time that the tide gate is closed. It is desirable to have the gate-closure elevation as high as possible in order to maintain a maximum head on the tide gate during the time that it is open and flowing.

The following discussion presents a method for determining the design capacity and size of a tide gate or gates. The example given is a continuation of the previous example used.

Step 1. Draw a straight line between points E_2 and E_1 in the first cycle of the tide curve, Figure 9-4. This is the approximate water surface elevation in the forebay during the time that the tide gate is open. The head on the tide gate (submerged flow) at any time is the vertical scaled distance between the tide elevation and the line drawn between E_1 and E_2 discussed above. For example, the head acting on the gate at 1:30 p.m. is 3.25 feet.

Step 2. Compute the volume of water to be discharged through the tide gate during the time that it is open from 9:30 a.m. to 5:10 p.m. This is the total volume of drainage inflow during one twelve hour cycle.

$$\text{Gate discharge one cycle} = \frac{(\text{inflow rate})(\text{time flow})}{12.1}$$

$$\text{Gate discharge one cycle} = \frac{(5.0 \text{ cfs})(12 \text{ hours})}{12.1} = 5.0 \text{ acre feet}$$

Step 3. Compute the average flow through the tide gate during the time that it is open. The tide gate is open from 9:30 a.m. to 5:10 p.m., or for a period of 7 hours and 40 minutes or 7.7 hours.

$$\text{Average flow} = \frac{(\text{inflow rate})(\text{cycle time})}{(\text{open-gate time})}$$

$$\text{Average flow} = \frac{(5.0 \text{ cfs})(12 \text{ hours})}{(7.7 \text{ hours})} = 7.8 \text{ cfs}$$

Step 4. Determine the average head on the tide gate during the time that the gate is open. The head at any time is equal to the vertical scaled distance between the tide curve and the straight line drawn between E_1 and E_2, as explained in step 1. Determine the average head by scaling the head at 30 minute intervals; record this information in column 2, Table 9-2; and compute the average head for the gate-open period.

$$\text{Average head} = \frac{31.51}{15} = 2.10 \text{ feet}$$

Step 5. Make a trial computation for the gate size, using the average flow of 7.8 cfs computed in step 3; and the average head of 2.10 feet computed in step 4. Computing the discharge through a tide gate attached to pipes of various lengths is a rather complex problem. To simplify this for the designer, charts shown as Figures 9-6 and 9-6a have been prepared from computer solutions. The family of curves shown on these charts covers the range of tide gates from 12 inch to 66 inch, and for each size gate there are four pipe lengths of 100, 80, 60, 40 and 20 feet. This coverage is thought to be adequate for most situations. The head in feet is shown on the vertical scale and the discharge in cfs is shown on the horizontal scale. Note that the two charts, Figures 9-6 and 9-6a, are actually the same chart with alternate gate sizes shown on each to separate the curves and avoid the clutter and confusion of closely spaced curves. Using these charts and assuming a pipe length of 80 feet for this example, find the intersection of the average head 2.10 feet and the average discharge of 7.8 feet on Figure 9-6. This point falls between the 18" x 80' and the 18" x 100' curves. The 18" x 80' gate and pipe assembly has a discharge capacity of 8.0 cfs which is a little more than the average of 7.8 cfs and indicates that the 18 inch gate may be adequate. The above approximation, using average head and average discharge is only approximate and it is now necessary to refine these computations to determine if the 18 inch gate is adequate.

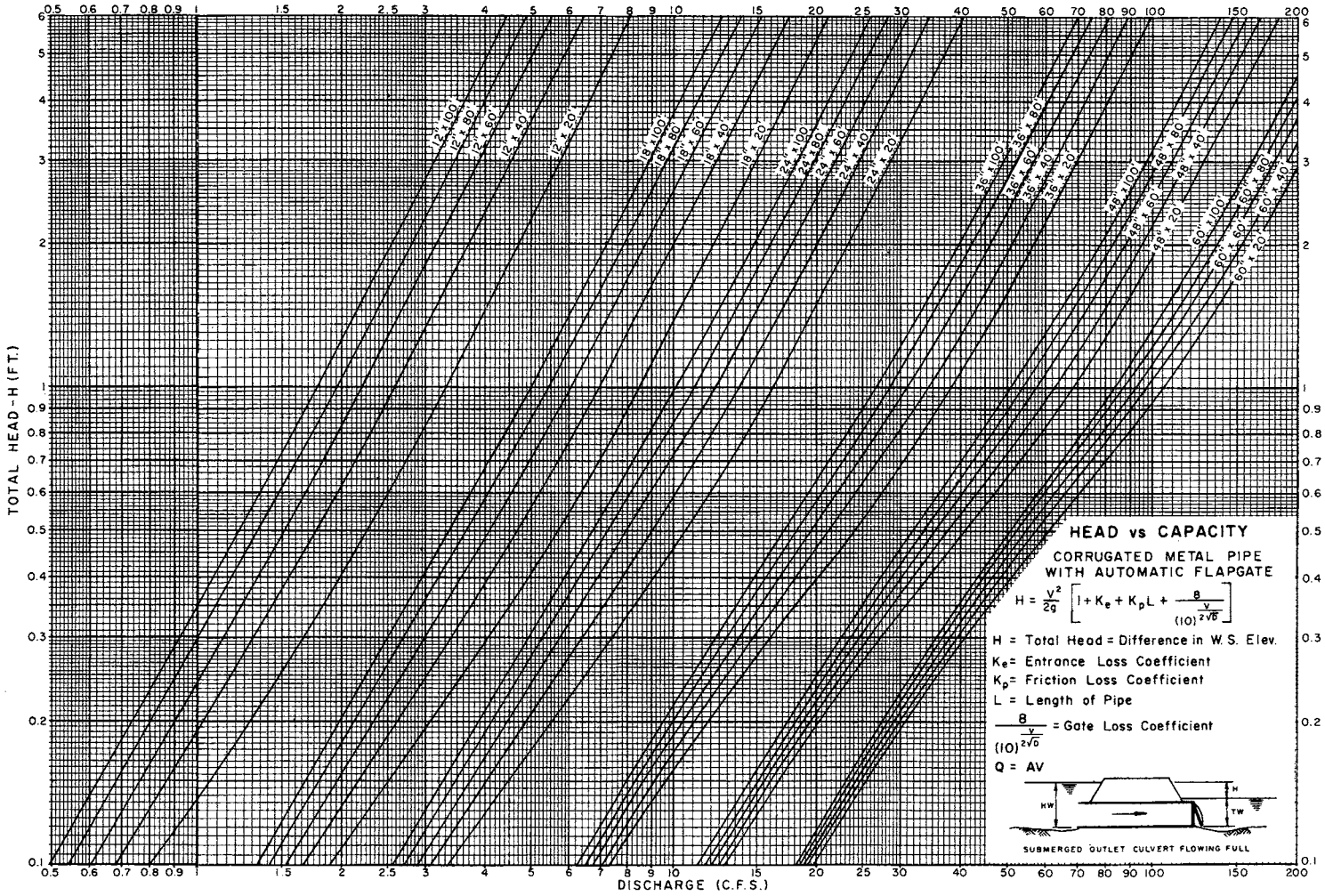
Step 6. Compute the average heads for each 30 minute increment of time that the tide gate is open, record these average heads in column 3, Table 9-2. Using Figure 9-6, determine the average discharge for each average head shown in column 3, and record these in column 4, Table 9-2. Compute the volume of discharge, in acre feet, for each 30 minute increment of time and record this in column 5, Table 9-2. Total column 5, to determine the total discharge of the tide gate during one tide cycle. This is equal to 4.82 acre feet which indicates that the 18" x 80' gate and pipe structure is adequate as the total drainage inflow, determined in step 2, was 5.0 acre feet. The difference between 4.82 and 5.00 is less than four percent which is within the allowable error for this determination. In the event that the discharge of the 18 inch

Table 9-2, Tide gate data sheet

(1) Time	(2) Measured Head (feet)	(3) Average Head 30 min. int. (feet)	(4) Average 1/ Discharge 18" X 80' (cfs)	(5) Volume 2/ Discharge 18" X 80' (acre feet)
Gate open 9:30a	0.00			
		0.25	2.43	0.10
10:00a	0.50			
		0.80	4.80	0.20
10:30a	1.10			
		1.37	6.42	0.27
11:00a	1.65			
		1.92	7.70	0.32
11:30a	2.20			
		2.47	8.70	0.36
12:00n	2.75			
		2.95	9.60	0.40
12:30p	3.15			
		3.21	10.03	0.42
1:00p	3.28			
		3.26	10.04	0.42
1:30p	3.25			
		3.17	9.90	0.41
2:00p	3.10			
		3.00	9.70	0.40
2:30p	2.90			
		2.75	9.25	0.39
3:00p	2.60			
		2.37	8.60	0.36
3:30p	2.15			
		1.89	7.60	0.32
4:00p	1.63			
		1.31	6.30	0.26
4:30p	1.00			
		0.62	4.15	0.17
5:00p	0.25			
		0.13	1.65	0.02 3/
Gate close 5:10p	0.00			
Totals	31.51	----	----	4.82

1/ Average discharge for 18" tide gate attached to 80' pipe.
2/ Volume of discharge for 18" tide gate attached to 80' pipe.
3/ Adjusted for 10 minute time interval.

DRAINAGE: HEAD vs DISCHARGE - FLAPGATE WITH ATTACHED LENGTHS OF CORRUGATED METAL PIPE



REFERENCE

These curves were developed by the Design Section, EWPU Portland.

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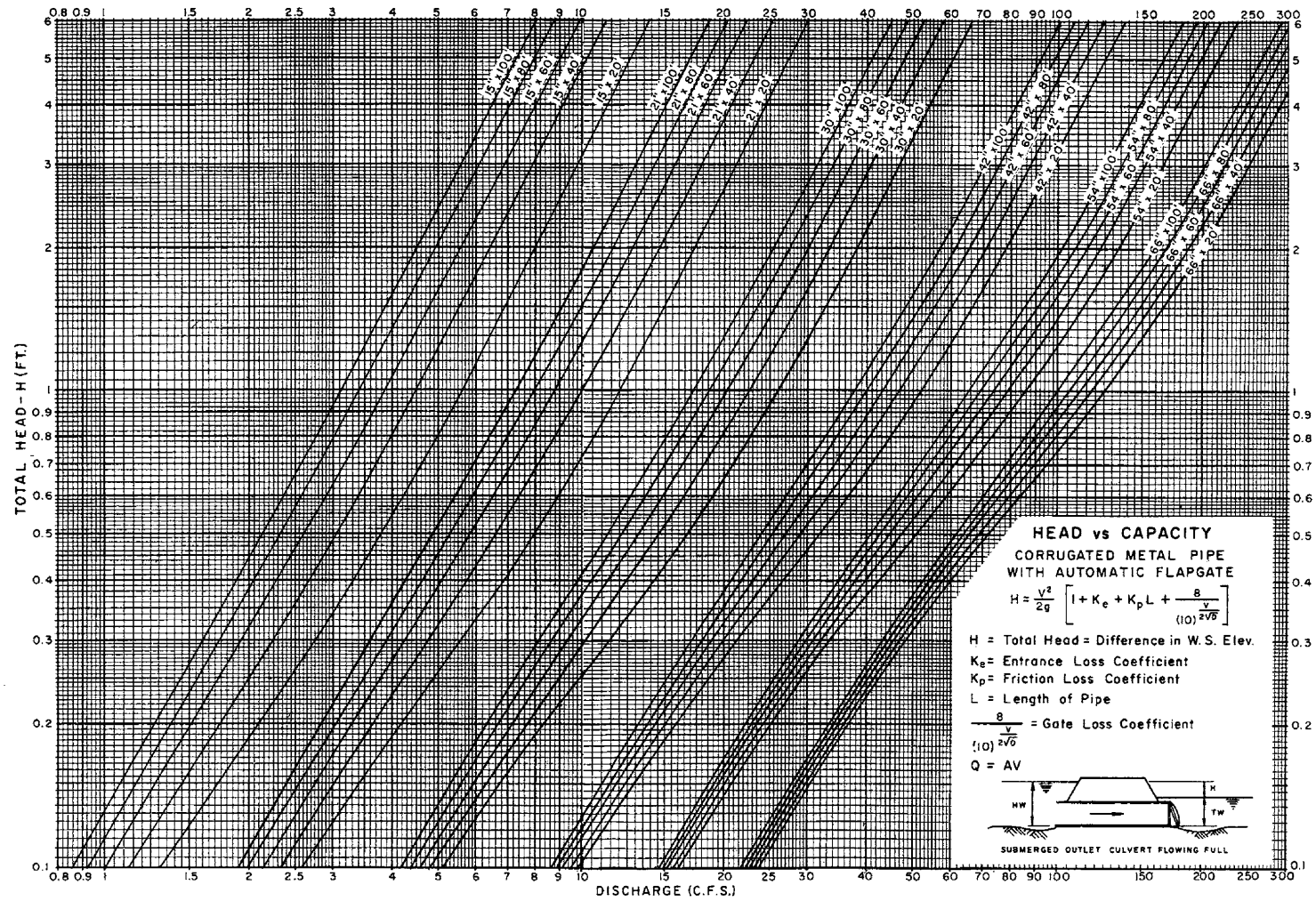
STANDARD DWG. NO
ES-723

DRAINAGE: HEAD vs DISCHARGE - FLAPGATE WITH ATTACHED LENGTHS OF CORRUGATED METAL PIPE



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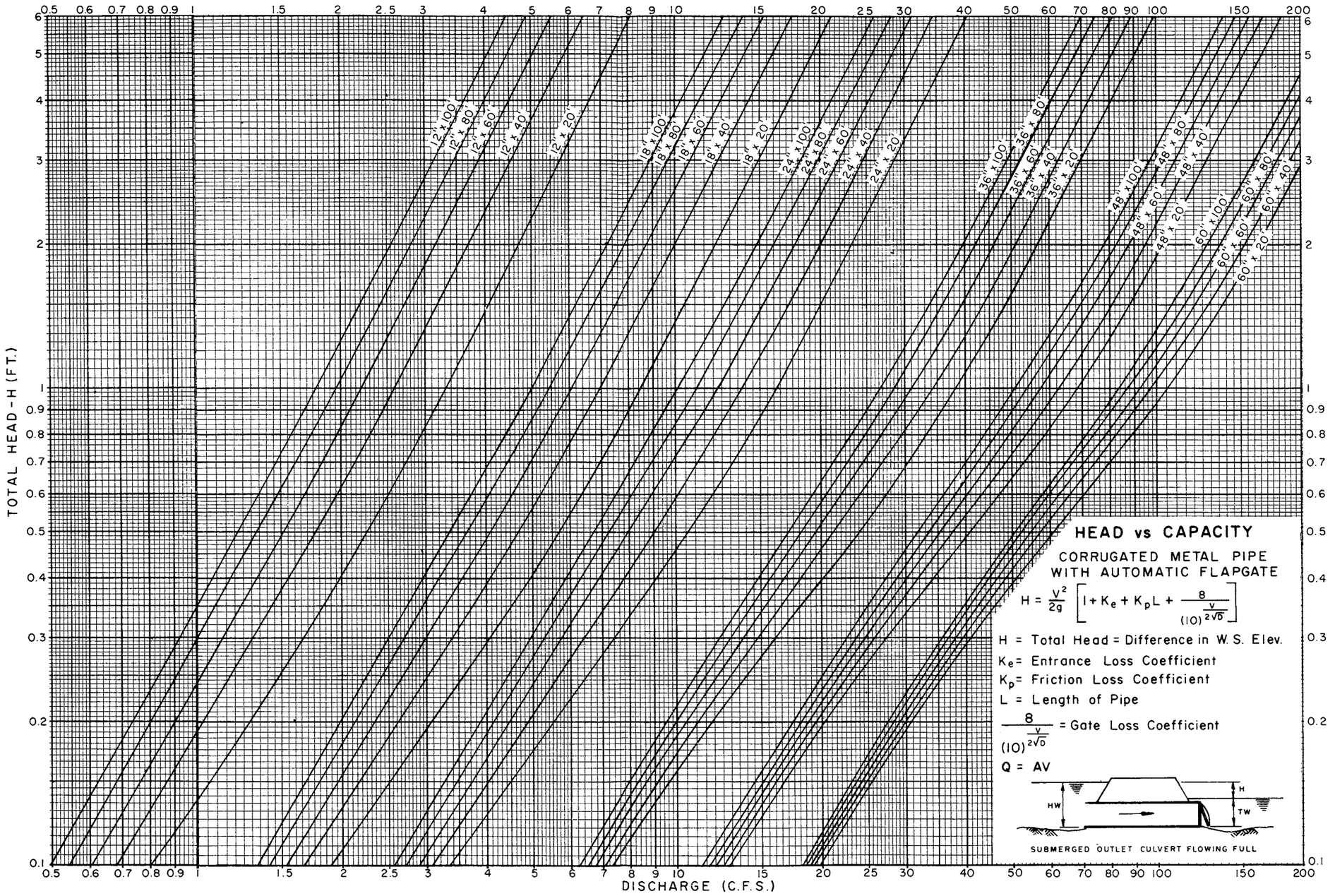
DRAINAGE: HEAD vs DISCHARGE - FLAPGATE WITH ATTACHED LENGTHS OF CORRUGATED METAL PIPE



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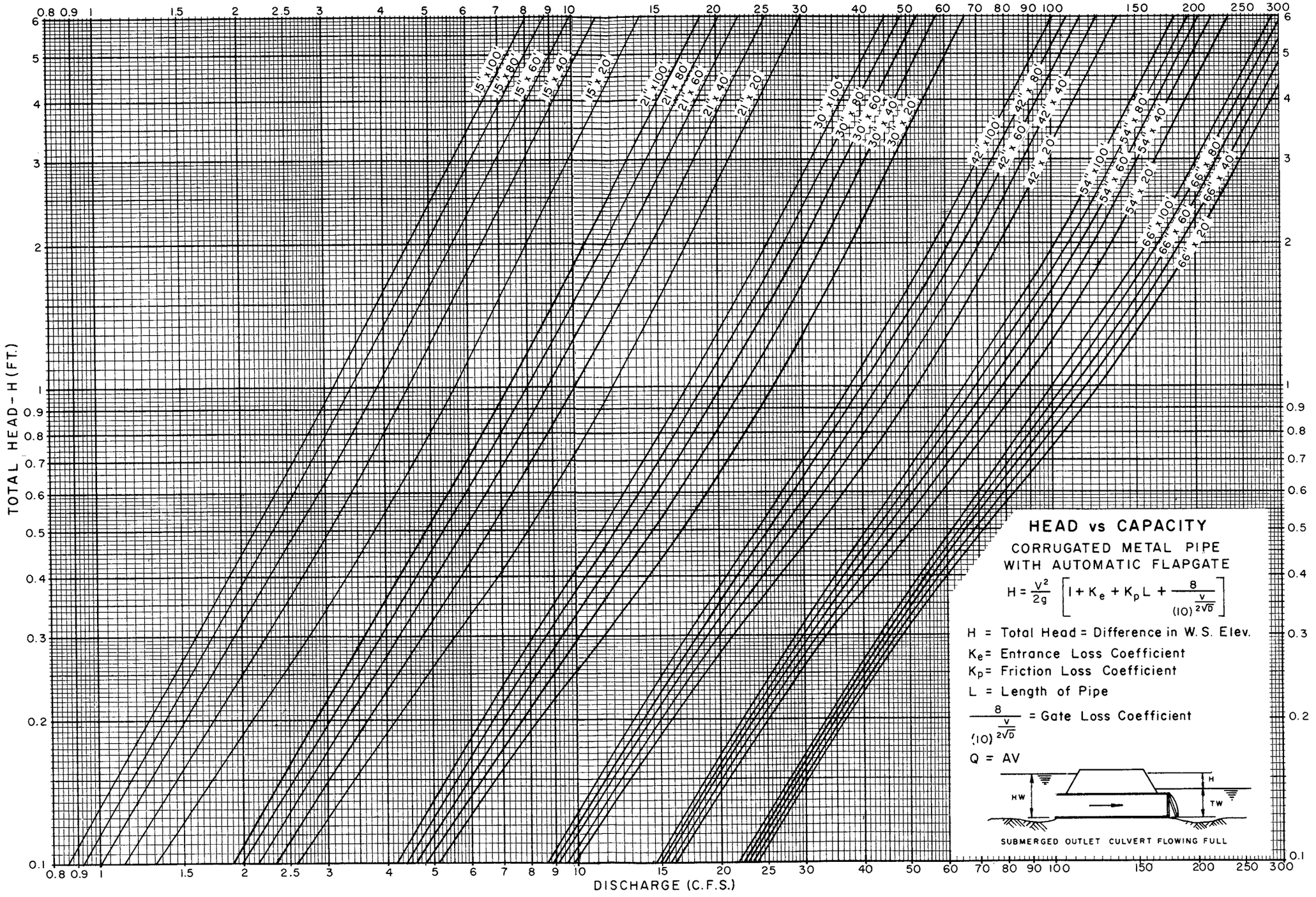
SHEET 1 OF 2
DATE 10-15-70

Figure 9-6, Curves for tide gate discharge

ES-723

DRAINAGE: HEAD vs DISCHARGE - FLAPGATE WITH ATTACHED LENGTHS OF CORRUGATED METAL PIPE



HEAD vs CAPACITY

CORRUGATED METAL PIPE
WITH AUTOMATIC FLAPGATE

$$H = \frac{V^2}{2g}\left[1 + K_e + K_p L + \frac{8}{\frac{V}{(10)^{2\sqrt{D}}}}\right]$$

H = Total Head = Difference in W. S. Elev.

K_e = Entrance Loss Coefficient

K_p = Friction Loss Coefficient

L = Length of Pipe

$\frac{8}{\frac{V}{(10)^{2\sqrt{D}}}}$ = Gate Loss Coefficient

Q = AV

SUBMERGED OUTLET CULVERT FLOWING FULL

REFERENCE

These curves were developed by the Design Section, EWPU Portland

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ES-723
SHEET 2 OF 2
DATE 10-15-70

Figure 9-6a, Curves for tide gate discharge

ES-723

tide gate was considerably less than the drainage inflow for the cycle, the next larger size gate, a 21 inch gate would be selected and it would be necessary to repeat step 6, to compute the discharge of this gate.

Figures 9-7, 9-7a, and 9-7b, showing the sequence of relative stage elevations, of ocean and forebay, versus time, have been included to help the reader to visualize the operation of a tide gate installation. These have been prepared to illustrate the previous problem, assuming an 18 inch tide gate installation and using the data tabulated in Table 9-2.

Supplemental Pumping

The previous example for the design of a tide gate installation was based on a situation where there was more than adequate reservoir area to store the accumulated drainage inflow during the period that the tide gate was closed. In actual practice this is not always the case, as there may not be adequate storage area in the forebay and natural or artificial channels on the land side of the dike and tide gate. Where the storage is not adequate, there are two alternatives open to the designer. The storage area can be increased by excavating a larger forebay and channel area or pumps can be installed to remove a part of the drainage inflow to balance the available storage to the drainage inflow for the time that the tide gate was closed. In the previous example the storage reservoir was lowered to elevation + 1.0 feet which was all that was necessary to provide the required 1.97 acre feet of storage needed while the tide gate was closed. This was only about one half of the total storage area available in this reservoir.

As a general rule, it is usually not practical to attempt to lower the water surface in the forebay and reservoir lower than elevation - 1.0 feet because of the construction problems involved with sloughing and in de-watering the site; however, this will depend on individual site conditions. Referring to the previous example, if the reservoir had been lowered to elevation - 1.0 feet it would have been possible to store 3.67 acre feet rather than 1.97 acre feet and the gate-closed time would have been 6 hours and 42 minutes rather than 4 hours and 57 minutes. This would be the maximum utilization of this storage reservoir, assuming that it could be lowered to elevation - 1.0 feet, and would provide for a drainage inflow rate of about 6.6 cfs. If the drainage inflow was greater than 6.6 cfs it would be necessary to resort to supplemental pumping to remove a portion of the drainage inflow during the time that the tide gate was closed.

The above discussion suggests a rapid method to estimate the capability of a given reservoir or storage area for a known drainage inflow rate. The procedure is as follows:

Step 1. Based on a consideration of site conditions and the intended use of the land being protected, establish the maximum and minimum stages for the reservoir. These stages will establish the gate-open and gate-closure times on the tide curve and the gate-closed time can be computed.

Step 2. The available storage between maximum and minimum reservoir stage can be determined by plotting these stages on the stage-storage curve and determining the storage between these points.

Step 3. Having determined the gate-open time and the available reservoir storage and knowing the inflow rate, the required storage can be computed and compared to the available storage. If the required storage is equal to or less than the available, the reservoir is adequate and pumping will not be

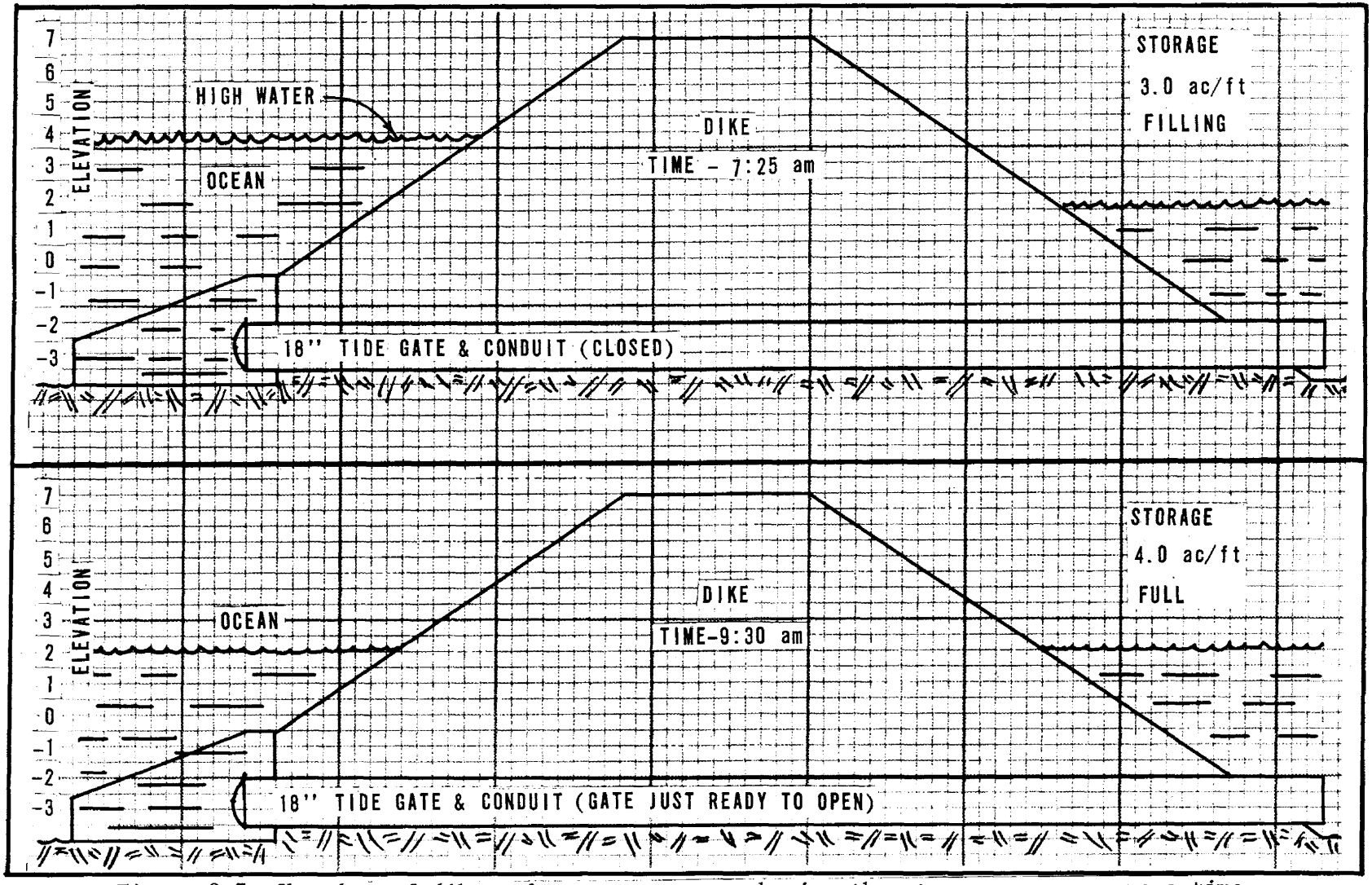


Figure 9-7, Sketches of dike and gate structure showing the stage sequence versus time

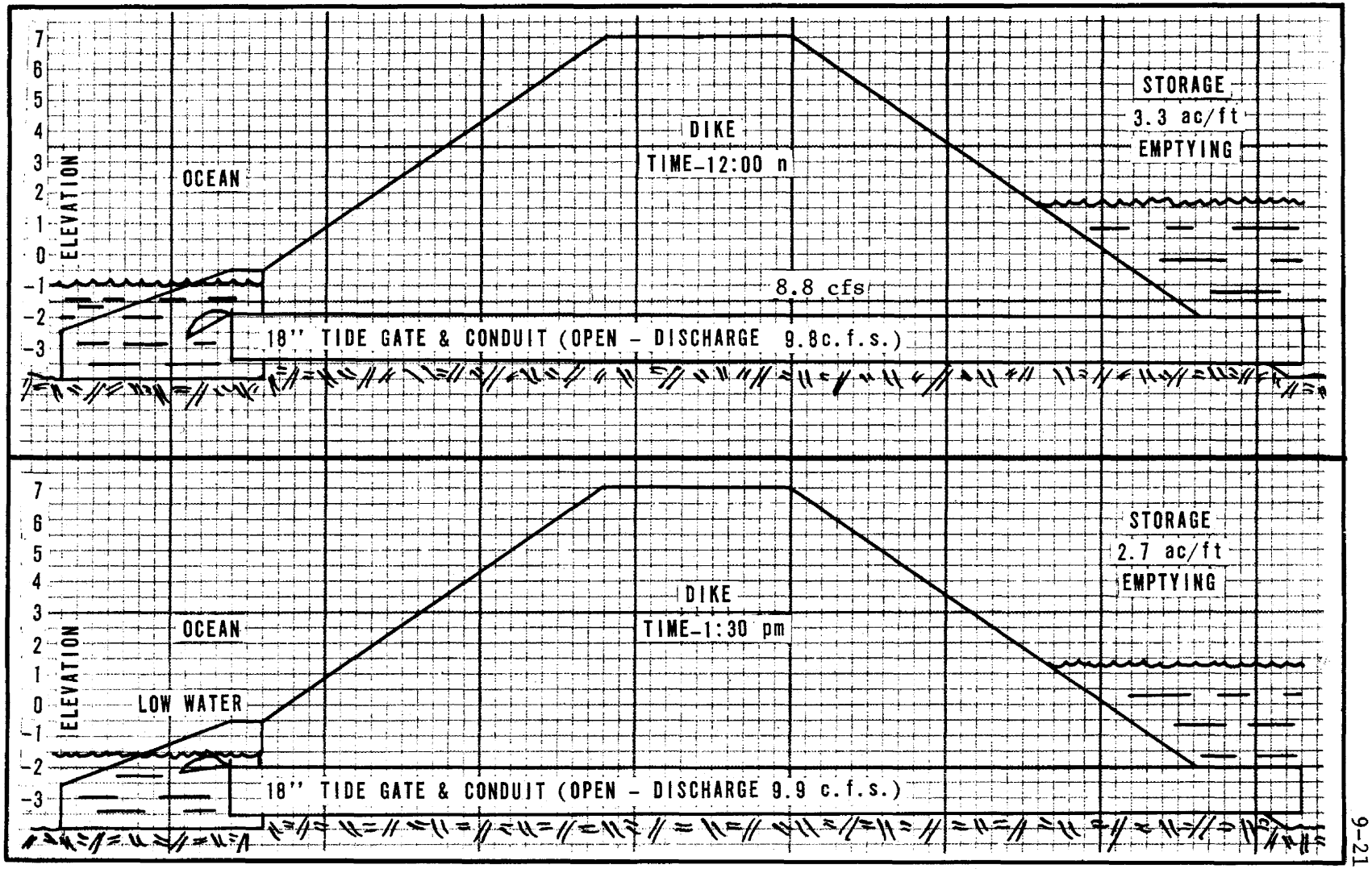


Figure 9-7a, Sketches of dike and gate structure showing the stage sequence versus time

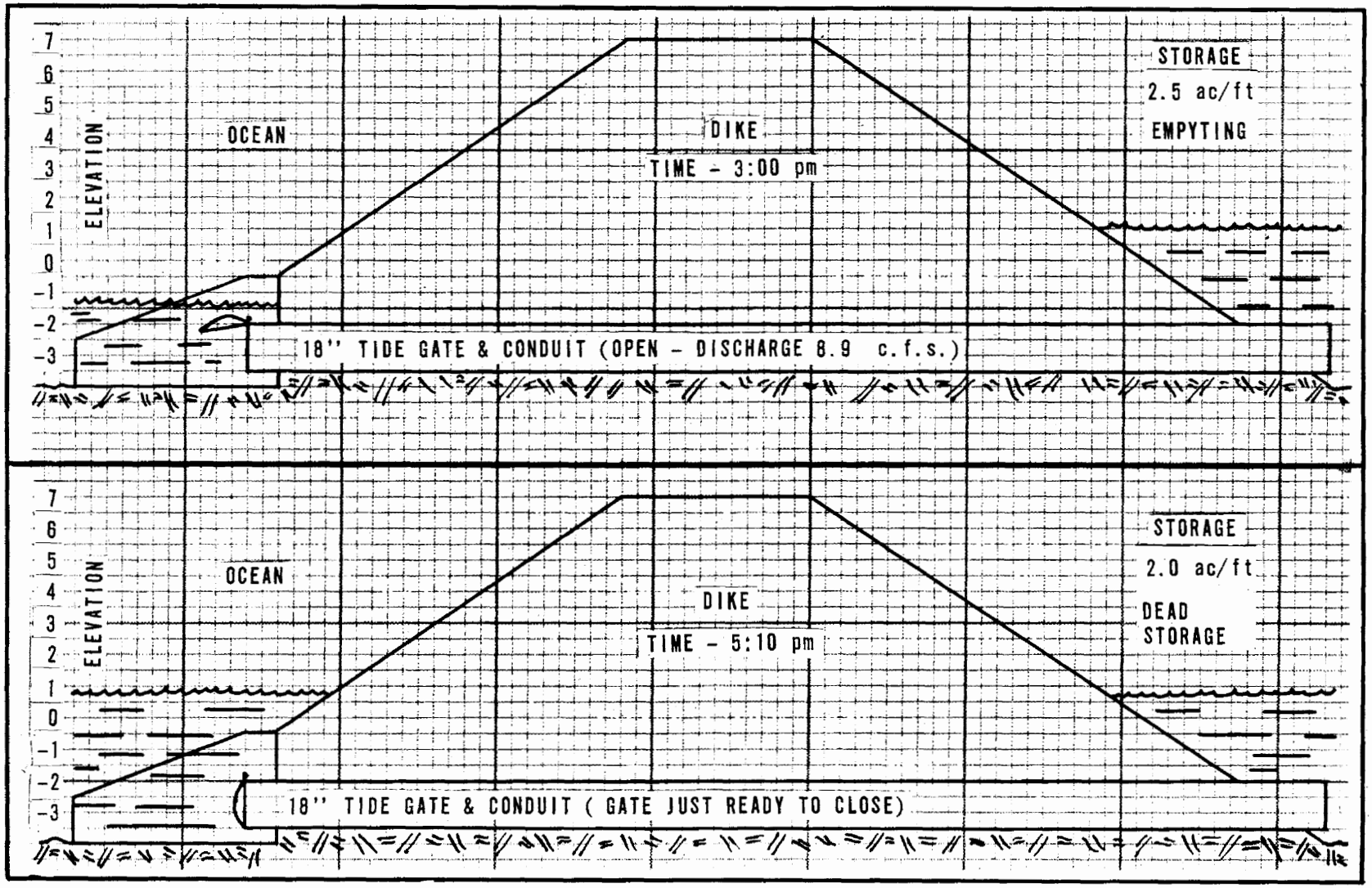


Figure 9-7b - Sketches of dike and gate structure showing the stage sequence versus time

required. If the required storage is greater than the available storage the reservoir might be enlarged or a part of the drainage inflow might be pumped.

This simple determination, to establish whether or not pumping will be required, may determine the feasibility of the project without proceeding further to calculate the tide gate capacity and size.

Pumping requirement

In the previous example it was determined that the storage reservoir had a usable capacity of 3.67 acre feet with drawdown to elevation - 1.0 feet. It was also determined that the capability of the reservoir was limited to a drainage inflow rate of about 6.6 cfs without supplemental pumping or enlargement of the reservoir. In the following example it will be assumed that the inflow rate is 9.0 cfs; that the reservoir can be drawn down to elevation - 1.0 feet, noted as point E_3 on Figure 9-4, and that all other conditions will remain the same as in the previous example.

Step 1. Determine the gate-closed time. The gate-closed time is the time difference between point E_3 and point E_1 on the ebb tide limb of the second cycle of the tide curve. The gate will close at 3:25 p.m. and open at 10:07 p.m.; therefore, the gate-closed time is 6 hours and 42 minutes or 6.70 hours.

Step 2. Determine the volume of accumulated drainage inflow that collects during the gate-closed period.

$$\text{Inflow volume} = \frac{(9.0 \text{ cfs})(6.7 \text{ hours})}{12.1} = 4.98 \text{ acre feet}$$

Step 3. Determine the storage available above elevation - 1.0 feet. Plot point E_3 on the stage-storage curve, Figure 9-5, and determine the volume of storage between points E_1 and E_3. This is equal to 3.67 acre feet.

Step 4. Determine the required pumping rate. If the required storage is 4.98 acre feet and the available storage is 3.67 acre feet this leaves a deficit of 1.31 acre feet which must be pumped during the gate closed period.

$$\text{Pumping rate - cfs} = \frac{(12.1)(1.31)}{6.70} = 2.37 \text{ cfs or } 1064 \text{ gpm}$$

A pump that will handle 1000 gpm would probably be adequate.

Lowering the reservoir to elevation - 1.0 feet will materially reduce the head on the tide gate during the time that it is open and will also shorten the time that the tide gate is open. Referring to the tide curve Figure 9-4, the straight line drawn between points E_1 in the first cycle and E_3 will be the approximate water surface elevation during the gate-open period and it will be necessary to recompute the capacity and size of tide gate required. From inspection it is obvious that this situation will require a much larger tide gate than the one in the first example.

Step 1. Compute the volume of water to be discharged through the tide gate during the time that it is open, from 9:30 a.m. to 3:25 p.m. which is 5.92 hours. This is the total volume of drainage inflow during one twelve hour cycle less the volume of water pumped during the gate-closed period. This is equal to 9.00 - 1.31 = 7.69 acre feet.

Step 2. Compute the average flow through the tide gate during the time that it is open.

$$\text{Average flow} = \frac{(7.69 \text{ cfs})(12 \text{ hours})}{5.92 \text{ hours}} = 15.59 \text{ cfs}$$

Step 3. Determine the average head on the tide gate during the time that the gate is open, and record this information in column 2, Table 9-3.

$$\text{Average head} = \frac{13.90}{12} = 1.16 \text{ feet}$$

Step 4. Make a trial computation for the gate size, using the average flow of 15.59 cfs as computed in step 2; and the average head of 1.16 as computed in step 3. Referring to the chart, Figure 9-6 it will be noted that the 24" x 80' gate and pipe assembly has a capacity of about 12.0 cfs and the next larger size, a 30" x 80' has a capacity of about 20.0 cfs; therefore, it is assumed that 30" gate assembly will be required. This trial computation is only approximate as the average head and the average discharge were used. It is necessary to refine these computations to determine if this gate and assembly will discharge the required 7.69 acre feet.

Step 5. Compute the average heads for each 30 minute increment or time that the tide gate is open, record these average heads in column 3, Table 9-3 and complete the table. The total discharge is 9.59 acre feet which is in excess of the requirement.

In this example it was assumed that the 1000 gpm pump would operate only during the gate-closed period, to reduce the demand on the available storage reservoir. This is the usual system employed for most tidal drainage projects. If the pump was set to operate on a continuous basis it will further reduce the volume of water that must pass through the tide gate, thereby permitting use of a tide gate with less capacity. This would reduce the cost of the tide gate installation and would increase the cost of pumping. This is seldom done as the cost of continuous pumping, over a long period of time, is usually much more than the increased initial cost for a larger tide gate installation.

Construction and Installation

Forebay channel section

The forebay channel section must have adequate capacity to deliver the design drainage discharge to the forebay and tide gate and it must also have capacity to pass the peak discharge of the tide gate. The peak discharge may be several times the drainage inflow. In case of the previous example, see column 5, Table 9-2, the peak discharge was about 10.0 cfs at 1:00 p.m., or twice the drainage inflow.

Discharge bay channel section

The discharge section of the outlet bay should be designed with ample capacity to discharge the peak flow to deep open water without causing backwater against the tide gate. Usually low foreshore banks have adequate overflow area to accommodate such discharge, however, they should be checked to insure that they do.

Table 9-3, Tide gate data sheet

(1) Time		(2) Measured Head (feet)	(3) Average Head 30 min. int. (feet)	(4) Average Discharge 30"X80' (cfs)	(5) Volume Discharge 30"X80' (acre feet)
Gate open	9:30a	0.00			
			0.18	6.23	0.26
	10:00a	0.35			
			0.55	12.80	0.53
	10:30a	0.75			
			0.92	17.75	0.74
	11:00a	1.10			
			1.27	21.30	0.89
	11:30a	1.45			
			1.64	24.80	1.03
	12:00n	1.84			
			1.92	26.80	1.17
	12:30p	2.00			
			1.95	27.25	1.13
	1:00p	1.90			
			1.81	26.00	1.08
	1:30p	1.72			
			1.54	23.85	0.99
	2:00p	1.36			
			1.16	20.10	0.84
	2:30p	0.97			
			0.71	15.20	0.63
	3:00p	0.46			
			0.23	7.30	0.30
Gate close	3:25p	0.00			
Totals		13.90	----	----	9.59

Tide gates, pipes and outlet structures

As a rule tide gate structures must be installed in wet, swampy areas and tidal flats where construction is difficult at best. The structure is usually placed at a minus elevation so that the gate functions as a submerged orifice, at all times. Structural sites and approach channels are usually excavated by dragline, which also serves as a crane for hoisting gates, pipes and other structural members in place. During construction the site is isolated from surface waters by encircling the site with spoil banks that act as temporary dikes and it may be necessary to dewater the excavation by pumping. Under very difficult conditions it may be necessary to shore the excavation with sheet piling, and encircle it with a battery of dewatering wells connected to a manifold and pump.

An outlet structure is usually necessary to protect the gate and connecting pipe from scour and floating debris. Treated timber structures, as illustrated in the standard design, Figure 9-8, have proven very practical and economical to install. This type structure can be prefabricated in whole or in part, depending upon the size, and set into place by dragline for attachment to preset piles that have been driven or jetted into the foundation. Cast in place concrete or masonry is not usually satisfactory for this kind of site unless the excavation can be kept free from salt water long enough for the cement to set.

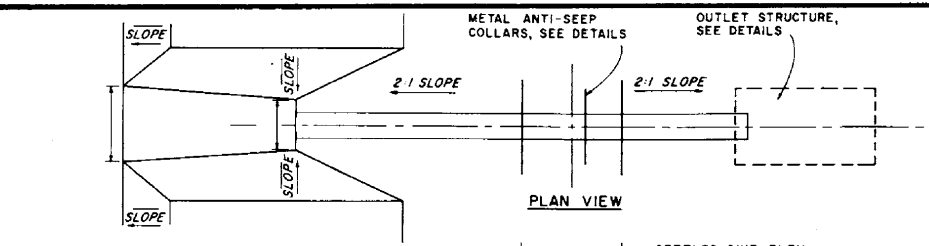
Corrugated metal pipes are commonly used in tide gate structures and must be asbestos bonded to resist salt water corrosion. The pipes should be 10 gauge or thicker to insure long life under these severe saline conditions. Experience has shown that thin gauge noncoated pipes have a very short life in brackish water.

The pipe conduit and gate, unless of very small size, should be installed as separate units. Usually, a short or stub section of conduit is fabricated to the gate and joined to the pipe after both are placed. Camber should be provided in the excavated trench bed to allow for consolidation and unequal loading by the superimposed dike. Conduit and gate must be set true to line and grade if the structure is to operate as designed. Prefabricated cast iron flap gates, usually called tide gates when used for tidal installations, have bronze bushings and are coated with water-resistant rust preventative compound at the factory and have a long life even under brackish water. Figure 9-9, shows a typical flap gate suitable for tidal drainage. This type flap gate is also available with stainless steel hardware as a further precaution against salt water corrosion.

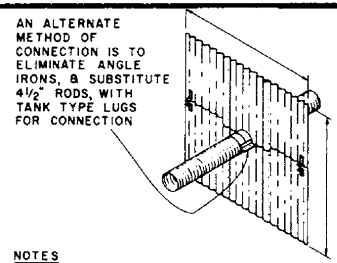
In those cases where tide gates must have a large capacity to discharge the drainage flow during the low tide period, it is usually desirable to install a battery of small gates rather than one or two large gates. As a general rule tide gates are placed so that the top of the gate is at about elevation - 1.0 feet, to provide for submergence at all times and to provide for full utilization of the available storage reservoir. These conditions dictate that the pipe and gate be placed entirely below zero elevation, which in turn imposes difficult construction conditions. From this it is obvious small diameter gates and pipe can be installed much more easily than large diameter structures. For example; five 24" gates in a battery will have about the same discharge as one 48" gate and reduce the depth of trench excavation by two feet. This two foot reduction in trench depth may materially simplify construction and more than pay for the additional cost of multiple gates.

Supplemental pump installations

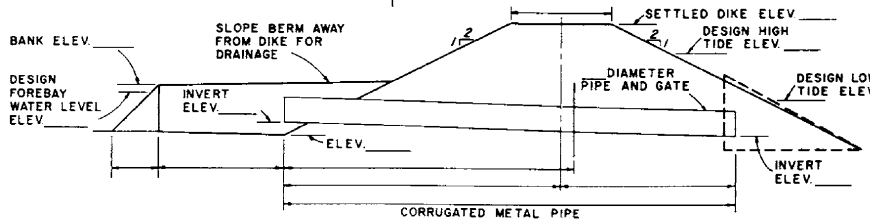
Pumps and pumping installations are covered in Chapter 7, Drainage Pumping, of this handbook. Pump installations for pumping drainage water, in connection with tidal drainage, are usually low head, high capacity type installations. In most parts of the country, excluding Alaska and coastal areas of the northeast, the tidal range seldom exceeds eight feet, except during periods of severe storms. The average pumping lift is usually small, in the range of zero to four feet and pumps selected for this condition should have a high efficiency at low head operation. Propeller and mixed flow type pumps are best suited for this situation. See Chapter 7, for a detailed discussion of pumps and structures for pumping installations.



PLAN VIEW



AN ALTERNATE METHOD OF CONNECTION IS TO ELIMINATE ANGLE IRONS, & SUBSTITUTE 4½" RODS, WITH TANK TYPE LUGS FOR CONNECTION



PROFILE ALONG CENTERLINE OF PIPE

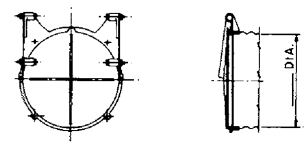
NOTES

HOLES MATCH PUNCHED IN SHOP TO PERMIT FIELD BOLTING

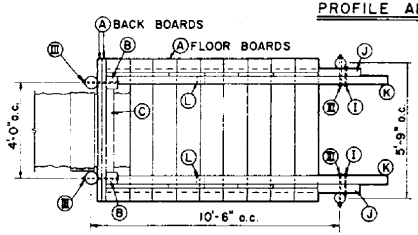
GALVANIZED BOLTS TO BE FURNISHED WITH DIAPHRAGM

LAP BETWEEN TWO SECTIONS TO RECEIVE EXTRA BITUMINOUS COATING AT TIME OF ASSEMBLY.

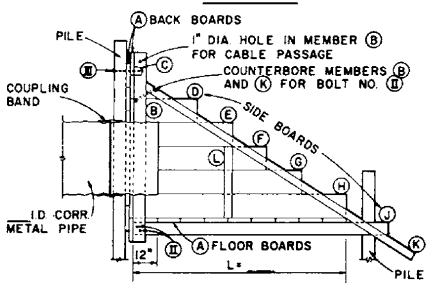
DIAPHRAGM TO BE FULLY BITUMINOUS COATED



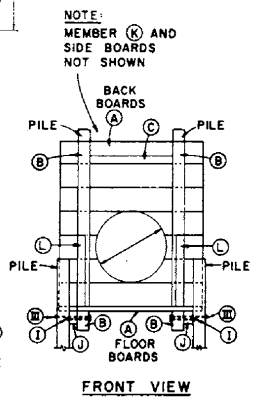
TYPICAL FLAP GATE



PLAN VIEW



PROFILE ALONG CENTERLINE

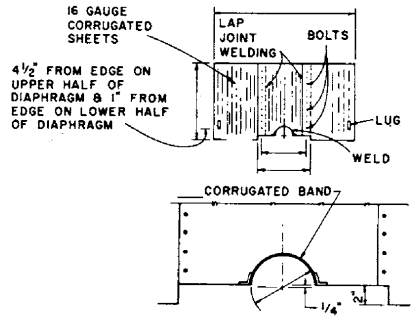


FRONT VIEW

TIMBER OUTLET STRUCTURE DETAILS

GENERAL NOTES

1. BACK BOARDS TO BE NAILED TO MEMBER (B) AND NAILED TOGETHER WITH NAILS AT 6" O.C.
2. SIDE BOARDS TO BE NAILED TO MEMBERS (B) AND (K).
3. FLOOR BOARDS SHALL BE NAILED TO MEMBER (J).
4. NAILS SHALL BE PAINTED OR COATED.
5. ALL NAILS AND BOLTS WILL BE GALVANIZED.
6. MINIMUM LENGTH OF PILES SHALL BE 2 TIMES THE LENGTH OF MEMBER (B).
7. TIP END OF PILES SHALL BE A MINIMUM OF 6" DIA.
8. PILES SHALL BE DRIVEN OR JETTED IN PLACE.
9. PILES AND TIMBER SHALL BE CREOSOTE TREATED.
10. L = LENGTH OF FLOOR.
11. ASBESTOS BONDED BITUMINOUS COATED CORRUGATED METAL PIPE SHALL BE USED.
12. WATER TIGHT COUPLING BANDS SHALL BE USED AT ALL PIPE JOINTS.
13. TIMBER QUANTITIES BASED ON DRESSED SIZES.
14. FLAP GATE SHALL BE ARMCO MODEL 10-C OR APPROVED EQUAL, WITH BRONZE BUSHINGS, BOLTS AND HINGE BARS



ANTI-SEEP COLLARS

NOT TO SCALE

Figure 9-8, Drainage flap gate installation

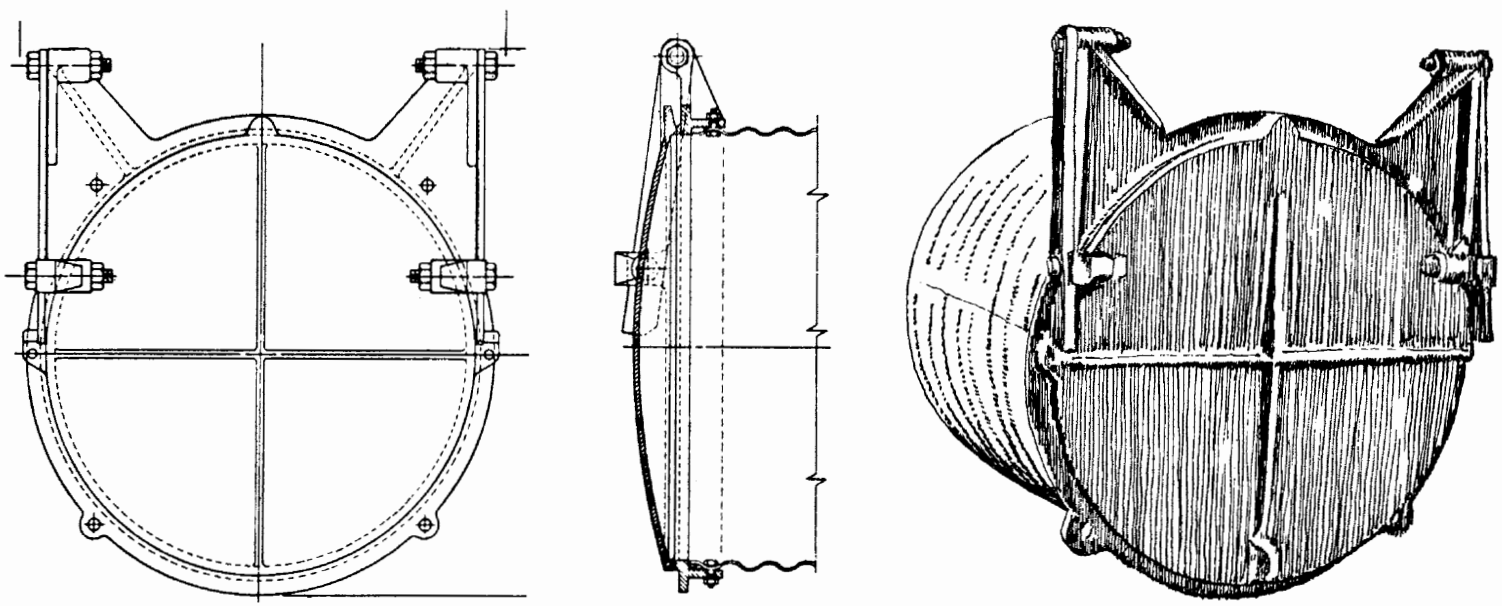


Figure 9-9, Typical low-head tide gate

As previously mentioned, pumps used in tidal drainage situations are usually operated only during the gate-closed period, to reduce the storage requirement. They can be operated during the gate-open period also; however, this is seldom done as the only benefit is to reduce the load on the tide gate permitting use of a smaller gate. Generally, the cost of pumping during this gate-open period, for the life of the project, is far in excess of the cost of a larger gate.

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