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RAPID CHANGES OF FLOW IN AN IRRIGATION RESERVOIR

BY

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S U M M A R Y

The paper deals with the problem of rapid changes of flow in an irrigation reservoir due to the inflow at the upstream end of a substantial flood for various tidal levels in the estuary downstream of the reservoir. The reservoir which was essentially the river channel to the bank full stage was modelled as a single open channel 8½ miles long with a known flood hydrograph at the upstream end and a gated weir at the downstream end.

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1. INTRODUCTION

This paper deals with the problem of rapid changes of flow in an irrigation reservoir constructed in a tidal estuary. It was proposed that an irrigation reservoir be created by construction of a weir across a creek entering the estuary and that fresh water be stored upstream of this weir in the existing river channel. The problem was to determine the height of water in the reservoir when a flood passed through it, taking into account various tidal levels in the estuary downstream of the weir, and hence determine if the adjacent low-lying land was flooded.

The reservoir was modelled as a single open channel 8½ (44,800 feet) miles long. At the upstream end of the reservoir a known flood hydrograph was injected and at the downstream end there was a gated weir which was subject to submergence by tides on the downstream side. (Figure 1).

2. FLOOD ROUTING

The method of flood routing adopted was the solution of shallow water wave equations by the method of characteristics. This method requires the selection of a constant such as Manning's n to represent the roughness of the channels and a detailed knowledge of the geometry of the channel.

The flow examined in the current study was entirely within the normal river channel. The channel was fairly regular with only a small amount of meandering. The channel or reservoir was assumed to be trapezoidal.

3. THE METHOD OF CHARACTERISTICS FOR FLOOD ROUTING

The equations of continuity and momentum for unsteady flow (or shallow water wave equations) can be solved by several different numerical methods. In this case the numerical method known as the method of characteristics was chosen. Liggett and Woolhiser (1) have carried out a comparison between this method and several others and conclude that this method is stable and relatively rapid.

The computer program used for this study used the basic numerical techniques developed by Streeter and Wylie(2). The program has been verified and used for a variety of unsteady flow problems.

4. BOUNDARY CONDITIONS

In order to obtain sufficient equations to solve for all unknowns, the depth, velocity, discharge or some relationship between depth and discharge must be defined at both ends of the channel.

The two boundary conditions used here were:-

1. An inflow hydrograph at the upstream end of the channel.
2. A stage/time relationship at the downstream end of the channel.

For the current problem, the first boundary condition was used to specify the 1 year flood hydrograph which was fed in at the upstream end of the reservoir. The second boundary condition was included in a special subroutine written to specify the conditions at the weir.

The gated weir at the downstream end of the channel can act either as a weir or an orifice depending on whether the water level upstream of the gate is below the lower edge of the gate. (Figure 2 a & b). Appropriate weir and orifice equations were developed in terms of the depths y_p, y_w, H, h_s & h_q (Figure 2).

The criterion for which equation to use depends on whether the end condition is an orifice or weir and this is governed by whether the upstream water surface is or is not touching the bottom of the gate.

5. NUMERICAL TESTS

In most of the tests the channel was subdivided into 17 reaches of length 2,640 feet. The total length of channel was about 8½ miles. A shorter reach length of 1,320 feet was used as a check on the accuracy on the longer reach length but most of the tests were done using the longer reach length and shorter computing time. Manning's n was selected as 0.03 or 0.035.

The geometry of the river was assumed to be trapezoidal. For the downstream 5.5 miles the bottom width was 190 feet and the side slopes 1 to 2.5. For the upstream 3 miles the bottom width was 146 feet and the side slopes 1 to 2.38.

All tests were carried out with a full reservoir at the start and with an initial dry weather discharge of approximately 100 cubic feet per second. The 1 year hydrograph was specified at the upstream end.

The operating conditions specified that the gates were to open when the water level at a point 35,000 feet upstream rose 1 foot. This was found to occur at approximately 2.25 hours and gate opening was then made to change from almost zero at this time to 10 feet, 15 minutes later. A 3 feet sinusoidal tide was assumed, and the time of high tide was varied from test to test.

6. RESULTS OF COMPUTATION

Typical results of tests are presented in Figures 3 and 4 which depict a stage and discharge hydrograph at the weir and at a point 35,000 feet upstream for two tidal conditions (i.e., no tide and high tide at 2.25 hours). Also included is the input hydrograph at the upstream end. The depth hydrographs are in terms of the depths of water above the channel bottom.

At the weir, the discharge and depth remain approximately constant up to the time at which the gate starts opening. The initial discharge is the dry weather discharge of approximately 100 cubic feet per second. In the case where the effect of tide is neglected (Figure 3), the discharge rapidly increases to approximately 8,000 cubic feet per second at which discharge the hydrograph is a sharply peaked maximum. The depth reduces very rapidly at this time. For a high tide at 2.25 hours (Figure 4) the effect of submergence of the weir means that a lower discharge occurs. In both cases the discharge remains fairly constant but then increases as the flood hydrograph arrives at the weir. After the peak, the hydrograph tails off in the normal fashion.

The discharge hydrograph at a distance 35,000 feet upstream initially rises in response to the input hydrograph even though the discharge at the weir is restricted to 100 cubic feet per second. There is a resulting increase in depth at all points along the channel to account for the increase of volume of water to be stored. The effect of the gate opening is not felt at this point until approximately 20 minutes after opening commences. Then the discharge increases and the depth decreases, though not to such an extent as at the weir because of attenuation and to the effect of the input hydrograph. The peak of the discharge hydrograph is reached at 4.2 hours and the depth hydrograph at 4.75 hours. The peaks of the depth and discharge hydrographs at the weir are concurrent at 5.25 hours since the weir possesses a single valued rating curve. In Figure 3 the dotted line indicates the depths that would have occurred at a distance 35,000 feet from the gate if the gate had not been opened.

The ability of the method of characteristics to handle sudden changes in discharge has been investigated by Martin and de Fazio (3) who found that the results produced were surprisingly accurate. However, the sudden change of discharge in this investigation from 100 cubic feet per second to 8,000 cubic feet per second in less than 10 minutes is not likely to have been calculated accurately. A test was conducted with a reach length of 1,320 feet (i.e., half of the original value) and hence correspondingly smaller time intervals and it was found that the peak discharge which was only 129 cubic feet per second greater than that shown in Figure 3 for the corresponding test. The effect of opening the gate very rapidly while the reservoir is full produces a very large increase in discharge at the weir and it is concluded that the method solves this situation satisfactorily.

The downstream boundary conditions had a considerable effect on the results. The high tide caused the weir to be partially or wholly drowned out and in some tests the discharge occasionally became zero because of the submergence of the weir.

The stage at a distance of 35,000 feet upstream of the weir was of interest because this was the lowest point in the bank. If the water level exceeded the height of the bank, flooding of the adjacent land would occur. Tests were conducted with high tide at different times and it was found that this depth ranged over 0.1 foot.

7. CONCLUSIONS

The method of characteristics has been used here to show the way in which a known flood effects the depth in the reservoir under various rapidly varying gate conditions.

References:

1. Liggett, J.A. and Woolhiser, D.A., "Diference Solutions of the Shallow-Water Wave Equation," Proc. ASCE, J. Eng. Mech. Divn., Vol. 93, No. EM2, pp 39-71, 1967.

2. Streeter, V.L. and Wylie, E.B., "Hydraulic Transients", McGraw Hill, 1967.

3. Martin, C.S. and De Fazio, F.G., "Open Channel Surge Simulation by Digital Computer", Proc. ASCE, J. Hydraulics Divn., Vol. 95, No. HY6, pp 2049-2070, 1969.

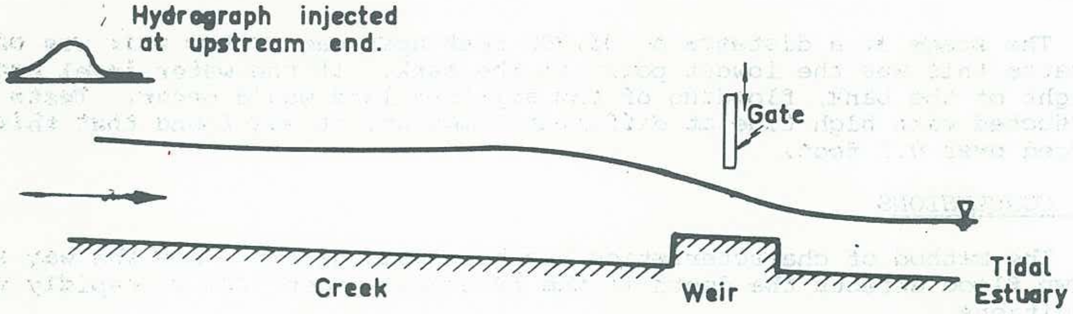
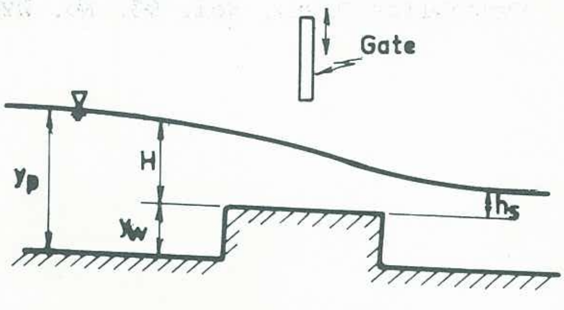
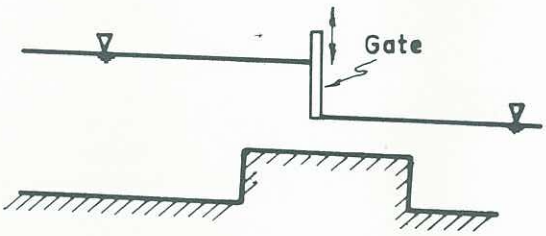


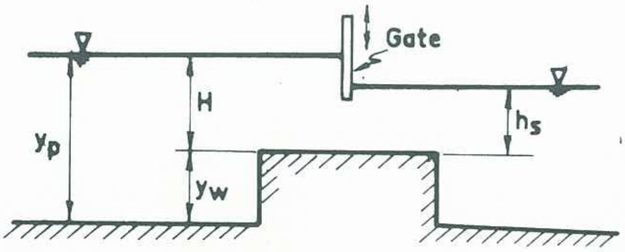
Fig 1 DIAGRAMATIC REPRESENTATION



a. Broad crested weir.



b. Orifice



c. Submerged orifice

Fig 2 DOWNSTREAM BOUNDARY CONDITIONS

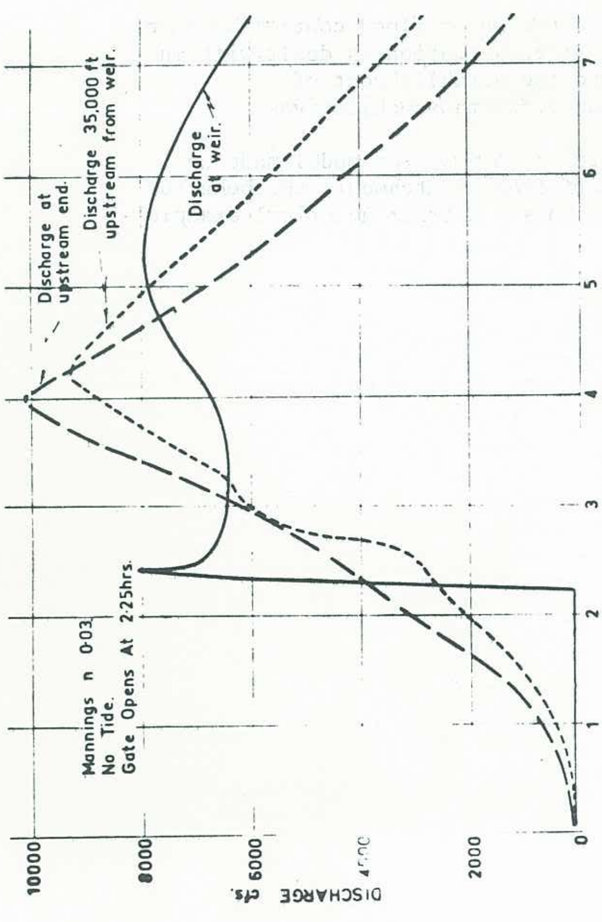


Fig 3a DISCHARGE HYDROGRAPH

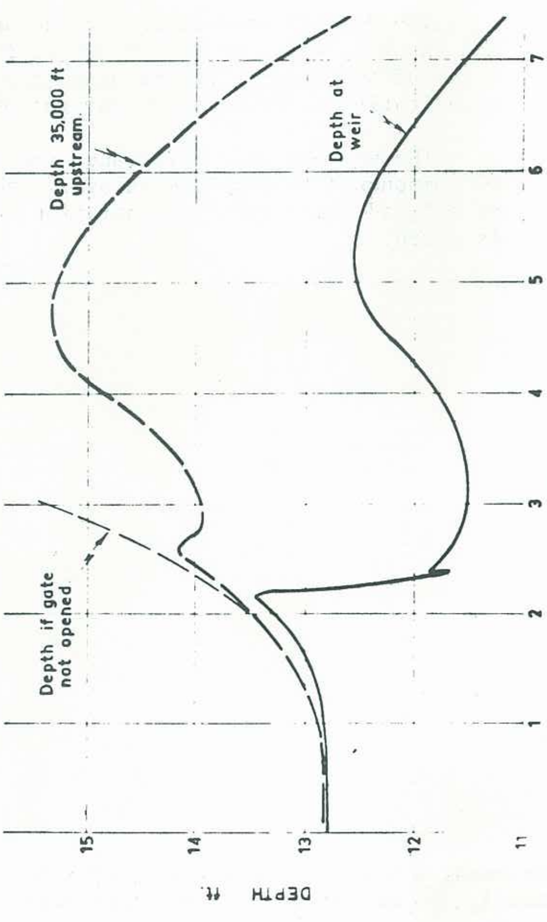


Fig 3b DEPTH HYDROGRAPH

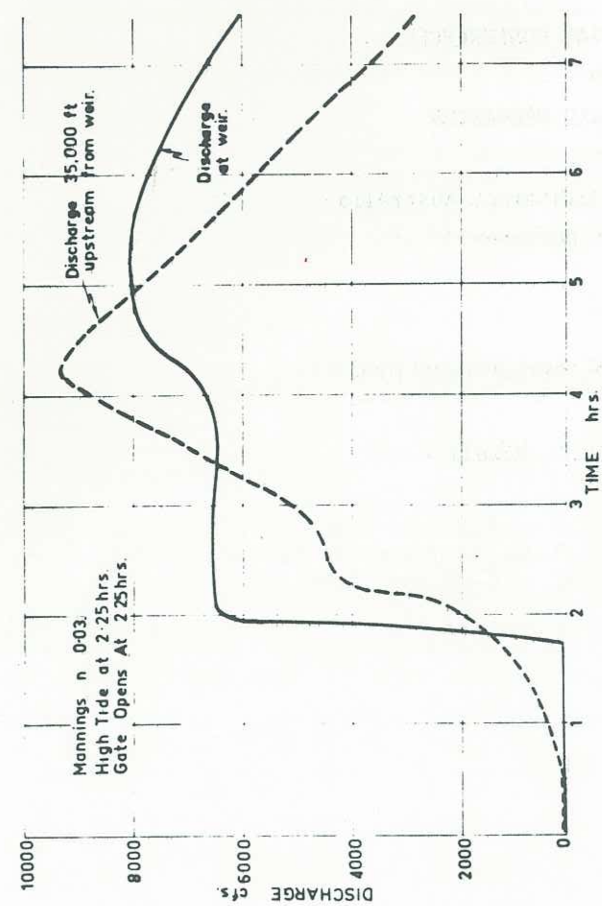


Fig 4a DISCHARGE HYDROGRAPH

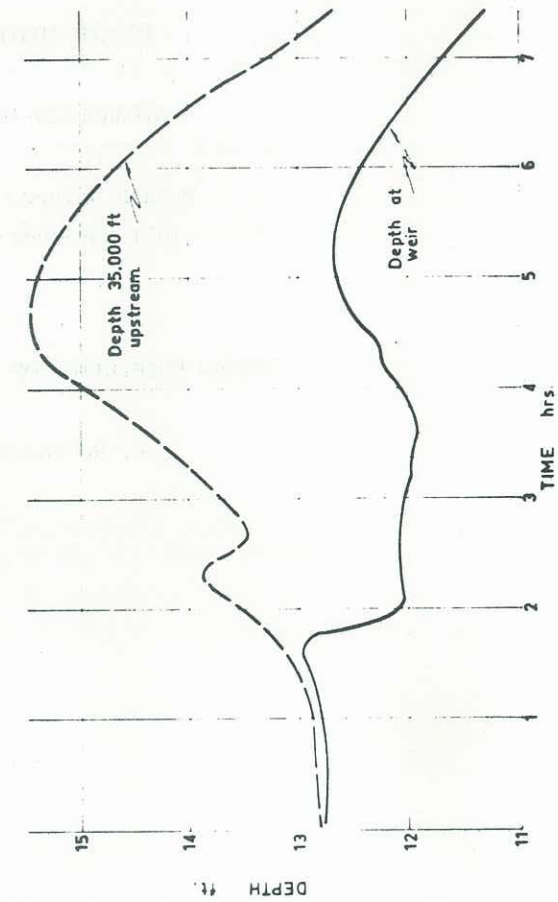


Fig 4b DEPTH HYDROGRAPH