Supplementary file:

Dam-break flood calculation results and investigation data of the Ertan Hydropower Station

We have conducted a detailed study on dam-break calculation of the Ertan Hydropower Station and compiled a report: *Analysis report on dam-break flood of the Ertan Hydropower Station*. This report contains eight chapters, 280 pages and all written in Chinese, it is difficult to translate all of them. Due to the flooding scenario is a full dam break in our study, we only translated the main content of full dam-break calculation process.

1. *Basic equation*

The evolution of the flood wave downstream is calculated by Saint Venant equation, and the Saint Venant equation is:

$$\begin{cases} \dfrac{\partial Q}{\partial x} + \dfrac{\partial (A + A_0)}{\partial t} - q = 0 \\ \dfrac{\partial Q}{\partial t} + \dfrac{\partial (Q^2 / A)}{\partial x} + gA\left(\dfrac{\partial h}{\partial x} + S_f + S_e\right) = 0 \end{cases} \tag{1}$$

Where:

Q: Discharge at dam site (m^3/s);

A: Effective cross section area (m^2);

A_0: Area of beach land (m^2);

x : Distance along the direction of water flow (m);

t: Time (s);

q: Lateral inflow or outflow (m^3/s);

g: Gravity (m/s^2)

S_f: Friction ratio drop, $S_f = \dfrac{n^2 |Q| Q}{2.21 A^2 R^{4/3}}$

S_e: local head loss, $S_e = \dfrac{k\Delta (Q/A)^2}{2g\Delta x}$

h: Water elevation (m);

k: Coefficient of contraction.

2. *Computational method*

Presimann's four-point eccentric implicit scheme is used to solve the equations (1) and the values of h and Q can be obtained at any time. Implicit difference equation can be written as:

$$\theta\left[\frac{Q_{i+1}^{j+1}-Q_i^{j+1}}{\Delta x_i}\right]+(1-\theta)\left[\frac{Q_{i+1}^{j}-Q_i^{j}}{\Delta x_i}\right]+\frac{1}{2\Delta t_j}\left[\left(A+A_0\right)_i^{j+1}+\left(A+A_0\right)_{i+1}^{j+1}-\left(A+A_0\right)_i^{j}-\left(A+A_0\right)_{i+1}^{j}\right]=0 \tag{2}$$

$$\begin{aligned}&\frac{1}{2\Delta t_j}\left[Q_i^{j+1}+Q_{i+1}^{j+1}-Q_i^{j}-Q_{i+1}^{j}\right]+\frac{\theta}{\Delta x_i}\left[\left(\frac{Q^2}{A}\right)_{i+1}^{j+1}-\left(\frac{Q^2}{A}\right)_{i}^{j+1}\right]\\&\quad+g\overline{A}^{j+1}\left[\frac{1}{\Delta x_i}\left(h_{i+1}^{j+1}-h_i^{j+1}\right)+\overline{S}_f^{j+1}+S_e^{j+1}\right]+\frac{1-\theta}{\Delta x_i}\left[\left(\frac{Q^2}{A}\right)_{i+1}^{j}-\left(\frac{Q^2}{A}\right)_{i}^{j}\right]\\&\quad+g\overline{A}^{j}\left[\frac{1}{\Delta x_i}\left(h_{i+1}^{j}-h_i^{j}\right)+\overline{S}_f^{j}+S_e^{j}\right]=0\end{aligned} \tag{3}$$

Where,

$\overline{A}=\left(A_i+A_{i+1}\right)/2$

$\overline{S}_f=n^2\overline{Q}\left|\overline{Q}\right|\Big/\left(2.21\overline{A}^2\overline{R}^{4/3}\right)$

$\overline{Q}=\left(\overline{Q}_i+\overline{Q}_{i+1}\right)/2$

$\overline{R}=\overline{A}/\overline{B}$

$\overline{B}=\left(B_i+B_{i+1}\right)/2$

3. *Initial and boundary conditions*

3.1 Initial conditions

In solving the unsteady flow equation by difference equations, the water level h and discharge Q of each section must be calculated at the initial time (t = 0).

$$Q_i=Q_{i-1}+q_{i-1}\quad (i=2,3,\cdots,N) \tag{4}$$

Where,

Q_i---------------- Discharge at cross section i;

Q_{i-1}-------------- Discharge at cross section i-1;

q_{i-1}--------------- Lateral inflow or outflow at cross section i-1 to cross section i;

The water level at the initial moment is calculated by the following formula:

$$\begin{aligned}&\frac{\left(Q^2/A\right)_{i+1}-\left(Q^2/A\right)_i}{\Delta x_i}\\&\quad+g\left[\frac{A_i+A_{i+1}}{2}\right]\left[\frac{h_{i+1}-h_i}{\Delta x_i}+\frac{n^2\left(Q_i+Q_{i+1}\right)^2\left(B_i+B_{i+1}\right)^{4/3}}{2\left(A_i+A_{i+1}\right)^{10/3}}\right]=0\end{aligned} \tag{5}$$

3.2 Boundary conditions

(1) Upstream boundary conditions, reservoir outflow line Q(t);

(2) Downstream boundary conditions, water level to discharge relation curve;

If the downstream end flow is controlled by the river channel, it can be calculated by Manning equation:

$$Q_N = \frac{1.49}{n} A_N^{5/3} B_N^{2/3} \left[\frac{h_{N-1} - h_N}{\Delta x_{N-1}} \right]^{1/2} \quad (6)$$

If the downstream end flow is controlled by a building (such as a dam), the following relationship can be used as the downstream boundary condition:

$$Q_N = Q_b + Q_s \quad (7)$$

4. *Roughness of river channels*

Different rivers have different rough coefficients.

When $\frac{n_{max}}{n_{min}} > 1.5 \sim 2$:

$$n = \left(\frac{\chi_1 n_1^{1.5} + \chi_2 n_2^{1.5} + \cdots\cdots + \chi_m n_m^{1.5}}{\chi_1 + \chi_2 + \cdots\cdots + \chi_m} \right)^{\frac{2}{3}} \quad (8)$$

When $\frac{n_{max}}{n_{min}} < 1.5 \sim 2$:

$$n = \frac{\chi_1 n_1 + \chi_2 n_2 + \cdots\cdots + \chi_m n_m}{\chi_1 + \chi_2 + \cdots\cdots + \chi_m} \quad (9)$$

Channel Material	***Constant n***
clean, uncoated cast iron	0.013-0.015
clean, coated cast iron	0.012-0.014
dirty, tuberculed cast iron	0.015-0.035
riveted steel	0.015-0.017
lock-bar and welded	0.012-0.013
galvanized iron	0.015-0.017
brass and glass	0.009-0.013
Wood Stove Pipe	
• small diameter	0.011-0.012
• large diameter	0.012-0.013

Concrete	
• with rough joints	0.016-0.017
• dry mix, rough forms	0.015-0.016
• wet mix, steel forms	0.012-0.014
• very smooth, finished	0.011-0.012
vitrified sewer	0.013-0.015
common-clay drainage tile	0.012-0.014
asbestos	0.011
planed timber	0.011
canvas	0.012
unplaned timber	0.014
brick	0.016
rubble masonry	0.017
smooth earth	0.018
firm gravel	0.023
corrugated metal pipe	0.022
natural channels, good condition	0.025
natural channels with stones and weeds	0.035
very poor natural channels	0.060

5. *Outlet flow analysis*

The dam break caused by many extreme cases, and there are many uncertainties in its formation. The Ertan Dam is a concrete hyperbolic arch dam, according to the construction materials and its density, this dam can be considered as a rigid dam and the collapse mode is transient collapse.

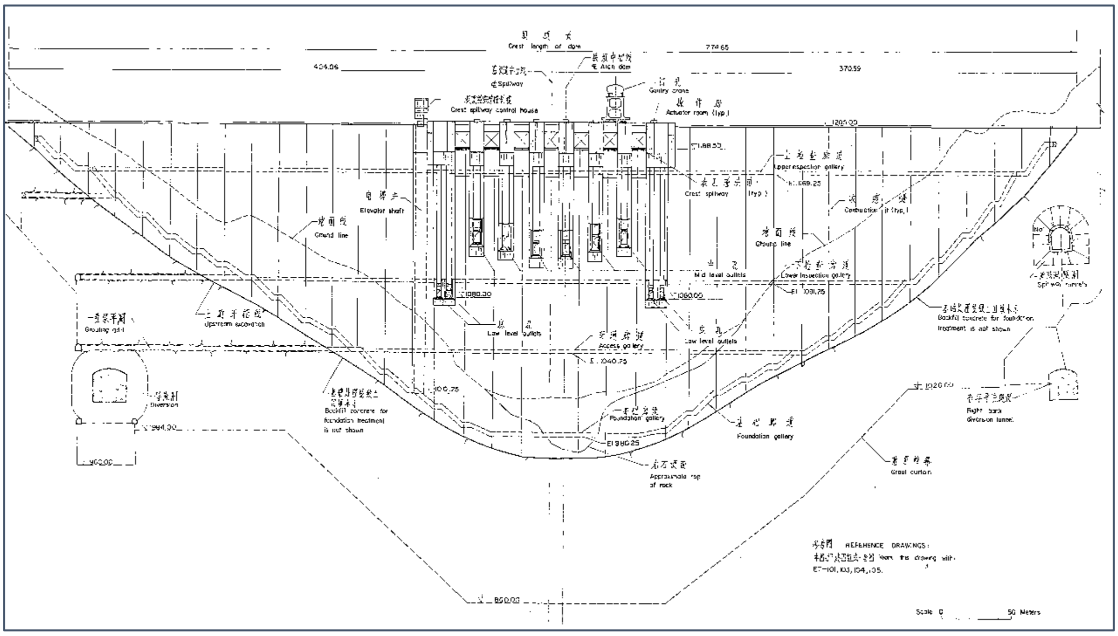

Figure1 Longitudinal section diagram of the Ertan dam

Figure1 is the longitudinal section diagram of the Ertan dam. The dam body was divided into 3 layers, with 7 surface holes, 6 middle holes and 4 bottom holes. There are also two spillways on the right bank. The flow of the dam site consists of two parts, one is the flow through a broad-crested weir; the other is the flow through flood releasing structures. That is:

$$Q = Q_b + Q_s \tag{10}$$

The flow through a broad-crested weir Q_b:

$$Q_b = C_1(h-h_b)^{1.5} + C_2(h-h_b)^{2.5} \tag{11}$$

Where:

$$C_1 = 3.1 b_i C_v K_s$$

$$C_2 = 2.45 Z C_v K_s$$

$$h_b = h_d - (h_d - h_{bm})\frac{t_b}{\tau} \quad (t_b \le \tau)$$

$$h_b = h_{bm} \quad (t_b > \tau)$$

$$b_i = b t_b / \tau \quad (t_b \le \tau)$$

$$b_i = b \quad (t_b > \tau)$$

$$C_v = 1.0 + 0.023 Q_b^2 / \left[B_d^2 (h-h_{bm})^2 (h-h_b)\right]$$

$$k_s = 1.0 \qquad \text{when } \frac{h_t - h_b}{h - h_b} \le 0.67$$

$$k_s = 1.0 - 27.8\left[\frac{h_t - h_b}{h - h_b} - 0.67\right]^3 \quad \text{when } \frac{h_t - h_b}{h - h_b} > 0.67$$

The flow through flood releasing structures Q_s:

$$Q_s = C_s L_s (h-h_s)^{1.5} + C_g A_g (h-h_g)^{0.5} + C_d L_d (h-h_d)^{1.5} + Q_t \tag{12}$$

$$I - Q = ds/dt$$

$$(I + I')/2 - (Q + Q')/2 = \Delta S / \Delta t$$

$$\Delta S = \left(A_s' + A_s\right)(h-h')/2$$

$$\left(A_s + A_s'\right)(h-h')/\Delta t + C_1(h-h_b)^{1.5} + C_2(h-h_b)^{2.5} + C_s L_s (h-h_b)^{1.5} + C_g A_g (h-h_g)^{0.5} + C_d L_d (h-h_d) + Q_t + Q' - I - I' = 0$$

6. *Theoretical Analysis of maximum Flood in Dam break*

Hydraulic calculation of dam-break wave is an important research subject, and the theoretical solution of dam-break flood is of great significance. The Ritter solution is a representative research result, which is briefly introduced in this supplementary file. For a rectangular channel with a flat-bottomed prism, no water downstream, and the dam collapse instantaneously, the Ritter equation is as follows:

Water depth of dam site section: $h = \frac{4}{9} h_0$

Maximum flow of dam section: $Q_{max} = \frac{8}{27} B_0 h_0 \sqrt{g h_0}$

Flood of dam break process is shown in Fugure2:

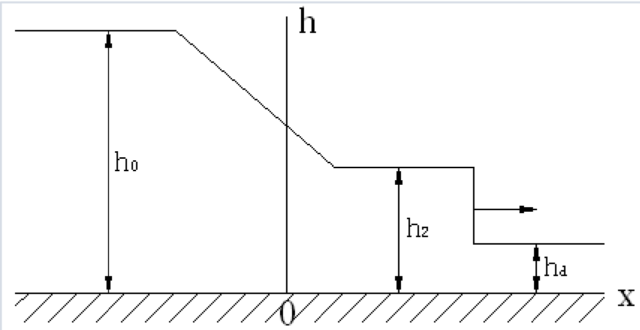

Figure2

In this figure, x is the distance between the dam site section and downstream section; h_0 is the upstream water depth; h_d is the downstream water depth, h_2 is the shock wave water depth and ζ is the shock wave velocity.

$$h = \frac{1}{9}(2 - x^*)^2 h_0 \tag{13}$$

$$Q = \frac{2}{27}(1 + x^*)(2 - x^*)^2 B_0 h_0 \sqrt{g h_0}\ ,\quad x^* = \frac{x}{t\sqrt{g h_0}} \tag{14}$$

$$(\frac{h_2}{h_0})^3 - 9(\frac{h_2}{h_0})^2(\frac{h_d}{h_0}) + 16(\frac{h_2}{h_0})^{3/2}(\frac{h_d}{h_0}) - (\frac{h_2}{h_0})(\frac{h_d}{h_0})(\frac{h_d}{h_0} + 8) + (\frac{h_d}{h_0})^3 = 0 \tag{15}$$

$$\frac{\xi}{\sqrt{g h_0}} = 2\frac{h_2}{h_0}\frac{1 - \sqrt{\frac{h_2}{h_0}}}{\frac{h_2}{h_0} - \frac{h_d}{h_0}} \tag{16}$$

7. *Full dam break*

Dam-break condition: water level in dam site: 1205m; instantaneous dam failure. According to the theory and mathematical formula above, we calculate the dam-break flood.

We listed 4 typical cross sections in Yalong River and Jinsha River, as shown in table1.

Table1 Typical cross section

Place name	Cross Section number	Place name	Section number
Ertan dam site	60	Jinsha River junction	90
Deshi town	70	Wande town	192

7.1 Flow and water level process of the typical CS

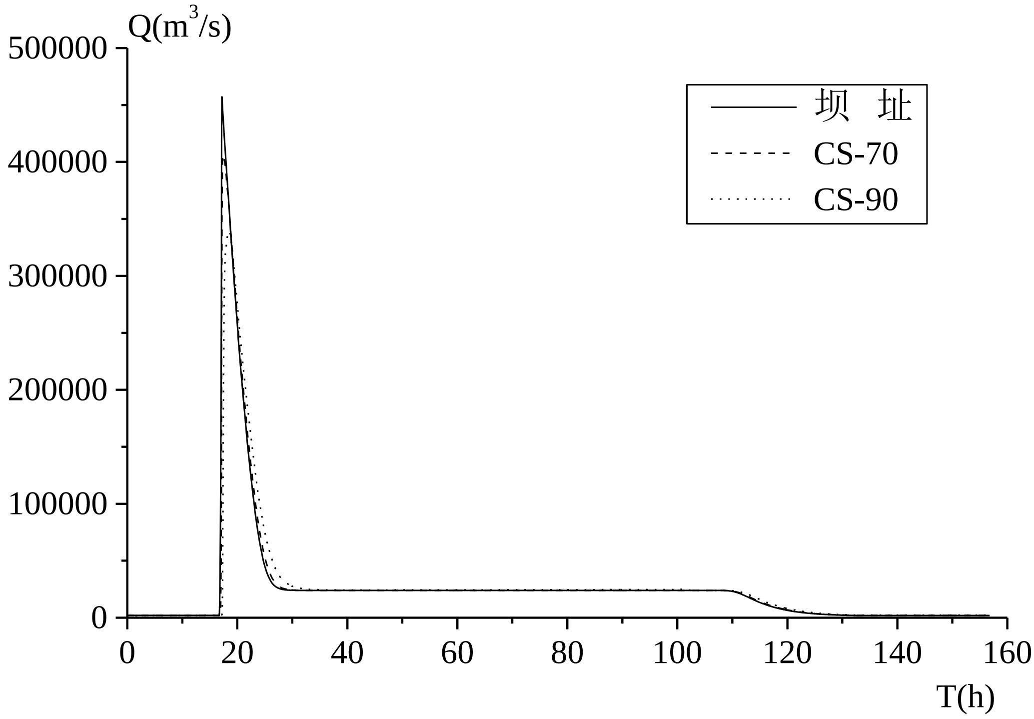

Figure3 Flow process line diagram of dam site, CS70 and CS90

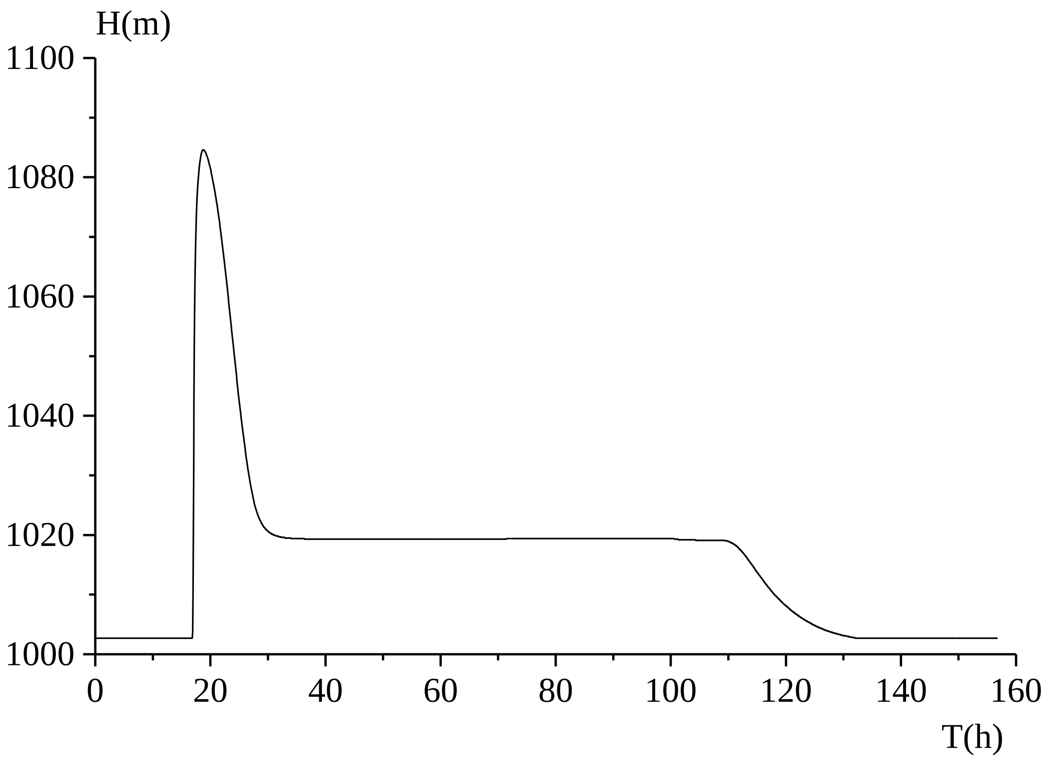

Figure4 Water level process line diagram of CS70

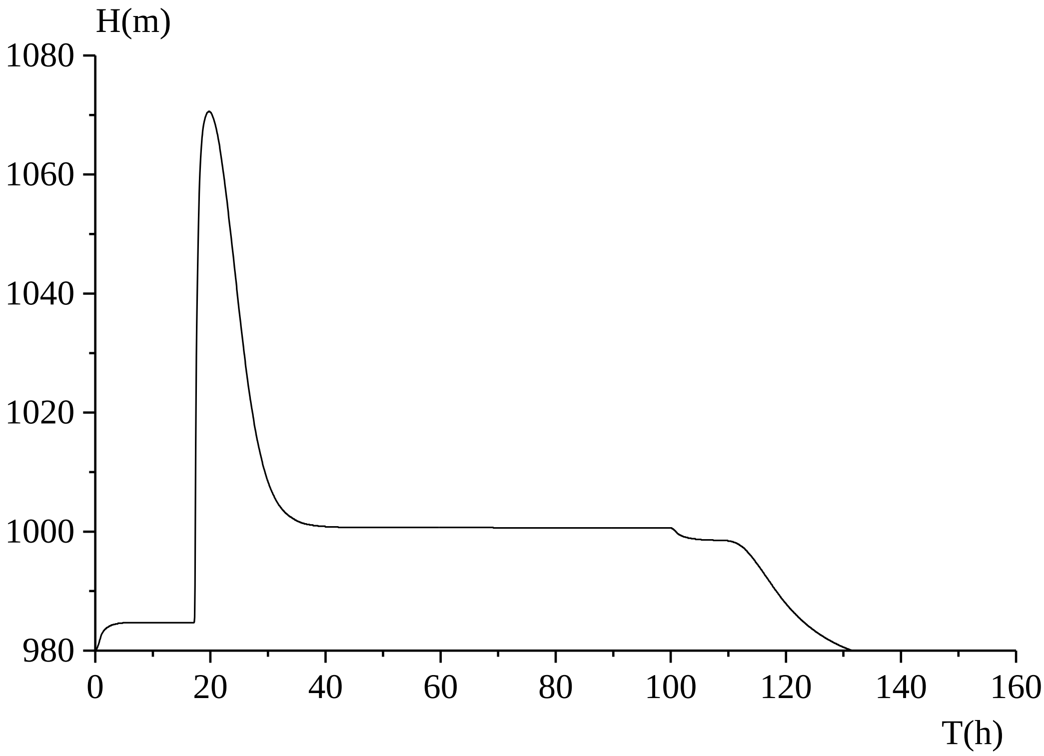

Figure5 Water level process line diagram of CS90

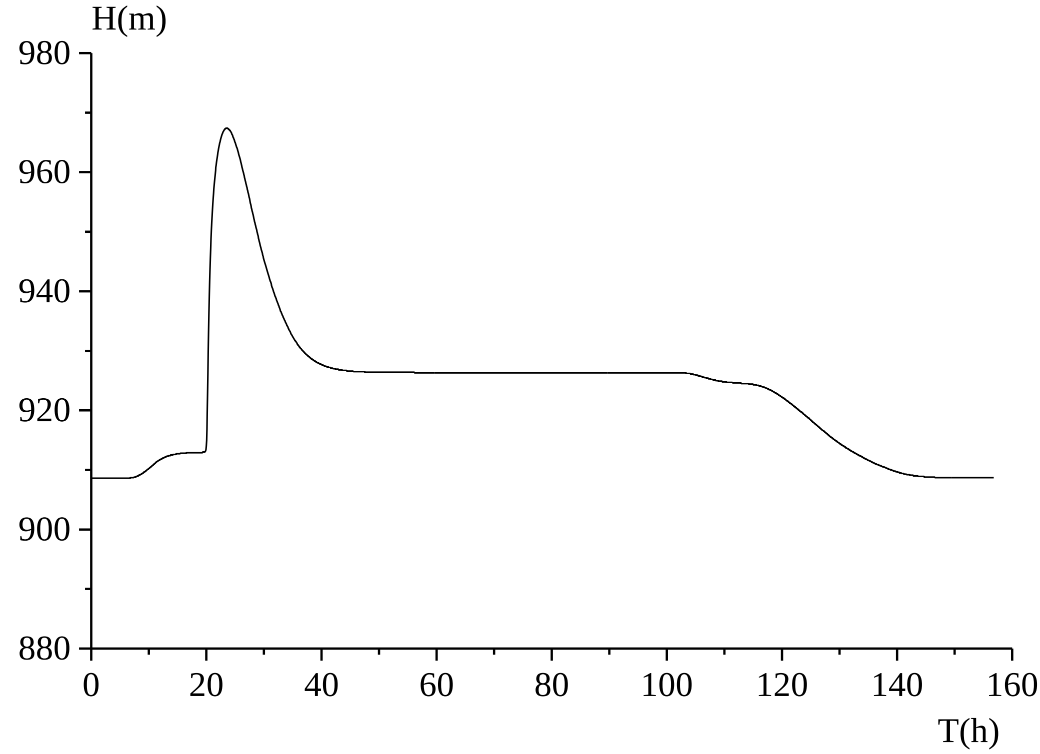

Figure6 Water level process line diagram of CS192

7.2 Evolution of dam-breaking floods

In the case of the full collapse of the Ertan dam, the maximum flood peak flow at the dam site is 457,000 m^3/s, which is more than 19 times that of the check flood flow of 23,900 m^3/s. After 1.67 hours, the flood peak moved from the Yalong River to Panzhihua city, with a flood level of 138 m (the flood level, which refers to the flood elevation calculated from the zero elevation of the Yellow Sea datum in China, is different from the flood depth). After 4.74 hours, the flood peak reached Jiangbian town, with a flood level of 106 m. After 9.94 hours later, the flood peak reached Tuobuka town, with a flood level of 74.5 m.

According to investigation data of the Ertan Hydropower Station and dam-break flood calculation results, the statistical data of 35 representative residential areas are shown in Table 2.

Table2 Statistical data of 35 representative residential areas

Num	Place Name	Population	Flood Arrival Time (h)	Flood Level (m)	Evacuation Time (h)	Local GDP (Million $)
1	Santan Village	500	0.24	147	0.267	12
2	Oufangyingdi	100	0.24	147	0.233	8
3	Gantianbao	200	0.60	144	0.50	10
4	Jinhe	200	0.60	144	0.50	26
5	Panzhihua City flooded area	129,100	1.67	138	2.0	1500
6	Xinlong Village	800	2.28	133	0.267	17.6
7	Hepiao Village	300	2.80	129.5	0.233	11
8	Yuzuo	1000	3.12	127	0.20	40
9	Lazuo	1000	3.12	127	0.233	40
10	Lumuzu	20	3.50	122	0.267	3
11	Yishala Ecological zone	200	3.70	120	0.247	11.3
12	Tuoji Factory	100	3.90	118.5	0.367	9
13	Luomodi	200	4.20	112	0.350	25
14	Yimoshidu	500	4.50	108	0.233	19
15	Jiangbian Town	800	4.74	106	0.40	75
16	Bingnong Village	100	5.22	103.5	0.40	9
17	Xikangzhi	150	5.28	103	0.433	14
18	Wande	200	5.28	103	0.667	21
19	Jiangzhu	200	5.64	100	0.564	20
20	Xinshan Village	150	6.10	96.2	0.530	10
21	Xin'an	200	6.36	95	0.636	60
22	Wumushu	40	6.85	92	0.233	3
23	Pulong	80	6.85	92	0.933	12
24	Yituzhuang	100	6.90	92.5	0.333	13
25	Luji	80	7.38	93	0.20	11
26	Longshu	30	7.62	91.6	0.60	16
27	Makou	40	7.78	89.5	0.8	5
28	Wujia Village	100	7.92	88.6	0.30	22
19	Huaizuo	50	8.39	85.3	0.56	6
30	Dalishu	100	8.55	84	0.43	24
31	Luhe Village	50	8.64	83.4	0.670	26
32	Pumie	50	8.82	82.7	0.187	21
33	Yanba	60	9.06	81	1	23
34	Yinmin	50	9.30	78	0.87	34
35	Tuobuka Town	200	9.94	74.5	0.400	60