A SunCam online continuing education course

Spillway Design for Small Dams

by

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Introduction:
This course presents an overview of the features that go into the design of spillways for small dams. "Small dams" in this course refer to a wide variety of water impoundments that includes detention basins, municipal ponds, agricultural ponds, and others. This course is based on a variety of reference materials including "Guidelines for Design of Dams" published by the New York State Department of Environmental Conservation and "Design of Small Dams" published by the United States Department of the Interior, Bureau of Reclamation.

When you complete this course you should also be familiar with the various types of spillways that are available and with the basic hydraulics of each type.

Overview of Spillways:
There are many types of spillways that are used in conjunction with small dams. These spillways come in a variety of shapes and configurations and are commonly divided into principal spillways and emergency spillways. Alternatively, these are sometimes referred to as service spillways and auxiliary spillways, respectively. Whatever they are called, all spillways are intended to safely pass the floodwaters without allowing the dam to overtop and without damaging downstream properties through flooding or erosion.

Principal Spillways:
The principal spillway represents the main outlet of the pond. It can take several forms and can either be a single phase or a multi-phase structure. The main functions of the principal spillway are as follows:

1. To safely pass the design storm flows.
2. To maintain the desired water level within the pond.
3. To allow the pond to be drained if necessary. (Note that this is a very important consideration in maintenance of the pond and, even more so, as a safety feature. The disastrous Johnstown Flood of 1889 was caused, at least in large part, by an inability of the manager to drain the South Fork Dam prior to the storm).

Some examples of principal spillways are shown below.

The photograph below shows the principal spillway of a small dam being rebuilt.

In order to set the forms shown in the photograph above, it was first necessary to carry out dewatering operations, bypassing this area with the outflow from the pond.

The photograph below shows this same spillway after completion of the renovation. The picture is taken from the downstream side of the concrete spillway. A chute has been constructed to direct water that passes over the spillway into the receiving channel below.

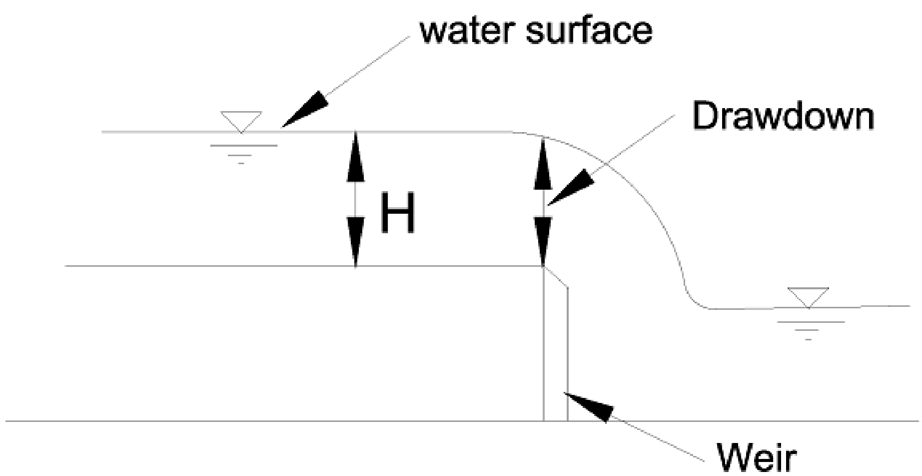
To calculate the flow through the spillway structure it is important to determine the type of structure and correctly model the outlet. Many spillways function as some type of weir. The most common types of weirs are as follows:

- Sharp crested weirs
- Broad crested weirs
- V-notch weirs

Flow over a sharp crested weir can be approximated by the equation: $Q = cLH^{3/2}$, where

Q= the flow over the weir in cubic feet per second (CFS)

C is the weir coefficient. This is generally assumed to be approximately equal to 3.2 for most sharp-crested weirs.
L is the effective length of the weir in feet, and
H is the head in feet. Note that it is important to measure the head far enough upstream of the weir that drawdown is not a factor. This is shown schematically below.



Discharge Over Weir

The effective weir length is determined by the equation: $L = L' - 0.1NH$, where:
L' is the measured weir length, in feet
N is the number of end contractions, and
H is the head as described above.
A slight variation of the sharp crested weir is the Cippoletti Weir. In this type of weir the shape is a trapezoid instead of a rectangle and the slightly sloping sides eliminate the need to correct the length for end contractions.

Rectangular Sharp-crested Weir Example:
The photograph below shows the spillway of a municipal pond in Somerset County, New Jersey. The spillway consists of a 15 foot wide overflow that can be modeled as a sharp-crested weir. There are abutments at each end of the weir, as can be seen in the photograph. The spillway crest is approximately 24 inches below the top of the dam elevation. A weir coefficient of 3.2

will be used for this example. Determine the flow over the weir when the head is measured at a depth of 6 inches.

Solution:
First determine the effective weir length. The measured length of the weir is given as 15 feet and there are two end contractions. Therefore, the effective length is calculated as:

$L = L' - 0.1NH$,

Where: L' is the measured length (15 feet),
N is the number of end contractions (2), and
H is the head in feet (0.5)

Substituting the known parameters yields: $L = 15 - (0.1)(2)(0.5) = 14.9'$

Next, calculate the discharge over the weir using: $Q = cLH^{3/2}$
Where: c is the coefficient (3.2),
L is the effective length (14.9 feet),
And h is the head in feet (0.5)
Once again, substituting the known parameters yields: $Q = (3.2)(14.9)(0.5)^{3/2} = 16.9CFS$
Therefore, the flow over this weir with a head of 6 inches is approximately 16.9 CFS.

The weir equation can also be used to determine if the spillway is adequate to pass the anticipated design flow. For example, staying with the municipal pond shown above, suppose that the 100 year peak discharge into the reservoir has been calculated as 90 CFS. What head would be required to pass this flow?

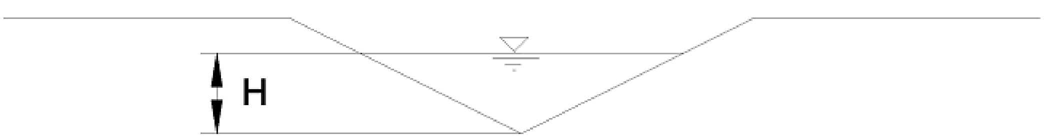
<u>Solution:</u>
In this case it is necessary to re-write the weir equation to solve for the head, as follows:
$H = (Q / cL)^{2/3}$
Substituting the known values:
$H = ((90/(3.2)(14.9)))^{2/3} = 1.5FT$
Note that this number is approximate and we can refine it by using 1.5 feet as the head and re-calculating the effective weir length as: $L = 15 - (0.1)(2)(2) = 14.6FT$
$H = ((90/(3.2)(14.6))^{2/3} = 1.5FT$ (This shows that the first approximation, using a weir length of 14.9 feet was fairly accurate).
The required head is 1.5 feet or 18 inches. Note that this is less than the 2 feet provided between the spillway crest and the top of dam elevation given in the original problem statement. Therefore, this spillway should be able to safely pass the peak 100 year discharge without overtopping the dam.

Depending on the geometry, weir type spillways are sometimes called chute spillways or drop spillways. However, they can all generally be modeled as a sharp-crested or broad-crested weir.

The photograph below shows a typical drop spillway servicing a small pond in a wildlife refuge. This type of spillway can be modeled as a sharp crested weir with the total weir length being the total of the lengths of each of the sides. Because all four sides are open, there are actually eight end contractions to be accounted for in the weir equation.

V-Notch Sharp Crested Weirs:

A common type of sharp-crested weir is the v-notch weir. As the name implies, this type of weir has a notched shape, as shown in the diagram below.



V-Notch Weir

If the angle of the notch is 90^0, the flow over a v-notch weir can be approximated using the following relatively simple equation:

$Q = 2.5H^{2.5}$, where H is the head over the weir in feet.

The following example will illustrate the use of this equation.

V-Notch Weir Example 1:

The v-notch weir shown in the diagram above has a v-notch angle of 90^0 and a depth of 15 inches. What is the capacity of the v-notch weir at this maximum head of 15 inches?

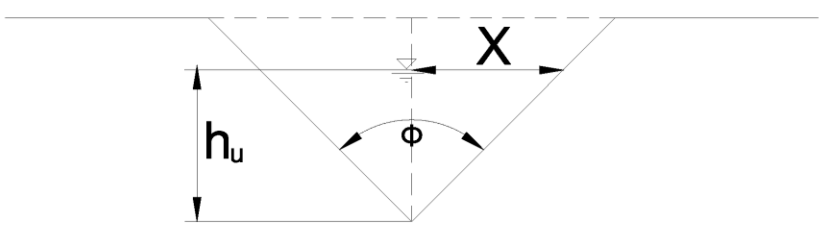
Solution:

To apply the equation above, we must first convert the head to feet. 15 inches = 1.25 feet. Then, using the V-notch weir equation given above, the capacity is calculated as follows:

$Q = 2.5(1.25)^{2.5} = 4.37CFS$

This is the capacity of the v-notch weir at a head of 15 inches.

Another v-notch weir is shown below, but in this case the angle is not 90 degrees.



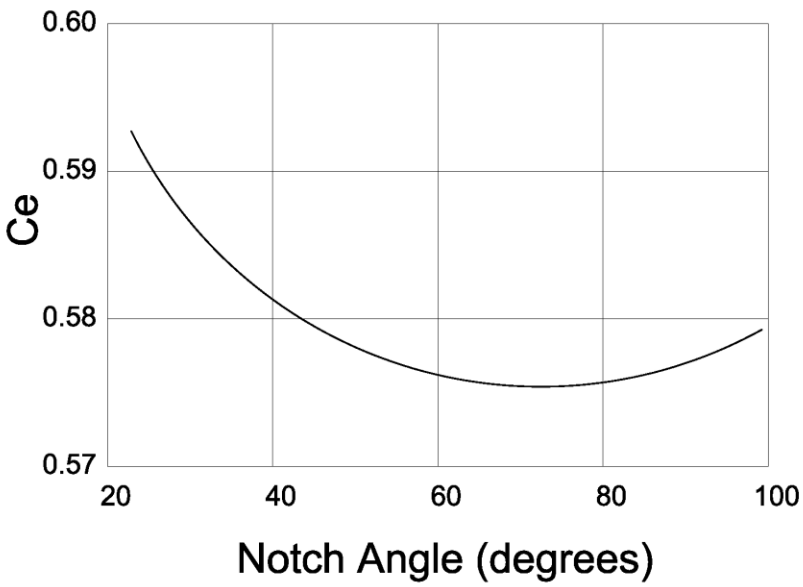
V-Notch Weir

If the notch angle is not equal to 90^0, the equation becomes much more cumbersome. In this case, the flow over the weir can be calculated using the following:

$Q = (8/15)(2g)^{1/2} C_e Tan(X/2) h_e^{5/2}$, where

g is the acceleration due to gravity (32.2 feet per second squared)

C_e is based on the notch angle and can be determined from the chart shown below

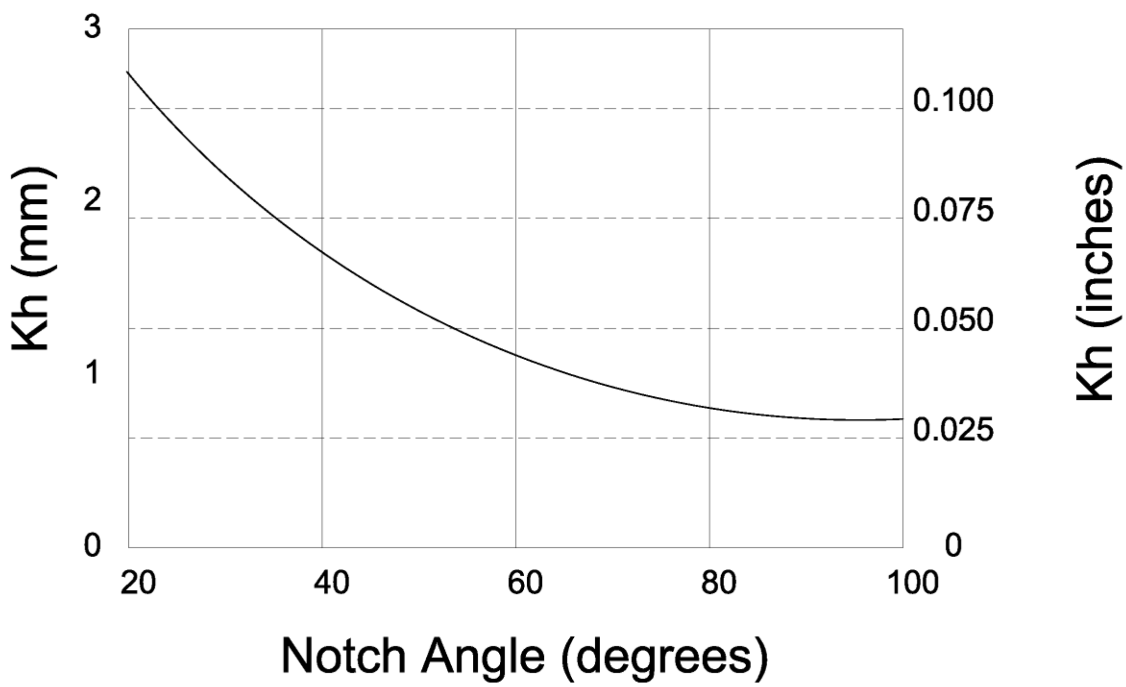


X is the angle of the notch in degrees

he can be calculated as follows: $h_e = h_u + K_h$

hu is the maximum depth of flow over the weir notch.

K_h is taken from the chart shown below:



V-Notch Weir Example 2:
Assume that the v-notch weir in example 1, above, has a v-notch angle of 60^0 and a depth of 12. What is the capacity of the v-notch weir at this maximum head of 12 inches?

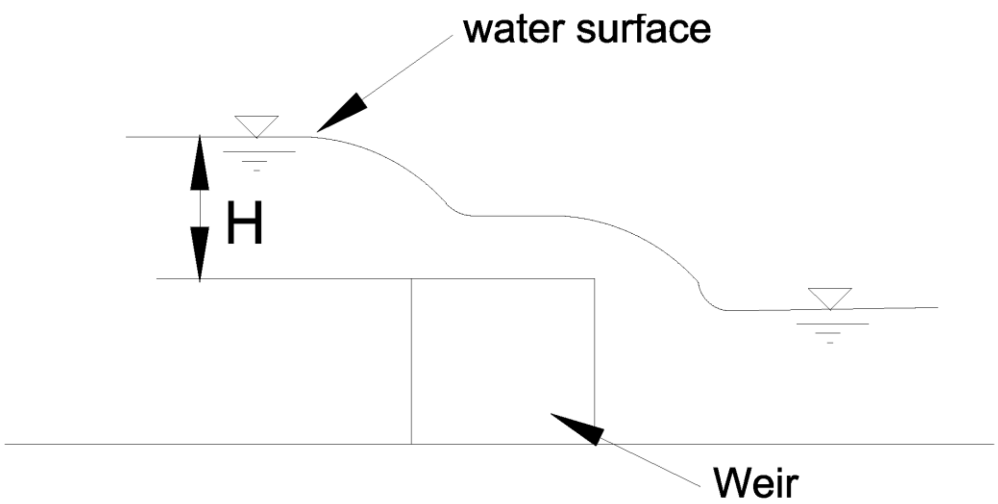
Solution:
To apply the equation $Q = (8/15)(2g)^{1/2} C_e Tan(X/2))h_e^{5/2}$, we must first determine the following parameters:
Ce is 0.576, based on the first of the two charts shown above.
X is 60 degrees
H_e can be calculated as follows: $H_e = 1 + 0.00375 = 1.00375$
H_u is 12" or 1 foot, as given in the problem
K_h is 0.045" or 0.00375 feet, based on the second of the two charts shown above

Then the capacity is calculated as follows:

$Q=(8/15)(2g)^{1/2}(0.576)\text{Tan}(60/2)(1.00375)^{5/2}=1.44\text{CFS}$

Comparing this with the result obtained in the 90^0 v-notch weir, we can see that this flow is somewhat less. This makes sense because the 60^0 angle is not as open as the 90^0 angle.

Broad-crested weirs:
As the name implies, a rectangular broad crested weir is very similar to a rectangular sharp crested weir except for the weir width, itself. Schematically, this is shown in the diagram below:



Broad Crested Weir

For this reason, rectangular broad-crested weirs are analyzed in the same way as rectangular sharp-crested weirs. However, the weir coefficient is less (generally a value of 2.6 is used for broad crested weirs as opposed to the 3.2 used for sharp crested weirs). Therefore, the general equation for flow over a broad-crested rectangular weir is the same as shown above for a sharp-crested weir:
$Q = cLH^{3/2}$. In this case, c is set equal to 2.6 and the other parameters are as defined above.
As explained later in this course, flow over an earthen emergency spillway is often approximated using a broad crested weir model.

Detention Basin Spillways:
Many small dams constructed today are intended for use as detention basins. These dams generally impound water for only a short time (immediately during and after a storm). The principal spillway of a detention basin consists of a controlled outlet. This is an outlet structure that is designed to reduce the flow out of the basin to pre-determined levels during specific storm events. Because of this, the outlet structure often includes a number of weirs and orifices of different sizes.

A typical detention basin outlet structure is shown in the photograph below. The aluminum grating bolted to the front of the controlled outlet is a trash rack and is intended to keep debris out of the outlet pipe.

In many cases, a trash rack is also provided at the top of the controlled outlet. This type of trash rack, such as the one shown below, also functions as a safety feature by keeping children out of harms way.

When designing a controlled outlet structure for a detention basin it is often necessary to determine the outflow from the basin for a variety of storms. In New Jersey, the 2 year, 10, and 100 year storm events are typically modeled. The design engineer must ensure that the outflow from the detention basin will not exceed the discharge from the area under pre-developed conditions during any of the design storms. In order to accomplish this, it is often necessary to employ a variety of orifices and weirs of varying sizes placed at various elevations.

In order to reduce the peak discharges, orifices are often used in place of weirs in controlled outlets. This is because the flow through an orifice varies with the square root of the head, whereas the flow over a weir varies with the 3/2 power of the head. Therefore, orifices generally allow less flow at higher heads. The flow through an orifice can be approximated by the orifice equation:

$Q = 0.6A((2)(g)(H))^{1/2}$, where

A is the area of the orifice opening.
g is the acceleration due to gravity (32 feet per second squared),
H is the head, in feet. Once again it is important to measure the head from the correct elevation. There is a fundamental difference in measuring the head over an orifice and the head over a weir. The depth of the head over a weir is simply the vertical distance from the weir crest to the water surface. However, in an orifice, the head is the vertical distance of the midpoint of the orifice to the water surface. This means that if the water depth is 10" above the bottom of a 4" diameter circular orifice, then the head is 8".

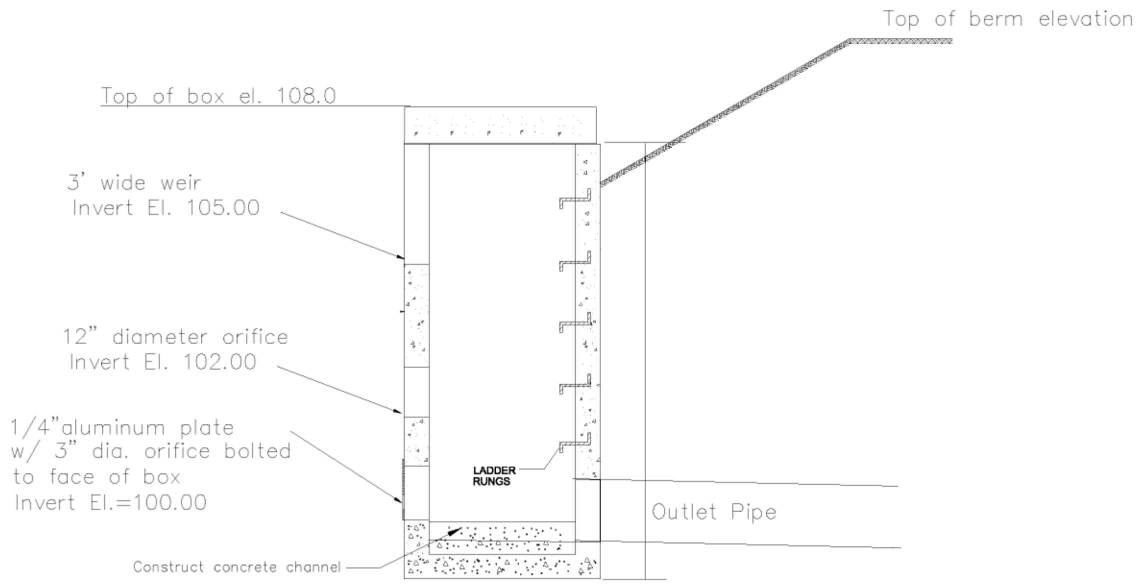
Orifices can come in a variety of shapes, but (for ease of construction) are generally either circular or rectangular.

<u>Orifice Example:</u>
A 6" diameter circular orifice is placed at elevation 110.0 in a controlled outlet structure. What would be the flow through this orifice when the elevation in the basin reaches elevation 115.5?

<u>Solution:</u>
Using the orifice equation, $Q = 0.6A(2gH)^{1/2}$ we can determine the outflow. First, calculate the area of the orifice as follows:
$A = 3.14(0.25)^2 = 0.196SF$ Note that in this example, 3.14 is used for Π and the radius of the circle is expressed in feet (3"=0.25').
The head over the orifice is calculated as the water surface minus the midpoint of the orifice (i.e. 115.5-110.25=5.25 feet). The discharge through the orifice can then be calculated as:
$Q=0.6x0.196x(2X32.2x5.25)^{1/2}=2.16CFS$

Therefore, the flow through the orifice, given these parameters, would be slightly more than 2 CFS.

A typical detention basin controlled outlet structure is shown schematically below:



Outlet Structure Detail: Side View
not to scale

In this controlled outlet, as in many detention basin outlet structures, the total outflow is a combination of weir and orifice flow. The resulting stage vs. discharge table is made up of the individual discharges from each of the openings in the controlled outlet box.
These openings (shown in the above diagram) are summarized below:

- A 3" diameter circular orifice with an invert elevation of 100.0
- A 12" diameter circular orifice with an invert elevation of 102.0
- A 3' wide weir with a crest elevation of 105.0

A stage vs. discharge table can be prepared for a controlled outlet like this one can be prepared by calculating the flows through each of the apertures at the various elevations and summing them to obtain the total flow a that elevation.

For the controlled outlet shown above, the stage vs. discharge table would look like the following:

Elevation	**Orifice "A" Head (FT)**	**Orifice "A" Q (CFS)**	**Orifice "B" Head (FT)**	**Orifice "B" Q (CFS)**	**Weir Head (FT)**	**Weir Q (CFS)**	**Total Q (CFS)**
100.0	-----	-----	-----	-----	-----	-----	0.0
101.0	0.875	0.22	-----	-----	-----	-----	0.22
102.0	1.875	0.32	-----	-----	-----	-----	0.32
103.0	2.875	0.40	0.5	2.67	-----	-----	3.07
104.0	3.875	0.47	1.5	4.63	-----	-----	5.10
105.0	4.875	0.52	2.5	5.98	-----	-----	6.50
106.0	5.875	0.57	3.5	7.07	1.0	9.60	17.24
107.0	6.875	0.62	4.5	8.02	2.0	27.15	35.79
108.0	7.875	0.66	5.5	8.87	3.0	49.88	59.41

There are several things about the above table that should be pointed out:

1. As mentioned previously, the head in the two orifices is measured from the mid-point of the orifice, whereas it is measured from the crest of the weir.
2. The orifice flows do not increase rapidly (as the weir flow does) with increasing head, because as pointed out before, the flow in the orifice equation is proportional to the square root of the head.
3. If there is an emergency spillway, the flow through this feature would be in addition to the principal spillway flow shown above.
4. Another important consideration in determining the stage vs. discharge relationship is the capacity of the outlet pipe from the controlled outlet. The values in the table above show the flow that approaches the outlet pipe for various head elevations. However, if the pipe is too small or not laid at a sufficient slope (which is often the case due to downstream considerations) the capacity of this pipe controls the flow out of the basin. If this is the case, an extra column should be added to the table above addressing this controlling flow rate.

Floodgates:

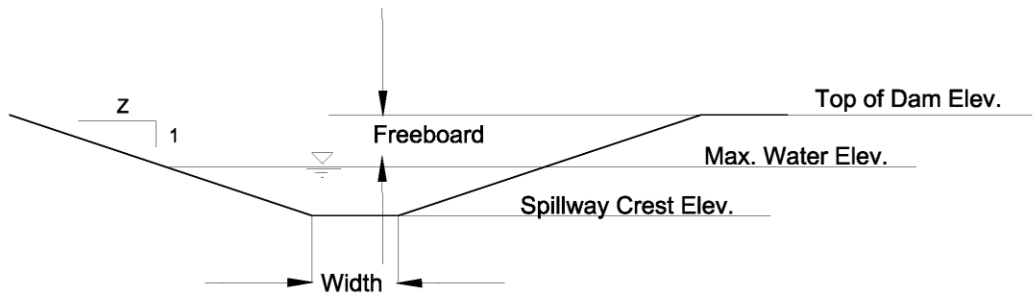
According to FEMA's "Federal Guidelines for Dam Safety", gated spillways are the usual hydraulic appurtenances for control of all or the major portion of the design flood and major emergency releases. Outlets, such as sluiceways, conduits, or tunnels, may be used alone in this regard or in conjunction with spillways to control flood discharges. In gated spillways gates are provided to increase the capacity of the spillway during extreme flood events. The flood gates are ordinarily left closed, but they can opened if a very large rainfall is forecast, or if other factors (e.g. an upstream dam breaching, etc.) is expected to overtax the normal spillway operation.

Emergency Spillways:

In addition to the principal spillway, all dams should also be equipped with an emergency spillway. The purpose of this spillway is just what it seems to be: to function in emergency situations. These situations could include a tremendous rainfall that overtops the principal spillway or the clogging of the principal spillway, itself. The emergency spillway then functions to pass the flood waters downstream without overtopping the dam and, potentially, causing a wash-out of the dam structure.

The emergency spillway can be a separate structure or it can be incorporated into the principal spillway. However, whenever possible, it is preferable to have the emergency spillway as a separate structure altogether. This way it will not be affected if, for instance, the outlet pipe of the principal spillway becomes clogged. In the case of a very small dam or if a dam failure will not adversely affect downstream properties, a combined principal/emergency spillway could be considered.

The emergency spillway is often an earth structure that can best be approximated by a broad-crested weir. However, the weir equation does not take into account the sloping sides of the spillway (which increases the flow because the weir equation assumes that the sides of the weir are vertical). With side slopes of Z:1, this is shown schematically in the emergency spillway cross section below:



Cross Section of Vegetated Emergency Spillway

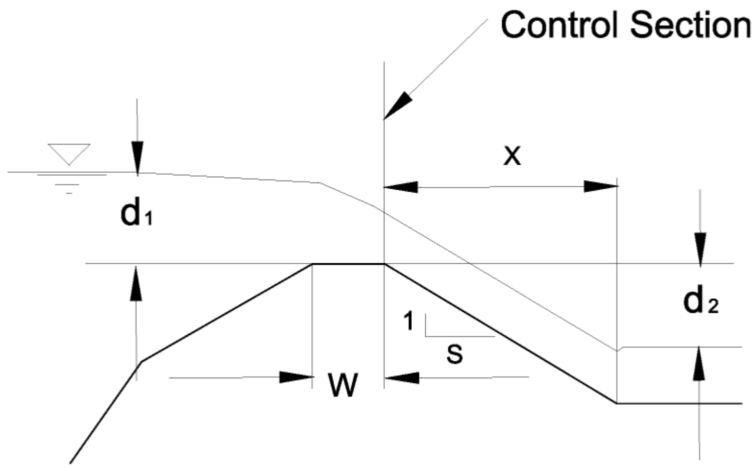
Therefore, the National Resource Conservation Service has a table of discharges for various earthen spillway configurations which can be used as well. The flow through the spillway is a function of the spillway width, the depth of flow, and the roughness (n) of the lining (vegetation or earth) of the spillway. This is the same n as is used in the Manning's Equation for open channel flow.

A portion of one of these tables (for a vegetated spillway with side slopes of 3:1 and an estimated n value of 0.040) is included below. Note that an n value of 0.040 is fairly typical for the dense grasses used to line an emergency spillway channel.

Spillway Bottom width (feet)	**Stage or head (feet)**	**Velocity (FPS)**	**Q (CFS)**	**Minimum slope of channel below control section (%)**	**Minimum length of channel below control section (feet)**
10	0.5	2.7	8	3.9	33
10	1.0	4.0	27	3.0	52
10	1.5	4.9	54	2.7	72
10	2.0	5.6	94	2.4	92
20	0.5	2.7	14	3.9	33
20	1.0	4.0	48	3.0	52
20	1.5	5.0	96	2.6	72
20	2.0	5.7	160	2.4	92
30	0.5	2.7	21	3.8	33
30	1.0	4.0	69	3.0	52
30	1.5	5.0	136	2.6	72
30	2.0	5.8	223	2.3	92
40	0.5	2.7	28	3.8	33
40	1.0	4.0	90	3.0	52
40	1.5	5.1	180	2.5	72
40	2.0	5.8	288	2.3	92

There are other, similar charts for other side slopes and n values.

The last two columns in the table above (minimum slope of channel below control section in % and in feet) deal with the exit channel below the spillway and are shown in the schematic profile below.



Profile Through ℄ of Earth Spillway

The actual design of the emergency spillway is based, in large part, on the dam classification. If a dam is a large structure, or if its failure would jeopardize public safety, then a larger spillway design criteria is used. This is discussed in more detail below.

Dam Classification:
The classification of the dam is based on both the size of the dam and on the potential for damage downstream in the case of a dam break. Various governmental agencies having oversight of dams use different classification schemes. The New York State Department of

Environmental Conservation's (NYDEC) "Guidelines for Design of Dams" classifies dams according to the following table:

NY State Dam Hazard Classification	Description
Class "A"	Dam failure will damage nothing more than isolated farm buildings, undeveloped lands, or township or county roads.
Class "B"	Dam failure can damage homes, main highways, minor railroads, or interrupt the use or service of relatively important public utilities.
Class "C"	Dam failure can cause loss of life, serious damage to homes, industrial, or commercial buildings, important public utilities, main highways, and railroads.

Conversely, the New Jersey Department of Environmental protection uses a slightly different classification scheme as described below:

NJ Dam Classification	Description
Class I	Dam failure would likely cause loss of life.
Class II	Dam failure would likely cause significant damage to downstream properties but would not be considered a threat to life.
Class III	Dam failure would cause little or no downstream damage.
Class IV	These dams are less than 15 feet in height, impound less than 15 acre-feet of water, and drain less than 150 acres. In addition, dam failure would not be expected to cause loss of life.

Naturally, the emergency spillways of dams that have the potential to pose significant risks to public health and safety are designed to higher standards than are spillways for less potential threatening dams.

The following table delineates the NYDEC's required emergency spillway requirements for the various dam sizes and hazard classifications:

Dam Size*	**Dam Hazard Classification**	**Emergency Design Flood**	**Minimum Required Freeboard**
Small	"A"	100 year	1 foot
Large	"A"	150% of 100 year	2 feet
Small	"B"	225% of 100 year	1 foot
Large	"B"	40% of PMP	2 feet
Small	"C"	50% of PMP	1 foot
Large	"C"	PMP	2 feet

- A "small" dam has a height of less than 40 feet and a storage at normal water surface of less than 1000 acre-feet.
- A "large" dam has a height of greater than 40 feet or a storage at normal water surface of greater than 1000 acre-feet.
- To qualify as a "large" dam either the height or the storage threshold listed above must be reached.
- PMP is the probable maximum precipitation and is described below.
- Freeboard refers to the clear height between the maximum flood elevation in the basin (and emergency spillway) during the design storm and the top of the dam.

In the table above, the 100 year storm can be either an SCS-type 24 hour storm or a Rational Method storm with a shorter durations. The PMP in the table is the "Probable Maximum Precipitation". The National Oceanic & Atmospheric Administration (NOAA) defines this as: "The theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage area at a certain time of year". The PMP values often look, at first glance, to be unrealistically large.
A table of PMP values for different areas of the country vs. the 100 year storm precipitations follows:

Region	**24 hour 100 year Storm (Inches)***	**24 hour PMP (inches)****
Chicago, IL	7.24	31.5
New York, NY	8.86	34
Oklahoma City, OK	9.17	37
New Orleans, LA	14.4	47.5
Atlanta, GA	7.46	42
Morgantown, WV	5.06	34
Boston, MA	7.86	31.5

*Taken from The National Oceanic & Atmospheric Administration's Hydrometeorological Design Studies Center website, which can be accessed at: http://hdsc.nws.noaa.gov/hdsc/pfds/
**Taken from the United States Department of Commerce National Oceanic and Atmospheric Administration, Us. Department of the Army, Corps of Engineers' Hydrometeorological Report No. 51, entitled "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian", dated June 1978.

Looking at the table above makes it seem like these PMP values are unrealistic. However, there have been historically documented storms in many regions that at least approach the precipitations shown in the PMP. A few of these storms are listed below. (These data are taken from the US Department of Commerce Hydrometeorological Report No 51, referenced above).

Region	**Date of Storm**	**Duration of Storm**	**Precipitation**
Jefferson, OH	September 1878	24 hours	12.2 inches
Woodward Ranch, TX	May 1935	6 hours	20.4 inches
Ewan, NJ	September 1940	12 hours	22.4 inches
Hallet, OK	September 1940	24 hours	23.6 inches
Yankeettown, FL	September 1950	24 hours	38.7 inches
Thrall, TX	September 1921	24 hours	36.5 inches
Elba, AL	November 1929	24 hours	20.0 inches
Boyden, IA	September 1926	24 hours	21.7 inches
Altapass, NC	July, 1916	24 hours	22.2 inches

The exit channel of the emergency spillway is an important component of the spillway as a whole and it is imperative that it be designed to provide adequate capacity for the flood flows and that it prevents erosion. The spillway itself and the exit channel sometimes must be armored with concrete, riprap, or other materials. In other cases, a grass channel will be adequate. In addition, the bottom of the spillway is often a critical location and must be armored against erosion. Energy dissipaters are often needed at this location.

The photograph below is taken from the top of a spillway and shows the energy dissipating blocks at the base of the outfall. This is the same spillway that was illustrated in the design example for sharp-crested weirs and it functions as a combined principal/emergency spillway for this pond.

If both the principal spillway and emergency spillway were to fail and the dam were overtopped, there is every reason to believe that the dam will fail. In order to determine the effect of this type of catastrophe, it is necessary to perform a Dam break Analysis. There are several ways to conduct such an analysis including one contained in the US Army Corps of Engineers program HEC-RAS. However, a detailed discussion of dam break analysis is beyond the scope of this course.

In the exit channel from an emergency spillway, the velocity is often quite rapid and the flow is often supercritical. In fact, many emergency spillway exit channels are provided with a control section in which the flow will move (across a hydraulic jump) from a supercritical to a subcritical regime. This is one reason the design engineer must often provide energy dissipating blocks or other means to reduce the velocity at the base of the spillway channel. It is imperative that the exit channel be designed to be stable under peak flow conditions. For this reason, the exit channel often needs to be lined with concrete, or riprap, or be provided with energy-dissipating blocks. The photograph below shows a concrete lined exit channel.

The photograph below shows the downstream side of an emergency spillway of a detention basin located in Somerset County, New Jersey. This spillway is armored against erosion by a gabion blanket.

The photograph below shows the same detention basin. The principal spillway (controlled outlet) is shown to the right and the armored emergency spillway is shown to the left of the photo. Note that the controlled outlet has an open top (with a trash rack to collect debris and for safety purposes). Therefore, a significant amount of flow would be expected to pass through this structure before the emergency spillway started functioning.

Hydraulics of Spillway exit Channels:
As was mentioned earlier, the energy in a spillway must often be dissipated before the water is returned to a downstream river channel. Often the flow exiting the spillway is supercritical and this must be brought to subcritical flow before entering a downstream channel. In order to transition from supercritical to subcritical flow, the water must pass through a hydraulic jump. The calculations associated with a hydraulic jump can be quite involved. However, the following describes the basic analysis of the jumps. Much of this information is based on "Design of Small Dams" published by the United States Department of the Interior, Bureau of Reclamation.

To determine the flow regime (critical, sub-critical, or supercritical), it is necessary to calculate the Froude number. The Froude Number is a measurement of bulk flow characteristics such as waves, flow/depth interactions at cross sections or between boulders, etc. It is defined as the ratio of inertial forces divided by gravitational forces.

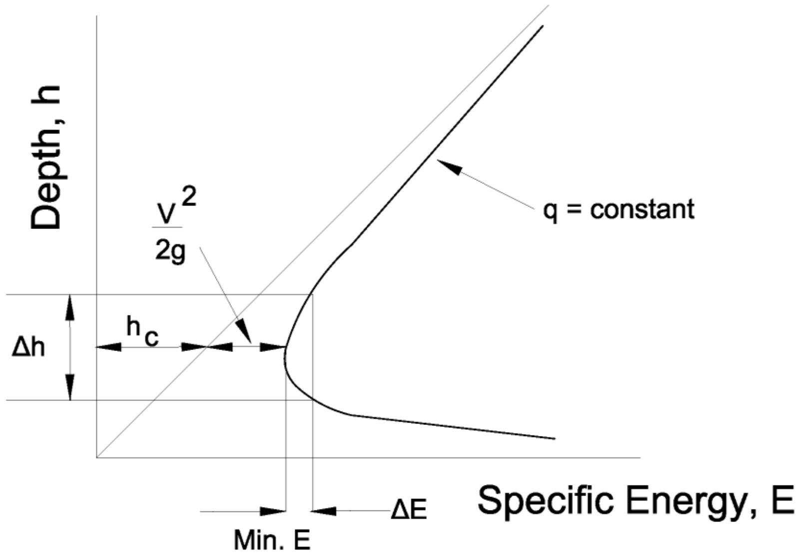
The Froude Number is calculated using the following formula:
$Fr=V/(gD)^{1/2}$, where:
Fr is the Froude Number
V is the velocity in feet per second
g is the acceleration due to gravity (32 feet per second squared)
D is the hydraulic depth (cross sectional area divided by flow top width)

If the Froude Number is greater than 1, the flow is supercritical.
If the Froude Number is equal to 1, the flow is critical.
If the Froude Number is less than 1, the flow is subcritical.

If the Froude Number is 1(i.e. critical; flow) the resulting depth is known as critical depth. Depths near critical depth are inherently unstable because small changes in surface energy can lead to large changes in the local flow depth.

The specific energy is calculated using the following equation:
$E = h + (V^2 / 2g)$, where:
E is the specific energy
v is the velocity
h is the flow depth
g is the acceleration due to gravity (32 feet per second squared)

The solution to this equation is a parabola as shown in the specific energy curve below.



Specific Energy Curve for Open Channel Flow

As can be seen by this curve, there are two separate possible flow depths for any specific energy. The higher depth (and lower velocity and energy) is subcritical. The lower depth (and higher velocity and energy) is supercritical. At the minimum E value the flow is critical. The two corresponding depths at a particular E value are known as the conjugate depths.

If a hydraulic jump occurs, a somewhat analogous equation to the one for the Froude Number can be used to find the conjugate depths:

$d_1 = -(d_2/2 + ((2v_2{}^2 d_2/g) + (d_2{}^2/4))^{1/2}$, or using the Froude Number:

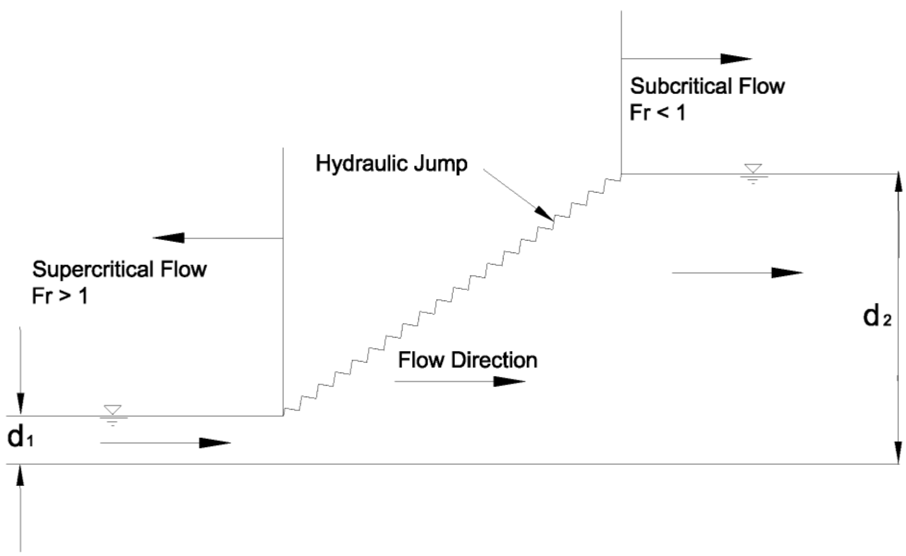
$d_1/d_2 = 1/2((8Fr+1)^{1/2} - 1)$, where

d_1 and d_2 are the conjugate depths as shown on the hydraulic jump schematic below.

The denominator of the above equation represents the speed of a small wave on the surface relative to the speed of the water and is called the wave celerity. When Fr = 1 and the flow is critical, the celerity equals the velocity. Therefore, any disturbance to the water surface will remain stationary under this condition. If Fr < 1 (sub-critical flow), the flow is controlled from a downstream point and disturbances are transmitted upstream. This leads to backwater effects. If

Fr > 1 (super-critical flow) the flow is controlled upstream and disturbances are transmitted downstream.

As indicated above, critical flow is an unstable flow regime. Supercritical flow is in a higher energy state and subcritical flow is in a lower energy state. In order to pass from supercritical to subcritical, the flow passes through a hydraulic jump, as shown schematically below. The conjugate depths, d_1 and d_2, are also shown.



Hydraulic Jump Schematic

The basic formula governing the geometry of a hydraulic jump is shown below:

$d_2 = -d_1/2 + ((d_1^{\,2}/4) + (2v_1^{\,2}d_1/g))^{1/2}$, where;

d_1 and d_2 are the subcritical and supercritical flow depths as shown in the schematic above.

v_1 is the velocity in the supercritical flow regime.

g is the acceleration due to gravity (32 feet per second squared).

The following example will illustrate the calculation of the conjugate depths in a hydraulic jump.

Hydraulic Jump Example:
A hydraulic jump is seen to from at the base of a spillway exit channel. If the depth coming down the channel is measured at 2 feet and the velocity in the channel has been measured at 14 FPS, what will be the conjugate depth below the hydraulic jump?
In order to determine the conjugate depth (d2), the equation given above for a hydraulic jump is employed:

$$d_2 = -d_1/2 + ((d_1^{\,2}/4) + (2v_1^{\,2}d_1/g))^{1/2}$$

Substituting the values given in the problem statement (d_1=2 feet, v_1=14 FPS), and using 32 feet per second squared for g, yields the following:

$$d_2 = -2/2 + ((2^2/4) + (2(14)^2 2/32))^{1/2} = 4.95$$

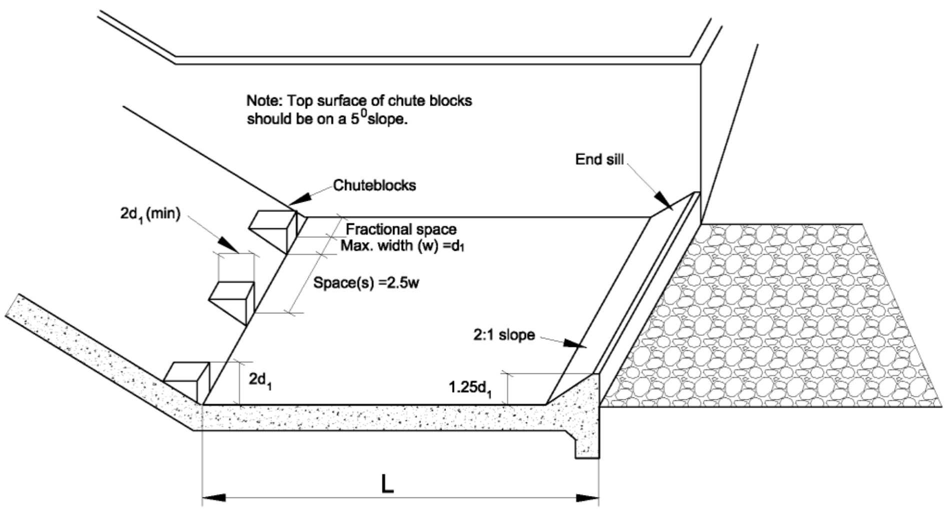
Therefore, the resulting depth below the hydraulic depth will be 4.95 feet.

Although the Froude Number of 1 divides subcritical and supercritical flow, a full-blown hydraulic depth will not from until the Froude Number reaches a value of 4.5. This has implications for the type of stilling basin required below the spillway exit channel. Because of the differences in the nature of the hydraulic jumps associated with different Froude Numbers, the type of stilling basin required depends on the Froude Number. This is discussed in some detail below.

At a Froude Number of 1.0, the flow is at critical depth and a hydraulic jump cannot form.
At Froude Numbers between 1.0 and 1.7, there will only be a gradual change from subcritical to supercritical flow, because the incoming flow is only slightly below critical depth. As the Froude Number approaches 1.7, the surface in the transition zone becomes more turbulent as small rollers begin to appear on the surface.
When the Froude Number is approximately 1.7 the conjugate depth, d_2, is about twice d_1 and about 40% greater than critical depth. Conversely, the exit velocity, v_2, is about half the incoming velocity.
For Froude Numbers below 1.7, no stilling basin is generally required. However, the channel length beyond the jump should be at least equal to $4d_2$. No other dissipating devices (e.g. baffles or sills) are required.

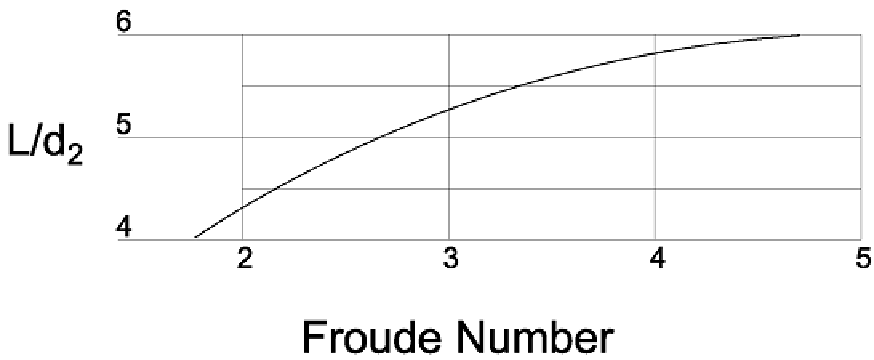
Flows associated with Froude Numbers in the range from 1.7 to 2.5 are considered to be in a transitional or pre-jump stage. Because flows in this range generally are not characterized by active turbulence, baffles and sills are generally not required. However, the channel length should be long enough to contain the downstream effects of the jump.

Flows associated with Froude Numbers in the range from 2.5 to 4.5 are still considered to be in a transitional stage because a true hydraulic jump does not occur. Stilling basins are usually not adequate in providing dissipation of these flows. There are types of stilling basins which can be adequate, however, it may be easier to adjust the flow parameters to avoid this condition. Manipulating the width of the stilling basin can be used to bring the Froude Number to 4.6, at which point a true hydraulic jump will form. If a stilling basin must be designed for a Froude Number in this range, the type of basin shown below can be somewhat effective.



Stilling Basin Dimensions for Lower Froude Numbers

The relationship of the Froude Number, length of basin and conjugate depth (d_2) for this basin is shown in the graph below:



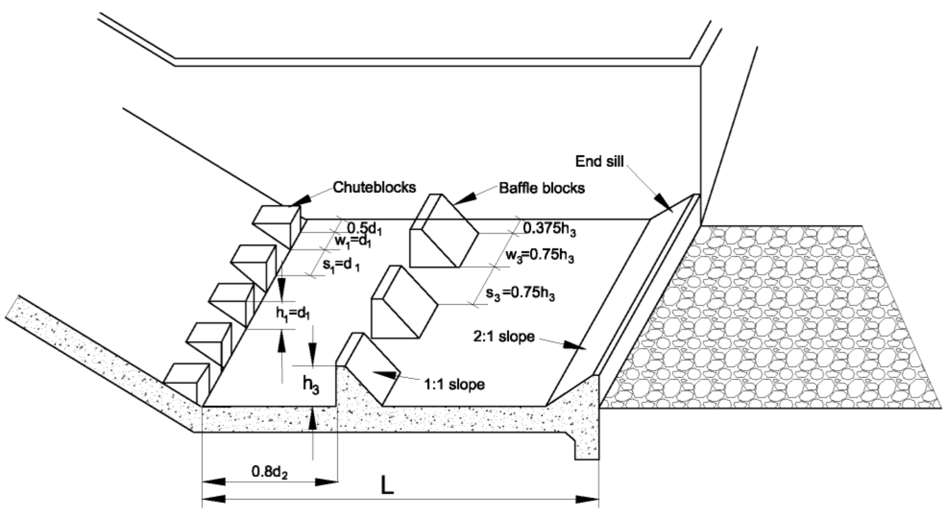
At Froude Numbers between 2.5 and 4.5 an oscillating form of hydraulic jump occurs. With a Froude Number in this range, however, the wave action propagated by the oscillating flow cannot be dampened entirely. Therefore, it is sometimes necessary to utilize auxiliary wave dampeners or wave suppressors to provide a relatively smooth water surface downstream of the basin. Also, because of the tendency of the jump to sweep out the water depths in the basin should be about 10% greater than the calculated conjugate depth. This will also act as an aid in wave suppression.

At Froude Numbers between 4.5 and 9 a stable, well balanced hydraulic jump occurs. The water surface below the jump is relatively smooth in this case.
Finally, at Froude Numbers above 9, the surface roller action and turbulence within the jump increases and causes strong wave action downstream of the jump.

Stilling Basin Design for Froude Numbers greater than 4.5: As stated above, at this point a true hydraulic jump will occur. In order to design a stilling basin that will be effective in such a case, the installation of blocks, baffles, sills, and other accessory devices will produce a stabilizing effect on the jump. When using these types of devices, it is important to take into account the added loads placed on the floor of the basin by dynamic force brought against the face of the baffle blocks. This force can be calculated by the following equation:
$Force = 2wA(d_1 + h_{v1})$, where;
W is the unit weight of water in pounds per cubic foot
A is the area of the upstream face of the block
(d_1+h_{v1}) represents the specific energy of the flow entering the basin, in feet.

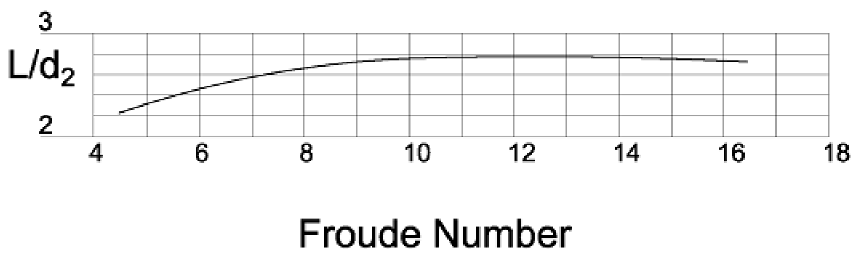
Of course, negative pressure on the back face of the baffle blocks will further increase the total load on the basin floor. However, because the baffle blocks are placed at a distance of $0.8d_2$ beyond the start of the jump, there will be some cushioning effect by the time the incoming jet reaches the face of the blocks, and the resulting force will actually be less then calculated using the above equation. Therefore, if this equation is used, the added load caused by the negative pressure can be ignored.

A typical example of a stilling basin designed for a Froude Number of greater than 4.5 is shown below:



Stilling Basin Dimensions

The relationship of the Froude Number, length of basin and conjugate depth (d_2) for this basin is shown in the graph below:



Inspection & Maintenance:
Periodic inspection and routine maintenance of spillways is absolutely essential to ensure the safe and proper functioning of the system. These inspections should be done as part of the overall dam safety inspections. The Association of State Dam Safety Officials (ASDSO) has a comprehensive set of guidelines on their website for conducting these types of inspections. Their website, www.damsafety.org, also includes numerous helpful technical publications (including both workbooks and videos) that will aid the engineer in preparing for, and conducting, safety inspections on dams. Much of this material deals with embankment integrity and other subjects which are beyond the limited scope of this course. However, technical data is also included on their website regarding inspections of principal and auxiliary spillways. Numerous state agencies also provide guidelines for inspecting and maintaining dams. The checklist below is adapted from a portion of the Oklahoma Water Resources Board Planning & Management Division – Dam Safety Program – Dam inspection Checklist. A checklist like this one makes for an easy and efficient inspection of the dam spillway.

Item	Description	Yes	No	N/A	Description (satisfactory, fair, poor, unsatisfactory)	Remarks
1	**Conduit & Outlet**	----	---	----	---------------------------	------------------
a.	Spalling/cracking/scaling?					
b.	Exposed reinforcement?					
c.	Joints displaced or offset?					
d.	Joint material lost?					
e.	Leakage of valve or gates?					Estimated gpm_____
f.	Other leakage?					Estimated gpm_____

						Clear or cloudy?
g	Conduit misaligned?					
h.	Outlet or channel obstructed?					
i.	Outlet channel eroding?					
2.	**Concrete Spillway**	----	---	----	---------------------------	------------------
a.	Spalling/cracking/scaling?					
b.	Exposed or deteriorating reinforcement?					
c.	Joints displaced or offset?					
d.	Joint material lost?					
e.	Leakage (joints, cracks, other)?					Estimated gpm______ Clear or cloudy?
f.	Wall displaced?					
g.	Dissipater deteriorating?					
h.	Dissipater clean of debris or vegetation?					
i.	Erosion at toe of spillway?					
j.	Spillway undercutting?					
3.	**Auxiliary (Emergency) Spillway**	----	---	----	---------------------------	------------------
a.	Obstructions, debris, or trees present?					
b.	Erosion or sinkholes?					
c.	Animal burrows or holes?					
d.	Evidence of livestock on spillway?					
4.	**Stilling Basin**					
a.	Spalling/cracking/scaling?					
b.	Exposed reinforcement?					
c.	Joints displaced or offset?					
d.	Joint material lost?					

e.	Joints leaking?					Estimated gpm_____ Clear or cloudy?
f.	Rock adequate?					
g.	Excessive vegetation or debris present?					
h.	Dissipater deteriorating?					
i.	Dissipaters clean of debris & vegetation?					
5.	**Flood Gates**					
a.	Floodgates broken or bent?					
b.	Floodgates eroded or rusted?					
c.	Floodgates operational?					
d.	Floodgates leaking?					Estimated gpm_____

After making the necessary inspections, the engineer must prioritize the maintenance to be conducted. If serious defects are found these should be addressed immediately. In extreme cases, it may be found that the dam is unsafe and public notification/temporary road closures or other preventative measures may be warranted.

The photograph below shows the downstream side of a pond spillway in Morris County, New Jersey. Several of the features discussed in this course are visible in this photo. The combined principal/emergency spillway is a weir shown in the upper central part of the picture. To either side of this weir are flood gates which can be removed to increase the flow out of the pond. The steep concrete exit channel below the spillway is constructed with steps to break up the flow and to act as energy dissipaters, reducing the velocity and erosive potential of the outflow. These steps also do not allow a hydraulic jump to form.

This dam spillway was constructed many years ago and serves a pond that is used for recreation purposes by a residential homeowners association. It can be seen that this spillway is properly functioning, well-maintained, and non erosive. These features must be built into the design of and dam spillway, no matter what the size of function of the pond.

