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Characteristics of Power Station Surges in a River — An Hydraulic Model Study

I. G. HAMPTON

Engineer, Fluid Mechanics Branch, Snowy Mountains Engineering Corporation

D. C. GREEN

Engineer-in-Charge (Dams), Civil and Architectural Department, State Electricity Commission of Victoria

and

E. J. LESLEIGHTER

Head, Fluid Mechanics Branch, Snowy Mountains Engineering Corporation

SUMMARY The paper describes an hydraulic model study of transient flow in the Mitta Mitta River (Victoria) downstream of the Dartmouth Power Station under conditions of very rapid load acceptance by the turbo-generator, resulting in increases in discharge from zero to 124 m^3s^{-1} in 8 s. The speed of travel of the disturbance and the change of water level were monitored at various points. It was found that under natural river conditions the disturbance was propagated as a well defined surge front or wave with relatively little attenuation in the stretch of river concerned, because the speed of propagation of the initial discharge was sensitive to frictional effects while that of the peak discharge was more influenced by pool storage. The rate of rise of the water level was found to be high enough to constitute a hazard to users of the river but it was also found that this could be reduced to acceptable limits by using control structures at critical sections.

1 INTRODUCTION

In many cases adequate attenuation of the effects of rapid increases in hydro power station discharges is facilitated by nearby tailwater storages. In the 150 MW Dartmouth Power Station case, the valley is very narrow close to the power station, and to obtain adequate storage the regulating dam will be 8.6 km downstream. Investigation of the surge problem from the power station was considered essential by the project's owners the State Electricity Commission of Victoria (SECV) with a view to determining what means were required to mitigate the potential hazard to the public due to a rapidly rising river when the power station started up. As part of the investigation a physical hydraulic model study of this reach of the river was undertaken in the Fluid Mechanics Laboratory of the Snowy Mountains Engineering Corporation (SMEC). The investigation covered a range of power station operating conditions, and a range of measures to alleviate the severity of surges. This paper describes some of the significant hydraulic behaviour exhibited by the surges. All units refer to the prototype unless otherwise stated.

2 MODEL DESCRIPTION

The extent of the model which included the 8.6 km reach of the river between the power station tail-bay and the regulating dam is shown on Figure 1. The Mitta Mitta River in the area of interest consists of a series of pools and rapids so that the flow regime passes from subcritical to supercritical at the control points at the upstream ends of rapid sections. This is depicted in the model invert profile in Figure 2.

The model was constructed to a horizontal linear scale of 1:70 and a vertical linear scale 1:50. Other scaling was according to Froude similarity. The model thus incorporated a distortion of 1.4 in order to improve the accuracy of water level measurements, and increase model discharges and Reynolds numbers. Model construction was based on contour plans, the 16 surveyed cross sections numbered in Figure 1, site inspection and large-scale coloured aerial photographs.

Both rapid starting and slower starting of the power station were simulated by manipulating valves to turn into the model a steady flow which had been established in a separate channel before the start of each test.

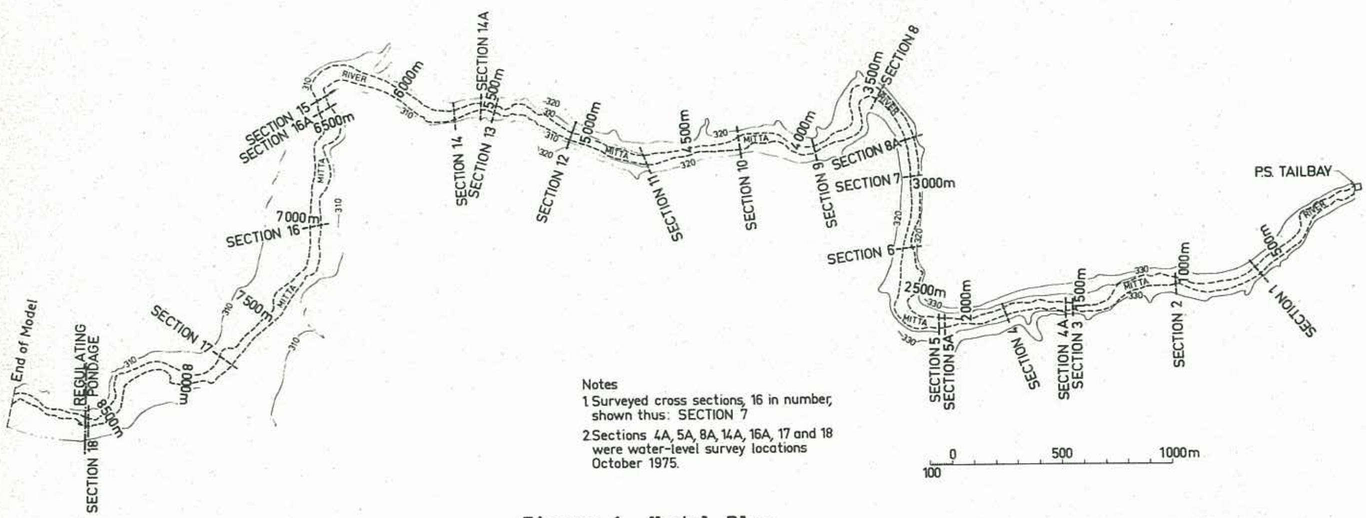

Figure 1 Model Plan

Water surface levels were observed from staff gauges installed at various locations along the model. In addition capacitance wave probe stations were set up at 17 positions along the model river. The wave probes provided a voltage output varying linearly with the water depth against them. During each test run the outputs from the wave probes were recorded on an FM tape recorder. Manual recording of start and finish water levels allowed correlation with wave probe outputs to establish datum and calibration, and subsequently the outputs from the tape recorder were used to generate plots of the water surface level versus time on an XY plotter.

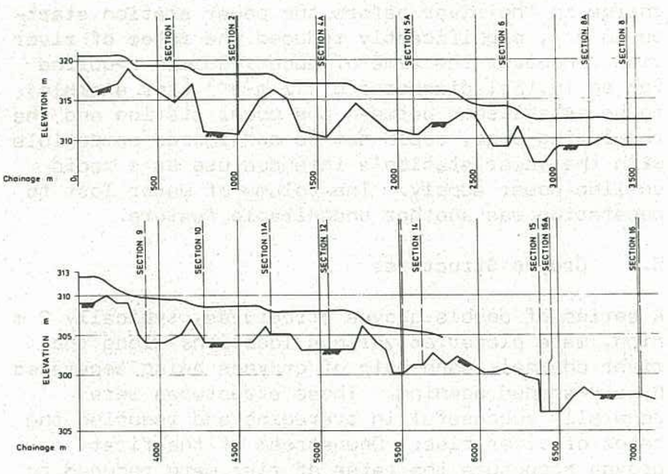

Figure 2 Invert profile showing water surface profile for discharge 124 m^3s^{-1}

3 VERIFICATION OF THE MODEL

The operating range of discharges from the power station was 0 to 124 m^3s^{-1} and, as the data provided by the SECV included surveyed water levels for discharges in the ranges 12 m^3s^{-1} to 19.5 m^3s^{-1} and 80 m^3s^{-1} to 136 m^3s^{-1}, an almost ideal situation prevailed for reliable verification of the model. The expected model roughness height required to reproduce prototype friction was deduced from the observed water surface slope between Sections 11A and 12 (Figure 1) on 13 October 1975 for which the rated discharge in the river was 120 m^3s^{-1}.

This roughness height, nominal 20 mm aggregate (model), was used as the basis of roughness modifications in the model during the verification procedure. The model proving process was carried out as a trial-and-error process of matching model to prototype water surface levels for known, steady discharges.

4 INITIAL TESTING: RAPID INCREASE IN POWER STATION DISCHARGE INTO NATURAL RIVER

The initial series of tests involved simulation of rapid load acceptance at the power station. The most severe case being an increase in power station discharge from 0 to 124 m^3s^{-1} (the maximum power station discharge) in 8 s into a non-flowing river. The plotted water surface versus time results for this test are depicted in Figure 3.

The maximum rates of rise of the water surfaces given in the following text were obtained by drawing a straight line tangent to the steepest portion of each curve.

The most significant features in these tests were:

(i) The formation of a small gravity wave on the surge front and a reflected wave in the first pool only downstream of the power station tailbay. The enlarged-scale plots of conditions at Chainage (Ch) 70 m and Ch 240 m in Figure 3 show the evidence of this oscillatory wave. The alternating steep and flat sections appear to relate to the wave travelling along the pool in the direction of flow and being reflected, in an upstream direction from the change of section or direction. Part of this wave at the time of reflection also spilled over the high point at Ch 310 but was dissipated in the dry rocky bed downstream before water reached the next pool at approximately Ch 430.

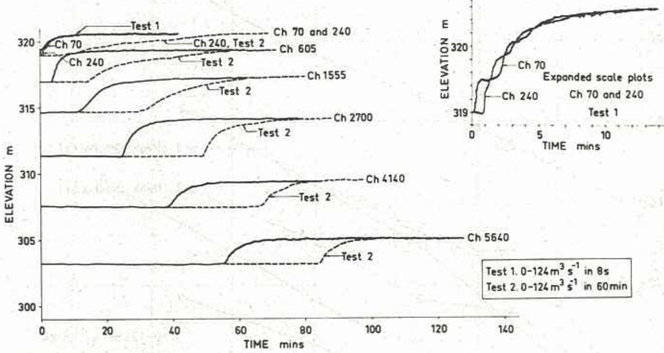

Figure 3 Water surface elevations vs time

(ii) The speed of propagation (celerity) of the initial disturbance was high(er) along the pools and low in dry bed portions downstream of the various high points. The time of arrival of the surge's initial disturbance and its peak discharge at points along the river for some of the cases discussed in this paper can be seen in Figure 4. The celerity of the initial disturbance along the pools was found to approach the value given by the normal shallow water equation

$$c = (gy)^{\frac{1}{2}} \tag{1}$$

where c is the celerity of a small disturbance in a uniform channel of depth y, and g is the acceleration due to gravity. This is illustrated in Table I for two areas where there were no sections of dry bed. The theoretical celerity in each case was based on the depth above the average invert level.

TABLE I

CELERITY OF INITIAL DISTURBANCE

REACH OF RIVER	SPEED OF PROPAGATION (ms^{-1})	
	Observed	Theoretical
Ch 70 to 240 m	4.8	5.0
Ch 3915 to 4140 m	4.3	5.2

By comparison the celerity across the initially dry bed portions was very low, with the result that the average speed from Ch 70 to Ch 4140 was 1.8 ms^{-1}. If the existing sections of dry bed were ignored and an overall average depth computed, the theoretical speed from Ch 70 to Ch 4140 was 4.6 ms^{-1}.

(iii) The maximum rates of rise at various points along the river are shown in Table II. In general the time rate of rise characteristics in each pool were determined mainly by the restriction to flow at the high level control point at the downstream end and the pool surface area (storage). These also controlled the rate of increase of discharge into the next pool.

(iv) With the influence of storage routing of the surges it would be expected that the rates of rise would diminish as the surge moved downstream. However, while attenuation did occur in the first 1500 m as depicted by the divergence of lines '1' on Figure 4, this process did not continue further downstream. Indeed, the physical features of pool and control point widths, and the surge-steepening characteristics associated with 'dry' rapids had such a marked effect that higher rates of rise of levels and discharge occurred at some points downstream compared with points upstream. For example, the rates of rise at Ch 2700 were significantly greater than those at Ch 1550 and Ch 2110.

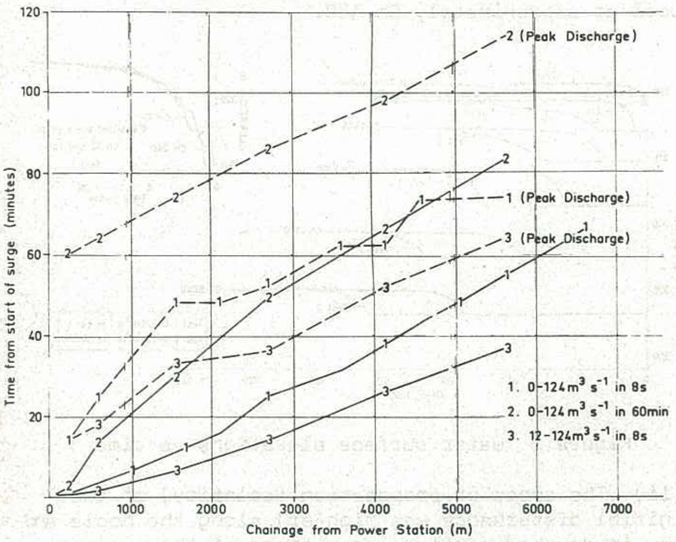

Figure 4 Progress of initial disturbance and peak discharge

5 REMEDIAL MEASURES

The above tests showed that when the power station started rapidly and discharged into an initially non-flowing river the resulting rates of river level rise were sufficiently high to be judged a potential hazard to users of the river, being over one metre per minute in the first 600 m below the power station. 'Remedial' tests were carried out with the aim of reducing the rates of rise to 0.3 m per minute or less. In an attempt to achieve this by either reducing the local rate of increase of discharge or by increasing the local surface width five basic approaches were adopted as discussed below.

5.1 Slower Power Station Start

Variation of the start-up time of the power station from 15 s to 60 s was found to have no appreciable effect on rates of rise of the water surface compared to the rates achieved for the 8 s start-up. Even varying the start-up time to full discharge in 60 min caused no appreciable changes at some downstream locations, and indeed resulted in a slightly higher rate of rise at a location 5640 m downstream of the power station.

Further testing established that start-up would have to be gradual over 220 min for the rate of rise to be kept below 0.30 m min^{-1} at all gauged locations.

5.2 Initial Discharge

It was found that establishing a small steady discharge in the river before the power station started in 8 s, significantly reduced the rates of river rise. However the time of about 150 min required for an initial discharge of 12 m^3s^{-1} (for example) to be established between the power station and the regulating pond, could not be considered compatible with the power station's intended use as a rapid on-line power supply. The volume of water lost to generation was another undesirable feature.

5.3 Groyne Structures

A series of double groyne structures, typically 2 m high, were placed at various locations along the river channel, each pair of groynes being separated by a V-shaped opening. These structures were generally successful in averaging and reducing the rates of river rise. Downstream of the first groyne structure the rates of rise were reduced to a maximum of approximately 0.25 m min^{-1}. Upstream of the first structure the rate of rise was increased due to its flow restricting effect and the total rises upstream of the structures were of course increased.

5.4 Secondary Regulating Dam

Placement of a secondary regulating dam downstream of the power station, with a low level free-draining culvert to control the releases into the reach of the river upstream of the main pondage, was an effective solution for the reach downstream only. Rates of rise tended to remain high upstream of such a dam.

5.5 Impermeable Banks

Impermeable banks of different heights were placed at selected locations along the river. These arrangements resulted in the initial disturbance travelling down river very rapidly, and the rates of rise were much reduced. The best results were obtained with banks approximately 6 m high at Ch 2500, 4650 and 6000. In this case the reach of

TABLE II

MAXIMUM RATES OF RIVER RISE FOR NATURAL RIVER AND REMEDIAL TESTS
(All results for discharge 0-124 m^3s^{-1} in 8 s unless otherwise stated)

CHAINAGE (m)	70	600	1555	2700	4140	5640
TEST	MAX. RATE OF RIVER RISE (m min^{-1})					
1. Natural Non Flowing River	1.75	0.88	0.29	0.75	0.36	0.40
2. Slow Start 60 min		0.21	0.18	0.57	0.39	0.48
2A. Slow Start 220 min		0.08	0.09	0.23	0.15	0.28
3. 12 m^3s^{-1} for 157.5 min, then 12-124 m^3s^{-1} in 8 s		0.44	0.21	0.19	0.09	0.11
4. Groynes at Ch 710, 1230, 2413, 3670, 4667, 5270		0.91	0.25	0.28	0.23	0.26
5. Secondary Regulating Dam Ch 1790, spillway 8 m above river bed		1.11	0.21	0.14	0.06	0.11
6. Secondary Regulating Dam Ch 2760, spillway 7 m above river bed		1.04	0.36	0.20	0.10	0.17
7. Impermeable banks at Ch 2500, 4650, 6000		GW*	0.43	0.35	0.09	0.04

*GW Gravity wave, small very rapid rise

TABLE III

CELERITY OF INITIAL DISTURBANCE AND PEAK DISCHARGE

TEST	CELERITY CH 240 TO CH 5640 (ms^{-1}) OF			
Power Station Discharge Characteristic	Initial Disturbance	% Diff.	Peak Discharge	% Diff.
1. 0-124 m^3s^{-1} in 8 s, natural river	1.66		1.45	
2. 0-124 m^3s^{-1} in 60 min, natural river	1.10	-34	1.67	+15
3. 12-124 m^3s^{-1} in 8 s, natural river	2.49	+50	1.80	+24
4. 0-124 m^3s^{-1} in 8 s, groyne structures	1.22	-27	1.05	-28
7. 0-124 m^3s^{-1} in 8 s, impermeable banks	3.57	+115	1.55	+ 7

river was permanently drowned and the waterway was significantly wider. The rate of rise was typically reduced to 0.1 m min^{-1} or less, but reached approximately 0.4 m min^{-1} at Ch 1555 and 2700.

5.6 Comparison of Remedial Measures

The relative effectiveness of the above remedial measures in reducing the rates of river rise are shown in Table II.

6 CHARACTERISTICS OF SURGES OBSERVED DURING REMEDIAL TESTING

Section 4(iv) mentioned the steepening effect of initially dry bed and shallow sections of the river channel resulting in non-attenuation of the surge downstream of Ch 1500. In Test 2 involving a linear increase in discharge over 60 min, the surge steepened as it travelled downstream. For example, at Ch 605 it took 51 minutes for the discharge to increase to 124 m^3s^{-1}, while at Ch 5640, 0-124 m^3s^{-1} took 30 minutes. The progressive steepening of the surge can be seen on Figure 3, and is shown by the convergence of the initial and peak discharge curves on Figure 4. The 'catching up' of the initial small discharge disturbance by later released higher discharges is illustrated in Table III which lists the celerities of the initial disturbance and peak discharge for a number of test cases.

The initial disturbance was found to have a much higher celerity over an already wet river bed. For example, in Test 3 of the table a steady discharge of 12 m^3s^{-1} was established along the river before being increased rapidly to 124 m^3s^{-1}. The increase in average depth in the pools due to a discharge of 12 m^3s^{-1} was from 3 m to 4 m, thereby increasing the theoretical celerity of a disturbance in those areas from Equation 1 by 17%. The overall increase of 50% compared with Test 1 demonstrates the markedly higher celerity in rapids areas having an initial discharge.

While the celerity of the initial disturbance was strongly dependent on initial depth it can be seen from Table III and Figure 4 that the celerity of the peak discharge (wave peak) was relatively insensitive to this, and to the discharge versus time curve and steepness of the wave at the upstream end of the reach. The celerity of a kinematic wave, that is, a wave whose properties follow principally from the equation of continuity is given by

$$c = \frac{dQ}{dA} \qquad (2)$$

where Q is discharge and A the area of flow. Comparison of celerities of the wave peaks for Tests 1 and 2 with the theoretical celerity from Equation (2) shows good agreement for Test 2 for the whole reach and for Test 1 downstream of Ch 1700. That is, for the most part the progress of the wave peak is approximately according to river storage, which in this river is mainly pool storage upstream of the form control sections. On the other hand, the celerity of the initial disturbance is principally influenced by the significant friction in the shallow rapids sections. The groyne structures mentioned in Section 5.3, by providing more restrictive form controls upstream of locations where high rates of rise occurred in the natural river, may be seen in the context of reducing the celerity of the wave peak by creating more storage. However, due to slower rates of increase of discharges onto the dry rapids the celerity of the initial disturbance was reduced in these areas and the wave steepened again, so that while in some locations rates of rise were reduced, in others they were increased.

The impermeable banks mentioned in Section 5.5 by drowning the dry rapids areas had the effect of increasing the celerity of the initial disturbance by 115% while increasing that of the wave peak by only 7%. Combined with greater water surface widths resulting in small increases in depth for a given increase in discharge this approach was successful in reducing the rates of river rise to acceptable figures.

7 CONCLUSIONS

The model study demonstrated the rates of river rise which could be expected in the natural river due to rapid starting of the Dartmouth Hydro-electric Power Station, at various locations between the power station and the regulating pond. At some locations these rates of rise were quite high, being of the order of 1 m min^{-1} or greater, in the first 600 m below the power station.

The progress of a surge along a natural river consisting of a series of pools and rapids is a complex phenomenon; for example, contrary to what may have been expected, it was shown that a wave front may not reduce in steepness as it progresses, but may actually increase in steepness.

The study also showed the relative importance of frictional and form control effects on the progress of a surge. This in turn enabled an evaluation to be made of various measures for reducing the rates of river rise. Measures involving changes to power station starting procedure were effective but were operationally undesirable. Measures involving control structures at critical points in the river were also successful to varying degrees, but were generally not capable of reducing the rate of river rise significantly in the first 600 m or so.

8 ACKNOWLEDGMENT

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