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JINNAH HYDROPOWER PROJECT
DEVELOPMENT OF SURGE DUE TO SUDDEN SHUTDOWN

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ABSTRACT

Jinnah Hydropower Project is presently under construction, on the right side of the Jinnah Barrage on river Indus. It is a 96 MW power station, fed from Jinnah Barrage Pond through a power channel with discharge capacity of 2360 m^3/sec. To ascertain the adequacy of freeboard in the power channel, the development of surge has been simulated in the case of sudden shutdown of the power plant. For this purpose, the nonlinear partial differential equations (continuity & momentum) for gradually varying flow in prismatic channel have been used. For initial condition, the normal flow was specified in the power channel, whereas a constant head boundary was assumed at the upstream end and a linearly decreasing discharge at power plant (downstream boundary) over the specified shutdown period. The numerical solution of the transient flow problem was obtained using finite difference technique with implicit method. The results show that for closure period of 15 seconds, the height of surge above the normal water level would be about 1.8 metre, and its speed in the upward direction would be 8.03 m/sec. The speed and height of surge compares favourably with that computed using an analytical solution by reducing the moving surge phenomenon to a stationary surge or hydraulic jump. The freeboard of 3.5 m as provided in the power channel would be sufficient to contain the surge within channel prism without over topping. However, since the speed of surge is nearly 4 times the normal velocity in the channel, the side slope stone protection designed to cater for normal flow condition is likely to be dislodged, particularly in the upper part of channel prism. Therefore, the design of stone protection may have to be re-examined for stability against the action of high surge speed.

1. INTRODUCTION

Jinnah Hydropower project is located on the right side of the Jinnah Barrage on the Indus River approximately 5 kilometers downstream of the town of Kalabagh in the district of Mianwali, location map in Figure-1. The optimized maximum power station output is 96 MW, corresponding to a designed maximum discharge of 2360 m^3/s at a mean head of 4.7 m. The optimized project will produce 688 GWh/annum. The construction of project started in 2006 and is expected to be completed in 2011. The total project cost is US$ 128 million.

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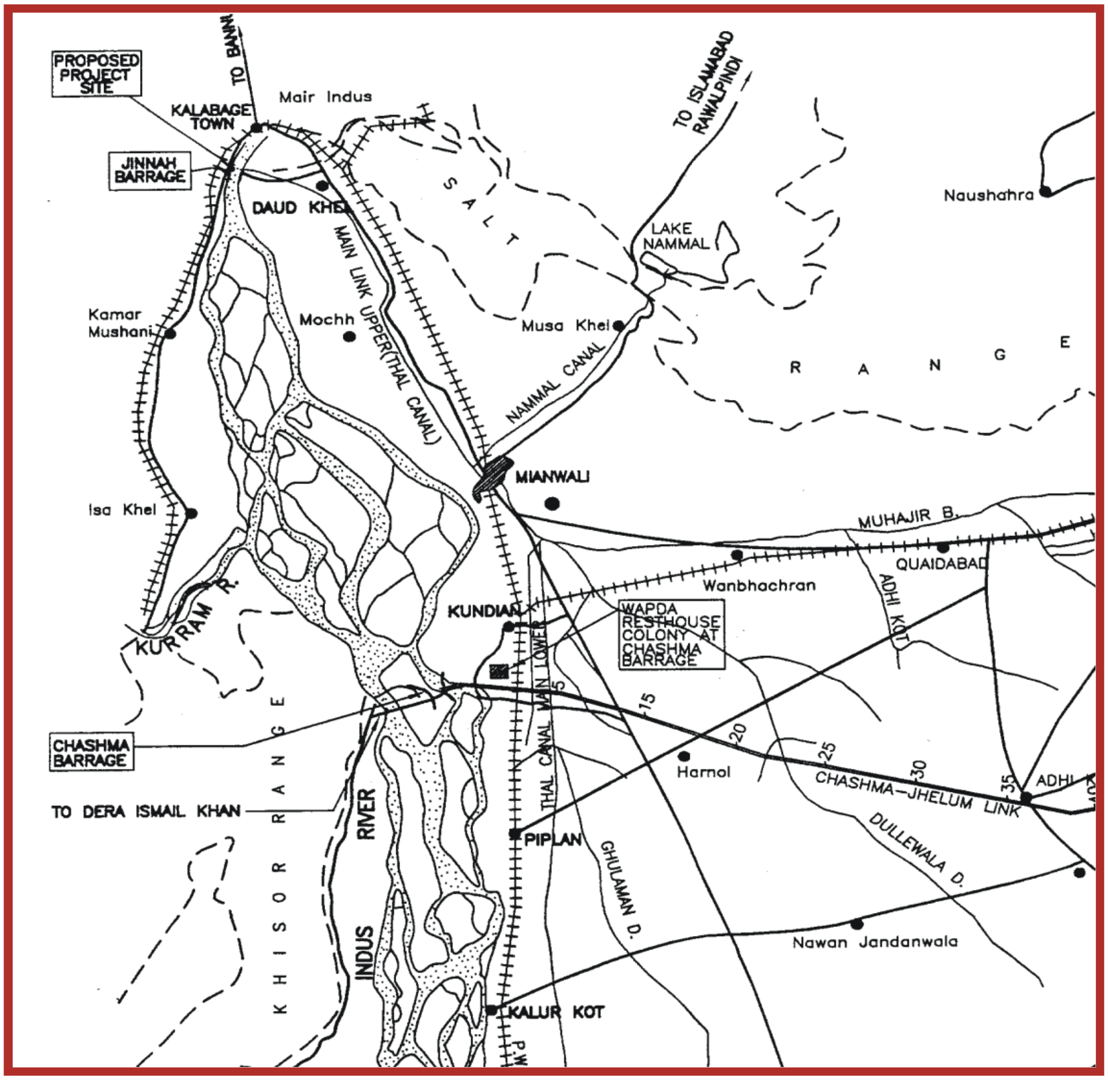


Figure-1: Geographical location of the Project

In the case of sudden shutdown of the power house, a surge of water shall travel from lower end of power channel towards the upper end, to be damped by channel resistance and the constant water level maintained in the barrage pond.

To ascertain the adequacy of freeboard and stability of stone protection on side slopes in the power channel, the development of surge has to be simulated in the case of sudden shutdown of the power plant. This problem has been analysed using an implicit finite difference technique applied to governing equations of the flow.

2. PROJECT LAYOUT

The headrace will off-take from right guide bank about 1300 m upstream of Jinnah Barrage (Figure-2). Near the inlet the head race channel has curved alignment and then becomes straight for about 730m upstream of powerhouse.

The maximum design discharge of the power channel is taken as 2360 m^3/s. The bed width is taken equal to the total width of 8 units which is 134 m. The maximum pond level is 211.50 m.a.s.l. The top level of embankment has been fixed at 215.00 m.a.s.l which is the same level as that of the existing guide bank. The freeboard provided in channel is 3.5 m. The side slopes of the power channel are 2.5:1. The bed level for the headrace channel has been fixed at 202.50 m.a.s.l to have a flow velocity in the range of 0.5 m/s to 2.0 m/s. The tailrace having a length of 500 m will take the powerhouse out flows to Indus River.

The powerhouse consists of 4 turbine blocks and one service bay. Service bay is provided on the left side of the powerhouse. Each turbine block consists of two units and is separated to other units with an expansion joint.

The powerhouse will be equipped with 8 pit type turbines having speed of 60 rpm. The generator will be coupled with turbine trough speed increaser having speed of 750 rpm. The generator output will be 12 MW. The power plant will be equipped with all necessary mechanical auxiliary equipment required for the smooth powerhouse operation.

The construction of project started in 2006 and expected to be completed in 2010 at a cost of Rs. 13546.8 million through an EPC contract.

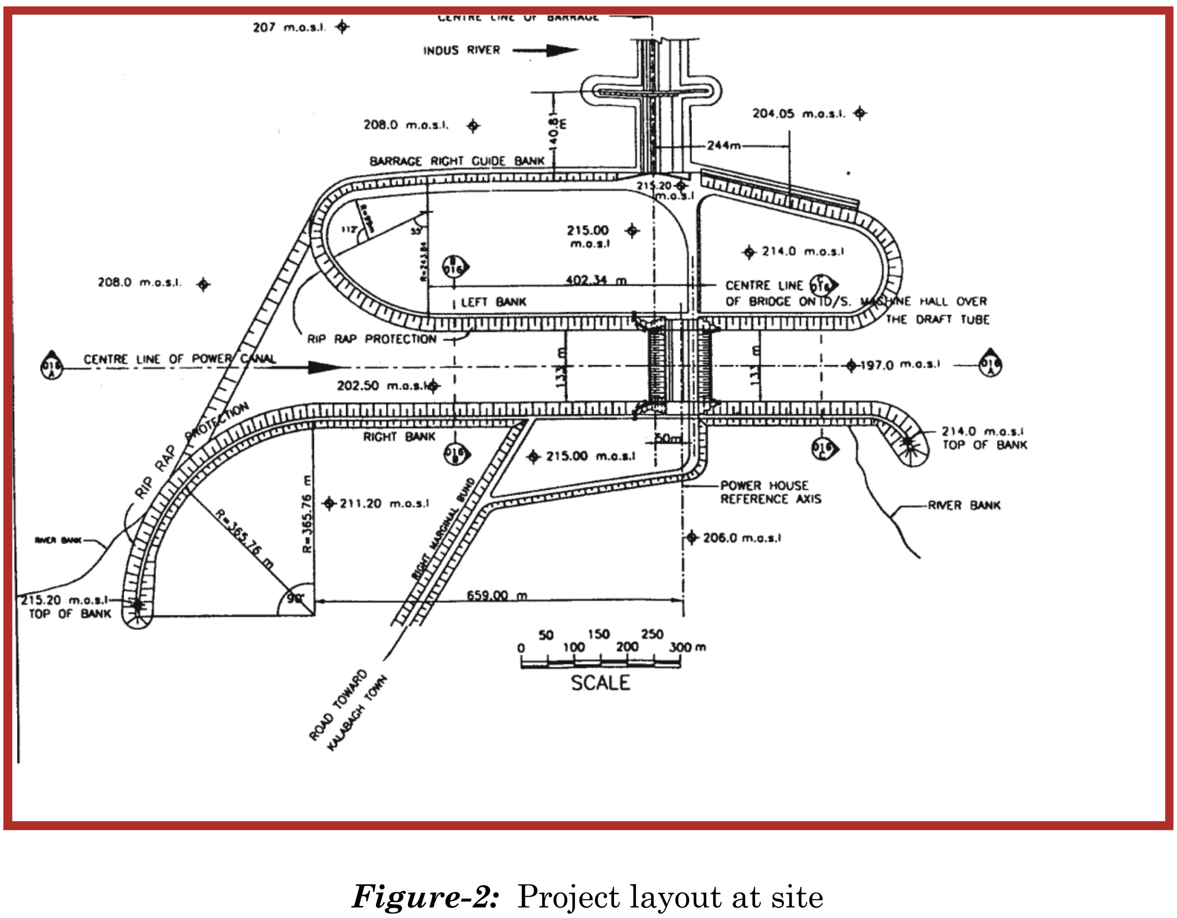


Figure-2: Project layout at site

3. GOVERNING EQUATIONS

The basic equations for one dimensional flow in prismatic channels have been used for solving the problem. It has been assumed that channel slope α is small enough, so that cos $\alpha = 1$. This condition is met in all but steep channels. Hydrostatic conditions are assumed to prevail along any vertical line in the fluid, which implies that vertical accelerations are not considered.

The nonlinear partial differential equations to describe the behavior of gradually varying unsteady free surface flow in prismatic channels is given as follows:

Momentum Equation

$$g\frac{\partial y}{\partial x} + g\,S - g\sin\alpha + V\frac{\partial V}{\partial x} + \frac{\partial V}{\partial t} + \frac{V}{A}q = 0 \qquad (1)$$

Continuity Equation

$$V\,T\frac{\partial y}{\partial x} + T\frac{\partial y}{\partial t} + A\frac{\partial V}{\partial x} - q = 0 \qquad (2)$$

Where

$V =$ Velocity

$R =$ Hydraulic Radius

$A =$ cross sectional area

$T =$ Top width

$q =$ Lateral inflow per unit length

$$S = \frac{n^2 V^2}{C_m^2 R^{3/4}}$$

$n =$ Manning Roughness

$C_m =$ 1.48 in English units

$=$ 1.0 in S.I. units

$So =$ Bed Slope

4. SOLUTION BY IMPLICIT METHOD

The implicit procedure is particularly suited to solving slow transients in natural channels. Thus it has been extensively used for flood routing in rivers. However it can also be utilized for rapid transients in power canals or pumping station channels with some understanding of its particular attributes. The method was first presented in a practical scheme by Preissman (1960), and further advanced by Cunge (1975), Liggett and Cunge (1975). The formulation used herein is adopted from Wylie & Streeter (1983)

By ignoring lateral inflow, and by noting that $Q = AV$ and $T = \frac{\partial A}{\partial y}$, the momentum and continuity equations, Eqs. (1) and (2), may be written in terms of discharge Q and depth y as the dependent variables:

$$F_1 = \left(1 - \frac{Q^2 T}{gA^3}\right)\frac{\partial y}{\partial x} + \frac{2Q}{gA^2}\frac{\partial Q}{\partial x} + \frac{1}{gA}\frac{\partial Q}{\partial t} + S - So = 0 \qquad (3)$$

$$F_2 = T\frac{\partial y}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (4)$$

The finite difference equivalent of Eq.3 and Eq.4 for use in the implicit method are obtained by considering the following substitutions for typical of the variables y, *A, T, P*, and *Q*, with reference to Figure-3.

$$y = \psi\left(\frac{y'_A + y'_B}{2}\right) + (1-\psi)\left(\frac{y_A + y_B}{2}\right) \qquad (5)$$

$$\frac{\partial y}{\partial x} = \frac{\psi(y'_B - y'_A) + (1-\psi)(y_B - y_A)}{\Delta x} \qquad (6)$$

$$\frac{\partial y}{\partial t} = \frac{(y'_A + y'_B) - (y_A + y_B)}{2\Delta t} \qquad (7)$$

In which $\psi = \Delta t'/\Delta t =$ a weighting factor. A stable numerical solution to Eqs.(3) and (4) may be obtained by use of values of ψ given by $0.5 < \psi \leq 1$. The solution is generally unstable if $\psi < 0.5$.

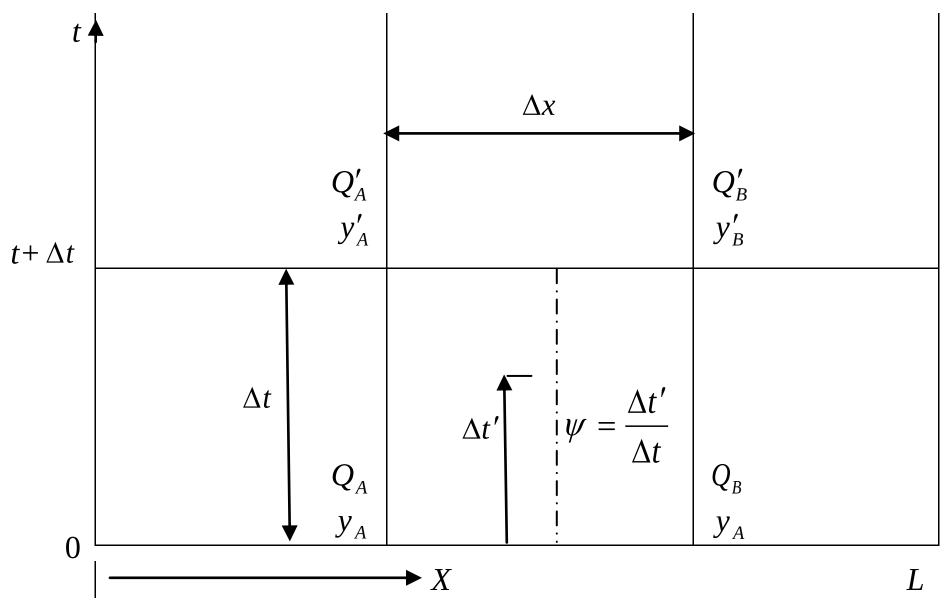


Figure-3: Rectangular grid for implicit method

Using the finite difference equivalents represented by Eqs. (5) to (7) for all variables in Eqs. (3) and (4), we obtain the following difference equations for the grid shown in Figure-3.

$$F_1 = \left(1 - \frac{\overline{Q}^2 \overline{T}}{g\overline{A}^3}\right)\left[\psi\left(y'_B - y'_A\right) + (1-\psi)(y_B - y_A)\right]$$
$$+ \frac{2\overline{Q}}{g\overline{A}^2}\left[\psi\left(Q'_B - Q'_A\right) + (1-\psi)(Q_B - Q_A)\right]$$
$$+ \frac{\Delta x}{\Delta t}\frac{1}{g\overline{A}}\left[\frac{Q'_A + Q'_B}{2} - \frac{Q_A + Q_B}{2}\right] + \frac{n^2 \Delta x \overline{P}^{4/3}}{C_m^2 \overline{A}^{10/3}}\overline{Q}\ \left|\overline{Q}\ \right| - So\Delta x = 0 \tag{8}$$

$$F_2 = \overline{T}\left[\frac{y'_A + y'_B}{2} - \frac{y_{A+} y_B}{2}\right]$$
$$+ \frac{\Delta t}{\Delta x}\left[\psi\left(Q'_B - Q'_A\right) + (1-\psi)(Q_B - Q_A)\right] = 0 \tag{9}$$

Equations (8) and (9) constitute a set of two nonlinear algebraic equations in four independent unknowns y'_A, y'_B , Q'_A and Q'_B. The unknowns are common to any two neighboring cells in Figure-3. A similar pair of equations may be written for each of the N cells in the channel. Thus there are 2 N equations in 2(N+1) unknowns. The boundary conditions at each end of the system provide the necessary two additional equations so that a unique solution may be obtained.

Different approaches are possible in finding the solution to the 2(N+1) nonlinear equations. A direct linearization of the equations is possible by writing the variables in terms of the known value plus a change in the value. Alternatively, the Newton-Raphson procedure may be used to provide a set of linear equations to be solved simultaneously. The linearized banded matrix arising from Eq. (8) and (9) may be solved using IBM Scientific Subroutine Package. The iterative solution converges quickly at each time step. The latter approach has been used in solving the present transient flow problem, by implicit method.

5. APPLICATION TO JINNAH POWER CHANNEL

Input Data

The power channel has been taken to be a straight prismatic channel, 730 metres long and 134 metres wide with a side slope of 2.5:1 and bed slope of 0.00025. Manning roughness has been assessed to be 0.033, so that normal depth computes to 8.44 metres, for a design discharge of 2360 m^3/sec. The program has been executed for two scenarios, namely the power house shut down time of 30 seconds and 15 seconds. The channel length has been divided in 10 segments for the shut down time of 30 seconds and in 20 segments for the shut down time of 15 seconds, for obtaining the finite difference solution. The computational time step used was 10 seconds, and 5 seconds respectively for the two scenarios. Weighting factor ψ was taken to 0.60 for a stable solution.

Output

For a shut down time of 15 seconds, the program outputs the discharge, depth and average velocity in the power channel at an interval of 10% of channel length, after every 5 seconds.

The development of surge and its travel in the upstream direction in the power channel has been shown in Figure.4. The height of surge continues to dampen only slightly as it travels in the upstream direction. The maximum height of surge develops at the downstream boundary, which is about 1.773 $\cong$ 1.8 meters, above initial water level (Figure-5). It is clear that the freeboard of 3.5 m as provided in the design is sufficient to contain the surge within channel prism without over topping.

There is little difference in the maximum height of surge for increasing the shut down time from 15 seconds to 30 seconds, refer Figure-6. However the face of surge front, was found to be less steep for a longer shut down time.

The program does not compute surge velocity (celerity) separately, but gives the average velocity over a cross section, at intervals of 0.1L, where L is the length of power channel. Surge velocity (celerity) can however be computed indirectly by observing the progression of surge front in the power channel at different time intervals, as shown in Figure.4. Maximum speed of surge as obtained from Figure-4 is about 8.03 m/sec.

The upstream speed of surge is more than four times the normal velocity in the power channel, and very likely to dislodge the stone protection on side slopes designed for normal flow condition, particularly in the upper part of channel prism. The design may therefore review the stone protection provided for stability, and if necessary, incorporate suitable modification to cater for high surge speed.

6. COMPARISON WITH AN APPROXIMATE ANALYTICAL SOLUTION

The moving surge phenomenon could be analysed by reducing the situation to one of stationary surge or hydraulic jump (Henderson, 1966). Let v_1 and y_1 be the velocity and depth of the flow before the jump and v_2 and y_2 be the velocity and depth after the jump and the c be the velocity of surge running upstream. Figure-7(a) shows the surge as seen by an observer on the bank as unsteady flow; whereas Figure-7(b) shows it as seen by observer riding along the surge, since the surge is now stationary. Figure-7(b) is produced from Figure-6(a) by superimposing the whole system, a velocity equal and opposite to that of the surge c; the stream velocities are therefore increased to $c+v_1$ and $c+v_2$ as shown.

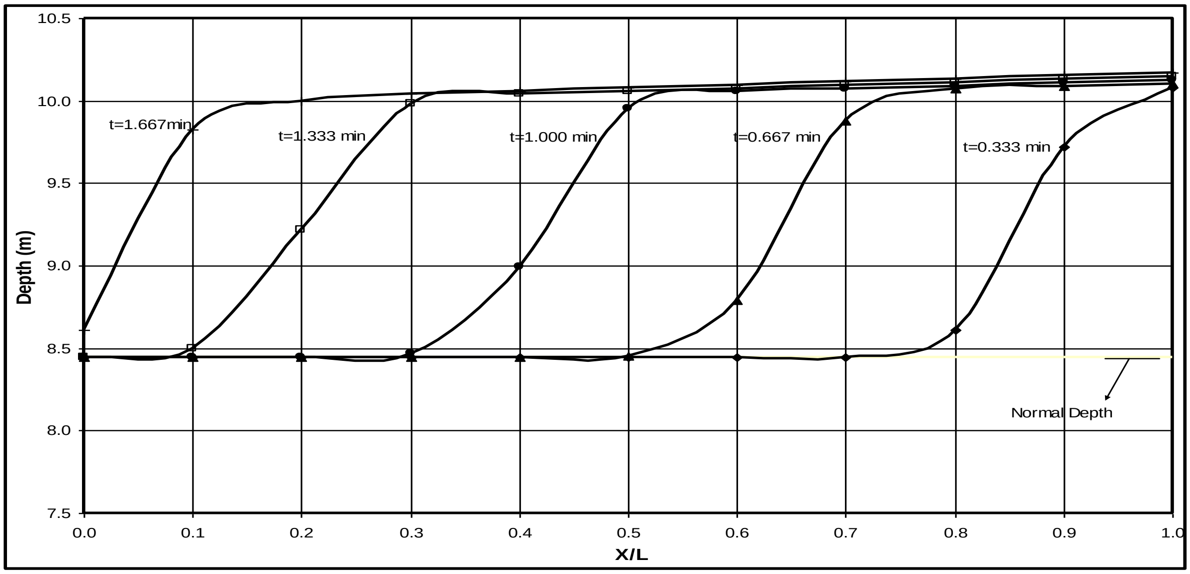


Figure 4: Development of Surcharge due to sudden closure (shutdown time=15 seconds)

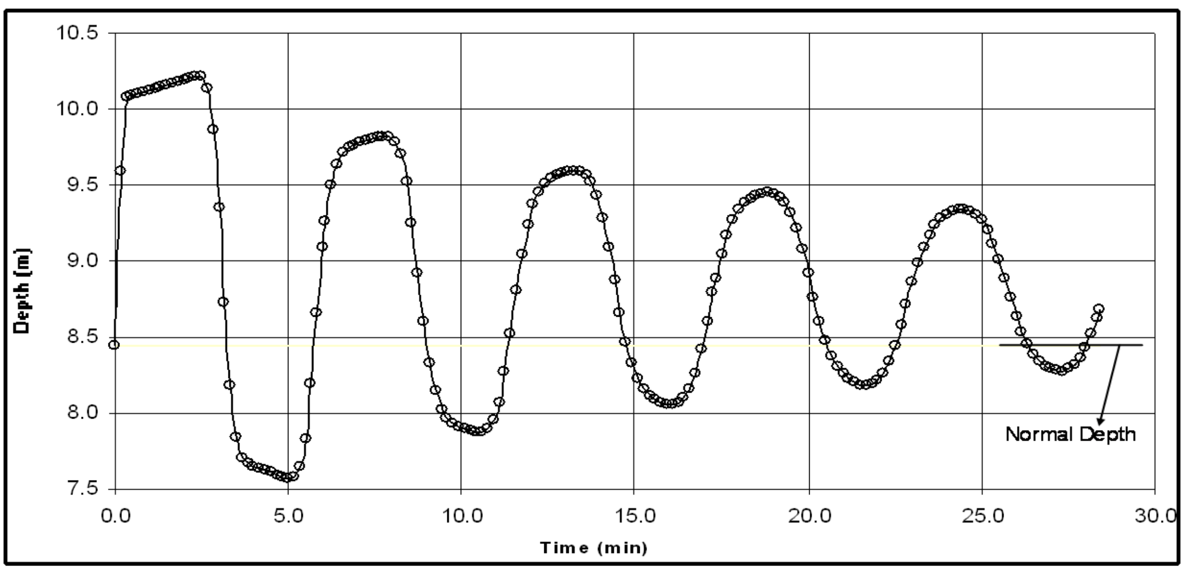


Figure 5: Variation in depth of flow at downstream boundary (shutdown time=15 seconds)

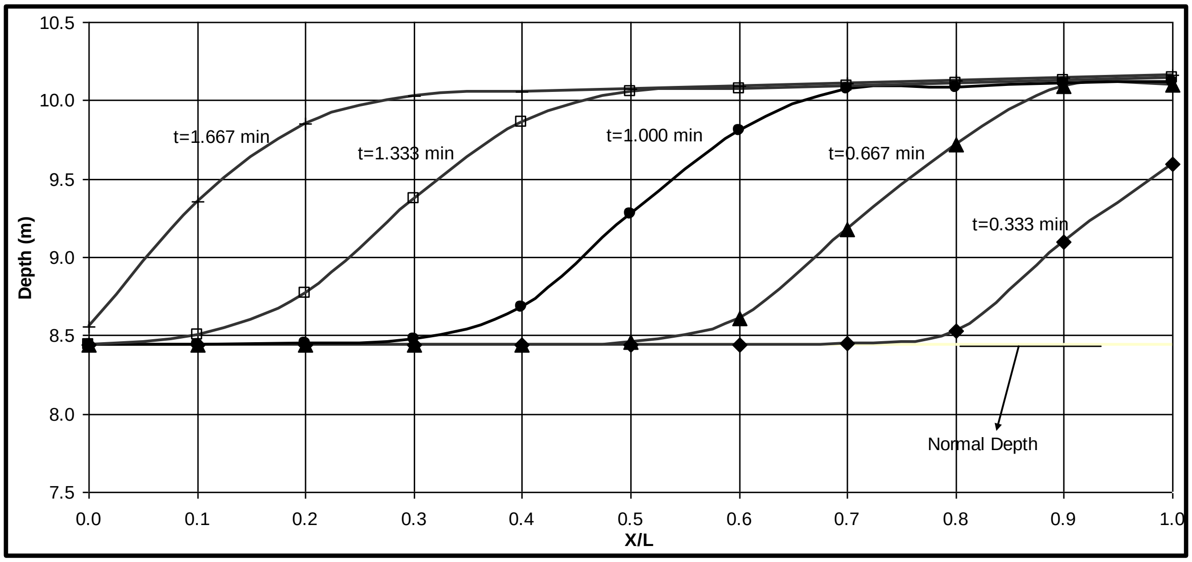
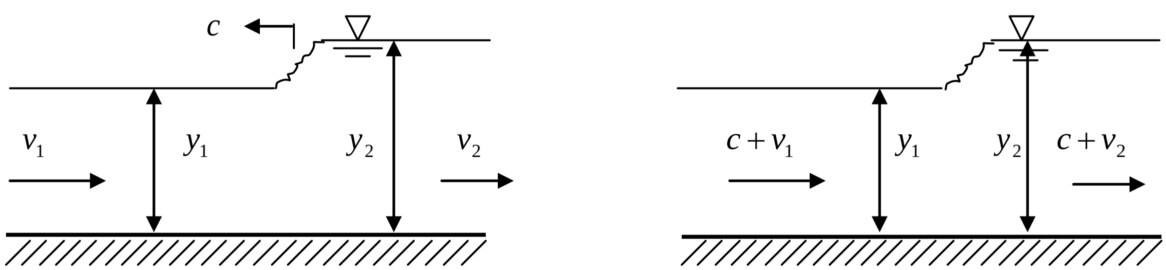


Figure 6: Development of Surcharge due to sudden closure (shutdown time=30 seconds)



(a) Unsteady flow - moving surge

(b) Steady flow - stationary surge

Figure 7: Reduction of the Moving surge to a Stationary Surge, or Hydraulic Jump

The equation of hydraulic jump of Figure-7(b) for a rectangular channel can be written as:

$$\frac{(c+v_1)^2}{gy_1} = \frac{1}{2}\frac{y_2}{y_1}\left(\frac{y_2}{y_1}+1\right) \qquad (10)$$

The continuity equation could also be applied to steady state situation of Figure-7(b).

$$(c+v_1)y_1 = (c+v_2)y_2 \qquad (11)$$

From Eq. (11) the surge speed could be computed as:

$$c = \frac{v_2 y_2 - v_1 y_1}{y_2 - y_1} = \frac{q_1 - q_1}{y_2 - y_1} \qquad (12)$$

Where q_1 is the initial discharge per unit width, and q_2 is the known unit discharge after shutdown. The surge height $(y_2 - y_1)$ could be computed for a iterative solution of Eq. (10) and Eq. (11).

For the given initial flow condition $(v_1 = 1.802 m/\text{sec.}, y_1 = 8.44 m)$ and for sudden shut down of power house $(v_2 = 0)$, Eq. (10) & (11) give:

Surge speed, c = 8.7m/sec.

$y_2 = 10.19$m

Surge height = $y_2 - y_1$ = 10.19 - 8.44 = 1.75 m.

These results compare favourably with those obtained from numerical solution.

7. SUMMARY AND CONCLUSIONS

The development of surge in the power channel of Jinnah Hydropower Project in the case of sudden shut down has been simulated using a numerical solution of nonlinear partial differential equation for conservation mass and momentum for prismatic channel using finite difference approach with implicit procedure. For the initial condition, the normal flow was specified in the power channel with design discharge of 2360 m^3/sec., bed width 134 m, side slope 2.5:1, bed slope 0.00025, and Manning roughness 0.033. A constant head boundary was assumed at the upstream, whereas a linearly decreasing discharge was specified at the downstream boundary. For a shut down period of 15 seconds, the height of surge was about 1.8 m and its speed 8.03 m/sec. The numerical solution compares favourably with an analytical solution obtained by reducing the moving surge phenomena to a stationary surge or hydraulic jump. It was found that the freeboard of 3.5 m as specified in the design would be adequate for containing the surge within channel prism without over topping. However, the surge speed is nearly four time the normal velocity in the power channel. Therefore, the design of stone protection may have to be re-examined for stability against the action of high surge speed.

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