Pak. J. Engg. & Appl. Sci. Vol. 8, *Jan.*, 2011 (*p.* 34-48)

Hydraulic Transient Analysis of Surge Tanks: Case Study of Satpara and Golen Gol Hydropower Projects in Pakistan

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Abstract

Surge tanks are used for dissipate the water hammer pressure in high head hydropower project. Commonly used surge tanks have one surge chamber. A double chamber surge tanks were introduced in two high head hydropower projects in Pakistan. In the present study hydraulic design of surge tanks for the two potential sites in Pakistan were analyzed for surge wave height and time to dissipate. Surge tanks designed for Golen Gol hydropower project and Satpara hydropower project were analyzed for the hydraulic transient under the two operational scenarios i.e. complete closure and complete opening. It was concluded that for Satpara hydropower plant, surge tank without chamber and surge tank with two chambers produces high range of surges to cause undesirably heavy governor movement. While surge tank with lower chamber produces the minimum surge height as compared to other types, so the hydraulic behavior of surge tank with lower chamber is more stable than other types of surge tanks. Similarly for Golen Gol hydropower plant surge tank with two chambers produces better surge protection as compared to surge tank with single chamber and no chamber.

Key Words: *Hydraulic Transient, Hydropower, Surge Chamber, High Head Hydropower, Pressure Conduit.*

1. Introduction

The surge tank or surge tower is an essential part of with high head hydropower to protect the low-pressure conduit system from high internal pressure. The surge tank is also useful to minimize the possible danger due to water hammer due to pressure change in closed pipes caused when flowing water in pipes is accelerated or decelerated by closing or opening a valve or changing the velocity of water rapidly in some other mean. Whenever there is an abrupt load rejection by the power system, the mass of water in the conveyance system in turn get suddenly decelerated, this process gives rise to water hammer phenomenon. The purpose of the surge tank is to intercept and dampen these high-pressure waves and not allow them in the low-pressure system. Its operation benefits in three different ways.

1. It shortens the distance between the turbine inlet and the nearest free water surface, and thereby greatly reduces the intensity of the water hammer waves.

2. With a reduction of turbine load, the water level in the chamber rises until it exceeds the level in the main reservoir, thus retarding the main conduit flow and absorbing the surplus kinetic energy.

3. In case of increase of turbine load, the chamber act as a reservoir which will provide sufficient water to enable the turbine to pick up their new load safely and quickly and to keep them running at the increased load until the water level in the surge chamber has fallen below its original level. Sufficient head is thereby created to accelerate the flow of water in the conduit until it meets the new demand.

The abrupt reduction of the electrical load the turbine governor will rapidly cause the turbine guide vanes to close, so that there will be an abrupt reduction of flow. This will initiate a surge wave which causes the water level to rise in the chamber until it exceeds reservoir level and produces a retarding force that will arrest and then reverse the direction of the flow in the conduit. The chamber water level will then drop it is below reservoir level; the conduit flow is again slowly stopped and reversed, and the cycle is repeated until damped out by frictional losses. These changes are illustrated in Figure 1a.

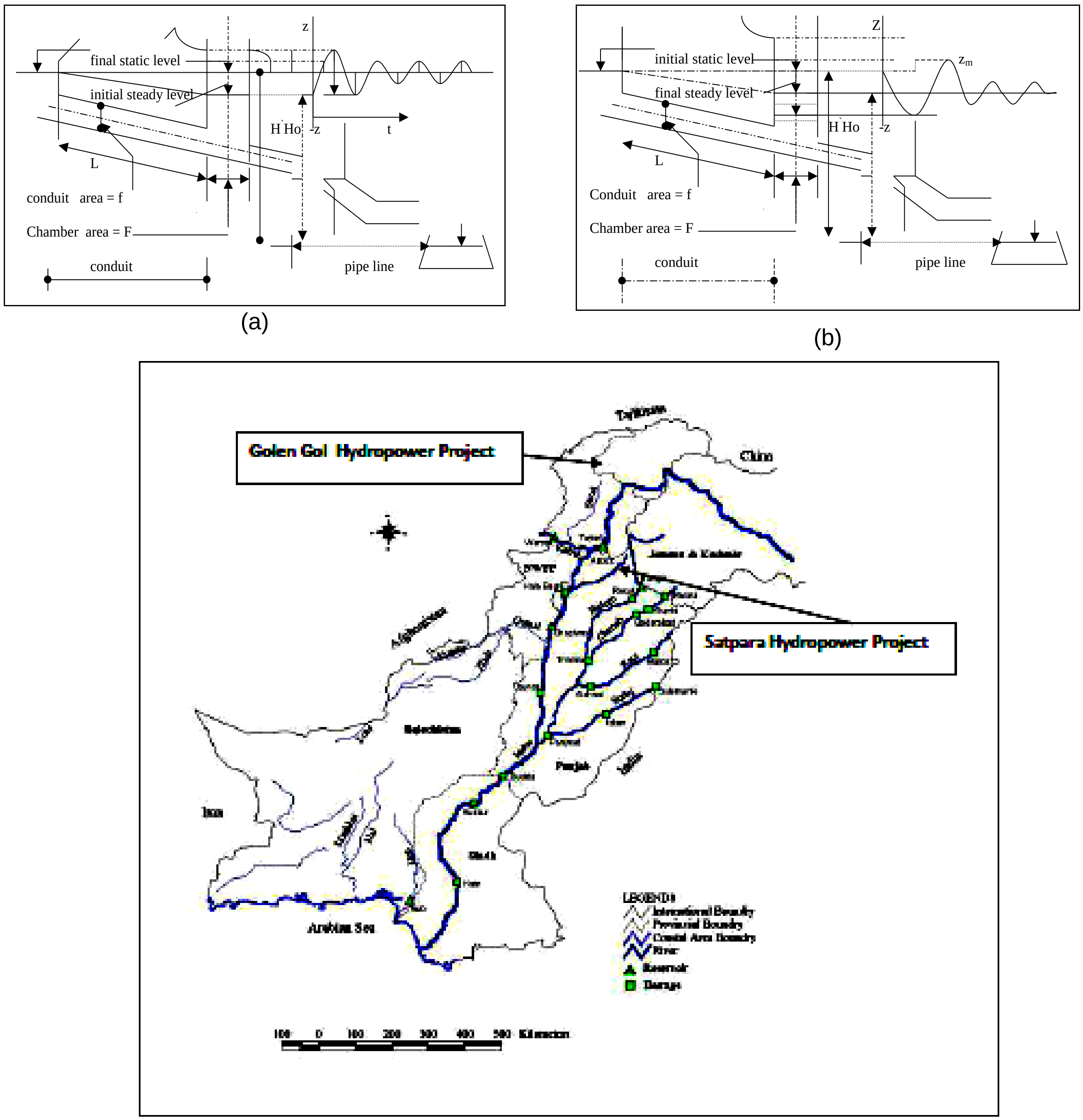


Figure 1: Water hammer effects due to (a) sudden opening, (b) sudden closure of turbine guide vanes and (c) location of the hydropower projects.

On starting up the turbine with water at rest in the tunnel, the water level in the chamber will fall rapidly below its initial position and will thus create sufficient head to accelerate the main mass of water in the tunnel. The conduit discharge will thus increase until it exceeds that required by the turbine; the surplus will cause a rise in chamber level, increasing the net head on the turbine and reducing its demand for water. The rise of chamber water level will retard the conduit flow and, as with load rejection, a surge motion will be set up and will continue until damped out by friction. The water level in the chamber will settle finally at its steady running level which, owing to friction and velocity effects, is lower than reservoir level. The conditions are illustrated in Figure1b.

For a given load change the surge amplitude in a simple chamber is approximately proportional to the diameter of the chamber, if the chamber is big enough , the surge becomes "dead beat " and will die away after the half cycle. A similar deadbeat condition will result if the load change is sufficiently slow. Deadbeat chambers are not usually economical.

1.1 Design considerations

The surge chambers are designed to meet the following conditions.

1. The surge chamber must be so located that pressure variations caused by water hammer are kept within acceptable limits.
2. The chamber must be stable, i.e. the surges resulting from small partial load changes must be naturally damped and must not under any condition be sustained or amplified.
3. The chamber must be of such size and so proportioned that it will contain the maximum possible upsurge (unless a spillway is provided). The lowest down surge will not allow air to be drawn into the tunnel. The range of surges must not be great enough to cause undesirably heavy governor movements or difficulty in startup load.

1.2 Extreme loading conditions for Turbines

Sudden shutdown of hydroelectric plants or change in water flow through hydraulic turbine may cause problems ranging from rupture of penstock due to water hammer to runner speed changes that cause the line current of the generators to vary from the desired frequency. As mentioned earlier, In case of electrical or mechanical failures the entire load would be rejected instantly; this might occur with the turbines at full load and with the reservoir at any level. Full load rejection must therefore be considered in every case. It is usual to consider full-load rejection under two conditions.

1. With the reservoir at its maximum level, in which case the maximum upsurge level will govern the top level of the chamber;
2. With the reservoir at its lowest draw down, in which case the first down surge level may control the bottom level of the chamber if air drawing is to be avoided.

The loading conditions (load acceptance) are not so critical because it depends on the turbine design and operation procedures. Several articles on the various aspects of water hammer have been published. Despite this a wide field still remains open to further research. The following equations apply for the case when the pipe is considered to be compressible. Restricted orifice surge tank analysis was introduced by Mosonyi and Seth (1975), he developed equations when the restricted orifice surge tank operates and water hammer causes significant pressure head rise in the penstock upstream of the surge tank. They developed and tested this theory in a laboratory in Germany for a particular cross sectional area of surge tank.

1.3 Basic Equations for Surge Analysis

The following continuity and momentum equations were used as explained in Chaudhry (1987), Wylie and Streeter (1993) and Parmakian (1963).

Continuity equation

$$\frac{dZ}{dt}=\frac{1}{A_s}(Q_{tun}-Q_{tur}) \tag{1}$$

Momentum equation

$$\frac{dV_{tun}}{dt}=\frac{g}{L_1}\left(-Z-C_e\frac{V_{tun}|V_{tun}|}{2g}-\frac{V_{tun}|V_{tun}|}{2g}-C_t\frac{V_{tun}|V_{tun}|}{2g}-(C_{orf}+C_c)\frac{V_s|V_s|}{2g}\right) \tag{2}$$

where A_s = cross sectional area of the surge tank (m^2); Q_{tun} is flow in the tunnel (m^3/s); Q_{tur} = turbine flow (m^3/s); Z = fluctuations in the surge tank with respect to static water level in the reservoir (m); V_{tun} = velocity of the water in the tunnel (m/s); L_1= length of the tunnel from reservoir to surge tank (m); C_e = coefficient of entrance loss (0.5); C_t = coefficient of frictional losses in the tunnel; C_{dc} =coefficient of confluence or diversion of flows due to either filling or emptying of the surge chamber; C_{orf} =coefficient of losses in orifice due to either inflow or outflow; V_s = velocity of water in the surge tank (m/s); L_2 = length of the penstock from surge tank to the turbine (m); K_{L1} = friction coefficient of tunnel from upper reservoir to surge tank. R_{H1} = Hydraulic radius of the tunnel connecting upper reservoir to surge tank.

Coefficient of frictional losses in the tunnel is computed using Strickler equation

$$C_t = \frac{2gL}{[K_{L1}(R_{H1})^{0.667}]^2} \tag{3}$$

Loss coefficients for flow diversion or confluence and coefficient of orifice are computed by the formulas developed by Gardel (1956), Blaisdell and Manson (1967), and Ito and Imai (1973) with respect to inflow or outflow into the surge tank.

(a) Filling the surge chamber (diversion)

$$C_{dc} = 0.99 - 0.82\left|\frac{Q_{tun} - Q_{tur}}{Q_{tun}}\right| + 1.02\left(\frac{Q_{tun} - Q_{tur}}{Q_{tun}}\right)^2 \tag{4}$$

$$C_{orf} = k_{01}\frac{A_{tun}}{A_{sao}}\left(\frac{Q_{tun} - Q_{tur}}{Q_{tun}}\right)^2 \tag{5}$$

Where

k_{01} = loss coefficient of orifice due to inflow discharge = 10

A_{sao} = cross sectional area of the surge tank above the orifice

A_{tun} = cross sectional area of the tunnel.

(b) Emptying the surge chamber (confluence)

$$C_{dc} = -1.0 + 4\left|\frac{Q_{tun} - Q_{tur}}{Q_{tun}}\right| - 2\left(\frac{Q_{tun} - Q_{tur}}{Q_{tun}}\right)^2 \tag{6}$$

$$C_{orf} = k_{02}\frac{A_{tun}}{A_{sbo}}\left(\frac{Q_{tun} - Q_{tur}}{Q_{tun}}\right)^2 \tag{7}$$

Where;

A_{sbo} = cross sectional area of the surge tank below the orifice

k_{02} = loss coefficient of orifice due to outflow discharge = 10

In this study Runge- kutta fourth order method was used to solve continuity and momentum equations as described below. Surge tank cross sectional areas were interpolated to solve equation (2) for known water levels in the surge tank.

A_s = function of (water level in the surge tank i.e., Z^t)

$$K1_1 = \frac{g}{L_1}\left((C_e + 1 + C_t)\frac{Q_{tun}|Q_{tun}|}{1gA_t^2} - (C_{orf} + C_{dc})\frac{Q_s|Q_s|}{2gA_s^2}\right) \tag{8}$$

$$K2_1 = \frac{1}{A_s}(Q_{tun} - Q_{tur}) \tag{9}$$

$$Q_1 = Q_{tun}^t + 0.5\Delta t K1_1 A_t \tag{10}$$

$$Z_1 = Z^t + 0.5\Delta t K2_1 \tag{11}$$

Thus, $K1_2$, $K2_2$, Q_2, Z_2 are obtained at t+0.5Δt using the slopes $K1_1$, $K2_1$.

$K1_3$, $K2_3$, Q_3, Z_3 are obtained at t+0.5Δt using the slopes $K1_2$, $K2_2$.

$K1_4$, $K2_4$, Q_4, Z_4 are obtained at t+Δt using the slopes $K1_3$, $K2_3$.

Averaging the four slopes gives the flow rate in the tunnel and water level in the surge tank at t+Δt time level

$$Q_{tun}^{t+\Delta t} = Q_{tun}^t + (K1_1 + 2K1_2 + 2K1_3 + K1_4)^{A_t\Delta t}/6 \tag{12}$$

$$Z^{t+\Delta t} = Z^t + (K2_1 + 2K2_2 + 2K2_3 + K2_4)^{\Delta t}/6 \tag{13}$$

These computations are carried out until computational time reaches stopping time.

The turbine discharge can be estimated using the following equations for a given power output of the plant.

$$Q = \frac{P}{pg\eta H_{net}} \tag{14}$$

$$H_{net} = H - h_{tun} - h_{pen} \tag{15}$$

where P = power capacity in watts, ρ = mass density of water (kg/m^3); η = total efficiency of the power station; H_{net} = net head available for power generation (m); H = gross head (m); h_{tun} = frictional losses in the tunnel (m); h_{pen} = frictional losses in the penstock (m).

1.4 Stability Criteria

Allievi's (1913) developed the basic water hammer equations for surge analysis. Jaeger (1955, 1958, 1960, and 1963) investigated variety of surge problems, generalized the Allievi's (1913) system of equations for surge tank and solved the stability problem. He proposed that a large surge tank is an excellent protection against pressure waves because all waves are totally reflected, and the additional pressures in the pipeline are always zero.

Thoma (1910) established stability criteria which is called Thoma criteria of the surge tanks. According to this criteria, to damp out the mass oscillations in the surge tank, the cross section of the riser of the surge tank should be greater than Thoma cross-section 'A_{th}'. If the riser area is smaller than this value the stability of the mass oscillations may not be guaranteed. Later investigations revealed the impracticability of a general criterion and established the necessity of specifying separate conditions for small and for great amplitudes. The formula suggested by Thoma in case of small oscillations for the limit cross-sectional area of the surge tank is

$$A_{th} = \frac{nV^2 LA_{tun}}{2g\beta V^2 H_{net}} \tag{16}$$

where n = Factor of safety; V = Tunnel velocity pertaining to the new dynamic equilibrium opening; β = Resistance factor of the tunnel; L = Length of the tunnel; A_t = Tunnel section; k = Manning –Strickler coefficient; H_{net} the net head (by subtracting the frictional head loss in the tunnel and penstock from the gross head= $H - \beta V^2$); The damping factor may be defined as:

$$m = 2\beta g \frac{A_{th}}{A_{tun}} \tag{17}$$

Substituting the damping factor m defined by above equation into relationship written Eq. (16), the minimum limit value of head ensuring surge stability in case of given cross-sectional area of the surge tank is

$$H_{net} = \frac{nL}{m} \tag{18}$$

Assuming that local resistance can be neglected with the respect to friction losses and substituting $\beta = \frac{1}{k^2 R^{\frac{4}{3}}}$ in equation (16) we get

$$A_{th} = \frac{nk^2 R_{tun}^{\frac{4}{3}} A_{tun}}{2gH_{net}} \tag{19}$$

which can be simplified in case of a tunnel of circular section, with A_{tun}= 3.14D_{tun}^2 /4 and R_{tun} = D_{tun} /4

$$A_{th} = \frac{nk^2 D_{tun}^{\frac{10}{3}} A_{tun}}{160 H_{net}} \tag{20}$$

A safety factor of 1.5 to 1.8 has recently been adopted. As it is clear from the above equations, to lower the friction, i.e. to higher the velocity factor in the Manning –Strickler formula, to larger the required cross- section of the surge tank and vice versa. Limit values of A_{th} are thus obtained by the simultaneous assumption of the highest factors of n and k.

2.0 Surge Analysis

The equations mentioned in previous sections were solved in Surge model. The model was developed by German Technical Cooperation (GTZ) in collaboration with Water and Power Development Authority (WAPDA). The hydraulic transient studies of two hydropower project Golen Gol and Satpara were carried out for operation of the turbines and behavior of the surge tank. The numerical study deals with the analysis of the surge produced by sudden load rejection and sudden load acceptance. The surge structure design data in connection with hydraulic analysis was used for the numerical simulation is given in Table 1. Descriptions of the terms used in this analysis are shown in Figure 2.

2.1 Golen Gol hydropower project

The project is located on Golen Gol Nullah, a tributary of Mastuj River, 25 km from Chitral Town in NWFP. The installed capacity of the project is 106 MW. The details of the input data for surge analysis are presented in Table 1.

The hydraulic analysis of the surge tank of Golen Gol hydropower project was analyzed under the two operational scenarios i,e. The complete closure and complete opening of the turbine governors. Three types of the surge tank system were analyzed under the above-mentioned scenarios. Three different types of surge tanks were studied which include surge wave without surge tank, surge wave in hydropower project having lower surge chamber and surge tank having two surge chambers (upper chamber and lower chamber). Two operating conditions are sudden

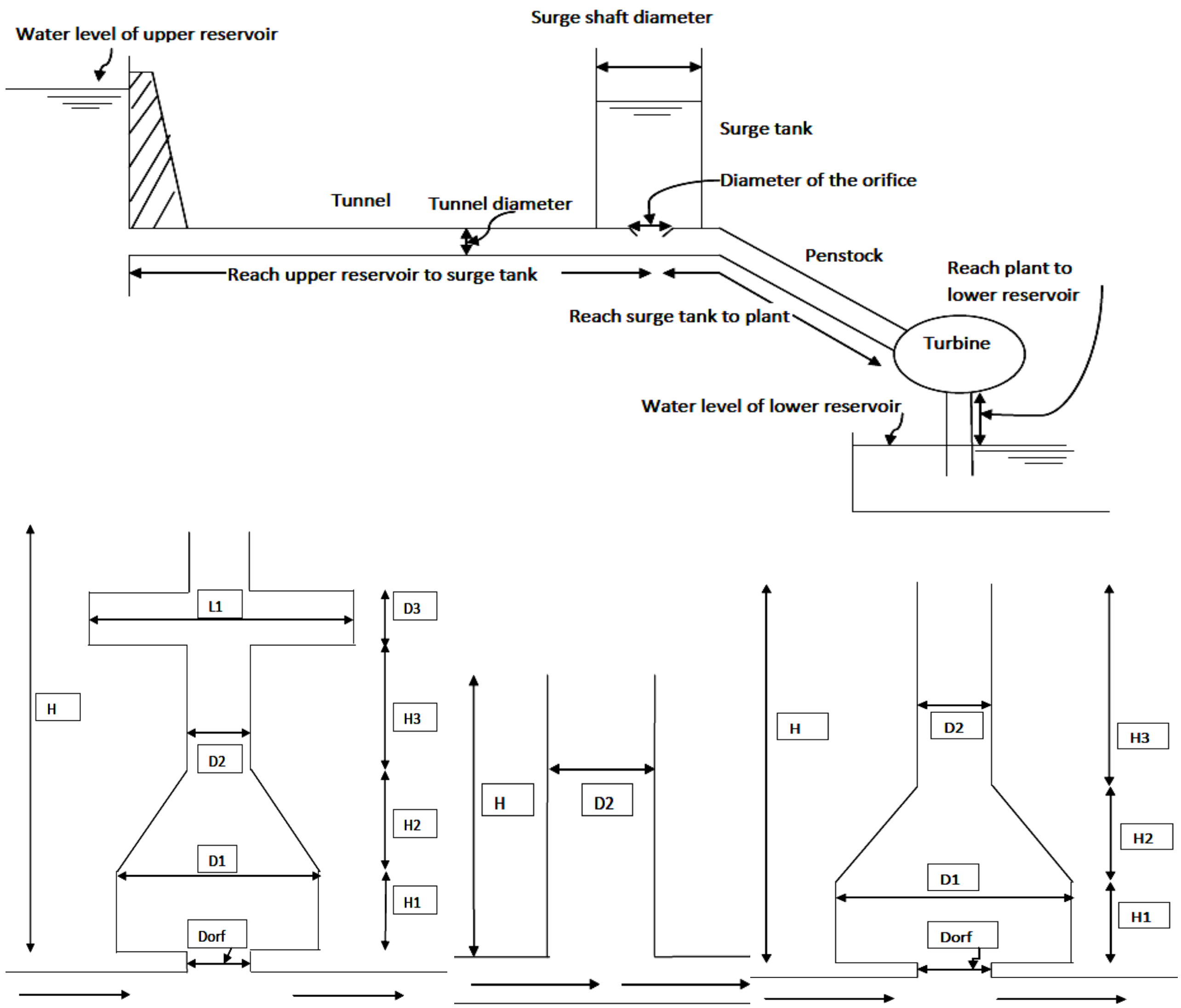


Figure 2: Description of the components used in modeling of hydropower system, the H is head D is diameter for different three types of surge tanks, H1, H2 and H3 are heads up to different stages from which the transient in section starts.

closure of the turbine due to mechanical or other failure of the system, and sudden operation of the turbine. Different scenarios are described below.

Table 1: Input data for the Golen Gol hydropower project

Descriptions of data	Data
Location of surge tank	Upstream
Water level of upper reservoir	2052.00m
Water level of lower reservoir	1612.00 m
Friction coefficient reach upper reservoir- surge chamber	80
Friction coefficient reach surge chamber – plant	80
Friction coefficient reach plant – lower reservoir	70
Tunnel length reach reservoir –surge chamber	3810.00m
Tunnel diameter reach reservoir – surge chamber	3.20m
Tunnel length reach surge chamber – plant	650.00 m
Tunnel diameter reach surge chamber- plant	3.00 m
Tunnel length reach plant- lower reservoir	80.00
Tunnel diameter reach plant- lower reservoir	5.00 m
Diameter of surge shaft	9 m
Height of surge shaft	30 m
Diameter of vertical shaft below orifice	3.00 m
Diameter of vertical shaft above orifice	9.00m
Diameter of orifice	3.25m
Design discharge	30 m^3/sec
Installed capacity	106 M watt
Total efficiency of power station	0.85

The surge wave was studied for the options that there is no chamber in the surge shaft, only lower chamber and two chambers in the surge shaft and turbine as suddenly operated. The design discharge 30 m^3/sec was attained with in 100 s of the operation. For shut down the turbine was shutdown in 120 s. The simulation was carried out for 2000 s with a computational time interval of 0.5 s. The results tabulated in Table-1 the result of complete closure and complete opening is given in table 2 and table 3. The behavior of the corresponding surge waves are shown in Figures 3 to 5.

Table 2: Results of maximum surge height in complete closure

Sr. No.	Type of surge tank	Maximum up surge (m)
1	Surge tank without chambers	2070.00
2	Surge tank with lower chamber	2069.80
3	Surge tank with two chamber	2062.00

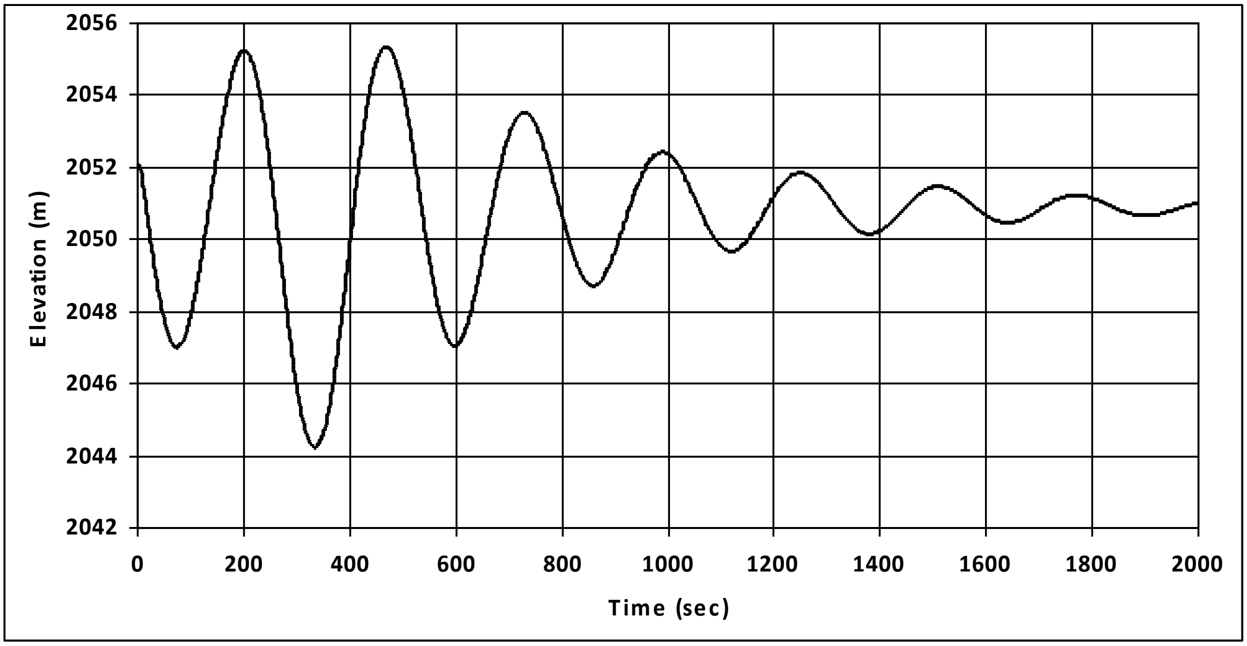
Table 3: Results of maximum surge height in complete opening

Sr. No.	Type of surge tank	Max. down surge (m)
1	Surge tank without chambers	2044.50
2	Surge tank with lower chamber	2044.20
3	Surge tank with two chamber	2044.50

From the table 2 and table 3, the surge height accumulated as the difference of maximum up surge and maximum down surge are presented in table 4.

Table 4: Results of accumulated surge height

Sr. No.	Type of surge tank	Surge accumu-lated (m)
1	Surge tank without chambers	25.5
2	Surge tank with lower chamber	25.3
3	Surge tank with two chamber	17.5



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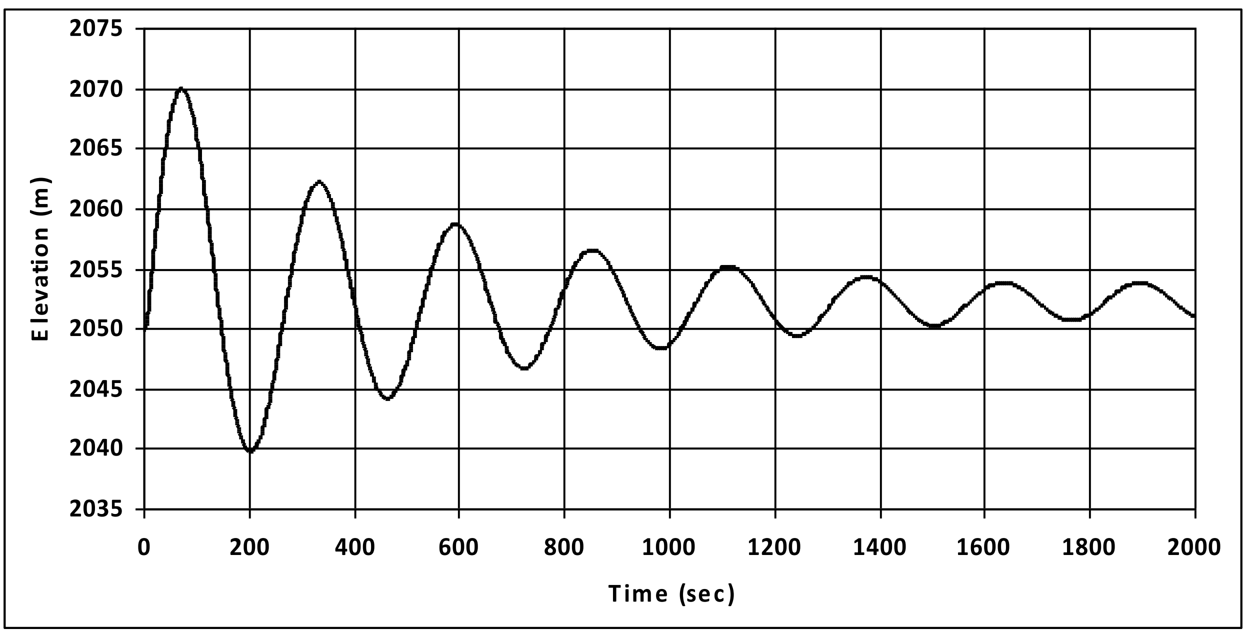
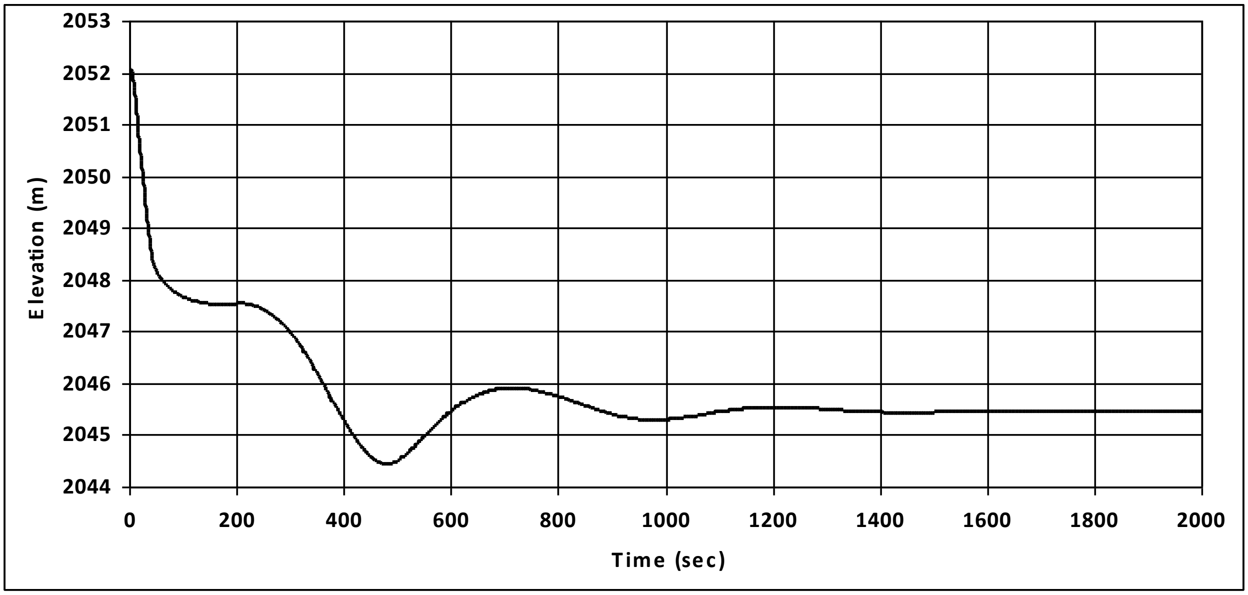


(b)

Figure 3: Surge analysis of Golen Gol Hydropower project without surge chamber **(a)** downsurge and (b) upsurge



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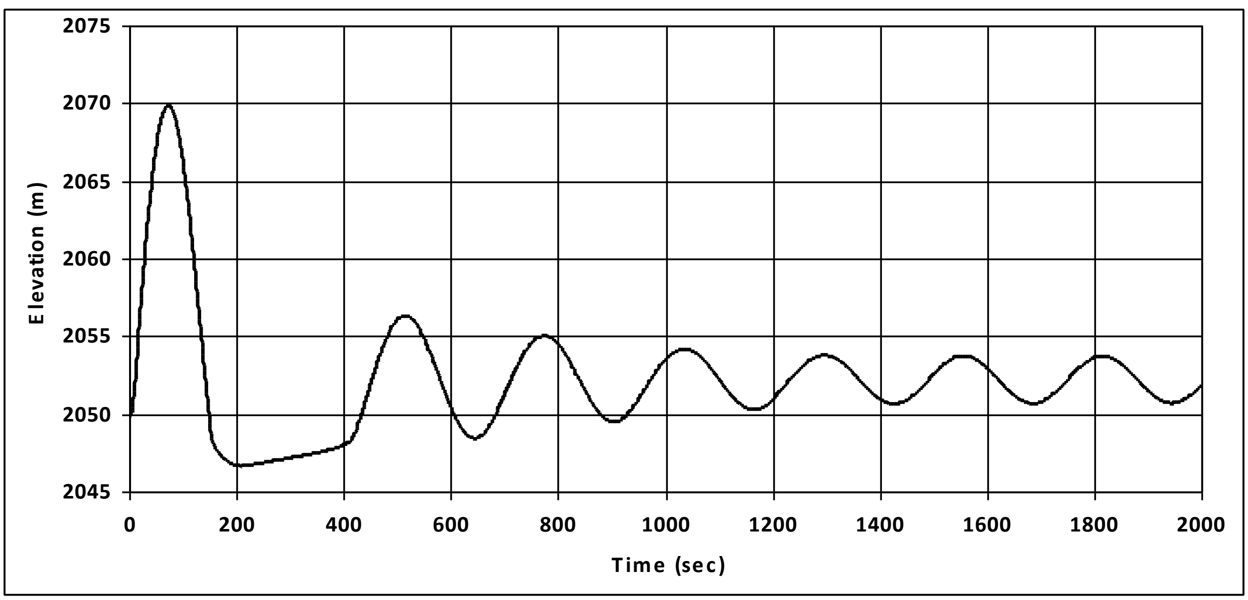

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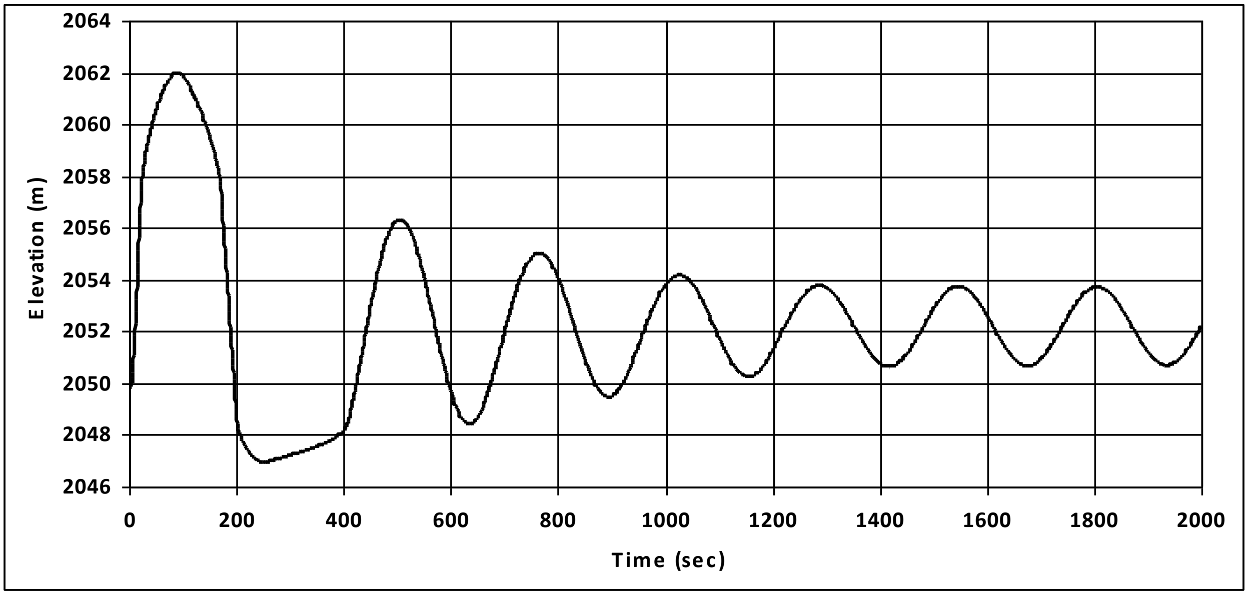
Figure 4: Surge analysis of Golen Gol Hydropower project with surge shaft with lower chamber (a) downsurge and (b) upsurge



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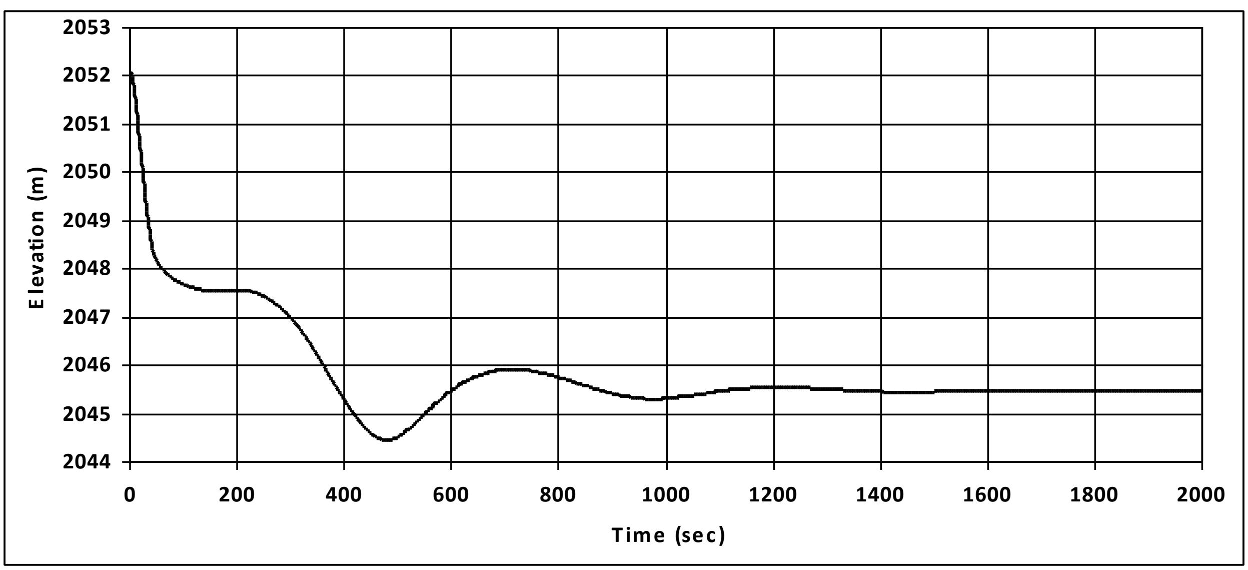


(b)

Figure 5: Surge analysis of Golen Gol hydropower project with surge shaft with lower and upper chambers (a) upsurge and (b) downsurge

2.2 Satpara Hydropower System

Satpara hydropower project is located in Northern areas of Pakistan at Satpara Lake, which is about 6 km south of Skardu town. The various input data for this project are given in table 5.

Table 5: Input data for the Satpara hydropower project

Location of surge tank	Upstream
Water level of upper reservoir	2664.31 m
Water level of lower reservoir	2570.00 m
Friction coefficient reach upper reservoir- surge chamber	40.00
Friction coefficient reach surge chamber – plant	80.00
Friction coefficient reach plant – lower reservoir	85.00
Tunnel length reach reservoir – surge chamber	236.10 m
Tunnel diameter reach reservoir – surge chamber	3.44 m
Tunnel length reach surge chamber – plant	567.0 m
Tunnel diameter reach surge chamber – plant	1.50 m
Tunnel length reach plant- lower reservoir	0.00 m
Tunnel diameter reach plant- lower reservoir	0. 00 m
Diameter of surge shaft	5.66 m
Height of surge shaft	33.0 m
Diameter of vertical shaft below orifice	5.66 m
Diameter of vertical shaft above orifice	5.66 m
Diameter of orifice	5.66 m
Design discharge	6.00 m^3/sec
Installed capacity	4.14 M watt
Total efficiency of power station	0.85

The hydraulic analysis of the surge tank of Satpara hydropower project was analyzed under the two operational scenarios i.e. the complete closure and complete opening of the turbine governors. The design discharge 6 m^3/sec was attained with in 15 s of the operation. For shut down the turbine was shutdown in 10 s. The simulation was carried out for 1000 s with a computational time interval of 0.5 s. Three types of the surge tank system were analyzed under the above-mentioned scenarios. The different typical surge tank systems analyzed are surge shaft without chambers, surge shaft with lower chamber and two chamber surge tank. The results tabulated in tables 6-8, were also shown in figure 6-8.

Operational Scenario-1 Complete Closure

Considering friction coefficients for the analysis and the maximum and minimum water levels respectively at the intake, the calculated surge levels are:

Table 6: Results of maximum surge height

Sr. No.	Type of surge tank	Maximum up surge (m)
1	Surge tank without chambers	2666.38
2	Surge tank with lower chamber	2663.01
3	Surge tank with two chamber	2666.71

Operational Scenario-2 complete opening

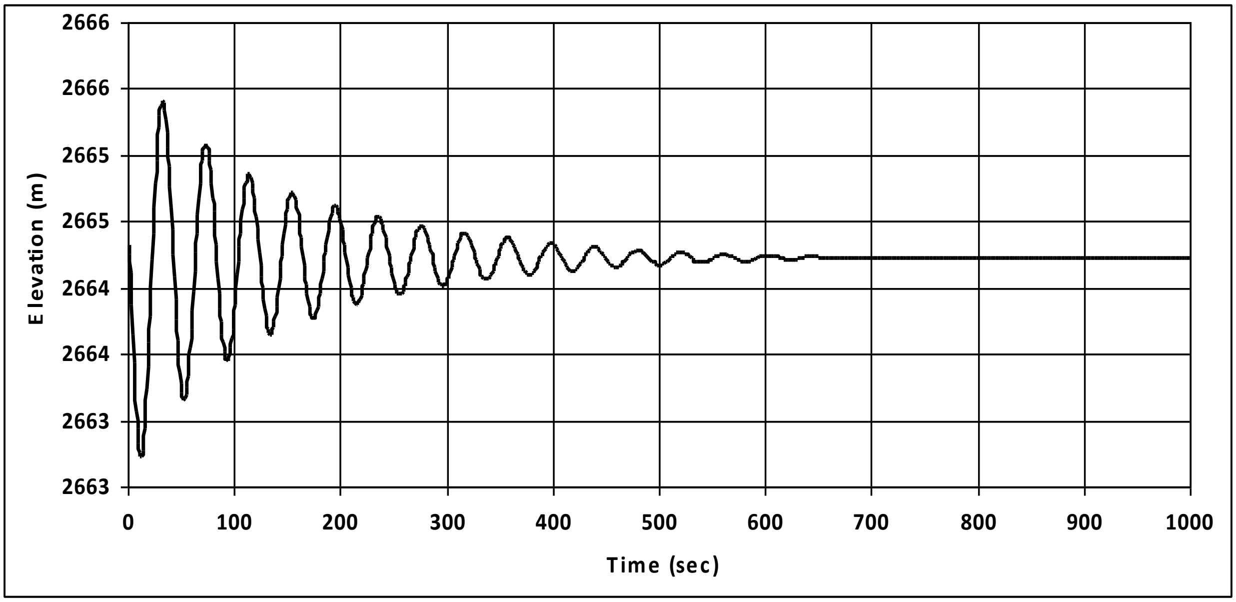
Table 7: Results of maximum surge height

Sr. No.	Type of surge tank	Maximum down surge (m)
1	Surge tank without chambers	2662.73
2	Surge tank with lower chamber	2662.49
3	Surge tank with two chamber	2662.71

From the table 6 and 7, the surge height accumulated as the difference of maximum up surge and maximum down surge shown in table 8.

Table 8: Results of accumulated surge height

Sr. No.	Type of surge tank	Surge accumu-lated (m)
1	Surge tank without chambers	3.65
2	Surge tank with lower chamber	3.37
3	Surge tank with two chamber	3.69



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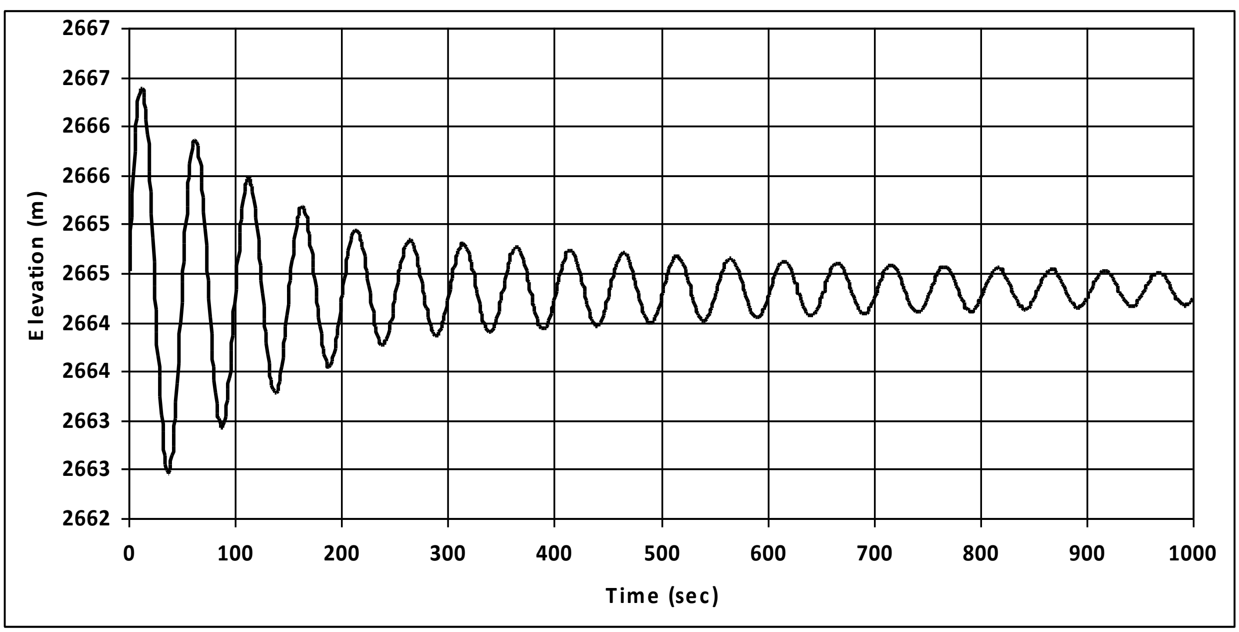
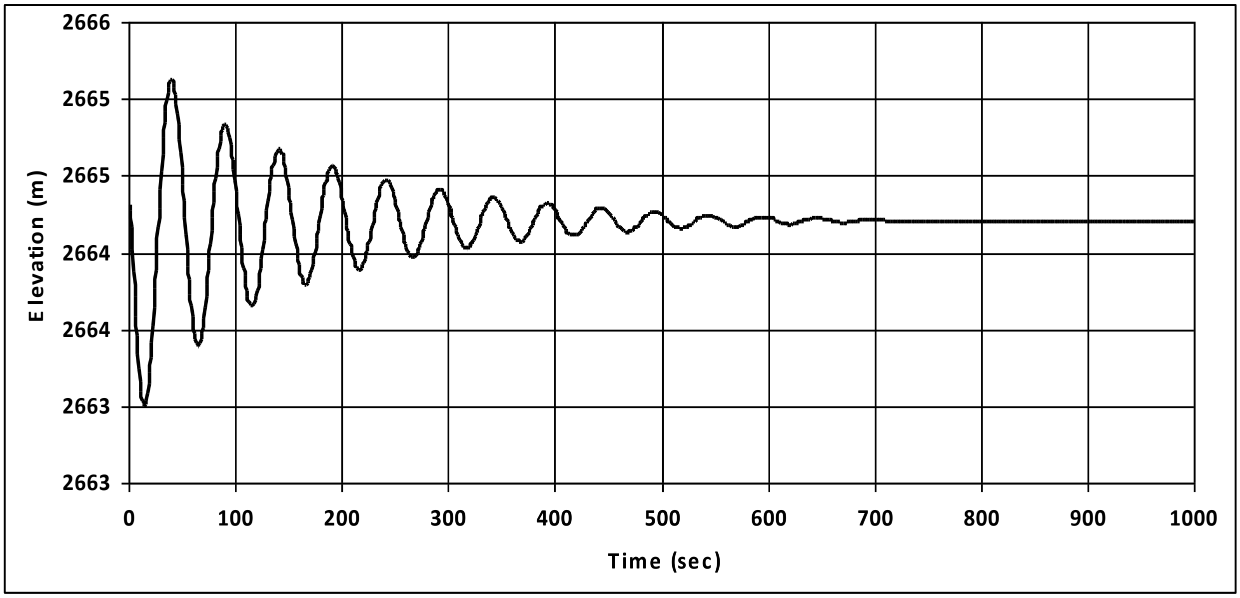


(b)

Figure 6: Surge analysis of Satpara hydropower project with surge shaft without chamber **(a)** downsurge and (b) upsurge



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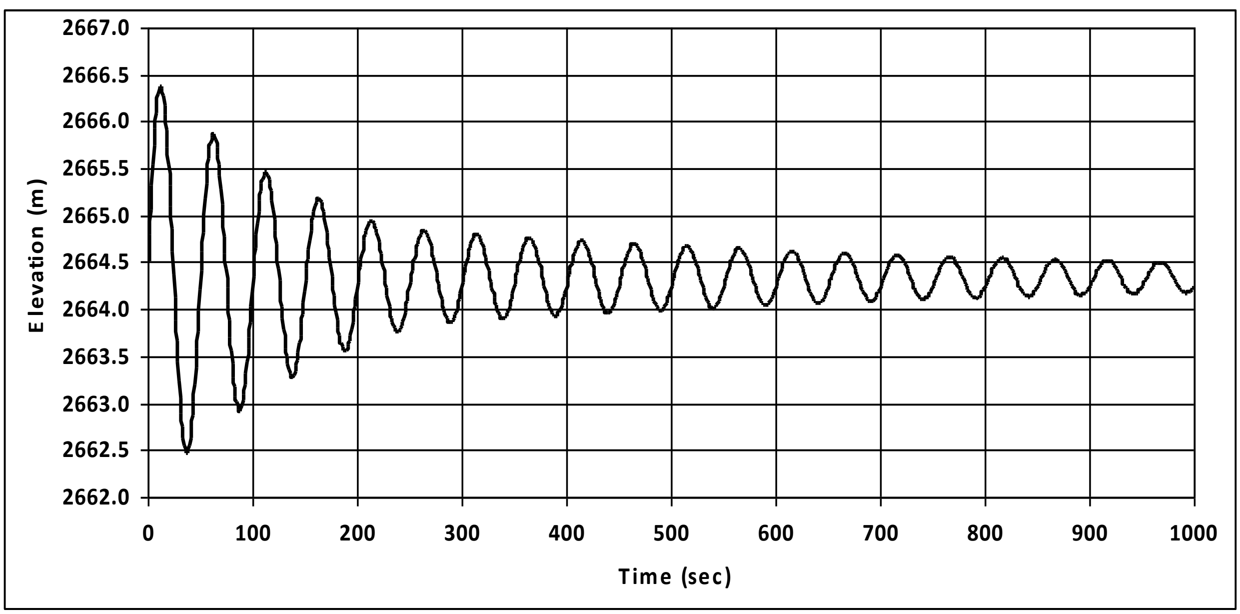

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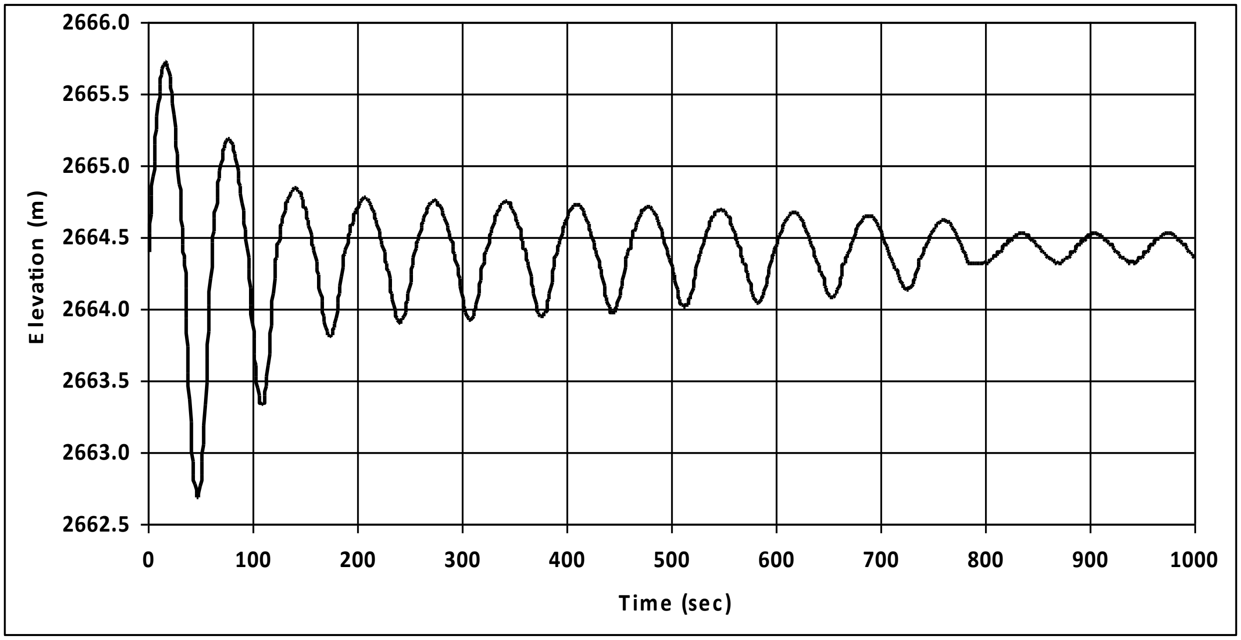
Figure 7 Surge analysis of Satpara hydropower project with surge shaft with lower chamber (a) downsurge and (b) upsurge



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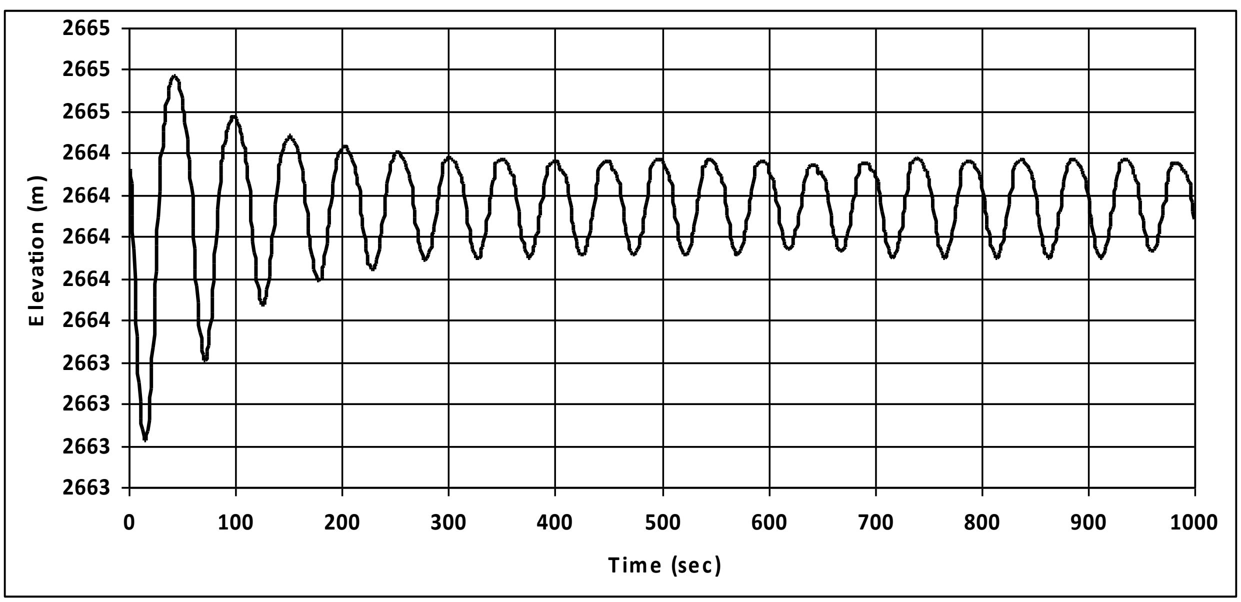

(b)

Figure 8: Surge analysis of Satpara hydropower project with surge tank with two chambers (a) upsurge and (b) down surge

3.0 Conclusions

From the results tabulated above, for Golen Gol hydropower plant the best surge tank system is two chambers as compared to surge tank with single chamber and without chamber. The dimensions of the two chamber surge tank are diameter of surge shaft is 9.0 m and height of the surge shaft is 100.0 m. This system gives minimum accumulated surge of 17.5 m.

Similarly for Satpara hydropower plant it was concluded that surge tank with lower chamber gives better results i.e accumulated minimum surge is 3.37 m, as compared to surge tank having two chambers and without chamber. The dimensions of the surge tank with lower chamber are diameter of surge shaft is.5.66 m and height of the surge shaft is 33.0 m.

4.0 References

[1] Allievi, L., (1913). ‘‘Teoria del colpo d’ariete,’’ Atti Collegio Ing. Arch. (English translation by Halmos EE 1929) ‘‘The Theory of Waterhammer,’’ Trans. ASME.

[2] Blaisdell, F. W., and Manson, P. W. (1967). "Energy loss at pipe junctions." *Irrigarion and Drainage Divsion* Proceedings of ASCE, vol. 93(IR3), September, 59-78

[3] Chaudhry, M. H. (1987).Applied Hydraulic Transients, Second ed. Van Nostrand Reinhold Co., New York, N. Y.

[4] Gardel, A. (1956). *Chambres d’Equilibre*, Lausanne, Switzerland.

[5] Ito, H., and Imai, K. (1973). “Energy losses at 90 degree pipe junctions.” *J. Hydr.Div.*, ASCE, 99(9), 1353-1368.

[6] Jaeger, C. (1954). “Present Trends In Surge Chamber Design”, Proc.Inst. Mech. E., vol. 108.

[7] Jaeger, C. (1958). “Contribution to the Stability Theory of Systems of Surge Tanks,” Trans. ASME, vol. 80 pp. 1574-1584, 1958.

[8] Jaeger, C. (1960). A review of Surge-Tank Stability Criteria, J. Basic Eng., Trans. ASME, pp 765-783, Dec., 1960.

[9] Jaeger, C. (1963). “The theory of Resonance in Hydropower Systems, Discussion of Incidents and Accidents Occurring in Pressure Systems,” Trans, ASME, vol, 85, ser. D. p. 631, Dec., 1963.

[10] Mosonyi, E., and Seth, H. B. S. (1975). “The surge tank a device for controlling water hammer,” Water power and Dam construction, Vol. 27(2).

[11] Parmakian, J. (1963). *Waterhammer Analysis*. Dover Publications, Inc., New York, New York.

[12] Thoma, D., (1910). *Zur Theorie des Wasserschlosses bei Selbsttaetig Geregelten Turbinenanlagen, Oldenburg, Munchen, Germany.*

[13] Wylie, E. B., and Streeter, V. L. (1993). *Fluid Transients in Systems*. Prentice-Hall, Englewood Cliffs, New Jersey.