# The Simulation of a Downstream Flooding Process with Dam-Break Model

**Lei Qiao, Wei Liu, Feng Xu**
**Beijing University of Technology, China PR**

## Abstract

This paper mainly discusses a dam-break model and its application to the downstream area of Lake Murray Dam. According to the model and the data obtained, we simulate the flooding downstream process in the event there is a catastrophic earthquake that breaches the dam and estimate the magnitude and extent of damage caused by the dam-break to the downstream area.

We analyze the effect of dam-break parameters on breach outflow. Since the dam-break can be approximately considered as an instantaneous total or partial dam- break model, we introduce a classic formula used to calculate the peak breach discharge, which affects the downstream flows. In addition, contour diagram about the area of Lake Murray Dam can be made after processing the data obtained from Internet [1], which make great contribution to simulating the flood process.

On the basis of efforts above, we propose a model for the lateral flow exchange between tributary and main channel based on flow equilibrium. For unsteady flows, Saint Venant equation [2] is one of the approaches to solving this kind of problems. Preissmann Method [2] is adopted in discretion of the Saint Venant equations to compile computer program to solve the water level $Z$ and discharge rate $Q$ at different cross sections. Considering the flood could overflow the river band in a short time, Saint Venant equations is not applicable in this case. So we design a flood routing algorithm based on concept of energy in open channel. This energy algorithm can well describe the flood distribution according to changes of elevation. Therefore, the entire dam-break process can be simulated and recorded to estimate the magnitude and extent of damage to the downstream area.

According to the test algorithm, we draw the conclusion that even in the worst situation, the flood can extend back to Rawls Creek about 1.13 miles, and if considering other creek's function of receiving flood other than Rawls Creek, the water would not reach up to the S.C. State Capitol Building at the peak of flood.

## Introduction

Dam-break process is a complex procedure which is affected by many factors, so modeling the flooding downstream in extreme events are difficult because of the steep gradient inherent in the problem. Therefore, we have to simplify the model by ignoring some minor ingredients.

We divide dam-break process into 3 phases, each of which has a model describing a session of the process respectively.

- Since the dam-break can be approximately considered as an instantaneous total or partial dam-break model, we introduce a classic formula used to calculate the peak breach discharge.

- Based on flow equilibrium, we develop a model for the lateral flow exchange between tributary and main channel, which can describe the relation between the flow of tributary and main channel.

- We adopt Preissmann Method in discretion of the Saint Venant equations to calculate the water level $Z$ and discharge rate $Q$ at different cross sections during the entire process.

## Assumptions

- Since the damage of the dam is caused by a catastrophic earthquake, the breach of dam is formed instantaneously and totally. In other words, the shape and area of the breach does not extend as time elapse.

- The position of the dam breach is near the river bank, that is, water flowing from the dam is mainly along the river, while water flowing along other direction is so little that we could ignore it.

- Because dam-break does not take place in flood season, the effect of precipitation is insignificant.

- During the simulation process, nobody participate in the repair of dam or make efforts to restrict the flood. Water flows spontaneously and is not affected by people.

- The water storage capacity of Lake Murray is enormous, so the discharge rate of water flowing from the dam is treated as constant in quite a few periods of time after the dam-break.

- Atlantic Ocean is seen as an infinite reservoir which can receive the water flowing from upstream without increased depth.

- The watercourse is an open-channel, so the discharge rate of water only depends on the altitude and the initial discharge rate of water, and has nothing to do with time.

# Relevant Parameters and Variables

**Table 1.** Table of symbols

| Symbol | Description | Units or Value |
|---|---|---|
| $B$ | overall width of the dam | $f$ |
| $H$ | depth of Lake before dam-break | $f$ |
| $Q_m$ | peak breach discharge | $f^3/s$ |
| $g$ | acceleration of gravity | $32.185 f/s^2$ |
| $A$ | cross sectional area of flood | $f^2$ |
| $\lambda$ | flow coefficient | / |
| $m$ | shape coefficient of cross section | / |
| $t$ | time | $s$ |
| $Z$ | water level of rivers | $f$ |
| $Q$ | discharge rate of main channel | $f^3/s$ |
| $q$ | discharge rate of tributary flow | $f^3/s$ |
| $V$ | velocity of flow | $f/s$ |

| | | |
|---|---|---|
| $E$ | specific energy | $f$ |
| $E_c$ | minimal value of $E$ | $f$ |
| $Hb$ | height of channel bottom | $f$ |

# Problem Analysis

### Two Particular Problems

- Rawls Creek is a year-round stream that flows into the Saluda River a short distance downriver from the dam. How much flooding will occur in Rawls Creek from a dam failure, and how far back will it extend?

- Could the flood be so massive downstream that water would reach up to the S.C. State Capitol Building, which is on a hill overlooking the Congaree River?

### Solution to the Problems

- The former is a problem dealing with the relationship between tributary and main channel during flood routing. The general method is to add the discharge of tributary to that of main channel of the same instant. In fact, the tributary acts as a conditioning reservoir, which is an important factor to sudden flood caused by dam collapse. According to flow equilibrium and flow continuity condition, the water flowing into the tributary from the main channel during the uprising period of main channel flood, which is called “lateral flow exchange” have a dynamic equilibrium relationship between the water flowing into the tributary from the main channel and water flowing into the main channel from the tributary. When the flow from the tributary in to the main channel equals the flow from the main channel into the tributary, the flow reach up the farthest distance of the tributary.

- The latter is a problem tackling with water flowing from low place into high place. Whether water would reach up to the S.C. State Capitol Building depends on the flow rate, the width of the Congaree River and the terrain characteristic near the S.C. State Capitol Building, having nothing to do with time.

# Data Processing

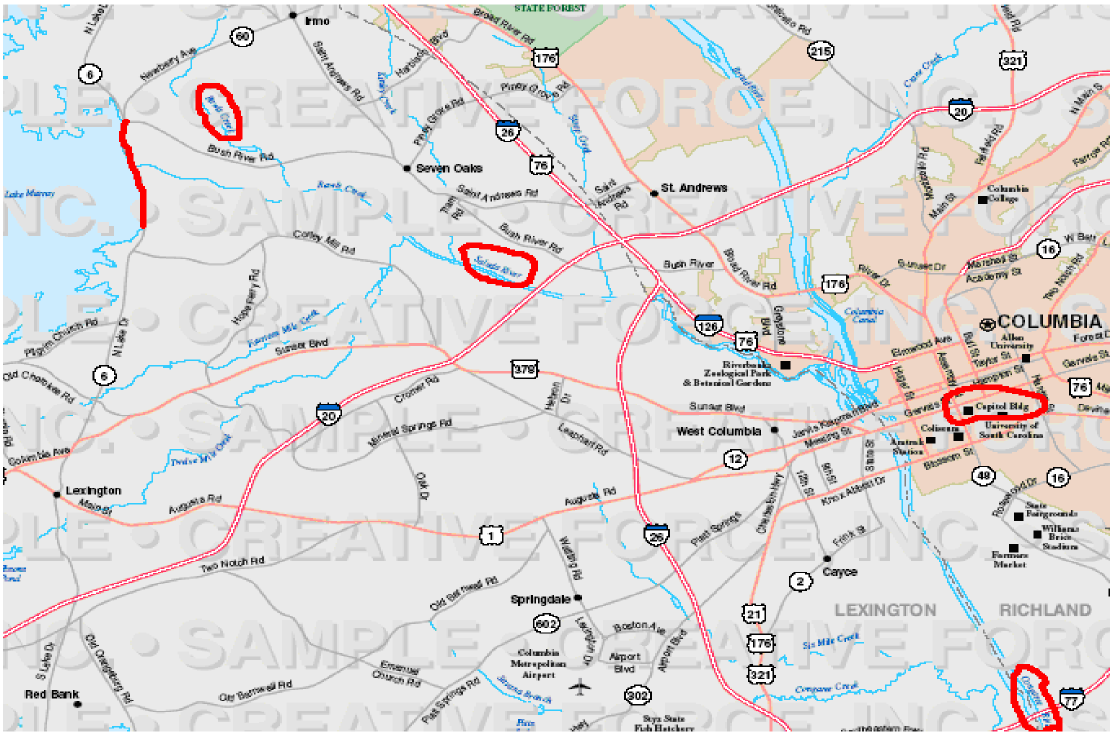
- **Geographical Background**

Lake Murray is named after William Murray, the engineer who, with his partner T. C. Williams, conceived and persevered until "the world's largest earthen dam" at that time was finished. Lake Murray lies in the center of South Carolina. Water in Lake Murray is used for hydropower generation and there is no dedicated flood storage.

Saluda River we research is the section lying form Murray Dam to Congaree River, which is main routing of Dam-flood. Within this section, there are many tributaries coming into Saluda River. Because of the limitation of the data resources, in our model, we only consider several relatively large tributaries which contribute much to the main river discharge rate, such as Rawle Creek, Fourteemale Creek, Twelvemile Creek, Kinley Creek, Stoops Creek, Broad River and Congaree River.

The main river begins near the North Carolina border. As it runs to the sea, it fills Greenville Water reservoir, Greenwood Lake, and then Lake Murray. Past the dam, the Saluda joins the Congaree and Wateree rivers to flow to the Atlantic Ocean. Rawls Creek is a year-round stream that flows into the Saluda River a short distance downriver from the dam. The Congaree River is formed by the Saluda and Broad River with the gauging station on the Saluda River near Columbia. State Capitol Building located near upstream of Congaree River, approximately two miles away. Key points are circled in following map.

**Figure 1 The map about the area of Lake Murray Dam**

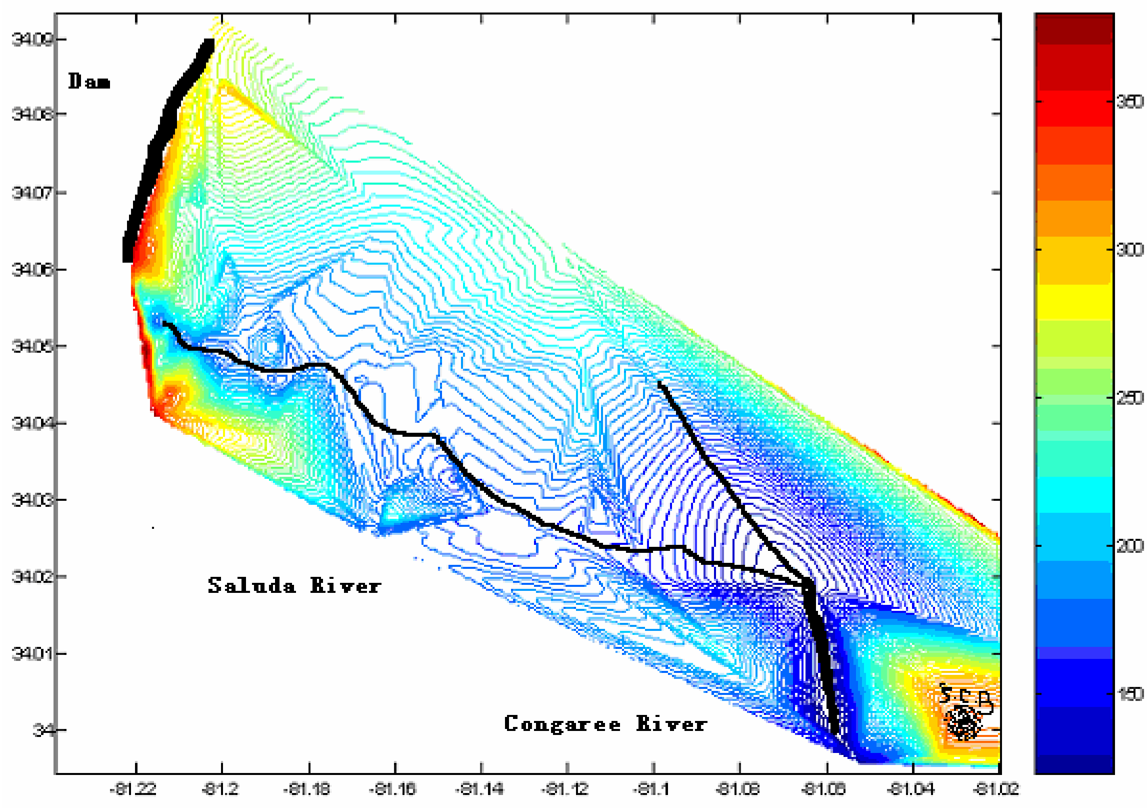



- **Physiognomy**

By making use of map-viewer at website [1] of the U.S. Geological Survey

(USGS), we could find elevation, distances, altitude, and longitude at every location of our interest scope. We sample hundreds points in that map system, covering a large enough area including area from Lake Murray to S.C. State Capitol Building. According to altitudes and longitudes, we locate the samples in Matlab, and drew a 3-demensional map of Lake. Murray - SC. State Capitol Building as follow, which is convenient for further modeling computation. By processing information shown in this map, we can numerate the changes of depth of rivers and height of dam and building, gauge the length and width of rivers and dam, and compute the area of lakes. All data are important for our flood-routing model. Our 3D map's contour diagram is as follow.

Figure 2 Contour diagram of Murray Lake - Sate Capitol Building




Lake Murray Dam itself was built over 200 feet tall. It runs a distance of a mile and a half across. The ground level of the dam is over a quarter of a mile thick. The lake that it forms is forty-one miles long and, in places, over fourteen miles wide.

- **Hydrograph**

We search about the information of the discharge rate of these rivers at water source website [3]. There is a monthly periodic discharge in Saluda River area and Congaree River. These information about the discharge rate can help us to estimate different damages of dam-breach caused by earthquake taking place in various seasons. From 1988 to present monthly mean discharges, we can learn that the greatest deal of discharge is in March, and the least one is in May. The monthly means of discharge rate in the four main water bodies are shown in following bar charts.

Figure 3 Saluda River mean discharge in a year

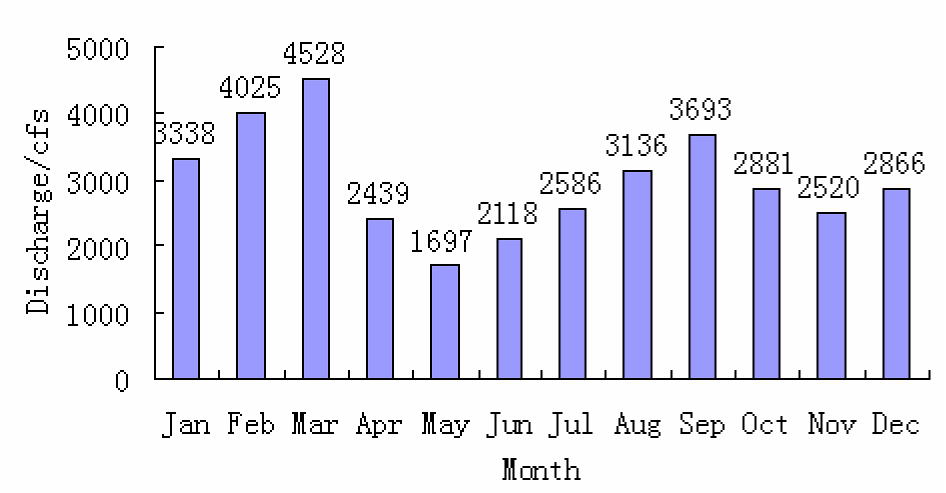
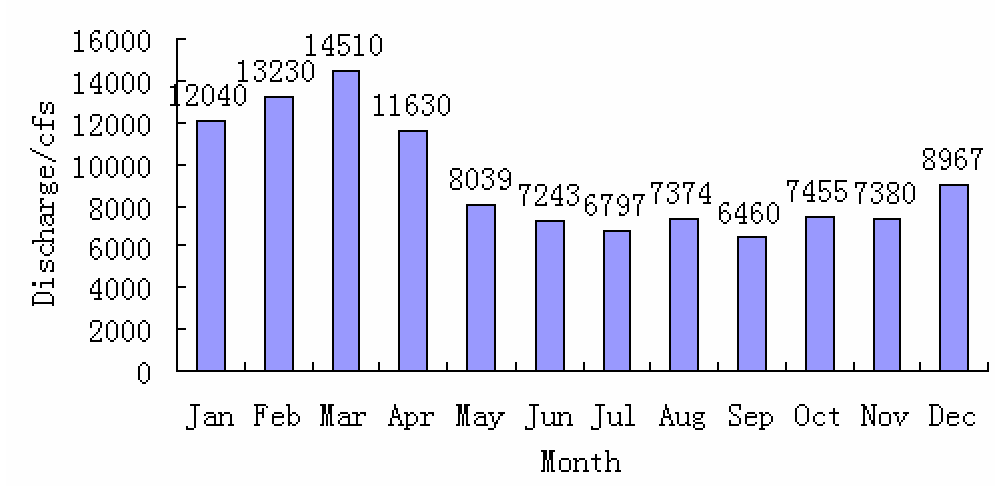



Figure 4 Congaree River mean discharge in a year




# Model Design

We simulate the flood disaster in 3 parts: the beginning of dam-break, lateral flow exchanges between tributary and main channel, and flood routing.

- **The Beginning of Dam-Break**

Because Lake Murray Dam is an earthen dam, and is breached by an abrupt and catastrophic earthquake, the dam-break can be approximately considered as an instantaneous total or partial dam-break process. We introduce a classic formula used to calculate the peak breach discharge.

Instantaneous total dam-break is a classical and simple problem. According to Saint Venant equations, assuming the river bed is horizontal and rounding off friction item, the reservoir is infinite and the depth of downstream river $h_2$ accords with requirement as follow:

$$h_2 \le \frac{H}{3.214}(\frac{2m}{2m+1})^2$$

The peak breach discharge formula is

$$Q_m = \lambda\sqrt{g}BH^{\frac{3}{2}} \tag{1}$$

$\lambda$ is a flow coefficient which is determined by the shape of cross section and initial discharge velocity of the river. The formula used to calculate $\lambda$ is

$$\lambda = m^{m-1}(\frac{2\sqrt{m}+\frac{u_0}{\sqrt{gH}}}{2m+1})^{2m+1} \tag{2}$$

Assuming $\frac{u_0}{\sqrt{gH}} \to 0$, thus $\lambda = m^{m-1}(\frac{2\sqrt{m}}{2m+1})^{2m+1}$

$m$ is the shape coefficient of cross section. If the shape of cross section is rectangle, $m=1$; if the shape is triangle, $m=2$; if the shape is between triangle and rectangle, $m \approx 1.1$; if the shape is parabola, $m = 1 \sim 2$

There is a table of calculation formula about the peak breach discharge with different shape of cross section.

**Table 2** the formula in different shape of cross section condition

| Shape of cross section | Discharge rate formula | Requirement | Shape coefficient |
|---|---|---|---|
| rectangle | $0.296\sqrt{g}BH^{\frac{3}{2}}$ | $h_2 \le 0.1384H$ | $m=1$ |
| triangle | $0.181\sqrt{g}BH^{\frac{3}{2}}$ | $h_2 \le 0.199H$ | $m=2$ |
| parabola | $0.23\sqrt{g}BH^{\frac{3}{2}}$ | $h_2 \le 0.175H$ | $m=1.5$ |

- **Lateral Flow Exchanges Between Tributary and Main Channel**

Some researches indicate that the lateral flow exchange of the tributary whose flow angle $\alpha$ is less than $70°$ is affected insignificantly by the angle value, and is only dependent on the tributary width at the junction of main channel and tributary, as well as on the gradient of the tributary.

As figure 5 shows, the main channel and the tributary converge together at the cross section $b-b$, $q_e(t)$ is the discharge rate flowing to the tributary, $q_0(t)$

is the original discharge rate of the tributary.

According to flow equilibrium and flow continuity condition, the water flowing

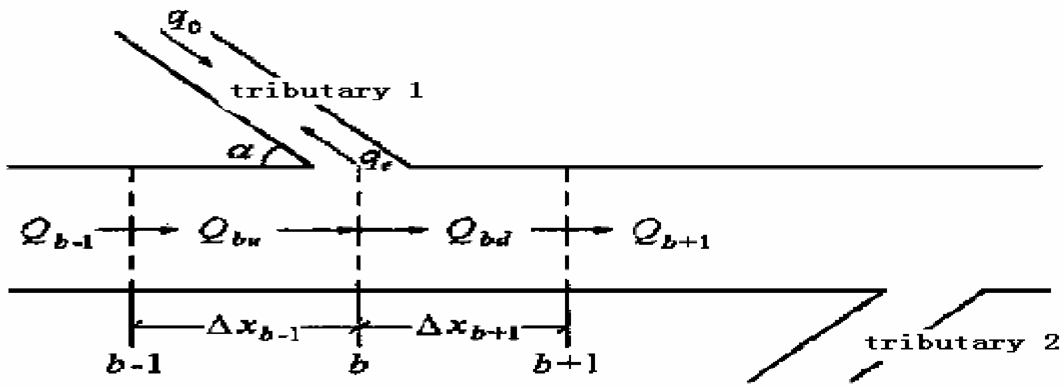



Figure 5 Junction of main channel and tributary

into the tributary from the main channel during the uprising period of main channel flood, which is called "lateral flow exchange" have a dynamic equilibrium relationship between the water flowing into the tributary from the main channel and water flowing into the main channel from the tributary as follows:

$$\begin{cases} \int_0^t [q_0(t) + q_e(t)]dt = \int_{h_0}^{h_b} F(h)dh \\ \frac{\partial Q}{\partial x} + \frac{\partial (A_0 + A)}{\partial t} = 0 \\ \frac{\partial Q}{\partial t} + \frac{\partial (Q^2 / A)}{\partial t} + gA\left(\frac{\partial h}{\partial x} + S_f + S_e\right) = 0 \end{cases} \tag{3}$$

In equations (3), $S_f = \frac{n^2 Q^2}{A^2 R^{4/3}}$ $S_e = \frac{\mathrm{k}(\mathrm{Q}/\mathrm{A})^2}{2\mathrm{gx}}$

- **Flood Routing**

The flood flow is a kind of unsteady and non-uniform flow, therefore the discharge rate $Q$ is a function of time and position, and the height of the flow $h$ is also a function of time and position, that is, $h = h(x,t)$ $Q = Q(x,t)$. So the flood routing model is developed based on the principles of Saint Venant equations:

$$\frac{\partial Q}{\partial x} + B_r \frac{\partial Z}{\partial t} = q \tag{4}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial (Q^2/A)}{\partial t}+gA\left(\frac{\partial h}{\partial x}+\frac{Q|Q|}{K^2}\right)+\frac{Q}{A}q=0 \tag{5}$$

Due to the difficulty of programming of partial differential equations, Preissmann Method [2] is adopted in discretion of the Saint Venant equations to compile computer program to solve the water level *Z* and discharge rate *Q* at different cross sections. The discrete equation form is as follows:

$$\begin{cases} f=\frac{\theta}{2}(\Delta f_{j+1}+\Delta f_j)+\frac{1}{2}(f_{j+1}+f_j) \\ \frac{\partial f}{\partial t}=\frac{\Delta f_{j+1}+\Delta f_j}{2\Delta t} \\ \frac{\partial f}{\partial x}=\frac{\Delta f_{j+1}-\Delta f_j}{2\Delta t}+\frac{f_{j+1}-f_j}{\Delta xj} \end{cases} \tag{6}$$

In Equation (6), $\Delta t$ is interval of time, $\Delta x$ is interval of distance along flow, $\theta$ is weight coefficient, $j$ is serial number of discrete samples of cross section.

Using Equation (6) to express $Q_j, \Delta Q_j$ in Equation (4) and Equation (5), we can obtain difference equations of $Q(x,t), Z(x,t)$ —— Equation (7) and Equation (8):

$$A_{1j}\Delta Q_j+B_{1j}\Delta Z_j+C_{1j}\Delta Q_{j+1}+D_{1j}\Delta Z_{j+1}=G_{1j} \tag{7}$$

$$A_{2j}\Delta Q_j+B_{2j}\Delta Z_j+C_{2j}\Delta Q_{j+1}+D_{2j}\Delta Z_{j+1}=G_{2j} \quad (j=1,2,.....,N+1) \tag{8}$$

In Equation (7) and Equation (8), coefficient is as follow:

$$A_{1j}=\theta; B_{1j}=\frac{\Delta x_j}{4\Delta t}(Br_{j+1}+Br_j); C_{1j}=\theta; D_{1j}=B_{1j}; G_{1j}=\Delta x_j(q_j+\theta\Delta q_j)-(Q_{j+1}-Q_j);$$

$$A_{2j}=\theta\Delta x_j\left|\frac{1}{2\theta\Delta t}-\frac{2Q_j}{A_j\Delta x_j}+g|Q_j|\frac{A_j+A_{j+1}}{K_j^2+K_{j+1}^2}+\frac{1}{2A_j}(q_j+\theta\Delta q_j)\right|;$$

$$B_{2j}=\theta Br_j\left|\left(\frac{Q_j}{A_j}\right)^2+0.5g(Z_{j+1}-Z_j)+\frac{g\Delta x_j(Q_j|Q_j|+Q_{j+1}|Q_{j+1}|)}{2(K_j^2+K_{j+1}^2)}-\frac{\Delta x_j Q_j}{2A_j^2}(q_j-\theta\Delta q_j)\right|$$

$$-\frac{g\theta\Delta x_j K_j (A_j + A_{j+1})(Q_j|Q_j| + Q_{j+1}|Q_{j+1}|)}{(K_j^2 + K_{j+1}^2)}\frac{dK_j}{dZ_j} - 0.5\theta g(A_j + A_{j+1});$$

$$C_{2j} = \theta\Delta x_j \left|\frac{1}{2\theta\Delta t} - \frac{2Q_j}{A_{j+1}\Delta x_j} + g|Q_{j+1}|\frac{A_j + A_{j+1}}{K_j^2 + K_{j+1}^2} + \frac{1}{2A_{j+1}}(q_j + \theta\Delta q_j)\right|;$$

$$D_{2j} = \theta Br_{j+1}\left|-\left(\frac{Q_j}{A_j}\right)^2 + 0.5g(Z_{j+1} - Z_j) + \frac{g\Delta x_j (Q_j|Q_j| + Q_{j+1}|Q_{j+1}|)}{2(K_j^2 + K_{j+1}^2)} - \frac{\Delta x_j Q_{j+1}}{2A_{j+1}^2}(q_j - \theta\Delta q_j)\right|$$

$$-\frac{g\theta\Delta x_j K_j (A_j + A_{j+1})(Q_j|Q_j| + Q_{j+1}|Q_{j+1}|)}{(K_j^2 + K_{j+1}^2)}\frac{dK_{j+1}}{dZ_{j+1}} + 0.5\theta g(A_j + A_{j+1});$$

$$G_{2j} = \frac{Q_j^2}{A_j} - \frac{Q_{j+1}^2}{A_{j+1}} - 0.5g(A_j + A_{j+1})(Z_{j+1} - Z_j) - \frac{g\Delta x_j (A_j + A_{j+1})(Q_j|Q_j| + Q_{j+1}|Q_{j+1}|)}{2(K_j^2 + K_{j+1}^2)}$$

$$-0.5\Delta x_j (q_j + \theta\Delta q_j)\left|\frac{Q_j}{A_j} + \frac{Q_{j+1}}{A_{j+1}}\right|;$$

Therefore, through calculating the coefficient in the discrete equations by computer, the entire dam-break process can be simulated and recorded to estimate the magnitude and extent of damage to the downstream area.

# Test Algorithm and Process Simulation

- **The Algorithm of Energy Concepts Theory**

To test our model's precision, we design a test algorithm to simulate the dam breach flood in real time. To fulfill this emulation, we make full use of our 3D map and knowledge of mechanics of fluids in order to generate connection between discharge and water depth, discharge and time. In this model we introduce Energy Concepts in open-channel flow [4].

Firstly, we find water depth and hypsography trend of spots along rivers. From our 3D map which logs elevation of surface of water, from website [3], which contains detailed discharge datum rivers, we can know:

$b = b(x, y)$: width of cross section along rivers

$El = El(x, y)$: elevation of surface flow along rivers

$Q' = Q'(x, y)$: discharge along rivers at no flood time

$x, y$ is coordinate of rivers, $x$ is heading with direction of river, and $y$ is vertically against $x$.

Then, we interpose the samples to obtain large enough quantity of data to locate the bottom of channels.

$$V'^2_{x+1,y} = 2g(El_{x+1,y} - El_{x,y}) \quad \& \quad V'_{0x} = \sqrt{2gH}$$

$$V'^2_{x,y+1} = 2g(El_{x,y+1} - El_{x,y}) \quad \& \quad V'_{0y} = V'_{0x}$$

$$h'_{x,y} = \frac{Q'_{x,y}}{V'_{x,y}}$$

$$Hb_{x,y} = H_{x,y} - h'_{x,y}$$

$V'$ : flow velocity at no flood time

$V'_{0x}$: initial flow velocity at $x$ direction

$V'_{0y}$: initial flow velocity at $y$ direction

$h'$: water depth at no flood time

$$E = h + \frac{Q^2}{2gA} \tag{9}$$

$V$ is the velocity of flow, $E$ is specific energy, which is the sum of the flow depth $h$ and kinetic energy head $V^2/2g$.

If the slope of depth changes very slowly gradually, $E_{x+1} = E_x + \Delta Hb$. In another word, equation is conditional and it has minimum value $E_c$.

$$\frac{dE}{dh} = 1 - \frac{Q}{gA^3}\frac{dA}{dh} \tag{10}$$

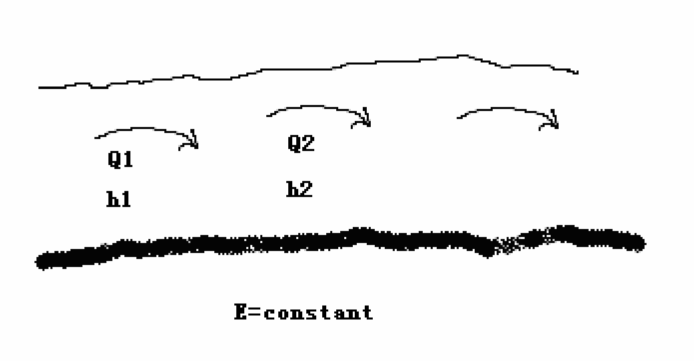
For changes in depth, the corresponding change in area is $dA = bdh$. Here $b$ is average free flow width. Thus setting (10) to zero, the minimum-energy condition becomes

$$1 - \frac{Q^2 b}{gA^3} = 0 \tag{11}$$

$$Q_c = \sqrt{\frac{gA^3}{b}} \quad , \quad E_c = h + \frac{Q_c^2}{2gA} \tag{12}$$

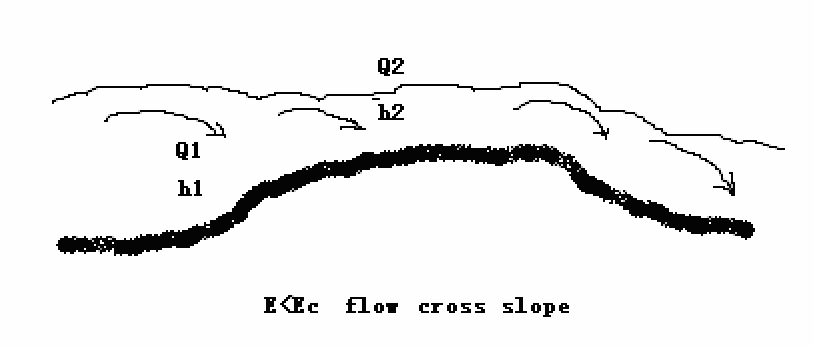
When $E < E_c$, we can see $Q$ decrease greatly. Thus, the flow must be climbing a little steep slope or down to a wide channel. In these two cases, energy is no longer constant, but discharge has still close relationship with depth. Under this two situations, discharge rate can be described by the broad crested weir equation [4].

$$Q = \frac{2}{3}\sqrt{\frac{2}{3}g}b(h - \Delta Hb)^{3/2} \tag{13}$$




When $E > Ec$, we can know that

$$h_2 + \frac{Q_2^2}{2gA_2} = h_1 + \frac{Q_1^2}{2gA_1} + Hb_2 - Hb_1, Q_2 = \frac{2}{3}\sqrt{\frac{2}{3}g}b_2\left|H_1 - Hb_2\right|^{3/2} \tag{14}$$




- **Simulation**

Based on analysis above, we can program iterative arithmetic to calculate $Q_{j+1}$, since $Q_j$, $h_j$ and $A_j$ have been known. After computing in foregoing steps, we can complete our model by Matlab. The flow chart of the computer program is as follow.

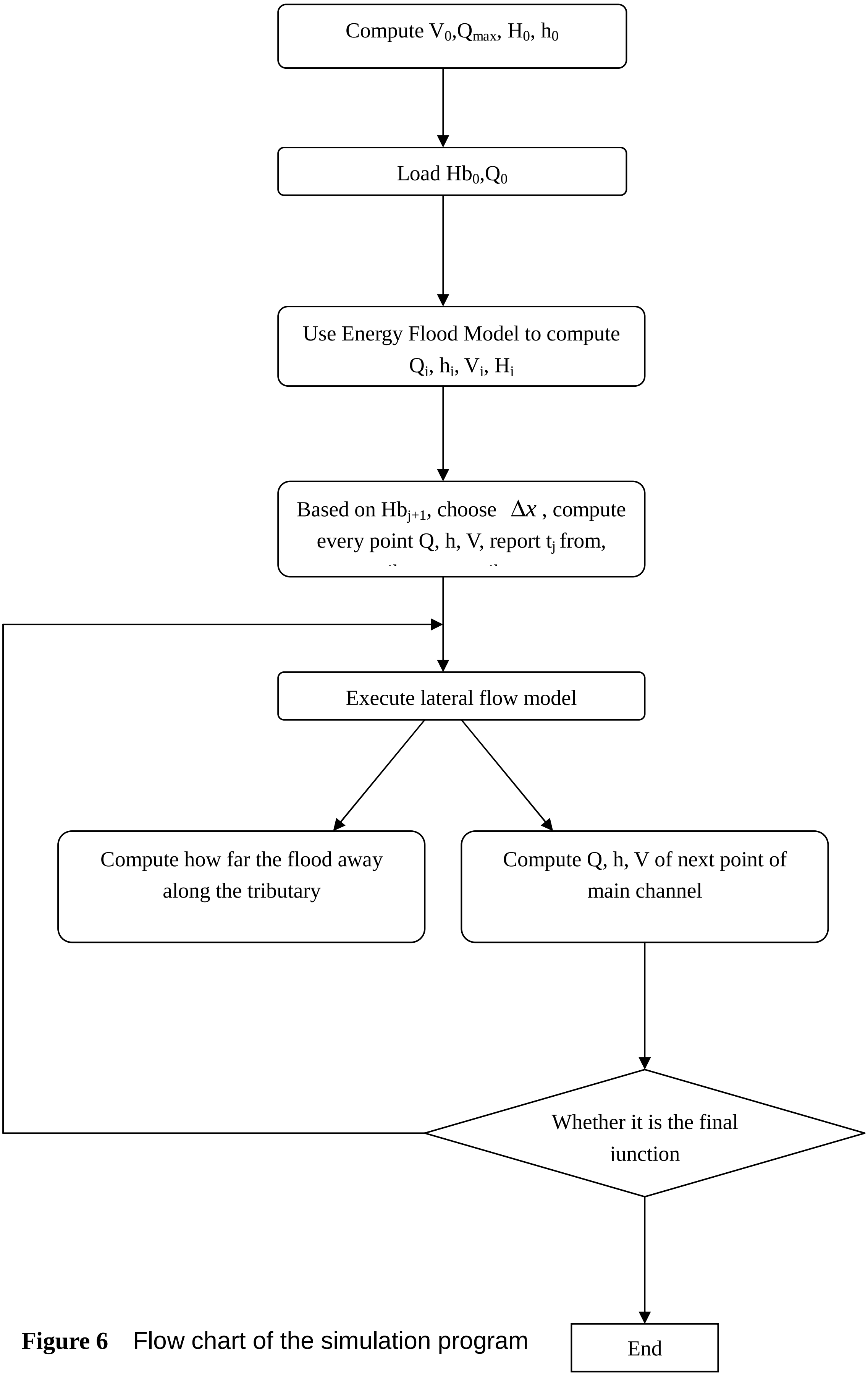



**Figure 6** Flow chart of the simulation program

## Conclusions

According to the test algorithm above, we draw the conclusion that even in the worst situation (that is, the dam has a maximal breach possible and there is no human factors to control the flood), the flood can extend back to Rawls Creek 1.13 miles, and if considering other Creek's function of receiving flood other than Rawls Creek, the water would not reach up to the S.C. State Capitol Building at the peak of flood.

## Strengths and Weaknesses

- **Strengths**

We have done a great deal of work on data searching and sampling, which sums up to more than two hundred sample spots and over five hundred data. According to the abundance of geographic and hydrological information about Lake Murray to State Capitol Building area, we make a contour diagram, which makes sure that the simulation can describe real situation of flood routing as much as possible.

Flood-routing model uses Saint Venant equation that can reflect both time and location information, but it is too complex to solve. So we optimize its solution to discrete style by referring to Preissmam method, which is a feasible measure to solve the equations.

Energy concept model does success in the flood simulation with regard to the difficulty in solving the partial differential equations, so our result can include the three-dimensional flow of flood distribution.

- **Weakness**

Although flood routing model is well backed up by mature theories, our solution skill is not mature enough to solve the partial differential equations precisely.

Saluda River has a number of tributaries; we cannot find enough data to describe the rivers one by one. Thus, their effect on the flood process is not considered in our model.

## References

[1] http://nmviewogc.cr.usgs.gov/viewer.htm
[2] Mahmood K, Yevjevich V. Unsteady Flow in Open Channels [M]. Water Resources Publications, 1975.
[3] http://www.dnr.state.sc.us/pls/hydro/river.home
[4] Merle C. Potter, David C. Wiggert, Mechanics of Fluids, 489-493,495-496

# Appendix

- **Matlab Main Program Based on Algorithm of Energy Concepts Theory**

```
function
[X_flood,Qx,Vx,Hx,Zx,X_Rawls_Creek,X_Kinley_Creek,X_Stoop_Creek]=river_iterate(V0,Qm
ax,h,time,X_Saluda)

D=264;

%Input ： Qmax, V0, h, X_Saluda
%Output:X_flood,Qx,Vx,Hx,Zx,X_Rawls_Creek,X_Kinley_Creek,X_Stoop_Creek
[Xi,Yi,Zi]=mapinfor(); %loading position data
river_data;
%[Xsa,Ysa,Zsa]=;%position of Saluda River
%[Xra,Yra,Zra]=;%position of Rawls Creek
%[Xki,Yki,Zki]=;%position of Kinley Creek
%[Xst,Yst,Zst]=;%position of Stoop Creek

T=0;t=0;
Hs=0;Hr=0;Hst=0;Hk=0;
Qfs=0;Qfr=0;Qfst=0;Qfk=0;Vs=0;
Xunderwater=0;Yunderwater=0;Zunderwater=0;

dr=5.28;ds=5.28;dk=5.28;dt=5.28;
%[X_uw,Y_uw,Z_uw,V,H,Qf]=upstream(ir,v1,y1,Q,d,Xi,Yi,Zi,Xr,Yr,Zr,dx,dy,heading)
%[Vj,Qj,VJ,QJ]=junction_convert(V,Q,y,D,d,Xr,Yr,Zr);

v1=52.417;
Q=7.5424*10^5;
y1=27.2523;
is=1;%irSaluda River Mark    ir=1;
ist=1;
ik=1;

i=1;
while is<8

[X_uw,Y_uw,Z_uw,V,H,Qf]=upstream(is,v1,y1,Q,D,Xi,Yi,Zi,Xsa,Ysa,Zsa,4.8*10^(-004),2.9000*
10^(-004),1);
    v1=V;Q=Qf;y1=H;
    td=dis(is,Xsa,Ysa,Zsa)/V;
    Qfs(is)=Qf;
```

```
        Hs(is)=H;
        Vs(is)=V;

        j=1;
        [xt,yt,zt]=matrixtran(X_uw,Y_uw,Z_uw,1)
        while j<size(xt)
                if xt(j)~=0
                        Xunderwater(uw)=xt(j);
                        Yunderwater(uw)=yt(j);
                        Zunderwater(uw)=zt(j);
                end
                j=j+1;
        end
        td=dis(is,Xsa,Ysa,Zsa)/V;
        t=t+td;
        T(is)=t;
        is=is+1;
end
is=is-1;
%到 Rawls Creek 河
Q=Qfs(is);H=Hs(is);Hr(ir)=H;V=Vs(is);
[Vj,Qj,VJ,QJ]=junction_convert(V,Q,H,D,dr,Xra,Yra,Zra);
Qfs(is)=QJ;Vs(is)=VJ;

i=1;v1=VJ;Q=QJ;y1=Hs(is);
while i<=5

[X_uw,Y_uw,Z_uw,V,H,Qf]=upstream(is,v1,y1,Q,D,Xi,Yi,Zi,Xsa,Ysa,Zsa,4.8*10^(-004),2.9*10^
(-004),1);
        v1=V;Q=Qf;y1=H;Vs(is)=V;
        Hs(is)=H;

        j=1;
        [xt,yt,zt]=matrixtran(X_uw,Y_uw,Z_uw,1)
        while j<size(xt)
                if xt(j)~=0
                        Xunderwater(uw)=xt(j);
                        Yunderwater(uw)=yt(j);
                        Zunderwater(uw)=zt(j);
                end
                j=j+1;
        end
        Qfs(is)=Qf;
        td=dis(is,Xsa,Ysa,Zsa)/V;
```

```
        t=t+td;
        T(is)=t;
        is=is+1;
        i=i+1;
end
is=is-1;
%Rawls Creek
i=1;v1=Vj;Q=Qj;y1=Hr(ir);VR=2930/(5.28*Hr(ir));
while i<26&(V>VR)

[X_uwr,Y_uwr,Z_uwr,V,H,Qf]=upstream(ir,v1,y1,Q,dr,Xi,Yi,Zi,Xra,Yra,Zra,4.8*10^(-004),2.9*1
0^(-004),-1);
        v1=V;Q=Qf;y1=H;Vr(ir)=V;
        Hr(ir)=H;
        Qfr(ir)=Qf;

        j=1;
        [xt,yt,zt]=matrixtran(X_uw,Y_uw,Z_uw,1)
        while j<size(xt)
                if xt(j)~=0
                        Xunderwater(uw)=xt(j);
                        Yunderwater(uw)=yt(j);
                        Zunderwater(uw)=zt(j);
                end
                j=j+1;
        end
        ir=ir+1;
        i=i+1;
end

%Saluda River to Twelevmiles River
Q=Qfs(is);H=Hs(is);V=Vs(is);Ht(it)=H;
[Vj,Qj,VJ,QJ]=junction_convert(V,Q,H,D,dt,Xtw,Ytw,Ztw);
Qfs(is)=QJ;Vs(is)=VJ;

%Saluda River to Kinley Creek
  i=1;v1=VJ;Q=QJ;y1=Hs(is);
while i<=5

[X_uw,Y_uw,Z_uw,V,H,Qf]=upstream(is,v1,y1,Q,D,Xi,Yi,Zi,Xsa,Ysa,Zsa,4.8*10^(-004),2.9*10^
(-004),1);
        v1=V;Q=Qf;y1=H;Vs(is)=V;
        Hs(is)=H;
        j=1;
```

```
    [xt,yt,zt]=matrixtran(X_uw,Y_uw,Z_uw,1)
    while j<size(xt)
        if xt(j)~=0
            Xunderwater(uw)=xt(j);
            Yunderwater(uw)=yt(j);
            Zunderwater(uw)=zt(j);
        end
        j=j+1;
    end
    Qfs(is)=Qf;
    td=dis(is,Xsa,Ysa,Zsa)/V;
    t=t+td;
    T(is)=t;
    is=is+1;
    i=i+1;
end
is=is-1
%Twelevmiles River


Q=Qfs(is);H=Hs(is);Hr(ir)=H;V=Vs(is);
[Vj,Qj,VJ,QJ]=junction_convert(V,Q,H,D,dk,Xki,Yki,Zki);
Qfs(is)=QJ;Vs(is)=VJ;

%Saluda to Stoop Creek
i=1;v1=VJ;Q=QJ;y1=Hs(is);
while i<=5

[X_uw,Y_uw,Z_uw,V,H,Qf]=upstream(is,v1,y1,Q,D,Xi,Yi,Zi,Xsa,Ysa,Zsa,4.8*10^(-004),2.9*10^
(-004),1);
    v1=V;Q=Qf;y1=H;Vs(is)=V;
    Hs(is)=H;

    j=1;
    [xt,yt,zt]=matrixtran(X_uw,Y_uw,Z_uw,1)
    while j<size(xt)
        if xt(j)~=0
            Xunderwater(uw)=xt(j);
            Yunderwater(uw)=yt(j);
            Zunderwater(uw)=zt(j);
        end
        j=j+1;
    end
```

```
        Qfs(is)=Qf;
        td=dis(is,Xsa,Ysa,Zsa)/V;
        t=t+td;
        T(is)=t;
        is=is+1;
        i=i+1;
end
is=is-1;
%Kinley Creek
i=1;v1=Vj;Q=Qj;y1=Hk(ik);VK=2280/(5.28*Hk(ik));
while i<=22&(V>VK)

[X_uwr,Y_uwr,Z_uwr,V,H,Qf]=upstream(ir,v1,y1,Q,dk,Xi,Yi,Zi,Xki,Yki,Zki,4.8*10^(-004),2.9*1
0^(-004),-1);
        v1=V;Q=Qf;y1=H;Vk(ik)=V;
        Hk(ik)=H;
        Qfk(ik)=Qf;

        j=1;
        [xt,yt,zt]=matrixtran(X_uw,Y_uw,Z_uw,1)
        while j<size(xt)
                if xt(j)~=0
                        Xunderwater(uw)=xt(j);
                        Yunderwater(uw)=yt(j);
                        Zunderwater(uw)=zt(j);
                end
                j=j+1;
        end
        ik=ik+1;
        i=i+1;
end

%Stoop Creek
Q=Qfs(is);H=Hs(is);Hst(ist)=H;V=Vs(is);
[Vj,Qj,VJ,QJ]=junction_convert(V,Q,H,D,dr,Xst,Yst,Zst);
Qfs(is)=QJ;Vs(is)=VJ;

%Saluda River to the junction
i=1;v1=VJ;Q=QJ;y1=Hs(is);
while i<=5

[X_uw,Y_uw,Z_uw,V,H,Qf]=upstream(is,v1,y1,Q,D,Xi,Yi,Zi,Xsa,Ysa,Zsa,4.8*10^(-004),2.9*10^
(-004),1);
        v1=V;Q=Qf;y1=H;Vs(is)=V;
```

```
        Hs(is)=H;

        j=1;
        [xt,yt,zt]=matrixtran(X_uw,Y_uw,Z_uw,1)
        while j<size(xt)
                if xt(j)~=0
                        Xunderwater(uw)=xt(j);
                        Yunderwater(uw)=yt(j);
                        Zunderwater(uw)=zt(j);
                end
                j=j+1;
        end
        Qfs(is)=Qf;
        td=dis(is,Xsa,Ysa,Zsa)/V;
        t=t+td;
        T(is)=t;
        is=is+1;
        i=i+1;
end
is=is-1;
%Stoop Creek
i=1;v1=Vj;Q=Qj;y1=Hk(ik);VK=1642/(5.28*Hst(ist));
while i<=26&(V>VK)

[X_uwr,Y_uwr,Z_uwr,V,H,Qf]=upstream(ir,v1,y1,Q,dk,Xi,Yi,Zi,Xst,Yst,Zst,4.8*10^(-004),2.9*1
0^(-004),-1);
        v1=V;Q=Qf;y1=H;Vst(ist)=V;
        Hk(ik)=H;
        Qfk(ik)=Qf;

        j=1;
        [xt,yt,zt]=matrixtran(X_uw,Y_uw,Z_uw,1)
        while j<size(xt)
                if xt(j)~=0
                        Xunderwater(uw)=xt(j);
                        Yunderwater(uw)=yt(j);
                        Zunderwater(uw)=zt(j);
                end
                j=j+1;
        end
        ik=ik+1;
        i=i+1;
end
```