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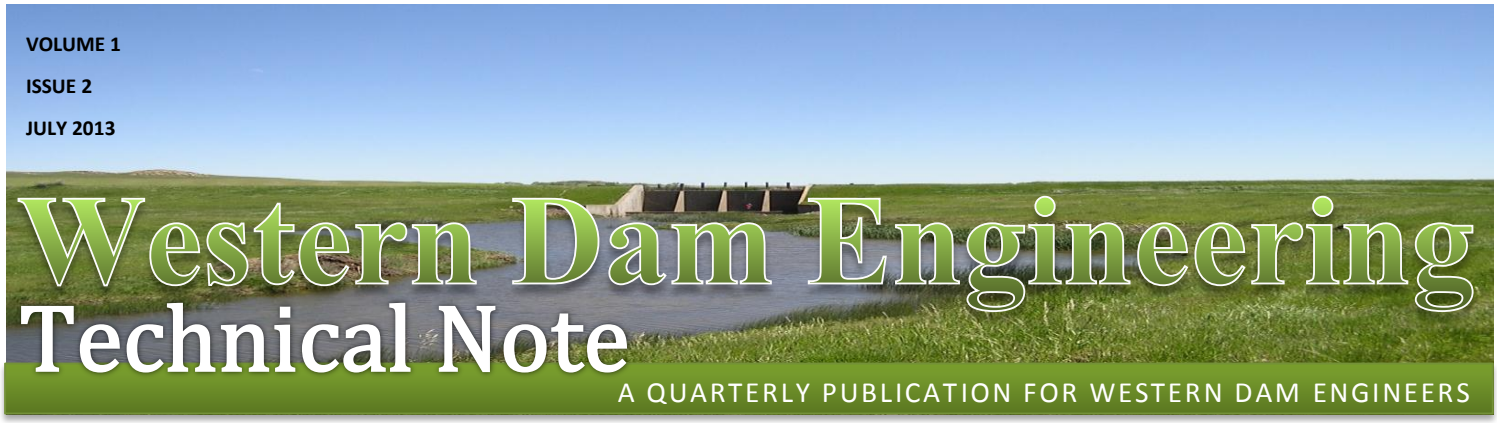

Western Dam Engineering Technical Note

A QUARTERLY PUBLICATION FOR WESTERN DAM ENGINEERS

In this issue of the ***Western Dam Engineering Technical Note*** we present articles on the importance of protecting dams against the erosive action of waves and also discuss how to protect conduits from cavitation damage through air venting. This quarterly newsletter is meant as an educational resource for civil engineers who practice primarily in rural areas of the western United States. This publication focuses on technical articles specific to the design, inspection, safety, and construction of small dams. It provides general information. The reader is encouraged to use the references cited and engage other technical experts as appropriate.

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- Wyoming State Engineer's Office

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Good to Know

Valuable Low-Cost Reference:
The Embankment Dam Reference Toolbox provides a comprehensive collection of design standards and references for dam engineering available from ASDSO.

Upcoming ASDSO Webinar Dam Safety Training:

- *Hydraulic Design of Labyrinth Weirs, by Blake P. Tullis, Ph.D., and Brian Crookston. August 20, 2013*
- *Stability Evaluations of Concrete Dams, by Guy S. Lund, P.E., and Robert Kline, Jr., P.E. October 8, 2013*

ASDSO Training Website Link

Correction

An error was found in Issue 01 of this publication. Please note the following correction to the article titled "*Simple Steps to Siphoning*": *H_{max} must be greater than the value of (DCE-RWS), H_{max}>(DCE-RWS)*
The corrected Issue 01 can be found via the following link:
Western Dam Tech Note Issue 01

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Predicting Wave Runup on Dam Slopes

Introduction

When wind blows over an open water surface, such as within a reservoir, wind-generated waves can strike the upstream slope of the dam embankment. This can cause erosion of the embankment material and if severe enough, waves can overtop the embankment, both of which are dam safety issues. Therefore, the dam embankment design must consider the potential effects of wave action and protect against erosion of the embankment materials and overtopping due to wave runup. This is done by extending the embankment up from the still water flood pool level to an elevation equal to the still water pool plus the maximum calculated wave runup and wind setup height.

This article describes a procedure for calculating the wind-generated wave characteristics for inland reservoirs and lakes and the resulting wave runup on a sloping dam embankment for small dams.

Dominant Factors and Procedure

The major variables used to calculate wind-generated wave height on open water surfaces, such as reservoirs, and influence embankment design are:

- Effective Fetch and Wind Direction
- Wind Speed over Water
- Wind Setup, Wave Height and Runup

The procedure presented in this article is based on information presented in TR-69 (USDA, 1983) and Bureau of Reclamation ACER TM-No. 2. Additional information related to US Army Corps of Engineers (USACE) procedures is presented in the reference documents included at the end of this article.

Effective Fetch and Design Wind Direction

The procedure in this article is limited to reservoirs where 1.) Effective fetch is less than 10 miles and 2.) Wave height is less than 5 feet.

The fetch is an overwater length blown on at a constant wind speed and direction. The longer the fetch and the faster the wind speed, the more wind energy is imparted to the water surface and proportionally higher waves will be produced. TR-69 recommends two approaches to determine the design fetch and wind direction: (1) U.S. Weather Service climatological data or (2) site orientation. Because most dams/reservoirs are ungauged, wind data does not typically exist and the site orientation method is preferred to define the effective fetch and design wind direction.

The design wind direction is obtained by determining the longest stretch of open water from a point on the shoreline opposite to the dam embankment. It is assumed that wind and waves are developed along the longest fetch of open water from the dam. According to Saville's 1954 study, the width of the fetch on inland reservoirs normally places a definite restriction on the length of effective fetch, which is the effective distance of the water over which the wind blows without appreciable change in direction. **Figure 1** diagrammatically shows the central (longest) and radial fetch lines for a hypothetical reservoir. Simplistically, this method involves drawing the central radial line and then drawing seven radial lines at 6-degree intervals on each side of the central radial line.

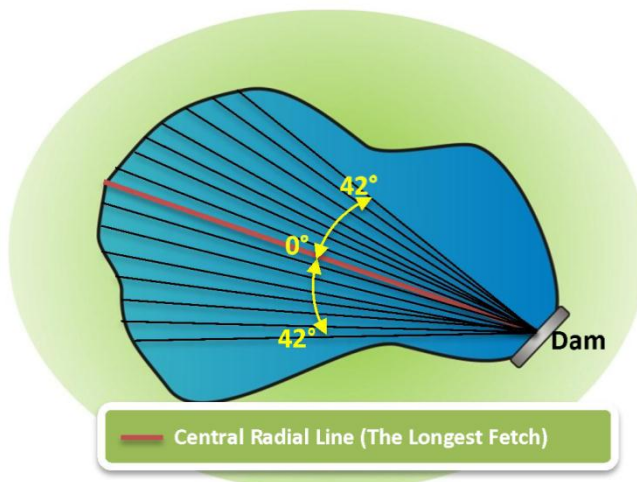

Figure 1: Central and radial fetch lines

The effective fetch, F_e, can then be computed using **Equation 1**.

$$F_e = \frac{\sum_{i=1}^{15}(x_i \cdot \cos^2\alpha_i)}{\sum_{i=1}^{15}(\cos\alpha_i)} \qquad \text{Eq.1}$$

x_i = Length of Radial Line i

α_i = Angle Degree between the Central Radial Line and the Radial Line i

Wind Speed over Water

There are two common procedures for determining the design wind speed. They are:

1. *A constant overwater wind speed of 100 mph (Reclamation, 1987)*
2. *Site-specific wind speed and duration curves*

The 100 mph wind speed recommended by Reclamation is a simple but conservative approach. The more detailed site-specific approach is presented in the following paragraphs.

According to the guidelines titled Reclamation ACER TM-No. 2 and TR-69, the design wind speed and duration can be selected by using the observed maximum wind speed and the effective fetch. Commonly, the observed fastest mile wind speed is considered as the maximum overland wind speed, U_L, and can be obtained from the National Oceanic and Atmospheric Administration (NOAA) National Climatic Data Center websites shown at the end of this article. The NOAA wind data, including wind speed, duration, and direction, indicates the overland wind characteristics at 25 feet above ground.

The duration of a given wind speed that needs to be maintained to fully develop the maximum waves is a function of the effective fetch. The longer the effective fetch, the longer the duration for the sustained wind speed. **Figure 2** graphically shows the selection of design wind speed based on the relationship between the maximum wind speed and the effective fetch response to wind speed. The intersection of the red curve and blue curve identifies the "design wind speed."

The red line on **Figure 2** can be developed using the observed fastest mile wind speed and the information contained in Figure 5 of TR-69. Alternatively, **Table 1** is provided as a simplification of the information shown in Figure 5 of TR-69.

Careful!! The units for effective fetch and wind speed vary for the various equations in this article. Make note of units required for each eqn.

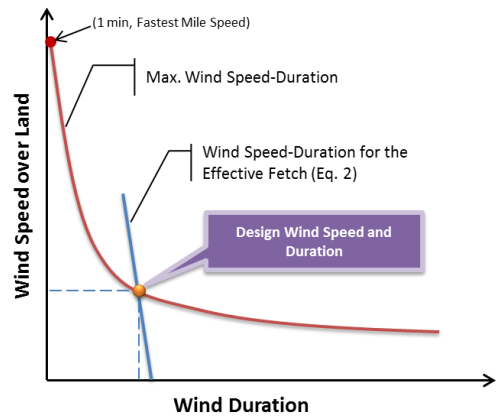

Figure 2: Plot of wind speed vs. duration

Table 1: Maximum wind speed relationship

Fastest Mile Wind Speed, mph	Ratio of Land Wind Speed to the Fastest Mile Wind for the Durations			
	1 min*	30 min	60 min	100 min
100	100%	52%	46%	41%
80	100%	57%	51%	47%
60	100%	65%	59%	55%

* Duration of fastest mile wind speed is one minute.

The blue curve in **Figure 2** needs to be generated using Figure 2 in TR-69 or the empirical relationship (**Equation 2**) of overland wind speed and duration for the site specific effective fetch.

$$g\frac{T}{U_L} = 27.99\left(\frac{g \cdot F_e}{{U_L}^2}\right)^{0.72} \qquad \text{Eq.2}$$

g = Gravitational Acceleration, 32.2 ft/sec^2
T = Wave Duration in seconds. Wave duration is equal to the minimum wind duration required for generation of wave heights for a specific effective fetch and wind speed.
U_L = Overland Wind Speed in ft/sec
F_e = Site Specific Effective Fetch in ft

Because of smoother and more uniform surface conditions, overwater wind speeds, U_w, are higher than overland wind speeds, U_L. To consider this speed enhancement, the overwater wind speed can be computed using the following equation.

$$U_w = \beta \cdot U_L \qquad \text{Eq.3}$$

β = Wind Speed Adjustment Factor or Ratio, U_w/U_L, Shown on **Figure 3**.

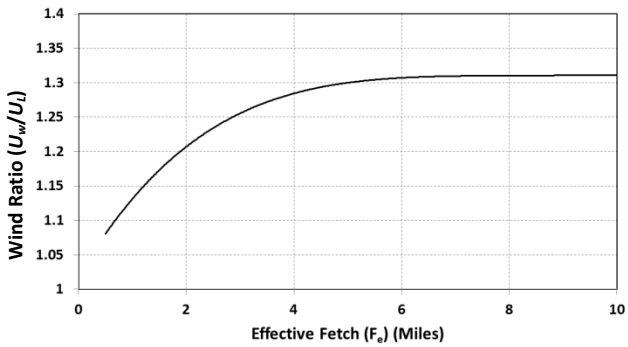

Figure 3: Wind speed relationship – water to land

Wind Setup, Wave Height, and Runup

A sketch of waves striking an embankment slope is illustrated in **Figure 4**. When wind is blowing over a water surface, horizontal shear stress acts on the water surface, and the water surface is tilted in the direction of the wind. This wind effect is termed "wind setup" and can be estimated using the empirical equation from TR-69 shown below.

$$S = \frac{{U_W}^2 \cdot F}{1400 \cdot D} \qquad \text{Eq. 4}$$

S = Wind Setup in feet
U_w = Wind Speed in miles per hour
F = Wind Fetch in miles (Approximately equal to F_e)
D = An approximation of the average water depth along the fetch length in feet

Slope protection is generally designed for what is known as the "significant wave height." The significant wave height is the average height of the highest one-third of the wind-generated waves. This means that 33 percent of the waves that hit the slope will be higher than this value. Based on the selected design overwater wind speed and the effective fetch, the significant wave height, H_s, and wave length, L, can be estimated using the following dimensionless equations from TR-69.

$$g\frac{H_S}{{U_L}^2} = 0.0026\left(\frac{g \cdot F_e}{{U_L}^2}\right)^{0.47} \qquad \text{Eq. 5}$$

$$g\frac{\sqrt{L}}{U_L} = 1.041\left(\frac{g \cdot F_e}{{U_L}^2}\right)^{0.28} \qquad \text{Eq. 6}$$

g = Gravitational Acceleration, 32.2 ft/sec^2
H_s = Significant Wave Height in feet
L = Wave Length in feet
U_L = Overland Wind Speed in ft/sec
F_e = Effective Fetch in feet

Equations 5 and **6** are empirical equations developed from deep-water waves, which are defined as waves having lengths equal to or less than $2D$. They also give conservative wave height estimations for shallow-water waves.

The significant wave height defined above would be exceeded by approximately 33 percent of the expected waves generated by the associated wind speed. If a lower potential of exceedance is desired, a wave height of 1.27H$_s$ and 1.67H$_s$ have a corresponding potential for exceedance of 10 percent and 1 percent, respectively.

When waves reach a sloping embankment, the waves will eventually break on the slope and run up to a height governed by the angle of the slope, and the surface roughness and permeability. Wave runup height, R, is the difference between the maximum elevation reached by wave runup on a slope and the storm water level. The steeper the embankment slope the greater the wave runup height. Many studies have been published that provide guidance for determining the wave runup height on slopes. The runup from a significant wave on an embankment slope with riprap protection can be predicted using:

$$R = \frac{H_S}{0.4 + \left(\frac{H_S}{L}\right)^{0.5} \cdot \cot\theta} \qquad \text{Eq. 7}$$

R = Wave Runup Height in feet
H_s = Significant Wave Height in feet
L = Wave Length in feet
θ = Angle of the Dam Face from Horizontal

Equation 7 should be used only for embankment slopes steeper than 5H:1V.

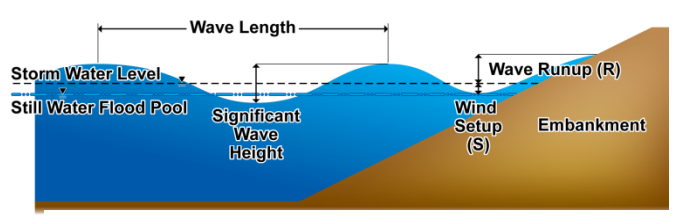

Figure 4: Sketch illustrating wave terms

Conclusions

The wind-generated wave characteristics and the related wind setup and wave runup on a sloping embankment within a reservoir must be considered for the purposes of designing embankments and embankment slope protection. Slope protection for the embankment must also be considered and a procedure for the design of riprap slope protection is described in the following article titled, "Design of Riprap for Slope Protection against Wave Action."

NOAA Climatological Data Links

Local Climatological Data:
http://www.ncdc.noaa.gov/IPS/lcd/lcd.html

Climate Maps of the United States:
http://cdo.ncdc.noaa.gov/cgi-bin/climaps/climaps.pl

NOAA Climate Data Online:
http://www.ncdc.noaa.gov/cdo-web/

References (with Links where available)

- USDA (1983), Technical Release No. 69: "Riprap for Slope Protection against Wave Action."
- Reclamation (1992), ACER Technical Memorandum No. 2: "Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams."
- Reclamation (1987), Design of Small Dams, Third Edition.
- USACE (1976), Engineering Technical Letter No. 1110-2-221: "Wave Runup and Wind Setup on Reservoir Embankments."
- Saville, Thorndike J. (1954), "The Effect of Fetch Width on Wave Generation," Technical Memorandum No. 70, Beach Erosion Board, USACE.

Example #1:

Find the wind setup, the wave height and the wave runup of a reservoir as shown on **Figure 1.** The observed fastest mile wind speed is 75 mph for this site. The average depth of the reservoir is 10 feet, and the riprap protected embankment has a 3H:1V or 18° slope.

Calculations:

1. To measure the lengths of the central (longest) and radial lines as shown in **Figure 1**, compute the effective fetch using **Equation 1.** The computation is shown in **Table 2**.

Table 2: Procedure to determine the effective fetch

Radial No.	Radial Length (mi), X_i	α (Degree)	cos α	$X_i \cdot \cos^2 \alpha$
1	1.7	42	0.74	0.96
2	1.8	36	0.81	1.20
3	1.9	30	0.87	1.45
4	2.0	24	0.91	1.70
5	2.2	18	0.95	2.02
6	2.3	12	0.98	2.23
7	2.4	6	0.99	2.41
8	2.6	0	1.00	2.63
9	2.5	6	0.99	2.51
10	2.4	12	0.98	2.33
11	2.3	18	0.95	2.11
12	2.1	24	0.91	1.78
13	2.0	30	0.87	1.53
14	1.8	36	0.81	1.20
15	1.7	42	0.74	0.96
		Sum=	13.51	27.02

$$F_e = \frac{\sum_{i=1}^{15}(X_i \cdot \cos^2\alpha_i)}{\sum_{i=1}^{15}(\cos\alpha_i)} = \frac{27.02}{13.51} = 2.0 \text{ miles}$$

This effective fetch of 2.0 miles or 10,560 feet from the given reservoir with a longest fetch of 2.6 miles is estimated.

2. Refer to Figure 5 of TR-69 or **Table 1** in this article, the generalized maximum wind speed-duration relationship is plotted as the red line on **Figure 5**. This is computed by using the observed fastest mile wind speed, 75 mph, interpolating the ratio of land wind speed to the fastest mile wind for each of the durations shown and then multiplying this ratio by the observed fastest wind speed. The results of these computations are shown in **Table 2**.

Table 2: Maximum Wind Speed-Duration Relationship for a Fastest Mile Wind of 75 mph

	1 min	30 min	60 min	100 min
Interpolated Ratio from **Table 1**	100%	59%	53%	49%
Corresponding Max. Wind Speed (mph)	75	44	40	37

3. By using **Equation 2** and the effective fetch, 2.0 miles, the relationship of overland wind speed-duration for the selected fetch is determined for a range of selected speeds (in this case, U_L= 90 mph, 60 mph, and 35 mph). Remember to first convert U_L to ft/sec and fetch length to feet. T is calculated in seconds with **Equation 2** and then converted to minutes for the plot. The results are shown as the blue curve in **Figure 5.**

$$32.2 \cdot \frac{T}{U_L} = 27.99 \left(\frac{32.2 \cdot 2.0 \cdot 5280}{{U_L}^2} \right)^{0.72}$$

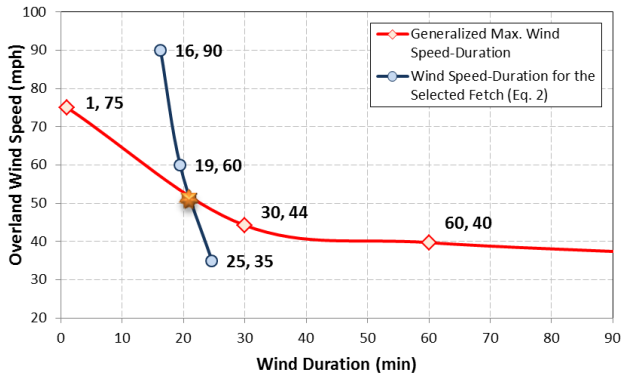

Figure 5: Plot of wind speed vs. duration

4. The intersection of the red curve and blue curve identifies the "Design Overland Wind Speed (U_L)" of 52 mph or 76 ft/sec. Find the overwater wind speed using **Figure 3** and **Equation 3**. This gives a wind ratio of 1.21 from the figure and an adjusted overwater wind speed (U_w) of 63 mph.

$$U_w = \beta \cdot U_L = 1.21 * 52\ mph = 63\ mph$$

5. Find the wind setup using **Equation 4**.

$$S = \frac{63^2 * 2.0}{1400 * 10} = 0.6\ feet$$

6. Find the wave height using **Equation 5**.

$$H_s = \frac{76^2}{32.2} \cdot 0.0026 \left(\frac{32.2 \cdot 10560}{76^2} \right)^{0.47} = 3.2\ ft$$

7. Find the wave length using **Equation 6**.

$$L = \left(\frac{76}{32.2} \cdot 1.041 \left(\frac{32.2 \cdot 10560}{76^2} \right)^{0.28} \right)^2 = 59\ ft$$

8. Find the wave runup height using **Equation 7**.

$$R = \frac{3.2}{0.4 + \left(\frac{3.2}{59}\right)^{0.5} \cdot \cot(18^0)} = 2.9\ ft$$

Results:

The estimated maximum significant wave height is 3.2 feet with an overwater wind speed of 63 mph. The corresponding maximum height the water will reach from the still water flood level is 3.5 feet, which is the sum of wind setup (0.6 foot) and runup (2.9 feet).

Design of Riprap for Slope Protection against Wave Action

Introduction

This article is intended to provide practical guidance to engineers for the design and construction of riprap for embankment dams, particularly small embankment dams. This article is not intended to be an all-inclusive guide. A list of commonly used references on the topic is provided at the end of this article.

As discussed in the previous article of this issue, earthen embankment dams can be subject to erosion by wave action within the reservoir. In 1983 the USDA developed a technical release (TR-69) that describes procedures for the design of rock riprap protection for earthen embankments to protect against wave action. TR-69 was used as the basis for this article. Detail not found herein can be found in TR-69 and the associated references. As mentioned in the previous article the design procedures described in TR-69 are generally limited to reservoirs having an effective fetch length of less than 10 miles and significant wave height of less than 5 feet.

Additional relevant publications for guidance on the design of riprap slope protection include Chapter 7 of "Embankment Dams" (Reclamation 1992), "Design of Small Dams" (Reclamation 1987), "Design of Coastal Revetments, Sea Walls, and Bulkheads" (USACE 1995) and "Design of Riprap Revetment" (FHWA 1989).

Why Riprap?

Slope armoring acts as primary protection against embankment erosion caused by wind and wave action within the reservoir. Excessive erosion of a dam embankment can lead to embankment failure. Inadequately designed or installed riprap can pose a dam safety risk. For successful performance, a riprap layer must be designed to:

- Protect the individual rock particles from displacement by the wave force, and
- Keep the protected earth, filter, and bedding underlying the riprap from being washed out through the voids in the riprap.

Figures 1 and **2** are examples of the embankment erosion that can occur without adequate protection against wave action.

Figure 1: Erosion of a small embankment dam in Montana caused by wave action.

Figure 2: Erosion of a small embankment dam in Montana caused by wave action.

Riprap is one material commonly used as armoring for upstream slope protection. There are other commercially available armoring materials, each with their own design considerations and methodologies.

Some of these alternate materials include articulated concrete blocks, cellular concrete mats, and in some low wave-energy sites, vegetation or geosynthetic reinforced vegetation. This article focuses on the design of riprap armoring, as it is the most commonly preferred and installed material.

Procedure

In general terms and in TR-69, the procedure for the design of riprap can be summarized as a flow chart as shown on **Figure 3**. This procedure is described in the following sections of this article and an example (Example #2) is provided at the end of this article. Example #2 is a continuation of Example #1 from the previous article in this newsletter.

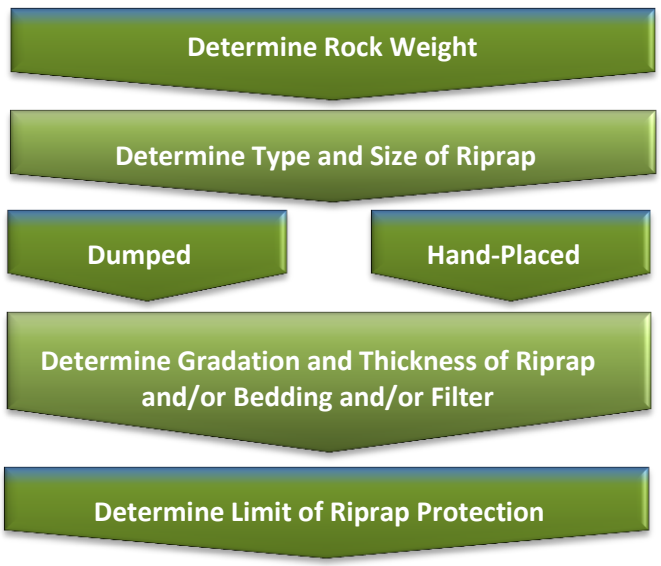

Figure 3: Summary flow chart procedure for design of riprap.

Determining Rock Weight

In accordance with TR-69, the equation to estimate the required riprap rock weight (W_{50}) can be given as:

$$W_{50} = \frac{19.5\, G_S\, H_S^3}{(G_S - 1)^3 \cot \alpha} \qquad \text{Eq.1}$$

G_S = Specific Gravity
H_S= Significant Wave Height (See previous article for calculation method)
cot α = Horizontal Component of Embankment Slope

Rock weight can also be estimated using Figure 8 in TR-69. As the embankment slope and or significant wave height increases, the calculated W_{50} rock weight also increases. Conversely as the embankment slope and or significant wave height decreases, the calculated W_{50} rock weight reduces.

Determining Type, Size, Thickness and Gradation

There are two types of rock placement described in TR-69:

- Type 1 – Dumped (Equipment-Placed) Rock
- Type 2 – Hand-Placed Rock

Dumped rock is regarded as superior to hand-placed rock because of historically low maintenance costs. Experience has also shown that in most cases dumped rock provides the best upstream slope protection at the lowest ultimate cost. For these reasons, only dumped rock is discussed further in this article.

The procedure for determining the physical riprap characteristics described in TR-69 for Type 1 (dumped) rock is as follows:

- Size: using the W_{50} weight of rock, find rock size (D_{50}) using Figure 9 (TR-69) or the equations provided with the figure. Usually the equation for spherically shaped rock is used to estimate rock size for riprap as follows:

$$D_{50} = 1.24 \sqrt[3]{\frac{W_{50}}{62.4\, G_S}} \qquad \text{Eq.2}$$

  Where, G_S = Specific Gravity
- Gradation: using the rock size, find the gradation limits using Figure 10 (TR-69).
- Thickness: two times the D_{50} rock size.

According to the hazard category of the dam a safety factor can also be applied to the calculated D_{50} rock size and this is described in "Slope Protection for Dams and Lakeshores" (USDA 1989). Alternative methods for determining riprap size, thickness, and gradation are described in Chapter 7 of "Embankment Dams" (USBR 1992).

Generally riprap should be hard, dense angular stone, graded as designed, comprising sound fragments resistant to abrasion and weathering and be free of cracks, seams, clay, organic material and other defects. Rounded boulders or cobbles are not generally acceptable as riprap.

Bedding and Filters

Once the gradation of the riprap is determined, the gradation and thickness of the bedding layer should be determined. In principal, the bedding layer provides a foundation for the riprap placement and also provides a filter-compatible transition layer to finer, underlying embankment materials. The finer embankment material underlying the riprap could be washed out through the rock particles during reservoir fluctuations and wave action. Retention of the underlying embankment materials is attained by placing a finer-grained layer of bedding under the riprap. Where very large riprap is used, a progressively finer two-stage bedding/filter layer may be required. The bedding layer needs to be filter-compatible with both the underlying embankment material and overlying riprap to limit the potential of erosion and washout of both embankment and bedding material between the voids of the riprap.

Generally bedding should be a well-graded mixture of gravel and sand that is filter-compatible with both the riprap and the embankment materials. There is some general guidance on developing the filter-compatible gradation and the recommended thickness provided in Chapter 7 of Embankment Dams (Reclamation 1992) and in Design of Riprap Revetment (FHWA 1989). The general guidance for bedding thickness is summarized in **Table 1**.

Table 1: Bedding layer thickness according to riprap layer thickness.

Riprap Layer Thickness	Bedding Layer Thickness
12-24”	9”
27-36”	12”
Over 36”	15”

Limit and Layout of Riprap Protection

According to TR-69, the lower limit of the riprap protection should be 1.5 times the significant wave height (H_s) below the reservoir normal water level at the lowest ungated opening, or below the lowest controlled outlet.

The upper limit of riprap is described by TR-69 as the vertical distance above the reservoir still water flood pool level equal to the sum of the wave runup (R) and wind setup (S). This can be calculated as described in the previous article. The lower limit of riprap is determined by the lower of either the (a) vertical distance of 1.5 times H_s below the still water flood pool, or (b) lowest controlled outlet elevation.

The upper and lower limits of riprap are shown on **Figure 4**.

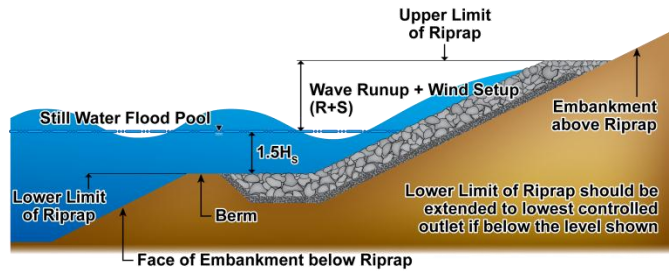

Figure 4: Typical upper and lower limits for riprap placement.

For owners of existing small dams the extent of a riprap revetment may be limited by the budget available to complete the project. When this is the case the owner and designer should carefully consider where the riprap can offer best value from a dam safety and operational perspective. Priorities could include, but may not be limited to, providing riprap on sections of the embankment where erosion has previously occurred, is deemed likely to occur (i.e., adjacent to concrete structures and other infrastructure), and or in horizontal bands at the reservoir normal water level or normal operational water level. Experience has shown that dam embankments built with interior or exterior bends or at angles that are perpendicular to prevailing winds, can be more susceptible to erosion. Armoring of these areas should be a priority.

Placement

According to TR-69, for dumped rock, the placement of bedding and riprap on a dam embankment should be as shown on **Figure 5**. This figure shows the riprap supported by a level berm (also refer to **Figure 4**), which facilitates placement.

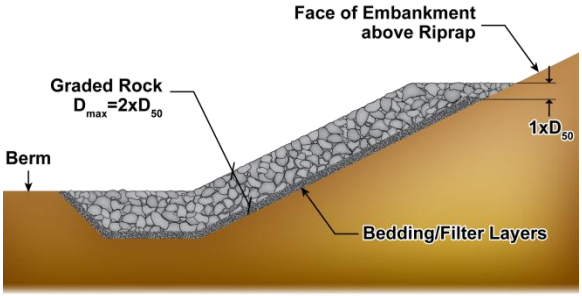

Figure 5: Dumped rock placement detail.

Where construction of a berm is impractical or on an existing slope, keying of the riprap into the slope is recommended to prevent displacement of riprap down the slope. A reference published by the Minnesota NRCS state office titled "Slope Protection for Dams and Lakeshores" (USDA 1989) provides alternatives for keying riprap into existing slopes where the riprap will not extend the full height or length of the dam.

The placement of riprap should be done by mechanical means, such as a hydraulic excavator. Dumping riprap from a truck down an embankment should be avoided as it can cause segregation of the rock by size and result in unsuitable gradation. Placement should be performed to produce a well-graded, even mass of rock with uniform cover and minimal voids. Laborers should be provided during placement for rearrangement of loose rock fragments, "chinking" of void spaces, and hand-placement as needed to provide a well-keyed and stable layer of riprap.

Figure 6 shows dumped riprap being placed over bedding on the upstream slope of a small dam in South Dakota.

Figure 6: Placing riprap bedding and riprap on the upstream slope of a small dam in South Dakota.

Conclusion

Properly designed and installed riprap can provide erosion protection from wave action that would otherwise cause significant damage of earthen embankment dams. For riprap to be effective the designer must calculate the required riprap weight, size and gradation, and specify acceptable material properties. The designer must carefully consider bedding and/or filter requirements to ensure that they are compatible with the embankment material and the riprap itself.

References (with Links where available)

To aid the designer through the process, the following is a list of design references that can be used:

- USDA (1983), Technical Release No. 69: Riprap for Slope Protection against Wave Action.
- USDA (1989), Minnesota Technical Note 2: Slope Protection for Dams and Lakeshores.
- Reclamation (1992), "Design Standards No. 13: Embankment Dams", Chapter 7 – Riprap Slope Protection, U.S. Department of Interior, Bureau of Reclamation, Technical Service Center, Denver, CO. *Look for the revised version of this reference at the following link soon:* Reclamation Dam Design Standards
- Reclamation (1987), "Design of Small Dams", U.S. Department of Interior, Bureau of Reclamation.
- FHWA (1989), "Design of Riprap Revetment", Federal Highway Administration, McLean, Virginia.

Example #2:

Design embankment riprap protection for the dam described as Example #1 in the previous article of this newsletter. The upstream dam embankment slope is 3H:1V, the significant wave height (H_s) was calculated as 3.2 ft and the specific gravity (G_s) of the riprap source is 2.65.

Calculations:

1. Determine the required W_{50} rock weight for the riprap using **Equation 1**:

$$W_{50} = \frac{19.5 * 2.65 * 3.2^3}{(2.65 - 1)^3 * 3.0}$$

$$W_{50} = 126\ lbs$$

2. Using the W_{50} rock weight determine the D_{50} rock size for the riprap using **Equation 2**:

$$D_{50} = 1.24 \sqrt[3]{\frac{126}{62.4 * 2.65}}$$

$$D_{50} = 1.15 \text{ feet}$$

The riprap layer thickness and maximum rock size is calculated as two times the D_{50} rock size. Using the D_{50} rock size of 1.15 feet, D_{MAX} is 2.3 feet.

3. Using the D_{50} rock size estimate the gradation limits using Figure 10 (TR-69). Gradation limits for a riprap with a D_{50} rock size of 1.15 feet are shown in Table 2.

Table 2: Gradation limits for a D_{50} rock size of 1.15 feet.

Rock Passing Sieve	Size
100%	21 to 28”
85%	19 to 26”
50%	14 to 20”
15%	2 to 9”

4. Using the guidance on the bedding layer thickness provided in Chapter 7 of “Embankment Dams” (Reclamation 1992), adopt the bedding layer thickness as 12”. Determine the gradation of the bedding and any requirements for a filter layer in accordance with the aforementioned reference, TR-69 and “Design of Riprap Revetment” (FHWA 1989).

5. Determine the limit and layout of the riprap protection. Consider the limits described in this article and in TR-69.

Design Considerations for Outlet Works Air Vents

Introduction

Outlet works air vent design is often a difficult, misunderstood, or even unknown subject for many design engineers. This article introduces the subject of air demand and air vent sizing, and discusses possible consequences of inadequate air vent design. Important design criteria and guidelines are summarized, and a conservative, generalized approach for estimating air demand and sizing air vents is provided.

Several references containing alternate design methodologies are presented in this article; however, it is cautioned that there are limitations associated with each design method. Designers should check these limitations to ensure the specifics of their projects are consistent with the methods being employed.

Why Air Vents?

An important consideration in any closed conduit design for an outlet works is the proper use of air venting. An air vent simply allows air under atmospheric pressure to flow into an outlet works conduit, introducing (or entraining) air into the flow. Specifically, a properly designed air vent serves the following purposes:

- Reduces potential for formation of low pressures within the flowing water;
- Reduces potential for unstable flow in the conduit; and
- Allows bleeding of air from a conduit prior to operation.

Air vents are typically installed downstream from a control gate or valve, where formation of low flow pressures can occur. In the absence of adequate air venting, low flow pressures can lead to cavitation, air blow back, pipe collapse, excessive vibration, and excessive noise. Each of these possible consequences is discussed below.

Consequences of Inadequate Air Vent Design

Cavitation, or the formation of vapor cavities (bubbles) in low pressure areas just downstream from the control gate/valve, is the most common consequence of inadequate air vent design. As cavitation bubbles are carried downstream from the gate into higher pressure flow areas, they rapidly collapse (implode), sending out high-pressure shock waves that can damage a conduit wall near the implosion. Cavitation damage generally occurs downstream of the gate slots in the outlet works, but can also occur on the invert downstream of the control gate. **Figure 1** shows typical cavitation damage on an outlet gate and conduit walls.

Figure 1: Typical cavitation damage on gate and conduit

Air blowback can occur as air collects on the crown of the conduit downstream of a control gate and forms a large pocket of air that can violently "blow back" toward the control gate and intake structure, causing damage to those structures.

Pipe collapse downstream from a gate can also occur if low pressure flow is extreme enough, as illustrated in **Figure 2**.

Excessive vibration in low pressure or unstable flow areas downstream of a control gate can lead to structural damage of the conduit and gate, if severe enough.

Excessive noise can occur at the air vent opening if the air vent is designed too small. The noise can even be so loud that it is damaging to hearing. At one dam, nearby residents complained of a popping noise coming from the air vent that was keeping them up at night.

Figure 2: Outlet pipe liner collapse due to cavitation

Air Demand

Estimating air demand is the most important component of adequate air vent sizing. Air demand refers to the amount of air that the flowing water pulls into the conduit (and entrains into the flow) through the air vent and through the downstream exit portal (if not submerged).

There are a number of variables that can influence air demand, including:

- Gate opening height
- Head
- Volume flow rate and velocity of water
- Flow type (e.g., free surface flow, or hydraulic jump that closes the conduit)
- Froude Number
- Gate geometry and roughness
- Conduit length, diameter, cross section shape, and roughness
- Water surface roughness
- Outlet submergence
- Air vent geometry (e.g., entrance, bends) and head loss
- Altitude

Air demand is usually greatest at small (5 to 10 percent open) and large (between 50 to 100 percent open) gate opening heights. **Figure 3** illustrates the effect of gate openings on air demand. At small gate openings and when flow is not influenced by tailwater conditions or by a hydraulic jump, "jet flow" occurs, which entrains large quantities of air as the water jet frays or breaks up. At large gate openings and free surface flow conditions, air demand is caused by the drag force between the water surface and the overlying air column. Air demand for flow involving a hydraulic jump has been shown by studies to represent the lower bound of free surface flow air demand. When the conduit flows full, or when the gate is at the downstream end of the conduit (open to atmospheric pressure), air demand is zero.

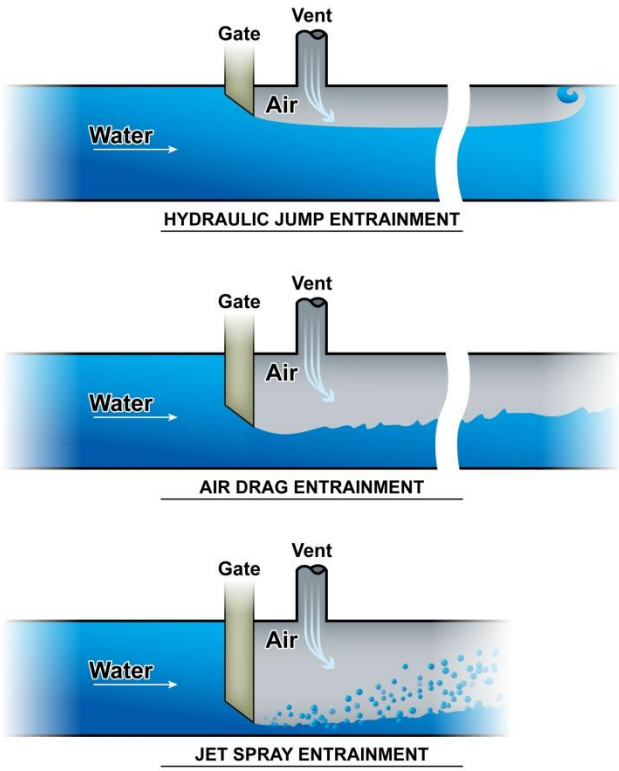

Figure 3: Gate Opening, Flow Type, and Air Entrainment

In addition to gate opening height and flow type, the other variables bulleted above influence air demand to varying degrees. Accounting for these variables in air demand estimation can be challenging for the practicing design engineer, as there is currently no known comprehensive methodology applicable to the wide range of possible outlet works configurations and hydraulic conditions represented by these variables.

Fortunately, for small to medium size dams where air vents are likely not nearly as costly as for large dams, a conservative design approach summarized below can be employed, wherein the air vent is oversized, negating the need for rigorous hydraulic analysis or model studies to account for all the variables. In cases where cost is a more significant issue, such as for low budget projects or for larger or more complex dams, a number of references describing alternate methodologies are provided below.

A Generalized, Conservative Design Approach

For flow in gated closed conduits with free surface open channel flow conditions (i.e., jet flow and air drag flow), the following equation, obtained from the 1980 publication *Air-Water Flow in Hydraulic Structures* (See references for full citation.) may be used to calculate maximum theoretical airflow rate:

$$\left(\frac{Q_a}{Q_w}\right) = \frac{A_d}{A_{wp}} - 1$$

where:

$\left(\frac{Q_a}{Q_w}\right)$ = Air Demand Ratio

Q_a = Volume Flow Rate of Air

Qw = Volume Flow Rate of Water

A_d = Cross Sectional Area of Conduit

A_{wp} = Maximum Cross Sectional Area of Water in Conduit

Ideally, a conduit water surface profile should be calculated for a range of gate opening heights to arrive at $A_{wp.}$ Alternatively, A_{wp} can be approximated from the water surface profile corresponding to a gate opening of 75 percent under maximum design head, as studies have shown that maximum air demand typically occurs at/near 75 percent gate opening and maximum design head. As a rough check, the design engineer should verify that the maximum volume flow rate of air is approximately equal to the maximum flow rate of water.

For cases where the water surface profile indicates that a hydraulic jump will occur, the following equation from *Air-Water Flow in Hydraulic Structures* may be used:

$$\left(\frac{Q_a}{Q_w}\right) = 0.0066(F_r - 1)^{1.4}$$

where:

F_r = Froude Number Upstream of the Hydraulic Jump *(Note: F_r is a dimensionless index of flow regime (i.e., subcritical or supercritical)).*

In a circular pipe, F_r can be calculated from the flow depth *y* by using the following equation:

$$F = \frac{V}{(gy_e)^{1/2}}$$

where:

V = Mean Flow Velocity

g = Gravitational Constant

y_e = Effective Depth = A/T

A = Cross Sectional Area of the Water in the Conduit

T = Top Width of Flow Passage = $2[y(D-y)]^{1/2}$

D = Conduit Diameter

Y = Flow Depth

After Q_a is calculated, a maximum design air velocity can be selected, and the cross sectional area and diameter of the air vent can be calculated. An example calculation using this design method is provided at the end of this article.

As a side note, the Bureau of Reclamation conservatively designs their outlet conduits so that a hydraulic jump will theoretically never occur, while the U.S. Army Corps of Engineers (USACE) allows hydraulic jumps in outlet conduits at their dams.

Alternative Design Methodologies

The 1980 USACE *Engineering Manual Hydraulic Design of Reservoir Outlet Works* (EM 1110-2-1602), together with "Hydraulic Design Criteria" 050-1 and 050-2, present a method of estimating air demand and sizing the air vent based on an envelope design curve that was developed from outlet works air demand data from 5 different dams with heads ranging from 24 to

370 feet. The method relates Froude number and air demand ratio and is generally applicable for slide and tractor gates operating in rectangular gate chambers. The envelop design curve may underestimate air demand in some cases, such as for Beltzville Dam, where actual air demand was 5 times higher than the air demand derived from the design envelop curve. This illustrates the necessity for the designer to check the limitations and applicability of a given method to ensure the specifics of their projects are consistent with the methods being employed. A spreadsheet that employs this design method is attached to this document.

The 2011 paper titled, *Determining Air Demand for Small- to Medium-Sized Embankment Dam Low-Level Outlet Works* presents a design method for estimating air demand and sizing the air vent based on laboratory-scale low-level outlet tests with an inclined gated inlet on a 3H:1V slope. The design methodology presents a series of design curves that relate gate geometry (and corresponding discharge coefficient), driving head, gate opening (10, 30, 50, 60, 70, and 90 percent), and air demand ratio. The design method uses an envelope curve of all the observed model data; with the limitation that parameters such as conduit length and air vent geometry (and associated head losses) were not considered in the model, and the method may not be applicable for gates with inclinations different than 3H:1V.

The 2008 thesis titled, *Air Demand in Free Flowing Gated Conduits* summarizes empirical design methodologies developed by previous researchers, and presents observations on significant parameters developed from a laboratory model study. The parameters studied included: Froude number, ratio of head to gate opening, surface water roughness, conduit length, and conduit slope. A possible limitation of this study is that the model air velocity measurements were not sufficiently detailed to draw conclusions.

Air Vent Design Criteria and Guidelines

The following criteria and guidelines are commonly employed in air vent design practice:

- Limit maximum air flow velocity in the air vent to approximately 100 feet/second by increasing the vent size as necessary; above this velocity an objectionable, whistling noise occurs that can be damaging to hearing.
- For safety reasons, keep children away from vent openings, and place personnel barriers around vents if the air velocity is expected to exceed approximately 50 feet/second.
- A minimum air vent diameter of 4 inches should be used for all cases to facilitate vent cleaning and maintenance.
- For valves, the air vent is typically located upstream from the point where the water jet impinges on the conduit walls.
- If the air vent is of sufficient size to interrupt rebar in the conduit wall, use a series of smaller, side-by-side air vents.
- Install an air vent through HDPE and CIPP pipe liners if there is susceptibility to internal vacuum pressures and liner collapse.
- If steel vent pipes are used and will be in contact with corrosive soils, design appropriate cathodic protection, or use a protective coating or wrap.
- A typical configuration for the end (open to atmosphere) of the air vent is to include a 90 degree elbow (see **Figure 4**) with an expanded or bell-mouth opening oriented away from the prevailing winds, with a stainless steel screen over the opening, which will help prevent debris from entering the vent, and help prevent water from entering the pipe, which could result in freezing blockage during the winter.
- Avoid air vent design features that could result in large head losses such as a small-mesh steel screen, or an excessive number of vent pipe bends.
- Take precautions against small objects (e.g., rodents, clipboards, etc.) getting sucked into the vent and creating a potential blockage; periodically inspect the air vent to ensure air is flowing freely through it and that there are no

blockages, corrosion, or structural damage that may affect performance.

- For cases where it is not possible for an air vent to have direct connection to the atmosphere, such as for control gates located in outlet works tunnels, air demand must be supplied by an air duct above the free surface of the flowing water, and the hydraulic design should ensure flow never rises to the level of the air duct.

Figure 4: Typical outlet works air vent for a small dam

It is also important to point out that there are several outlet works hydraulic flow issues that are commonly misattributed to insufficient air vent size, but are actually associated with inadequate hydraulic design or operations errors. These include surging, structural damage due to filling the pipe too rapidly, and bi-stable flow in the conduit.

References (with Links where available)

- Air-Water Flow in Hydraulic Structures, A Water Resources Technical Publication, Engineering Monograph No. 41, United States Department of the Interior, Water and Power Resources Service, by Henry T. Falvey, Engineering and Research Center, Denver, CO, December 1980.
- *Cavitation in Chutes and Spillways*, A Water Resources Technical Publication, Engineering Monograph No. 42, United States Department of the Interior, Bureau of Reclamation, by Henry T. Falvey, Research Engineer, Denver, CO, April 1990.
- *Air Demand in Free Flowing Gated Conduits*, D. Peter Oveson, A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science, Civil and Environmental Engineering, Utah State Engineering, Logan, Utah, 2008.
- *Determining Air Demand for Small- to Medium-Sized Embankment Dam Low-Level Outlet Works*, Journal of Irrigation and Drainage Engineering, American Society of Civil Engineers, B.P. Tullis, and J. Larchar, December 2011.
- Hydraulic Design of Reservoir Outlet Works, EM 1110-2-1602, U.S. Army Corps of Engineers, October 15, 1980 ; together with HDC 050-1 and HDC-050-2.

Air vent sizing example using method from the 1980 publication *Air-Water Flow in Hydraulic Structures:*

Given:

- Conduit diameter = 2 feet
- Maximum water depth in conduit corresponding to 75% gate opening = 1.5 feet
- Volume flow rate of water (Q_w) = 50 ft^3/s

Calculate:

$$A_d = \pi \frac{D^2}{4} = \pi \frac{2^2}{4} = 3.14 \text{ ft}^2$$

A_{wp} = 2.53 ft^2 *(obtained from table typically found in hydraulic textbooks that provides numerical values for area, wetted perimeter, and hydraulic radius for a partially filled circular pipe)*

$$\left(\frac{Q_a}{Q_w}\right) = \frac{A_d}{A_{wp}} - 1 = \frac{3.14}{2.53} - 1 = 0.24$$

$$Q_a = 0.24 * Q_w = 0.24*50 \text{ ft}^3/\text{s} = 12 \text{ ft}^3/\text{s}$$

Setting maximum velocity at 100 ft/s,

$$A=Q/V = (12 \text{ ft}^3/\text{s})/(100 \text{ ft/s}) = 0.12 \text{ ft}^2 = 17.3 \text{ in}^2$$

$$D_{pipe} = \sqrt{\frac{4*A}{\pi}} = \sqrt{\frac{4*17.3}{\pi}} = 4.7 \text{ inches}$$

Increase D_{pipe} to commonly available pipe size of 6 inches.