BIE 5300/6300 Assignment #9
Open-Channel Transition Design

23 Nov 04 (due 2 Dec 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments.

Given:

- The design flow rate is 20.0 m^3/s
- The upstream trapezoidal section side slopes of m = 1.25
- The upstream trapezoidal section bed width is b = 3.40 m
- The downstream rectangular section has b = 2.75 m
- The bed slope of the upstream trapezoidal section is 0.000220 m/m
- The bed slope of the downstream rectangular flume is 0.00332 m/m
- The upstream and downstream channels are concrete-lined
- The length of the transition will be L = 10.0 m
- Uniform flow will prevail upstream & downstream of the transition

Required:

1. For uniform flow, do you expect subcritical flow both in the trapezoidal and rectangular sections?
2. Specify the rate of change of bed width with distance, using a 3rd-degree polynomial, through the transition.
3. Specify the rate of change of side slope with distance through the transition.
4. Determine the elevation of the bed (invert) of the transition versus distance along the transition so that the energy line has a constant slope through the transition, matching the upstream and downstream uniform flow depths.
5. Make sure the water surface through the transition has a constant, uniform slope.
6. What is the total bed elevation change across the transition?
7. Show your results graphically, with a side view and a plan view of the transition.

BIE 5300/6300 Assignment #10
Drop Spillway Design

2 Dec 04 (due 7 Dec 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments.

Given:

- The design flow rate for an earthen canal is 120.0 cfs.
- There is a 7.5-ft drop in canal bed elevation at a location in the canal.
- The average base width of the canal is 10 ft, and the average inverse side slope is 2:1, for a trapezoidal cross section.
- The canal longitudinal bed slope is 0.000125 ft/ft upstream of the drop location, and downstream of the drop location it is 0.000129 ft/ft.
- The average water temperature in the canal is 65°F.
- Assume a Manning roughness of 0.018 for the earthen canal.
- Assume uniform flow conditions at the design flow rate in the downstream channel.
- Forty percent of the canal flow rate is delivered to agricultural water users, and the rest goes to municipal water users.

Required:

1. Design a drop spillway for the given conditions at the location of the 7.5-ft drop.
2. Use English units for the design.
3. Use the design procedure given in the lecture notes, but iterate to make the basin area (b x L) as small as feasible at the design flow rate; however, if possible, do not make the basin width, b, greater than the average base width of the earthen canal.
4. Add 10% to the upstream normal depth for freeboard, determining the height of the headwall at the upstream sides of the stilling basin.
5. Produce side view and plan view technical drawings of the drop spillway, indicating the dimensions of the energy dissipation structure.

A Design Solution:

I. Uniform-Flow Depths

- From the ACA program, the following uniform-flow depths were found:
    - Upstream normal depth: 3.91 ft
    - Downstream normal depth: 3.88 ft

  both of which are for the design flow rate of 120 cfs.

II. Minimum Stilling Basin Area

- The stilling basin width is limited to a maximum of b = 10.0 ft in this problem.
- Start at this limit and decrease b incrementally to find the minimum area (L x b).
- Set up calculations in a spreadsheet, using the equations from the lecture notes.

b (ft)	h_c (ft)	y_{end} (ft)	h_t (ft)	$2.15h_c$ (ft)	Eq. 14	y_{drop} (ft)	x_f (ft)	x_t (ft)	x_s (ft)	x_a (ft)	x_b (ft)	x_c (ft)	L (ft)	L x b (sq ft)
10.0	1.648	0.659	4.540	3.542	OK	-8.159	7.555	5.232	8.013	7.784	1.318	2.883	11.985	119.850
9.5	1.705	0.682	4.563	3.665	OK	-8.182	7.702	5.337	8.167	7.935	1.364	2.983	12.282	116.680
9.0	1.767	0.707	4.588	3.800	OK	-8.207	7.861	5.450	8.334	8.098	1.414	3.093	12.605	113.442
8.5	1.836	0.734	4.615	3.948	OK	-8.234	8.034	5.572	8.515	8.274	1.469	3.213	12.956	110.130
8.0	1.912	0.765	4.646	4.110	OK	-8.265	8.223	5.706	8.712	8.467	1.529	3.346	13.342	106.739
7.5	1.996	0.798	4.679	4.291	OK	-8.298	8.429	5.854	8.928	8.679	1.597	3.493	13.768	103.260
7.0	2.090	0.836	4.717	4.493	OK	-8.336	8.657	6.016	9.166	8.912	1.672	3.657	14.241	99.685
6.5	2.196	0.878	4.759	4.721	OK	-8.378	8.911	6.198	9.431	9.171	1.757	3.842	14.770	96.004
6.0	2.316	0.926	4.807	4.979	invalid	-8.426	9.195	6.401	9.728	9.462	1.853	4.053	15.367	92.205
5.5	2.454	0.982	4.863	5.277	invalid	-8.482	9.518	6.633	10.064	9.791	1.963	4.295	16.049	88.272
5.0	2.615	1.046	4.927	5.623	invalid	-8.546	9.887	6.898	10.450	10.168	2.092	4.577	16.838	84.188

- It is noted that b ≤ 6.0 ft violates Eq. 14.
- In the above table, b = 6.5 ft gives the lowest basin area (96 ft^2).
- More precisely, b ≈ 6.41 ft gives the minimum value without violating Eq. 14:

b (ft)	h_c (ft)	y_{end} (ft)	h_t (ft)	$2.15h_c$ (ft)	Eq. 14	y_{drop} (ft)	x_f (ft)	x_t (ft)	x_s (ft)	x_a (ft)	x_b (ft)	x_c (ft)	L (ft)	L x b (sq ft)
6.50	2.196	0.878	4.759	4.721	OK	-8.378	8.911	6.198	9.431	9.171	1.757	3.842	14.770	96.004
6.49	2.198	0.879	4.760	4.725	OK	-8.379	8.916	6.201	9.437	9.177	1.758	3.846	14.781	95.930
6.48	2.200	0.880	4.761	4.730	OK	-8.380	8.922	6.205	9.442	9.182	1.760	3.850	14.792	95.855
6.47	2.202	0.881	4.762	4.735	OK	-8.381	8.927	6.209	9.448	9.188	1.762	3.854	14.804	95.780
6.46	2.205	0.882	4.763	4.740	OK	-8.382	8.932	6.213	9.454	9.193	1.764	3.858	14.815	95.705
6.45	2.207	0.883	4.764	4.745	OK	-8.383	8.938	6.217	9.459	9.199	1.766	3.862	14.826	95.630
6.44	2.209	0.884	4.765	4.750	OK	-8.384	8.943	6.221	9.465	9.204	1.767	3.866	14.838	95.555
6.43	2.212	0.885	4.766	4.755	OK	-8.385	8.949	6.225	9.471	9.210	1.769	3.870	14.849	95.480
6.42	2.214	0.886	4.767	4.760	OK	-8.386	8.954	6.229	9.476	9.215	1.771	3.874	14.861	95.405
6.41	2.216	0.886	4.767	4.765	OK	-8.386	8.960	6.232	9.482	9.221	1.773	3.878	14.872	95.330
6.40	2.218	0.887	4.768	4.770	invalid	-8.387	8.965	6.236	9.488	9.226	1.775	3.882	14.883	95.254

- Choose a stilling basin width of b = 6.5 ft (rounding up to the nearest half foot).
- Then,

  - h_c = 2.20 ft
  - x_a = 9.17 ft
  - x_b = 1.76 ft
  - x_c = 3.84 ft
  - L = 14.77 ft
  - h_t = 4.76 ft
  - y_{end} = 0.88 ft
  - y_{drop} = -8.38 ft

- Some of the above values could be rounded up, but in this design they will remain as calculated (other dimensions will be rounded, as shown below).

III. Headwall & Wingwalls

- Adding 10% to the upstream normal depth (as specified), the headwall height should be 1.1(3.91) = 4.30 ft above the origin, which is at the crest height.
- There will need to be a converging section at the stilling basin inlet because the stilling basin width is less than the upstream channel width. This is given a 45-degree convergence, as shown in the plan view drawing (see below).
- According to design procedures, the wingwall height at the end sill is to be $0.85h_c$ = 0.85(2.20) = 1.87 ft above the tail water surface.
- The side walls should slope linearly from the headwall to the beginning of the wingwalls over the length, L, of the stilling basin.
- Also according to design procedures, the wingwalls splay out at 45 degrees, and the tops slope downward at 45 degrees.
- Extending the wingwalls to intersect with the base of the downstream channel side slopes, the length of each wingwall will be:

$$L_{wing} = \sqrt{2\left(\frac{10.0 - 6.5}{2}\right)^2} = 2.47 \text{ ft}$$

- Round this up to 2.50 ft.
- Riprap and or other measures to help prevent erosion at and just downstream of the wingwalls will also be necessary to complete this design.

IV. Floor Block Sizing & Spacing

- This design will follow the guidelines in which the floor blocks have the same width and length, which is equal to:

$$L_{blocks} = 0.5h_c = 0.5(2.20) = 1.10 \text{ ft}$$

- Floor block height will be:

$$y_{blocks} = 0.8h_c = 0.8(2.20) = 1.76 \text{ ft}$$

which in this design will be rounded down to 1.75 ft.

- Number of floor blocks for 50% occupation of the stilling basin width:

$$N = \frac{6.5}{2(1.10)} = 2.95$$

which in this design will be rounded up to three blocks.

- Equal block spacing across the stilling basin width gives:

$$\text{block spacing} = \frac{6.50 - 3(1.10)}{4} = 0.80 \text{ ft}$$

- Finally, the proportion of the stilling basin width occupied by the three floor blocks will be:

$$\text{occupied width} = 100\left(\frac{3(1.10)}{6.50}\right) = 50.8\%$$

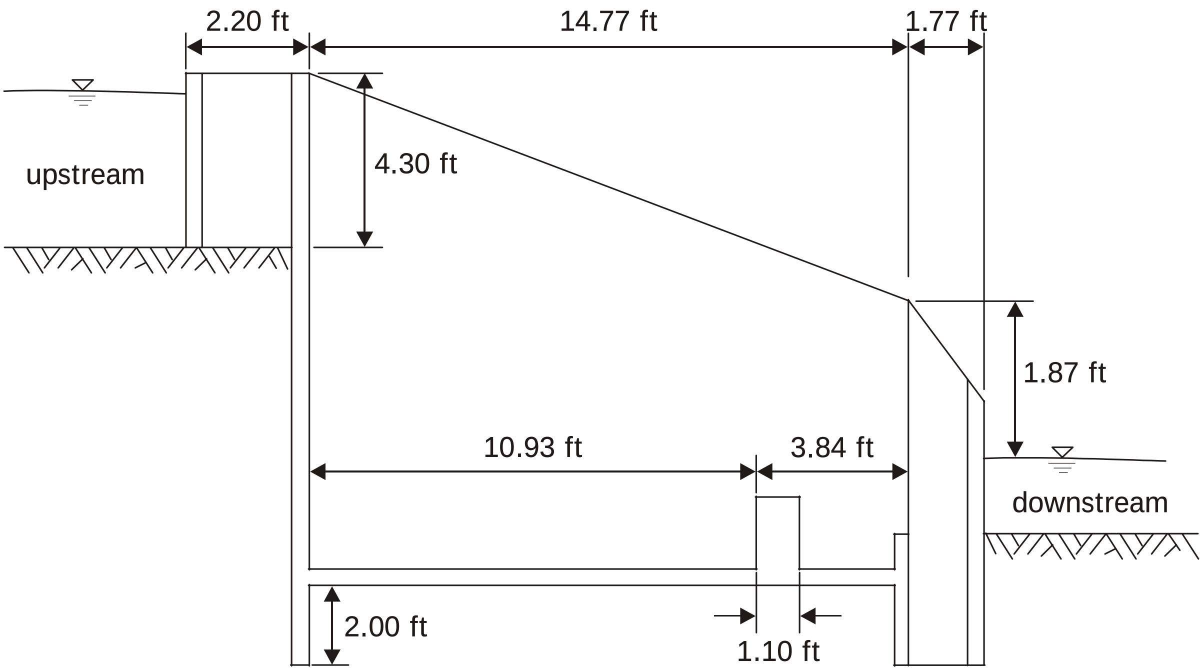
which is in the recommended range of 50 – 60%.

V. Footings & Other Details

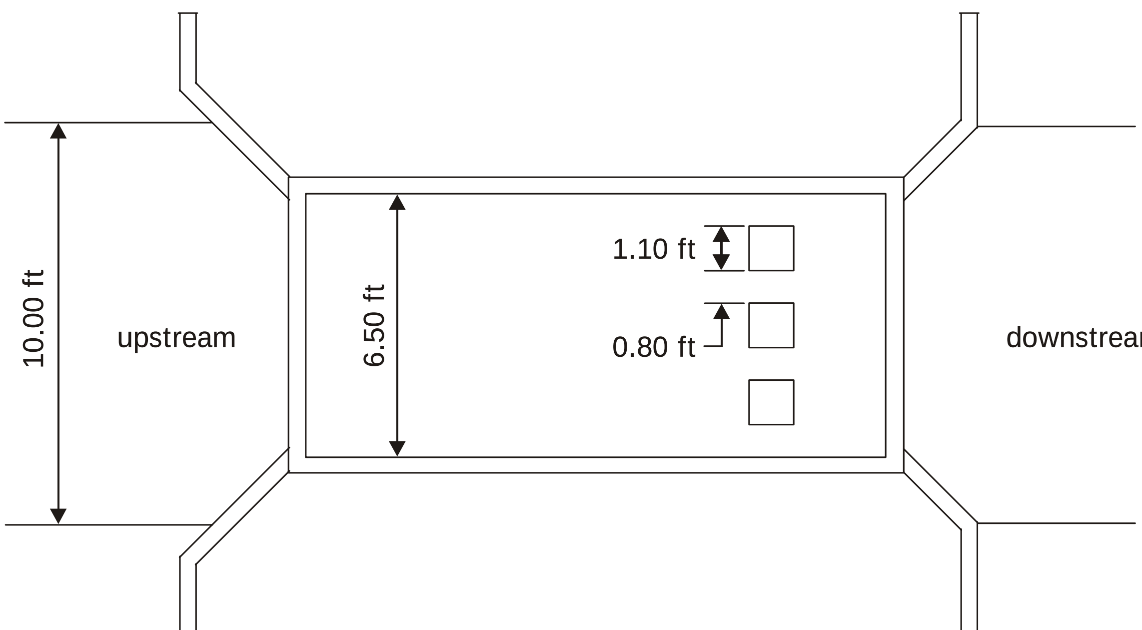
- Footing depth: 2.00 ft.
- All concrete work to be steel reinforced.
- Concrete floor and wall thicknesses: 5 inches.
- No longitudinal sills required.

VI. Drawings

- Side and plan views: see below.



Side View



Plan View

BIE 5300/6300 Assignment #11
Siphon Spillway Design

7 Dec 04 (due 10 Dec 04)

Show your calculations in an organized, neat format. Indicate any assumptions and or relevant comments.

Given:

- An overflow spillway needs to be designed for a location along a canal.
- The concrete-lined canal has a maximum flow rate of 4.0 m^3/s.
- The concrete lining depth is 2.3 m, and the depth to the top of the earthen berms is 2.5 m.
- The canal bed elevation at this location is 239 m above mean sea level (msl).
- The maximum water surface elevation in the open drainage channel on the downhill side of the canal at this location is 238.5 m.
- The canal cross section is trapezoidal in shape.
- The canal bed width is 1.65 m, and the inverse side slopes are 1.5.
- The longitudinal bed slope is 0.000284 m/m.

Required:

1. Use metric units for the design.
2. Use the Chezy equation with C = 60.
3. Design a siphon spillway for the given conditions.
4. Assume a C_d value of 0.67 when applying the orifice equation for the siphon spillway capacity.
5. Refer to section 4-14 to 4-16 of the USBR book, and to the lecture notes.
6. Determine the following:

   a. What is the depth of flow in the canal at the maximum discharge? Take this as FSL (full supply level).
   b. What siphon spillway crest elevation (referenced to msl) do you recommend?
   c. What is the estimated available head, H, across the siphon spillway?
   d. Is $H < h_{atm}$, where h_{atm} is mean atmospheric pressure head?
   e. What are the barrel dimensions (b and D), given a rectangular barrel section (according to USBR guidelines, $D \geq 2.0$ ft)?
   f. What are R_{CL}, R_C and R_S?
   g. What is the estimated full-pipe unit discharge, q, in the siphon spillway?
   h. What is the estimated maximum unit discharge through the siphon spillway ("vortex" equation)? Make sure this is more than the discharge calculated from the "orifice equation," otherwise the design is not acceptable.
   i. What is the minimum required vent (siphon breaker) pipe inside diameter?
   j. What is the required height of the outlet deflector sill (1.5D)?

k. What is the required height of the outlet ceiling, h_2?
l. What is the hydraulic seal in the canal, above the top of the opening to the siphon spillway, on the downhill canal back at Q_{max}? Is it greater than the minimum values of $1.5h_v$ +0.5 ft, or 1.0 ft (whichever is greater)?
m. Create a side-view drawing of your design, with Fig. 4-17 as a model.

Solution:

a) *Normal depth in the canal*

By iteration, the normal depth is found to be $\mathbf{h_n = 1.289}$ m at 4.0 m^3/s, with C = 60. This is taken to be the full supply level (FSL).

b) *Elevation of siphon spillway crest*

By USBR design guidelines, the crest elevation is 0.2 ft (0.061 m) above FSL. Then, the crest elevation is: 239.000 + 1.289 + 0.061 = **240.350 m** above msl.

c) *Available head*

The available head, H, is measured from the crest elevation to the downstream water surface elevation (see Fig. 4-17). Taking the maximum downstream water surface elevation: H = 240.350 – 238.500 = **1.850 m**.

d) *Is $H < h_{atm}$?*

The mean atmospheric pressure head at an elevation of 239 m is estimated as:

$$h_{atm} = 10.3 - 0.00105(239.00) = 10.05 \text{ m}$$

Then, $H < h_{atm}$, which is as required for siphonic operation.

e) *Barrel dimensions*

Try the minimum recommended barrel height, **D = 2.0 ft (0.610 m)**. The width, b, will be determined below.

f) *R_{CL}, R_C and R_S*

Use the recommended ratio $R_{CL}/D = 2.0$. Then, $R_{CL} = 2.0(0.610) = 1.22$ m. $R_C = R_{CL} - ½D = 0.915$ m. And, $R_S = R_{CL} + ½D = 1.525$ m.

g) *Full pipe unit discharge*

Use the orifice equation with $C_d = 0.67$:

$$q = C_d D\sqrt{2gH} = 0.67(0.610)\sqrt{2(9.81)(1.850)} = 2.46\ \text{m}^2/\text{s}$$

h) Maximum unit discharge

This is based on the "vortex" equation:

$$\begin{aligned} q_{max} &= R_C\sqrt{2g(0.7h)}\ln\left(\frac{R_S}{R_C}\right) \\ &= 0.915\sqrt{2(9.81)(0.7)(10.05)}\ln\left(\frac{1.525}{0.915}\right) \\ &= 5.49\ \text{m}^2/\text{s} \end{aligned}$$

Thus, $q_{max} > q$, as required. Then, the required barrel width is:

$$b = \frac{Q}{q} = \frac{4.00}{2.46} = 1.63\ \text{m}$$

Finally, the area of the barrel is: A = bD = (1.63)(0.61) = 0.992 m^2.

i) Siphon breaker diameter

This is taken as 1/24th of the barrel area, or 0.992/24 = 0.0413 m^2. For a circular pipe cross section, this give an inside **diameter of 0.229 m**. Round up to the nearest available steel pipe size.

j) Outlet deflector sill height

This height is 1.5D = 1.5(0.61) = **0.915 m**.

k) Outlet ceiling height

This is equal to:

$$h_2 = 1.5D + E_{critical} + 0.3048$$

Critical depth for the design discharge (4.0 m^3/s) is:

$$h_c = \sqrt[3]{\frac{(Q/b)^2}{g}} = \sqrt[3]{\frac{(4.00/1.63)^2}{9.81}} = 0.850\ \text{m}$$

The velocity at critical depth would be:

$$V_c = \frac{Q}{bh_c} = \frac{4.00}{(1.63)(0.850)} = 2.89 \text{ m/s}$$

And, the velocity head at critical flow is:

$$\frac{V_c^2}{2g} = \frac{(2.89)^2}{2(9.81)} = 0.425 \text{ m}$$

Finally,

$$h_2 = 1.5(0.61) + 0.425 + 0.850 + 0.3048 = 2.49 \text{ m}$$

l) Hydraulic seal at inlet

Using the minimum dimensions, the inlet along the side slope of the canal is b x 2D, or 1.63 wide by 1.22 high (in meters). This gives a total inlet area of (1.63)(1.22) = 1.99 m^2. Then, the maximum inlet velocity head is:

$$h_v = \frac{Q^2}{2gA^2} = \frac{(4.00)^2}{2(9.81)(1.99)^2} = 0.206 \text{ m}$$

Minimum hydraulic seal is defined as:

$$1.5h_v + 0.152 = 1.5(0.206) + 0.152 = 0.461 \text{ m}$$

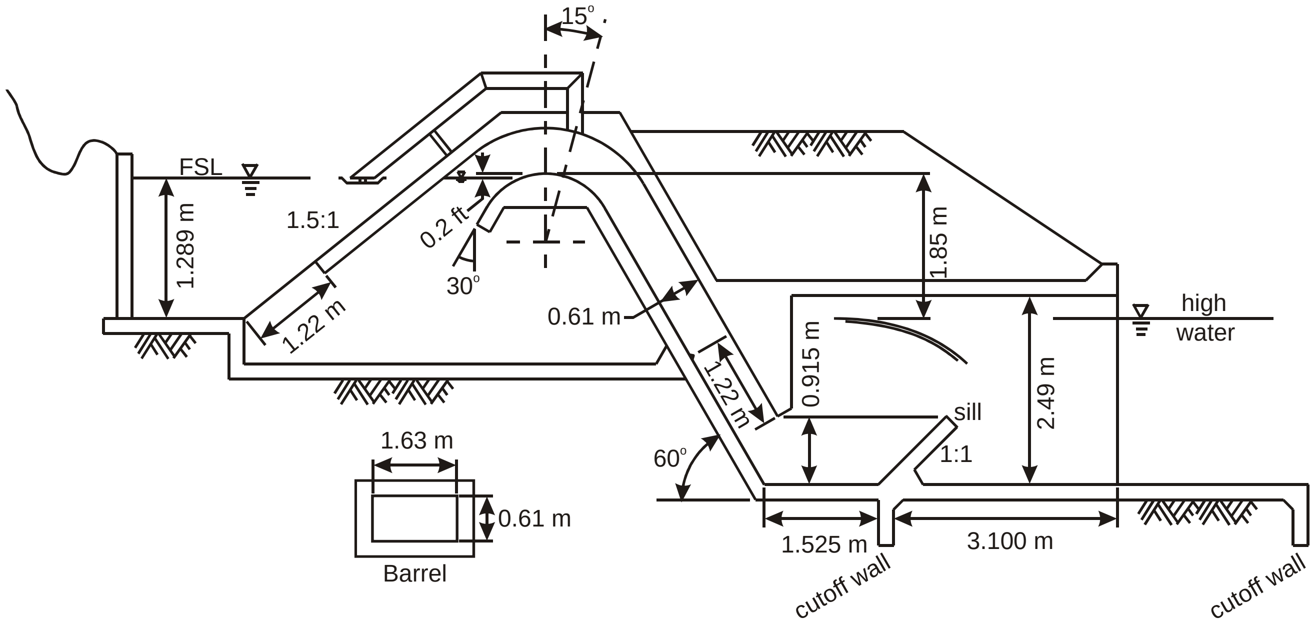
If the inlet begins at the base of the canal side slope (see Fig. 4-17), this hydraulic seal corresponds to a vertical depth of:

$$0.461 \sin\left[\tan^{-1}\left(1/1.5\right)\right] = 0.256 \text{ m}$$

The depth of water above the inlet is, then: 1.289 – 0.256 = 1.033 m, which exceeds the hydraulic seal requirement of 0.461 m. Thus, the hydraulic seal is sufficient.

m) Side-view drawing

See the drawing below.



15°
FSL
1.289 m
1.5:1
0.2 ft
30°
1.22 m
0.61 m
1.85 m
high
water
0.915 m
2.49 m
1.22 m
sill
1:1
60°
1.63 m
0.61 m
Barrel
1.525 m
3.100 m
cutoff wall
cutoff wall

Lecture 2

Flumes for Open-Channel Flow Measurement

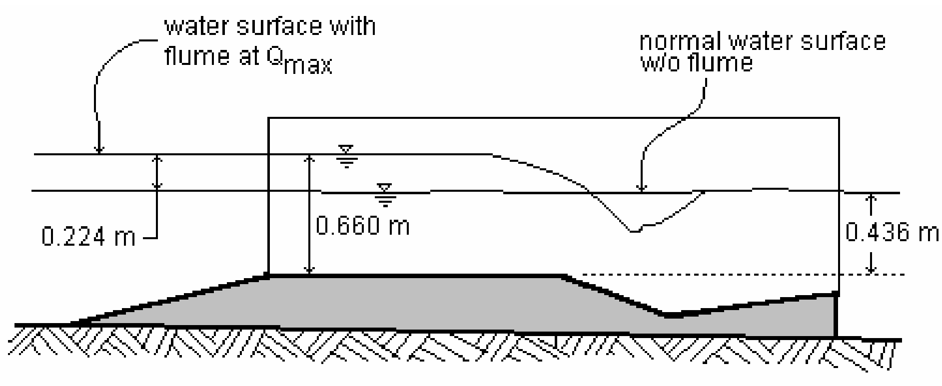
"Superb accuracy in water measurement, Jessica thought."
Dune, F. Herbert (1965)

I. Procedure for Installing a Parshall Flume to Ensure Free Flow

- If possible, you will want to specify the installation of a Parshall flume such that it operates under free-flow conditions throughout the required flow range
- To do this, you need to specify the minimum elevation of the upstream floor of the flume
- Follow these simple steps to obtain a free-flow in a Parshall flume, up to a specified maximum discharge:

1. Determine the maximum flow rate (discharge) to be measured
2. Locate the high water line on the canal bank where the flume is to be installed, or otherwise determine the maximum depth of flow on the upstream side
3. Select a standard flume size and calculate h_u from the free-flow equation corresponding to the maximum discharge capacity of the canal
4. Place the floor of the flume at a depth not exceeding the transition submergence, S_t, multiplied by h_u below the high water line

- In general, the floor of the flume should be placed as high in the canal as grade and other conditions permit, but not so high that upstream free board is compromised.
- The downstream water surface elevation will be unaffected by the installation of the flume (at least for the same flow rate)
- As an example, a 0.61-m Parshall flume is shown in the figure below
- The transition submergence, S_t, for the 0.61-m flume is 66% (see table)
- The maximum discharge in the canal is given as 0.75 m^3/s, which for free-flow conditions must have an upstream depth of (see Eq. 3): $h_u = (0.75/1.429)^{1/1.55} = 0.66$ m
- With the transition submergence of 0.66, this gives a depth to the flume floor of 0.66(0.660 m) = 0.436 m from the downstream water surface
- Therefore, the flume crest (elevation of h_u tap) should be set no lower than 0.436 m below the normal maximum water surface for this design flow rate, otherwise the regime will be submerged flow
- However, if the normal depth for this flow rate were less than 0.436 m, you would place the floor of the flume on the bottom of the channel and still have free flow conditions
- The approximate head loss across the structure at the maximum flow rate will be the difference between 0.660 and 0.436 m, or 0.224 m
- This same procedure can be applied to other types of open-channel measurement flumes



II. Non-Standard Parshall Flume Calibrations

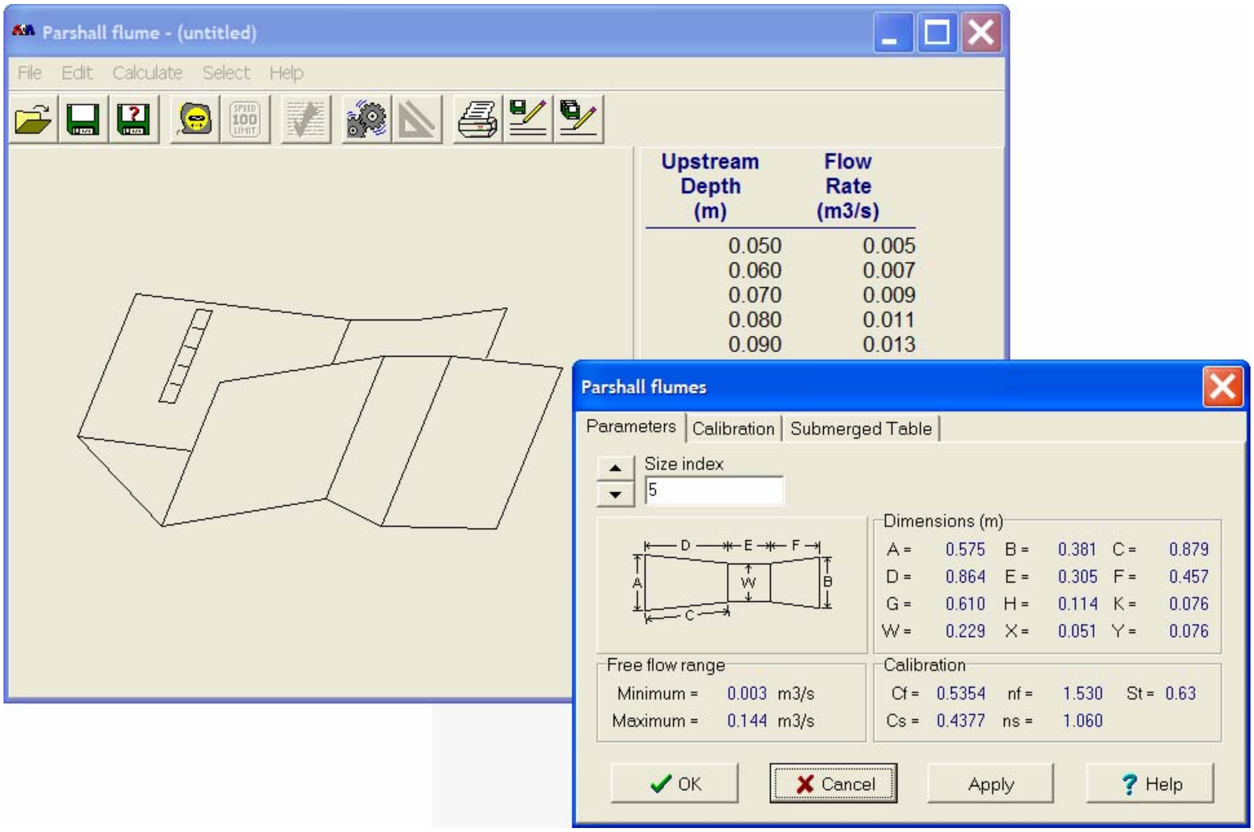
- Some Parshall flumes were incorrectly constructed or were intentionally built with a non-standard size
- Others have settled over time such that the flume is out of level either cross-wise or longitudinally (in the direction of flow), or both
- Some flumes have the taps for measuring h_u and h_d at the wrong locations (too high or too low, or too far upstream or downstream)
- Some flumes have moss, weeds, sediment or other debris that cause the calibration to shift from that given for standard conditions
- Several researchers have worked independently to develop calibration adjustments for many of the unfortunate anomalies that have befallen many Parshall flumes in the field, but a general calibration for non-standard flumes requires 3-D modeling
- There are calibration corrections for out-of-level installations and for low-flow conditions

III. Hysteresis Effects in Parshall Flumes

- There have been reports by some researchers that hysteresis effects have been observed in the laboratory under submerged-flow conditions in Parshall flumes
- The effect is to have two different flow rates for the same submergence, S, value, depending on whether the downstream depth is rising or falling
- There is no evidence of this hysteresis effect in Cutthroat flumes, which are discussed below

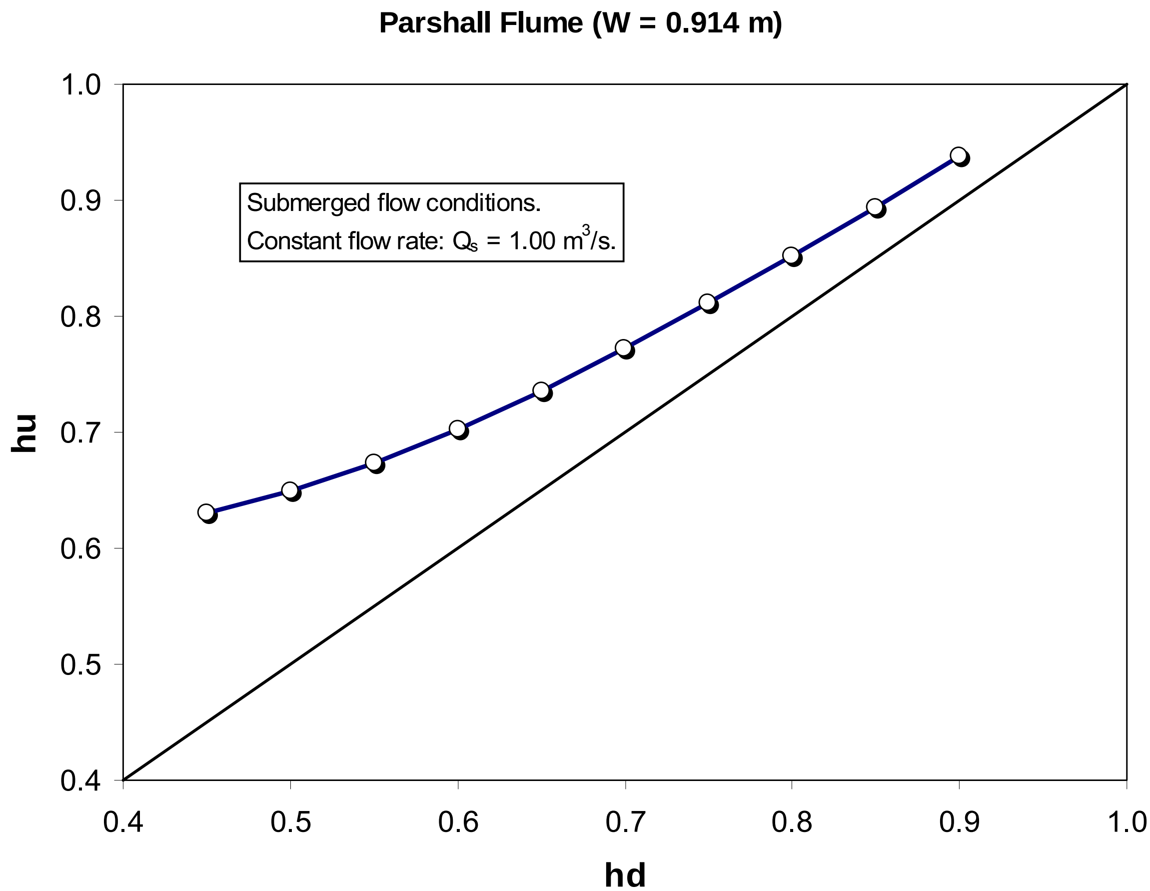
IV. Software

- You can use the ***ACA*** program to develop calibration tables for Parshall, Cutthroat, and trapezoidal flumes
- Download ***ACA*** from: http://www.engineering.usu.edu/bie/faculty/merkley/Software.htm
- You can also download the ***WinFlume*** program from: http://www.usbr.gov/pmts/hydraulics_lab/winflume/index.html



V. Submerged-Flow, Constant Flow Rate

- Suppose you have a constant flow rate through a Parshall flume
- How will h_u change for different h_d values under submerged-flow conditions?
- This situation could occur in a laboratory flume, or in the field where a downstream gate is incrementally closed, raising the depth downstream of the Parshall flume, but with a constant upstream inflow
- The graph below is for steady-state flow conditions with a 0.914-m Parshall flume
- Note that h_u is always greater than h_d (otherwise the flow would move upstream, or there would be no flow)



hd	hu	Q	S	Regime
(m)	(m)	(m3/s)		
0.15	0.714	0.999	0.210	free
0.20	0.664	0.999	0.301	free
0.25	0.634	0.999	0.394	free
0.30	0.619	1.000	0.485	free
0.35	0.615	1.002	0.569	free
0.40	0.619	1.000	0.646	free
0.45	0.631	1.000	0.713	submerged
0.50	0.650	1.001	0.769	submerged
0.55	0.674	1.000	0.816	submerged
0.60	0.703	1.000	0.853	submerged
0.65	0.736	1.000	0.883	submerged
0.70	0.772	0.999	0.907	submerged
0.75	0.811	1.001	0.925	submerged
0.80	0.852	1.004	0.939	submerged
0.85	0.894	1.000	0.951	submerged
0.90	0.938	1.002	0.959	submerged

VI. Cutthroat Flumes

- The Cutthroat flume was developed at USU from 1966-1990
- A Cutthroat flume is a rectangular open-channel constriction with a flat bottom and zero length in the throat section (earlier versions did have a throat section)
- Because the flume has a throat section of zero length, the flume was given the name “Cutthroat” by the developers (Skogerboe, et al. 1967)
- The floor of the flume is level (as opposed to a Parshall flume), which has the following advantages:

    1. ease of construction – the flume can be readily placed inside a concrete-lined channel
    2. the flume can be placed on the channel bed

- The Cutthroat flume was developed to operate satisfactorily under both free-flow and submerged-flow conditions
- Unlike Parshall flumes, all Cutthroat flumes have the same dimensional ratios
- It has been shown by experiment that downstream flow depths measured in the diverging outlet section give more accurate submerged-flow calibration curves than those measured in the throat section of a Parshall flume
- The centers of the taps for the US and DS head measurements are both located ½-inch above the floor of the flume, and the tap diameters should be ¼-inch

<u>Cutthroat Flume Sizes</u>

- The dimensions of a Cutthroat flume are identified by the flume width and length (W x L, e.g. 4” x 3.0’)
- The flume lengths of 1.5, 3.0, 4.5, 6.0, 7.5, 9.0 ft are sufficient for most applications
- The most common ratios of W/L are 1/9, 2/9, 3/9, and 4/9
- The recommended ratio of h_u/L is equal to or less than 0.33

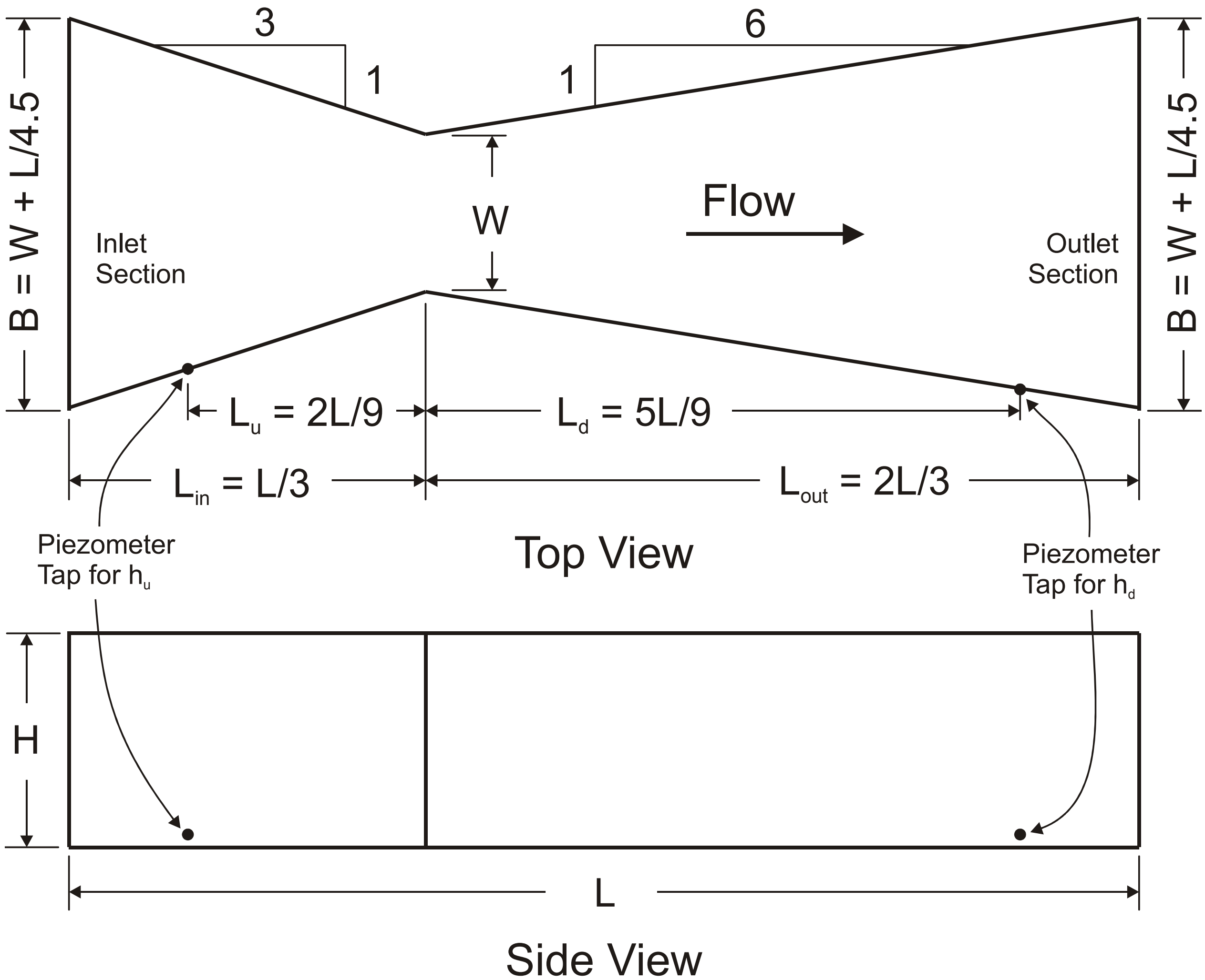
<u>Free-flow equation</u>

- For Cutthroat flumes the free-flow equation takes the same general form as for Parshall flumes, and other channel “constrictions”:

$$Q_f = C_f W \left(h_u\right)^{n_f} \tag{1}$$

where Q_f is the free-flow discharge; W is the throat width; C_f is the free-flow coefficient; and n_f is the free-flow exponent

- That is, *almost any* non-orifice constriction in an open channel can be calibrated using Eq. 1, given free-flow conditions
- The depth, h_u, is measured from the upstream tap location (½-inch above the flume floor)



- For any given flume size, the flume wall height, H, is equal to h_u for Q_{max}, according to the above equation, although a slightly larger H-value can be used to prevent the occurrence of overflow
- So, solve the above free-flow equation for h_u, and apply the appropriate Q_{max} value from the table below; the minimum H-value is equal to the calculated h_u

Submerged-flow equation

- For Cutthroat flumes the submerged-flow equation also takes the same general form as for Parshall flumes, and other channel constrictions:

$$Q_s = \frac{C_s W (h_u - h_d)^{n_f}}{[-(\log_{10} S)]^{n_s}} \tag{2}$$

where C_s = submerged-flow coefficient; W is the throat width; and $S = h_d/h_u$

- Equation 2 differs from the submerged-flow equation given previously for Parshall flumes in that the C_2 term is omitted
- The coefficients C_f and C_s are functions of flume length, L, and throat width, W
- The generalized free-flow and submerged-flow coefficients and exponents for standard-sized Cutthroat flumes can be taken from the following table (metric units: for Q in m^3/s and head (depth) in m, and using a *base 10 logarithm* in Eq. 2)
- *Almost any* non-orifice constriction in an open channel can be calibrated using Eq. 2, given submerged-flow conditions

Cutthroat Flume Calibration Parameters for metric units (depth and W in m and flow rate in m^3/s)

W (m)	L (m)	C_f	n_f	S_t	C_s	n_s	Discharge (m^3/s)	
							min	max
0.051	0.457	5.673	1.98	0.553	3.894	1.45	0.0001	0.007
0.102	0.457	5.675	1.97	0.651	3.191	1.58	0.0002	0.014
0.152	0.457	5.639	1.95	0.734	2.634	1.67	0.0004	0.022
0.203	0.457	5.579	1.94	0.798	2.241	1.73	0.0005	0.030
0.102	0.914	3.483	1.84	0.580	2.337	1.38	0.0002	0.040
0.203	0.914	3.486	1.83	0.674	1.952	1.49	0.0005	0.081
0.305	0.914	3.459	1.81	0.754	1.636	1.57	0.0008	0.123
0.406	0.914	3.427	1.80	0.815	1.411	1.64	0.0011	0.165
0.152	1.372	2.702	1.72	0.614	1.752	1.34	0.0005	0.107
0.305	1.372	2.704	1.71	0.708	1.469	1.49	0.0010	0.217
0.457	1.372	2.684	1.69	0.788	1.238	1.50	0.0015	0.326
0.610	1.372	2.658	1.68	0.849	1.070	1.54	0.0021	0.436
0.203	1.829	2.351	1.66	0.629	1.506	1.30	0.0007	0.210
0.406	1.829	2.353	1.64	0.723	1.269	1.39	0.0014	0.424
0.610	1.829	2.335	1.63	0.801	1.077	1.45	0.0023	0.636
0.813	1.829	2.315	1.61	0.862	0.934	1.50	0.0031	0.846
0.254	2.286	2.147	1.61	0.641	1.363	1.28	0.0009	0.352
0.508	2.286	2.148	1.60	0.735	1.152	1.37	0.0019	0.707
0.762	2.286	2.131	1.58	0.811	0.982	1.42	0.0031	1.056
1.016	2.286	2.111	1.57	0.873	0.850	1.47	0.0043	1.400
0.305	2.743	2.030	1.58	0.651	1.279	1.27	0.0012	0.537
0.610	2.743	2.031	1.57	0.743	1.085	1.35	0.0025	1.076
0.914	2.743	2.024	1.55	0.820	0.929	1.40	0.0039	1.611
1.219	2.743	2.000	1.54	0.882	0.804	1.44	0.0055	2.124

- Note that n_f approaches 1.5 for larger W values, but never gets down to 1.5
- As for the Parshall flume data given previously, the submerged-flow calibration is for base 10 logarithms
- Note that the coefficient conversion to English units is as follows:

$$C_{f\,(English)} = \frac{(0.3048)^{1+n_f}}{(0.3048)^3} C_{f\,(metric)} \tag{3}$$

- The next table shows the calibration parameters for English units

Cutthroat Flume Calibration Parameters for English units (depth and W in ft and flow rate in cfs)

W (ft)	L (ft)	C_f	n_f	S_t	C_s	n_s	Discharge (cfs)	
							min	max
0.167	1.50	5.796	1.98	0.553	3.978	1.45	0.004	0.24
0.333	1.50	5.895	1.97	0.651	3.315	1.58	0.008	0.50
0.500	1.50	5.956	1.95	0.734	2.782	1.67	0.013	0.77
0.667	1.50	5.999	1.94	0.798	2.409	1.73	0.018	1.04
0.333	3.00	4.212	1.84	0.580	2.826	1.38	0.009	1.40
0.667	3.00	4.287	1.83	0.674	2.400	1.49	0.018	2.86
1.000	3.00	4.330	1.81	0.754	2.048	1.57	0.029	4.33
1.333	3.00	4.361	1.80	0.815	1.796	1.64	0.040	5.82
0.500	4.50	3.764	1.72	0.614	2.440	1.34	0.016	3.78
1.000	4.50	3.830	1.71	0.708	2.081	1.49	0.034	7.65
1.500	4.50	3.869	1.69	0.788	1.785	1.50	0.053	11.5
2.000	4.50	3.897	1.68	0.849	1.569	1.54	0.074	15.4
0.667	6.00	3.534	1.66	0.629	2.264	1.30	0.024	7.43
1.333	6.00	3.596	1.64	0.723	1.940	1.39	0.050	15.0
2.000	6.00	3.633	1.63	0.801	1.676	1.45	0.080	22.5
2.667	6.00	3.662	1.61	0.862	1.478	1.50	0.111	29.9
0.833	7.50	3.400	1.61	0.641	2.159	1.28	0.032	12.4
1.667	7.50	3.459	1.60	0.735	1.855	1.37	0.068	25.0
2.500	7.50	3.494	1.58	0.811	1.610	1.42	0.108	37.3
3.333	7.50	3.519	1.57	0.873	1.417	1.47	0.151	49.4
1.000	9.00	3.340	1.58	0.651	2.104	1.27	0.042	19.0
2.000	9.00	3.398	1.57	0.743	1.815	1.35	0.088	38.0
3.000	9.00	3.442	1.55	0.820	1.580	1.40	0.139	56.9
4.000	9.00	3.458	1.54	0.882	1.390	1.44	0.194	75.0

Unified Discharge Calibrations

- Skogerboe also developed “unified discharge” calibrations for Cutthroat flumes, such that it is *not necessary to select from the above standard flume sizes*
- A regression analysis on the graphical results from Skogerboe yields these five calibration parameter equations:

$$C_f = 6.5851 L^{-0.3310} W^{1.025} \tag{4}$$

$$n_f = 2.0936L^{-0.1225} - 0.128(W/L) \quad (5)$$

$$n_s = 2.003(W/L)^{0.1318}L^{-0.07044(W/L)-0.07131} \quad (6)$$

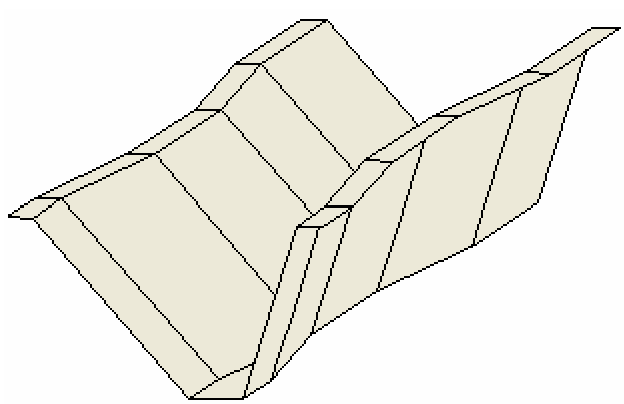
$$S_t = 0.9653(W/L)^{0.2760}L^{0.04322(W/L)^{-0.3555}} \quad (7)$$

$$C_s = \frac{C_f\left(-\log_{10} S_t\right)^{n_s}}{\left(1-S_t\right)^{n_f}} \quad (8)$$

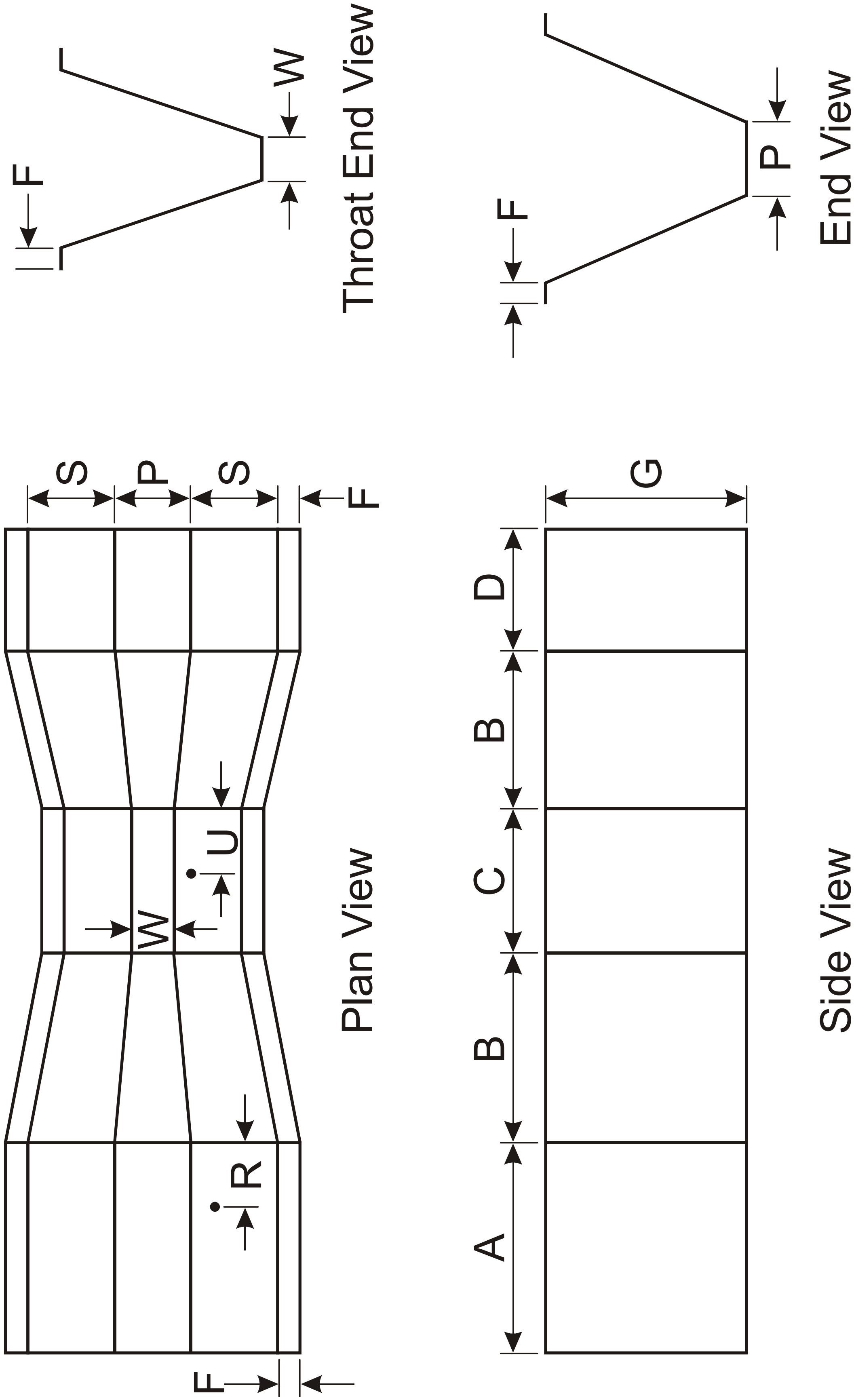
- Note that Eqs. 4-8 are for English units (L and W in ft; Q in cfs)
- The maximum percent difference in the Cutthroat flume calibration parameters is less than 2%, comparing the results of Eqs. 4-8 with the calibration parameters for the 24 standard Cutthroat flume sizes

VII. Trapezoidal Flumes

- Trapezoidal flumes are often used for small flows, such as for individual furrows in surface irrigation evaluations
- The typical standard calibrated flume is composed of five sections: approach, converging, throat, diverging, and exit
- However, the approach and exit sections are not necessary part of the flume itself



- Ideally, trapezoidal flumes can measure discharge with an accuracy of ±5% under free-flow conditions
- But the attainment of this level of accuracy depends on proper installation, accurate stage measurement, and adherence to specified tolerances in the construction of the throat section
- Discharge measurement errors are approximately 1.5 to 2.5 times the error in the stage reading for correctly installed flumes with variations in throat geometry from rectangular to triangular sections


Throat End View
End View
Plan View
Side View
F
W
P
S
G
A
B
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D
U
R

- In the following table with seven trapezoidal flume sizes, the first two flumes are V-notch (zero base width in the throat, and the last five have trapezoidal throat cross sections

Flume Number	Description	Dimensions (inches)											
		A	B	C	D	E	F	G	P	R	S	U	W
1	Large 60°-V	7.00	6.90	7.00	3.00	7.00	1.00	6.75	2.00	1.50	4.00	3.50	0.00
2	Small 60°-V	5.00	6.05	5.00	2.00	4.25	1.00	4.00	2.00	1.00	2.38	2.50	0.00
3	2”-60° WSC	8.00	6.41	8.50	3.00	8.50	1.00	13.50	4.90	1.50	6.00	4.30	2.00
4	2”-45° WSC	8.00	8.38	8.50	3.00	8.50	1.00	10.60	4.90	1.50	10.60	4.30	2.00
5	2”-30° WSC	8.00	8.38	8.50	3.00	8.50	1.00	10.00	4.90	1.50	17.30	4.30	2.00
6	4”-60° WSC	9.00	9.81	10.00	3.00	10.00	1.00	13.90	8.00	1.50	8.00	5.00	4.00
7	2”-30° CSU	10.00	10.00	10.00	3.00	10.80	1.00	9.70	10.00	1.50	16.80	5.00	2.00

Note: All dimensions are in inches. WSC are Washington State Univ Calibrations, while CSU are Colorado State Univ Calibrations (adapted from Robinson & Chamberlain 1960)

- Trapezoidal flume calibrations are for free-flow regimes only (although it would be possible to generate submerged-flow calibrations from laboratory data)
- The following equation is used for free-flow calibration

$$Q_f = C_{ft}\left(h_u\right)^{n_{ft}} \tag{9}$$

where the calibration parameters for the above seven flume sizes are given in the table below:

Flume Number	C_{ft}	n_{ft}	Q_{max} (cfs)
1	1.55	2.58	0.35
2	1.55	2.58	0.09
3	1.99	2.04	2.53
4	3.32	2.18	2.53
5	5.92	2.28	3.91
6	2.63	1.83	3.44
7	4.80	2.26	2.97

Note: for h_u in ft and Q in cfs

<u>V-Notch Flumes</u>

- When the throat base width of a trapezoidal flume is zero (W = 0, usually for the smaller sizes), these are called “V-notch flumes”
- Similar to the V-notch weir, it is most commonly used for measuring water with a small head due to a more rapid change of head with change in discharge
- Flume numbers 1 and 2 above are V-notch flumes because they have W = 0

VIII. Flume Calibration Procedure

- Sometimes it is necessary to develop site-specific calibrations in the field or in the laboratory
- For example, you might need to develop a custom calibration for a “hybrid” flume, or a flume that was constructed to nonstandard dimensions
- To calibrate based on field data for flow measurement, it is desired to find flow rating conditions for both free-flow and submerged-flow
- To analyze and solve for the value of the unknown parameters in the flow rating equation the following procedure applies:

  1. Transform the exponential equation into a linear equation using logarithms
  2. The slope and intersection of this line can be obtained by fitting the transformed data using linear regression, or graphically with log-log paper
  3. Finally, back-calculate to solve for the required unknown values

The linear equation is:

$$Y = a + bX \tag{10}$$

The transformed flume equations are:

<u>Free-flow:</u>

$$\log(Q_f) = \log\left(C_f W\right) + n_f \log(h_u) \tag{11}$$

So, applying Eq. 10 with measured pairs of Q_f and h_u, “a” is log C_f and “b” is n_f

<u>Submerged-flow:</u>

$$\log\left[\frac{Q_s}{(h_u - h_d)^{n_f}}\right] = \log(C_s W) - n_s \log\left[-(\log S)\right] \tag{12}$$

Again, applying Eq. 10 with measured pairs of Q_s and h_u and h_d, “a” is log C_s and “b” is n_s

- Straight lines can be plotted to show the relationship between log h_u and log Q_f for a free-flow rating, and between log (h_u-h_d) and log Q_S with several degrees of submergence for a submerged-flow rating
- If this is done using field or laboratory data, any base logarithm can be used, but the base must be specified
- Multiple linear regression can also be used to determine C_s, n_f, and n_s for submerged flow data only – this is discussed further in a later lecture

IX. Sample Flume Calibrations

Free Flow

- Laboratory data for free-flow conditions in a flume are shown in the following table
- Free-flow conditions were determined for these data because a hydraulic jump was seen downstream of the throat section, indicating supercritical flow in the vicinity of the throat

Q (cfs)	hu (ft)
4.746	1.087
3.978	0.985
3.978	0.985
2.737	0.799
2.737	0.798
2.211	0.707
1.434	0.533
1.019	0.436
1.019	0.436
1.019	0.436
1.019	0.436
0.678	0.337

- Take the logarithm of Q and of h_u, then perform a linear regression (see Eqs. 10 and 11)
- The linear regression gives an R^2 value of 0.999 for the following calibration equation:

$$Q_f = 4.04 h_u^{1.66} \tag{13}$$

where Q_f is in cfs; and h_u is in ft

- We could modify Eq. 13 to fit the form of Eq. 6, but for a custom flume calibration it is convenient to just include the throat width, W, in the coefficient, as shown in Eq. 13
- Note that the coefficient and exponent values in Eq. 13 have been rounded to three significant digits each – never show more precision than you can justify

Submerged Flow

- Data were then collected under submerged-flow conditions in the same flume
- The existence of submerged flow in the flume was verified by noting that there is not downstream hydraulic jump, and that any slight change in downstream depth produces a change in the upstream depth, for a constant flow rate
- Note that a constant flow rate for varying depths can usually only be obtained in a hydraulics laboratory, or in the field where there is an upstream pump, with an unsubmerged outlet, delivering water to the channel

- Groups of (essentially) constant flow rate data were taken, varying a downstream gate to change the submergence values, as shown in the table below

Q (cfs)	hu (ft)	hd (ft)
3.978	0.988	0.639
3.978	1.003	0.753
3.978	1.012	0.785
3.978	1.017	0.825
3.978	1.024	0.852
3.978	1.035	0.872
3.978	1.043	0.898
3.978	1.055	0.933
3.978	1.066	0.952
3.978	1.080	0.975
3.978	1.100	1.002
3.978	1.124	1.045
2.737	0.800	0.560
2.736	0.801	0.581
2.734	0.805	0.623
2.734	0.812	0.659
2.733	0.803	0.609
2.733	0.808	0.642
2.733	0.818	0.683
2.733	0.827	0.714
2.733	0.840	0.743
2.733	0.858	0.785
2.733	0.880	0.823
2.733	0.916	0.876
2.733	0.972	0.943
1.019	0.437	0.388
1.019	0.441	0.403
1.010	0.445	0.418
1.008	0.461	0.434
1.006	0.483	0.462
1.006	0.520	0.506

- In this case, we will use n_f in the submerged-flow equation (see Eq. 12), where n_f = 1.66, as determined above
- Perform a linear regression for $\ln[Q/(h_u - h_d)^{1.66}]$ and $\ln[-\log_{10}S]$, as shown in Eq. 12, giving an R^2 of 0.998 for

$$Q_s = \frac{1.93\left(h_u - h_d\right)^{1.66}}{\left(-\log_{10} S\right)^{1.45}} \tag{14}$$

where Q_s is in cfs; and h_u and h_d are in ft

- You should verify the above results in a spreadsheet application

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UtahState
UNIVERSITY

BIE 5300/6300
Fall Semester 2004

Irrigation Conveyance & Control:
Flow Measurement & Structure Design

Lecture Notes

Biological & Irrigation Engineering Department
Utah State University, Logan, Utah

Preface

These lecture notes were prepared by Gary P. Merkley of the Biological & Irrigation Engineering Department at USU for use in the BIE 5300/6300 courses. The material contained in these lecture notes is the intellectual property right of G.P. Merkley, except where otherwise stated.

Many thanks are extended to USU engineering students, past and present, whose numerous suggestions and corrections have been incorporated into these lecture notes.

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Contents

These lecture notes are formatting for printing on both sides of the page, with odd-numbered pages on the front. Each lecture begins on an odd-numbered page, so some even-numbered pages are blank.

Units, Constants and Conversions

28.35 g/oz
15.85 gpm/lps (= 60/3.785)
7.481 gallons/ft^3
448.86 gpm/cfs (= 7.481*60)
3.7854 litres/gallon

6.89 kPa/psi
1 cb = 1 kPa
10 mb/kPa, or 100 kPa/bar
2.308 ft/psi, or 9.81 kPa/m (head of water)
14.7 psi = 101.3 kPa = 10.34 m (head of water) = 1,013 mbar = 1 atm
62.4 lbs/ft^3, or 1000 kg/m^3 (max density of pure water at 4°C)
0.1333 kPa/mmHg

1 ppm ≈ 1 mg/liter (usually)
1 mmho/cm = 1 dS/m = 550 to 800 mg/liter

0.7457 kW/HP
1 langley = 1 cal/cm^2
0.0419 MJ/m^2 per cal/cm^2

0.3048 m/ft
1.609 km/mile
2.471 acre/ha
43,560 ft^2/acre
1,233 m^3/acre-ft

57.2958 degrees/radian
$\pi \approx 3.14159265358979323846$
$e \approx 2.71828182845904523536$

ºC = (ºF – 32)/1.8
ºF = 1.8(ºC) + 32

Ratio of weight to mass at sea level and 45° latitude: $g = 9.80665\ m/s^2$

Lecture 1

Course Introduction

"What is never measured is never managed"
Rep. Stephen Urquhart (2003)

I. Textbook and Other Materials

- The main references are *Design of Small Canal Structures*, USBR; and *Water Measurement Manual*, USBR. At least one copy of each will be on reserve in the library.
- Some material will also be referred to from *Irrigation Fundamentals*, by Hargreaves & Merkley, as well as from other books and sources
- BIE 5300/6300 lecture notes by G.P. Merkley are required

II. Homework

- All work must be organized and neat
- There will be some computer programming and or spreadsheet exercises
- Submitting work late:10% reduction per day, starting after class

III. Tests

- One mid-term exam
- Final exam is not comprehensive

IV. Subject Areas

- Flow measurement
    - Open channels
    - Full pipe flow

- Design of conveyance infrastructure
    - Canals, flumes, chutes
    - Canal linings
    - Siphons
    - Culverts
    - Energy dissipation structures

V. Why Measure Water?

Flow measurement is a key element in:

1. ***Water Management***. Without knowing flow rates it is usually difficult to quantify deliveries to water users, and in this case the evaluation of water management practices is only vague
2. ***Water Quality Analysis***. This relates to concentrations, rate of movement, direction of movement, and dispersion of contaminants, and other issues

3. ***Water Rights and Water Law***. This includes volumetric delivery allotments, groundwater pumping, and excess water (e.g. irrigation runoff), among others

- Good quality, fresh water, is becoming more and more scarce as people exploit water resources more aggressively, and as the world population increases
- This increases the importance of water measurement
- It is unlikely that the regional and global situations on water availability and water quality will improve in the foreseeable future

When a resource is measured, it is implied that it has significant value; when not measured, the implication is of little or no value

VI. Some Fundamental Flow Measurement Concepts

- Most flow measurement devices and techniques are based on the measurement of head (depth or pressure) or velocity
- One exception to this is the salt dilution method (described below)
- Here, the term "flow rate" refers to volumetric rate, or volume per unit time
- Thus, we apply mathematical relationships between head and discharge, or take products of velocity and cross-sectional area

- Strictly speaking, all open-channel and most pipe flow measurement techniques cause head loss

"*The inability to make accurate measurements is not necessarily because of instrumentation deficiencies, but is a fundamental property of the physical world – you cannot measure something without changing it*" (paraphrased) W.K. Heisenberg (1901-76)

- However, some methods incur negligible losses (e.g. ultrasonic)
- It is usually desirable to have only a small head loss because this loss typically translates into an increased upstream flow depth in subcritical open-channel flow
- In open-channel flow measurement, devices can operate under *free flow* and *submerged flow* regimes
- In free flow, we are concerned with the upstream head because critical flow occurs in the vicinity of the flow measurement device. As long as this is true, changes in downstream depth will not affect discharge at that location.
- In submerged flow, we are concerned with a head differential across the flow measurement device.
- In this class, the terms *flow rate* and *discharge* will be used interchangeably.

VII. Flow Measurement Accuracy

- Perhaps the most accurate method for measuring flow rate is by timing the filling of a container of known volume
- However, this is often not practical for large flow rates
- Typical flow measurement accuracies are from ±2% to ±20% of the true discharge, but this range can be much greater
- Measurements of head, velocity, and area are subject to errors for a variety of reasons:

1. Approach Conditions

   - High approach velocity
   - Approach velocity not perpendicular to measurement device

2. Turbulence and Eddies

   - Rough water surface
   - Swirling flow near or at measurement location

3. Equipment Problems

   - Staff gauges, current meters, floats, etc., in disrepair
   - Shifting calibrations on pressure transducers and other meters
   - Poor installation (non-horizontal crest, wrong dimensions, etc.)

4. Measurement Location

   - Local measurements (uniform flow assumption?)
   - Stream gauging stations (need steady flow)

5. Human Errors

   - Misreading water levels, etc.
   - Misuse of equipment, or improper application of equipment

VIII. Simple Flow Measurement Methods

- The following are considered to be special methods, because they are mostly simple and approximate, and because they are not usually the preferred methods for flow measurement in open channels

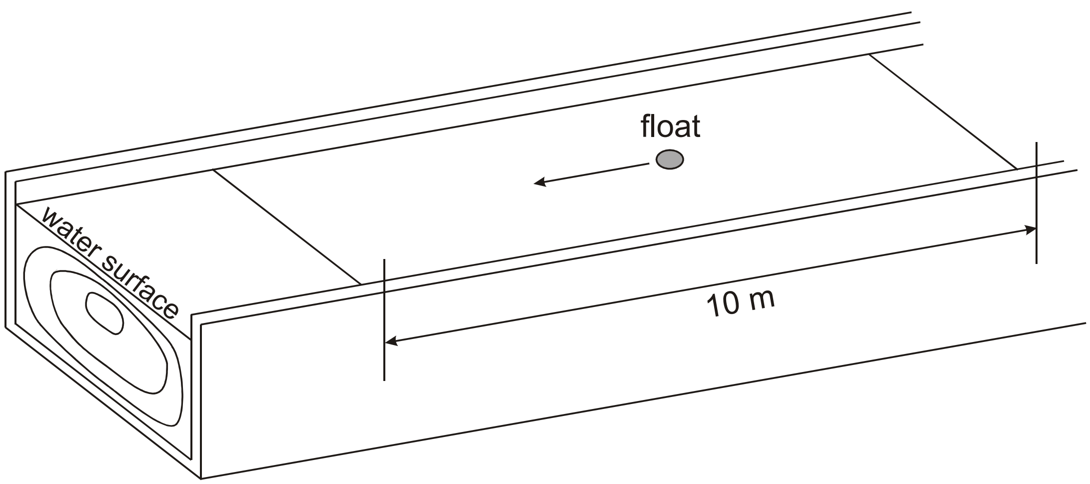
- Preferred methods are through the use of calibrated structures (weirs, flumes, orifices, and others), and current metering

1. Measurement by Observation

- In this method one must rely on experience to estimate the discharge in an open channel, simply by observing the flow in the channel and mentally comparing it to similar channels from which the flow rate was measured and known
- This method is usually not very accurate, especially for large flows, but some very experienced hydrographers can (with some luck) arrive at a close estimate

2. Measurement by Floats

- The average flow velocity in an open channel can be estimated by measuring the speed of a floating object on the surface of the water
- This can be done by marking uniform distances along the channel and using a watch to measure the elapsed time from a starting location to respective downstream locations
- However, in practice, usually only a single distance (say 10 m) is used



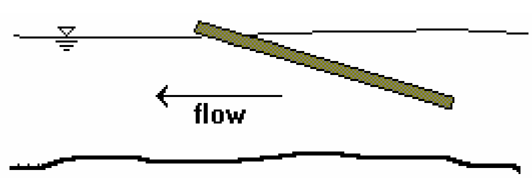
- It is a good idea to have more than one measurement point so that the velocity can be averaged over a reach, and to lessen the chance of an error
- Then, a graph can be made of float travel distance versus time, with the slope equal to the surface velocity of the water
- Select a location in which the channel is fairly straight, not much change in cross-section, smooth water surface, and no abrupt changes in bed elevation or longitudinal slope
- Note that wind can affect the velocity of the float, changing the relationship between surface velocity and average flow velocity

- Care should be taken to obtain measurements with the float moving near the center of the surface width of flow, not bumping into the channel sides, and not sinking

- The float speed will be higher than the average flow velocity in the channel, unless perhaps the float travels near one of the channel banks or is obstructed by vegetation
- You can estimate the average velocity in the channel by reducing the float speed by some fraction
- The following table is from the U.S. Bureau of Reclamation
- It gives coefficients to multiply by the measured float velocity, as a function of average depth, to obtain the approximate average flow velocity in the channel

Average Depth		**Coefficient**
(ft)	**(m)**	
1	0.30	0.66
2	0.61	0.68
3	0.91	0.70
4	1.22	0.72
5	1.52	0.74
6	1.83	0.76
9	2.74	0.77
12	3.66	0.78
15	4.57	0.79
>20	>6.10	0.80

- To obtain average depth, divide the cross-sectional area by the top width of the water surface (do not use an area-weighted average of subsection depths)
- The coefficients in the above table only give approximate results; you can typically expect errors of 10 to 20% in the flow rate
- What happens to the above coefficient values when the average water depth is below 1 ft (or 0.3 m)?
- Some hydrographers have used partially submerged wooden sticks which are designed to approximate the mean flow velocity, precluding the need for coefficients as in the above table
- One end of the stick is weighted so that is sinks further



- The stick will give the correct velocity only for a small range of water depths
- The float method is not precise because the relationship between float speed and true average flow velocity is not well known in general
- Other methods should be used if an accurate measurement is desired

Sample calculation:

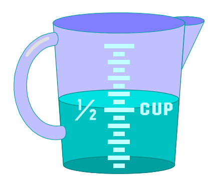
The float method is applied in a rectangular channel with a base width of 0.94 m and a uniform water depth of 0.45 m. Ten float travel times are recorded over a distance of 5.49 m (18 ft), with calculated surface velocities:

Trial	Time (s)	V (m/s)
1	7.33	0.749
2	6.54	0.839
3	7.39	0.743
4	7.05	0.779
5	6.97	0.788
6	6.83	0.804
7	7.27	0.755
8	6.87	0.799
9	7.11	0.772
10	6.86	0.800
Avg:	**7.02**	**0.783**

Alternatively, the average surface velocity can be taken as 5.49/7.02 = 0.782 m/s, which is very close to the average velocity of 0.783 m/s from the above table (as is usually the case). The surface velocity coefficient can be taken as 0.67, interpolating in the previous table of USBR data, and the cross-sectional area of the channel is (0.45)(0.94) = 0.42 m^2. Then, the estimated flow rate is:

$$(0.67)(0.783 \text{ m/s})(0.42 \text{ m}^2) = \mathbf{0.22\ m^3/s}$$

3. Dye Method



- The dye method, or "color-velocity method", can be used to measure the flow velocity, similar to the float method
- However, in this method a slug of dye is injected into the stream, and the time for the slug of dye to reach a downstream location is measured
- This time can be taken as the average of the time for the first portion of the dye to reach the downstream location, and the time for the last portion of the dye to reach that location (the dye will disperse and elongate as it moves downstream).
- The test section should not be too long, otherwise the dye will have dispersed too much and it is difficult to visually detect the color differential in the water
- Usually, it is appropriate to use a test section of approximately 3 m
- Dyes used in this type of measurement should be nontoxic so as not to pollute the water
- Food coloring can be used, as can other colored chemicals, such as "flouricine"

4. Salt Dilution

- In this method, an aqueous solution of known salt concentration, C_1, is poured into the stream at a constant rate, q
- The completely mixed solution is measured at a downstream location, providing the concentration C_2
- The downstream location should be at least 5 m (perhaps up to 10 or 15 m) from the point of injection, otherwise incomplete mixing may result in large errors; that is, you might measure a highly-concentrated slug of water, or you might miss the slug altogether, if you try to measure too close to the point of injection
- After measuring the existing salt concentration in the flow (without adding the concentrated solution), C_0, the stream discharge, Q, can be calculated as,

$$\Delta t\left[QC_0 + qC_1\right] = \Delta t\left[(Q+q)C_2\right] \quad (1)$$

or,

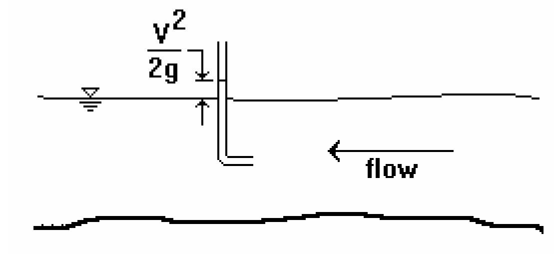
$$Q = q\left(\frac{C_1 - C_2}{C_2 - C_0}\right) \quad (2)$$

- The above two equations represent a mass balance, where: Δt can be in s (doesn't really matter, because it cancels out); C can be in mg per liter; and q and Q can be in lps
- This method is not used to measure velocity, but total volumetric flow rate
- Concentrations are normally expressed as mmho/cm, or dS/m

5. Uniform Flow

- In this method the channel bed slope, average cross-section, and average depth are measured
- A roughness value is estimated, and the Manning or Chezy equation is applied to calculate the discharge
- This method is valid only for steady uniform flow, and is severely limited by an inability to accurately estimate the roughness value
- And because it is only valid for steady uniform flow, it cannot be applied in general since these flow conditions are often not found in open channels
- Ideally, both bed slope and water surface slope are measured to verify whether the flow is uniform or not
- The discharge can be estimated by giving a range of probable flow rates for maximum and minimum roughness values (also estimated), based on the channel appearance and size
- The roughness can be estimated by experience, or by consulting hydraulics handbooks which provide tables and figures, or photographs

6. Pitot Tube

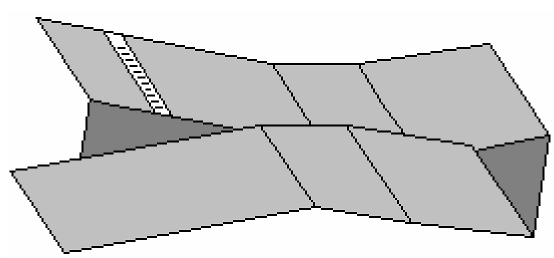


- A simple pitot tube can be positioned into the flow to measure the velocity head
- One end of the tube is pointed into the flow, and the other end is pointed up vertically out of the water – both ends are open
- The submerged end of the tube is positioned to be essentially parallel to the flow
- Solving for velocity in the equation for velocity head:

$$V = \sqrt{2gh} \tag{3}$$

- This method is best applied for higher flow velocities because it is difficult to read the head differential at low velocities, in which large errors in the estimation of velocity can result

IX. Introduction to Flumes



- Measurement flumes are open-channel devices with a specially-shaped, partially-constricted throat section
- The flume geometry is often designed to cause the flow regime to pass through critical depth, providing a means for determining the rate of flow from a single (upstream) water depth measurement – this is the advantage of a free-flow regime
- When the water surface exceeds specified limits, submerged-flow conditions occur and two water depth measurements are required (upstream and downstream)
- In channels with small longitudinal bed slopes, it may be desirable to install a flume to operate under conditions of submerged-flow rather than free flow in order to:

    1. reduce energy losses
    2. place the flume on the channel bed to minimize the increase in upstream water surface elevation

    (the above two reasons are essentially the same thing)

- Many different flow measurement flumes have been designed and tested, but only a few are commonly found in practice today

X. Flume Classifications

There are two principal classes of flumes: short-throated and long-throated

Short-throated flumes:

- Critical flow conditions occur in regions of curvilinear flow (assuming the regime is free flow)
- These include flumes with side contractions and bottom contractions, and some type of transition section
- In general, laboratory calibrations are required to obtain flow coefficients for rating
- Under favorable operating conditions the discharge can be determined with an accuracy of ±2 to ±5% for free flow

Long-throated flumes:

- Critical flow conditions are created in a region of parallel flow in the control section, again, assuming free flow conditions
- These linear-stream flow conditions are much better theoretically defined; thus, rating relations can be reasonably well predicted
- Generally, flows larger than 10 lps can be measured with an error of less than ±2% in an appropriately dimensioned flume
- Broad-crested weirs are an example of long-throated flumes

XI. Advantages and Disadvantages of Flumes

Advantages

1. capable of operation with relatively small head loss, and a high transition submergence value (compared to sharp-crested weirs)
2. capable of measuring a wide range of free-flow discharges with relatively high tail-water depths, using a single water depth measurement
3. capable of measuring discharge under submerged flow conditions using two water depth measurements
4. both sediment and floating debris tend to pass through the structure
5. no need for a deep and wide upstream pool to reduce the velocity of approach

Disadvantages

1. usually more expensive to construct than weirs
2. must be constructed carefully and accurately for satisfactory performance
3. cannot be used as flow control structures (compared to adjustable weirs, orifice gates, and other structures)

4. often need to use standard design dimensions, unless you want to develop your own calibration curve

XII. Free, Submerged, and Transitional Flow

- When critical flow occurs the flow rate through the flume is uniquely related to the upstream depth, h_u
- That is, the free flow discharge can be obtained with only a single water depth measurement

$$Q_f = f(h_u) \tag{4}$$

- When the tail-water depth is increased such that the flume operates under submerged-flow conditions, both upstream, h_u, and downstream, h_d, depth measurements are required.
- Let S be the submergence ratio, or $S = h_d/h_u$. Then, Q_s is a function of the head differential, $(h_u - h_d)$, and S

$$Q_s = f(h_u - h_d, S) \tag{5}$$

- The value of submergence which marks the change from free flow to submerged flow, and vice versa, is referred to as the transition submergence, S_t.
- At this condition the discharge given by the free-flow equation is exactly the same as that given by submerged-flow equation

XIII. Parshall Flumes

- The Parshall flume is perhaps the most commonly used open-channel flow-measuring device in irrigation systems in the U.S. and elsewhere
- It was developed at Colorado State University by Ralph Parshall from 1915-1922
- Some characteristics of this flume design are:

  1. This flume has specially designed converging, throat and diverging sections
  2. It has been designed to measure flow from 0.01 to 3,000 cfs (1 lps to 85 m^3/s), or more
  3. Under typical conditions, free-flow accuracy is ±5% of the true discharge
  4. Under favorable conditions (calm upstream water surface, precise flume construction, level upstream flume floor) free-flow accuracy can be ±2% of the true discharge
  5. The head loss across a comparably-sized sharp-crested weir under free-flow conditions is roughly four times that of a Parshall flume operating under free-flow conditions
  6. It is usually designed to operate under free-flow conditions

7. Size selection is based on the flume width which best fits the channel dimensions and hydraulic properties
8. As a general rule, the width of the throat of a Parshall flume should be about one-third to one-half the width of the upstream water surface in the channel at the design discharge and at normal depth

- The general forms of the free-flow and submerged-flow equations for flumes, including the Parshall flume, are:

Free Flow

$$Q_f = C_f W \left(h_u\right)^{n_f} \tag{6}$$

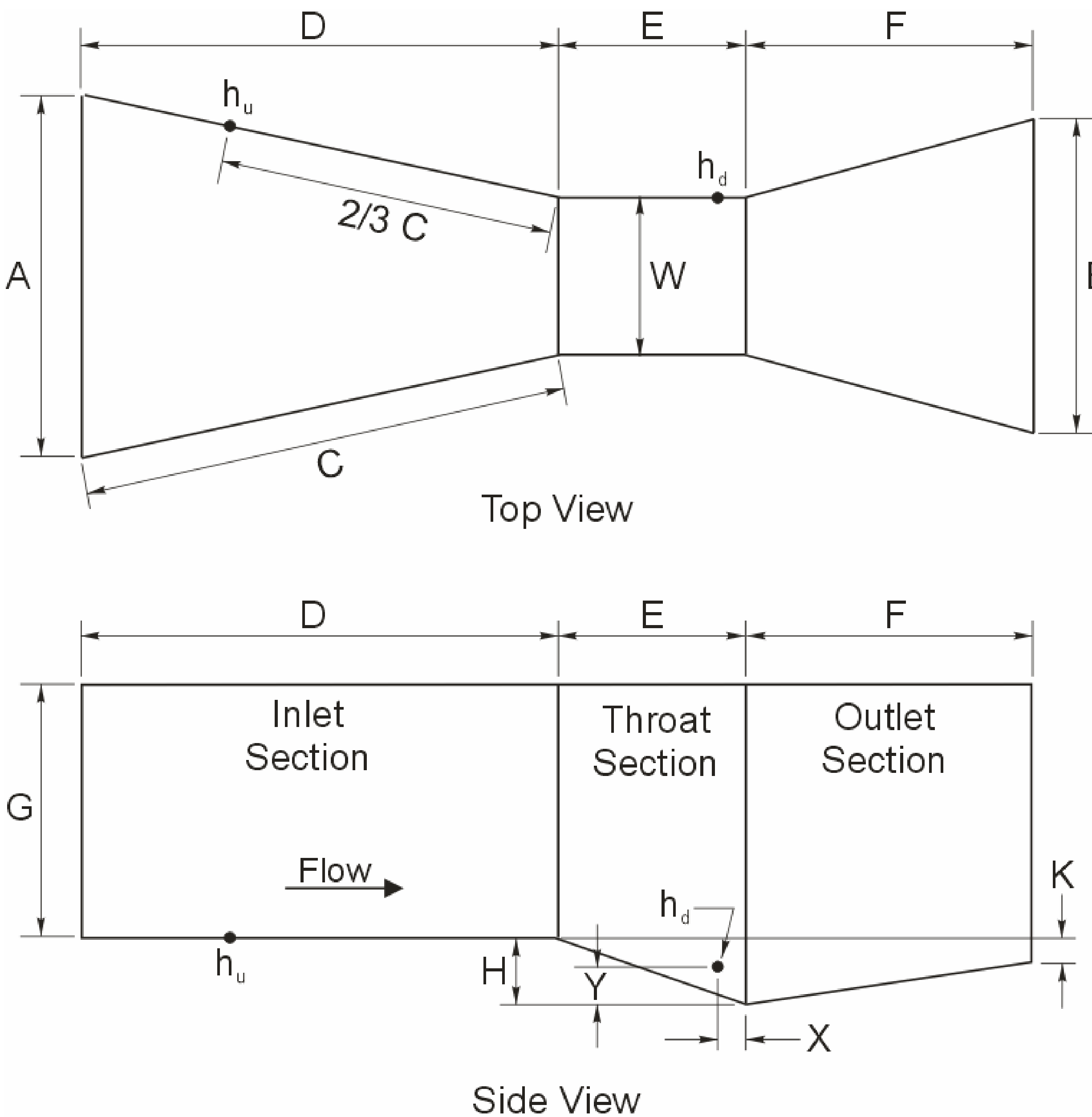
Submerged Flow

$$Q_s = \frac{C_s W (h_u - h_d)^{n_f}}{[-(\log_{10} S + C_2)]^{n_s}} \tag{7}$$

where n_f and n_s are the free-flow and submerged-flow exponents, respectively; and W is the throat width.

- It is strongly recommended that you use the same units for W and depth (h_u and h_d) in Eqs. 6 and 7 (i.e. don't put W in inches and h_u in feet)
- Below are two views of a Parshall flume
- Note that both h_u and h_d are measured from the upstream floor elevation, that is, from a common datum
- This is in spite of the fact that the downstream tap is supposed to be located at an elevation equal to H – Y below the upstream floor, as shown in the figure below

- The diverging outlet section of the flume is not required when the structure is placed at a drop in bed elevation, whereby it would always operate under free-flow conditions
- The USBR (1974) discusses "modified Parshall flumes" which fit a particular canal profile
- The following table gives dimensions (A-H, K, X & Y) and discharge ranges for the 23 standard Parshall flume sizes (see the following figure showing the dimensional parameters) in metric units



D
E
F
h_u
h_d
2/3 C
A
W
B
C
Top View
D
E
F
G
Inlet Section
Throat Section
Outlet Section
Flow
K
h_d
h_u
H
Y
X
Side View

Parshall Flume Dimensions in <u>metric</u> units (see the above figure)

	Dimensions (m)											Q (m^3/s)	
W (m)	A	B	C	D	E	F	G	H	K	X	Y	min	max
0.025	0.167	0.093	0.363	0.356	0.076	0.203	0.152	0.029	0.019	0.008	0.013	0.00028	0.0057
0.051	0.214	0.135	0.414	0.406	0.114	0.254	0.203	0.043	0.022	0.016	0.025	0.00057	0.011
0.076	0.259	0.178	0.467	0.457	0.152	0.305	0.381	0.057	0.025	0.025	0.038	0.00085	0.017
0.152	0.394	0.394	0.621	0.610	0.305	0.610	0.457	0.114	0.076	0.051	0.076	0.00142	0.082
0.229	0.575	0.381	0.879	0.864	0.305	0.457	0.610	0.114	0.076	0.051	0.076	0.00283	0.144
0.305	0.845	0.610	1.372	1.343	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.0113	0.453
0.457	1.026	0.762	1.448	1.419	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.0142	0.680
0.610	1.207	0.914	1.524	1.495	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.0198	0.934
0.762	1.391	1.067	1.632	1.600	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.0227	1.16
0.914	1.572	1.219	1.676	1.645	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.0283	1.42
1.219	1.937	1.524	1.829	1.794	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.0368	1.93
1.524	2.302	1.829	1.981	1.943	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.0623	2.44
1.829	2.667	2.134	2.134	2.092	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.0736	2.94
2.134	3.032	2.438	2.286	2.242	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.116	3.43
2.438	3.397	2.743	2.438	2.391	0.610	0.914	0.914	0.229	0.076	0.051	0.076	0.130	3.96
3.048	4.756	3.658	4.350	4.267	0.914	1.829	1.219	0.343	0.152	0.305	0.229	0.170	5.66
3.658	5.607	4.470	4.972	4.877	0.914	2.438	1.524	0.343	0.152	0.305	0.229	0.227	9.91
4.572	7.620	5.588	7.772	7.620	1.219	3.048	1.829	0.457	0.229	0.305	0.229	0.227	17.0
6.096	9.144	7.315	7.772	7.620	1.829	3.658	2.134	0.686	0.305	0.305	0.229	0.283	28.3
7.620	10.668	8.941	7.772	7.620	1.829	3.962	2.134	0.686	0.305	0.305	0.229	0.425	34.0
9.144	12.313	10.566	8.084	7.925	1.829	4.267	2.134	0.686	0.305	0.305	0.229	0.425	42.5
12.192	15.481	13.818	8.395	8.230	1.829	4.877	2.134	0.686	0.305	0.305	0.229	0.566	56.6
15.240	18.529	17.272	8.395	8.230	1.829	6.096	2.134	0.686	0.305	0.305	0.229	0.708	85.0

- It is noted that Parshall flumes were developed using English units, but these days we often prefer metric units
- Anyway, many of the dimensions in English units were not even “round” numbers, often being specified to the 32^{nd} of an inch
- The next table shows Parshall flume dimensions for the same 23 standard sizes, but in feet, rounded to the thousandth of a foot, with discharge ranges in cubic feet per second

Parshall Flume Dimensions in English units (see the above figure)

W (ft)	Dimensions (ft)											Q (cfs)	
	A	B	C	D	E	F	G	H	K	X	Y	min	max
0.083	0.549	0.305	1.190	1.167	0.250	0.667	0.500	0.094	0.063	0.026	0.042	0.01	0.2
0.167	0.701	0.443	1.359	1.333	0.375	0.833	0.667	0.141	0.073	0.052	0.083	0.02	0.4
0.250	0.849	0.583	1.531	1.500	0.500	1.000	1.250	0.188	0.083	0.083	0.125	0.03	0.6
0.500	1.292	1.292	2.036	2.000	1.000	2.000	1.500	0.375	0.250	0.167	0.250	0.05	2.9
0.750	1.885	1.250	2.885	2.833	1.000	1.500	2.000	0.375	0.250	0.167	0.250	0.10	5.1
1.000	2.771	2.000	4.500	4.406	2.000	3.000	3.000	0.750	0.250	0.167	0.250	0.40	16.0
1.500	3.365	2.500	4.750	4.656	2.000	3.000	3.000	0.750	0.250	0.167	0.250	0.50	24.0
2.000	3.958	3.000	5.000	4.906	2.000	3.000	3.000	0.750	0.250	0.167	0.250	0.70	33.0
2.500	4.563	3.500	5.354	5.250	2.000	3.000	3.000	0.750	0.250	0.167	0.250	0.80	41.0
3.000	5.156	4.000	5.500	5.396	2.000	3.000	3.000	0.750	0.250	0.167	0.250	1.0	50.0
4.000	6.354	5.000	6.000	5.885	2.000	3.000	3.000	0.750	0.250	0.167	0.250	1.3	68.0
5.000	7.552	6.000	6.500	6.375	2.000	3.000	3.000	0.750	0.250	0.167	0.250	2.2	86.0
6.000	8.750	7.000	7.000	6.865	2.000	3.000	3.000	0.750	0.250	0.167	0.250	2.6	104
7.000	9.948	8.000	7.500	7.354	2.000	3.000	3.000	0.750	0.250	0.167	0.250	4.1	121
8.000	11.146	9.000	8.000	7.844	2.000	3.000	3.000	0.750	0.250	0.167	0.250	4.6	140
10.000	15.604	12.000	14.271	14.000	3.000	6.000	4.000	1.125	0.500	1.000	0.750	6.0	200
12.000	18.396	14.667	16.313	16.000	3.000	8.000	5.000	1.125	0.500	1.000	0.750	8.0	350
15.000	25.000	18.333	25.500	25.000	4.000	10.000	6.000	1.500	0.750	1.000	0.750	8.0	600
20.000	30.000	24.000	25.500	25.000	6.000	12.000	7.000	2.250	1.000	1.000	0.750	10	1000
25.000	35.000	29.333	25.500	25.000	6.000	13.000	7.000	2.250	1.000	1.000	0.750	15	1200
30.000	40.396	34.667	26.521	26.000	6.000	14.000	7.000	2.250	1.000	1.000	0.750	15	1500
40.000	50.792	45.333	27.542	27.000	6.000	16.000	7.000	2.250	1.000	1.000	0.750	20	2000
50.000	60.792	56.667	27.542	27.000	6.000	20.000	7.000	2.250	1.000	1.000	0.750	25	3000

- The minimum flow rate values represent the limits of the validity of the free-flow rating equation
- For submerged flow conditions, a minimum flow rate also applies because if it is very low, the difference between h_u and h_d will be virtually indistinguishable (perhaps 1 mm or less)

- The next table gives calibration parameters (C_f, C_s, n_f, n_s) and transition submergence (S_t) for standard Parshall flume sizes (metric units)
- Use Eq. (3) or (4) to get flow rate in m^3/s, where depths are in metres
- The C_2 value in Eq. (4) is equal to about 0.0044 (dimensionless) for all of the standard Parshall flume sizes
- Standard sizes were developed in English units, so the throat width values show below are "odd" numbers, but the ft-inch equivalents are given in parentheses
- Note that S_t is transition submergence – the value tends to increase with the size of the flume, up to a maximum of about 0.80
- Be aware that the S_t values in the table below are for the maximum flow rate; for other flow rates it is different
- Also note that the values in the table below are for a base 10 logarithm in Eq. (4)
- In practice, under extreme submerged-flow conditions, the head differential, h_u-h_d, can be less than 1 mm and no measurement is possible with the flume

Parshall Flume Calibration Parameters for metric units (depth and W in m, flow rate in m^3/s)

Throat Width (m)	C_f	C_s	n_f	n_s	S_t	Metric Units
0.025 (1”)	2.38	2.10	1.55	1.000	0.56	
0.051 (2”)	2.38	2.15	1.55	1.000	0.61	
0.076 (3”)	2.32	2.14	1.55	1.000	0.64	
0.152 (6”)	2.50	2.02	1.58	1.080	0.55	
0.229 (9”)	2.34	1.91	1.53	1.060	0.63	
0.305 (12”)	2.26	1.76	1.52	1.080	0.62	
0.457 (18”)	2.32	1.71	1.54	1.115	0.64	
0.610 (24”)	2.34	1.74	1.55	1.140	0.66	
0.762 (30”)	2.36	1.70	1.56	1.150	0.67	
0.914 (3’)	2.37	1.70	1.56	1.160	0.68	
1.219 (4’)	2.40	1.66	1.57	1.185	0.70	
1.524 (5’)	2.43	1.65	1.58	1.205	0.72	
1.829 (6’)	2.46	1.62	1.59	1.230	0.74	
2.134 (7’)	2.49	1.61	1.60	1.250	0.76	
2.438 (8’)	2.49	1.59	1.60	1.260	0.78	
3.048 (10’)	2.47	1.52	1.59	1.275	0.80	
3.658 (12’)	2.43	1.50	1.59	1.275	0.80	
4.572 (15’)	2.40	1.48	1.59	1.275	0.80	
6.096 (20’)	2.37	1.46	1.59	1.275	0.80	
7.620 (25’)	2.35	1.45	1.59	1.275	0.80	
9.144 (30’)	2.33	1.44	1.59	1.275	0.80	
12.192 (40’)	2.32	1.43	1.59	1.275	0.80	
15.240 (50’)	2.31	1.42	1.59	1.275	0.80	

- The following table gives calibration parameters (C_f, C_s, n_f, n_s) and transition submergence (S_t) for standard Parshall flume sizes in English units

Parshall Flume Calibration Parameters for <u>English</u> units (depth and W in ft, flow rate in cfs)

Throat Width	C_f	C_s	n_f	n_s	S_t	English Units
1 inches	4.06	3.59	1.550	1.000	0.56	
2 inches	4.06	3.67	1.550	1.000	0.61	
3 inches	3.97	3.66	1.550	1.000	0.64	
6 inches	4.12	3.32	1.580	1.080	0.55	
9 inches	4.09	3.35	1.530	1.060	0.63	
12 inches	4.00	3.11	1.520	1.080	0.62	
18 inches	4.00	2.95	1.540	1.115	0.64	
24 inches	4.00	2.97	1.550	1.140	0.66	
30 inches	4.00	2.89	1.555	1.150	0.67	
3 feet	4.00	2.87	1.560	1.160	0.68	
4 feet	4.00	2.78	1.570	1.185	0.70	
5 feet	4.00	2.71	1.580	1.205	0.72	
6 feet	4.00	2.64	1.590	1.230	0.74	
7 feet	4.00	2.59	1.600	1.250	0.76	
8 feet	4.00	2.55	1.600	1.260	0.78	
10 feet	4.01	2.48	1.590	1.275	0.80	
12 feet	3.96	2.45	1.590	1.275	0.80	
15 feet	3.90	2.41	1.590	1.275	0.80	
20 feet	3.85	2.38	1.590	1.275	0.80	
25 feet	3.82	2.36	1.590	1.275	0.80	
30 feet	3.80	2.34	1.590	1.275	0.80	
40 feet	3.77	2.33	1.590	1.275	0.80	
50 feet	3.75	2.32	1.590	1.275	0.80	

- It is seen that n_f, n_s, and S_t are dimensionless, but C_f & C_s depend on the units
- Also, the submerged-flow coefficient, C_s, is for a base-10 logarithm
- Note that, for 1 ft ≤ W ≤ 8 ft, the n_f value can be approximated as:

$$n_f \approx 1.522\,W^{0.026} \tag{8}$$

where W is in ft

- For W > 8 ft, n_f is constant, at a value of 1.59
- For W > 8 ft, n_s and S_t also remain constant at 1.275 and 0.80, respectively

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Lecture 2

Flumes for Open-Channel Flow Measurement

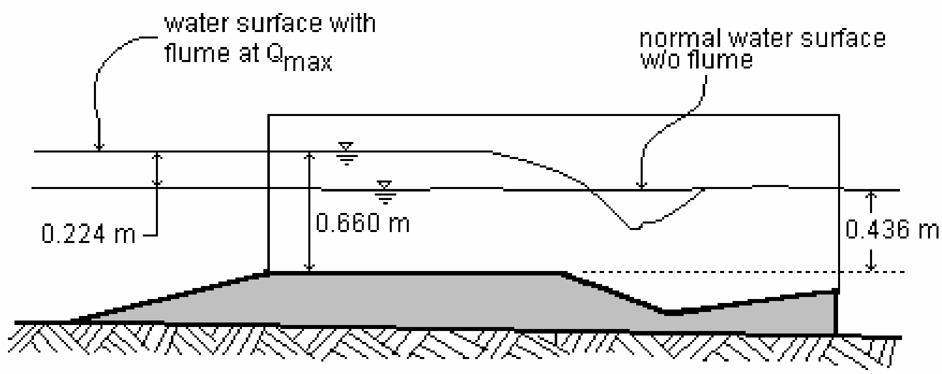
"Superb accuracy in water measurement, Jessica thought."
Dune, F. Herbert (1965)

I. Procedure for Installing a Parshall Flume to Ensure Free Flow

- If possible, you will want to specify the installation of a Parshall flume such that it operates under free-flow conditions throughout the required flow range
- To do this, you need to specify the minimum elevation of the upstream floor of the flume
- Follow these simple steps to obtain a free-flow in a Parshall flume, up to a specified maximum discharge:

1. Determine the maximum flow rate (discharge) to be measured
2. Locate the high water line on the canal bank where the flume is to be installed, or otherwise determine the maximum depth of flow on the upstream side
3. Select a standard flume size and calculate h_u from the free-flow equation corresponding to the maximum discharge capacity of the canal
4. Place the floor of the flume at a depth not exceeding the transition submergence, S_t, multiplied by h_u below the high water line

- In general, the floor of the flume should be placed as high in the canal as grade and other conditions permit, but not so high that upstream free board is compromised.
- The downstream water surface elevation will be unaffected by the installation of the flume (at least for the same flow rate)
- As an example, a 0.61-m Parshall flume is shown in the figure below
- The transition submergence, S_t, for the 0.61-m flume is 66% (see table)
- The maximum discharge in the canal is given as 0.75 m^3/s, which for free-flow conditions must have an upstream depth of (see Eq. 3): $h_u = (0.75/1.429)^{1/1.55} = 0.66$ m
- With the transition submergence of 0.66, this gives a depth to the flume floor of 0.66(0.660 m) = 0.436 m from the downstream water surface
- Therefore, the flume crest (elevation of h_u tap) should be set no lower than 0.436 m below the normal maximum water surface for this design flow rate, otherwise the regime will be submerged flow
- However, if the normal depth for this flow rate were less than 0.436 m, you would place the floor of the flume on the bottom of the channel and still have free flow conditions
- The approximate head loss across the structure at the maximum flow rate will be the difference between 0.660 and 0.436 m, or 0.224 m
- This same procedure can be applied to other types of open-channel measurement flumes



II. Non-Standard Parshall Flume Calibrations

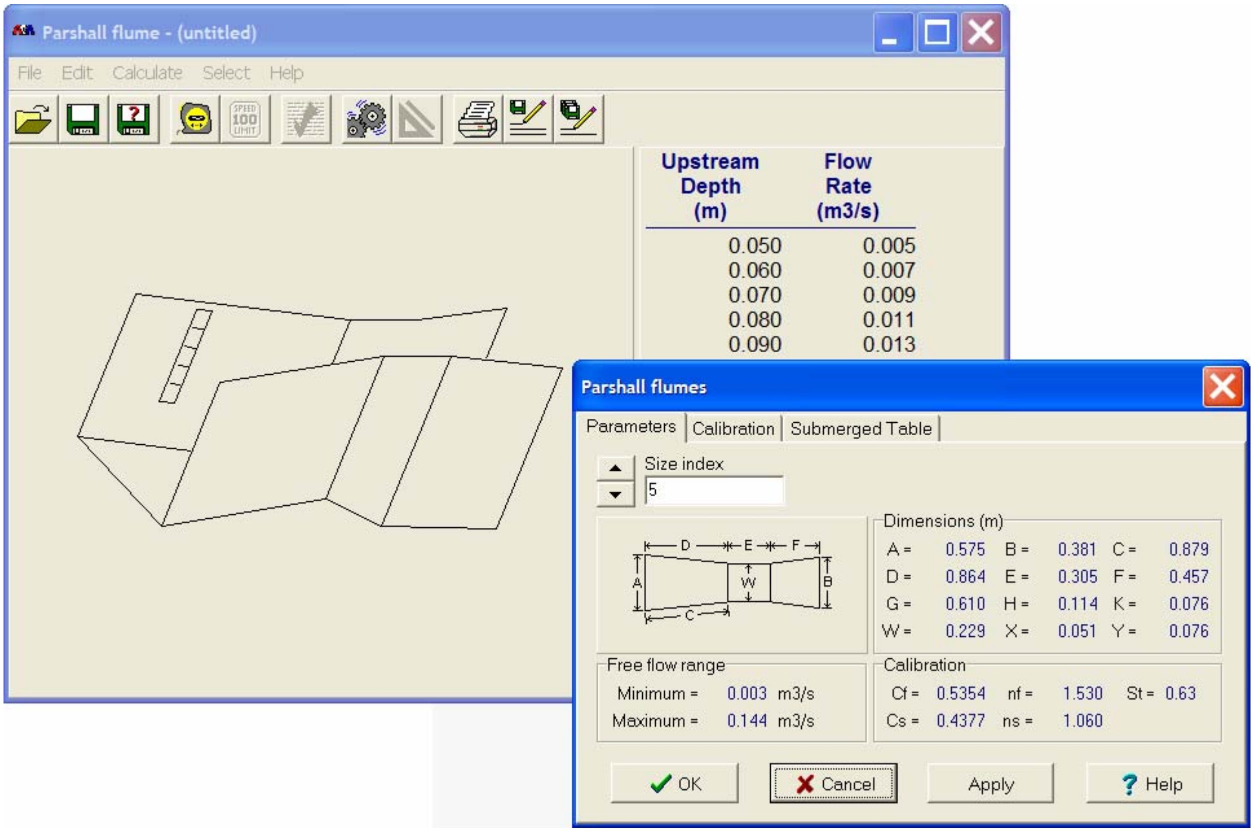
- Some Parshall flumes were incorrectly constructed or were intentionally built with a non-standard size
- Others have settled over time such that the flume is out of level either cross-wise or longitudinally (in the direction of flow), or both
- Some flumes have the taps for measuring h_u and h_d at the wrong locations (too high or too low, or too far upstream or downstream)
- Some flumes have moss, weeds, sediment or other debris that cause the calibration to shift from that given for standard conditions
- Several researchers have worked independently to develop calibration adjustments for many of the unfortunate anomalies that have befallen many Parshall flumes in the field, but a general calibration for non-standard flumes requires 3-D modeling
- There are calibration corrections for out-of-level installations and for low-flow conditions

III. Hysteresis Effects in Parshall Flumes

- There have been reports by some researchers that hysteresis effects have been observed in the laboratory under submerged-flow conditions in Parshall flumes
- The effect is to have two different flow rates for the same submergence, S, value, depending on whether the downstream depth is rising or falling
- There is no evidence of this hysteresis effect in Cutthroat flumes, which are discussed below

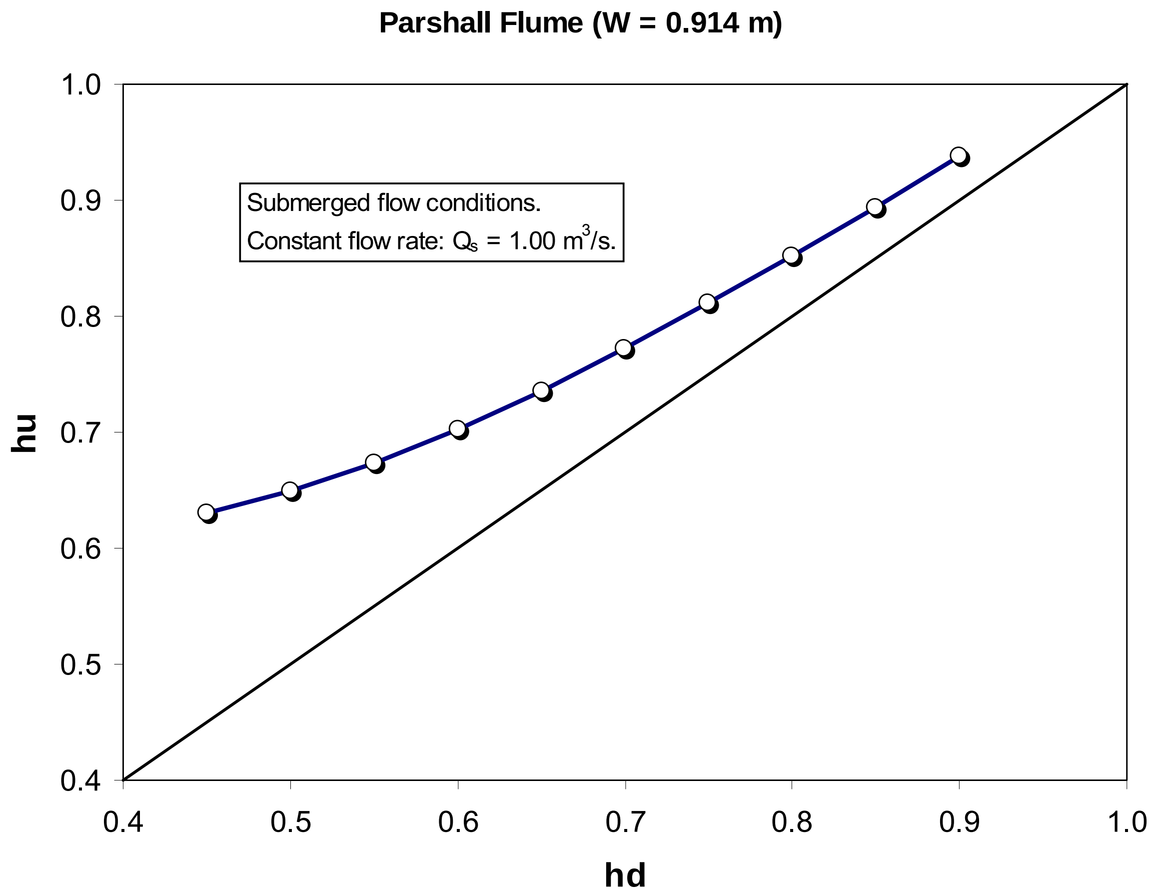
IV. Software

- You can use the ***ACA*** program to develop calibration tables for Parshall, Cutthroat, and trapezoidal flumes
- Download ***ACA*** from: http://www.engineering.usu.edu/bie/faculty/merkley/Software.htm
- You can also download the ***WinFlume*** program from: http://www.usbr.gov/pmts/hydraulics_lab/winflume/index.html



V. Submerged-Flow, Constant Flow Rate

- Suppose you have a constant flow rate through a Parshall flume
- How will h_u change for different h_d values under submerged-flow conditions?
- This situation could occur in a laboratory flume, or in the field where a downstream gate is incrementally closed, raising the depth downstream of the Parshall flume, but with a constant upstream inflow
- The graph below is for steady-state flow conditions with a 0.914-m Parshall flume
- Note that h_u is always greater than h_d (otherwise the flow would move upstream, or there would be no flow)



hd (m)	hu (m)	Q (m3/s)	S	Regime
0.15	0.714	0.999	0.210	free
0.20	0.664	0.999	0.301	free
0.25	0.634	0.999	0.394	free
0.30	0.619	1.000	0.485	free
0.35	0.615	1.002	0.569	free
0.40	0.619	1.000	0.646	free
0.45	0.631	1.000	0.713	submerged
0.50	0.650	1.001	0.769	submerged
0.55	0.674	1.000	0.816	submerged
0.60	0.703	1.000	0.853	submerged
0.65	0.736	1.000	0.883	submerged
0.70	0.772	0.999	0.907	submerged
0.75	0.811	1.001	0.925	submerged
0.80	0.852	1.004	0.939	submerged
0.85	0.894	1.000	0.951	submerged
0.90	0.938	1.002	0.959	submerged

VI. Cutthroat Flumes


A Cutthroat flume

- The Cutthroat flume was developed at USU from 1966-1990
- A Cutthroat flume is a rectangular open-channel constriction with a flat bottom and zero length in the throat section (earlier versions did have a throat section)
- Because the flume has a throat section of zero length, the flume was given the name "Cutthroat" by the developers (Skogerboe, et al. 1967)
- The floor of the flume is level (as opposed to a Parshall flume), which has the following advantages:

    1. ease of construction – the flume can be readily placed inside a concrete-lined channel
    2. the flume can be placed on the channel bed

- The Cutthroat flume was developed to operate satisfactorily under both free-flow and submerged-flow conditions
- Unlike Parshall flumes, all Cutthroat flumes have the same dimensional ratios
- It has been shown by experiment that downstream flow depths measured in the diverging outlet section give more accurate submerged-flow calibration curves than those measured in the throat section of a Parshall flume
- The centers of the taps for the US and DS head measurements are both located ½-inch above the floor of the flume, and the tap diameters should be ¼-inch

Cutthroat Flume Sizes

- The dimensions of a Cutthroat flume are identified by the flume width and length (W x L, e.g. 4" x 3.0')
- The flume lengths of 1.5, 3.0, 4.5, 6.0, 7.5, 9.0 ft are sufficient for most applications
- The most common ratios of W/L are 1/9, 2/9, 3/9, and 4/9
- The recommended ratio of h_u/L is equal to or less than 0.33

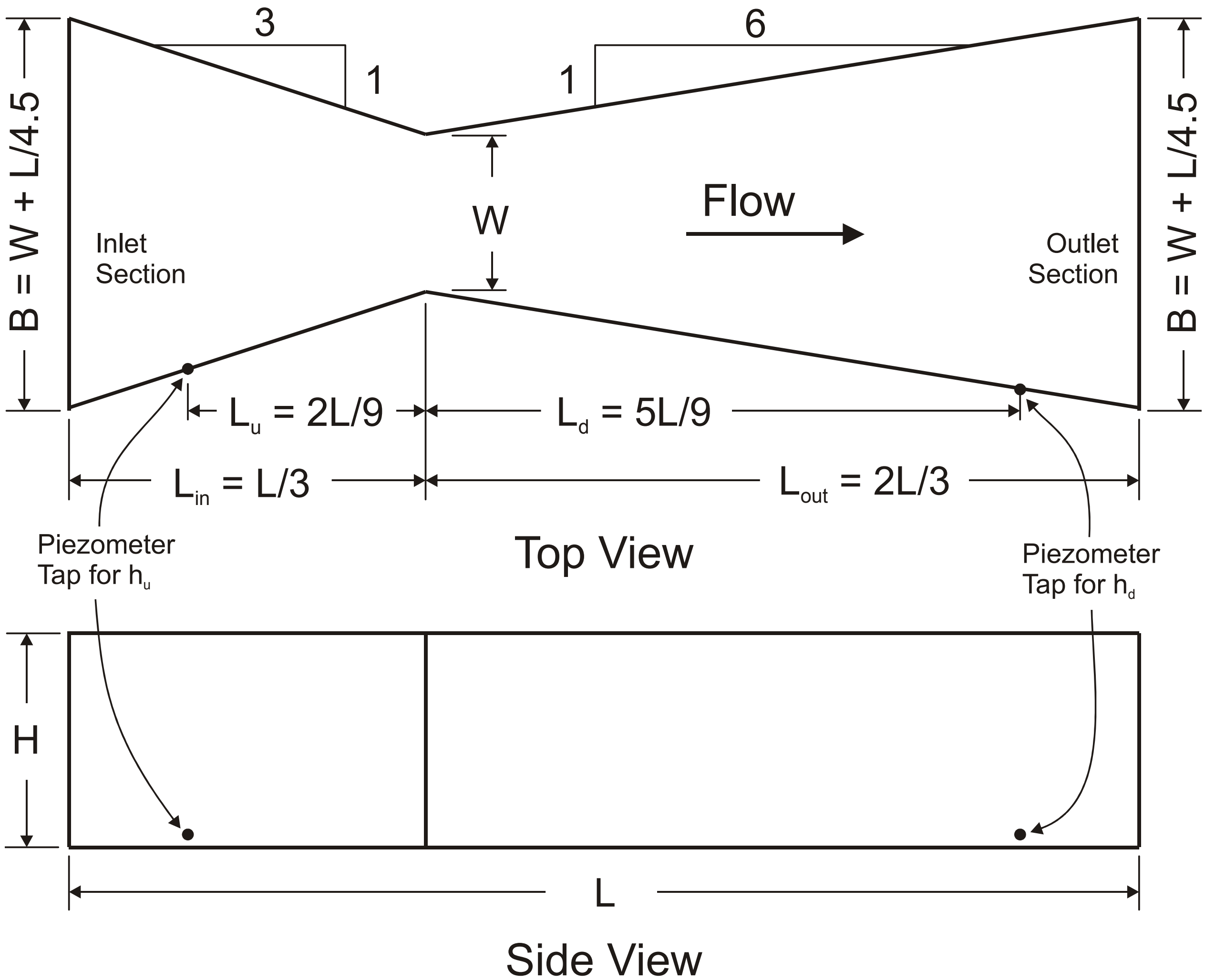
Free-flow equation

- For Cutthroat flumes the free-flow equation takes the same general form as for Parshall flumes, and other channel "constrictions":

$$Q_f = C_f W \left(h_u\right)^{n_f} \quad (1)$$

where Q_f is the free-flow discharge; W is the throat width; C_f is the free-flow coefficient; and n_f is the free-flow exponent

- That is, *almost any* non-orifice constriction in an open channel can be calibrated using Eq. 1, given free-flow conditions
- The depth, h_u, is measured from the upstream tap location (½-inch above the flume floor)



- For any given flume size, the flume wall height, H, is equal to h_u for Q_{max}, according to the above equation, although a slightly larger H-value can be used to prevent the occurrence of overflow
- So, solve the above free-flow equation for h_u, and apply the appropriate Q_{max} value from the table below; the minimum H-value is equal to the calculated h_u

Submerged-flow equation

- For Cutthroat flumes the submerged-flow equation also takes the same general form as for Parshall flumes, and other channel constrictions:

$$Q_s = \frac{C_s W (h_u - h_d)^{n_f}}{[-(\log_{10} S)]^{n_s}} \tag{2}$$

where C_s = submerged-flow coefficient; W is the throat width; and $S = h_d/h_u$

- Equation 2 differs from the submerged-flow equation given previously for Parshall flumes in that the C_2 term is omitted
- The coefficients C_f and C_s are functions of flume length, L, and throat width, W
- The generalized free-flow and submerged-flow coefficients and exponents for standard-sized Cutthroat flumes can be taken from the following table (metric units: for Q in m^3/s and head (depth) in m, and using a *base 10 logarithm* in Eq. 2)
- *Almost any* non-orifice constriction in an open channel can be calibrated using Eq. 2, given submerged-flow conditions

Cutthroat Flume Calibration Parameters for metric units (depth and W in m and flow rate in m^3/s)

W (m)	L (m)	C_f	n_f	S_t	C_s	n_s	Discharge (m^3/s)	
							min	max
0.051	0.457	5.673	1.98	0.553	3.894	1.45	0.0001	0.007
0.102	0.457	5.675	1.97	0.651	3.191	1.58	0.0002	0.014
0.152	0.457	5.639	1.95	0.734	2.634	1.67	0.0004	0.022
0.203	0.457	5.579	1.94	0.798	2.241	1.73	0.0005	0.030
0.102	0.914	3.483	1.84	0.580	2.337	1.38	0.0002	0.040
0.203	0.914	3.486	1.83	0.674	1.952	1.49	0.0005	0.081
0.305	0.914	3.459	1.81	0.754	1.636	1.57	0.0008	0.123
0.406	0.914	3.427	1.80	0.815	1.411	1.64	0.0011	0.165
0.152	1.372	2.702	1.72	0.614	1.752	1.34	0.0005	0.107
0.305	1.372	2.704	1.71	0.708	1.469	1.49	0.0010	0.217
0.457	1.372	2.684	1.69	0.788	1.238	1.50	0.0015	0.326
0.610	1.372	2.658	1.68	0.849	1.070	1.54	0.0021	0.436
0.203	1.829	2.351	1.66	0.629	1.506	1.30	0.0007	0.210
0.406	1.829	2.353	1.64	0.723	1.269	1.39	0.0014	0.424
0.610	1.829	2.335	1.63	0.801	1.077	1.45	0.0023	0.636
0.813	1.829	2.315	1.61	0.862	0.934	1.50	0.0031	0.846
0.254	2.286	2.147	1.61	0.641	1.363	1.28	0.0009	0.352
0.508	2.286	2.148	1.60	0.735	1.152	1.37	0.0019	0.707
0.762	2.286	2.131	1.58	0.811	0.982	1.42	0.0031	1.056
1.016	2.286	2.111	1.57	0.873	0.850	1.47	0.0043	1.400
0.305	2.743	2.030	1.58	0.651	1.279	1.27	0.0012	0.537
0.610	2.743	2.031	1.57	0.743	1.085	1.35	0.0025	1.076
0.914	2.743	2.024	1.55	0.820	0.929	1.40	0.0039	1.611
1.219	2.743	2.000	1.54	0.882	0.804	1.44	0.0055	2.124

- Note that n_f approaches 1.5 for larger W values, but never gets down to 1.5
- As for the Parshall flume data given previously, the submerged-flow calibration is for base 10 logarithms
- Note that the coefficient conversion to English units is as follows:

$$C_{f\,(English)} = \frac{(0.3048)^{1+n_f}}{(0.3048)^3} C_{f\,(metric)} \tag{3}$$

- The next table shows the calibration parameters for English units

Cutthroat Flume Calibration Parameters for <u>English</u> units (depth and W in ft and flow rate in cfs)

W (ft)	L (ft)	C_f	n_f	S_t	C_s	n_s	Discharge (cfs)	
							min	max
0.167	1.50	5.796	1.98	0.553	3.978	1.45	0.004	0.24
0.333	1.50	5.895	1.97	0.651	3.315	1.58	0.008	0.50
0.500	1.50	5.956	1.95	0.734	2.782	1.67	0.013	0.77
0.667	1.50	5.999	1.94	0.798	2.409	1.73	0.018	1.04
0.333	3.00	4.212	1.84	0.580	2.826	1.38	0.009	1.40
0.667	3.00	4.287	1.83	0.674	2.400	1.49	0.018	2.86
1.000	3.00	4.330	1.81	0.754	2.048	1.57	0.029	4.33
1.333	3.00	4.361	1.80	0.815	1.796	1.64	0.040	5.82
0.500	4.50	3.764	1.72	0.614	2.440	1.34	0.016	3.78
1.000	4.50	3.830	1.71	0.708	2.081	1.49	0.034	7.65
1.500	4.50	3.869	1.69	0.788	1.785	1.50	0.053	11.5
2.000	4.50	3.897	1.68	0.849	1.569	1.54	0.074	15.4
0.667	6.00	3.534	1.66	0.629	2.264	1.30	0.024	7.43
1.333	6.00	3.596	1.64	0.723	1.940	1.39	0.050	15.0
2.000	6.00	3.633	1.63	0.801	1.676	1.45	0.080	22.5
2.667	6.00	3.662	1.61	0.862	1.478	1.50	0.111	29.9
0.833	7.50	3.400	1.61	0.641	2.159	1.28	0.032	12.4
1.667	7.50	3.459	1.60	0.735	1.855	1.37	0.068	25.0
2.500	7.50	3.494	1.58	0.811	1.610	1.42	0.108	37.3
3.333	7.50	3.519	1.57	0.873	1.417	1.47	0.151	49.4
1.000	9.00	3.340	1.58	0.651	2.104	1.27	0.042	19.0
2.000	9.00	3.398	1.57	0.743	1.815	1.35	0.088	38.0
3.000	9.00	3.442	1.55	0.820	1.580	1.40	0.139	56.9
4.000	9.00	3.458	1.54	0.882	1.390	1.44	0.194	75.0

<u>Unified Discharge Calibrations</u>

- Skogerboe also developed “unified discharge” calibrations for Cutthroat flumes, such that it is *not necessary to select from the above standard flume sizes*
- A regression analysis on the graphical results from Skogerboe yields these five calibration parameter equations:

$$C_f = 6.5851 L^{-0.3310} W^{1.025} \tag{4}$$

$$n_f = 2.0936 L^{-0.1225} - 0.128(W/L) \tag{5}$$

$$n_s = 2.003 (W/L)^{0.1318} L^{-0.07044(W/L) - 0.07131} \tag{6}$$

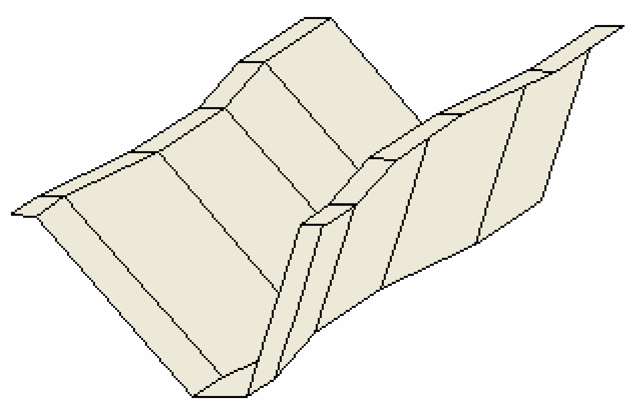
$$S_t = 0.9653(W/L)^{0.2760} L^{0.04322(W/L)^{-0.3555}} \tag{7}$$

$$C_s = \frac{C_f \left(-\log_{10} S_t\right)^{n_s}}{\left(1 - S_t\right)^{n_f}} \tag{8}$$

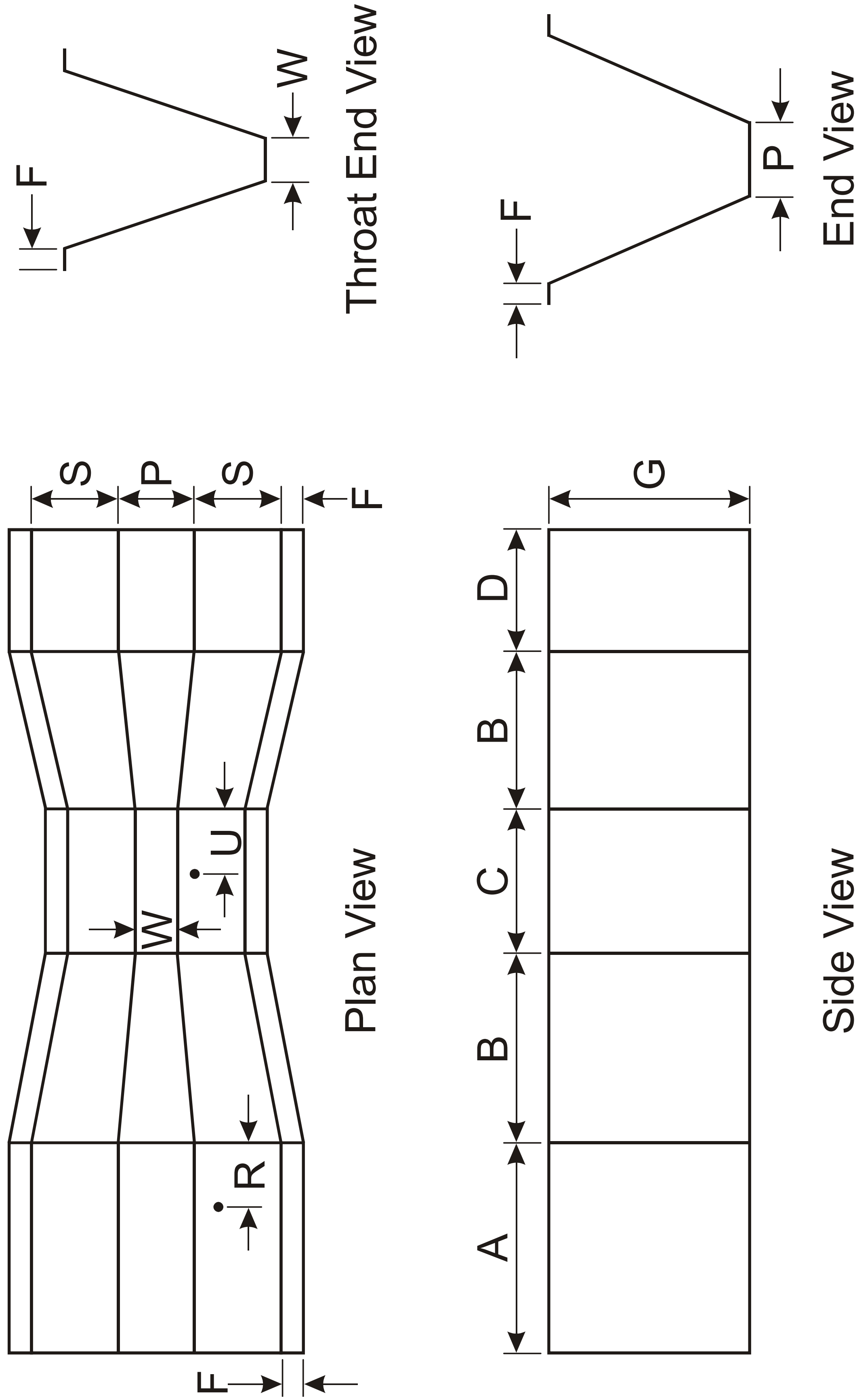
- Note that Eqs. 4-8 are for English units (L and W in ft; Q in cfs)
- The maximum percent difference in the Cutthroat flume calibration parameters is less than 2%, comparing the results of Eqs. 4-8 with the calibration parameters for the 24 standard Cutthroat flume sizes

VII. Trapezoidal Flumes

- Trapezoidal flumes are often used for small flows, such as for individual furrows in surface irrigation evaluations
- The typical standard calibrated flume is composed of five sections: approach, converging, throat, diverging, and exit
- However, the approach and exit sections are not necessary part of the flume itself



- Ideally, trapezoidal flumes can measure discharge with an accuracy of ±5% under free-flow conditions
- But the attainment of this level of accuracy depends on proper installation, accurate stage measurement, and adherence to specified tolerances in the construction of the throat section
- Discharge measurement errors are approximately 1.5 to 2.5 times the error in the stage reading for correctly installed flumes with variations in throat geometry from rectangular to triangular sections



Throat End View
End View
Plan View
Side View
F
W
P
S
U
R
A
B
C
D
G

- In the following table with seven trapezoidal flume sizes, the first two flumes are V-notch (zero base width in the throat, and the last five have trapezoidal throat cross sections

Flume Number	Description	Dimensions (inches)											
		A	B	C	D	E	F	G	P	R	S	U	W
1	Large 60°-V	7.00	6.90	7.00	3.00	7.00	1.00	6.75	2.00	1.50	4.00	3.50	0.00
2	Small 60°-V	5.00	6.05	5.00	2.00	4.25	1.00	4.00	2.00	1.00	2.38	2.50	0.00
3	2”-60° WSC	8.00	6.41	8.50	3.00	8.50	1.00	13.50	4.90	1.50	6.00	4.30	2.00
4	2”-45° WSC	8.00	8.38	8.50	3.00	8.50	1.00	10.60	4.90	1.50	10.60	4.30	2.00
5	2”-30° WSC	8.00	8.38	8.50	3.00	8.50	1.00	10.00	4.90	1.50	17.30	4.30	2.00
6	4”-60° WSC	9.00	9.81	10.00	3.00	10.00	1.00	13.90	8.00	1.50	8.00	5.00	4.00
7	2”-30° CSU	10.00	10.00	10.00	3.00	10.80	1.00	9.70	10.00	1.50	16.80	5.00	2.00

Note: All dimensions are in inches. WSC are Washington State Univ Calibrations, while CSU are Colorado State Univ Calibrations (adapted from Robinson & Chamberlain 1960)

- Trapezoidal flume calibrations are for free-flow regimes only (although it would be possible to generate submerged-flow calibrations from laboratory data)
- The following equation is used for free-flow calibration

$$Q_f = C_{ft}\left(h_u\right)^{n_{ft}} \tag{9}$$

where the calibration parameters for the above seven flume sizes are given in the table below:

Flume Number	C_{ft}	n_{ft}	Q_{max} (cfs)
1	1.55	2.58	0.35
2	1.55	2.58	0.09
3	1.99	2.04	2.53
4	3.32	2.18	2.53
5	5.92	2.28	3.91
6	2.63	1.83	3.44
7	4.80	2.26	2.97

Note: for h_u in ft and Q in cfs

V-Notch Flumes

- When the throat base width of a trapezoidal flume is zero (W = 0, usually for the smaller sizes), these are called “V-notch flumes”
- Similar to the V-notch weir, it is most commonly used for measuring water with a small head due to a more rapid change of head with change in discharge
- Flume numbers 1 and 2 above are V-notch flumes because they have W = 0

VIII. Flume Calibration Procedure

- Sometimes it is necessary to develop site-specific calibrations in the field or in the laboratory
- For example, you might need to develop a custom calibration for a "hybrid" flume, or a flume that was constructed to nonstandard dimensions
- To calibrate based on field data for flow measurement, it is desired to find flow rating conditions for both free-flow and submerged-flow
- To analyze and solve for the value of the unknown parameters in the flow rating equation the following procedure applies:

1. Transform the exponential equation into a linear equation using logarithms
2. The slope and intersection of this line can be obtained by fitting the transformed data using linear regression, or graphically with log-log paper
3. Finally, back-calculate to solve for the required unknown values

The linear equation is:

$$Y = a + bX \tag{10}$$

The transformed flume equations are:

<u>Free-flow:</u>

$$\log(Q_f) = \log\left(C_f W\right) + n_f \log(h_u) \tag{11}$$

So, applying Eq. 10 with measured pairs of Q_f and h_u, "a" is log C_f and "b" is n_f

<u>Submerged-flow:</u>

$$\log\left[\frac{Q_s}{(h_u - h_d)^{n_f}}\right] = \log(C_s W) - n_s \log\left[-(\log S)\right] \tag{12}$$

Again, applying Eq. 10 with measured pairs of Q_s and h_u and h_d, "a" is log C_s and "b" is n_s

- Straight lines can be plotted to show the relationship between log h_u and log Q_f for a free-flow rating, and between log (h_u-h_d) and log Q_S with several degrees of submergence for a submerged-flow rating
- If this is done using field or laboratory data, any base logarithm can be used, but the base must be specified
- Multiple linear regression can also be used to determine C_s, n_f, and n_s for submerged flow data only – this is discussed further in a later lecture

IX. Sample Flume Calibrations

Free Flow

- Laboratory data for free-flow conditions in a flume are shown in the following table
- Free-flow conditions were determined for these data because a hydraulic jump was seen downstream of the throat section, indicating supercritical flow in the vicinity of the throat

Q (cfs)	hu (ft)
4.746	1.087
3.978	0.985
3.978	0.985
2.737	0.799
2.737	0.798
2.211	0.707
1.434	0.533
1.019	0.436
1.019	0.436
1.019	0.436
1.019	0.436
0.678	0.337

- Take the logarithm of Q and of h_u, then perform a linear regression (see Eqs. 10 and 11)
- The linear regression gives an R^2 value of 0.999 for the following calibration equation:

$$Q_f = 4.04 h_u^{1.66} \tag{13}$$

where Q_f is in cfs; and h_u is in ft

- We could modify Eq. 13 to fit the form of Eq. 6, but for a custom flume calibration it is convenient to just include the throat width, W, in the coefficient, as shown in Eq. 13
- Note that the coefficient and exponent values in Eq. 13 have been rounded to three significant digits each – never show more precision than you can justify

Submerged Flow

- Data were then collected under submerged-flow conditions in the same flume
- The existence of submerged flow in the flume was verified by noting that there is not downstream hydraulic jump, and that any slight change in downstream depth produces a change in the upstream depth, for a constant flow rate
- Note that a constant flow rate for varying depths can usually only be obtained in a hydraulics laboratory, or in the field where there is an upstream pump, with an unsubmerged outlet, delivering water to the channel

- Groups of (essentially) constant flow rate data were taken, varying a downstream gate to change the submergence values, as shown in the table below

Q (cfs)	hu (ft)	hd (ft)
3.978	0.988	0.639
3.978	1.003	0.753
3.978	1.012	0.785
3.978	1.017	0.825
3.978	1.024	0.852
3.978	1.035	0.872
3.978	1.043	0.898
3.978	1.055	0.933
3.978	1.066	0.952
3.978	1.080	0.975
3.978	1.100	1.002
3.978	1.124	1.045
2.737	0.800	0.560
2.736	0.801	0.581
2.734	0.805	0.623
2.734	0.812	0.659
2.733	0.803	0.609
2.733	0.808	0.642
2.733	0.818	0.683
2.733	0.827	0.714
2.733	0.840	0.743
2.733	0.858	0.785
2.733	0.880	0.823
2.733	0.916	0.876
2.733	0.972	0.943
1.019	0.437	0.388
1.019	0.441	0.403
1.010	0.445	0.418
1.008	0.461	0.434
1.006	0.483	0.462
1.006	0.520	0.506

- In this case, we will use n_f in the submerged-flow equation (see Eq. 12), where n_f = 1.66, as determined above
- Perform a linear regression for $\ln[Q/(h_u - h_d)^{1.66}]$ and $\ln[-\log_{10}S]$, as shown in Eq. 12, giving an R^2 of 0.998 for

$$Q_s = \frac{1.93\left(h_u - h_d\right)^{1.66}}{\left(-\log_{10} S\right)^{1.45}} \tag{14}$$

where Q_s is in cfs; and h_u and h_d are in ft

- You should verify the above results in a spreadsheet application

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Lecture 3
Current Metering

> "Six hours the waters run in, and six hours they run out, and the reason is this: when there is higher water in the sea than in the river, they run in until the river gets to be highest, and then it runs out again"
>
> The Last of the Mohicans, J.F. Cooper (1826)

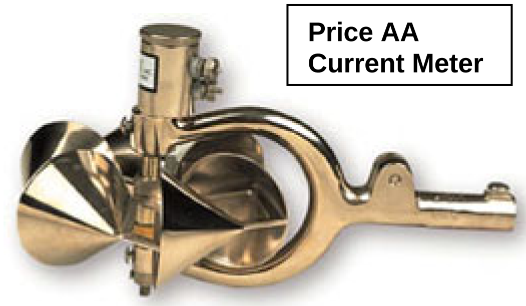
I. Introduction

- Current metering in open channels is both a science and an art
- One cannot learn how to do current metering well by only reading a book
- This is a *velocity-area* flow measurement method for open-channel flow
- A current meter is used to measure the velocity at several points in a cross section
- The velocities are multiplied by respective subsection areas to obtain flow rates

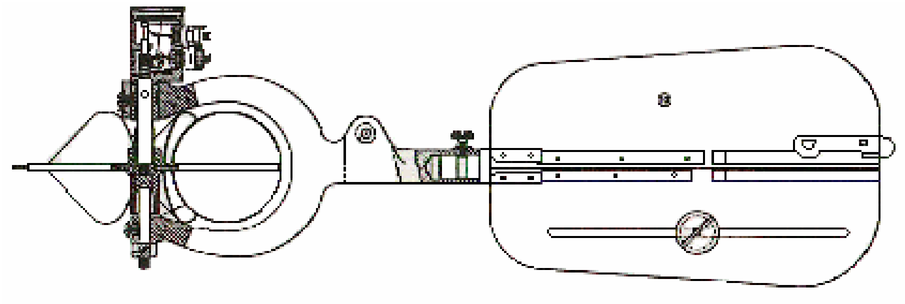

Current metering in an irrigation canal

II. Types of Current Meters

- There are many companies that manufacture good quality current meters, and there are many types of current meters, including mechanical and electromagnetic versions
- Current meters with a rotating unit that senses the water velocity are either vertical-shaft or horizontal-shaft types
- The vertical-axis current meter has a rotating cup with a bearing system that is simpler in design, more rugged, and easier to service and maintain than horizontal-shaft (axis) current meters
- Because of the bearing system, the vertical-shaft meters will operate at lower velocities than horizontal-axis current meters
- The bearings are well protected from silty water, the adjustment is usually less sensitive, and the calibration at lower velocities is more stable
- Two of the commonly used vertical-axis current meters are the Price Type A (or AA) current meter and the Price Pygmy current meter, the latter intended for use with shallow flow depths and relatively low velocities (less than 0.5 fps, or 0.15 m/s)

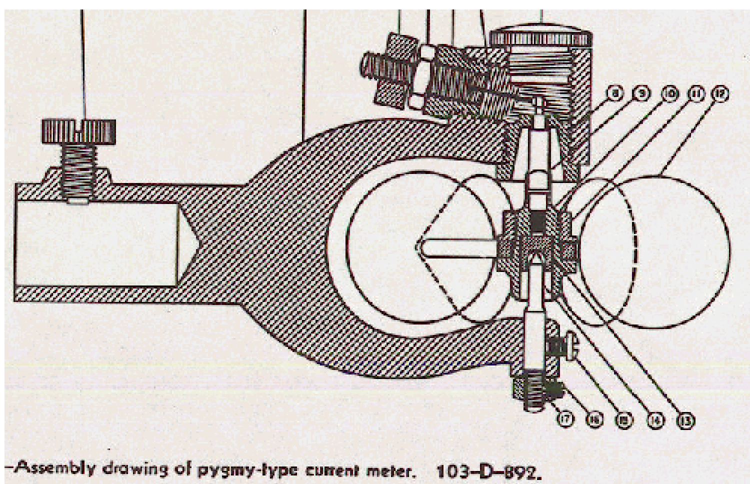

Price AA Current Meter

- The commonly-used Price AA current meter can measure velocities up to about 8 fps (2.4 m/s)
- None of the Price current meters can accurately measure velocities less than about 0.2 fps (0.07 m/s)



Price Type AA Current Meter

- The horizontal-shaft current meters use a propeller
- These horizontal-axis rotors disturb the flow less than vertical-axis cup rotors because of axial symmetry in the flow direction
- Also, the horizontal-shaft current meters are less sensitive to vertical velocity components in the channel



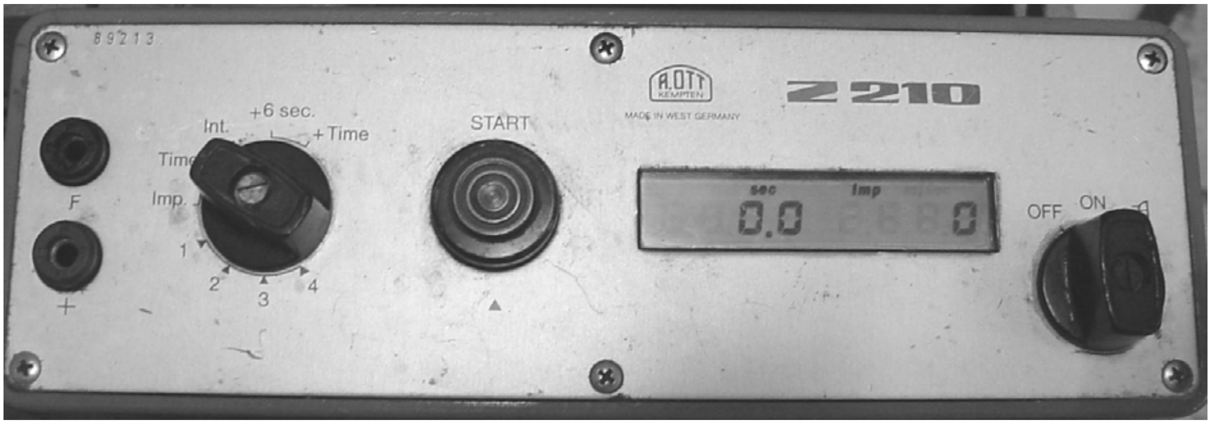
Price Pygmy Current Meter

- Because of its shape, the horizontal-axis current meter is less susceptible to becoming fouled by small debris and vegetative material moving with the water
- Some common horizontal-axis current meters are the Ott (German), the Neyrpic (France) and the Hoff (USA)

Ott Current Meters

- Some recent models have proven to be both accurate and durable when used in irrigation channels
- Electromagnetic current meters are available that contain a sensor with the point velocity displayed digitally
- Some earlier models manifested considerable electronic noise under turbulent flow conditions (even the latest models still have problems if near steel-reinforced concrete infrastructure, such as bridge piers)
- Present models yield more stable velocity readings, with averaging algorithms
- However, recent lab tests have shown that the Price current meters are more accurate than at least two types of electromagnetic meter throughout a range of velocities, and significantly more accurate at low velocities (J.M. Fulford 2002)

- Mechanical current meters automatically integrate and average velocities when rotations per specific time interval are counted (e.g. count the rotations of the meter in a 30-s or 60-s interval)
- The photograph below shows a digital display from an Ott current meter. The number on the left is the elapsed time in seconds (to tenths of a second), and the number on the right is the number of revolutions of the propeller



- With a traditional current meter, you use an electronic counter as shown above, earphones that emit a beep for each rotation of the propeller, or other device, using a stopwatch if necessary
- Then you divide the number of revolutions by the elapsed seconds to get a value in revolutions per second
- The longer the duration (elapsed time) per measurement, the greater the integration effect as the propeller speeds up and slows down with fluctuations in the flow (unless the flow is perfectly stable at the location)
- The revolutions per second are directly proportional to the velocity of the flow, according to the calibration of the instrument (see below)

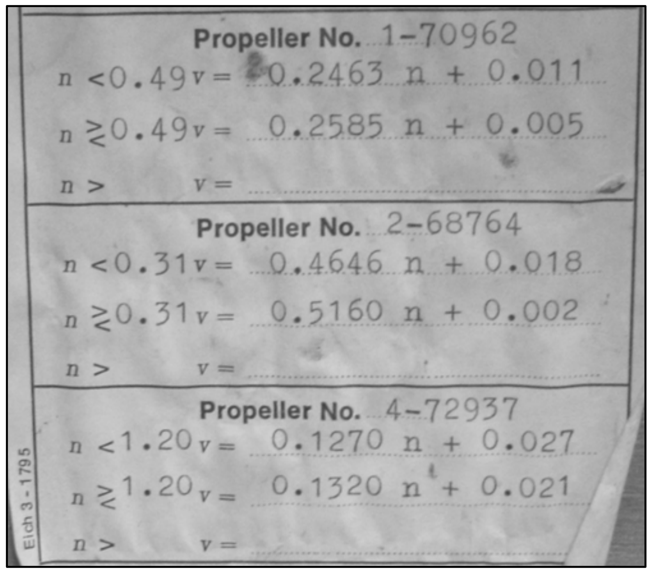
III. Care of the Equipment

- Accuracy in velocity measurements can only be expected when the equipment is properly assembled, adjusted, and maintained
- The current meter should be treated as a delicate instrument that needs meticulous care and protective custody, both when being used and when being transported
- The current meter necessarily receives a certain amount of hard usage that may result in damage, such as a broken pivot, chipped bearing, or bent shaft that will result in the current meter giving velocity readings that are lower than actual velocities
- Measurements near bridge piers and abutments, water depth readings taken at cross-sections having irregular bed profiles with the current meter attached to the measuring line, and floating debris, represent the greatest hazards to the equipment (Corbett, et al. 1943)
- Damage to current meter equipment during transport is generally due to careless packing or negligence
- A standard case is provided by all manufacturers of current meter equipment, which should be used before and after taking discharge measurements
- The equipment case should always be used when transporting the current meter, even when the distance is relatively short
- Transport of assembled equipment from one location to another is one of the most common sources of damage

IV. Spin Test

- A “spin test” can be performed on a current meter to determine whether it is spinning freely or not; this is done while the current meter is out of the water
- For a Price-type current meter, the USBR (1981) recommends putting the shaft in a vertical position and giving the cups a quick turn by hand
- Ideally, the cups should spin for at least 3 minutes, but if it is only about 1½ minutes the current meter can still be used, provided the velocity is not very low
- The cups should come to a smooth and gradual stop

V. Current Meter Ratings



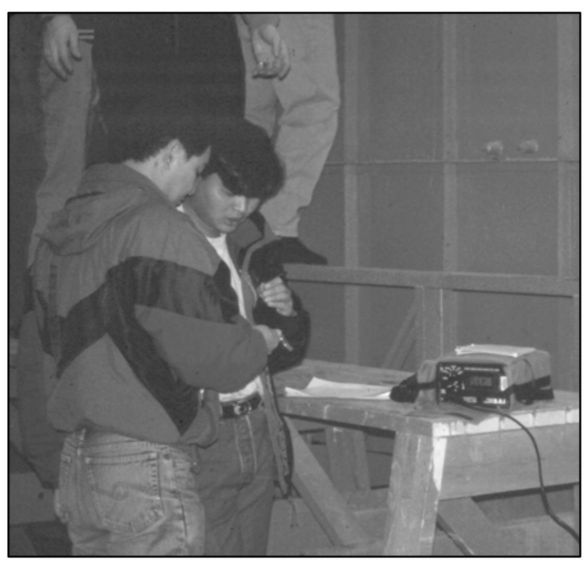
- Usually, a current meter is calibrated in a towing tank, which is a small, straight open channel with stagnant water
- The current meter is attached to a carriage that travels on rails (tracks) placed on the top of the towing tank
- Then, a series of trials are conducted wherein the current meter is towed at different constant velocities
- For each trial, the constant velocity of the carriage is recorded, as well as the revolutions per second (rev/s) of the current meter
- These data are plotted on rectangular coordinates to verify that a straight-line relationship exists; then, the equation is determined by regression analysis
- The table below is an example of a velocity rating based on the rating equation for a current meter:

$$\text{Velocity (m/s)} = 0.665\ \text{(rev/s)} + 0.009 \qquad (1)$$

Sample Velocity Rating for a Current Meter, with Velocity in m/s

Time (seconds)	REVOLUTIONS									
	5	10	15	20	25	30	40	50	60	80	100
40	0.092	0.175	0.258	0.342	0.425	0.508	0.674	0.840	1.007	1.339	1.672
41	0.090	0.171	0.252	0.333	0.415	0.496	0.658	0.820	0.982	1.307	1.631
42	0.088	0.167	0.247	0.326	0.405	0.484	0.642	0.801	0.959	1.276	1.592
43	0.086	0.164	0.241	0.318	0.396	0.473	0.628	0.782	0.937	1.246	1.556
44	0.085	0.160	0.236	0.311	0.387	0.462	0.614	0.765	0.916	1.218	1.520
45	0.083	0.157	0.231	0.305	0.378	0.452	0.600	0.748	0.896	1.191	1.487
46	0.081	0.154	0.226	0.298	0.370	0.443	0.587	0.732	0.876	1.166	1.455
47	0.080	0.151	0.221	0.292	0.363	0.434	0.575	0.716	0.858	1.141	1.424
48	0.078	0.148	0.217	0.286	0.355	0.425	0.563	0.702	0.840	1.117	1.394
49	0.077	0.145	0.213	0.280	0.348	0.416	0.552	0.688	0.823	1.095	1.366
50	0.076	0.142	0.209	0.275	0.342	0.408	0.541	0.674	0.807	1.073	1.339
51	0.074	0.139	0.205	0.270	0.335	0.400	0.531	0.661	0.791	1.052	1.313
52	0.073	0.137	0.201	0.265	0.329	0.393	0.521	0.648	0.776	1.032	1.288
53	0.072	0.135	0.197	0.260	0.323	0.385	0.511	0.636	0.762	1.013	1.264
54	0.071	0.132	0.194	0.255	0.317	0.378	0.502	0.625	0.748	0.994	1.241
55	0.070	0.130	0.190	0.251	0.311	0.372	0.493	0.614	0.735	0.976	1.218
56	0.068	0.128	0.187	0.247	0.306	0.365	0.484	0.603	0.722	0.959	1.197
57	0.067	0.126	0.184	0.242	0.301	0.359	0.476	0.592	0.709	0.942	1.176
58	0.066	0.124	0.181	0.238	0.296	0.353	0.468	0.582	0.697	0.926	1.156
59	0.065	0.122	0.178	0.234	0.291	0.347	0.460	0.573	0.685	0.911	1.136
60	0.064	0.120	0.175	0.231	0.286	0.342	0.452	0.563	0.674	0.896	1.117

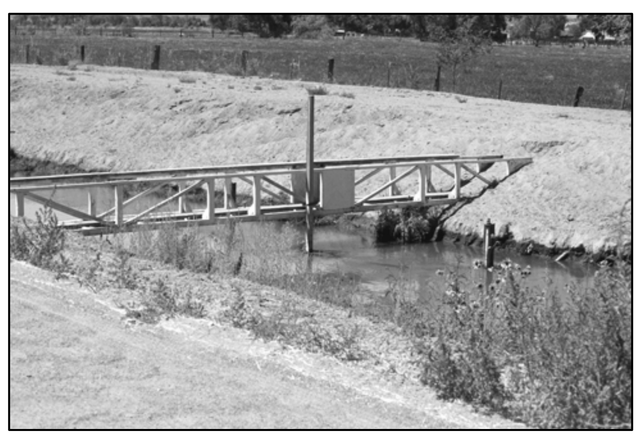
- Of course, if you have a calculator or spreadsheet software, it may be preferable to use the equation directly rather than interpolating in a table, which is based on the equation anyway
- The calibration equation is always linear, where the constant term is the threshold flow velocity at which the current meter just begins to rotate; thus, current meters have limits on the velocities that can be measured
- Laboratory nozzles with uniform velocity distributions at a circular cross section are sometimes used (instead of a towing tank) to calibrate current meters, as in the Utah Water Research Lab



VI. Methods Of Employing Current Meters

Wading

- The wading method involves having the hydrographer stand in the water holding a wading rod with the current meter attached to the rod
- The wading rod is graduated so that the water depth can be measured. The rod has a metal foot pad which sets on the channel bed
- The current meter can be placed at any height on the wading rod and is readily adjusted to another height by the hydrographer while standing in the water
- A tag line is stretched from one bank to the other, which can be a cloth or metal tape
- This tag line is placed perpendicular to the flow direction
- The zero length on the tag line does not have to correspond with the edge of the water on one of the banks
- This tag line is used to define the location of the wading rod each time that a current meter measurement is made (*recheck measurements each time, and check units*)
- The wading rod is held at the tag line
- The hydrographer stands sideways to the flow direction, facing toward one of the banks
- The hydrographer stands 5-10 cm downstream from the tag line and approximately 50 cm to one side of the wading rod
- During the measurement, the rod needs to be held in a vertical position and the current meter must be parallel with the flow direction
- An assistant can signal to the hydrographer whether or not the rod is vertical in relation to the flow direction
- If the flow velocity at the bank is not zero, then this velocity should be estimated as a percentage of the velocity at the nearest measuring point (vertical)
- Thus, the nearest measuring point should be as close to the bank as possible in order to minimize the error in the calculated discharge for the section adjacent to the bank



Bridge

- Many of the larger irrigation channels have bridges at various locations, such as headworks and cross regulators, but they may not

be located at an appropriate section for current meter measurements

- However, culverts often prove to be very good locations, with current meter measurements usually being made on the downstream end of the culvert where parallel streamlines are more likely to occur
- Bridges often have piers, which tend to collect debris on the upstream face, that should be removed prior to undertaking current meter measurements
- Either a hand line or a reel assembly may be used from a bridge
- In either case, a weight is placed at the bottom of the line, which sets on the channel bed in order that the line does not move as a result of the water flow
- The current meter is then placed at whatever location is required for each measurement
- For a hand line assembly, the weight is lowered from the bridge to the channel bed and the reading on the graduated hand line is recorded; then, the weight is lifted until it is setting on the water surface and the difference in the two readings on the hand line is recorded as the water depth
- Afterwards, the current meter is placed at the appropriate location on the hand line in order to make the velocity measurement
- If a weight heavier than 10-15 kg is required in order to have a stable, nearly vertical, cable line, then a crane-and-reel assembly is used
- The reel is mounted on a crane designed to clear the handrail of the bridge and to guide the meter cable line beyond any interference with bridge members
- The crane is attached to a movable base for convenience in transferring the equipment from one measuring point (vertical) to another

Cableway

- For very wide canals, or rivers, with water depths exceeding 150 cm, a cable is placed above the water with vertical supports on each bank that are heavily anchored for stability
- The cable supports a car (box) that travels underneath the cable using pulleys. This car carries the hydrographer and the current meter equipment
- The cable has markers so that the location across the channel is known
- A hand line or a cable reel assembly is used depending on the size of the weight that must be used

Boat

- For some very wide channels, such as those often encountered in the Indian subcontinent (and many other places), the installation of a cableway is a significant expense
- Consequently, a boat is commonly employed instead of the cableway
- Some friends on the banks should help hold the boat in place with ropes while the velocity measurements are taken
- Either a hand line or a cable reel assembly is used in this case
- This method is not as convenient as the wading method, and it takes longer to make measurements, but it is sometimes the best alternative

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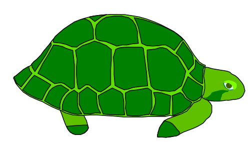
Lecture 4

Current Metering

I. Velocity Measurement Techniques

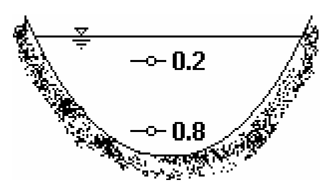
Vertical Velocity Method

- The most complete method for establishing the mean velocity at a vertical section is to take a series of current meter velocity measurements at various depths in the vertical
- Often, the current meter is placed below the water surface at one-tenth of the water depth and a velocity measurement is made, then the current meter is placed at two-tenths of the water depth; this procedure is continued until the velocity has finally been measured at nine-tenths below the water surface
- Of particular importance are the velocity measurements at relative water depths of 0.2, 0.6 and 0.8 because they are used in the simpler methods
- When the above field procedure has been completed for a number of verticals in the cross section, the data are plotted
- The relative water depth, which varies from zero at the water surface to unity at the channel bed, is plotted on the ordinate starting with zero at the top of the ordinate scale and unity at the bottom of the ordinate scale
- Velocity is plotted on the abscissa
- A smooth curve can be fitted on the data points for each vertical, from which the mean velocity for the vertical can be determined
- Also, the relative water depth(s) corresponding with the mean velocity on the velocity profile can be compared between each vertical
- Because the field procedure and data analysis for this method are time consuming, simpler methods are commonly used
- Some of the more common methods are described in the following sections
- However, the vertical velocity method provides an opportunity to determine whether or not the simpler procedures are valid, or if some adjustments are required



Two-Point Method

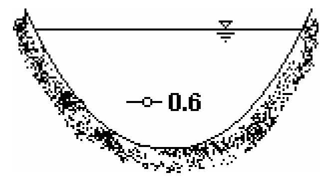
- The most common methodology for establishing the mean velocity in a vertical is the Two-Point Method
- Based on many decades of experience, a current meter measurement is made at two relative water depths -- 0.2 and 0.8
- The average of the two measurements is taken as the mean velocity in the vertical



$$\overline{V} = \frac{V_{0.2} + V_{0.8}}{2} \tag{1}$$

- In some field cases the velocity profile is distorted
- For example, measurements taken downstream from a structure may have very high velocities near the water surface that can be visually observed, or near the channel bed which can be sensed by the hydrographer when using the wading method
- If there is any suspicion that an unusual velocity profile might exist in the cross section, the vertical velocity method can be used to establish a procedure for determining the mean velocity in a vertical for that cross section

Six-Tenths Method



- For shallow water depths, say less than 75 cm, the Six-Tenths Method is used
- However, shallow is a relative term that is dependent on the type (size) of current meter being used
- A single current meter measurement is taken at a relative water depth of 0.6 below the water surface and the resulting velocity is used as the mean velocity in the vertical
- In irrigation canals, this method is commonly used at the first vertical from each bank, while the two points method is used at all of the other verticals in the cross-section
- Frequently, the first vertical from each bank has a low velocity so the discharge in each section adjacent to the left and right (looking downstream) banks represents a very small portion of the total discharge in the cross-section
- In situations where shallow flow depths exist across most of the cross section, and the six-tenths method must be used because of the type of current meter that is available, then it is likely there will be considerable error in the velocity measurement, perhaps more than ten percent

Three-Point Method

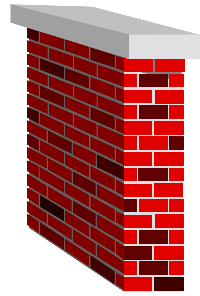
- This is a combination of the previous two methods
- The mean velocity is calculated as the sum of the results from the previous two methods, divided by two:

$$\overline{V} = \frac{1}{2}\left(\frac{V_{0.2} + V_{0.8}}{2} + V_{0.6}\right) \tag{2}$$

Integration Method

- In this approach, experienced hydrographers can slowly lower and raise the current meter two or three times along a vertical line in the stream
- The resulting "integrated" velocity along the vertical is then used to determine the flow rate in a cross-section
- This method is <u>subject to large errors</u>, however, and should only be used for quick checks

II. Velocity at Vertical Walls



- Vertical walls are frequently encountered in irrigation systems
- Usually, this occurs in rectangular channels lined with concrete or brick-and-mortar
- Even earthen canals will likely have some structures with a rectangular cross section
- In some cases, there may be a vertical retaining wall along only one side of the canal to stabilize the embankment

- In such cases, visual observation will usually disclose that the velocity very near the vertical wall is significantly greater than zero
- These data are given in the table below
- The mathematical relationship between the parameters is:

$$\frac{\bar{V}_w}{\bar{V}_x} = \frac{\bar{V}_w / \bar{V}_D}{\bar{V}_x / \bar{V}_D} \tag{3}$$

and,

$$\bar{V}_w = \bar{V}_x \left(\frac{\bar{V}_w / \bar{V}_D}{\bar{V}_x / \bar{V}_D} \right) = \frac{0.65\ \bar{V}_x}{\bar{V}_x / \bar{V}_D} \tag{4}$$

where,

D =	depth of flow measured at the vertical wall;
x =	horizontal distance from the wall toward the center of the channel (x equals zero at the wall);
$\bar{V}_D$ =	mean velocity in the vertical at x = D;
$\bar{V}_x$ =	mean velocity measured in the vertical at a horizontal distance x from the wall ($x \leq D$);
$\bar{V}_w$ =	calculated mean velocity in the vertical at the wall, where x = 0;
$\bar{V}_x / \bar{V}_D$ =	relative mean velocity in the vertical at a horizontal distance x from the wall;
$\bar{V}_w / \bar{V}_D$ =	relative mean velocity in the vertical at the wall, where x = 0

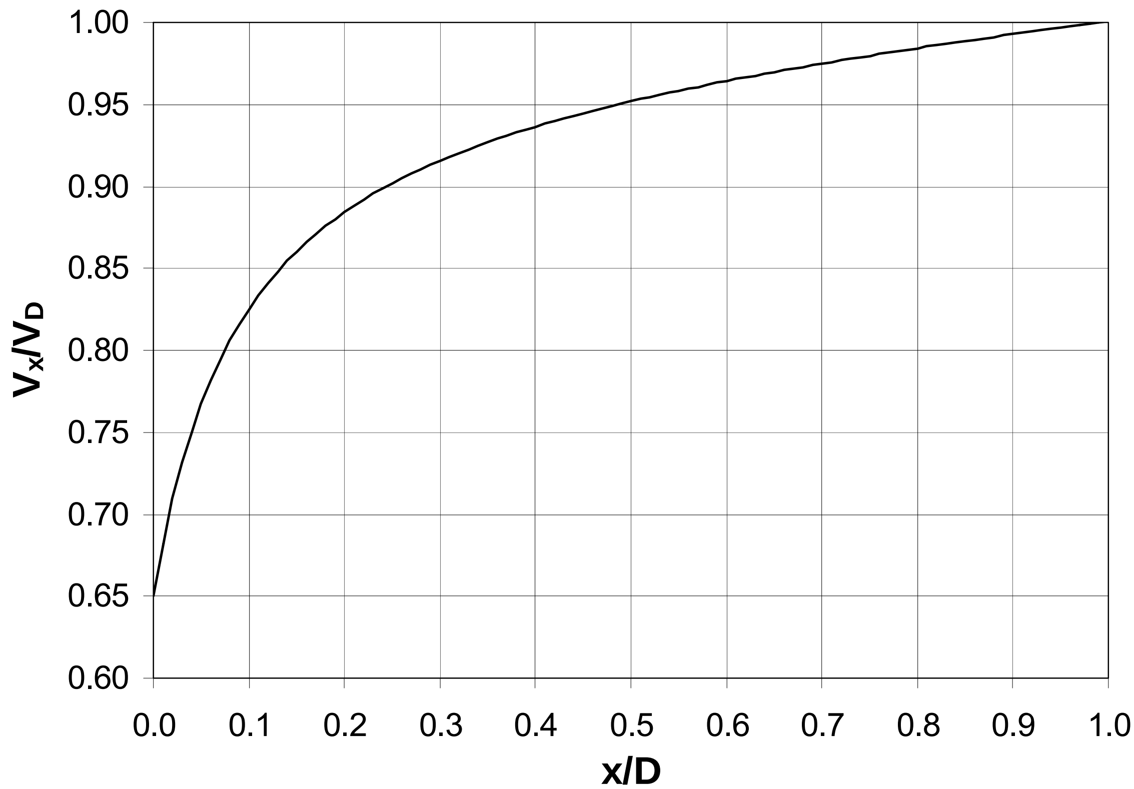
- The following table gives some values for the relationship between x/D and V_x/V_D:

x/D	$\bar{V}_x / \bar{V}_D$
0.0	0.650
0.1	0.825
0.2	0.884
0.3	0.916
0.4	0.936
0.5	0.952
0.6	0.964
0.7	0.975
0.8	0.984
0.9	0.993
1.0	1.000

- When applying this procedure in a spreadsheet or other computer application, you can use the following equation to accurately define the same relationship:

$$\frac{\bar{V}_x}{\bar{V}_D} = \frac{0.65 + 10.52(x/D)}{1 + 10.676(x/D) - 0.51431(x/D)^2} \tag{5}$$

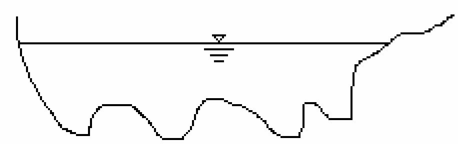
- The figure below is a graphical representation of Eq. 5



- The ratio $\bar{V}_x / \bar{V}_D$ is obtained from the above table after having measured $\bar{V}_x$ at a horizontal distance x from the wall
- The accuracy of the estimated mean velocity at the wall will be enhanced by measuring the mean velocity in a vertical located as close to the vertical wall as the current meter equipment will allow
- Thus, if a current meter measurement could be made at a distance D/4 from the wall, then the estimated mean velocity at the vertical wall would be the mean velocity measured at D/4 from the wall multiplied by the ratio 0.65/0.90, where the 0.90 value is obtained from interpolating in the table
- In this example, the relative horizontal distance from the wall is x/D = (D/4)/D = 0.25
- Note that if x/D = 0, $\bar{V}_x / \bar{V}_D$ = 0.65, giving $\bar{V}_w = \bar{V}_x$ (which is logical because the measurement is at the wall)
- Note also that x should be less than D
- Special current meters exist for measuring velocities very close to vertical walls, but they are expensive and not very common

III. Selection of Measuring Cross Section

A poor cross section for current metering



Another poor cross section

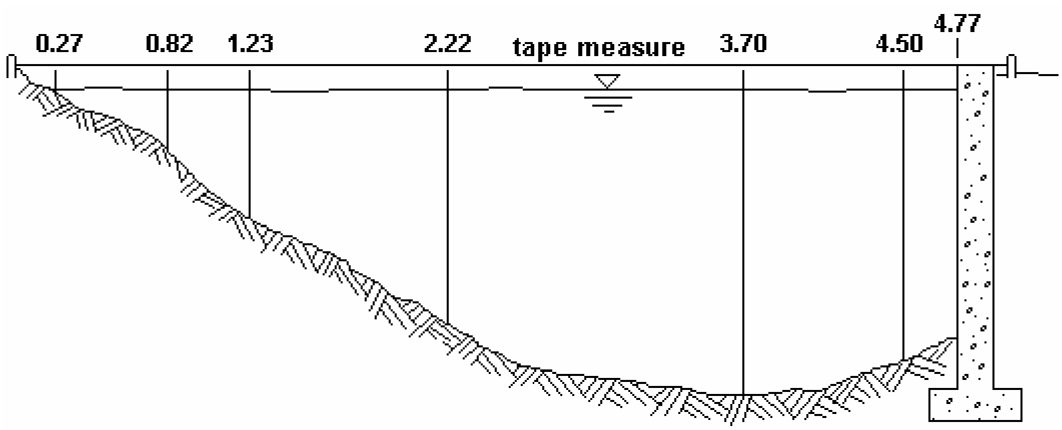
- The most commonly used criterion in selecting a channel cross section for current meter measurements is that it be located in a straight channel
- Cross sections having large eddies and excessive turbulence are to be avoided
- A cross section with stagnant water near one of the banks should be avoided
- Avoid cross sections where the flow depths are shallow (except near the banks) and the flow velocities are too low
- Rantz (1982) recommends that the flow depths should exceed 15 cm and the flow velocities should exceed 15 cm/s
- It is preferable to select a cross section with little or no aquatic growth that can cause problems with the rotation of the current meter – but this is true for electromagnetic current meters too, although to a lesser extent, because vegetation in the canal tends to cause velocity fluctuations
- A cross-section is preferred where the channel bed is not highly irregular so that the area of the cross section can be accurately determined
- An irregular channel bed will affect the velocity profiles

IV. Subdivision of a Cross Section into Verticals

- The current meter is used to measure the mean velocity of each vertical in the cross section
- In addition, the spacing of the verticals is used in determining the cross-sectional area of each section, where a section is defined as the cross-sectional area of flow between two verticals.

- In natural channels, the measuring cross-section should be subdivided into twenty (20) or more verticals for a relatively smooth channel bed; but for lined canals, twenty verticals is usually excessive and unnecessary
- For an irregular channel bed, more verticals are needed, not only to better define the cross-sectional area of flow, but also because an irregular bed causes more variation in the velocity distribution
- Verticals do not need to be spaced closer than 0.3 m across the width of the channel (Corbett et al. 1943), but for small canals the verticals can be closer than that, especially when there are only 3 or 4 verticals across the section
- For concrete-lined trapezoidal cross-section canals of small and medium size, it is typical to take verticals at the mid-points of the side slopes on each side, and at the two vertices where the side slopes meet the canal bottom, then dividing the base width into 3 to 5 equally-spaced verticals

- An example earthen canal cross-section is illustrated in the figure below



- The most important verticals for defining the cross-sectional area of flow are shown in this figure (for this example)
- The data from the above figure will be used below in sample calculations

V. Measuring Water Depths

- The water depth must be known at each vertical in order to calculate the cross-sectional area of flow for two sections, one on each side of the vertical
- Accurately determining the flow areas is just as important as accurate velocity measurements
- The greatest sources of error in measuring the depth of water are:

  1. an irregular channel bed
  2. a channel bed that is soft so that a weight or a rod sinks into the material, indicating a water depth greater than actually exists
  3. human error (simple mistakes… not paying attention to detail?)

- Another source of error: water “piles up” on the upstream edge of the rod and is lower on the downstream edge, requiring the hydrographer to sight across the rod, looking both upstream and downstream to get a reading

VI. Recording of Data

- There are various formats for recording current metering data, and various computational procedures (all of which are similar)
- These days, it is usually convenient to transfer the data to a spreadsheet application and do the computations therein

Date: ________________ **Channel:** ____________________________ **Station:** _______________________

distance from start point	depth	observed depth	revolu-tions	time	velocity			mean depth	width	area	flow rate
					at point	mean in vertical	mean in section				

Hydrographers: ____

No. _____ **of** _____ **page(s)** **Computations** ____

Checked by: ____

VII. Computational Procedure

- The computational procedure for an example current meter discharge measurement is given in the table below (see the figure above also)
- The major verticals had readings of 0.82, 1.23, 2.22, 3.70 and 4.50 m along the tag line
- Intermediate verticals were selected as listed in the table
- The water surface is contained between 0.27 and 4.77 m along the tag line
- The first cross-section is contained between 0.27 and 0.82 m along the tag line
- The velocity at the bank is roughly estimated to be 10% of the mean velocity in the vertical at 0.82 m along the tag line (the velocity at the bank is often listed as zero)

- Because of the shallow water depth at 0.82 m, the six-tenths method was used in making the current meter measurement, which resulted in a velocity of 0.208 m/s
- The discharge in this cross section is less than 0.5% of the total discharge

- For the last cross-section, which contains a vertical wall, a set of current meter measurements were made at 4.50 m along the tag line, with the mean velocity in the vertical being 0.553 m/s
- The distance, x, from the vertical wall was 4.77 - 4.50 = 0.27 m
- The depth of water, D, at the vertical wall was 0.92 m
- Thus, x/D is 0.27 /0.92 = 0.29

Current Metering Data

Location: __________ **Date:** __________ **Hour:** __________

Data Collector(s): __________

distance from start (m)	depth (m)	fraction of depth	velocity (m/s)			average depth (m)	width (m)	area (m^2)	flowrate (m^3/s)
			from point	average of the points	average of the section				

- The relative mean velocity in the vertical is 0.91, whereas the relative mean velocity at the wall is 0.65
- The total flow rate in the cross section is estimated to be 1.831 m^3/s (see the following table), but this can be rounded to 1.83 m^3/s because it is almost certain that the accuracy is less than four significant digits

distance from edge (m)	depth (m)	depth fraction	revolutions	time (s)	velocity (m/s) at point	velocity (m/s) mean in vertical	velocity (m/s) mean in section	mean depth (m)	width (m)	area (m2)	flow rate (m3/s)
0.27	0.00	--	--	--	10%	0.0208					
							0.1144	0.135	0.55	0.0743	0.008
0.82	0.27	0.6	20	67	0.208	0.2080					
							0.2245	0.480	0.41	0.1968	0.044
1.23	0.69	0.2	20	54	0.255	0.2410					
		0.8	20	61	0.227		0.2580	0.775	0.32	0.2480	0.064
1.55	0.86	0.2	25	58	0.296	0.2750					
		0.8	25	68	0.254		0.3020	0.905	0.35	0.3168	0.096
1.90	0.95	0.2	25	50	0.342	0.3290					
		0.8	30	65	0.316		0.3583	0.965	0.32	0.3088	0.111
2.22	0.98	0.2	30	49	0.416	0.3875					
		0.8	30	57	0.359		0.4310	1.000	0.28	0.2800	0.121
2.50	1.02	0.2	40	53	0.511	0.4745					
		0.8	40	62	0.438		0.4903	1.050	0.30	0.3150	0.154
2.80	1.08	0.2	40	49	0.552	0.5060					
		0.8	40	59	0.46		0.5310	1.105	0.30	0.3315	0.176
3.10	1.13	0.2	40	43	0.628	0.5560					
		0.8	40	56	0.484		0.5683	1.155	0.30	0.3465	0.197
3.40	1.18	0.2	50	52	0.648	0.5805					
		0.8	50	66	0.513		0.6008	1.200	0.30	0.3600	0.216
3.70	1.22	0.2	50	49	0.688	0.6210					
		0.8	50	61	0.554		0.6120	1.175	0.30	0.3525	0.216
4.00	1.13	0.2	50	51	0.661	0.6030					
		0.8	50	62	0.545		0.5893	1.105	0.25	0.2763	0.163
4.25	1.08	0.2	50	55	0.614	0.5755					
		0.8	50	63	0.537		0.5643	1.025	0.25	0.2563	0.145
4.50	0.97	0.2	40	47	0.575	0.5530					
		0.8	40	51	0.531		0.4740	0.945	0.27	0.2552	0.121
4.77	0.92	At vertical wall: 0.553 (0.65/0.91) =				0.3950		**Totals:**	**4.50**	**3.9178**	**1.831**

- The two tables below give sample current metering data and flow rate calculations (one in Spanish, the other in English)

Aforo con Molinete en Sistema de Riego Chacuey
Ubicación: Inicio del canal principal — **Método: Tres Puntos**
15 agosto 01 08h30 — **Sección: Rectangular, de concreto**

Distancia desde el inicio (m)	Profundidad (m)	Fracción de Profundidad	Velocidad (m/s) del punto	promedio de los puntos	promedio de subsección	Profundidad Promedio (m)	Ancho (m)	Area (m2)	Caudal (m3/s)
derecha	pared vertical: (0.93)(0.65)/0.893=								
0.00	0.88		0.68	0.68					
					0.80	0.88	0.20	0.176	0.141
0.20	0.88	0.2	1.00						
		0.6	0.92	0.93					
		0.8	0.87		0.93	0.88	0.20	0.176	0.163
0.40	0.88	0.2	1.01						
		0.6	0.92	0.92					
		0.8	0.84		0.93	0.88	0.20	0.176	0.163
0.60	0.88	0.2	0.99						
		0.6	0.94	0.93					
		0.8	0.84		0.94	0.88	0.20	0.176	0.165
0.80	0.88	0.2	1.01						
		0.6	0.95	0.94					
		0.8	0.86		0.90	0.88	0.20	0.176	0.158
1.00	0.88	0.2	0.89						
		0.6	0.87	0.85					
		0.8	0.76		0.73	0.88	0.18	0.158	0.116
1.18	0.88		0.62	0.62					
izquierda	pared vertical: (0.85)(0.65)/0.886=								
					Totales:		**1.18**	**1.038**	**0.905**

Logan & Northern Canal Company
Location: Upstream of Parshall flume — **Method: Two-point & six-tenths**
09 Aug 01 10h30 — **Section: Earthen**

Distance from edge (m)	Depth (m)	Depth Fraction	Velocity (m/s) point	avg of points	avg of subsection	Average Depth (m)	Width (m)	Area (m2)	Flow Rate (m3/s)
0.000	0.000		0.02	0.02					
					0.10	0.200	0.61	0.122	0.012
0.610	0.400	0.6	0.18	0.18					
					0.23	0.488	0.61	0.297	0.067
1.219	0.575	0.2	0.29	0.27					
		0.8	0.25		0.30	0.638	0.61	0.389	0.115
1.829	0.700	0.2	0.35	0.32					
		0.8	0.30		0.31	0.755	0.61	0.460	0.143
2.438	0.810	0.2	0.35	0.30					
		0.8	0.25		0.30	0.825	0.61	0.503	0.149
3.048	0.840	0.2	0.32	0.29					
		0.8	0.27		0.28	0.815	0.61	0.497	0.138
3.658	0.790	0.2	0.29	0.26					
		0.8	0.23		0.23	0.743	0.61	0.453	0.103
4.267	0.695	0.2	0.20	0.19					
		0.8	0.18		0.18	0.598	0.61	0.364	0.065
4.877	0.500	0.6	0.16	0.16					
					0.14	0.378	0.61	0.230	0.031
5.486	0.255	0.6	0.11	0.11					
					0.06	0.128	0.61	0.078	0.005
6.096	0.000		0.01	0.01					
					Totals:		**6.10**	**3.392**	**0.829**

References & Bibliography

Sontek. 2003. 6837 Nancy Ridge Dr., Suite A, San Diego, CA. www.sontek.com

USBR. 1997. Water Measurement Manual. U.S. Government Printing Office, Denver, CO.

Lecture 5
Field Exercise for Flume Calibration

I. Introduction

- In this field exercise, we will check the dimensions of a flow measurement flume, also applying the:

    1. Observation method;
    2. Float method;
    3. Uniform flow method; and,
    4. Dye method.

- You will write this up as a homework exercise with the following sections:

    1. Date, location, participant names;
    2. Introduction (describe what was done);
    3. Procedure;
    4. Data analysis;
    5. Summary & conclusions; and,
    6. References and or bibliography.

- Include a few digital photographs in the report (we will bring a camera)
- You may turn the report in by groups, if desired, but everyone in the group must contribute significantly to the work

II. Field Activities (Procedure)

- Dress appropriately for field work
- Guess (observation) the flow rate by looking at the channel and or flume
- Use the “float method” to estimate the flow rate
- Use the “dye method” to estimate the flow rate
- Take several dimensional and elevational measurements on the flume, including the water surface elevations
- Notice if the flume is operating under free or submerged conditions
- Elevational measurements should include at least five points on the upstream floor of the flume, and the top of the flume walls
- Measure the channel bed elevations downstream of the flume at 15 – 20 ft distance intervals for a total distance of at least 300 ft
- Measure data to define the channel cross section, downstream of the flume, at two locations (try to get representative locations)
- Estimate the Manning roughness value
- Write down any notes or observations which might be relevant to the work and the flow measurement results

Lecture 6

Field Exercise for Current Metering

I. Introduction

- In this field exercise, we will do current metering in a canal, at the same locations as the flume from the previous field work
- We will use three current meters:

1. A Price current meter;
2. A Marsh-McBirney electromagnetic current meter; and,
3. An Ott-type current meter.

- The results of the current metering will be compared to the known calibration of a measurement flume
- You will write this up as a homework exercise with the following sections:

1. Date, location, participant names;
2. Introduction (describe what was done);
3. Procedure;
4. Data analysis;
5. Summary & conclusions; and,
6. References and or bibliography.

- Include a few digital photographs in the report (we will bring a camera)
- You may turn the report in by groups, if desired, but everyone in the group must contribute significantly to the work

II. Field Activities (Procedure)

- Dress appropriately for field work
- Use the current meters to measure the flow rate
- Measure the upstream & downstream depths at the flume, including the water surface elevations

Lecture 7

Weirs for Flow Measurement

I. Introduction

- Weirs are overflow structures built across open channels to measure the volumetric rate of water flow
- The crest of a measurement weir is usually perpendicular to the direction of flow
- If this is not the case, special calibrations must be made to develop a stage discharge relationship
- Oblique and "duckbill" weirs are sometimes used to provide nearly constant upstream water depth, but they can be calibrated as measurement devices

- Some general terms pertaining to weirs are:

notch....... the opening which water flows through
crest....... the edge which water flows over
nappe....... the overflowing sheet of water
length....... the "width" of the weir notch

II. Advantages and Disadvantages of Weirs

Advantages

1. Capable of accurately measuring a wide range of flows
2. Tends to provide more accurate discharge ratings than flumes and orifices
3. Easy to construct
4. Can be used in combination with turnout and division structures
5. Can be both portable and adjustable
6. Most floating debris tends to pass over the structure

Disadvantages

1. Relatively large head required, particularly for free flow conditions. This precludes the practical use of weirs for flow measurement in flat areas.
2. The upstream pool must be maintained clean of sediment and kept free of weeds and trash, otherwise the calibration will shift and the measurement accuracy will be compromised

III. Types of Weirs

- Weirs are identified by the shape of their opening, or notch
- The edge of the opening can be either sharp- or broad-crested

<u>(1) Sharp-crested weir</u>

- A weir with a sharp upstream corner, or edge, such that the water springs clear of the crest
- Those most frequently used are sharp-crested rectangular, trapezoidal, Cipoletti, and triangular or 90° V-notch weirs
- According to the USBR, the weir plate thickness at the crest edges should be from 0.03 to 0.08 inches
- The weir plate may be beveled at the crest edges to achieve the necessary thickness

<u>(2) Broad-crested weir</u>

- A weir that has a horizontal or nearly horizontal crest sufficiently long in the direction of flow so that the nappe will be supported and hydrostatic pressures will be fully developed for at least a short distance
- Broad-crested weirs will be covered in detail later in the course
- Some weirs are not sharp- nor broad-crested, but they can be calibrated for flow measurement

Weirs may also be designed as *suppressed* or *contracted*

(1) Suppressed weir

- A rectangular weir whose notch (opening) sides are coincident with the sides of the approach channel, also rectangular, which extend unchanged downstream from the weir
- It is the lateral flow contraction that is "suppressed"

(2) Contracted weir

- The sides and crest of a weir are far away from the sides and bottom of the approach channel
- The nappe will fully contract laterally at the ends and vertically at the crest of the weir
- Also called an "unsuppressed" weir
- Calibration is slightly more complex than for a suppressed weir

IV. Types of Flow

(1) Free flow

- Also called "modular" flow, is a condition in which the nappe discharges into the air
- This exists when the downstream water surface is lower than the lowest point of the weir crest elevation
- Aeration is automatic in a contracted weir
- In a suppressed weir the sides of the structure may prevent air from circulating under the nappe, so the underside of the nappe should be vented (if used for flow measurement)
- If not vented, the air beneath the nappe may be exhausted, causing a reduction of pressure beneath the nappe, with a corresponding increase in discharge for a given head

(2) Submerged flow

- Also referred to as "non-modular" flow, is a condition in which the discharge is partially under water, where changes in the downstream depth will affect the flow rate
- A condition which indicates the change from free-flow to submerged-flow is called *transition submergence*, where submergence is defined as the ratio of downstream to upstream specific energy (E_d/E_u)

- For practical application of weirs as flow measurement devices, it is preferable that they operate under free-flow conditions so that only the upstream depth need be measured to arrive at a discharge value

- The calibration of free-flow weirs is more accurate that the calibration of submerged-flow weirs

V. Approach Velocity and Gauge Location

- Large errors in flow measurement can occur because of poor flow conditions, high-velocity and turbulence in the area just upstream of weir
- In general, the approaching flow should be the same as the flow in a long, straight channel of the same size
- The upstream section of channel is sometimes called the "weir pool"
- For best flow measurement accuracy, the velocity of approach to a weir should be less than 0.5 fps, or about 0.15 m/s
- This value is approximately obtained by dividing the maximum discharge by the product of channel width and water depth (for a rectangular channel section), which measured at the upstream point 4 to 6 times the weir head
- This point is the preferred staff gauge location upstream of the weir
- A tranquil flow condition should extend upstream from the weir a distance of 15 to 20 times the head on the weir
- The weir pool can be a wide channel section just upstream, thereby obtaining a sufficiently low approach velocity
- Never place a weir in an open-channel reach with supercritical flow; a hydraulic jump will form upstream and the water surface at the weir will not be tranquil

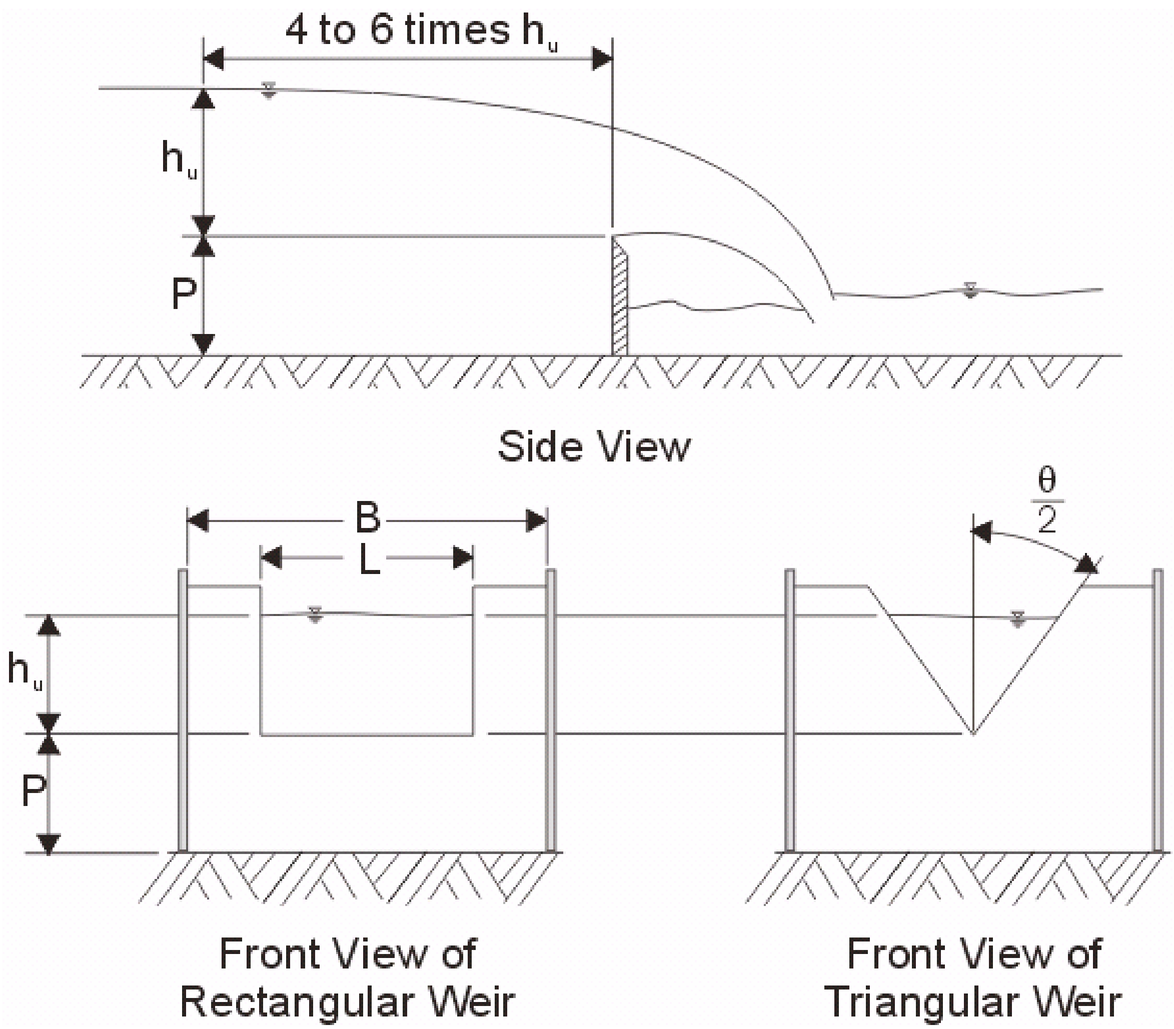
You can install a weir in a supercritical channel and a hydraulic jump will occur upstream of the weir, but there will be too much turbulence (unless the sill is very high). Always check the upstream Froude number in weir designs.

VI. Guidelines for Designing & Operating Weirs

1. The weir should be set at the lower end of a long pool sufficiently wide and deep to give an even, smooth flow
2. The centerline of the weir notch should be parallel to the direction of the flow
3. The face of the weir should be vertical, not leaning upstream nor downstream
4. The crest of the weir should be level, so the water passing over it will be of the same depth at all points along the crest (does not apply to V-notch weirs, but the centerline of the V-notch opening should be vertical)
5. The upstream edge should be sharp so that the nappe touches the crest only at the leading (upstream) edge
6. Ideally, though not always practical, the height of the crest above the bottom of the pool, P, should be at least three times the depth of water flowing over the weir crest (check this condition for the maximum flow rate) – note that some calibrations do not have this restriction, as described below
7. The sides of the pool should be at a distance from the sides of the crest not less than twice the depth of the water passing over the crest (for unsuppressed rectangular weirs):

$$\left(\frac{B-L}{2}\right) > 2h_u \tag{1}$$

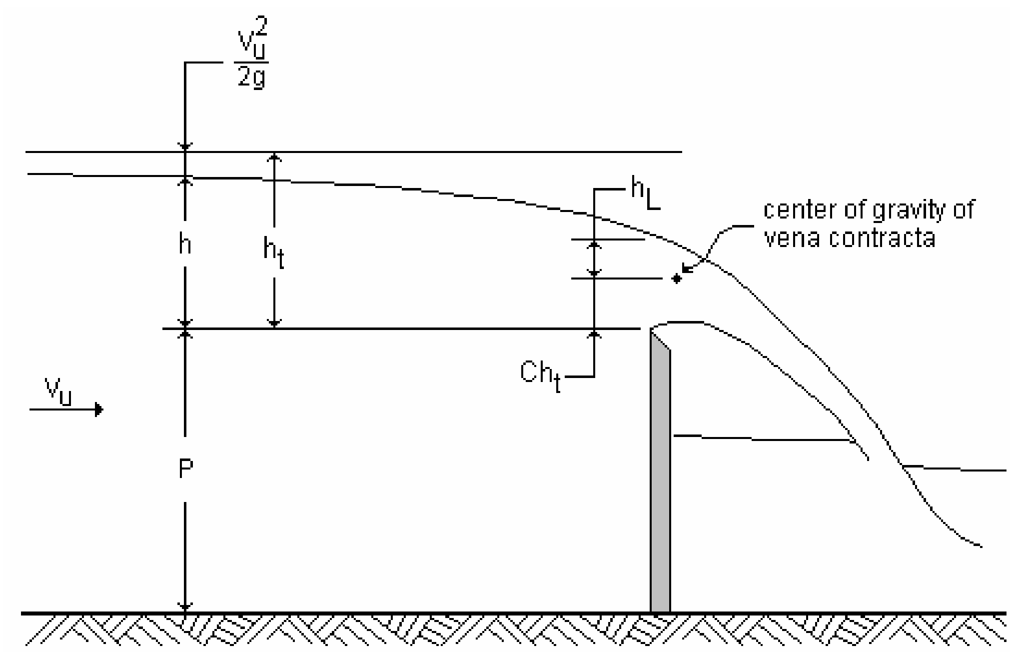
8. For accurate measurements the depth over the crest should be no more than one-third the length of the crest
9. The depth of water over the crest should be no less than two inches (50 mm), as it is difficult to obtain sufficiently accurate depth readings with smaller depths
10. The crest should be placed high enough so water will fall freely below the weir, leaving an air space under the over-falling sheet of water. If the water below the weir rises above the crest, this free fall is not possible, and the weir is then operating under submerged-flow conditions.
11. To prevent erosion by the falling and swirling water, the channel downstream from the weir should be protected by loose rock or by other material
12. You can assume that the discharge measurement accuracy of a sharp-crested weir under free-flow conditions is within ±2% under the best field conditions
13. Don't design a weir in which the minimum measurable flow rate is less than 2% of the maximum flow rate, because you will not be able to accurately measure such small flows.



- Note that it is not always possible to achieve the above guidelines when using sharp-crested weirs for flow measurement in open channels
- But some things can be compensated for, such as an approach velocity which is greater than 0.5 fps (0.15 m/s), as described below
- Also, the $P > 3h_u$ restriction is not always necessary (e.g. the C_e graphs below have h_u/P up to a value of 2.4)
- As the ratio of P/h_u decreases, the calculated flow rate over the weir is increasingly underestimated
- Never let $P < h_u$ unless you are prepared to develop a custom calibration

VII. Derivation of the Free-Flow Weir Equations

- An equation for accurately describing the head-discharge relationship over a weir under free-flow conditions cannot be derived purely from theoretical considerations assuming one-dimensional flow
- Theoretical calibrations can be derived based on 3-D flow analysis and a few assumptions, but so far this can only be done with models using numerical approximations
- In terms of one-dimensional flow, the Bernoulli equation can be written from a point upstream of the weir to the crest location, as follows:



$$h_t = h + \frac{V_u^2}{2g} = Ch_t + h_L + \frac{V_v^2}{2g} \tag{2}$$

- Solving for the mean flow velocity at the vena contracta, V_v,

$$V_v = \sqrt{2g}\sqrt{h_t(1-C)-h_L} \tag{3}$$

- Taking the liberty to combine some terms,

$$V_v \approx C'\sqrt{2gh_t} \tag{4}$$

- From continuity, Q = A_vV_v, and expressing the area of the vena contracta in terms of the weir opening, $A_v = C_cA$, where C_c is the contraction coefficient,

$$Q = C_cA\,C'\sqrt{2gh_t} \tag{5}$$

- Letting $C_d = C_cC'\sqrt{2g}$,

$$Q = C_dA\sqrt{h_t} \tag{6}$$

- For a horizontal-crested <u>rectangular weir</u>, A = hL. Therefore,

$$Q = C_dLh\sqrt{h_t} \approx C_dLh^{3/2} \tag{7}$$

- For a <u>V-notch weir</u>, A = $h^2\tan(\theta/2)$, and,

$$Q = C_d\tan\left(\frac{\theta}{2}\right)h^2\sqrt{h_t} \approx C_d\tan\left(\frac{\theta}{2}\right)h^{5/2} \tag{8}$$

- Letting $C_{dv} = C_d\tan\frac{\theta}{2}$,

$$Q = C_{dv}h^{5/2} \tag{9}$$

- For field calibrations it is useful to apply Eq. 7 for rectangular weirs and Eq. 9 for triangular weirs
- These coefficients will include the effects of approach velocity, nappe shape, weir opening contraction, and head loss
- Note that Eqs. 7 and 9 are dimensionally correct for either cfs or m^3/s, given the above definition for C_d
- Note also that Eq. 9 is of the same form as the free-flow calibration equation for nonorifice open-channel constrictions
- The general form of Eq. 9 can be used to calibrate <u>most weirs</u>, regardless of whether they are sharp-crested or not, when both the coefficient and the exponent on the "h" term are taken to be calibration parameters (based on field or lab data)

VIII. Sharp-Crested Rectangular Weirs

- A convenient method of including the variation in the velocity of approach and the contraction of the water jet over the weir is to relate C_d to the ratio h_u/P, where P is the vertical distance from the upstream channel bed to the weir crest
- A larger discharge for a given h_u would be passed when h_u/P is large
- In other words, when h_u/P is large, the influence of the vertical component is relatively small, and there is less contraction
- This is done through a coefficient called "C_e"

Kindsvater and Carter (1957) weir equation, for Q in cfs:

$$Q = C_e L_e h_e^{3/2} \tag{10}$$

$$L_e = L + K_L \tag{11}$$

$$h_e = h_u + K_H \tag{12}$$

where
L_e = the effective weir length
L = the measured weir length
h_e = the effective head
h_u = the measured head above the weir crest (ft)
C_e = the effective discharge coefficient
K_H = a small correction to the measured head (ft)

*For weirs with L/B = 1 (**suppressed weirs**)*

(a) According to the Kindsvater and Carter tests:

$$C_e = 3.22 + 0.40\frac{h_u}{P} \tag{13}$$

for K_H = 0.003 ft and K_L = -0.003 ft, with Q in cfs and head in feet.

(b) According to the Bazin (1886) tests:

$$C_e = 3.25 + 0.445\frac{h_u}{P} \tag{14}$$

for K_H = 0.012 ft and K_L = 0, with Q in cfs and head in feet.

(c) According to the Schroder and Turner (1904-1920) tests:

$$C_e = 3.21 + 0.45\frac{h_u}{P} \tag{15}$$

for K_H = 0.004 ft and K_L = 0, with Q in cfs and head in feet.

(d) According to USBR tests:

$$C_e = 3.22 + 0.44 \frac{h_u}{P} \tag{16}$$

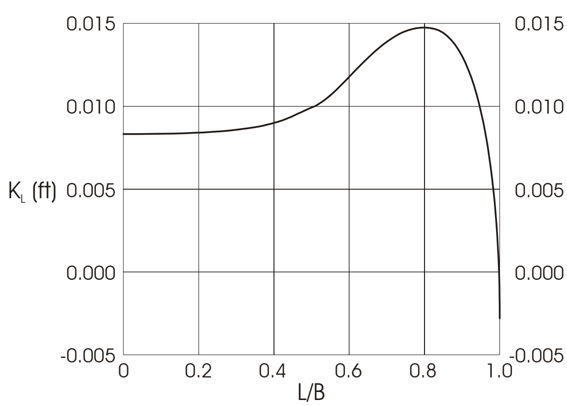
for K_H = 0.003 ft and K_L = 0, with Q in cfs and head in feet.

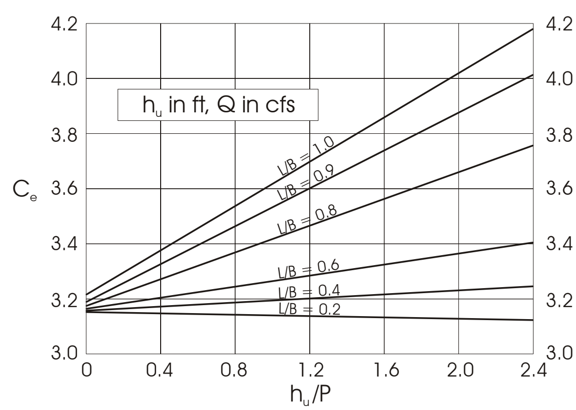
- It is seen that Eqs. 13 through 16 will give very similar results
- Note also that K_L is either zero or very small, and often negligible
- You can see that some of the above relationships were developed 100 years ago

*For weirs with L/B < 1 (**unsuppressed weirs**)*

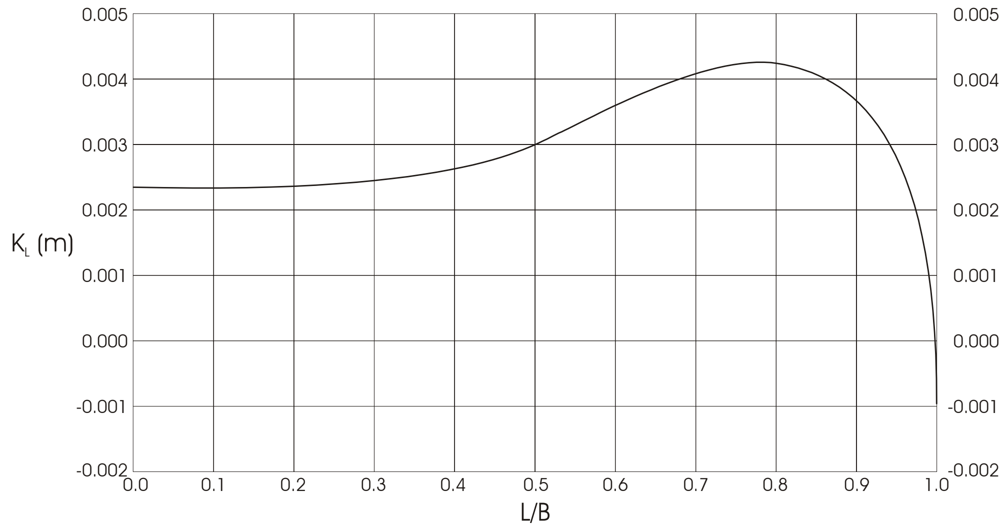
- Equations 10 - 12 still apply in this case
- The contraction effect is to decrease the magnitude of the coefficient, C_e
- The relationship of C_e to the constriction ratio L/B can be found in figures (see below) presented by Kindsvater and Carter (1957)
- The K_H values remain the same (but multiply the respective K_H values in Eqs. 13 - 16 by 0.3048 to use meters instead of feet)
- K_L values can also be determined graphically (see below)

Sharp-crested, rectangular weirs, English units:



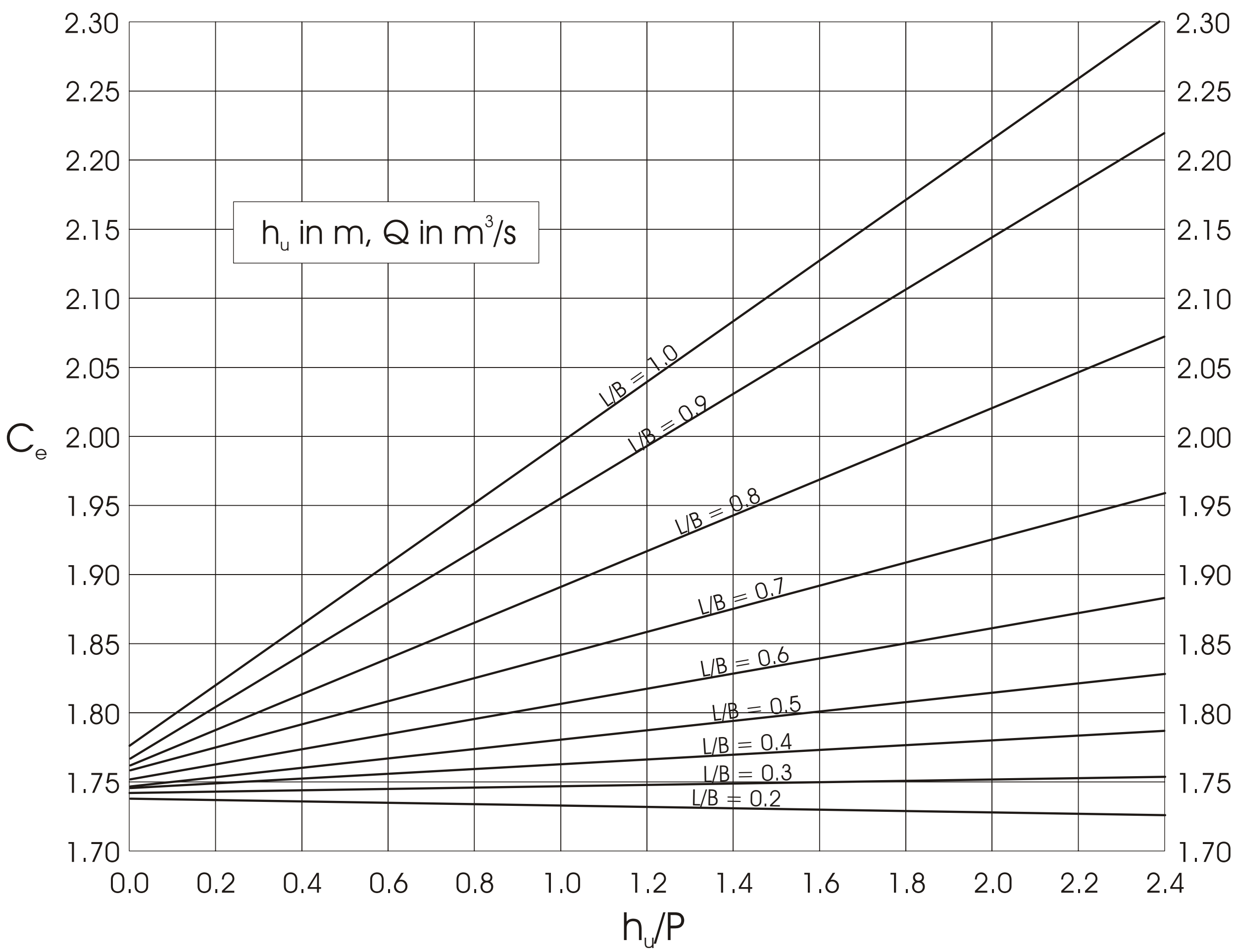


Sharp-crested, unsuppressed, rectangular weirs, metric units:



Note: suppression occurs at L/B = 1

Sharp-crested, unsuppressed, rectangular weirs, metric units:



- Observe that the abscissa scale in the above graph for C_e goes up to a maximum of $h_u/P = 2.4$, which exceeds the recommended maximum of 0.333, as discussed previously in this lecture
- Nevertheless, the above calibration procedure allows for $h_u/P > 0.333$

"B" for Rectangular Weirs in Non-rectangular Sections

- Note that rectangular-notch weirs in non-rectangular channel sections are always unsuppressed
- When applying the above calibrations to rectangular weirs in non-rectangular channel sections, let B equal the width of the upstream cross-section at the elevation of the weir crest

Equations Instead of Graphs

- It may be more convenient to approximate the above graphical solutions for K_L and C_e by equations when applying the relationships on a computer or calculator
- A rational function fits the C_e lines in the above graph (in metric units):

$$C_e = \alpha_{ce}\left(\frac{h_u}{P}\right) + \beta_{ce} \tag{17}$$

where C_e is for Q in m^3/s, and,

$$\beta_{ce} = 1.724 + 0.04789\left(\frac{L}{B}\right) \tag{18}$$

and,

$$\alpha_{ce} = \frac{-0.00470432 + 0.030365\left(\frac{L}{B}\right)}{1 - 1.76542\left(\frac{L}{B}\right) + 0.879917\left(\frac{L}{B}\right)^2} \tag{19}$$

- A combination of a straight line and a polynomial approximates the K_L curve, for K_L in meters:

For $0 \leq L/B \leq 0.35$:

$$K_L = 0.002298 + 0.00048\left(\frac{L}{B}\right) \tag{20}$$

For $0.35 < L/B \leq 1.00$:

$$
\begin{aligned}
K_L = &-0.10609\left(\frac{L}{B}\right)^4 + 0.1922\left(\frac{L}{B}\right)^3 - 0.11417\left(\frac{L}{B}\right)^2 \\
&+0.028182\left(\frac{L}{B}\right) - 0.00006
\end{aligned}
\tag{21}
$$

where K_L is in meters

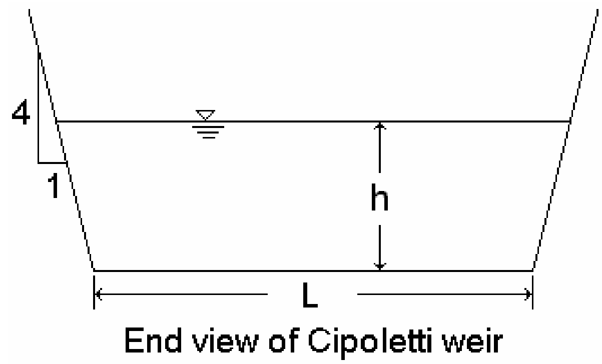
References & Bibliography

Kindsvater and Carter (1957)

Lecture 8

Weirs for Flow Measurement

I. Cipoletti Weirs



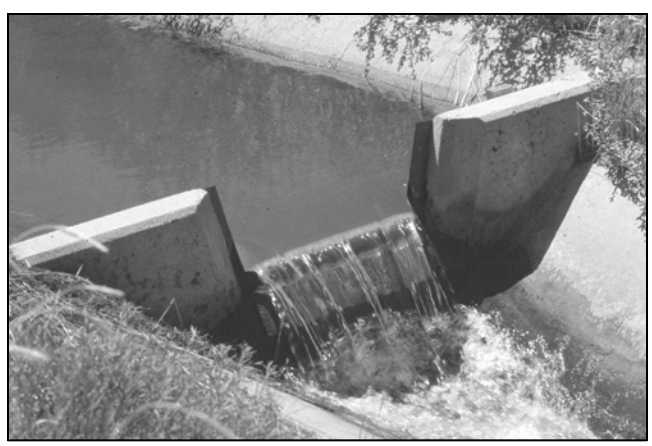
- The trapezoidal weir that is most often used is the so-called Cipoletti weir, which was reported in ASCE Transactions in 1894
- This is a fully contracted weir in which the notch ends (sides) are not vertical, as they are for a rectangular weir
- The effects of end contraction are compensated for by this trapezoidal notch shape, meaning that mathematical corrections for end contraction are unnecessary, and the equation is simpler
- The side slopes of the notch are designed to correct for end contraction (as manifested in a rectangular weir), splayed out at angle of 14° with the vertical, or nearly 1 horizontal to 4 vertical (tan 14° ≈ 0.2493, not 0.25 exactly)
- Some researchers have claimed than the side slopes should be greater than 1:4 in order to eliminate the effects of end contraction
- The sloping sides provides the advantage of having a stable discharge coefficient and true relationship of:

$$Q = CLh^{3/2} \tag{1}$$

The discharge equation by Addison (1949) is:

$$Q = \left(0.63\left(\frac{2}{3}\right)\sqrt{2g}\right)Lh^{3/2} = C_{cip}\,L\,h^{3/2} \tag{2}$$

where L is the weir length (equal to the width of the bottom of the crest, as shown above); and h is the upstream head, measured from the bottom (horizontal part) of the weir crest



- The units for L & h are feet for Q in cfs, with $\mathbf{C_{cip} = 3.37}$
- The units for L & h are m for Q in m^3/s, with $\mathbf{C_{cip} = 1.86}$

- Eq. 2 is of the same form as a rectangular sharp-crested weir
- Eq. 2 (right-most side) is simpler than that for unsuppressed rectangular and triangular sharp-crested weirs because the coefficient is a *simple constant* (i.e. no calibration curves are needed)

X. V-Notch Weirs

A small V-notch weir (for furrows)

- Triangular, or V-notch, weirs are among the most accurate open channel constrictions for measuring discharge
- For relatively small flows, the notch of a rectangular weir must be very narrow so that H is not too small (otherwise the nappe clings to the downstream side of the plate)
- Recall that the minimum h_u value for a rectangular weir is about 2 inches (50 mm)
- But with a narrow rectangular notch, the weir cannot measure large flows without correspondingly high upstream heads
- The discharge of a V-notch weir increases more rapidly with head than in the case of a horizontal crested weir (rectangular or trapezoidal), so for the same maximum capacity, it can measure much smaller discharges, compared to a rectangular weir

- A simplified V-notch equation is:

$$Q = Ch^{5/2} \tag{3}$$

- Differentiating Eq. 3 with respect to h,

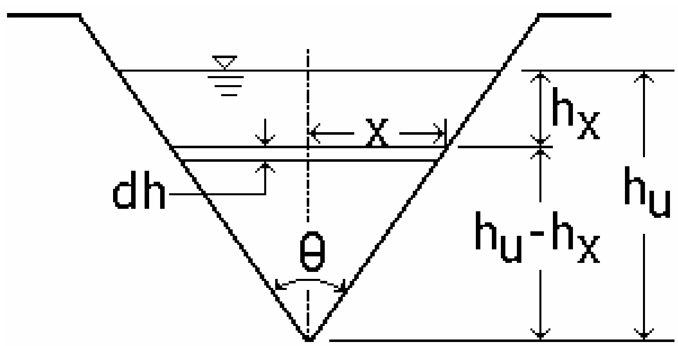
$$\frac{dQ}{dh} = \frac{5}{2} C h^{3/2} \tag{4}$$

- Dividing Eq. 4 by Eq. 3 and rearranging,

$$\frac{dQ}{Q} = \frac{5}{2} \frac{dh}{h} \tag{5}$$

- It is seen that the variation of discharge is around 2.5 times the change in head for a V-notch weir
- Thus, it can accurately measure the discharge, even for relatively small flows with a small head: h is not too small for small Q values, but you still must be able to measure the head, h, accurately
- A rectangular weir can accurately measure small flow rates only if the length, L, is sufficiently small, because there is a minimum depth value relative to the crest; but small values of L also restrict the maximum measurable flow rate
- The general equation for triangular weirs is:

$$Q = C_d 2\sqrt{2g} \tan\left(\frac{\theta}{2}\right) \int_0^{h_u} (h_u - h_x)\sqrt{h_x}\, dh \tag{6}$$



because,

$$dA = 2x\, dh \tag{7}$$

$$\frac{x}{h_u - h_x} = \tan(\theta/2) \tag{8}$$

$$dQ = C_d \sqrt{2gh}\, dA \tag{9}$$

- Integrating Eq. 6:

$$Q = C_d \frac{8}{15}\sqrt{2g}\tan\left(\frac{\theta}{2}\right)h_u^{2.5} \tag{10}$$

- For a given angle, θ, and assuming a constant value of C_d, Eq. 10 can be reduced to Eq. 3 by clumping constant terms into a single coefficient

- A modified form of the above equation was proposed by Shen (1981):

$$Q = \frac{8}{15}\sqrt{2g}\,C_e\tan\left(\frac{\theta}{2}\right)h_e^{5/2} \tag{11}$$

where,

$$h_e = h_u + K_h \tag{12}$$

- Q is in cfs for h_u in ft, or Q is in m^3/s for h_u in m
- The K_h and C_e values can be obtained from the two figures below
- Note that C_e is dimensionless and that the units of Eq. 11 are L^3T^{-1} (e.g. cfs, m^3/s, etc.)

Sharp-crested triangular (V-notch weirs):

- Shen (ibid) produced the following calibration curves based on hydraulic laboratory measurements with sharp-crested V-notch weirs

- The curves in the two figures below can be closely approximated by the following equations:

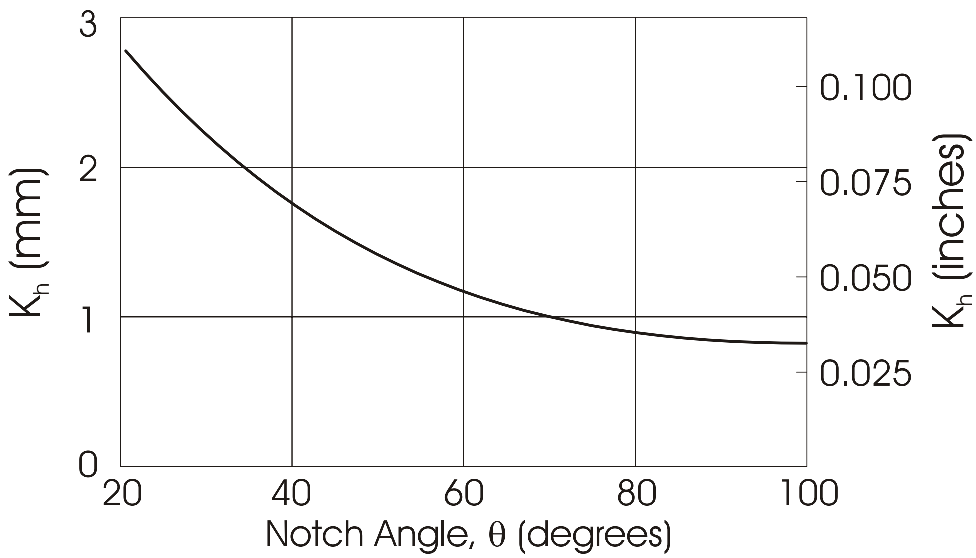
$$K_h \cong 0.001\left[\theta\left(1.395\theta - 4.296\right) + 4.135\right] \tag{13}$$

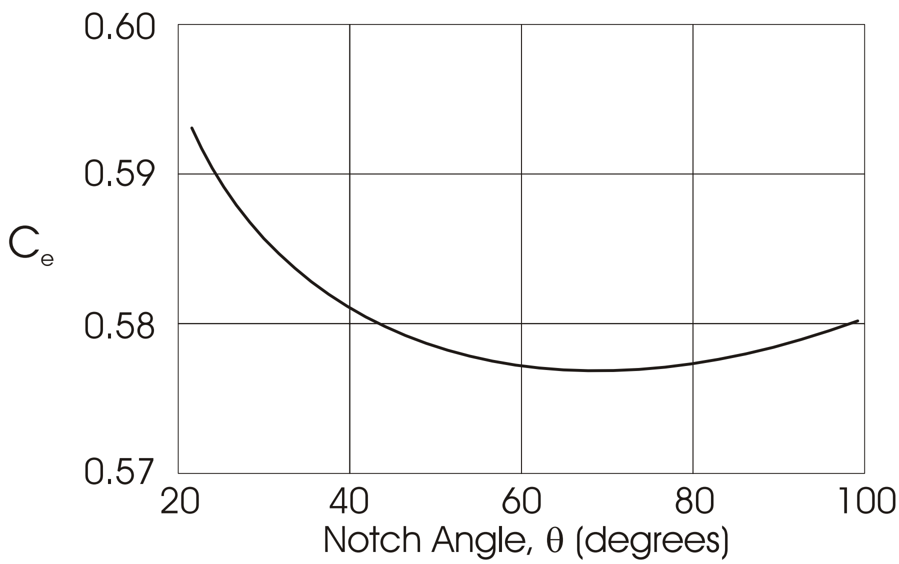
for K_h in meters; and,

$$C_e \cong \theta\left(0.02286\theta - 0.05734\right) + 0.6115 \tag{14}$$

for θ in radians

- Of course, you multiply a value in degrees by π/180 to obtain radians
- Some installations have an insertable metallic V-notch weir that can be placed in slots at the entrance to a Parshall flume to measure low flow rates during some months of the year



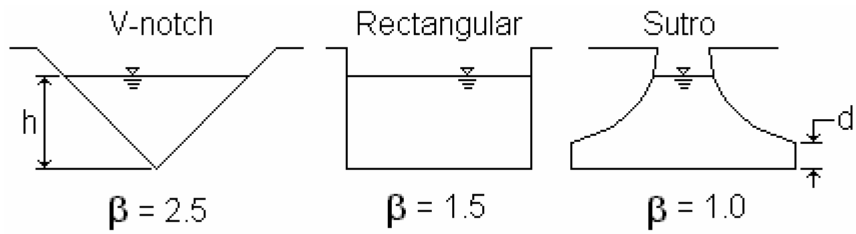


XI. Sutro Weir

- Sutro weirs have a varying cross-sectional shape with depth
- This weir design is intended to provide high flow measurement accuracy for both small and large flow rates
- A Sutro weir has a flow rate that is linearly proportional to h (for free flow)
- A generalized weir equation can be written as:

$$Q_f = k + \alpha h^{\beta} \tag{15}$$

where k = 0 for the V-notch and rectangular weirs, but not for the Sutro; and β is as defined below:



- The Sutro weir functions like a rectangular weir for $h \leq d$
- This type of weir is designed for flow measurement under free-flow conditions
- It is not commonly found in practice

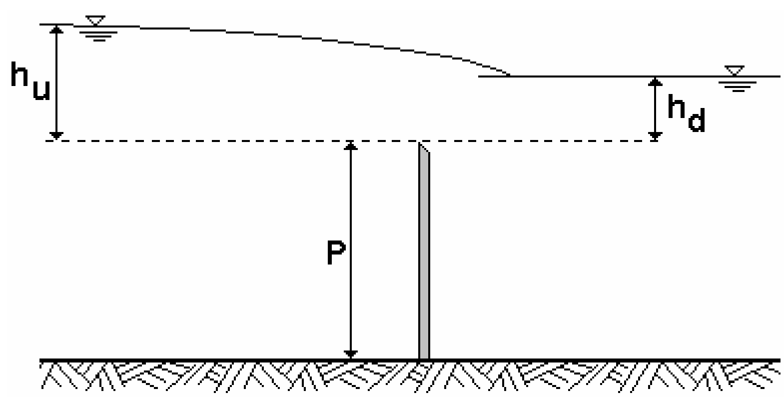
XII. Submerged Flow over Weirs

Single Curve

- Villamonte (1947) presented the following from his laboratory results:

$$Q_s = Q_f \left(1 - \left(\frac{h_d}{h_u} \right)^{n_f} \right)^{0.385} = K_s Q_f \tag{16}$$

- For $h_d \leq 0$, $K_s = 1.0$ and the flow is free
- For $h_d > h_u$, there will be backflow across the weir
- For $h_u = h_d$, the value of Q_s becomes zero (this is logical)
- The value of Q_f is calculated from a free-flow weir equation
- The exponent, n_f, is that which corresponds to the free-flow equation (usually, $n_f = 1.5$, or $n_f = 2.5$)
- The figure below shows that in applying Eq. 16, h_u & h_d are measured from the sill elevation



Sharp-Crested Weir, Submerged Flow

- Eq. 16 is approximately correct, but may give errors of more than 10% in the calculated flow rate, especially for values of h_d/h_u near unity

Multiple Curves

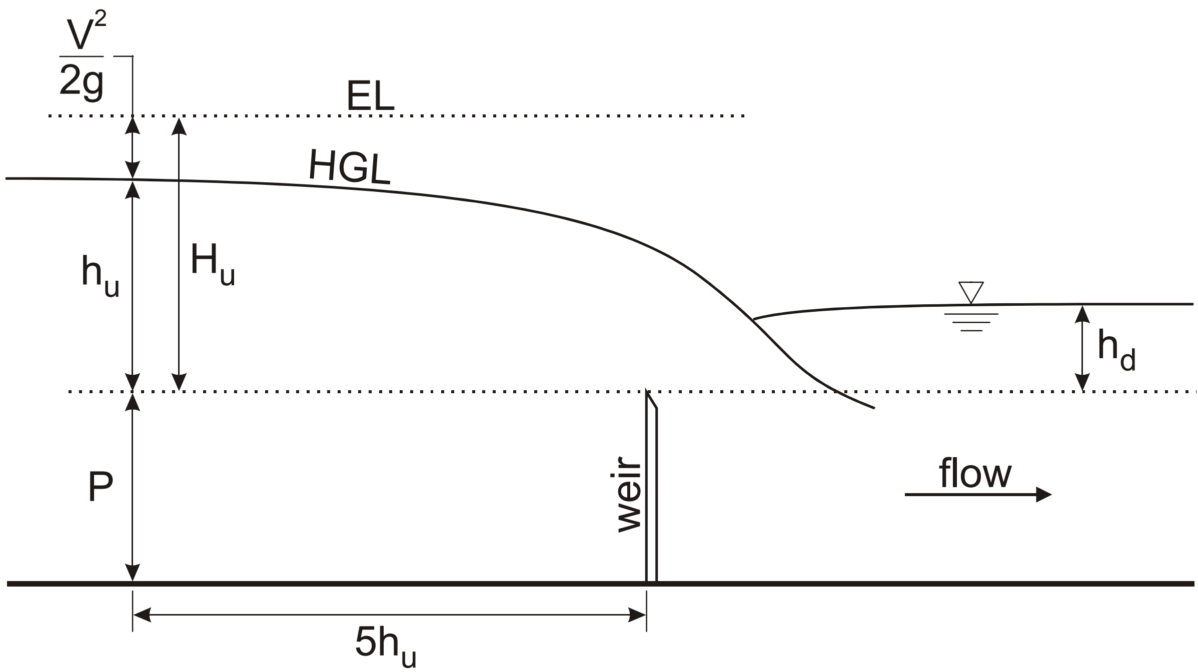
- Scoresby (1997) expanded on this approach, making laboratory measurements which could be used to generate a family of curves to define the submerged-flow coefficient, K_s
- The following is based on an analysis of the laboratory data collected by Scoresby (ibid). The flow rate through a weir is defined as:

$$Q = K_s C_f L H_u^{n_f} \tag{17}$$

where Q is the flow rate; C_f and n_f are calibration parameters for free-flow conditions; L is the "length" of the crest; H_u is the total upstream hydraulic head with respect to the crest elevation; and K_s is a coefficient for submerged flow, as defined above. As before, the coefficient K_s is equal to 1.0 (unity) for free flow and is less than 1.0 for submerged flow. Thus,

$$K_s \leq 1.0 \tag{18}$$

- Below is a figure defining some of the terms:



- The coefficient K_s can be defined by a family of curves based on the value of H_u/P and h_d/H_u
- Each curve can be approximated by a combination of an exponential function and a parabola
- The straight line that separates the exponential and parabolic functions in the graph is defined herein as:

$$K_s = A\left(\frac{h_d}{H_u}\right) + B \quad (19)$$

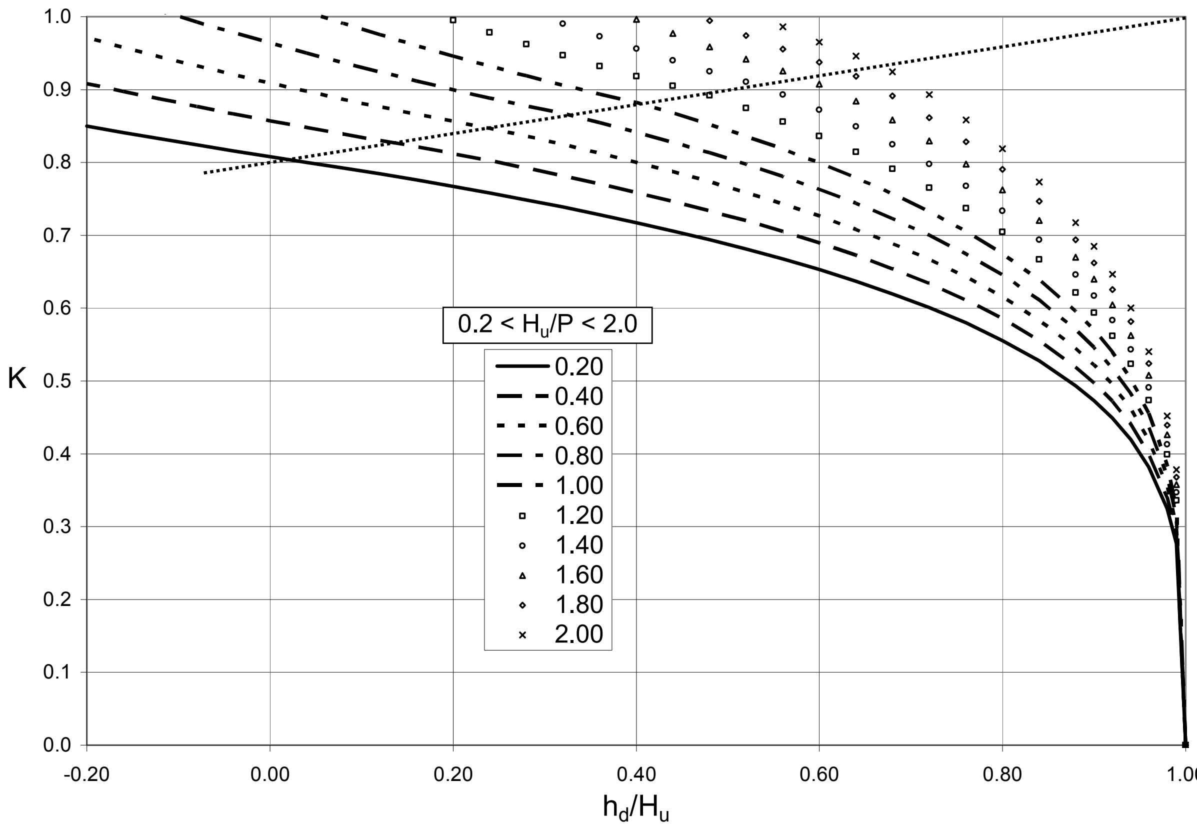
- The exponential function is:

$$K_s = \alpha\left(1 - \frac{h_d}{H_u}\right)^{\beta} \quad (20)$$

- The parabola is:

$$K_s = a\left(\frac{h_d}{H_u}\right)^2 + b\left(\frac{h_d}{H_u}\right) + c \quad (21)$$

- Below the straight line (Eq. 19) the function from Eq. 20 is applied
- And, Eq. 21 is applied above the straight line
- In Eq. 19, let A = 0.2 y B = 0.8 (other values could be used, according to judgment and data analysis)
- In any case, A+B should be equal to 1.0 so that the line passes through the point (1.0, 1,0) in the graph (see below).



- This curve is defined by Eq. 20, but the values of α and β depend on the value of H_u/P

- The functions are based on a separate analysis of the laboratory results from Scoresby (ibid) and are the following:

$$\alpha = 0.24\left(\frac{H_t}{P}\right) + 0.76 \tag{22}$$

$$\beta = 0.014\left(\frac{H_t}{P}\right) + 0.23 \tag{23}$$

- The point at which the two parts of the curves join is calculated in the following:

$$A\left(\frac{h}{H_t}\right) + B = \alpha\left(1 - \frac{h}{H_t}\right)^{\beta} \tag{24}$$

- Defining a function F, equal to zero,

$$F = A\left(\frac{h_d}{H_u}\right) + B - \alpha\left(1 - \frac{h_d}{H_u}\right)^{\beta} = 0 \tag{25}$$

$$\frac{\partial F}{\partial\left(\frac{h_d}{H_u}\right)} = A + \alpha\beta\left(1 - \frac{h_d}{H_u}\right)^{\beta-1} \tag{26}$$

- With Eqs. 25 and 26, a numerical method can be applied to determine the value of h_d/H_u
- Then, the value of K_s can be determined as follows:

$$K_s = A\left(\frac{h_d}{H_u}\right) + B \tag{27}$$

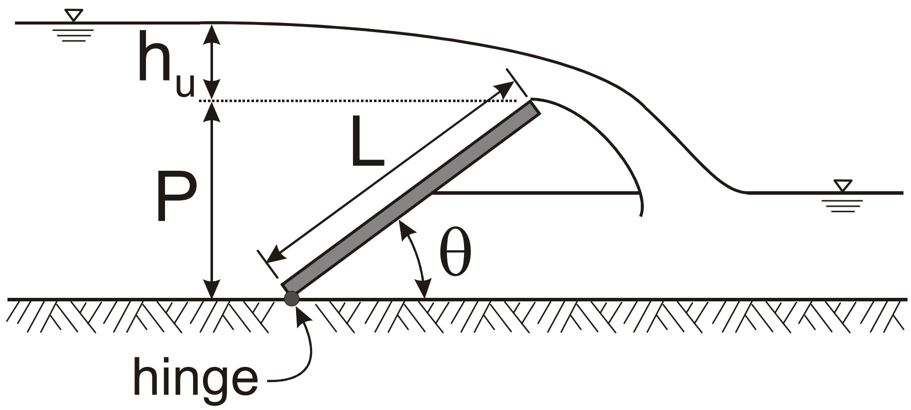
- The resulting values of h_d/H_u and K_s define the point at which the two parts of the curves join together on the graph

XIII. Overshot Gates

- So-called “overshot gates” (also known as “leaf gates,” “Obermeyers,” “Langeman,” and other names) are weirs with a hinged base and an adjustable angle setting (see the side-view figure below)
- Steel cables on either side of the gate leaf are attached to a shaft above and upstream of the gate, and the shaft rotates by electric motor to change the setting
- At large values of the angle setting the gate behaves like a weir, and at lower

angles it approximates a free overfall (but this distinction is blurred when it is recognized that these two conditions can be calibrated using the same basic equation form)

- These gates are manufactured by the Armtec company (Canada), Rubicon (Australia), and others, and are easily automated
- The figure below shows an overshot gate operating under free-flow conditions



- The calibration equations presented below for overshot gates are based on the data and analysis reported by Wahlin & Replogle (1996)
- The representation of overshot gates herein is limited to rectangular gate leafs in rectangular channel cross sections, whereby the specified leaf width is assumed to be the width of the cross section, at least in the immediate vicinity of the gate; this means that weir end contractions are suppressed

- The equation for both free and submerged flow is:

$$Q = K_s C_a C_e \frac{2\sqrt{2g}}{3} G_w h_e^{1.5} \tag{28}$$

where Q is the discharge; θ is the angle of the opening ($10° \le \theta \le 65°$), measured from the horizontal on the downstream side; G_w is the width of the gate leaf; and h_e is the *effective head*

- The effective head is defined as: $h_e = h_u + K_H$, where K_H is equal to 0.001 m, or 0.0033 ft
- K_H is insignificant in most cases
- For $\theta = 90°$, use the previously-given equations for rectangular weirs
- For h_e in m, Q is in m^3/s; for h_e in ft, Q is in cfs
- The calibration may have significant error for opening angles outside of the specified range
- The coefficient C_e is a function of θ and can be approximated as:

$$C_e = 0.075\left(\frac{h_u}{P}\right) + 0.602 \tag{29}$$

where P is the height of the gate sill with respect to the gate hinge elevation (m or ft)

- The value of P can be calculated directly based on the angle of the gate opening and the length of the gate leaf ($P = L \sin\theta$, where L is the length of the gate)
- The coefficient C_a is a function of the angle setting, θ, and can be adequately described by a parabola:

$$C_a = 1.0333 + 0.003848\,\theta - 0.000045\,\theta^2 \tag{30}$$

where θ is in degrees

- The submerged-flow coefficient, K_s, is taken as defined by Villamonte (1947), but with custom calibration parameters for the overshot gate type.

$$K_s = C_1\left[1 - \left(\frac{h_d}{h_u}\right)^{1.5}\right]^{C_2} \tag{31}$$

where,

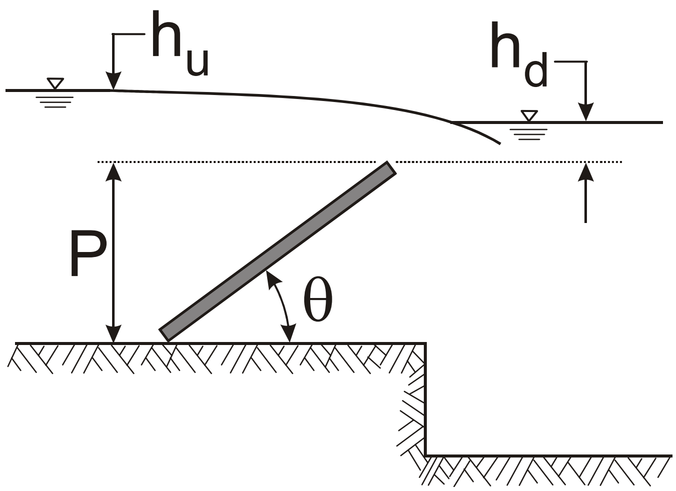
$$\begin{aligned} C_1 &= 1.0666 - 0.00111\,\theta && \text{for } \theta < 60^\circ \\ C_1 &= 1.0 && \text{for } \theta \geq 60^\circ \end{aligned} \tag{32}$$

and,

$$C_2 = 0.1525 + 0.006077\,\theta - 0.000045\,\theta^2 \tag{33}$$

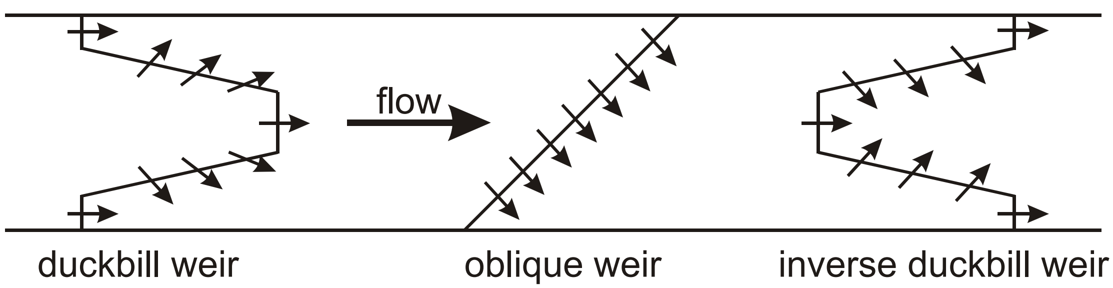
in which θ is in degrees

- The submerged-flow coefficient, K_s, is set equal to 1.0 when $h_d \leq 0$
- See the figure below for an example of an overshot gate with submerged flow



XIV. Oblique and Duckbill Weirs

- What about using oblique or duckbill weirs for flow measurement?
- The problem is that with large L values, the h_u measurement is difficult because small Δh values translate into large ΔQ
- Thus, the h_u measurement must be extremely accurate to obtain accurate discharge estimations



XV. Approach Velocity

- The issue of approach velocity was raised above, but there is another standard way to compensate for this
- The reason this is important is that all of the above calibrations are based on zero (or negligible) approach velocity, but in practice the approach velocity may be significant
- To approximately compensate for approach velocity, one ~~approach (ha ha!)~~ method is to add the upstream velocity head to the head term in the weir equation
- For example, instead of this…

$$Q_f = C_f \left(h_u\right)^{n_f} \tag{34}$$

…use this (where V is the mean approach velocity, Q/A):

$$Q_f = C_f \left(h_u + \frac{V^2}{2g} \right)^{n_f} \tag{35}$$

or,

$$Q_f = C_f \left(h_u + \frac{Q_f^2}{2gA^2} \right)^{n_f} \tag{36}$$

which means it is an iterative solution for Q_f, which tends to complicate matters a lot, because the function is not always well-behaved

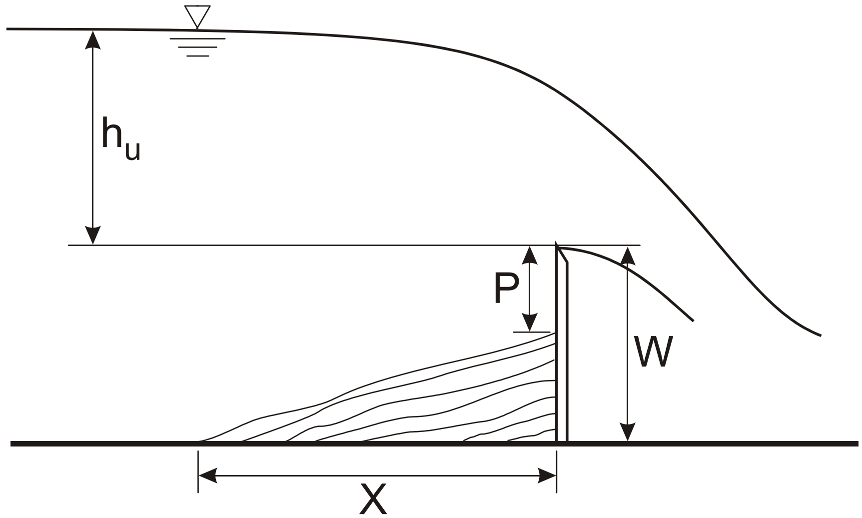
- For known h_u and A, and known C_f and n_f, the solution to Eq. 36 may have multiple roots; that is, multiple values of Q_f may satisfy the equation (e.g. there may be two values of Q_f that are very near each other, and both positive)
- There may also be no solution (!*%&!#@^*) to the equation
- Conclusion: it is a logical way to account for approach velocity, but it can be difficult to apply

XVI. Effects of Siltation

- One of the possible flow measurement errors is the effect of siltation upstream of the weir
- This often occurs in a canal that carries a medium to high sediment load
- Some weirs have underflow gates which can be manually opened from time to time, flushing out the sediment upstream of the weir
- The effect is that the discharge flowing over the weir can be increased due to a higher upstream "apron", thus producing less flow contraction
- The approximate percent increase in discharge caused by silting in front of a rectangular weir is given below:

Percent Increase in Discharge

P/W	X/W: 0	0.5	1.0	1.5	2.0	2.5
0.00	zero	**10%**	**13%**	**15%**	**16%**	**16%**
0.25	zero	**5%**	**8%**	**10%**	**10%**	**10%**
0.50	zero	**3%**	**4%**	**5%**	**6%**	**6%**
0.75	zero	**1%**	**2%**	**2%**	**3%**	**3%**
1.00	zero	**zero**				



- W is the value of P when there is no sediment deposition upstream of the weir
- X is the horizontal distance over which the sediment has been deposited upstream of the weir – if X is very large, use the top of the sediment for determining P, and do not make the discharge correction from the previous table
- The reason for the increase in discharge is that there is a change in flow lines upstream of the weir
- When the channel upstream of the weir becomes silted, the flow lines tend to straighten out and the discharge is higher for any given value of h_u

References & Bibliography

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Scoresby, P. 1997. Unpublished M.S. thesis, Utah State Univ., Logan, UT.

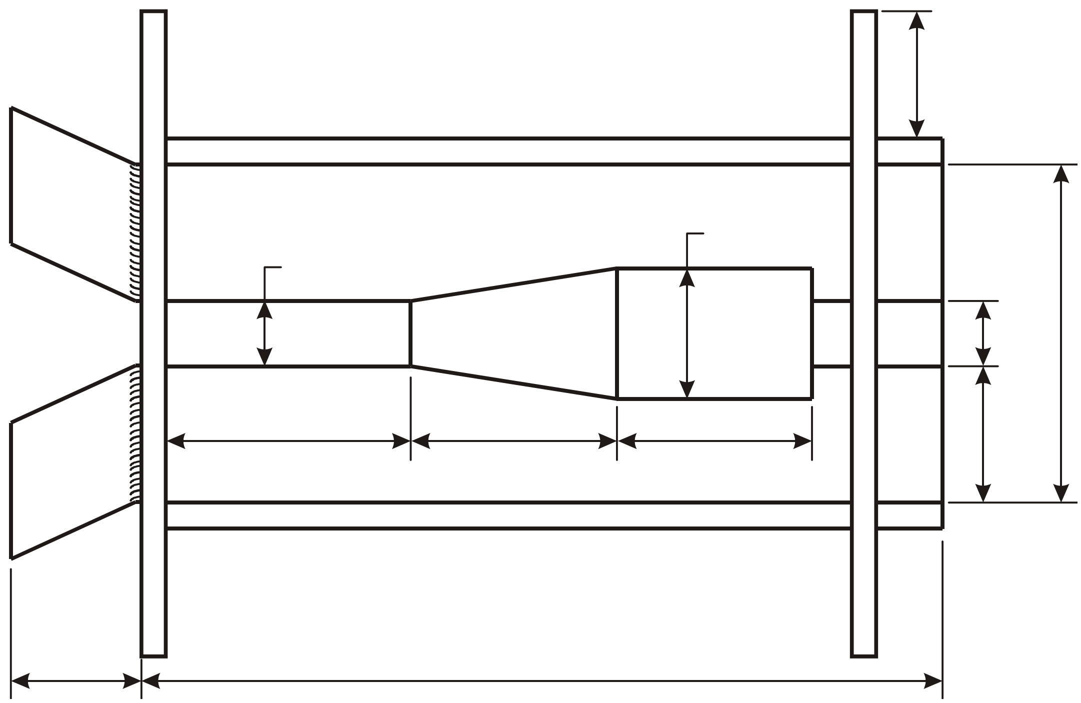
Wahlin T., and J. Replogle. 1996.

Lecture 9

Broad Crested Weirs

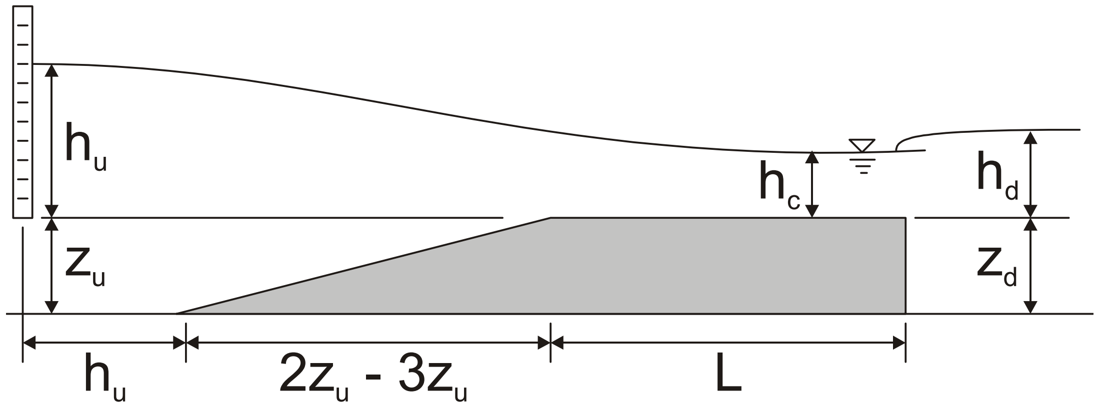
I. Introduction

- The broad-crested weir is an open-channel flow measurement device which combines hydraulic characteristics of both weirs and flumes
- Sometimes the name “ramp flume” is used in referring to broad-crested weirs
- As with related open-channel measurement devices, the broad-crested weir has an upstream converging section, a throat section, and a downstream diverging section
- The broad-crested weir can be calibrated for submerged flow conditions; however, it is desirable to design this device such that it will operate under free-flow conditions for the entire range of discharges under which it is intended to function



- When operating under free-flow conditions, critical flow will occur over the crest (sill), and the discharge is uniquely related to the upstream flow depth – in this case, the downstream conditions do not affect the calibration
- The broad-crested weir can be calibrated in the field or laboratory; however, a major advantage of the structure is that it can be accurately calibrated based on theoretical equations without the need for independent laboratory measurements

- The flow depth upstream of the measurement structure must always be higher than it would be in the absence of the structure because there is always some head loss
- Downstream of the structure the depth will not be affected; so, the required head loss is manifested (in one way) as an increase in the upstream depth



II. Transition Submergence

- The typical transition submergence ranges for modular flow are:

Parshall flume .... 58 to 80%
Cutthroat flume .... 55 to 88%
Broad-crested weir................ 70 to 95%

- This means that the broad-crested weir can usually function as a free-flow measurement device with less increase in the upstream water depth, which can be a significant advantage

III. Advantages and Disadvantages

Advantages

1. the design and construction of the structure is simple, thus it can be relatively inexpensive to install
2. a theoretical calibration based on post-construction dimensions can be obtained, and the accuracy of the calibration is such that the discharge error is less that two percent (this is assuming correct design and installation of the structure)
3. as with other open-channel flow measurement structures operating under free-flow (modular) conditions, a staff gauge which is marked in discharge units can be placed upstream; this allows a direct reading of the discharge without the need for tables, curves, or calculators
4. the head loss across the structure is usually small, and it can be installed in channels with flat slopes without greatly affecting existing upstream flow depths
5. floating debris tends to pass over and through the structure without clogging

Disadvantages

1. for water supplies with sediment, there will be deposition upstream of the structure
2. the upstream water depth will be somewhat higher than it was without the structure
3. farmers and other water users tend to oppose the installation of this structure because they believe that it significantly reduces the channel flow capacity. Although this is a false perception for a correctly designed broad-crested weir, it

does represent an important disadvantage compared to some other flow measurement devices

IV. Site Selection

- The channel upstream of the broad-crested weir should be fairly straight and of uniform cross-section
- The flow regime in the upstream section should be well into the subcritical range so that the water surface is stable and smooth ($F_r^2 < 0.20$, if possible). For this reason it is best to avoid locating the structure just downstream of a canal gate or turnout, for example, because the water surface is often not stable enough for an accurate staff gauge reading
- The use of a stilling well and float assembly (or other water level sensing device) to measure water level can partially compensate for fluctuating water levels, although it involves additional cost
- Preferably, there are no gates or channel constrictions downstream of the structure which would cause non-modular flow
- In fact, it is desirable to locate the structure just upstream of an elevation drop if possible
- The presence of adjustable gates downstream complicates the design even more than for fixed constrictions because the depth will depend on both discharge and gate setting
- Other factors involved in the site selection are the stability of the channel bed and side slopes in the upstream direction (in the case of earthen canals), and the accessibility for measurement readings and maintenance
- If the upstream channel is not stable, the calibration may change significantly, and sediment can accumulate rapidly at the structure, also affecting the calibration

V. Design Considerations

- One of the important advantages of the broad-crested weir is that it can be accurately calibrated according to theoretical and empirical relationships
- This means that it is not necessary to install "standard" structure sizes and rely on laboratory calibration data
- The ability to calibrate the structure using equations instead of measurements is based on the existence of parallel streamlines in the control section over the crest
- In many other open-channel flow measurement devices the streamlines are not straight and parallel in the control section, and although a theoretical calibration would be possible, it requires complex hydraulic modeling
- On the other hand, theoretical calibration of the broad-crested weir is relatively simple
- The broad-crested weir should be located and dimensioned so that the flow is modular over the full operating range of the device
- If there is a significant drop in the channel bed immediately downstream of the structure, then the height of the crest may not be important in achieving critical depth

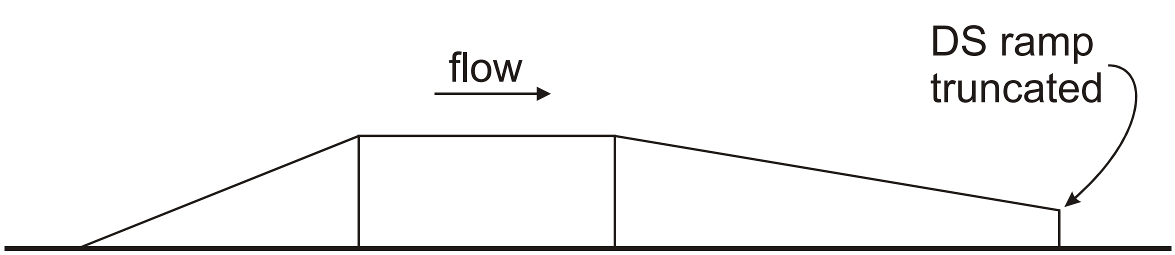
- However, the relative dimensions of the structure are important to obtain "favorable" flow conditions over the crest, that is, flow conditions which conform to the inherent assumptions for accurate theoretical calibration
- Thus, the height and length of the crest are important dimensions with relation to the upstream flow depth
- In any case, adequate design of the structure dimensions is essentially a process of trial-and-error, and therefore can be greatly facilitated through use of a calculator or computer program

<u>Sill Height</u>

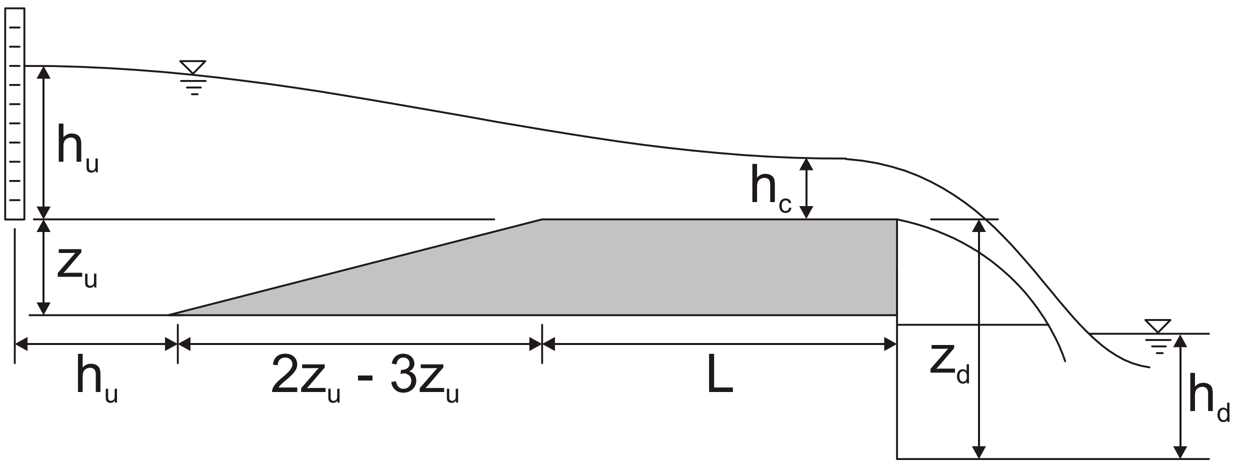
- One of the most important design parameters is the height of the sill above the upstream channel bed
- This height should be sufficient to provide modular flow for the entire range of discharges that the broad-crested weir is intended to measure; however, it should not be higher than necessary because this would cause undue increases in the upstream water level after installation
- Thus, a design objective is to determine the minimum crest height for which modular flow can be obtained, and not to exceed this minimum height
- Excessively tall broad-crested weirs are not a problem in terms of water measurement or calibration, they are only troublesome with respect to unnecessarily raising the upstream water level
- The lower limit on sill height is based on the Froude number in the upstream channel section ($F_r^2 < 0.20$)

<u>Upstream and Downstream Ramps</u>

- The converging upstream ramp should have a slope of between 2:1 and 3:1 (H:V). If flatter, the ramp is longer than necessary and there will be additional hydraulic losses which detract from the calibration accuracy
- If the ramp is steeper than 2:1, unnecessary turbulence may be created in the converging section, also causing addition head loss
- The diverging ramp at the downstream end of the crest should have a slope of between 4:1 and 6:1 (H:V), or should be truncated (non-existent). The 6:1 ratio is preferred in any case, and this same ratio is used in the diverging sections of other flow measurement devices, in both open-channel and pipe flow, to minimize head losses from turbulence
- If the 6:1 ratio causes an excessively long downstream ramp, then the length should be abruptly truncated (see the figure below), not rounded off



- Many broad-crested weirs do not have a downstream ramp – the structure is terminated with a vertical wall just downstream of the throat section. In many cases, the energy that could be "recovered" by the inclusion of a downstream ramp is not enough to justify the additional expense. Also, the benefits of a downstream ramp are more significant in large broad-crested weirs (more than 1 m high)
- Most of the head loss across the structure occurs due to turbulence in the diverging section, and in many cases the losses in the converging and throat sections may be neglected in calibration calculations

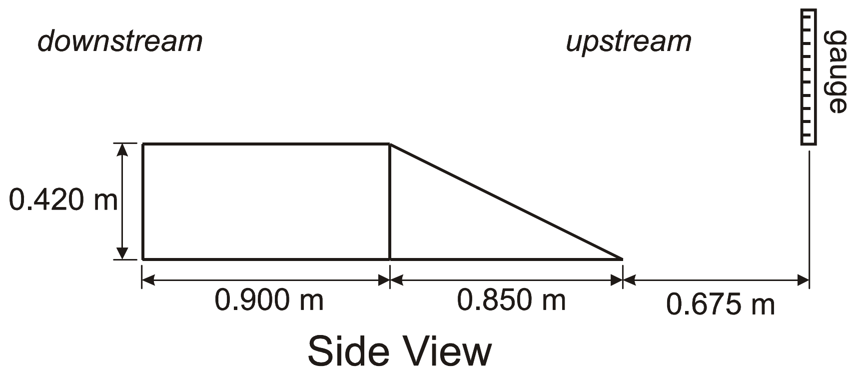


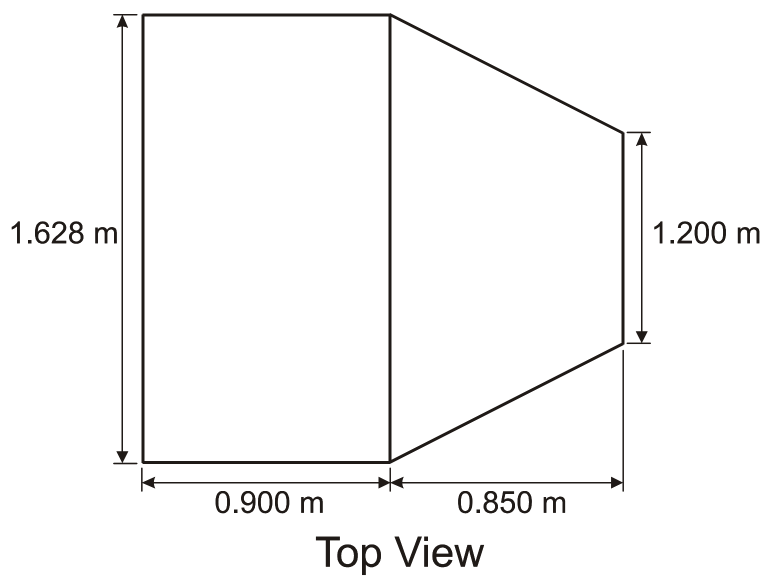
Lateral Flow Contraction

- The side slope in the throat section of the broad-crested weir is usually the same as that in the upstream section, but it does not need to be the same
- In very wide and earthen channels it is common practice to reduce the width of the throat section and design for a zero side slope (i.e. a rectangular section)
- When the side slope is reduced it is usually because the vertical flow contraction obtained by the crest height is insufficient to induce modular flow conditions. Therefore, in some cases lateral flow contraction is also required

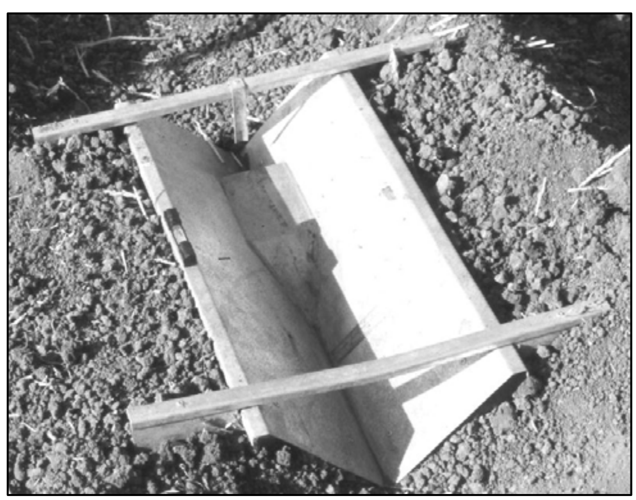
Ratio of h_u/L

- The ratio of upstream head to crest length is limited by a maximum of approximately 0.75, and a minimum of approximately 0.075
- The lower limit is imposed to help maintain a reasonably small ratio of head loss to total upstream head (relative to the crest elevation)
- The upper limit is meant to avoid a non-hydrostatic pressure distribution on the crest
- The calibration procedure is valid only for horizontal, parallel streamlines in the control section in the throat. When the ratio is between these limits, the theoretical calibration should be accurate to within two percent of the actual discharge
- The ratio of upstream flow depth (referenced from the sill elevation) to the throat length is approximately 0.5 for the average discharge over a correctly-designed broad-crested weir
- The figure below shows dimensions for a sample BCW design





VI. Modular Limit



- The modular limit is defined as the ratio of specific energies in the downstream and upstream sections (E_d/E_u) at which transitional flow exists
- The velocity heads in the upstream and downstream sections will normally be small compared to the flow depths, so this ratio may be approximated by the transition submergence, $S_t = h_d/h_u$ for $Q_f = Q_s$
- When the ratio is greater than the modular limit, the structure is submerged and the flow is non-modular

- Under non-modular conditions the theoretical calibration is invalid since it assumes that critical flow occurs somewhere over the crest in the throat section
- Field calibration of the structure for submerged-flow conditions is possible, but the results will be less accurate and the structure's use as a measurement device will be less convenient

Determining Downstream Depth

- For existing channels with a straight section downstream of the broad-crested weir, and without hydraulic controls such as sluice gates, the value of h_d can be determined according to normal flow conditions. That is, for a given discharge, the value of h_d can be calculated using the Manning or Chezy equations
- In the case of a downstream control which causes a backwater effect at the broad-crested weir, the issue becomes complicated since the actual submergence ratio across the structure depends not only on the discharge, but also on the control setting (which creates an M1 profile upstream toward the broad-crested weir). This is a common situation because the water surface profile in most irrigation channel reaches is affected by downstream flow control structures
- For this reason, it is preferable to have a drop in elevation immediately downstream of the broad-crested weir, or to have a straight canal section without any nearby control structures in the downstream direction

Energy Balance and Losses

- Given an upstream depth and its associated discharge (according to the calibration), the value of downstream specific energy, E_d, for the modular limit is calculated by subtracting estimated head losses from the upstream specific energy, E_u
- These head losses include friction and turbulence across the broad-crested weir, and equations exist to approximate the respective values according to cross-sectional geometry, expansion ratios, and roughness coefficients (Bos, et al. 1984)
- Computer programs for developing the theoretical calibration contain these equations; however, it is worth noting that the losses due to wall friction are very minimal, and accurate estimation of roughness coefficients is not necessary for calibration
- The majority of the losses occur due to the sudden expansion downstream of the throat section, and these losses are either estimated or calculated (using empirical relationships) depending on the downstream ramp dimensions

Calculating the Modular Limit

- The modular limit will vary according to discharge for a given installation, and its calculation can be summarized as follows:

  1. Given an upstream depth, channel cross-section, and weir calibration, calculate the discharge (assuming modular flow), and add depth plus velocity head to produce E_u

2. Calculate head losses due to wall friction in the converging and throat sections, then estimate the expansion loss downstream of the throat, and add these two values
3. Subtract the combined losses from E_u, giving the value of E_d. The modular limit is, then, E_d/E_u
4. Determine the downstream depth for the given discharge. This can be done using the Manning equation for uniform flow, or by another equation when a backwater profile exists from a downstream control
5. Compare the calculated downstream water level with the value of h_d, which is equal to E_d minus the corresponding downstream velocity head. If the calculated water level is less than or equal to h_d, the flow will be modular because the actual head loss is greater than that required for modular flow

Lecture 10

Broad Crested Weirs

I. Calibration by Energy Balance

- The complete calibration of a broad-crested weir includes the calculation of head losses across the structure. However, the calibration can be made assuming no losses in the converging and throat sections, and the resulting values will usually be very close to those obtained by the complete theoretical calibration
- The procedure which is presented below is useful to illustrate the hydraulic principles which govern the broad-crested weir characteristics, and to check the calibration of an existing structure in the field with a programmable calculator
- The simplified calibration approach does not include the calculation of the modular limit; however, this is an important consideration in the design and operation of a broad-crested weir because the structure is usually intended to operate under modular flow conditions

(1) The specific energy of the flow upstream of the broad-crested weir can be set equal to the specific energy over the crest (or sill) of the structure. The energy balance can be expressed mathematically as follows:

$$h_u + z_u + \frac{V_u^2}{2g} = h_c + z_u + \frac{V_c^2}{2g} \tag{1}$$

where h_u is the upstream flow depth, *referenced from the sill elevation*; V_u is the average velocity in the upstream section, based on a depth of $(h_u + z_u)$; z_u is the height of the sill above the upstream bed; h_c is the depth over the crest where critical flow is *assumed* to occur; and V_c is the average velocity in the critical flow section over the crest.

Note that the z_u term cancels from Eq. 1.

Recognizing that Q = VA, where Q is the volumetric flow rate (discharge) and A is the area of the flow cross-section,

$$h_c - h_u + \frac{Q^2}{2gA_c^2} - \frac{Q^2}{2gA_u^2} = 0 \tag{2}$$

which can be reduced to,

$$h_c - h_u + \frac{Q^2}{2g}\left(\frac{1}{A_c^2} - \frac{1}{A_u^2}\right) = 0 \tag{3}$$

(2) Critical flow over the crest can be defined by the Froude number, which is equal to unity for critical flow. Thus, the square of the discharge over the crest can be defined as follows:

$$Q^2 = \frac{g A_c^3}{T_c} \tag{4}$$

where T_c is the width of the water surface over the crest. This last equation for Q^2 can be combined with the equation for energy balance to produce the following:

$$h_c - h_u + \frac{A_c^3}{2 T_c}\left(\frac{1}{A_c^2} - \frac{1}{A_u^2}\right) = 0 \tag{5}$$

This last equation can be solved by trial-and-error, or by any other iterative method, knowing h_u, z_u, and the geometry of the upstream and throat cross-sections. The geometry of the sections defines the relationship between h_c and A_c, and between h_u and A_u (<u>important:</u> if you look carefully at the above equations, you will see that A_u must be calculated based on a depth of $h_u + z_u$). The solution to Eq. 5 gives the value of h_c.

(3) The final step is to calculate the discharge corresponding to the value of A_c, which is calculated directly from h_c. This is done using the following form of the Froude number equation:

$$Q = \sqrt{\frac{g A_c^3}{T_c}} \tag{6}$$

This process is repeated for various values of the upstream flow depth, and in the end a table of values for upstream depth and discharge will have been obtained. From this table a staff gauge can be constructed. This simple calibration assumes that the downstream flow level is not so high that non-modular flow exists across the structure.

- See the computer program listing on the following two pages
- In the design of broad-crested weirs it is often necessary to consider other factors which limit the allowable dimensions, and which restrict the flow conditions for which the calibration is accurate
- Complete details on broad-crested weir design, construction, calibration, and application can be found in the book "Flow Measurement Flumes for Open Channel Systems", 1984, by M.G. Bos, J.A. Replogle, y A.J. Clemmens

```
//--------------------------------------------------------------------------
// Broad crested weir calibration for free flow by energy balance equation.
// Written in Object Pascal (Delphi 6) by Gary Merkley.  September 2004.
//--------------------------------------------------------------------------
unit BCWmain;

interface

uses
  Windows, Messages, SysUtils, Classes, Graphics, Controls, Forms, Dialogs,
  StdCtrls, Buttons;

type
  TWmain = class(TForm)
    btnStart: TBitBtn;
    procedure btnStartClick(Sender: TObject);
  private
    function NewtonRaphson(hu:double):double;
    function EnergyFunction(hc:double):double;
    function Area(h,b,m:double):double;
    function TopWidth(h,b,m:double):double;
  end;

var
  Wmain: TWmain;

implementation
{$R *.DFM}

const
  g =  9.810;           // weight/mass (m/s2)
  bu = 2.000;           // base width upstream (m)
  mu = 1.250;           // side slope upstream (H:V)
  zu = 1.600;           // upstream sill height (m)
  bc = 6.000;           // base width at control section (m)
  mc = 1.250;           // side slope at control section (H:V)
  L =  1.500;           // sill length (m)

var
  hu,Au,hc: double;

function TWmain.NewtonRaphson(hu:double):double;
//--------------------------------------------------------------------------
// Newton-Raphson method to solve for critical depth.  Returns flow rate.
//--------------------------------------------------------------------------
var
  i,iter: integer;
  dhc,F,Fdhc,change,Ac,Tc: double;
begin
  result:=0.0;

  for i:=1 to 9 do begin

    hc:=0.1*i*hu;

    for iter:=1 to 50 do begin

      dhc:=0.0001*hc;
      F:=EnergyFunction(hc);
      Fdhc:=EnergyFunction(hc+dhc);
      change:=Fdhc-F;
      if abs(change) < 1.0E-12 then break;
      change:=dhc*F/change;
      hc:=hc-change;

      if (abs(change) < 0.001) and (hc >= 0.001) then begin
        Ac:=Area(hc,bc,mc);
        Tc:=TopWidth(hc,bc,mc);
        result:=sqrt(g*Ac*Ac*Ac/Tc);
```

```
        Exit;
      end;
    end;
  end;
end;

function TWmain.EnergyFunction(hc:double):double;
//-----------------------------------------------------------------------------
// Energy balance function (specific energy), equal to zero.
//-----------------------------------------------------------------------------
var
   Ac,Tc: double;
begin
  Ac:=Area(hc,bc,mc);
  Tc:=TopWidth(hc,bc,mc);
  result:=hc-hu+0.5*Ac*Ac*Ac*(1.0/(Ac*Ac)-1.0/(Au*Au))/Tc;
end;

function TWmain.Area(h,b,m:double):double;
//-----------------------------------------------------------------------------
// Calculates cross-section area for symmetrical trapezoidal shape.
//-----------------------------------------------------------------------------
begin
  result:=h*(b+m*h);
end;

function TWmain.TopWidth(h,b,m:double):double;
//-----------------------------------------------------------------------------
// Calculates top width of flow for symmetrical trapezoidal shape.
//-----------------------------------------------------------------------------
begin
  result:=b+2.0*m*h;
end;

procedure TWmain.btnStartClick(Sender: TObject);
//-----------------------------------------------------------------------------
// Entry point for calculations (user clicked the Start button).
//-----------------------------------------------------------------------------
var
  i: integer;
  F: TextFile;
  strg: string;
  Q,humin,humax: double;
begin
  AssignFile(F,'BCWenergy.txt');
  Rewrite(F);
  Writeln(F,'      hc (m)        hu (m)        Q (m3/s)');

  humin:=0.075*L;
  humax:=0.75*L;

  hu:=humin;

  for i:=0 to 100 do begin
    Au:=Area(hu+zu,bu,mu);
    Q:=NewtonRaphson(hu);
    strg:=Format('%12.3f%12.3f%12.3f',[hc,hu,Q]);
    Writeln(F,strg);

    hu:=hu+0.03;
    if hu > humax then break;
  end;

  CloseFile(F);
end;

end.
```

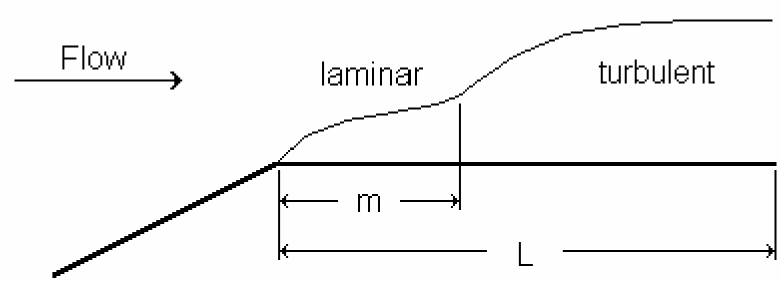
II. Calculation of Head Loss

Throat (Control Section)

Head loss in the throat (where the critical flow control section is assumed to be located) can be estimated according to some elements from boundary layer theory. The equation is (Schlichting 1960):

$$\left(h_f\right)_{throat} = \frac{C_F L V_c^2}{2gR} \tag{7}$$

where L is the length of the sill; V_c is the average velocity in the throat section; and R is the hydraulic radius of the throat section. The values of V and R can be taken for critical depth in the throat section. The *drag coefficient*, C_F, is estimated by assuming the sill acts as a thin flat plat with both laminar and turbulent flow, as shown in the figure below (after Bos, Replogle and Clemmens 1984).



The drag coefficient is calculated by assuming all turbulent flow, subtracting the turbulent flow portion over the length L_m, then adding the laminar flow portion for the length L_m. Note that C_F is dimensionless.

$$C_F = C_{T,L} - \left(\frac{m}{L}\right) C_{T,m} + \left(\frac{m}{L}\right) C_{L,m} \tag{8}$$

where $C_{L,m}$ is the thin-layer laminar flow coefficient over the distance m, which begins upstream of the weir crest:

$$C_{L,m} = \frac{1.328}{\sqrt{\left(R_e\right)_m}} \tag{9}$$

When $(R_e)_L < (R_e)_m$, the flow is laminar over the entire crest and $C_F = C_{L,L}$, where $C_{L,L}$ is defined by Eq. 9.

The $C_{T,L}$ and $C_{T,m}$ coefficients are calculated by iteration from:

$$0.544\sqrt{C_{T,x}} = C_{T,x}\left[5.61\sqrt{C_{T,x}} - \ln\left(\frac{1}{Re_x\, C_{T,x}} + \frac{k}{4.84x\sqrt{C_{T,x}}}\right) - 0.638\right] \quad (10)$$

where x is equal to L or m, for $C_{T,L}$ and $C_{T,m}$, respectively; Re is the Reynolds number; and k is the absolute roughness height. All values are in m and m^3/s. Below are some sample values for the roughness, k.

Material and Condition	Roughness, k (mm)
Glass	0.001 to 0.01
Smooth Metal	0.02 to 0.1
Rough Metal	0.1 to 1.0
Wood	0.2 to 1.0
Smooth Concrete	0.1 to 2.0
Rough Concrete	0.5 to 5.0

The Reynolds number can be calculated as follows:

$$\left(R_e\right)_x = \frac{V\,x}{\nu} \quad (11)$$

where ν is the kinematic viscosity (a function of temperature); x is equal to m or L; and V is the average velocity in the throat section for critical flow. If $(R_e)_L$ is less than $(R_e)_m$, the boundary layer over the sill is laminar only, and $C_F = C_{L,m}$.

The value of m can be estimated as:

$$m = \frac{\nu}{V}\left(350{,}000 + \frac{L}{k}\right) \quad (12)$$

where the units are $(m^2/s)/(m/s) = m$

Diverging Section

The head loss in the downstream diverging section is estimated as:

$$\left(h_f\right)_{ds} = \frac{\xi\left(V_c - V_d\right)^2}{2g} \quad (13)$$

where $(h_f)_{ds}$ is the head loss in the diverging section (m); V_c is the average velocity in the control section, at critical depth (m/s); and V_d is the average velocity in the downstream section (m/s), using h_d referenced to the downstream channel bed elevation (not the sill crest).

The coefficient ξ is defined as:

$$\xi = \frac{\log_{10}\left[114.6\tan^{-1}\left(z_d/L_d\right)\right] - 0.165}{1.742} \tag{14}$$

where L_d is the length of the downstream ramp; and z_d is the height of the downstream ramp, also equal to the difference in elevation between the sill and the downstream bed elevation. Note that the recommended value for the z_d/L_d ratio is 1/6.
If the downstream ramp is not used, L_d equals zero. In this case, assume ξ = 1.2.

Converging Section

After having calculated the C_F drag coefficient for the throat section, and assuming the same roughness value for the channel and structure from the gauge location to the control section, the upstream losses can be estimated. These are the losses from the gauge location to the beginning of the sill.

For the section from the gauge to the beginning of the upstream ramp, the head loss is estimated as:

$$\left(h_f\right)_{gauge} = \left(\frac{C_F\,L_{gauge}}{R_u}\right)\frac{V_u^{\,2}}{2g} \tag{15}$$

where L_{gauge} is the distance from the gauge to the beginning of the upstream ramp; V_u is the average velocity in the upstream section (at the gauge); and R_u is the hydraulic radius at the gauge. All values are in metric units (m and m/s).

For the upstream ramp, the same equation can be used, but the hydraulic radius changes along the ramp. Therefore,

$$\left(h_f\right)_{us} = \frac{1}{2}\left(\frac{C_F\,L_u}{2g}\right)\left(\frac{V_u^2}{R_u} + \frac{V_r^2}{R_r}\right) \tag{16}$$

where the values of V_r and R_r, at the entrance to the throat section (top of the ramp), are estimated by calculating the depth at this location:

$$h_r = h_c + 0.625\left(h_u - h_c\right) \tag{17}$$

III. Photographs of BCW Construction

References & Bibliography

Lecture 11

Calibration of Canal Gates

I. Suitability of Gates for Flow Measurement

Advantages

- relatively low head loss
- often already exists as a *control device*
- sediment passes through easily

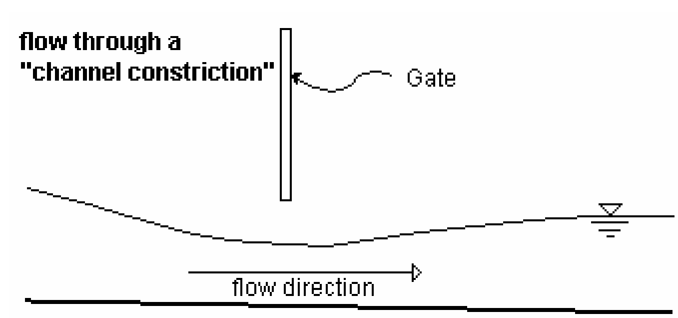
Disadvantages

- usually not as accurate as weirs
- floating debris tends to accumulate
- calibration can be complex for all flow conditions

II. Orifice and Non-Orifice Flow

FO	SO
FN	SN

- Canal gates can operate under orifice flow conditions and as channel constrictions
- In general, either condition can occur under free or submerged (modular or non-modular) regimes
- Orifice flow occurs when the upstream depth is sufficient to "seal" the opening – in other words, the bottom of the gate is lower than the upstream water surface elevation
- The difference between free and submerged flow for a gate operating as an orifice is that for free flow the downstream water surface elevation is less than C_cG_o, where C_c is the contraction coefficient and G_o is the vertical gate opening, referenced from the bottom of the gate opening (other criteria can be derived from momentum principles)
- The distinguishing difference between free and submerged flow in a channel constriction is the occurrence of critical velocity in the vicinity of the constriction (usually a very short distance upstream of the narrowest portion of the constriction)
- This lecture focuses on the calibration of gates under non-orifice flow conditions



III. Rating Open Channel Constrictions

- Whenever doing structure calibrations, examine the data carefully and try to identify erroneous values or mistakes
- Check the procedures used in the field or laboratory because sometimes the people who take the measurements are not paying attention and make errors
- Never blindly enter data into a spreadsheet or other computer program and accept the results at "face value" because you may get incorrect results and not even realize it
- Always graph the data and the calibration results; don't perform regression and other data analysis techniques without looking at a graphical representation of the data and comparison to the results

Free Flow

- Gates perform hydraulically as open channel constrictions (non-orifice flow) when the gate is raised to the point that it does not touch the water surface
- The general form of the free-flow equation is:

$$Q_f = C_f h_u^{n_f} \tag{1}$$

where the subscript f denotes free flow; Q_f is the free-flow discharge; C_f is the free-flow coefficient; and n_f is the free-flow exponent

- The value of C_f increases as the size of the constriction increases, but the relationship is usually not linear
- The value of n_f is primarily dependent upon the geometry of the constriction with the theoretical values being 3/2 for a rectangular constriction and 5/2 for a triangular constriction

Sample Free-Flow Constriction Calibration

- Sample field data for developing the discharge rating for a rectangular open-channel constriction are listed in the table below
- The discharge rate in the constriction was determined by taking current meter readings at an upstream location, and again at a downstream location

- This is a good practice because the upstream and downstream flow depths are often significantly different, so that the variation in the measured discharge between the two locations is indicative of the accuracy of the current meter equipment and the methodology used by the field staff

Date	Discharge (m^3/s)	Water Surface Elevation in Stilling Well (m)
21 Jun 86	0.628	409.610
21 Jun 86	1.012	409.935
21 Jun 86	1.798	410.508
21 Jun 86	2.409	410.899

Note: The listed discharge is the average discharge measured with a current meter at a location 23 m upstream of the constriction, and at another location 108 m downstream.

- The free-flow equation for the flow depths measured below the benchmark (at 411.201 m) is:

$$Q_f = 0.74\left(h_u\right)_x^{1.55} \tag{2}$$

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5
Discharge (m^3/s)	**Water Surface Elevation (m)**	**$(h_u)_{sw}$ (m)**	**Tape Measurement (m)**	**$(h_u)_x$ (m)**
0.628	409.610	0.918	1.604	0.905
1.009	409.935	1.243	1.294	1.215
1.797	410.508	1.816	0.734	1.775
2.412	410.899	2.207	0.358	2.151

Notes: The third column values equal the values in column 2 minus the floor elevation of 408.692 m. The values in column 5 equal the benchmark elevation of 411.201 m minus the floor elevation of 408.692 m, minus the values in column 4.

- The "Tape Measurement" in the above table is for the vertical distance from the benchmark down to the water surface
- If a regression analysis is performed with the free-flow data using the theoretical value of $n_f = 3/2$,

$$Q_f = 0.73\left(h_u\right)_{sw}^{1.5} \tag{3}$$

or,

$$Q_f = 0.75\left(h_u\right)_x^{1.5} \tag{4}$$

- The error in the discharge resulting from using $n_f = 3/2$ varies from -1.91% to +2.87%.

Submerged Flow

- The general form of the submerged-flow equation is:

$$Q_s = \frac{C_s\left(h_u - h_d\right)^{n_f}}{\left(-\log S\right)^{n_s}} \tag{5}$$

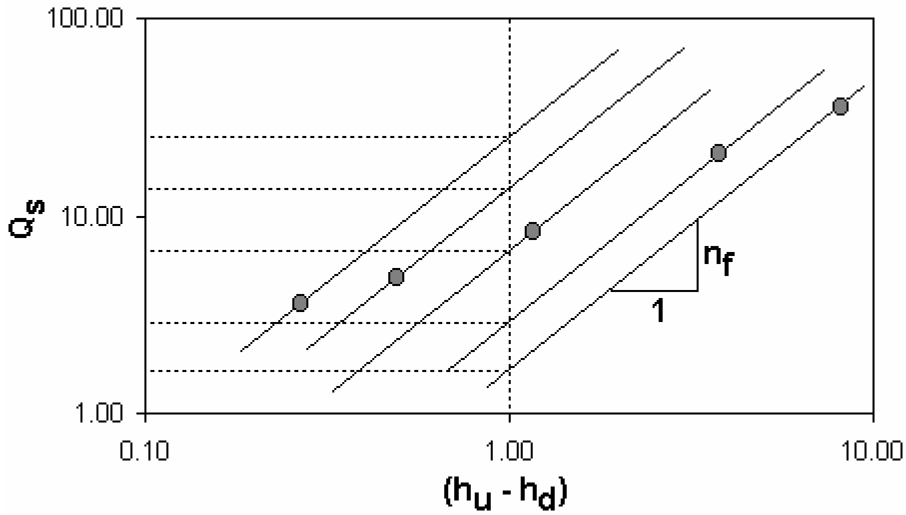
where the subscript s denotes submerged flow, so that Q_s is the submerged-flow discharge, C_s is the submerged-flow coefficient, and n_s is the submerged-flow exponent

- Base 10 logarithms have usually been used with Eq. 5, but other bases could be used, so the base should be specified when providing calibration values
- Note that the free-flow exponent, n_f, is used with the term h_u - h_d
- Consequently, n_f is determined from the free-flow rating, while C_s and n_s must be evaluated using submerged-flow data
- The theoretical variation in n_s is between 1.0 and 1.5
- Note that the logarithm term in the denominator of Eq. 5 can be estimated by taking the first two or three terms of an infinite series (but this is not usually necessary):

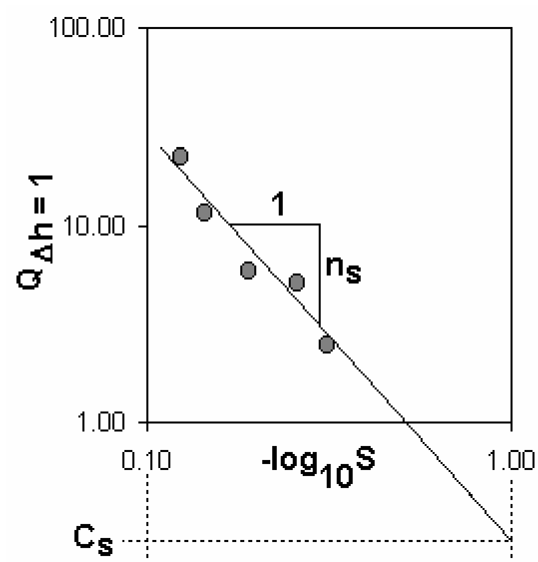
$$\log_e(1+x) = x - \frac{1}{2}x^2 + \frac{1}{3}x^3 - \frac{1}{4}x^4 + \ldots \tag{6}$$

Graphical Solution for Submerged-Flow Calibration

- The graphical solution used to be performed by hand on log-log paper before PCs and programmable calculators became widely available
- This is essentially how the form of the submerged-flow equation was derived in the 1960's
- This solution technique assumes that the free-flow exponent, n_f, is known from a prior free-flow calibration at the same structure
- So, assuming you already know n_f, and you have data for submerged flow conditions, you can plot Q_s versus (h_u – h_d), as shown in the figure below where there are five measured points



- The above graph has a log-log scale
- The slope of the parallel lines is equal to n_f, as shown above (measured using a linear, not log, scale), and each line passes through one of the plotted data points
- The five values of Q_s at each of the horizontal dashed lines are for $\Delta h = 1.0$
- Get another sheet of log-log paper and plot the five $Q_{\Delta h = 1.0}$ values versus -$\log_{10}S$, as shown in the figure below



- Draw a straight line through the five data points and extend this line to $-\log_{10}S = 1.0$ (as shown above)
- The value on the ordinate at $-\log_{10}S = 1.0$ is C_s
- The slope of the line is $-n_s$ (measured with a linear scale)
- You might not prefer to use this method unless you have no computer

Submerged-Flow Calibration by Multiple Regression

- Multiple regression analysis can be used to arrive directly at all three values (C_s, n_f, and n_s) without free-flow data
- This can be done by taking the logarithm of Eq. 5 as follows:

$$\log Q_s = \log C_s + n_f \log(h_u - h_d) - n_s \log(-\log S) \tag{7}$$

- Equation 7 is linear with respect to the unknowns C_s, n_f, and n_s
- Such a procedure may be necessary when a constriction is to be calibrated in the field and only operates under submerged-flow conditions
- You can also compare the calculated n_f values for the free- and submerged-flow calibrations (they should be nearly the same)
- Spreadsheet applications and other computer software can be used to perform multiple least-squares regression conveniently
- This method can give a good fit to field or laboratory data, but it tends to complicate the calculation of transition submergence, which is discussed below

Sample Submerged-Flow Constriction Calibration

- In this example, a nearly constant discharge was diverted into the irrigation channel and a check structure with gates located 120 m downstream was used to incrementally increase the flow depths
- Each time that the gates were changed, it took 2-3 hours for the water surface elevations upstream to stabilize
- Thus, it took one day to collect the data for a single flow rate
- The data listed in the table below were collected in two consecutive days

Date	Discharge (m^3/s)	Tape Measurement from U/S Benchmark (m)	Tape Measurement from D/S Benchmark (m)
22 Jun 86	0.813	1.448	1.675
22 Jun 86	0.823	1.434	1.605
22 Jun 86	0.825	1.418	1.548
22 Jun 86	0.824	1.390	1.479
22 Jun 86	0.793	1.335	1.376
23 Jun 86	1.427	0.983	1.302
23 Jun 86	1.436	0.966	1.197
23 Jun 86	1.418	0.945	1.100
23 Jun 86	1.377	0.914	1.009
23 Jun 86	1.241	0.871	0.910

Q_s (m^3/s)	$(h_u)_x$ (m)	$(h_d)_x$ (m)	S	$-\log_{10}S$	$Q_{\Delta h=1}$
0.813	1.061	0.832	0.784	0.1057	7.986
0.823	1.075	0.902	0.839	0.0762	12.486
0.825	1.091	0.959	0.879	0.0560	19.036
0.824	1.119	1.028	0.919	0.0367	33.839
0.793	1.174	1.131	0.963	0.0164	104.087
1.427	1.526	1.205	0.790	0.1024	8.305
1.436	1.543	1.310	0.849	0.0711	13.733
1.418	1.564	1.407	0.900	0.0458	25.005
1.377	1.595	1.498	0.939	0.0273	51.220
1.241	1.638	1.597	0.975	0.0110	175.371

- A logarithmic plot of the submerged-flow data can be made
- Each data point can have a line drawn at a slope of n_f = 1.55 (from the prior free-flow data analysis), which can be extended to where it intercepts the abscissa at h_u - h_d = 1.0
- Then, the corresponding value of discharge can be read on the ordinate, which is listed as $Q_{\Delta h=1.0}$ in the above table
- The value of $Q_{\Delta h=1.0}$ can also be determined analytically because a straight line on logarithmic paper is an exponential function having the simple form:

$$\frac{Q_s}{Q_{\Delta h=1}} = \frac{\dfrac{C_s\left(h_u - h_d\right)^{n_f}}{\left(-\log S\right)^{n_s}}}{\dfrac{C_s\left(1\right)^{n_f}}{\left(-\log S\right)^{n_s}}} = \left(h_u - h_d\right)^{n_f} \tag{8}$$

then,

$$Q_s = Q_{\Delta h=1.0}\left(h_u - h_d\right)^{n_f} \tag{9}$$

or,

$$Q_{\Delta h=1.0} = \frac{Q_s}{\left(h_u - h_d\right)^{n_f}} \tag{10}$$

where $Q_{\Delta h=1.0}$ has a different value for each value of the submergence, S

- Using the term $Q_{\Delta h=1.0}$ implies that h_u - h_d = 1.0 (by definition); thus, Eq. 10 reduces to:

$$Q_{\Delta h=1.0} = \frac{C_s\left(1.0\right)^{n_f}}{\left(h_u - h_d\right)^{n_f}} = C_s\left(-\log S\right)^{-n_s} \tag{11}$$

- Again, this is a power function where $Q_{\Delta h=1.0}$ can be plotted against (-log S) on logarithmic paper to yield a straight-line relationship
- Note that the straight line in such a plot would have a negative slope ($-n_s$) and that C_s is the value of $Q_{\Delta h=1.0}$ when (-log S) is equal to unity
- For the example data, the submerged-flow equation is:

$$Q_s = \frac{0.367\left(h_u - h_d\right)^{1.55}}{\left(-\log S\right)^{1.37}} \tag{12}$$

Transition Submergence

- By setting the free-flow discharge equation equal to the submerged-flow discharge equation, the transition submergence, S_t, can be determined
- Consider this:

$$C_f h_u^{n_f} = \frac{C_s\left(1 - S_t\right)^{n_f} h_u^{n_f}}{\left(-\log S_t\right)^{n_s}} \tag{13}$$

then,

$$f\left(S_t\right) = C_f\left(-\log S_t\right)^{n_s} - C_s\left(1 - S_t\right)^{n_f} = 0 \tag{14}$$

- In our example, we have:

$$0.74\left(-\log S\right)^{1.37} = 0.367\left(1 - S\right)^{1.55} \tag{15}$$

or,

$$0.74 h_u^{1.55} = \frac{0.367\left(h_u - h_d\right)^{1.55}}{\left(-\log S\right)^{1.37}} \tag{16}$$

and,

$$0.74\left(-\log S\right)^{1.37} = 0.367\left(1 - S\right)^{1.55} \tag{17}$$

- The value of S in this relationship is S_t provided the coefficients and exponents have been accurately determined
- Again, small errors will dramatically affect the determination of S_t

$$0.74\left(-\log S_t\right)^{1.37} = 0.367\left(1 - S_t\right)^{1.55} \tag{18}$$

- Equation 18 can be solved to determine the value of S_t, which in this case is 0.82
- Thus, free flow exists when S < 0.82 and submerged flow exists when the submergence is greater than 82%
- The table below gives the submergence values for different values of Q_s/Q_f for the sample constriction rating

S	**Q_s/Q_f**	**S**	**Q_s/Q_f**
0.82	1.000	0.91	0.9455
0.83	0.9968	0.92	0.9325
0.84	0.9939	0.93	0.9170
0.85	0.9902	0.94	0.8984
0.86	0.9856	0.95	0.8757
0.87	0.9801	0.96	0.8472
0.88	0.9735	0.97	0.8101
0.89	0.9657	0.98	0.7584
0.90	0.9564	-	-

$$\frac{Q_s}{Q_f} = \frac{0.367\left(h_u - h_d\right)^{1.55}}{\left(-\log S\right)^{1.37}} = \frac{1}{0.74 h_u^{1.55}} \tag{19}$$

which is also equal to:

$$\frac{Q_s}{Q_f} = \frac{0.496\left(1 - S\right)^{1.55}}{\left(-\log S\right)^{1.37}} \tag{20}$$

- For example, if h_u and h_d are measured and found to be 1.430 and 1.337, the first step would be to compute the submergence, S,

$$S = \frac{1.337}{1.430} = 0.935 \tag{21}$$

- Thus, for this condition submerged flow exists in the example open-channel constriction
- In practice, there may only be a “trivial” solution for transition submergence, in which $S_t = 1.0$. In these cases, the value of C_s can be slightly lowered to obtain another mathematical root to the equation. This is a “tweaking” procedure.
- Note that it is almost always expected that $0.50 < S_t < 0.92$. If you come up with a value outside of this range, you should be suspicious that the data and or the analysis might have errors.

IV. Constant-Head Orifices

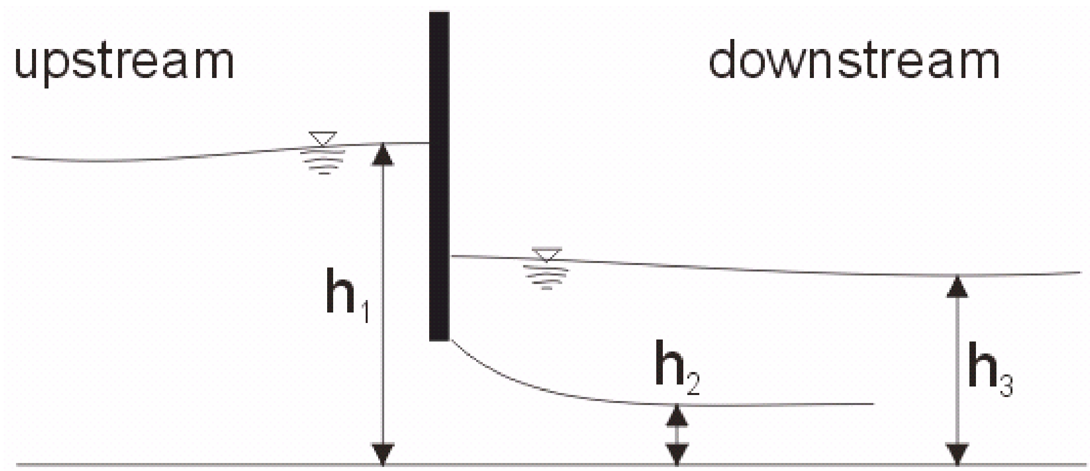
- A constant-head orifice, or CHO, is a double orifice gate, usually installed at the entrance to a lateral or tertiary canal
- This is a design promoted for years by the USBR, and it can be found in irrigation canals in many countries
- The idea is that you set the downstream gate as a meter, and set the upstream gate as necessary to have a constant water level in the mid-gate pool
- It is kind of like the double doors in the engineering building at USU: they are designed to act as buffers whereby the warm air doesn’t escape so easily when people enter an exit the building
- But, in practice, CHOs are seldom used as intended; instead, one of the gates is left wide open and the other is used for regulation (this is a waste of materials because one gate isn’t used at all)
- Note the missing wheel in the upstream gate, in the above figure
- Few people know what CHOs are for, and even when they do, it is often considered inconvenient or impractical to operate both gates
- But these gates can be calibrated, just as with any other gate

V. General Hydraulic Characteristics of Gates for Orifice Flow

- It is safe to assume that the exponent on the head for orifice flow (free and submerged) is 0.50, so it is not necessary to treat it as an empirically-determined calibration parameter
- The basic relationship for orifice flow can be derived from the Bernoulli equation
- For orifice flow, a theoretical contraction coefficient, C_c, of 0.611 is equal to $\pi/(\pi+2)$, derived from hydrodynamics for vertical flow through an infinitely long slot
- Field-measured discharge coefficients for orifice flow through gates normally range from 0.65 to about 0.9 – there is often a significant approach velocity
- Radial gates can be field calibrated using the same equation forms as vertical sluice gates, although special equations have been developed for them

VI. How do You Know if it is Free Flow?

- If the water level on the upstream side of the gate is above the bottom of the gate, then the flow regime is probably that of an orifice
- In this case, the momentum function (from open-channel hydraulics) can be used to determine whether the flow is free or submerged
- In some cases it will be obviously free flow or obviously submerged flow, but the following computational procedure is one way to distinguish between free orifice and submerged orifice flow
- In the figure below, h_1 is the depth just upstream of the gate, h_2 is the depth just downstream of the gate (depth at the vena-contracta section) and h_3 is the depth at a section in the downstream, a short distance away from the gate



- If the value of the momentum function corresponding to h_2 is greater than that corresponding to h_3, free flow will occur; otherwise, it is submerged
- The momentum function is:

$$M = A h_c + \frac{Q^2}{gA} \tag{22}$$

where A is the cross-sectional area and h_c is the depth to the centroid of the area from the water surface

- The following table shows the values of A and Ah_c for three different channel sections

Section	A	Ah_c
Rectangular	bh	$\frac{bh^2}{2}$
Trapezoidal	$\left[b+\left(\frac{m_1+m_2}{2}\right)h\right]h$	$\left[\frac{b}{2}+\left(\frac{m_1+m_2}{6}\right)h\right]h^2$
Circular	$\frac{D^2}{8}(\theta-\sin\theta)$	$\frac{D^3}{4}\left[\frac{\sin^3(\theta/2)}{3}-\frac{\cos(\theta/2)(\theta-\sin\theta)}{4}\right]$

- For rectangular and trapezoidal sections, b is the base width
- For trapezoidal sections, m_1 and m_2 are the inverse side slopes (zero for vertical sides), which are equal for symmetrical sections

- For circular sections, θ is defined as: $\theta = 2\cos^{-1}\left(1-\frac{2h}{D}\right)$

  where D is the inside diameter of the circular section

- There are alternate forms of the equations for circular sections, but which yield the same calculation results.

- The depth h_2 can be determined in the either of the following two ways:

  1. $h_2 = C_c\, G_o$ where C_c is the contraction coefficient and G_o is the vertical gate opening
  2. Equate the specific energies at sections 1 and 2, where $E = h + \frac{Q^2}{2gA^2}$, then solve for h_2

- Calculate M_2 and M_3 using the depths h_2 and h_3, respectively
- If $M_2 \le M_3$, the flow is submerged; otherwise the flow is free

- Another (simpler) criteria for the threshold between free- and submerged-orifice flow is:

$$C_cG_o = h_d \tag{23}$$

where C_c is the contraction coefficient ($\approx$ 0.61)

- But this is not the preferred way to make the distinction and is not as accurate as the momentum-function approach

VII. How do You Know if it is Orifice Flow?

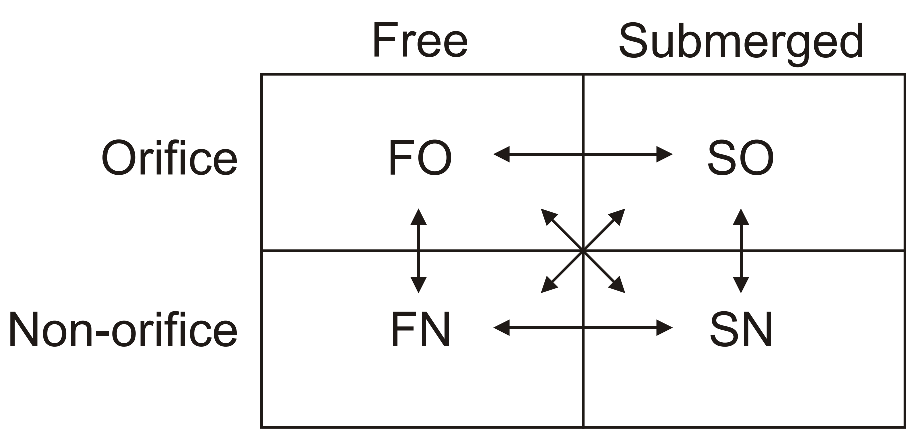
- The threshold between orifice and nonorifice flow can be defined as:

$$C_o h_u = G_o \tag{24}$$

where C_o is an empirically-determined coefficient ($0.80 \le C_o \le 0.95$)

- Note that if C_o = 1.0, then when $h_u = G_o$, the water surface is on the verge of going below the bottom of the gate (or vice-versa), when the regime would clearly be nonorifice
- However, in moving from orifice to nonorifice flow, the transition would begin before this point, and that is why C_o must be less than 1.0
- It seems that more research is needed to better defined the value of C_o

- In practice, the flow can move from any regime to any other at an underflow (gate) structure:



VII. Orifice Ratings for Canal Gates

- For free-flow conditions through an orifice, the discharge equation is:

$$Q_f = C_d C_v A \sqrt{2gh_u} \tag{25}$$

where C_d is the dimensionless discharge coefficient; C_v is the dimensionless velocity head coefficient; A is the area of the orifice opening, g is the ratio of weight to mass; and h_u is measured from the centroid of the orifice to the upstream water level

- The velocity head coefficient, C_v, approaches unity as the approach velocity to the orifice decreases to zero
- In irrigation systems, C_v can usually be assumed to be unity since most irrigation channels have very flat gradients and the flow velocities are low
- The upstream depth, h_u, can also be measured from the bottom of the orifice opening if the downstream depth is taken to be about 0.61 times the vertical orifice opening
- Otherwise, it is assumed that the downstream depth is equal to one-half the opening, and h_u is effectively measured from the area centroid of the opening
- The choice will affect the value of the discharge coefficient

- If h_u is measured from the upstream canal bed, Eq. 4 becomes

$$Q_f = C_d C_v A \sqrt{2g(h_u - C_g G_o)} \quad (26)$$

where C_g is usually either 0.5 or 0.61, as explained above.

- If the downstream water level is also above the top of the opening, submerged conditions exist and the discharge equation becomes:

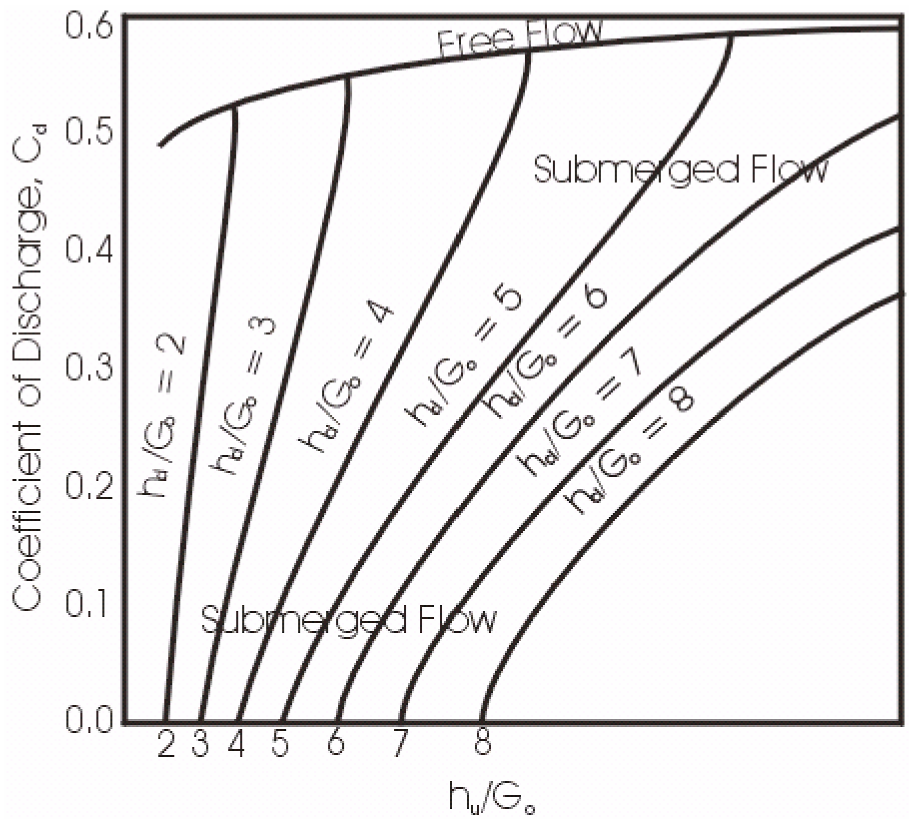
$$Q_s = C_d C_v A \sqrt{2g\left(h_u - h_d\right)} \quad (27)$$

where h_u - h_d is the difference in water surface elevations upstream and downstream of the submerged orifice.

- An orifice can be used as an accurate flow measuring device in an irrigation system
- If the orifice structure has not been previously rated in the laboratory, then it can easily be rated in the field
- As mentioned above, the hydraulic head term, (h_u - $C_g G_o$) or (h_u - h_d), can be relied upon to have the exponent ½, which means that a single field rating measurement *could* provide an accurate determination of the coefficient of discharge, C_d
- However, the use of a single rating measurement implies the assumption of a constant C_d value, which is not the case in general
- Adjustments to the basic orifice equations for free- and submerged-flow are often made to more accurately represent the structure rating as a function of flow depths and gate openings
- The following sections present some alternative equation forms for taking into account the variability in the discharge coefficient under different operating conditions

VIII. Variation in the Discharge Coefficient

- Henry (1950) made numerous laboratory measurements to determine the discharge coefficient for free and submerged flow through an orifice gate
- The figure below shows an approximate representation of his data



Lecture 12

Calibration of Canal Gates

I. Free-Flow Rectangular Gate Structures

- For a rectangular gate having a gate opening, G_o, and a gate width, W, the free-flow discharge equation can be obtained from Eq. 26 of the previous lecture, assuming that the dimensionless velocity head coefficient is equal to unity:

$$Q_f = C_d G_o G_w \sqrt{2g\left(h_u - C_g G_o\right)} \tag{1}$$

where G_o is the vertical gate opening; G_w is the gate width; G_oG_w is the area, A, of the gate opening; and, C_g is between 0.5 and 0.61

- The upstream flow depth, h_u, can be measured anywhere upstream of the gate, including the upstream face of the gate
- The value of h_u will vary only a small amount depending on the upstream location chosen for measuring h_u
- Consequently, the value of the coefficient of discharge, C_d, will also vary according to the location selected for measuring h_u

- One of the most difficult tasks in calibrating a gate structure is obtaining a highly accurate measurement of the gate opening, G_o
- For gates having a threaded rod that rises as the gate opening is increased, the gate opening is read from the top of the hand-wheel to the top of the rod with the gate closed, and when set to some opening, G_o
- This very likely represents a measurement of gate opening from where the gate is totally seated, rather than a measurement from the gate lip; therefore, the measured value of G_o from the thread rod will usually be greater than the true gate opening, unless special precautions are taken to calibrate the thread rod

- Also, when the gate lip is set at the same elevation as the gate sill, there will undoubtedly be some flow or leakage through the gate
- This implies that the datum for measuring the gate opening is below the gate sill
- In fact, there is often leakage from a gate even when it is totally seated because of inadequate maintenance
- An example problem will be used to illustrate the procedure for determining an appropriate zero datum for the gate opening

Sample Calibration

- Calibration data (listed in the table below) were collected for a rectangular gate structure
- The data reduction is listed in the next table, where the coefficient of discharge, C_d, was calculated from Eq. 27

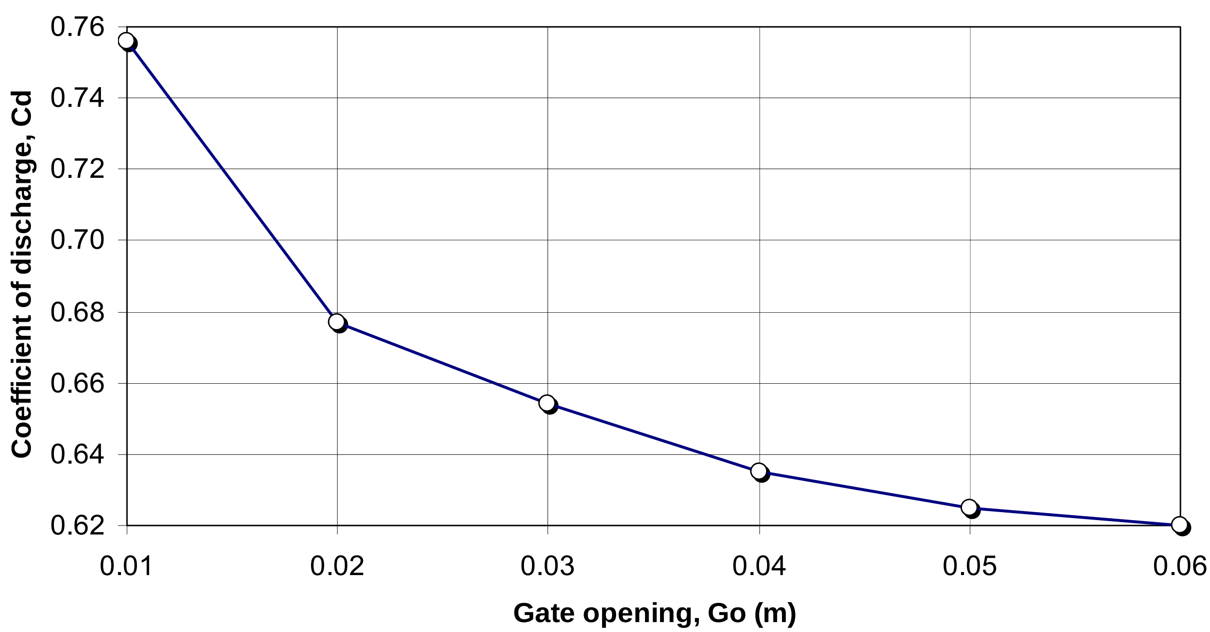
Discharge, Q_f (m^3/s)	Gate Opening, G_o (m)	Upstream Benchmark Tape Measurement (m)
0.0646	0.010	0.124
0.0708	0.020	1.264
0.0742	0.030	1.587
0.0755	0.040	1.720
0.0763	0.050	1.787
0.0767	0.060	1.825

Q_f (m^3/s)	G_o (m)	h_u (m)	C_d
0.0646	0.010	1.838	0.756
0.0708	0.020	0.698	0.677
0.0742	0.030	0.375	0.654
0.0755	0.040	0.242	0.635
0.0763	0.050	0.175	0.625
0.0767	0.060	0.137	0.620

Note: The discharge coefficient, C_d, was calculated using the following equation:

$$Q_f = C_d G_o G_w \sqrt{2g\left(h_u - \frac{G_o}{2}\right)}$$

- A rectangular coordinate plot of C_d versus the gate opening, G_o, is shown in the figure below



- Notice that the value of C_d continues to decrease with larger gate openings
- One way to determine if a constant value of C_d can be derived is to rewrite Eq. 5 in the following format (Skogerboe and Merkley 1996):

$$Q_f = C_d \left(G_o + \Delta G_o \right) G_w \sqrt{2g \left(\left(h_u \right)_{\Delta G_o} - \frac{G_o + \Delta G_o}{2} \right)} \qquad (2)$$

where ΔG_o is a measure of the zero datum level below the gate sill, and

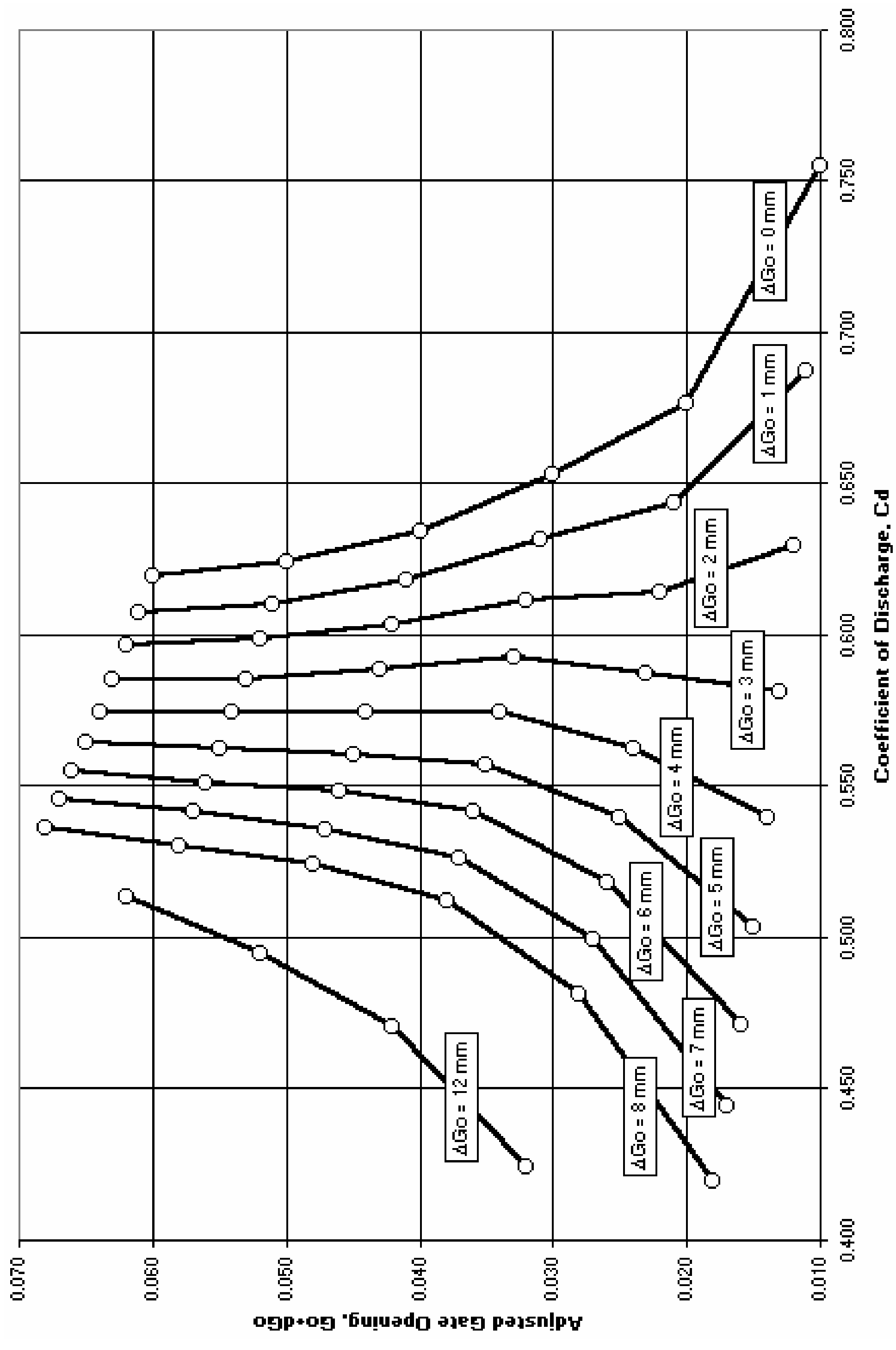
$$\left(h_u \right)_{\Delta G_o} = h_u + \Delta G_o \qquad (3)$$

- Assuming values of ΔG_o equal to 1 mm, 2 mm, 3 mm, etc., the computations for determining C_d can be made from Eq. 3
- The results for ΔG_o equal to 1 mm, 2 mm, 3 mm, 4 mm, 5 mm, 6 mm, 7 mm, 8 mm and 12 mm (gate seated) are listed in the table below
- The best results are obtained for ΔG_o of 3 mm – the results are plotted in the figure below, which shows that C_d varies from 0.582 to 0.593 with the average value of C_d being 0.587
- For this particular gate structure, the discharge normally varies between 200 and 300 lps, and the gate opening is normally operated between 40-60 mm, so that a constant value of C_d = 0.587 can be used when the zero datum for G_o and h_u is taken as 3 mm below the gate sill
- Another alternative would be to use a constant value of C_d = 0.575 for ΔG_o = 4 mm and G_o greater than 30 mm

Q_f (m^3/s)	G_o (m)	h_u (m)	Discharge Coefficient, C_d									
			ΔG_o 0 mm	ΔG_o 1 mm	ΔG_o 2 mm	ΔG_o 3 mm	ΔG_o 4 mm	ΔG_o 5 mm	ΔG_o 6 mm	ΔG_o 7 mm	ΔG_o 8 mm	ΔG_o 12mm
0.0646	0.010	1.838	0.756	0.688	0.630	0.582	0.540	0.504	0.472	0.445	0.420	0.344
0.0708	0.020	0.698	0.677	0.644	0.615	0.588	0.563	0.540	0.519	0.500	0.482	0.425
0.0742	0.030	0.375	0.654	0.632	0.612	0.593	0.575	0.558	0.542	0.527	0.513	0.471
0.0755	0.040	0.242	0.635	0.619	0.604	0.589	0.575	0.561	0.549	0.536	0.525	0.495
0.0763	0.050	0.175	0.625	0.611	0.599	0.586	0.575	0.563	0.552	0.542	0.531	0.514
0.0767	0.060	0.137	0.620	0.608	0.597	0.586	0.575	0.565	0.556	0.546	0.537	0.531

Notes: The last column with G_o = 12 mm is for the gate totally closed. The discharge coefficient, C_d, was calculated from:

$$Q_f = C_d\left(G_o + \Delta G_o\right) W \sqrt{2g\left((h_u)_{\Delta G_o} - \frac{G_o + \Delta G_o}{2}\right)}$$



Coefficient of Discharge, Cd
Adjusted Gate Opening, Go+dGo
ΔGo = 0 mm
ΔGo = 1 mm
ΔGo = 2 mm
ΔGo = 3 mm
ΔGo = 4 mm
ΔGo = 5 mm
ΔGo = 6 mm
ΔGo = 7 mm
ΔGo = 8 mm
ΔGo = 12 mm
0.400
0.450
0.500
0.550
0.600
0.650
0.700
0.750
0.800
0.070
0.060
0.050
0.040
0.030
0.020
0.010

II. Submerged-Flow Rectangular Gate Structures

- Assuming that the dimensionless velocity head coefficient in Eq. 27 is unity, the submerged-flow discharge equation for a rectangular gate having an opening, G_o, and a width, W, becomes:

$$Q_s = C_d G_o G_w \sqrt{2g\left(h_u - h_d\right)} \tag{4}$$

where $G_o G_w$ is the area, A, of the orifice

- Field calibration data for a rectangular gate structure operating under submerged-flow conditions are listed in the table below
- Note that for this type of slide gate, the gate opening can be measured both on the left side, $(G_o)_L$, and the right side, $(G_o)_R$, because the gate lip is not always horizontal
- The calculations are shown in the second table below

Discharge, Q_s (m^3/s)	**Gate Opening**		**Benchmark Tape Measurements**	
	$(G_o)_{left}$ (m)	**$(G_o)_{right}$ (m)**	**Upstream (m)**	**Downstream (m)**
0.079	0.101	0.103	0.095	0.273
0.095	0.123	0.119	0.099	0.283
0.111	0.139	0.139	0.102	0.296
0.126	0.161	0.163	0.105	0.290
0.141	0.180	0.178	0.108	0.301
0.155	0.199	0.197	0.110	0.301

Q_s (m^3/s)	**G_o (m)**	**h_u (m)**	**h_d (m)**	**$h_u - h_d$ (m)**	**C_d**
0.079	0.102	0.823	0.643	0.180	0.676
0.095	0.121	0.819	0.633	0.187	0.674
0.111	0.139	0.816	0.620	0.196	0.668
0.126	0.162	0.813	0.626	0.187	0.666
0.141	0.179	0.810	0.615	0.195	0.660
0.155	0.198	0.808	0.615	0.193	0.659

- As in the case of the free-flow orifice calibration in the previous section, a trial-and-error approach can be used to determine a more precise zero datum for the gate opening
- In this case, the submerged flow equation would be rewritten as:

$$Q_s = C_d \left(G_o + \Delta G_o\right) G_w \sqrt{2g\left(h_u - h_d\right)} \tag{5}$$

where ΔG_o is the vertical distance from the gate sill down to the zero datum level, as previously defined in Eq. 2

Q_s (m^3/s)	G_o (m)	$h_u - h_d$ (m)	C_d		
			ΔG_o = 4 mm	ΔG_o = 6 mm	ΔG_o = 8 mm
0.079	0.102	0.1801	0.650	0.638	0.626
0.095	0.121	0.1865	0.651	0.641	0.631
0.111	0.139	0.1960	0.649	0.640	0.631
0.126	0.162	0.1869	0.650	0.642	0.635
0.141	0.179	0.1949	0.646	0.639	0.632
0.155	0.198	0.1931	0.646	0.640	0.634

Note: The discharge coefficient, C_d, was calculated from Eq. 32:

- As before, the criteria for determining ΔG_o is to obtain a nearly constant value of the discharge coefficient, C_d
- The above table has the example computational results for determining the discharge coefficient, C_d, according to adjusted gate openings, G_o, under submerged flow conditions

III. Calibrating Medium- and Large-Size Gate Structures

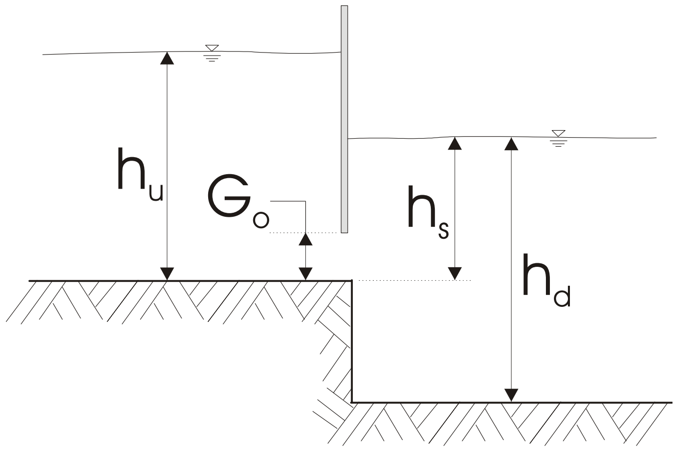
- A different form of the submerged-flow rating equation has been used with excellent results on many different orifice-type structures in medium and large canals
- The differences in the equation involve consideration of the gate opening and the downstream depth as influential factors in the determination of the discharge coefficient
- The equation is as follows:

$$Q_s = C_s h_s G_w \sqrt{2g\left(h_u - h_d\right)} \tag{6}$$

and,

$$C_s = \alpha \left(\frac{G_o}{h_s} \right)^{\beta} \tag{7}$$

where h_s is the downstream depth referenced to the bottom of the gate opening, α and β are empirically-fitted parameters, and all other terms are as described previously



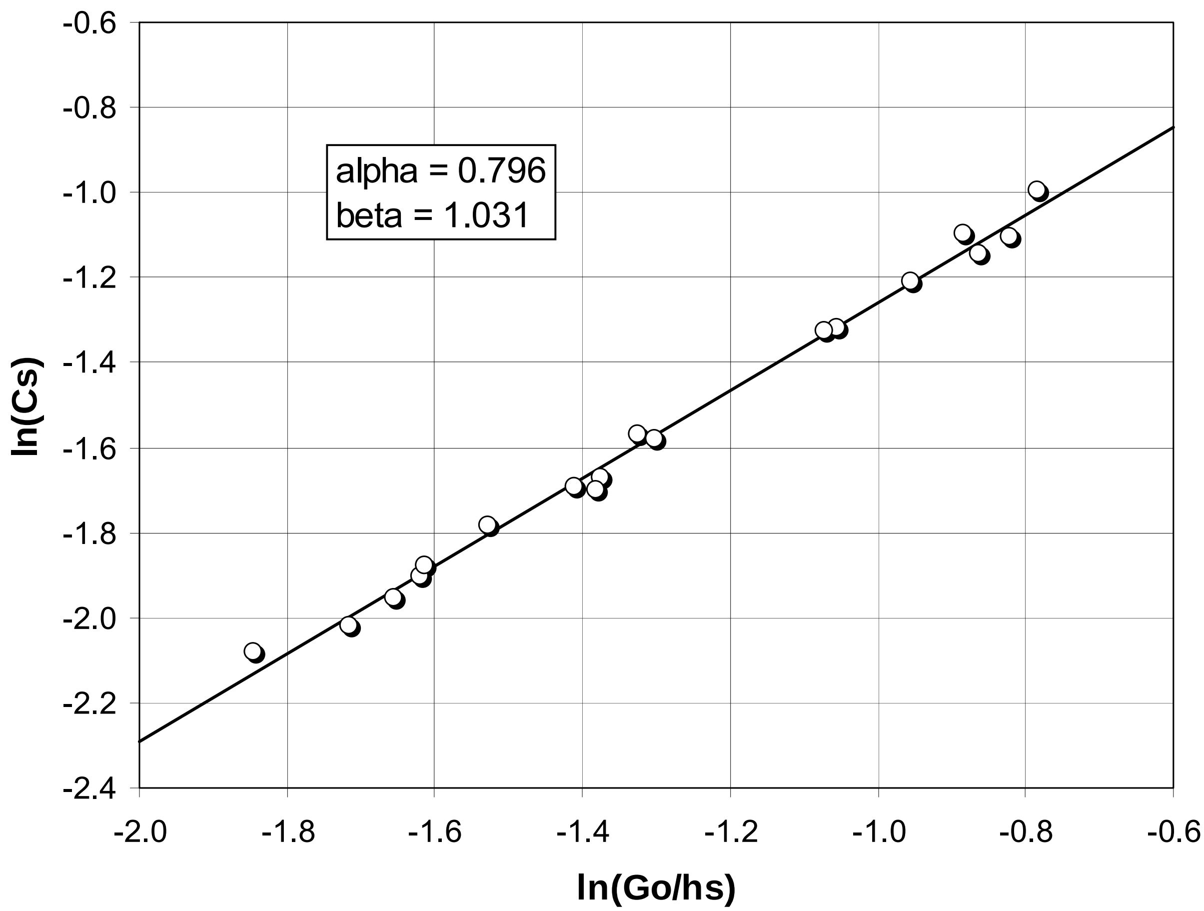
- Note that C_s is a dimensionless number
- The value of the exponent, β, is usually very close to unity
- In fact, for β equal to unity the equation reverts to that of a constant value of C_s equal to α (the h_s term cancels)

- The next table shows some example field calibration data for a large canal gate operating under submerged-flow conditions
- The solution to the example calibration is: α = 0.796, and β = 1.031
- This particular data set indicates an excellent fit to Eqs. 6 and 7, and it is typical of other large gate structures operating under submerged-flow conditions

Data Set	Discharge (m^3/s)	G_o (m)	Δh (m)	h_s (m)	G_o/h_s	C_s
1	8.38	0.60	3.57	2.205	0.272	0.206
2	9.08	0.70	3.00	2.010	0.348	0.268
3	5.20	0.38	3.31	1.750	0.217	0.168
4	4.27	0.30	3.41	1.895	0.158	0.125
5	5.45	0.40	3.43	2.025	0.198	0.149
6	12.15	0.95	2.63	2.300	0.413	0.334
7	5.49	0.38	3.72	1.905	0.199	0.153
8	13.52	1.10	2.44	2.405	0.457	0.369
9	14.39	1.00	3.84	2.370	0.422	0.318
10	16.14	1.13	3.79	2.570	0.440	0.331
11	6.98	0.50	3.70	1.980	0.253	0.188
12	11.36	0.58	7.64	2.310	0.251	0.183
13	7.90	0.42	6.76	2.195	0.191	0.142
14	7.15	0.38	6.86	2.110	0.180	0.133
15	7.49	0.51	3.98	2.090	0.244	0.184
16	10.48	0.70	3.92	2.045	0.342	0.266
17	12.41	0.85	3.76	2.205	0.385	0.298
18	8.26	0.55	3.91	2.065	0.266	0.208

Note: the data are for two identical gates in parallel, both having the same opening for each data set, with a combined opening width of 2.20 m.

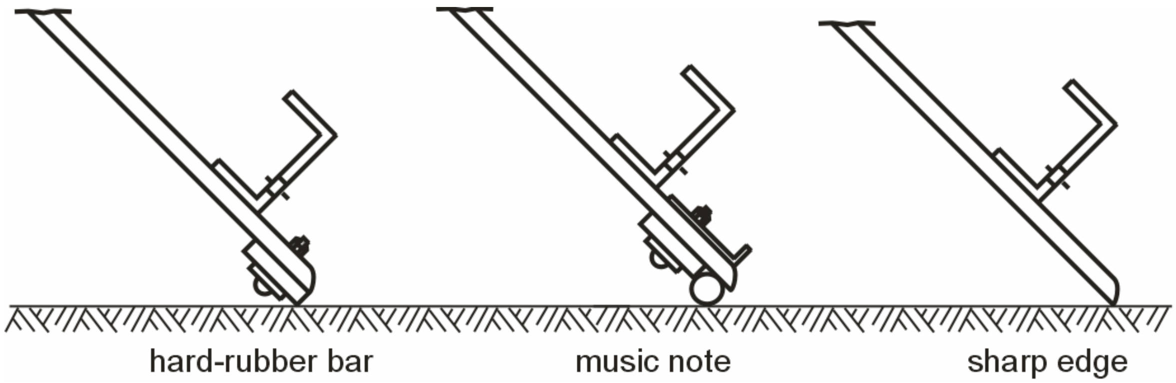
- A similar equation can be used for free-flow through a large gate structure, with the upstream depth, h_u, replacing the term h_s, and with (h_u - G_o/2) replacing (h_u - h_d)
- As previously mentioned, the free-flow equation can be calibrated using (h_u - 0.61G_o) instead of (h_u - G_o/2)
- The following figure shows a graph of the 18 data points; the straight line is the regression results which gives α and β



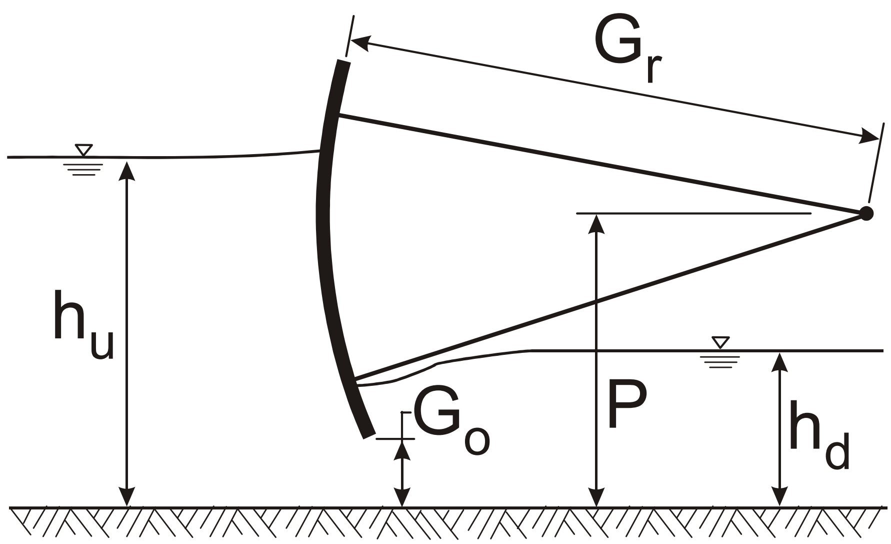
IV. Radial Gate Orifice-Flow Calibrations

- This structure type includes radial (or "Tainter") gates as calibrated by the USBR (Buyalski 1983) for free and submerged orifice-flow conditions
- The calibration of the gates follows the specifications in the USBR "REC-ERC-83-9" technical publication, which gives calibration equations for free and submerged orifice flow, and corrections for the type of gate lip seal
- The calibration requires no field measurements other than gate dimensions, but you can add another coefficient to the equations for free flow and orifice flow in an attempt to accommodate calibration data, if available
- Three gate lip seal designs (see figure below) are included in the calibrations:

1. Hard-rubber bar;
2. Music note; and,
3. Sharp edge



- The gate lip seal is the bottom of the gate leaf, which rests on the bottom of the channel when the gate is closed
- The discharge coefficients need no adjustment for the hard-rubber bar gate lip, which is the most common among USBR radial gate designs, but do have correction factors for the other two lip seal types
- These are given below for free and submerged orifice flow



- The gate radius divided by the pinion height should be within the range $1.2 \le G_r/P \le 1.7$
- The upstream water depth divided by the pinion height should be less than or equal to 1.6 ($h_u/P \le 1.6$)
- If these and other limits are observed, the accuracy of the calculated flow rate from Buyalski's equations should be within 1% of the true flow rate

Free Orifice Flow Free, or modular orifice flow is assumed to prevail when the downstream momentum function corresponding to C_cG_o, where G_o is the vertical gate opening, is less than or equal to the momentum function value using the downstream depth. Under these conditions the rating equation is:

$$F_f = Q_f - C_{fcda} G_o G_w \sqrt{2gh_u} = 0 \tag{8}$$

where C_{fcda} is the free-flow discharge coefficient; G_o is the vertical gate opening (m or ft); G_w is the width of the gate opening (m or ft); and h_u is the upstream water depth (m or ft); C_{fcda} is dimensionless

- C_{fcda} is determined according to a series of conic equations as defined by Buyalski (1983) from an analysis of over 2,000 data points
- The equations are lengthy, but are easily applied in a computer program

Eccentricity

$$AFE = \sqrt{0.00212\left(1.0+31.2\left(\frac{G_r}{P}-1.6\right)^2\right)} + 0.901 \tag{9}$$

$$BFE = \sqrt{0.00212\left(1.0+187.7\left(\frac{G_r}{P}-1.635\right)^2\right)} - 0.079 \tag{10}$$

$$FE = AFE - BFE\left(\frac{G_o}{P}\right) \tag{11}$$

where G_r is the gate radius (m or ft); and P is the pinion height (m or ft)

Directrix

$$AFD = 0.788 - \sqrt{0.04\left(1.0+89.2\left(\frac{G_r}{P}-1.619\right)^2\right)} \tag{12}$$

$$BFD = 0.0534\left(\frac{G_r}{P}\right) + 0.0457 \tag{13}$$

$$FD = 0.472 - \sqrt{BFD\left(1.0\text{-}\left(\frac{G_o}{P} - AFD\right)^2\right)} \tag{14}$$

Focal Distances

$$\begin{aligned} FX_1 &= 1.94\left(\frac{G_o}{P}\right) - 0.377 & \frac{G_o}{P} &\leq 0.277 \\ FX_1 &= 0.18\left(\frac{G_o}{P}\right) + 0.111 & \frac{G_o}{P} &> 0.277 \end{aligned} \tag{15}$$

$$FY_1 = 0.309 - 0.192\left(\frac{G_o}{P}\right) \tag{16}$$

and,

$$FXV = \frac{h_u}{P} - FX_1 \tag{17}$$

The correction on C_{fcda} for the "<u>music note</u>" gate lip seal design is:

$$C_{correct} = 0.125\left(\frac{G_o}{P}\right) + 0.91 \quad \text{(music note)} \tag{18}$$

The correction on C_{fcda} for the "<u>sharp edge</u>" gate lip seal design is:

$$C_{correct} = 0.11\left(\frac{G_o}{P}\right) + 0.935 \quad \text{(sharp edge)} \tag{19}$$

For the hard-rubber bar gate lip seal design, $C_{correct}$ = 1.0. The preceding corrections on C_{fcda} for the "music note" and "sharp edge" gate lip seal designs were chosen from the linear options proposed by Buyalski (ibid).

Finally,

$$C_{fcda} = C_{correct}\left(\sqrt{FE^2\left(FD + FXV\right)^2 - FXV^2} + FY_1\right) \quad (20)$$

Radial gates in parallel at a check structure

Submerged Orifice Flow The submerged orifice rating equation is:

$$F_s = Q_s - C_{scda} G_o G_w \sqrt{2gh_u} = 0 \quad (21)$$

where C_{scda} is the submerged-flow discharge coefficient; and all other terms are as previously defined; both C_{scda} and C_{ds} are dimensionless

- Note that the square-root term does not include the downstream depth, h_d, but it is included in the lengthy definition of C_{scda}
- As in the free-flow case, C_{scda} is determined according to a series of conic equations:

Directrix

$$ADA = \left(11.98\left(\frac{G_r}{P}\right) - 26.7\right)^{-1} \quad (22)$$

$$ADB = 0.62 - 0.276\left(\frac{P}{G_r}\right) \quad (23)$$

$$AD = \left(ADA\left(\frac{G_o}{P}\right) + ADB\right)^{-1} \quad (24)$$

$$BDA = 0.025\left(\frac{G_r}{P}\right) - 2.711 \quad (25)$$

$$BDB = 0.071 - 0.033\left(\frac{G_r}{P}\right) \quad (26)$$

$$BD = BDA\left(\frac{G_o}{P}\right) + BDB \quad (27)$$

$$DR = AD\left(\frac{h_d}{P}\right) + BD \quad (28)$$

$$D = DR^{-1.429} \quad (29)$$

Eccentricity

$$AEA = 0.06 - 0.019\left(\frac{G_r}{P}\right) \quad (30)$$

$$AEB = 0.996 + 0.0052\left(\frac{G_r}{P}\right) \quad (31)$$

$$AE = \left(AEA\left(\frac{G_o}{P}\right) + AEB\right)^{-1} \quad (32)$$

$$BEK = 0.32 - 0.293\left(\frac{G_r}{P}\right) \tag{33}$$

$$BE = BEK + \sqrt{0.255\left(1.0 + 1.429\left(\frac{G_o}{P} - 0.44\right)^2\right)} \tag{34}$$

$$ER = AE(D) + BE \tag{35}$$

$$E = \sqrt{\ln\left(\frac{ER}{D}\right)} \tag{36}$$

Vector V_1

$$V_1 = \frac{E(D)}{1.0 + E} \tag{37}$$

Focal Distance

$$AFA = 0.038 - 0.158\left(\frac{P}{G_r}\right) \tag{38}$$

$$AFB = 0.29 - 0.115\left(\frac{G_r}{P}\right) \tag{39}$$

$$AF = AFA\left(\frac{G_o}{P}\right) + AFB \tag{40}$$

$$BFA = 0.0445\left(\frac{P}{G_r}\right) - 0.321 \tag{41}$$

$$BFB = 0.155 - 0.092\left(\frac{P}{G_r}\right) \tag{42}$$

$$BF = BFA\left(\frac{P}{G_o}\right) + BFB \tag{43}$$

$$FY = BF - \frac{AF\left(h_d\right)}{P} \tag{44}$$

- If $FY \le 0$, then let FY = 0 and FX = 0. Otherwise, retain the calculated value of FY and,

$$FX = \sqrt{V_1^2 + FY^2} - V_1 \quad \text{(for } FY > 0\text{)} \tag{45}$$

$$VX = \frac{h_u}{P} - V_1 - \frac{h_d}{P} - FX \tag{46}$$

The correction on C_{scda} for the "music note" gate lip seal design is:

$$C_{correct} = 0.39\left(\frac{G_o}{P}\right) + 0.85 \quad \text{(music note)} \tag{47}$$

The correction on C_{scda} for the "sharp edge" gate lip seal design is:

$$C_{correct} = 0.11\left(\frac{G_o}{P}\right) + 0.9 \quad \text{(sharp edge)} \tag{48}$$

- For the hard-rubber bar gate lip seal design, $C_{correct}$ = 1.0
- The preceding corrections on C_{scda} for the "music note" and "sharp edge" gate lip seal designs were chosen from the linear options proposed by Buyalski (ibid)

Finally,

$$C_{scda} = C_{correct}\left(\sqrt{E^2\left(D+VX\right)^2 - VX^2} + FY\right) \quad (49)$$

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Lecture 13

Flow Measurement in Pipes

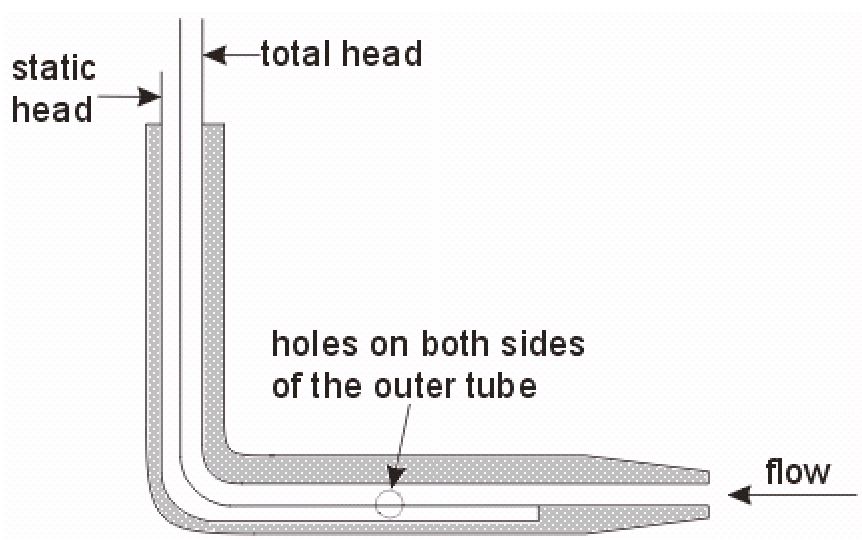
I. Introduction

- There are a wide variety of methods for measuring discharge and velocity in pipes, or closed conduits
- Many of these methods can provide very accurate measurements
- Others give only rough estimates
- But, in general, it is easier to obtain a given measurement accuracy in pipes when compared to measurement in open channels
- Some of the devices used are very expensive and are more suited to industrial and municipal systems than for agricultural irrigation systems

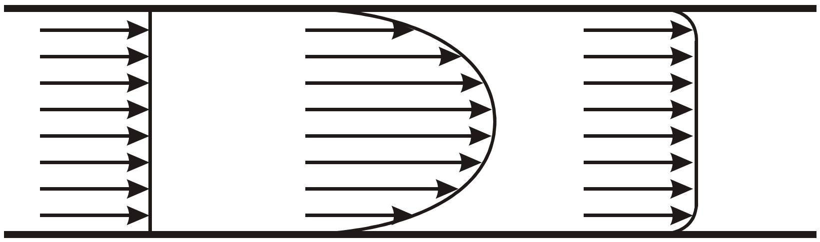
II. Pitot Tubes

- The pitot tube is named after Henri Pitot who used a bent glass tube to measure velocities in a river in France in the 1700s
- The pitot tube can be used not only for measuring flow velocity in open channels (such as canals and rivers), but in closed conduits as well
- There are several variations of pitot tubes for measuring flow velocity, and many of these are commercially available

- Pitot tubes can be very simple devices with no moving parts
- More sophisticated versions can provide greater accuracy (e.g. differential head meters that separate the static pressure head from the velocity head)
- The pitot static tube, shown in the figure below, is one variation of the device which allows the static head (P/γ) and dynamic (total) head ($P/\gamma + V^2/2g$) to be separately measured



- The static head equals the depth if open-channel flow
- Calibrations are required because the velocity profile can change with the flow rate, and because measurement(s) are only a sampling of the velocities in the pipe



- The measurement from a pitot tube can be accurate to ±1% of the true velocity, even if the submerged end of the tube is up to ±15% out of alignment from the flow direction
- The velocity reading from a pitot tube must be multiplied by cross-sectional area to obtain the flow rate (it is a *velocity-area* method)
- Pitot tubes tend to become clogged unless the water in the pipe is very clean
- Also, pitot tubes may be impractical if there is a large head, unless a manometer is used with a dense liquid like mercury

III. Differential Producers

- This is a class of flow measurement devices for full pipe flow
- “Differential producers” cause a pressure differential which can be measured and correlated to velocity and or flow rate in the pipe
- Examples of differential producers:

    - Venturis
    - Nozzles
    - Orifices

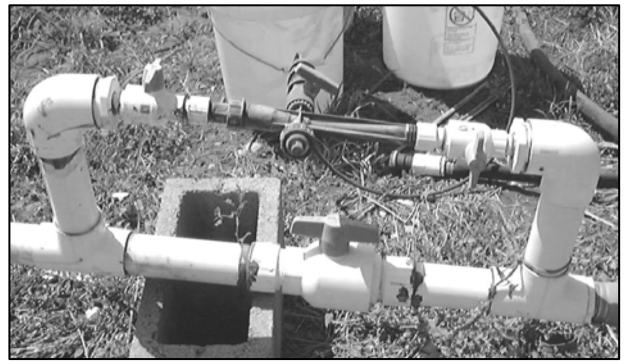
- Measured ΔP at a differential producer depends on:

    - Flow rate
    - Fluid properties
    - Element geometry

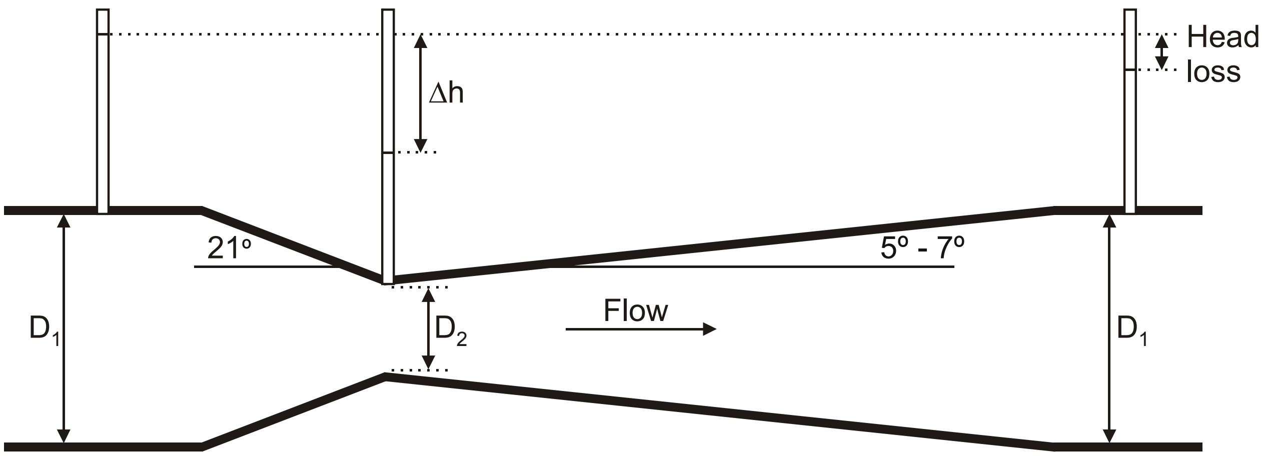
IV. Venturi Meters

- The principle of this flow measurement device was first documented by J.B. Venturi in 1797 in Italy
- Venturi meters have only a small head loss, no moving parts, and do not clog easily

- The principle under which these devices operate is that some pressure head is converted to velocity head when the cross-sectional area of flow decreases (Bernoulli equation)



- Thus, the head differential can be measured between the upstream section and the throat section to give an estimation of flow velocity, and this can be multiplied by flow area to arrive at a discharge value
- The converging section is usually about 21º, and the diverging section is usually from 5 to 7º



- A form of the calibration equation is:

$$Q = C A_2 \frac{\sqrt{2g\Delta h(sg-1)}}{\sqrt{1-\beta^4}} \quad (1)$$

where C is a dimensionless coefficient from approximately 0.935 (small throat velocity and diameter) to 0.988 (large throat velocity and diameter); β is the ratio of D_2/D_1; D_1 and D_2 are the inside diameters at the upstream and throat sections, respectively; A_2 is the area of the throat section; Δh is the head differential; and "sg" is the specific gravity of the manometer liquid

- The discharge coefficient, C, is a constant value for given venturi dimensions
- Note that if $D_2 = D_1$, then $\beta = 1$, and Q is undefined; if $D_0 > D_1$, you get the square root of a negative number (but neither condition applies to a venturi)
- The coefficient, C, must be adjusted to accommodate variations in water temperature

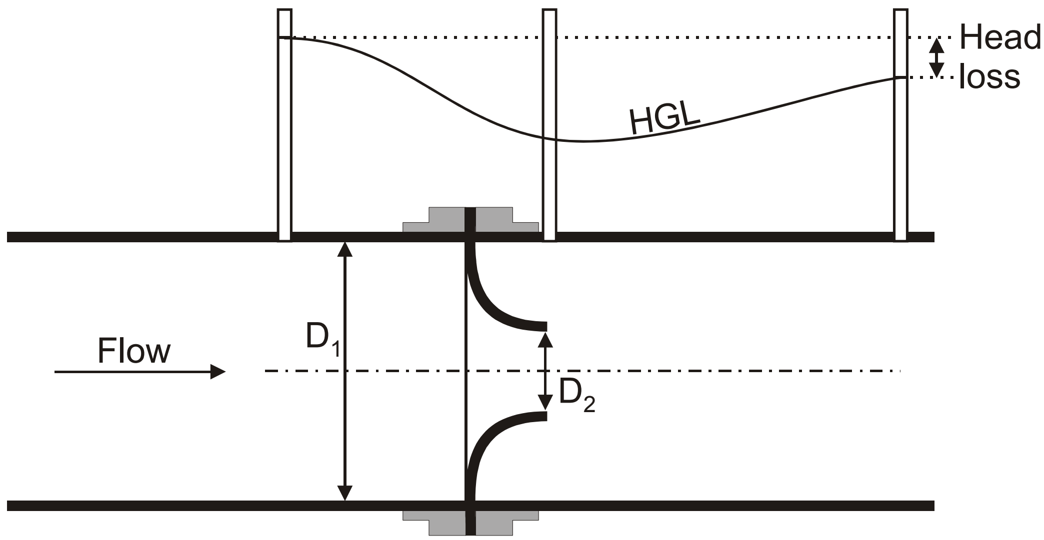
- The value of β is usually between 0.25 and 0.50, but may be as high as 0.75
- Venturi meters have been made out of steel, iron, concrete, wood, plastic, brass, bronze, and other materials
- Most modern venturi meters of small size are made from plastic (doesn't corrode)
- Many commercial venturi meters have patented features
- The upstream converging section usually has an angle of about 21° from the pipe axis, and the diverging section usually has an angle of 5° to 7° (1:6 divergence, as for the DS ramp of a BCW, is about 9.5°)

- Straightening vanes may be required upstream of the venturi to prevent swirling flow, which can significantly affect the calibration
- It is generally recommended that there should be a distance of at least $10D_1$ of straight pipe upstream of the venturi
- The head loss across a venturi meter is usually between 10 and 20% of Δh
- This percentage decreases for larger venturis and as the flow rate increases
- Venturi discharge measurement error is often within ±0.5% to ±1% of the true flow rate value

V. Flow Nozzles

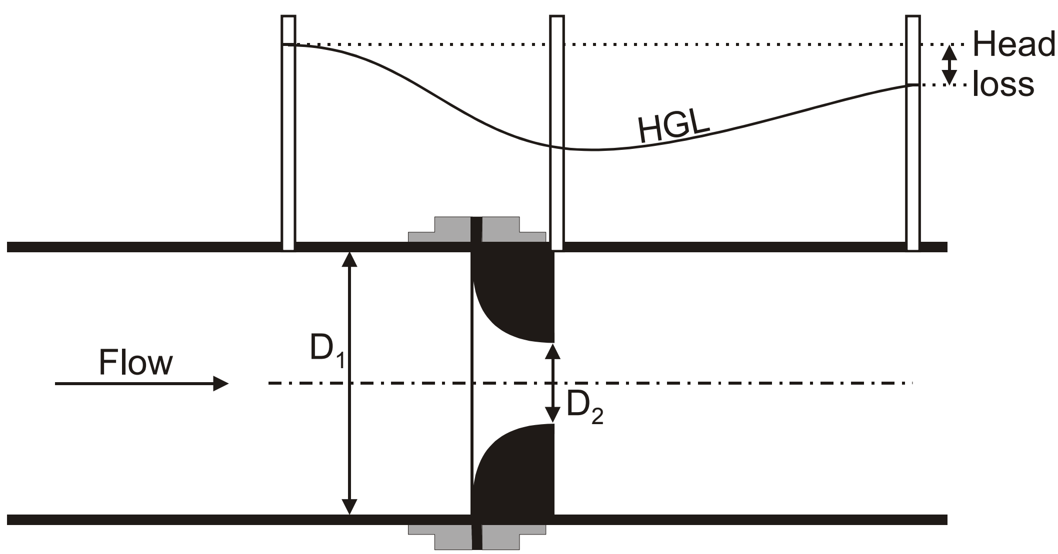
- Flow nozzles operate on the same principle as venturi meters, but the head loss tends to be much greater due to the absence of a downstream diverging section
- There is an upstream converging section, like a venturi, but there is no downstream diverging section to reduce energy loss

- Flow nozzles can be less expensive than venturi meters, and can provide comparable accuracy
- The same equation as for venturi meters is used for flow nozzles
- The head differential across the nozzle can be measured using a manometer or some kind of differential pressure gauge
- The upstream tap should be within ½D_1 to D_1 upstream of the entrance to the nozzle
- The downstream tap should be approximately at the outlet of the nozzle (see the figure below)



A Flow Nozzle in a Pipe

- The space between the nozzle and the pipe walls can be filled in to reduce the head loss through the nozzle, as seen in the following figure



A "Solid" Flow Nozzle in a Pipe

VI. Orifice Meters

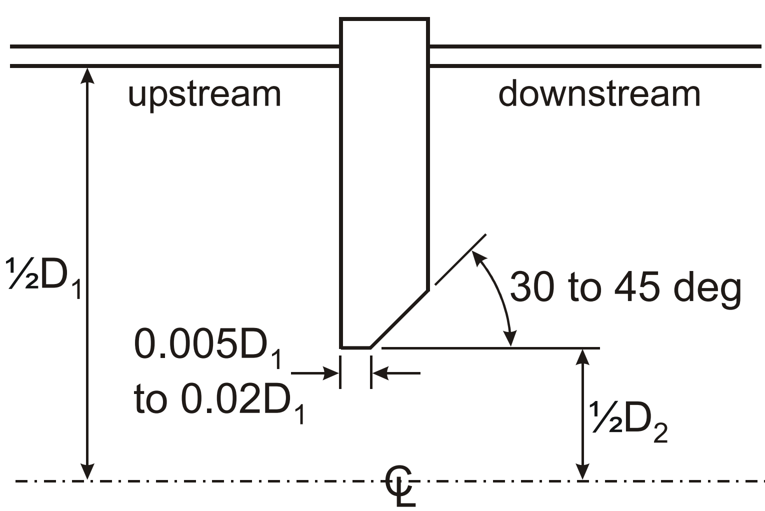
- These devices use a thin plate with an orifice, smaller than the pipe ID, to create a pressure differential
- The orifice opening is usually circular, but can be other shapes:

  - Square
  - Oval
  - Triangular
  - Others

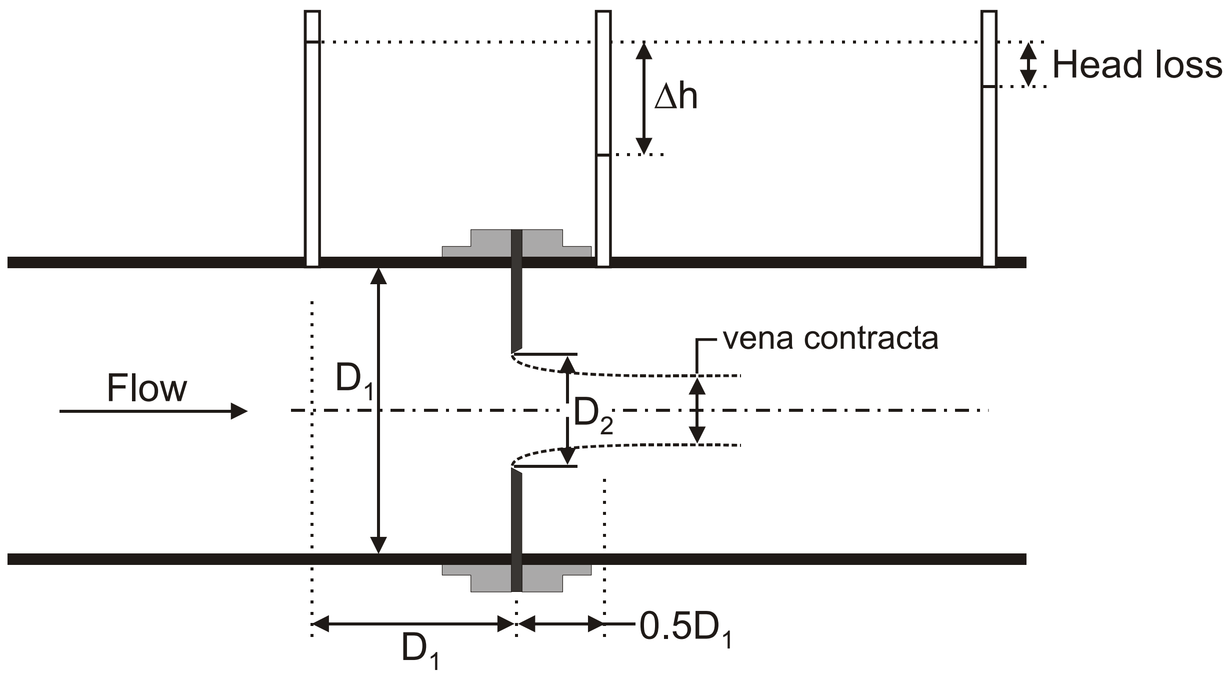
- The pressure differential can be measured, as in venturi and nozzle meters, and the same equation as for venturi meters can be used
- However, the discharge coefficient is different for orifice meters
- It is easy to make and install an orifice meter in a pipeline – easier than a nozzle

- Orifice meters can give accurate measurements of Q, and they are simple and inexpensive to build
- But, orifice meters cause a higher head loss than either the venturi or flow nozzle meters
- As with venturi meters and flow nozzles, orifice meters can provide values within $\pm$1% (or better) of the true discharge
- As with venturi meters, there should be a straight section of pipe no less than 10 diameters upstream
- Some engineers have used eccentric orifices to allow passage of sediments – the orifice is located at the bottom of a horizontal pipe, not in the center of the pipe cross section

- The orifice opening can be "sharp" (beveled) for better accuracy
- But don't use a beveled orifice opening if you are going to use it to measure flow in both directions
- These are the beveling dimensions:



- The upstream head is usually measured one pipe diameter upstream of the thin plate, and the downstream head is measured at a variable distance from the plate
- Standard calibrations are available, providing C values from which the discharge can be calculated for a given Δh value
- In the following, the coefficient for an orifice plate is called "K", not "C"
- The coefficient values depend on the ratio of the diameters and on the Reynold's number of approach; they can be presented in tabular or graphical formats

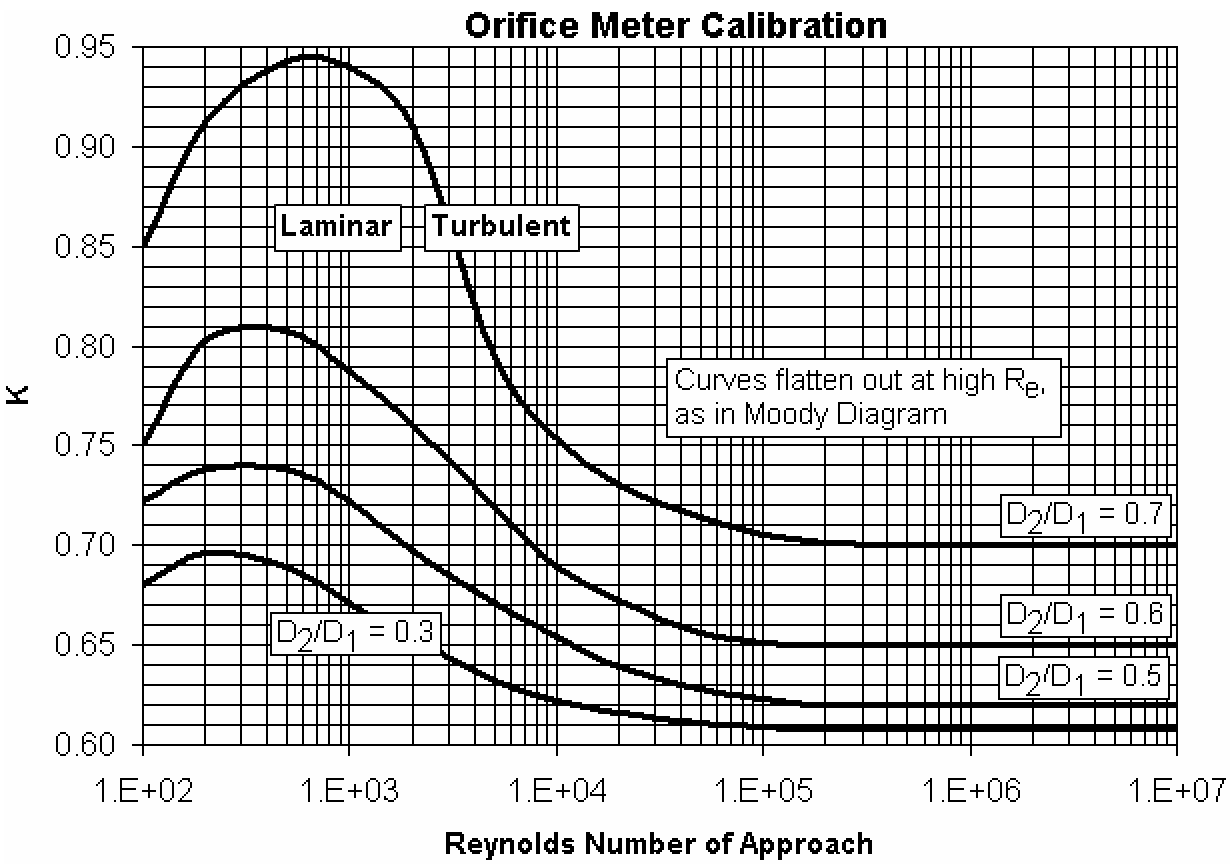


An Orifice Meter in a Pipe

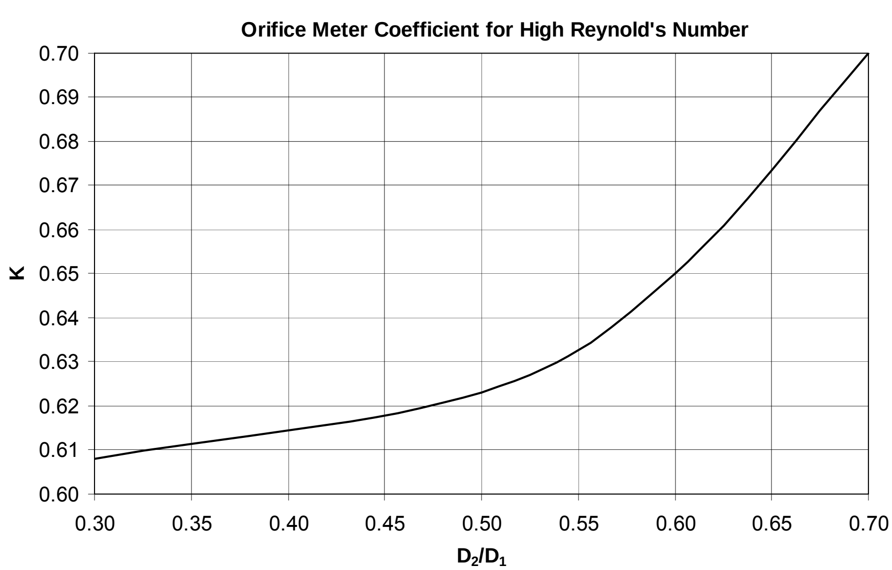
- In the figure below, the Reynold's number of approach is calculated for the pipe section upstream of the orifice plate (diameter D_1, and the mean velocity in D_1)
- Note also that pipe flow is seldom laminar, so the curved parts of the figure are not of great interest
- An equation for use with the curves for K:

$$Q = KA_2\sqrt{2g\left[\left(\frac{P_u}{\gamma}+z_u\right)-\left(\frac{P_d}{\gamma}+z_d\right)\right]} \tag{2}$$

- The above equation is the same form as for canal gates operating as orifices
- The ratio β is embedded in the K term
- Note that z_u equals z_d for a horizontal pipe (they are measured relative to an arbitrary elevation datum)
- Note that P_u/γ is the same as h_u (same for P_d/γ and h_d)
- Also, you can let $\Delta h = h_u - h_d$



- P_d is often measured at a distance of about ½D_1 downstream of the orifice plate, but the measurement is not too sensitive to the location, within a certain range (say ¼ D_1 to D_1 downstream)
- The following graph shows the K value for an orifice meter as a function of the ratio of diameters when the Reynold's number of approach is high enough that the K value no longer depends on R_e



Orifice Plate Calibrations

- A perhaps better way to calibrate sharp-edged orifice plates in pipes is based on the following equations
- Flow rate can be calculated through the orifice using the following equation:

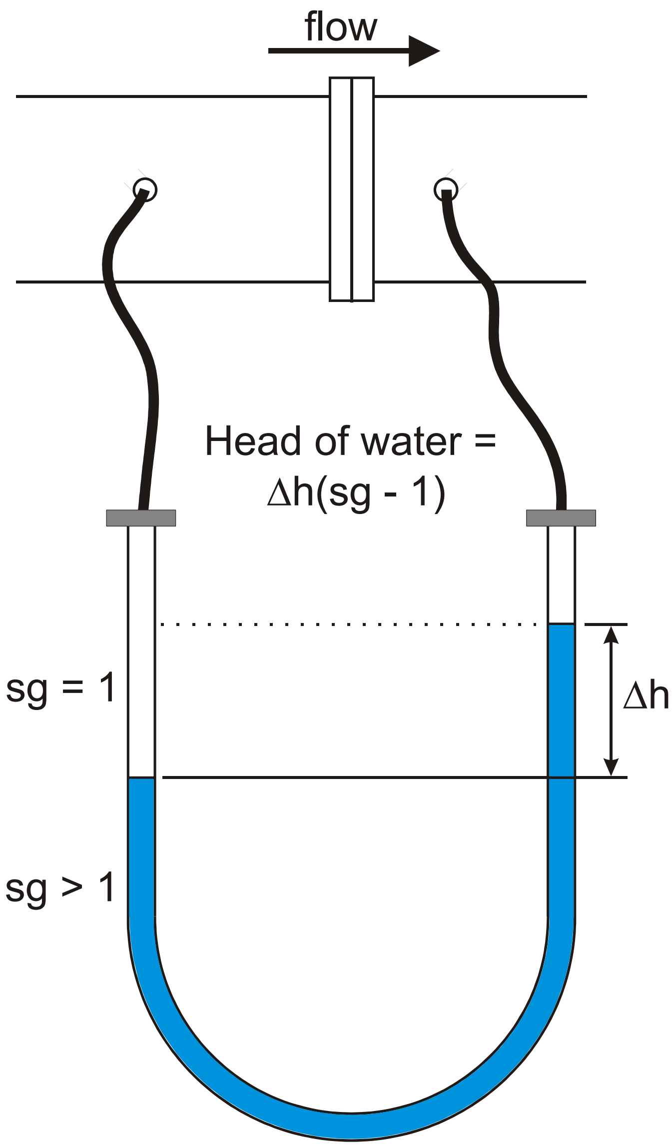
$$Q = C_d A_2 \frac{\sqrt{2g\Delta h(sg-1)}}{\sqrt{1-\beta^4}} \tag{3}$$

where C_d is a dimensionless orifice discharge coefficient, as defined below; A_2 is the cross-sectional area of the orifice plate opening; g is the ratio of weight to mass; Δh is the change in piezometric head across the orifice; and, β is a dimensionless ratio of the orifice and pipe diameters:

$$\beta = \frac{D_2}{D_1} \tag{4}$$

where D_2 is the diameter of the circular orifice opening; and, D_1 is the inside diameter of the upstream pipe

- In Eq. 3, "sg" is the specific gravity of the manometer fluid, and the constant "1" represents the specific gravity of pure water
- The specific gravity of the manometer liquid must be greater than 1.0
- Thus, if a manometer is used to measure the head differential across the orifice plate, the term "Δh(sg - 1)" represents the head in depth (e.g. m or ft) of water
- If both ends of the manometer were open to the atmosphere, and there's no water in the manometer, then you will see $\Delta h = 0$
- But if both ends of the manometer are open to the atmosphere, and you pour some water in one end, you'll see $\Delta h > 0$, thus the need for the "(sg – 1)" term
- Note that the specific gravity of water can be slightly different than 1.000 when the water is not pure, or when the water temperature is not exactly 5°C
- See the figure below
- Note also that the manometer liquid must not be water soluble!



- The inside pipe diameter, D_1, is defined as:

$$D_1 = \left[1 + \alpha_p \left(T_{\circ C} - 20\right)\right]\left(D_1\right)_{meas} \qquad (5)$$

in which $T_{\circ C}$ is the water temperature in °C; $(D_1)_{meas}$ is the measured inside pipe diameter; and α_p is the coefficient of linear thermal expansion of the pipe material (1/°C)

- The coefficient of linear thermal expansion is the ratio of the change in length per degree Celsius to the length at 0°C
- See the following table for linear thermal expansion values

	Material	Coefficient of Linear Thermal Expansion (1/°C)
Metal	Cast iron	0.0000110
	Steel	0.0000120
	Tin	0.0000125
	Copper	0.0000176
	Brass	0.0000188
	Aluminum	0.0000230
	Zinc	0.0000325
Plastic	PVC	0.0000540
	ABS	0.0000990
	PE	0.0001440
Other	Glass	0.0000081
	Wood	0.0000110
	Concrete	0.0000060 – 0.0000130

- For the range 0 to 100 °C, the following two equations can be applied for the density and kinematic viscosity of water
- The density of pure water:

$$\rho = 1.4102(10)^{-5}T^3 - 0.005627(10)^{-5}T^2 + 0.004176(10)^{-6}T + 1{,}000.2 \qquad (6)$$

where ρ is in kg/m^3; and T is in °C

- The kinematic viscosity of pure water:

$$\nu = \frac{1}{83.9192\,T^2 + 20{,}707.5\,T + 551{,}173} \qquad (7)$$

where ν is in m^2/s; and T is in °C

- Similarly, the orifice diameter is corrected for thermal expansion as follows:

$$D_2 = \left[1 + \alpha_{op}\left(T_{^\circ C} - 20\right)\right]\left(D_2\right)_{meas} \qquad (8)$$

where α_{op} is the coefficient of linear thermal expansion of the orifice plate material (1/°C); and $(D_2)_{meas}$ is the measured orifice diameter

- Note that the water temperature must be substantially different than 20°C for the thermal expansion corrections to be significant

- The coefficient of discharge is defined by Miller (1996) for a circular pipe and orifice plate in which the upstream tap is located at a distance D_1 from the plate, and the downstream tap is at a distance ½D_1:

$$C_d = 0.5959 + 0.0312\beta^{2.1} - 0.184\beta^8 + \frac{0.039\beta^4}{1-\beta^4} - 0.0158\beta^3 + \frac{91.71\beta^{2.5}}{R_e^{0.75}} \quad (9)$$

in which R_e is the Reynolds number.

- Similar C_d equations exist for other orifice plate configurations, and for venturis
- The C_d expression for venturis is much simpler than that for orifice plates
- The Reynold's number is a function of the flow rate, so the solution is iterative
- The calculated value of C_d is typically very near to 0.6, so if this is taken as the initial value, usually only one or two iterations are needed:

1. Specify T, Δh, α_p, and α_{op}
2. Calculate or specify ρ and ν
3. Calculate D_1 and D_2
4. Calculate $\beta = D_1/D_2$
5. Let $C_d = 0.60$
6. Calculate Q
7. Calculate R_e
8. Calculate C_d

- Repeat steps 6 - 8 until Q converges to the desired precision

References & Bibliography

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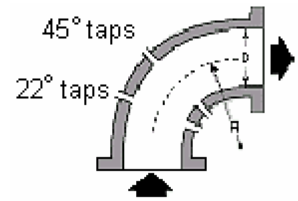
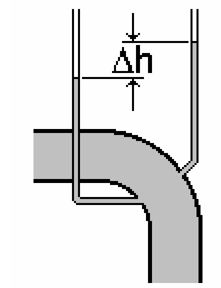
Lecture 14

Flow Measurement in Pipes

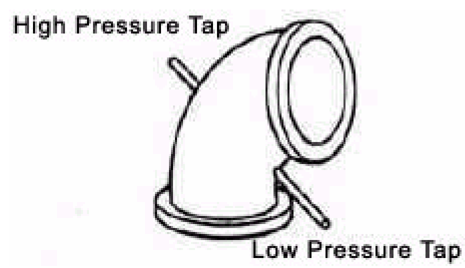
I. Elbow Meters



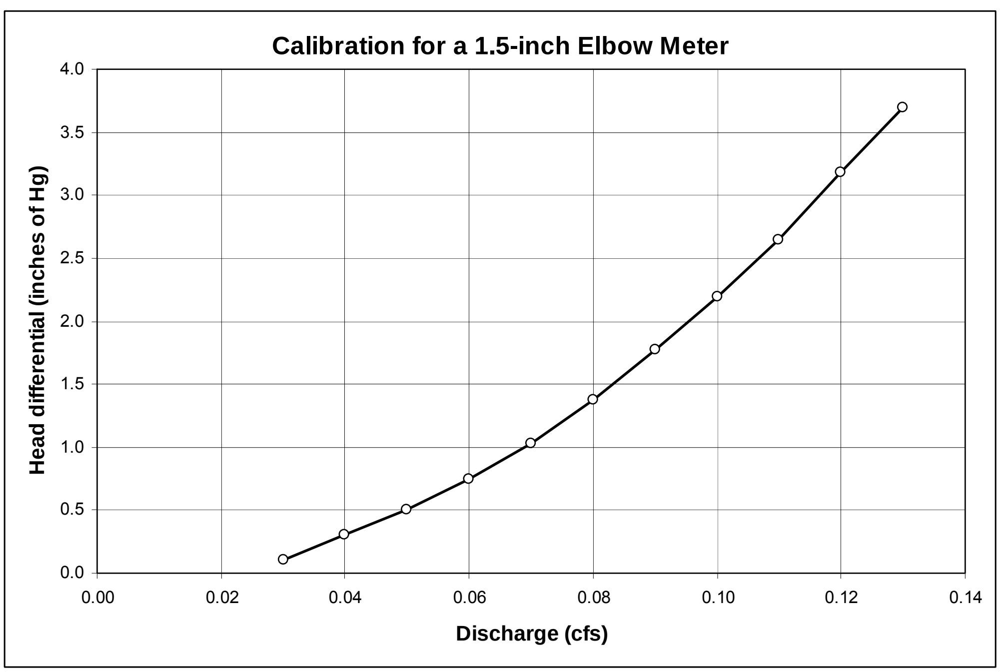
- An elbow in a pipe can be used as a flow measuring device much in the same way as a venturi or orifice plate
- The head differential across the elbow (from inside to outside) is measured, and according to a calibration the discharge can be estimated
- The taps are usually located in the center of the elbow (e.g. at a 45° angle for a 90° elbow), but can be at other locations toward the upstream side of the elbow



- Some companies manufacture elbow meters for flow measurement, but almost any pipe elbow can be calibrated
- Elbow meters are not as potentially accurate as venturi, nozzle, and orifice meters
- Typical accuracy is about ±4% of Q
- One advantage of elbow meters is that there need not be any additional head loss in the piping system as a result of flow measurement

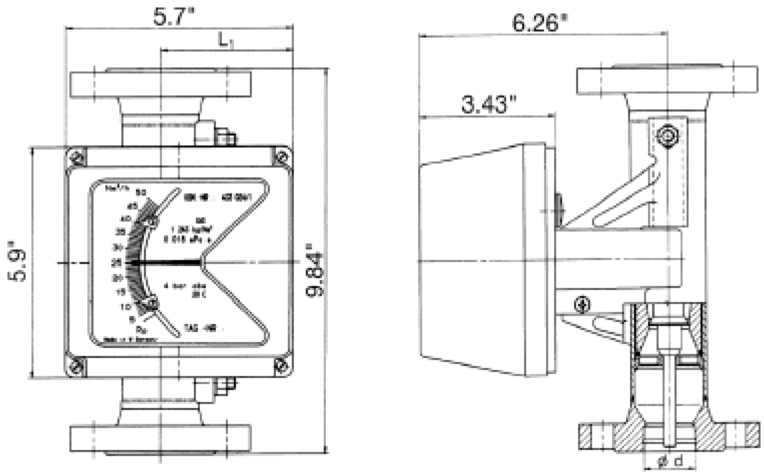


- The graph below is a sample calibration curve for a 1½-inch elbow meter in a USU hydraulic lab where the head differential (inside to outside tap) is measured in inches of mercury, using a manometer (data are from Dr. L.S. Willardson)



II. Variable Area Meters

- These are vertical cylinders with a uniformly expanding cross-section in the upward direction
- A float inside the cylinder stabilizes at a certain elevation depending on the flow rate through the cylinder



- Note that the outside walls are usually transparent to allow direct readings by eye

III. Horizontal Trajectory Method

- From physics, an accelerating object will travel a distance x in time t according to the following equation (based on Newton's 2nd law):

$$x = v_o t + \frac{a t^2}{2} \tag{1}$$

where x is the distance; v_o is the initial velocity at time 0; t is the elapsed time; and a is the acceleration

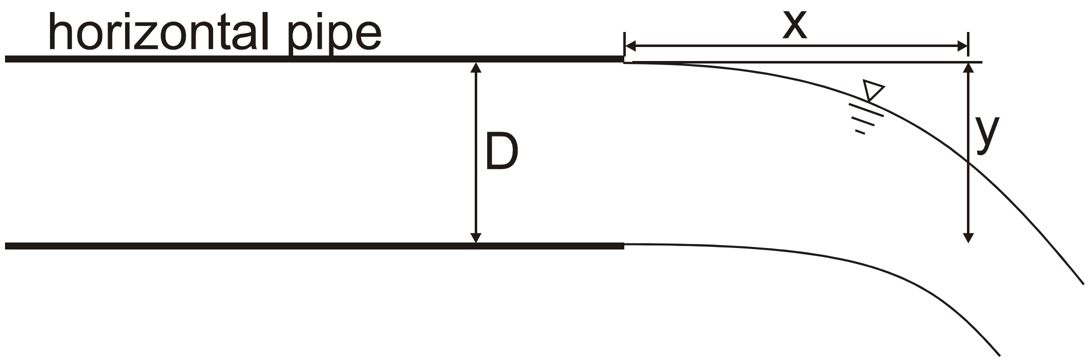
- Flow emanating from a horizontal pipe will fall a height y over a distance x
- The horizontal component (x-direction) has almost no acceleration, and the vertical component (y-direction) has an initial velocity of zero
- The vertical acceleration is equal to the ratio of weight to mass, or g = 9.81 m/s^2 (32.2 ft/s^2)
- Therefore,

$$x = v_o t \quad \text{and,} \quad y = \frac{g t^2}{2} \tag{2}$$

- Then by getting rid of t, knowing that Q = VA, and the equation for the area of a circle, the flow rate is calculated as follows:

$$Q = \frac{\pi D^2 x}{4\sqrt{\frac{2y}{g}}} \quad (3)$$

where D is the inside diameter of the circular pipe



- This equation is approximately correct if x and y are measured to the center of mass of the discharge trajectory
- Errors occur because in practice it is difficult to measure exactly to the center, and because of possible wind and other turbulent effects
- Also, the pipe might not be exactly horizontal (although a correction could take this into account, according to the same analysis given above)
- Tables of coefficient values derived from experiments allow x and y to be measured from the top of the trajectory
- However, measurements can be difficult because the flow is often very turbulent at a distance from the pipe end

- The previous equation can be simplified as:

$$Q = 3.151 C D^2 \frac{x}{\sqrt{y}} \quad (4)$$

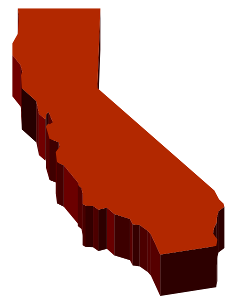
where C is a coefficient to adjust the calculated discharge value when the ratios of x/D or y/D are smaller than 8 and 5, respectively (otherwise, C equals unity)

- Equation 4 is valid for x, y and D in ft, and Q in cfs
- The following table is for C values for use with Eq. 4

y/D	x/D							
	1.00	**1.50**	**2.00**	**2.50**	**3.00**	**4.00**	**5.00**	**8.00**
0.5	1.44	1.28	1.18	1.13	1.10	1.06	1.03	1.00
1.0	1.37	1.24	1.17	1.12	1.09	1.06	1.03	1.00
2.0		1.11	1.09	1.08	1.07	1.05	1.03	1.00
3.0			1.04	1.04	1.04	1.04	1.03	1.00
4.0			1.01	1.01	1.02	1.03	1.02	1.00
5.0			0.97	0.99	1.00	1.01	1.01	1.00

- This method can also be used for pipes flowing partially full (i.e. $A < \pi D^2/4$), and experimental data are available to assist in the estimation of discharge for these conditions

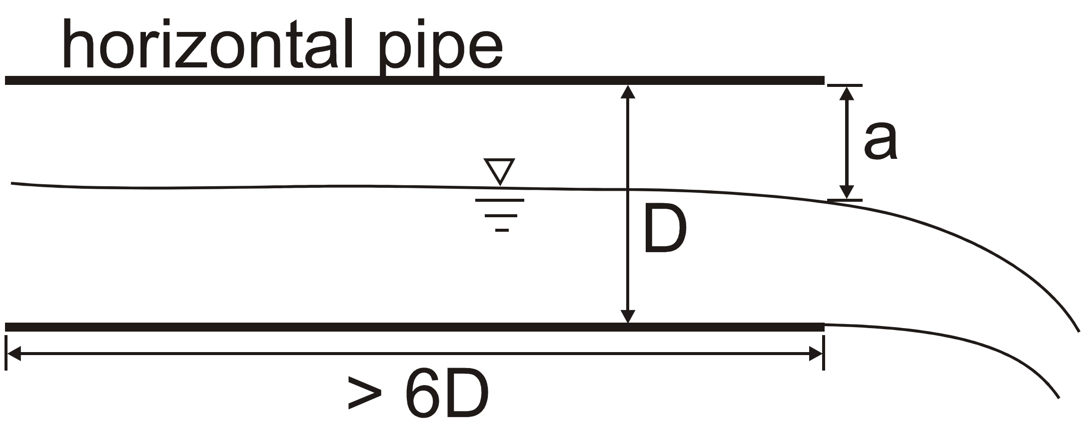
IV. California Pipe Method



- This is the horizontal pipe method for partially-full pipes
- It is somewhat analogous to the calibration for a weir or free overfall
- The following equation is in English units:

$$Q = 8.69\left(1 - \frac{a}{D}\right)^{1.88} D^{2.48} \tag{5}$$

where a and D are defined in the figure below (ft); and Q is discharge in cfs



- The ratio a/D is limited to: $a/D > 0.45$
- This method was published in the 1920’s
- Measurement accuracy is only ±10%, at best
- The pipe must be exactly horizontal (level), with circular cross section
- The pipe must discharge freely into the air, unsubmerged

V. Vertical Trajectory Method

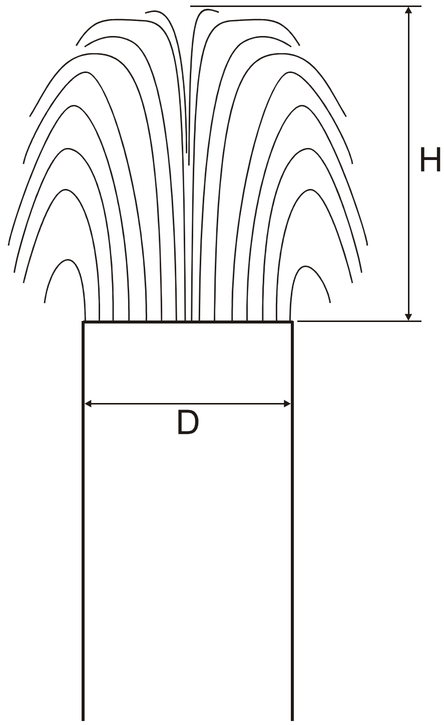
- As with pipes discharging horizontally into the air, there is a method to measure the flow rate from vertical pipes
- This is accomplished by assuming a translation of velocity head into the measurable height of a column of water above the top of the pipe
- Thus, to estimate the flow rate from pipes discharging vertically into the air it is only necessary to measure the:

1. inside diameter of the pipe, D; and,
2. the height of the jet, H, above the pipe

- This is a nice idea on "paper," but in practice, it can be difficult to measure the height of the column of water because of sloshing, surging, and splashing
- Also, the act of measuring the height of the column can significantly alter the measured value
- The table below gives flow rate values in gpm for several pipe diameters in inches

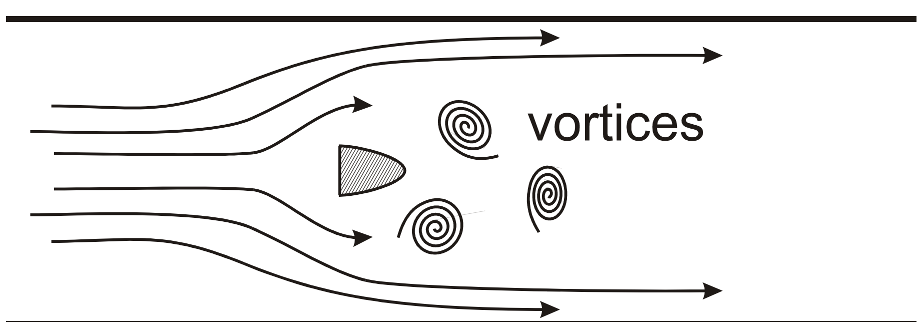
Jet	Pipe Diameter (inch)							
Height	2	3	4	5	6	8	10	12
(inch)	(gpm)	(gpm)	(gpm)	(gpm)	(gpm)	(gpm)	(gpm)	(gpm)
2	28	57	86	115	150	215	285	355
2½	31	69	108	150	205	290	385	480
3	34	78	128	183	250	367	490	610
3½	37	86	145	210	293	440	610	755
4	40	92	160	235	330	510	725	910
4½	42	98	173	257	365	570	845	1060
5	45	104	184	275	395	630	940	1200
6	50	115	205	306	445	730	1125	1500
7	54	125	223	336	485	820	1275	1730
8	58	134	239	360	520	890	1420	1950
9	62	143	254	383	550	955	1550	2140
10	66	152	268	405	585	1015	1650	2280
12	72	167	295	450	650	1120	1830	2550
14	78	182	320	485	705	1220	2000	2800
16	83	195	345	520	755	1300	2140	3000
18	89	208	367	555	800	1400	2280	
20	94	220	388	590	850	1480	2420	
25	107	248	440	665	960	1670	2720	
30	117	275	485	740	1050	1870	3000	
35	127	300	525	800	1150	2020		
40	137	320	565	865	1230	2160		

From Utah Engineering Experiment Station Bulletin 5, June 1955.
"Jet Height" (first column) is the height from the top of the pipe to the top of the jet.



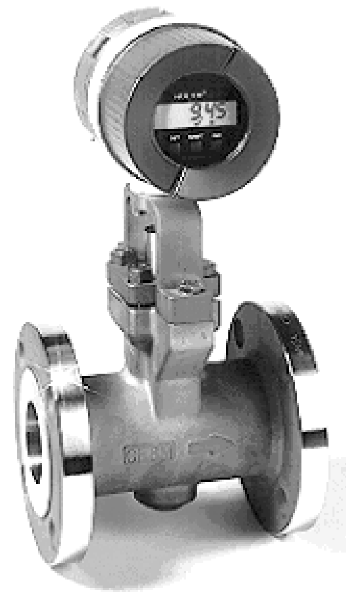
VI. Vortex Shedding Meters

- The vortex shedding meter can be accurate to within ±½% to ±1% of the true discharge
- The basic principal is that an object placed in the flow will cause turbulence and vortices in the downstream direction, and the rate of fluctuation of the vortices can be measured by detecting pressure variations just downstream



- This rate increases with increasing velocity, and it can be used to give an estimate of the discharge
- This requires calibration for a particular pipe material, pipe size, element shape and size, fluid type, and temperature
- It is essentially a velocity-area flow measurement method, but it is calibrated to give discharge directly

- Vortex shedding meters are commercially available and are used with a variety of fluids, not only with water, and can operate well over a large pressure range and high flow velocities
- Size selection for these meters is important in order to avoid cavitation
- There need not be any moving parts in the meter
- This meter can be used for velocities up to approximately 50 m/s, or 180 kph
- The response of the device is linear for Reynolds numbers of 10,000 or more
- Errors can result from pipe vibration due to external machinery or other causes, or when the velocity is too low; however, some sophisticated devices have been developed and tested to correct for such errors



VII. Ultrasonic Meters

1. Doppler

    - An emitted pressure wave reflects off a deflector plate
    - Difference between transmitted and reflected frequencies correlates to flow velocity
    - Liquid does not have to be clean – in fact, it may not work well if the liquid is “too clean” because it needs particles to reflect the signal

2. Transit-time

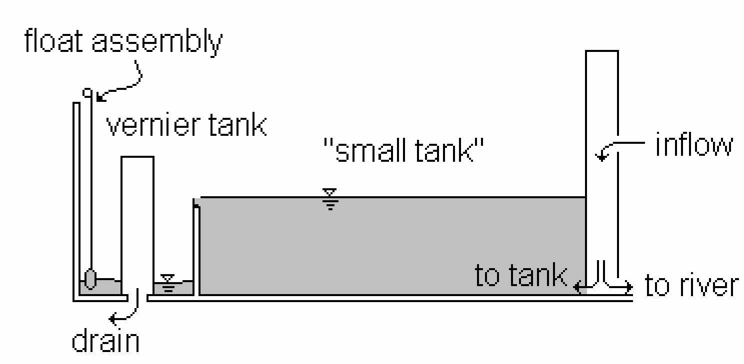
    - Also called “time-of-flight”
    - The liquid should be fairly clean with this method
    - Devices generates high-frequency ($\approx$1 MHz) pressure wave(s)
    - Time to reach an opposing wall (inside the pipe) depends on:

        a) Flow velocity
        b) Beam orientation (angle)
        c) Speed of sound through the liquid medium

    - Upstream straightening vanes may be needed to avoid swirling flow
    - May have a single or multiple transmitted sound beams

VIII. Other Measurement Devices

- Collins meters
- Commercial propeller flow meters
- Electromagnetic flow meters
- Volumetric tank



- Weight tank

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Lecture 15

Canal Design Basics

I. Canal Design Factors

- There are a number of criteria to consider when designing canals
- Below is a list of main criteria, not necessarily in order of importance:

1. Flow rate capacity requirements (demand)
2. Expected flow rate entering the canal (supply)
3. Construction cost
4. Safety considerations
5. Hydraulic operational characteristics
6. Water management needs
7. Maintenance requirements
8. Environmental conservation
9. Need for emergency spill structures
10. Cross-channel surface drainage needs
11. Need for drainage directed *into* the canal
12. Right-of-way (easements) along the canal path
13. Secondary uses (clothes washing, swimming, others)
14. Aesthetics

- Historically, flow rate capacity and construction cost have been the dominant design criteria, but it is better to take into account all of the above factors before finalizing a design
- This is not to say that you necessarily have to dwell on an issue like aesthetics, for example

- However, issues such as dynamic operation, maintenance requirements and need for spillways have often been given only cursory attention during the design phase, requiring subsequent post-construction modifications to the infrastructure
- Water management and operational needs are very similar

- Secondary uses can include things like navigation (large canals), clothes washing, other domestic uses, aquatic production, bathing, and many others
- Remember that every design has both common and unique *(site-specific)* features, compared to other canals

II. Capacity-Based Design

- This is an important consideration because a canal must have sufficient capacity, but not "too much"
- Construction and maintenance costs increase significantly with larger canals
- Actual required delivery system capacity depends on:

  1. size of the irrigated area
  2. cropping patterns (crop types, planting & rotation schedules)
  3. climatological conditions
  4. conveyance efficiencies
  5. on-farm efficiencies
  6. availability & exploitation of other water sources (conjunctive use)
  7. type of delivery schedule (continuous, rotation, on-demand)
  8. non-agricultural water needs

- It is often recommendable to allow for a safety factor by increasing capacities by 10% to 20% in case crops change, an expansion in irrigated area occurs, conveyance losses increase, and other possible factors
- The magnitude of design safety factors is very subjective and debatable
- Capacity requirements can change with different crop types, different total area, different planting schedules, and different efficiencies due to maintenance and rehabilitation (or lack thereof)
- On-demand delivery schedules require higher capacities because the combined requirements will tend to peak higher at some point during each growing season (on-demand delivery schemes can fail when there is not enough water or not enough conveyance capacity)
- Administrative losses can be significant, especially if the delivery schedule is very flexible (need extra water running through the system to buffer sudden turnout deliveries, else spill the excess)

- The required design flow rate capacity is usually known from independent calculations
- For example, irrigation project canal capacities are based on peak crop evapotranspiration requirements and net irrigated area
- A typical main canal capacity is approximately 1 lps per irrigated hectare
- Irrigation canal capacity may also be partially based on non-irrigation requirements, such as municipal supply, industry, fishery & wildlife conservation, and others
- Of course, the capacity of the canals will depend on location, whereby the capacity requirements tend to decrease in the downstream direction due to deliveries in upstream reaches
- But, if the capacity for each reach is known based on crop water and other requirements, and one or more canal layouts have been identified, the design problem becomes one of cross-sectional shape and size, and longitudinal bed slope

- An important point in capacity-based designs is that most canal designs are "static", based only on the hydraulic ability to carry up to a specified maximum flow rate
- The problem with this is that many designs did not consider the "dynamics" of canal operation, nor the type of delivery schedules envisioned
- This oversight has caused many operational difficulties and has limited the operational flexibility of many systems, sometimes severely
- The dynamics of canal operation can be taken into account through design-phase modeling, either with physical models or mathematical models

- In earthen canals, and for canals in general, the most efficient cross section is a secondary consideration to erodibility, maintenance, safety, and convenience
- The ratio of flow depth, h, to canal bottom width, b, usually varies from 1:2 to 1:4 for small canals, and from 1:4 to 1:8 in large canals
- Freeboard can be designed into the canal size at ¼ of the maximum water depth plus one foot (maximum of 6 ft)
- Less freeboard is required if the canal is carefully controlled during operation
- Top width of the bank should allow for a vehicle to pass on one side; the other side can be more narrow

III. System Layout Considerations

- A primary concern in the layout of the system is that it serves the purpose of conveying and distributing water to key locations in the area of service
- Another concern is that the excavation and earthen fill volumes not be excessive
- When large volumes of excavation and or fill are required, the construction costs can increase tremendously
- In fill areas, compaction of the soil material is very important, to avoid settlement problems and possible structural failure

- In reaches constructed over fill, the seepage losses tend to be high, even if the canal is lined
- For these reasons, canals are often designed to follow the existing topography for the design bed slope, which often means routing the canals indirectly so that earth moving work can be minimized, or at least held to an acceptable level
- The selection of longitudinal bed slope should also take into account the existing slopes of the terrain, so as to minimize deviations in canal routing
- Curves in canals should not be too sharp; following are some recommended limits:

Channel Capacity (m^3/s)	Minimum Curve Radius (m)
< 15	300
15-30	600
30 -90	1,000
> 90	1,500

- In bends, the radius of curvature should usually be between 3 and 7 times the top width of flow at maximum design discharge (larger radius for larger canals)

IV. Designing for Maximum Discharge and Uniform Flow

- For a known design discharge, known longitudinal bed slope, and selected cross-sectional shape, the Manning or Chezy equation can be solved for the required depth
- Or, for a known design discharge, known longitudinal bed slope, and specified maximum depth, the Manning equation can be solved for the required base width of a rectangular section
- In general, the equation can be solved for any “unknown”, where all other parameters are specified
- You can also go to the field and measure everything but roughness under steady, uniform flow conditions, then calculate the value of n
- Avoid critical flow at or near design discharge (unstable water surface)

V. Manning Equation

- The Manning equation has been used to size canals all over the world
- It is an empirical equation for approximating uniform flow conditions in open channels
- A roughness factor, n, is used in the equation

- This factor is dependent on the type and condition of the canal lining
- But in reality, the factor also depends on the Reynold's number, N_R (that is, the size and shape of the cross section, not just the roughness of the lining material)
- In practice, it is often erroneously assumed that n is independent of N_R

- In **Metric units**:

$$Q = \frac{1}{n} A R^{2/3} \sqrt{S_o} \tag{1}$$

where Q is in m^3/s; A is cross-section flow area (m^2); R is hydraulic radius (m), equal to A divided by wetted perimeter; and S_o is the longitudinal bed slope (dimensionless)

- In English units, a coefficient must be added to the equation:

$$\frac{1}{(0.3048\ m/ft)^{1/3}} \approx 1.49 \tag{2}$$

- In **English units**:

$$Q = \frac{1.49}{n} A R^{2/3} \sqrt{S_o} \tag{3}$$

where Q is in cfs; A is in ft^2; and R is in (ft)

- An alternative to the Manning equation is the Chezy equation

VI. Chezy Equation

- The Chezy equation is an alternative to the Manning equation, and can be applied as described above
- It is also an empirical equation for approximating uniform flow conditions in open channels, but it has more of a theoretical basis
- The Chezy equation has a diagram analogous to the Moody diagram for the Darcy-Weisbach equation (pipe flow head loss) that takes the Reynold's number into account, which makes it technically more attractive than the Manning equation
- Another advantage is that the Chezy equation can be applied successfully on steeper slopes than the Manning equation

$$Q = CA\sqrt{RS_o} \tag{4}$$

where Q is in m^3/s; A is cross-section flow area (m^2); R is hydraulic radius (m), equal to A divided by wetted perimeter; and S_o is the longitudinal bed slope (dimensionless)

VII. Chezy C Value

- The units of C are $m^{½}/s$
- Note that the numerical value of C increases for smoother surfaces, which is opposite to the behavior of Manning's n
- The relationship between C and Manning's n is (for m and m^3/s):

$$C = \frac{R^{1/6}}{n} \tag{5}$$

- The relationship between C and the Darcy-Weisbach f is:

$$C \approx \sqrt{\frac{8g}{f}} \tag{6}$$

- Thus, C can be defined as a function of relative roughness (ε/R) and Reynold's number, and the resulting graph looks much like the Moody diagram, vertically inverted
- Reynold's number can be defined like this:

$$N_R = \frac{4RV}{\nu} \tag{7}$$

where R is the hydraulic radius (m), A/W_p; V is the mean flow velocity in a cross section (m/s); and ν is the kinematic viscosity of water (m^2/s)

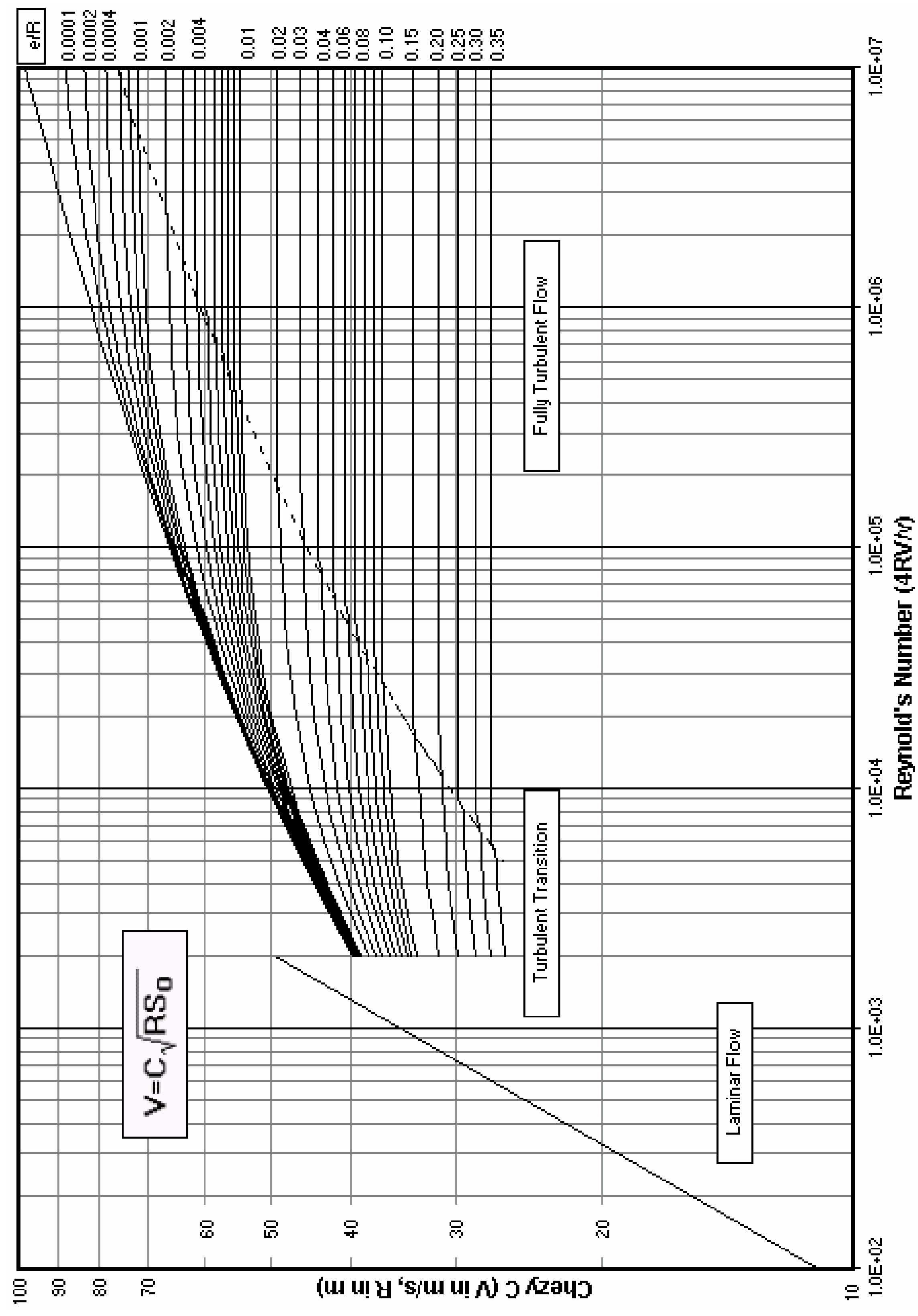
- For a full circle:

$$R = \frac{A}{W_p} = \frac{\pi r^2}{2\pi r} = \frac{r}{2} \tag{8}$$

whereby 4R = D (diameter), so use 4R in general for non-circular sections

- Kinematic viscosity is a function of water temperature

Water Temperature (°C)	**Kinematic Viscosity (m^2/s)**
0	0.000001785
5	0.000001519
10	0.000001306
15	0.000001139
20	0.000001003
25	0.000000893
30	0.000000800
40	0.000000658
50	0.000000553
60	0.000000474



$V = C\sqrt{RS_o}$
Laminar Flow
Turbulent Transition
Fully Turbulent Flow
Chezy C (V in m/s, R in m)
Reynold's Number (4RV/ν)
e/R
0.0001
0.0002
0.0004
0.001
0.002
0.004
0.01
0.02
0.03
0.04
0.06
0.08
0.10
0.15
0.20
0.25
0.30
0.35
10
20
30
40
50
60
70
80
90
100
1.0E+02
1.0E+03
1.0E+04
1.0E+05
1.0E+06
1.0E+07

- For laminar flow (N_R < 2000) and units of m and m^3/s:

$$C = 1.107\sqrt{N_R} \tag{9}$$

which is analogous to the Blasius equation (Darcy-Weisbach f)

- For turbulent smooth flow (N_R > 2000 & $\varepsilon \approx 0$) and units of m and m^3/s:

$$C = -17.7\log_{10}\left(\frac{0.28\,C}{N_R}\right) \tag{10}$$

- For turbulent transitional flow (N_R > 2000 & ε > 0) and units of m and m^3/s:

$$C = -17.7\log_{10}\left(\frac{\varepsilon/R}{12} + \frac{0.28\,C}{N_R}\right) \tag{11}$$

- For turbulent rough flow (N_R > 20,000 & ε > 0), where C is no longer a function of N_R, and units of m and m^3/s:

$$C = 17.7\log_{10}\left(\frac{12}{\varepsilon/R}\right) \tag{12}$$

which gives the flat (horizontal) lines for fully turbulent flow

- To determine the threshold between turbulent transition and turbulent rough flow for a given ε/R ratio, first determine C from Eq. 11, then calculate N_R as:

$$N_R = \frac{75\,C}{\varepsilon/R} \tag{13}$$

- Other equations exist to define the C value as a function of N_R and relative roughness, and these can be found in hydraulics textbooks & handbooks
- For R in ft, A in ft^2, and Q in cfs, multiply C by:

$$\frac{\sqrt{0.3048}}{0.3048} = 2.006 \tag{14}$$

- That is, in English units:

$$Q = 2.006\,C\,A\sqrt{RS_o} \tag{15}$$

where Q is in cfs; A is in ft^2; and R is in ft

- Note that for all but laminar flow, you must iterate to solve for C
- This can be done quickly and easily in a computer program, and the results can be presented as in the graph above

VIII. Chezy Epsilon Values

- Epsilon (roughness height) values depend on channel lining material type & condition:

Material & Condition	ε (m)
Very smooth and essentially seamless concrete	0.0003
Smooth concrete with joints between panels	0.0005
Rough concrete surfaces	0.0012
Very rough concrete surfaces	0.004 to 0.005
Gunite with a smooth finish	0.0005 to 0.0015
Untreated gunite	0.003 to 0.010

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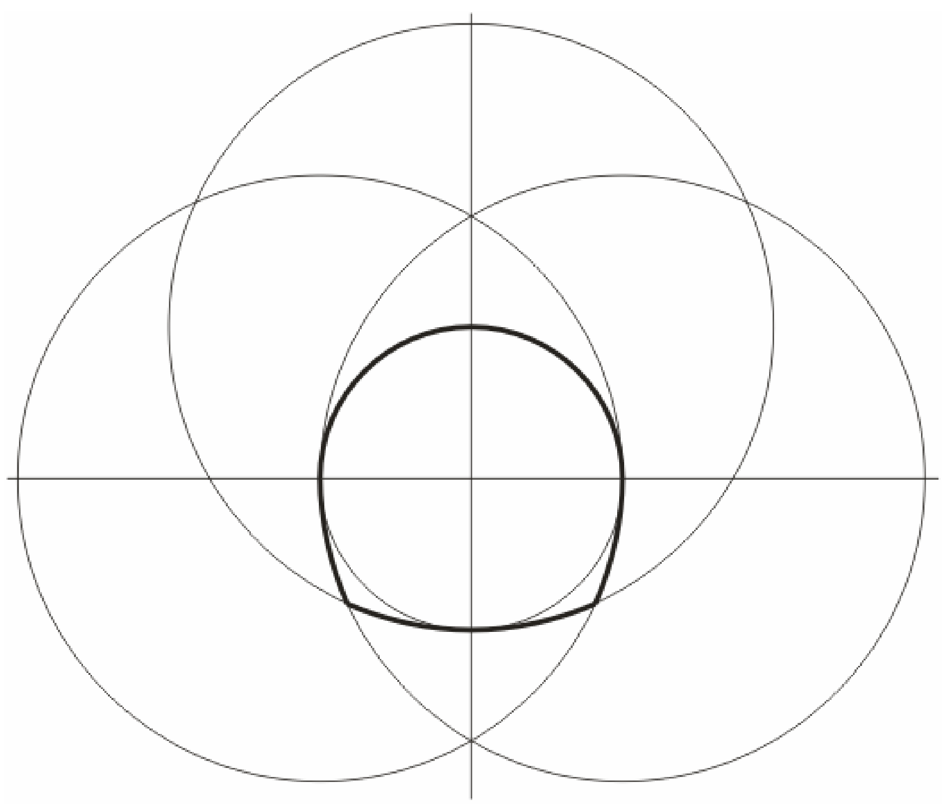
Lecture 16

Channel Cross Sections

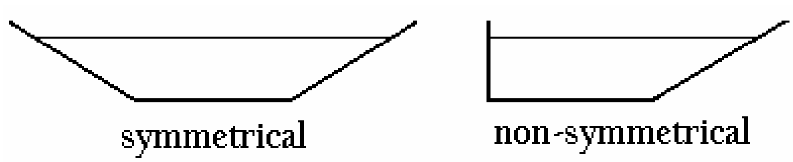
I. Channel Cross Section Parameters

- Common cross-sectional shapes are rectangular, trapezoidal and circular
- Geometrically, rectangular cross-sections are just special cases of trapezoidal sections
- Circular cross sections are hydraulically more “efficient” than other cross-sectional shapes, but they are only used for small channel sizes
- Circular cross sections are usually made of precast concrete mixes, and elevated above the ground
- Tunnels designed for open-channel flow are sometimes built with special cross sections (e.g. “horseshoe” sections)
- The standard horseshoe cross section has a semicircular top portion, and an intersection of three larger circles for the lower portion – it can be considered a modification of a circular section
- A semi-circular channel cross section is the best shape for an open-channel, including open-channel flow in tunnels, but the horseshoe shape has been used in dozens of tunnels to allow for greater floor width, thereby facilitating the passage of equipment through the tunnel



Standard horseshoe cross section (bold curves): diameter of upper section is half of the diameter of the three larger circles

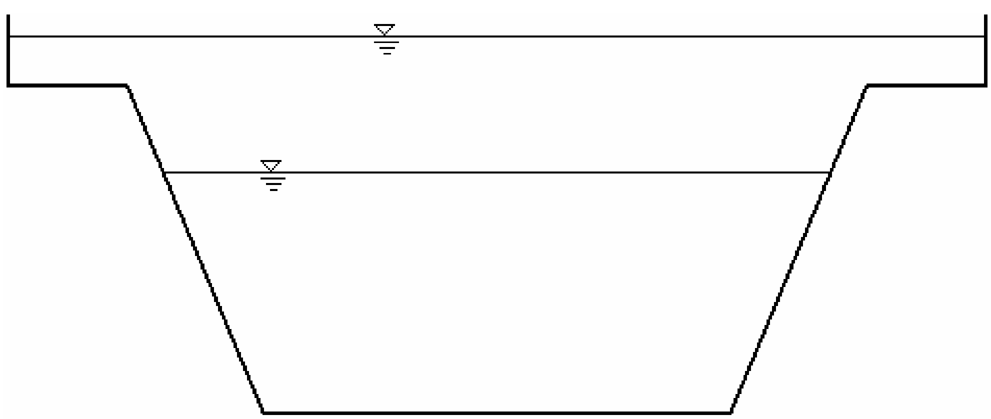
- Trapezoidal cross sections can be symmetrical or non-symmetrical



- For trapezoidal cross sections, the inverse side slope (H:V) is usually between zero and 2.0
- Common inverse side slopes are zero (rectangular section), 1.0 and 1.5
- There are tradeoffs between low and high values of side slope:

  1. canals with low inverse side slopes occupy less land area
  2. high inverse side slopes are more stable and may require less maintenance
  3. high inverse side slopes are safer, if animals or people could fall into the canal, because it is easier to climb out
  4. rectangular cross sections can be simpler to build, when lined with concrete (especially for small cross sections)
  5. it may be easier to build and install structures and transitions for rectangular sections
  6. medium-range side slopes correspond to greater hydraulic efficiency

- Compound sections are not uncommon
- For example, a combination of a trapezoidal and rectangular section:



II. Freeboard Recommendations

- Freeboard means the extra depth of a canal section, above the water surface for 100% flow rate capacity, usually for uniform-flow conditions
- A freeboard value should be added to the maximum expected depth to allow for:

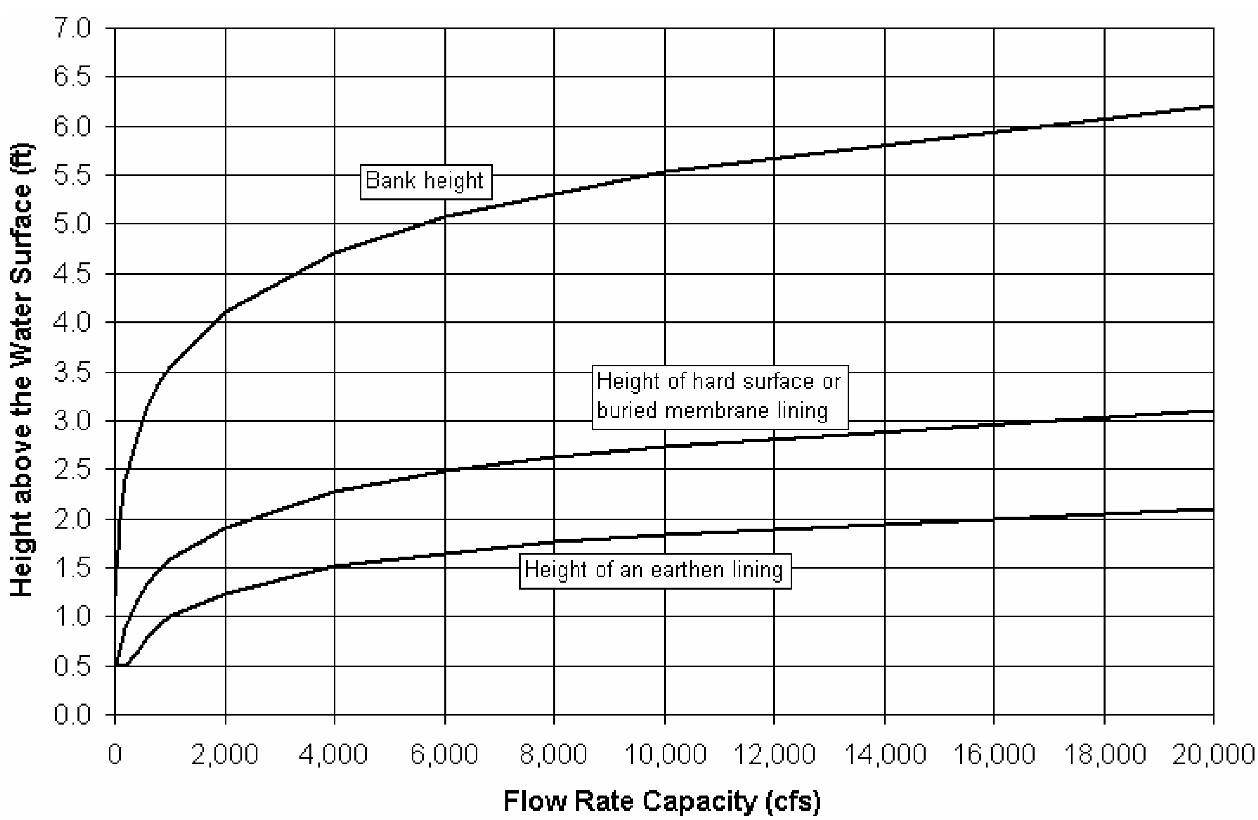
1. deviations between design and construction (these are ubiquitous, only varying in magnitude from place to place)
2. post-construction, non-uniform land settlement
3. operational flexibility (including operator mistakes)
4. accommodate transient flow conditions
5. provide a more conservative design (in terms of flow capacity)
6. increase in hydraulic roughness due to lining deterioration, weed growth, and for other reasons
7. wind loading
8. other reasons

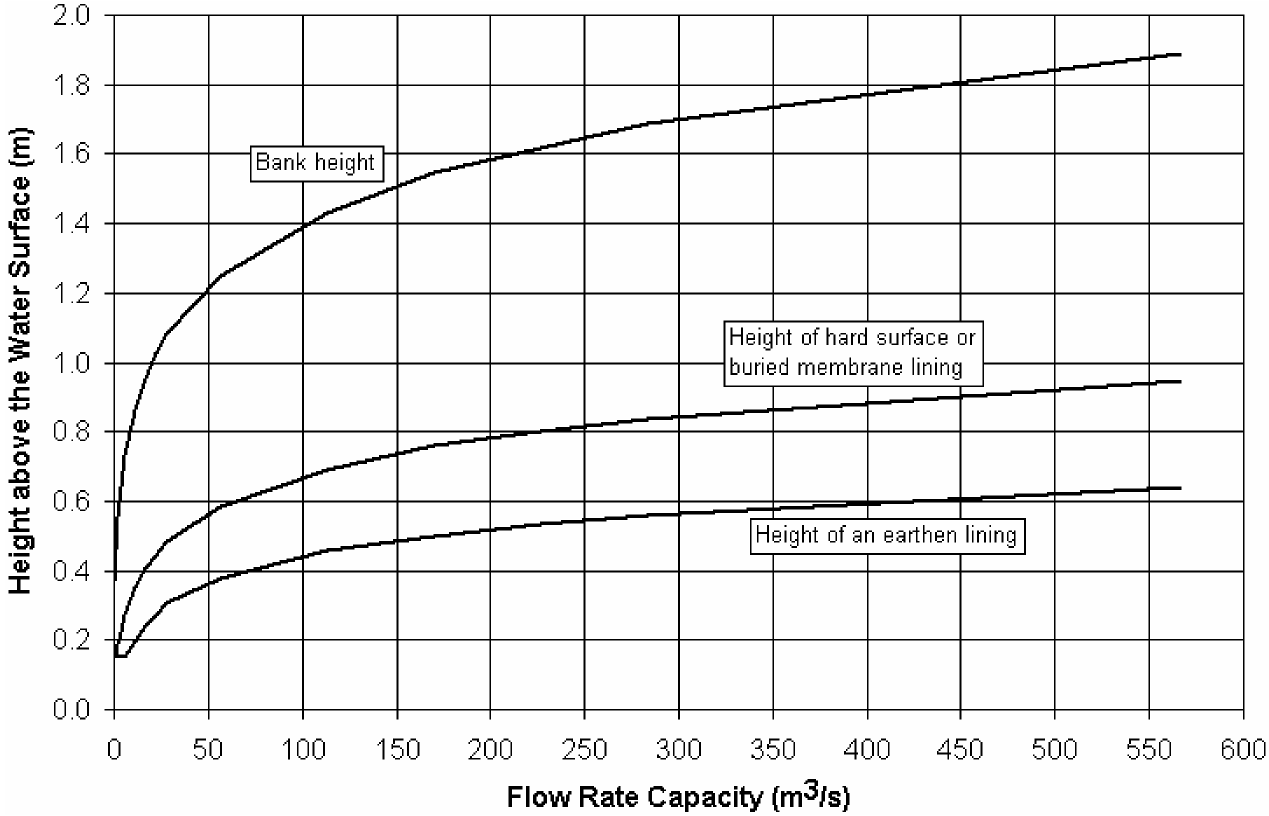
- Thus, with freeboard, under maximum flow conditions (*full supply level*, FSL), canal overflow is not impending
- If the canal starts to overflow, enormous erosive damage can occur in just a few minutes
- According to *Murphy's Law*, these things usually happen about 3:00 am, when no one is around. Then, everyone finds out at about 6:30 am after it has been spilling for hours.
- Many reaches of canal in many countries are routinely operated with virtually no freeboard, and disasters often occur

A canal which is overtopping the banks and spilling water.

A canal with impending spillage (zero freeboard).

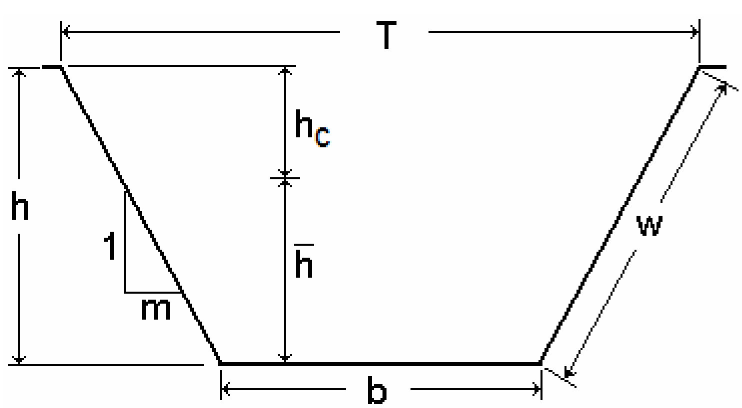
- Traditional wisdom says that the minimum design freeboard for "small canals" is 1 ft (0.3048 m), so this would be the minimum for most any canal, but for very small canals it would certainly be excessive
- For canals with flow rates up to 3,000 cfs (85 m^3/s), the freeboard should be up to 4 ft (1.2 m), and for intermediate flow rates, the freeboard should be 1 ft plus 25% of the maximum expected water depth
- However, the design freeboard can extend above the lined portion of a concrete-lined canal, and it often does (the berm of a lined canal is almost always higher than the top of the lining)
- Also, considerable judgment is required to know what the required freeboard might actually be, including knowledge of the operational modes in the canal (these will help determine the need for freeboard)
- Special analysis should go into the determination of required freeboard for canals with capacities exceeding 3,000 cfs, but such analysis can also be included for any size canal
- The analysis should also include economic criteria, because on large and or long canals, a difference of a few centimeters in freeboard is likely to mean a difference of millions of dollars in construction costs
- Nevertheless, the USBR published data on freeboard guidelines for up to 20,000 cfs (560 m^3/s) capacities, and these are found in the plots below
- To put things in perspective, note that very few irrigation canals exceed a capacity of 100 m^3/s; most main canals have a capacity of less than 20 m^3/s





III. Trapezoidal Cross Section

<u>Symmetrical section</u>:



$$A = h(b + mh) \tag{16}$$

$$T = b + 2mh \tag{17}$$

$$W_p = b + 2h\sqrt{m^2 + 1} \tag{18}$$

$$w = h\sqrt{m^2 + 1} \tag{19}$$

$$m = \frac{T - b}{\sqrt{4w^2 - (T - b)^2}} \tag{20}$$

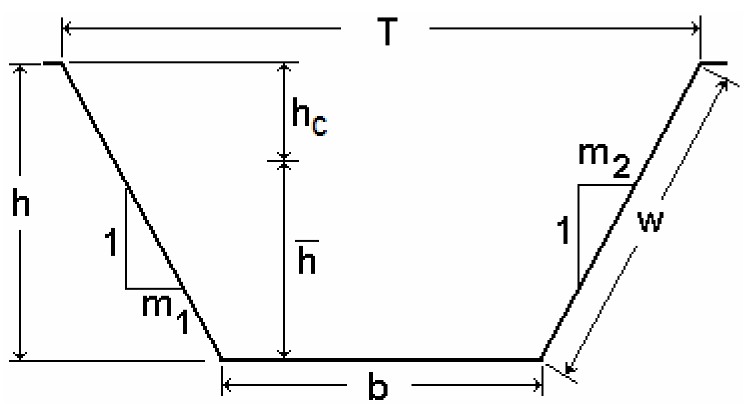
$$h_c = \frac{h^2}{2A}\left(b + \frac{2hm}{3}\right) \tag{21}$$

$$\bar{h} = \frac{h^2}{2A}\left(b + \frac{4hm}{3}\right) \tag{22}$$

where $\bar{h}$ is the depth from the bottom (or "invert") of the cross section up to the centroid of the cross-sectional area; and h_c is the depth from the water surface down to the area centroid:

$$h_c = h - \bar{h} \tag{23}$$

IV. Nonsymmetrical Trapezoidal Cross Section



$$A = h\left[b + 0.5(m_1 + m_2)h\right] \tag{24}$$

$$T = b + h\left(m_1 + m_2\right) \tag{25}$$

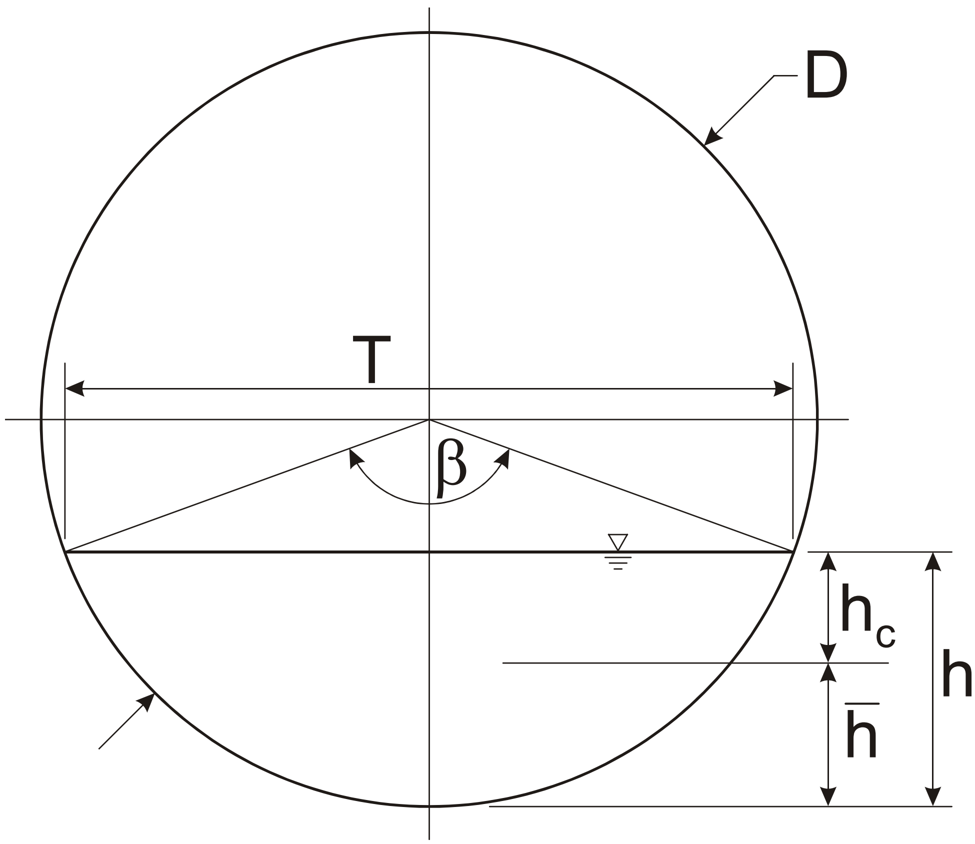
$$W_p = b + h\left(\sqrt{m_1^2 + 1} + \sqrt{m_2^2 + 1}\right) \tag{26}$$

$$w = h\sqrt{m_1^2 + 1} \text{ or } h\sqrt{m_2^2 + 1} \tag{27}$$

$$h_c = \frac{h^2}{2A}\left[b + \frac{h}{3}\left(m_1 + m_2\right)\right] \tag{28}$$

$$\bar{h} = \frac{h^2}{2A}\left[b + \frac{2h}{3}\left(m_1 + m_2\right)\right] \tag{29}$$

V. Circular Cross Section



- In the following, angle β is in radians

$$\beta = 2\cos^{-1}\left(1 - \frac{2h}{D}\right) \tag{30}$$

$$A = \frac{D^2}{8}\left(\beta - \sin\beta\right) \tag{31}$$

or,

$$A = (h - r)\sqrt{2hr - h^2} + r^2\left[\sin^{-1}\left(\frac{h - r}{r}\right) + \frac{\pi}{2}\right] \tag{32}$$

where r = D/2

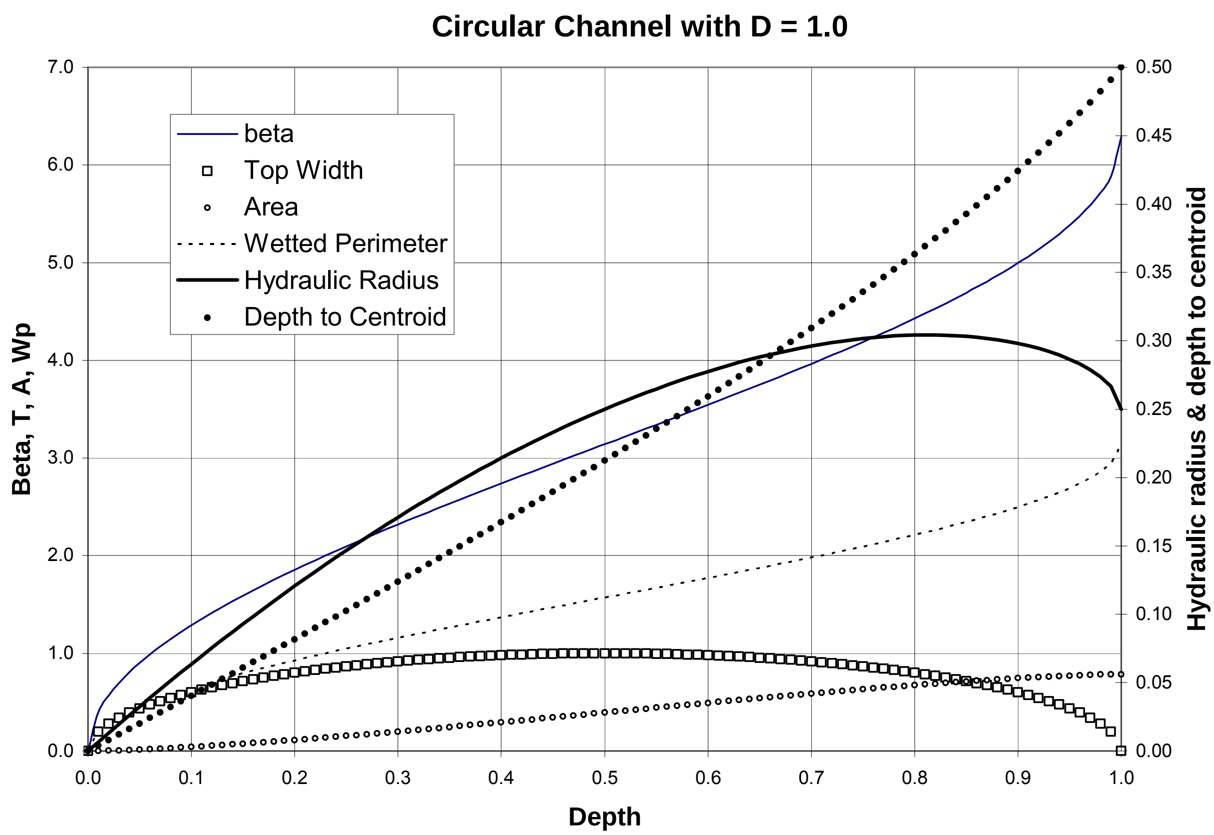
$$T = D\,\sin\left(\frac{\beta}{2}\right) = D\sqrt{1-\left(1-\frac{2h}{D}\right)^2} \tag{33}$$

$$W_p = \frac{\beta D}{2} \tag{34}$$

$$h = \frac{D}{2}\left(1-\cos\frac{\beta}{2}\right) \tag{35}$$

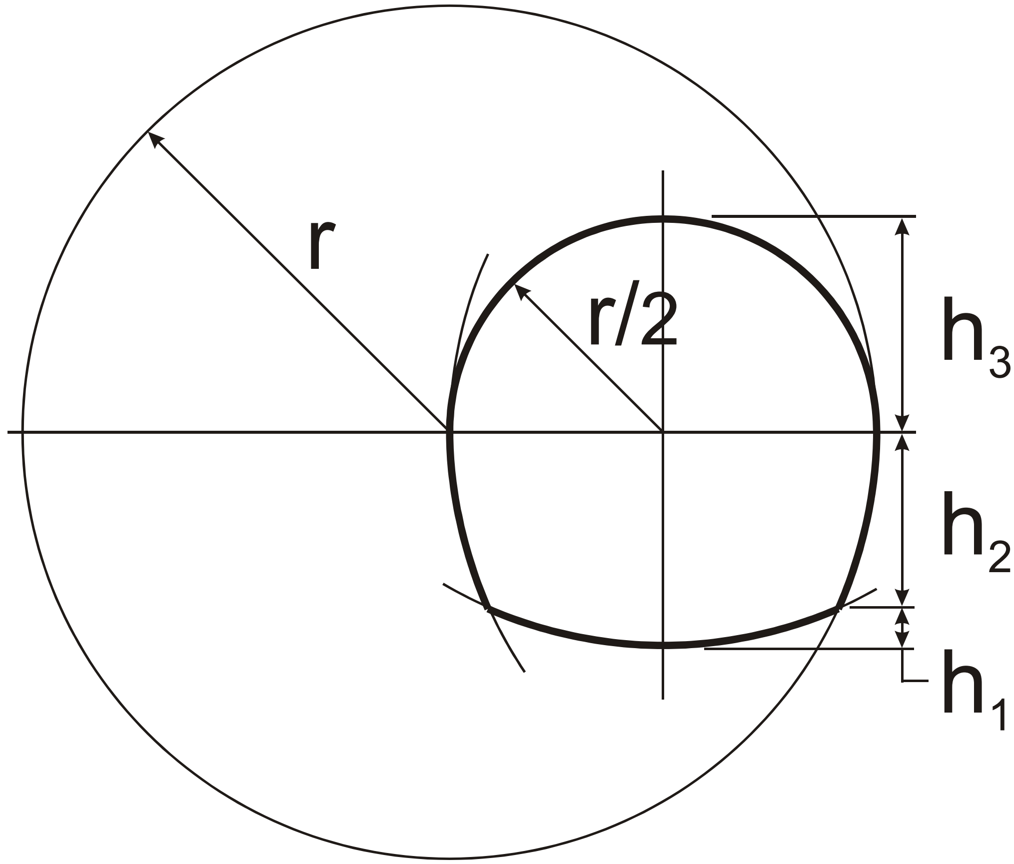
$$\bar{h} = \frac{D}{2} - \frac{2}{3A}\left(hD-h^2\right)^{3/2} \tag{36}$$

$$h_c = h - \bar{h} \tag{37}$$



Nondimensional curves of circular cross-section geometry.

VI. Standard Horseshoe Cross Section



- The following is for a “standard” horseshoe cross section
- Divide the depth into three segments
- Note that $h_1 + h_2 + h_3 = r$ (see the above figure)
- Determine h_1 by solving for the intersection of two of the circles

$$h_1 = r\left[1 - \left(\frac{1+\sqrt{7}}{4}\right)\right] \tag{38}$$

then,

$$h_2 = \frac{r}{2} - h_1 \tag{39}$$

and,

$$h_3 = \frac{r}{2} \tag{40}$$

- In the following, all angles are in radians

Top width (at water surface):

- For $0 < h \le h_1$:

$$T = 2r\sqrt{1-\left(1-\frac{h}{r}\right)^2} \tag{41}$$

- For $h_1 < h \le r/2$:

$$T = \left(\sqrt{r^2-\left(h-\frac{r}{2}\right)^2}-\frac{r}{2}\right)-\left(-\sqrt{r^2-\left(h-\frac{r}{2}\right)^2}+\frac{r}{2}\right) \tag{42}$$

or,

$$T = 2\sqrt{r^2-\left(h-\frac{r}{2}\right)^2}-r \tag{43}$$

- For $r/2 < h < r$:

$$T = r\sqrt{1-\left(1-\frac{2h}{r}\right)^2} \tag{44}$$

and $T = 0$ when $h = 0$ or $h = r$

Cross-sectional area:

- For $0 \le h \le h_1$:

$$A = (h-r)\sqrt{h(2r-h)}+r^2\left[\sin^{-1}\left(\frac{h-r}{r}\right)+\frac{\pi}{2}\right] \tag{45}$$

- For $h_1 < h \le r/2$:

$$A = r^2\left(\alpha_2-\alpha_1-\frac{1}{4}\left[\cot(\alpha_1)-\cot(\alpha_2)\right]\right)-A_a+A_b+A_1 \tag{46}$$

where A_1 is the cross-sectional area corresponding to $h = h_1$; $\cot(\alpha_1)$ is the cotangent of α_1, equal to $1/\tan(\alpha_1)$; and,

$$\varphi_1 = \frac{\sqrt{r^2 - h_2^2}}{h_2} \tag{47}$$

$$\varphi_2 = \frac{\sqrt{r^2 - \left(\frac{r}{2} - h\right)^2}}{\frac{r}{2} - h} \tag{48}$$

$$\alpha_1 = \tan^{-1}\left(\varphi_1\right) \tag{49}$$

$$\alpha_2 = \tan^{-1}\left(\varphi_2\right) \tag{50}$$

$$A_a = \frac{1}{\varphi_2}\left(\sqrt{r^2 - \left(\frac{r}{2} - h\right)^2} - \frac{r}{2}\right)^2 \tag{51}$$

and,

$$A_b = \frac{1}{\varphi_1}\left(\sqrt{r^2 - h_2^2} - \frac{r}{2}\right)^2 \tag{52}$$

- Note that $h_2 = r/2-h_1$
- Note that α_1 and A_b are constants for a given value of r
- Note that $\alpha_2 = \pi/2$ and $A_a = 0$ when $h = r/2$

- Another way to calculate this ($h_1 < h \leq r/2$) area is by integration:

$$A = 2\int_{y_1}^{y_2} x\,dy + A_1 = 2\int_{y_1}^{y_2}\left(-\frac{r}{2} + \sqrt{r^2 - y^2}\right)dy + A_1 \tag{53}$$

which yields the following expression:

$$A = \left[-r\,y + y\sqrt{r^2 - y^2} + r^2 \sin^{-1}\left(\frac{y}{r}\right)\right]_{y_1}^{y_2} + A_1 \tag{54}$$

where y_2 and y_1 are the integration limits:

$$y_2 = h - \frac{r}{2} \tag{55}$$

$$y_1 = \frac{r\,C_1}{2} \tag{56}$$

where,

$$C_1 = 1 - \left(\frac{1+\sqrt{7}}{2}\right) \tag{57}$$

and,

$$C_2 = \frac{C_1}{2}\left(1 - \sqrt{1 - \frac{C_1^2}{4}}\right) - \sin^{-1}\left(\frac{C_1}{2}\right) \tag{58}$$

Finally, applying the integration limits:

$$A = r^2\left[C_2 + \sin^{-1}\left(\frac{2h-r}{2r}\right)\right] - \left(h - \frac{r}{2}\right)\left(r - \sqrt{r^2 - \left(h - \frac{r}{2}\right)^2}\right) + A_1 \tag{59}$$

where A_1 is the area corresponding to $h = h_1$ (Eq. 45)

- Equation 59 is preferred over Eq. 46 because it is simpler and yields the same result for $h_1 < h \leq r/2$

- For $r/2 < h \leq r$:

$$A = \left(h - \frac{r}{2}\right)\sqrt{h(r-h)} + \frac{r^2}{4}\sin^{-1}\left(\frac{2h-r}{r}\right) + A_2 \tag{60}$$

where A_2 is the area corresponding to $h = r/2$ (Eq. 59)

Depth to area centroid:

- For $0 \leq h \leq h_1$:

$$\bar{h} = \frac{r^3}{A}\left[\frac{\pi}{2} - \sin^{-1}\left(1 - \frac{h}{r}\right)\right] - \frac{\sqrt{h(2r-h)}}{3A}\left(hr - 2h^2 + 3r^2\right) \tag{61}$$

where A is as calculated by Eq. 45; and $\bar{h}$ is the depth measured from the area centroid to the bottom of the cross section

- For $h_1 < h \leq r/2$, the moment of area with respect to x is:

$$M_x = \int (yx)\,dy = \int y\left(-r + 2\sqrt{r^2 - y^2}\right)dy$$

$$M_x = -r\int y\,dy + 2\int y\sqrt{r^2 - y^2}\,dy \tag{62}$$

$$M_x = \left[-\frac{ry^2}{2} - \frac{2}{3}\left(r^2 - y^2\right)^{3/2}\right]_{y_1}^{y_2}$$

where y_1 and y_2 are integration limits, exactly as defined above for cross-sectional area. Applying the integration limits:

$$M_x = r^3 C_3 - \frac{r}{2}\left(h - \frac{r}{2}\right)^2 - \frac{2}{3}\left[r^2 - \left(h - \frac{r}{2}\right)^2\right]^{3/2} \tag{63}$$

where,

$$C_3 = \frac{C_1^2}{8} + \frac{2}{3}\left(1 - \frac{C_1^2}{4}\right)^{3/2} \tag{64}$$

where C_3 is a constant; and C_1 is as defined in Eq. 57

- The value of M_x will be negative because it is calculated based on coordinate origins at $h = r/2$, so the depth to centroid for a given depth, h, must be shifted upward by the amount r/2:

$$\bar{h}_x = \frac{r}{2} + \frac{M_x}{A_x} \tag{65}$$

which will be a positive value, with A_x being the cross-sectional area corresponding to the same integration limits, y_1 and y_2:

$$A_x = r^2\left[C_2 + \sin^{-1}\left(\frac{2h - r}{2r}\right)\right] - \left(h - \frac{r}{2}\right)\left(r - \sqrt{r^2 - \left(h - \frac{r}{2}\right)^2}\right) \tag{66}$$

and C_2 is also as previously defined

- The composite value of $\bar{h}$ must account for the calculations up to h = h_1, so for depths from h_1 to r/2, the following area-weighted relationship is used to obtain the exact depth to the area centroid:

$$\bar{h} = \frac{A_x\left(\frac{r}{2} + \frac{M_x}{A_x}\right) + A_1\bar{h}_1}{A_x + A_1} \tag{67}$$

where A_1 and $\bar{h}_1$ are the values corresponding to h = h_1 (Eqs. 45 and 61)

- For r/2 < h ≤ r, the moment of area with respect to x is:

$$M_x = \frac{r^3}{12} - \frac{2}{3}\left[h(r-h)\right]^{3/2} \tag{68}$$

- The cross-sectional area from r/2 up to some h value is:

$$A_x = \left(h - \frac{r}{2}\right)\sqrt{h(r-h)} + \frac{r^2}{4}\sin^{-1}\left(\frac{2h-r}{r}\right) \tag{69}$$

which is Eq. 60 minus the A_2 term

- The composite value of $\bar{h}$ must account for the calculations up to h = r/2, so for depths from r/2 to r, the following area-weighted relationship is used to obtain the exact depth to the area centroid:

$$\bar{h} = \frac{A_x\left(\frac{r}{2} + \frac{M_x}{A_x}\right) + A_2\bar{h}_2}{A_x + A_2} \tag{70}$$

where A_2 and $\bar{h}_2$ are the values corresponding to h = r/2 (Eqs. 59 and 67)

Wetted perimeter:

- For $0 \le h \le h_1$:

$$W_p = 2r\cos^{-1}\left(1 - \frac{h}{r}\right) \tag{71}$$

- For $h_1 < h \le r/2$:

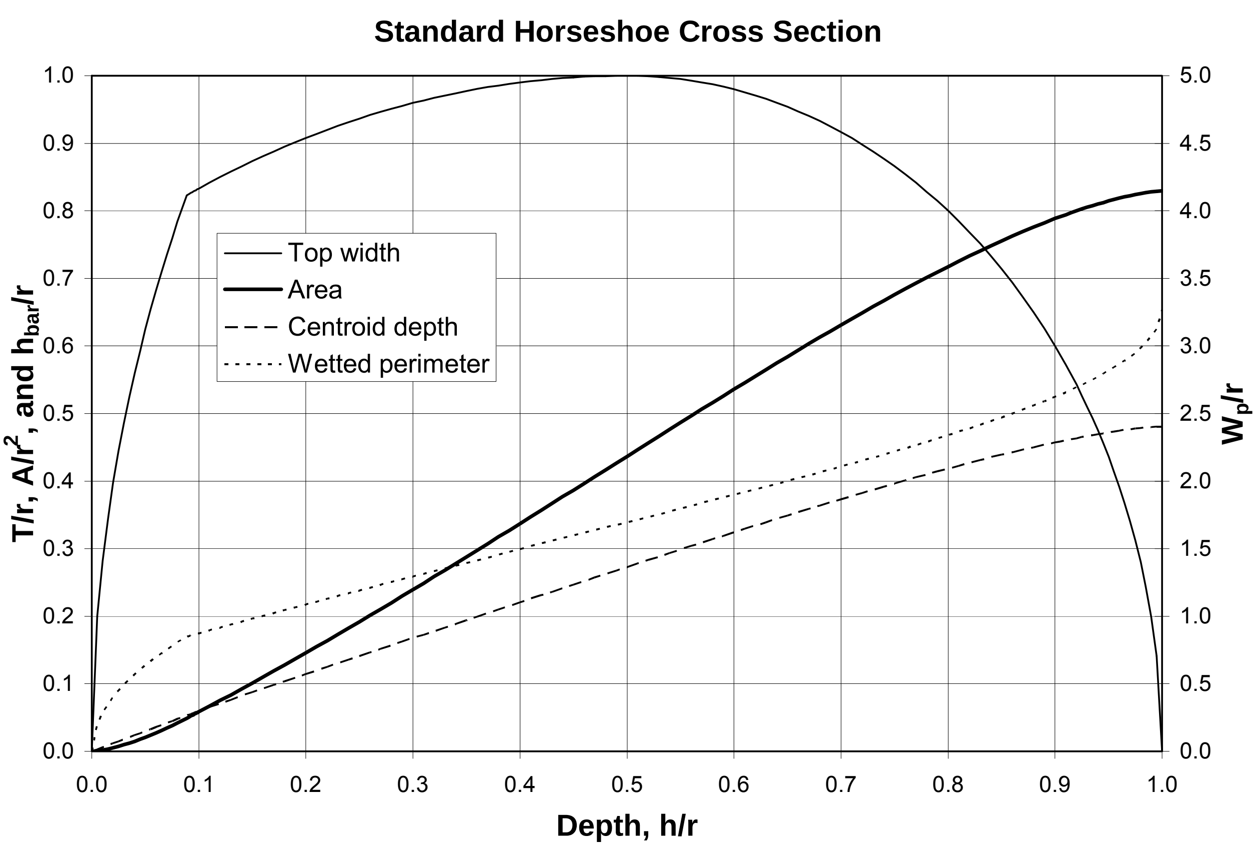
$$W_p = 2r\left[\cos^{-1}\left(\frac{r-2h}{2r}\right) - \cos^{-1}\left(-\frac{C_1}{2}\right)\right] + W_{p1} \tag{72}$$

where C_1 is as defined in Eq. 57; and W_{p1} is the wetted perimeter corresponding to $h = h_1$ (Eq. 71)

- Note that the term "$\cos^{-1}(-C_1/2)$" is a constant, based on h_1
- For $r/2 < h \leq r$:

$$W_p = r\left[\cos^{-1}\left(1-\frac{2h}{r}\right) - \frac{\pi}{2}\right] + W_{p2} \tag{73}$$

where W_{p2} is the wetted perimeter corresponding to $h = r/2$ (Eq. 72)



Nondimensional geometric values in a standard horseshoe cross section.

- The increase in area with the standard horseshoe cross section (compared to a circular section with a diameter of r) is only about 5.6% for a full section

VII. "Efficient" Canal Sections

- Sometimes it is useful to apply an "efficient" cross section to maximize channel capacity for a given bed slope and roughness
- However, other considerations such as side slope stability, safety, and lining material may be more important
- Comparisons between the most efficient section and other sections show that relative changes in the section often do not affect the capacity significantly – capacity is much more sensitive to changes in roughness and bed slope

VIII. Most Efficient Trapezoidal Canal Section

- How to calculate the most efficient trapezoidal cross section?
- Minimize the wetted perimeter with respect to cross-sectional area of flow (or depth), or maximize the hydraulic radius ($R = A/W_p$)
- Express the wetted perimeter as a function of A, m, and h, where h is depth
- Keep area, A, as a constant, otherwise you will get $W_p = 0$ for the most efficient section
- Differentiate W_p with respect to depth, h, and set it equal to zero
- For a symmetrical trapezoidal cross section:

$$A = h(b + mh) \tag{1}$$

$$W_p = b + 2h\sqrt{m^2 + 1} \tag{2}$$

1. Write the wetted perimeter in terms of A, h, and m (get rid of b by combining Eqs. 1 and 2):

$$b = \frac{A}{h} - mh \tag{3}$$

$$W_p = \frac{A}{h} - mh + 2h\sqrt{m^2 + 1} \tag{4}$$

2. Differentiate W_p with respect to h (A and m constant) and equate to zero (to minimize W_p for a given area):

$$\frac{\partial W_p}{\partial h} = \frac{-A}{h^2} - m + 2\sqrt{m^2 + 1} = 0 \tag{5}$$

3. Solve Eq. 5 for A

$$A = h^2\left(2\sqrt{m^2+1} - m\right) \tag{6}$$

4. For R = A/W_p, use Eq. 6 to obtain

$$R = \frac{h^2\left(2\sqrt{m^2+1} - m\right)}{b + 2h\sqrt{m^2+1}} \tag{7}$$

5. Now, manipulate Eq. 7

$$R = \frac{bh + 2h^2\sqrt{m^2+1} - bh - mh^2}{b + 2h\sqrt{m^2+1}} \tag{8}$$

$$R = \frac{h\left(b + 2h\sqrt{m^2+1}\right) - h(b + mh)}{b + 2h\sqrt{m^2+1}} \tag{9}$$

$$R = \frac{hW_p - A}{W_p} = h - R \tag{10}$$

6. Therefore, h = 2R, or,

$$R = \frac{h}{2} \tag{11}$$

You could also directly manipulate Eq. 6 to get the same result: 2A = hW_p

7. For the most efficient *rectangular section*,

$$R = \frac{A}{W_p} = \frac{bh}{b + 2h} = \frac{h}{2} \tag{12}$$

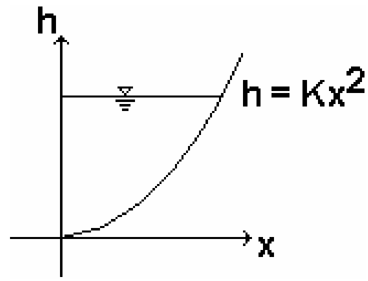
which results in b = 2h (bed width twice the maximum flow depth).

8. For the most efficient *trapezoidal section* we will get W_p = T + b, where T is the top width of flow (b+2mh), which for a symmetrical trapezoid means that the length of each side slope (for depth h) is T/2. It also means that b = T/2, and this corresponds to half of a regular six-sided polygon, or a hexagon. The interior angle of a hexagon is 120 degrees, so m = 1/tan(60°) = **0.577**.

IX. Parabolic Canal Section

- Suppose you have a parabolic channel section…
- Define half of a symmetrical parabolic section as:

$$h = Kx^2 \tag{13}$$



- The cross-sectional area of flow (for half of the section) is:

$$A = \int_0^h x\,dh = \int_0^h \sqrt{\frac{h}{K}}\;dh = \frac{2h^{3/2}}{3\sqrt{K}} \tag{14}$$

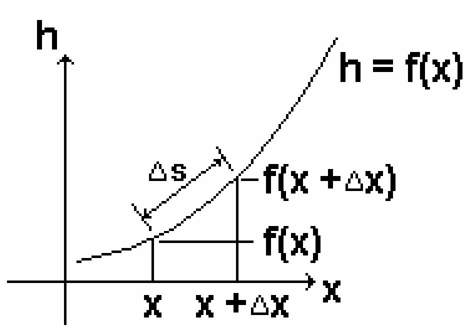
- The wetted perimeter (again, half of the section) is:

$$W_p = \lim_{\Delta x \to 0} \sum \sqrt{\left[f\left(x+\Delta x\right) - f\left(x\right)\right]^2 + \left(\Delta x\right)^2} \tag{15}$$

or,

$$W_p = \int \left(\left(\frac{df}{dx} \right)^2 + 1 \right)^{1/2} dx \tag{16}$$

where f = Kx^2. This derivation can be described graphically as follows:



where the curve is broken up (discretized) into successive linear segments...

$$\Delta s \approx \sqrt{\left[f\left(x+\Delta x\right)-f\left(x\right)\right]^2+\left(\Delta x\right)^2} \tag{17}$$

- For $y = f(x) = Kx^2$,

$$\left(\frac{df}{dx}\right)^2 = 4K^2x^2 \tag{18}$$

- Then,

$$W_p = \int_0^{\sqrt{h/K}} \sqrt{4K^2x^2+1}\,dx \tag{19}$$

- After integration (using integration tables), the wetted perimeter for half of the parabolic section is:

$$W_p = K\sqrt{\frac{h}{K}\left(\frac{h}{K}+\frac{1}{4K^2}\right)}+\frac{1}{4K}\ln\left[2K\left(\sqrt{\frac{h}{K}}+\sqrt{\frac{h}{K}+\frac{1}{4K^2}}\right)\right] \tag{20}$$

which of course is a function of both K (curvature) and depth (h)

- An analysis of the hydraulic radius for such a parabolic section shows that the hydraulic radius decreases monotonically as K increases from an infinitesimally small value, so there is no “most efficient” value of K
- Chow (1959) has some equations (exact and approximate) for various channel section shapes, including the parabola defined in this case

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Lecture 17

Design of Earthen Canals

I. General

A borrow pit

- Much of this information applies in general to both earthen and lined canals
- Attempt to balance cuts and fills to avoid waste material and or the need for “borrow pits” along the canal
- It is expensive to move earth long distances, and or to move it in large volumes
- Many large canals zigzag across the terrain to accommodate natural slopes; this makes the canal longer than it may need to be, but earthwork is less
- Canals may also follow the contours along hilly or mountainous terrain
- Of course, canal routing must also consider the location of water delivery points
- In hilly and mountainous terrain, canals generally follow contour gradients equal to the design bed slope of the canal
- Adjustments can be made by applying geometrical equations, but usually a lot of hand calculations and trial-and-error are required
- As previously discussed, it is generally best to follow the natural contour of the land such that the longitudinal bed slope is acceptable
- Most large- and medium-size irrigation canals have longitudinal slopes from 0.00005 to 0.001 m/m
- A typical design value is 0.000125 m/m, but in mountainous areas the slope may be as high as 0.001 m/m: elevation change is more than enough
- With larger bed slopes the problems of sedimentation can be lessened
- In the technical literature, it is possible to find many papers and articles on canal design, including application of mathematical optimization techniques (e.g. FAO Irrig & Drain Paper #44), some of which are many years old
- The design of new canals is not as predominant as it once was

An earthen channel

II. Earthen Canal Design Criteria

- Design cross sections are usually trapezoidal
- Field measurements of many older canals will also show that this is the range of averaged side slopes, even though they don’t appear to be trapezoidal in shape

- When canals are built on hillsides, a berm on the uphill side should be constructed to help prevent sloughing and landslides, which could block the canal and cause considerable damage if the canal is breached

III. Earth Canal Design: Velocity Limitations

- In designing earthen canals it is necessary to consider erodibility of the banks and bed -- this is an "empirical" exercise, and experience by the designer is valuable
- Below are four methods applied to the design of earthen channels
- The first three of these are entirely empirical
- All of these methods apply to open channels with erodible boundaries in alluvial soils carrying sediment in the water

1. Kennedy Formula
2. Lacey Method
3. Maximum Velocity Method
4. Tractive-Force Method

1. Kennedy Formula

- Originally developed by British on a canal system in Pakistan
- Previously in wide use, but not used very much today

$$V_o = C_1 \left(h_{avg} \right)^{C_2} \tag{1}$$

where V_o is the velocity (fps); and h_{avg} is the mean water depth (ft)

- The resulting velocity is supposed to be "just right", so that neither erosion nor sediment deposition will occur in the channel
- The coefficient (C_1) and exponent (C_2) can be adjusted for specific conditions, preferably based on field measurements
- C_1 is mostly a function of the characteristics of the earthen material in the channel
- C_2 is dependent on the silt load of the water
- Below are values for the coefficient and exponent of the Kennedy formula:

Table 1. Calibration values for the Kennedy formula.

C_1	Material
0.56	extremely fine soil
0.84	fine, light sandy soil
0.92	coarse, light sandy soil
1.01	sandy, loamy silt
1.09	coarse silt or hard silt debris

C_2	Sediment Load
0.64	water containing very fine silt
0.50	clear water

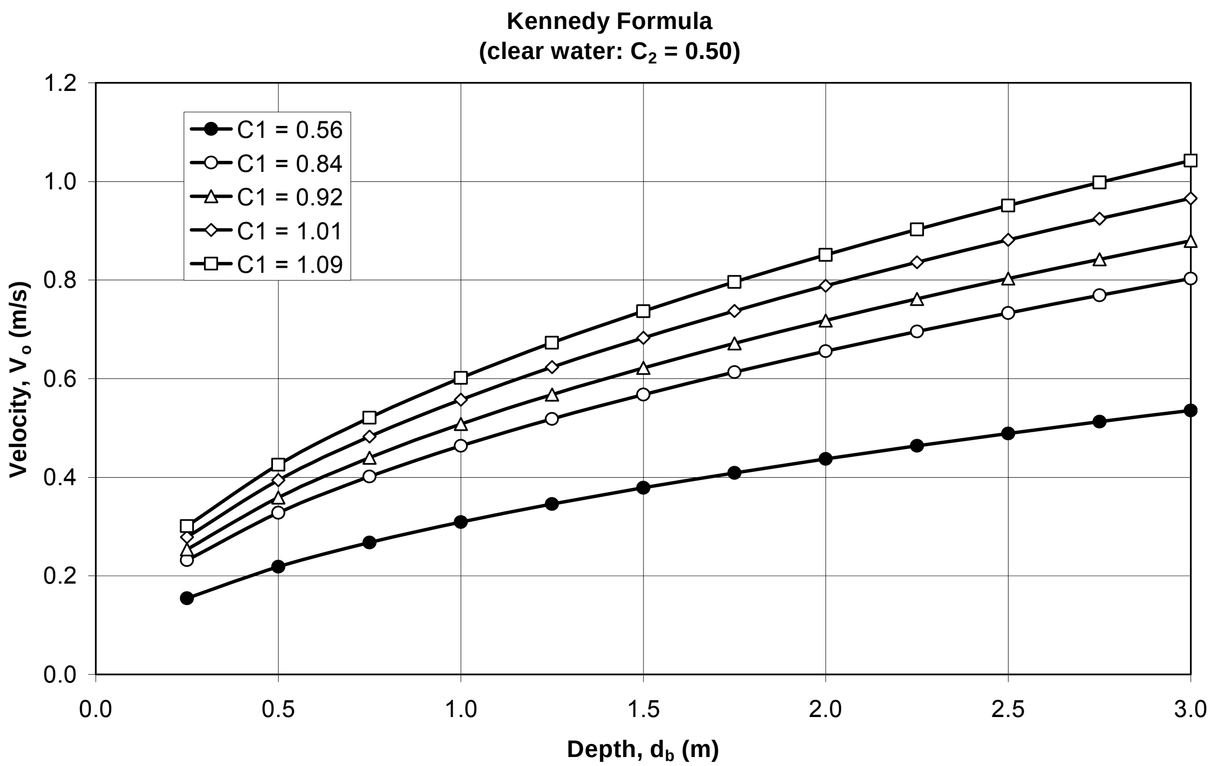


Figure 1. Velocity values versus water depth for the Kennedy formula with clear water.

2. Lacey Method

- Developed by G. Lacey in the early part of the 20th century based on data from India, Pakistan, Egypt and elsewhere
- Supports the "Lindley Regime Concept", in which Lindley wrote:

> "*when an artificial channel is used to convey silty water, both bed and banks scour or fill, changing depth, gradient and width, until a state of balance is attained at which the channel is said to be in regime"*

- There are four relationships in the Lacey method
- All four must be satisfied to achieve "regime" conditions

1. Velocity

$$V = 1.17\sqrt{fR} \tag{2}$$

2. Wetted Perimeter

$$W_p = 2.67\sqrt{Q} \tag{3}$$

3. Hydraulic Radius

$$R = 0.47\sqrt[3]{Q/f} \tag{4}$$

4. Bed Slope

$$S = 0.000547\frac{f^{2/3}}{Q^{1/6}} \tag{5}$$

where,

$$f = 1.76\sqrt{d_m} \tag{6}$$

and, d_m is the mean diameter of the bed and side slope materials (mm); V is the mean velocity over the cross-section (fps); W_p is the wetted perimeter (ft); R is the hydraulic radius (ft); S is the longitudinal bed slope (ft/ft); and Q is discharge (cfs)

- The above relationships can be algebraically manipulated to derive other dependent relationships that may be convenient for some applications
- For example, solve for S in terms of discharge
- Or, solve for d_m as a function of R and V
- Here are two variations of the equations:

$$V = 0.00124\, d_m^{11/12} / S \tag{7}$$

and,

$$V = 0.881\, Q^{1/6} d_m^{1/12} \tag{8}$$

- A weakness in the above method is that it considers particle size, d_m, but not cohesion & adhesion

<u>Lacey General Slope Formula</u>:

$$V = \frac{1.346}{N_a} R^{0.75}\sqrt{S} \tag{9}$$

where N_a is a roughness factor, defined as:

$$N_a = 0.0225 f^{0.25} \cong 0.9 n R^{0.083} \tag{10}$$

where n is the Manning roughness factor

- This is for uniform flow conditions
- Applies to both regime and non-regime conditions
- Appears similar to the Manning equation, but according to Lacey it is more representative of flow in alluvial channels

3. Maximum Velocity Method

- This method gives the maximum permissible mean velocity based on the type of bed material and silt load of the water
- It is basically a compilation of field data, experience, and judgment
- Does not consider the depth of flow, which is generally regarded as an important factor in determining velocity limits

Table 2. Maximum permissible velocities recommended by Fortier and Scobey

Material	Velocity (fps)	
	Clear water	Water with colloidal silt
Fine sand, colloidal	1.5	2.5
Sandy loam, non-colloidal	1.75	2.5
Silt loam, non-colloidal	2	3
Alluvial silt, non-colloidal	2	3.5
Firm loam soil	2.5	3.5
Volcanic ash	2.5	3.5
Stiff clay, highly colloidal	3.75	5
Alluvial silt, colloidal	3.75	5
Shales and hard "pans"	6	6
Fine gravel	2.5	5
Coarse gravel	4	6
Cobble and shingle	5	5.5

Table 3. USBR data on permissible velocities for non-cohesive soils

Material	Particle diameter (mm)	Mean velocity (fps)
Silt	0.005-0.05	0.49
Fine sand	0.05-0.25	0.66
Medium sand	0.25	0.98
Coarse sand	1.00-2.50	1.80
Fine gravel	2.50-5.00	2.13
Medium gravel	5.00	2.62
Coarse gravel	10.00-15.00	3.28
Fine pebbles	15.00-20.00	3.94
Medium pebbles	25.00	4.59
Coarse pebbles	40.00-75.00	5.91
Large pebbles	75.00-200.00	7.87-12.80

IV. Introduction to the Tractive Force Method

- This method is to prevent scouring, not sediment deposition
- This is another design methodology for earthen channels, but it is not 100% empirical, unlike the previously discussed methods
- It is most applicable to the design of earthen channels with erodible boundaries (wetted perimeter) carrying clear water, and earthen channels in which the material forming the boundaries is much coarser than the transported sediment
- The tractive force is that which is exerted on soil particles on the wetted perimeter of an earthen channel by the water flowing in the channel
- The "tractive force" is actually a shear stress multiplied by an area upon which the stress acts
- A component of the force of gravity on the side slope material is added to the analysis, whereby gravity will tend to cause soil particles to roll or slide down toward the channel *invert* (bed, or bottom)
- The design methodology treats the bed of the channel separately from the side slopes

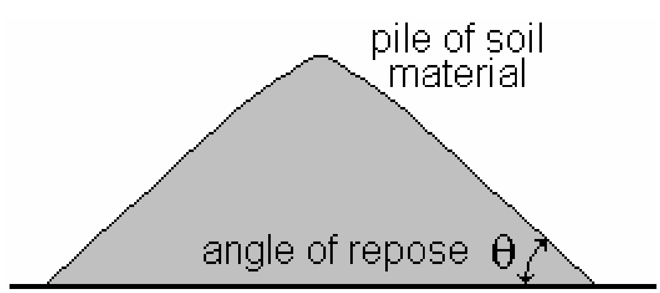
- The key criterion is whether the tractive + gravity forces are less than the "critical" tractive force of the materials along the wetted perimeter of the channel
- If this is true, the channel should not experience scouring (erosion) from the flow of water within
- Thus, the critical tractive force is the threshold value at which scouring would be expected to begin
- This earthen canal design approach is for the prevention of scouring, but not for the prevention of sediment deposition
- The design methodology is for trapezoidal or rectangular cross sections
- This methodology was developed by the USBR

V. Forces on Bed Particles

- The friction force (resisting particle movement) is:

$$W_s \tan\theta \tag{11}$$

where θ is the angle of repose of the bed material and W_s is the weight of a soil particle



- Use the angle of repose for wet (not dry) material
- θ will be larger for most wet materials
- Note that "tan θ" is the angle of repose represented as a slope

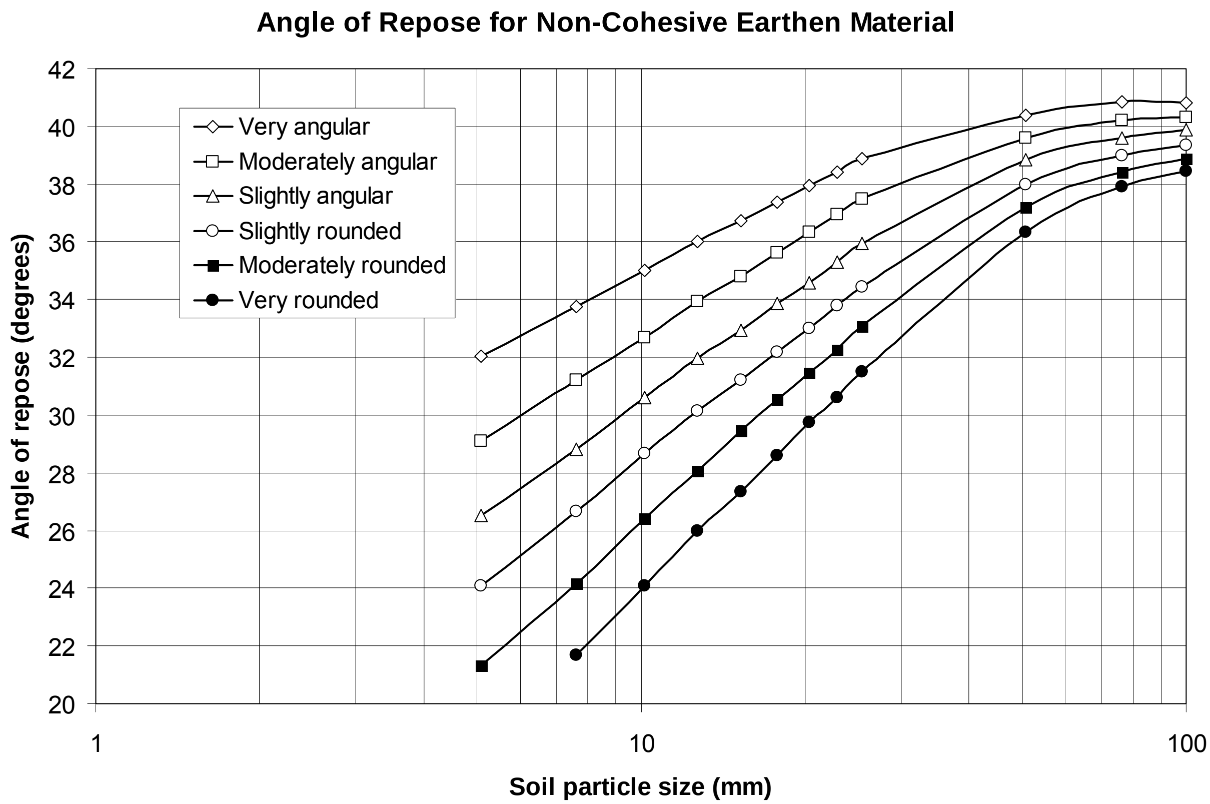


Figure 2. Angle of repose (degrees from horizontal), θ, for non-cohesive earthen materials (adapted from USBR Hyd Lab Report Hyd-366).

- The shear force on a bed particle is:

$$aT_{bed} \tag{12}$$

where "a" is the effective particle area and T_{bed} (lbs/ft^2 or N/m^2) is the shear stress exerted on the particle by the flow of water in the channel

- When particle movement is impending on the channel bed, expressions 1 and 2 are equal, and:

$$W_s \tan\theta = aT_{bed} \tag{13}$$

or,

$$T_{bed} = \frac{W_s \tan\theta}{a} \tag{14}$$

VI. Forces on Side-Slope Particles

- The component of gravity down the side slope is:

$$W_s \sin\phi \tag{15}$$

where ϕ is the angle of the side slope, as defined in the figure below

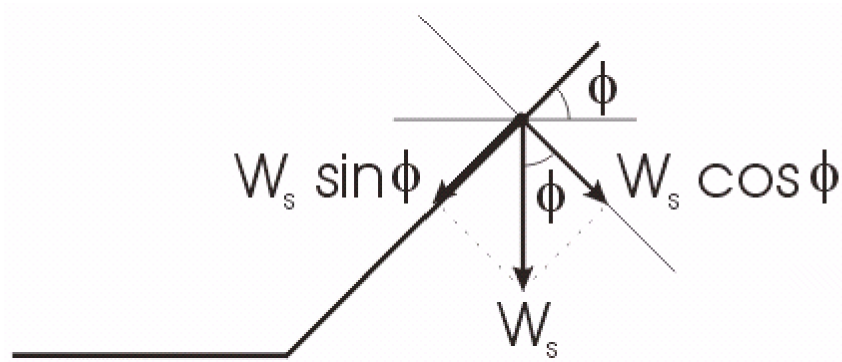


Figure 3. Force components on a soil particle along the side slope of an earthen channel.

- If the inverse side slope is m, then:

$$\phi = \tan^{-1}\left(\frac{1}{m}\right) \tag{16}$$

- The force on the side slope particles in the direction of water flow is:

$$aT_{side} \tag{17}$$

where T_{side} is the shear stress (lbs/ft^2 or N/m^2) exerted on the side slope particle by the flow of water in the channel

- Note: multiply lbs/ft^2 by 47.9 to convert to N/m^2
- Combining Eqs. 15 & 17, the resultant force on the side slope particles is downward and toward the direction of water flow, with the following magnitude:

$$\sqrt{W_s^2 \sin^2\phi + a^2 T_{side}^2} \tag{18}$$

- The resistance to particle movement on the side slopes is due to the orthogonal component of Eq. 15, $W_s\cos\phi$, as shown in the above figure, multiplied by the coefficient of friction, tan θ
- Thus, when particle movement is impending on the side slopes:

$$W_s \cos\phi \tan\theta = \sqrt{W_s^2 \sin^2\phi + a^2 T_{side}^2} \tag{19}$$

- Solving Eq. 19 for T_{side}:

$$T_{side} = \frac{W_s}{a}\sqrt{\cos^2\phi \tan^2\theta - \sin^2\phi} \tag{20}$$

- Applying trigonometric identities and simplifying:

$$T_{side} = \frac{W_s}{a}\cos\phi \tan\theta\sqrt{1 - \frac{\tan^2\phi}{\tan^2\theta}} \tag{21}$$

or,

$$T_{side} = \frac{W_s}{a}\tan\theta\sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}} \tag{22}$$

VII. Tractive Force Ratio

- As defined in Eq. 14, T_{bed} is the critical shear on bed particles
- As defined in Eqs. 20-22, T_{side} is the critical shear on side slope particles
- The *tractive force ratio*, K, is defined as:

$$K = \frac{T_{side}}{T_{bed}} \tag{23}$$

where T_{side} and T_{bed} are the critical (threshold) values defined in Eqs. 4 & 9-11

- Then:

$$K = \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}} = \cos\phi\sqrt{1 - \frac{\tan^2\phi}{\tan^2\theta}} \tag{24}$$

VIII. Design Procedure

- The design procedure is based on calculations of maximum depth of flow, h
- Separate values are calculated for the channel bed and the side slopes, respectively

- It is necessary to choose values for inverse side slope, m, and bed width, b to calculate maximum allowable depth in this procedure
- Limits on side slope will be found according to the angle of repose and the maximum allowable channel width
- Limits on bed width can be set by specifying allowable ranges on the ratio of b/h, where b is the channel base width and h is the flow depth
- Thus, the procedure involves some trial and error

Step 0

- Specify the desired maximum discharge in the channel
- Identify the soil characteristics (particle size gradation, cohesion)
- Determine the angle of repose of the soil material, θ
- Determine the longitudinal bed slope, S_o, of the channel

Step 1

- Determine the critical shear stress, T_c (N/m^2 or lbs/ft^2), based on the type of material and particle size from Fig. 3 or 4 (note: 47.90 N/m^2 per lbs/ft^2)
- Fig. 3 is for cohesive material; Fig. 4 is for non-cohesive material
- Limit ϕ according to θ (let $\phi \leq \theta$)

Step 2

- Choose a value for b
- Choose a value for m

Step 3

- Calculate ϕ from Eq. 16
- Calculate K from Eq. 24
- Determine the max shear stress fraction (dimensionless), K_{bed}, for the channel bed, based on the b/h ratio and Fig. 6
- Determine the max shear stress fraction (dimensionless), K_{side}, for the channel side slopes, based on the b/h ratio and Fig. 7

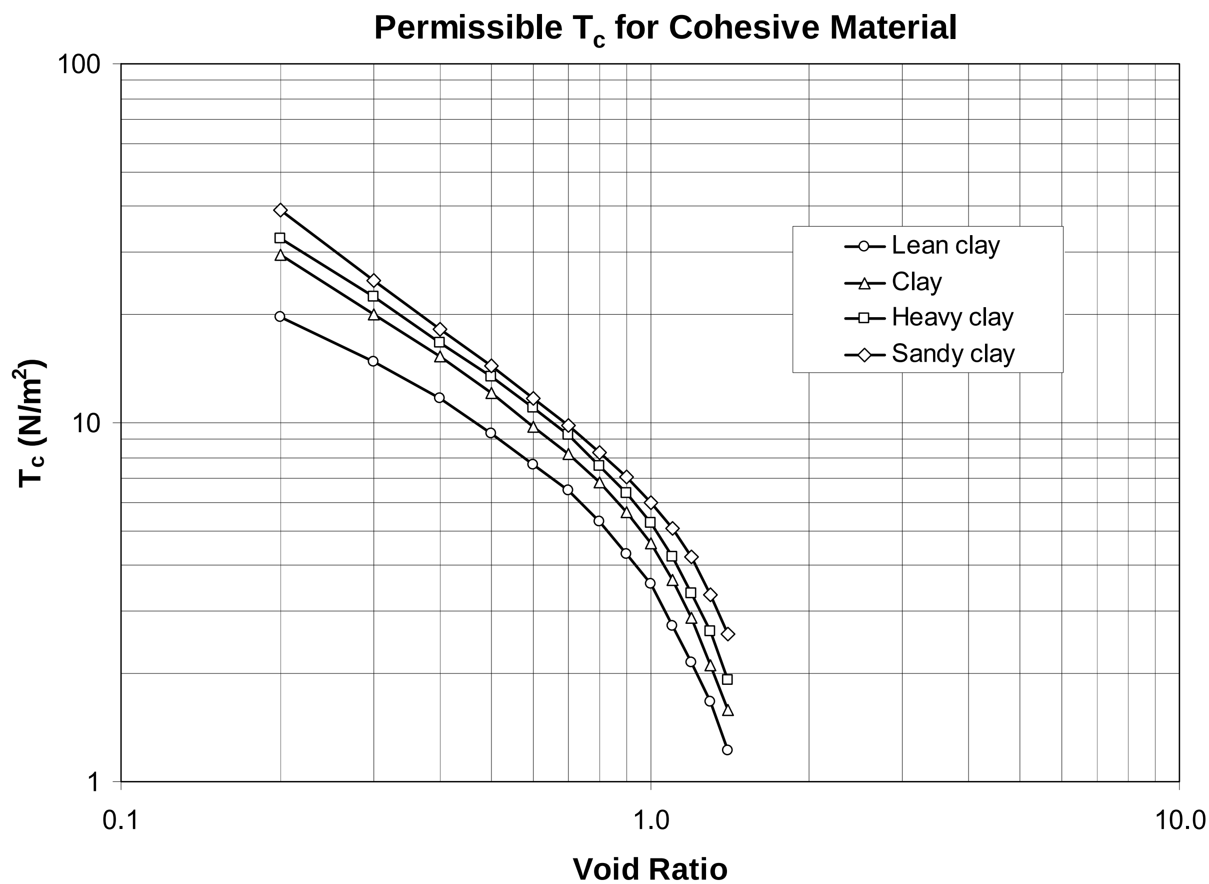


Figure 4. Permissible value of critical shear stress, T_c, in N/m^2, for cohesive earthen material (adapted from USBR Hyd Lab Report Hyd-352).

About Figure 4: The "void ratio" is the ratio of volume of pores to volume of solids. Note that it is greater than 1.0 when there is more void space than that occupied by solids. The void ratio for soils is usually between 0.3 and 2.0.

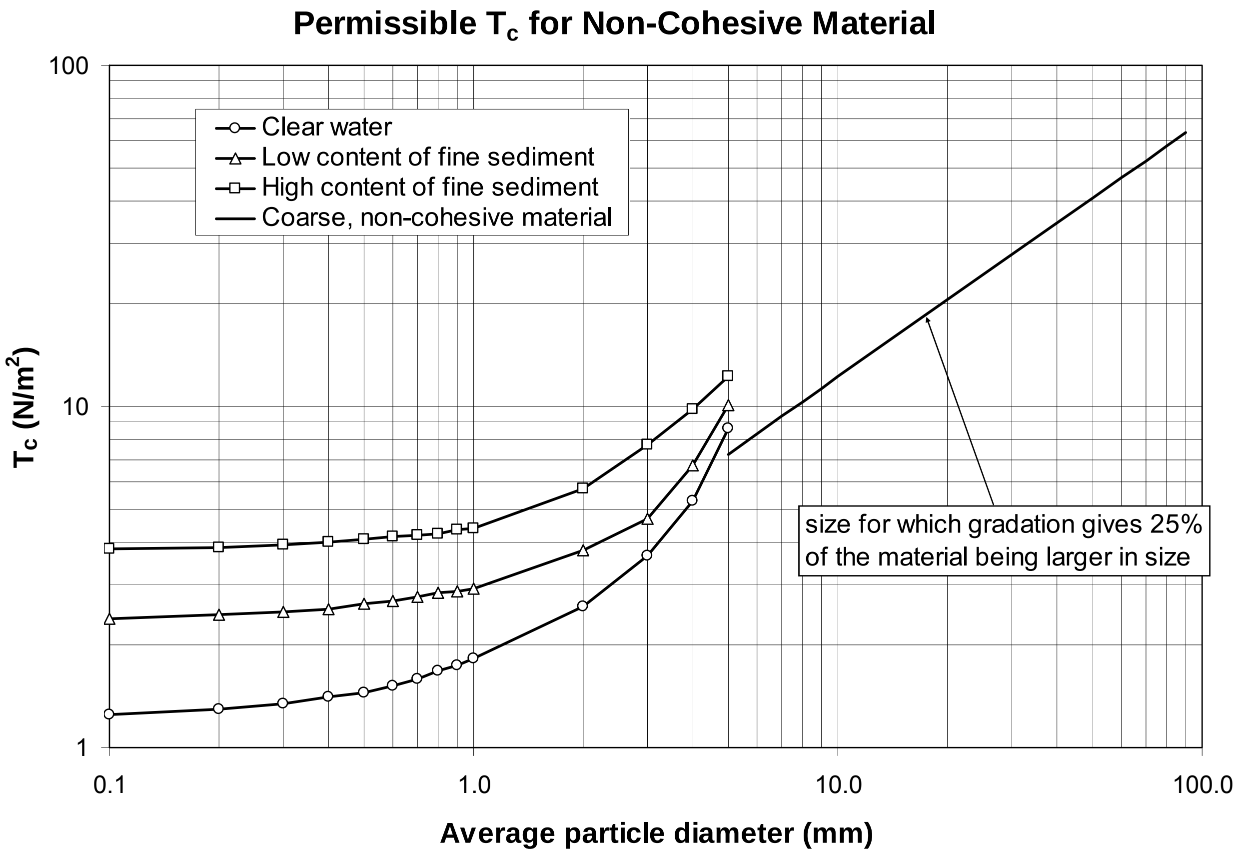
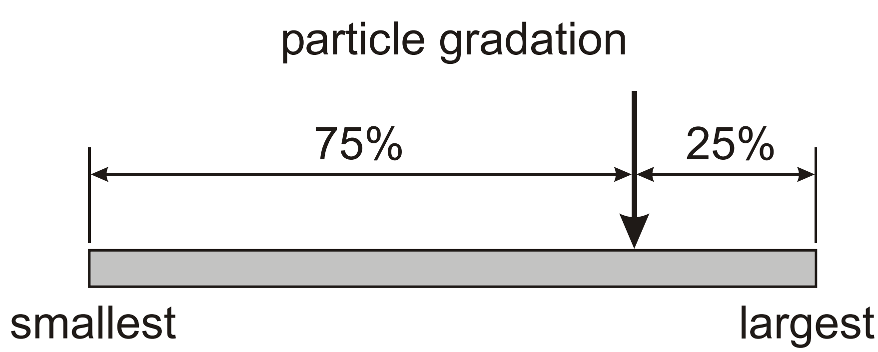


Figure 5. Permissible value of critical shear stress, T_c, in N/m^2, for non-cohesive earthen material (adapted from USBR Hyd Lab Report Hyd-352).

- The three curves at the left side of Fig. 5 are for the average particle diameter
- The straight line at the upper right of Fig. 5 is not for the "average particle diameter," but for the particle size at which 25% of the material is larger in size
- This implies that a gradation (sieve) analysis has been performed on the earthen material



- The three curves at the left side of Fig. 5 ($d \leq 5$ mm) can be approximated as follows:

  Clear water:

$$T_c = 0.0759\,d^3 - 0.269\,d^2 + 0.947\,d + 1.08 \tag{25}$$

  Low sediment:

$$T_c = 0.0756\,d^3 - 0.241d^2 + 0.872\,d + 2.26 \tag{26}$$

  High sediment:

$$T_c = -0.0321d^3 + 0.458\,d^2 + 0.190\,d + 3.83 \tag{27}$$

  where T_c is in N/m^2; and d is in mm

- The portion of Fig 5. corresponding to "coarse material" ($d > 5$ mm) is approximated as:

  Coarse material:

$$T_c = 2.17\,d^{0.75} \tag{28}$$

- Equations 25-28 are for diameter, d, in mm; and T_c in N/m^2
- Equations 25-28 give T_c values within $\pm$1% of the USBR-published data
- Note that Eq. 28 is exponential, which is required for a straight-line plot with log-log scales

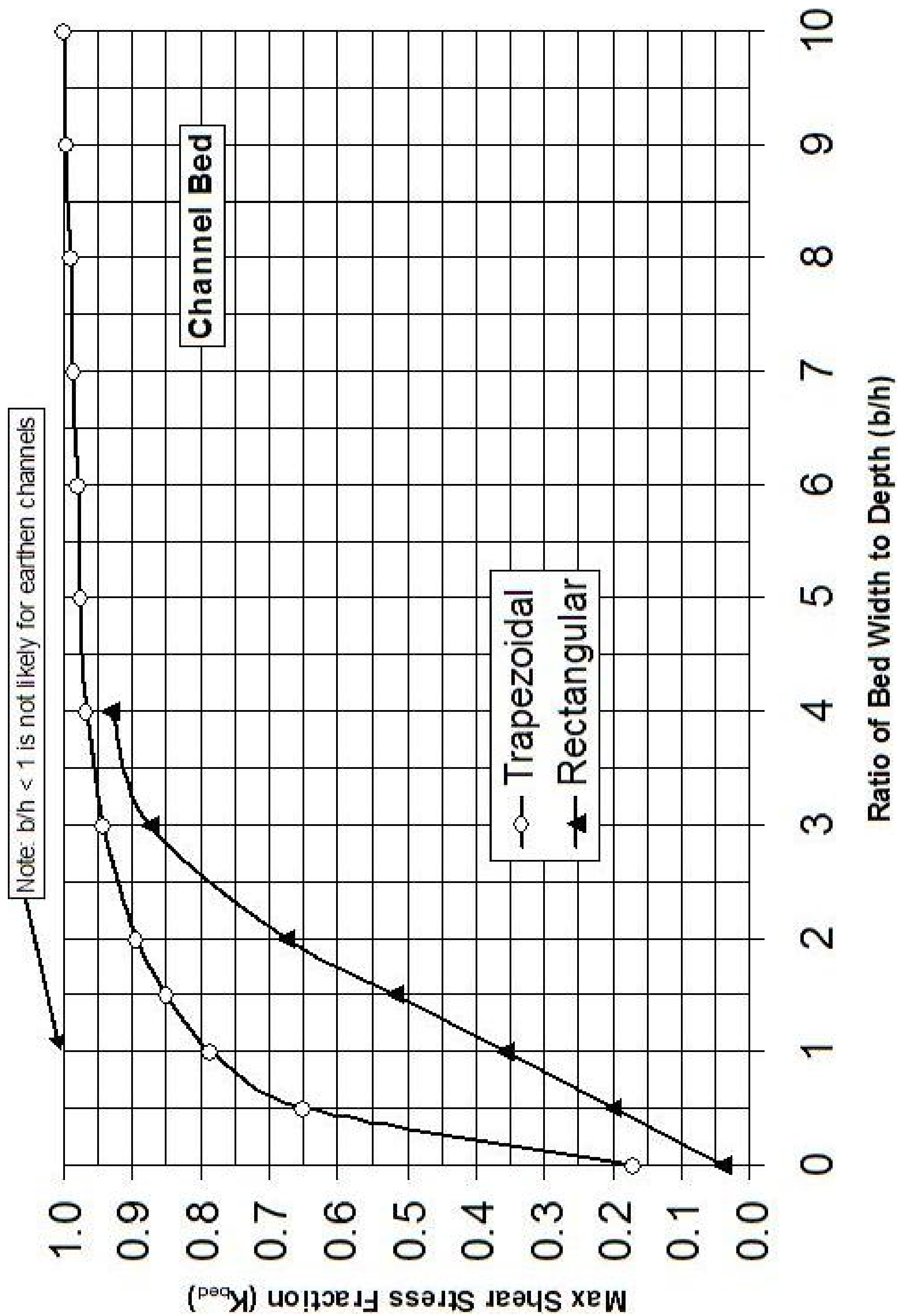


Figure 6. K_{bed} values as a function of the b/h ratio.

Notes: This figure was made using data from USBR Hydraulic Lab Report Hyd-366. The ordinate values are for maximum shear stress divided by γhS_o, where $\gamma = \rho g$, h is water depth, and S_o is longitudinal bed slope. Both the ordinate & abscissa values are dimensionless.

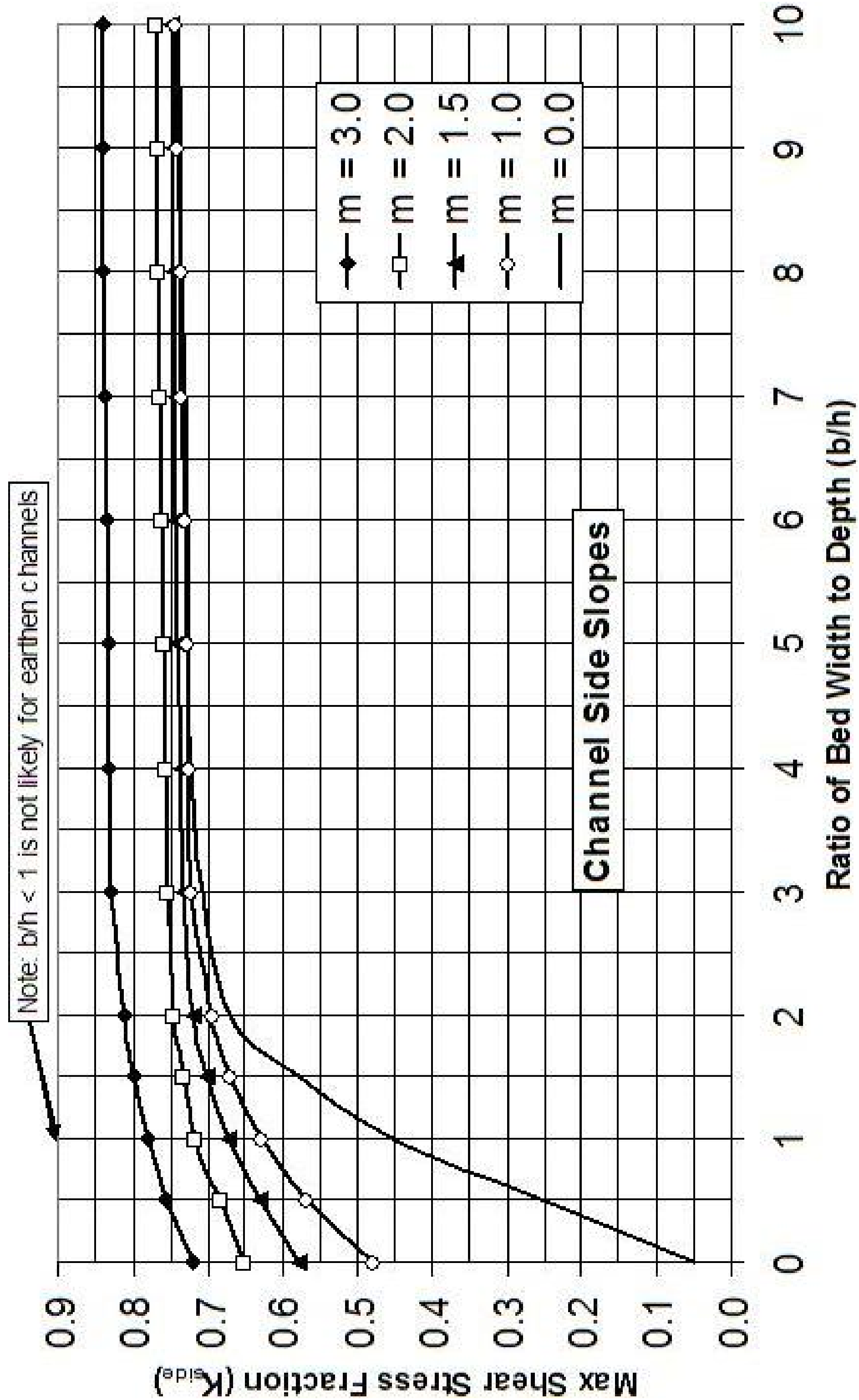


Figure 7. K_{side} **values as a function of the b/h ratio.**

Notes: This figure was made using data from USBR Hydraulic Lab Report Hyd-366. The ordinate values are for maximum shear stress divided by $\gamma h S_o$*, where* $\gamma = \rho g$*, h is water depth, and* S_o *is longitudinal bed slope. Both the ordinate and abscissa values are dimensionless.*

- Regression analysis can be performed on the plotted data for K_{bed} & K_{side}
- This is useful to allow interpolations that can be programmed, instead of reading values off the curves by eye
- The following regression results give sufficient accuracy for the max shear stress fractions:

$$
\begin{aligned}
K_{bed} &\cong 0.792\left(\frac{b}{h}\right)^{0.153} && \text{for } 1 \le b/h \le 4 \\
K_{bed} &\cong 0.00543\left(\frac{b}{h}\right) + 0.947 && \text{for } 4 \le b/h \le 10
\end{aligned}
\tag{29}
$$

for trapezoidal cross sections; and,

$$
K_{side} \cong \frac{AB + C\left(b/h\right)^{D}}{B + \left(b/h\right)^{D}} \tag{30}
$$

where,

$$
A = -0.0592\left(m\right)^{2} + 0.347\left(m\right) + 0.193 \tag{31}
$$

$$
B = 2.30 - 1.56\,e^{-0.000311\left(m\right)^{7.23}} \tag{32}
$$

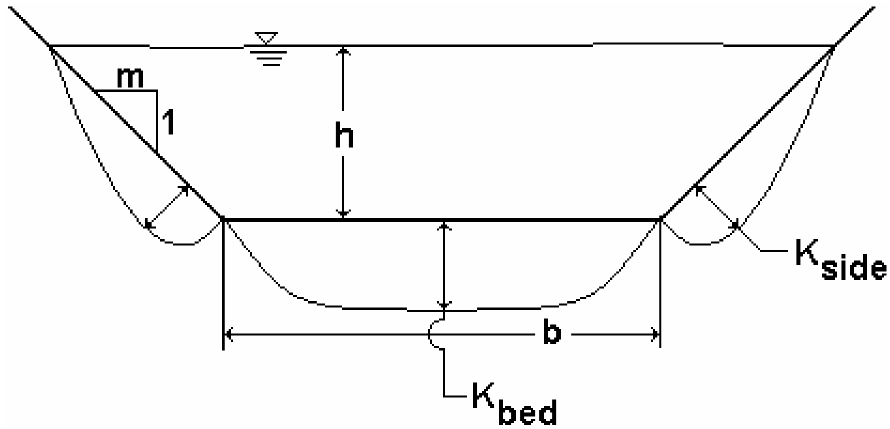
$$
C = 1.14 - 0.395\,e^{-0.00143\left(m\right)^{5.63}} \tag{33}
$$

$$
D = 1.58 - 3.06\,e^{-35.2\left(m\right)^{-3.29}} \tag{34}
$$

for $1 \le m \le 3$, and where e is the base of natural logarithms

- Equations 29 give K_{bed} to within ±1% of the values from the USBR data for $1 \le b/h \le 10$
- Equations 30-34 give K_{side} to within ±2% of the values from the USBR data for $1 \le m \le 3$ (where the graphed values for m = 3 are extrapolated from the lower m values)

- The figure below is adapted from the USBR, defining the inverse side slope, and bed width
- The figure below also indicates locations of measured maximum tractive force on the side slopes, K_{side}, and the bed, K_{bed}
- These latter two are proportional to the ordinate values of the above two graphs (Figs. 6 & 7)



Step 4

- Calculate the maximum depth based on K_{bed}:

$$h_{max} = \frac{KT_c}{K_{bed}\gamma S_o} \tag{35}$$

- Recall that K is a function of ϕ and θ (Eq. 13)
- Calculate the maximum depth based on K_{side}:

$$h_{max} = \frac{KT_c}{K_{side}\gamma S_o} \tag{36}$$

where γ is 62.4 lbs/ft^3, or 9,810 N/m^3

- Note that K, K_{bed}, K_{side}, and S_o are all dimensionless; and T_c/γ gives units of length (ft or m), which is what is expected for h
- The smaller of the two h_{max} values from the above equations is applied to the design (i.e. the "worst case" scenario)

Step 5

- Take the smaller of the two depth, h, values from Eqs. 35 & 36
- Use the Manning or Chezy equations to calculate the flow rate
- If the flow rate is sufficiently close to the desired maximum discharge value, the design process is finished
- If the flow rate is not the desired value, change the side slope, m, and or bed width, b, checking the m and b/h limits you may have set initially
- Return to Step 3 and repeat calculations
- There are other ways to attack the problem, but it's almost always iterative

- For a "very wide" earthen channel, the channel sides become negligible and the critical tractive force on the channel bed can be taken as:

$$T_c \cong \gamma h S_o \tag{37}$$

- Then, if S_o is known, h can be calculated

IX. Definition of Symbols

a	effective particle area (m^2 or ft^2)
b	channel base width (m or ft)
h	depth of water (m or ft)
h_{max}	maximum depth of water (m or ft)
K	tractive force ratio (function of ϕ and θ)
K_{bed}	maximum shear stress fraction (bed)
K_{side}	maximum shear stress fraction (side slopes)
m	inverse side slope
S_o	longitudinal bed slope
T_{bed}	shear stress exerted on a bed soil particle (N/m^2 or lbs/ft^2)
T_c	critical shear stress (N/m^2 or lbs/ft^2)
T_{side}	shear stress exerted on a side slope soil particle (N/m^2 or lbs/ft^2)
W_s	weight of a soil particle (N or lbs)
ϕ	inverse side slope angle
γ	weight of water per unit volume (N/m^3 or lbs/ft^3)
θ	angle of repose (wet soil material)

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Lecture 18

Sample Earthen Channel Designs

I. Example "A": Design Procedure for an Earthen Canal

- Design an earthen canal section in an alluvial soil such that the wetted boundaries do not become eroded
- The canal will follow the natural terrain at an estimated S_o = 0.000275 ft/ft with a preliminary design side slope of 1.5:1.0 (h:v)
- The bed material has been determined to be a non-cohesive "coarse light sand" with an average particle diameter of 10 mm, 25% of which is larger than 15 mm
- Thus, 15 mm will be used to determine T_c in Fig. 5
- Tests have shown that the angle of repose for the bed material is approximately 34°, measured from the horizontal
- For the Manning equation, use a roughness value of 0.030

- The design discharge (Q_{max}) is 650 cfs
- The source of water is such that there will be a low content of fine sediment
- The canal can be assumed to be straight, even though there will be bends at several locations

- Design the section using a trapezoidal shape with a bed width to depth ratio, b/h, of between 1.0 and 5.0
- The design should also be such that the canal bed and sides do not erode
- Adjust the side slope if necessary, but keep it within the range 0.5:1 to 2.0:1 (h:v)
- Note that $\phi < \theta$ must be true to allow for a stable side slope

1. Design the canal using the tractive force method
2. Compare the results for the case in which it is assumed that the channel is very wide (i.e. critical tractive "force" = $\gamma h S_o$)
3. Compare with results from the Kennedy formula
4. Compare with results from the Lacey method
5. Compare with results from the maximum velocity method using Table 2, and again using Table 3

Solution to Example "A" Design Problem:

1. Tractive Force Method

Critical Tractive Force

- The critical tractive force can be estimated from Figure 5 (see above)
- The material is non-cohesive, and 25% of the particles are larger than 15 mm
- This gives $T_c \approx 16.3$ N/m^2 (0.34 lbs/ft^2) for the 15 mm abscissa value

Angle of Repose

- The angle of repose, θ, is given as 34°
- Then, the ratio of T_{side} to T_{bed} is:

$$K = \frac{T_{side}}{T_{bed}} = \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}} = \sqrt{1 - 3.2\sin^2\phi} \tag{1}$$

- Design requirements for this example call for a side slope between 0.5 & 2.0
- Actually, the range is restricted to 1.5 to 2.0 because the preliminary design side slope of 1.5:1 corresponds to an angle ϕ = 33.7°
- This is less than the angle of repose, θ = 34°, but it is very close
- Make a table of K values:

Table 1. K values for different side slopes

m	ϕ	K
1.5	33.7°	0.122
1.6	32.0°	0.318
1.7	30.5°	0.419
1.8	29.1°	0.493
1.9	27.8°	0.551
2.0	26.6°	0.599

Maximum Shear Stress Fractions

- From Figure 7 (see above), the maximum shear stress fraction for sides, K_{side}, is approximately equal to 0.74 in the range 1.0 < (b/h) < 5.0, and for side slopes from 1.5 to 2.0
- Take K_{side} as a constant for this problem: $K_{side} \approx 0.74$
- The maximum shear stress fraction on the channel bed, K_{bed}, will fall on the curve for trapezoidal sections, and will vary from 0.79 to 0.97 within the acceptable range1.0 < (b/h) < 5.0
- Make a table of K_{bed} values according to b/h ratio (from Figure 6):

Table 2. Values of K_{bed} for different b/h ratios

b/h	K_{bed}
1	0.79
2	0.90
3	0.94
4	0.96
5	0.97

Manning Equation

- The Manning roughness, n, is given as 0.030. The longitudinal bed slope is given as 0.000275 ft/ft
- The side slope can be any value between 1.5 and 2.0
- Construct a table of depths (normal depths) for the maximum design discharge of 650 cfs, then make another table showing bed width to depth ratios:

Table 3. Flow depths (ft) for 650 cfs

b (ft)	inverse side slope, m					
	1.5	**1.6**	**1.7**	**1.8**	**1.9**	**2.0**
10	10.11	9.92	9.74	9.58	9.43	9.29
15	9.01	8.87	8.74	8.62	8.51	8.41
20	8.10	8.00	7.91	7.83	7.75	7.67
25	7.36	7.29	7.23	7.16	7.10	7.04
30	6.75	6.70	6.65	6.60	6.56	6.52
35	6.25	6.21	6.17	6.14	6.10	6.07

Table 4. Bed width to depth ratios for 650 cfs

b (ft)	inverse side slope, m					
	1.5	**1.6**	**1.7**	**1.8**	**1.9**	**2.0**
10	**0.989**	1.008	1.027	1.044	1.060	1.076
15	1.665	1.691	1.716	1.740	1.763	1.784
20	2.469	2.743	2.766	2.554	2.581	2.608
25	3.397	3.429	3.458	3.492	3.521	3.551
30	4.444	4.478	4.511	4.545	4.573	4.601
35	**5.600**	**5.636**	**5.672**	**5.700**	**5.738**	**5.766**

Note: bold values fall outside the acceptable range of $1.0 < (b/h) < 5.0$.

- In Table 4 it is seen that the bed width must be less than 35 ft, otherwise the required b/h ratio will be greater than 5.0
- Also, it can be seen that bed widths less than 10 ft will have problems because the b = 10 and m = 1.5 combination gives b/h < 1.0

Allowable Depth for Tractive Force: Side Slopes

- Taking K_{side} from Figure 7, the maximum allowable depth according to the tractive force method for side slopes is:

$$h_{max} = \frac{T_c K}{K_{side} \gamma S_o} = \frac{0.34\sqrt{1-3.2\sin^2\phi}}{(0.74)(62.4)(0.000275)} = 26.8\sqrt{1-3.2\sin^2\phi} \qquad (2)$$

where the unit weight of water is taken as γ = 62.4 lbs/ft^3

- Make a table of h_{max} values (essentially independent of b/h) for different values of the angle ϕ:

Table 5. Maximum depth values for different side slopes

m	ϕ	h_{max}
1.5	33.7°	3.3
1.6	32.0°	8.5
1.7	30.5°	11.2
1.8	29.1°	13.2
1.9	27.8°	14.8
2.0	26.6°	16.0

- Now the design possibilities will narrow further
- Compare depths calculated by the Manning equation for 650 cfs with the maximum allowable depths by tractive force method, according to side slope traction (combine Tables 3 & 5):

Table 6. Ratio of Manning depths to h_{max} for 650 cfs

b (ft)	**inverse side slope, m**					
	1.5	**1.6**	**1.7**	**1.8**	**1.9**	**2.0**
10	**3.06**	**1.17**	0.87	0.73	0.64	0.58
15	**2.73**	**1.04**	0.78	0.65	0.58	0.53
20	**2.45**	0.94	0.71	0.59	0.52	0.48
25	**2.23**	0.86	0.65	0.54	0.48	0.44
30	**2.05**	0.79	0.59	0.50	0.44	0.41

Note: bold values are out of range (uniform flow depths are too high).

- From the above table it is seen that the side slope must now be between 1.6 and 2.0, otherwise the required flow depths will exceed the limit imposed by the tractive force method for side slopes

Allowable Depth for Tractive Force: Channel Bed

Again, taking K_{bed} from Figure 6, the maximum allowable depth according to the tractive force method for the channel bed is:

$$h_{max} = \frac{KT_c}{K_{bed}\gamma S_o} = \frac{0.34\sqrt{1-3.2\sin^2\phi}}{K_{bed}(62.4)(0.000275)} = \frac{19.8}{K_{bed}}\sqrt{1-3.2\sin^2\phi} \tag{3}$$

where K_{bed} is a function of the b/h ratio

Table 7. Maximum allowable depths (ft) according to bed criterion

b (ft)	side slope, m					
	1.5	**1.6**	**1.7**	**1.8**	**1.9**	**2.0**
10	**n/a**	**8.00**	10.50	12.40	13.80	15.00
15	**n/a**	**7.20**	9.70	11.20	12.50	13.60
20	**n/a**	**6.80**	8.90	10.60	11.90	12.90
25	**n/a**	**6.60**	8.70	10.30	11.50	12.50
30	**n/a**	6.50	8.60	10.10	11.30	12.30

Note: bold depth values are not acceptable.

- Comparing with Table 3 from Manning equation, most of the depths required for 650 cfs at m = 1.6 are higher than allowed by tractive force method (bed criterion)
- However, the combination of m= 1.6 and b = 30 falls within acceptable limits

Final Tractive Force Design

- The allowable depths according to the bed criterion are all less than the allowable depths (for the same m values) from the side slope criterion
- Therefore, use the bed criterion as the basis for the design
- The permissible values for m = 2.0 are all much higher than those from the Manning equation
- Permissible values for m = 1.9, 1.8, and 1.7 are also higher than those from the Manning equation
- For m = 1.6, only the b = 30 ft bed width is within limits (less than that required by the Manning equation for 650 cfs)

- Make a judgment decision based on economics, convenience of construction, area occupied by the channel (channel width), safety considerations, and other factors

- Reject the 30-ft bed width; it will be wider than necessary
- Due to lack of other information, recommend the 20-ft bed width and 1.7 side slope option
- At this point it would not be useful to consider other intermediate values of b and m
- For b = 20 ft and m = 1.7, the depth will be about 7.91 ft (allowable is 8.9 ft from Table 4), and the mean flow velocity at 650 cfs is: V = 2.5 fps

2. Tractive Force Method: Assume "very wide channel"

In this case, K is equal to 1.0, and,

$$h_{max} = \frac{T_c}{\gamma S} = \frac{0.34}{(62.4)(0.000275)} = 19.8 \text{ ft} \tag{4}$$

- Thus, the depth would have to be less than 19.8 ft
- This is a much less conservative value than that obtained above (OK)

3. Kennedy Formula

- For a "coarse light sand", C = 0.92
- For fine sediment in water, m = 0.64. Then,

$$V_o = CD^m = 0.92(7.91)^{0.64} = 3.5 \text{ fps} \tag{5}$$

- This is more than the tractive force design velocity of 2.5 fps. (OK)

4. Lacey Method

- Mean diameter of bed material, d_m = 10 mm
- The hydraulic radius at 650 cfs from the tractive force design is R = A/W_p = 265/51 = 5.2 ft
- Then,

$$f = 1.76\sqrt{d_m} = 5.57 \tag{6}$$

$$V = 1.17\sqrt{fR} = 1.17\sqrt{(5.57)(5.2)} = 6.3 \text{ fps} \tag{7}$$

- This is also more than the tractive force design velocity of 2.5 fps. (OK)

5. Maximum Velocity Method

- From Table 1, the maximum permissible velocity for "coarse light sand", say "fine gravel", is 5.0 fps for water transporting colloidal silt
- For the same material and clear water, the maximum is 2.5 fps -- a large difference based on a fairly subjective determination
- Also, from the given information of this example problem, an exact match the materials listed in Table 1 is not possible
- From Table 2, for "coarse sand", the maximum permissible velocity is 1.8 fps

II. Example “B”: Design Procedure for an Earthen Canal

- Design an earthen canal section using the tractive force method such that the bed and side slopes are stable
- The design flow rate is 90 m^3/s and the water is clear
- The earthen material is non-cohesive fine sand with average particle size of 0.5 mm and an angle of repose of 27°
- Assume that the inverse side slope is fixed at m = 3.0 for this design. Use a Manning’s n of 0.02
- Determine the minimum bed width, b
- Determine the maximum longitudinal bed slope, S_o
- Recommend a freeboard value for the design discharge
- Make a sketch to scale of the channel cross section and the water surface at the design discharge

Solution to Example “B” Design Problem:

1. Convert to English units

- 90 m^3/s = **3,178 cfs**

2. Check angle of repose

- θ = 27°, or 0.471 rad (angle of repose)
- $\phi = \tan^{-1}(1/m) = \tan^{-1}(1/3.0) = 0.322$ rad (18.4°)
- $\phi < \theta$, so the side slopes are potentially stable

3. Tractive force ratio, K

$$K = \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}} = \sqrt{1 - \frac{\sin^2(0.322)}{\sin^2(0.471)}} = 0.718$$

4. Critical shear stress, T_c

- From Figure 5, for non-cohesive material, with an average particle size of 0.5 mm and clear water, $\mathbf{T_c \approx 0.03\ lb/ft^2}$

5. Max shear stress fractions

- Arbitrarily limit b/h to a minimum of 1 and maximum of 9 (note that b/h < 1 is usually not reasonable or feasible for an earthen channel)
- This is a very wide range of b/h values anyway
- The table below shows K_{bed} and K_{side} for $1 \le b/h \le 9$
- The K_{side} values are extrapolated from the curves for $1 \le m \le 2$ because in our case, m = 3

b/h	K_{bed}	K_{side}
1	0.79	0.78
2	0.90	0.81
3	0.94	0.83
4	0.97	0.83
5	0.98	0.83
6	0.98	0.84
7	0.99	0.84
8	0.99	0.84
9	1.00	0.84

6. Uniform flow depths

- Calculate uniform flow depths for various b and S_o values, using n = 0.02 and the Manning equation
- Note that b will have to be fairly large because it is an earthen channel and the design discharge is rather large itself
- The uniform flow calculations are done in a computer program and the results are given in the table below

Uniform flow depths (in ft) for varying b & S_o (Manning; n = 0.02)

b (ft)	Longitudinal bed slope, S_o									
	0.00001	*0.00002*	*0.00003*	*0.00004*	*0.00005*	*0.00006*	*0.00007*	*0.00008*	*0.00009*	*0.00010*
30	23.750	20.379	18.609	17.437	16.573	15.894	15.340	14.874	14.473	14.122
40	22.479	19.161	17.426	16.278	15.434	14.773	14.233	13.780	13.390	13.050
50	21.317	18.063	16.367	15.250	14.429	13.787	13.264	12.825	12.448	12.119
60	20.256	17.074	15.423	14.337	13.541	12.920	12.415	11.991	11.628	11.312
70	19.289	16.184	14.579	13.527	12.757	12.158	11.671	11.263	10.913	10.609
80	18.406	15.382	13.825	12.806	12.063	11.485	11.016	10.623	10.287	9.995
90	17.602	14.659	13.149	12.164	11.446	10.889	10.437	10.059	9.737	9.456
100	16.867	14.005	12.542	11.589	10.896	10.359	9.923	9.560	9.250	8.980
110	16.194	13.413	11.994	11.073	10.404	9.885	9.465	9.115	8.816	8.557
120	15.578	12.874	11.499	10.607	9.960	9.459	9.054	8.717	8.429	8.179

7. Base width to depth ratio, b/h

- Calculate b/h values for each of the uniform-flow depths in the above table
- Values in the following table are shown in **bold** where greater than the specified maximum of 9

Bed width to depth ratios (b/h) for varying b & S_o

b (ft)	Longitudinal bed slope, S_o									
	0.00001	*0.00002*	*0.00003*	*0.00004*	*0.00005*	*0.00006*	*0.00007*	*0.00008*	*0.00009*	*0.00010*
30	1.263	1.472	1.612	1.720	1.810	1.888	1.956	2.017	2.073	2.124
40	1.779	2.088	2.295	2.457	2.592	2.708	2.810	2.903	2.987	3.065
50	2.346	2.768	3.055	3.279	3.465	3.627	3.770	3.899	4.017	4.126
60	2.962	3.514	3.890	4.185	4.431	4.644	4.833	5.004	5.160	5.304
70	3.629	4.325	4.801	5.175	5.487	5.758	5.998	6.215	6.414	6.598
80	4.346	5.201	5.787	6.247	6.632	6.966	7.262	7.531	7.777	8.004
90	5.113	6.140	6.845	7.399	7.863	8.265	8.623	8.947	**9.243**	**9.518**
100	5.929	7.140	7.973	8.629	**9.178**	**9.653**	**10.078**	**10.460**	**10.811**	**11.136**
110	6.793	8.201	**9.171**	**9.934**	**10.573**	**11.128**	**11.622**	**12.068**	**12.477**	**12.855**
120	7.703	**9.321**	**10.436**	**11.313**	**12.048**	**12.686**	**13.254**	**13.766**	**14.237**	**14.672**

- The following two tables are interpolated K_{bed} and K_{side} values
- Values shown in **bold** (following two tables) are greater than 1.0 and considered to be infeasible

Interpolated K_{bed} values

b (ft)	Longitudinal bed slope, S_o									
	0.00001	*0.00002*	*0.00003*	*0.00004*	*0.00005*	*0.00006*	*0.00007*	*0.00008*	*0.00009*	*0.00010*
30	0.821	0.840	0.852	0.861	0.867	0.873	0.878	0.882	0.885	0.889
40	0.865	0.886	0.899	0.909	0.916	0.922	0.928	0.932	0.936	0.940
50	0.902	0.926	0.940	0.950	0.958	0.965	0.970	0.975	0.969	0.969
60	0.935	0.960	0.975	0.970	0.971	0.972	0.973	0.974	0.975	0.976
70	0.965	0.970	0.973	0.975	0.977	0.978	0.980	0.981	0.982	0.983
80	0.971	0.975	0.978	0.981	0.983	0.985	0.986	0.988	0.989	0.990
90	0.975	0.980	0.984	0.987	0.990	0.992	0.994	0.996	0.997	0.999
100	0.979	0.986	0.990	0.994	0.997	0.999	**1.002**	**1.004**	**1.006**	**1.007**
110	0.984	0.992	0.997	**1.001**	**1.004**	**1.007**	**1.010**	**1.013**	**1.015**	**1.017**
120	0.989	0.998	**1.004**	**1.008**	**1.012**	**1.016**	**1.019**	**1.022**	**1.024**	**1.027**

Interpolated K_{side} values

b (ft)	Longitudinal bed slope, S_o									
	0.00001	*0.00002*	*0.00003*	*0.00004*	*0.00005*	*0.00006*	*0.00007*	*0.00008*	*0.00009*	*0.00010*
30	0.775	0.775	0.776	0.776	0.777	0.777	0.777	0.777	0.777	0.778
40	0.777	0.777	0.778	0.778	0.779	0.779	0.779	0.779	0.780	0.780
50	0.778	0.779	0.780	0.780	0.780	0.781	0.781	0.781	0.828	0.828
60	0.779	0.780	0.781	0.828	0.828	0.828	0.828	0.828	0.828	0.828
70	0.781	0.828	0.828	0.828	0.828	0.828	0.828	0.828	0.828	0.828
80	0.828	0.828	0.828	0.828	0.828	0.828	0.828	0.828	0.828	0.828
90	0.828	0.828	0.828	0.828	0.828	0.828	0.828	0.828	0.828	0.828
100	0.828	0.828	0.828	0.828	0.828	0.828	0.829	0.829	0.829	0.829
110	0.828	0.828	0.828	0.829	0.829	0.829	0.829	0.829	0.829	0.829
120	0.828	0.828	0.829	0.829	0.829	0.829	0.829	0.829	0.829	0.829

8. Ratio of max depth to uniform flow depth

- Using the interpolated max shear stress fractions, apply the following equations to calculate maximum water depth:

$$h_{max} = \frac{KT_c}{K_{bed}\gamma S_o}$$

and,

$$h_{max} = \frac{KT_c}{K_{side}\gamma S_o}$$

where the smaller of the two values is taken for the design

- **Bold** values in the following two tables have a uniform flow depth which exceeds the calculated maximum depth, and are removed from consideration

Ratio of max depth (based on K_{bed}) to uniform-flow depth

b (ft)	Longitudinal bed slope, S_o									
	0.00001	*0.00002*	*0.00003*	*0.00004*	*0.00005*	*0.00006*	*0.00007*	*0.00008*	*0.00009*	*0.00010*
30	1.771	1.008	**0.726**	**0.575**	**0.480**	**0.415**	**0.366**	**0.329**	**0.299**	**0.275**
40	1.775	1.016	**0.734**	**0.583**	**0.488**	**0.422**	**0.373**	**0.336**	**0.306**	**0.281**
50	1.795	1.032	**0.748**	**0.596**	**0.500**	**0.433**	**0.383**	**0.345**	**0.318**	**0.294**
60	1.822	1.053	**0.765**	**0.621**	**0.525**	**0.458**	**0.408**	**0.369**	**0.338**	**0.313**
70	1.855	1.099	**0.811**	**0.654**	**0.554**	**0.484**	**0.431**	**0.391**	**0.358**	**0.331**
80	1.932	1.151	**0.851**	**0.687**	**0.582**	**0.509**	**0.454**	**0.411**	**0.377**	**0.349**
90	2.012	1.201	**0.889**	**0.719**	**0.609**	**0.533**	**0.475**	**0.431**	**0.395**	**0.366**
100	2.090	1.250	**0.926**	**0.749**	**0.636**	**0.556**	**0.496**	**0.450**	**0.412**	**0.382**
110	2.167	1.298	**0.962**	**0.779**	**0.661**	**0.578**	**0.516**	**0.468**	**0.429**	**0.397**
120	2.241	1.344	**0.997**	**0.807**	**0.685**	**0.599**	**0.535**	**0.484**	**0.444**	**0.411**

Ratio of max depth (based on K_{side}) to uniform-flow depth

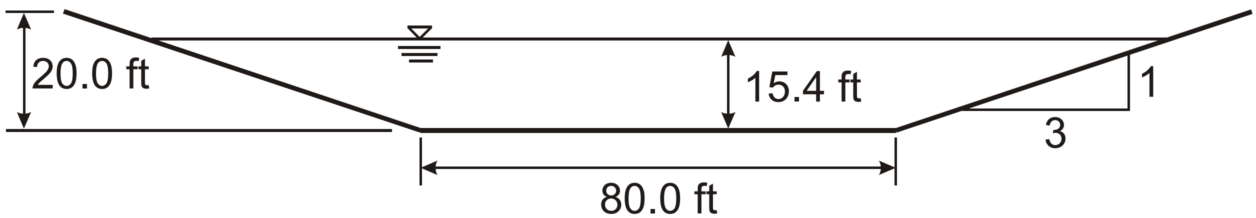
b (ft)	Longitudinal bed slope, S_o									
	0.00001	*0.00002*	*0.00003*	*0.00004*	*0.00005*	*0.00006*	*0.00007*	*0.00008*	*0.00009*	*0.00010*
30	1.876	1.092	**0.797**	**0.637**	**0.536**	**0.466**	**0.414**	**0.373**	**0.341**	**0.314**
40	1.977	1.159	**0.849**	**0.681**	**0.574**	**0.500**	**0.445**	**0.402**	**0.367**	**0.339**
50	2.081	1.226	**0.902**	**0.725**	**0.613**	**0.535**	**0.476**	**0.431**	**0.372**	**0.344**
60	2.186	1.295	**0.955**	**0.727**	**0.615**	**0.537**	**0.479**	**0.434**	**0.398**	**0.368**
70	2.292	1.287	**0.953**	**0.770**	**0.653**	**0.571**	**0.510**	**0.462**	**0.424**	**0.393**
80	2.264	1.354	1.005	**0.813**	**0.691**	**0.605**	**0.540**	**0.490**	**0.450**	**0.417**
90	2.367	1.421	1.056	**0.856**	**0.728**	**0.638**	**0.570**	**0.518**	**0.475**	**0.441**
100	2.470	1.488	1.107	**0.899**	**0.765**	**0.670**	**0.600**	**0.545**	**0.500**	**0.464**
110	2.573	1.553	1.158	**0.941**	**0.801**	**0.702**	**0.629**	**0.571**	**0.525**	**0.487**
120	2.675	1.618	1.208	**0.982**	**0.837**	**0.734**	**0.657**	**0.597**	**0.549**	**0.509**

- It is clear that the bed slope must be less than S_o = 0.00003 ft/ft
- This is a very small slope
- But there is a large range of possible bed widths
- The lower b values will result in great depths, and the higher b values will take up a wide “swath” of land. Most feasible will probably be a compromise between these extremes, perhaps 60 < b < 90 ft.
- To complete one design possibility, recommend **b = 80 ft & S_o = 0.00002 ft/ft**
- This corresponds to a uniform flow depth of **h = 15.4 ft** (see above)

9. Freeboard

- Using the freeboard curves from the previous lecture, with Q = 3,178 cfs, the height of the bank above the water surface should be about 4.5 ft
- Then, the depth of the channel is 15.4 + 4.5 = 19.9 ft. **Round up to 20 ft.**

10. Cross-section sketch



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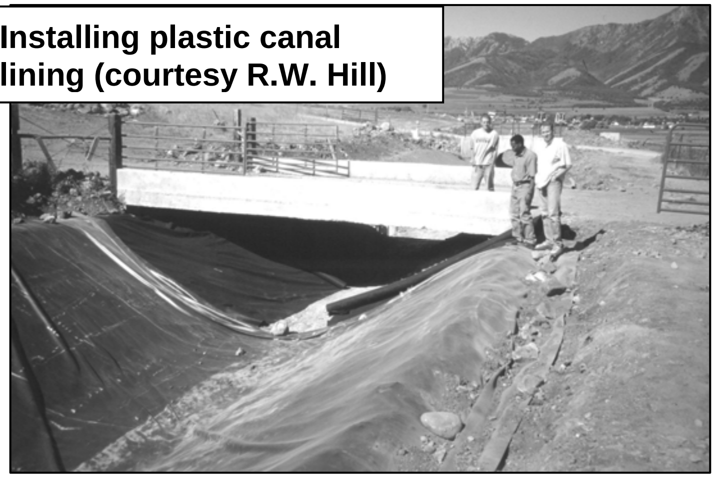
Lecture 19

Canal Linings

I. Reasons for Canal Lining



Installing plastic canal lining (courtesy R.W. Hill)

1. To save water (reduce seepage)
2. To stabilize channel bed and banks (reduce erosion)
3. To avoid piping through and under channel banks
4. To decrease hydraulic roughness (flow resistance)
5. To promote movement, rather than deposition, of sediments
6. To avoid waterlogging of adjacent land
7. To control weed growth
8. To decrease maintenance costs and facilitate cleaning
9. To reduce excavation costs (when extant material is unsuitable)
10. To reduce movement of contaminated groundwater plumes

- The most common and (usually) most important reason is to reduce seepage losses (and this may be for a variety of reasons)
- The assumption that lining will solve seepage problems is often unfounded, simply because poor maintenance practices (especially with concrete linings) will allow cracking and panel failures, and tears and punctures in flexible membranes
- Seepage losses from canals can be beneficial in that it helps recharge aquifers and makes water accessible to possibly larger areas through groundwater pumping. The extent of aquifers is more continuous than that of canals and canal turnouts. But, pumping ($energy$) is usually necessary with groundwater, unless perhaps you are downhill and there is an artesian condition (this is the case in some places).
- “Administrative losses” and over-deliveries can add up to a greater volume of water than seepage in many cases (that means that canal lining is not always the most promising approach to saving water in the distribution system)

- Sometimes, only the bottom of a canal is lined when most of the seepage has been found to be in the vertical direction
- It may be advisable to perform soil compaction testing under concrete linings to determine if steps need to be taken to avoid subsequent settlement of the canal
- Lining to decrease maintenance costs can backfire (costs may actually increase)
- Concrete pipe is an alternative to lined canals, but for large capacities the pipes tend to cost more
- Many billions of dollars have been spent world-wide during the past several decades to line thousands of miles of canals

II. Some Types of Lining and Costs

Type	Typical Costs

1. Soil

   - Lime
   - Bentonite clay
   - “High-swell” Bentonite & coarse clay or other “bridging material”
   - Geosynthetic clay liner (“Bentomat”)
   - Soil mixed with portland cement
   - Thin compacted earth (6 - 12 inches)
   - Thick compacted earth (12 - 36 inches)

2. Fly Ash .... $3.00/yd^2
3. Masonry (stone, rock, brick)
4. Concrete (portland cement)

   - Nonreinforced concrete .... $5.00/yd^2
   - Reinforced concrete (with steel)
   - Gunite, a.k.a. shotcrete, a.k.a. cement mortar (hand or pneumatically applied; w/o steel reinforcement)................ $12.00/yd^2
   - Gunite, a.k.a. shotcrete, a.k.a. cement mortar (hand or pneumatically applied; w/ steel reinforcement).................. $15.00/yd^2

5. Plastic

   - Polyvinyl Chloride (PVC) .... $5.00/yd^2
   - Oil Resistant PVC
   - Chlorinated Polyethylene (PE)
   - Low Density Polyethylene.. $4.00/yd^2
   - High Density Polyethylene... $10.00/yd^2
   - Polyurethane foam with or without coatings

6. Asphalt (bituminous)

   - Sprayed (“blown”) asphalt
   - Asphaltic Concrete .... $4.00/yd^2

7. Synthetic Rubber

   - Butyl Rubber.. $8.00/yd^2
   - Neoprene Rubber
   - Shotcrete over geosynthetic .... $37.00/yd^2
   - Concrete over geosynthetic .... $26.00/yd^2

III. Comments on Different Lining Materials

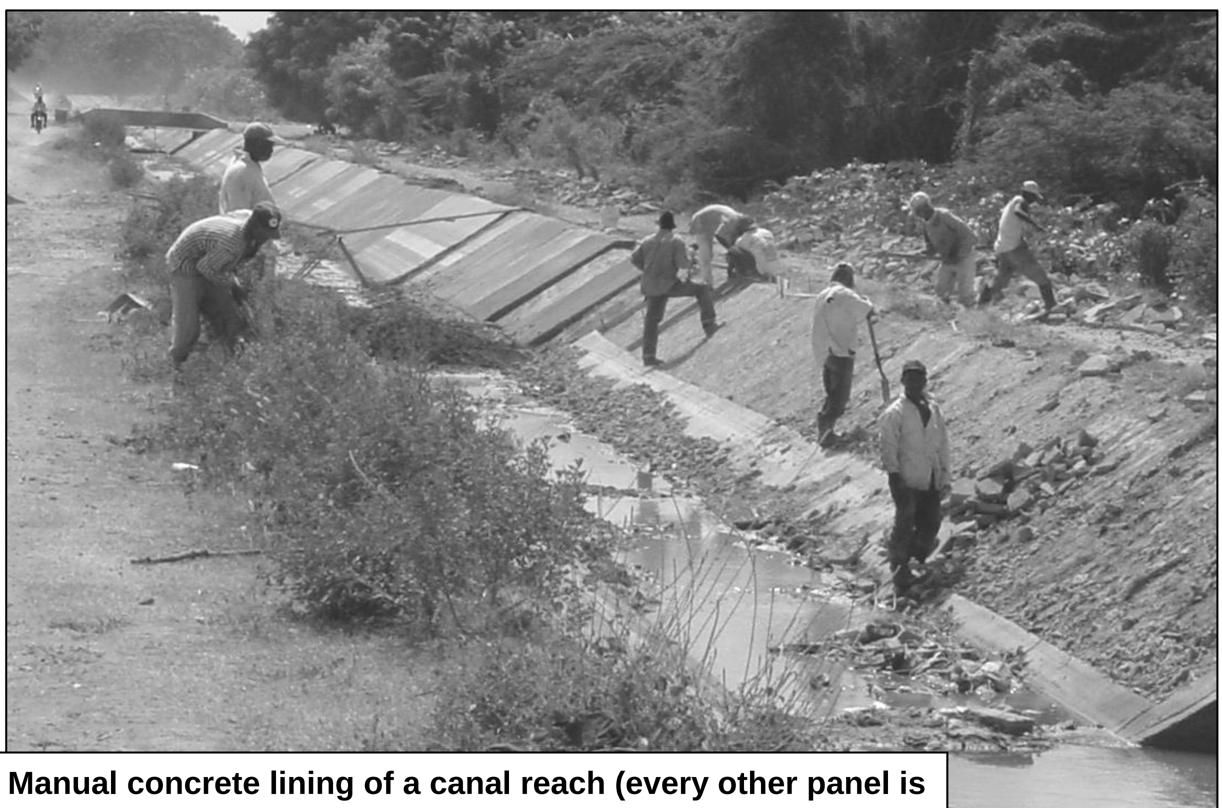
- The USBR has had a long-standing research program on canal lining materials and installation techniques (began in 1946, but essentially discontinued in recent years)
- There are many publications with laboratory and field data, design guidelines and standards, and other relevant information (but you have to dig it all up because it doesn't come in one book)
- Many technical articles can be found in the journals on canal lining materials, construction methods, and experience with different types of linings

Earthen Linings

- Earthen linings usually require significant over-excavation, and transport of suitable material (in large volumes) from another site
- Many earthen linings are 2 - 3 ft thick; "thin" linings are 6 - 12 inches thick
- Clay linings can crack after only a few cycles of wetting and drying, causing increased seepage loss. Bentonite clay swells considerably when wet, but cracks may not completely seal after the canal has been dried, then filled with water again.
- Bentonite is a special kind of clay, usually made of up decomposed volcanic ash, and containing a high percentage of colloidal particles (less than 0.0001 cm in diameter)
- High-swell Bentonite may swell 8 to 12 times in volume when wetted; other types may swell less than 8 times in volume
- Bentonite disperses well when mixed with soft water, but may flocculate (clump up) when mixed with hard water. Flocculation can be avoided by adding one or more dispersing agents (e.g. tetrasodium pyrophosphate, sodium tripolyphosphate, sodium hexametaphosphate). Low-swell Bentonite tends to flocculate easier.
- Repeated drying-wetting cycles can cause loss of lining density, loss of stability, and progressive deterioration of the lining
- Other than Bentonite, clay linings may be of montmorillonite, or montmorillonite - chlorite
- Some clay linings have been treated with lime to stabilize the material. The addition of lime to expansive soils (e.g. Bentonite) improves workability and increases structural strength

Portland Concrete

- Small concrete-lined canals are usually non-reinforced
- Steel reinforcement (rebar or steel mesh) is also not commonly used on large canals anymore unless there are compelling structural reasons
- The elimination of steel reinforcement from concrete canal linings saves about 10 to 15% of the total cost (USBR 1963)
- During the past several years it has become popular to install concrete linings in small canals at the same time as final excavation and finishing, often using a laser to control the alignment and longitudinal slope
- Some “underwater” concrete lining operations have been performed in recent years on full canals (so as not to disrupt delivery operations)
- Careful shaping, or finishing, of the native soil is an important step in the preparation for concrete lining simply because it can greatly reduce the required volume of concrete (significantly lowering the cost)



Manual concrete lining of a canal reach (every other panel is poured first to facilitate formwork)

- Reinforced concrete can contain rebar and or wire mesh. Reinforcement is usually for structural reasons, but also to control cracking of the lining
- Concrete panel joints may have rubber strips to prevent seepage
- Weep holes or flap valves are often installed in cut sections of a concrete-lined canal to relieve back pressures which can cause failure of the lining
- Flap valves may be installed both in side slopes and in the canal bed
- Some concrete-lined canals have (measured) high seepage loss rates, particularly in "fill" sections of canal, and in soils with high permeability (usually sandy soils) -- but, seepage rates are rarely measured; they are "assumed" based on tables in books
- British researchers report that their investigations show that if 0.01% of the area of a concrete canal lining is cracked (0.01% are cracks), the average seepage rate may be the same as that of an unlined canal
- Soil mixed with Portland cement, especially sandy soil, can be an acceptable cost-saving approach to canal lining

IV. Concrete Lining Thickness

- Lining thickness is often chosen in a somewhat arbitrary manner, but based on experience and judgment, and based on the performance of existing linings on other canals
- Thinner linings may crack, but this does not have to be a problem if the cracks are sealed during routine maintenance (not all concrete-lined canals enjoy routine maintenance)
- Concrete lined channels often have high seepage loss rates due to cracks and unsealed panel joints
- Grooves are often specified to control the location and extent of cracking, which can be expected even under the best conditions
- The selection of lining thickness is an economic balance between cost and durability (canals perceived to be very important will have more conservative designs -- municipal supplies, for example)
- The USBR has suggested the following guidelines:

Lining Type	**Thickness (inch)**	**Discharge (cfs)**
Unreinforced concrete	2.00	0-200
	2.50	200-500
	3.00	500-1,500
	3.50	1,500-3,500
	4.00	> 3,500
Asphaltic concrete	2.00	0-200
	3.25	200-1,500
	4.00	> 1,500
Reinforced concrete	3.50	0-500
	4.00	500-2,000
	4.50	> 2,000

Lining Type	Thickness (inch)	Discharge (cfs)
Gunite (shotcrete)	1.25	0-100
	1.50	100-200
	1.75	200-400
	2.00	> 400

Plastic and Rubber

- Plastic linings are also referred to as “geomembranes” or “flexible membrane” linings
- Plastic canal linings have been in use for approximately 40 years
- Plastic and rubber linings are covered with soil, soil and rock, bricks, concrete, or other material for

    1. protection
        - ozone “attack” and UV radiation
        - puncture due to maintenance machinery and animal feet, etc.
        - vandalism

    2. anchoring
        - flotation of the lining (high water table)
        - resist gravity force along side slope
        - wind loading

- Plastic linings are typically 10 to 20 mil (0.010 to 0.020 inches, or 0.25 to 0.5 mm) -- thicker membranes are usually recommendable because of increased durability, and because the overall installation costs only increase by about 15% for a doubling in thickness
- The USBR previously used 10 mil plastic linings, but later changed most specifications to 20 mil linings
- Plastic linings of as low as 8 mil (PE), and up to 100 mil have been used in canals and retention ponds
- Low density polyethylene (LDPE) is made of nearly the same material as common trash bags (such as “Hefty” and “Glad” brands), but these trash bags have a thickness of only 1.5 - 2 mils
- Plastic canal linings are manufactured in rolls, 5 to 7 ft in width, then seamed together in a factory or shop to create sheets or panels of up to 100 ft (or more) in width
- Rubber membrane linings can have a thickness ranging from 20 to 60 mil
- Flexible plastic and synthetic rubber linings are susceptible to damage (punctures, tears) both during and after installation
- Flatter than normal side slopes (say 3:1) are sometimes preferred with plastic linings to help prevent the possible migration of the lining down the slope, and to help prevent uncovering of the lining by downward movement of soil

- Correctly installed plastic and synthetic rubber linings are completely impervious, provided they have not been damaged, and provided that the flow level in the channel does not exceed the height of the lining
- Plastic liners will “age” and lose plasticizer, causing a loss of flexibility and greater potential for damage. Increased plasticizer during fabrication has been shown to be effective in this regard

plas-ti-ciz-er (plas'tuh sie zuhr) n. a group of substances that are used in plastics to impart viscosity, flexibility, softness, or other properties to the finished product

- Some canals in central Utah have had plastic linings for more than 30 years, and most of it is still in good condition (measured seepage is essentially zero in the lined sections, but some evidence of puncture/tearing has been found)
- Plastic lining material is sometimes used to retrofit existing concrete-lined canals after the concrete lining canal fails and or continued maintenance is considered infeasible

Preparing a canal section for buried membrane lining (courtesy R.W. Hill)

- In the former Soviet Union, thin PE lining has been placed under precast slabs of concrete lining in some canals
- In India, some canals have been lined with plastic (PE) on the bottom, and bricks or tiles on the side slopes
- Polyethylene (PE) is the lowest cost geomembrane material, PVC is next lowest. Some newer materials such as polyolefin are more expensive

Exposed and Buried Membranes

- Exposed membrane linings have been tried, but tend to deteriorate quickly for various reasons
- Exposed membrane linings have recently been installed in some full (operating) canals

- Buried membrane lining should have a cover layer of soil of approximately 1/12th of the water depth, plus 10 inches
- Some vegetation can penetrate these types of linings (asphaltic too), so sometimes soil sterilant is applied to the soil on the banks and bed before lining

Fly Ash

- Fly ash is a fine dust particulate material (roughly the size of silt) produced by coal-burning power plants, usually in the form of glassy spheres
- Fly ash contains mostly SiO_2 (silicon dioxide), Al_2O_3 (aluminum oxide), and Fe_2O_3 (iron oxide)
- Fly ash is often mixed with soil to form canal linings, the mixture being more dense and less permeable than soil alone
- Fly ash is sometimes mixed with both soil and portland cement

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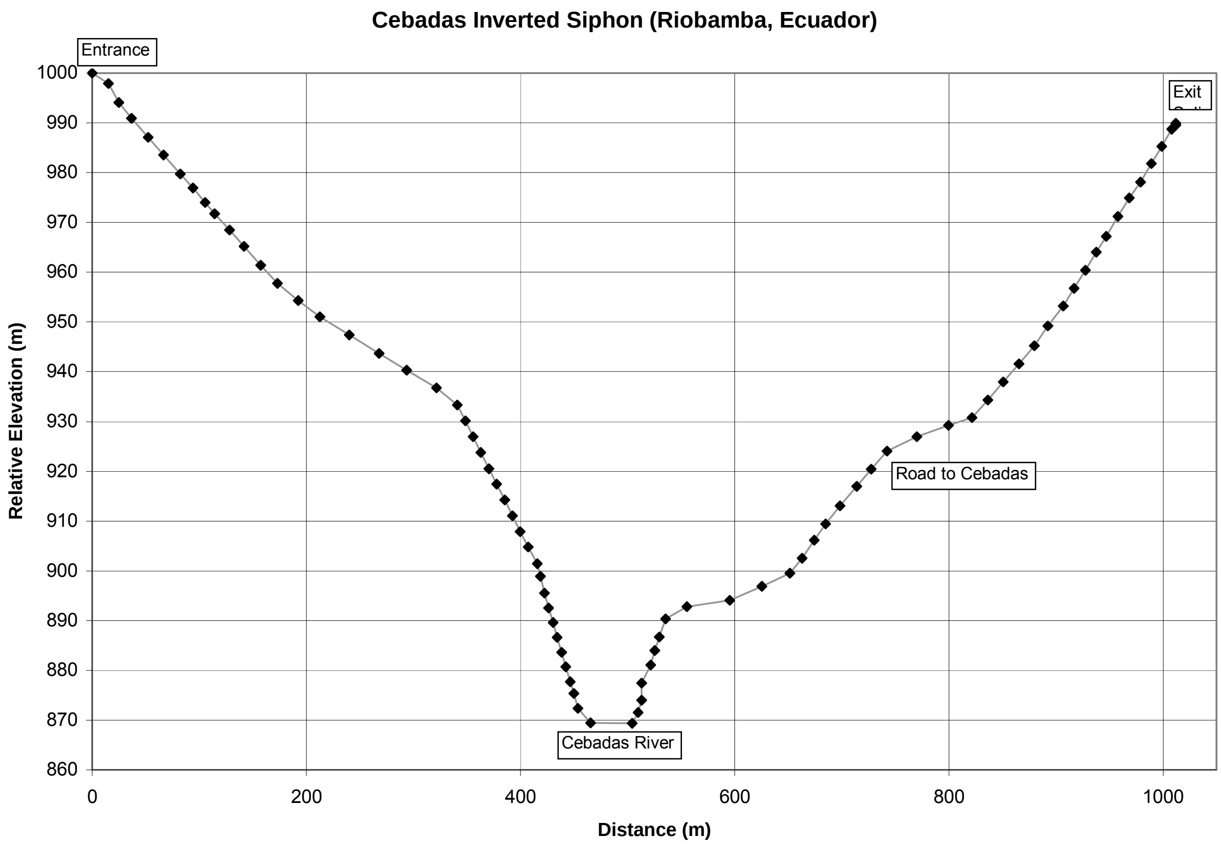
Lecture 20

Inverted Siphons

Portions of the following were adapted from the USBR publication "Design of Small Canal Structures" (1978)

I. Introduction

- Siphons, or *inverted siphons*, are used to convey water across a natural depression, under a road, or under a canal
- Siphons are usually made of circular concrete pipe or PVC, connecting two canal reaches in series
- Some siphons have rectangular cross-sections
- Siphons may have a straight lateral alignment, or may have changes in direction



- When going across a depression, the siphon should be completely buried, usually with a minimum of about 1 m of cover
- Siphons are like culverts, but instead of sloping down from inlet to outlet, they slope down, then back up to the outlet
- Also, siphons usually have only one pipe (not two or three in parallel, as with many culverts)
- Some siphons have multiple pipes in parallel; for example, when the original flow capacity is to be increased
- In general, siphons are longer than culverts

- Open-channel flumes are alternatives to siphons, but may be more expensive to build and/or to maintain, and may not be aesthetically acceptable
- Siphons can be very dangerous (the time to travel from inlet to outlet is usually longer than a someone can hold their breath)
- Siphons can be very problematic with sediment-laden water because sediment may tend to deposit at the low point(s)
- With large siphons, periodic cleaning is possible, but may be impractical with smaller siphons
- Also, cleaning usually means a significant interruption in water delivery service
- Sometimes gravel and rock can enter the siphon
- However, trash racks and/or screens should always be provided at the inlet

II. Structural Components of Siphons

1. Pipe

- Most siphons are built out of pre-cast concrete pipe (PCP), reinforced with steel in larger diameters
- Concrete pipe head class can go up to 200 ft (86 psi) or more
- Pre-stressed concrete cylinder pipe has steel wire wrapped around the pipe with a mortar coating
- Pre-stressed concrete pipe is typically used on large siphons (over 10 ft in diameter), with lengths of 20 ft, and placed in trench with a special vehicle
- Pre-stressed concrete pipe was previously considered the most economical option for large siphons, but experience has shown that the wire may corrode in only 15-18 years of service (e.g. Salt River project in Arizona)
- Older siphons have been built out of asbestos-concrete (AC) mixtures, but this has been discontinued in the USA due to health risks from exposure to asbestos
- Siphons can also be built with plastic or steel pipe, including other less common possibilities (even wood)
- USBR siphon designs always have a single pipe, or barrel, but this is not a design restriction in general, and you can find siphons with two or more pipes in parallel
- USBR siphon designs always have circular pipe cross-sections

- A siphon in the Narmada canal in India was recently built to carry 40,000 cfs (1,100 m^3/s) across a depression; it has multiple rectangular conduits in parallel

- The Central Arizona Project (CAP) has four large siphons with 21-ft diameter pre-stressed concrete pipe; some of these have already been replaced because of corrosion and subsequent structural failure

2. Transitions

Concrete pipe repairs on an inverted siphon

- Transitions for siphons are the inlet and outlet structures
- Most siphons have inlet and outlet structures to reduce head loss, prevent erosion and piping, and maintain submergence ("hydraulic seal")
- It can be very hazardous to omit inlet and outlet structures because these locations are often at steep embankments that would erode very quickly in the event of a breach or overflow
- An emergency spillway is sometimes located just upstream of a siphon inlet
- Transitions in smaller siphons may be of the same design at the inlet and outlet, and standard designs can be used to reduce costs
- With larger siphons, it may be desirable to do a "site-specific" transition design, possibly with different designs for the inlet and outlet

3. Gates and Checks

- Gates and checks can be installed:

  (a) at the entrance of a siphon to control the upstream water level
  (b) at the outlet of a siphon to control upstream submergence

- Operation of a gate at the entrance of a siphon may ensure hydraulic seal, but will not ensure full pipe flow in the downhill section(s) of pipe at discharges below the design value
- It is not common to install a gate at the outlet of a siphon (this is never done in USBR designs)

4. Collars

- Collars may be used, as with culverts, to prevent "piping" and damage due to burrowing animals
- However, with siphons they are not always necessary because the inlet and outlet structures should be designed and built to direct all water into the entrance and exit all water to the downstream channel

5. Blowoff and Vent Structures

- A "blow-off" structure is a valved outlet on top of the pipe at a low point in the siphon
- Smaller siphons often do not have a blow-off structure
- These structures are used to help drain the siphon in an emergency, for routine maintenance, or for winter shut-down
- Blow-off structures may have man holes (or "person access holes") on large siphons to allow convenient entry for manual inspection
- Blow-off structures can be used to periodically remove sediment from the pipe
- Continuous-acting vents are installed in some siphons to remove air during operation (this is usually an after thought when "blow-back" (surging) problems are manifested)
- Others have simple vertical pipes to vent air from the pipe, but these can have the opposite effect
- Air can become trapped, especially in long siphons, during filling; filling of the siphon should be gradual, not sudden
- Some blow-off structures are of the "clamshell" type, with top and bottom leafs off of a tee at the bottom of the siphon
- Clamshell blow-offs are not so common, but have definite advantages in terms of avoiding cavitation (handling high velocity flows at large heads) compared to butterfly valves, for example

6. Canal Wasteways

- A wasteway (side-spill weir) is sometimes built in the canal just upstream of a siphon inlet to divert the canal flow in the case of clogging of the siphon or other emergency situation
- Also, the inlet to a siphon should always have trash racks and/or screens to prevent rocks and other debris from entering the pipe
- If the inlet is at a canal turnout, the design should have a forebay to calm the turbulence before entering the siphon; otherwise, air entrainment may be significant

7. Safety Features

- In operation, siphons can appear to be harmless, especially in a large canal, but can be deadly
- Just upstream of the siphon entrance the following may be used:

    - posted signs with warnings
    - ladder rungs on the canal banks
    - steps on the canal banks
    - cable with floats across the water surface
    - safety net with cables and chains
    - gratings or trash racks

III. Design of Siphons

- The design of siphons has many similarities to the design of culverts; however, unlike the design of culverts:

    1. siphons are usually designed for full pipe flow
    2. siphons are usually designed to minimize head loss
    3. siphons take the water down, then back up

- USBR siphon designs are generally for an assumed 50-year *useful life*

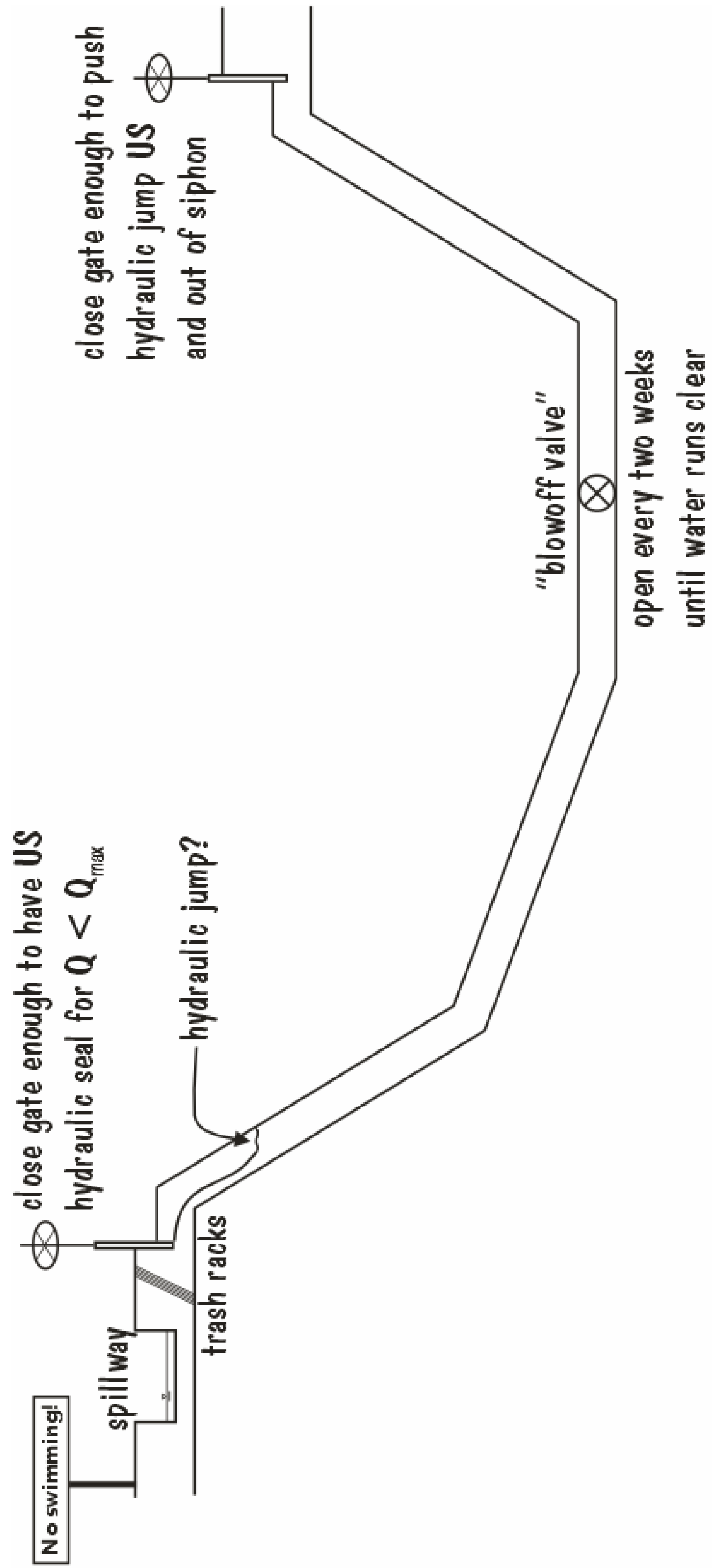
1. Pipe Velocity Limit

- According to the USBR, pipe velocities at design discharge should be between 3.5 and 10 fps
- Recent USBR designs have mostly called for 8 fps velocity
- Many small culverts are designed for 10 fps
- In general, lower pipe velocities are fine for small siphons, but in large capacity and or long siphons it is justifiable to design for higher velocities
- Long siphons can cost much less with even a slightly smaller pipe size

2. Head Losses

- Culverts are usually designed for full pipe flow from inlet to outlet (the outlet is almost always submerged, and it is highly unlikely that open channel flow would prevail throughout the siphon – the change in elevation is usually too great)
- Total head loss is the sum of: inlet, outlet, pipe, and minor head losses
- Convergence losses at the entrance are usually negligible, but divergence losses at the exit (outlet) can be significant
- Most of the loss in a siphon is from pipe friction
- Outlet losses are typically about twice the inlet losses
- Minor losses in pipe bends are usually insignificant
- Most siphons are designed to carry the full design discharge without causing an "M1" profile (backwater) in the upstream channel – to achieve this, it is important to carefully estimate head losses
- If the total siphon head loss at the design discharge exceeds available head (difference in upstream and downstream canal elevations and water depths) the siphon will operate at a lower discharge and cause the upstream water level to increase

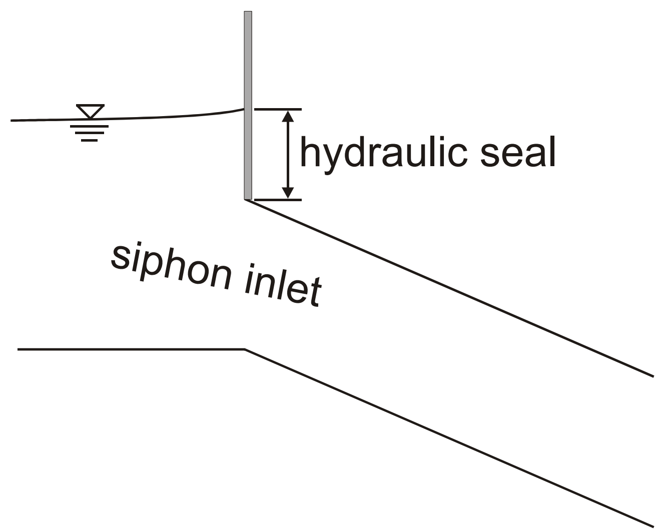
- A hydraulic jump in the descending part of the siphon (upstream side) will greatly increase the head loss, and may cause problems of surging and "blow-back"
- Blow-back occurs when air is entrained into the water due to a hydraulic jump in the pipeline, or due to movement of a hydraulic jump within the pipe; water and air periodically surge backwards through the inlet


close gate enough to push
hydraulic jump US
and out of siphon
"blowoff valve"
open every two weeks
until water runs clear
close gate enough to have US
hydraulic seal for $Q < Q_{max}$
hydraulic jump?
trash racks
spillway
No swimming!

- Blow-back is usually more problematic in siphons with relatively flat descending (upstream) slopes -- a small change in the downstream head can cause a hydraulic jump to move within the pipe at the upstream end

3. Hydraulic Seal

- The "hydraulic seal" is the minimum required upstream head, relative to the upper edge of the siphon pipe at the siphon inlet, to prevent the entrainment of air at that location
- The hydraulic seal recommended by the USBR is equal to $1.5\Delta h_v$, where Δh_v is the difference in velocity heads in the upstream open channel and in the pipe (when flowing full)
- For a more conservative value of the hydraulic seal, use $1.5h_{pipe}$, where h_{pipe} is the velocity head in the siphon pipe when flowing full



4. Design Steps

- Determine the route that the siphon will follow
- Determine the required pipe diameter according to the design discharge and allowable velocity
- Determine the appropriate transition structure types at the inlet and outlet, or design custom transitions for the particular installation
- Design the siphon layout according to the existing terrain, and the proposed (or existing) canal elevations at the inlet and outlet
- Determine the pressure requirements of the pipe according to the head (at the lowest point) during operation
- Determine the total head loss in the siphon at the design discharge
- If the head loss is too high, choose a larger pipe or different pipe material; or, consider adjusting the canal elevations at the inlet and outlet
- Use Fig. 2-7 on page 30 of *Design of Small Canal Structures* (USBR) to determine whether blow-back might be a problem, and make adjustments if necessary

IV. Siphon Pressure Rating

- What is the maximum pressure in the inverted siphon pipe? This must be calculated at the design stage so that a suitable pipe is selected.
- The maximum pressure is equal to the maximum of:

    1. Water surface elevation at the outlet minus the elevation of the lowest point in the siphon (unless there is a closed valve at the outlet); or
    2. Water surface elevation at the inlet minus the elevation of the lowest point in the siphon, minus the friction loss from the entrance to the low point.

- In any case, the maximum pressure will be at, or very near, the location of minimum elevation in an inverted siphon
- If a gate or valve is at the siphon exit, and it is completely closed, the maximum pressure will be according to #2, without subtracting friction loss (i.e. full pipe, zero flow condition); otherwise, the zero flow condition pertains to #1
- Note that the above assumes that an open channel is upstream of the siphon entrance, and an open channel is at the siphon exit
- Note that in order to calculate friction loss, you need to assume a pipe diameter (ID) and a pipe material
- Due to possible water surging in the pipe, the pressure may be somewhat higher than that calculated above, so consider adding a 10% safety factor

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Lecture 21

Culvert Design & Analysis

Much of the following is based on the USBR publication:
"Design of Small Canal Structures" (1978)

I. Cross-Drainage Structures

- Cross-drainage is required when a canal will carry water across natural drainage (runoff) channels, or across natural streams; otherwise, the canal may be damaged
- In some cases, cross-drainage flows are collected in a small channel paralleling the canal, with periodic cross-drainage structures over or under the canal; this is especially prevalent where there are poorly defined natural drainage channels
- In culvert design for carrying runoff water, usually one of the big questions is what the capacity should be
- When the canal capacity is less than the natural channel capacity, it may be economical to build an inverted siphon so the canal crosses the natural channel
- With siphon crossings, it is not nearly as important to accurately estimate the maximum flow in the natural channel because the structure is for the canal flow
- In other cases, it may be more economical to provide cross-drainage by building a culvert to accommodate natural flows after the canal is constructed
- In these cases, the cross-drainage structure does one of the following:

1. Carry water under the canal
2. Carry water over the canal
3. Carry water into the canal

- Here are the common cross-drainage solutions:

1. Culverts

    - These are often appropriate where natural flows cross a fill section of the canal
    - Culverts may tend to clog with weeds, debris, rock, gravel, and or sediments, especially at or near the inlet

2. Over-chutes

    - These are appropriate where the bottom of the natural channel is higher than the full supply level of the canal
    - For example, over-chutes might be used in a cut section of the canal

Canal over-chute & bridge

- Open-channel over-chutes can carry debris and sediment that might clog a culvert, but pipe over-chutes may be equally susceptible to clogging

3. Drain Inlets

- With these structures, the flow of the natural channel is directed into the canal
- These may be appropriate where the natural flows are small compared to the canal capacity, and or when the natural flows are infrequent
- These may be appropriate when the canal traverses a steep slope, and cross-drainage might cause excessive downhill erosion, compromising the canal
- These may be less expensive than over-chute or culvert structures, but may require more frequent maintenance of the canal
- Drain inlets may be problematic insofar as rocks, sediment and other debris can clog the inlet and or fill the canal near the inlet, obstructing the canal flow

II. Alignment

- Align the culvert along natural open channels where possible so that the natural runoff pattern is not disturbed any more than necessary
- If the natural drainage channel is not perpendicular to the canal, it is best to have a skewed alignment of the culvert
- One or more bends in the culvert can be used to help follow the natural channel, especially in longer culverts
- If there is no apparent natural runoff channel, consider using the shortest straight path from inlet to outlet
- In some cases it may be unnecessary or undesirable to follow a natural channel

III. Barrel Profile

- Knowing the inlet and outlet locations will determine the length and slope of the culvert
- The invert of the inlet and outlet should correspond approximately to the natural ground surface elevations at the two respective locations -- otherwise, sedimentation and or erosion will likely occur, requiring maintenance
- However, a compound slope may be needed if:

1. The culvert would not have enough vertical clearance under a canal (about 2 ft for an earth canal, or 0.5 ft for a concrete canal), road, etc.;
2. The slope of the culvert would cause supercritical open-channel flow, which might require a downstream energy dissipation structure (making the design more costly); or,
3. You want to force a hydraulic jump to dissipate energy.

- The USBR recommends, in general, a minimum slope of 0.005 and a maximum slope of somewhat less than the critical slope (maintain subcritical flow)
- The minimum slope is imposed in an effort to prevent sediment deposition in the culvert barrel
- The barrel of the culvert is usually circular (perhaps corrugated pipe) or rectangular
- The maximum slope is imposed in an effort to avoid the additional cost of an energy dissipation structure at the outlet (channels upstream and downstream of culverts are typically unlined, although there may be some riprap)
- With a compound slope, the upstream slope is steeper than critical, and the downstream slope is mild, thereby forcing significant energy dissipation through a hydraulic jump in the vicinity of the break in grade, inside the barrel

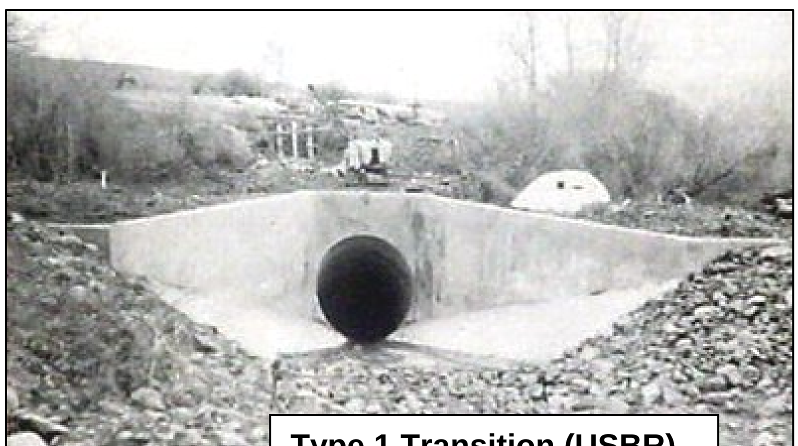
IV. Inlets and Outlets

- USBR Culvert Inlets

Type 1: "broken-back transition", appropriate for natural channels with well-defined upstream cross-section (USBR Figs. 7-1 & 7-2)
Type 2: suitable for wide natural channels with poorly-defined upstream cross section (USBR Fig. 7-4)
Type 3: "box inlet", also for use in a poorly-defined natural channel, but allows for a lower barrel invert at the inlet (USBR Fig. 7-5)
Type 4: similar to Type 3, but with a sloping invert, allowing for an even lower barrel inlet (USBR Figs. 7-6 & 7-7)

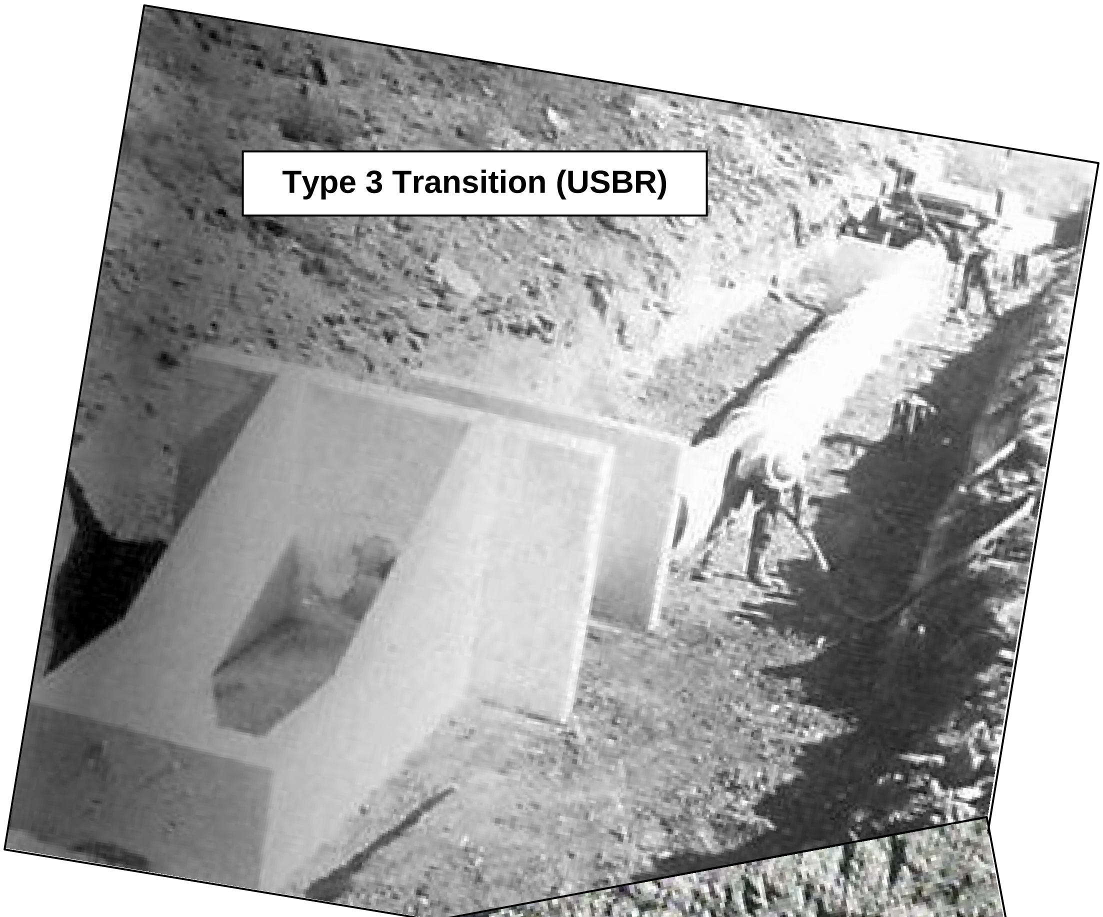
- USBR Culvert Outlets

  1. With energy dissipation structure
  2. Without energy dissipation structure

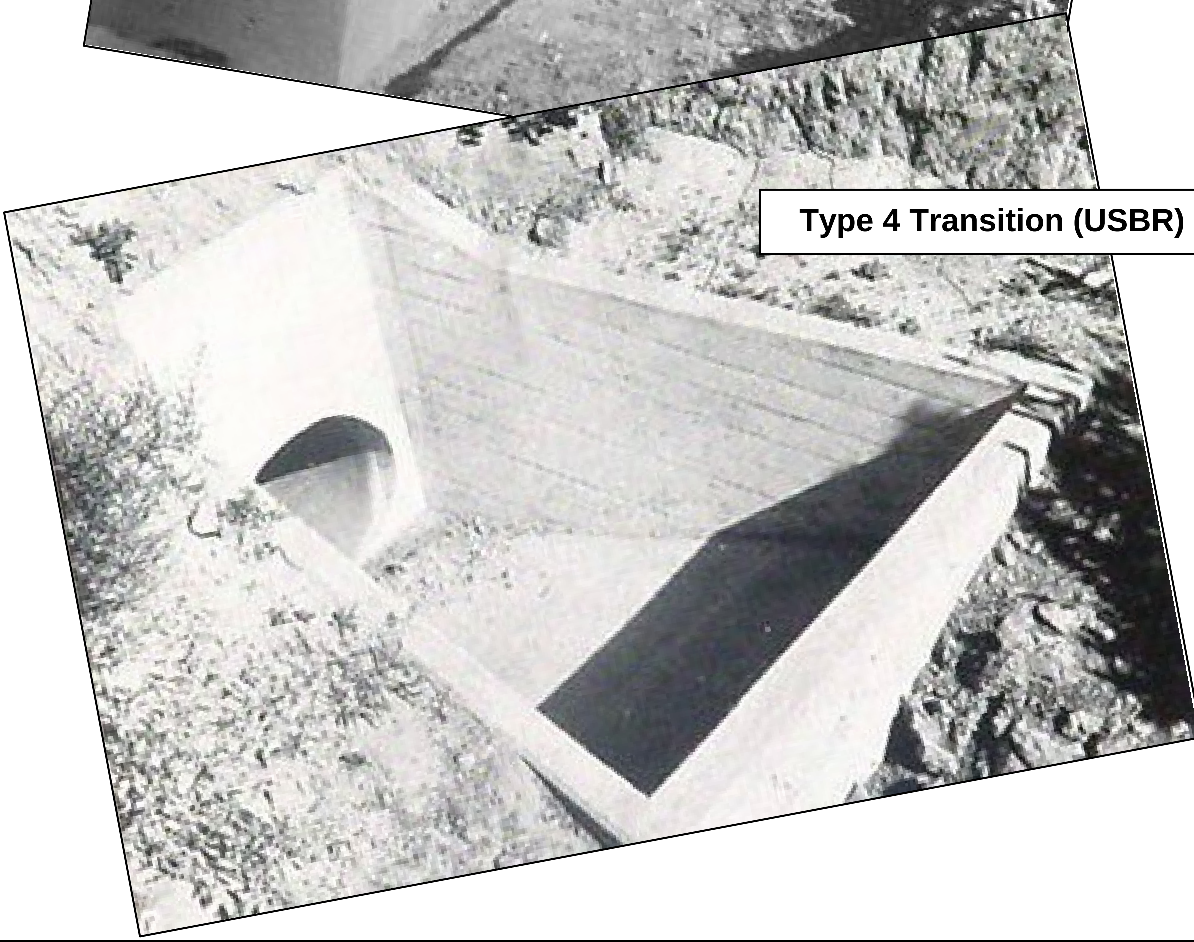


Type 1 Transition (USBR)

- There are other USBR standard inlet designs (besides the above four)
- USBR-type culvert inlets and outlets are made almost exclusively of concrete
- Some corrugated metal culverts have a circular or elliptical cross section with smooth metalic inlet and outlet transitions
- Use standard inlet & outlet designs if possible to save time and to avoid operational and or maintenance problems



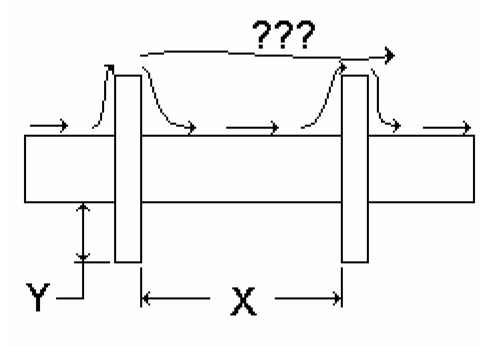
Type 3 Transition (USBR)



Type 4 Transition (USBR)

V. Pipe Collars

- Pipe collars are used to prevent "piping" along the outside of the barrel and or damage by burrowing animals
- For culverts under canals, the typical USBR design calls for three collars: one under the center of the upstream canal bank, and two under the downstream canal bank
- A "short path" between two adjacent collars means that the collars are too close together and or their diameters are too small



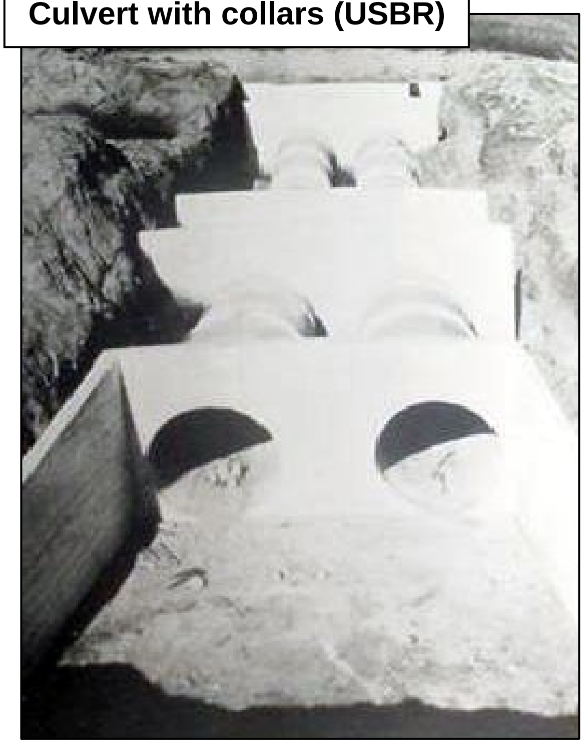
- The USBR recommends the following for minimum collar spacing:

$$X_{min} = 1.2Y \tag{1}$$

VI. Basic Design Hydraulics

- Culverts are typically designed for full-pipe flow in the barrel at the design discharge value
- This means that pressurized pipe flow is impending at the design discharge, but at lower flow rates open-channel flow exists in the barrel
- The upper limit on barrel velocity is usually specified at about 10 fps, or perhaps 12 fps with an energy dissipation structure at the outlet
- For full pipe flow without inlet and outlet structures, in which case the culvert is simply a buried pipe, you can use a limit of 5 fps

Culvert with collars (USBR)



- Knowing the design discharge and the velocity limit, the diameter (circular barrels) for full pipe flow can be directly calculated
- For rectangular barrel sections, you need to determine both width & height

- Discharge capacity can be checked using the Manning (or Chezy) equation for a circular section running full (again, impending pressurization)
- For new pre-cast concrete pipe, the Manning "n" value is about 0.013, but for design purposes you can use a higher value because the pipe won't always be new

- You can also check the discharge using the Darcy-Weisbach equation, with specified values for upstream and downstream water surface elevations in the inlet and outlet structures, respectively

- The head loss through a typical inlet structure with *inlet control* can be estimated as a "minor loss" by:

$$h_f = K\frac{V^2}{2g} \tag{2}$$

  where the coefficient K may vary from 0.05 for a smooth, tapered inlet transition, flush with the culvert barrel, to 0.90 for a projecting, sharp-edged barrel inlet
- Note that the inlet and or outlet losses may or may not be "minor" losses when dealing with culverts, especially when the barrel is short

- For *outlet control*, the head loss is estimated as in the above equation for inlet control, except that there will also be expansion losses downstream
- For *barrel control*, the head loss is the sum of the inlet, barrel, and outlet losses

References & Bibliography

USBR. 1978. *Design of small canal structures*. U.S. Government Printing Office, Washington, D.C. 435 pp.

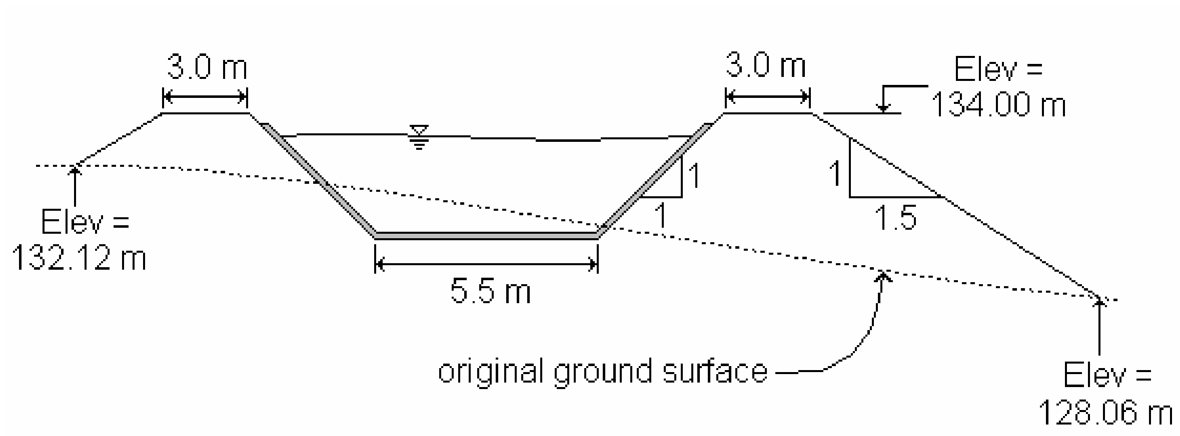
Lecture 22

Example Culvert Design

Much of the following is based on the USBR technical publication "Design of Small Canal Structures" (1978)

I. An Example Culvert Design

- Design a concrete culvert using the procedures given by the USBR (*Design of Small Canal Structures*, USBR 1978)
- The culvert will go underneath a concrete-line canal as shown in the figure below, perpendicular to the canal alignment (shortest path across)



- The outside slope of the canal banks is 1.5:1.0 on both sides (H:V)
- The inside side slope of the concrete lining is 1:1 (both sides)
- The concrete lining thickness is 0.05 m
- Elevation of the top of the canal banks is given as 134.00 m, with elevations of the original ground surface at intersections with canal banks given in the figure
- The berm on both banks is 3.0 m wide
- The depth of the canal from the bottom to the top of the berms is 4.0 m, with the upper 30 cm unlined

- The barrel will be pre-cast circular concrete pipe, and the inlet and outlet structures can be specified as Type 1, 2, 3, or 4, as given by the USBR
- The available concrete pipe has an inside diameter of: 60, 70, 80, and 90 cm
- You must select from one of these diameters for the culvert barrel
- Length of the pipe is 1.5 m per section

- The hydrological assessment of the area came up with a maximum surface runoff of 2.4 m^3/s for a 20-year flood in the area uphill from the canal
- This is the runoff rate that the culvert must be designed to carry

- The upstream and downstream natural channels are wide and poorly defined in cross-section, and no effort will be made to develop prismatic channels upstream of the culvert inlet, nor downstream of the culvert outlet

II. A Culvert Design Solution

1. Determine the Horizontal Distance

- The horizontal distance from the culvert inlet to the pipe outlet is (from left to right):

  1.5*(134.00-132.12)+3.0+4.0+5.5+4.0+3.0+1.5*(134.00-128.06) = 31.23 m

2. Determine the Required Pipe Size

- Use a maximum average barrel velocity of 3.0 m/s
- Then, for the design discharge of 2.4 m^3/s:

$$D = \sqrt{\frac{4Q}{\pi V}} = \sqrt{\frac{4(2.4)}{\pi(3.0)}} = 1.01\,\text{m} \tag{1}$$

- The largest available pipe size is 90 cm; therefore, two or more pipes are needed in parallel for this culvert design

- For half the design discharge, 1.2 m^3/s, the required diameter is:

$$D = \sqrt{\frac{4Q}{\pi V}} = \sqrt{\frac{4(1.2)}{\pi(3.0)}} = 0.71\,\text{m} \tag{2}$$

- Then, we can use two 80-cm ID pipes at a full pipe flow velocity of 2.39 m/s
- It would also be possible to use three 60-cm ID pipes at a full pipe flow velocity of 2.83, which is closer to the maximum velocity of 3.0 m/s
- But, choose two 80-cm ID pipes because it will simplify installation, require less excavation work, and may reduce the overall pipe cost

3. Determine the Energy Loss Gradient

- With the full pipe flow impending, the energy loss gradient can be estimated by the Manning equation for open-channel flow
- Use a Manning n value of 0.015 for new concrete pipe, with a slight safety factor for aging (typical useful life is estimated as 40 to 50 years)
- Use half the design discharge because two 80-cm ID pipes will be installed in parallel

$$S_f = \frac{Q^2 n^2 W_p^{4/3}}{A^{10/3}} = \frac{(1.2)^2 (0.015)^2 (2.51)^{4/3}}{(0.503)^{10/3}} = 0.011\ \text{m/m} \tag{3}$$

where the wetted perimeter, W_p, for full pipe flow is πD; and the area, A, is $\pi D^2/4$, for an inside diameter of 0.80 m and half the design discharge, 1.2 m^3/s

4. <u>Determine the Critical Slope</u>

- For critical flow, the Froude number is equal to unity:

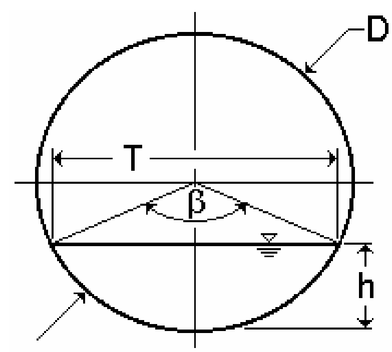
$$F_r^2 = \frac{Q^2 T}{gA^3} = 1.0 \tag{4}$$

- For circular pipes, the following definitions apply:

$$\beta = 2\cos^{-1}\left(1 - \frac{2h}{D}\right) \tag{5}$$

$$T = D\sin\left(\frac{\beta}{2}\right) \quad \text{and,} \quad W_p = \frac{\beta D}{2} \tag{6}$$

$$A = \frac{D^2}{8}\left(\beta - \sin\beta\right) \tag{7}$$



- Solve for depth, h, such that $F_r^2 = 1.0$ for Q = 1.2 m^3/s and D = 0.80 m
- Using the Newton method, h_c = <u>0.663 m</u>

- Calculate the energy loss gradient (critical slope) corresponding to this depth
- For a depth of 0.663 m, the flow area is 0.445 m^2, and the wetted perimeter is 1.83 m
- Applying the Manning equation:

$$\left(S_f\right)_{crit} = \frac{(1.2)^2 (0.015)^2 (1.83)^{4/3}}{(0.445)^{10/3}} = 0.011 \text{ m/m} \tag{8}$$

- This is essentially the same loss gradient as for impending full pipe flow, but note that the critical flow depth is 83% of the pipe ID
- If the slope of the pipe is 0.011 m/m or greater, critical flow can occur

5. Determine the Minimum Upstream Pipe Slope

- The upstream pipe will be situated so as to begin at Elev 132.12, and just clear the canal base at the left side
- The elevation of the canal base is 134.00 - 4.0 = 130.00 m

- The horizontal distance from the culvert inlet to the left side of the canal base is 1.5*(134.00-132.12)+3.0+4.0 = 9.82 m
- The pipe must drop at least 132.12-130.00+0.05+0.2 = 2.37 m over this horizontal distance
- This corresponds to a pipe slope of 2.37/9.82 = 0.24 m/m (24%)

- The critical slope is 1.1% (< 24%), so the culvert will have inlet flow for the design discharge (and for lower discharge values)
- At the design discharge, we will expect a hydraulic jump in the pipe upstream of the bend, because the pipe slope will be lower in the remaining (downstream) portion of the culvert

- It is necessary to check that the slope of the downstream pipe does not exceed the critical slope

6. Determine the Downstream Pipe Slope

- The downstream part of the culvert barrel will travel a horizontal distance of 31.23 - 9.82 = 21.41 m
- The change in elevation over this distance will be 129.75 - 128.06 = 1.69 m
- Then, the slope of the downstream part of the pipe will be 1.69/21.41 = 0.079 m/m (7.9%)

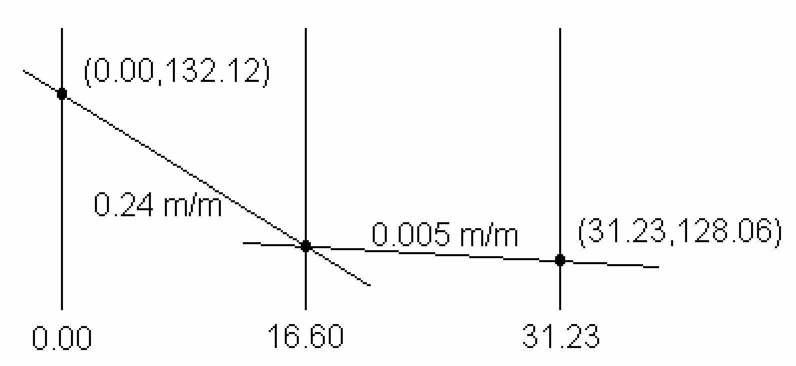
- This slope is greater than the critical slope, and is not acceptable because it would cause supercritical flow throughout, from inlet to outlet, causing erosion downstream (unless erosion protection is used)
- Use the USBR recommended downstream slope of 0.005 (0.5%), which is less than the critical slope of 1.1%

- To accomplish this, the upstream (steep) portion of the culvert pipe can be extended further in the downstream direction (to the right)
- Equations can be written for the tops of the upstream and downstream pipes:

Upstream: .... $y = -0.24x + 132.12$

Downstream:.............................. $y = -0.005x + 128.22$

[Note: 128.06+(31.23)(0.005) = 128.22]



- Solving the two equations for distance, a value of x = 16.60 m is obtained
- This is the distance from the inlet at which the top of the upstream pipe intersects the top of the downstream pipe
- The elevation of the intersection point is y = -0.24(16.60) + 132.12 = 128.14 m
- The minimum clearance of 0.2 m under the canal bed is still provided for, but the excavation for the culvert will require more work
- Note that in many locations the natural ground slope is insufficient to justify a critical slope on the upstream side, and a subcritical slope on the downstream side of the culvert

7. Determine the Pipe Lengths

- The approximate length of the upstream (steep) pipe is:

$$L_{us} = \sqrt{(16.60)^2 + (132.12 - 128.14)^2} = 17.07 \text{ m} \tag{9}$$

- The approximate length of the downstream pipe is:

$$L_{ds} = \sqrt{(31.23 - 16.60)^2 + (128.14 - 128.06)^2} = 14.63 \text{ m} \tag{10}$$

- At 1.5 m per pipe, this corresponds to (17.07 + 14.63)/1.5 ≈ 21 pipes
- For a double-barreled culvert, there must be about 42 pipe lengths

8. Specify Inlet and Outlet Types

- The inlet and outlet can be USBR Type 2, 3, or 4
- Type 1 would not be appropriate because the upstream and downstream channels are not well defined
- An energy dissipation structure at the outlet is not needed (outlet velocity will be < 15 fps)

9. <u>Specify Collar Placement and Size</u>

- Use the standard USBR culvert design, calling for two collars under the downhill canal bank, and one collar under the uphill bank
- The distance between the two collars under the downhill bank will be approximately 3.0 m plus 2.0 ft, or 3.61 m
- Then, the Y value is X/1.2, or Y = 3.61/1.2 = 3.0 m

- This gives very large collars
- There are other methods for determining collar size, but in this case the Y value can be taken as 1.0 m, which would be only about one meter below the uphill canal berm
- More information about the site and soil would be required to verify the adequacy of the collar design
- Many culverts don't have collars anyway, and in some cases they are problematic because they impede effective soil compaction – "piping" may be worse <u>with</u> the collars

References & Bibliography

USBR. 1978. *Design of small canal structures*. U.S. Government Printing Office, Washington, D.C. 435 pp.

Lecture 23
Culvert Hydraulic Behavior

I. Open-Channel Definitions

- The following open-channel definitions apply in most cases:

Canal	Open channel of mild slope (subcritical flow) and relatively long; could be lined or unlined
Flume	A channel built above the natural ground surface, usually of mild slope and rectangular or circular cross section, crossing a depression or running along the contour of a hillside
Chute	Like a flume, but having a steep slope (supercritical flow; $F_r^2 > 1.0$) and usually with some type of energy dissipation structure on the downstream side (outlet)
Culvert	One or more circular or rectangular pipes/conduits in parallel, crossing under a road, canal, or other structure, either flowing full (pressurized) or part full (open-channel flow); often used as a cross-drainage structure
Prismatic	This means a constant cross-sectional shape with distance, constant and uniform bed slope, and straight channel alignment, applied to any of the above

II. Culvert Hydraulics & Flow Regimes

- A culvert can serve as a combination open-channel and closed conduit structure, depending upon the type of flow condition in the culvert
- Most of the research involving the hydraulics of culverts has been concerned with the use of such structures under highways
- Some of the research has focused on inlet control (free orifice flow) and submerged outlet control (submerged orifice flow)
- For culverts placed in an irrigation conveyance channel (i.e. not as a cross-drainage structure), free surface (open-channel) flow usually occurs in the culvert
- Culverts as part of an irrigation canal often pass under vehicular bridges
- Typically, downstream hydraulic conditions will likely control the depth of flow and the discharge in the culvert
- Culvert hydraulics can be much more complex than might appear at first glance

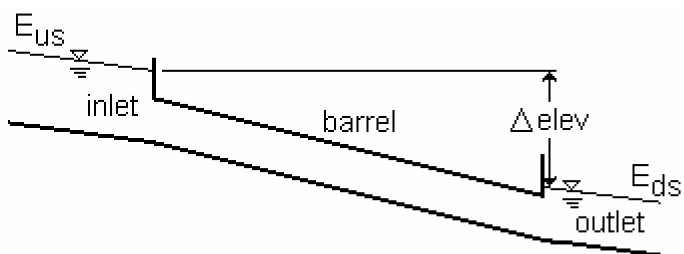
III. Flow Regimes

- The classification of the hydraulic behavior of culverts can take several forms
- Three primary groupings can be used to describe the hydraulics of culverts
- These groups are based on the three parts of the culvert that exert primary control on culvert performance and capacity:

(1) the *inlet;*
(2) the *barrel;* and
(3) the *outlet*

- Usually, one of these three primary controls determines the performance and capacity of the culvert
- An example of this is a projecting, square-edged inlet with the barrel on a steep slope (F_r^2 > 1.0) and flowing partly full: if the inlet is not submerged, the upstream water level (headwater) is determined by the inlet characteristics alone
- In other cases, two or even all three primary controls can simultaneously affect the performance and discharge capacity

- For example, if the inlet and outlet are submerged and the barrel is full, then a designer can determine the headwater elevation by adding the outlet losses, the barrel friction losses, and the inlet losses to the tailwater (downstream) elevation (assuming the same specific energy in both the upstream & downstream open channels)



Culvert with Full Barrel Flow

- The classification is further subdivided under each main group, as shown in the table below (Blaisdell 1966)
- The classification indicates the number of factors the designer must consider when determining the performance of a culvert and computing its capacity under different regimes
- Only those items that exert a control on the hydraulic performance of a culvert are listed in the table
- Many alternatives are possible for each control

- For example, each type of inlet will have a different effect on the culvert performance
- Many of the items listed in the table are inter-related, which further complicates an already difficult problem
- For instance, the depth of the flow just inside the culvert entrance depends on the inlet geometry
- If this depth is less than the normal depth of flow, a water surface profile must be computed beginning with the contracted depth of flow to determine the flow depth at the culvert outlet

INLET CONTROL

A. Unsubmerged (Free Surface)
    1. Weir (Modular Flow)
    2. Surface profile (Non-Modular Flow)

B. Submerged (Inlet Crown Under Water)
    1. Orifice (Free Orifice Flow)
    2. Vortex (Non-Aerated Jet)
    3. Full (Submerged Orifice Flow)

BARREL CONTROL

C. Length
    1. Short
    2. Long

D. Slope
    1. Mild
        i. Barrel slope less than critical slope
            a. Part full, normal depth greater than critical depth
            b. Full, not applicable
        ii. Barrel slope less than friction slope
            a. Part full, depth increases along barrel
            b. Full, barrel under pressure
    2. Steep
        i. Barrel slope steeper than critical slope
            a. Part full, normal depth less than critical depth
            b. Full, not applicable
        ii. Barrel slope steeper than friction slope
            a. Part full, depth decreases along barrel (increases if the inlet causes the depth inside the inlet to be less than normal depth)
            b. Full, barrel under suction

E. Discharge
    1. Partially Full (Free-Surface Open-Channel Flow)
    2. Slug and Mixture (Unsteady Flow)
    3. Full (Closed Conduit Flow)

OUTLET CONTROL

F. Part Full (Free Surface Open Channel Flow)
    1. Critical Depth (Free Flow)
    2. Tailwater (Submerged Flow)

G. Full (Closed Conduit Flow)
    1. Free (Free Orifice Flow)
    2. Submerged (Submerged Orifice Flow)

IV. Hydraulically Short & Long

- If the computed outlet depth exceeds the barrel height, the culvert is hydraulically long, the barrel will fill, and the control will be the inlet, the barrel, and the outlet
- If the computed depth at the outlet is less than the barrel height, the barrel is only part full and the culvert is considered hydraulically short, will not fill, and the control will remain at the inlet
- Whether a culvert is hydraulically long or short depends on things such as the barrel slope and the culvert material
- For example, changing from corrugated pipe to concrete pipe can change the hydraulic length of a culvert from long to short
- A similar effect could result from a change in the inlet geometry
- Flow in culverts is also controlled by the hydraulic capacity of one section of the installation
- The discharge is either controlled at the culvert entrance or at the outlet, and is designated inlet control and outlet control, respectively
- In general, inlet control will exist as long as the ability of the culvert pipe to carry the flow exceeds the ability of water to enter the culvert through the inlet
- Outlet control will exist when the ability of the pipe barrel to carry water away from the entrance is less than the flow that actually enters the inlet
- The location of the control section will shift as the relative capacities of the entrance and barrel sections change with increasing or decreasing discharge
- This means that it cannot be assumed that a given culvert will always operate under the same hydraulic regime

V. Three Hydraulic Classifications

Inlet Control

- Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater and the entrance geometry, including the barrel shape and cross-sectional area
- With inlet control, the roughness and length of the culvert barrel, as well as outlet conditions (including depth of tailwater), are not factors in determining culvert capacity
- An increase in barrel slope reduces the headwater (inlet) depth, and any correction for slope can be neglected for conventional or commonly used culverts operating under inlet control

Barrel Control

- Under barrel control, the discharge in the culvert is controlled by the combined hydraulic effects of the entrance (inlet), barrel length & slope, and roughness of the pipe barrel
- The characteristics of the flow do not always identify the type of flow
- It is possible, particularly at low flows, for length, slope, and roughness to control the discharge without causing the pipe to flow full

- But, this is not common at design discharges for most culverts
- The usual condition for this type of flow at the design discharge is one in which the pipe cross section flows full for a major portion (but not all) of the length of the culvert
- The discharge in this case is controlled by the combined effect of all hydraulic factors

Outlet Control

- Culverts flowing with outlet control can have the barrel full of water or partly full for either all or part of the barrel length
- If the entire cross section of the barrel is filled with water for the total length of the barrel, the culvert is said to be flowing full

VI. Culvert Flow Regimes

- The following is a slightly different classification of culvert flow
- The flow through culverts can be divided into six categories (French 1985; Bodhaine 1976), depending on the upstream and downstream free-surface water elevations, and the elevations of the culvert inlet and outlet
- The following categories are defined based on the design (maximum) discharge capacity of a culvert

Type I Flow Inlet control. Critical depth occurs at or near the inlet:

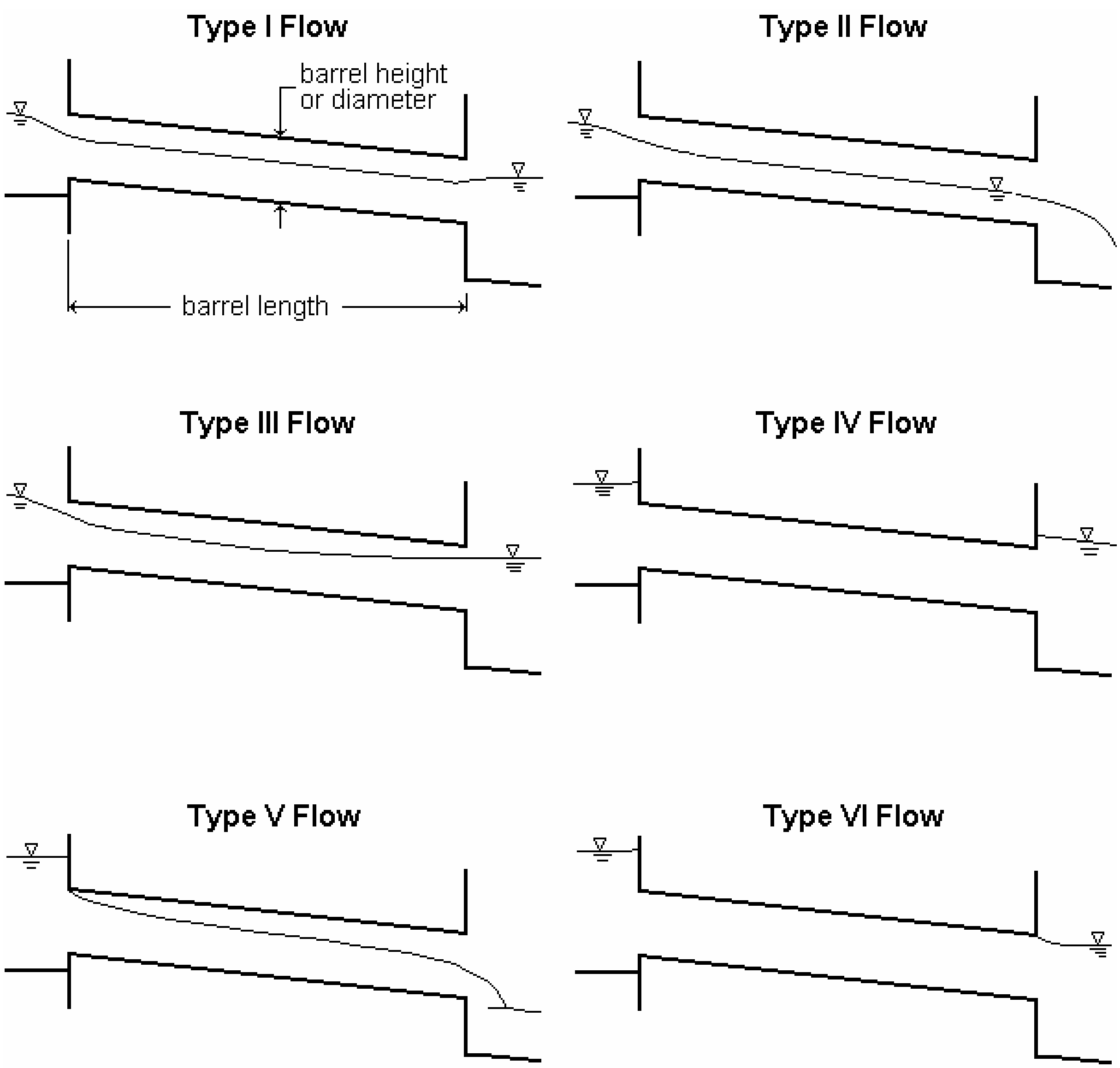
(a) The slope of the culvert barrel is greater than the critical slope
(b) The downstream water surface elevation is lower than the elevation of the water surface where critical flow occurs at the inlet
(c) The upstream water depth is less than approximately 1.5 times the barrel height (or diameter)

Type II Flow Outlet control. Critical depth occurs at or near the outlet:

(a) The slope of the culvert barrel is less than critical slope
(b) The downstream water surface elevation is lower than the elevation of the water surface where critical flow occurs at the outlet
(c) The upstream water depth is less than approximately 1.5 times the barrel height (or diameter)

Type III Flow Barrel control. Subcritical barrel flow, a gradually-varied flow profile:

(a) The downstream water surface elevation is less than the height (or diameter) of the barrel, but is more than the critical depth at the outlet
(b) The upstream water depth is less than approximately 1.5 times the barrel height (or diameter)



Type IV Flow Barrel control. Both the upstream and downstream ends of the culvert are submerged, and the barrel is completely full of water. The culvert behaves essentially like an orifice, but with additional head loss due to the barrel.

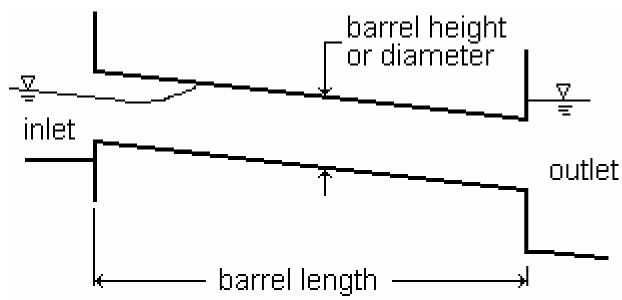
Type V Flow Inlet control. The barrel flows partially full and supercritical flow occurs in the barrel downstream of the inlet:

(a) The slope of the culvert barrel is greater than the critical slope
(b) The upstream water depth is greater than approximately 1.5 times the barrel height (or diameter)

Type VI Flow Barrel control. The culvert is completely full of water:

(a) The upstream water depth is greater than approximately 1.5 times the barrel height (or diameter)
(b) The outlet is unsubmerged (downstream depth less than the barrel height or diameter)

VII. Additional Culvert Flow Regimes



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Lecture 24

Flumes & Channel Transitions

I. General Characteristics of Flumes

- Flumes are often used:

    1. Along contours of steep slopes where minimal excavation is desired
    2. On flat terrain where it is desired to minimize pumping, except perhaps at the source
    3. On flat terrain where it is desired to avoid pumping, except maybe at the water source
    4. Where cross-drainage is required over a depression

Flume/chute on a steep slope

- A pipeline or siphon can be considered to be an alternative to a flume in many cases, or to a canal – these days the choice is essentially one of economics
- Aesthetics may make an inverted siphon preferable to a flume for the crossing of a depression, even though the flume could be less costly
- Unlike most canals, flumes seldom have gates or other flow control structures; that is, they are generally used strictly for conveyance

Pipeline… aesthetics?

- The average flow velocity in a flume is higher than that for most canals
- But the flow regime in flumes is usually subcritical, as opposed to *chutes*, which usually operate under supercritical flow conditions
- Flume cross-section shapes are typically rectangular, but may also be semi-circular or parabolic
- Several irrigation systems in Morocco have networks of elevated semi-circular flumes
- Flumes with non-rectangular sections are usually pre-cast concrete, or concrete mixed with other materials
- Flumes may have under-drains, side inlet structures, and over-pass structures to handle cross flows, especially for cross flows going down a slope

Flume with parabolic section

- Some flumes are elevated for crossing natural channels, depressions, roads, railroads, etc.

II. Flume Design

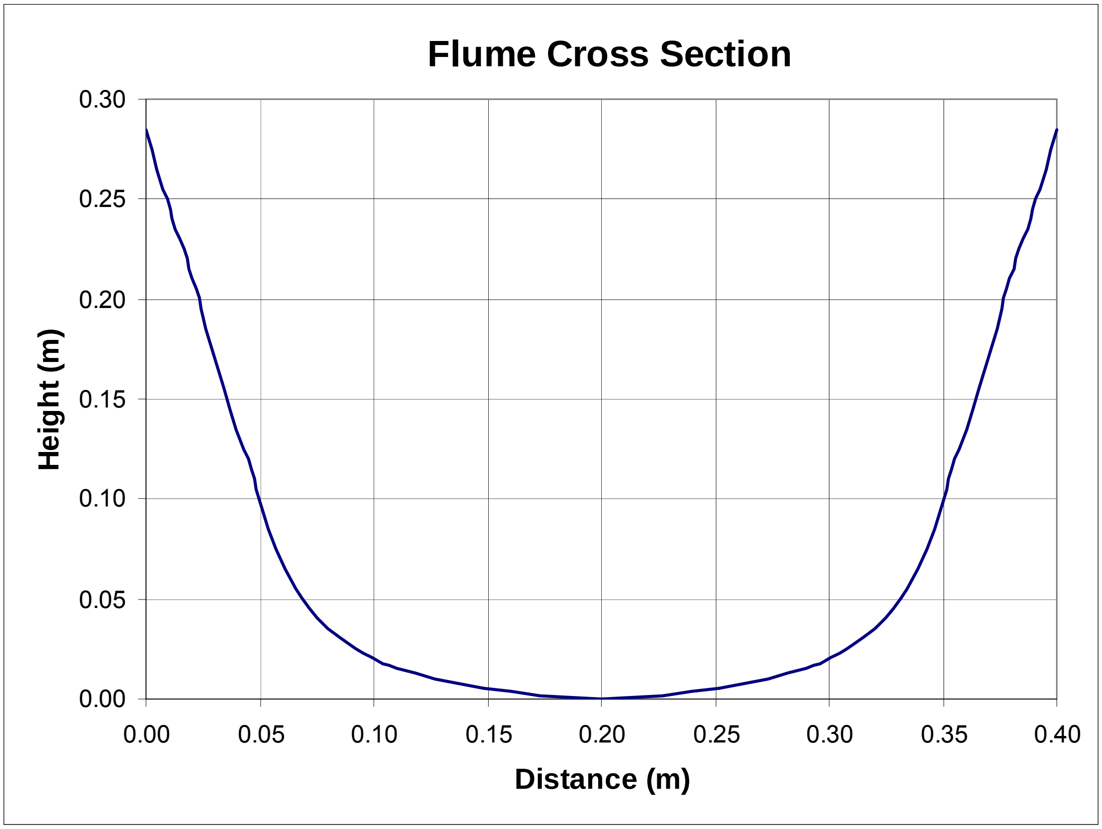
- The hydraulic design of flumes involves calculations of maximum steady-state capacity:

  1. Determine the design discharge
  2. Select an appropriate route with consideration to total length, construction costs, longitudinal bed slope, need for cross-drainage, and other factors
  3. Select a cross-sectional shape (usually rectangular, sometimes circular)
  4. Select a material for the flume channel (this will determine the roughness)
  5. Use the Chezy or Manning equations to determine the size of the cross section (unless you expect nonuniform flow conditions)
  6. Make adjustments as necessary to accommodate the design discharge and other technical, aesthetic, safety, and economic factors

- As noted in a previous lecture, the hydraulic efficiency of the cross section may be a consideration, semi-circular sections being the most efficient
- Inlet and outlet water levels may be a consideration in the flume design
- For example, the flume may be connected to a reservoir with a specified range of water surface elevation
- As in any open-channel, avoid flume designs that would produce near-critical flow conditions at the design capacity; attempt to arrive at a design with $F_r < 0.9$
- Consider USBR and or other guidelines for the inclusion of freeboard at the design discharge
- Note that an overflow could quickly cause severe erosion under the flume

III. Flume Cross-Sections

- A variety of cross section shapes have been used for flumes
- The most hydraulically efficient section is that which has a maximum value of the hydraulic radius for a given discharge, roughness and side slope
- The most hydraulically efficient rectangular section is that with a bed width to depth ratio, b/h, of 2.0
- According to the USBR, the most cost-effective rectangular sections have a range of ratios from $1 \leq b/h \leq 3$



Cross section of a pre-cast concrete flume used in several irrigation systems in the Dominican Republic

IV. Function of Channel Transitions

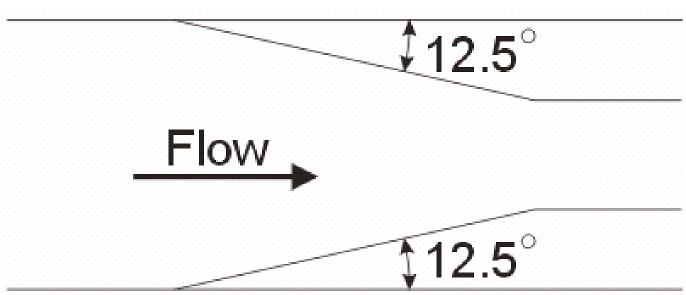
- Channel transitions occur at locations of cross-sectional change, usually over a short distance
- Transitions are also used at entrances and exits of pipes such as culverts and inverted siphons
- Below are some of the principal reasons for using transitions:

  1. Provide a smooth change in channel cross section
  2. Provide a smooth (possibly linear) change in water surface elevation
  3. Gradually accelerate flow at pipe inlets, and gradually decelerate flow at pipe outlets
  4. Avoid unnecessary head loss through the change in cross section
  5. Prevent occurrence of cross-waves, standing waves, and surface turbulence in general
  6. Protect the upstream and downstream channels by reducing soil erosion
  7. To *cause head loss* for erosion protection downstream; in this case, it is an energy dissipation & transition structure

V. General Comments about Transitions

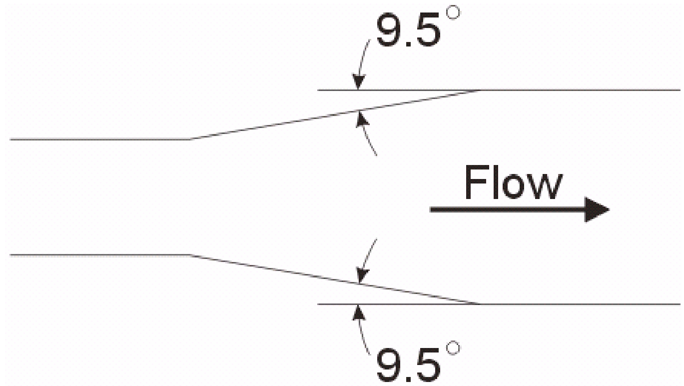
- Transitions at pipe, flume, and canal outlets (ends) often have energy dissipation structures included or added to the design
- Transitions are typically made of concrete or earth, the latter often having some sort of riprap protection
- Earthen transitions between open-channel and pipe flow (culverts and siphons) are often acceptable when the velocity in the pipe is less than about 3.5 fps (1 m/s)
- Transitions can have both lateral and vertical (bed) contraction or expansion

- The optimum angle of lateral convergence (at contractions) is given as 12.5° by Chow (1959), corresponding to a 4.5:1 ratio



- The optimum angle of lateral divergence (at expansions) is often taken as approximately 9.5°, or a 6:1 ratio

- This divergence ratio is used in a number of expansion transitions in open channels, pipes, nozzles and other devices



- It is very difficult to design transitions that work well over a wide range of flows
- Consequently, transitions are often designed for specified maximum flow rates

- The theoretical analysis of hydraulics in transitions is limited, especially when the analysis is for one-dimensional flow
- Three-dimensional analysis is usually required to predict the occurrence of waves and to estimate head loss under different conditions, but even this is often inadequate
- Physical models are required, in general, for reliable evaluation of the performance of a particular transition design under various flow conditions
- Some day, mathematical models will be up to the task

VI. Standard USBR Transitions

- Standard designs are *selected* for many small transitions because the time and effort to design a special transition for a particular case may not be justified, and because the engineer may not know how to go about designing a transition based on the application of hydraulic equations
- The most common type of transition used in small canal structures by the USBR is called “broken back” (Type I), which has vertical walls on the converging/diverging sides

- For small structures (about 100 cfs or less), the USBR usually applies one of five standard transition designs
- Standard USBR transitions are given in Chapter VII of the “Design of Small Canal Structures” (1978) book and other publications and design reports
- More sophisticated transitions may be designed for larger flow rates

- Many of the USBR transitions are inlet and outlet structures for pipes, not transitions between channel cross section changes
- Many open-channel-to-pipe transition designs call for a transition length (in the direction of flow) of about three times the diameter of the pipe

* The length can also be defined by a specified angle of convergence or divergence, knowing the upstream and downstream channel widths
* For earthen transitions the minimum length may be given as about 5 ft or 2 m

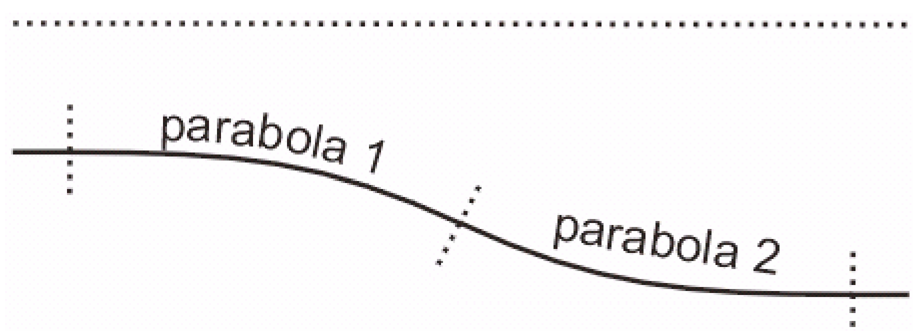
* Inlet transitions to inverted siphons are generally designed such that the top of the pipe is below the upstream open-channel water surface for the design discharge (this is to help prevent the continuous entry of air into the siphon, which reduces capacity)
* The USBR calls the difference in elevation between water surface and top of pipe opening the "hydraulic seal"
* Inlet transitions for culverts are generally designed such that inlet control occurs, possibly causing supercritical flow and a downstream hydraulic jump inside the pipe (barrel)

VII. Computational Design Procedures

* A change in cross-sectional size or shape will generally cause a change in water surface elevation
* However, in some cases it is desirable to avoid surface elevation changes and backwater profiles
* This can be done by calculating one or more dimensions of the cross sections such that the water surface elevation continues on a uniform downhill gradient through the transition for a specified "design" discharge
* This is possible for subcritical flow only, not for supercritical flow

* The head loss in inlet transitions is typically taken to be about $0.1\Delta h_v$, where Δh_v is the change in velocity head from upstream to downstream across the transition
* For outlet transitions, the loss is usually about double this, or $0.2\Delta h_v$
* In some cases, these losses may be twice these values, or more, but Δh_v is usually very small anyway, compared to the specific energy

* Another approach, sometimes used in changes of width for rectangular sections, is the *reverse parabola* transition, consisting of two parabolas which define the bed elevation through the transition, or the channel width through the transition



* The second parabola is the same as the first, but inverted vertically & horizontally

- The transition length is determined before the parabola is defined, and the total length of the transition is divided into two halves with the parabola in the second half being the inverted parabola of the first half
- This procedure is described in some hydraulics books
- You can also use a unique 3^{rd-}degree polynomial instead of two parabolas by fixing the end points and specifying zero slope at each end
- In many cases the transition can be designed to give a very smooth water surface
- The reverse parabola approach can also be applied to trapezoidal to rectangular cross-section transitions, but the calculations are more involved
- A typical design approach is to use a continuous and uniform reduction in side slope along the length of the transition, with a calculated bed elevation such that the water surface continues as smoothly as possible from upstream normal depth to downstream normal depth

Lecture 25

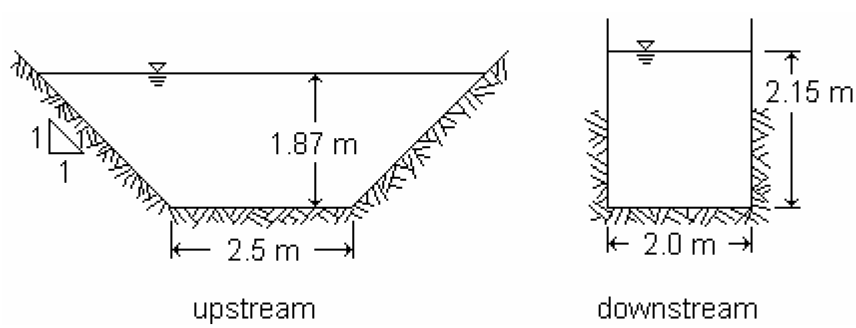
Design Example for a Channel Transition

I. Introduction

- This example will be for a transition from a trapezoidal canal section to a rectangular flume section
- The objective of the transition design is to avoid backwater (GVF) profiles in the transition, and upstream & downstream of the transition
- We will specify a length for the transition, but the total net change in canal invert elevation across the transition will be defined as part of the solution
- The main design challenge will be to determine the shape (profile) of the canal invert across the transition

II. Given Information

- The design flow rate is 15.0 m^3/s
- The upstream trapezoidal section has 1:1 side slopes (m = 1)
- The bed slope of the upstream trapezoidal section is 0.000516 m/m
- The bed slope of the downstream rectangular flume is 0.00292 m/m
- The upstream and downstream channels are concrete-lined, as will be the transition
- In this example, the length of the transition is specified to be L = 8.0 m; in other cases the invert elevation change, Δz might be specified
- Both L and Δz cannot be specified beforehand because it would unnecessarily constrain the solution



- The base widths and uniform flow depths for the upstream and downstream channels are shown in the figure above; these were determined during the design procedures for the respective channels (canal & flume)
- These calculations can be confirmed by applying the Manning or Chezy equations

- The reduction in bottom width of the channel will be accomplished with a reverse parabola, from b = 2.5 m to b = 2.0 m
- The reduction in side slope from m = 1 to m = 0 will be done linearly across the length L of the transition

III. Confirm Subcritical Flow

- In the upstream channel, for uniform flow, the squared Froude number is:

$$F_r^2 = \frac{Q^2 T}{gA^3} = \frac{Q^2(b+2mh)}{g\left[h(b+mh)\right]^3} = \frac{(15)^2(2.5+2(1)(1.87))}{9.81\left[1.87(2.5+(1)(1.87))\right]^3} = 0.262 \qquad (1)$$

- In the downstream channel (flume), for uniform flow, the squared Froude number is:

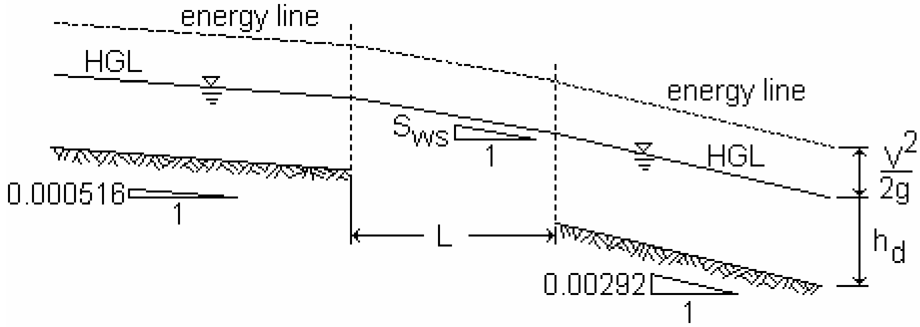
$$F_r^2 = \frac{Q^2 T}{gA^3} = \frac{Q^2 b}{g\left[hb\right]^3} = \frac{(15)^2(2.0)}{9.81\left[(2.15)(2.0)\right]^3} = 0.577 \qquad (2)$$

- Therefore, $F_r^2 < 1.0$ for both the upstream canal and downstream flume
- Then, the flow regime in the transition should also be subcritical
- It would probably also be all right if the flow were supercritical in the flume, as long as the flow remained subcritical upstream; a hydraulic jump in the transition would cause a problem with our given design criterion

IV. Energy Balance Across the Transition

- For uniform flow, the slope of the water surface equals the slope of the channel bed
- Then, the slope of the upstream water surface is 0.000516, and for the downstream water surface it is 0.00292
- Since the mean velocity is constant for uniform flow, the respective energy lines will have the same slopes as the hydraulic grade lines (HGL), upstream and downstream
- For our design criterion of no GVF profiles, we will make the slope of the energy line through the transition equal to the average of the US and DS energy line slopes:

$$S_{EL} = \frac{0.000516 + 0.00292}{2} = 0.001718 \qquad (3)$$



- This means that the total hydraulic energy loss across the transition will be:

$$\Delta E = (0.001718)(8.0) = 0.0137 \text{ m} \tag{4}$$

where the length of the transition was given as L = 8.0 m

- The energy balance across the transition is:

$$h_u + \frac{Q^2}{2gA_u^2} + \Delta z = h_d + \frac{Q^2}{2gA_d^2} + \Delta E \tag{5}$$

where h_u is the upstream depth (m); Q is the design flow rate (m^3/s); A_u is the upstream cross-sectional flow area (m^2); Δz is the total net change in canal invert across the transition (m); h_d is the downstream depth (m); A_d is the downstream cross-sectional flow area (m^2); and ΔE is the hydraulic energy loss across the transition (m)

- The Δz value is unknown at this point, but the slope of the water surface across the transition should be equal to:

$$S_{ws} = \frac{h_u + \Delta z - h_d}{L} \tag{6}$$

where S_{ws} is the (constant) slope of the water surface across the transition (m/m); and L is the length of the transition (m)

- Combining Eqs. 5 & 6:

$$S_{ws} = \frac{\dfrac{Q^2}{2g}\left(\dfrac{1}{A_d^2} - \dfrac{1}{A_u^2}\right) + \Delta E}{L} \tag{7}$$

- For Q = 15 m^3/s; A_d = (2.15)(2.0) = 4.30 m^2; A_u = (1.87)(2.5)+(1.0)$(1.87)^2$ = 8.172 m^2; ΔE = 0.0137 m; and L = 8.0 m:

$$S_{ws} = \frac{\dfrac{(15)^2}{2(9.81)}\left(\dfrac{1}{(4.3)^2} - \dfrac{1}{(8.172)^2}\right) + 0.0137}{8.0} = 0.0578 \tag{8}$$

- Note that $S_{ws} \neq S_{EL}$

V. Change in Side Slope

- The side slope will change linearly from 1 to 0 over the length of the transition
- The equation for m, with x = 0 at the upstream end of the transition, is:

$$m = 1 - 0.125\,x \tag{9}$$

where $0 \leq x \leq 8$ m

VI. Change in Bed Width

- The bed width decreases from 2.5 to 2.0 m over the length of the transition
- This reduction is specified to be a reverse parabola, defined over L/2 = 4.0 m
- Specific criteria could be used to define the shape of the parabola, but a reduction of 0.5 m in bed width over an 8.0-m distance can be accomplished in a simpler way
- Define the bed width, b, for the first half of the transition as follows:

$$b = 2.5 - \frac{x^2}{64} \tag{10}$$

where $0 \leq x \leq 4$ m

- For x > 4 m, the equation is:

$$b = 2.0 + \frac{(x-8)^2}{64} \tag{11}$$

where $4 \leq x \leq 8$ m

- You can also do this with a 3rd-degree polynomial:

$$b = Ax^3 + Bx^2 + Cx + D \tag{12}$$

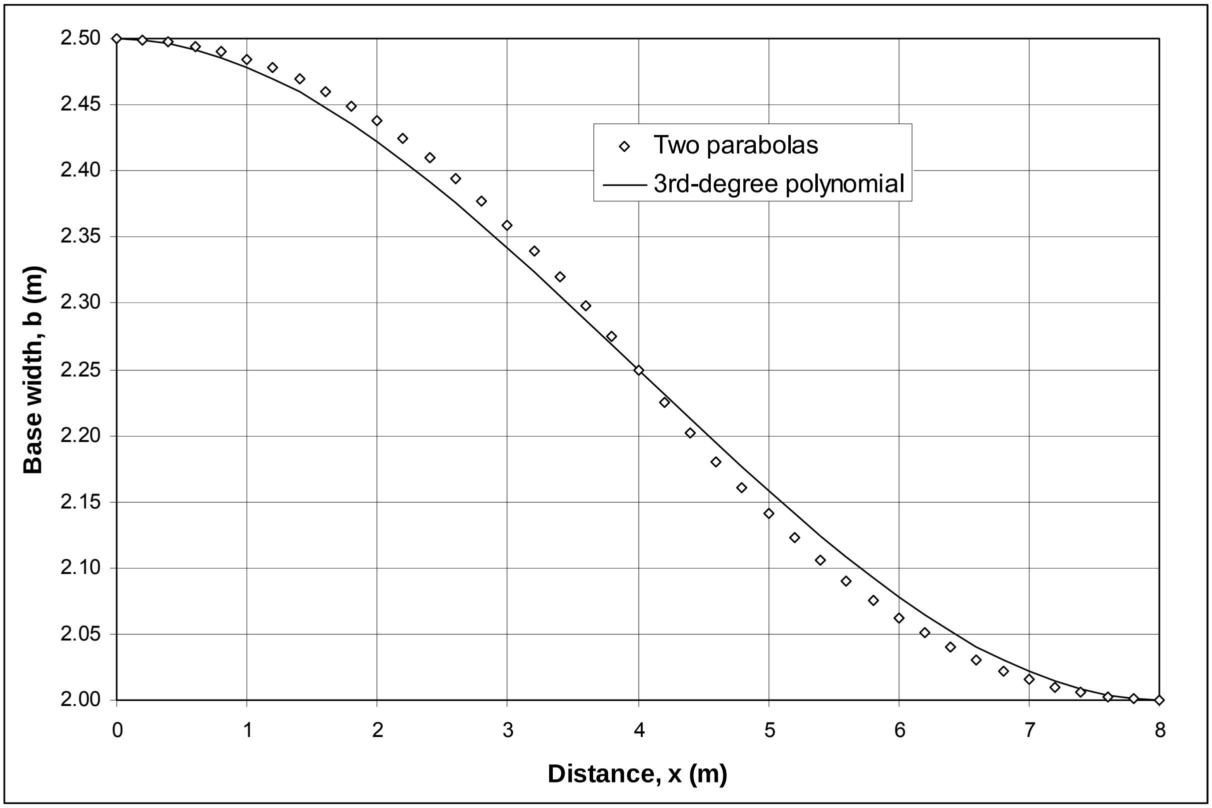
where A, B, C, D are fitted so that the slope is zero at x = 0 and at x = 8

- By quick inspection of Eq. 12, it is seen that b = 2.5 at x = 0, so D = 2.5
- And, at x = 8, b = 2.0
- The other two conditions are that the slope equal zero at the end points:

$$3Ax^2 + 2Bx + C = 0 \tag{13}$$

where x = 0 and x = 8

- So, C must be equal to zero, and then A and B can be determined after a small amount of algebra: A = 0.001953125, B = -0.0234375
- The results are not identical, but very close (see the figure below)



VII. Change in Bed Elevation

- The change in bed elevation can be determined by setting up and solving a differential equation, or by the known change in velocity head across the transition
- Setting up and solving the differential equation can be done, but it is easier to apply the velocity head, which is the difference between the energy line (EL) and the hydraulic grade line (HGL)
- The slope of the EL is S_{EL} = 0.001718 in the transition, and the slope of the water surface is S_{ws} = 0.0578

- The velocity head can be described as follows:

$$\frac{V^2}{2g} = \frac{Q^2}{2gA^2} + x(S_{ws} - S_{EL}) = \frac{(15)^2}{2(9.81)(8.172)^2} + 0.0561x \qquad (14)$$

or,

$$\frac{V^2}{2g} = 0.172 + 0.0561x \tag{15}$$

- And, the cross-sectional area of flow, A, is equal to Q/V, which equals h(b+mh):

$$A = \frac{Q}{\sqrt{2g(0.172 + 0.0561x)}} = h(b + mh) \tag{16}$$

where b and m are defined as functions of x in Eqs. 9, 10, 11; and $0 \le x \le 8$ m

- Eq. 16 is quadratic in terms of h:

$$h = \frac{-b + \sqrt{b^2 + 4mA}}{2m} \tag{17}$$

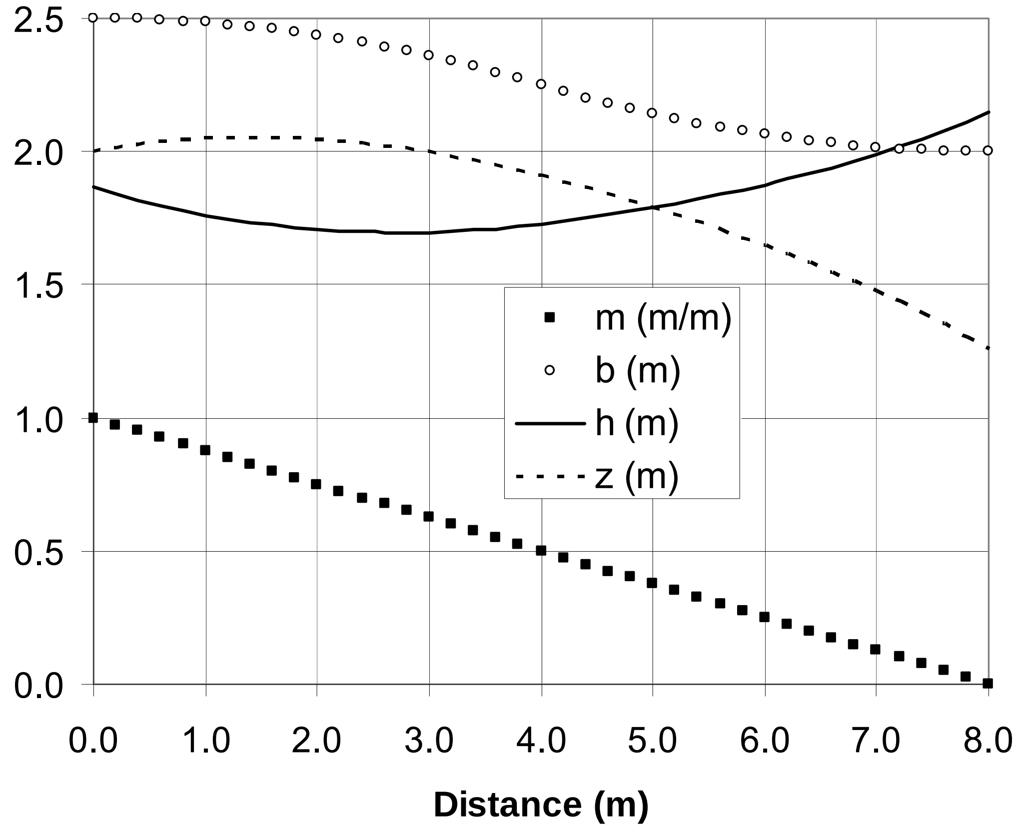
- Use Eq. 16 to calculate A as a function of x, then insert A into Eq. 17 and solve for h at each x value
- Using an arbitrary invert elevation of 2.0 m at the transition inlet, the relationship between depth of water, h, and canal bed elevation, z, across the 8-m transition is:

$$h = 3.87 - S_{ws}x - z(x) \tag{18}$$

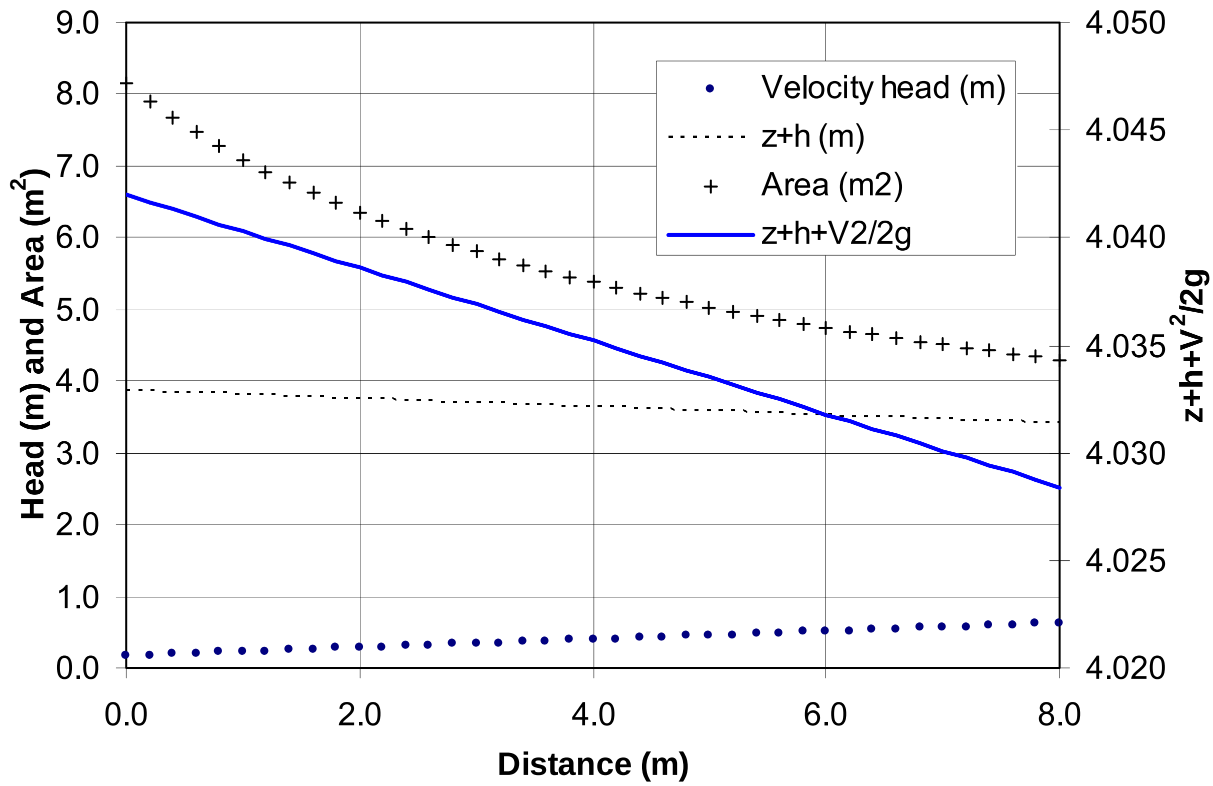
where $0 \le x \le 8$ m; and z = 2.0 at x = 0

- Once h is known, use Eq. 18 to solve for z, then go to the next x value

- The graph below shows the results of calculations using the above equations
- The numerical results are shown in the table below
- Note that the sum "z+h" decreases linearly through the transition (the water surface has a constant slope)
- Note that the velocity head increases linearly through the transition
- Note that the summation, $z+h+V^2/2g$, in the last column of the table (to the right) decreases linearly at the rate of 0.001718 m per meter of distance, x, as we have specified (see Eq. 3): the energy line has a constant slope



2.5
2.0
1.5
1.0
0.5
0.0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
Distance (m)
m (m/m)
b (m)
h (m)
z (m)



9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Head (m) and Area (m2)
4.050
4.045
4.040
4.035
4.030
4.025
4.020
z+h+V2/2g
0.0
2.0
4.0
6.0
8.0
Distance (m)
Velocity head (m)
z+h (m)
Area (m2)
z+h+V2/2g

- Note that the bed elevation, z, increases with x at first, then decreases to the final value of 1.26 m
- Note that the cross-sectional area decreases non-linearly from 0 to 8 m, but the inverse of the area squared increases linearly, which is why the velocity head also increases at a linear rate
- This transition design will produce a smooth water surface for the design flow rate of 15 m^3/s, but not for any other flow rate
- Below are the transition design results using an arbitrary invert elevation of 2.00 m at the inlet to the transition
- Why would you want to have a smooth water surface for the design flow rate in such a transition?

Transition Design Results

x (m)	m (m/m)	b (m)	A (m2)	h (m)	$V^2/2g$ (m)	z (m)	$z+h+V^2/2g$ (m)
0.0	1.000	2.500	8.165	1.87	0.17200	2.00	4.0420
0.2	0.975	2.499	7.911	1.84	0.18322	2.02	4.0417
0.4	0.950	2.498	7.680	1.82	0.19444	2.03	4.0413
0.6	0.925	2.494	7.467	1.80	0.20566	2.04	4.0410
0.8	0.900	2.490	7.272	1.78	0.21688	2.05	4.0406
1.0	0.875	2.484	7.091	1.76	0.22810	2.05	4.0403
1.2	0.850	2.478	6.922	1.75	0.23932	2.05	4.0400
1.4	0.825	2.469	6.766	1.73	0.25054	2.05	4.0396
1.6	0.800	2.460	6.619	1.72	0.26176	2.05	4.0393
1.8	0.775	2.449	6.482	1.72	0.27298	2.05	4.0389
2.0	0.750	2.438	6.352	1.71	0.28420	2.05	4.0386
2.2	0.725	2.424	6.230	1.70	0.29542	2.04	4.0383
2.4	0.700	2.410	6.115	1.70	0.30664	2.03	4.0379
2.6	0.675	2.394	6.007	1.70	0.31786	2.02	4.0376
2.8	0.650	2.378	5.903	1.70	0.32908	2.01	4.0372
3.0	0.625	2.359	5.805	1.70	0.34030	2.00	4.0369
3.2	0.600	2.340	5.712	1.70	0.35152	1.99	4.0366
3.4	0.575	2.319	5.623	1.70	0.36274	1.97	4.0362
3.6	0.550	2.298	5.538	1.71	0.37396	1.95	4.0359
3.8	0.525	2.274	5.456	1.72	0.38518	1.93	4.0355
4.0	0.500	2.250	5.379	1.73	0.39640	1.91	4.0352
4.2	0.475	2.226	5.304	1.74	0.40762	1.89	4.0349
4.4	0.450	2.203	5.233	1.75	0.41884	1.87	4.0345
4.6	0.425	2.181	5.164	1.76	0.43006	1.84	4.0342
4.8	0.400	2.160	5.098	1.78	0.44128	1.82	4.0338
5.0	0.375	2.141	5.034	1.79	0.45250	1.79	4.0335
5.2	0.350	2.123	4.973	1.81	0.46372	1.76	4.0332
5.4	0.325	2.106	4.914	1.82	0.47494	1.74	4.0328
5.6	0.300	2.090	4.857	1.84	0.48616	1.71	4.0325
5.8	0.275	2.076	4.802	1.86	0.49738	1.68	4.0321
6.0	0.250	2.063	4.748	1.88	0.50860	1.65	4.0318
6.2	0.225	2.051	4.697	1.90	0.51982	1.62	4.0315
6.4	0.200	2.040	4.647	1.92	0.53104	1.58	4.0311
6.6	0.175	2.031	4.599	1.94	0.54226	1.55	4.0308
6.8	0.150	2.023	4.552	1.96	0.55348	1.51	4.0304
7.0	0.125	2.016	4.506	1.99	0.56470	1.48	4.0301
7.2	0.100	2.010	4.462	2.02	0.57592	1.44	4.0298
7.4	0.075	2.006	4.419	2.05	0.58714	1.40	4.0294
7.6	0.050	2.003	4.378	2.08	0.59836	1.35	4.0291
7.8	0.025	2.001	4.337	2.11	0.60958	1.31	4.0287
8.0	0.000	2.000	4.298	2.15	0.62080	1.26	4.0284

Lecture 26

Energy Dissipation Structures

I. Introduction

- Excess energy should usually be dissipated in such a way as to avoid erosion in unlined open channels
- In this context, “excess energy” means excess water velocity which causes erosion and or scouring in an open channel
- Erosive damage can occur even at low flow velocities when the water is swirling, although at a slower rate
- Energy dissipation structures and other protective infrastructure are used at locations that are prone to erosion

II. Locations of Excess Energy

- What are the locations of excess energy in open channels?

  1. Channel constrictions (such as gates, weirs, others)
  2. Steep longitudinal bed slopes
  3. Drops in elevation

- Energy dissipation is almost always needed downstream of supercritical flow sections
- Energy dissipation may also be desired in lined channels
- Energy dissipation structures are typically located at:

  1. Sudden drops in bed elevation
  2. Downstream ends of channel branches, flumes and chutes (especially where discharging into earthen sections)
  3. Outlets of culverts and inverted siphons
  4. Structures causing supercritical flow (e.g. underflow gates)
  5. Structures causing downstream turbulence and eddies

III. Energy Dissipation Structure Types

- Most energy dissipation structures in open channels are based on:

  1. the creation of a stable ***hydraulic jump***
  2. head-on ***impact*** on a solid, immovable obstruction

- Both of these energy-dissipation structure classes can cause significant turbulence, reducing the hydraulic energy
- USBR publications state that the impact-type energy dissipation structures are more “efficient” than hydraulic-jump energy dissipaters (see Chapter VI in the *Design of Small Canal Structures* book by the USBR)

- Here, a more “efficient” design is defined as one which results in a smaller and or cheaper structure for the same energy dissipation capacity
- Design dimensions for energy dissipation structures are important because an inappropriate design can worsen an erosion and or scouring problem, as has been manifested in the field and in laboratory experiments
- There have been cases in which the installation of an energy dissipation structure caused more erosion than that which occurred without it

- The USBR has published design specifications for:

    1. baffled apron drops
    2. baffled pipe outlets
    3. vertical sleeve valve stilling wells

- The vertical sleeve structure is designed for energy dissipations at pipe-to-open channel interfaces (flow from a pipe into an open channel)
- All three of the above USBR energy dissipation structures are of the impact type
- In practice, many variations of baffled energy dissipation structures can be found

IV. Hydraulic Jumps for Energy Dissipation

- In open channels, a transition from subcritical to supercritical flow regimes results in very little localized hydraulic energy loss
- But, the opposite transition, from supercritical to subcritical, involves a hydraulic jump and energy loss
- The energy loss through a hydraulic jump can be significant, so jumps can be applied to energy dissipation applications in open channels

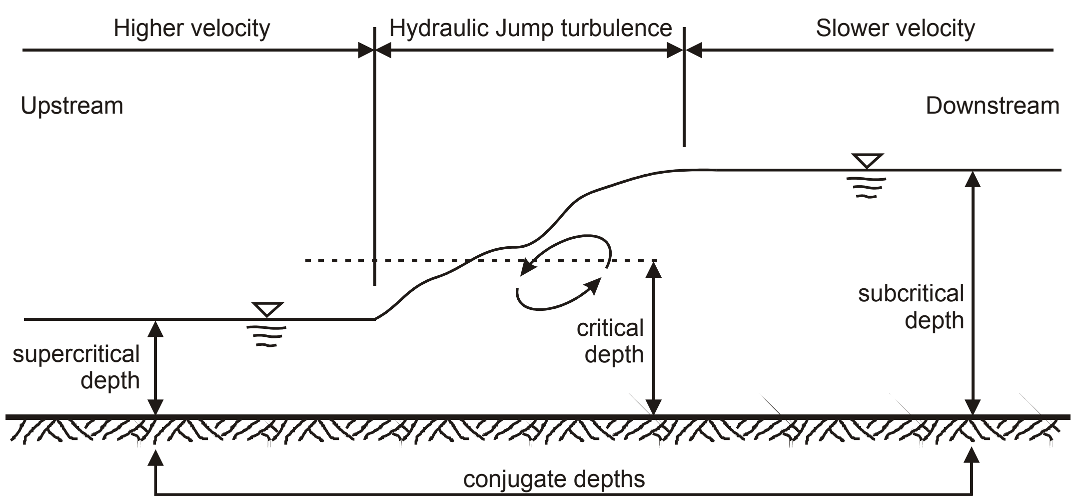


Figure 1. Side view of a hydraulic jump (flow is from left to right)

- The energy loss across a hydraulic jump (upstream to downstream) is equal to the difference in specific energy:

$$\Delta E = E_u - E_d \tag{1}$$

- Energy loss can be calculated based on measurements of depth and flow rate
- In designs, hydraulic jump energy loss is unknown, so you must apply the momentum function to determine a conjugate depth, then apply Eq. 1
- For a given Froude number, flow rate, and upstream depth:

    1. a rectangular cross section gives the least energy loss
    2. a triangular cross sections gives the greatest energy loss

- Cross sections with sloping sides provide more pronounced secondary currents (essentially orthogonal to the stream-wise direction), which also help dissipate hydraulic energy
- Thus, hydraulic jumps in trapezoidal cross sections give energy dissipation magnitudes somewhere between the extremes of rectangular and triangular cross-sectional shapes

- Some important hydraulic jump parameters, such as jump length and location, are determined experimentally, not theoretically
- Thus, design procedures for hydraulic jump energy dissipaters always include empirical equations
- The length of the "roller," L_r, is always less than the length of the jump, L_j

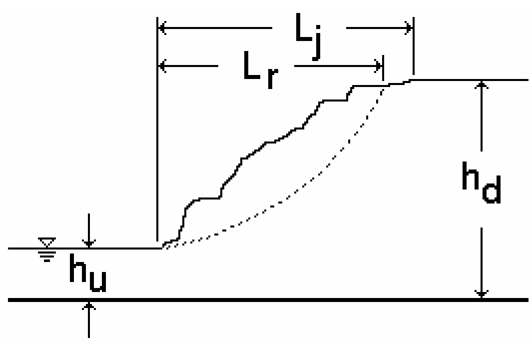


Figure 2. Another side view of a hydraulic jump (flow is from left to right)

- Small (weak) hydraulic jumps do not have a roller
- The length of a hydraulic jump in a rectangular cross section can be approximated by the following function:

$$L_j \approx 9.75 h_u \left(F_{us} - 1.0\right)^{1.01} \tag{2}$$

where F_{us} is the Froude number on the upstream side of the jump

- There are several classifications for hydraulic jumps

- Procedures exist to determine the type of jump that might occur in a given situation
- One classification groups hydraulic jumps into types “A” to “F”

V. Drop Spillways

- Drop spillways (also known as “drop structures”) are abrupt decreases in channel bed elevation with a downstream stilling basin, used to dissipate hydraulic energy in open channels
- Drop spillways often combine both hydraulic jump and impact features, although not all design situations are associated with a hydraulic jump
- Much research and experimentation has been done on drop spillways in efforts to adequately define design procedures and parameters
- Part of the reason for this is that, when incorrectly dimensioned, drop spillways can actually worsen an erosion problem in the downstream channel
- Most drop spillways have the following basic features:

    1. Inlet section
    2. Drop section
    3. Rectangular stilling basin
    4. Outlet section

- The flow through a drop spillway:

    1. Spills over a crest at a vertical headwall
    2. Falls on a horizontal (level) apron
    3. Impinges on floor blocks inside the basin
    4. Exits the stilling basin over an end sill

- Energy dissipation occurs via:

    1. Floor blocks
    2. End sill at DS of basin
    3. Turbulence in the “tail water”
    4. Hydraulic jump (in some cases)

- The following drop structure design elements are adapted principally from Donnelly & Blaisdell (1965) and involve mostly empirically-determined relationships

Stilling Basin Length

- How long does the stilling basin need to be for effective energy dissipation?
- According to experimental results, a series of simple iterative calculations are needed to answer this question
- Base dimensions on a design discharge and critical depth in a rectangular basin:

$$h_c = \sqrt[3]{\frac{(Q/b)^2}{g}} \tag{3}$$

where h_c is the critical depth of water in a rectangular open-channel section (m or ft); Q is the flow rate (m^3/s or cfs); b is the channel base width (m or ft); and g is the ratio of weight to mass (9.81 m/s^2 or 32.2 ft/s^2)

- In the present context, b represents the width of the stilling basin
- Note that Eq. 3 is based on the squared Froude number, set equal to unity
- Critical depth, h_c, may or may not actually occur in the stilling basin (if it does not, there will be no hydraulic jump), but in any case the value of h_c is still used in the following design calculations

Where the Nappe Hits the Floor

- Consider the following figure where flow goes from left to right (note that the coordinate origin is located at the brink of the overfall):

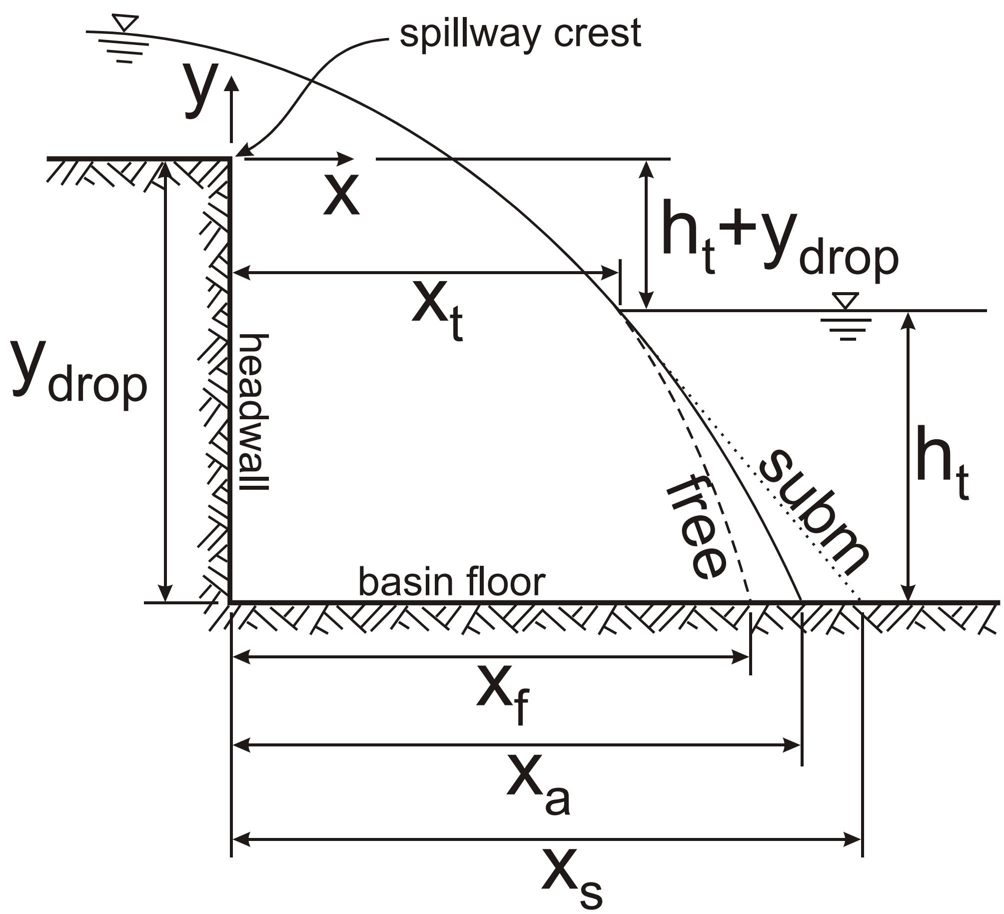


Figure 3. Side view of a drop spillway showing the free and submerged nappes (flow is from left to right)

- This is the equation for the "free nappe" is:

$$\frac{x_f}{h_c} = -0.406 + \sqrt{3.195 - 4.386\left(\frac{y_{drop}}{h_c}\right)} \tag{4}$$

where h_c is as defined in Eq. 3; and the other variables are defined in Fig. 3

- Note that $y_{drop} < 0$, and $h_c > 0$, in all cases
- This means that the ratio y_{drop}/h_c is always negative
- Thus, x_f increases with increasing absolute magnitude of y_{drop}
- Note also that x_f defines the upper nappe surface
- Each of the terms in Eq. 4 are dimensionless

- This is the equation for the "submerged nappe:"

$$\frac{x_s}{h_c} = \frac{0.691 + 0.228\left(\frac{x_t}{h_c}\right)^2 - \frac{y_{drop}}{h_c}}{0.185 + 0.456\left(\frac{x_t}{h_c}\right)} \tag{5}$$

again, where h_c is the critical depth, as defined in Eq. 3; x_t is defined by Eq. 6; and the other variables are defined in Fig. 3

- The variable x_t is the distance to where the upper nappe surface plunges into the tail water
- The nappe plunge location, x_t, is defined by an equation which is similar to Eq. 4 for the free nappe:

$$\frac{x_t}{h_c} = -0.406 + \sqrt{3.195 - 4.386\left(\frac{h_t + y_{drop}}{h_c}\right)} \tag{6}$$

where h_t is the tail water depth in the stilling basin, as seen in Fig. 3, and is referenced to the stilling basin floor

- The term in parentheses in Eq. 6 will be positive in those cases in which the tail water is above the spillway crest
- To avoid a negative square root term in Eq. 6, limit $(h_t + y_{drop})/h_c$ to a maximum of 0.7 when applying Eq. 6
- This is not a significant restriction because the required stilling basin length is not affected when:

$$\frac{h_t + y_{drop}}{h_c} > 0.67 \tag{7}$$

- All water depths (including h_c and h_t) are greater than zero
- All "x" values downstream of the spillway crest are greater than zero
- But all "y" values are negative below the spillway crest, positive above (this follows the convention introduced by Donnelly and Blaisdell), as seen in Fig. 3

- The average of the results from Eqs. 4 and 5 are used for drop structure design:

$$x_a = \frac{(x_f + x_s)}{2} \tag{8}$$

where the value of x_a is can be determined mathematically (preferred) or graphically, as shown in the following plot (Fig. 4) of the above equations

- The stilling basin length, L, will always be greater than x_a ($L > x_a$)

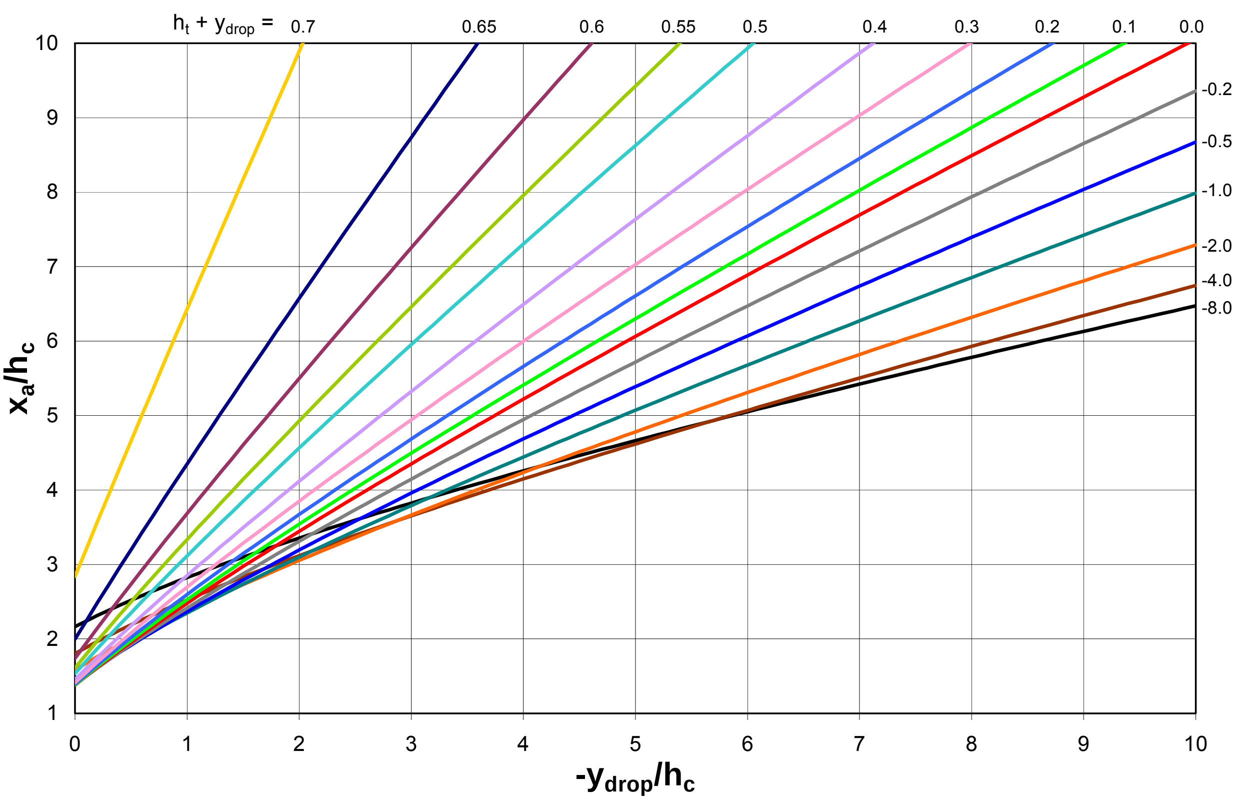


Figure 4. Plot of drop spillway design equations for determining the value of x_a

Floor Blocks

- Floor blocks are usually included in drop structure designs to help dissipate hydraulic energy before the flow exits the stilling basin
- There is a required minimum distance from x_a to the blocks so the flow becomes parallel to the floor before impinging on the upstream face of the blocks
- If the blocks are too close to the location of x_a, water splashes ("boils") off the blocks, and may go over the sides of the stilling basin

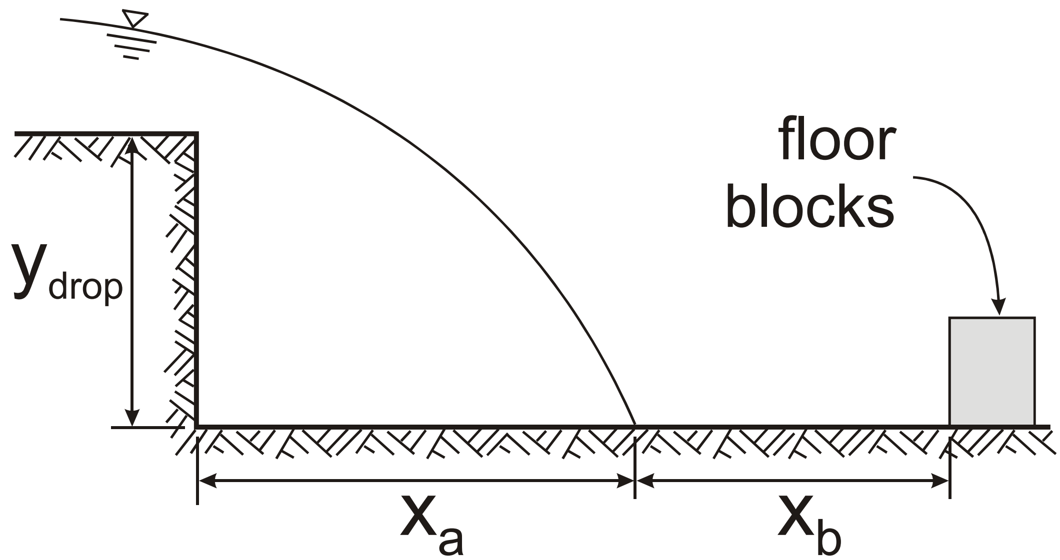


Figure 5. Side view of a drop spillway showing the recommended location of floor blocks (flow is from left to right)

- If $x_b < ½h_c$, the floor blocks are mostly ineffective in terms of energy dissipation
- Thus, for stilling basin design, let

$$x_b = 0.8 h_c \tag{9}$$

- The recommended height of the floor blocks is $0.8\,h_c$
- The recommended length of the floor blocks is 0.5 to $0.75\,h_c$
- The recommended width of the floor blocks is also 0.5 to $0.75\,h_c$
- Usually, make the floor blocks square (length = width)
- The upstream faces of the floor blocks should occupy from 50 to 60% of the basin width for effective energy dissipation
- Use equal spacing of floor blocks across the width of the stilling basin, but make slight adjustments as necessary to accommodate the total width, b

Longitudinal Sills

- Longitudinal sills are sometimes placed on the floor of the stilling basin, parallel to the basin walls, as seen in a plan-view (Fig. 6)

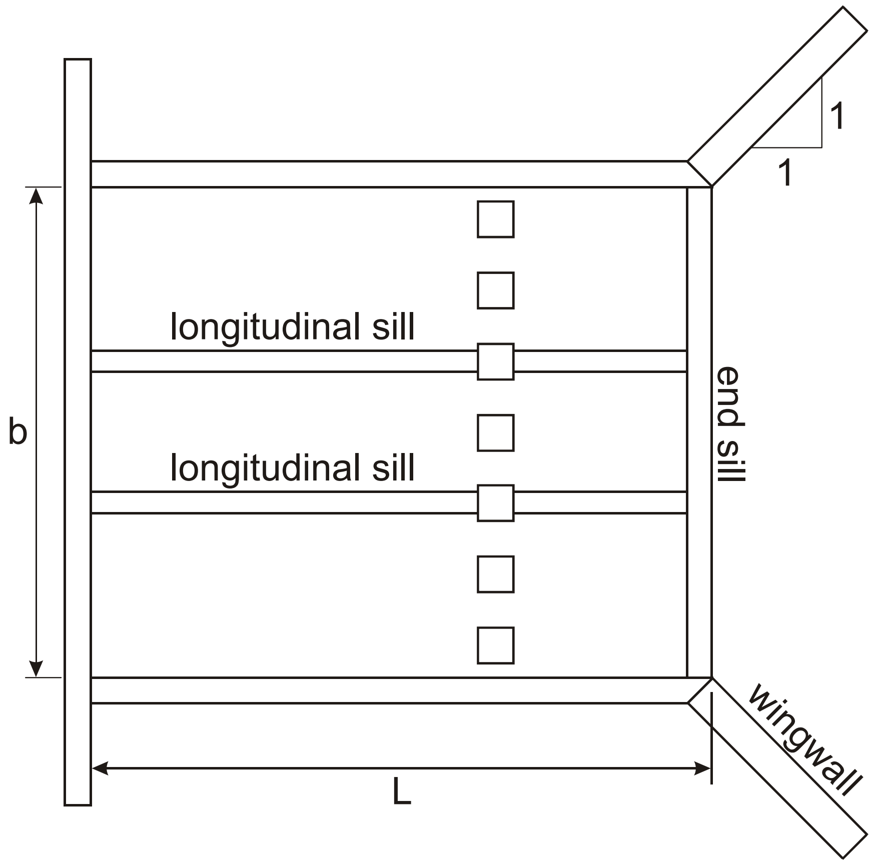


Figure 6. Plan view of a drop spillway showing longitudinal sills and square floor blocks (flow is from left to right)

- These sills are unnecessary if the floor blocks are properly:

1. Proportioned
2. Spaced

- Longitudinal sills are sometimes included in a design for structural reasons
- If they are included, they should pass through (not between) the floor blocks, as shown in Fig. 6

End Sill Location

- There is a minimum distance from the floor blocks to the end sill, which is located at the downstream end of the stilling basin
- This minimum distance is intended to maximize the energy dissipation from both the floor blocks and the end sill
- For design purposes, let:

$$x_c \geq 1.75h_c \tag{10}$$

where x_c is defined in Fig. 7

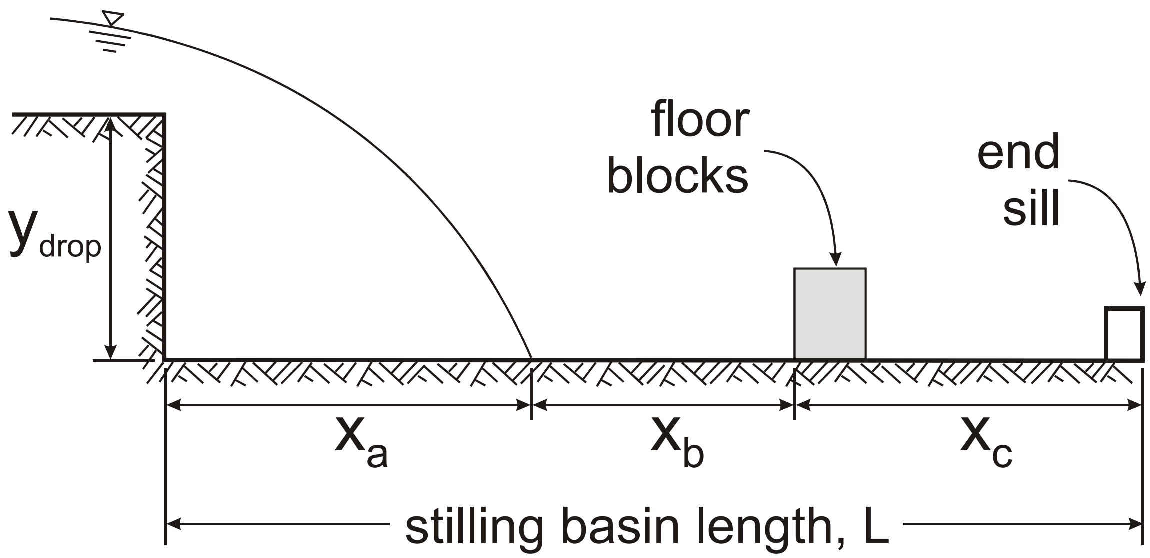


Figure 7. Side view of a drop spillway showing the location of the end sill and the total basin length (flow is from left to right)

- However, in most design cases, x_c is set equal to $1.75\,h_c$
- In other cases, it may be necessary to provide a longer stilling basin length to accommodate the site-specific conditions

Stilling Basin Length

- In summary, the stilling basin length is:

$$L = x_a + x_b + x_c \tag{11}$$

or,

$$L = x_a + 2.55h_c \tag{12}$$

End Sill Height

- The end sill height is:

$$y_{end} = 0.4h_c \tag{13}$$

where y_{end} is the end sill height, as shown in Fig. 8

- Observe that $y_{end} > 0$ in all cases

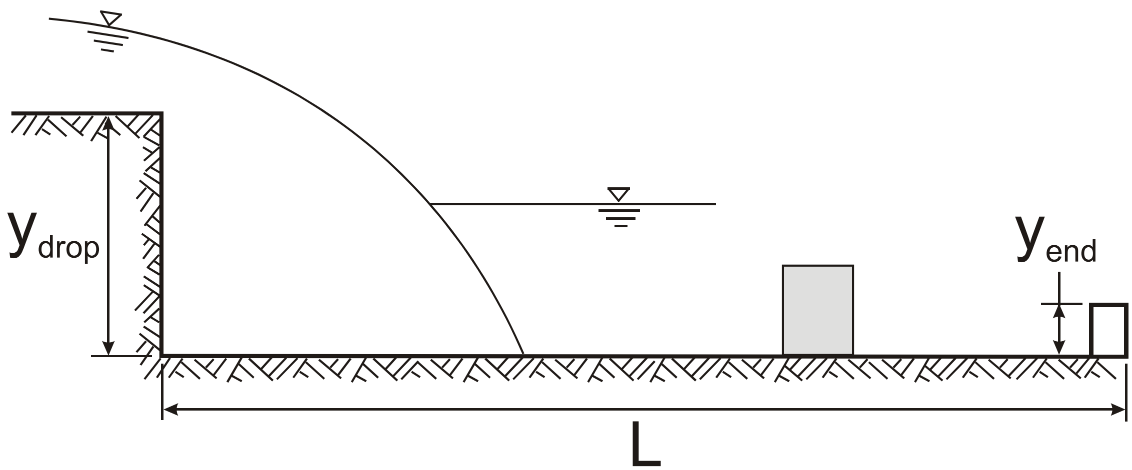


Figure 8. Side view of a drop spillway showing the height of the end sill

- The top of the end sill should be at or slightly above the invert (bottom) elevation of the downstream channel (or downstream channel transition), as shown in the following figure

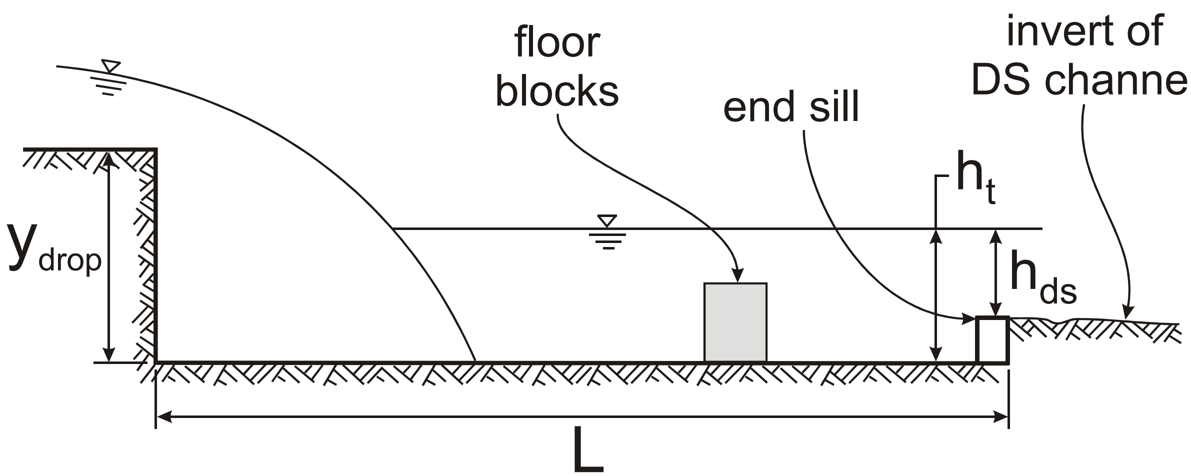


Figure 9. Side view of a drop spillway showing the downstream channel invert

Tail Water Depth

- A minimum tail water depth is required in the design of a drop spillway
- To prevent downstream scouring, the tail water depth should be "about the same" as the depth in the stilling basin
- If this is true, the hydraulic jump is submerged inside the basin length
- For design, let

$$h_t \geq 2.15 h_c \tag{14}$$

where h_t is from the downstream water surface to the stilling basin floor, as seen in Fig. 9

- In most drop spillway designs, let

$$h_t = 2.15 h_c \tag{15}$$

- Note that the recommended ratio of h_t/h_c (= 2.15) is independent of the drop height, y_{drop}
- There may be a hydraulic jump up to the tail water depth, in some cases

- If the tail water depth, h_t, is too low (i.e. $h_t < 2.15\ h_c$)

    1. Increase the stilling basin width, b, which will decrease h_c; or,
    2. Increase $|y_{drop}|$, deepening the stilling basin floor

- An increase in $|y_{drop}|$ and or b may increase construction and maintenance costs
- An increase in $|y_{drop}|$ also increases the end sill height

- Note that the depth from the spillway crest to the stilling basin floor can be increased not only by deepening the basin floor, but also by providing a weir at the overfall location
- This solution can be convenient for the drop structure design, but care must be taken with the freeboard in the upstream channel because increasing the spillway crest height will result in a corresponding upstream water depth increase

- How to determine the value of tail water depth, h_t?
- If uniform flow conditions prevail in the downstream channel, use the Manning or Chezy equation to calculate h_{ds}
- Otherwise, apply gradually-varied flow analysis for subcritical conditions to determine h_t
- Thus, h_{ds} is calculated independently of the drop structure dimensions
- Finally,

$$h_t = h_{ds} + y_{end} \tag{16}$$

Side and Wing Walls

- The tops of the sidewalls should be at least 0.85dc above the tail water surface
- Wing walls are DS of the end sill, at 45º angle, and with a top slope of 1:1
- Wing wall length depends on the width of the DS channel section
- Wing walls are not necessary if the DS channel is a lined canal

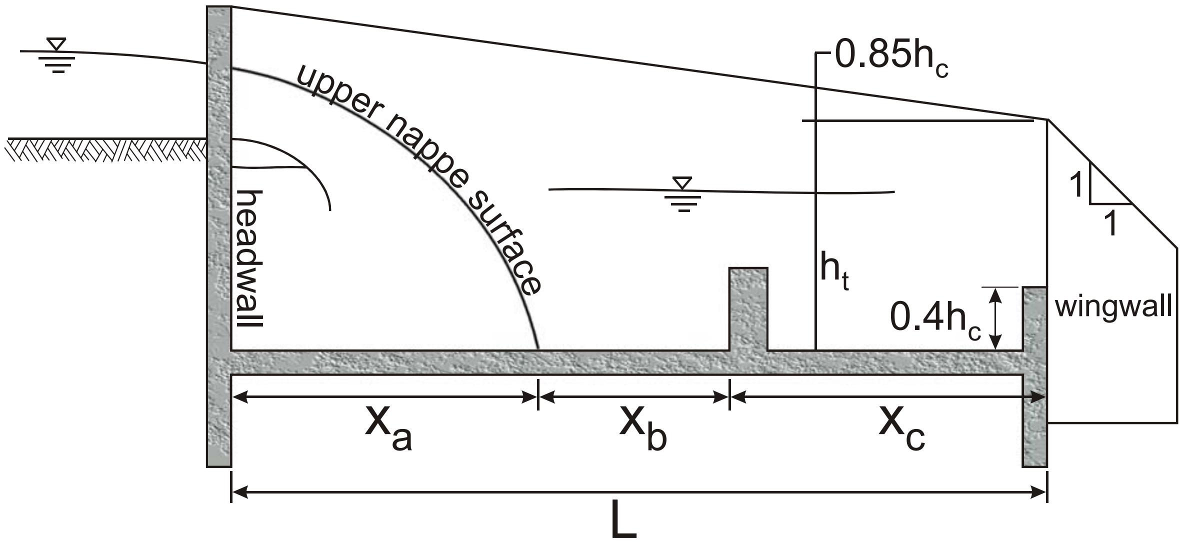


Figure 10. Side view of a drop spillway showing footings and wing walls

Drop Spillway Construction

- Construction is usually of steel-reinforced concrete
- The basin floor should be level, both longitudinally and transversely
- Upstream and or downstream channel transitions may be needed
- Concrete floor and wall thickness is usually 5-8 inches (12-20 cm)
- The depth of the concrete footings should be 2-3 ft for most designs in small- and medium-size channels
- May need riprap or other form of erosion protection upstream and downstream of the drop structure where earthen channels exist
- The approach channel bed elevation should be the same as the spillway crest elevation at the headwall
- The required headwall height at the crest location depends on the expected upstream depth at the design discharge, plus freeboard
- The side walls slope down from the top of the headwall to the top of the wing walls at the end sill location (see Fig. 10)
- In some cases it is convenient and appropriate to make the stilling basin width, b, equal to the width of the upstream or downstream channel (may eliminate the need for transitions)

VI. Drop Spillway Design Procedure

- The best design procedure depends on the given site conditions and requirements for a particular location
- However, in general, the following procedure can be applied

    1. Define the total available bed elevation change at the proposed drop structure location.
    2. Define the design discharge, Q.

3. Calculate h_{ds} based on the downstream channel conditions (cross section, bed slope, roughness) using a uniform-flow equation, or the gradually-varied flow equation, as appropriate.
4. Choose a reasonable value for the stilling basin width, b.
5. Calculate critical depth in the stilling basin, h_c (Eq. 3).
6. Calculate y_{end} (Eq. 13).
7. Calculate h_t (Eq. 16).
8. Is Eq. 14 satisfied? If not, use Eq. 14 to recalculate the stilling basin width, b, then go back to Step 5.
9. Calculate y_{drop} based on the total available bed elevation change and y_{end} (see Fig. 9), where y_{drop} should be less than zero. If $y_{drop} \geq 0$, consider raising the spillway crest by including a weir.
10. Calculate x_f (Eq. 4).
11. Calculate x_t (Eq. 6).
12. Calculate x_s (Eq. 5).
13. Calculate x_a (Eq. 8).
14. Calculate x_b (Eq. 9).
15. Calculate x_c (Eq. 10).
16. Calculate the stilling basin length, L (Eq. 11). If the length is not acceptable, adjust b and go back to Step 5.
17. Calculate the floor-block dimensions and spacing.
18. Calculate the head wall height based on the upstream depth at Q_{max}, plus freeboard
19. Calculate the height of the wing walls at the end sill ($0.85\, h_c$).
20. Prepare side view and plan view drawings of the drop spillway structure.

VII. Example Drop Spillway Design

Given:

- The design flow rate is $Q_{max} = 9.0\ m^3/s$
- There is a drop of 2.25 m in channel bed invert at this location
- The upstream channel is earthen, as is the downstream channel
- The upstream channel has a base width of approximately 5 m
- The downstream channel has an approximately trapezoidal cross section: base width is b = 5 m, side slopes have m = 1.64, and the bed slope is $S_o = 0.000112$
- For the downstream channel, use a Manning n value of 0.019
- The depth in the downstream channel is at the uniform flow depth at Q_{max}

Solution:

1. The total available bed elevation change is given as 2.25 m.
2. The design discharge is given as 9 m^3/s
3. Uniform flow conditions are expected in the downstream channel. Using the ***ACA*** program, the normal depth in the downstream channel is 1.80 m at the design capacity of 9 m^3/s, with a Manning roughness of n = 0.019.
4. Try a stilling basin width of b = 5 m, matching the upstream channel base width

5. Critical depth in the stilling basin (Eq. 3):

$$h_c = \sqrt[3]{\frac{(9/5)^2}{9.81}} = 0.691\,\text{m}$$

6. The end sill height will be (Eq. 13):

$$y_{end} = 0.4(0.691) = 0.276\text{ m}$$

7. The tail water depth will be (Eq. 16):

$$h_t = 1.80 + 0.276 = 2.076\text{ m}$$

8. Check to see if Eq. 14 is satisfied:

$$h_t = 2.076\text{ m}$$
$$2.15h_c = 2.15(0.691) = 1.486\text{ m}$$

Thus,

$$h_t > 2.15h_c$$

and Eq. 14 is satisfied.

9. The value of y_{drop} will be:

$$y_{drop} = -2.25 - 0.276 = -2.526\text{ m}$$

Notice that y_{drop} is negative (as required).

10. Calculate x_f (Eq. 4):

$$x_f = 0.691\left[-0.406 + \sqrt{3.195 - 4.386\left(\frac{-2.526}{0.691}\right)}\right] = 2.75\text{ m}$$

11. Calculate x_t (Eq. 6):

$$x_t = 0.691\left[-0.406 + \sqrt{3.195 - 4.386\left(\frac{2.076 - 2.526}{0.691}\right)}\right] = 1.42\text{ m}$$

12. Calculate x_s (Eq. 5):

$$x_s = 0.691\left[\frac{0.691 + 0.228\left(\frac{1.42}{0.691}\right) + \left(\frac{2.526}{0.691}\right)}{0.185 + 0.456\left(\frac{1.42}{0.691}\right)}\right] = 3.27 \text{ m}$$

13. Calculate x_a (Eq. 8):

$$x_a = \frac{(2.75 + 3.27)}{2} = 3.01 \text{ m}$$

14. Calculate x_b (Eq. 9):

$$x_b = 0.8(0.691) = 0.553 \text{ m}$$

15. Calculate x_c (Eq. 10):

$$x_c = 1.75(0.691) = 1.209 \text{ m}$$

16. Calculate the stilling basin length (Eq. 11):

$$L = x_a + x_b + x_c = 4.77 \text{ m}$$

Notice that L < b in this design.

17. Floor block dimensions and spacing:

Floor block height: $0.8\,h_c$ = 0.8 (0.691) = 0.55 m
Floor block width: $0.5\,h_c$ = 0.8 (0.691) = 0.35 m
Floor block length: $0.5\,h_c$ = 0.8 (0.691) = 0.35 m

At 50% basin width, the required number of floor blocks is:

$$N = \frac{0.5b}{0.35} = \frac{2.5}{0.35} = 7.1 \text{ blocks}$$

Round up to N = 8 blocks, giving a percent area of 56%. Placing a block against each side wall of the stilling basin, the uniform spacing between the blocks will be:

$$\text{spacing} = \frac{b - 0.35N}{N - 1} = 0.314 \text{ m}$$

18. The height of the headwall, from the basin floor to the top, should be:

$$-y_{drop} + 1.1\left(h_{ds}\right) = 2.526 + 1.1(1.80) \approx 4.50 \text{ m}$$

where the coefficient 1.1 is to allow for freeboard.

19. Wing wall height at end sill:

$$0.85h_c = 0.85(0.691) = 0.587 \text{ m}$$

20. Design notes:

- Complete the design by specifying wall & floor thickness
- Specify the depth of footings (see Fig. 10)
- Specify the length of the wing walls (they should be at least long enough to meet the side slopes of the downstream channel)
- Make design drawings (side and plan views)
- A more iterative design approach could be used to minimize the size (b x L) of the drop spillway, thereby reducing its cost

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Lecture 27
Protective Structures

I. Introduction

- Protective structures include energy dissipation and erosion control structures, structures to divert excess water to prevent over-topping of canals, and others
- In this context, protective structures are for the canals and other infrastructure, and not for the protection of animals and people
- For example, there should be a wasteway weir or other structure in a canal, upstream of an in-line pumping plant -- the pump(s) could shut off unexpectedly, possibly causing over-topping of the canal upstream
- An inverted siphon could become clogged, or a landslide could block the flow in the canal, also causing the canal to overflow on the upstream side
- Of course, canal over-topping can also occur due to design deficiencies, construction problems, and operational error
- This means that spillway structures are generally designed to carry the full flow of the canal
- Side spillways sometimes have radial or vertical slide gates to facilitate dewatering of the channel in emergencies, for sediment removal, and other reasons
- Some in-line gate structures in canals have fixed-crest side weirs to allow for water to pass downstream in the event of an operational error
- The flow from spillway structures is generally directed into a natural channel which can safely carry the maximum spill rate away from the canal

II. Wasteway Weirs

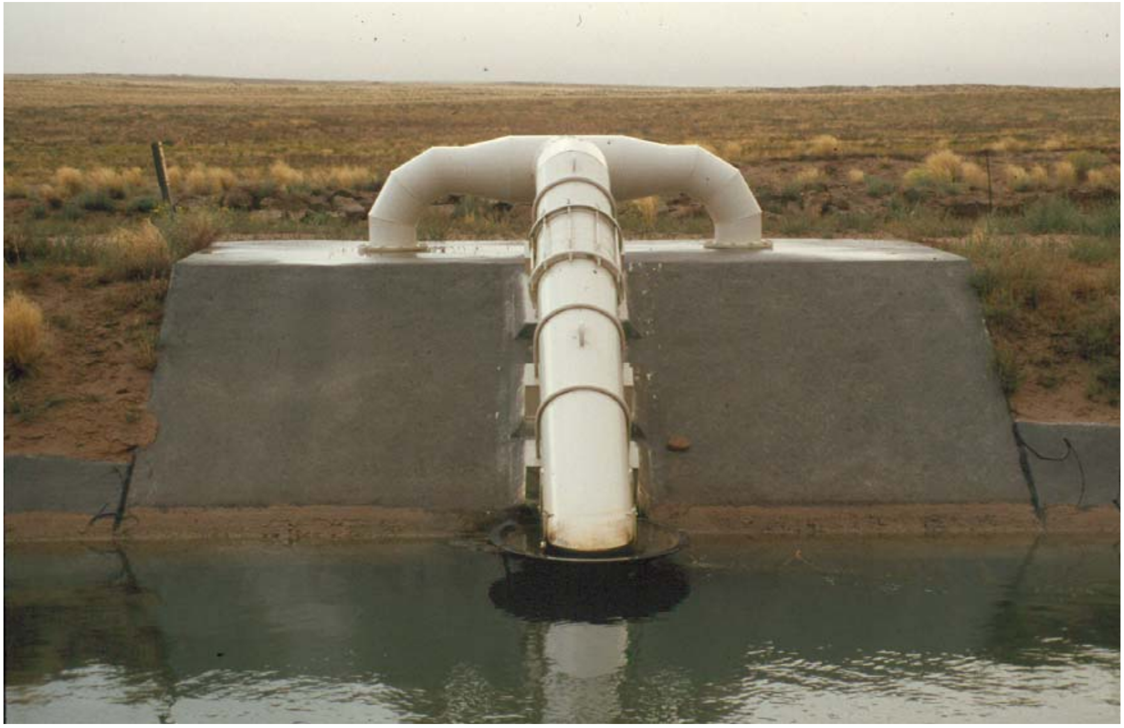
- A wasteway weir is a sharp- or blunt-crested weir located along one bank of the canal

- The weir may be designed to accept “stop logs” to allow for changes in the operating level of the canal
- The crest of the weir is equal to the maximum operating level of the canal, taking into consideration that a higher water surface elevation will result in the canal for the full design discharge
- The wider the weir is, the less difference in water surface elevation between impending spill and full-flow spill through the structure
- Wasteway weirs are located at places where the canal would be most likely to overtop in the event of an operational error or the clogging of a flow control structure (e.g. cross regulator):

    1. Upstream of an inverted siphon entrance
    2. Upstream of a gate or weir structure
    3. Upstream of a pumping station

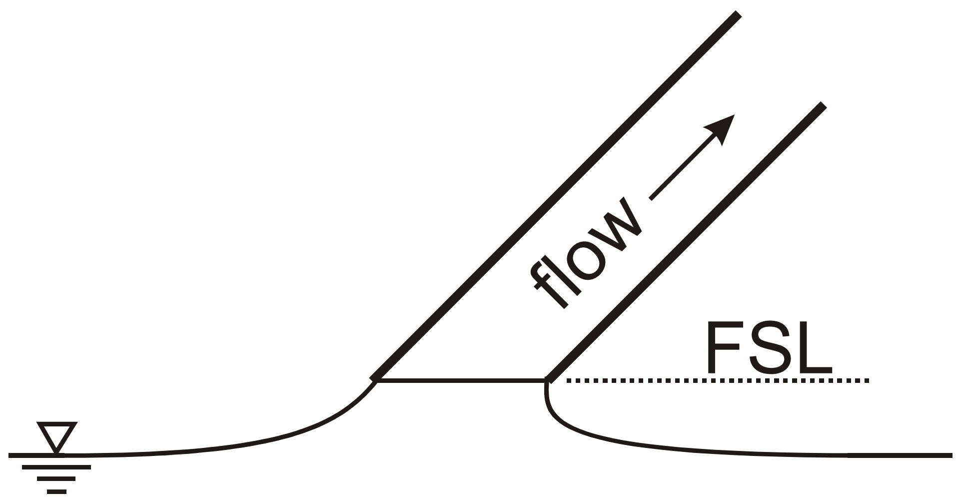
III. Siphon Spillways

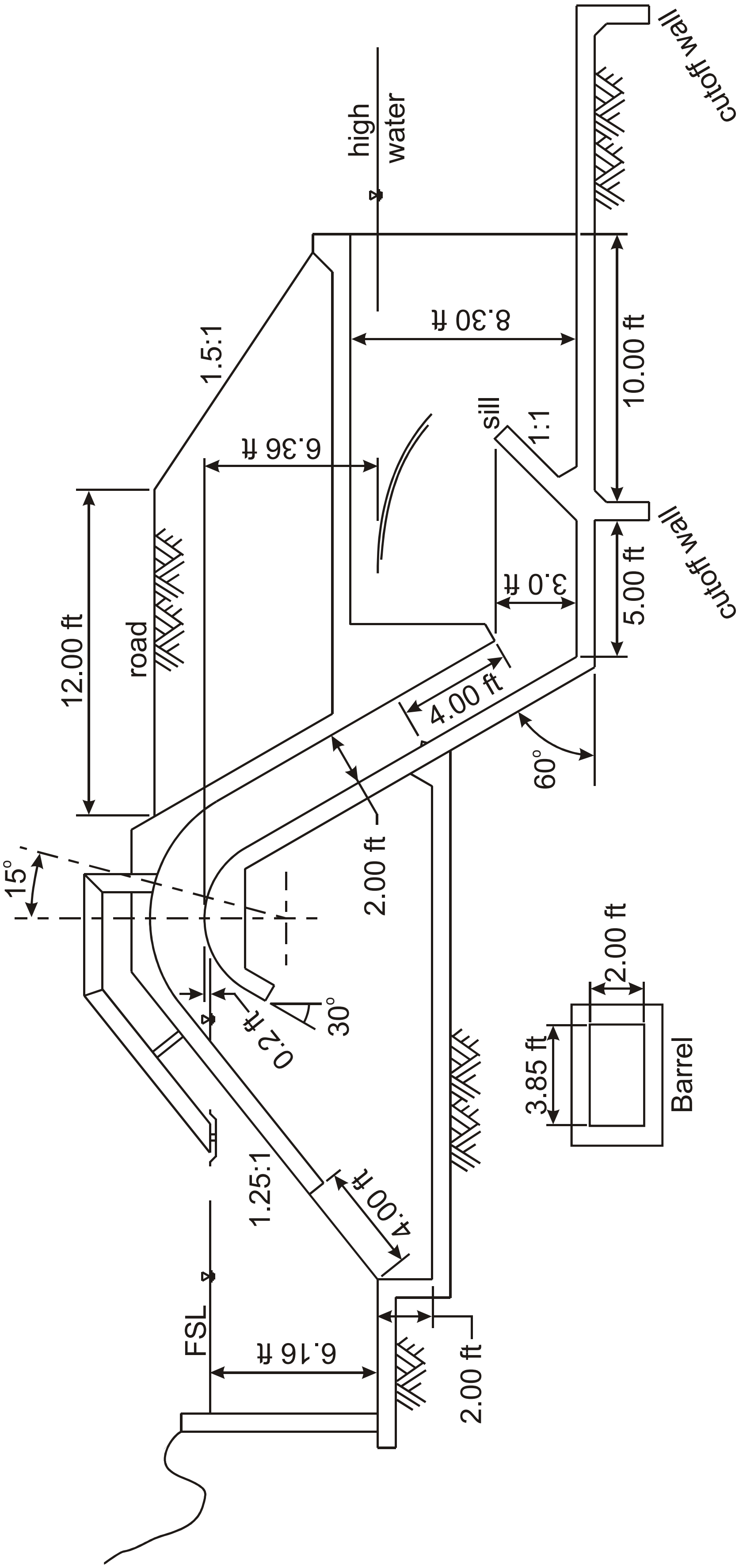
- As opposed to inverted siphons, *siphon spillways* operate under negative pressures (below atmospheric)
- Once a siphon spillway is primed, water will continue to flow through the structure as long as the downstream water elevation is lower than that in the canal, or until suction is broken by other means
- Siphon spillways are generally more expensive to build than side spillway weirs, but for the same discharge they require a much smaller width
- Siphon spillways can discharge more water than a weir for a small increase in upstream water surface elevation
- As the water level in the canal increases, the siphon spillway acts like a weir
- As the water level continues to increase, the siphon will become “primed” and operate under full pipe flow conditions
- The USBR design calls for a crest elevation of about 0.2 ft above the normal water surface (or full supply level) in the canal
- Note that the inlet to the siphon can serve as a sediment trap, requiring periodic manual cleaning
- Also note that it is impractical to change the FSL of the canal once the siphon spillway is installed – with side spill weirs, you can always raise the level, if needed
- See Section 4-14 of the USBR *Small Canal Structures* book



Air Vent for Breaking Suction

- An air vent is located just downstream of the top of the siphon spillway to break suction when the upstream water level in the canal drops below a certain level
- This location is about 15° (from vertical) downstream of the top of the siphon pipe, as shown in Figure 4-17 of the USBR *Small Canal Structures* book
- The upstream end of the vent is open at the normal water surface level (or FSL) of the canal
- A potential operating problem with this type of structure is that when the suction is broken, the discharge will suddenly cease, and this can cause surges in the canal
- A pan can be attached to the upstream end of the vent to help prevent the vent from acting as a siphon itself, possibly causing the water level in the canal to drop below full supply level
- The pan helps reduce the amount of water level fluctuation in the canal




high water
cutoff wall
1.5:1
8.30 ft
10.00 ft
sill
1:1
6.36 ft
12.00 ft
road
3.0 ft
5.00 ft
cutoff wall
4.00 ft
60°
2.00 ft
15°
0.2 ft
30°
2.00 ft
3.85 ft
Barrel
1.25:1
4.00 ft
FSL
6.16 ft
2.00 ft

Deflector at the Downstream Side of the Siphon

- A small angled deflector can be installed in the downstream end of the siphon to help direct water up to the top of the barrel under non-full-flow conditions
- This helps to mix air and water and cause the siphon to prime to full flow quicker
- The roof of the siphon structure at the outlet should be above the expected downstream water surface elevation
- This helps to evacuate air from the siphon

IV. Design of a Siphon Spillway

This design example is adapted from an example given by the USBR (1978)

Given:

Suppose there is a canal with a design discharge of 120 cfs in which an in-line pump station is used to lift the water up to a downstream reach. The canal is trapezoidal in cross-section, with a base width of 8.0 ft, side slope of 1½:1 (H:V).
The canal is at an elevation of about 6000 ft above msl. The available head across the siphon spillway, H, is 6.0 ft.

Solution:

(a) Preliminary Calculations According to USBR recommendations, use a ratio R_{CL}/D of 2.0. Assume an initial value of D = 2.0 ft. Then, the radius of the centerline of the siphon is R_{CL} = 4.0 ft, and the radius of the siphon invert is R_C = 4.0 ft - ½(2.0) = 3.0 ft. The radius of the top of the siphon is R_S = 4.0 ft + ½(2.0) = 5.0 ft.

(b) Full Pipe Discharge Estimate the full pipe discharge by assuming: (1) orifice flow; and (2) a discharge coefficient of 0.65. This will give a flow rate per "unit width" of the barrel:

$$q = C_d D\sqrt{2gH} = (0.65)(2.0)\sqrt{2(32.2)(6.0)} = 25.6 \text{ cfs/ft} \tag{1}$$

Note that this is an estimate, using an assumed C_d value, and D instead of area.

(c) Maximum Possible Discharge Now, estimate the discharge per unit width according to the "vortex" equation, which takes into account the atmospheric pressure available to "push" the water up over the invert of the siphon crest. The so-called vortex equation looks like this:

$$q_{max} = R_C\sqrt{2g(0.7h)}\,\ln\left(\frac{R_S}{R_C}\right) \tag{2}$$

where h is the available atmospheric pressure head, based on the density of water; and q_{max} is the flow rate per foot of barrel width (cfs/ft).

The change in average atmospheric pressure with elevation can be approximated by the following linear relationship:

$$P_{atm} \approx (33.9 - 0.00105\,\text{Elev})/2.31 \tag{3}$$

where P_{atm} is in psi and Elev is the elevation above mean sea level, in ft. For 6000-ft of elevation, the atmospheric pressure is about 11.9 psi, or h = 27.6 ft of head (water).

Then, q_{max} is equal to:

$$q_{max} = (3.0)\sqrt{2g(0.7)(27.6)}\,\ln\left(\frac{5.0}{3.0}\right) = 54.1\ \text{cfs/ft} \tag{4}$$

This means that the previously-calculated unit discharge of 25.6 cfs/ft (from the orifice equation) is acceptable. If q were greater than q_{max}, it would have been necessary to decrease H or change R_{CL}.

(d) Barrel Width The width of the barrel is determined as:

$$b = \frac{Q}{q} = \frac{120\ \text{cfs}}{25.6\ \text{cfs/ft}} = 4.7\ \text{ft} \tag{5}$$

This value could be rounded up to provide a margin of safety, but we will leave it at 4.7 ft (at least for now).

(e) Vent Diameter The diameter of the siphon breaker pipe, D_p, should be such that the cross-sectional area is at least 1/24th of the cross-sectional area of the barrel (according to USBR guidelines). This gives an area of (2.0)(4.7)/24 = 0.39 ft^2. The corresponding ID is 0.70 ft, or 8.5 inches. Thus, use whatever pipe size would be closest to this diameter (perhaps 9-inch nominal size), noting that steel pipe is usually used (for strength).

(f) Outlet Sill Height The height of the deflector sill at the outlet of the siphon is given as 1.5D, or 3.0 ft in our case.

(g) Outlet Ceiling Height The ceiling of the outlet is defined as h_2. Referring to Figure 4-17, this is given by:

$$h_2 = 1.5D + E_{critical} + 1.0\ \text{ft} \tag{6}$$

where $E_{critical}$ is the specific energy for critical flow conditions, in feet. This is how the dimensions are defined in the USBR design procedures.

The width of the downstream pool will be the same as the barrel width, or 4.7 ft, and the section will be rectangular. Critical depth for the design discharge is:

$$y_c = \sqrt[3]{\frac{Q^2}{gb^2}} = \sqrt[3]{\frac{(120)^2}{(32.2)(4.7)^2}} = 2.7 \text{ ft} \tag{7}$$

The velocity is Q/A = 120/(2.7*4.7) = 9.5 fps, so the velocity head is:

$$\frac{V^2}{2g} = \frac{(9.5)^2}{2(32.2)} = 1.4 \text{ ft} \tag{8}$$

Then, $E_{critical}$ is 2.7 + 1.4 = 4.1 ft. And, h_2 = 1.5(2.0) + 4.1 + 1.0 = 8.1 ft.

(h) Other Design Details The inlet structure from the canal can be designed with a height along the side slope of 2D (minimum). The inlet structure should provide a minimum submergence of $1.5h_v$ + 0.5 ft, where h_v is the velocity at the inlet, and the inlet area is at least 2Db.

Lecture 28

Safety Considerations

"Here lies one whose name was writ in water"
John Keats (1821)

I. Introduction

- Canals and related infrastructure can be very dangerous to people and animals
- People drown in canals, inverted siphons and other facilities every year
- One of the most important considerations is the number of people that might be exposed to dangerous facilities (canals, siphons, etc.) at a given site
- It is difficult to determine generally applicable design standards for safety features because of many factors that should be considered
- Note that design engineers can be held legally liable for mishaps & accidents

II. USBR Hazard Classifications

- The kind of safety protection applied to a given canal and canal structures normally depends on the safety classification:

Class A	Canals nearby or adjacent to schools and recreational areas, or where children are often present
Class B	Canals nearby or adjacent to urban areas, county roads or highways that would have frequent public access or recreational use
Class C	Canals nearby or adjacent to farms, county roads or highways that would have a possibility for children to occasionally be present
Class D	Canals far from roads and houses that would usually not be visited by the public
Class E	Canals that might be a hazard to domestic animals
Class F	Canals that would be very hazardous to large game animals

III. Safety Devices for Canals

1. Preventative

- Fencing
- Sign Posting
- Guard Railings and other Barriers

2. Escape Devices (usually only upstream of the hazardous location)

- Safety Nets
- Ladders
- Cables with Floats
- Pipe Inlet Racks

BIE 5300/6300

Fall Semester 2004
Exam #1 – 19 Oct 04

Show your work neatly on this or separate pages. Show units for all calculated values. Make note of any important assumptions.

Name______________________________

1. **(20 pts)** A Cutthroat flume needs to be installed in an irrigation canal at a location with a maximum flow rate of 50 cfs. The canal is rectangular in cross section, with a base width of 6.0 ft, and a longitudinal bed slope of 0.00065 ft/ft. Normal depth at the maximum flow rate is 2.40 ft.

   (a) Select a standard Cutthroat flume size (specify L & W in feet).
   (b) Will the canal cross section need to be modified to accommodate the flume? Be specific.
   (c) Determine the minimum height (in feet) of the flume floor with respect to the channel bed such that the flume operates under free-flow conditions at Q_{max}.

2. **(15 pts)** Current metering data is given below for an open channel with a top width of 3.03 m. Complete the calculations to estimate the total flow rate in m^3/s.

distance from edge (m)	depth (m)	depth fraction	velocity (m/s)			mean depth (m)	width (m)	area (m^2)	flow rate (m^3/s)
			at point	mean in vertical	mean in section				
0.05	0.00	n/a	10%	0.019					
					0.105	0.090	0.400	0.036	0.0038
0.45	0.18	0.6	0.190	0.190					
					0.201	0.375	0.800	0.300	0.0602
1.25	0.57	0.2	0.208	0.211					
		0.8	0.214		0.215	0.700	0.500	0.350	0.0751
1.75	0.83	0.2	0.211	0.218					
		0.8	0.225		0.221	0.900	0.250	0.225	
2.00	0.97	0.2	0.218	0.224					
		0.8	0.229		0.220	1.010	0.500	0.505	
2.50	1.05	0.2	0.210	0.216					
		0.8	0.222		0.213	1.050	0.500		
3.00	1.05	0.2	0.208	0.210					
		0.8	0.212						
3.08	1.05	n/a							
						Totals:			

3. **(15 pts)** You need to design a suppressed, rectangular sharp-crested weir for a rectangular channel with a base width of 2.10 m, and a Q_{max} of 4.5 m^3/s. A normal depth of 0.678 m at Q_{max} was measured in the field. Manning roughness is 0.014. Design the weir, specifying crest height, P, provided the conditions are appropriate.

4. **(25 pts)** A BCW, operating under free-flow conditions, has a flow rate of 17.00 m^3/s when h_u (referenced from the sill) is 1.813 m. At this same flow rate, h_d (referenced from the downstream bed elevation) is 2.661 m. The upstream and throat sections are rectangular with a width of 3.75 m. The sill height is $z_u = z_d = 1.15$ m. This BCW has a downstream ramp with a 6:1 slope.

   (a) Determine the upstream specific energy, E_u.
   (b) Determine h_c (referenced from the sill) for this flow rate.
   (c) Estimate the hydraulic head loss in the diverging section, $(h_f)_{ds}$.

5. **(25 pts)** A circular pipe has a circular, sharp-edged orifice plate (D_2 = 5.50 inches), centered in the pipe cross section, which has an inside diameter of D_1 = 7.90 inches. Taps at D_1 (upstream) and ½D_1 (downstream) are connected to a manometer with mercury, from which a head differential of 186 mm is measured. Water temperature is 13°C. Ignoring thermal expansion adjustments:

   (a) Determine the ratio β.
   (b) Determine the kinematic viscosity, ν.
   (c) Determine the discharge coefficient, C_d.
   (d) Determine the upstream Reynolds number, R_e.
   (e) Determine the flow rate, Q.

Solutions:

1. **(20 pts)** A Cutthroat flume needs to be installed in an irrigation canal at a location with a maximum flow rate of 50 cfs. The canal is rectangular in cross section, with a base width of 6.0 ft, and a longitudinal bed slope of 0.00065 ft/ft. Normal depth at the maximum flow rate is 2.40 ft.

(a) Select a standard Cutthroat flume size (specify L & W in feet).

For 50 cfs, select the W = 3 ft, L = 9 ft Cutthroat flume from the table in the lecture notes, with Q_{max} = 56.9 cfs.

(b) Will the canal cross section need to be modified to accommodate the flume? Be specific.

B = W + L/4.5 = 3 + 9/4.5 = 5 ft. Thus, no modifications to the channel section (which is 6-ft wide) except for the inclusion of headwalls (US & DS) to direct all flow through the flume.

(c) Determine the minimum height (in feet) of the flume floor with respect to the channel bed such that the flume operates under free-flow conditions at Q_{max}.

From Cutthroat flume table in lecture notes: S_t = 0.820, n_f = 1.55, and C_f = 3.442 (ft & cfs) for the selected flume size. At 50 cfs:

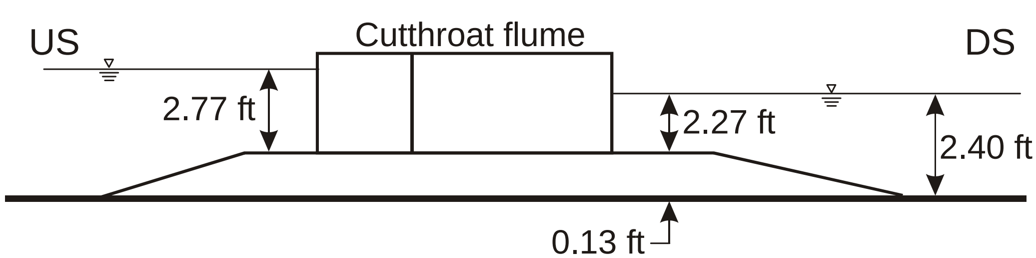
$$h_u = \left(\frac{Q_f}{C_f W}\right)^{1/n_f} = \left(\frac{50.0}{(3.442)(3.0)}\right)^{1/1.55} = 2.77 \text{ ft}$$

Then,

$$(h_d)_{max} = S_t h_u = (0.820)(2.77) = 2.27 \text{ ft}$$

Finally, the minimum floor height with respect to the channel bed is:

$$(h_{floor})_{min} = 2.40 - 2.27 = 0.13 \text{ ft}$$



2. **(15 pts)** Current metering data is given below for an open channel with a top width of 3.03 m. Complete the calculations to estimate the total flow rate in m^3/s.

Note that there is a vertical wall at the 3.08 distance from the edge (otherwise, the depth would be zero at this location). The ratio x/D is:

$$\frac{x}{D} = \frac{(3.08 - 3.00)}{1.05} = 0.0762$$

Then,

$$\frac{\overline{V}_x}{\overline{V}_D} = \frac{0.65 + 10.52(0.0762)}{1 + 10.676(0.0762) - 0.51431(0.0762)^2} = 0.802$$

and,

$$\overline{V}_w = \frac{0.65(0.210)}{0.802} = 0.170 \text{ m/s}$$

The rest of the calculations are simple and are given in the table below.

distance from edge (m)	depth (m)	depth fraction	velocity (m/s)			mean depth (m)	width (m)	area (m^2)	flow rate (m^3/s)
			at point	mean in vertical	mean in section				
0.05	0.00	n/a	10%	0.019					
					0.105	0.090	0.400	0.036	0.0038
0.45	0.18	0.6	0.190	0.190					
					0.201	0.375	0.800	0.300	0.0602
1.25	0.57	0.2	0.208	0.211					
		0.8	0.214		0.215	0.700	0.500	0.350	0.0751
1.75	0.83	0.2	0.211	0.218					
		0.8	0.225		0.221	0.900	0.250	0.225	0.0497
2.00	0.97	0.2	0.218	0.224					
		0.8	0.229		0.220	1.010	0.500	0.505	0.1110
2.50	1.05	0.2	0.210	0.216					
		0.8	0.222		0.213	1.050	0.500	0.525	0.1118
3.00	1.05	0.2	0.208	0.210					
		0.8	0.212		0.190	1.050	0.080	0.084	0.0160
3.08	1.05	n/a		0.170					
						Totals:	3.030	2.025	0.427

3. (15 pts) You need to design a suppressed, rectangular sharp-crested weir for a rectangular channel with a base width of 2.10 m, and a Q_{max} of 4.5 m^3/s. A normal depth of 0.678 m at Q_{max} was measured in the field. Manning roughness is 0.014. Design the weir, specifying crest height, P, provided the conditions are appropriate.

Check the Froude number in this channel at Q_{max}.

$$F_r^2 = \frac{Q^2T}{gA^3} = \frac{(4.5)^2(2.10)}{(9.81)\left[(2.10)(0.678)\right]^3} = 1.50$$

Thus, the regime is supercritical, so it is suggested that this is not a good location for a measurement weir.

4. (25 pts) A BCW, operating under free-flow conditions, has a flow rate of 17.00 m^3/s when h_u (referenced from the sill) is 1.813 m. At this same flow rate, h_d (referenced from the downstream bed elevation) is 2.661 m. The upstream and throat sections are rectangular with a width of 3.75 m. The sill height is $z_u = z_d =$ 1.15 m. This BCW has a downstream ramp with a 6:1 slope.

(a) Determine the upstream specific energy, E_u.

$$E_u = h_u + \frac{Q^2}{2gA_u^2} =$$

$$(1.813 + 1.15) + \frac{(17.0)^2}{2(9.81)\left[(1.813 + 1.15)(3.75)\right]^2} = 3.082 \text{ m}$$

(b) Determine h_c (referenced from the sill) for this flow rate.

For critical flow, the Froude number equals unity. Thus,

$$F_r^2 = \frac{Q^2T}{gA^3} = \frac{(17.0)^2(3.75)}{9.81(3.75h_c)^3} = 1$$

whereby h_c = 1.280 m.

(c) Estimate the hydraulic head loss in the diverging section, $(h_f)_{ds}$.

See the equations in the lecture notes:

$$\xi = \frac{\log_{10}\left[114.6\tan^{-1}(1/6)\right] - 0.165}{1.742} = 0.638$$

The velocity at the critical-flow section is V_c = 17.0/((3.75)(1.280)) = 3.54 m/s. The velocity at the DS section is V_d = 17.0/((3.75)(2.661)) = 1.70 m/s. Then,

$$(h_f)_{ds} = \frac{\xi(V_c - V_d)^2}{2g} = \frac{0.638(3.54 - 1.70)^2}{2(9.81)} \approx 0.11\,\text{m}$$

5. **(25 pts)** A circular pipe has a circular, sharp-edged orifice plate (D_2 = 5.50 inches), centered in the pipe cross section, which has an inside diameter of D_1 = 7.90 inches. Taps at D_1 (upstream) and ½D_1 (downstream) are connected to a manometer with mercury, from which a head differential of 186 mm is measured. Water temperature is 13°C. Ignoring thermal expansion adjustments:

(a) Determine the ratio β.

$$\beta = \frac{D_2}{D_1} = \frac{5.50}{7.90} = 0.696$$

(b) Determine the kinematic viscosity, ν.

$$\nu = \frac{1}{83.9192(13)^2 + 20{,}707.5(13) + 551{,}173} = 1.198(10)^{-6}\ \text{m}^2/\text{s}$$

(c) Determine the discharge coefficient, C_d.

This requires one or two iterations. The diameter of the orifice is: D_2 = 0.3048(5.50/12) = 0.140 m. The cross-sectional area of the orifice opening is:

$$A_2 = \frac{\pi D^2}{4} = \frac{\pi(0.140)^2}{4} = 0.0154\ \text{m}^2$$

Start with C_d = 0.6.

$$Q_1 = 0.6(0.0154)\frac{\sqrt{2g(0.186)(13.6-1)}}{\sqrt{1-(0.696)^4}} = 0.6(0.1194) = 0.0716\ \text{m}^3/\text{s}$$

Then,

$$R_e = \frac{4Q}{\pi D \nu} = \frac{4(0.0716)}{\pi(0.140)(0.000001198)} = 543{,}500$$

and,

$$\begin{aligned} C_d &= 0.5959 + 0.0312(0.696)^{2.1} - 0.184(0.696)^8 \\ &\quad + \frac{0.039(0.696)^4}{1-(0.696)^4} - 0.0158(0.696)^3 + \frac{91.71(0.696)^{2.5}}{(543{,}500)^{0.75}} \\ &= 0.607 + \frac{37.06}{R_e^{0.75}} \\ &= 0.609 \end{aligned}$$

The adjusted flow rate is:

$$Q_2 = 0.609(0.1194) = 0.0727 \text{ m}^3/\text{s}$$

Another iteration gives: R_e = 551,900, and C_d = 0.609, so it is converged.

(d) Determine the upstream Reynolds number, R_e.

R_e = 551,900

(e) Determine the flow rate, Q.

Q = 0.0727 m^3/s

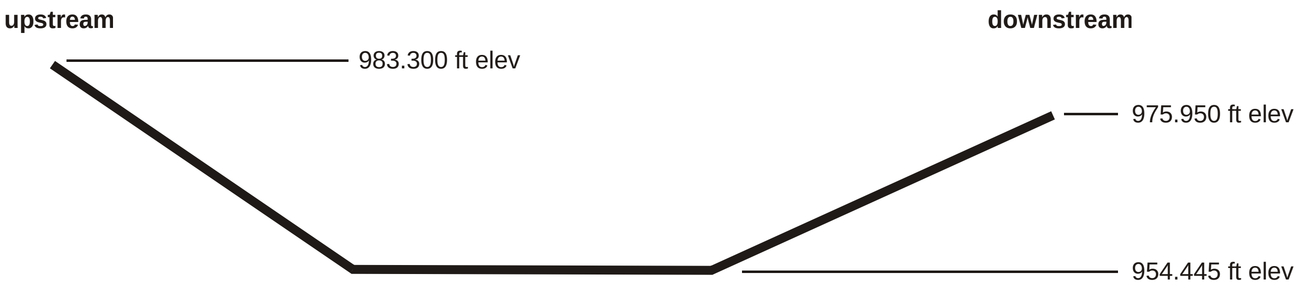
BIE 6300

Fall Semester 2004
Exam #2 – 14 Dec 04

Show your work neatly on this or separate pages. Show units for all calculated values. Make note of any important assumptions.

Name________________________________

An inverted siphon with a single barrel of circular concrete pipe (2.25-ft inside diameter) has a total length of 635 ft. The descending part of the barrel has a length of 185 ft. The inlet and outlet structures are ungated Type 1 USBR transitions. See the side view (profile) figure below:



The inverted siphon connects upstream and downstream open canals which have the same trapezoidal in cross section: base width is b = 4.0 ft; inverse side slope is 1.00; and, longitudinal bed slope is 0.000212 ft/ft. The Manning roughness is estimated to be n = 0.015 in the open canals. Assume uniform-flow conditions in the open channels both upstream and downstream of the inverted siphon.

a) Estimate the flow rate through the inverted siphon pipe under steady-state flow conditions. Use the Darcy-Weisbach equation with friction factor "f" defined by the Swamee-Jain equation:

$$f = \frac{0.25}{\left[\log_{10}\left(\frac{\varepsilon}{3.75D} + \frac{5.74}{N_R^{0.9}}\right)\right]^2}$$

where the roughness height is ε = 0.001 ft; and the kinematic viscosity of the water is $\nu = 1.2(10)^{-5}$ ft^2/s.

b) Calculate the maximum pressure in the inverted siphon pipe for steady-state flowing conditions.
c) Calculate the maximum pressure in the inverted siphon pipe for non-flowing conditions.
d) Calculate the uniform-flow depth in the upstream open canal.
e) Is the hydraulic seal sufficient at the inverted siphon entrance? (the barrel invert has the same elevation as the canal invert at the siphon entrance).

Solution:

(a) Steady-State Flow Rate

- Recognize that the open canals US and DS of the inverted siphon have the same cross-section, longitudinal bed slope, and roughness. Thus, if uniform-flow conditions prevail US and DS of the inverted siphon, the depth just US of the siphon must be the same as the depth just DS. This means that the head differential on the siphon is equal to the difference in entrance and exit elevations:

$$\Delta \text{Elev} = 983.300 - 975.95 = 7.350 \text{ ft}$$

- Under steady-state flow conditions, the velocity heads are the same in the open channels just US and just DS of the inverted siphon. This means that the hydraulic energy loss through the siphon under these conditions is very nearly equal to ΔElev. Then,

$$\Delta \text{Elev} = h_f$$

  where h_f is determined from Darcy-Weisbach and Swamee-Jain (English units):

$$h_f = f\frac{L}{D}\frac{V^2}{2g} = \frac{8fLQ^2}{g\pi^2 D^5} = \frac{8f(635)Q^2}{(32.2)\pi^2(2.25)^5} = 0.277fQ^2$$

- The friction factor, f, is defined as:

$$f = \frac{0.25}{\left[\log_{10}\left(\frac{0.001}{3.75(2.25)} + \frac{5.74}{N_R^{0.9}}\right)\right]^2} = \frac{0.25}{\left[\log_{10}\left(0.000119 + \frac{5.74}{N_R^{0.9}}\right)\right]^2}$$

- The Reynolds number is:

$$N_R = \frac{VD}{\nu} = \frac{4Q}{\nu\pi D} = \frac{4Q}{1.2(10)^{-5}\pi(2.25)} = 4.72(10)^4 Q$$

- Then:

$$f = \frac{0.25}{\left[\log_{10}\left(0.000119 + 0.000357Q^{-0.9}\right)\right]^2}$$

- Combine the above equation with the equation for h_f:

$$h_f = \frac{0.0693Q^2}{\left[\log_{10}\left(0.000119 + 0.000357Q^{-0.9}\right)\right]^2} = 7.350 \text{ ft}$$

  where the only unknown is Q (cfs). Rearrange as follows:

$$Q = -10.3\log_{10}\left(0.000119 + 0.000357\,Q^{-0.9}\right)$$

- Using the above equation, make an initial guess of 10 cfs (for example), then iterate to determine the solution to the equation:

Q (cfs)	next Q (cfs)
10.00	38.99
38.99	39.95
39.95	39.96

- From the above table, Q = 40.0 cfs (to three significant digits). This gives f = 0.0166.

(d) Uniform Flow Depth in the Canal

- Use the Manning equation:

$$40.0 = \frac{1.49}{0.015}\frac{\left[h(4.0+h)\right]^{5/3}}{\left[4.0+2h\sqrt{1+1}\right]^{2/3}}\sqrt{0.000212}$$

or,

$$27.7 = \frac{\left[h(4.0+h)\right]^{5/3}}{\left[4.0+2.83h\right]^{2/3}}$$

- Solving the above equation by iteration: **h = 2.89 ft** (to three significant digits).

(b) Maximum Pressure: Steady-State Flow

- In this case, the maximum pressure will occur at the lowest point in the inverted siphon, at the end of the descending portion of the barrel. This pressure is equal to the change in elevation from the US free water surface to the end of the descending part of the barrel, minus the friction loss. Using Darcy-Weisbach for h_f:

$$h_{max} = \left(2.89 + 983.300 - 954.445\right) - \frac{8f(185)(40.0)^2}{(32.2)\pi^2(2.25)^5}$$

or,

$$h_{max} = 31.7 - 129\,f$$

- Note that f is the same as previously calculated (same D, same Q, etc.): f = 0.0166. Then,

$$h_{max} = 31.7 - 129(0.0166) = 29.6\text{ ft}$$

and,

$$P_{max} = \frac{29.6 \text{ ft}}{2.31 \text{ ft/psi}} = 12.8 \text{ psi}$$

(c) Maximum Pressure: Zero Flow

- In this case, the maximum pressure will again occur at the lowest point in the inverted siphon. For zero flow, there is zero friction loss.
- Assuming zero depth in the downstream channel:

$$h_{max} = 975.950 - 954.445 = 21.5 \text{ ft}$$

and,

$$P_{max} = \frac{21.5 \text{ ft}}{2.31 \text{ ft/psi}} = 9.31 \text{ psi}$$

- Thus, the pressure is very low under both steady and non-flowing conditions, well within the pressure rating for concrete pipe joints.

(e) Hydraulic Seal at Siphon Entrance

- The hydraulic seal is the difference in elevation between the US free water surface and the crown (highest point) of the barrel inlet. In our case, this is approximately equal to:

$$\text{seal} = 2.89 - 2.25 = 0.64 \text{ ft}$$

- According to USBR design criteria, the required hydraulic seal is 1.5 times the difference in velocity heads between the pipe and the open canal:

$$\text{required seal} = 1.5\Delta h_v$$

- Velocity head in the pipe:

$$\left(\frac{V^2}{2g}\right)_{pipe} = \frac{8(40.0 \text{ cfs})^2}{(32.2)\left(\pi(2.25)^2\right)^2} = 1.57 \text{ ft}$$

- Velocity head in the canal:

$$\left(\frac{V^2}{2g}\right)_{canal} = \frac{(40.0 \text{ cfs})^2}{2(32.2)\left(2.89(4+2.89)\right)^2} = 0.0627 \text{ ft}$$

- Then,

$$\text{required seal} = 1.5\left(1.57 - 0.0627\right) = 2.26 \text{ ft}$$

- Thus, the actual hydraulic seal is much less than the required hydraulic seal.

BIE 5300/6300 Assignment #1
Simple Flow Measurement and Flumes

02 Sep 04 (due 07 Sep 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments about each of the problems.

I. A float test was conducted in a straight reach of open channel. The rectangular cross section had a base width of 1.55 m. A small piece of a wooden twig was timed with a stopwatch over a distance of 10 m, and the three trials produced transit times of: 7.42, 7.58, and 7.49 s. The depth of water in the center of the channel was 1.32 m. Apply the float method to estimate the discharge in m^3/s.

II. You have field data for a concrete-lined, rectangular irrigation canal. The base width is 3.04 m. The data show that the channel is prismatic (straight in alignment, with constant cross-sectional shape and size) and the water depth was constant at 2.15 m along a 2.23-km reach. The water depth had not changed over a period of two hours before the measurements were taken. If the longitudinal bed slope of the channel is 0.00012 m/m, what is the estimated discharge range (min & max), in m^3/s, for this canal reach? (use the Manning and or Chezy equations).

III. Data were taken using the dye method in an earthen canal. A slug of dye was injected into the center of the stream and the leading (front) edge traveled a distance of 12 m in 9.05 s, while the trailing edge crossed the 12-m distance in 10.11 s. If the cross-sectional area of the channel was 5.13 m^2, what is the estimated discharge in m^3/s?

IV. The table below has free-flow calibration data for a Cutthroat flume with L = 3.0 ft and W = 8 inches. Analyze the data to determine the values of C_f and n_f and compare these values to those published in the lecture notes.

Q (cfs)	h_u (ft)
2.688	1.003
2.688	1.003
2.686	1.003
2.687	1.003
2.687	1.003
1.865	0.811
1.865	0.811
3.375	1.095
2.263	0.907
1.461	0.696
1.122	0.611
1.123	0.611
1.123	0.611
1.123	0.611
0.844	0.518

V. The table below has submerged-flow calibration data for a Cutthroat flume with L = 3.0 ft and W = 8 inches. Analyze the data to determine the values of C_s and n_s, using the n_f value from the previous calibration for the same flume size. Compare these values to those published in the lecture notes.

Q (cfs)	h_u (ft)	h_d (ft)
2.687	1.005	0.575
2.687	1.008	0.602
2.687	1.009	0.627
2.687	1.011	0.660
2.687	1.013	0.689
2.687	1.021	0.739
2.687	1.030	0.781
2.685	1.055	0.863
2.685	1.083	0.931
2.685	1.120	0.993
2.683	1.183	1.093
1.865	0.813	0.581
1.865	0.815	0.606
1.865	0.820	0.639
1.865	0.828	0.673
1.863	0.840	0.702
1.863	0.905	0.826
1.86	0.870	0.763
1.86	0.953	0.893
1.86	1.077	1.025
1.858	1.015	0.968
1.123	0.613	0.443
1.123	0.617	0.468
1.123	0.618	0.485
1.123	0.623	0.523
1.123	0.638	0.560
1.123	0.666	0.608
1.123	0.698	0.658
1.12	0.828	0.805

VI. A Cutthroat flume is to be installed in an existing concrete-lined rectangular canal with a base width of 5.0 ft. The maximum flow rate to be measured at that location is 40 cfs. The longitudinal bed slope of the channel is 0.00045 ft/ft, and the depth of the concrete lining is 3.7 ft. Assume a Manning roughness value of n = 0.015.

(a) Select an appropriate Cutthroat flume size, in English units, from the table in the lecture notes.
(b) Determine the minimum height of the Cutthroat flume floor, relative to the existing canal bed, such that free-flow conditions prevail up to the maximum discharge of 40 cfs. Do not specify a floor elevation below the existing canal bed.
(c) Will the upstream canal banks need to be raised if the Cutthroat flume is installed for free-flow conditions?

Solutions:

I. The average transit time of the three trials is: (7.42 + 7.58 + 7.49)/3 = 7.50 s. The average estimated surface velocity is: 10 m/7.50 s = 1.33 m/s. Alternatively, the average surface velocity can be taken as (10/7.42 + 10/7.58 + 10/7.49)/3 = 1.33 m/s (same result). The cross-sectional area of the channel is:

$$A = (1.55)(1.32) = 2.05 \text{ m}^2$$

The average depth in the channel is 1.32 m (because the section is rectangular). Interpolating linearly (which is the simplest option, and probably as valid as anything else in this case) in the table from the lecture notes, the surface velocity coefficient is:

$$\frac{1.32 - 1.22}{1.52 - 1.22} = \frac{C - 0.72}{0.74 - 0.72}$$

which gives C = 0.73. The average velocity in the cross section is estimated as: (0.73)(1.33 m/s) = 0.97 m/s. Finally, the flow rate is estimated to be:

$$A\overline{V} = \left(2.05 \text{ m}^2\right)\left(0.97 \text{ m/s}\right) \approx 2.0 \text{ m}^3\text{/s}$$

II. For this channel cross section size and type of lining, the Manning "n" value might be in the range 0.012 < n < 0.018, depending on the condition of the concrete lining and the presence (or absence) of vegetation and sediment. It probably won't be less than 0.012, but could be greater than 0.018.

The cross-sectional area is: A = (3.04 m)(2.15 m) = 6.54 m^2. The wetted perimeter is: W_p = 3.04 + 2(2.15) = 7.34 m. Applying the Manning equation:

$$Q = \frac{1}{n}\frac{A^{5/3}}{W_p^{2/3}}\sqrt{S_o} = \frac{1}{0.012}\frac{(6.54)^{5/3}}{(7.34)^{2/3}}\sqrt{0.00012} \approx 5.5 \text{ m}^3\text{/s}$$

at the lower range. Applying the equation again for n = 0.018, we get Q ≈ 3.7 m^3/s. Thus, by the Manning equation, you might agree that: **3.7 < Q < 5.5 m^3/s**.

Note that the Manning "n" value has no more than two significant digits in this case, so Q can have no more than that. You could also apply the Chezy equation. These two equations will be discussed further in a future lecture.

III. With this method, you don't apply a coefficient to the measured velocity, which in this case is:

$$V = \frac{12\text{ m}}{0.5(9.05 + 10.11)} = 1.25\text{ m/s}$$

Finally, the estimated discharge is:

$$Q = AV = \left(5.13\text{ m}^2\right)(1.25\text{ m/s}) \approx 6.4\text{ m}^3\text{/s}$$

IV. The free-flow data were analyzed as shown in the table below, giving C_f = 2.71 and n_f = 1.78. Three significant digits are the most that can be justified for this kind of calibration. Note the high coefficient of determination, R^2, of 0.998. These results were done in MS Excel®, using the LINEST spreadsheet function.

Q (cfs)	h_u (ft)	ln(Q)	ln(h_u)	Predicted Q (cfs)	Percent Difference
2.688	1.003	0.9888	0.0025	2.7234	-1.31
2.688	1.003	0.9888	0.0025	2.7234	-1.31
2.686	1.003	0.9881	0.0025	2.7234	-1.38
2.687	1.003	0.9884	0.0025	2.7234	-1.35
2.687	1.003	0.9884	0.0025	2.7234	-1.35
1.865	0.811	0.6233	-0.2097	1.8665	-0.08
1.865	0.811	0.6233	-0.2097	1.8665	-0.08
3.375	1.095	1.2164	0.0908	3.1868	5.74
2.263	0.907	0.8167	-0.0980	2.2773	-0.63
1.461	0.696	0.3791	-0.3626	1.4215	2.74
1.122	0.611	0.1151	-0.4929	1.1272	-0.46
1.123	0.611	0.1160	-0.4929	1.1272	-0.37
1.123	0.611	0.1160	-0.4929	1.1272	-0.37
1.123	0.611	0.1160	-0.4929	1.1272	-0.37
0.844	0.518	-0.1696	-0.6587	0.8390	0.59

Note: L = 3.0 ft and W = 8 inches

Linear Regression	
1.7805	0.9974
0.0212	0.0070
0.9982	0.0198

C_f=	**4.07**
n_f=	**1.78**

Thus,

$$Q_f = 4.07\,W\,h_u^{1.78}$$

for W = 8/12 ft; h_u in feet; and Q_f in cfs.

V. The submerged-flow data were analyzed as shown in the table below, also in a spreadsheet application, giving C_s = 1.60 and n_s = 1.31. Three significant digits are the most that can be justified for this kind of calibration. The free-flow exponent, n_f, was used in this calibration. Note the coefficient of determination, R^2, of 0.999.

							Predicted	Percent
Q (cfs)	h_u (ft)	h_d (ft)	S=hd/hu	hu-hd	LHS	RHS	Q (cfs)	Difference
2.687	1.005	0.575	0.572	0.430	2.356	-1.417	2.61	2.85
2.687	1.008	0.602	0.597	0.406	2.449	-1.497	2.64	1.75
2.687	1.009	0.627	0.621	0.383	2.545	-1.575	2.66	1.01
2.687	1.011	0.660	0.653	0.351	2.685	-1.687	2.68	0.42
2.687	1.013	0.689	0.680	0.324	2.813	-1.787	2.69	0.04
2.687	1.021	0.739	0.724	0.282	3.041	-1.965	2.70	-0.46
2.687	1.030	0.781	0.758	0.249	3.240	-2.118	2.71	-0.71
2.685	1.055	0.863	0.818	0.192	3.664	-2.441	2.70	-0.66
2.685	1.083	0.931	0.859	0.153	4.034	-2.720	2.69	-0.19
2.685	1.120	0.993	0.887	0.127	4.335	-2.954	2.71	-0.86
2.683	1.183	1.093	0.925	0.089	4.903	-3.380	2.68	0.09
1.865	0.813	0.581	0.715	0.232	2.992	-1.926	1.87	-0.22
1.865	0.815	0.606	0.743	0.209	3.158	-2.049	1.86	0.10
1.865	0.820	0.639	0.779	0.181	3.394	-2.224	1.85	0.82
1.865	0.828	0.673	0.812	0.156	3.635	-2.402	1.84	1.51
1.863	0.840	0.702	0.835	0.138	3.827	-2.549	1.84	1.44
1.863	0.905	0.826	0.913	0.079	4.731	-3.225	1.80	3.21
1.86	0.870	0.763	0.877	0.107	4.246	-2.868	1.83	1.55
1.86	0.953	0.893	0.937	0.060	5.178	-3.567	1.80	3.06
1.86	1.077	1.025	0.952	0.052	5.421	-3.846	2.04	-9.31
1.858	1.015	0.968	0.954	0.047	5.584	-3.890	1.83	1.32
1.123	0.613	0.443	0.723	0.170	2.987	-1.959	1.18	-5.20
1.123	0.617	0.468	0.759	0.148	3.207	-2.124	1.18	-4.79
1.123	0.618	0.485	0.784	0.133	3.380	-2.249	1.17	-3.88
1.123	0.623	0.523	0.838	0.101	3.833	-2.569	1.13	-0.52
1.123	0.638	0.560	0.877	0.078	4.242	-2.867	1.11	1.24
1.123	0.666	0.608	0.914	0.058	4.743	-3.238	1.09	2.63
1.123	0.698	0.658	0.944	0.039	5.365	-3.685	1.05	6.27
1.12	0.828	0.805	0.972	0.023	6.201	-4.389	1.15	-2.47

Notes: "LHS" is ln(Qs) - nf ln(hu-hd). "RHS" is ln(-log10(S))

Linear Regression	
-1.311555	0.468977
0.007212	0.019525
0.999184	0.030379

Cs=	**2.40**
ns=	**1.31**

Thus,

$$Q_s = \frac{2.40\,W\left(h_u - h_d\right)^{1.78}}{\left(-\log_{10} S\right)^{1.31}}$$

for W = 8/12 ft; h_u & h_d in feet; and Q_f in cfs.

VI. Referring to the Cutthroat flume table for English units in the lecture notes, and the Cutthroat flume top view figure, it is seen that the width of the flume is B = W + L/4.5.

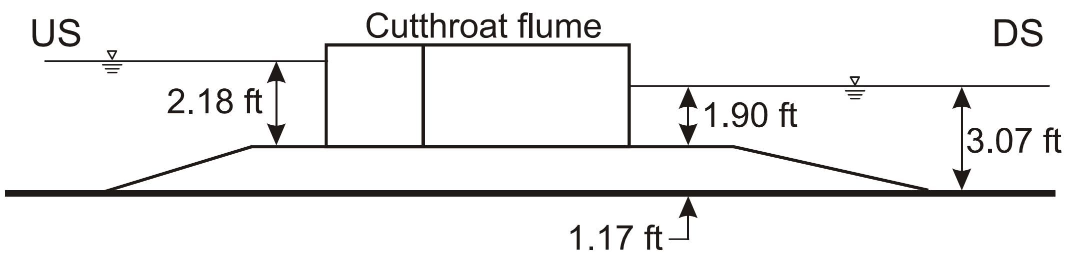
(a) In the table, the smallest Cutthroat flume with a capacity of at least 40 cfs is for W = 3.333 ft and L = 7.50 ft. This gives B = 3.333 + 7.50/4.5 = 5.0 ft, which is the same as the base width of the rectangular canal. This works out just right in this case, so choose the **W = 3.333 ft** and **L = 7.50 ft** size. Note that W/L = 4/9.

(b) From the calibration table (English units), S_t = 0.873 for our selected flume size. Free-flow conditions at Q_{max} give:

$$h_u = \left(\frac{Q_{max}}{C_f W}\right)^{1/n_f} = \left(\frac{40}{(3.519)(3.333)}\right)^{1/1.57} = 2.18\ \text{ft}$$

Then, $S_t h_u$ = (0.873)(2.18) = 1.90 ft, which is the maximum downstream depth with respect to the upstream floor elevation.

Using the ACA program to determine the normal depth corresponding to the given conditions, at Q = 40 cfs, it is found that the downstream depth would be 3.07 ft. Thus, the floor of the flume must be at least 3.07 – 1.90 = **1.17 ft above the existing canal bed**. See the side view figure below.



(c) The depth of the concrete lining is given as 3.7 ft. The upstream depth at 40 cfs with the Cutthroat flume in place will be 2.18 + 1.17 = 3.35 ft, which is less than 3.7 ft by a margin of 0.35 ft, which may be enough freeboard in this case. The upstream canal banks **do not need to be raised**.

BIE 5300/6300 Assignment #2
Current Metering Calculations

09 Sep 04 (due 14 Sep 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments.

I. You are given the electromagnetic current metering measurements below. Calculate the total flow rate in the channel using a spreadsheet. Plot the cross-section profile in a graph.

Distance	Depth	Depth	Velocity (m/s)			Average	Width	Area	Flow
				mean at	subsection				
(m)	(m)	Fraction	At point	vertical	mean	depth (m)	(m)	(m2)	(m3/s)
Left									
0.000	0.00		0.25						
0.305	0.58	0.2	0.51						
		0.8	0.46						
0.615	0.58	0.2	0.54						
		0.8	0.46						
0.925	0.58	0.2	0.49						
		0.8	0.43						
1.230	0.00		0.25						
Right						Totals:			

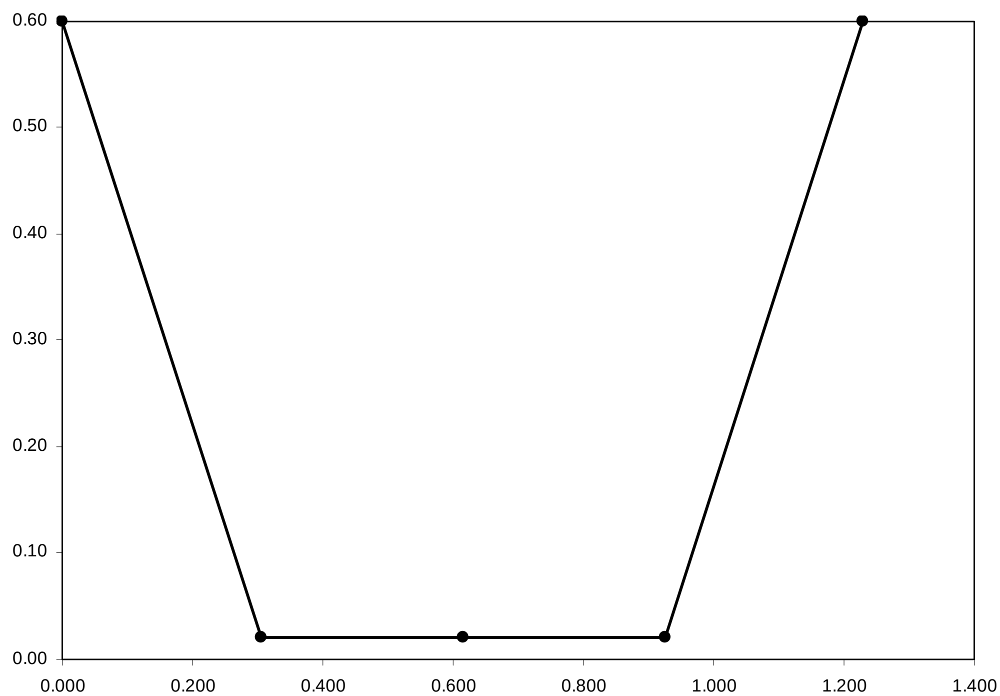
II. You are given the electromagnetic current metering measurements below. Calculate the total flow rate in the channel using a spreadsheet. Plot the cross-section profile in a graph.

Distance	Depth	Depth	Velocity (m/s)			Average	Width	Area	Flow
		Fraction	At point	mean at vertical	subsection mean	Depth			
(m)	(m)					(m)	(m)	(m2)	(m3/s)
Left									
0.00	0.500								
0.16	0.500	0.2	1.40						
		0.8	1.51						
0.32	0.500	0.2	1.85						
		0.8	1.45						
0.48	0.500	0.2	1.96						
		0.8	1.50						
0.64	0.500	0.2	1.94						
		0.8	1.56						
0.80	0.500	0.2	1.62						
		0.8	1.46						
0.96	0.500								
Right									
					Totals:				

Solutions:

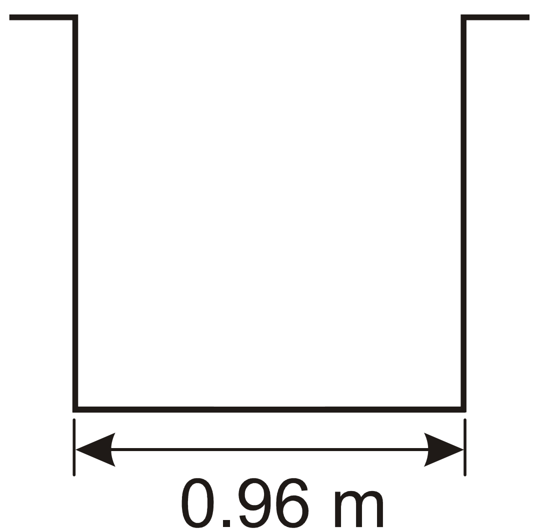
I. The calculations were performed in a spreadsheet and the results are:

Distance	Depth	Depth	Velocity (m/s)			Average	Width	Area	Flow
				mean at	subsection				
(m)	(m)	Fraction	At point	vertical	mean	depth (m)	(m)	(m2)	(m3/s)
Left									
0.000	0.00		0.25	0.250					
					0.368	0.290	0.305	0.088	0.033
0.305	0.58	0.2	0.51	0.485					
		0.8	0.46		0.493	0.580	0.310	0.180	0.089
0.615	0.58	0.2	0.54	0.500					
		0.8	0.46		0.480	0.580	0.310	0.180	0.086
0.925	0.58	0.2	0.49	0.460					
		0.8	0.43		0.355	0.290	0.305	0.088	0.031
1.230	0.00		0.25	0.250					
Right						**Totals:**	**1.230**	**0.537**	**0.239**



II. Again, the calculations were performed in a spreadsheet, applying the algorithm for vertical walls (see the lecture notes), and the results are:

Distance	**Depth**	**Depth**	**Velocity (m/s)**			**Average**	**Width**	**Area**	**Flow**
		Fraction	**At point**	**mean at vertical**	**subsection mean**	**Depth**			
(m)	**(m)**					**(m)**	**(m)**	**(m2)**	**(m3/s)**
Left		*x/D=*	*0.32*						
0.00	0.500	*Vx/VD=*	*0.92*	1.03					
					1.24	0.500	0.16	0.080	0.099
0.16	0.500	0.2	1.40	1.46					
		0.8	1.51		1.55	0.500	0.16	0.080	0.124
0.32	0.500	0.2	1.85	1.65					
		0.8	1.45		1.69	0.500	0.16	0.080	0.135
0.48	0.500	0.2	1.96	1.73					
		0.8	1.50		1.74	0.500	0.16	0.080	0.139
0.64	0.500	0.2	1.94	1.75					
		0.8	1.56		1.65	0.500	0.16	0.080	0.132
0.80	0.500	0.2	1.62	1.54					
		0.8	1.46		1.31	0.500	0.16	0.080	0.105
0.96	0.500	*x/D=*	*0.32*	1.09					
Right		*Vx/VD=*	*0.92*						
						Totals:	**0.960**	**0.480**	**0.735**



Note: Never show open-channel current metering results with more than two or three significant digits – four or more significant digits cannot be justified.

BIE 5300/6300 Assignment #3
Weir Calculations

24 Sep 04 (due 28 Sep 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments.

I. Hydraulically, what is the difference between a suppressed weir and a contracted weir?

II. A rectangular sharp-crested weir is installed in a straight section of a rectangular open channel with B = 10 ft, L = 7 ft, P = 3 ft, and h_u = 1.37 ft. The downstream water surface is well below the weir crest elevation.

(a) Do the stated conditions meet all the guidelines for setting and operating weirs, as given in the lecture notes? If no, which are violated?

(b) You are required to estimate the discharge over the weir for the stated conditions. Assuming negligible approach velocity, estimate Q_f in cfs.

(c) Estimate the discharge, Q_f, without assuming a negligible approach velocity (hint: use H_u instead of h_u in the calibration equation).

(d) Now suppose h_d = 0.31 ft and everything else is the same as given above. Estimate Q_s in cfs.

III. A Cipoletti weir is installed in an open channel. The approach velocity is negligible. If the crest length is 0.90 m and the upstream depth, h_u, is 0.22 m, referenced to the crest elevation, what is the estimated free-flow discharge?

IV. An overshot gate with L = 8.0 ft and G_w = 12.0 ft is installed in a canal. At the downstream side of the gate is a reservoir with a constant water surface elevation which is 0.29 ft above the gate hinge. The irrigation district needs you to develop and plot calibration curves for gate openings of: θ = 15, 20, 25, 30, 35, 40, 45, 50, 55, and 60 degrees. The plot should have h_u on the abscissa and Q on the ordinate. Both the abscissa and ordinate must start at zero. Each curve on the plot must be labeled with its corresponding gate opening angle.

V. A new trapezoidal concrete canal with a base width of 2.0 m and inverse side slopes of 1.5 has a total lined depth of 2.5 m. The bed slope of the canal is 0.00015 m/m and the length is 2.35 km, all straight in alignment (no curves or bends). At the end of the section there is a sudden drop in the bed elevation of 3.5 m, then the same channel cross section continues downstream, with the same bed slope. Design a sharp-crested weir, just upstream of the elevation drop, for a maximum flow rate of 7.0 m^3/s. Make sure the canal lining won't be overtopped upstream of the weir.

BIE 5300/6300 Assignment #5
Open-Channel Constriction Calibrations

06 Oct 04 (due 12 Oct 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments.

You have measured data in a spreadsheet file (attached) for an open-channel constriction (non-orifice flow). One data set is for free-flow conditions, the other for submerged-flow.

I. Develop a free-flow rating for this constriction using the following equation:

$$Q_f = C_f h_u^{n_f}$$

(a) Determine C_f and n_f.
(b) Make a graph with plotted symbols for $(Q_f)_{measured}$ vs. $(Q_f)_{calculated}$. The ordinate range should be the same as the abscissa range, with a diagonal line representing $(Q_f)_{measured}/(Q_f)_{calculated} = 1.0$.
(c) Comment on the data fit, using correlation or other such indices, as appropriate.

II. Develop a submerged-flow rating for this constriction using the following equation:

$$Q_s = \frac{C_s\left(h_u - h_d\right)^{n_f}}{\left(-\log_{10} S\right)^{n_s}}$$

(a) Determine C_s and n_s, using n_f from the free-flow rating.
(b) Make a graph with plotted symbols for $(Q_s)_{measured}$ vs. $(Q_s)_{calculated}$. The ordinate range should be the same as the abscissa range, with a diagonal line representing $(Q_s)_{measured}/(Q_s)_{calculated} = 1.0$.
(c) Comment on the data fit, using correlation or other such indices, as appropriate.

III. Solve for transition submergence, S_t, for the above calibration.

(a) Determine S_t.
(b) Make a graph of S_t (from 0.1 to 0.99 on the abscissa) vs. the function value (Q_f - Q_s = 0) and indicate where the solution(s) exist, if any.
(c) If you don't get any solution for S_t, try adjusting C_s slightly so that you get a solution. If you do this, show the adjusted C_s value.

IV. Re-do the submerged-flow rating using the following equation:

$$Q_s = \frac{C_s \left(h_u - h_d\right)^{n_{s1}}}{\left(-\log_{10} S\right)^{n_{s2}}}$$

(a) Determine C_s, n_{s1} and n_{s2} based only on the submerged-flow data.
(b) Make a graph with plotted symbols for $(Q_s)_{measured}$ vs. $(Q_s)_{calculated}$. The ordinate range should be the same as the abscissa range, with a diagonal line representing $(Q_s)_{measured}/(Q_s)_{calculated} = 1.0$.
(c) Comment on the data fit, using correlation or other such indices, as appropriate.
(d) Comment on the data fit using this equation, as opposed to using the Q_s equation from (II) above.

Solutions:

I. Develop a free-flow rating for this constriction using the following equation:

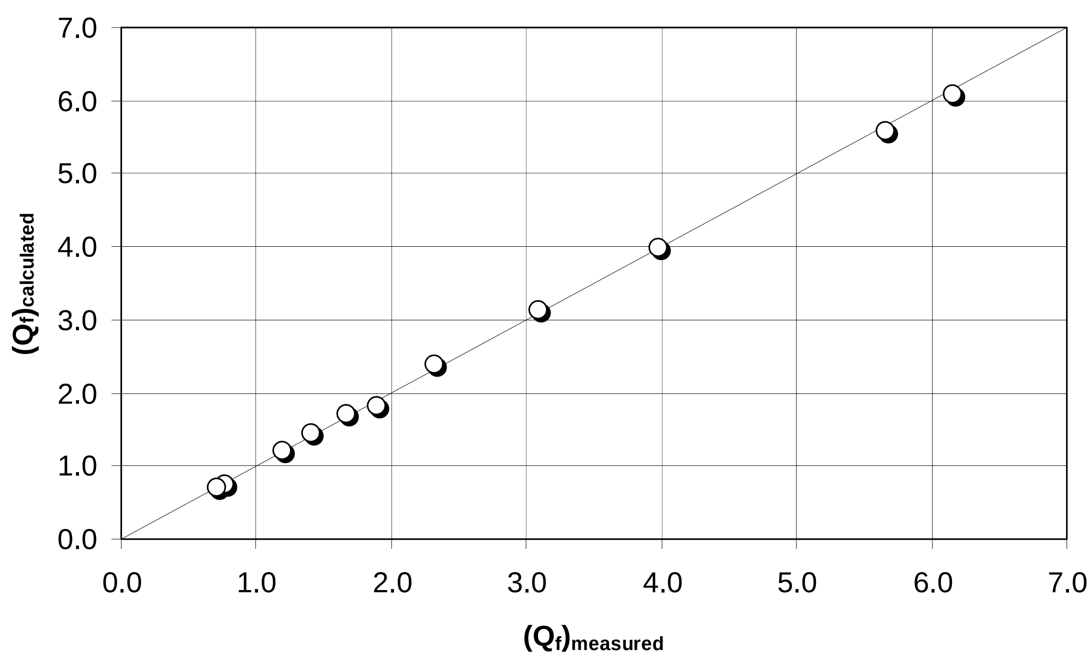
$$Q_f = C_f h_u^{n_f}$$

Make two new columns for $\ln(Q_f)$ and $\ln(h_u)$ in the spreadsheet. Do a linear regression using the LINEST spreadsheet function. The regression gives:

$C_f = 5.57$
$n_f = 1.61$

for Q_f in cfs; and h_u in ft.

The R^2 value is 0.999, indicating a very good fit, and this is also seen in the comparison graph:



II. Develop a submerged-flow rating for this constriction using the following equation:

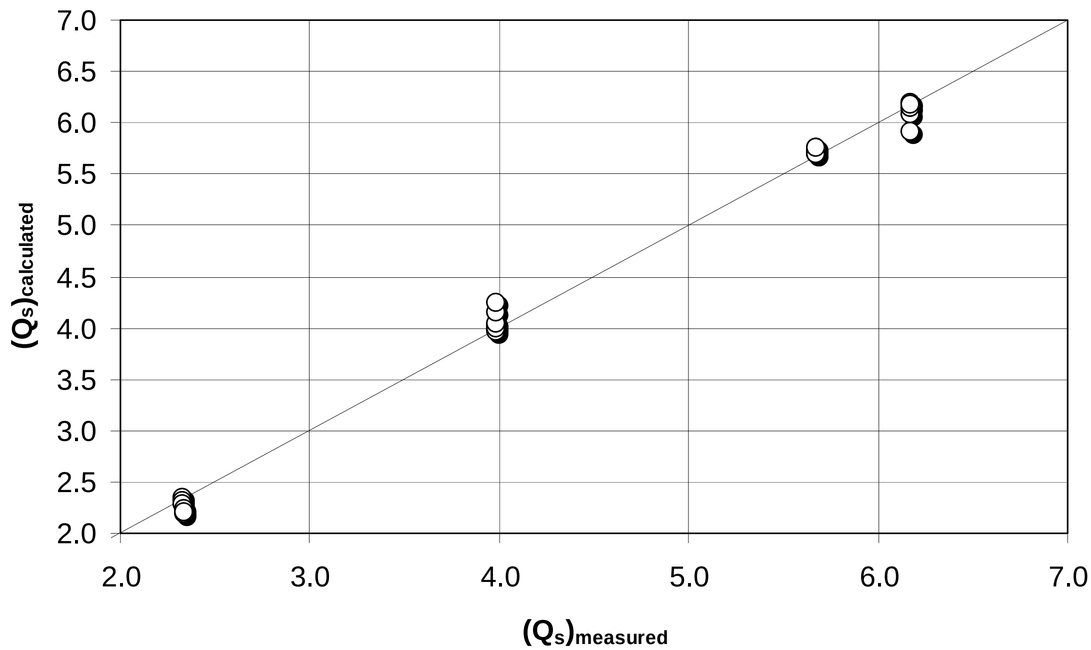
$$Q_s = \frac{C_s \left(h_u - h_d\right)^{n_f}}{\left(-\log_{10} S\right)^{n_s}}$$

Make two new columns for $\ln(Q_s/(h_u-h_d)^{nf})$ and $\ln(-\log_{10}S)$ in the spreadsheet. Do a linear regression using the LINEST spreadsheet function. The regression gives:

C_s= 2.62
n_s = 1.43

for Q_s in cfs; and h_u & h_d in ft.

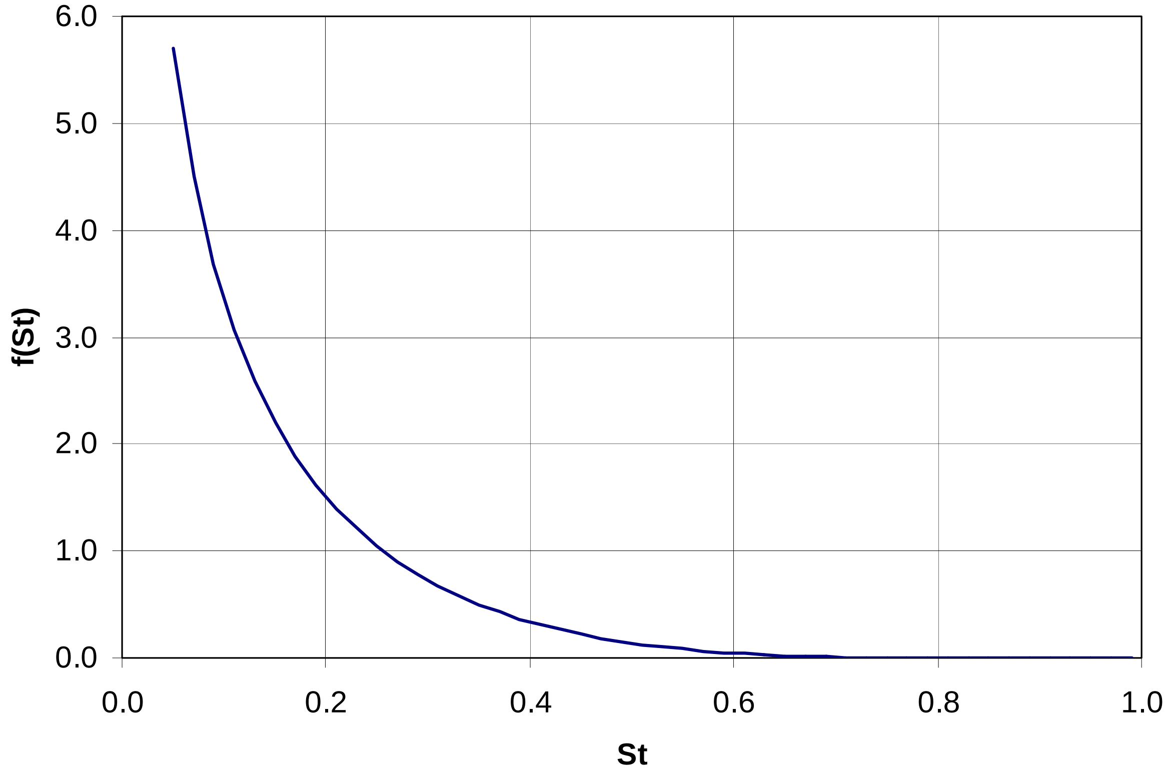
The R^2 value is 0.999, indicating a very good fit, and this is also seen in the comparison graph:

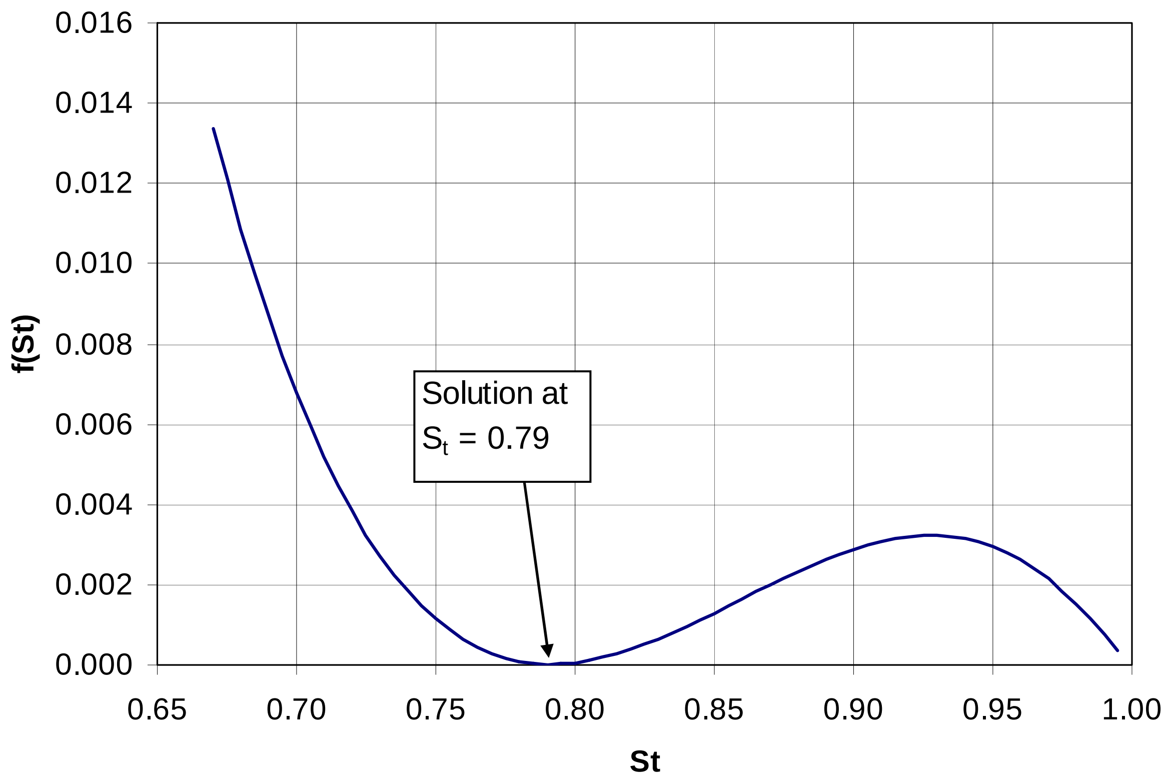


III. Solve for transition submergence, S_t, for the above calibration.

Use the equation for $f(S_t) = 0$, as shown in the lecture notes. This equation is derived by setting $Q_f = Q_s$. Make a table of S_t versus $f(S_t)$, then plot the results. The only solution is for $S_t = 1.00$, which is mathematically correct, but physically impossible.

Adjust C_s slightly, from 2.62 to 2.639, whereby a solution is found at about $S_t \approx$ 0.79, as shown in the next graph.


6.0
5.0
4.0
3.0
2.0
1.0
0.0
f(St)
0.0
0.2
0.4
0.6
0.8
1.0
St


0.016
0.014
0.012
0.010
0.008
0.006
0.004
0.002
0.000
f(St)
Solution at
S_t = 0.79
0.65
0.70
0.75
0.80
0.85
0.90
0.95
1.00
St

IV. Re-do the submerged-flow rating using the following equation:

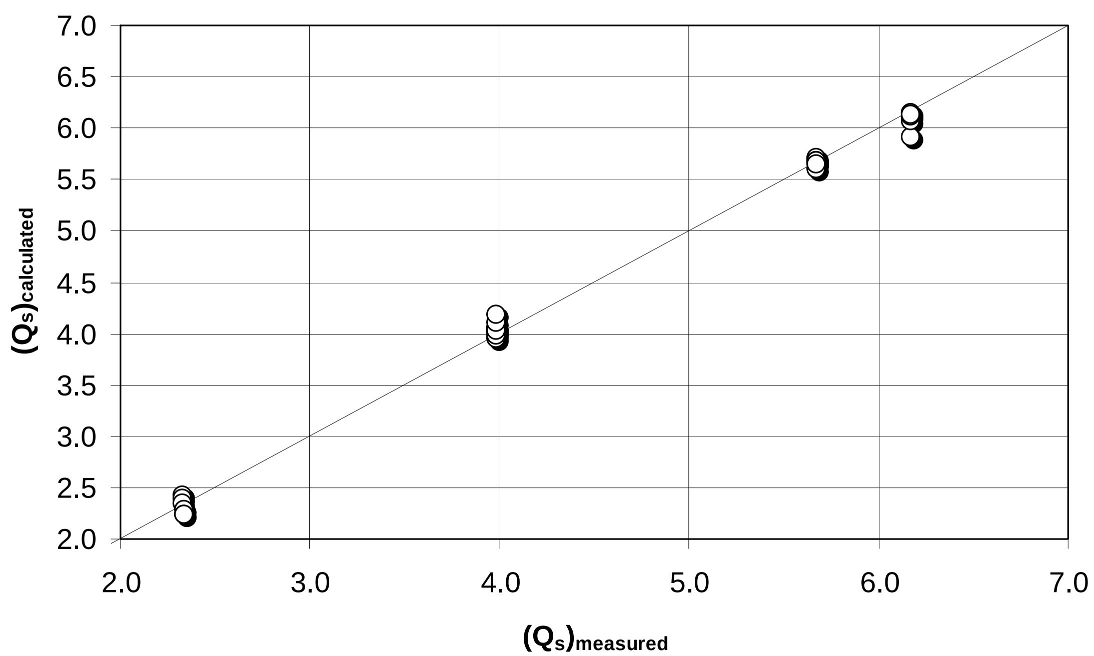
$$Q_s = \frac{C_s \left(h_u - h_d\right)^{n_{s1}}}{\left(-\log_{10} S\right)^{n_{s2}}}$$

Make three new columns for $\ln(Q_s)$, $\ln(h_u\text{-}h_d)$, and $\ln(-\log_{10}S)$ in the spreadsheet. Do a multiple linear regression using the LINEST spreadsheet function. The regression gives:

$C_s = 2.78$
$n_{s1} = 1.54$
$n_{s2} = 1.36$

for Q_s in cfs; and h_u & h_d in ft.

The R^2 value is 0.996, indicating a very good fit, and this is also seen in the comparison graph, which is very similar to the previous plot:



Even though the R^2 value is slightly lower than for the previous form of the submerged-flow equation, the sum of absolute deviations in measured and calculated discharges is less in this case.

BIE 5300/6300 Assignment #6
Pipe Flow Measurement

13 Oct 04 (due 18 Oct 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments. E-mail the assignment to me, or drop it by my office, by 12:00 pm Monday, then I will post my solutions at 12:30 pm the same day so that you can study for the test on Tuesday.

I. You use a simple Pitot tube to measure the total head at the center of a circular pipe with an inside diameter of 336 mm. The tip of the tube points in the upstream direction. You find a total head of 42.35 m of water when connecting the Pitot tube to a manometer. Separately, you measure the pressure in the pipe at the same location, obtaining P = 413 kPa.

(a) Estimate the velocity in the pipe at the center of the cross section.
(b) Estimate the flow rate in the pipe, in liters per second.

II. You have a venturi connected to a manometer with mercury, whereby the manometer is connected to an upstream tap, and to a tap just at the throat of the venturi. The head differential on the mercury is 456 mm. The diameters are: D_1 = 100 mm (upstream), and D_2 = 50 mm (throat). The calibration coefficient for a "machined inlet" is C = 0.995. Calculate the flow rate through the venturi.

III. You have a sharp-crested circular orifice at a gasketed pipe flange fitting. The upstream pipe ID is D_1 = 12.0 inches and the orifice diameter is 9.05 inches. The orifice opening is centered in the pipe cross section. The upstream tap is at a distance D1 upstream of the orifice plate, and the downstream tap is at a distance ½D_1 downstream of the plate. When the taps are connected to a manometer with "blue" fluid (sg = 1.75), the head differential is observed to be 0.519 m. The water temperature is measured and found to be 8°C. Calculate the flow rate to three significant digits, taking into account the Reynold's number.

IV. You have to estimate the discharge from a partially-full horizontal pipe which discharges freely into a canal. The end of the pipe is 20 cm above the water surface in the canal. The pipe inside diameter is 35 cm, and the depth of water at the pipe end is measured, giving 13 cm. Estimate the discharge in m^3/s.

Solutions:

I. You use a simple Pitot tube to measure the total head at the center of a circular pipe with an inside diameter of 336 mm. The tip of the tube points in the upstream direction. You find a total head of 42.35 m of water when connecting the Pitot tube to a manometer. Separately, you measure the pressure in the pipe at the same location, obtaining P = 413 kPa.

(a) Velocity in the pipe at the center of the cross section.

Pipe area:

$$A = \frac{\pi(0.336)^2}{4} = 0.08867 \text{ m}^2$$

Pressure head:

$$\frac{P}{\gamma} = \frac{413 \text{ kPa}}{9.81} = 42.1 \text{ m}$$

Velocity head:

$$\frac{V^2}{2g} = 42.35 - 42.1 = 0.25 \text{ m}$$

Velocity (at center):

$$V = 2.21 \text{ m/s}$$

(b) Flow rate in the pipe, in liters per second.

The maximum flow rate would be:

$$Q_{max} = VA = (2.21)(0.08867) = 0.196 \text{ m}^3/\text{s}$$

or, 196 lps. The true flow rate is probably slightly lower than this because the velocity at the center of the cross section is greater than the average velocity, even for fully turbulent flow.

II. You have a venturi connected to a manometer with mercury, whereby the manometer is connected to an upstream tap, and to a tap just at the throat of the venturi. The head differential on the mercury is 456 mm. The diameters are: D_1 = 100 mm (upstream), and D_2 = 50 mm (throat). The calibration coefficient for a "machined inlet" is C = 0.995. Calculate the flow rate through the venturi.

First, the beta ratio is:

$$\beta = \frac{D_2}{D_1} = \frac{50}{100} = 0.50$$

The cross-sectional area is:

$$A_2 = \frac{\pi(0.05)^2}{4} = 0.001963 \text{ m}^2$$

The flow rate is:

$$Q = C_d A_2 \frac{\sqrt{2g\Delta h(sg-1)}}{\sqrt{1-\beta^4}} =$$

$$0.995(0.001963)\frac{\sqrt{2g(0.456)(13.6-1)}}{\sqrt{1-(0.50)^4}} = 0.0214 \text{ m}^3\text{/s}$$

or, 0.756 cfs.

III. You have a sharp-crested circular orifice at a gasketed pipe flange fitting. The upstream pipe ID is D_1 = 12.0 inches and the orifice diameter is 9.05 inches. The orifice opening is centered in the pipe cross section. The upstream tap is at a distance D1 upstream of the orifice plate, and the downstream tap is at a distance ½D_1 downstream of the plate. When the taps are connected to a manometer with "blue" fluid (sg = 1.75), the head differential is observed to be 0.519 m. The water temperature is measured and found to be 8°C. Calculate the flow rate to three significant digits, taking into account the Reynold's number.

First, the beta ratio is:

$$\beta = \frac{D_2}{D_1} = \frac{9.05}{12.00} = 0.7542$$

The cross-sectional area is:

$$A_2 = \frac{\pi(9.05/12)^2}{4} = 0.4467 \text{ ft}^2$$

or, 0.04150 m^2.

The kinematic viscosity is:

$$\nu = \frac{1}{83.9192(8)^2 + 20{,}707.5(8) + 551{,}173} = 1.385(10)^{-6} \text{ m}^2\text{/s}$$

Assume that the linear expansion due to pipe and element temperature is negligible (which it probably is). Next, assume a starting C_d value of 0.6.

$$Q_1 = 0.6(0.0415)\frac{\sqrt{2g(0.519)(1.75-1)}}{\sqrt{1-(0.7542)^4}} = 0.0837 \text{ m}^3/\text{s}$$

The Reynold's number is:

$$R_e = \frac{4Q}{\pi D\nu} = \frac{4(0.0837)}{\pi(0.2299)(0.000001385)} = 334{,}700$$

The first calculated C_d value is:

$$\begin{aligned} C_d &= 0.5959 + 0.0312(0.7542)^{2.1} - 0.184(0.7542)^8 \\ &\quad + \frac{0.039(0.7542)^4}{1-(0.7542)^4} - 0.0158(0.7542)^3 + \frac{91.71(0.7542)^{2.5}}{(334{,}700)^{0.75}} \\ &= 0.609 \end{aligned}$$

The updated flow rate is:

$$Q_2 = 0.609(0.0415)\frac{\sqrt{2g(0.519)(1.75-1)}}{\sqrt{1-(0.7542)^4}} = 0.0850 \text{ m}^3/\text{s}$$

The updated Reynold's number is:

$$R_e = \frac{4Q}{\pi D\nu} = \frac{4(0.0850)}{\pi(0.2299)(0.000001385)} = 339{,}900$$

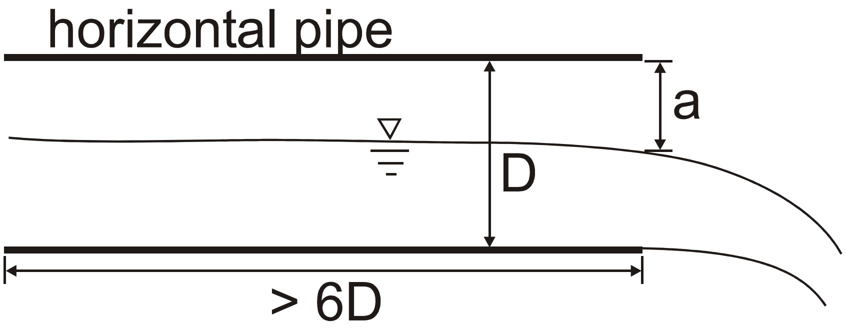
The first calculated C_d value is:

$$\begin{aligned} C_d &= 0.5959 + 0.0312(0.7542)^{2.1} - 0.184(0.7542)^8 \\ &\quad + \frac{0.039(0.7542)^4}{1-(0.7542)^4} - 0.0158(0.7542)^3 + \frac{91.71(0.7542)^{2.5}}{(339{,}900)^{0.75}} \\ &= 0.609 \end{aligned}$$

Thus, the coefficient has converged to within three significant digits after only one iteration. The flow rate is 0.0850 m^3/s (3.00 cfs).

IV. You have to estimate the discharge from a partially-full horizontal pipe which discharges freely into a canal. The end of the pipe is 20 cm above the water surface in the canal. The pipe inside diameter is 35 cm, and the depth of water at the pipe end is measured, giving 13 cm. Estimate the discharge in m^3/s.

Use the "California pipe method."



a/D = (35 - 13)/35 = 0.629 (which is greater than 0.45… OK), and

$$Q = 8.69\left(1-0.629\right)^{1.88}\left(\frac{0.35}{0.3048}\right)^{2.48} = 1.90 \text{ cfs}$$

or, 0.0538 m^3/s.

BIE 5300/6300 Assignment #7
Earthen Canal Design

28 Oct 04 (due 2 Nov 04)

Show your calculations in an organized and neat format, including all relevant calculations. Indicate any assumptions or relevant comments.

Given:

- An earthen canal is to be designed
- Accommodating the natural terrain, the longitudinal bed slope will be $S_o = 0.0001$
- Bed material is a non-cohesive fine sandy soil material with an average particle diameter of 2 mm
- The angle of repose for wet material is 26 degrees
- Assume a Manning n of 0.019
- Design discharge is $Q_{max} = 12$ m^3/s
- The source water has a low content of fine sediment (silt)
- The b/h ratio should be limited to values between 2 and 6
- The inverse side slope should be limited as follows: $m < 3.5$

Required:

- Design the earthen channel section by applying the tractive force method
- Compare your results with those using the assumption of a very wide channel, in which the critical tractive force is $\gamma h S_o$
- Compare your design with the velocity as obtained from the Kennedy formula
- Compare your design with the velocity as obtained from the Lacey method
- Compare your design with the velocity as obtained from the maximum velocity method, both for values by Fortier and Scobey, and by the USBR

A Design Solution:

<u>Critical Tractive Force</u>

- The critical tractive force is taken from Fig. 5 (non-cohesive material) of the lecture notes, using the curve labeled "low content of fine sediment."
- Instead of reading the graph by eye, use the appropriate equation from the lecture notes:

$$T_c = 0.0756(2)^3 - 0.241(2)^2 + 0.872(2) + 2.26$$
$$T_c \approx 3.64\ N/m^2$$

Angle of Repose

- The angle of repose, θ, is given as 26°
- Then, the ratio of T_{side} to T_{bed} is:

$$K = \frac{T_{side}}{T_{bed}} = \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}} = \sqrt{1 - 5.20\sin^2\phi}$$

- Design requirements for this example call for a side slope between 0.0 & 3.5
- Then,

$$\phi_{min} = \tan^{-1}\left(\frac{1}{3.5}\right) = 15.9^\circ$$

- Let ϕ_{min} = 16° (round to nearest whole degree)
- θ = 26° is the upper limit for ϕ, so: 16° ≤ ϕ ≤ 26°

- Make a table of tractive force ratio, K, values for the acceptable range of ϕ:

ϕ (deg)	**m**	**K**
16	3.487	*0.778*
17	3.271	*0.745*
18	3.078	*0.710*
19	2.904	*0.670*
20	2.747	*0.626*
21	2.605	*0.576*
22	2.475	*0.520*
23	2.356	*0.454*
24	2.246	*0.374*
25	2.145	*0.267*
26	2.050	*0.027*

Maximum Shear Stress Fractions

- These are K_{bed} and K_{side}
- The range of inverse side slopes is: 3.487 ≤ m ≤ 2.050
- Recall that the range of bed width to depth is: 2.0 ≤ b/h ≤ 6.0
- For the bed, apply the equations from the lecture notes, where K_{bed} is a function of the ratio b/h for trapezoidal cross sections…

b/h	**K_{bed}**
2.0	*0.881*
2.4	*0.906*
2.8	*0.927*
3.2	*0.946*
3.6	*0.963*
4.0	*0.969*
4.4	*0.971*
4.8	*0.973*
5.2	*0.975*
5.6	*0.977*
6.0	*0.980*

- For the sides, apply the equations from the lecture notes, where K_{side} is a function of inverse side slope, m, and the ratio b/h

		b/h										
φ (deg)	**m**	2.0	2.4	2.8	3.2	3.6	4.0	4.4	4.8	5.2	5.6	6.0
16	3.487	*0.794*	*0.793*	*0.791*	*0.789*	*0.788*	*0.787*	*0.786*	*0.785*	*0.784*	*0.784*	*0.783*
17	3.271	*0.805*	*0.806*	*0.807*	*0.807*	*0.808*	*0.809*	*0.809*	*0.810*	*0.810*	*0.811*	*0.811*
18	3.078	*0.808*	*0.812*	*0.815*	*0.818*	*0.820*	*0.822*	*0.824*	*0.825*	*0.827*	*0.828*	*0.830*
19	2.904	*0.807*	*0.812*	*0.816*	*0.820*	*0.823*	*0.826*	*0.828*	*0.831*	*0.833*	*0.835*	*0.837*
20	2.747	*0.800*	*0.806*	*0.811*	*0.815*	*0.819*	*0.822*	*0.824*	*0.827*	*0.829*	*0.831*	*0.833*
21	2.605	*0.791*	*0.797*	*0.802*	*0.806*	*0.810*	*0.812*	*0.815*	*0.817*	*0.819*	*0.821*	*0.822*
22	2.475	*0.782*	*0.787*	*0.792*	*0.796*	*0.799*	*0.801*	*0.803*	*0.805*	*0.807*	*0.808*	*0.809*
23	2.356	*0.772*	*0.777*	*0.782*	*0.785*	*0.788*	*0.790*	*0.792*	*0.793*	*0.795*	*0.796*	*0.797*
24	2.246	*0.763*	*0.768*	*0.772*	*0.775*	*0.778*	*0.780*	*0.781*	*0.783*	*0.784*	*0.785*	*0.786*
25	2.145	*0.755*	*0.760*	*0.764*	*0.767*	*0.770*	*0.771*	*0.773*	*0.774*	*0.775*	*0.776*	*0.777*
26	2.050	*0.748*	*0.754*	*0.757*	*0.760*	*0.762*	*0.764*	*0.766*	*0.767*	*0.768*	*0.768*	*0.769*

Note: values in italics are K_{side}

Maximum Allowable Water Depths

- These are h_{max} based on: (1) bed; and, (2) side slopes
- For water, use γ = 9,810 N/m^3
- The following table has h_{max} values based on K_{bed}:

	b/h										
m	2.0	2.4	2.8	3.2	3.6	4.0	4.4	4.8	5.2	5.6	6.0
3.487	*3.28*	*3.19*	*3.11*	*3.05*	*3.00*	*2.98*	*2.97*	*2.97*	*2.96*	*2.95*	*2.95*
3.271	*3.14*	*3.05*	*2.98*	*2.92*	*2.87*	*2.85*	*2.85*	*2.84*	*2.84*	*2.83*	*2.82*
3.078	*2.99*	*2.91*	*2.84*	*2.78*	*2.73*	*2.72*	*2.71*	*2.71*	*2.70*	*2.69*	*2.69*
2.904	*2.82*	*2.75*	*2.68*	*2.63*	*2.58*	*2.57*	*2.56*	*2.55*	*2.55*	*2.54*	*2.54*
2.747	*2.64*	*2.56*	*2.50*	*2.45*	*2.41*	*2.40*	*2.39*	*2.39*	*2.38*	*2.38*	*2.37*
2.605	*2.43*	*2.36*	*2.31*	*2.26*	*2.22*	*2.21*	*2.20*	*2.20*	*2.19*	*2.19*	*2.18*
2.475	*2.19*	*2.13*	*2.08*	*2.04*	*2.00*	*1.99*	*1.99*	*1.98*	*1.98*	*1.97*	*1.97*
2.356	*1.91*	*1.86*	*1.82*	*1.78*	*1.75*	*1.74*	*1.74*	*1.73*	*1.73*	*1.72*	*1.72*
2.246	*1.58*	*1.53*	*1.50*	*1.47*	*1.44*	*1.43*	*1.43*	*1.43*	*1.42*	*1.42*	*1.42*
2.145	*1.12*	*1.09*	*1.07*	*1.05*	*1.03*	*1.02*	*1.02*	*1.02*	*1.02*	*1.01*	*1.01*
2.050	*0.11*	*0.11*	*0.11*	*0.11*	*0.10*	*0.10*	*0.10*	*0.10*	*0.10*	*0.10*	*0.10*

Note: values in italics are h_{max} based on K_{bed}

- The next table has h_{max} values based on K_{side}:

	b/h										
m	2.0	2.4	2.8	3.2	3.6	4.0	4.4	4.8	5.2	5.6	6.0
3.487	*3.63*	*3.64*	*3.65*	*3.66*	*3.66*	*3.67*	*3.67*	*3.68*	*3.68*	*3.68*	*3.69*
3.271	*3.44*	*3.43*	*3.43*	*3.43*	*3.42*	*3.42*	*3.42*	*3.42*	*3.41*	*3.41*	*3.41*
3.078	*3.26*	*3.24*	*3.23*	*3.22*	*3.21*	*3.20*	*3.20*	*3.19*	*3.18*	*3.18*	*3.17*
2.904	*3.08*	*3.06*	*3.05*	*3.03*	*3.02*	*3.01*	*3.00*	*2.99*	*2.98*	*2.98*	*2.97*
2.747	*2.90*	*2.88*	*2.86*	*2.85*	*2.84*	*2.83*	*2.82*	*2.81*	*2.80*	*2.79*	*2.79*
2.605	*2.70*	*2.68*	*2.67*	*2.65*	*2.64*	*2.63*	*2.62*	*2.62*	*2.61*	*2.61*	*2.60*
2.475	*2.47*	*2.45*	*2.44*	*2.42*	*2.42*	*2.41*	*2.40*	*2.40*	*2.39*	*2.39*	*2.38*
2.356	*2.18*	*2.17*	*2.15*	*2.15*	*2.14*	*2.13*	*2.13*	*2.12*	*2.12*	*2.12*	*2.11*
2.246	*1.82*	*1.81*	*1.80*	*1.79*	*1.78*	*1.78*	*1.77*	*1.77*	*1.77*	*1.77*	*1.77*
2.145	*1.31*	*1.30*	*1.30*	*1.29*	*1.29*	*1.28*	*1.28*	*1.28*	*1.28*	*1.28*	*1.28*
2.050	*0.13*	*0.13*	*0.13*	*0.13*	*0.13*	*0.13*	*0.13*	*0.13*	*0.13*	*0.13*	*0.13*

Note: values in italics are h_{max} based on K_{side}

- The above two tables show that h_{max} from K_{side} is greater for every combination of b/h and m (within the given ranges)
- Therefore, use only the table for h_{max} from K_{side} to determine the maximum allowable depth of water
- Note that the full range of “m” and the full range of “b/h” is represented in the above two tables

Channel Base Width Limits

- Calculate the uniform flow (normal) depth for values of “m” from 2.050 to 3.487 (as shown in the above tables), and various values of base width, “b.”

	base width, b (m)										
m	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
3.487	*1.93*	*1.88*	*1.83*	*1.79*	*1.74*	*1.70*	*1.66*	*1.62*	*1.58*	*1.55*	*1.51*
3.271	*1.96*	*1.91*	*1.86*	*1.81*	*1.76*	*1.72*	*1.68*	*1.64*	*1.60*	*1.56*	*1.53*
3.078	*2.00*	*1.94*	*1.89*	*1.83*	*1.79*	*1.74*	*1.70*	*1.65*	*1.62*	*1.58*	*1.54*
2.904	*2.02*	*1.97*	*1.91*	*1.86*	*1.81*	*1.76*	*1.71*	*1.67*	*1.63*	*1.59*	*1.55*
2.747	*2.05*	*1.99*	*1.93*	*1.88*	*1.83*	*1.78*	*1.73*	*1.69*	*1.64*	*1.60*	*1.56*
2.605	*2.08*	*2.02*	*1.96*	*1.90*	*1.84*	*1.79*	*1.75*	*1.70*	*1.66*	*1.61*	*1.58*
2.475	*2.11*	*2.04*	*1.98*	*1.92*	*1.86*	*1.81*	*1.76*	*1.71*	*1.67*	*1.63*	*1.59*
2.356	*2.13*	*2.06*	*2.00*	*1.94*	*1.88*	*1.83*	*1.77*	*1.73*	*1.68*	*1.64*	*1.60*
2.246	*2.16*	*2.09*	*2.02*	*1.96*	*1.90*	*1.84*	*1.79*	*1.74*	*1.69*	*1.65*	*1.60*
2.145	*2.18*	*2.11*	*2.04*	*1.97*	*1.91*	*1.85*	*1.80*	*1.75*	*1.70*	*1.66*	*1.61*
2.050	*2.21*	*2.13*	*2.06*	*1.99*	*1.93*	*1.87*	*1.81*	*1.76*	*1.71*	*1.66*	*1.62*

Note: values in italics are depths, h, based on the Manning equation

- Next, divide the base width by each respective depth, over the range of inverse side slope values, giving a table of b/h ratios:

	base width, b (m)										
m	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
3.487	*2.07*	*2.39*	*2.73*	*3.08*	*3.44*	*3.82*	*4.22*	*4.63*	*5.05*	*5.49*	*5.94*
3.271	*2.04*	*2.35*	*2.69*	*3.04*	*3.40*	*3.78*	*4.17*	*4.58*	*5.00*	*5.44*	*5.89*
3.078	*2.00*	*2.32*	*2.65*	*3.00*	*3.36*	*3.73*	*4.13*	*4.53*	*4.95*	*5.39*	*5.84*
2.904	*1.98*	*2.29*	*2.62*	*2.96*	*3.32*	*3.70*	*4.08*	*4.49*	*4.91*	*5.34*	*5.79*
2.747	*1.95*	*2.26*	*2.59*	*2.93*	*3.29*	*3.66*	*4.05*	*4.45*	*4.87*	*5.30*	*5.75*
2.605	*1.92*	*2.23*	*2.56*	*2.90*	*3.25*	*3.62*	*4.01*	*4.41*	*4.83*	*5.26*	*5.71*
2.475	*1.90*	*2.21*	*2.53*	*2.87*	*3.22*	*3.59*	*3.98*	*4.38*	*4.80*	*5.23*	*5.68*
2.356	*1.88*	*2.18*	*2.50*	*2.84*	*3.19*	*3.56*	*3.95*	*4.35*	*4.76*	*5.20*	*5.64*
2.246	*1.85*	*2.16*	*2.48*	*2.81*	*3.16*	*3.53*	*3.92*	*4.32*	*4.73*	*5.16*	*5.61*
2.145	*1.83*	*2.14*	*2.45*	*2.79*	*3.14*	*3.51*	*3.89*	*4.29*	*4.70*	*5.13*	*5.58*
2.050	*1.81*	*2.11*	*2.43*	*2.76*	*3.11*	*3.48*	*3.86*	*4.26*	*4.68*	*5.11*	*5.55*

Note: values in italics are ratios of b/h, using h from the Manning equation

- These calculations show that the range of base widths is limited (approximately) as follows (with b in meters):

$$4 < b < 9$$

whereby b < 4 gives b/h less than the minimum of 2, and b > 9 gives b/h greater than the maximum of 6, as specified for this problem

Ratios of h to h_{max}

- Divide depths, h, from Manning by h_{max} based on K_{side}
- Only values less than unity are acceptable for this ratio

	base width, b (m)										
m	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
3.487	*0.53*	*0.52*	*0.50*	*0.49*	*0.48*	*0.46*	*0.45*	*0.44*	*0.43*	*0.42*	*0.41*
3.271	*0.57*	*0.56*	*0.54*	*0.53*	*0.52*	*0.50*	*0.49*	*0.48*	*0.47*	*0.46*	*0.45*
3.078	*0.61*	*0.60*	*0.58*	*0.57*	*0.56*	*0.54*	*0.53*	*0.52*	*0.51*	*0.50*	*0.49*
2.904	*0.66*	*0.64*	*0.63*	*0.61*	*0.60*	*0.58*	*0.57*	*0.56*	*0.54*	*0.53*	*0.52*
2.747	*0.71*	*0.69*	*0.67*	*0.66*	*0.64*	*0.63*	*0.61*	*0.60*	*0.59*	*0.57*	*0.56*
2.605	*0.77*	*0.75*	*0.73*	*0.71*	*0.70*	*0.68*	*0.66*	*0.65*	*0.63*	*0.62*	*0.60*
2.475	*0.85*	*0.83*	*0.81*	*0.79*	*0.77*	*0.75*	*0.73*	*0.71*	*0.70*	*0.68*	*0.66*
2.356	*0.97*	*0.95*	*0.92*	*0.90*	*0.88*	*0.85*	*0.83*	*0.81*	*0.79*	*0.77*	*0.75*
2.246	***1.18***	***1.15***	***1.12***	***1.09***	***1.06***	***1.03***	***1.00***	*0.98*	*0.95*	*0.93*	*0.91*
2.145	***1.66***	***1.61***	***1.57***	***1.52***	***1.48***	***1.44***	***1.40***	***1.36***	***1.33***	***1.30***	***1.26***
2.050	***16.51***	***16.04***	***15.58***	***15.13***	***14.71***	***14.30***	***13.90***	***13.53***	***13.17***	***12.84***	***12.52***

Note: values in bold are ratios of h/h_{max} that exceed unity, and are therefore unacceptable

- The above table shows that acceptable inverse side slopes for this design problem are (rounding to two significant digits):

$$2.3 < m < 3.5$$

- The above table also shows that the base width can be any value between 4 and 9 m
- These ranges of base width and inverse side slope represent the domain of feasible design solutions for this channel
- It would often be best to limit the width of the channel, possibly choosing a value of b = 4, and accepting a somewhat greater depth of water for uniform flow

Very Wide Channel

- With this assumption, $T_c = \gamma h S_o$
- Then,

$$h_{max} = \frac{T_c}{\gamma S_o} = \frac{3.64}{9{,}810(0.0001)} = 3.71\,\text{m}$$

- The largest h_{max} value based on K_{side}, for the acceptable range of m and b, is 3.69 m (see the above table)
- Therefore, the "very wide channel" solution is less restrictive than the previous solution and will not have any bearing on the range of feasible m and b values

Kennedy Formula

- For a "fine sandy soil," $C_1 = 0.84$
- For water containing "fine silt," $C_2 = 0.64$
- Applying the Kennedy formula:

$$V_o = 0.84(0.3048)\left(\frac{h}{0.3048}\right)^{0.64}$$

where V_o is the "regime" velocity (m^3/s); and h is depth (m) from the Manning equation

- Multiply V_o by area, A, to obtain flow rate according to the Kennedy formula, where:

$$A = h(b + mh)$$

for a trapezoidal channel section

- The following table gives Q values (m^3/s) based on depths from the Manning equation and V_o from the Kennedy formula for the previously-established range of acceptable b and m values:

	base width, b (m)										
m	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
3.487	*17.3*	*17.1*	*16.9*	*16.6*	*16.4*	*16.2*	*16.1*	*15.9*	*15.7*	*15.6*	*15.5*
3.271	*17.3*	*17.0*	*16.8*	*16.6*	*16.4*	*16.2*	*16.0*	*15.8*	*15.7*	*15.5*	*15.4*
3.078	*17.2*	*17.0*	*16.7*	*16.5*	*16.3*	*16.1*	*15.9*	*15.8*	*15.6*	*15.4*	*15.3*
2.904	*17.2*	*16.9*	*16.7*	*16.5*	*16.3*	*16.1*	*15.9*	*15.7*	*15.5*	*15.4*	*15.2*
2.747	*17.2*	*16.9*	*16.7*	*16.4*	*16.2*	*16.0*	*15.8*	*15.6*	*15.5*	*15.3*	*15.2*
2.605	*17.1*	*16.9*	*16.6*	*16.4*	*16.2*	*15.9*	*15.8*	*15.6*	*15.4*	*15.3*	*15.1*
2.475	*17.1*	*16.8*	*16.6*	*16.3*	*16.1*	*15.9*	*15.7*	*15.5*	*15.4*	*15.2*	*15.1*
2.356	*17.1*	*16.8*	*16.6*	*16.3*	*16.1*	*15.9*	*15.7*	*15.5*	*15.3*	*15.2*	*15.0*
2.246	*17.1*	*16.8*	*16.5*	*16.3*	*16.0*	*15.8*	*15.6*	*15.4*	*15.3*	*15.1*	*15.0*
2.145	*17.1*	*16.8*	*16.5*	*16.2*	*16.0*	*15.8*	*15.6*	*15.4*	*15.2*	*15.1*	*14.9*
2.050	*17.1*	*16.8*	*16.5*	*16.2*	*16.0*	*15.8*	*15.6*	*15.4*	*15.2*	*15.1*	*14.9*

Note: values in italics are flow rate (m^3/s) based on V_o from Kennedy ($Q = V_oA$)

- Note that all flow rates in the above table are greater than Q_{max} (12 m^3/s)
- This means that channel scouring would not be expected, at the design discharge, according to the Kennedy formula
- However, some sediment deposition might occur, assuming the Kennedy formula is correct for these site-specific conditions
- Recall that the Kennedy formula is 100% empirical

Lacey Method

- The average particle diameter is given as d_m = 2 mm
- Then, the “f” value from Lacey is:

$$f = 1.76\sqrt{2} = 2.49$$

- Longitudinal bed slope for “regime” flow:

$$S = 0.000547\left(\frac{f^{2/3}}{Q^{1/6}}\right) = 0.000547\left(\frac{2.49^{2/3}}{(12*35.31)^{1/6}}\right) = 0.00037$$

- This is greater than the specified design bed slope of 0.0001
- Thus, Lacey’s method would predict some scouring at the design discharge
- Note that other relationships from the Lacey method could also be checked
- However, recall that the Lacey method is 100% empirical

Maximum Velocity Method

- Refer to the tables from the lecture notes
- From Fortier & Scobey, use the column for “water with colloidal silt”: the value for fine sand is V_{max} = 2.5 fps (0.76 m/s)

- The following table gives average velocity (m/s) at a channel cross section for uniform flow conditions, where V = Q/A, and Q = 12 m^3/s:

	base width, b (m)										
m	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
3.487	*0.58*	*0.58*	*0.57*	*0.57*	*0.57*	*0.57*	*0.57*	*0.56*	*0.56*	*0.56*	*0.55*
3.271	*0.59*	*0.58*	*0.58*	*0.58*	*0.58*	*0.58*	*0.57*	*0.57*	*0.57*	*0.56*	*0.56*
3.078	*0.59*	*0.59*	*0.59*	*0.59*	*0.58*	*0.58*	*0.58*	*0.58*	*0.57*	*0.57*	*0.57*
2.904	*0.60*	*0.60*	*0.60*	*0.59*	*0.59*	*0.59*	*0.58*	*0.58*	*0.58*	*0.58*	*0.57*
2.747	*0.61*	*0.60*	*0.60*	*0.60*	*0.60*	*0.59*	*0.59*	*0.59*	*0.58*	*0.58*	*0.58*
2.605	*0.61*	*0.61*	*0.61*	*0.60*	*0.60*	*0.60*	*0.60*	*0.59*	*0.59*	*0.58*	*0.58*
2.475	*0.62*	*0.62*	*0.61*	*0.61*	*0.61*	*0.60*	*0.60*	*0.60*	*0.59*	*0.59*	*0.59*
2.356	*0.62*	*0.62*	*0.62*	*0.62*	*0.61*	*0.61*	*0.61*	*0.60*	*0.60*	*0.59*	*0.59*
2.246	*0.63*	*0.63*	*0.62*	*0.62*	*0.62*	*0.61*	*0.61*	*0.61*	*0.60*	*0.60*	*0.59*
2.145	*0.63*	*0.63*	*0.63*	*0.62*	*0.62*	*0.62*	*0.61*	*0.61*	*0.61*	*0.60*	*0.60*
2.050	*0.64*	*0.64*	*0.63*	*0.63*	*0.63*	*0.62*	*0.62*	*0.61*	*0.61*	*0.60*	*0.60*

Note: values in italics are average velocity (m/s)

- In this case, all average velocity values within the feasible range (tractive force) of m and b are below 0.76 m/s
- Thus, no scouring is to be expected, although perhaps some sediment deposition might occur, according to Fortier & Scobey
- Again, the maximum velocity method is 100% empirical and will not provide accurate results in all cases
- From the USBR, an average particle diameter of 2 mm falls under the “coarse sand” category, giving a maximum velocity of V_{max} = 1.8 fps (0.55 m/s)
- This is more restrictive than the V_{max} from Fortier & Scobey, and also exceeds almost all of the velocity values from the above table, but not by very much

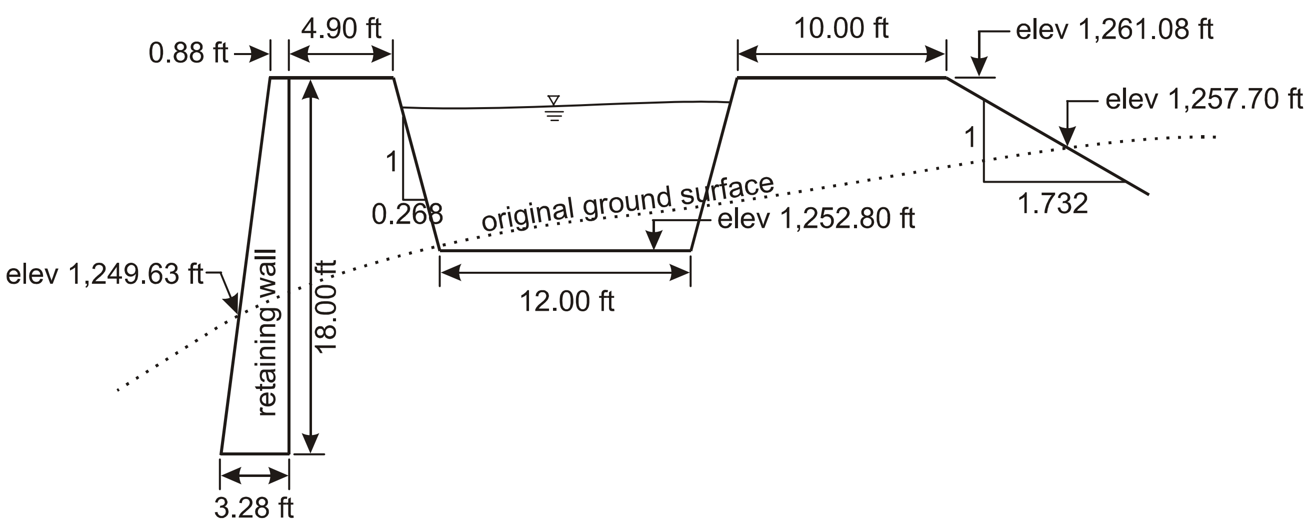
BIE 5300/6300 Assignment #8
Culvert Design

16 Nov 04 (due 23 Nov 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments.

Given:

- A culvert is to be designed to carry cross-drainage water under a new irrigation canal (see the drawing below).
- The outlet of the culvert will protrude through the retaining wall on the downhill side.
- The canal will be concrete lined, with a lining thickness of 2.5 inches.



- The natural channel (a gulch) which carries the cross-drainage flow has a well-defined and fairly stable cross-section.
- The alignment of the gulch is orthogonal to the canal alignment.
- The design flow rate for the culvert will be 100 cfs.
- Circular concrete pipe will be used for the culvert barrel, and it comes with inside diameters of 2.0 and 3.0 ft.
- Let the upper top of the inlet to the culvert barrel be located at or just below elevation 1,257.70 ft (this will require some excavation work just upstream of the culvert inlet, but you don't have enough information here to know how much excavation will be needed).
- Let the upper top of the outlet from the culvert barrel be located at or just below elevation 1,249.63 ft.

Required:

- Design a culvert for cross-drainage under the canal.
- Follow USBR design guidelines, as given in the “Small Canal Structures” book.
- Use English units for this design.
- Attempt to make the design so that an energy dissipation structure is not required at the culvert outlet. Don’t have supercritical flow at the outlet.
- Determine the number of culvert barrels (in parallel) for this design.
- Determine the slope(s) of the culvert barrel from inlet to outlet.
- Specify collar size and locations.
- Specify the total length of concrete pipe required for the barrel in your design.
- Specify a standard USBR design for the inlet & outlet transitions.
- Make a drawing showing the location of the culvert barrel relative to the canal cross section and retaining wall, and the location of collars.
- Show your calculations and units, indicate any important assumptions, and provide any important comments you might have about the design.

Solution:

1. Determine the Required Pipe Size

- According to USBR design guidelines, use a maximum average barrel velocity of 10.0 fps
- Then, for the design discharge of 100 cfs:

$$D = \sqrt{\frac{4Q}{\pi V}} = \sqrt{\frac{4(100)}{\pi(10)}} = 3.57 \text{ ft} \tag{1}$$

- The available pipe sizes are 2 and 3 feet, inside diameter.
- Try two 2-ft pipes:

$$V = \frac{0.5(100 \text{ cfs})}{\left(\frac{\pi(2)^2}{4}\right)} = 15.9 \text{ fps} \tag{2}$$

which is too high.

- Try two 3-ft pipes:

$$V = \frac{0.5(100 \text{ cfs})}{\left(\frac{\pi(3)^2}{4}\right)} = 7.07 \text{ fps} \tag{3}$$

which is acceptable. Therefore, use two 3-ft pipes for this culvert design, giving two barrels.

2. Determine the Energy Loss Gradient

- With the full pipe flow impending, the energy loss gradient can be estimated by the Manning equation for open-channel flow, in which h = D
- Use a Manning n value of 0.015 for new concrete pipe, with a slight safety factor for aging:

$$W_p = \pi D = \pi(3.0) = 9.425 \text{ ft} \tag{4}$$

$$A = \frac{\pi D^2}{4} = \frac{\pi(3.0)^2}{4} = 7.069 \text{ ft} \tag{5}$$

$$S_f = \left(\frac{Qn}{1.49}\right)^2 \left(\frac{W_p^{4/3}}{A^{10/3}}\right) = \left(\frac{(50)(0.015)}{1.49}\right)^2 \left(\frac{(9.425)^{4/3}}{(7.069)^{10/3}}\right) = 0.00744 \tag{6}$$

where half of the design flow rate is used per barrel.

3. Determine the Critical Slope

- For critical flow, the Froude number is equal to unity:

$$F_r^2 = \frac{Q^2 T_c}{g A_c^3} = 1.0 \tag{7}$$

- For circular pipes, the following definitions apply:

$$\beta_c = 2\cos^{-1}\left(1 - \frac{2h_c}{D}\right) \tag{8}$$

$$T_c = D\sin\left(\frac{\beta_c}{2}\right) \tag{9}$$

$$A_c = \frac{D^2}{8}\left(\beta_c - \sin\beta_c\right) \tag{10}$$

- Solve for depth, h, such that F_r^2 = 1.0 for Q = 50 cfs and D = 3.0 ft (for English units, g ≈ 32.2 ft/s^2)

- Using the Newton method, β_c = 4.268 rad, and <u>h_c = 2.301 ft</u>
- Calculate the energy loss gradient (critical slope) corresponding to this depth
- For a depth of 2.301 ft, the flow cross-sectional area is 5.818 ft^2, and the wetted perimeter is 6.402 ft
- Applying the Manning equation:

$$\left(S_f\right)_{crit} = \left(\frac{(50)(0.015)}{1.49}\right)^2 \left(\frac{(6.402)^{4/3}}{(5.818)^{10/3}}\right) = 0.00850 \qquad (11)$$

- If the slope of the pipe is 0.00850 or greater, critical flow can occur

4. <u>Determine the Minimum Upstream Pipe Slope</u>

- According to USBR guidelines, the top of the culvert barrel (pipe) should clear the bottom of the canal by at least 0.5 ft
- Note that if more information were provided about the uphill topography, it would also be possible to move the culvert inlet uphill, away from the canal berm
- The elevation of the inside top of the barrel at the culvert inlet is 1,257.70 ft, as specified above
- Assume a pipe thickness of 2 inches
- Recall the 2.5-inch canal lining thickness (specified)
- The elevation of the inside top of the barrel at the right canal base:

1,252.80 - (2.5+2.0)/12 - 0.50 = 1,251.93 ft (12)

- The horizontal distance of the steep descending part of the culvert barrel:

1.732(1,261.08 - 1,257.70) + 10.00 + 0.268(1,261.08 - 1,252.80) = 18.07 ft (13)

- The minimum slope of the steep descending part of the culvert barrel:

$$\text{slope} = \frac{1{,}257.70 - 1{,}251.93}{18.07} = \frac{5.77}{18.07} = 0.319 \qquad (14)$$

- It is also necessary to check that the slope of the downstream pipe does not exceed the critical slope

5. <u>Determine the Downstream Pipe Slope</u>

- The flatter downstream part of the culvert barrel would traverse a horizontal distance of:

Length = 12.00 + 0.268(1,261.08 - 1,252.80) + 4.90 + 0.88 +

(1,261.08 - 1,249.63)(3.28 - 0.88)/18.00 = <u>21.53 ft</u>

- The change in elevation over this distance will be 1,251.93 - (1,249.63 - 2/12) = 1,251.93 - 1,249.46 = 2.47 ft (where the 2/12 value is the assumed 2 inches of pipe wall thickness)
- Then, the slope of the flatter downstream part of the barrel would be 2.47/21.53 = 0.115
- Also, the total horizontal distance is, then: 18.07 + 21.53 = 39.60 ft

- This slope is greater than the critical slope, and is not acceptable because it would cause supercritical flow throughout, from inlet to outlet, causing erosion downstream (unless erosion protection is used)

- Use the USBR-recommended downstream slope of 0.005 (0.5%), which is less than the calculated critical slope of 0.850%
- To accomplish this, the upstream (steep) portion of the culvert pipe can be extended further in the downstream direction (to the left)
- Linear equations can be written for the inside tops of the upstream & downstream barrel segments:

  Upstream: .... $y = 0.319x + 1{,}245.07$

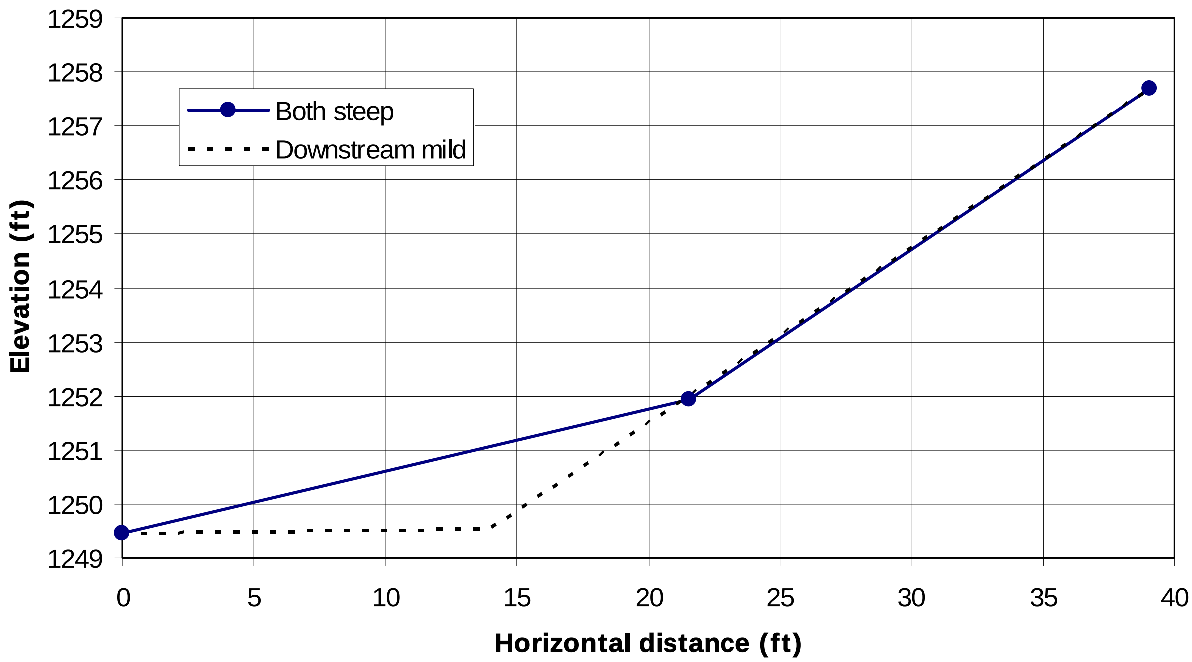
  Downstream:.............................. $y = 0.005x + 1{,}249.46$

  where the intercept in the upstream equation is calculated as 1,257.70 - (0.319)(39.60) = 1,245.07 ft

- Solving the two linear equations for distance:

$$x = \frac{1{,}249.46 - 1{,}245.07}{0.319 - 0.005} = \frac{4.39}{0.314} = 13.98 \text{ ft} \tag{15}$$

- This is the distance from the outlet at which the inside top of the upstream pipe intersects the inside top of the downstream pipe
- The elevation of the intersection point is y = 0.319(13.98) + 1,245.07 = 1,249.53 ft, and the steep part of the barrel has the same slope as before
- Below, a profile of the inside top of the culvert barrel is shown



6. *Determine the Pipe Length*

- The approximate length of the upstream (steep) pipe is:

$$L_{us} = \sqrt{(39.60 - 13.98)^2 + (1{,}257.70 - 1{,}249.53)^2} = 26.89 \text{ ft} \tag{16}$$

- The approximate length of the downstream (mild) pipe is:

$$L_{ds} = \sqrt{(13.98)^2 + (1{,}249.53 - 1{,}249.46)^2} = 13.98 \text{ ft} \tag{17}$$

- A total length of 2(26.89 + 13.98) ≈ 82 ft of 3-ft diameter pipe is needed

7. *Specify Inlet and Outlet Types*

- The inlet and outlet can be USBR Type 1 (well-defined earthen section)
- The capacity of the downstream (mild) part of the barrel at impending full pipe flow (but assuming open-channel flow) is approximated as:

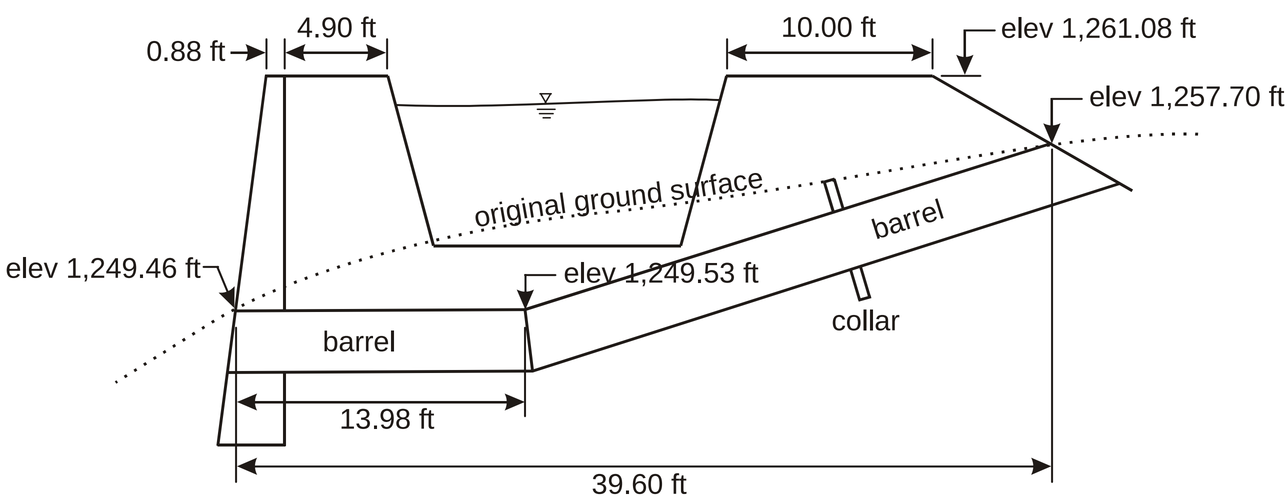
$$Q = \frac{1.49}{0.015} \frac{(7.069)^{5/3}}{(9.425)^{2/3}} \sqrt{0.005} = 41 \text{ cfs} \tag{18}$$

- Then, at the design capacity of 50 cfs (per barrel), the downstream portion of the barrel would flow full and there would be a hydraulic jump inside the upstream (steep) part of the barrel
- The outlet velocity would be approximately 7.07 fps (see Eq. 3 above)
- An energy dissipation structure at the outlet is not needed (because the outlet velocity will be < 15 fps)

8. <u>Specify Collar Placement and Size</u>

- The standard USBR culvert design, calling for two collars under the downhill canal bank, and one collar under the uphill bank, is not appropriate in this design because of the downstream retaining wall
- Use a single collar under the upstream canal bank, with Y = ½D = 1.5 ft, and a thickness of 6 inches
- Note that some excavation will be required at the downhill side of the retaining wall because the top of the culvert barrel has been set at the elevation of 1,249.63 (the inside top is about 2 inches lower, in this design)

9. <u>Side View Drawing</u>



Sprinkle & Trickle Irrigation
Lecture Notes

BIE 5110/6110
Fall Semester 2004

Biological and Irrigation Engineering Department
Utah State University, Logan, Utah

Preface

These lecture notes were prepared by Gary P. Merkley of the Biological and Irrigation Engineering (BIE) Department at USU, and Richard G. Allen of the University of Idaho, for use in the BIE 5110/6110 courses. The notes are intended to supplement and build upon the material contained in the textbook *Sprinkle and Trickle Irrigation* by Jack Keller and Ron D. Bliesner (Chapman-Hall Publishers 1990). Due to the close relationship between the lecture notes and the textbook, some equations and other material presented herein is taken directly from Keller and Bliesner (1990) – in these instances the material is the intellectual property of the textbook authors and should be attributed to them. In all other cases, the material contained in these lecture notes is the intellectual property right of G.P. Merkley and R.G. Allen.

Copyright Notice

These lecture notes are formatting for printing on both sides of the page, with odd-numbered pages on the front. Each lecture begins on an odd-numbered page, so some even-numbered pages are blank.

Contents

Note: Equations are numbered consecutively in these lecture notes as (xxx). Equations with the (xxx.xx) format refer to those found in the textbook by Keller & Bliesner.

Units, Constants and Conversions

28.35 g/oz
15.85 gpm/lps (= 60/3.785)
7.481 gallons/ft^3
448.86 gpm/cfs (= 7.481*60)
3.7854 litre/gallon

6.89 kPa/psi
1 cb = 1 kPa
10 mb/kPa, or 100 kPa/bar
2.308 ft/psi, or 9.81 kPa/m (head of water)
14.7 psi = 101.3 kPa = 10.34 m (head of water) = 1,013 mbar = 1 atm
62.4 lbs/ft^3, or 1000 kg/m^3 (max density of pure water at 4°C)
0.1333 kPa/mmHg

1 ppm ≈ 1 mg/liter (usually)
1 mmho/cm = 1 dS/m = 550 to 800 mg/liter

0.7457 kW/HP
1 langley = 1 cal/cm^2
0.0419 MJ/m^2 per cal/cm^2

0.3048 m/ft
1.609 km/mile
2.471 acre/ha
43,560 ft^2/acre
1,233 m^3/acre-ft

57.2958 degrees/radian
$\pi \approx 3.14159265358979323846$
$e \approx 2.71828182845904523536$

ºC = (ºF – 32)/1.8
ºF = 1.8(ºC) + 32

Ratio of weight to mass at sea level and 45° latitude: $g = 9.80665$ m/s^2

PVC = Polyvinyl chloride
PE = Polyethylene
ABS = Acrylonitrile-Butadiene-Styrene

Equation Chapter 1 Section 1**Lecture 1**

Course Introduction

I. Course Overview

- Design of sprinkle and trickle systems – perhaps the most comprehensive course on the subject anywhere
- Previously, this was two separate courses at USU
- Everyone must be registered at least for audit
- Prerequisites: BIE 5010/6010; computer programming; hydraulics
- There will be two laboratory/field exercises
- Review of lecture schedules for sprinkle and trickle

II. Textbook and Other Materials

- Textbook by Keller and Bliesner
- Two textbooks are on reserve in the Merrill Library
- Lecture notes by Merkley and Allen are required
- We will also use other reference materials during the semester

III. Homework and Design Project

- Work must be organized and neat
- Working in groups is all right, but turn in <u>your own work</u>
- Computer programming and spreadsheet exercises
- Submitting work late (10% per day, starting after class)
- Sprinkle or trickle system design project

IV. Tests, Quizzes, and Grading Policy

- Maybe some quizzes (these will not be announced)
- Two mid-term exams
- Final exam is comprehensive

V. Units

- It is often necessary to convert units in design calculations
- Make it a habit to perform dimensional analysis when using equations; only in some of the empirical equations will the units not work out correctly

VI. Irrigation Systems

- On-farm level (field)
- Project level (storage, conveyance, tertiary)

VII. General Types of On-Farm Irrigation Systems

Type	U.S. Area	World Area
Surface	65%	95%
Sprinkler	30%	3%
Micro Irrigation	3%	1%
Sub-Irrigation	2%	1%

These are approximate percent areas

VIII. Sprinkler Systems

Important Advantages

1. effective use of small continuous streams of water
2. greater application uniformity on non-homogeneous soils (provided there is no appreciable surface runoff)
3. ability to adequately irrigate steep or undulating topographies w/o erosion
4. good for light and frequent irrigation where surface irrigation may be used later in the growing season
5. labor is only needed for a short time each day (unless there are many fields)
6. labor can be relatively unskilled (pipe moving)
7. automation is readily available for many sprinkler systems
8. can be effective for weather (micro-climate) modification

Important Disadvantages

1. initial cost can be high (compared to surface irrigation systems) at $500 to $3500 per ha
2. operating cost (energy) can be high compared to non-pressurized systems, unless sufficient head is available from a gravity-fed supply
3. water quality can be a problem with overhead sprinklers if water is saline, and in terms of clogging and nozzle wear. Also, some types of water are corrosive to sprinkler pipes and other hardware
4. some fruit crops cannot tolerate wet conditions during maturation (unless fungicides, etc., are used)
5. fluctuating flow rates at the water source can be very problematic
6. irregular field shapes can be difficult to accommodate
7. very windy and very dry conditions can cause high losses
8. low intake rate soils (< 3 mm/hr) cannot be irrigated by sprinkler w/o runoff

IX. Slides of Sprinkler Systems

[these will be shown in class]

Lecture 2

Types of Sprinkler Systems

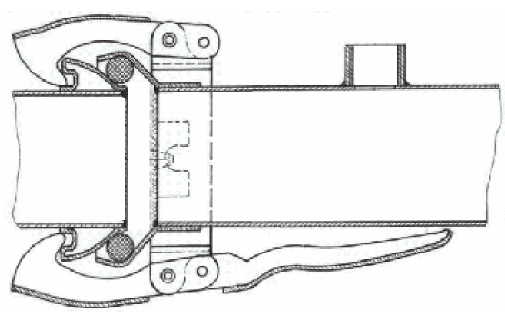
I. Sprinkler System Categories

- Two broad categories: ***set*** and ***continuous-move***
- Set systems can be further divided into: ***fixed*** and ***periodic-move***

II. Set Systems:

Hand-Move

- very common type of sprinkler system
- costs about $30 - $90 per acre, or $75 - $220 per ha
- requires relatively large amount of labor
- laterals are usually aluminum: strong enough, yet light to carry
- usually each lateral has one sprinkler (on riser), at middle or end of pipe
- to move, pull end plug and begin draining of line, then pull apart
- lateral pipe is typically 3 or 4 inches in diameter
- usually for periodic move, but can be set up for a fixed system
- sprinklers are typically spaced at 40 ft along the pipe
- laterals are typically moved at 50- or 60-ft intervals along mainline



End-Tow

- similar to hand-move, but pipes are more rigidly connected
- tractor drags the lateral from position to position, straddling a mainline
- has automatically draining values (open when pressure drops)
- pipe is protected from wear during dragging by mounting it on skid plates or small wheels
- least expensive of the mechanically-moved systems
- needs a 250-ft (75-m) “turning strip” about the mainline

Side-Roll

- very common in western U.S.
- costs about $150 - $300 per acre, or $360 - $750 per ha
- wheels are usually 65 or 76 inches in diameter

- lateral is the axle for the wheels; lateral pipe may have thicker walls adjacent to a central "mover" to help prevent collapse of the pipe during moving
- uses "movers" or motorized units to roll the lateral; these may be mounted in middle and or at ends, or may be portable unit that attaches to end of line
- self-draining when pressure drops
- must drain lines before moving, else pipe will break
- windy conditions can cause difficulties when moving the lateral, and can blow empty lateral around the field if not anchored down
- can have trail tubes (*drag lines*) with one or two sprinklers each
- need to "dead-head" back to the starting point
- mainline is often portable
- has swivels at sprinkler and trail tube locations to keep sprinklers upright
- low growing crops only (lateral is about 3 ft above ground)
- can be automated, but this is not the typical case

Side-Move

- almost the same as side-roll, but lateral pipe is not axle: it is mounted on A frames with two wheels each
- clearance is higher than for side-roll
- not as common as side-roll sprinklers

Gun

- 5/8-inch (16 mm) or larger nozzles
- rotate by rocker arm mechanism
- discharge is 100 to 600 gpm at 65 to 100 psi
- large water drops; commonly used on pastures, but also on other crops

Boom

- have big gun sprinklers mounted on rotating arms, on a trailer with wheels
- arms rotate due to jet action from nozzles
- arms supported by cables
- large water drops; commonly used on pastures, but also on other crops

Other Set Sprinklers

- Perforated Pipe
- Hose-Fed Sprinklers
- Orchard Sprinklers

Fixed (Solid-Set) Systems

- enough laterals to cover entire field at same time
- will not necessarily irrigate entire field at the same time, but if you do, a larger system capacity is needed
- only fixed systems can be used for: *frost protection, crop cooling, blossom delay*
- easier to automate that periodic-move systems

- laterals and mainline can be portable and above ground (aluminum), or permanent and buried (PVC or steel, or PE)

III. Continuous-Move Systems

Traveler

- could be big gun or boom on platform with wheels
- usually with a big gun (perhaps 500 gpm & 90 psi) sprinkler
- long flexible hose with high head loss
- may reel up the hose or be pulled by a cable
- large water drops; commonly used on pastures, but also on other crops
- some travelers pump from open ditch, like linear moves
- sprinkler often set to part circle so as not to wet the travel path

Center Pivot

- cost is typically $35,000 ($270/ac or $670/ha), plus $15,000 for corner system
- easily automated
- typical maximum (fastest) rotation is about 20 hrs
- don't rotate in 24-hr increment because wind & evaporation effects will concentrate
- returns to starting point after each irrigation
- typical lateral length is 1320 ft (400 m), or ¼ mile (quarter "section" area)
- laterals are about 10 ft above the ground
- typically 120 ft per tower (range: 90 to 250 ft) with one horsepower electric motors (geared down)
- IPS 6" lateral pipe is common (about 6-5/8 inches O.D.); generally 6 to 8 inches, but can be up to 10 inches for 2640-ft laterals
- typical flow rates are 45 - 65 lps (700 to 1000 gpm)
- typical pressures are 140 - 500 kPa (20 to 70 psi)
- older center pivots can have water driven towers (spills water at towers)
- end tower sets rotation speed; micro switches & cables keep other towers aligned
- corner systems are expensive; can operate using buried cable; corner systems don't irrigate the whole corner
- w/o corner system, $\pi/4$ = 79% of the square area is irrigated
- for 1320 ft (not considering end gun), area irrigated is 125.66 acres
- with corner system, hydraulics can be complicated due to end booster pump
- center pivots are ideal for allowing for effective precipitation
- ignore soil water holding capacity (WHC)
- requires almost no labor; but must be maintained, or it will break down
- can operate on very undulating topography
- known to run over farmers' pickups (when they leave it out there)!
- many variations: overhead & underneath sprinklers; constant sprinkler spacing; varied sprinkler spacing; hoses in circular furrows, etc.
- sprinkler nearest the pivot point may discharge only a fine spray; constant radial velocity but variable tangential speeds (fastest at periphery)
- some center pivots can be moved from field to field

Linear Move

- costs about $40,000 for 400 m of lateral

- field must be rectangular in shape
- typically gives high application uniformity
- usually guided by cable and trip switches (could be done by laser)
- usually fed by open ditch with moving pump, requiring very small (or zero slope) in that direction
- can also be fed by dragging a flexible hose, or by automated arms that move sequentially along risers in a mainline
- need to "dead-head" back to other side of field, unless half the requirement is applied at each pass
- doesn't have problem of variable tangential speeds as with center pivots

IV. LEPA Systems

- ***L**ow **E**nergy **P**recision **A**pplication* (LEPA) is a concept developed in the mid to late 1970s in the state of Texas to conserve water and energy in pressurized irrigation systems
- The principal objective of the technology was to make effective use of all available water resources, including the use of rainfall and minimization of evaporation losses, and by applying irrigation water near the soil surface
- Such applications of irrigation water led to sprinkler system designs emphasizing lower nozzle positions and lower operating pressures, thereby helping prevent drift and evaporative losses and decreasing pumping costs
- For example, many center pivot systems with above-lateral sprinklers have been refitted to position sprinkler heads under the lateral, often with lower pressure nozzle designs
- The commercialization of the LEPA technology has led to many modifications and extensions to the original concept, and is a term often heard in discussions about agricultural sprinkler systems
- The LEPA concept can be applied in general to all types of sprinkler systems, and to many other types of irrigation systems

Soil-Water-Plant Relationships

I. Irrigation Depth

$$d_x = \frac{MAD}{100} W_a Z \tag{1}$$

where d_x is the maximum net depth of water to apply per irrigation; MAD is *management allowed deficit* (usually 40% to 60%); W_a is the water holding capacity, a function of soil texture and structure, equal to FC – WP (field capacity minus wilting point); and Z is the root depth

- For most agricultural soils, field capacity (FC) is attained about 1 to 3 days after a complete irrigation
- The d_x value is the same as "*allowable depletion*." Actual depth applied may be less if irrigation frequency is higher than needed during peak use period.
- MAD can also serve as a safety factor because many values (soil data, crop data, weather data, etc.) are not precisely known
- Assume that crop yield and crop ET begins to decrease below maximum potential levels when actual soil water is below MAD (for more than one day)
- Water holding capacity for agricultural soils is usually between 10% and 20% by volume
- W_a is sometimes called "TAW" (total available water), "WHC" (water holding capacity), "AWHC" (available water holding capacity)
- Note that it may be more appropriate to base net irrigation depth calculations on soil water *tension* rather than soil water content, also taking into account the crop type – this is a common criteria for scheduling irrigations through the use of tensiometers

II. Irrigation Interval

- The maximum irrigation frequency is:

$$f_x = \frac{d_x}{U_d} \tag{2}$$

where f_x is the *maximum* interval (frequency) in days; and U_d is the average daily crop water requirement during the peak-use period

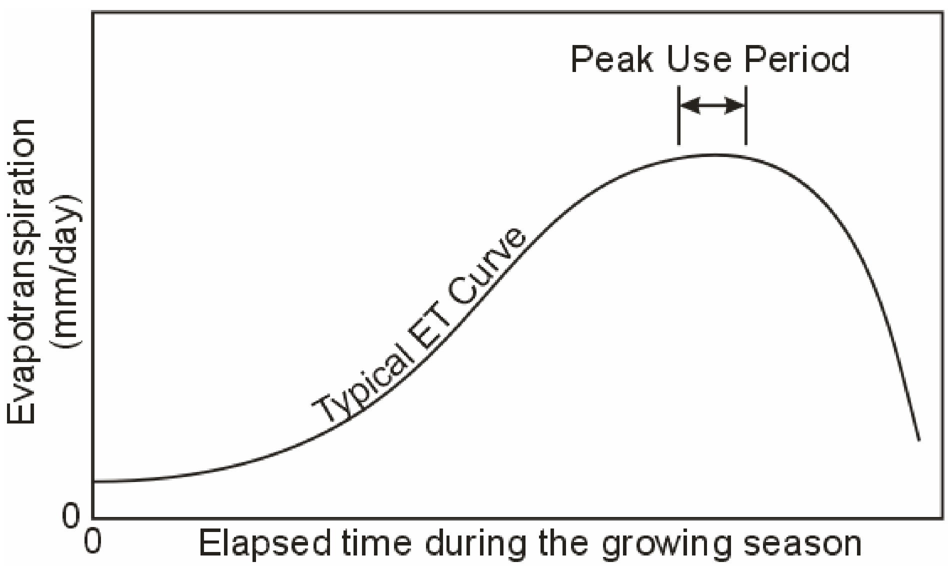
- The range of f_x values for agricultural crops is usually:

$$0.25 < f_x < 80 \text{ days} \tag{3}$$

- Then *nominal* irrigation frequency, f', is the value of f_x rounded down to the nearest whole number of days (
- But, it can be all right to round up if the values are conservative and if f_x is near the next highest integer value
- f' could be fractional if the sprinkler system is automated
- f' can be further reduced to account for nonirrigation days (e.g. Sundays), whereby $f \leq f'$
- The net application depth per irrigation during the peak use period is $d_n = f'U_d$, which will be less than or equal to d_x. Thus, $d_n <= d_x$, and when $d_n = d_x$, f' becomes f_x (the maximum allowable interval during the peak use period).
- Calculating d_n in this way, it is assumed that U_d persists for f' days, which may result in an overestimation if f' represents a period spanning many days

III. Peak Use Period

- Irrigation system design is usually for the most demanding conditions:



- The value of ET during the peak use period depends on the crop type and on the weather. Thus, the ET can be different from year to year for the same crop type.
- Some crops may have peak ET at the beginning of the season due to land preparation requirements, but these crops are normally irrigated by surface systems.
- When a system is to irrigate different crops (in the same or different seasons), the crop with the highest peak ET should be used to determine system capacity.
- Consider design probabilities for ET during the peak use period, because peak ET for the same crop and location will vary from year-to-year due to weather variations.

- Consider *deficit irrigation*, which may be feasible when water is very scarce and or expensive (relative to the crop value). However, in many cases farmers are not interested in practicing deficit irrigation.

IV. Leaching Requirement

- Leaching may be necessary if annual rains are not enough to flush the root zone, or if deep percolation from irrigation is small (i.e. good application uniformity and or efficiency).
- If EC_w is low, it may not be necessary to consider leaching in the design (system capacity).
- Design equation for leaching:

$$LR = \frac{EC_w}{5EC_e - EC_w} \tag{4}$$

where LR is the leaching requirement; EC_w is the EC of the irrigation water (dS/m or mmho/cm); and EC_e is the estimated saturation extract EC of the soil root zone for a given yield reduction value

- Equation 4 is taken from FAO Irrigation and Drainage Paper 29
- When LR > 0.1, the leaching ratio increases the depth to apply by 1/(1-LR); otherwise, LR does not need to be considered in calculating the gross depth to apply per irrigation, nor in calculating system capacity:

$$LR \leq 0.1: \quad d = \frac{d_n}{E_a} \tag{5}$$

$$LR > 0.1 \quad d = \frac{0.9\,d_n}{(1-LR)E_a} \tag{6}$$

- When LR < 0.0 (a negative value) the irrigation water is too salty, and the crop would either die or suffer severely
- Standard salinity vs. crop yield relationships (e.g. FAO) are given for electrical conductivity as saturation extract
- Obtain saturation extract by adding pure water in lab until the soil is saturated, then measure the electrical conductivity
- Here are some useful conversions: 1 mmho/cm = 1 dS/m = 550 to 800 mg/l (depending on chemical makeup, but typically taken as 640 to 690). And, it can usually be assumed that 1 mg/l $\approx$ 1 ppm, where ppm is by weight (or mass).

V. Leaching Requirement Example

Suppose EC_w = 2.1 mmhos/cm (2.1 dS/m) and EC_e for 10% reduction in crop yield is 2.5 dS/m. Then,

$$LR = \frac{EC_w}{5EC_e - EC_w} = \frac{2.1}{5(2.5) - 2.1} = 0.20 \tag{7}$$

Thus, LR > 0.1. And, assuming no loss of water due to application nonuniformity, the gross application depth is related to the net depth as follows:

$$d = d_n + (LR)d = \frac{d_n}{1 - LR} \tag{8}$$

and,

$$d = \frac{d_n}{1 - 0.20} = 1.25\,d_n \tag{9}$$

See Eq. 5.3 from the textbook regarding nonuniformity losses.

Sprinkle Irrigation Planning Factors

I. Farm Systems vs. Field Systems

- The authors of the textbook only devote a few paragraphs to this topic, but it is one of great importance
- A complete understanding of the distinctions between farm and field systems comes only through years of experience
- Variability in design, operation and management conditions is limitless

"A poorly designed system that is well managed can often perform better than a well designed system that is poorly managed"

- Farm systems may have many field systems
- Planning considerations should include the possibility of future expansions and extra capacity
- Permanent buried mainlines should generally be oversized to allow for future needs -- it is much cheaper to put a larger pipe in at the beginning than to install a secondary or larger line later
- Consider the possibility of future automation
- Consider the needs for land leveling before burying pipes
- How will the system be coordinated over many fields?

- What if the cropping patterns change? (tolerance to salinity, tolerance to foliar wetting, peak ET rate, root depth, need for crop cooling or frost protection, temporal shifting of peak ET period)
- What if energy costs change?
- What if labor availability and or cost change?
- What if the water supply is changed (e.g. from river to groundwater, or from old well to new well)?
- What if new areas will be brought into production?

II. Outline of Sprinkler Design Procedure

1. Make an inventory of resources

   - visit the field site personally if at all possible, and talk with the farmer
   - get data on soil, topography, water supply, crops, farm schedules, climate, energy, etc.
   - be suspicious of parameter values and check whether they are within reasonable ranges

2. Calculate a preliminary value for the maximum net irrigation depth, d_x
3. Obtain values for peak ET rate, U_d, and cumulative seasonal ET, U (Table 3.3)
4. Calculate maximum irrigation frequency, f_x, and nominal frequency, f'

   - this step is unnecessary for automated fixed systems and center pivots

5. Calculate the required system capacity, Q_s

   - first, calculate gross application depth, d
   - for center pivots use $d/f = U_d$, and $T \approx 90\%$ of 24 hrs/day = 21.6

6. Determine the "optimum" (or maximum) water application rate

   - a function of soil type and ground slope (Table 5.4)

7. Consider different types of feasible sprinkle systems
8. For periodic-move and fixed (solid-set) systems:

   (a) determine S_e, q_a, nozzle size, and P for optimum application rate (Tables 6.4 to 6.7)
   (b) determine number of sprinklers to operate simultaneously to meet Q_s ($N_n = Q_s/q_a$) (Chapter 7)
   (c) decide upon the best layout of laterals and mainline (Chapter 7)
   (d) Adjust f, d, and/or Q_s to meet layout conditions
   (e) Size the lateral pipes (Chapter 9)
   (f) Calculate the maximum pressure required for individual laterals
9. Calculate the mainline pipe size(s), then select from available sizes
10. Adjust mainline pipe sizes according to the "economic pipe selection method" (Chapter 10)

11. Determine extreme operating pressure and discharge conditions (Chapter 11)
12. Select the pump and power unit (Chapter 12)
13. Draw up system plans and make a list of items with suggestions for operation

III. Summary

- Note that MAD is not a precise value; actual precision is less than two significant digits; this justifies some imprecision in other values (don't try to obtain very precise values for some parameters when others are only rough estimates)
- Why use **f** to determine Q_s but **f'** to determine net application depth? (because Q_s must be based on gross requirements; not irrigating 24 hrs/day and 7 days/week does not mean that the crop will not transpire water 7 days/week)
- When determining the seasonal water requirements we subtract P_e from U. However, to be safe, the value of P_e must be reliable and consistent from year to year, otherwise a smaller (or zero) value should be used.
- Note that lateral and sprinkler spacings are not infinitely adjustable: they come in standard dimensions from which designers must choose. The same goes for pipe diameters and lengths.
- Note that design for peak U_d may not be appropriate if sprinklers are used only to germinate seeds (when later irrigation is by a surface method).

IV. Example Calculations for a Periodic-Move System

Given:

Crop is alfalfa. Top soil is 1.0 m of silt loam, and subsoil is 1.8 m of clay loam. Field area is 35 ha. MAD is 50% and EC_w is 2.0 dS/m. Application efficiency is estimated at 80%, and the soil intake rate is 15 mm/hr. Lateral spacing is 15 m and lateral length is 400 m. Assume it takes ½ hour to change sets. Seasonal effective rainfall is 190 mm; climate is hot. Assume one day off per week (irrigate only 6 days/week).

From tables in the textbook:

Hot climate, table 3.3 gives............U_d = 7.6 mm/day, and U = 914 mm/season
Top soil, table 3.1 gives ....W_a = 167 mm/m
Sub soil, table 3.1 gives..W_a = 183 mm/m
Root depth, table 3.2 gives ....Z = (1.2 + 1.8)/2 = 1.5 m
Salinity for 10% yield reduction, table 3.5 gives........................EC_e = 3.4 dS/m

1. Average water holding capacity in root zone:

top soil is 1.0 m deep; root zone is 1.5 m deep...

$$W_a = \frac{1.0(167) + (1.5 - 1.0)(183)}{1.5} = 172.3 \text{ mm/m} \tag{10}$$

2. Max net application depth (Eq. 3.1):

$$d_x = \frac{MAD}{100} W_a Z = \left(\frac{50}{100}\right)(172.3)(1.5) = 129.2 \text{ mm} \tag{11}$$

3. Maximum irrigation interval (Eq. 3.2):

$$f_x = \frac{d_x}{U_d} = \frac{129.2 \text{ mm}}{7.6 \text{ mm/day}} = 17.0 \text{ days} \tag{12}$$

4. Nominal irrigation interval (round down, or truncate):

$$f' = \text{trunc}(f_x) = 17 \text{ days} \tag{13}$$

5. Net application depth:

$$d_n = f' U_d = (17 \text{ days})(7.6 \text{ mm/day}) = 129.2 \text{ mm} \tag{14}$$

6. Operating time for an irrigation:

17 days is just over two weeks, and depending on which day is off, there could be 3 off days in this period. So, with one day off per week, we will design the system capacity to finish in 17 - 3 = 14 days. Thus, f = 14 days. But, remember that we still have to apply 17 days worth of water in these 14 days (we irrigate 6 days/week but crop transpires 7 days/week)

7. Leaching requirement (Eq. 3.3):

$$LR = \frac{EC_w}{5EC_e - EC_w} = \frac{2.0}{5(3.4) - 2.0} = 0.13 \tag{15}$$

LR > 0.1; therefore, use Eq. 5.3 b...

8. Gross application depth (Eq. 5.3b):

$$d = \frac{0.9 d_n}{(1 - LR)(E_a / 100)} = \frac{0.9(129.2)}{(1 - 0.13)(0.8)} = 167.1 \text{ mm} \tag{16}$$

9. Nominal set operating time:

With 167.1 mm to apply and a soil intake rate of 15 mm/hr, this gives 11.14 hrs minimum set time (so as not to exceed soil intake rate). Then, we can make the nominal set time equal to 11.5 hours for convenience. With 0.5 hrs to move each set, there are a total of 12.0 hrs/set, and the farmer can change at 0600 and 1800 (for example).

At this point we could take the lateral spacing, S_l, sprinkler spacing, S_e, and actual application rate to determine the flow rate required per sprinkler.

10. Sets per day:

From the above, we can see that there would be two sets/day.

11. Number of sets per irrigation:

(14 days/irrigation)(2 sets/day) = 28 sets

12. Area per lateral per irrigation:

Lateral spacing on mainline is S_l = 15 m. Lateral length is 400 m. Then, the area per lateral is:

(15 m/set)(28 sets)(400 m/lateral) = 16.8 ha/lateral

13. Number of laterals needed:

$$\frac{35\text{ ha}}{16.8\text{ ha/lateral}} = 2.08\text{ laterals} \qquad (17)$$

Normally we would round up to the nearest integer, but because this is so close to 2.0 we will use two laterals in this design.

14. Number of irrigations per season:

$$\frac{U - P_e}{d_n} = \frac{914\text{ mm - }190\text{ mm}}{129.2\text{ mm/irrig}} = 5.6\text{ irrigations} \qquad (18)$$

Thus, it seems there would be approximately six irrigations in a season. But, the initial R_z value is less than 1.5 m, so there may actually be more than six irrigations.

15. System flow capacity (Eq. 5.4):

with 11.5 hours operating time per set and two sets per day, the system runs 23 hrs/day...

$$Q_S = 2.78\frac{Ad}{fT} = 2.78\frac{(35\ \text{ha})(167.1\ \text{mm})}{(14\ \text{days})(23\ \text{hrs/day})} = 50.5\ \text{lps (800 gpm)} \tag{19}$$

This is assuming no effective precipitation during the peak ET period.

Lecture 3

Sprinkler Characteristics

I. Hardware Design Process

1. Sprinkler selection
2. Design of the system layout
3. Design of the laterals
4. Design of the mainline
5. Pump and power unit selection

II. Classification of Sprinklers and Applicability

(see Table 5.1 from the textbook)

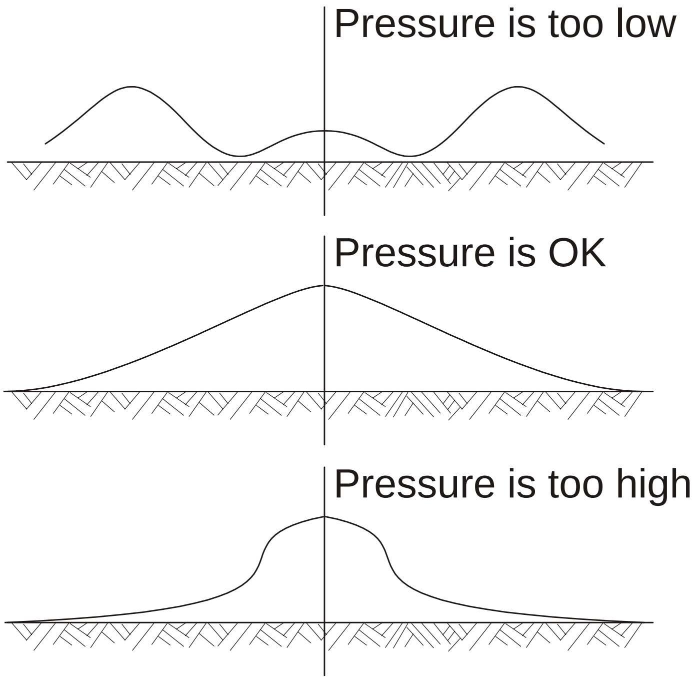
- Agricultural sprinklers typically have flow rates from 4 to 45 lpm (1 to 12 gpm), at nozzle pressures of 135 to 700 kPa (20 to 100 psi)
- "Gun" sprinklers may have flow rates up to 2,000 lpm (500 gpm; 33 lps) or more, at pressures up to 750 kPa (110 psi) or more
- Sprinklers with higher manufacturer *design pressures* tend to have larger wetted diameters
- But, deviations from manufacturer's recommended pressure may have the opposite effect (increase in pressure, decrease in diameter), and uniformity will probably be compromised

- Sprinklers are usually made of plastic, brass, and or steel
- Low pressure nozzles save pumping costs, but tend to have large drop sizes and high application rates
- Medium pressure sprinklers (210 - 410 kPa, or 30 to 60 psi) tend to have the best application uniformity
- Medium pressure sprinklers also tend to have the lowest minimum application rates
- High pressure sprinklers have high pumping costs, but when used in periodic-move systems can cover a large area at each set
- High pressure sprinklers have high application rates

- Rotating sprinklers have lower application rates because the water is only wetting a "sector" (not a full circle) at any given instance...
- For the same pressure and discharge, rotating sprinklers have larger wetted diameters
- Impact sprinklers always rotate; the "impact" action on the stream of water is designed to provide acceptable uniformity, given that much of the water would otherwise fall far from the sprinkler (the arm breaks up part of the stream)
- Check out Web sites such as www.rainbird.com

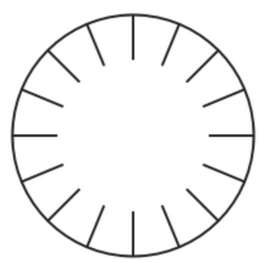
III. Precipitation Profiles

- Typical examples of low, correct, and high sprinkler pressures (see Fig 5.5).

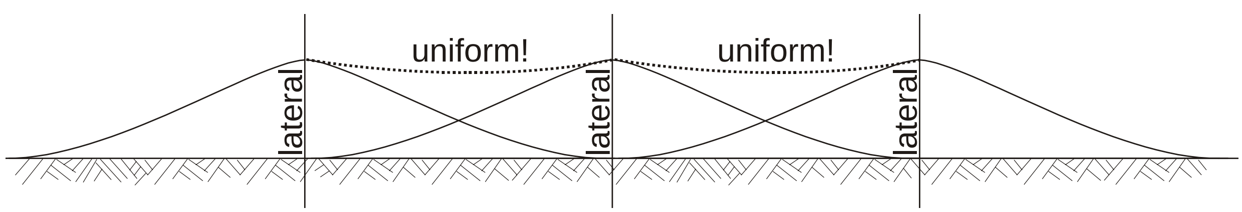


- The precipitation profile (and uniformity) is a function of many factors:

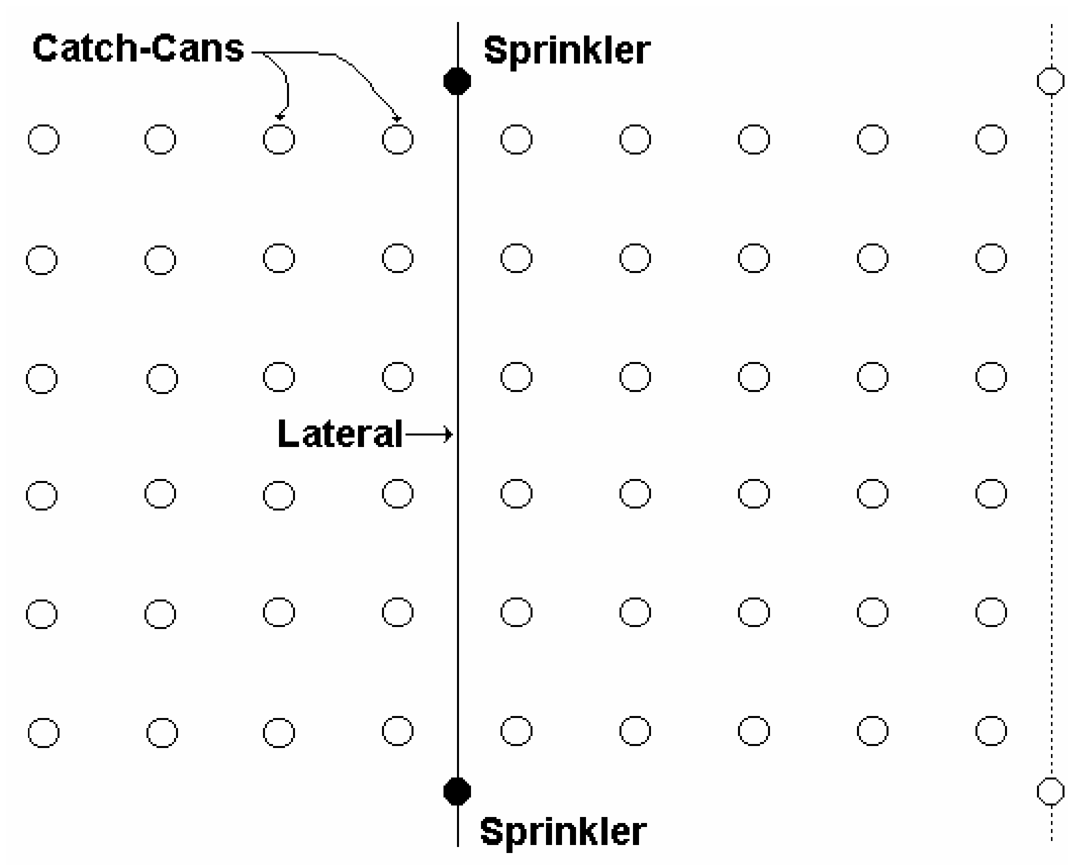
1. nozzle pressure
2. nozzle shape & size
3. sprinkler head design
4. presence of straightening vanes
5. sprinkler rotation speed
6. trajectory angle
7. riser height
8. wind



- Straightening vanes can be used to compensate for consistently windy conditions
- Overlapping sprinkler profiles (see Fig. 5.7)



- Simulate different lateral spacings by “overlapping” catch-can data in the direction of lateral movement (overlapping along the lateral is automatically included in the catch-can data, unless it’s just one sprinkler)



IV. Field Evaluation of Sprinklers

- Catch-can tests are typically conducted to evaluate the uniformities of installed sprinkler systems and manufacturers’ products
- Catch-can data is often overlapped for various sprinkler and lateral spacings to evaluate uniformities for design and management purposes
- A computer program developed at USU does the overlapping: ***CATCH3D***; you can also use a spreadsheet program to simulate overlapping (e.g. *Ctrl-C, Edit* | *Paste Special, Operation: Add*)
- Note that catch-can tests represent a specific wind and pressure situation and must be repeated to obtain information for other pressures or wind conditions
- Typical catch-can spacings are 2 or 3 m on a square grid, or 1 to 2 m spacings along one or more “radial legs”, with the sprinkler in the center
- Set up cans with half spacing from sprinklers (in both axes) to facilitate overlap calculations
- See Merriam & Keller (1978); also see ASAE S398.1 and ASAE S436

V. Choosing a Sprinkler

- the system designer doesn't "design" a sprinkler, but "selects" a sprinkler
- there are hundreds of sprinkler designs and variations from several manufacturers, and new sprinklers appear on the market often
- the system designer often must choose between different nozzle sizes and nozzle designs for a given sprinkler head design
- <u>the objective</u> is to combine sprinkler selection with S_e and S_l to provide acceptable application uniformity, acceptable pumping costs, and acceptable hardware costs
- manufacturers provide recommended spacings and pressures
- there are special sprinklers designed for use in frost control

VI. Windy Conditions

- When winds are consistently recurring at some specific hour, the system can be shut down during this period (T in Eq. 5.4 is reduced)
- For center pivots, rotation should not be a multiple of 24 hours, even if there is no appreciable wind (evaporation during day, much less at night)
- If winds consistently occur, special straightening vanes can be used upstream of the sprinkler nozzles to reduce turbulence; wind is responsible for breaking up the stream, so under calm conditions the uniformity could decrease
- For periodic-move systems, laterals should be moved in same direction as prevailing winds to achieve greater uniformity (because $S_e < S_l$)
- Laterals should also move in the direction of wind to mitigate problems of salt accumulating on plant leaves
- Wind can be a major factor on the application uniformity on soils with low infiltration rates (i.e. low application rates and small drop sizes)
- In windy areas with periodic-move sprinkler systems, the use of offset laterals ($½S_l$) may significantly increase application uniformity
- Alternating the time of day of lateral operation in each place in the field may also improve uniformity under windy conditions
- Occasionally, wind can help <u>increase uniformity</u>, as the randomness of wind turbulence and gusts helps to smooth out the precipitation profile

<u>Wind effects on the diameter of throw:</u>

0-3 mph wind:	reduce manufacturer's listed diameter of throw by 10% for an effective value (i.e. the diameter where the application of water is significant)
over 3 mph wind:	reduce manufacturer's listed diameter of throw by an additional 2.5% for every 1 mph above 3 mph (5.6% for every 1 m/s over 1.34 m/s)

In equation form:

For 0-3 mph (0-1.34 m/s):

$$\text{diam} = 0.9\,\text{diam}_{\text{manuf}} \tag{20}$$

For > 3 mph (> 1.34 m/s):

$$\text{diam} = \text{diam}_{\text{manuf}}\left[0.9 - 0.025\left(\text{wind}_{\text{mph}} - 3\right)\right] \tag{21}$$

or,

$$\text{diam} = \text{diam}_{\text{manuf}}\left[0.9 - 0.056\left(\text{wind}_{\text{m/s}} - 1.34\right)\right] \tag{22}$$

Example: a manufacturer gives an 80-ft diameter of throw for a certain sprinkler and operating pressure. For a 5 mph wind, what is the effective diameter?

$$80 \text{ ft} - (0.10)(0.80) = 72 \text{ ft} \tag{23}$$

$$72 \text{ ft} - (5 \text{ mph} - 3 \text{ mph})(0.025)(72 \text{ ft}) = 68 \text{ ft} \tag{24}$$

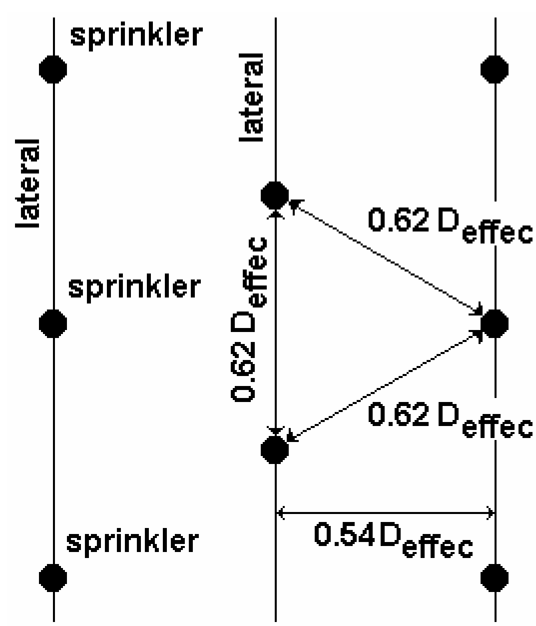
or,

$$\text{diam} = 80(0.9\text{-}0.025(5\text{-}3))\text{=}68 \text{ ft} \tag{25}$$

VII. General Spacing Recommendations

- Sprinkler spacing is usually rectangular or triangular
- Triangular spacing is more common under fixed-system sprinklers
- Sprinkler spacings based on average (moderate) wind speeds:

  1. ***Rectangular*** spacing is 40% (S_e) by 67% (S_l) of the effective diameter
  2. ***Square*** spacing is 50% of the effective diameter
  3. Equilateral ***triangle*** spacing is 62% of the effective diameter [lateral spacing is 0.62 cos (60°/2) = 0.54, or 54% of the effective diameter, D_{effec}]



- See Fig. 5.8 about profiles and spacings

VIII. Pressure-Discharge Relationship

- Equation 5.1:

$$q = K_d \sqrt{P} \tag{26}$$

where q is the sprinkler flow rate; K_d is an empirical coefficient; and P is the nozzle pressure

- The above equation is for a simple round orifice nozzle
- Eq. 5.1 can be derived from Bernoulli's equation like this:

$$\frac{P}{\gamma} = \frac{V^2}{2g} = \frac{q^2}{2gA^2} \tag{27}$$

$$\sqrt{\frac{2gA^2P}{\rho g}} = K_d \sqrt{P} = q \tag{28}$$

where the elevations are the same ($z_1 = z_2$) and the conversion through the nozzle is assumed to be *all pressure to all velocity*

- P can be replaced by H (head), but the value of K_d will be different
- Eq. 5.1 is accurate within a certain range of pressures
- See Table 5.2 for P, q, and K_d relationships

- K_d can be separated into an orifice coefficient, K_o, and nozzle bore area, A:

$$q = K_o A \sqrt{P} \tag{29}$$

whereby,

$$K_o = \sqrt{2/\rho} \tag{30}$$

where the value of K_o is fairly consistent across nozzle sizes for a specific model and manufacturer

- From Table 5.2 in the textbook, the values of K_o are as follows:

Flow Rate q	**Head or Pressure H or P**	**Nozzle Bore d**	K_o
lps	m	mm	0.00443
lps	kPa	mm	0.00137
lpm	m	mm	0.258
lpm	kPa	mm	0.0824
gpm	ft	inch	24.2
gpm	psi	inch	36.8

- Similar values can be determined from manufacturer's technical information
- Note also that nozzle diameter (bore) can be determined by rearranging the above equation as follows:

$$d = \sqrt{\frac{4q}{\pi K_o \sqrt{P}}} \quad (31)$$

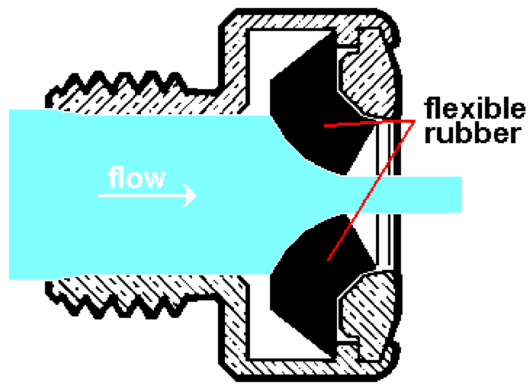
- The value of d can then be rounded up to the nearest available diameter (64^{ths} of an inch, or mm)
- Then, either P or q are adjusted as necessary in the irrigation system design
- Below is a sample *pressure versus discharge* table for a RainBird© sprinkler

Straight Bore Nozzle (SBN-4) with Plug (Stream Height: 8 ft.)

	NOZZLE SIZE													
	7/32"		1/4"		9/32"		15/16"		11/32"		3/8"		13/32"	
PSI @ Nozzle	Rad.	GPM	Rad.	GPM	Rad.	GPM	Rad.	GPM	Rad.	GPM	Rad.	GPM	Rad.	GPM
40	57	8.80	60	11.50	62	14.60	65	17.70	66	21.10	68	24.40	70	28.50
45	58	9.40	61	12.20	64	15.50	66	18.90	68	22.50	70	26.00	72	30.40
50	59	9.90	62	12.90	65	16.30	68	20.00	70	23.80	71	27.50	73	32.30
55	60	10.40	63	13.60	66	17.20	70	21.00	71	25.00	73	29.10	75	34.00
60	61	10.90	64	14.20	67	18.00	71	22.00	73	26.20	74	30.60	77	35.70
65	62	11.40	65	14.80	68	18.80	72	23.00	74	27.40	76	32.00	78	37.30
70	63	11.80	66	15.40	70	19.50	73	23.90	76	28.50	77	33.20	79	38.90
75	64	12.20	67	16.00	71	20.30	74	24.80	77	29.60	78	34.50	81	40.40
80	65	12.60	68	16.50	72	20.90	75	25.70	78	30.60	80	35.70	82	41.80

Application Rates

I. Flow Control Nozzles



Flow Control Nozzle

- More expensive than regular nozzles (compare $0.60 for a brass nozzle to about $2.70 for a flow control nozzle)
- May require more frequent maintenance
- The orifice has a flexible ring that decreases the opening with higher pressures, whereby the value of $A\sqrt{P}$ in the equation remains approximately constant
- It can be less expensive to design laterals and mainline so that these types of nozzles are not required, but this is not always the case
- FCNs are specified for nominal discharges (4, 4.5, 4.8, 5.0 gpm, etc.)
- The *manufacturer's coefficient of variation* is about ±5% of q; don't use FCNs unless pressure variation is greater than about 10% (along lateral and for different lateral positions)

$$\sqrt{1.10P} \approx 1.05\sqrt{P} \tag{32}$$

II. Low-Pressure Sprinklers

1. Pressure alone is not sufficient to break up the stream in a standard nozzle design for acceptable application uniformity
2. Need some mechanical method to reduce drop sizes from the sprinkler:

    - pins that partially obstruct the stream of water
    - sharp-edged orifices
    - triangular, rectangular, oval nozzle shapes

3. Some sprinkler companies have invested much into the design of such devices for low-pressure sprinkler nozzles
4. Low-pressure nozzles can be more expensive, possibly with reduced uniformity and increased application rate, but the trade-off is in operating cost

III. Gross Application Depth

$$d = \frac{d_n}{E_{pa}}, \quad \text{for } LR \leq 0.1 \tag{33}$$

where E_{pa} is the "designer" application efficiency (decimal; Eq. 6.9). And,

$$d = \frac{0.9\,d_n}{(1-LR)E_{pa}}, \quad \text{for } LR > 0.1 \tag{34}$$

- The gross application depth is the total equivalent depth of water which must be delivered to the field to replace (all or part of) the soil moisture deficit in the root zone of the soil, plus any seepage, evaporation, spray drift, runoff and deep percolation losses
- The above equations for d presume that the first 10% of the leaching requirement will be satisfied by the E_{pa} (deep percolation losses due to application variability). This presumes that areas which are under-irrigated during one irrigation will also be over-irrigated in the following irrigation, or that sufficient leaching will occur during non-growing season (winter) months.
- When the LR value is small ($EC_w \leq ½EC_e$), leaching may be accomplished both before and after the peak ET period, and the first equation (for $LR \leq 0.1$) can be used for design and sizing of system components. This will reduce the required pipe and pump sizes because the "extra" system capacity during the non-peak ET periods is used to provide water for leaching.

IV. System Capacity

- Application volume can be expressed as either Qt or Ad, where Q is flow rate, t is time, A is irrigated area and d is gross application depth
- Both terms are in units of volume
- Thus, the system capacity is defined as (Eq. 5.4):

$$Q_s = K\frac{A\,d}{f\,T} \tag{35}$$

where,

Q_s = system capacity;
T = hours of system operation per day (obviously, $T \leq 24$; also, t = fT)
K = coefficient for conversion of units (see below)
d = gross application depth (equals U_d/Eff during f' period)
f = time to complete one irrigation (days); equal to f' minus the days off
A = net irrigated area supplied by the discharge Q_s

Value of K:

- **Metric**: for d in mm, A in ha, and Q_s in lps: **K = 2.78**
- **English**: for d in inches, A in acres, and Q_s in gpm: **K = 453**

Notes about system capacity:

- Eq. 5.4 is normally used for periodic-move and linear-move sprinkler systems
- The equation can also be used for center pivots if f is decimal days to complete one revolution and d is the gross application depth per revolution
- For center pivot and solid-set systems, irrigations can be light and frequent ($d_{applied}$ < d): soil water is maintained somewhat below field capacity at all times (assuming no leaching requirement), and there is very little deep percolation loss
- Also, there is a margin of safety in the event that the pump fails (or the system is temporarily out of operation for whatever reason) just when MAD is reached (time to irrigate), because the soil water deficit is never allowed to reach MAD
- However, light and frequent irrigations are associated with higher evaporative losses, and probably higher ET too (due to more optimal soil moisture conditions). This corresponds to a higher basal crop coefficient (K_{cb} + K_s), where K_s is increased, and possibly K_{cb} too.
- When a solid-set (fixed) system is used for frost control, all sprinklers must operate simultaneously and the value of Q_s is equal to the number of sprinklers multiplied by q_a. This tends to give a higher Q_s than that calculated from Eq. 5.4.

V. Set Sprinkler Application Rate

- The average application rate is calculated as (after Eq. 5.5):

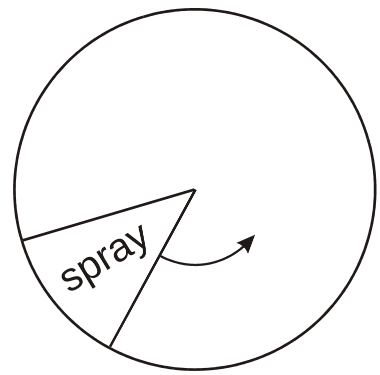
$$I = \frac{3600\,qR_e}{S_e S_l} \qquad (36)$$

where I is the application rate (mm/hr); q is the flow rate (lps); S_e is the sprinkler spacing (m); S_l is the lateral spacing (m); and R_e is the fraction of water emitted by the nozzle that reaches the soil (takes into account the evaporative/wind loss)

- R_e is defined in Chapter 6 of the textbook
- The instantaneous application rate for a rotating sprinkler (after Eq. 5.6):

$$I_i = \frac{3600\,q R_e}{\pi R_j^2 \left(\frac{S_a}{360}\right)} \tag{37}$$

where I_i is the application rate (mm/hr); R_j is the radius of throw, or wetted radius (m); and S_a is the segment wetted by the sprinkler when the sprinkler is not allowed to rotate (degrees)



- Note that due to sprinkler overlap, the instantaneous application rate may actually be higher than that given by I_i above
- For a non-rotating sprinkler, the instantaneous application rate is equal to the average application rate
- For a rotating sprinkler, the instantaneous application rate may be allowed to exceed the basic intake rate of the soil because excess (ponded) water has a chance to infiltrate while the sprinkler completes each rotation
- See sample calculation 5.3 in the textbook
- Higher pressures can give lower instantaneous application rates, but if the wetted radius does not increase significantly with an increase in pressure, the instantaneous rate may increase
- The minimum tangential rotation speed at the periphery of the wetted area should normally be about 1.5 m/s. For example, for 1 rpm:

$$\frac{(1.5\text{ m/s})(60\text{ s/min})}{(1\text{ rev/min})(2\pi\text{ rad/rev})} = 14.3\text{m (radius)} \tag{38}$$

  - Thus, a sprinkler with a wetted radius of 14.3 m should rotate at least 1 rpm
  - "Big gun" sprinklers can rotate slower than 1 rpm and still meet this criterion

VI. Intake & Optimum Application Rates

- Factors influencing the rate at which water should be applied:

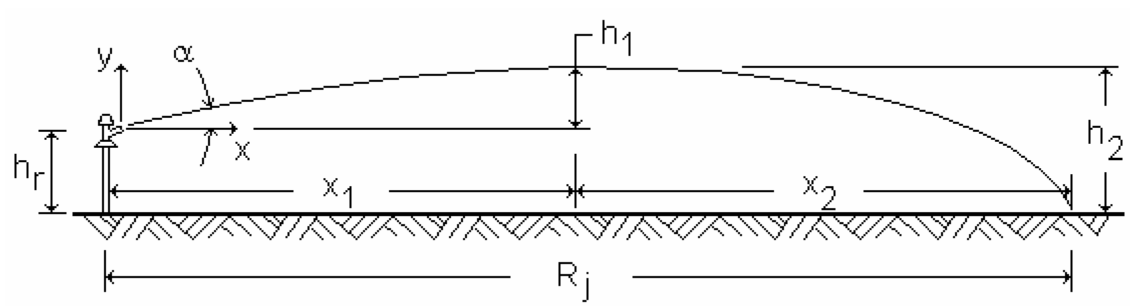
  1. Soil intake characteristics, field slope, and crop cover
  2. Minimum application rate that will give acceptable uniformity
  3. Practicalities regarding lateral movement in periodic-move systems

- Impact of water drops on bare soil can cause “surface sealing” effects, especially on heavy-textured (clayey) soils
- The result is a reduction in infiltration rate due to the formation of a semi-impermeable soil layer

- Sprinklers typically produce drops from ½ to 5 mm
- Terminal velocity of falling drops is from 2 to 22 m/s
- Water drops from sprinklers typically reach their terminal velocity before arriving at the soil surface (especially sprinklers with high risers)
- See Tables 5.3 and 5.4 in the textbook

V. Approximate Sprinkler Trajectory

- The trajectory of water from a sprinkler can be estimated according to physics equations
- The following analysis does not consider aerodynamic resistance nor wind effects, and is applicable to the largest drops issuing from a sprinkler operating under a recommended pressure
- Of course, smaller water drops tend to fall nearer to the sprinkler
- In the figure below, R_j refers to the approximate wetted radius of the sprinkler



- If the velocity in the vertical direction at the nozzle, V_y, is taken as zero at time t_1, then,

$$\left(V_y\right)_{t_1} = V_0 \sin\alpha - g\,t_1 = 0 \qquad (39)$$

where V_0 is the velocity of the stream leaving the nozzle (m/s); α is the angle of the nozzle; t_1 is the time for a drop to travel from the nozzle to the highest point in the trajectory (s); and g is the ratio of weight to mass (9.81 m/s^2)

- Note that the term $V_o \sin\alpha$ in Eq. 37 is the initial velocity component in the vertical direction, and the term gt_1 is the downward acceleration over time t_1
- The above equation can be solved for t_1
- The initial velocity, V_0, can be calculated based on the sprinkler discharge and the nozzle diameter
- Values of α can be found from manufacturers’ information

- Now, what is the highest point in the trajectory?
- First, solve for t_1 in the previous equation:

$$t_1 = \frac{V_o \sin\alpha}{g} \tag{40}$$

then,

$$h_1 = V_0 \sin\alpha\ t_1 - \frac{g t_1^2}{2} = \frac{V_0^2 \sin^2\alpha}{2g} \tag{41}$$

- Assuming no acceleration in the horizontal direction,

$$x_1 = V_0 \cos\alpha\ t_1 \tag{42}$$

solving for h_2,

$$h_2 = h_r + h_1 = V_y t_2 + \frac{g t_2^2}{2} \tag{43}$$

where h_r is the riser height (m); t_2 is the time for a drop of water to travel from the highest point in the trajectory to impact on the ground; and $V_y = 0$

- Then, x_2 is defined as:

$$x_2 = V_0 \cos\alpha\ t_2 = V_0 \cos\alpha \sqrt{\frac{2\left(h_r + h_1\right)}{g}} \tag{44}$$

And, the approximate wetted radius of the sprinkler is:

$$R_j = x_1 + x_2 \tag{45}$$

- In summary, if air resistance is ignored and the sprinkler riser is truly vertical, the theoretical value of R_j is a function of:

1. Angle, α
2. Nozzle velocity (q_a/A)
3. Riser height, h_r

- And, q_a is a function of P

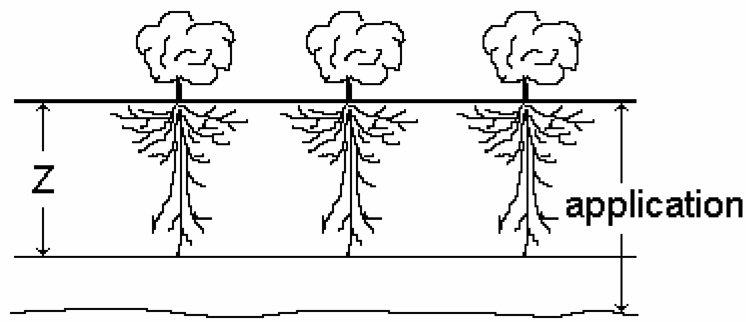
Lecture 4

Set Sprinkler Uniformity & Efficiency

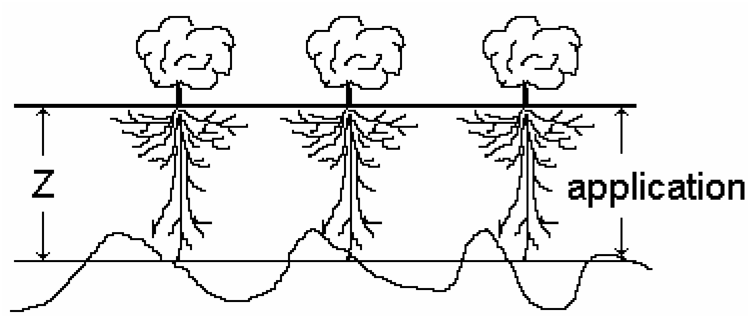
I. Sprinkler Irrigation Efficiency

1. Application uniformity
2. Losses (deep percolation, evaporation, runoff, wind drift, etc.)

- It is not enough to have uniform application if the average depth is not enough to refill the root zone to field capacity
- Similarly, it is not enough to have a correct average application depth if the uniformity is poor
- Consider the following examples:



Uniform, but average depth applied exceeds the soil water deficit (too much deep percolation)



Average depth is correct, but application is highly nonuniform, with underirrigation and DP

- We can design a sprinkler system that is capable of providing good application uniformity, but depth of application is a function of the set time (in periodic-move systems) or "on time" (in fixed systems)
- Thus, uniformity is mainly a function of design and subsequent system maintenance, but application depth is a function of management

II. Quantitative Measures of Uniformity

Distribution uniformity, DU (Eq. 6.1):

$$DU = 100\left(\frac{\text{avg depth of low quarter}}{\text{avg depth}}\right) \tag{46}$$

- The average of the low quarter is obtained by measuring application from a catch-can test, mathematically overlapping the data (if necessary), ranking the values by magnitude, and taking the average of the values from the low ¼ of all values
- For example, if there are 60 values, the low quarter would consist of the 15 values with the lowest "catches"

Christiansen Coefficient of Uniformity, CU (Eq. 6.2):

$$CU = 100\left(1.0 - \frac{\sum_{j=1}^{n} \text{abs}\left(z_j - m\right)}{\sum_{j=1}^{n} z_j}\right) \tag{47}$$

where z are the individual catch-can values (volumes or depths); n is the number of observations; and m is the average of all catch volumes.

- Note that CU can be negative if the distribution is very poor
- There are other, equivalent ways to write the equation
- These two measures of uniformity (CU & DU) date back to the time of slide rules (more than 50 years ago; no electronic calculators), and are designed with computational ease in mind
- More complex statistical analyses can be performed, but these values have remained useful in design and evaluation of sprinkler systems
- For CU > 70% the data usually conform to a normal distribution, symmetrical about the mean value. Then,

$$CU \approx 100\left(\frac{\text{avg depth of low half}}{\text{avg depth}}\right) \tag{48}$$

another way to define CU is through the standard deviation of the values,

$$CU = 100\left(1.0 - \frac{\sigma}{m}\sqrt{\frac{2}{\pi}}\right) \tag{49}$$

where σ is the standard deviation of all values, and a normal distribution is assumed (as previously)

- Note that CU = 100% for $\sigma = 0$
- The above equation assumes a normal distribution of the depth values, whereby:

$$\sum |z - m| = n\sigma\sqrt{2/\pi} \tag{50}$$

- By the way, the ratio σ/m is known in statistics as the *coefficient of variation*
- Following is the approximate relationship between CU and DU:

$$CU \approx 100 - 0.63(100 - DU) \tag{51}$$

or,

$$DU \approx 100 - 1.59(100 - CU) \tag{52}$$

- These equations are used in evaluations of sprinkler systems for both design and operation
- Typically, 85 to 90% is the practical upper limit on DU for set systems
- DU > 65% and CU > 78% is considered to be the minimum acceptable performance level for an economic system design; so, you would not normally design a system for a CU < 78%, unless the objective is simply to "get rid of water or effluent" (which is sometimes the case)
- For shallow-rooted, high value crops, you may want to use DU > 76% and CU > 85%

III. Alternate Sets (Periodic-Move Systems)

- The effective uniformity (over multiple irrigations) increases if "alternate sets" are used for periodic-move systems (½S_l)
- This is usually practiced by placing laterals halfway between the positions from the previous irrigation, alternating each time
- The relationship is:

$$\begin{aligned} CU_a &\approx 10\sqrt{CU} \\ DU_a &\approx 10\sqrt{DU} \end{aligned} \tag{53}$$

- The above are also valid for "double" alternate sets (S_l/3)
- Use of alternate sets is a good management practice for periodic-move systems
- The use of alternate sets approaches an S_l of zero, which simulates a continuous-move system

IV. Uniformity Problems

- Of the various causes of non-uniform sprinkler application, some tend to cancel out with time (multiple irrigations) and others tend to concentrate (get worse)
- In other words, the "composite" CU for two or more irrigations may be (but not necessarily) greater than the CU for a single irrigation

1. Factors that tend to Cancel Out

   - Variations in sprinkler rotation speed
   - Variations in sprinkler discharge due to wear
   - Variations in riser angle (especially with hand-move systems)
   - Variations in lateral set time

2. Factors that may both Cancel Out and Concentrate

   - Non-uniform aerial distribution of water between sprinklers

3. Factors that tend to Concentrate

   - Variations in sprinkler discharge due to elevation and head loss
   - Surface ponding and runoff
   - Edge effects at field boundaries

V. System Uniformity

- The uniformity is usually less when the entire sprinkler system is considered, because there tends to be greater pressure variation in the system than at any given lateral position.

$$\text{system CU} \approx \text{CU}\left[\frac{1}{2}\left(1+\sqrt{P_n / P_a}\right)\right] \qquad (54)$$

$$\text{system DU} \approx \text{DU}\left[\frac{1}{4}\left(1+3\sqrt{P_n / P_a}\right)\right] \qquad (55)$$

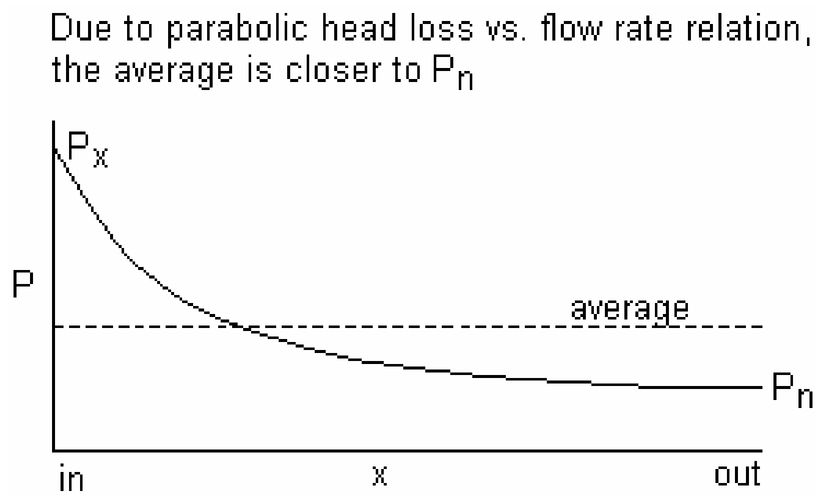
where P_n is the minimum sprinkler pressure in the whole field; and P_a is the average sprinkler pressure in the entire system, over the field area.

- These equations can be used in design and evaluation
- Note that when $P_n = P_a$ (no pressure variation) the system CU equals the CU
- If pressure regulators are used at each sprinkler, the system CU is approximately equal to 0.95CU (same for DU)

- If flexible orifice nozzles are used, calculate system CU as 0.90CU (same for DU)
- The P_a for a system can often be estimated as a weighted average of P_n & P_x:

$$P_a = \frac{2P_n + P_x}{3} \tag{56}$$

where P_x is the maximum nozzle pressure in the system



VI. Computer Software and Standards

- There is a computer program called "Catch-3D" that performs uniformity calculations on sprinkler catch-can data and can show the results graphically
- Jack Keller and John Merriam (1978) published a handbook on the evaluation of irrigation systems, and this includes simple procedures for evaluating the performance of sprinkler systems
- The ASAE S436 (Sep 92) is a detailed standard for determining the application uniformity under center pivots (not a set sprinkler system, but a continuous move system)
- ASAE S398.1 provides a description of various types of information that can be collected during an evaluation of a set sprinkler system

VII. General Sprinkle Application Efficiency

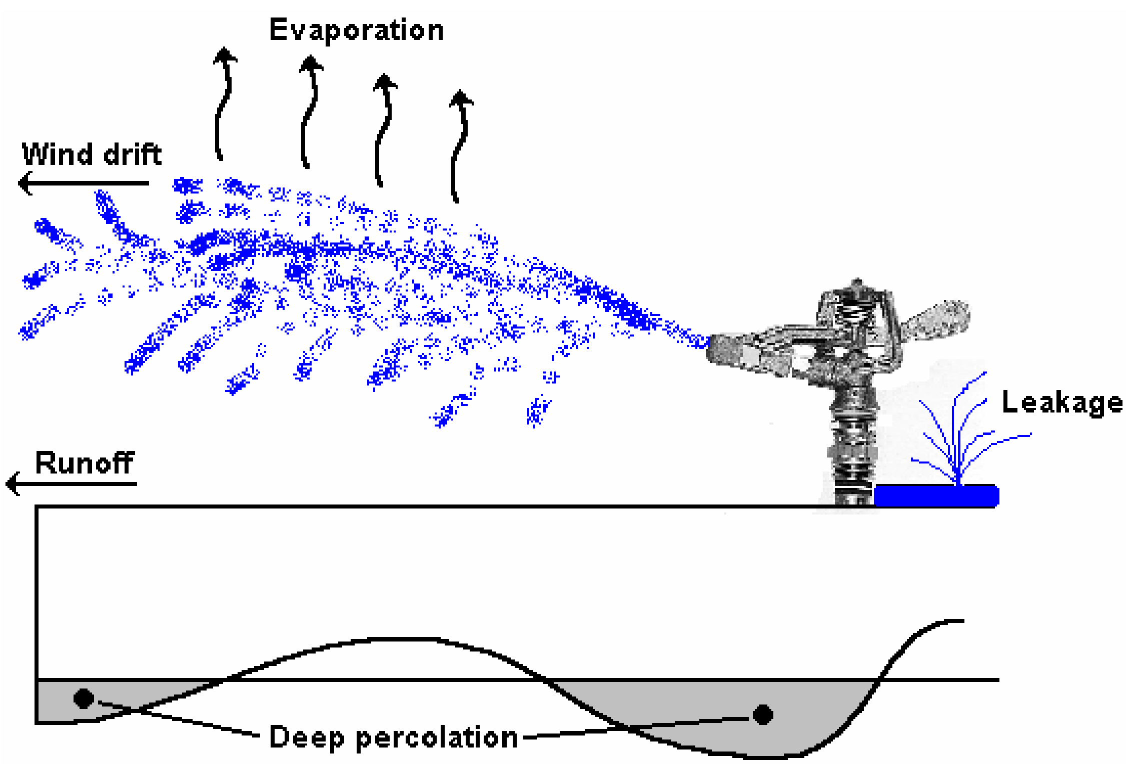
The following material leads up to the development of a general sprinkle application efficiency term (Eq. 6.9) as follows:

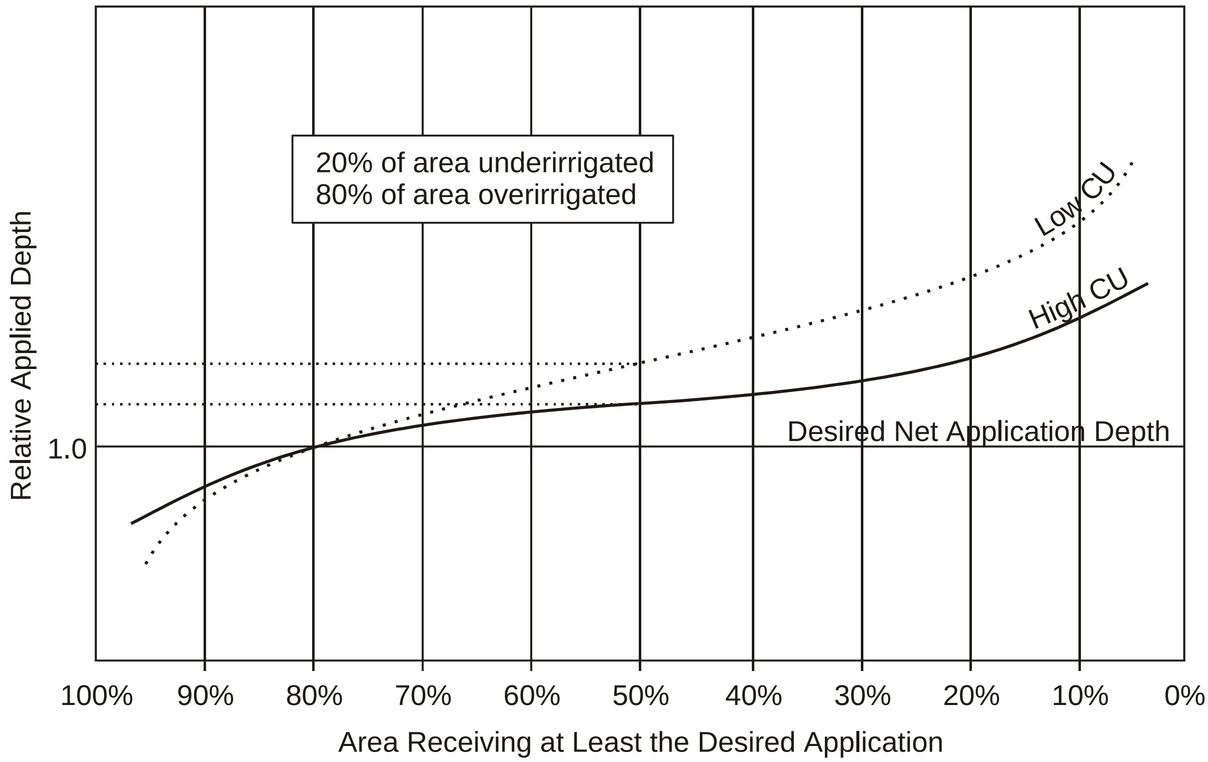
Design Efficiency:

$$E_{pa} = DE_{pa} R_e O_e \tag{57}$$

where DE_{pa} is the distribution efficiency (%); R_e is the fraction of applied water that reaches the soil surface; and O_e is the fraction of water that does not leak from the system pipes.

- The design efficiency, E_{pa}, is used to determine gross application depth (for design purposes), given the net application depth
- In most designs, it is not possible to do a catch-can test and data analysis – you have to install the system in the field first; thus, use the "design efficiency"
- The subscript "pa" represents the "percent area" of the field that is adequately irrigated (to d_n, or greater) – for example, E_{80} and DE_{80} are the application and distribution efficiencies when 80% of the field is adequately irrigated
- Question: can "pa" be less than 50%?





VIII. Distribution Efficiency

- This is used to define the *uniformity* and *adequacy* of irrigation
- DE is based on statistical distributions and application uniformity
- For a given uniformity (CU) and a given percent of land adequately irrigated (equal to or greater than required application depth), Table 6.2 gives values of DE that determine how much water must be applied in excess of the required depth so that the given percent of land really does receive at least the required depth

CU	**Percent area adequately irrigated (pa)**									
	95	**90**	**85**	**80**	**75**	**70**	**65**	**60**	**55**	**50**
94	87.6	90.4	92.2	93.7	94.9	96.1	97.1	98.1	99.1	100.0
92	83.5	87.1	89.6	91.6	93.2	94.7	96.1	97.5	98.7	100.0
90	79.4	83.9	87.0	89.4	91.5	93.4	95.2	96.8	98.4	100.0
88	75.3	80.7	84.4	87.3	89.8	92.1	94.2	96.2	98.1	100.0
86	71.1	77.5	81.8	85.2	88.2	90.8	93.2	95.6	97.8	100.0
84	67.0	74.3	79.2	83.1	86.5	89.5	92.3	94.9	97.5	100.0
82	62.9	71.1	76.6	81.0	84.8	88.2	91.3	94.3	97.2	100.0
80	58.8	67.9	74.0	78.9	83.1	86.8	90.3	93.6	96.8	100.0
78	54.6	64.7	71.4	76.8	81.4	85.5	89.4	93.0	96.5	100.0
76	50.5	61.4	68.8	74.7	79.7	84.2	88.4	92.4	96.2	100.0
74	46.4	58.2	66.2	72.6	78.0	82.9	87.4	91.7	95.9	100.0
72	42.3	55.0	63.6	70.4	76.3	81.6	86.5	91.1	95.6	100.0
70	38.1	51.8	61.0	68.3	74.6	80.3	85.5	90.5	95.3	100.0

- See Fig. 6.7

IX. Wind Drift and Evaporation Losses

- These losses are typically from 5% to 10%, but can be higher when the air is dry, there is a lot of wind, and the water droplets are small
- *Effective portion of the applied water*, R_e. This is defined as the percentage of applied water that actually arrives at the soil surface of the irrigated field.
- This is based on:

  - climatic conditions
  - wind speed
  - spray coarseness

- Figure 6.8 gives the value of R_e for these different factors
- The *Coarseness Index*, CI, is defined as (Eq. 6.7):

$$CI = 0.032\left(\frac{P^{1.3}}{B}\right) \tag{58}$$

where P is the nozzle pressure (kPa) and B is the nozzle diameter (mm)

CI > 17	**17 ≥ CI ≥ 7**	**CI < 7**
fine spray	**between fine and coarse**	**coarse spray**

- When the spray is between fine and coarse, R_e is computed as a weighted average of $(R_e)_{fine}$ and $(R_e)_{coarse}$ (Eq. 6.8):

$$R_e = \frac{(CI-7)}{10}\left(R_e\right)_{fine} + \frac{(17-CI)}{10}\left(R_e\right)_{coarse} \tag{59}$$

- Allen and Fisher (1988) developed a regression equation to fit the curves in Fig. 6.8:

$$\begin{aligned} R_e = {} & 0.976 + 0.005ET_o - 0.00017ET_o^2 + 0.0012W \\ & -0.00043(CI)(ET_o) - 0.00018(CI)(W) \\ & -0.000016(CI)(ET_o)(W) \end{aligned} \tag{60}$$

where ET_o is the reference ET in mm/day (grass-based); CI is the coarseness index ($7 \le CI \le 17$); and W is the wind speed in km/hr
- For the above equation, if CI < 7 then set it equal to 7; if CI > 17 then set it equal to 17

X. Leaks and Drainage Losses

1. Losses due to drainage of the system after shut-down

    - upon shut-down, most sprinkler systems will partially drain
    - water runs down to the low elevations and or leaves through automatic drain valves that open when pressure drops
    - fixed (solid-set) systems can have anti-drain valves at sprinklers that close when pressure drops (instead of opening, like on wheel lines)

2. Losses due to leaky fittings, valves, and pipes

    - pipes and valves become damaged with handling, especially with hand-move and side-roll systems, but also with orchard sprinklers and end-tow sprinklers
    - gaskets and seals become inflexible and fail

- These losses are quantified in the O_e term
- For systems in good condition these losses may be only 1% or 2%, giving an O_e value of 99% or 98%, respectively
- For system in poor condition these losses can be 10% or higher, giving an O_e value of 90% or less

XI. General Sprinkle Application Efficiency

- As given above, Eq. 6.9 from the textbook, it is:

$$E_{pa} = DE_{pa} R_e O_e \tag{61}$$

where DE_{pa} is in percent; and R_e and O_e are in fraction (0 to 1.0). Thus, E_{pa} is in percent.

XII. Using CU or DU instead of DE_{pa}

1. Application Efficiency of the **Low Quarter**, E_q

    - Given by Eq. 6.9 when DU replaces DE_{pa}
    - Useful for design purposes for medium to high-value crops
    - Only about 10% of the area will be under-irrigated
    - Recall that DU is the average of low quarter divided by average

2. Application Efficiency of the **Low Half**, E_h

    - Given by Eq. 6.9 when CU replaces DE_{pa}
    - Useful for design purposes for low-value and forage crops

- Only about 20% of the area will be under-irrigated
- Recall that CU is the average of low half divided by average

XIII. Procedure to Determine CU, Required Pressure, S_e and S_l for a Set System

1. Specify the minimum acceptable E_{pa} and target pa
2. Estimate R_e and O_e (these are often approximately 0.95 and 0.99, respectively)
3. Compute DE_{pa} from E_{pa}, R_e and O_e
4. Using DE_{pa} and pa, determine the CU (Table 6.2) that is required to achieve E_{pa}
5. Compute the set operating time, t_{so}, then adjust f' and d_n so that t_{so} is an appropriate number of hours
6. Compute q_a based on I, S_e and S_l (Eq. 5.5)
7. Search for nozzle size, application rate, S_e and S_l to obtain the CU
8. Repeat steps 5, 6 and 7 as necessary until a workable solution is found

XIV. How to Measure R_e

- The textbook suggests a procedure for estimating R_e
- You can also measure R_e from sprinkler catch-can data:

    1. Compute the average catch depth over the wetted area (if a single sprinkler), or in the area between four adjacent sprinklers (if in a rectangular grid)
    2. Multiply the sprinkler flow rate by the total irrigation time to get the volume applied, then divide by the wetted area to obtain the gross average application depth
    3. Divide the two values to determine the effective portion of the applied water

XV. Line- and Point-Source Sprinklers

- Line-source sprinklers are sometimes used by researches to determine the effects of varying water application on crop growth and yield
- A line-source sprinkler system consists of sprinklers spaced evenly along a straight lateral pipe in which the application rate varies linearly with distance away from the lateral pipe, orthogonally
- Thus, a line-source sprinkler system applies the most water at the lateral pipe, decreasing linearly to zero to either side of the lateral pipe
- A point-source sprinkler is a single sprinkler that gives linearly-varying application rate with radial distance from the sprinkler

- With a point-source sprinkler, the contours of equal application rate are concentric circles, centered at the sprinkler location (assuming the riser is vertical and there is no wind)

Lecture 5

Layout of Laterals for Set Sprinklers

I. Selecting Sprinkler Discharge, Spacing, and Pressure

- In Chapter 6 of the textbook there are several tables that provide guidelines for nozzle sizes for different:
  - Wind conditions
  - Application rates
  - Sprinkler spacings

- For selected values of wind, application rate, and spacing, the tables provide recommended nozzle sizes for single and double-nozzle sprinklers, recommended sprinkler pressure, and approximate uniformity (CU)

- Table values are for standard (non-flexible) nozzles
- Table values are for standard sprinkler and lateral spacings
- More specific information can be obtained from manufacturers' data

- Recall that the maximum application rate is a function of soil texture, soil structure, and topography (Table 5.4)
- For a given spacing and application rate, the sprinkler discharge, q_a, can be determined from Eq. 5.5

$$q_a = \frac{I(S_e S_l)}{3600} = \frac{d_n S_e S_l O_e}{3600 E_{pa} S_{to}} \tag{62}$$

where q_a is in lps; I is in mm/hr; d_n is in mm; S_{to} is the operating time for each set, in hours; and S_l and S_e are in m

- Why is the O_e term included in the above equation? (because E_{pa} includes O_e, as previously defined, and must be cancelled out when considering an individual sprinkler)

II. Number of Operating Sprinklers

- After calculating the system capacity and the design flow rate for sprinklers, the number of sprinklers that will operate at the same time is:

$$N_n = \frac{Q_s}{q_a} \tag{63}$$

where N_n is the minimum number of sprinklers operating, and Q_s and q_a have the same units

- It is recommendable to always operate the same number of sprinklers when the system is running. This practice can help avoid the need for pressure regulation, and can avoid uniformity problems. It can also help avoid wasting energy at the pump.
- For odd-shaped fields, and sometimes for rectangular fields, it is not possible to operate the same number of sprinklers for all sets. In this case, pressure regulation may be necessary, or other steps can be taken (multiple pumps, variable-speed motor, variable application rates).

III. Lateral Design Criteria

- Lateral pressure varies from inlet to extreme end due to:

  1. friction loss
  2. elevation change

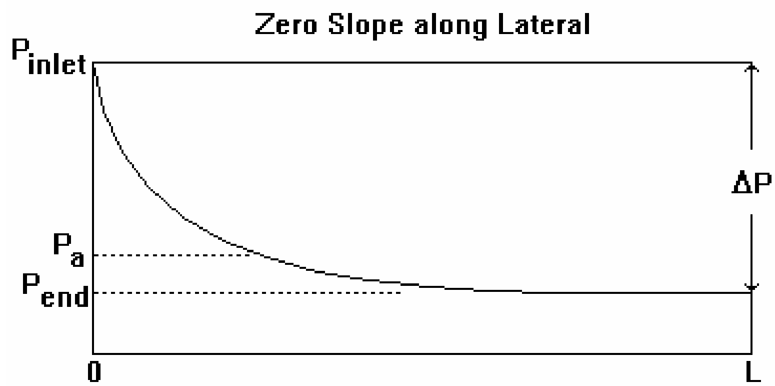
- The fundamental basis upon which sprinkler laterals are designed is:

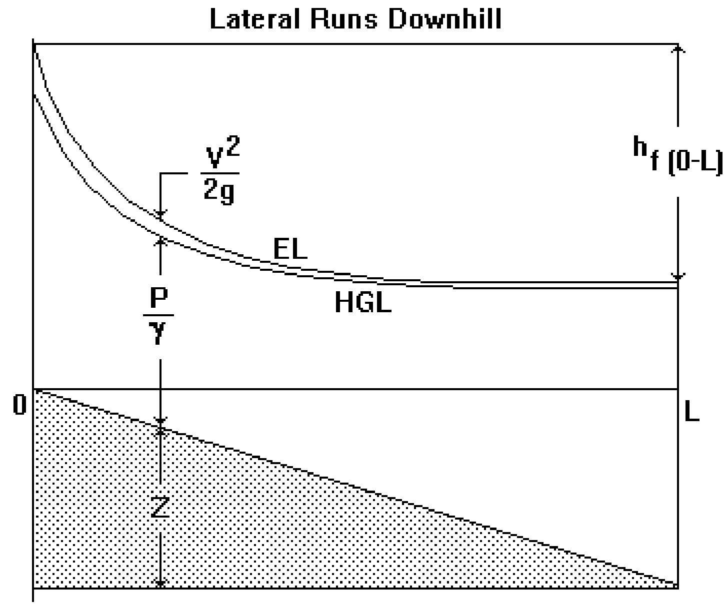
> *"pressure head variation in the lateral should not exceed 20% of the average design pressure for the sprinklers"*

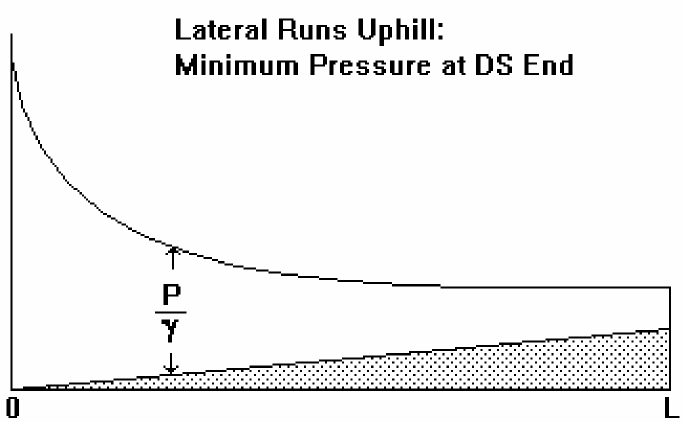
- This is a design assumption that has been used for many years, and is based on a great deal of experience
- The 20% for pressure variation is not an "exact" value; rather, it is based on judgment and some cost comparisons
- A designer could change this value, but it would affect system performance (uniformity), initial system cost, operating cost, and possibly other factors
- Computer programs could be written to search for an "optimal" percent pressure variation according to initial and operating costs, and according to crop value -- such an "optimal" value would vary from system to system

IV. Sprinkler Lateral Orientation

- It is usually preferable to run laterals on contours (zero slope) so that pressure variation in the lateral pipes is due to friction loss only
- It is advantageous to run laterals downhill, if possible, because the gain in energy due to elevation change will allow longer laterals without violating the 20% rule. But, if the slope is too steep, pressure regulators or flow control nozzles may be desirable.
- If the ground slope is equal to the friction loss gradient, the pressure in the lateral will be constant. However, the friction loss gradient is nonlinear because the flow rate is decreasing with distance along the lateral.


Zero Slope along Lateral
Pinlet
ΔP
Pa
Pend
0
L


Lateral Runs Downhill
V2/2g
hf (0-L)
EL
HGL
P/γ
0
L
Z


Lateral Runs Uphill:
Minimum Pressure at DS End
P/γ
0
L

- It is usually not recommendable to run laterals in an uphill direction. In this case:

    1. both friction loss and elevation are working to reduce pressure toward the end of the lateral, and length is more restricted if the 20% rule is still used
    2. However, for small slopes, running laterals uphill may be required to reduce the total length of the mainline pipe

- Note that $V^2/2g$ in the lateral pipe is normally converted into total head as the water flows through the nozzle body. Therefore, the velocity head (and EL) should normally be considered in lateral design. However, since a portion of the velocity head is lost during deceleration of the water at the entrance into risers and as turbulence inside the sprinkler head, and since $V^2/2g$ in a lateral pipe is typically small (< 1 ft of head, or 0.2 psi, or 0.3 m head, or 3 kPa), it is normally neglected during design, and the HGL is used.
- Aside from limits on pressure variation, laterals should be oriented so that they move in the direction of the prevailing winds -- this is because of salinity problems and application uniformity

- Figure 7.1 gives examples of layouts on different topographies

V. Lateral Sizing Limitations

- lateral pipes can be designed with multiple diameters to accommodate desirable pressure distributions, but...
- hand-move laterals should have only one or two different pipe sizes to simplify handling during set changes
- in practice, hand-move systems and wheel lines usually have only one size of lateral pipe
- some wheel lines, greater than 400 m in length, may have 5-inch pipe near the inlet and then 4-inch pipe at the end

Layout of Mainline for Set Sprinklers

I. Mainline Layout and Sizing

- if possible, run the mainline up or down slope so the laterals can be on contours (lateral pressure variation due to friction loss only)
- can also run the mainline along a ridge so the laterals run downhill on both sides (lateral friction loss partially offset by elevation change)
- should consider possible future expansions when sizing the mainline

"Split-Line" Lateral Operation:

- laterals operate on both sides of the mainline
- the mainline can be sized for only half capacity halfway down the mainline if laterals are run in different directions
- sometimes interferes with cultural practices
- it is convenient to have the water supply in the center of one side of the field, but this is seldom a design variable (the well is already there, or the canal is already there)
- may not need pumping if the water supply is at a higher elevation than the field elevation (e.g. 50 psi = 115 ft or 35 m of head) -- when pumping is not required, this changes the mainline layout and pipe sizing strategy
- in some cases it will be justifiable to include one or more booster pumps in the design -- even when the water source is a well (the well pump may not provide enough pressure for <u>any</u> of the lateral settings)
- we will discuss mainline economics in the next few lectures, then we will look at mainline design in more detail later

II. Design Variables to Accommodate Layout

- Number of sprinklers operating
- Average application rate
- Gross application depth
- Average sprinkler discharge
- Sprinkler spacing
- Operating hours per day
- Irrigation frequency
- Total operating time (fT)
- System capacity
- Percent probability of rain during peak-use period
- MAD

- It may be necessary to adjust the layout if a suitable combination of the above variables cannot be found
- Can also use flow control nozzles or pressure regulators to accommodate a given layout

III. Sample Calculation

- Consider a periodic-move system with S_l = 50 ft, S_e = 40 ft, f = 8 days, T = 11.5 hrs @ 2 sets/day, d = 2.7", and q_a = 4.78 gpm
- The field size is 80 acres (½ of a "quarter section"), 2,640 ft on one side and 1,320 ft on the other, rectangular
- The laterals will have to be 1,320-ft long

- System capacity:

$$Q_s = \frac{453(80\text{ ac})(2.7\text{ inch})}{(8\text{ days})(2\text{ sets/day})(11.5\text{ hrs/set})} = 532\text{ gpm} \quad (64)$$

- Number of sprinklers operating:

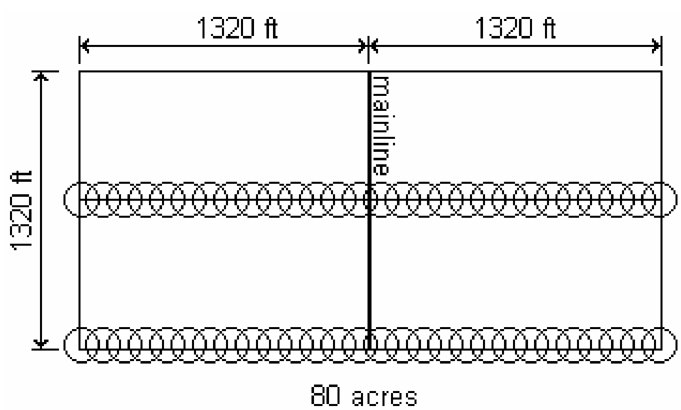
$$N_s = \frac{Q_s}{q_a} = \frac{532}{4.78} = 111\text{ sprinklers} \quad (65)$$

- Number of laterals,

$$\frac{1320\text{ ft/lateral}}{40\text{ ft/sprinkler}} = 33\text{ sprinklers/lateral} \quad (66)$$

$$\frac{111\text{ sprinklers}}{33\text{ sprinklers/lateral}} = 3.36\text{ laterals} \quad (67)$$

...so, round up to <u>4 laterals</u>



- Thus, two laterals on each side of the mainline (symmetry)

$$\frac{1320\text{ ft per lateral pair}}{50\text{ ft/position}} = 26.4 \quad (68)$$

- Round this up from 26.4 to 27 positions per lateral pair
- This gives 2 x 27 = 54 total lateral positions, and 54/4 = 13.5 sets/lateral
- Use 13 sets for two laterals and 14 sets for the other two laterals
- Then, there will be 14 sets per irrigation, even though the last set will only have two laterals operating

- Adjusted irrigation frequency:

$$f = \frac{14\text{ sets}}{2\text{ sets/day}} = 7\text{ days} \qquad (69)$$

- Note that the value of f was for an 8-day interval
- Thus, we need to increase Q_s to complete the irrigation in less time

- Adjusted system capacity:

$$\begin{aligned} Q_s &= (4\text{ laterals})(33\text{ sprinklers/lateral})(4.78\text{ gpm/sprinkler}) \\ &= 631\text{ gpm} \end{aligned} \qquad (70)$$

- Another way to adjust the system capacity:

$$Q_s = \left(\frac{8\text{ days}}{7\text{ days}}\right)(532\text{ gpm}) = 608\text{ gpm} \qquad (71)$$

- You might say that we are "effectively" finishing in somewhat less than 7 days, because the last set has only two laterals in operation, giving a system capacity of 608 instead of 631
- Consider this calculation: there are 2 x 13 + 2 x 14 = 54 sets, but the last 2 sets have only 2 laterals. So, (52/54) x 631 = 608 gpm, as calculated above.
- Which is correct?

- There are (52/54)*(4 laterals) = 3.85 laterals operating *on average* during each irrigation of the field
- However, you cannot always base the system capacity on the average number of laterals operating
- The system capacity should be based on the "worst case", which is when all four laterals operate simultaneously
- This means that the required capacity is 631 gpm, not 608 gpm
- Note that many farmers will accept some increase in system capital cost to provide more operational flexibility and safety
- In summary, we have essentially lowered f to accommodate the system configuration (layout), but:

    - same gross depth
    - same number of hours per set
    - same sprinkler flow rate
    - same sprinkler spacing
    - increased system capacity

Pipeline Hydraulics

I. Review

- Read Chapter 8 of the textbook to review the hydraulics of pipelines
- For pipe friction loss we will be using the Hazen-Williams and Darcy-Weisbach equations
- Be familiar with the Moody diagram, for use with the Darcy-Weisbach equation
- You can use the Swamee-Jain equation instead of the Moody diagram:

$$f = \frac{0.25}{\left[\log_{10}\left(\frac{\varepsilon}{3.75D} + \frac{5.74}{N_R^{0.9}}\right)\right]^2} \tag{72}$$

which is valid for turbulent flow in the range: $4{,}000 \le N_R \le 1.0(10)^8$. The ratio ε/D is called "relative roughness." The roughness height, ε, varies widely

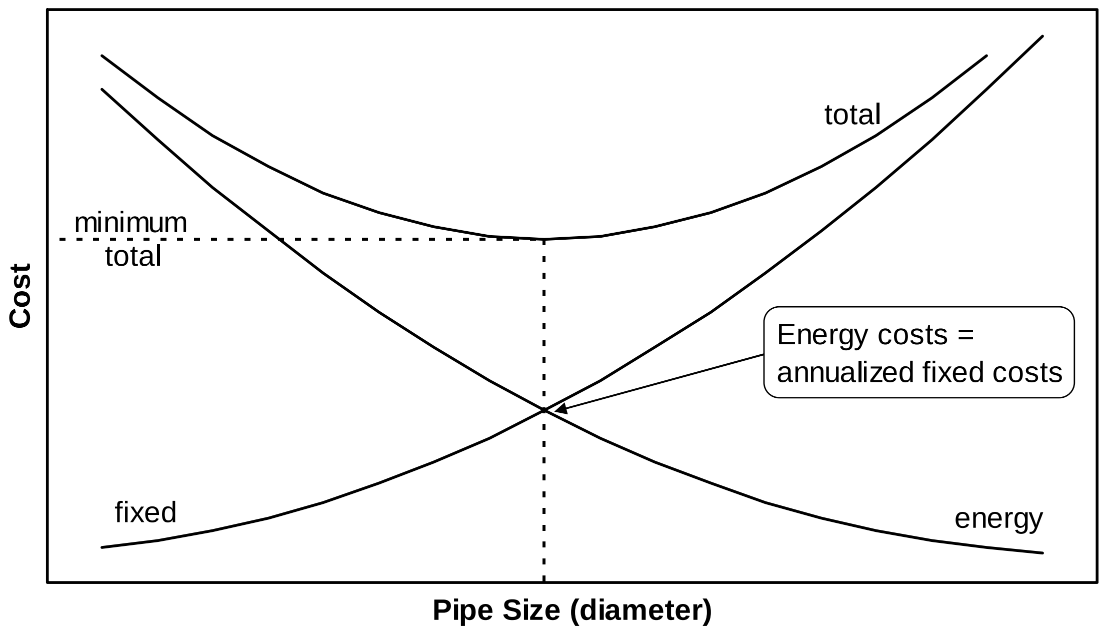
- We will also use the Blasius equation (Eq. 8.6) to determine the value of "f," in some cases, for "smooth pipes"

Lecture 6

Economic Pipe Selection Method

I. Introduction

- The *economic pipe selection method* (Chapter 8 of the textbook) is used to balance fixed (initial) costs for pipe with annual energy costs for pumping
- With larger pipe sizes the average flow velocity for a given discharge decreases, causing a corresponding decrease in friction loss
- This reduces the head on the pump, and energy can be saved
- However, larger pipes cost more to purchase



- To balance these costs and find the minimum cost we will annualize the fixed costs, compare with annual energy (pumping) costs
- We can also graph the results so that pipe diameters can be selected according to their maximum flow rate
- We will take into account interest rates and inflation rates to make the comparison
- This is basically an "engineering economics" problem, specially adapted to the selection of pipe sizes

- This method involves the following principal steps:

  1. Determine the equivalent annual cost for purchasing each available pipe size
  2. Determine the annual energy cost of pumping

3. Balance the annual costs for adjacent pipe sizes
4. Construct a graph of system flow rate versus section flow rate on a log-log scale for adjacent pipe sizes

- We will use the method to calculate "cut-off" points between adjacent pipe sizes so that we know which size is more economical for a particular flow rate
- We will use HP and kW units for power, where about ¾ of a kW equals a HP
- Recall that a Watt (W) is defined as a joule/second, or a N-m per second
- Multiply W by elapsed time to obtain Newton-meters ("work", or "energy")

II. Economic Pipe Selection Method Calculations

1. Select a period of time over which comparisons will be made between fixed and annual costs. This will be called the *useful life* of the system, n, in years.

   - The "useful life" is a subjective value, subject to opinion and financial amortization conditions
   - This value could alternatively be specified in months, or other time period, but the following calculations would have to be consistent with the choice

2. For several different pipe sizes, calculate the *uniform annual cost* of pipe per unit length of pipe.

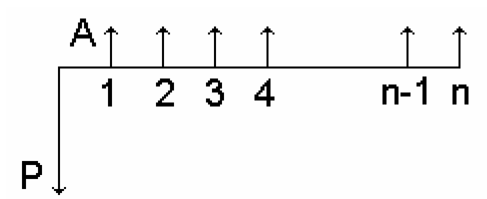
   - A unit length of 100 (m or ft) is convenient because J is in m/100 m or ft/100 ft, and you want a fair comparison (the actual pipe lengths from the supplier are irrelevant for these calculations)
   - You must use consistent units ($/100 ft or $/100 m) throughout the calculations, otherwise the ΔJ values will be incorrect (see Step 11 below)
   - So, you need to know the cost per unit length for different pipe sizes
   - PVC pipe is sometimes priced by weight of the plastic material (weight per unit length depends on diameter and wall thickness)
   - You also need to know the annual interest rate upon which to base the calculations; this value will take into account the time value of money, whereby you can make a fair comparison of the cost of a loan versus the cost of financing it "up front" yourself
   - In any case, we want an equivalent uniform annual cost of the pipe over the life of the pipeline
   - Convert fixed costs to equivalent uniform annual costs, UAC, by using the "capital recovery factor", CRF

$$UAC = P(CRF) \tag{73}$$

$$CRF = \frac{i(1+i)^n}{(1+i)^n - 1} \tag{74}$$

where P is the cost per unit length of pipe; i is the annual interest rate (fraction); and n is the number of years (useful life)

- Of course, i could also be the monthly interest rate with n in months, etc.



- Make a table of UAC values for different pipe sizes, per unit length of pipe
- The CRF value is the same for all pipe sizes, but P will change depending on the pipe size
- Now you have the equivalent annual cost for each of the different pipe sizes

3. Determine the number of operating (pumping) hours per year, O_t:

$$O_t = \frac{\text{(irrigated area)(gross annual depth)}}{\text{(system capacity)}} = \text{hrs / year} \tag{75}$$

- Note that the maximum possible value of O_t is 8,760 hrs/year (for 365 days)
- Note also that the "gross depth" is annual, so if there is more than one growing season per calendar year, you need to include the sum of the gross depths for each season (or fraction thereof)

4. Determine the pumping plant efficiency:

- The total plant efficiency is the product of pump efficiency, E_{pump}, and motor efficiency, E_{motor}

$$E_p = E_{pump} E_{motor} \tag{76}$$

- This is equal to the ratio of "water horsepower", WHP, to "brake horsepower", BHP ($E_{pump} \equiv$ WHP/BHP)

- Think of BHP as the power going into the pump through a spinning shaft, and WHP is what you get out of the pump – since the pump is not 100% efficient in energy conversion, WHP < BHP
- WHP and BHP are archaic and confusing terms, but are still in wide use
- E_{motor} will usually be 92% or higher (about 98% with newer motors and larger capacity motors)
- E_{pump} depends on the pump design and on the operating point (Q vs. TDH)
- WHP is defined as:

$$\mathrm{WHP} = \frac{QH}{102} \tag{77}$$

where Q is in lps; H is in m of head; and WHP is in kilowatts (kW)

- If you use m in the above equation, UAC must be in $/100 m
- If you use ft in the above equation, UAC must be in $/100 ft

- Note that for fluid flow, "power" can be expressed as $\rho gQH = \gamma QH$
- Observe that 1,000/g = 1,000/9.81 ≈ 102, for the above units (other conversion values cancel each other and only the 102 remains)
- The denominator changes from 102 to 3,960 for Q in gpm, H in ft, and WHP in HP

5. Determine the present annual energy cost:

$$E = \frac{O_t C_f}{E_p} \tag{78}$$

where C_f is the cost of "fuel"

- For electricity, the value of C_f is usually in dollars per kWh, and the value used in the above equation may need to be an "average" based on potentially complex billing schedules from the power company
- For example, in addition to the energy you actually consume in an electric motor, you may have to pay a monthly fee for the installed capacity to delivery a certain number of kW, plus an annual fee, plus different *time-of-day* rates, and others
- Fuels such as diesel can also be factored into these equations, but the power output per liter of fuel must be estimated, and this depends partly on the engine and on the maintenance of the engine
- The units of E are dollars per WHP per year, or dollars per kW per year; so it is a marginal cost that depends on the number of kW actually required

6. Determine the marginal equipment cost:

- Note that C_f can include the "marginal" cost for the pump and power unit (usually an electric motor)
- In other words, if a larger pump & motor costs more than a smaller pump, then C_f should reflect that, so the full cost of friction loss is considered
- If you have higher friction loss, you may have to pay more for energy to pump, but you may also have to buy a larger pump and/or power unit (motor or engine)
- It sort of analogous to the Utah Power & Light *monthly power charge*, based solely on the capacity to deliver a certain amount of power

  C_f ($/kWh) = energy cost + marginal cost for a larger pump & motor

  where "marginal" is the incremental unit cost of making a change in the size of a component

- This is not really an "energy" cost per se, but it is something that can be taken into account when balancing the fixed costs of the pipe (it falls under the *operating costs* category, increasing for decreasing pipe costs)
- That is, maybe you can pay a little more for a larger pipe size and avoid the need to buy a bigger pump, power unit and other equipment

- To calculate the marginal annual cost of a pump & motor:

$$MAC = \frac{CRF\left(\$_{big} - \$_{small}\right)}{O_t\left(kW_{big} - kW_{small}\right)} \tag{79}$$

  where MAC has the same units as C_f; and $\$_{big}$ -$\$_{small}$ is the difference in pump+motor+equipment costs for two different capacities

- The difference in fixed purchase price is annualized over the life of the system by multiplying by the CRF, as previously calculated
- The difference in pump size is expressed as ΔBHP, where ΔBHP is the difference in brake horsepower, expressed as kW
- To determine the appropriate pump size, base the smaller pump size on a low friction system (or low pressure system)
- For BHP in kW:

$$BHP = \frac{Q_s H_{pump}}{102 E_p} \tag{80}$$

- Round the BHP up to the next larger available pump+motor+equipment size to determine the size of the larger pump
- Then, the larger pump size is computed as the next larger available pump size as compared to the smaller pump
- Then, compute the MAC as shown above
- The total pump cost should include the total present cost for the pump, motor, electrical switching equipment (if appropriate) and installation
- C_f is then computed by adding the cost per kWh for energy

- Note that this procedure to determine MAC is approximate because the marginal costs for a larger pump+motor+equipment will depend on the magnitude of the required power change
- Using $\$_{big}$ -$\$_{small}$ to determine MAC only takes into account two (possibly adjacent) capacities; going beyond these will likely change the marginal rate
- However, at least we have a simple procedure to attempt to account for this potentially real cost

7. Determine the equivalent annualized cost factor:

- This factor takes inflation into account:

$$EAE = \left[\frac{(1+e)^n - (1+i)^n}{e-i}\right]\left[\frac{i}{(1+i)^n - 1}\right] \tag{81}$$

where e is the annual inflation rate (fraction), i is the annual interest rate (fraction), and n is in years

- Notice that for e = 0, EAE = unity (this makes sense)
- Notice also that the above equation has a mathematical singularity for e = i (but i is usually greater than e)

8. Determine the equivalent annual energy cost:

$$E' = (EAE)(E) \tag{82}$$

- This is an adjustment on E for the expected inflation rate
- No one really knows how the inflation rate might change in the future
- How do you know when to change to a larger pipe size (based on a certain sectional flow rate)?

Beginning with a smaller pipe size (e.g. selected based on maximum velocity limits), you would change to a larger pipe size along a section of pipeline if the *difference in cost for the next larger pipe size is less than the difference in energy (pumping) savings*

- Recall that the velocity limit is usually taken to be about 5 fps, or 1.5 m/s

9. Determine the difference in WHP between adjacent pipe sizes by equating the annual plus annualized fixed costs for two adjacent pipe sizes:

$$E'\left(HP_{s1}\right) + UAC_{s1} = E'\left(HP_{s2}\right) + UAC_{s2} \tag{83}$$

or,

$$\Delta WHP_{s1-s2} = \frac{\left(UAC_{s2} - UAC_{s1}\right)}{E'} \tag{84}$$

- The subscript s_1 is for the smaller of the two pipe sizes
- The units of the numerator might be \$/100 m per year; the units of the denominator might be \$/kW per year
- This is the WHP (energy) savings needed to offset the annualized fixed cost difference for purchasing two adjacent pipe sizes; it is the economic balance point

10. Determine the difference in friction loss gradient between adjacent pipe sizes:

$$\Delta J_{s1-s2} = 102\left(\frac{\Delta WHP_{s1-s2}}{Q_s}\right) \tag{85}$$

- This is the head loss difference needed to balance fixed and annual costs for the two adjacent pipe sizes
- The coefficient 102 is for Q_s in lps, and ΔWHP in kW
- You can also put Q_s in gpm, and ΔWHP in HP, then substitute 3,960 for 102, and you will get exactly the same value for ΔJ
- As before, ΔJ is a head loss gradient, in head per 100 units of length (m or ft, or any other unit)
- Thus, ΔJ is a dimensionless "percentage": head, H, can be in m, and when you define a unit length (e.g. 100 m), the H per unit meter becomes dimensionless
- This is why you can calculate ΔJ using any consistent units and you will get the same result

11. Calculate the flow rate corresponding to this head loss difference:

- Using the Hazen-Williams equation:

$$\Delta J = J_{s1} - J_{s2} = 16.42(10)^6 \left(\frac{q}{C}\right)^{1.852} \left(D_{s1}^{-4.87} - D_{s2}^{-4.87}\right) \tag{86}$$

where q is in lps, and D is the inside diameter of the pipe in cm

- Or, using the Darcy-Weisbach equation:

$$\Delta J = \frac{800\, f\, q^2}{g\pi^2}\left(D_{s1}^{-5} - D_{s2}^{-5}\right) \tag{87}$$

- Solve for the flow rate, q (with q in lps; D in cm):

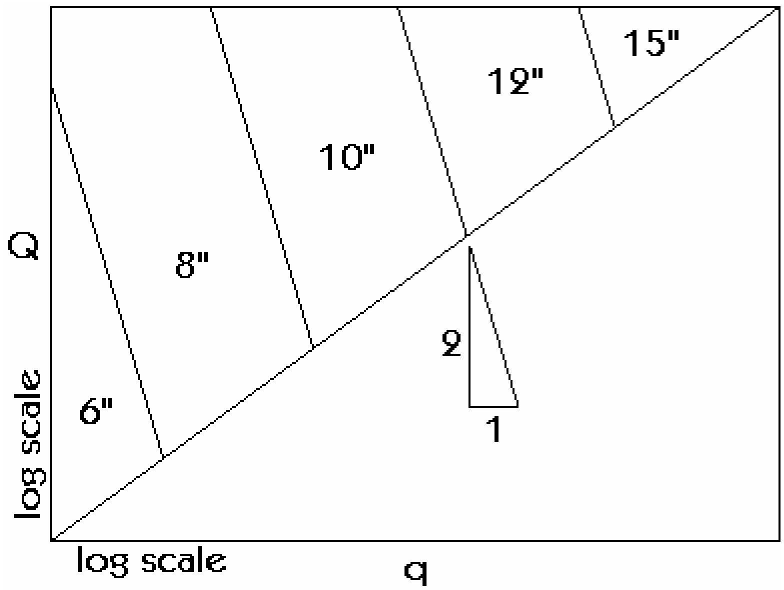
$$q = C\left[\frac{\Delta J}{16.42(10)^6\left(D_{s1}^{-4.87} - D_{s2}^{-4.87}\right)}\right]^{0.54} \tag{88}$$

- This is the flow rate for which either size (D_{s1} or D_{s2}) will be the most economical (it is the balancing point between the two adjacent pipe sizes)
- For a larger flow rate you would choose size D_{s2}, and vice versa

12. Repeat steps 8 through 11 for all other adjacent pipe sizes.

13. You can optionally create a graph with a log-log scale with the system flow rate, Q_s, on the ordinate and the section flow rate, q, on the abscissa:

    - Plot a point at Q_s and q for each of the adjacent pipe sizes
    - Draw a straight diagonal line from lower left to upper right corner
    - Draw a straight line at a slope of -1.852 (or -2.0 for Darcy-Weisbach) through each of the points
    - The slope will be different if the log scale on the axes are not the same distance (e.g. if you do the plot on a spreadsheet, the ordinate and abscissa may be different lengths, even if the same number of log cycles).
    - In constructing the graph, you can get additional points by changing the system flow rate, but in doing so you should also increase the area, A, so that O_t is approximately the same as before. It doesn't make sense to change the system flow rate arbitrarily.
    - Your graph should look similar to the one shown below



14. Applying the graph.

- Find the needed flow rate in a given section of the pipe, q, make an intersection with the maximum system capacity (Q_s, on the ordinate), then see which pipe size it is
- You can use the graph for different system capacities, assuming you are considering different total irrigated areas, or different crop and or climate values
- Otherwise, you can just skip step 13 and just do the calculations on a spreadsheet for the particular Q_s value that you are interested in
- The graph is perhaps didactic, but doesn't need to be constructed to apply this economic pipe selection method

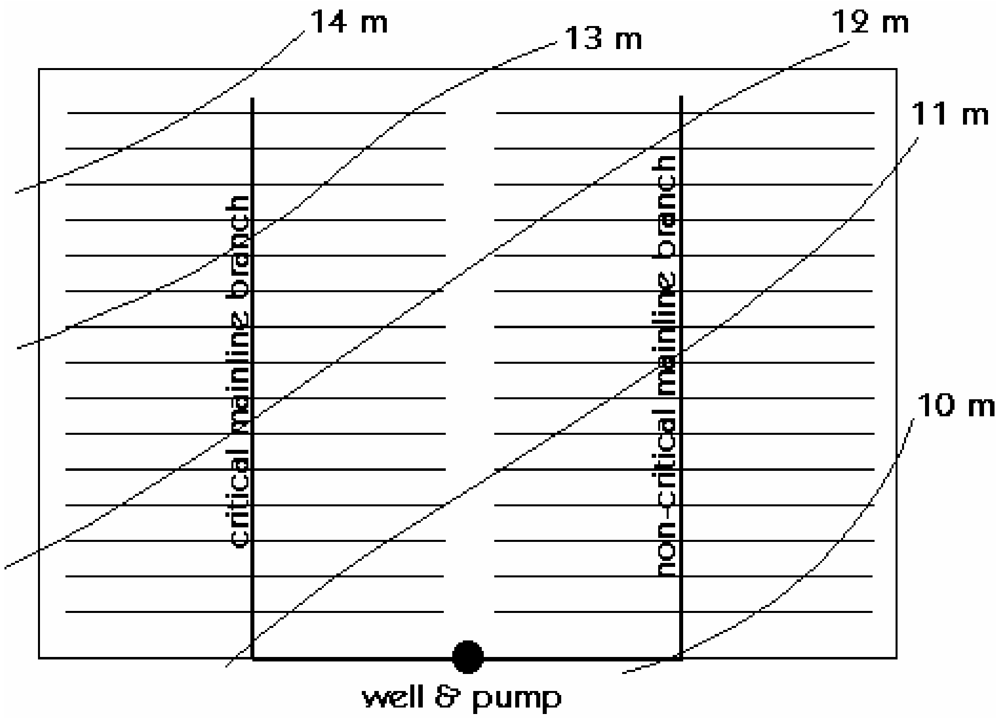
III. Notes on the Use of this Method

1. If any of the economic factors (interest rate, inflation rate, useful life of the system) change, the lines on the graph will shift up or down, but the slope remains the same (equal to the inverse of the velocity exponent for the head loss equation: 1.852 for Hazen-Williams and 2.0 for Darcy-Weisbach).

2. Computer programs have been developed to use this and other economic pipe selection methods, without the need for constructing a graphical solution on log-log paper. You could write such a program yourself.

3. The economic pipe selection method presented above is ***not necessarily valid*** for:
   - looping pipe networks
   - very steep downhill slopes
   - non-"worst case" pipeline branches

4. For loops, the flow might go in one direction some of the time, and in the opposite direction at other times. For steep downhill slopes it is not necessary to balance annual operation costs with initial costs because there is essentially no cost associated with the development of pressure – there is no need for pumping. Non-“worst case” pipeline branches may not have the same pumping requirements (see below).



5. Note that the equivalent annual pipe cost considers the annual interest rate, but not inflation. This is because financing the purchase of the pipe would be done at the time of purchase, and we are assuming a fixed interest rate. The uncertainty in this type of financing is assumed by the lending agency.

6. This method is not normally used for designing pipe sizes in laterals. For one thing, it might recommend too many different sizes (inconvenient for operation of periodic-move systems). Another reason is that we usually use different criteria to design laterals (the “20%” rule on pressure variation).

7. Other factors could be included in the analysis. For example, there may be certain taxes or tax credits that enter into the decision making process. In general, the analysis procedure in determining pipe sizes can get as complicated as you want it to – but higher complexity is better justified for larger, more expensive irrigation systems.

IV. Other Pipe Sizing Methods

- Other methods used to size pipes include the following:

  1. **Unit head loss method**: the designer specifies a limit on the allowable head loss per unit length of pipe
  2. **Maximum velocity method**: the designer specifies a maximum average velocity of flow in the pipe (about 5 to 7 ft/s, or 1.5 to 2.0 m/s)
  3. **Percent head loss method**: the designer sets the maximum pressure variation in a section of the pipe, similar to the 20%P_a rule for lateral pipe sizing

- It is often a good idea to apply more than one pipe selection method and compare the results
- For example, don't accept a recommendation from the economic selection method if it will give you a flow velocity of more than about 10 ft/s (3 m/s), otherwise you may have water hammer problems during operation
- However, it is usually advisable to at least apply the economic selection method unless the energy costs are very low
- In many cases, the same pipe sizes will be selected, even when applying different methods

- For a given average velocity, V, in a circular pipe, and discharge, Q, the required inside pipe diameter is:

$$D = \sqrt{\frac{4Q}{\pi V}} \tag{89}$$

- The following tables show maximum flow rates for specified average velocity limits and different pipe inside diameters

Gallons per Minute			
		Velocity Limit	
D (inch)	A (ft2)	5 fps	7 fps
0.5	0.00136	**3.1**	**4.3**
0.75	0.00307	**6.9**	**9.6**
1	0.00545	**12.2**	**17.1**
1.25	0.00852	**19.1**	**26.8**
1.5	0.01227	**27.5**	**38.6**
2	0.02182	**49.0**	**68.5**
3	0.04909	**110**	**154**
4	0.08727	**196**	**274**
5	0.13635	**306**	**428**
6	0.19635	**441**	**617**
8	0.34907	**783**	**1,097**
10	0.54542	**1,224**	**1,714**
12	0.78540	**1,763**	**2,468**
15	1.22718	**2,754**	**3,856**
18	1.76715	**3,966**	**5,552**
20	2.18166	**4,896**	**6,855**
25	3.40885	**7,650**	**10,711**
30	4.90874	**11,017**	**15,423**
40	8.72665	**19,585**	**27,419**
50	13.63538	**30,602**	**42,843**

Cubic Feet per Second			
		Velocity Limit	
D (ft)	A (ft2)	5 fps	7 fps
1	0.785	**3.93**	**5.50**
2	3.142	**15.71**	**21.99**
3	7.069	**35.34**	**49.48**
4	12.566	**62.83**	**87.96**
5	19.635	**98.17**	**137.44**
6	28.274	**141.37**	**197.92**
7	38.485	**192.42**	**269.39**
8	50.265	**251.33**	**351.86**
9	63.617	**318.09**	**445.32**
10	78.540	**392.70**	**549.78**
11	95.033	**475.17**	**665.23**
12	113.097	**565.49**	**791.68**
13	132.732	**663.66**	**929.13**
14	153.938	**769.69**	**1,077.57**
15	176.715	**883.57**	**1,237.00**
16	201.062	**1,005.31**	**1,407.43**
17	226.980	**1,134.90**	**1,588.86**
18	254.469	**1,272.35**	**1,781.28**
19	283.529	**1,417.64**	**1,984.70**
20	314.159	**1,570.80**	**2,199.11**

Litres per Second			
		Velocity Limit	
D (mm)	A (m2)	1.5 m/s	2 m/s
10	0.00008	**0.1**	**0.2**
20	0.00031	**0.5**	**0.6**
25	0.00049	**0.7**	**1.0**
30	0.00071	**1.1**	**1.4**
40	0.00126	**1.9**	**2.5**
50	0.00196	**2.9**	**3.9**
75	0.00442	**6.6**	**8.8**
100	0.00785	**11.8**	**15.7**
120	0.01131	**17.0**	**22.6**
150	0.01767	**26.5**	**35.3**
200	0.03142	**47.1**	**62.8**
250	0.04909	**73.6**	**98.2**
300	0.07069	**106**	**141**
400	0.12566	**188**	**251**
500	0.19635	**295**	**393**
600	0.28274	**424**	**565**
700	0.38485	**577**	**770**
800	0.50265	**754**	**1,005**
900	0.63617	**954**	**1,272**
1000	0.78540	**1,178**	**1,571**
1100	0.95033	**1,425**	**1,901**
1200	1.13097	**1,696**	**2,262**
1300	1.32732	**1,991**	**2,655**
1400	1.53938	**2,309**	**3,079**
1500	1.76715	**2,651**	**3,534**
1600	2.01062	**3,016**	**4,021**
1700	2.26980	**3,405**	**4,540**
1800	2.54469	**3,817**	**5,089**
1900	2.83529	**4,253**	**5,671**
2000	3.14159	**4,712**	**6,283**
2100	3.46361	**5,195**	**6,927**
2200	3.80133	**5,702**	**7,603**
2300	4.15476	**6,232**	**8,310**
2400	4.52389	**6,786**	**9,048**
2500	4.90874	**7,363**	**9,817**
2600	5.30929	**7,964**	**10,619**
2700	5.72555	**8,588**	**11,451**
2800	6.15752	**9,236**	**12,315**
2900	6.60520	**9,908**	**13,210**
3000	7.06858	**10,603**	**14,137**
3100	7.54768	**11,322**	**15,095**
3200	8.04248	**12,064**	**16,085**
3300	8.55299	**12,829**	**17,106**
3400	9.07920	**13,619**	**18,158**

Lecture 7

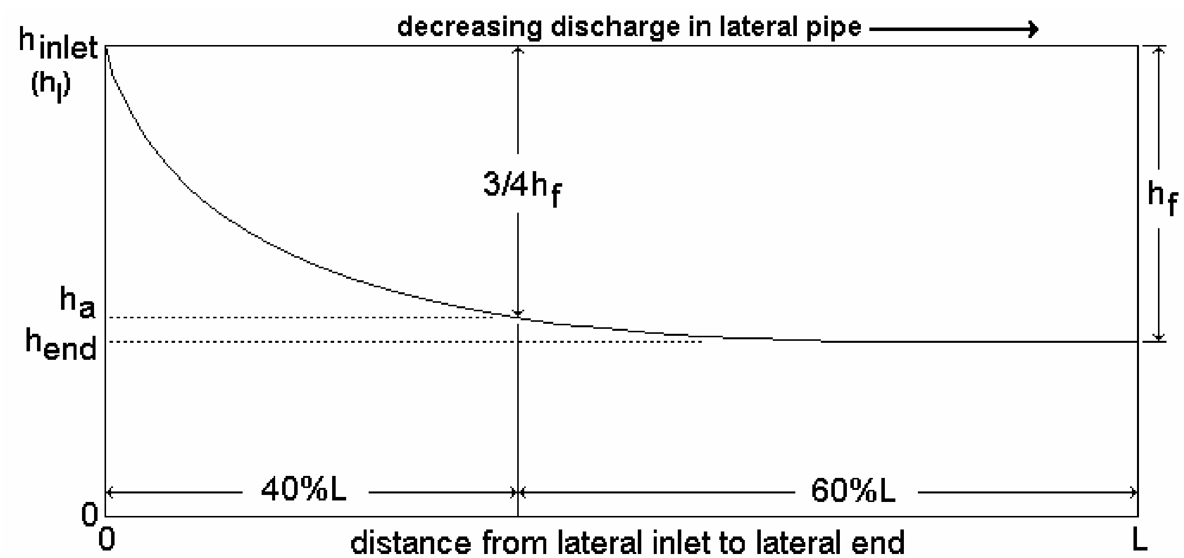
Set Sprinkler Lateral Design

I. Basic Design Criterion

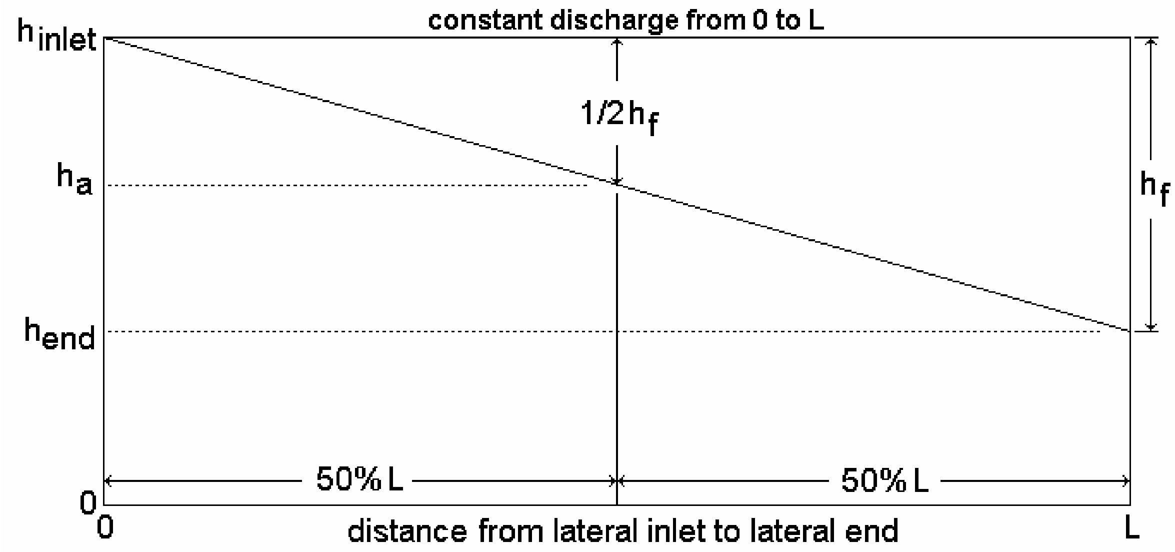
- The basic design criterion is to size lateral pipes so that pressure variation along the length of the lateral does not exceed *20% of the nominal design pressure* for the sprinklers
- This criterion is a compromise between cost of the lateral pipe and application uniformity in the direction of the lateral
- Note that the locations of maximum and minimum pressure along a lateral pipe can vary according to ground slope and friction loss gradient

II. Location of Average Pressure in the Lateral

- We are interested in the location of average pressure along a lateral pipe because it is related to the design of the lateral
- Recall that friction head loss along a multiple-outlet pipe is nonlinear
- The figure below is for a lateral laid on level ground – pressure variation is due to friction loss only...



- For equally-spaced outlets (sprinklers) and approximately thirty outlets (or more), three-quarters of the pressure loss due to friction will occur between the inlet and the location of average pressure
- The location of average pressure in the lateral is approximately 40% of the lateral length, measured from the lateral inlet
- If there were only one outlet at the end of the lateral pipe, then one-half the pressure loss due to friction would take place between the lateral inlet and the location of average pressure, as shown below



- A computer program can be written to solve for the head loss in the lateral pipe between each sprinkler
- Consider the following equations:

Total friction head loss:

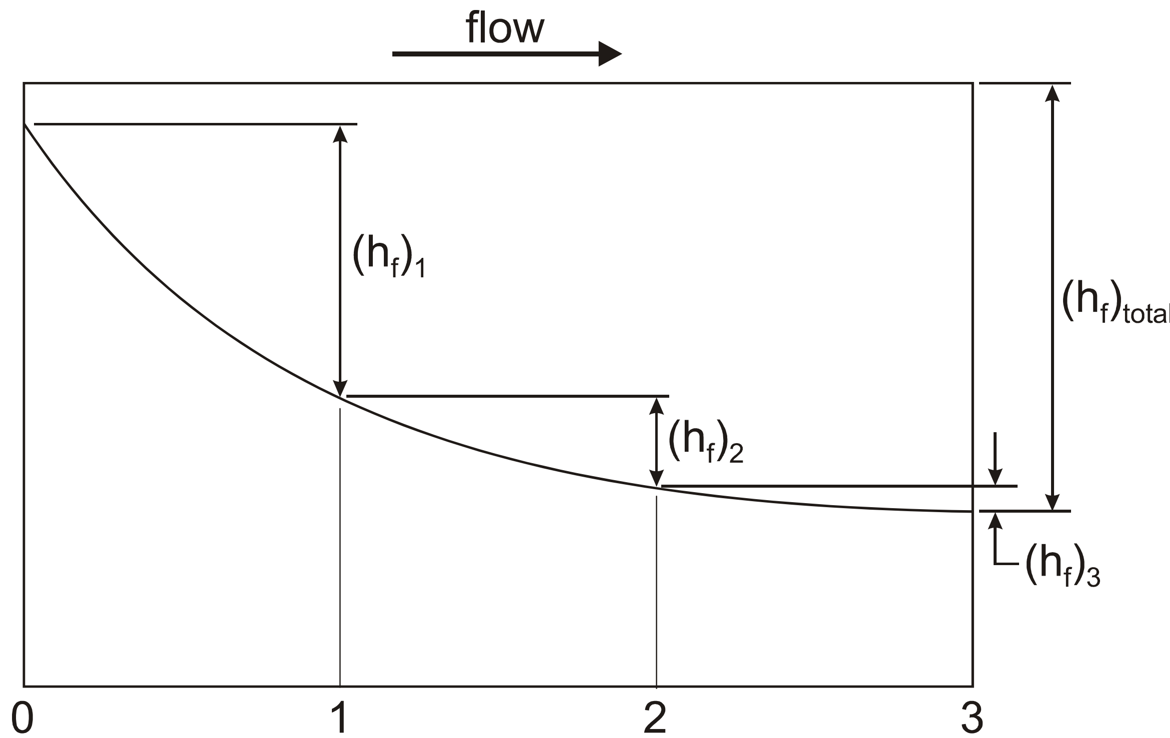
$$\left(h_f\right)_{total} = \sum_{i=1}^{n} \left(h_f\right)_i \tag{90}$$

Friction head loss to location of h_a:

$$\left(h_f\right)_a = \frac{\sum_{i=1}^{n} \sum_{j=1}^{i} \left(h_f\right)_j}{n+1} \tag{91}$$

where n is the number of sprinklers; $(h_f)_{total}$ is the total friction head loss from 0 to L; $(h_f)_i$ is the friction head loss in the lateral pipe between sprinklers i-1 and i; and $(h_f)_a$ is the friction loss from the lateral inlet to the location of h_a

- As indicated above, $(h_f)_a$ occurs over approximately the first 40% of the lateral
- Note that between sprinklers, the friction head loss gradient is linear in the lateral pipe
- Note also that $(h_f)_0 = 0$, but it is used in calculating $(h_a)_f$, so the denominator is (n+1), not n



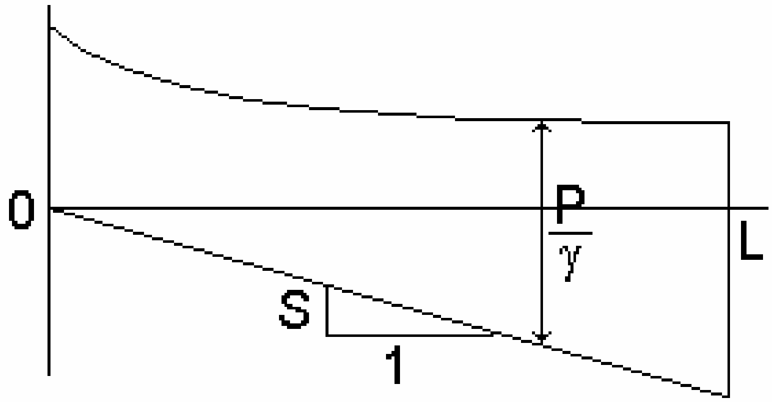
- In applying these equations with sample data, the following result can be found:

$$\frac{(h_f)_a}{(h_f)_{total}} \approx 0.73 \qquad (92)$$

- This supports the above claim that approximately ¾ of the friction head loss occurs between the lateral inlet and the location of h_a
- Also, from these calculations it can be seen that the location of h_a is approximately 38% of the lateral length, measured from the inlet, for laterals with approximately 30 or more sprinklers
- But, this analysis assumes a constant q_a, which is not quite correct unless flow control nozzles and or pressure regulators are used at each sprinkler
- We could eliminate this assumption of constant q_a, but it involves the solution of a system of nonlinear equations

III. Location of Minimum Pressure in Laterals Running Downhill

- The location of minimum pressure in a lateral running downhill is where the slope of the friction loss curve, J, equals the ground slope



- The above assertion is analogous to a pre-calculus “max-min problem”, where you take the derivative of a function and set it equal to zero (zero slope)
- Here we are doing the same thing, but the slope is not necessarily zero

Hazen-Williams Equation:

$$J = 1.21(10)^{12}\left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \tag{93}$$

for J in meters of friction head loss per 100 m (or ft/100 ft); Q in lps; and D in mm

- In this equation we will let:

$$Q = Q_l - \left(\frac{q_a}{S_e}\right)x \tag{94}$$

for multiple, equally-spaced sprinkler outlets spaced at S_e (m) from each other, with constant discharge of q_a (lps). Q_l is the flow rate at the lateral inlet (entrance).

- To find the location of minimum pressure, **let J = S**, where S is the ground slope (in %, because J is per 100 m), which is negative for downhill-sloping laterals
- Combining the two above equations and solving for x,

$$x = \frac{S_e}{q_a}\left[Q_l - 3(10)^{-7}\left(C(-S)^{0.54} D^{2.63}\right)\right] \tag{95}$$

where x is the distance, in m, from the lateral inlet to the minimum pressure

- S is in percent; S_e and x are in m; D is in mm; and Q_l and q_a are in lps
- Note that the valid range of x is: $0 \le x \le L$, and that you won't necessarily get J = S over this range of x values:

    - If you get x < 0 then the minimum pressure is at the inlet
    - If you get x > L then the minimum pressure is at the end

- This means that the above equation for x is valid for all ground slopes: S = 0, S > 0 and S < 0

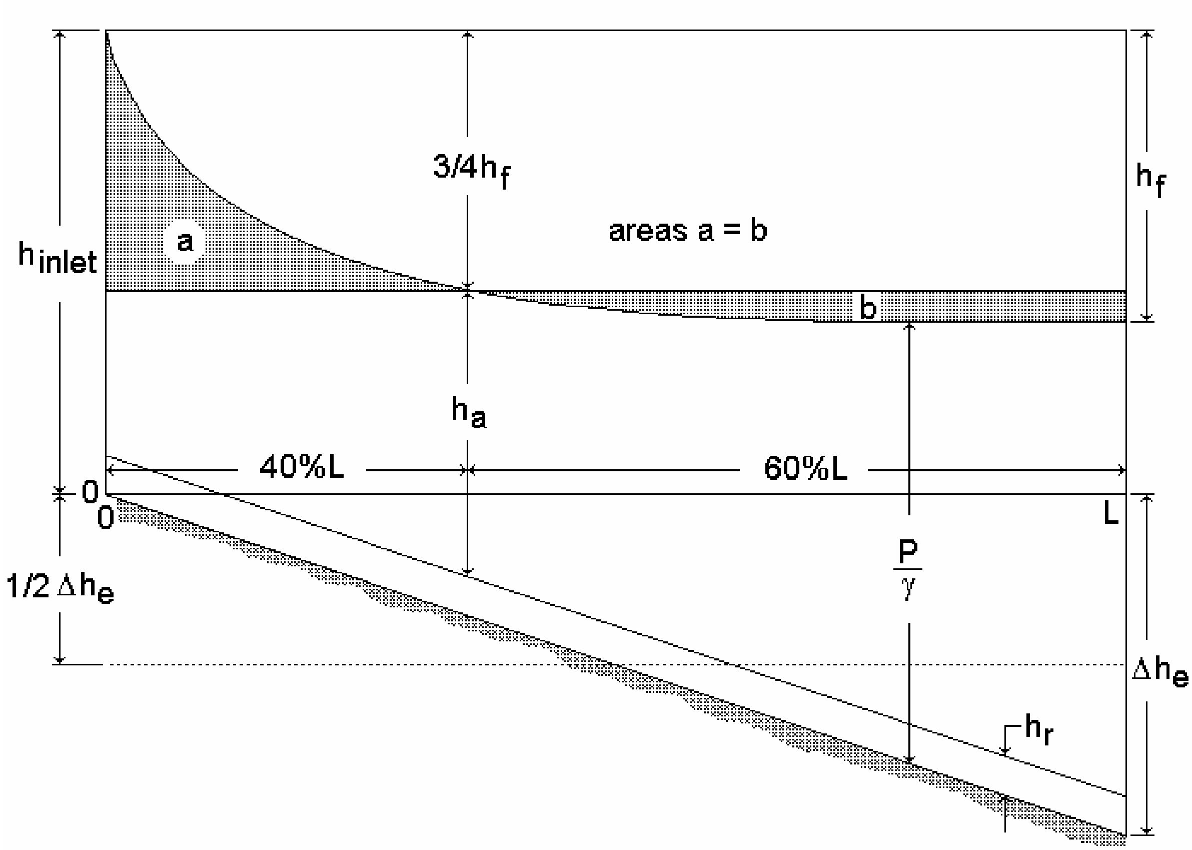
IV. Required Lateral Inlet Pressure Head

- Except for the most unusual circumstances (e.g. non-uniform downhill slope that exactly matches the shape of the h_f curve), the pressure will vary with distance in a lateral pipe
- According to Keller & Bliesner's design criterion, the required inlet pressure head to a sprinkler lateral is that which makes the average pressure in the lateral pipe equal to the required sprinkler pressure head, h_a

- We can force the average pressure to be equal to the desired sprinkler operating pressure by defining the lateral inlet pressure head as:

$$h_l = h_a + \frac{3}{4} h_f + \frac{1}{2} \Delta h_e \qquad (96)$$

- h_l is the required pressure head at the lateral inlet
- Strictly speaking, we should take approximately 0.4Δhe in the above equation, but we are taking separate averages for the friction loss and elevation gradients – and, this is a *design equation*
- Of course, instead of head, h, in the above equation, pressure, P, could be used if desired



The value of Δh_e is negative for laterals running downhill

- For steep downhill slopes, where the minimum pressure would be at the lateral inlet, it is best to let

$$h_f = -\Delta h_e \tag{97}$$

- Thus, we would want to consume, or "burn up", excess pressure through friction loss by using smaller pipes
- To achieve this equality for steep downhill slopes, it may be desirable to have more than one pipe diameter in the lateral
- A downhill slope can be considered "steep" when (approximately)…

$$-\Delta h_e > 0.3 h_a \tag{98}$$

- We now have an equation to calculate lateral inlet pressure based on h_a, h_f, and h_e

- However, for large values of h_f there will be correspondingly large values of h_l
- Thus, for zero ground slope, to impose a limit on h_f we will accept:

$$h_f = 0.20\,h_a \quad \text{(for S = 0 only)} \tag{99}$$

- This is the same as saying that we will not allow pipes that are too small, that is, pipes that would produce a large h_f value
- An additional head term must be added to the equation for h_l to account for the change in elevation from the lateral pipe to the sprinkler (riser height):

$$h_l = h_a + \frac{3}{4}h_f + \frac{1}{2}\Delta h_e + h_r \tag{100}$$

or, in terms of pressure…

$$P_l = P_a + \frac{3}{4}P_f + \frac{1}{2}\Delta P_e + P_r \tag{101}$$

V. Friction Losses in Pipes with Multiple Outlets

- Pipes with multiple outlets have decreasing flow rate with distance (in the direction of flow), and this causes the friction loss to decrease by approximately the square of the flow rate (for a constant pipe diameter)
- Sprinkler and trickle irrigation laterals fall into this hydraulic category
- Multiply the head loss for a constant discharge pipe by a factor "F" to reduce the total head loss for a lateral pipe with multiple, equally spaced outlets:

$$h_f = \frac{JFL}{100} \tag{102}$$

where F is from Eq. 8.9a

$$F = \frac{1}{b+1} + \frac{1}{2N} + \frac{\sqrt{b-1}}{6N^2} \tag{103}$$

for equally spaced outlets, each with the same discharge, and going all the way to the end of the pipe.

- All of the flow is assumed to leave through the outlets, with no "excess" spilled out the downstream end of the pipe
- N is the total number of equally spaced outlets
- The value of b is the exponent on Q in the friction loss equation

- Darcy-Weisbach: b = 2.0
- Hazen-Williams: b = 1.852

- The first sprinkler is assumed to be located a distance of S_e from the lateral inlet
- Eq. 8.9b (see below) gives $F(\alpha)$, which is the F factor for initial outlet spacings less than or equal to S_e

$$F(\alpha) = \frac{NF - (1-\alpha)}{N - (1-\alpha)} \tag{104}$$

where $0 < \alpha \leq 1$

- Note that when $\alpha = 1$, $F(\alpha) = F$
- Many sprinkler systems have the first sprinkler at a distance of $½S_e$ from the lateral inlet ($\alpha = 0.5$), when laterals run in both orthogonal directions from the mainline

VI. Lateral Pipe Sizing for a Single Pipe Size

- If the minimum pressure is at the end of the lateral, which is the case for no ground slope, uphill, and slight downhill slopes, then the change in pressure head over the length of the lateral is:

$$\Delta h = h_f + \Delta h_e \tag{105}$$

If we allow $\Delta h = 0.20\, h_a$, then

$$0.20 h_a = h_f + \Delta h_e \tag{106}$$

$$0.20 h_a - \Delta h_e = \frac{J_a F L}{100} \tag{107}$$

and,

$$J_a = 100\left(\frac{0.20 h_a - \Delta h_e}{FL}\right) \tag{108}$$

where J_a is the allowable friction loss gradient.

- Lateral pipe diameter can be selected such that $J \leq J_a$
- The above is part of a standard lateral design criteria and will give a system CU of approximately 0.97CU if lateral inlet pressures are the same for each lateral position, for set sprinkler systems

- If the lateral is sloping downhill and the minimum pressure does not occur at the end of the lateral, then we will attempt to consume the elevation gain in friction loss as follows:

$$h_f = -\Delta h_e \tag{109}$$

$$J_a = 100\left(\frac{-\Delta h_e}{FL}\right) \tag{110}$$

- Note that in this case $\Delta h \neq h_f + \Delta h_e$. Rather, $\Delta h = h_{max} - h_{min}$, where:

  1. h_{max} is either at the lateral inlet or at the end of the lateral, and
  2. h_{min} is somewhere between the lateral inlet and the end

- Given a value of J_a, the inside diameter of the lateral pipe can be calculated from the Hazen-Williams equation:

$$D = \left[\frac{K}{J_a}\left(\frac{Q_l}{C}\right)^{1.852}\right]^{0.205} \tag{111}$$

  where Q_l is the flow rate at the lateral inlet (Nq_a) and K is the units coefficient in the Hazen-Williams equation

- The calculated value of D would normally be <u>rounded up</u> to the next available internal pipe diameter

VII. Lateral Design Example

VI.1. Given information:

L =	396 m (lateral length)
q_a =	0.315 lps (nominal sprinkler discharge)
S_e =	12 m (sprinkler spacing)
h_r =	1.0 m (riser height)
slope =	-2.53% (going downhill)
P_a =	320 kPa (design nozzle pressure)
pipe material =	aluminum

VI.2. Calculations leading to allowable pressure head loss in the lateral:

N_n = 396/12 = 33 sprinklers
F = 0.36
Q_l = (0.315)(33) = 10.4 lps
Δh_e = SL = (-0.0253)(396) = -10.0 m

$(P_f)_a$ = 0.20Pa - Δh_e = 0.20(320 kPa) – 9.81(-10.0 m) = 162 kPa
$(h_f)_a$ = 162/9.81 = 16.5 m

VI.3. Calculations leading to required lateral pipe inside diameter:

$0.3P_a$ = 0.3(320 kPa) = 96.0 kPa
$0.3h_a$ = 96.0/9.81 = 9.79 m

Now, $0.3h_a < -\Delta h_e$ (steep downhill). Therefore, may want to use $h_f = -\Delta h_e$. Then, J_a is:

$$J_a = 100\left(\frac{-\Delta h_e}{FL}\right) = 100\left(\frac{-(-10.0\,\mathrm{m})}{(0.36)(396)}\right) = 7.01\,\mathrm{m}/100\,\mathrm{m} \tag{112}$$

However, if $0.3h_a > -\Delta h_e$, J_a would be calculated as:

$$J_a = 100\left(\frac{0.20h_a - \Delta h_e)}{FL}\right) = 100\left(\frac{16.5}{(0.36)(396)}\right) = 11.6\,\mathrm{m}/100\,\mathrm{m} \tag{113}$$

For now, let's use J_a = 7.01 m/100 m. Then, the minimum pipe inside diameter is (C ≈ 130 for aluminum):

$$D = \left[\frac{1.21E12}{7.01}\left(\frac{10.4}{130}\right)^{1.852}\right]^{0.205} = 77.7\,\mathrm{mm} \tag{114}$$

which is equal to 3.06 inches.

In the USA, 3" aluminum sprinkler pipe has an ID of 2.9" (73.7 mm), so for this design it would be necessary to round up to a 4" nominal pipe size (ID = 3.9", or 99.1 mm).

However, it would be a good idea to also try the 3" size and see how the lateral hydraulics turn out (this is done below; note also that for J_a = 11.6, D = 70.0 mm).

VI.4. Check the design with the choices made thus far

The real friction loss will be:

$$J = 1.21E12\left(\frac{10.4}{130}\right)^{1.852}\left(99.1\,\text{mm}\right)^{-4.87} = 2.14\,\text{m}/100\,\text{m} \tag{115}$$

$$h_f = \frac{JFL}{100} = \frac{(2.14)(0.36)(396)}{100} = 3.06\text{ m} \tag{116}$$

The required lateral inlet pressure head is:

$$\begin{aligned} h_l &= h_a + 0.75h_f + 0.5\Delta h_e + h_r \\ h_l &= 320/9.81 + 0.75(3.06) + 0.5(-10.0) + 1.0 = 30.9\text{ m} \end{aligned} \tag{117}$$

Thus, P_l is (30.9)(9.81) = 303 kPa, which is less than the specified P_a of 320 kPa, and this is because the lateral is running downhill

VI.5. Calculate the pressure and head at the end of the lateral pipe

$$h_{end} = h_l - h_f - \Delta h_e = 30.9 - 3.06 - (-10.0) = 37.8\text{ m} \tag{118}$$

which is equal to 371 kPa. Thus, the pressure at the end of the lateral pipe is greater than the pressure at the inlet.

To determine the pressure at the last sprinkler head, subtract the riser height to get 37.8 m – 1.0 m = 36.8 m (361 kPa)

VI.6. Calculate the location of minimum pressure in the lateral pipe

$$\begin{aligned} x &= \frac{S_e}{q_a}\left[Q_l - 3(10)^{-7}\left(C(-S)^{0.54}D^{2.63}\right)\right] \\ x &= \frac{12}{0.315}\left[10.4 - 3(10)^{-7}\left(130(2.53)^{0.54}(99.1)^{2.63}\right)\right] = -39.6\text{ m} \end{aligned} \tag{119}$$

The result is negative, indicating that that minimum pressure is really at the entrance (inlet) to the lateral pipe. The minimum sprinkler head pressure is equal to $h_l - h_r$ = 30.9 – 1.0 = 29.9 m, or 293 kPa.

VI.7. Calculate the percent pressure variation along the lateral pipe

The maximum pressure is at the last sprinkler (end of the lateral), and the minimum pressure is at the first sprinkler (lateral inlet). The percent pressure variation is:

$$\Delta P = \frac{P_{max} - P_{min}}{P_a} = \frac{361-293}{320} = 0.21\% \qquad (120)$$

That is, 21% pressure variation at the sprinklers, along the lateral

This is larger than the design value of 0.20, or 20% variation. But it is very close to that design value, which is somewhat arbitrary anyway.

VI.8. Redo the calculations using a 3" lateral pipe instead of the 4" size

In this case, the location of the minimum pressure in the lateral pipe is:

$$x = \frac{12}{0.315}\left[10.4 - 3(10)^{-7}\left(130(2.53)^{0.54}(73.7)^{2.63}\right)\right] = 196\text{ m} \qquad (121)$$

which is the distance from the upstream end of the lateral.

There are about 196/12 = 16 sprinklers from the lateral inlet to the location of minimum pressure, and about 17 sprinklers from x to the end of the lateral.

Friction loss from x to the end of the lateral is:

$$J_{x-end} = 1.21E12\left(\frac{(17)(0.315)}{130}\right)^{1.852}(73.7)^{-4.87} = 2.65\text{ m}/100\text{ m} \qquad (122)$$

$$\left(h_f\right)_{x-end} = \frac{(2.65)(0.38)(396-196)}{100} = 2.01\text{ m} \qquad (123)$$

Friction loss from the inlet to the end is:

$$J_{inlet-end} = 1.21E12\left(\frac{10.4}{130}\right)^{1.852}(73.7)^{-4.87} = 9.05\text{ m}/100\text{ m} \qquad (124)$$

$$\left(h_f\right)_{inlet-end} = \frac{(9.05)(0.36)(396)}{100} = 12.9 \text{ m} \tag{125}$$

Then, friction loss from inlet to x is:

$$\left(h_f\right)_{inlet-x} = 12.9 - 2.01 = 10.9 \text{ m} \tag{126}$$

The required lateral pipe inlet head is:

$$\begin{aligned} h_l &= h_a + 0.75h_f + 0.5\Delta h_e + h_r \\ h_l &= 320/9.81 + 0.75(12.9) + 0.5(-10.0) + 1.0 = 38.3 \text{ m} \end{aligned} \tag{127}$$

giving a P_l of (38.3)(9.81) = 376 kPa, which is higher than P_l for the 4" pipe

The minimum pressure head (at distance x = 196 m) is:

$$\begin{aligned} h_x &= h_l - \left(h_f\right)_{inlet-x} - \left(\Delta h_e\right)_{inlet-x} \\ h_x &= 38.3 - 10.9 - (-0.0253)(196) = 32.4 \text{ m} \end{aligned} \tag{128}$$

giving a P_x of (32.4)(9.81) = 318 kPa, which is very near P_a.

The pressure head at the end of the lateral pipe is:

$$h_{end} = h_l - h_f - \Delta h_e = 38.3 - 12.9 + 10.0 = 35.4 \text{ m} \tag{129}$$

giving P_{end} of (35.4)(9.81) = 347 kPa, which is less than P_l. So, the maximum lateral pipe pressure is at the inlet.

The percent variation in pressure at the sprinklers is based on P_{max} = 376 -(1.0)(9.81) = 366 kPa, and P_{min} = 318 - (1.0)(9.81) = 308 kPa:

$$\%\Delta P = \frac{P_{max} - P_{min}}{P_a} = \frac{366 - 308}{320} = 0.18 \tag{130}$$

which turns out to be slightly less than the design value of 20%

VI.9. What if the lateral ran uphill at 2.53% slope?

In this case, the maximum allowable head loss gradient is:

$$
\begin{aligned}
J_a &= 100\left(\frac{0.20h_a - \Delta h_e)}{FL}\right) \\
&= 100\left(\frac{0.2(320/9.81) - 10.0}{(0.36)(396)}\right) = -2.44\,m/100\,m
\end{aligned}
\tag{131}
$$

which is negative because $\Delta h_e > 0.2h_a$, meaning that it is not possible to have only a 20% variation in pressure along the lateral, that is, unless flow control nozzles and or other design changes are made.

VI.10. Some observations about this design example

Either the 3” or 4” aluminum pipe size could be used for this lateral design. The 4” pipe will cost more than the 3” pipe, but the required lateral inlet pressure is less with the 4” pipe, giving lower pumping costs, assuming pumping is necessary.

Note that it was assumed that each sprinkler discharged 0.315 lps, when in reality the discharge depends on the pressure at each sprinkler. To take into account the variations in sprinkler discharge would require an iterative approach to the mathematical solution (use a computer).

Most sprinkler laterals are laid on slopes less than 2.5%, in fact, most are on fields with less than 1% slope.

Lecture 8

Set Sprinkler Lateral Design

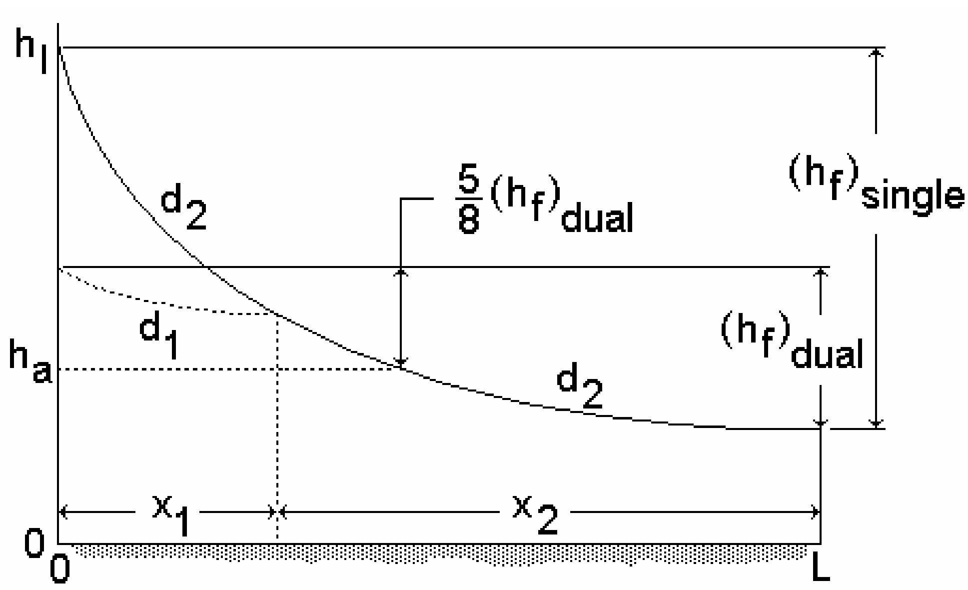
I. Dual Pipe Size Laterals

- Sometimes it is useful to design a lateral pipe with two different diameters to accomplish either of the following:

  1. a reduction in h_f
  2. an increase in h_f

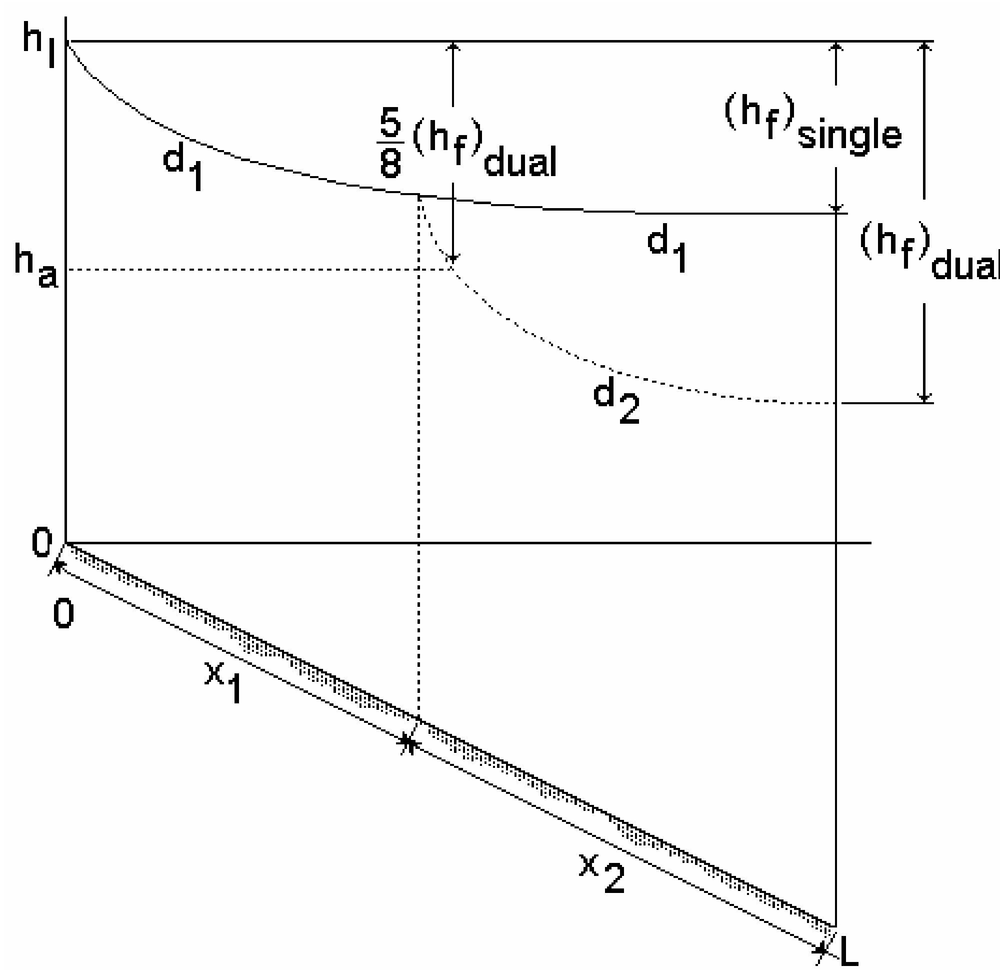
- In either case, the basic objective is to reduce pressure variations along the lateral pipe by arranging the friction loss curve so that it *more closely parallels the ground slope*
- It is not normally desirable to have more than one pipe size in portable laterals (hand-move, wheel lines), because it usually makes set changes more troublesome
- For fixed systems with buried laterals, it may be all right to have more than two pipe diameters along the laterals
- For dual pipe size laterals, approximately 5/8 of the pressure loss due to friction occurs between the lateral inlet and the location of average pressure

- **Case 1**: a lateral on level ground where one pipe size is too small, but the next larger size is too big...



- d_1 is the larger diameter, and d_2 is the smaller diameter
- note that $(h_f)_{single}$ is much larger than $(h_f)_{dual}$

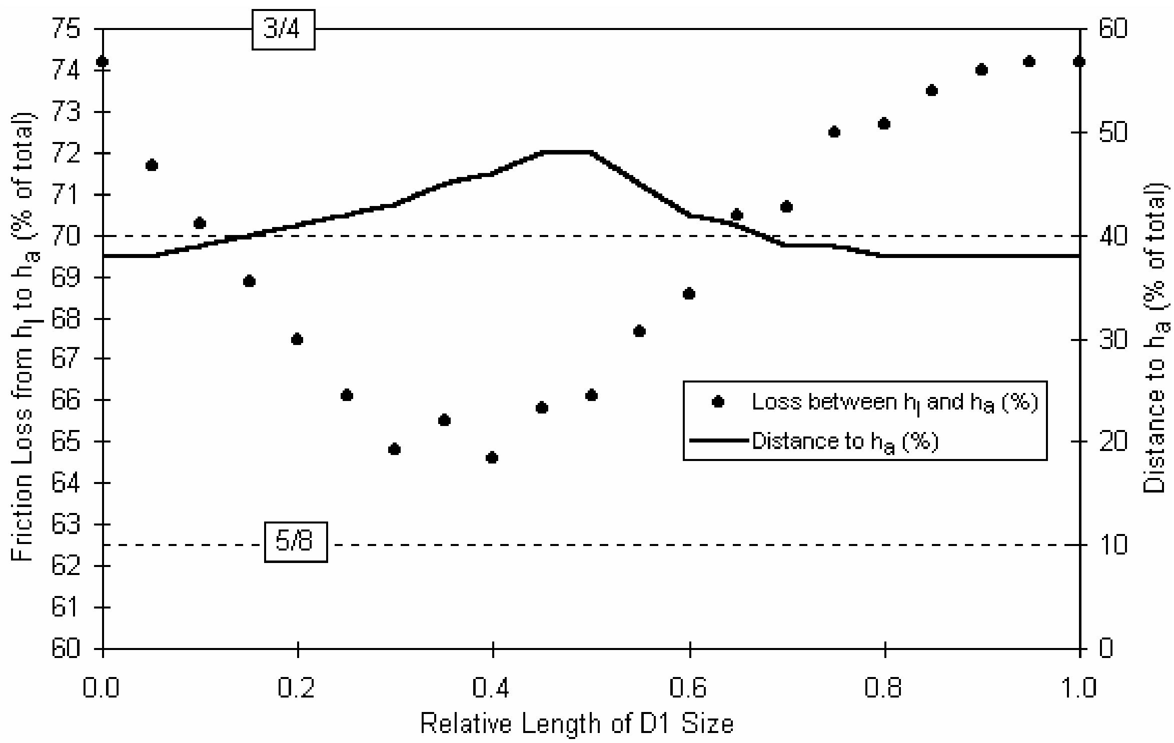
- **Case 2**: a lateral running downhill where one pipe size is too big, but the next smaller size is too small...



- The composite friction loss curve for d_1 and d_2 more closely parallels the ground slope than the curve with only d_1, which means that the pressure variation along the lateral is less with the dual pipe size design

II. Location of Average Pressure in Dual Size Laterals

- Do you believe that $5/8(h_f)_{dual}$ occurs between h_l and h_a?
- Consider the analysis shown graphically below (5/8 = 0.625)
- The plot is for a dual pipe size lateral with D_1 = 15 cm, D_2 = 12 cm, 100 equally-spaced outlets, 900 m total lateral length, Hazen-Williams C factor of 130, uniform sprinkler flow rate of 0.4 lps, and zero ground slope



- Notice where the ¾ value is on the left-hand ordinate
- Notice that the head loss from h_l to h_a is approximately 74% when the pipe size is all D_1 (1.0 on the abscissa) and when the pipe size is all D_2 (0.0 on the abscissa)
- The h_l values (inlet pressure head) would be different for each point on the curve if it were desired to maintain the same h_a for different lateral designs
- Notice that the distance from the lateral inlet to the location of average pressure head is roughly 40% of the total lateral length, but varies somewhat depending on the ratio of lengths of D_1 to D_2 (in this example)

- These calculations can be set up on a spreadsheet to analyze any particular combination of pipe sizes and other hydraulic conditions. Below is an example:

Section	Flow (lps)	Distance (m)	Diameter (cm)	h_f (m)	Sum (h_f) (m)	d(h_e) (m)	head (m)	diff from ha (%)	$h_f/(h_f)_{total}$
1	40.00	9.00	15.00	0.31	0.31	0	49.69	12.08	0.016
2	39.60	18.00	15.00	0.31	0.62	0	49.38	11.77	0.032
3	39.20	27.00	15.00	0.30	0.92	0	49.08	11.47	0.048
4	38.80	36.00	15.00	0.29	1.21	0	48.79	11.18	0.064
5	38.40	45.00	15.00	0.29	1.50	0	48.50	10.89	0.079
6	38.00	54.00	15.00	0.28	1.79	0	48.21	10.61	0.094

III. Determining X_1 and X_2 in Dual Pipe Size Laterals

- The friction loss is:

$$h_f = \left(\frac{J_1F_1L}{100} - \frac{J_2F_2x_2}{100}\right) + \left(\frac{J_3F_2x_2}{100}\right) \tag{132}$$

where,

h_f = total lateral friction head loss for dual pipe sizes

J_1 = friction loss gradient for D_1 and Q_{inlet}
J_2 = friction loss gradient for D_1 and Q_{inlet} - $(q_a)(x_1)/S_e$
J_3 = friction loss gradient for D_2 and Q_{inlet} - $(q_a)(x_1)/S_e$

F_1 = multiple outlet reduction coefficient for L/S_e outlets
F_2 = multiple outlet reduction coefficient for x_2/S_e outlets

x_1 = length of D_1 pipe (larger size)
x_2 = length of D_2 pipe (smaller size)

$x_1 + x_2 = L$

- As in previous examples, we assume constant q_a
- As for single pipe size laterals, we will fix h_f by

$$\Delta h = h_f + \Delta h_e = 20\% h_a \tag{133}$$

and,

$$h_f = 20\% h_a - \Delta h_e \tag{134}$$

- Find d_1 and d_2 in tables (or by calculation) using Q_{inlet} and...

$$(J)_{d1} \le J_a \le (J)_{d2} \tag{135}$$

for,

$$J_a = 100\left(\frac{20\% h_a - \Delta h_e}{FL}\right) \tag{136}$$

- Now there are two adjacent pipe sizes: d_1 and d_2
- Solve for x_1 and x_2 by trial-and-error, or write a computer program, and make $h_f = 0.20h_a - \Delta h_e$ (you already have an equation for h_f above)

IV. Setting up a Computer Program to Determine X_1 and X_2

- If the Hazen-Williams equation is used, the two F values will be:

$$F_1 \approx 0.351 + \frac{1}{2N_1}\left(1 + \frac{4}{13N_1}\right) \tag{137}$$

$$F_2 \approx 0.351 + \frac{1}{2N_2}\left(1 + \frac{4}{13N_2}\right) \tag{138}$$

where

$$N_1 = \frac{L}{S_e} \tag{139}$$

$$N_2 = \frac{L - x_1}{S_e} \tag{140}$$

- The three friction loss gradients are:

$$J_1 = K\left(\frac{Q_1}{C}\right)^{1.852} D_1^{-4.87} \tag{141}$$

$$J_2 = K\left(\frac{Q_2}{C}\right)^{1.852} D_1^{-4.87} \tag{142}$$

$$J_3 = K\left(\frac{Q_2}{C}\right)^{1.852} D_2^{-4.87} \tag{143}$$

where

$$Q_1 = \left(\frac{L}{S_e}\right) q_a \tag{144}$$

$$Q_2 = \left(\frac{L - x_1}{S_e}\right) q_a \tag{145}$$

- The coefficient K in Eqs. 141-143 is 1,050 for gpm & inches; $16.42(10)^6$ for lps and cm; or $1.217(10)^{12}$ for lps and mm

- Combine the above equations and set it equal to zero:

$$f\left(x_1\right) = \alpha_1\left[\alpha_2 - \alpha_3\left(L - x_1\right)^{2.852}F_2\right] - 0.2h_a + \Delta h_e = 0 \qquad (146)$$

where

$$\alpha_1 = \frac{K}{100\,C^{1.852}} \qquad (147)$$

$$\alpha_2 = \left(\frac{q_a L}{S_e}\right)^{1.852} D_1^{-4.87} F_1 L \qquad (148)$$

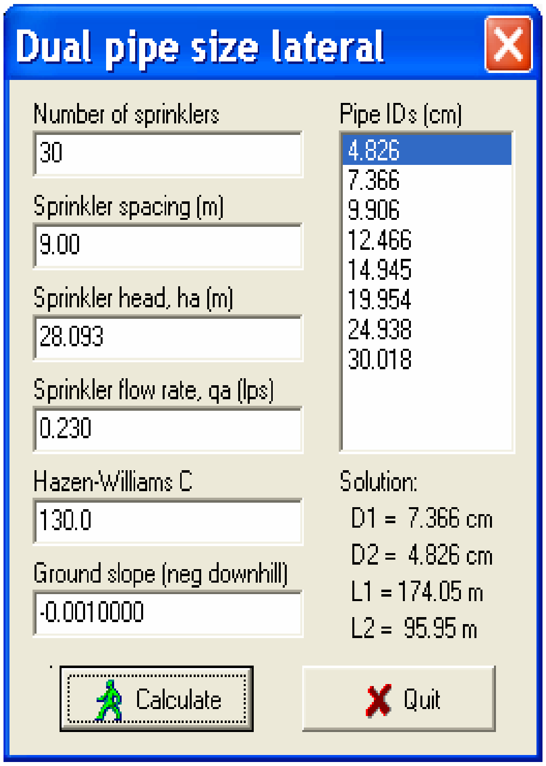
$$\alpha_3 = \left(D_1^{-4.87} - D_2^{-4.87}\right)\left(\frac{q_a}{S_e}\right)^{1.852} \qquad (149)$$

- The three alpha values are constants
- Eq. 146 can be solved for the unknown, x_1, by the Newton-Raphson method
- To accomplish this, we need the derivative of Eq. 146 with respect to x_1

$$\frac{df(x_1)}{dx_1} =$$

$$= \alpha_1\alpha_3\left[2.852F_2(L-x_1)^{1.852} - \frac{S_e(L-x_1)^{0.852}}{2}\left(1+\frac{8\,S_e}{13(L-x_1)}\right)\right] \qquad (150)$$

- Note that the solution may fail if the sizes D_1 & D_2 are inappropriate
- To make things more interesting, give the computer program a list of inside pipe diameters so that it can find the most appropriate available values of D_1 & D_2
- Note that the Darcy-Weisbach equation could be used instead of Hazen-Williams
- In Eq. 146 you could adjust the value of the 0.2 coefficient on h_a to determine its sensitivity to the pipe diameters and lengths
- The following screenshot is of a small computer program for calculating diameters and lengths of dual pipe size sprinkler laterals



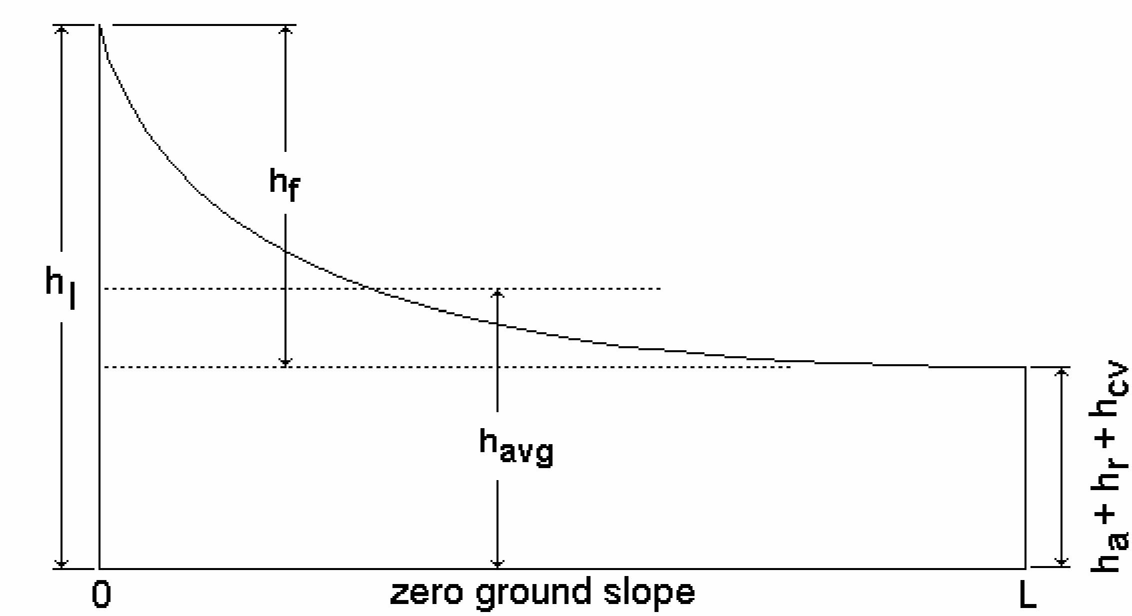
V. Inlet Pressure for Dual Pipe Size Laterals

$$h_l = h_a + \frac{5}{8}h_f + \frac{1}{2}\Delta h_e + h_r \tag{151}$$

- This is the same as the lateral inlet pressure head equation for single pipe size, except that the coefficient on h_f is 5/8 instead of 3/4
- Remember that for a downhill slope, the respective pressure changes due to friction loss and due to elevation change are <u>opposing</u>

VI. Laterals with Flow Control Devices

- Pressure regulating valves can be located at the base of each sprinkler: These have approximately 2 to 5 psi (14 to 34 kPa) head loss
- Also, flow control nozzles (FCNs) can be installed in the sprinkler heads
- FCNs typically have negligible head loss
- For a lateral on level ground, the minimum pressure is at the end:



- The lateral inlet pressure head, h_l, is determined such that the minimum pressure in the lateral is enough to provide h_a at each sprinkler...

$$h_l = h_a + h_f + \Delta h_e + h_r + h_{cv} \tag{152}$$

where h_{cv} is the pressure head loss through the flow control device

- For a lateral with flow control devices, the average pressure is not equal to the nominal sprinkler pressure

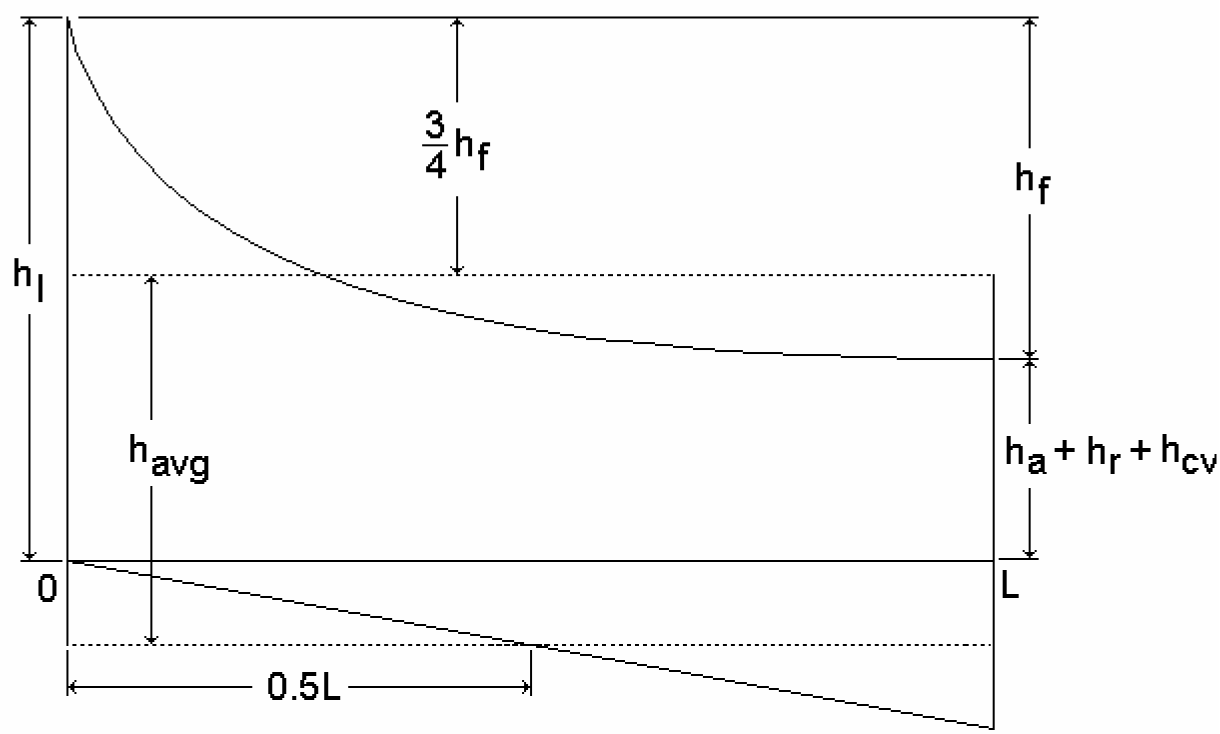
$$h_{avg} \neq h_a \tag{153}$$

- If the pressure in the lateral is enough everywhere, then

$$h_a = \left(\frac{q_a}{K_d}\right)^2 \tag{154}$$

where h_a is the pressure head at the sprinklers

- Below is a sketch of the hydraulics for a downhill lateral with flow control devices



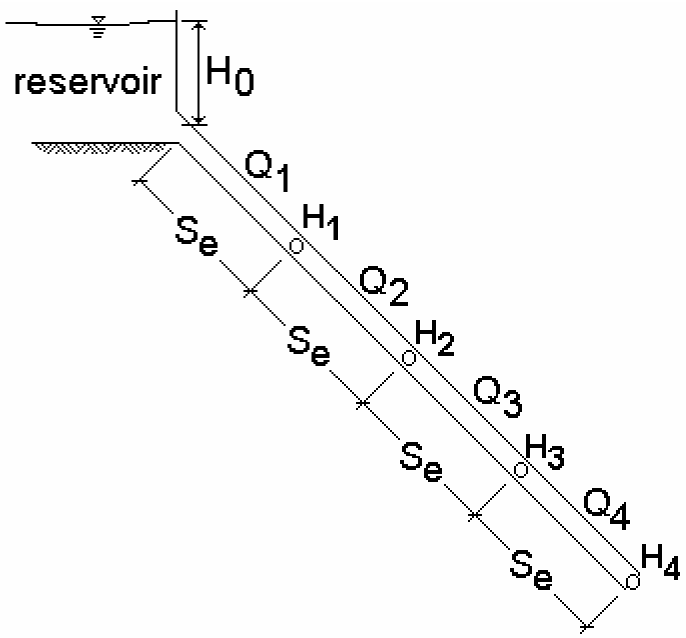
VII. Anti-Drain Valves

- Valves are available for preventing flow through sprinklers until a certain minimum pressure is reached
- These valves are installed at the base of each sprinkler and are useful where sprinkler irrigation is used to germinate seeds on medium or high value crops
- The valves help prevent seed bed damage due to low pressure streams of water during startup and shutdown
- But, for periodic-move, the lines still must be drained before moving

Gravity-Fed Lateral Hydraulic Analysis

I. Description of the Problem

- A gravity-fed sprinkler lateral with evenly spaced outlets (sprinklers), beginning at a distance S_e from the inlet:



- The question is, for known inlet head, H_0, pipe diameter, D, sprinkler spacing, S_e, ground slope, S_o, sprinkler discharge coefficient, K_d, riser height, h_r, and pipe material (C factor), <u>what is the flow rate through each sprinkler?</u>
- Knowing the answer will lead to predictions of application uniformity
- In this case, we won't assume a constant q_a at each sprinkler

II. Friction Loss in the Lateral

Hazen-Williams equation:

$$h_f = \frac{JL}{100} \tag{155}$$

$$J = 16.42(10)^6 \left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \tag{156}$$

for Q in lps; D in cm; J in m/100 m; L in m; and h_f in m.

- Between two sprinklers,

$$h_f = \frac{JS_e}{100} = 16.42(10)^4 S_e \left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \tag{157}$$

or,

$$h_f = h_w Q^{1.852} \tag{158}$$

where Q is the flow rate in the lateral pipe between two sprinklers, and

$$h_w = 16.42(10)^4 S_e C^{-1.852} D^{-4.87} \tag{159}$$

III. Sprinkler Discharge

typically,

$$q = K_d \sqrt{h} \tag{160}$$

where q is the sprinkler flow rate in lps; h is the pressure head at the sprinkler in m; and K_d is an empirical coefficient: $K_d = K_o A$, where A is the cross sectional area of the inside of the pipe

IV. Develop the System of Equations

- Suppose there are only four sprinklers, evenly spaced (see the above figure)
- Suppose that we know H_0, K_d, C, D, h_r, S_o, and S_e

$$q_1 = K_d \sqrt{H_1 - h_r} \quad \rightarrow \quad H_1 = h_r + \left(\frac{q_1}{K_d}\right)^2 = h_r + \frac{(Q_1 - Q_2)^2}{K_d^2} \tag{161}$$

$$q_2 = K_d \sqrt{H_2 - h_r} \quad \rightarrow \quad H_2 = h_r + \left(\frac{q_2}{K_d}\right)^2 = h_r + \frac{(Q_2 - Q_3)^2}{K_d^2} \tag{162}$$

$$q_3 = K_d \sqrt{H_3 - h_r} \quad \rightarrow \quad H_3 = h_r + \left(\frac{q_3}{K_d}\right)^2 = h_r + \frac{(Q_3 - Q_4)^2}{K_d^2} \tag{163}$$

$$q_4 = K_d \sqrt{H_4 - h_r} \quad \rightarrow \quad H_4 = h_r + \left(\frac{q_4}{K_d}\right)^2 = h_r + \frac{{Q_4}^2}{K_d^2} \tag{164}$$

- Pressure heads can also be defined independently in terms of friction loss along the lateral pipe

$$H_1 = H_0 - h_w Q_1^{1.852} - \Delta h_e \tag{165}$$

$$H_2 = H_1 - h_w Q_2^{1.852} - \Delta h_e \tag{166}$$

$$H_3 = H_2 - h_w Q_3^{1.852} - \Delta h_e \tag{167}$$

$$H_4 = H_3 - h_w Q_4^{1.852} - \Delta h_e \tag{168}$$

where,

$$\Delta h_e = \frac{S_e}{\sqrt{S_o^{-2} + 1}} \tag{169}$$

and S_o is the ground slope (m/m)

- The above presumes a uniform, constant ground slope
- Note that in the above equation, Δh_e is always positive. So it is necessary to multiply the result by –1 (change the sign) whenever $S_o < 0$.
- Note also that $S_o < 0$ means the lateral runs in the downhill direction
- Combining respective H equations:

$$\frac{\left(Q_1 - Q_2\right)^2}{K_d^{\,2}} = H_0 - h_w Q_1^{1.852} - \Delta h_e - h_r \tag{170}$$

$$\frac{\left(Q_2 - Q_3\right)^2}{K_d^{\,2}} = H_1 - h_w Q_2^{1.852} - \Delta h_e - h_r \tag{171}$$

$$\frac{\left(Q_3 - Q_4\right)^2}{K_d^{\,2}} = H_2 - h_w Q_3^{1.852} - \Delta h_e - h_r \tag{172}$$

$$\frac{Q_4^{\,2}}{K_d^{\,2}} = H_3 - h_w Q_4^{1.852} - \Delta h_e - h_r \tag{173}$$

- Setting the equations equal to zero:

$$f_1 = H_0 - \frac{\left(Q_1 - Q_2\right)^2}{K_d^2} - h_w Q_1^{1.852} - \Delta h_e - h_r = 0 \tag{174}$$

$$f_2 = \frac{\left(Q_1 - Q_2\right)^2}{K_d^2} - \frac{\left(Q_2 - Q_3\right)^2}{K_d^2} - h_w Q_2^{1.852} - \Delta h_e - h_r = 0 \tag{175}$$

$$f_3 = \frac{\left(Q_2 - Q_3\right)^2}{K_d^2} - \frac{\left(Q_3 - Q_4\right)^2}{K_d^2} - h_w Q_3^{1.852} - \Delta h_e - h_r = 0 \tag{176}$$

$$f_4 = \frac{\left(Q_3 - Q_4\right)^2}{K_d^2} - \frac{{Q_4}^2}{K_d^2} - h_w Q_4^{1.852} - \Delta h_e - h_r = 0 \tag{177}$$

The system of equations can be put into matrix form as follows:

$$\begin{bmatrix} \frac{\partial f_1}{\partial Q_1} & \frac{\partial f_1}{\partial Q_2} & & \\ \frac{\partial f_2}{\partial Q_1} & \frac{\partial f_2}{\partial Q_2} & \frac{\partial f_2}{\partial Q_3} & \\ & \frac{\partial f_3}{\partial Q_2} & \frac{\partial f_3}{\partial Q_3} & \frac{\partial f_3}{\partial Q_4} \\ & & \frac{\partial f_4}{\partial Q_3} & \frac{\partial f_4}{\partial Q_4} \end{bmatrix} \begin{bmatrix} \delta Q_1 \\ \delta Q_2 \\ \delta Q_3 \\ \delta Q_4 \end{bmatrix} = \begin{bmatrix} f_1 \\ f_2 \\ f_3 \\ f_4 \end{bmatrix} \tag{178}$$

where the two values in the first row of the square matrix are:

$$\frac{\partial f_1}{\partial Q_1} = \frac{-2\left(Q_1 - Q_2\right)}{K_d^2} - 1.852 h_w Q_1^{0.852} \tag{179}$$

$$\frac{\partial f_1}{\partial Q_2} = \frac{2\left(Q_1 - Q_2\right)}{K_d^2} \tag{180}$$

The two values in the last row of the square matrix, for n sprinklers, are:

$$\frac{\partial f_n}{\partial Q_{n-1}} = \frac{2\left(Q_{n-1} - Q_n\right)}{K_d^2} \tag{181}$$

$$\frac{\partial f_n}{\partial Q_n} = \frac{-2Q_{n-1}}{K_d^2} - 1.852 h_w Q_n^{0.852} \tag{182}$$

and the three values in each intermediate row of the matrix are:

$$\frac{\partial f_i}{\partial Q_{i-1}} = \frac{2\left(Q_{i-1} - Q_i\right)}{K_d^2} \tag{183}$$

$$\frac{\partial f_i}{\partial Q_i} = \frac{2\left(Q_{i+1} - Q_{i-1}\right)}{K_d^2} - 1.852 h_w Q_i^{0.852} \tag{184}$$

$$\frac{\partial f_i}{\partial Q_{i+1}} = \frac{2\left(Q_i - Q_{i+1}\right)}{K_d^2} \tag{185}$$

where i is the row number

- This is a system of nonlinear algebraic equations
- The square matrix is a *Jacobian matrix*; all blank values are zero
- Solve for Q_1, Q_2, Q_3, and Q_4 (or up to Q_n, in general) using the Newton-Raphson method, Gauss elimination, and backward substitution (or other solution method for a linear set of equations)
- Knowing the flow rates, you can go back and directly calculate the pressure heads one by one

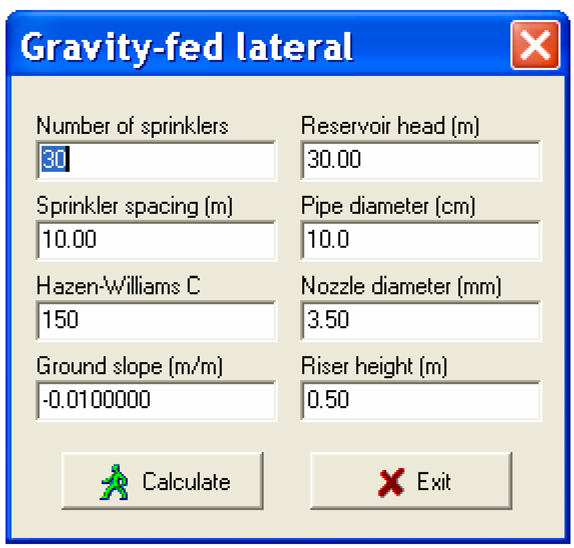
- The problem could be further generalized by allowing for different pipe sizes in the lateral, by including minor losses, by allowing variable elevation changes between sprinkler positions, etc.
- However, it is still a problem of solving for x unknowns and x equations
- For pumped systems (not gravity, as above), we could include a mathematical representation of the pump characteristic curve to determine the lateral hydraulic performance; that is, don't assume a constant H_0, but replace it by some function

V. Brute-Force Approach

- There is a computer program that will do the above calculations for a gravity-fed lateral with multiple sprinklers
- But, if you want to write your own program in a simpler way, you can do the calculations by “brute-force” as follows:

1. Guess the pressure at the end of the lateral
2. Calculate q for the last sprinkler
3. Calculate h_f over the distance S_e to the next sprinkler upstream
4. Calculate Δh_e over the same S_e
5. Get the pressure at that next sprinkler and calculate the sprinkler flow rate
6. Keep moving upstream to the lateral inlet
7. If the head is more than the available head, reduce the end pressure and start over, else increase the pressure and start over

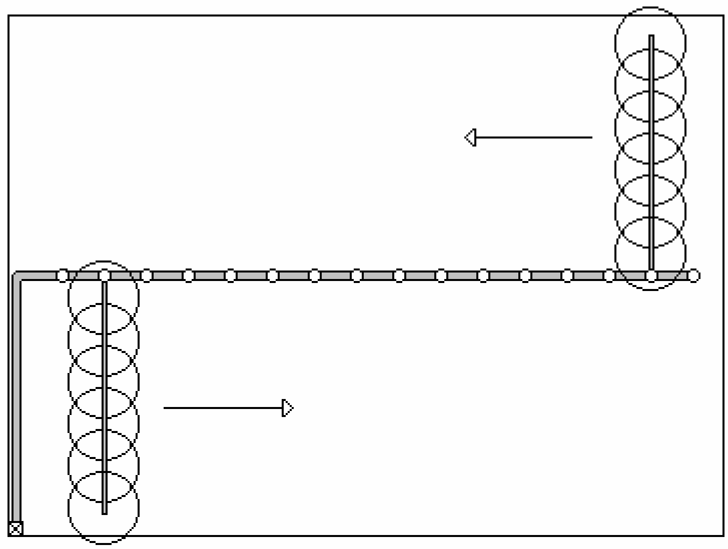
- Below is a screenshot of a computer program that will do the above calculations for a gravity-fed lateral with multiple sprinklers



Lecture 9

Mainline Pipe Design

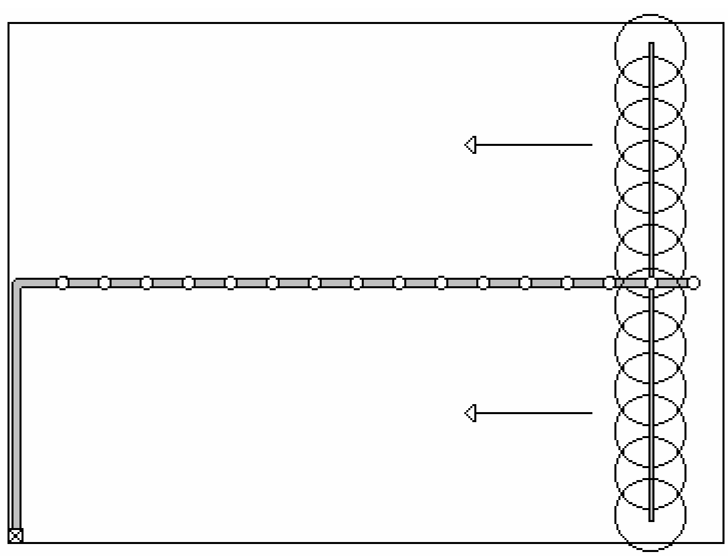
I. Split-Line Laterals



Laterals are usually distributed evenly along a mainline because:

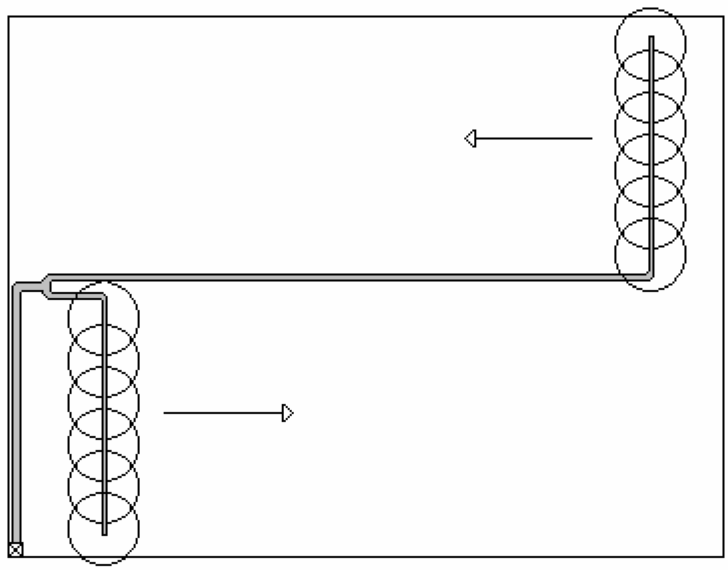
- More equal pump load at different lateral positions
- Reduced mainline cost
- Don't need to "dead-head" back when finished (cross over to other side)
- But, split-line laterals may interfere with cultural operations (wet areas at both ends of field)

Consider twin laterals operating in the same direction:



- In the above case, and for only a single lateral on the mainline, the design of the mainline is relatively simple – it is easy to find the most extreme operating position
- However, the friction loss along the mainline is about four times greater than for the split-line configuration
- Note that for the above two configurations the first sprinkler on the laterals would be at $0.5S_e$ from the inlet, unless the mainline is laid upon a roadway in the field

Twin split-line laterals with dual mainline...



- Same combined mainline length
- No valves on mainline -- elbow at each lateral inlet
- More labor required, but mainline costs less because no valves, and because the mainline is sized for the flow rate of one lateral over the entire mainline length (not half at twice the capacity)
- For different lateral positions, you remove pieces of the mainline from the longer section and put on the shorter section

II. General Design Considerations

- Look at extreme operating conditions for the mainline by varying lateral positions (this can be more complicated for irregular field shapes and non-uniform field slopes)
- Can use economic pipe selection method, but don't make a big sacrifice in terms of pressure uniformity along the mainline to save pumping costs
- Buried mainlines do not obstruct traffic, nor do they remove any land from production. But, they cannot be moved from field to field

- Buried mainlines tend to last longer, because they are not handled and banged up after installation

Uphill Split-Line Mainline Design

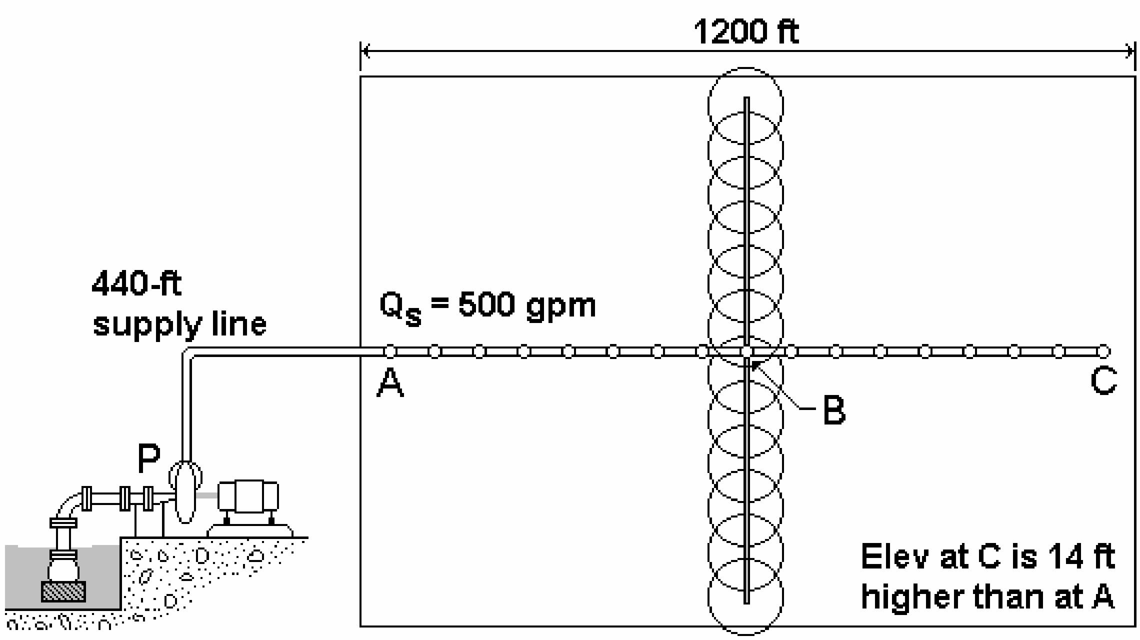
I. Definition of the Example Problem

- See example 10.1 from the textbook, an uphill mainline design for two split-line laterals
- For design, consider the two extreme lateral positions:

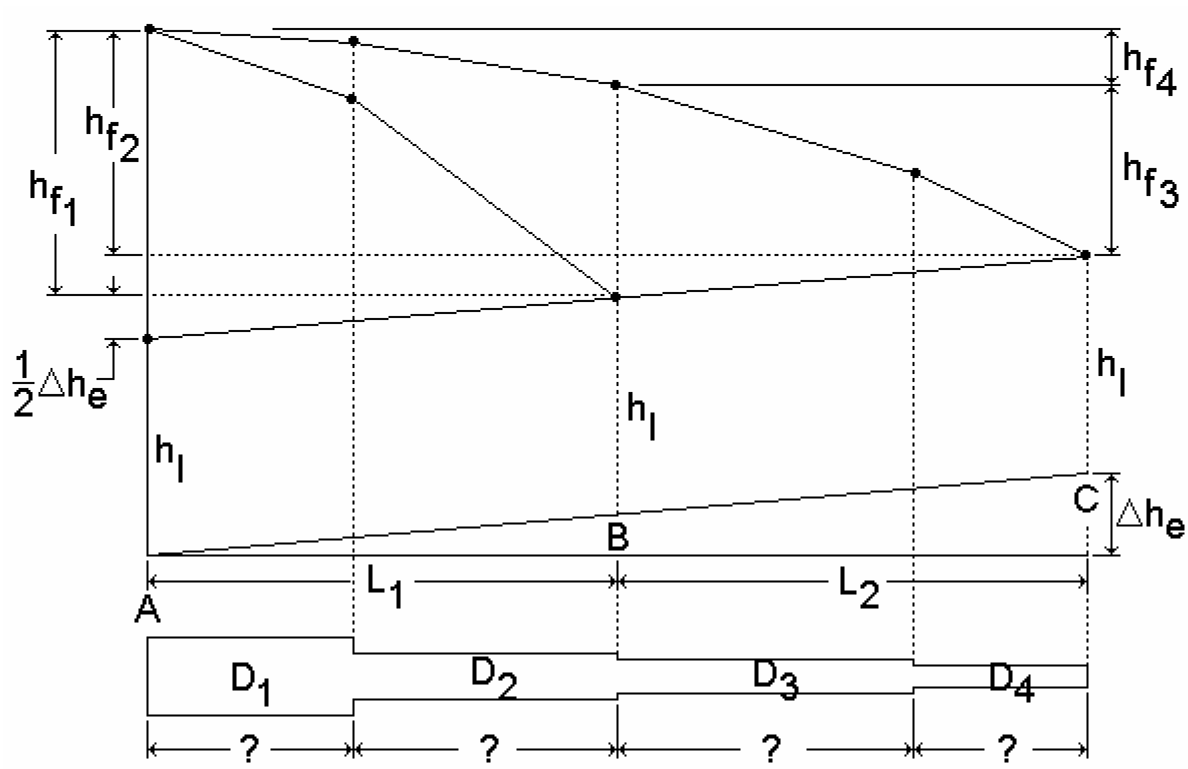
    1. Both laterals at position B (mid-point of mainline)
    2. One lateral at position A and the other at position C

- Divide the mainline into two logical lengths, at the mid-point, according to the two extreme lateral positions
- Determine the total allowable head loss due to friction in each of these logical lengths, then find two adjacent pipe sizes for each length
- Determine the lengths of each pipe size so that the total head loss is just equal to the allowable head loss
- This is somewhat analogous to the procedure for designing dual pipe size laterals

- This is the system layout (shown with both laterals at position B):



- Pump provides at least 172 ft of head at P
- Lateral inlet pressure head is given as 125 ft of head (Eq. 9.2)
- Supply line and mainline are to be aluminum, in 30-ft lengths
- The figure below shows the hydraulic schematic for this mainline, with separate friction loss profiles for the two extreme lateral positions
- The mainline is tentatively divided into sizes D_1 and D_2 for the first half (L_1), and D_3 and D_4 for the second half (L_2). So, there are *potentially* four different pipe sizes in the mainline from A to C.



- We do not yet know what these pipe diameters will be
- We do not yet know the required lengths of the different diameter pipes

- hf_1 is for the case when both laterals are at B
- hf_2, hf_3, and hf_4 are for the case when one lateral is at A and the other at C

- hf_1 is for 500 gpm over L_1
- hf_2 is for 250 gpm over $L_1 + L_2$
- hf_3 is for 250 gpm over L_2
- hf_4 is for 250 gpm over L_1

II. Select the Size of the Supply Line

- We need to select the size of the supply line to know what the head loss is from P to A (pressure at P is given as 172 ft of head)
- Assume no elevation change between P and A

- From continuity, Q = A V, then for an allowable velocity of 5 ft/s:

$$D = \sqrt{\frac{4Q}{\pi V}} = \sqrt{\frac{4(1.11\,\text{cfs})}{\pi(5\text{ fps})}} = 0.53\text{ ft} \tag{186}$$

- This is 6.4 inches. In Table 8.4, the 6-inch pipe has an inside diameter of 5.884 inches. With this size, the velocity at 500 gpm would be 5.9 ft/s, which we will accept (could use 8-inch pipe, but 6-inch is probably OK)

- From Table 8.4, the head loss gradient in the 6-inch supply line at 500 gpm is 2.27 ft/100 ft. Then,

$$(h_f)_{P-A} = (2.27\text{ ft}/100\text{ ft})\left(\frac{440\text{ ft}}{100}\right) = 10.0\text{ ft} \tag{187}$$

- This means that the pressure head at A is 172 ft - 10.0 ft = 162 ft

III. Determine D_1 and D_2 for both Laterals at B

- We will tolerate $(h_f)_1$ head loss over section L_1 of the mainline when both laterals are operating at B. This will give the required h_l at B.
- We can see that $(h_f)_1$ is defined as:

$$h_{f1} = 162\text{ ft} - h_l + 0.5\Delta h_e \tag{188}$$

$$h_{f1} = 162\text{ ft} - 125\text{ ft} - 7\text{ ft} = 30\text{ ft} \tag{189}$$

- The allowable loss gradient in section L_1 for both laterals operating at B is

$$(J_a)_{L1} = 100\left(\frac{30\text{ ft}}{600\text{ ft}}\right) = 5\text{ ft per }100\text{ ft} \tag{190}$$

- From Table 8.4, this is between the 5- and 6-inch pipe sizes, which have respective loss gradients of 5.54 ft/100 ft and 2.27 ft/100 ft for the 500 gpm flow rate. Therefore, choose D_1 = 6 inch and D_2 = 5 inch.

- Now we must find out how long D_1 should be so that the friction loss is really equal to 30 ft of head...

$$L_{D1}(2.27)+(600-L_{D1})(5.54)=100(30\text{ ft}) \tag{191}$$

- Solving the above, L_{D1} = 99.1 ft
- Using 30-ft pipe lengths, we adjust the length to...

$$L_{D1}=90\text{ ft of }6\text{" pipe }(3\text{ sections}) \tag{192}$$

$$L_{D2}=510\text{ ft of }5\text{" pipe }(17\,\text{sections}) \tag{193}$$

- With the adjusted lengths, we will get 30.3 ft of head loss over section L_1 for 500 gpm (this is close enough to the allowable 30 ft)

IV. Determine D_3 and D_4 for Laterals at A and C

- We will tolerate $(h_f)_3 + (h_f)_4$ head loss over the whole length of the mainline when one lateral is operating at A and the other at C
- We can calculate $(h_f)_4$ straight away because we already know the pipe sizes and lengths in section L_1...

$$h_{f4}=\frac{(90\text{ ft})(0.63)+(510\text{ ft})(1.53)}{100}=8.37\text{ ft} \tag{194}$$

where the friction loss gradients for 250 gpm are 0.63 ft/100 ft (6” size) and 1.53 ft/100 ft (5” size). These values were taken from Table 8.4.

- Now we need to know the allowable loss for $(h_f)_3$, such that the pressure in the mainline at C will be equal to h_l (we know that the pressure at A is 162 ft - - it is more than enough)...

$$h_{f2}=h_{f1}+0.5\,\Delta h_e=23.0\text{ ft} \tag{195}$$

$$h_{f3}=h_{f2}-h_{f4}=23.0\text{ ft}-8.37\text{ ft}=14.6\text{ ft} \tag{196}$$

- The allowable loss gradient in section L_2 for laterals at A and C is

$$(J_a)_{L2}=100\left(\frac{14.6\text{ ft}}{600\text{ ft}}\right)=2.43\text{ ft per 100 ft} \tag{197}$$

- From Table 8.1, this is between the 4- and 5-inch pipe sizes, which have respective loss gradients of 4.66 ft/100 ft and 1.53 ft/100 ft for the 250 gpm flow rate. Therefore, choose D_3 = 5 inch and D_4 = 4 inch.
- Now we must find out how long D_3 should be so that the friction loss is really equal to 14.6 ft of head...

$$L_{D3}(1.53) + (600 - L_{D3})(4.66) = 100(14.6\text{ ft}) \tag{198}$$

- Solving the above, L_{D3} = 427 ft
- Using 30-ft pipe lengths, we adjust the length to...

$$L_{D3} = 420\text{ ft of } 5\text{" pipe (14 sections)} \tag{199}$$

$$L_{D4} = 180\text{ ft of } 4\text{" pipe (6 sections)} \tag{200}$$

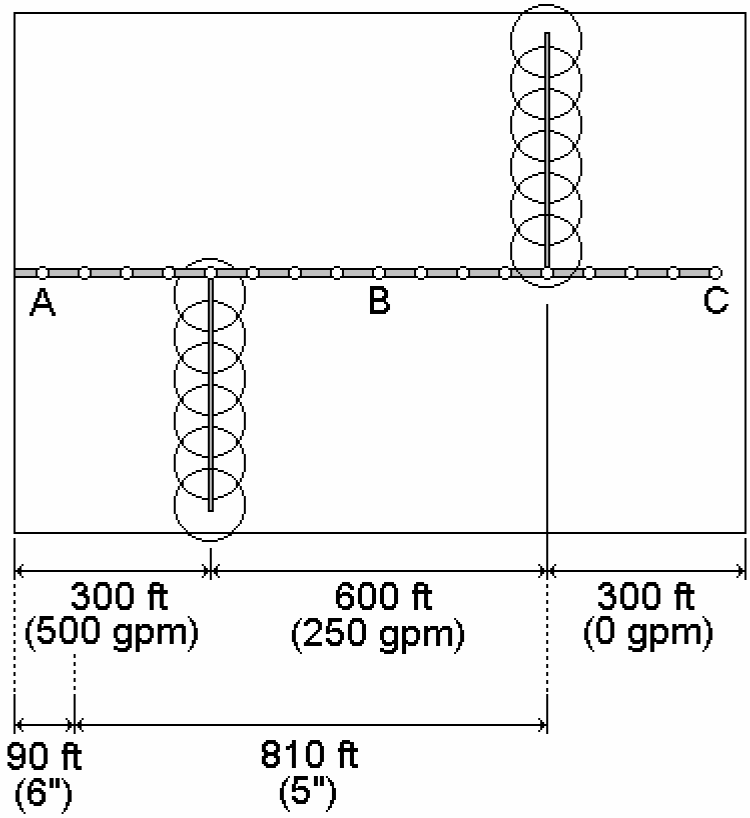
- With the adjusted lengths, we will get 14.8 ft of head loss over section L_2 for 250 gpm (this is close enough to the allowable 14.6 ft)

V. Check this Mainline Design for an Intermediate Position

- Just to be sure, suppose that one lateral is operating halfway between A and B, and the other halfway between B and C
- The allowable friction loss from point A to the furthest lateral is $(h_f)_2 + ¼\Delta h_e$, or 23.0 ft + 3.5 ft = 26.5 ft. The actual friction loss would be:

$$h_f = 0.01\left[(2.27)(90) + (5.54)(210) + (1.53)(600)\right] = 22.9\text{ ft} \tag{201}$$

- OK, the head in the mainline at the furthest lateral is more than enough
- See the figure below for a graphical interpretation of the two laterals in intermediate positions



VI. Comments About the Mainline Design

- Both D_2 and D_3 are the same size in this example
- If we were lucky, both D_1 and D_2 (or D_3 and D_4) could be the same size, but that means the friction loss gradient would have to be just right

- The lateral inlet pressure will be just right when both laterals operate at B
- The lateral inlet pressure will be just right for a lateral operating at C
- The lateral inlet pressure will always be too high for a lateral operating between A and B (the inlet pressure to the mainline, at A, is always 162 ft)

- We designed D_1 and D_2 for the condition when both laterals are at B. This is a more demanding condition for L_1 than when one lateral is at A and the other at C (in this case, only half the system flow rate is in L_1). So, we don't need to "check" D_1 and D_2 again for the case when the laterals are at A and C.

- We didn't consider the hydrant loss from the mainline into the sprinkler lateral, but this could be added to the requirements (say, *effective* h_l)

- This design could be also done using the economic pipe selection method (or another pipe selection method. It would be a good idea to check to see if the

172 ft at the pump (point A) could be reduced by using larger supply and mainline pipes, thus reducing the annual energy costs. However, if the 172 ft were due to gravity supply, the design would still be all right.

- However, the velocity in the 5-inch pipe at 500 gpm is too high, at 8.5 fps (always check velocity limits when sizing pipes!)

VII. Both Laterals Operating at Point C

- How would the mainline design change if it were not split line operation, and both laterals were operating at location C?
- In this case, intuition and past experience tells us location C is the critical lateral position – if you don't agree, then you should test other lateral positions to convince yourself
- We will tolerate $(h_f)_2$ head loss over the entire 1,200-ft length of the mainline when both laterals are operating at C. This will give the required h_l at C.
- We can see that $(h_f)_2$ is defined as:

$$h_{f2} = 162 \text{ ft} - h_l + \Delta h_e \tag{202}$$

$$h_{f2} = 162 \text{ ft} - 125 \text{ ft} - 14 \text{ ft} = 23 \text{ ft} \tag{203}$$

- The allowable loss gradient over the length of the mainline for both laterals operating at C is

$$J_a = 100\left(\frac{23 \text{ ft}}{1{,}200 \text{ ft}}\right) = 1.92 \text{ ft per 100 ft} \tag{204}$$

- From Table 8.4, this is between the 6- and 8-inch pipe sizes, which have respective loss gradients of 2.27 ft/100 ft and 0.56 ft/100 ft for the 500 gpm flow rate
- Determine the respective pipe lengths so that the friction loss is really equal to 23 ft of head...

$$L_{8''}(0.56) + (1{,}200 - L_{8''})(2.27) = 100(23 \text{ ft}) \tag{205}$$

- Solving the above, $L_{8''}$ = 248 ft
- Using 30-ft pipe lengths, we adjust the length to...

$$L_{8''} = 270 \text{ ft of } 8'' \text{ pipe } (9 \text{ sections}) \tag{206}$$

$$L_{6''} = 930 \text{ ft of } 6'' \text{ pipe } (31 \text{ sections}) \tag{207}$$

Lecture 10

Minor Losses & Pressure Requirements

I. Minor Losses

- Minor (or "fitting", or "local") hydraulic losses along pipes can often be estimated as a function of the velocity head of the water within the particular pipe section:

$$h_{ml} = K_r \frac{V^2}{2g} \tag{208}$$

where h_{ml} is the minor loss (m or ft); V is the mean flow velocity, Q/A (m/s or fps); g is the ratio of weight to mass (9.81 m/s^2 or 32.2 ft/s^2); and K_r is a coefficient, dependent on the type of fitting (valve, bend, transition, constriction, etc.)

- Minor losses include head losses through/past hydrants, couplers, valves, pipe elbows, "tees" and other fittings (see Tables 11.1 and 11.2)
- For example, there is some loss when water flows through a hydrant, but also some loss when water flows in a pipe past the location of a closed hydrant
- K_r = 0.3 to 0.6 for flow in a pipeline going past a closed hydrant, whereby the velocity in the pipeline is used to compute h_{ml}
- K_r = 0.4 to 0.8 for flow in a pipeline going past an open hydrant; again, the velocity in the pipeline is used to compute h_{ml}
- K_r = 6.0 to 8.0 for flow from a pipeline through a completely open hydrant. In this case, compute h_{ml} using the velocity of the flow through the lateral fitting on the hydrant, not the flow in the source pipeline.
- For flow through a partially open hydrant, K_r increases beyond the 6.0 to 8.0 magnitude, and the flow rate decreases correspondingly (but not linearly)

- In using Tables 11.1 and 11.2 for hydrants, the nominal diameter (3, 4, 5, and 6 inches) is the diameter of the hydrant and riser pipe, not the diameter of the source pipeline
- Use the diameter of the hydrant for K_r and for computing V_r. However, for line flow past a hydrant, use the velocity in the source pipeline, as indicated above.
- Always use the largest velocity along the path which the water travels – this may be either upstream or downstream of the fitting
- Do not consider velocities along paths through which the water does not flow

- In Table 11.2, for a sudden contraction, K_r should be defined as:

$$K_r = 0.7\left(1 - D_r^2\right)^2 \tag{209}$$

where D_r is the ratio of the small to large inside diameters (D_{small}/D_{large})

- Allen (1991) proposed a regression equation for gradual contractions and expansions using data from the *Handbook of Hydraulics* (Brater & King 1976):

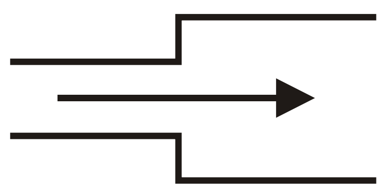
$$K_r = K_f\left(1 - D_r^2\right)^2 \tag{210}$$

where K_f is defined as:

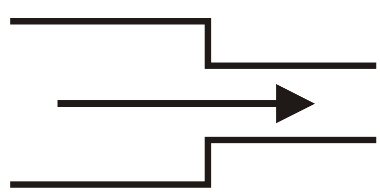
$$K_f = 0.7 - \cos(f)\left[\cos(f)\left(3.2\cos(f) - 3.3\right) + 0.77\right] \tag{211}$$

and f is the angle of the expansion or contraction in the pipe walls (degrees or radians), where $f \geq 0$

- For straight sides (no expansion or contraction), f = 0° (whereby K_f = 0.03)
- For an abrupt change in pipe diameter (no transition), f = 90° (whereby K_f = 0.7)
- The above regression equation for K_f gives approximate values for approximate measured data, some of which has been disputed
- In any case, the true minor head loss depends on more than just the angle of the transition



Expansion



Contraction

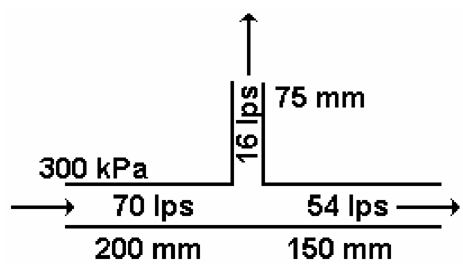
- For a sudden (abrupt) expansion, the head loss can also be approximated as a function of the difference of the mean flow velocities upstream and downstream:

$$h_{ml} = \frac{\left(V_{us} - V_{ds}\right)^2}{2g} \tag{212}$$

- An extreme (albeit unrealistic) case is for $V_{ds} = 0$ and $h_{ml} = V_{us}^2/2g$ (total conversion of velocity head)
- Various other equations (besides those given above) for estimating head loss in pipe expansions and contractions have been proposed and used by researchers and engineers

Minor Loss Example

- A mainline with an open lateral hydrant valve has a diameter of 200 mm ID upstream of the hydrant, and 150 mm downstream of the hydrant
- The diameter of the hydrant opening to the lateral is 75 mm
- $Q_{upstream}$ = 70 lps and $Q_{lateral}$ = 16 lps
- The pressure in the mainline upstream of the hydrant is 300 kPa



- The mean flow velocities are:

$$V_{200} = \frac{0.070 \text{ m}^3/\text{s}}{\left(\dfrac{\pi(0.200 \text{ m})^2}{4}\right)} = 2.23 \text{ m/s} \tag{213}$$

$$V_{150} = \frac{0.070 - 0.016 \text{ m}^3/\text{s}}{\left(\dfrac{\pi(0.150 \text{ m})^2}{4}\right)} = 3.06 \text{ m/s} \tag{214}$$

$$V_{hydrant} = \frac{0.016 \text{ m}^3/\text{s}}{\left(\dfrac{\pi(0.075 \text{ m})^2}{4}\right)} = 3.62 \text{ m/s} \tag{215}$$

- Note that V_{200} and V_{150} are both above the normal design limit of about 2 m/s
- The head loss past the open hydrant is based on the higher of the upstream and downstream velocities, which in this example is 3.06 m/s
- From Table 11.1, the K_r for flow past the open hydrant (line flow; 6" mainline) is 0.5; thus,

$$\left(h_{ml}\right)_{past} = 0.5\frac{(3.06)^2}{2(9.81)} = 0.24\,m \tag{216}$$

- The head loss due to the contraction from 200 mm to 150 mm diameter (at the hydrant) depends on the transition
- If it were an abrupt transition, then:

$$K_r = 0.7\left[1-\left(\frac{150}{200}\right)^2\right]^2 = 0.13 \tag{217}$$

- And, if it were a 45° transition, K_f = 0.67, also giving a K_r of 0.13
- Then, the head loss is:

$$\left(h_{ml}\right)_{contraction} = 0.13\frac{(3.06)^2}{2(9.81)} = 0.06\,m \tag{218}$$

- Thus, the total minor loss in the mainline in the vicinity of the open hydrant is about 0.24 + 0.06 = 0.30 m (0.43 psi).

- The loss through the hydrant is determined by taking K_r = 8.0 (Table 11.1; 3" hydrant):

$$\left(h_{ml}\right)_{through} = 8.0\frac{(3.62)^2}{2(9.81)} = 5.3\,m \tag{219}$$

- This is a high loss through the hydrant (about 7.6 psi), so it may be advisable to use a larger diameter hydrant.
- The pressure in the mainline downstream of the hydrant is (9.81 kPa/m):

$$\begin{aligned} P_{150} &= P_{200} - \gamma\left(h_{ml}\right)_{past} + \gamma\left(\frac{V_{200}^2 - V_{150}^2}{2g}\right) \\ P_{150} &= 300-(9.81)(0.24)+9.81\left(\frac{(2.23)^2-(3.06)^2}{2(9.81)}\right) = 295\,kPa \end{aligned} \tag{220}$$

II. Total Dynamic Head

- The Total Dynamic Head (TDH) is the head that the pump "feels" or "sees" while working, and is calculated to determine the pump requirements
- It includes the elevation that the water must be lifted from the source (not necessarily from the pump elevation itself) to the outlet, the losses due to "friction", the pressure requirement at the outlet, and possibly the velocity head in the pipeline
- For a sprinkler system, the value of TDH depends on the positions of the laterals, so that it can change with each set. Pump selection is usually made for the "critical" or extreme lateral positions, that is, for the "worst case scenario".
- Keller & Bliesner recommend the addition of a "miscellaneous" loss term, equal to 20% of the sum of all "friction" losses. This accounts for:

    1. Uncertainty in the K_r values (minor losses)
    2. Uncertainty in the Hazen-Williams C values
    3. Aging of pipes (increase in losses)
    4. Wear of pump impellers and casings

- Losses in connectors or hoses from the mainline to laterals, if present, must also be taken into account when determining the TDH
- See Example Calculation 11.2 in the textbook
- The next two lectures will provide more information about TDH and pumps

III. The System Curve

- The system curve determines the relationship between TDH and flow rate
- This curve is approximately parabolic, but can take more complex shapes
- Note that head losses in pipe systems are approximately proportional to the square of the flow rate (Q^2 or V^2)
- For the Hazen-Williams equation, these losses are actually proportional to $Q^{1.852}$ or $V^{1.852}$
- For standard, non-FCN, sprinkler nozzles, the head at the sprinkler is also proportional to Q^2
- Sprinkler systems can have a different system curve for each position of the lateral(s)
- Defining the system curve, or the "critical" system curve, is important for pump selection because it determines, in part, the operating point (TDH and Q) for the system

IV. Valving a Pump

- A throttle valve may be required at a pump:

  (a) Filling of the system's pipes

  - The head is low, and the flow rate is high
  - Pump efficiency is low and power requirements may be higher
  - Water hammer damage can result as the system fills
  - Air vents and other appurtenances can be "blown off"
  - For the above reasons, it is advisable to fill the system slowly

  (b) To avoid cavitation, which damages the pump, pipes and appurtenances
  (c) To control the TDH as the sprinklers are moved to different sets

- Throttle valves can be automatic or manual

Pressure Requirements & Pumps

I. Types of Pumps

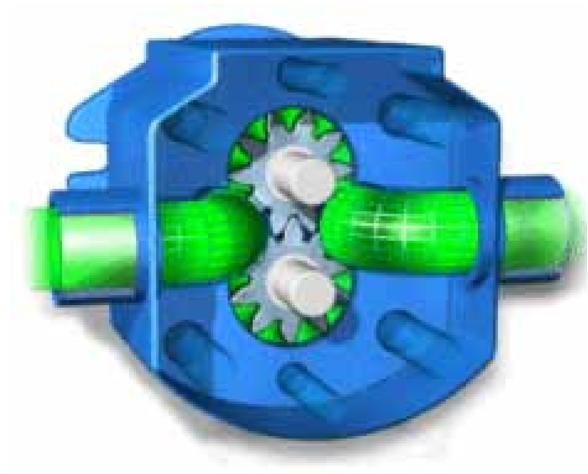
1. Positive Displacement

   - Piston pumps
   - Rotary (gear) pumps
   - Extruding (flexible tube) pumps

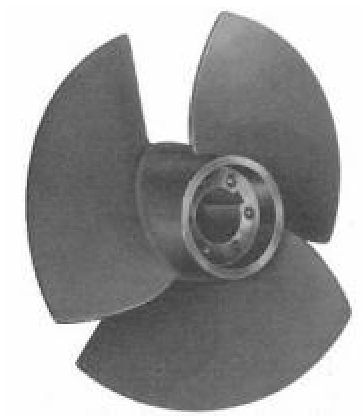
2. Variable Displacement

   - Centrifugal pumps
   - Injector pumps
   - Jet pumps

- The above lists of pump types are not exhaustive
- Positive displacement pumps have a discharge that is nearly independent of the downstream (resistive) pressure. That is, they produce a flow rate that is relatively independent of the total dynamic head, TDH

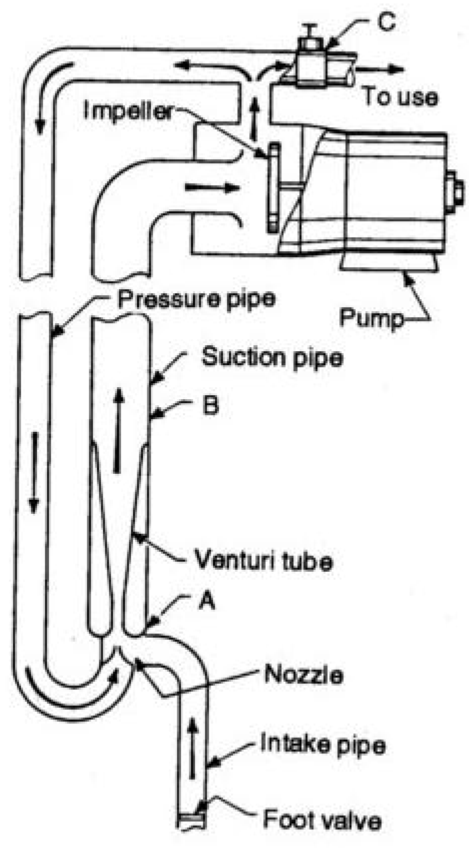


Positive Displacement Pumps



Axial-Flow Impeller

Closed Centrifugal Pump Impeller



Jet Pump

- But, with positive displacement pumps, the required pumping energy is a linear function of the pressure
- Positive displacement pumps can be used with thick, viscous liquids. They are not commonly used in irrigation and drainage, except for the injection of chemicals into pipes and for sprayers
- Piston-type pumps can develop high heads at low flow rates
- Air injection, or jet pumps are typically used in some types of well drilling operations. The air bubbles effectively reduce the liquid density and this assists in bringing the drillings up out of the well. Needs a large capacity air compressor.
- *Homologous* pumps are geometrically similar pumps, but of different sizes

II. Centrifugal Pumps

1. <u>Volute Case</u> This is the most common type of irrigation and drainage pump (excluding deep well pumps). Produce relatively high flow rates at low pressures.

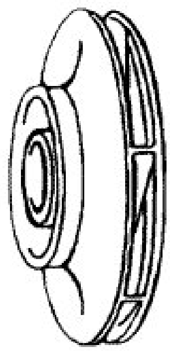
2. <u>Diffuser (Turbine)</u> The most common type for deep wells. Designed to lift water to high heads, typically using multiple identical "stages" in series, stacked up on top of each other.

3. <u>Mixed Flow</u> Uses a combination of centrifugal and axial flow action. For high capacity at low heads.

4. <u>Axial Flow</u> Water flows along the axis of impeller rotation, like a boat propeller. Appropriate for high discharge under very low lift (head). An example is the pumping plant on the west side of the Great Salt Lake.

5. <u>Regenerative</u> The characteristics of these pumps are those of a combination of centrifugal and rotary, or gear, pumps. Shut-off head is well-defined, but efficiency is relatively low. Not used in irrigation and drainage.

- In general, larger pumps have higher maximum efficiencies (they are more expensive, and more effort is given toward making them more efficient)
- Impellers can be open, semi-open, or closed. Open impellers are usually better at passing solids in the pumped liquid, but they are not as strong as closed impellers
- Double suction inlet pumps take water in from both sides and can operate without axial thrust



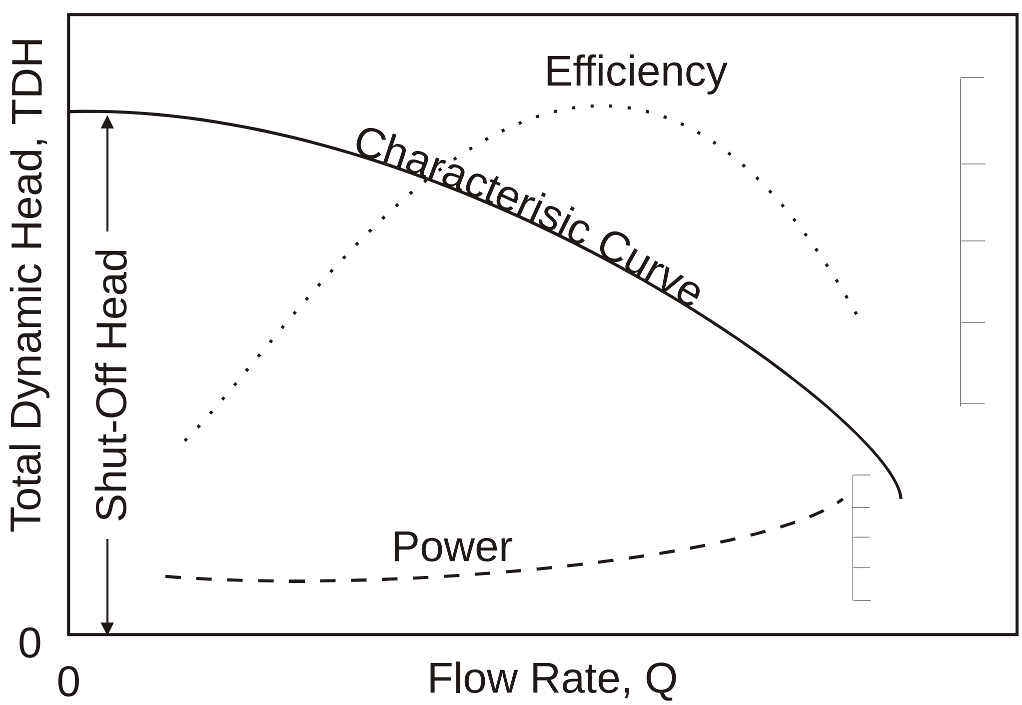
Closed Impeller

Semi-Open Impeller

Open Impeller

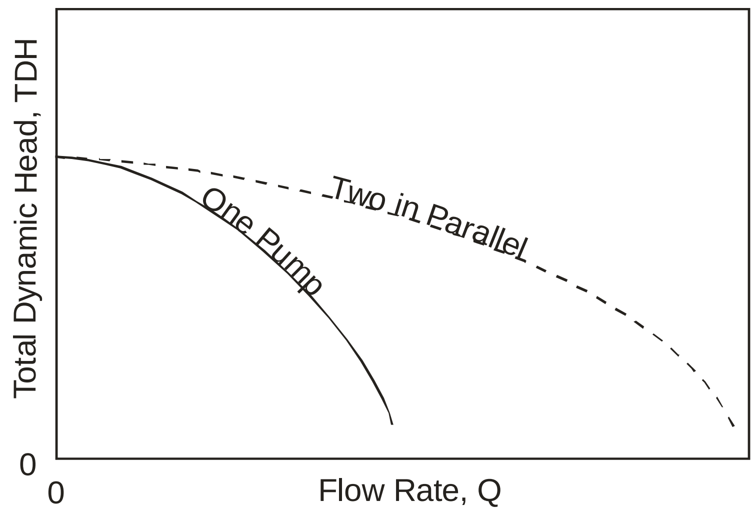
Characteristic Curve

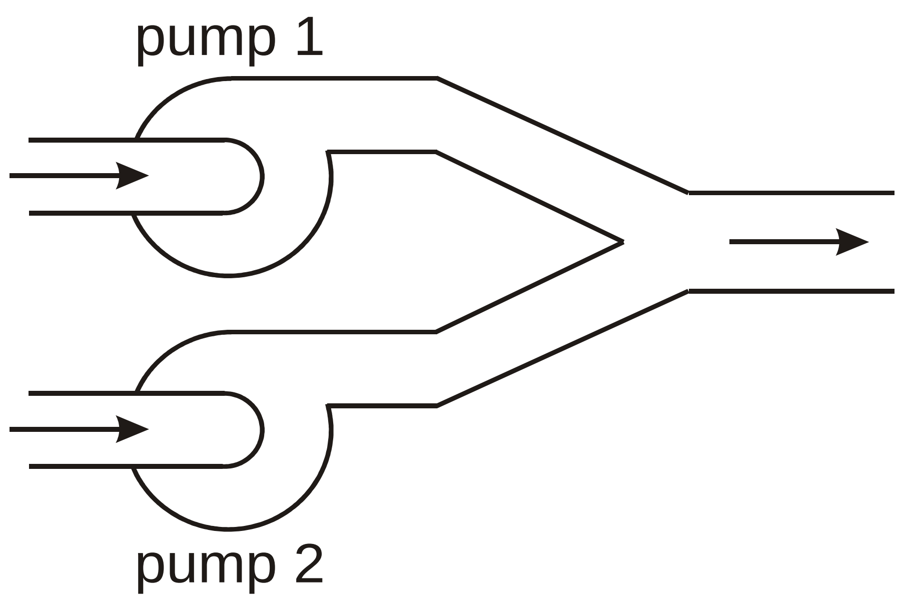
- The pump "characteristic curve" defines the relationship between total dynamic head, TDH, and discharge, Q
- The characteristic curve is unique for a given pump design, impeller diameter, and pump speed
- The characteristic curve has nothing to do with the "system" in which the pump operates
- The "shut-off" head is the TDH value when Q is zero (but the pump is still operating)
- The shut-off head can exceed the recommended operating pressure, or even the bursting pressure, especially with some thin-wall plastic pipes



III. Centrifugal Pumps in Parallel

- Pumps in **PARALLEL** means that the total flow is divided into two or more pumps
- Typical installations are for a single inlet pipe, branched into two pumps, with the outlets from the pumps converging to a single discharge pipe
- If only one of the pumps operates, some type of valve may be required so that flow does not flow backwards through the idle pump
- Flow rate is additive in this case

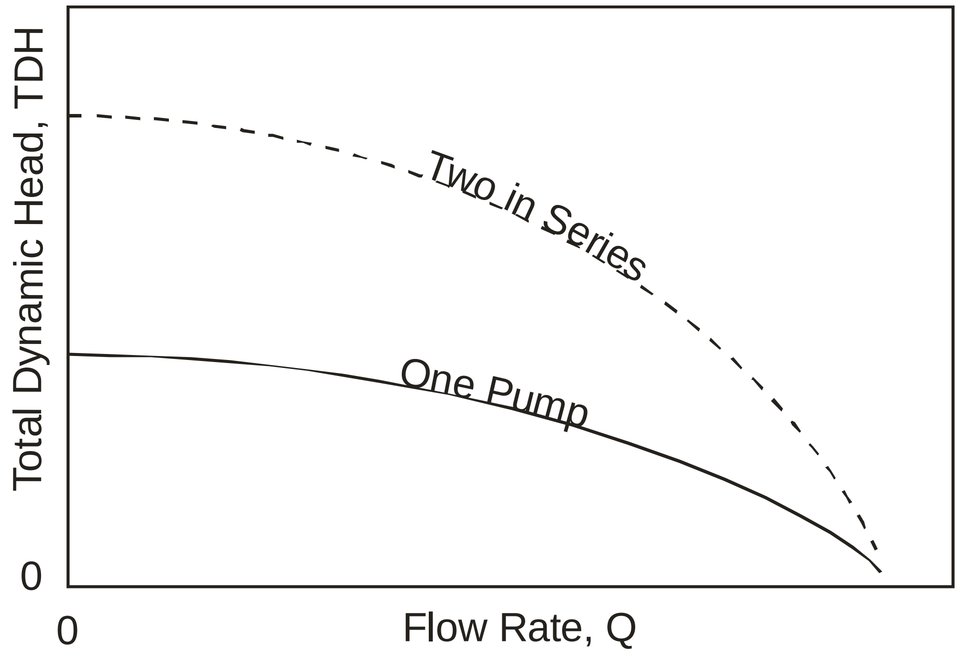




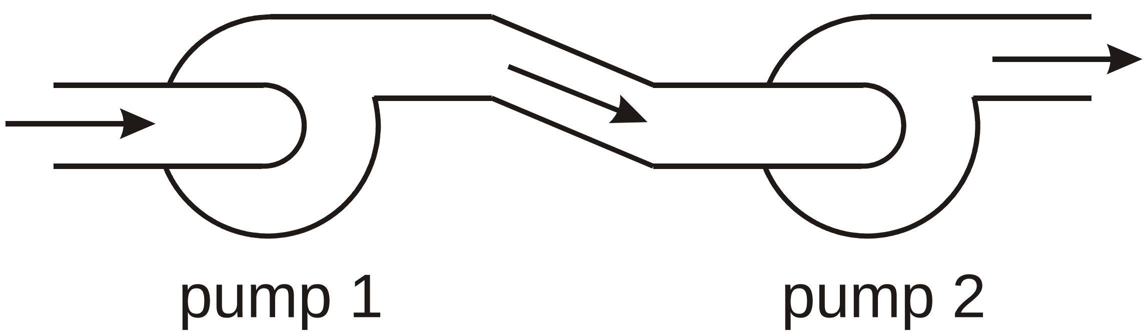
Two Pumps in Parallel

IV. Centrifugal Pumps in Series

- Pumps in **SERIES** means that the total flow passes through each of two or more pumps in line
- Typical installations are for increasing pressure, such as with a booster pump
- Head is additive in this case



- It is common for turbine (well) pumps to operate in series
- For centrifugal pumps, it is necessary to exercise caution when installing in series because the efficiency can be adversely affected
- May need straightening vanes between pumps to reduce swirling
- Note that the downstream pump could cause negative pressure at the outlet of the US pump, which can be a problem



Two Pumps in Series

Lecture 11

Pumps & System Curves

I. Pump Efficiency and Power

- Pump efficiency, E_{pump}

$$E_{pump} = \frac{\text{water horsepower}}{\text{brake horsepower}} = \frac{\text{WHP}}{\text{BHP}} \tag{221}$$

where *brake horsepower* refers to the input power needed at the pump shaft (not necessarily in "horsepower"; could be watts or some other unit)

- Pump efficiency is usually given by the pump manufacturer
- Typically use the above equation to calculate required BHP, knowing E_{pump}
- Water horsepower is defined as:

$$\text{WHP} = \frac{QH}{3956} \tag{222}$$

where WHP is in horsepower; Q in gpm; and H in feet of head. The denominator is derived from:

$$\gamma QH = \frac{\left(62.4\ \text{lbs/ft}^3\right)\left(\text{gal/min}\right)\left(\text{ft}\right)}{\left(33{,}000\ \text{ft-lbs/min-HP}\right)\left(7.481\ \text{gal/ft}^3\right)} \approx \frac{QH}{3956} \tag{223}$$

where $\gamma = \rho g$, and ρ is water density. In metric units:

$$\text{WHP} = \rho g QH = \frac{\left(1000\ \text{kg/m}^3\right)\left(9.81\ \text{m/s}^2\right)\left(\text{l/s}\right)\left(\text{m}\right)}{\left(1000\ \text{l/m}^3\right)\left(1000\ \text{W/kW}\right)} = \frac{QH}{102} \tag{224}$$

where WHP is in kW; Q in lps; and H in meters of head

$$1\ \text{HP} = 0.746\ \text{kW} \tag{225}$$

- *Total Dynamic Head*, TDH, is defined as:

$$\text{TDH} = \Delta\text{Elev} + h_f + \frac{P}{\gamma} + \frac{V^2}{2g} \tag{226}$$

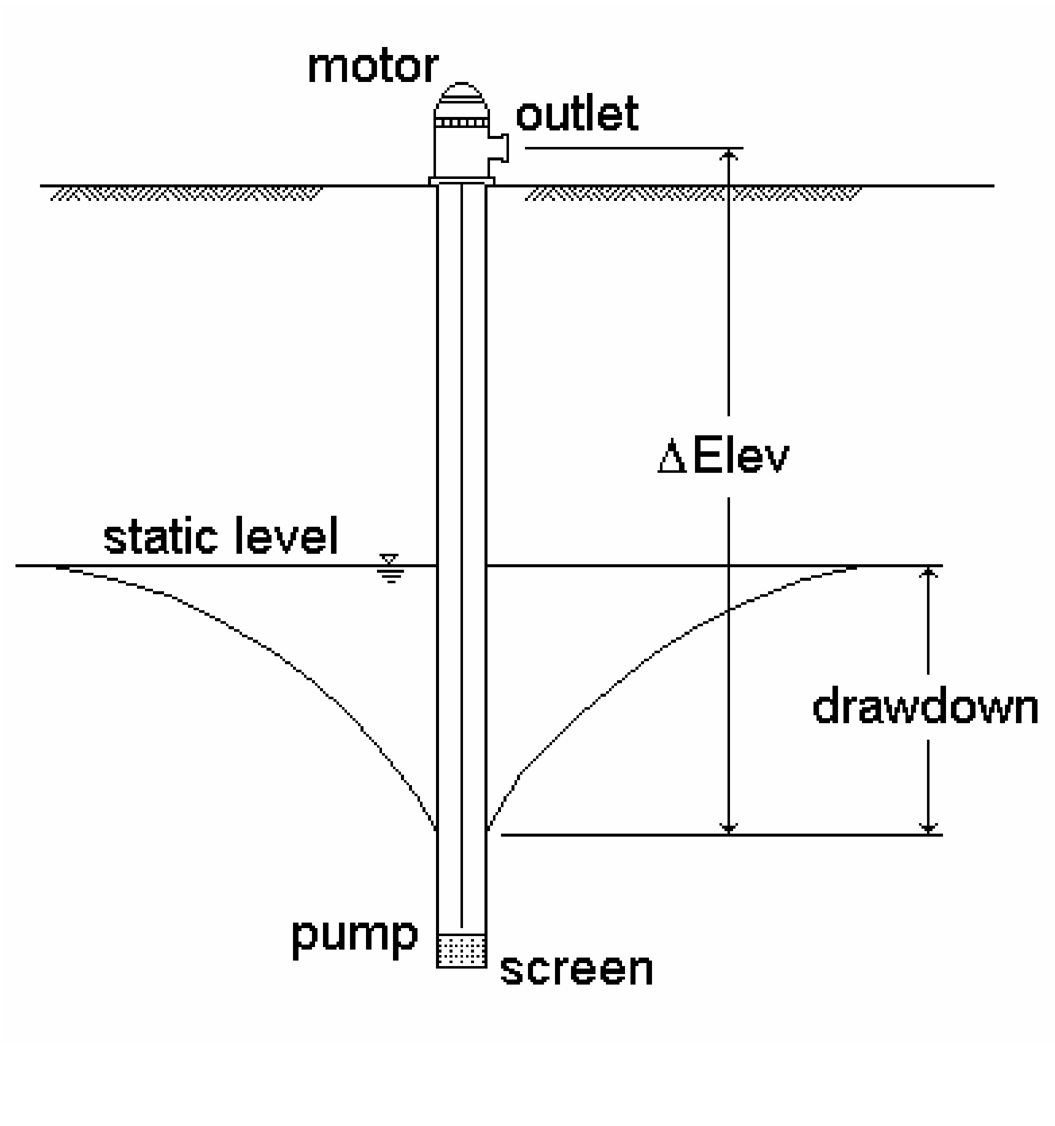
where the pressure, P, and velocity, V, are measured at the pump outlet, and h_f is the total friction loss from the entrance to the exit, including minor losses

- At zero flow, with the pump running,

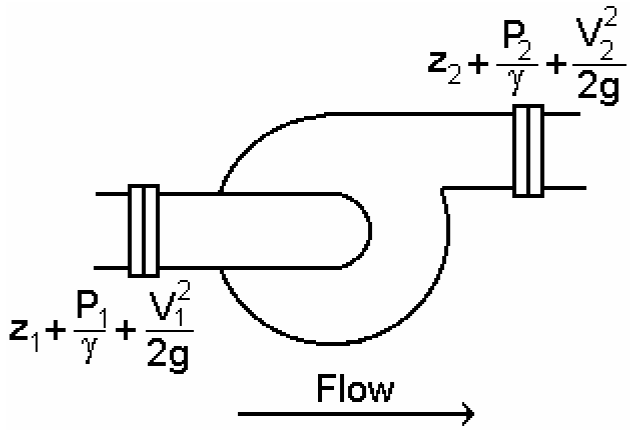
$$TDH = \Delta Elev + \frac{P}{\gamma} \tag{227}$$

but recognizing that in some cases P/γ is zero for a zero flow rate

- The elevation change, ΔElev, is positive for an increase in elevation (i.e. lifting the water)
- Consider a turbine pump in a well:

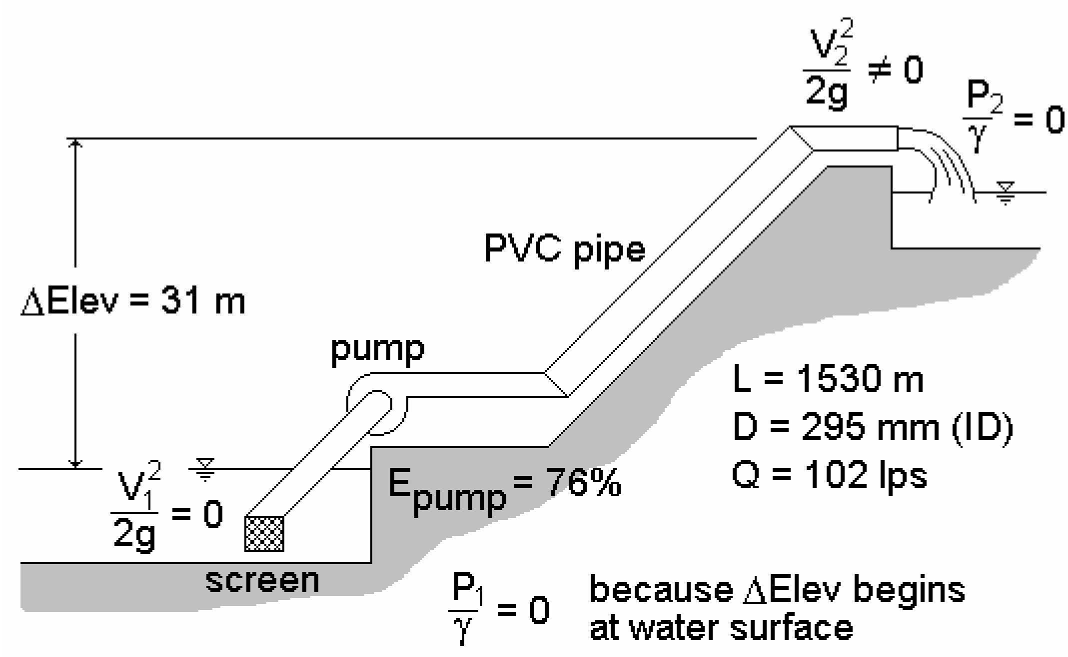


Consider a centrifugal pump:



II. Example TDH & WHP Calculation

- Determine TDH and WHP for a centrifugal pump discharging into the air...



Head loss due to friction:

$$h_f = h_{screen} + 3h_{elbow} + h_{pipe} \tag{228}$$

for PVC, $\varepsilon \approx 1.5(10)^{-6}$ m, relative roughness is:

$$\frac{\varepsilon}{D}=\frac{1.5(10)^{-6}}{0.295}=0.0000051 \tag{229}$$

Average velocity,

$$V=\frac{Q}{A}=\frac{4(0.102)}{\pi(0.295)^2}=1.49\text{ m/s} \tag{230}$$

Reynolds number, for 10°C water:

$$N_R=\frac{VD}{\nu}=\frac{(1.49\text{ m/s})(0.295\text{ m})}{1.306(10)^{-6}\text{ m}^2/\text{s}}=336{,}600 \tag{231}$$

- From the Moody diagram, f = 0.0141
- From the Blasius equation, f = 0.0133
- From the Swamee-Jain equation, f = 0.0141 (same as Moody)

Using the value from Swamee-Jain,

$$h_{pipe}=f\frac{L}{D}\frac{V^2}{2g}=0.0141\left(\frac{1{,}530}{0.295}\right)\frac{(1.49)^2}{2(9.81)}=8.27\text{ m} \tag{232}$$

Water Temperature (°C)	**Kinematic Viscosity (m^2/s)**
0	0.000001785
5	0.000001519
10	0.000001306
15	0.000001139
20	0.000001003
25	0.000000893
30	0.000000800
40	0.000000658
50	0.000000553
60	0.000000474

The values in the above table can be closely approximated by:

$$\nu=\left(83.9192T^2+20{,}707.5T+551{,}173\right)^{-1} \tag{233}$$

where T is in ºC; and ν is in m^2/s

From Table 11.2, for a 295-mm (12-inch) pipe and long radius 45-deg flanged elbow, the K_r value is 0.15

$$h_{elbow} = K_r \frac{V^2}{2g} = (0.15)\frac{(1.49)^2}{2(9.81)} = (0.15)(0.11) = 0.017 \text{ m} \tag{234}$$

For the screen, assume a 0.2 m loss. Then, the total head loss is:

$$h_f = 0.2 + 3(0.017) + 8.27 = 8.5 \text{ m} \tag{235}$$

With the velocity head of 0.11 m, the total dynamic head is:

$$\text{TDH} = 31 + 8.5 + 0.11 \approx 40 \text{ m} \tag{236}$$

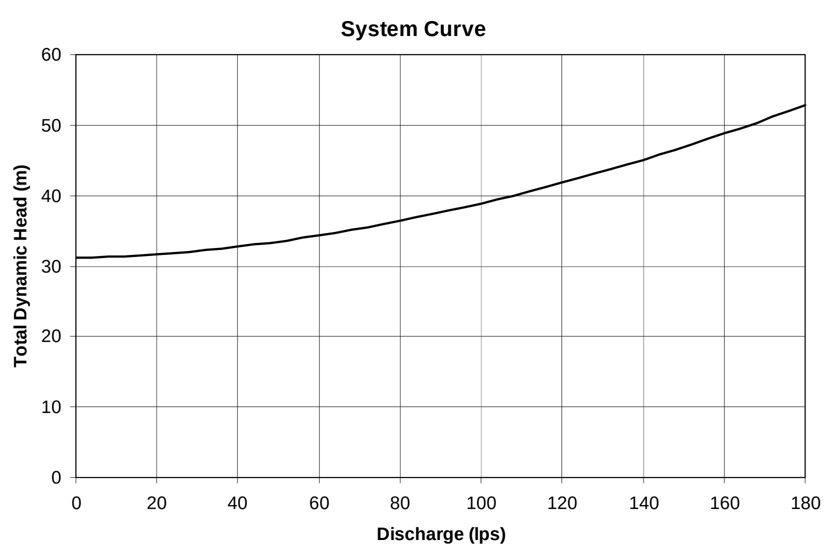
The water horsepower is:

$$\text{WHP} = \frac{QH}{102} = \frac{(102 \text{ lps})(40 \text{ m})}{102} = 40 \text{ kW (54 HP)} \tag{237}$$

The required brake horsepower is:

$$\text{BHP} = \frac{\text{WHP}}{E_{pump}} = \frac{40 \text{ kW}}{0.76} \approx 53 \text{ kW (71 HP)} \tag{238}$$

- This BHP value would be used to select a motor for this application
- These calculations give us one point on the system curve (Q and TDH)
- In this simple case, there would be only one system curve:

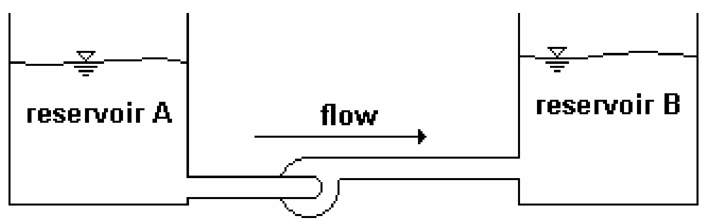


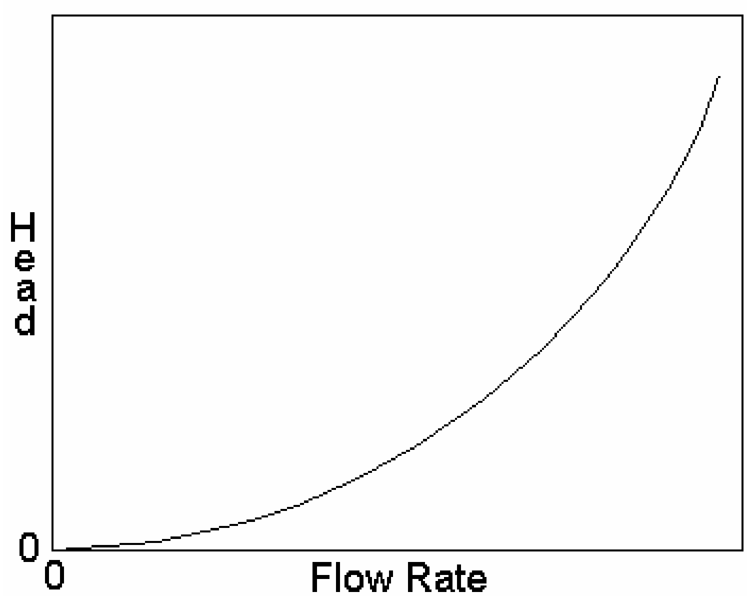
III. System Curves

- The “system curve” is a graphical representation of the relationship between discharge and head loss in a system of pipes
- The system curve is completely independent of the pump characteristics
- The basic shape of the system curve is parabolic because the exponent on the head loss equation (and on the velocity head term) is 2.0, or nearly 2.0
- The system curve will start at zero flow and zero head if there is no static lift, otherwise the curve will be vertically offset from the zero head value

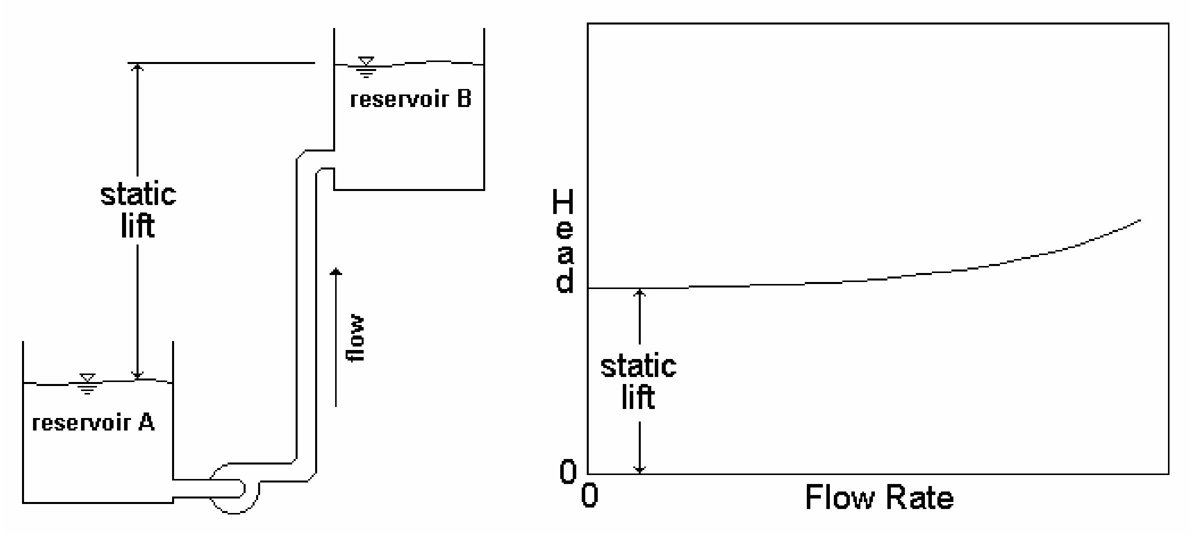
- Most sprinkle and trickle irrigation systems have more than one system curve because either the sprinklers move between sets (periodic-move systems), move continuously, or “stations” (blocks) of laterals are cycled on and off
- The intersection between the system and pump characteristic curves is the operating point (Q and TDH)
- A few examples of system curves:

1. All Friction Loss and No Static Lift

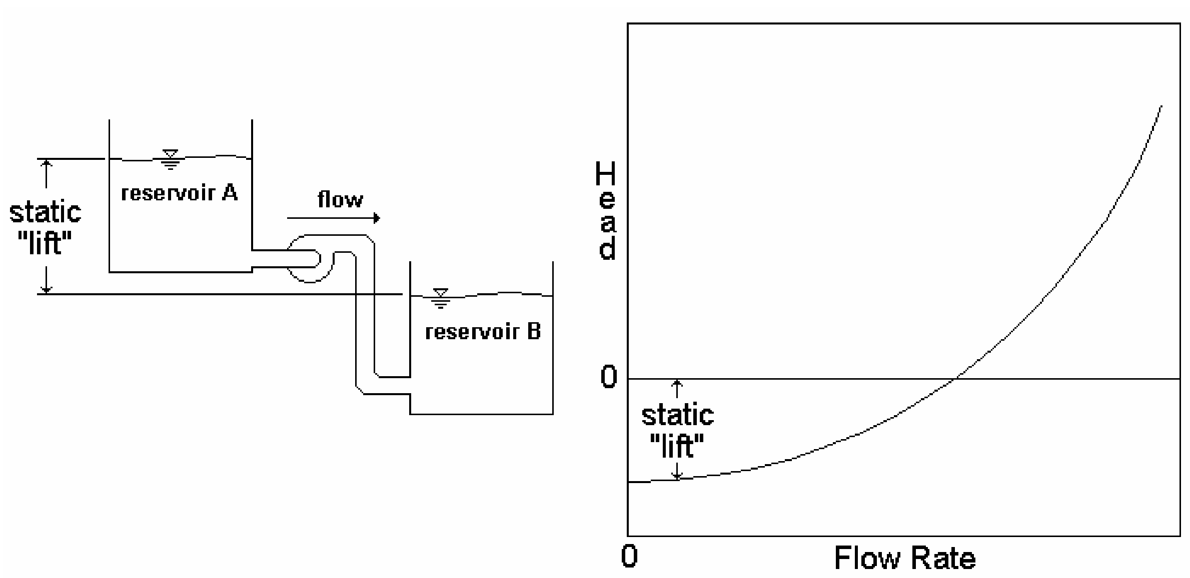




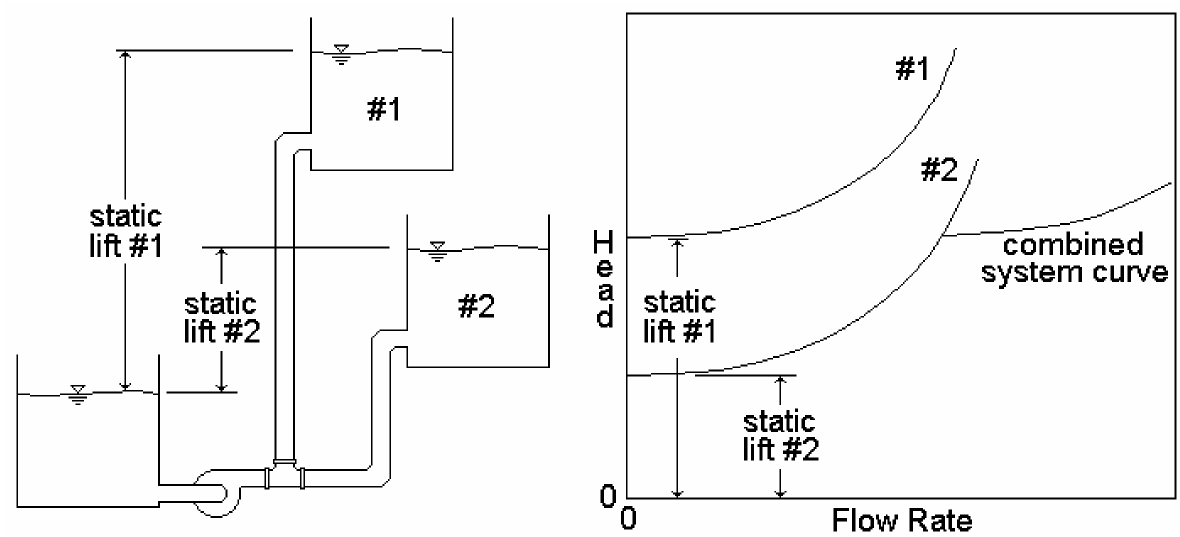
2. Mostly Static Lift, Little Friction Loss



3. Negative Static Lift

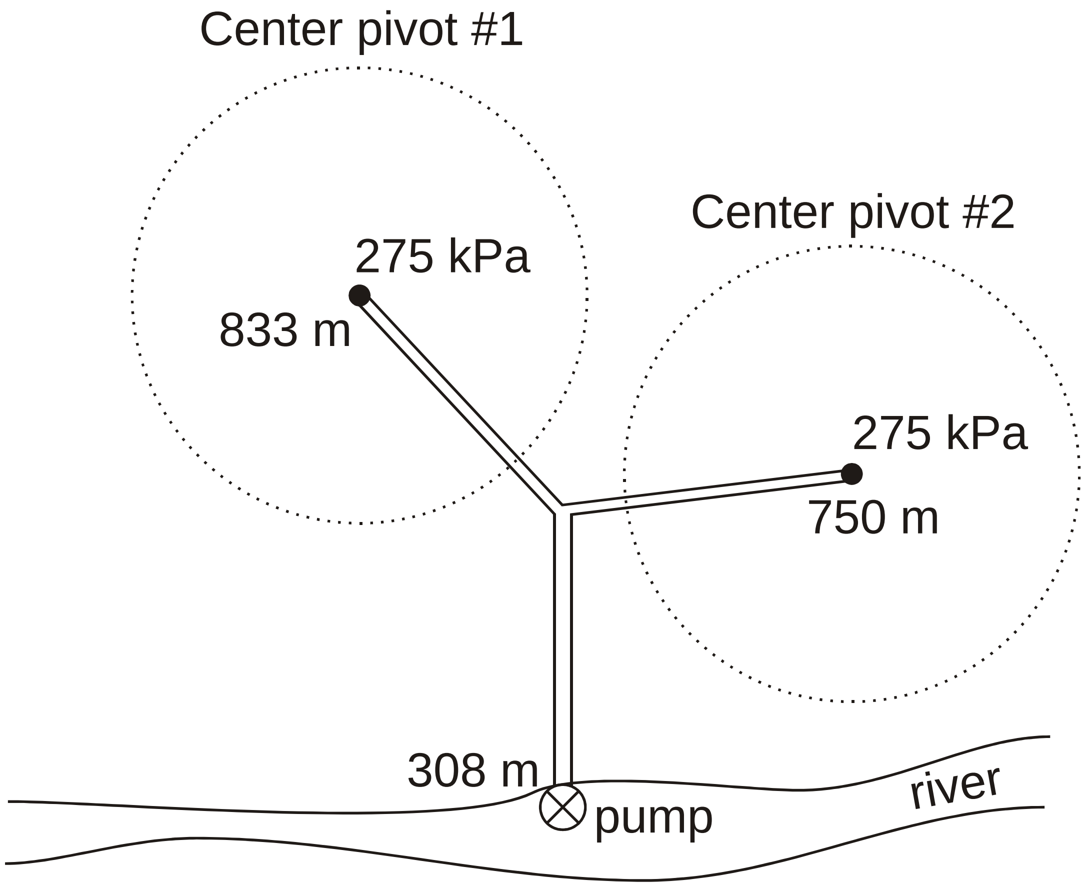


4. Two Different Static Lifts in a Branching Pipe



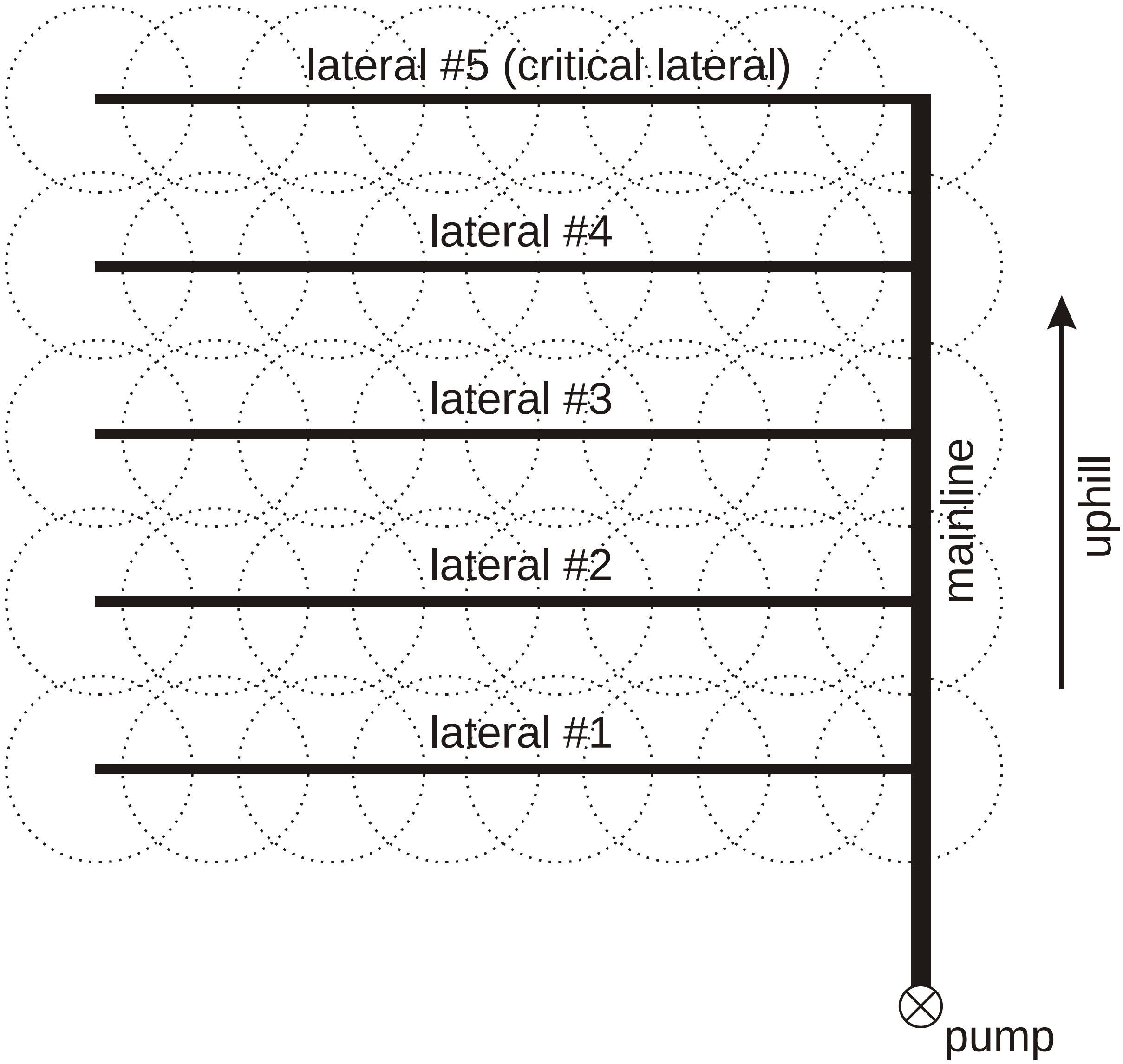
5. Two Center Pivots in a Branching Pipe Layout

- The figure below shows two center pivots supplied by a single pump on a river bank
- One of the pivots (#1) is at a higher elevation than the other, and is further from the pump – it is the "critical" branch of the two-branch pipe system
- Center pivot #2 will have excess pressure when the pressure is correct at Center pivot #1, meaning it will need pressure regulation at the inlet to the pivot lateral
- Use the critical branch (the path to Center pivot #1, in this case) when calculating TDH for a given operating condition – Do Not Follow Both Branches when calculating TDH
- if you cannot determine which is the critical branch by simple inspection, you must test different branches by making calculations to determine which is the critical one
- Note that the system curve will change with center pivot lateral position when the topography is sloping and or uneven within the circle
- Of course, the system curve will also be different if only one of the center pivots is operating



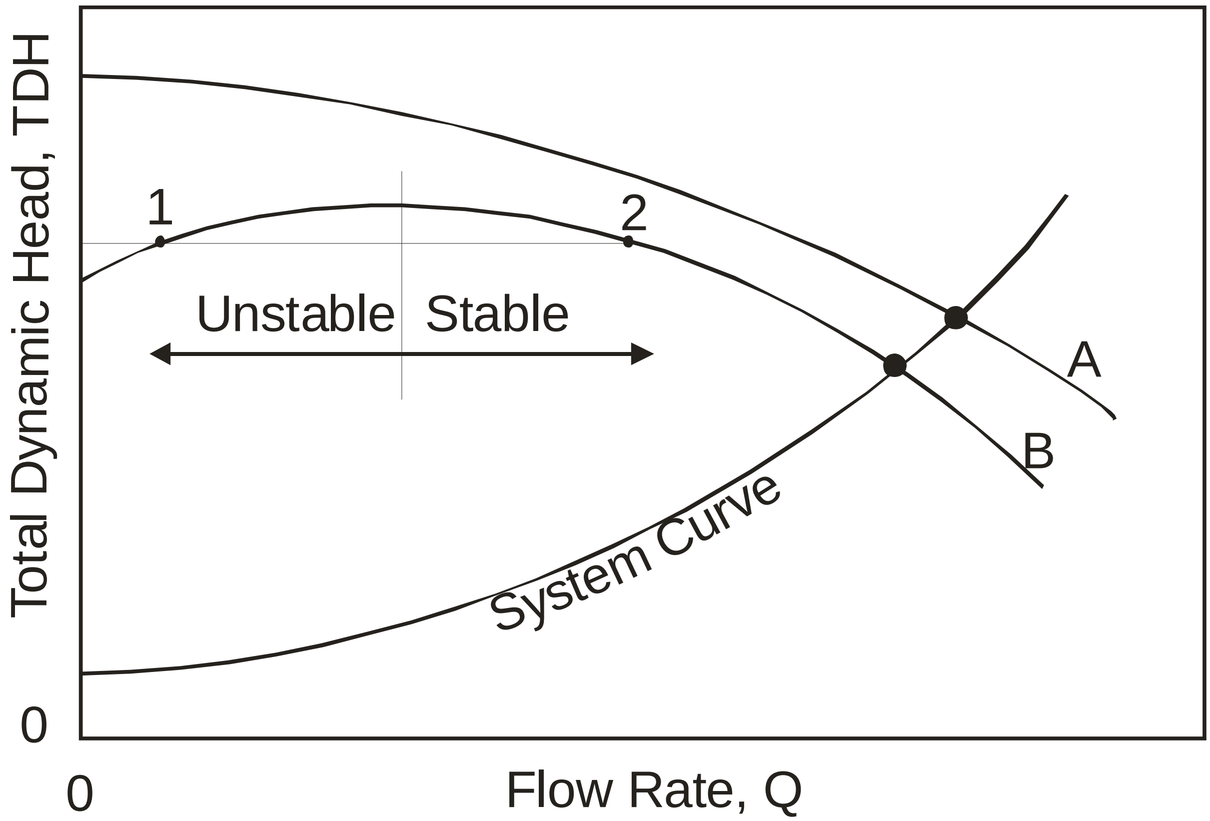
6. A Fixed Sprinkler System with Multiple Operating Laterals

- The next figure shows a group of laterals in parallel, attached to a common mainline in a fixed sprinkler system
- All of the sprinklers operate at the same time (perhaps for frost control or crop cooling purposes, among other possibilities)
- This is another example of a branching pipe system
- Since the mainline runs uphill, it is easy to determine by inspection that the furthest lateral will be the critical branch in this system layout – use this branch to determine the TDH for a given system flow rate
- Hydraulic calculations would be iterative because you must also determine the flow rate to each of the laterals since the flow rate is changing with distance along the mainline
- But in any case, <u>Do</u> <u>Not</u> <u>Follow</u> <u>Multiple</u> <u>Branches</u> when determining the TDH for a given system flow rate
- Remember that TDH is the resistance "felt" by the pump for a given flow rate and system configuration



7. Two Flow Rates for Same Head on Pump Curve

- Consider the following graph
- “A” has a unique Q for each TDH value
- “B” has two flow rates for a given head, over a range of TDH values
- Pumps with a characteristic curve like “B” should usually be avoided



Affinity Laws and Cavitation

I. Affinity Laws

1. Pump operating speed:

$$\frac{Q_1}{Q_2} = \frac{N_1}{N_2} \qquad \frac{H_1}{H_2} = \left(\frac{N_1}{N_2}\right)^2 \qquad \frac{BHP_1}{BHP_2} = \left(\frac{N_1}{N_2}\right)^3 \tag{239}$$

where Q is flow rate; N is pump speed (rpm); H is head; and BHP is "brake horsepower"

- The first relationship involving Q is valid for most pumps
- The second and third relationships are valid for centrifugal, mixed-flow, and axial-flow pumps

2. Impeller diameter:

$$\frac{Q_1}{Q_2} = \frac{D_1}{D_2} \qquad \frac{H_1}{H_2} = \left(\frac{D_1}{D_2}\right)^2 \qquad \frac{BHP_1}{BHP_2} = \left(\frac{D_1}{D_2}\right)^3 \tag{240}$$

- These three relationships are valid only for centrifugal pumps
- These relationships are not as accurate as those involving pump operating speed, N (rpm)

Comments:

- The affinity laws are only valid within a certain range of speeds, impeller diameters, flow rates, and heads
- The affinity laws are more accurate near the region of maximum pump efficiency (which is where the pump should operate if it is selected correctly)

- It is more common to apply these laws to reduce the operating speed or to reduce the impeller diameter (diameter is never increased)
- We typically use these affinity laws to fix the operating point by shifting the pump characteristic curve so that it intersects the system curve at the desired Q and TDH

II. Fixing the Operating Point

Combine the first two affinity law relationships to obtain:

$$\frac{H_1}{H_2} = \left(\frac{Q_1}{Q_2}\right)^2 \tag{241}$$

- If this relationship is plotted with the pump characteristic curve and the system curve, it is called the “equal efficiency curve”
- This is because there is typically only a small change in efficiency with a small change in pump speed
- Note that the “equal efficiency curve” will pass through the origin (when Q is zero, H is zero)

- Follow these steps to adjust the: (1) speed; or, (2) impeller diameter, such that the actual operating point shifts up or down along the system curve:

    1. Determine the head, H_2, and discharge, Q_2, at which the system should operate (the desired operating point)
    2. Solve the above equation for H_1, and make a table of H_1 versus Q_1 values (for fixed H_2 and Q_2):

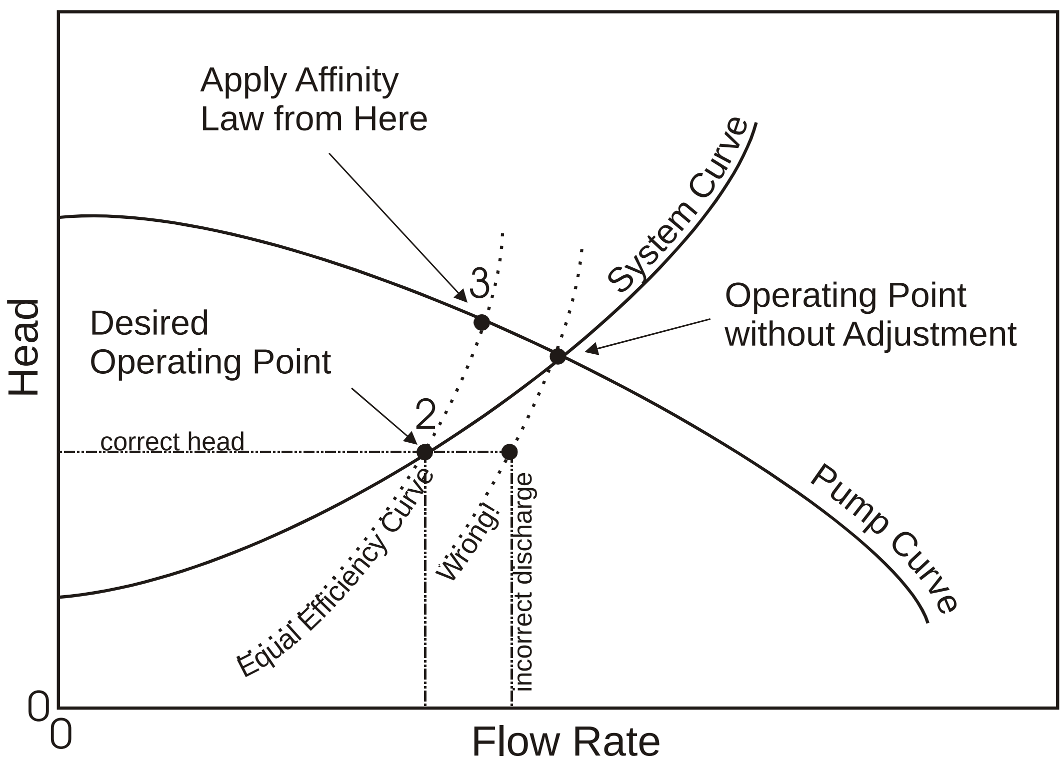
$$H_1 = H_2\left(\frac{Q_1}{Q_2}\right)^2 \tag{242}$$

3. Plot the values from this table on the graph that already has the pump characteristic curve
4. Locate the intersection between the pump characteristic curve and the "equal efficiency curve", and determine the Q_3 and H_3 values at this intersection
5. Use either of the following equations to determine the new pump speed (or use equations involving D to determine the trim on the impeller):

$$N_{new} = N_{old}\left(\frac{Q_2}{Q_3}\right) \quad \text{or,} \quad N_{new} = N_{old}\sqrt{\frac{H_2}{H_3}} \tag{243}$$

6. Now your actual operating point will be the desired operating point (at least until the pump wears appreciably or other physical changes occur)

- <u>You cannot directly apply any of the affinity laws</u> in this case because you will either get the right discharge and wrong head, or the right head and wrong discharge



III. Specific Speed

- The specific speed is a dimensionless index used to classify pumps
- It is also used in pump design calculations

Pump Type	***Specific Speed***
Centrifugal (volute case)	500 - 5,000
Mixed Flow	4,000 - 10,000
Axial Flow	10,000 - 15,000

- To be truly dimensionless, it is written as:

$$N_s = \frac{2\pi N\sqrt{Q}}{(gH)^{0.75}} \tag{244}$$

where the 2π is to convert revolutions (dimensional) to radians (dimensionless)

- Example: units could be N = rev/s; Q = m^3/s; g = m/s^2; and H = m
- However, in practice, units are often mixed, the 2π is not included, and even g may be omitted
- This means that N_s must not only be given numerically, but the exact definition must be specified

IV. Cavitation

- Air bubbles will form (the water boils) when the pressure in a pump or pipeline drops below the vapor pressure
- If the pressure increases to above the vapor pressure downstream, the bubbles will collapse
- This phenomenon is called “cavitation”
- Cavitation often occurs in pumps, hydroelectric turbines, pipe valves, and ship propellers
- Cavitation is a problem because of the energy released when the bubbles collapse; formation and subsequent collapse can take place in only a few thousandths of a second, causing local pressures in excess of 150,000 psi, and local speeds of over 1,000 kph
- The collapse of the bubbles has also been experimentally shown to emit small flashes of light (“sonoluminescence”) upon implosion, followed by rapid expansion on shock waves
- Potential problems:

  1. noise and vibration
  2. reduced efficiency in pumps

3. reduced flow rate and head in pumps
4. physical damage to impellers, volute case, piping, valves

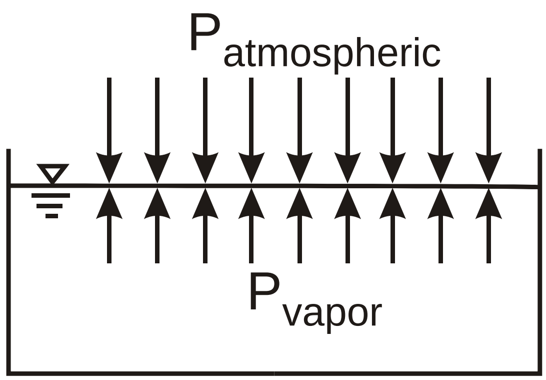
- From a hydraulics perspective cavitation is to be avoided
- But, in some cases cavitation is desirable. For example,

1. acceleration of chemical reactions
2. mixing of chemicals and or liquids
3. ultrasonic cleaning

- Water can reach the boiling point by:

1. reduction in pressure (often due to an increase in velocity)
2. increase in temperature

- At sea level, water begins to boil at 100°C (212°F)
- But it can boil at lower temperatures if the pressure is less than that at mean sea level (14.7 psi, or 10.34 m)

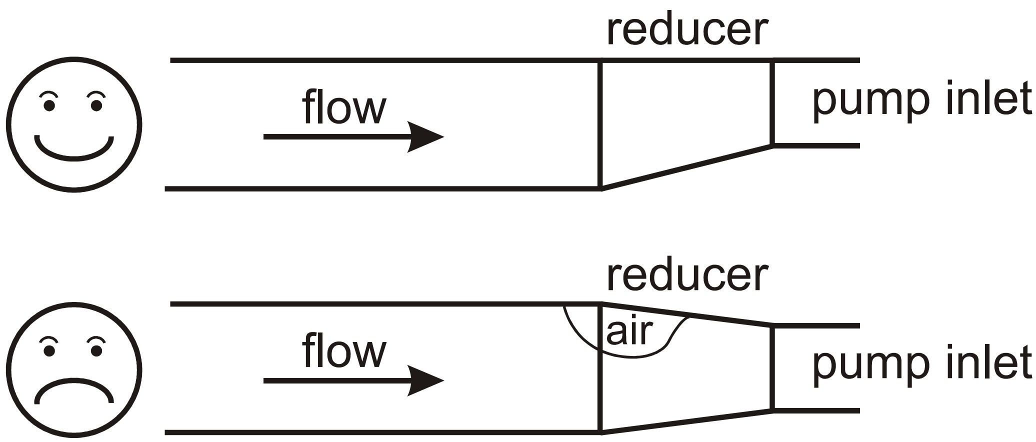


container with water

- Pump inlets often have an eccentric reducer (to go from a larger pipe diameter to the diameter required at the pump inlet:

1. Large suction pipe to reduce friction loss and increase NPSHa, especially where $NPSH_a$ is already too close to $NPSH_r$ (e.g. high-elevation pump installations where the atmospheric pressure head is relatively low)
2. Eccentric reducer to avoid accumulation of air bubbles at the top of the pipe

- See the following figure…



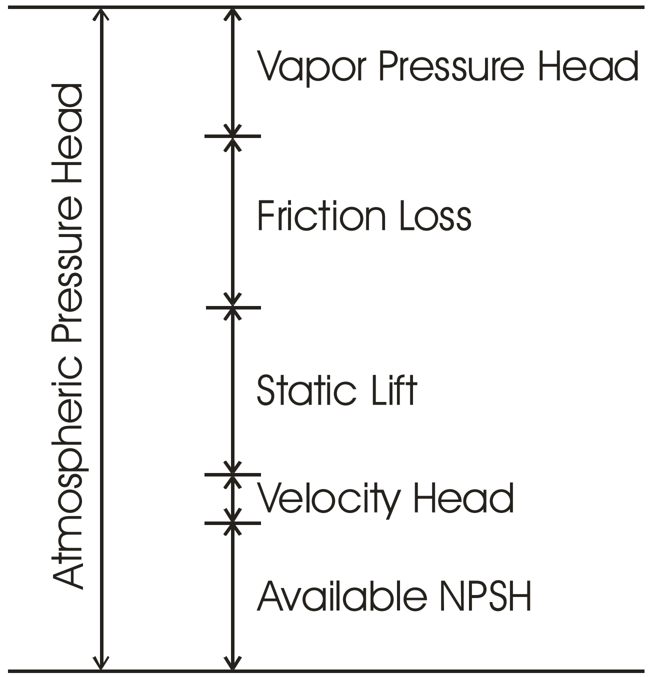
Required NPSH

- Data from the manufacturer are available for most centrifugal pumps
- Usually included in this data are recommendations for required *Net Positive Suction Head*, $NPSH_r$
- $NPSH_r$ is the minimum pressure head at the entrance to the pump, such that cavitation does not occur in the pump
- The value depends on the type of pump, its design, and size
- $NPSH_r$ also varies with the flow rate at which the pump operates
- $NPSH_r$ generally increases with increasing flow rate in a given pump
- This is because higher velocities occur within the pump, leading to lower pressures
- Recall that according to the Bernoulli equation, pressure will tend to decrease as the velocity increases, elevation being the same
- $NPSH_r$ is usually higher for larger pumps, meaning that cavitation can be more of a problem in larger pump sizes

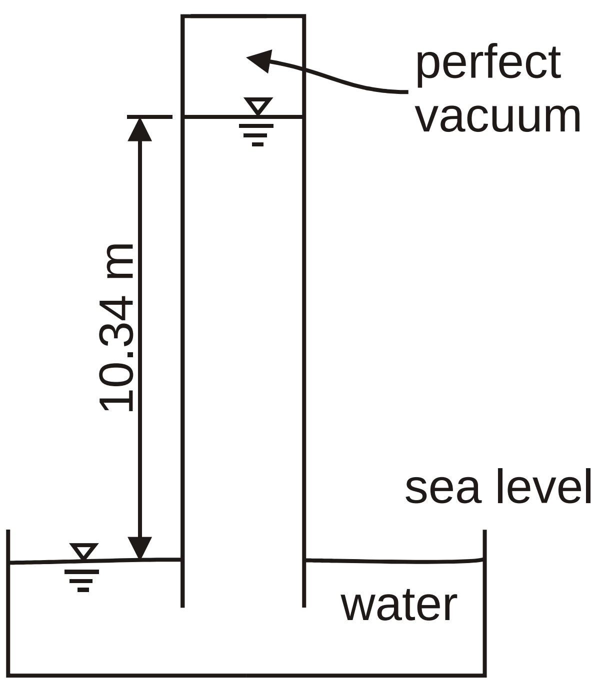
Available NPSH

- The available NPSH, or $NPSH_a$, is equal to the atmospheric pressure minus all losses in the suction piping (upstream side of the pump), vapor pressure, velocity head in the suction pipe, and static lift
- When there is suction at the pump inlet (pump is operating, but not yet primed), the only force available to raise the water is that of the atmospheric pressure
- But, the suction is not perfect (pressure does not reduce to absolute zero in the pump) and there are some losses in the suction piping

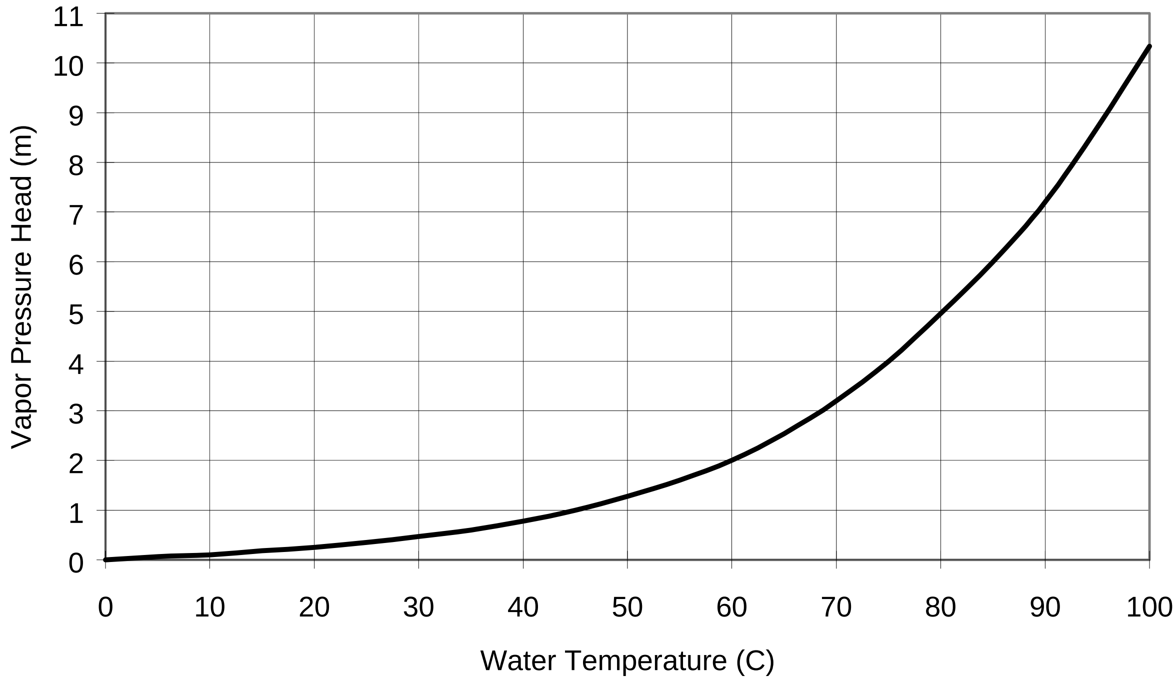
$$NPSH_a = h_{atm} - h_{vapor} - h_f - h_{lift} - \frac{V^2}{2g} \tag{245}$$



- If the pump could create a “perfect vacuum” and there were no losses, the water could be “sucked up” to a height of 10.34 m (at mean sea level)
- Average atmospheric pressure is a function of elevation above msl



- 10.34 m is equal to 14.7 psi, or 34 ft of head
- Vapor pressure of water varies with temperature



- Herein, when we say “vapor pressure,” we mean “saturation vapor pressure”
- Saturation vapor pressure head (as in the above graph) can be calculated as follows:

$$h_{vapor} = 0.0623 \exp\left(\frac{17.27\,T}{T + 237.3}\right) \quad (246)$$

for h_{vapor} in m; and T in ^{o}C

- Mean atmospheric pressure head is essentially a function of elevation above mean sea level (msl)
- Two ways to estimate mean atmospheric pressure head as a function of elevation:

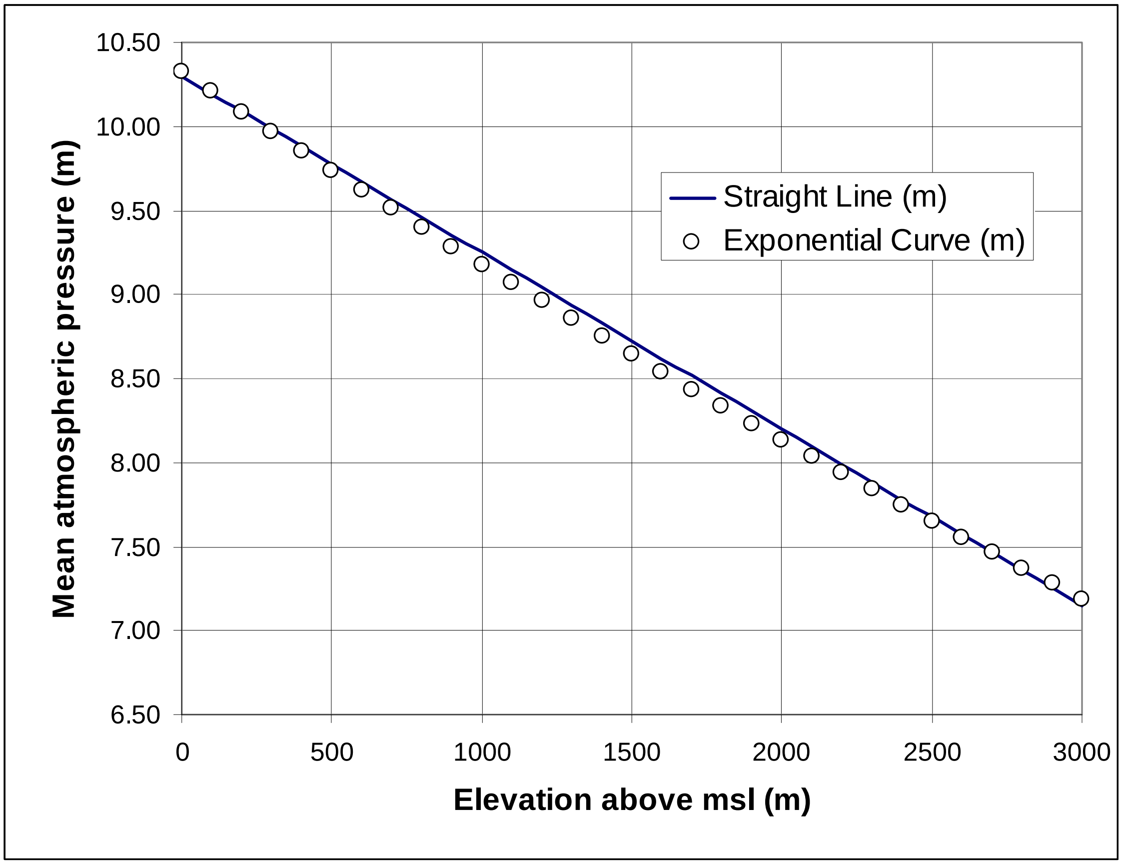
Straight line:

$$h_{atm} = 10.3 - 0.00105\,z \quad (247)$$

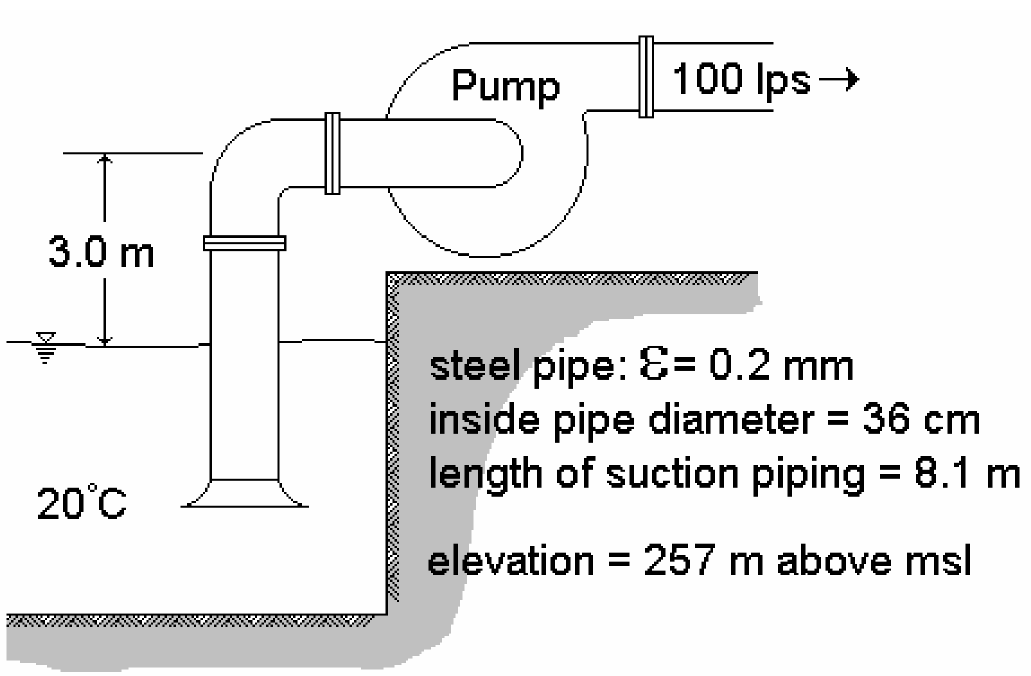
Exponential curve:

$$h_{atm} = 10.33\left(\frac{293 - 0.0065\,z}{293}\right)^{5.26} \quad (248)$$

where h_{atm} is atmospheric pressure head (m of water); and z is elevation above mean sea level (m)



V. Example Calculation of $NPSH_a$



1. Head Loss due to Friction

$$\frac{\varepsilon}{D} = \frac{0.2\text{ mm}}{360\text{ mm}} = 0.000556 \tag{249}$$

viscosity at 20°C, $\nu = 1.003(10)^{-6}$ m^2/s

flow velocity,

$$V = \frac{Q}{A} = \frac{0.100\text{ m}^3/\text{s}}{\frac{\pi}{4}(0.36)^2} = 0.982\text{ m/s} \tag{250}$$

Reynold's Number,

$$N_R = \frac{VD}{\nu} = \frac{(0.982)(0.36)}{1.003(10)^{-6}} = 353{,}000 \tag{251}$$

Darcy-Weisbach friction factor, f = 0.0184

velocity head,

$$\frac{V^2}{2g} = \frac{(0.982)^2}{2g} = 0.049\text{ m} \tag{252}$$

head loss in suction pipe,

$$(h_f)_{pipe} = f\frac{L}{D}\frac{V^2}{2g} = 0.0184\left(\frac{8.1}{0.36}\right)(0.049) = 0.0203\text{ m} \tag{253}$$

local losses, for the bell-shaped entrance, K_r = 0.04; for the 90-deg elbow, K_r = 0.14. Then,

$$(h_f)_{local} = (0.04+0.14)(0.049) = 0.0088\text{ m} \tag{254}$$

finally,

$$(h_f)_{total} = (h_f)_{pipe} + (h_f)_{local} = 0.0203 + 0.0088 = 0.0291\text{ m} \tag{255}$$

2. Vapor Pressure

for water at 20°C, h_{vapor} = 0.25 m

3. Atmospheric Pressure

at 257 m above msl, h_{atm} = 10.1 m

4. Static Suction Lift

- the center of the pump is 3.0 m above the water surface
- (the suction lift would be negative if the pump were below the water surface)

5. Available NPSH

$$NPSH_a = h_{atm} - h_{vapor} - (h_f)_{total} - h_{lift} - \frac{V^2}{2g} \tag{256}$$

$$NPSH_a = 10.1 - 0.25 - 0.0291 - 3.0 - 0.049 = 6.77 \text{ m}$$

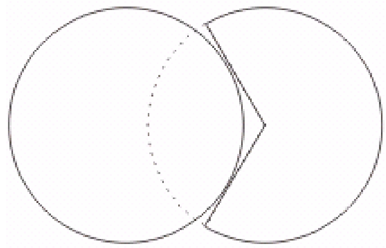
VI. Relationship Between NPSHr and NPSHa

- If $NPSH_r < NPSH_a$, there should be no cavitation
- If $NPSH_r = NPSH_a$, cavitation is impending
- As the available NPSH drops below the required value, cavitation will become stronger, the pump efficiency will drop, and the flow rate will decrease
- At some point, the pump would “break suction” and the flow rate would go to zero (even with the pump still operating)

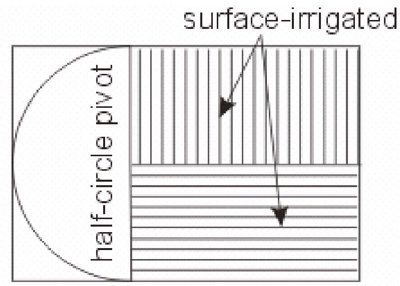
Lecture 12
Center Pivot Design & Operation

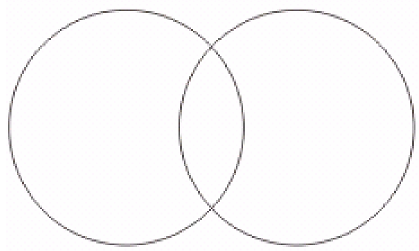
I. Introduction and General Comments

- Center pivots are used on about half of the sprinkler-irrigated land in the USA
- Center pivots are also found in many other countries
- Typical lateral length is 1,320 ft (400 m), or ¼ mile
- The lateral is often about 10 ft above the ground
- Typically, 120 ft pipe span per tower (range: 90 to 250 ft), often with one-horsepower electric motors (geared down)
- At 120 ft per tower, a 1,320-ft lateral has about 10 towers; with 1-HP motors, that comes to about 10 HP just for moving the pivot around in a circle
- The cost for a ¼-mile center pivot is typically about $55,000 (about $435/ac or $1,100/ha), plus about $20,000 (or more) for a corner system
- For a ½-mile lateral, the cost may be about $75,000 (w/o corner system)
- In the state of Nebraska there are said to be 43,000 installed center pivots, about 15% of which have corner systems
- Center pivots are easily (and commonly) automated, and can have much lower labor costs than periodic-move sprinkler systems
- Center pivot maintenance costs can be high because it is a large and fairly complex machine, operating under "field" conditions
- The typical maximum complete rotation is 20 hrs or so, but some (120-acre pivots) can go around in only about 6 hrs
- IPS 6" lateral pipe is common (about 6-5/8 inches OD); lateral pipe is generally 6 to 8 inches, but can be up to 10 inches for 2,640-ft laterals
- Long pivot laterals will usually have two different pipe sizes
- Typical lateral inflow rates are 45 - 65 lps (700 to 1,000 gpm)
- At 55 lps with a 6-inch pipe, the entrance velocity is a bit high at 3 m/s
- Typical lateral operating pressures are 140 - 500 kPa (20 to 70 psi)
- The end tower sets the rotation speed; micro switches & cables keep other towers aligned
- Corner systems are expensive; can operate using buried cable; corner systems don't necessarily irrigate the whole corner
- Without a corner system or end gun, $\pi/4$ = 79% of the square area is irrigated
- For a 1,320-ft lateral (without an end gun), the irrigated area is 125.66 acres
- For design purposes, usually ignore soil WHC (W_aZ); but, refill root zone at each irrigation (even if daily)
- Center pivots can operate on very undulating topography
- Some center pivots can be moved from field to field
- Below are some sample center pivot arrangements

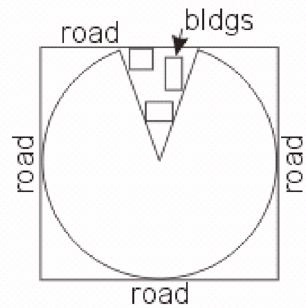


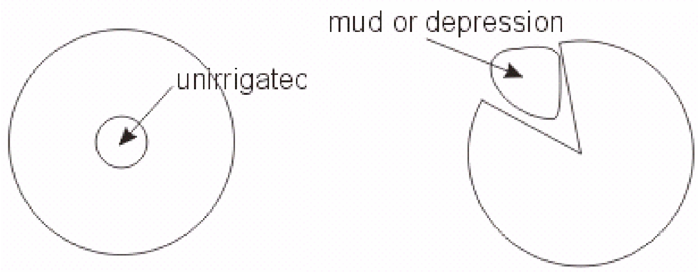
one pivot full circle,
the other partial circle

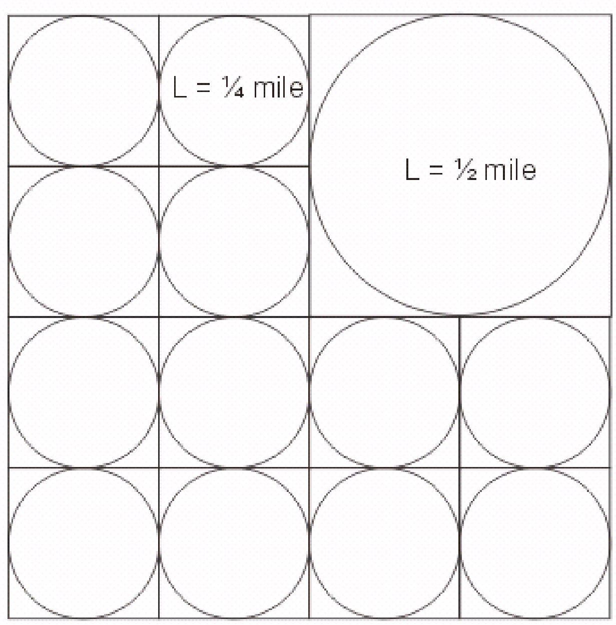




both full circle, overlapping








DANGER

- Some pivots have an end gun that turns on in the corners, in which all other sprinklers shut off via individual solenoid-actuated valves. The pivot stops in the corner while the end gun runs for a few minutes.
- Others just slow down in the corners, turning on an end gun, but leaving the other sprinklers running (at lower discharges)
- Many farmers like extra capacity in the center pivot so they can shut off during windy times of the day, and still complete the irrigations in time
- Corner systems have angle detectors so that sprinklers in the corner arm turn on and off individually (or in groups) as the arm swings out and then back in again
- Center pivots have safety switches to shut the whole thing off if any tower gets too far out of alignment. Some also have safety switches to shut them off if the temperatures gets below freezing (ice builds up and gets heavy, possibly collapsing the structure). Some have safety switches connected to timers: if a tower has not moved in a specified number of minutes, the system shuts down. There may also be safety switches associated with the chemical injection equipment at the lateral inlet location.

- Center pivots on rolling terrain almost always have pressure regulators at each sprinkler
- Some engineers claim that center pivots can have up to about 90% application efficiency

II. System Capacity

- The general center pivot design equation for system capacity is based on Eq. 5.4 from the textbook:

$$Q_s = K\frac{Ad}{fT} = \frac{R^2 d}{k_1 fT} = \frac{R^2 U_d k_f}{k_1 T E_{pa}} \tag{257}$$

where,

K is 2.78 for metric units and 453 for English units
k_1 is (3,600 s/hr)/π = 1,146 for metric units; 30.6 for English units
k_f is the peak period evaporation factor (Table 14.1 in the textbook)
A is area (ha or acre)
d is gross application depth (mm or inch)
f is frequency in days per irrigation
T is operating time (hrs/day)
R is the effective radius (m or ft)
U_d is the peak-use ET rate of the crop (mm/day or inch/day)
Q_s is the system capacity (lps or gpm)

- The gross application depth, d, is equal to d_n/E_{pa}, where E_{pa} is the design application efficiency, based on uniformity and percent area (pa) adequately irrigated
- The operating time, T, is generally 20-22 hrs/day during the peak-use period
- R is the *effective* radius, based on the wetted area from the center pivot
- The effective radius is about 400 m for many pivots
- $R \approx L + 0.4w$, where L is the physical length of the lateral pipe, and w is the wetted diameter of the end sprinkler
- This assumes that approximately 0.8 of the sprinkler radius beyond the lateral pipe is effective for crop production
- Note that, for center pivots, Q_s is proportional to U_d, and d and f are generally not used, which is similar to drip irrigation design

III. Gross Application Depth

- If a center pivot is operated such that the water holding capacity of the soil is essentially ignored, and water is applied frequently enough to satisfy peak-use crop water requirements, then use $d_n/f = U_d$, and

$$d' = \frac{k_f U_d}{E_{pa}} = \frac{k_f U_d}{DE_{pa} R_e O_e} \tag{258}$$

where d' is the gross application depth (mm/day or inches/day); and k_f is a peak-use period evaporation factor, which accounts for increased soil and foliage evaporation due to high frequency (daily) irrigation

- When LR > 0.1, the LR can be factored into the equation as:

$$d' = \frac{0.9 k_f U_d}{(1 - LR) DE_{pa} R_e O_e} \tag{259}$$

which is the same as Eq. 14.1b from the textbook, except that DE_{pa}, R_e and O_e are all as fractions (not percent)

- Values of k_f can be selected for the peak period from Table 14.1 of the textbook for varying values of frequency, f
- Values for non-peak periods can be computed as described in the textbook on page 314:

$$k_f' = \left(k_f - 1\right) \frac{(100 - PT')/PT'}{(100 - PT)/PT} + 1.0 \tag{260}$$

where k_f and PT are for the peak-use period (Table 14.1), and k'_f and PT' are the frequency coefficient and transpiration percentage (PT) for the non-peak period

$$PT = \frac{T}{ET} \tag{261}$$

- PT and PT' can be thought of as the basal crop coefficient (K_{cb}), or perhaps K_{cb} - 0.1 (relative to alfalfa, as per the note in Table 14.1)
- It represents the transpiration of the crop relative to an alfalfa reference

IV. Water Application along the Pivot Lateral

- A major design difficulty with a center pivot is maintaining the application rate so that it is less than the intake rate of the soil
- This is especially critical near the end of the lateral where application rates are the highest
- As one moves along the center pivot lateral, the area irrigated by each unit length of the lateral (each 1 ft or 1 m of length) at distance r from the pivot point can be calculated as:

$$a = \pi(r + 0.5)^2 - \pi(r - 0.5)^2 = 2\pi r \tag{262}$$

which is equal to the circumference at the radial distance r

- The portion of Q_s (called q) which is applied to the unit strip at distance r is:

$$\frac{q}{Q_s} = \frac{a}{A} = \frac{2\pi r}{\pi R^2} = \frac{2r}{R^2} \tag{263}$$

or,

$$q = \frac{2rQ_s}{R^2} \tag{264}$$

where q can be in units of lps per m, or gpm per ft

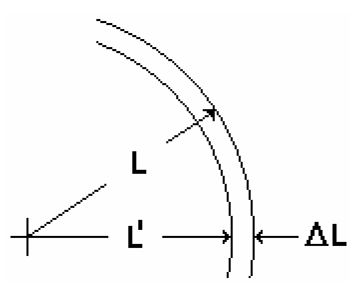
- This gives the amount of water which should be discharging from a specific unit length of lateral at a radial distance r from the pivot point
- The q value at the end of the lateral (r = R) per ft or m is:

$$q_{end} = \frac{2Q_s}{R} \tag{265}$$

- Use q to select the nozzle size, where $q_{nozzle} = q\ S_e$

V. End-Gun Discharge

- This last equation is very similar to Eq. 14.20a, except for the omission of the S_j term
- Equation 14.20b is for the end gun discharge, assuming that the end gun is used primarily to compensate for the lack of pattern overlap at the end of the lateral
- Equation 14.20b can be justified as follows:



- Assuming the "basic" circle discharge, Q_b, includes the end gun discharge, q_g, we can write:

$$\frac{Q_b}{\pi L^2} \approx \frac{q_g}{\Delta L\left(2\pi L'\right)} \tag{266}$$

or, perhaps more precisely,

$$\frac{Q_b}{\pi L^2} \approx \frac{q_g}{\Delta L\left(2\pi\left(L' + \Delta L/2\right)\right)} \tag{267}$$

but $\Delta L/2$ is generally very small compared to L', and this is ostensibly assumed in Eq. 14.20b, so after solving the above for q_g you will arrive at Eq. 14.20b:

$$q_g \cong \frac{2L'\Delta L}{L^2} Q_b; \text{ for } \Delta L < 0.03L \tag{268}$$

VI. Application Rate

- For a center pivot, $S_e = 1$ (based on a unit distance along the lateral) and S_l = w (wetted width in the tangential direction), so the average application rate (called AR) at a distance r along the lateral is:

$$AR = \frac{k_3 2r\, Q_s R_e O_e}{R^2 w} = \frac{2\pi r\, k_f d R_e O_e}{60 f\, T\, w} = \frac{2\pi r\, U'_d R_e O_e}{60\, T\, w} \tag{269}$$

where AR is the average application rate over width w (mm/min or inch/min); k_3 is 1.61 for English units and 60 for metric units; and f is the time to complete one revolution (days)

- w is equal to the wetted diameter of the spray or sprinkler nozzles on the lateral
- U'_d is the gross daily irrigation water requirement (mm/day or inch/day) and includes the effect of k_f

$$U'_d = \frac{k_f d}{f} = \frac{k_f\left(U_d - P_e\right)}{DE_{pa}} \tag{270}$$

- The three forms of the above equation assume a rectangular application pattern across the width w (that is, the application rate is uniform across w)
- Note that AR is proportional to r and is at a maximum at the end of the lateral
- Note that if w could be equal to $2\pi r$, the application rate would be equal to the gross application depth divided by the hours of operation per day (just like a fixed or solid-set sprinkler system) – but this is never the case with a center pivot machine

- At the end of the lateral (r = R), the average application rate can be calculated as:

$$AR_{r=R} = \frac{2\pi R U'_d R_e O_e}{60 T w} \tag{271}$$

again, where a rectangular application pattern is assumed

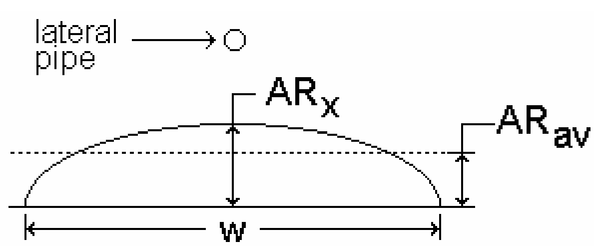
VII. Application Rate with an Elliptical Pattern

- If the application pattern perpendicular to the lateral were elliptical in shape:

$$AR_x = \frac{4}{\pi}\left(\frac{2k_3 r Q_s R_e O_e}{R^2 w}\right) = \frac{4}{\pi}\left(\frac{2\pi r U'_d R_e O_e}{60 T w}\right) = \frac{r U'_d R_e O_e}{7.5 T w} \tag{272}$$

where AR_x is the maximum application rate (in the center of the pattern) (AR_x is in mm/min for U'_d in mm/day)

- In the above equation, k_3 is 1.61 for English units, or 60 for metric units
- It is usually a better approximation to assume an elliptical pattern under the sprinklers than to assume a rectangular pattern, even though both are only approximations



- For example, if U'_d = 9 mm/day (which includes k_f), T is 22 hrs/day, w is 30 m, R is 400 m, R_e is 0.95 and O_e is 1.0, and the sprinkler application pattern is elliptical, then the maximum application rate at the far end of the lateral is:

$$AR_x = \frac{(400)(9)(0.95)(1.0)}{(7.5)(22)(30)} = 0.69 \text{ mm/min} \tag{273}$$

- AR_x is the peak AR (at the top of the ellipse, or directly beneath the lateral), so an "average" (AR_{av}) can be calculated, representing the average AR beneath the wetted area perpendicular to the lateral pipe
- The calculated value of 0.69 mm/min is 41.4 mm/hr, which could be tolerated only by a very sandy soil

- For a rectangular pattern, $AR_{av} = AR_x$
- For an elliptical pattern, $AR_{av} = (\pi/4)AR_x$
- Therefore, in the example, $AR_{av} = (\pi/4)(0.69) = 0.54$ mm/min
- If d were 10 mm, it would take $t_t = 10/0.54 = 18$ minutes to apply the water at the rate AR_{av}. (may want to use d R_e O_e instead of just d in such a calculation)
- R_e can be taken from Fig. 6.8 or from examples in Table 14.3
- Guidelines for determining CI are given in Table 14.4
- The center pivot speed (at the end of the lateral) is w/t_t, where t_t is the time of wetting
- In the preceding example, w is 30 m and t_t is 18 min
- Therefore, the speed should be about 30 m/18 min = 1.7 m/min at the end
- Note that with spray booms, w is larger, and AR is smaller for the same q value

Lecture 13
Center Pivot Nozzling

I. Center Pivot Nozzling

- The wetted width of the application package can be reduced closer to the pivot point because the towers are moving at a slower speed at inner points; therefore, the application intensity (AR) is less ($q_r \propto r$)
- Generally, if spray booms are required near the end of the center pivot, spray drops can be used toward the center, and the spray drops nearest the pivot point will produce something like a fine mist
- At the far end of the lateral the application may be more like a torrential rain
- Generally, impact and spray sprinklers would not be mixed on a center pivot because the pressure requirements are substantially different
- The minimum wetted width at any radius r along the pivot (for an elliptical pattern) can be calculated as:

$$w_r = \frac{8\,r\,U_d}{60\,T\,AR_x\,DE_{pa}} \tag{274}$$

where AR_x is the maximum permissible application rate (mm/min) according to limits imposed by the soil, slope, and vegetative cover; U_d is in mm/day; T is in hrs/day; and DE_{pa} is expressed as a fraction

- Note that w_r approaches zero for $r \rightarrow 0$
- A suitable application device can then be selected for radius r such that the wetted diameter of the device, w_d, is greater or equal to w_r ($w_d \geq w_r$)
- The actual application rate (mm/min) at radius r at the ground using a device with wetted diameter w_d should be:

$$AR_r \leq \frac{r\,U_d^{'}R_e O_e}{7.5\,T\,w_d} \tag{275}$$

- The term "R_eO_e" is included in the above equation to account for evaporation and wind drift losses, and pipe leakage
- Note that 60/8 = 7.5 and that we are using f = 1 day
- Divide by R_e in the above equation to obtain AR at the nozzle
- The wetting time at any radius r (assuming an elliptical pattern) along the lateral is:

$$\left(t_t\right)_r = \frac{4\,D_f}{\pi\,AR_r} \tag{276}$$

where D_f is the total cumulative application ($d\,R_e\,O_e$)

II. Center Pivot Nozzling Example

Given:

- U_d = 8 mm/day
- P_e = 0 mm/day
- T = 22 hrs/day
- Q_s = 73.6 lps
- R = 400 m
- Speed = 21.6 hrs/revolution
- AR_x = 2.3 mm/min (allowable)
- k_f = 1.02
- DE_{pa} = 0.74
- R_e = 0.94
- O_e = 0.99

Calculations:

- Calculate U_d':

$$U_d^{'} = \frac{k_f\left(U_d - P_e\right)}{DE_{pa}} = \frac{1.02\left(8-0\right)}{0.74} = 11.0 \text{ mm/day} \tag{277}$$

- Tangential speed of the pivot at distance "r" along the lateral:

$$S_r = \frac{2\pi r}{t} \tag{278}$$

where S_r is the speed in m/min; t is the minutes per full-circle revolution; and r is the radius from the pivot point in m

- The wetting time is, then:

$$\tau = \frac{w_r}{S_r} \tag{279}$$

where τ is wetting time in minutes; and w_r is the minimum wetted width in m.

- The values of w_d can be selected from available boom lengths, which in this case is 6, 8, 10, and 12 m. For less than 4-m width, no boom is required. Select w_d values such that AR_x is not exceeded.

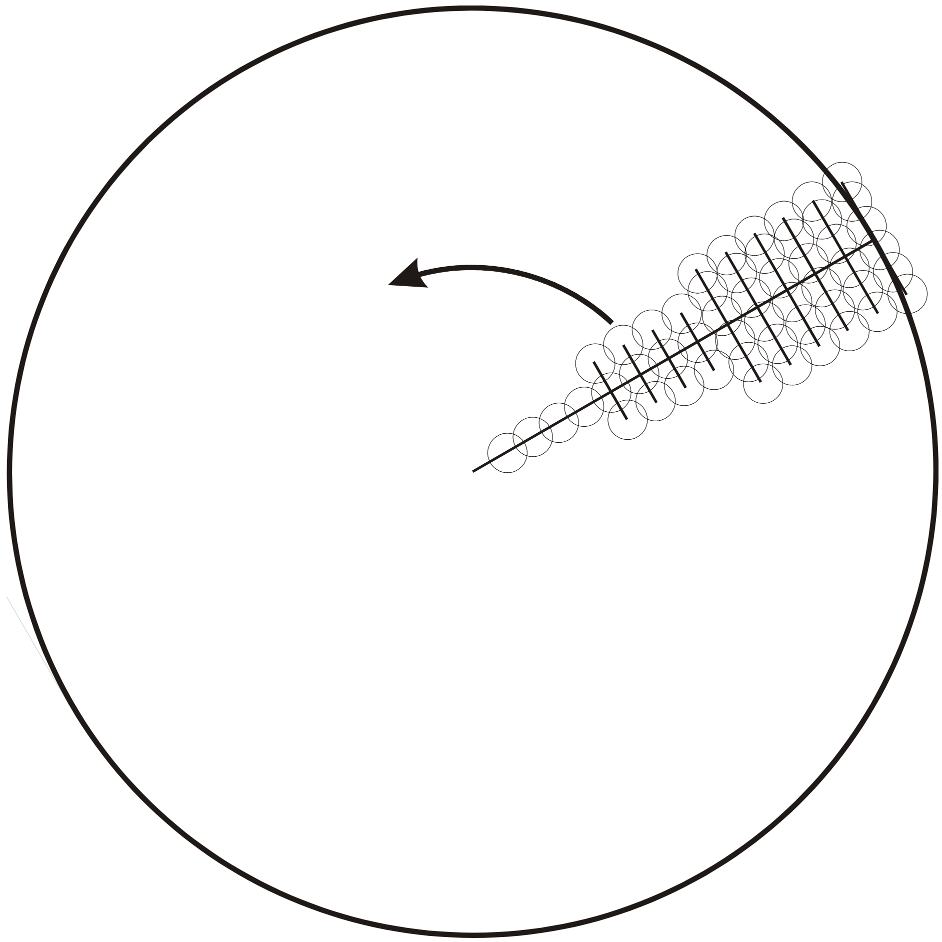
- Spreadsheet calculations:

r (m)	**q (lps/m)**	**w_r (m)**	**w_d (m)**	**AR_r (mm/min)**	**tau (s)**	**Q_r (lps)**
0	0.0000					73.6
40	0.0368	1.14	3	0.829	928	72.9
80	0.0736	2.28	3	1.658	464	70.7
120	0.1104	3.42	6	1.244	619	67.0
160	0.1472	4.56	6	1.658	464	61.8
200	0.1840	5.70	6	2.073	371	55.2
240	0.2208	6.84	8	1.866	413	47.1
280	0.2576	7.98	8	2.177	354	37.5
320	0.2944	9.12	10	1.990	387	26.5
360	0.3312	10.26	10	2.239	344	14.0
400	0.3680	11.39	12	2.073	371	0.0

- Notes about the variables:

  - "q" is the sprinkler discharge per unit length of lateral at r
  - "w_r" is the minimum wetted width at r
  - "w_d" is the actual (selected) wetted width at r
  - "AR_r" is the actual application rate at r
  - "wetting" is the wetting time at a given location
  - Q_r is the flow rate in the lateral pipe at a given location

- Most manufacturers prefer to specify the actual nozzle sizing and spacing along center pivots at the factory (rather than have the buyer specify these) for reasons of liability (they have specialized computer programs which attempt to maximize uniformity)
- Therefore, the designer will generally only specify the flow rate, pressure, and type of nozzle (spray drop, booms, impacts, etc.), and the manufacturer will specify individual nozzle sizes
- The following figure shows a center pivot with booms (the booms are greatly exaggerated in width to show the concept)

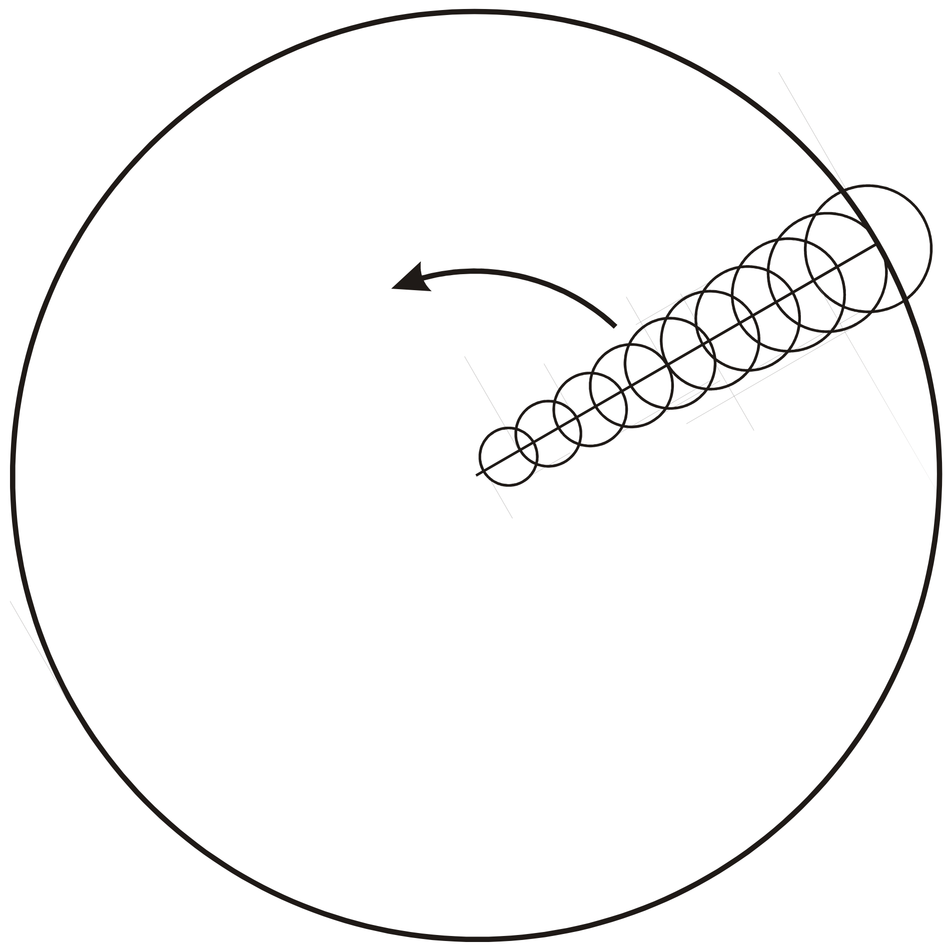


III. Sprinkler/Nozzle Configurations

- Because the required discharge per unit of lateral is linearly proportional to the distance along the lateral, discharge rates of sprinklers (or the sprinkler spacing) need to be varied to match this linear variation in flow rate
- Various configurations of locating sprinklers along a center pivot lateral are used

1. **Uniform Sprinkler Spacing**

- q_a of nozzles increases with r
- Large heads are required near the end of the lateral
- A large w (due to large heads) results near the end which is good, but large drop sizes may cause soil surface sealing and infiltration reduction
- The next figure shows a center pivot lateral with uniform sprinkler spacing and increasing w values toward the downstream end of the lateral

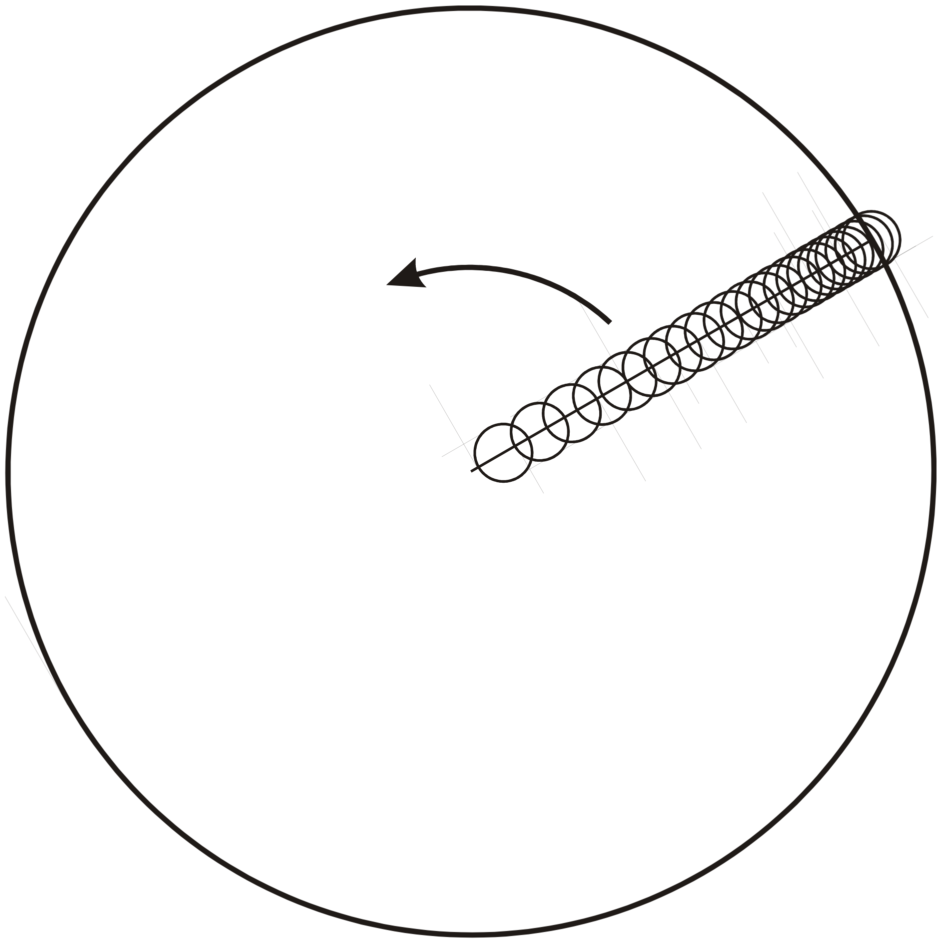


2. **Uniform Sprinkler Size** where the distance between sprinklers decreases with r

- Sprinkler density is higher near the downstream end of the lateral
- May allow the use of small nozzles with lower pressures and less soil sealing
- The value of w is smaller, so the soil must support a higher application rate
- See the following figure

3. **Combination of 1 and 2**

- This is the most common configuration because it is easier to match the required q/m of the lateral
- Smaller drop size
- Intermediate wetted width, w
- May have uniform spacing of outlets on lateral spans, but unused outlets are plugged (w/o any sprinkler), so pipes in each span are identical and interchangeable



Sizing Individual Nozzles:

1. Determine q for each spacing along the lateral (see Eq. 14.20a):

$$q_r = r\,S_r\,\frac{2Q_s}{R^2} \tag{280}$$

   where R is the maximum effective radius of the center pivot (approximately equal to L + 0.4w); S_r is the sprinkler spacing at a distance r from the pivot point; and r, S_r and R have the same units (m or ft)

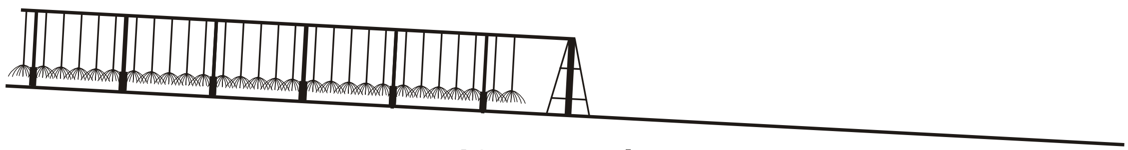
2. Beginning at the design pressure at the end of the lateral, L (where q is known), determine P_r:

$$P_r = (P_a)_{end} + (h_{fr})_{end} + (\Delta H_e)_{end-r} \tag{281}$$

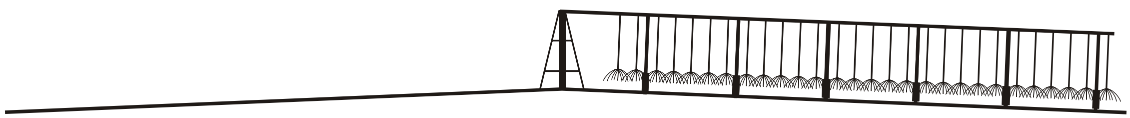
   where $(h_{fr})_{end}$ is the friction loss from point r to the far (downstream) end of the lateral. Note that $(\Delta H_e)_{end\text{-}r}$ often averages out to zero as the pivot makes its way around the circle, if the field slope is uniform (see the next figure).

3. Select the best nozzle size to provide q_r at a pressure of P_r
4. Return to Step 1 and repeat for the next r-S_r location
5. The required pressure at the pivot point is $P_r = 0$

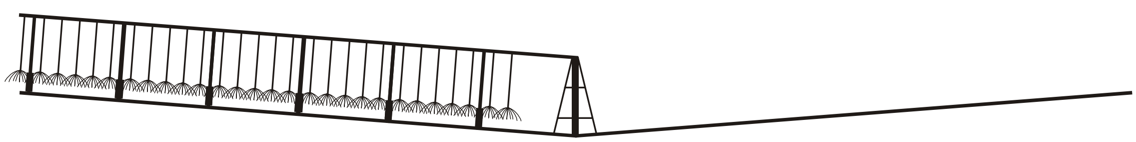
no slope



uniform slope



centered on a hill



centered on a depression

IV. Trajectory Angles of Impact Sprinklers

- For center pivots, sprinklers with 6° to 18° trajectory angles (low angle) are preferred because drift losses are minimized (see Table 14.3 in the textbook)
- Other things being the same, wind drift and evaporation losses can be higher with center pivots than with other types of sprinkler systems because of the relative height of the sprinklers above the ground
- But, you can use drop-down sprayers on a “goose-neck” pipe – some of these may be only a few centimeters from the mature crop canopy

V. End Guns

- The discharge for an end gun can be computed as (see Eq. 14.21):

$$Q_g = \frac{\left(\frac{2S_rQ}{R^2}\left(L + \frac{S_r}{2}\right)\right)}{0.93} \tag{282}$$

where L is the lateral length; R is the effective length of the pivot ($R = L + S_r$); Q_g is the end gun discharge; Q is the total center pivot flow rate (includes Q_g); S_r is R - L, which equals the effective wetted radius (or 75% of the gun radius)

- The above equation is essentially Eq. 14.20a with $r = L + S_r/2$
- The 0.93 factor (taken as O_e) is to account for ineffective discharge beyond 75% of the gun's wetted width, w (see page 349 of the textbook)
- Also, Q_g can be approximated as:

$$Q_g \approx Q\left(1 - \frac{L^2}{R^2}\right) \tag{283}$$

where Q includes Q_g and $R = L + S_r$

- Note that you can derive this last equation by substituting R - L for S_r in the previous equation as follows:

$$\begin{aligned} &\frac{2(R-L)Q\left[L + 0.5(R-L)\right]}{0.93R^2} = \\ &\frac{2QL(R-L) + Q(R-L)^2}{0.93R^2} \\ &= \frac{Q}{0.93}\left(1 - \frac{L^2}{R^2}\right) \end{aligned} \tag{284}$$

where the only difference is the 0.93 in the denominator

- A *part circle* rotation (typically about 150°) is generally used to achieve best uniformity under the area covered by the gun sprinkler, which is beyond the end of the lateral pipe
- If the rotation of the end gun covered 180° or more, it might make it too muddy for the wheels of the end tower – so with 150° (or so) the path in front of the end tower stays relatively dry

Booster Pump

- If an end gun is used primarily on corners, and if $Q_g > 0.2Q$, then a booster pump in parallel or series may be necessary
- Without a booster, when the gun is turned on (at the corner), the pressure along the lateral will drop, and individual sprinkler flow rates will be approximately $q[Q/(Q+Q_g)]$
- An alternative to a booster pump is to automatically decrease the center pivot speed at the corners: $S_{corner} = S[Q/(Q+Q_g)]$, where S is the speed in all places except the corners. That is, turn the gun on in the corners without any booster pump. This can help provide for equivalent, more nearly uniform application in all parts of the field (friction losses will be greater when the gun is operating; therefore, uniformity will not be perfect)
- If the end gun requires higher pressure than the nozzles along the center pivot lateral, then an electric-motor-driven booster pump may be mounted on the last tower, upstream of the end gun. This pump will increase the pressure to the end gun only. All other nozzles will be operated at lower pressure.

Center Pivot Hydraulic Analysis

I. Center Pivot Hydraulics

- The total discharge in the lateral pipe (not the flow rate from sprinklers at r) at any point r is approximately:

$$Q_r = Q_s\left(1 - \frac{r^2}{R^2}\right) \qquad (285)$$

where Q_s is total system capacity (possibly including an end gun); r is measured radially from the pivot point; and R includes S_r

Friction Loss

(1) For a center pivot without an end gun:

$$h_f = k_h F_p L\left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \qquad (286)$$

where k_h is 10.50 for h_f and L in ft, Q in gpm, and D in inches; k_h is $16.42(10)^4$ for h_f and L in m, Q in lps, and D in cm

- F_p is the multiple outlet friction factor for a center pivot (see Fig. 14.12)
- F_p = 0.555 for a center pivot with a "large" number of outlets and no end gun when using the Hazen-Williams equation.
- Other sources suggest using F_p = 0.543
- The value of C is about 130 for galvanized steel, or 145 for epoxy-coated steel

(2) For a center pivot with an end gun:

Compute friction loss as though the center pivot were R m long rather than L, and then subtract the non-existent friction past the point L, where R is the effective (wetted) radius and L is the physical length of the lateral pipe.

A traditional way to consider the effects of an end gun on friction loss is:

$$h_f = k_h F_p R \left(\frac{Q}{C}\right)^{1.852} D^{-4.87} - k_h F_g (R - L)\left(\frac{Q_g}{C}\right)^{1.852} D^{-4.87} \tag{287}$$

where Q is the total flow rate of the pivot plus the end gun; and Q_g is the flow rate of the end gun

- F_g should represent F_p for the distance R - L
- Base this on the bottom of page 355 of the textbook, where the number of outlets is (R - L)/S_e, where S_e is the sprinkler spacing on the last (downstream) span
- This F_g value will be conservative (it will underestimate the imaginary friction loss, therefore, will overestimate the total friction in the pivot lateral), because F_g will be greater than that stated
- Note that, near the end gun, the change in Q with distance is small; therefore, the value of F_g will usually be near unity

- A different method for calculating friction loss is suggested in the textbook (Eq. 14.26a)
- This may be a better method than that given above

Dual Pipe Sizes

- A center pivot may be assembled with dual pipe sizes (8- and 6-inch pipe, or 8- and 6 5/8-inch, for example)

- One requirement to note is that the tower spacing must be closer for the 8-inch pipe due to the added weight of water in the 8-inch pipe, and traction problems from towers which are too heavy and sink into the soil
- Therefore, balance the savings in h_f with added cost for the system
- Tower spacing is often 100 ft (30 m) for 8-inch pipe, and 150 ft (45 m) for 6-inch pipe
- Weight per tower = Wt of tower + Wt of 1 span (steel) + Wt of water in the span

Pressure Balance Equation

$$H_l = H_a + h_f + \Delta H_e + H_r + H_{minor} \tag{288}$$

where H_l is the pressure head required at ground level, at the pivot point; H_a is the pressure head requirement of the last nozzle (or end gun); h_f is the total friction loss along the pivot lateral; ΔH_e is the elevation increase between the pivot point and lateral end; H_r is the height of the lateral pipe less the vertical length of any drop tubes; and H_{minor} is the sum of all minor losses along the lateral

- H_a is, then, the pressure head requirement of the last nozzle
- If end guns require higher pressure than the nozzles along the lateral, then an electric booster pump can be installed at the last tower on the pivot
- See Chapter 14 of the textbook for the hydraulic effect of turning end guns or corner systems on and off, and for additional elevation effects

II. An Alternative Method for Determining H_f and H_l

- An alternative method to compute the exact value of H_l is to start at the pivot end and progress toward the pivot, adding friction losses
- This is a "stepwise" computational procedure, and is generally more accurate than using the JF_pL equations
- Friction loss, h_f, in pipe segments between nozzles on the pivot can be computed using the Hazen-Williams or Darcy-Weisbach equations with F_p =1
- In this case, include ΔH_e if H_{end} is the minimum pressure head at the highest elevation position of the lateral

$$H_l = H_{end} + \sum_{i=0}^{n} \left[\left(h_{f_i} + \Delta H_{e_i} \right) + H_r + H_{minor} \right] \tag{289}$$

where H_{end} is the desired nozzle pressure head at the pivot end; i is the outlet number along the lateral (i = 0 at the end, and i = n at the pivot point); n is the number of outlets (sprinklers) on the lateral; ΔH_{ei} is the elevation difference between two adjacent points (i and i+1) along the lateral; and h_{fi} is the friction loss between outlet i and i+1, where i+1 is upstream of i. This last term is defined as:

$$h_{f_i} = \frac{J_i \Delta L_i}{100} \quad (290)$$

where,

$$J_i = 1.21(10)^{12} \left(\frac{Q_i}{C} \right)^{1.852} D_i^{-4.87} \quad (291)$$

with Q_i in lps and D_i in mm

and,

$$Q_i = \sum_{j=0}^{i} q_j \quad (292)$$

in which j is the outlet number (j = 0 at the downstream end of the lateral); and q_j is a function of h_j and K_d

- H_{minor} includes short hose connections between pipe segments (at towers)
- Therefore, actual computed h_j values should be used with the selected nominal nozzle size (or FCN size), where h_j is the pressure head at outlet j
- The desired q_j is:

$$q_j = r_j S_{r_j} \left(\frac{2Q}{R^2} \right) = r_j \Delta L_i \left(\frac{2Q}{R^2} \right) \quad (293)$$

where ΔL_i is the distance between outlet i and outlet i+1.

- ΔL_i is constant for constant spacing, variable nozzle size
- ΔL_i is variable for variable spacing, constant nozzle size

Point of "Average" Elevation

The point of "average" elevation along a lateral may be determined by weighting elevations according to areas as (Allen 1991):

$$H_{ew} = \int_{r=0}^{L} \int_{\alpha}^{2\pi} \frac{r \left(H_e \right)_{r,\alpha}}{2\pi L^2} dr\, d\alpha \quad (294)$$

where r is a weighting term; and $(H_e)_{r,\alpha}$ is the elevation at radius r and pivot rotation angle α.

- Use $\alpha = 0$ for the whole irrigated area
- Note that on uniform slopes, the weighted elevation, H_{ew}, equals $(H_e)_{pivot}$
- Note also that you probably won't have data for elevations in polar coordinates, so this equation may be rather "academic"
- See the textbook for additional equations which consider elevation effects

Lecture 14

Center Pivot Uniformity Evaluation

I. Introduction

- The calculation of an application uniformity term must take into account the irrigated area represented by each catch container
- It is more important to have better application uniformity further from the pivot point than nearer, because the catch containers at larger distances represent larger irrigated areas
- If the catch containers are equally spaced in the radial direction, the area represented by each is directly proportional to the radial distance

II. Equation for Center Pivot CU

- The equation for CU proposed by Heermann and Hein is (ASAE/ANSI S436):

$$CU = 100\left(1.0 - \frac{\sum_{i=1}^{n}\left(r_i\left|d_i - \frac{\sum_{i=1}^{n}(d_i r_i)}{\sum_{i=1}^{n} r_i}\right|\right)}{\sum_{i=1}^{n}(d_i r_i)}\right) \tag{295}$$

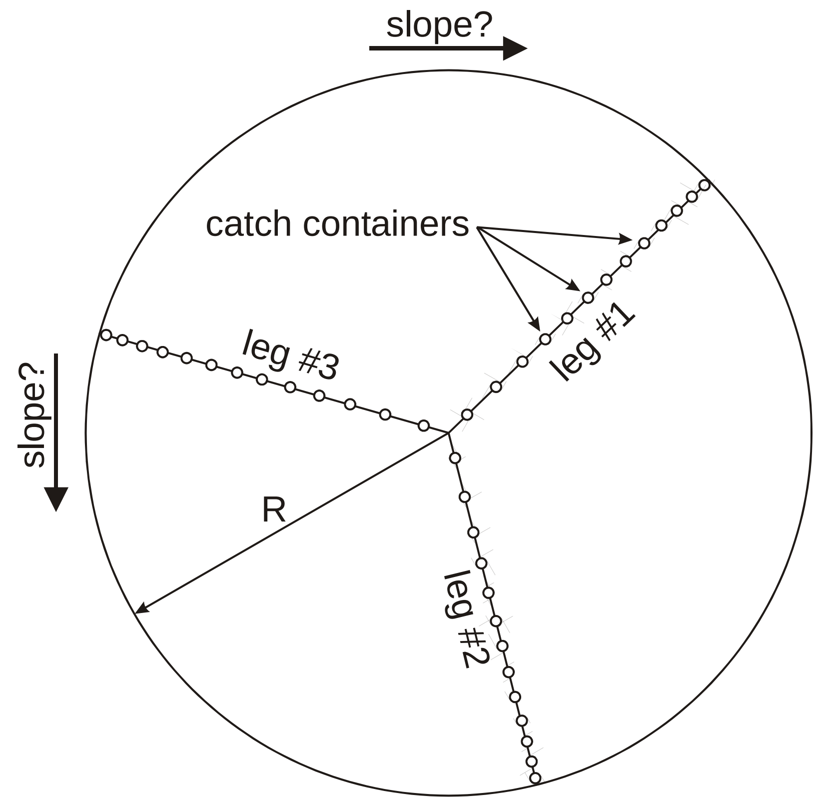
where CU is the coefficient of uniformity; d_i is the depth from an individual container; r_i is the radial distance from the pivot point; and n is the number of containers

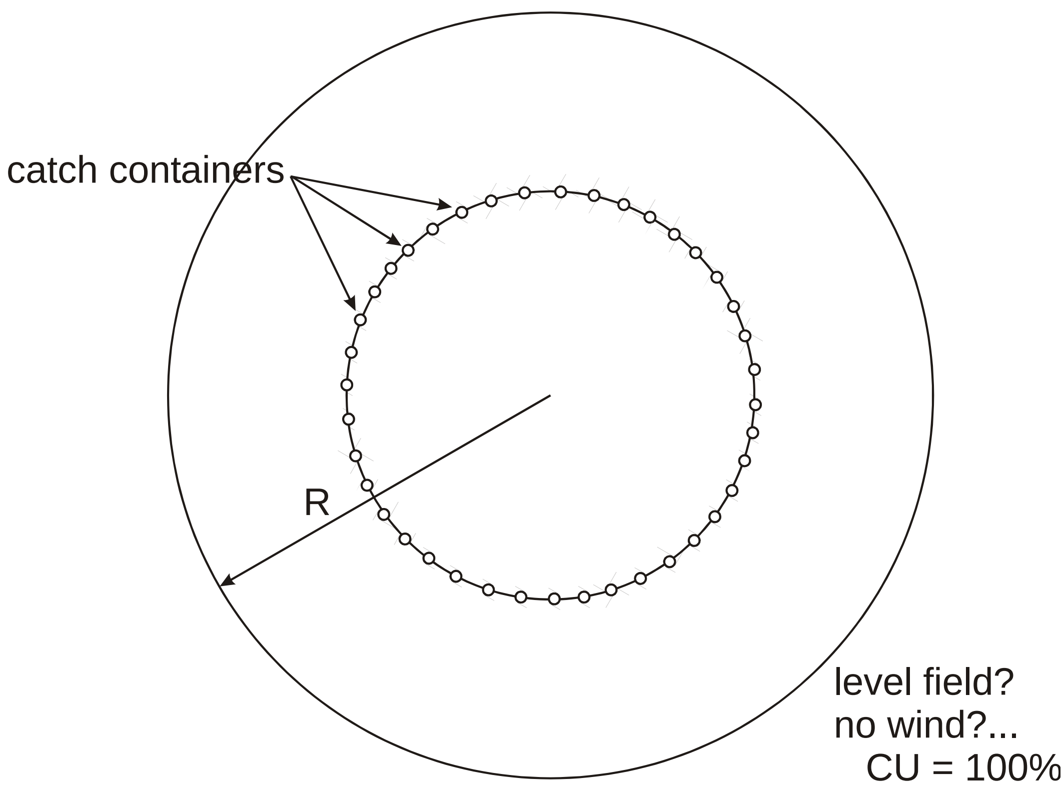
- First calculate the summations:

$$\sum_{i=1}^{n} r_i \quad \text{and,} \quad \sum_{i=1}^{n}(d_i r_i) \tag{296}$$

- Then, perform the outer summation to determine the CU value
- That is, don't recalculate the inner summation values for every iteration of the outer summation – it isn't necessary
- It is usually considered that a center pivot CU should be greater than 85%

- If the radial distances, r_i, are equal, the sequence number of the can (increasing with increasing radius) can be used instead of the actual distance for the purpose of calculating application uniformity
- Consider the following two figures:


slope?
catch containers
leg #3
leg #1
slope?
R
leg #2


catch containers
R
level field?
no wind?...
CU = 100%

III. Standard Uniformity Values

- You can also calculate the "standard" CU or DU if you weight each catch value by multiplying it by the corresponding radial distance
- To obtain the low ¼, rank the unweighted catches, then start summing radii (beginning with the radius for the lowest catch value) until the cumulative value is approximately equal to ¼ of the total cumulative radius
- This may or may not be equal to ¼ of the total catch values, because each catch represents a different annular area of the field
- Finally, divide the sum of the catches times the radii for this approximately ¼ area by the cumulative radius
- This gives the average catch of the low ¼
- <u>Don't</u> rank the weighted catches (depth x radius) because you will mostly get the values from the low r values (unless the inner catches are relatively high for some reason), and your answer will be wrong
- <u>Don't</u> calculate the average of the low ¼ like this… (because the lowest ¼ of the catches generally represents something different than ¼ of the irrigated area):

$$\cancel{\frac{\sum_{i=1}^{n/4}\left(d_i r_i\right)}{\sum_{i=1}^{n/4} r_i}}$$

- Actually, the equation at the right is all right, except for the value "n/4", which is probably the wrong number of ranked values to use in representing the low ¼
- You can set up a table like this in a spreadsheet application:

Ranked Center Pivot Catches				
Radius, r	**Cumulative r**	**Depth, d**	**d*r**	**Cumulative d*r**
		smallest		
		largest		
Totals:	**----**			**----**

- Note that when you rank the depths, the radius values should stay with the same depth values (so that the radius values will now be "unranked"; all mixed up)
- To get the average weighted depth for the whole pivot area, divide the total "Cumulative d*r" by the total "Cumulative r" (column 5 divided by column 2)
- Find the row corresponding closest to ¼ of the total "Cumulative r" value, and take the same ratio as before to get the weighted average of the low ¼ area
- Look at the example data analysis below:

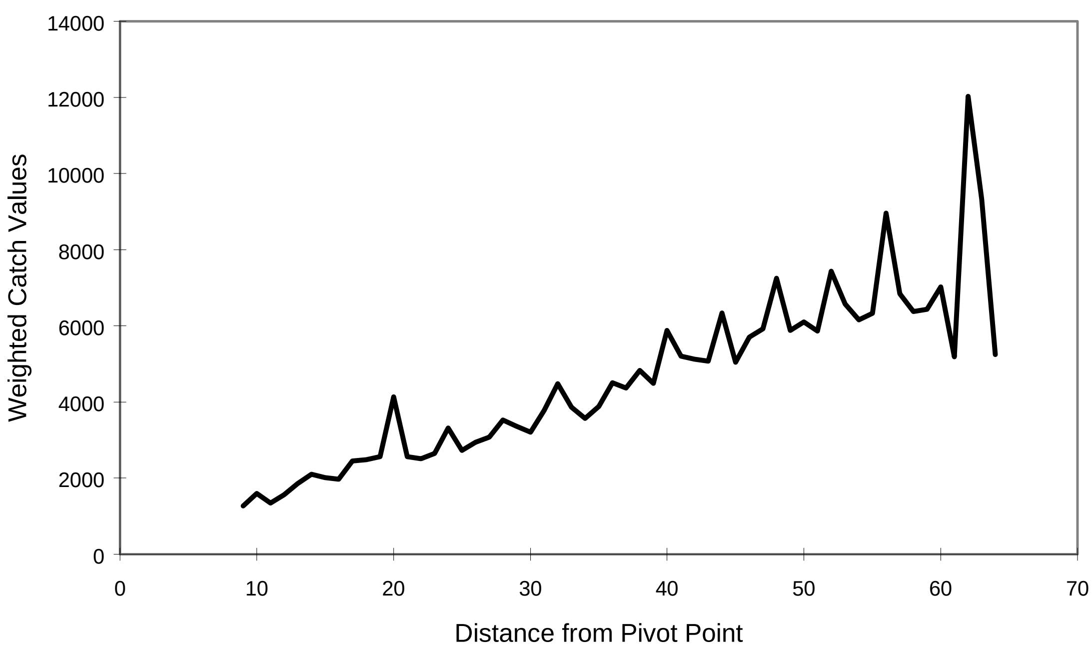
Ranked catches					
Radius, r	**Cum. r**	**Depth, d**	**d*r**	**Cum. d*r**	
120	120	0.52	62.6	62.6	
900	1,020	0.95	851.9	915	
160	1,180	1.29	205.8	1,120	
340	1,520	1.31	445.6	1,566	
1000	2,520	1.46	1,456.3	3,022	
1040	3,560	1.46	1,514.6	4,537	
240	3,800	1.48	355.3	4,892	
800	4,600	1.50	1,203.9	6,096	
860	5,460	1.53	1,315.0	7,411	
480	5,940	1.58	757.3	8,168	
1280	7,220	1.58	2,019.4	10,188	
980	8,200	1.60	1,569.9	11,758	
540	8,740	1.63	878.2	12,636	
360	9,100	1.65	594.2	13,230	
460	9,560	1.67	770.4	14,000	
880	10,440	1.70	1,495.1	15,496	
320	10,760	1.72	551.5	16,047	**1/4 area (11,055)**
1140	11,900	1.75	1,992.2	18,039	
1160	13,060	1.75	2,027.2	20,067	
280	13,340	1.82	509.7	20,576	
720	14,060	1.82	1,310.7	21,887	
1300	15,360	1.82	2,366.5	24,253	
200	15,560	1.84	368.9	24,622	
420	15,980	1.84	774.8	25,397	
440	16,420	1.89	833.0	26,230	
1020	17,440	1.89	1,931.1	28,161	
1200	18,640	1.92	2,301.0	30,462	
600	19,240	1.94	1,165.0	31,627	
640	19,880	1.94	1,242.7	32,870	
1060	20,940	1.94	2,058.3	34,928	
1100	22,040	1.94	2,135.9	37,064	
220	22,260	1.97	432.5	37,497	**1/2 area (22,110)**
1080	23,340	1.97	2,123.3	39,620	
380	23,720	1.99	756.3	40,376	
740	24,460	1.99	1,472.8	41,849	
920	25,380	1.99	1,831.1	43,680	
1220	26,600	1.99	2,428.2	46,108	
300	26,900	2.01	604.4	46,713	
180	27,080	2.03	364.8	47,077	
820	27,900	2.04	1,671.8	48,749	
1260	29,160	2.04	2,568.9	51,318	
660	29,820	2.06	1,361.7	52,680	
1180	31,000	2.06	2,434.5	55,114	
680	31,680	2.09	1,419.4	56,534	
940	32,620	2.11	1,985.0	58,519	
560	33,180	2.14	1,196.1	59,715	
260	33,440	2.16	561.7	60,276	
1120	34,560	2.18	2,446.6	62,723	
700	35,260	2.23	1,563.1	64,286	
760	36,020	2.23	1,697.1	65,983	
100	36,120	2.25	224.5	66,208	
960	37,080	2.26	2,167.0	68,375	
520	37,600	2.28	1,186.4	69,561	
620	38,220	2.28	1,414.6	70,976	
1240	39,460	2.28	2,829.1	73,805	
500	39,960	2.33	1,165.0	74,970	
140	40,100	2.35	329.6	75,300	
400	40,500	2.40	961.2	76,261	
780	41,280	2.52	1,968.9	78,230	
40	41,320	2.57	102.9	78,333	
80	41,400	2.57	205.8	78,538	
840	42,240	2.79	2,344.7	80,883	
580	42,820	2.82	1,633.0	82,516	
60	42,880	3.23	193.7	82,710	
1320	44,200	3.79	4,998.1	87,708	
20	44,220	3.83	76.7	87,784	

- Notice that the depth values (3rd column) are ranked from low to high
- Notice that the maximum value of cumulative r is 44,220 & maximum cumulative d*r is 87,784. Then, the weighted average depth for the entire center pivot is equal to 87,784/44,220 = 1.985 (whatever units)
- One quarter of 44,220 is equal to 11,055 which corresponds most closely to the row in the table with depth = 1.72. For the same row, divide the two cumulative columns (Col 5/Col 2) to get 16,047/10,760 = 1.491, which is approximately the average of the low ¼.
- Finally, estimate the distribution uniformity for this data set as:

$$DU \cong 100\left(\frac{1.491}{1.985}\right) \cong 75\% \qquad (297)$$

- Note that in this example, the average of the low ¼ was, in fact, based on approximately the first n/4 ranked values

- Consider the weighed catch-can data plotted below:



- As in any application uniformity evaluation, there is no “right” answer. The results are useful in a comparative sense with evaluations of other center pivots and other on-farm irrigation systems.
- However, a plot of the catches can give indications of localized problems along the center pivot radius

IV. The Field Work

- It may take a long time for the full catch in containers near the pivot point, and because these represent relatively small areas compared to the total irrigated area, it is usually acceptable to ignore the inside 10% or 20% of the radius
- The pivot quickly passes the outer cans, but takes longer to completely pass the inner cans, so you can collect the data from the outer cans sooner
- The pivot should not be moving so fast that the application depth is less than about 15 mm
- Catch containers can be placed beyond the physical length of the lateral pipe, but if they are so far out that the catches are very low, these can be omitted from the uniformity calculations
- Catch containers should be spaced in the radial direction no further than about 30% of the average wetted diameter of the sprinklers
- There is often an access road leading to the pivot point for inspection, manual operation, maintenance, and other reasons
- If the crop is dense and fairly tall (e.g. wheat or maize) it will be difficult to perform the evaluation unless the cans are placed on the access road
- Otherwise, you can wait until the crop is harvested, or do the test when the crop is still small
- Some people recommend two radial rows of catch cans, or even two parallel rows, to help smooth out the effects of the non continuous movement of towers (they start and stop frequently to keep the pivot lateral in alignment)
- Some have used troughs instead of catch cans to help ameliorate this problem.
- Note that if the field is sloping or undulating, the results from one radial row of catch cans may be quite different from those of a row on another part of the irrigated circle
- See Merriam and Keller (1978)

Linear Move Systems

I. Introduction

- Mechanically, a linear move system is essentially the same as a center pivot lateral, but it moves sideways along a rectangular field, perpendicular to the alignment of the lateral pipe
- The variation of flow rate in a linear move lateral is directly proportional to distance along the lateral pipe, whereas with center pivots it is proportional to a function of the square of the distance from the pivot point
- A center or end tower sets the forward speed of the machine, and the other towers just move to keep in line with the guide tower (this is like the far end tower on a center pivot)
- Usually, each tower is independently guided by cables and micro-switches as for a center pivot – this keeps the lateral pipe in a straight line (aligned with itself)
- Alignment with the field is usually not mechanically “enforced”, but it is “monitored” through switches in contact with a straight cable along the center of the field, or along one end of the field
- The center tower has two "fingers", one on each side of the cable, usually slightly offset in the direction of travel (they aren’t side by side). The fingers should be in constant contact with the cable – if one is lifted too far a switch will be tripped, shutting the system down (because the whole lateral is probably getting out of alignment with the field)
- If the cable is broken for any reason, this should also shut the system down because the fingers will lose physical contact
- If the lateral gets out of alignment with the field and shuts off, it will be necessary to back up one side and or move the other side forward until it is in the correct position
- This can involve electrical “jumps” between contacts in the control box, but some manufacturers and some installers put manual switches in just for this purpose
- Some linear moves are fitted with spray nozzles on drop tubes or booms
- If they are spaced closely along the lateral, it may be necessary to put booms out beyond the wheels at tower locations, either in back of the lateral or on both sides of the lateral

Water Supply

- Water is usually supplied to the lateral via:

    1. a concrete-lined trapezoidal-sectioned ditch, or
    2. a flexible hose (often 150 m in length), or
    3. automatic hydrant coupling devices with buried mainline

- Hose-fed systems require periodic manual reconnection to hydrants on a mainline – it is kind of like a period-move system, and you have to ask yourself whether the linear move machine is worth the cost in this case!
- With the automatic hydrant coupling machines (see Fig. 15.3) there are two arms with pipes and an elbow joint that bends as the linear move travels down the field. The two arms alternate in connecting to hydrants so as not to disrupt the irrigation nor the forward movement of the machine. These are mechanically complex.
- The advantage of hose-fed and automatic coupling linear moves is that you don't need to have a small, uniform slope in the direction of travel, because water is supplied from a pressurized mainline instead of an open channel
- On ditch-fed systems there can be a structure at the end of the field that a switch on the linear move contacts, shutting down the pump and reversing the direction of movement so that it automatically returns to the starting end of the field.
- The advantages and disadvantages of the ditch-feed system are:

<u>Pros</u>

- Low pressure (energy) requirement
- Totally automated system
- More frequent irrigations than hose-fed, since no one needs to be available to move the hose

<u>Cons</u>

- Trash and seeds and sediment pass through screen and may plug nozzles
- The pump must be on (move with) the lateral, causing extra weight
- Should have uniform slope along the lateral route

Pros and Cons Compared to a Center Pivot

<u>Pros</u>

- Easy irrigation of a rectangular field (important if land is expensive, but not important if land is cheap and water is scarce)
- Application rate is uniform over length of lateral, rather than twice the average value at the end of the center pivot
- No end gun is required

<u>Cons</u>

- The lateral does not end up right back at the starting point immediately after having traversed the irrigated area – you either have to "deadhead" back or irrigate in both directions
- May be more expensive than a pivot due to extra controls, pump on ditch feed, or more friction loss in the flexible feed hose (the hose is fairly expensive)

II. System Costs

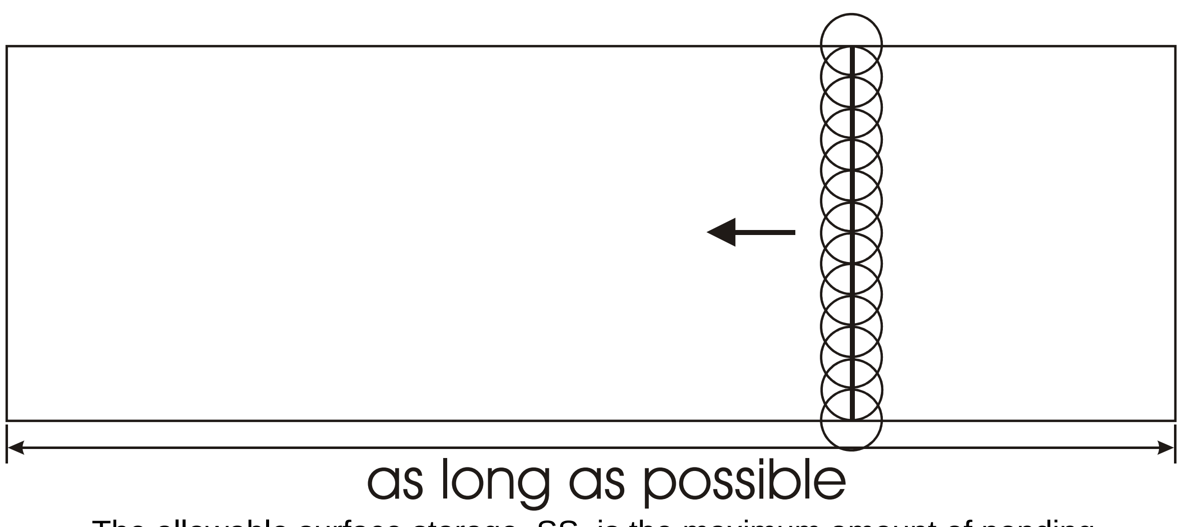
Relative costs for linear move systems are:

- $50,000 for a 1,280-ft hose-fed machine (perhaps 160 acres)
- $55,000 for a winch-tow hose fed 1,280-ft machine (perhaps 160 acres)
- $55,000 for a center pivot (1,280 ft) with a corner system (about 150 acres)
- $140,000 for a ½-mile (2,560-ft) linear with automated hydrant coupling system (no ditch or hose required). The mainline is down the middle (a ¼-mile lateral on each side). Perhaps 320 acres or more irrigated.

III. System Design

- A main strategy in linear move design is to minimize the cost per unit area. This is done by maximizing the area covered per lateral (length of field)
- Generally, the lateral length is limited to 400 to 800 m. Therefore, the major difficulties and objectives in linear design are to:

  1. Maximize the irrigated area per lateral (this minimizes $/area). In other words, how large a field can be irrigated by one machine?
  2. Prevent runoff by matching AR_x with $I_{soil} + SS/t_i$ (this tends to limit the field length, because if AR is small, it won't be possible to finish in f' days), where SS is the *allowable surface storage* in (mm or inches); and, t_i is the *time of irrigation*
  3. Determine whether spray nozzles can be used without causing runoff
  4. Minimize labor (for moving hoses and supervising)



- The allowable surface storage, SS, is the maximum amount of ponding without incurring surface runoff
- SS is a function of the general topography and the microtopography, and of the amount of foliar interception (water can "pond" on the crop leaves too)

- SS is usually less than about 5 mm unless small basins are created along furrows, for example
- AR_x limits the field length because it corresponds to some <u>minimum</u> time to finish an irrigation, for a given gross application depth, whereby a <u>maximum</u> interval (f) is calculated in the preliminary design steps

Lateral Inlet Head

- This is the same as for periodic-move systems
- The pressure balance equation for linear move systems is similar to set systems (both are linear, with uniform discharge from each outlet).

$$H_l = \frac{P_a}{\gamma} + \frac{3}{4}h_f + H_r + \frac{1}{2}\Delta H_e + \left(h_f\right)_{minor} + \left(h_f\right)_{hose} \tag{298}$$

- Or, if using flow control nozzles, with a minimum pressure required at the end (assuming the minimum pressure occurs at the end):

$$H_l = \frac{P_{end}}{\gamma} + h_f + H_r + \Delta H_e + \left(h_f\right)_{minor} + \left(h_f\right)_{hose} \tag{299}$$

where H_r is the height of the lateral or spray boom above the ground; and, $(h_f)_{minor}$ are the hydrant coupler and tower connection losses.

- The parameter $(h_f)_{hose}$ is the loss in the flexible hose connection on a hose-fed system
- Note that $(h_f)_{hose}$ may be a major loss, since the hose diameter is usually less than 5" or 6".

$$h_f = k_h F L \left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \tag{300}$$

where k_h = 10.50 for h_f and L in ft, Q in gpm, and D in inches; $K_h = 1.21(10)^{10}$ for h_f and L in m, Q in lps, and D in mm. F is the multiple outlet friction factor for a linear move ($F \approx 0.36$).

- For hose-fed systems, the maximum hose length for dragging the hose is 220 ft. Therefore, there could be about 400 ft between hydrants.
- For hose-fed systems with a cable/winch system for assisting in dragging the hose (towers only have a moderate amount of tractive power), the maximum hose length is 330 feet (640 feet between hose hydrants).
- Flexible hoses normally come in 5-inch (\$18/ft) and 6-inch (\$25/ft) diameters
- The Hazen-Williams C value for the hose can usually be taken as 150

Lecture 15

Maximizing Linear Move Field Length

I. The Procedure

- The following procedure for maximizing field length is from Allen, 1983, Univ. Idaho and Allen, 1990 (Irrig. Symp. Paper), and is used in the USUPIVOT computer program
- The basic strategy is to examine different application depths and different w values to maximize the area covered by the sprinkler system, and or to minimize labor requirements

1. Calculate the maximum application depth per irrigation ($d_x \le MAD*Z*W_a$). Note that the maximum application depth may be less than $MAD*Z*W_a$ with an automatic system to maintain optimal soil water conditions and to keep soil water content high in case of equipment failure (i.e. don't need to take full advantage of TAW)

$$f' = d_x/U_d \text{ (round down to even part of day)}$$

2. Calculate net and gross application depths:

$$d_n = f\,(U_d)$$

$$d = d_n/E_{pa}$$

3. Calculate the (presumed) infiltrated depth per irrigation:

$$(D_f)_{max} = d \cdot R_e$$

where $(D_f)_{max}$ is the maximum depth to be evaluated, and assuming no runoff

4. For a series of 10 or so infiltration depths, d_f, beginning with d_f equal to some fraction (say 1/10) of $(D_f)_{max}$:

$$d_f = (i/10)(D_f)_{max} \quad \text{where } i = 1 \text{ to } 10$$

and,

$$f' = d_f\, DE_{pa}/(100\, U_d)$$

$f = f'$ - days off (days off may be zero because the system is automatic), where f' = irrigation frequency for depth d_f. DE_{pa} is used here (in percent) because U_d is net, not gross

5. Determine the maximum AR_x for a particular d_f value using the following two equations (assuming an elliptical pattern):

$$AR_x = \frac{\left(1 - \frac{(SF)(AR_x)}{k}\right)\left(k^{\frac{1}{n+1}}(n+1)^{\frac{n}{n+1}}(D - SS - c)^{\frac{n}{n+1}}\right)}{\sqrt{1.05 - 1.6\left(\frac{\pi}{2}\right)^2\left(\frac{D}{d_f} - 0.5\right)^2}} \qquad (301)$$

where,

$$D = \left(\frac{\left(1.05\,AR_x^2 - 1.6\,AR_x^2\left(\frac{\pi}{2}\right)^2\left(\frac{D}{d_f} - 0.5\right)^2\right)^{-0.5}\left(-1.6\,AR_x^2\left(\frac{\pi}{2}\right)^2\left(\frac{D}{d_f} - 0.5\right)\right)}{d_f\, n\left(1 - \frac{(SF)(AR_x)}{k}\right)\left((n+1)^{\frac{-1}{n+1}}\right)\left(k^{\frac{1}{n+1}}\right)}\right)^{-n-1}$$
$$+ SS + c \qquad (302)$$

and AR_x is the peak application per pass (mm/min); D is the applied depth at time t = ∫ (AR) dt (mm); SS is the allowable surface storage (after ponding) before runoff occurs (usually less than about 5 mm); c is the instantaneous soil infiltration depth, from SCS soil intake families (mm); k is the coefficient in the Kostiakov-Lewis equation; and d_f is the total depth of water applied to the ground surface (mm)

- The parameter "n" is defined as: n = a -1, where "a" is the Kostiakov exponent (see NRCS soil curves at www.wcc.nrcs.usda.gov/nrcssirrig)
- Note that SS is a function of the field topography and micro-topography, and is affected by foliar interception of applied water
- These last two equations have π in them because there is an inherent assumption of an elliptical water application profile from the sprinklers or sprayers
- Recall that $AR_{av} = (\pi/4)AR_x$ for an elliptical pattern
- SF is a relative sealing factor (in terms of soil water infiltration), and may have values in the range of 0 to about 0.36

- The higher values of SF tend to be for freshly tilled soils, which are generally most susceptible to surface sealing from the impact of water drops
- Lower values of SF are for untilled soils and vegetative cover, such as alfalfa or straw, which tend to reduce the impact of water drops on the soil and help prevent runoff too
- If the linear move irrigates in both directions (no deadheading), then d_f is one-half the value from these two equations

6. Compute the total wetting time, t_i, in minutes:

$$t_i = \frac{d_f}{\frac{\pi}{4}\left(AR_x\right)} \tag{303}$$

7. Compute the speed of the system for the required t_i:

$S = w/t_i$ (m/min)   (w is for a specific nozzle type)

If $S \geq S_{max}$ (this may occur for a high intake soil or for a very light application with surface storage) then reduce the application rate and increase time as follows:

$$t_i = \frac{w}{S_{max}} \tag{304}$$

$$AR_x = \frac{4\,d_f}{\pi\,t_i} \tag{305}$$

Thus,

$$S = S_{max} \tag{306}$$

8. Calculate maximum field length, X:

8(a). For irrigation in one direction, only (dry return, or deadheading):

$$X = \frac{60\,f\,T - 2\,t_{reset}}{\left(\frac{1}{S_{wet}} + \frac{1}{S_{dry}} + \frac{t_{hose}}{100}\right)} \tag{307}$$

where,

X = maximum length of field (m);
f = system operating time per irrigation (days);
T = hours per day system is operated (21-23);
t_{reset} = time to reset lateral at each end of the field (min);
t_{hose} = time to change the hose (min/100 m);
S_{wet} = maximum speed during irrigation (m/min); and
S_{dry} = maximum dry (return) speed (m/min)

$$\text{labor} = \frac{2\,t_{reset} + 0.01X\left(t_{hose} + 2\,t_{super}\right)}{60\,f} \tag{308}$$

where labor is in hrs/day; and t_{super} is minutes of supervisory time per 100 m of movement

8(b). For irrigation in both directions (no deadheading):

$$X = \frac{60\,f\,T - 2\,t_{reset}}{2\left(\frac{1}{S_{wet}} + \frac{t_{hose}}{100}\right)} \tag{309}$$

and labor is calculated as above in 8(a)

9. Calculate the irrigated area:

$$\text{Area}_{max} = \frac{XL}{10,000} \tag{310}$$

where Area_{max} is in ha; and L is the total lateral length (m)

10. Labor per hectare per irrigation, L_{ha}:

$$L_{ha} = \frac{\text{labor}}{\text{Area}_{max}} \tag{311}$$

11. Repeat steps 5-10 for a different value of d_f

12. Repeat steps 4-11 for a new w (different application device or different operating pressure)

13. Select the nozzle device and application depth which maximizes the field length (or fits available field length) and which minimizes labor requirements per ha

14. System capacity:

$$Q_s = \frac{\pi AR_x wL}{4k_3 R_e} \quad (312)$$

where k_3 = 96.3 for L and w in ft, Q_s in gpm, and AR_x in in/hr; and k_3 = 60 for L and w in m, Q_s in lps, and AR_x in mm/min

The system capacity can also be computed as:

$$Q_s = \frac{d_f wL}{t_i k_3 R_e} \quad (313)$$

II. Assumptions & Limitations of the Above Procedure

- In the above procedure (and in the USUPIVOT computer program), when designing for a system which irrigates in both directions, the second pass is assumed to occur immediately after the first pass, so that the infiltration curve is decreased due to the first pass before the AR_x of the second pass is computed
- This will occur near the ends of the field, where the design is most critical. The proposed procedure assumes that:

  - There is no "surge" effect of soil surface sealing due to a brief time period between irrigation passes (when irrigating in both directions)
  - The infiltration curve used represents soil moisture conditions immediately before the initiation of the first pass
  - The infiltration curve used holds for all frequencies (f) or depths (d_f) evaluated, while in fact, as f↑, θ↓, so that the Kostiakov coefficients will change. Therefore, the procedure (and field ring infiltration tests) should be repeated using coefficients which represent the Kostiakov equation for the soil moisture condition which is found to be most optimal in order to obtain the most representative results.

Linear Move Design Example

I. Given Parameters

- Hose-fed linear move, irrigating in only one direction in a 64-ha field (400 m wide and 1,600 m long)
- The pressure is 140 kPa (20 psi) for spray booms with a preliminary w if 10 m (33 ft)
- The soil infiltration characteristics are defined for the Kostiakov-Lewis equation as:

$$Z = 5.43\tau^{0.49} \tag{314}$$

with Z in mm of cumulative infiltrated depth; and τ is intake opportunity time in minutes. Other design parameters:

U_d = 7.7 mm/day
MAD = 50%
Z = 0.9 m
W_a = 125 mm/m

O_e = 1.00
R_e = 0.94
E_{pa} = 85%

- Maximum dry (returning) speed = 3.5 m/min
- Maximum wet (irrigating) speed = 3.0 m/min
- Reset time = 0.5 hours per end of field
- Hose Reset time = 10 min/100 m of travel distance
- Supervisory time = 5 min/100 m of travel distance

II. One Possible Design Solution

- This design will consider only spray booms with w = 10 m
- Note that the full procedure would normally be performed with a computer program or spreadsheet, not by hand calculations

1. Calculate the maximum application depth per irrigation [d_x = MAD(z)(W_a), or less]

$$d_x = (0.5)(0.9)(125) = 56 \text{ mm} \tag{315}$$

$$f' = \frac{d_x}{U_d} = \frac{56}{7.7} = 7.3 \Rightarrow f' = 7 \text{ days} \tag{316}$$

2. Net and gross application depths:

$$d_n = f\,U_d = (7)(7.7) = 54 \text{ mm} \tag{317}$$

$$d = \frac{d_n}{E_{pa}} = \frac{54}{0.85} = 64 \text{ mm} \tag{318}$$

3. Infiltrated depth at each irrigation:

$$D_f = dR_e = (64)(0.94) = 60 \text{ mm} \tag{319}$$

4. For a series of 10 infiltration values, calculate d_f, beginning with $d_f = D_f$ /10:

$$d_f = D_f\left(\frac{i}{10}\right) \tag{320}$$

where i = 1 to 10. For this example, let i = 4 and, d_f = (0.4)(60 mm) = 24 mm. Then,

$$f' = \frac{d_f\,DE_{pa}}{U_d} = \frac{(24)(0.85/0.94)}{7.7} = 2.8 \text{ days} \tag{321}$$

Assume no days off (no down time during the peak use period)

$$f = f' - \text{days off} = 2.8 - 0 = 2.8 \text{ days} \tag{322}$$

5. Determine the maximum AR_x for the particular d_f depth:

From Eq. 282:

AR_x = 0.97 mm/min

AR_x reaching the soil surface = 0.97 (R_e) = 0.91 mm/min

6. Compute the total wetting time, t_i, in minutes:

$$t_i = \frac{4\,d_f}{\pi\,AR_x} = \frac{4(24)}{\pi(0.91)} = 34 \text{ min} \tag{323}$$

7. Compute the speed of the system for the required t_i:

$$S = \frac{w}{t_i} = \frac{10}{34} = 0.3\ \text{m/min} \tag{324}$$

Thus, S < S_{max} (3.0 m/min), so this is OK.

8. Calculate maximum field length, X:

For irrigation in one direction, only (deadhead back):

$$X = \frac{60\,f\,T - 2\,t_{reset}}{\left(\frac{1}{S_{wet}} + \frac{1}{S_{dry}} + \frac{t_{hose}}{100}\right)} = \frac{60(2.8)(22) - 2(30)}{\left(\frac{1}{0.3} + \frac{1}{3.5} + \frac{10}{100}\right)} = 970\ \text{m} \tag{325}$$

and, the labor requirements are:

$$\frac{2\,t_{reset} + 0.01\left(t_{hose} + 2\,t_{super}\right)X}{60\,f} = \frac{2(30) + 0.01[10 + 2(5)][970]}{60(2.8)} = 1.5\ \text{hrs/day} \tag{326}$$

where t_{reset} is the reset time at the end of the field (min); t_{hose} is the hose reconnection time (min/100 m); and t_{super} is the “supervisory” time (min/100 m)

9. Maximum irrigated area:

$$Area_{max} = XL/10000 = 970(400)/10000 = 38.8\ \text{ha}$$

which is only about half of the actual field area!

10. Labor per ha per irrigation, L/ha:

$$L/ha = (labor/area)_{max} = 1.5/38.8 = 0.039\ \text{hr/ha/day}$$

11. Repeat steps 5 - 10 for a new d_f (not done in this example)

12. Repeat steps 4-11 for a new w (different application device or operating pressure). (not done in this example).

13. Select the nozzle, device and application depth that maximizes the field length (or fits the available field length), and which minimizes labor requirements per ha.

Note: 38.8 ha << 64 ha, which is the size of the field, (970 m << 1600 m which is the length of the field). Therefore, it is important to continue iterations (steps 11 and 12) to find an application depth and or new w (different sprinkler or spray device) to reach 1600 m and 64 ha, if possible.

Additional Observations:

- For a 6-m spray boom, applying a 12-mm depth per each 1.4 days would almost irrigate the 64 ha. However, the labor requirement is doubled, as the machine must be moved twice as often. This additional cost must be considered and weighed against the larger area irrigated with one linear move machine.
- If larger spray booms were used (w = 16 m rather than 10 m) (these would be more expensive) then 18 mm could be applied each 2.1 days, and all 64 ha could be irrigated with one machine.
- If low pressure impact sprinklers were used (these would be less expensive than spray booms, but energy costs would be higher), then w = 22 m, and 30 mm could be applied each 3.5 days (more water can be applied since the application rate is spread over a wider area from the lateral), and all 64 ha could be irrigated. In addition, ET_c would be less since the soil would be wetted less often. Also, the soil intake rate would be higher each irrigation because of a drier antecedent moisture at the time of irrigation.
- Notice that required wetting time for rotation times (f) greater than 2 days are identical between all types of spray devices. This is because, for the large depths applied, a minimum wetting time is required. The system speed is adjusted to fit the w value of the water application device.
- If no acceptable solution for this problem were found, then alternatives to be evaluated would be to irrigate in <u>both</u> directions, or to consider a ditch-fed linear move (this requires a leveled ditch, but does not required time for moving hoses and hose friction losses).
- You could also consider a "robot" controlled machine that automatically connects alternating arms to hydrants on a buried mainline (but this is a very expensive alternative)
- You might begin to wonder whether an investment in a linear move machine is justifiable when there is a significant labor requirement for reconnecting the supply hose, resetting at the end of the field, and

supervising operation. That is, why not put in a center pivot or a side roll system instead?

- If one linear move cannot cover the entire field length in the available period, "f" (days), you could consider two linear move machines for the same field

14. System Capacity:

$$Q_s = \frac{\pi A R_x w L}{4 k_3 R_e} = \frac{\pi (0.91)(10)(400)}{4(60)(0.94)} = 51 \text{ lps} \;\; (809 \text{ gpm}) \tag{327}$$

alternatively,

$$Q_s = \frac{d_f w L}{t_i k_3 R_e} = \frac{(24)(10)(400)}{(33.6)(60)(0.94)} = 51 \text{ lps} \;\; (809 \text{ gpm}) \tag{328}$$

Note that the computed Q_s is larger than one based strictly on U_d and T, because the machine is shut off during reset and hose moving

For Q_s based only on f, A, d and T, with no consideration for t_{hose},

$$Q_s = 2.78 \frac{A d}{f T} = 2.78 \frac{(38.3)(24)}{(2.8)(22)(0.94)} = 44 \text{ lps} \;\; (700 \text{ gpm}) \tag{16}$$

But this flow rate is too low – it does not consider hose moving and reset time. So, the 51 lps system capacity should be used for design

Lecture 16
Trickle Irrigation System Components & Layout

I. Introduction and Descriptions

- “Trickle” and “drip” are terms used to describe what can be generally called “micro-irrigation systems”, in which water is applied in relatively precise quantities and precise times and at precise locations
- Land-leveling costs notwithstanding, trickle irrigation systems are usually the most expensive types of on-farm water application system to install
- They can also be expensive to operate and maintain
- Usually, trickle irrigation systems are installed in areas where water is scarce and or expensive, crop value is very high, or topographical and other conditions might preclude the successful use of other types of irrigation systems
- Not all micro-irrigation systems are complex and expensive
- Labor-intensive forms of micro-irrigation continue to be practiced in many areas of the world, especially for vegetable and other “cash” crops
- For example, people may carry water in buckets or shoulder harnesses to carefully pour at each plant in a field
- Or, porous pots are buried at regular intervals along rows and filled with water individually, which seeps out into the surrounding soil
- Sometimes water is merely splashed onto crop beds by hand

II. Advantages and Disadvantages of Trickle Systems

<u>Advantages</u>

1. Significant water, fertilizer, and operating cost (labor and power) **savings** are *possible*
2. Ease of field **operations** due to reduced weed problems and non-wetted soil surface (e.g. strawberries)
3. Ability to apply **saline water** because of frequent (daily) irrigation; thus, soil water salinity is nearly the same as the irrigation water salinity
4. Ability to operate on **steep slopes** and rough terrain
5. The ratio of crop yield to evapotranspiration can be higher under trickle irrigation because of reduced soil surface evaporation, continuously high soil water (near FC), and lower root zone salinity due to frequent application
6. Relatively easy to automate the system
7. Can be less labor intensive than some other irrigation systems

Disadvantages

1. Systems are **expensive** to purchase and install ($1,000 to $6,000 per ha)
2. Susceptibility to **clogging** of emitters, which usually have very small openings – so, it is important to spend time and money on maintaining the system, applying chemicals, and keeping filters clean
3. Possibly low distribution **uniformity** due to low operating pressures and possibly due to steep slopes, especially along laterals, and due to clogging
4. Where laterals are on steep slopes, the water will drain out the downhill end at every startup and shut-down.
5. Soils with very low intake rates will exhibit ponding and **runoff**
6. **Salt** tends to accumulate at the soil surface and around the wetted area -- when it rains, these accumulated salts may be driven into the root zone
7. These systems tend to require more capable and diligent **management** because of the susceptibility to clogging, and because the systems are usually designed to operate continuously during peak ET periods (can't afford to let the system shut down during these periods). These systems do not usually take full advantage of the soil storage (buffer) capacity.

III. Trickle System Components

- The following is a list of many of the common components found in modern trickle irrigation systems:

    1. Pump & motor
    2. Control head
        - Valves
        - Filters and Screens
        - Chemical Injection Equipment
        - Flow Rate or Volume Meters (U/S of acid injection)
    3. Mainline
        - Submains
        - Manifolds
        - Laterals
    4. Water applicators
        - Emitters
        - Bubblers
        - Sprayers
        - Misters
        - Others

bubbler

5. Other equipment
   - Valves & air vents
   - Vacuum Relief Valves
   - Pressure Relief Valves
   - Various Pipe Fittings and Appurtenances

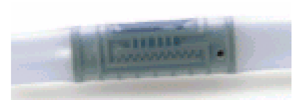
- Not all trickle irrigation systems will have all of these components
- For example, some systems are gravity-fed and require no pumping
- Simple systems may not have submains and manifolds
- Some systems do not have pressure relief or other types of safety valves
- Systems with relatively dirty water will have multiple levels of filtration, others may have only minimal screening

IV. Types of Water Applicators

- The basic purpose of water applicators in trickle irrigation systems is to ***dissipate energy***
- This is because lateral pressures must be high enough to provide adequate uniformity, yet emitters must yield small flow rates
- Otherwise, one could simply punch holes in a plastic pipe (in some cases this is exactly what people do)

1. Drip Emitters

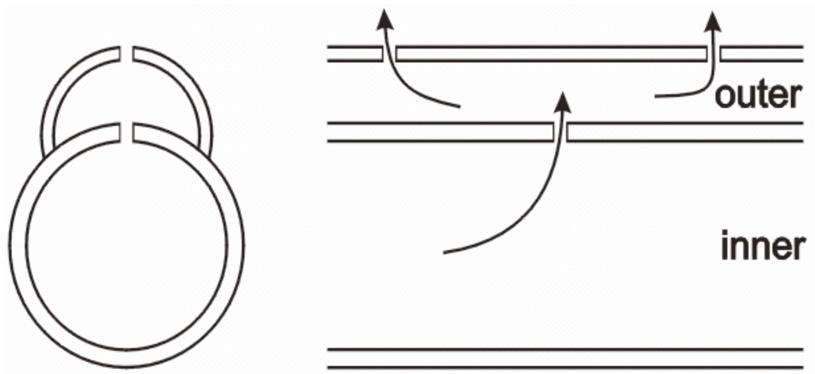
- Emitters are the typical water application device in many systems
- Can be "on-line" (usually with barb) or "in-line" (inserted into tubing by the tubing manufacturer)
- Various approaches are used to provide energy dissipation: long-path ("spaghetti tubing"), tortuous path (labyrinth), orifice, vortex,
- Some emitters have flushing, or continuous flushing features to help prevent clogging while still having small flow rates (some designs are very clever)
- Some emitters have pressure compensating features to provide a more constant flow rate over a range of operating pressures
- Some emitters are designed with multiple outlets



2. Line Source Tubing

- These are typically buried
- Single chamber with small orifices

- Double chamber with orifices between chambers and orifices to discharge water into the soil (acts something like a manifold to control pressures and provide greater uniformity)



- Can be removed and reused next year (typically 4-5 years life)
- Can be "disked up" and left in the field as chunks of plastic
- Porous or "leaky" pipe, made from old tires or new materials

3. Micro-Sprayers

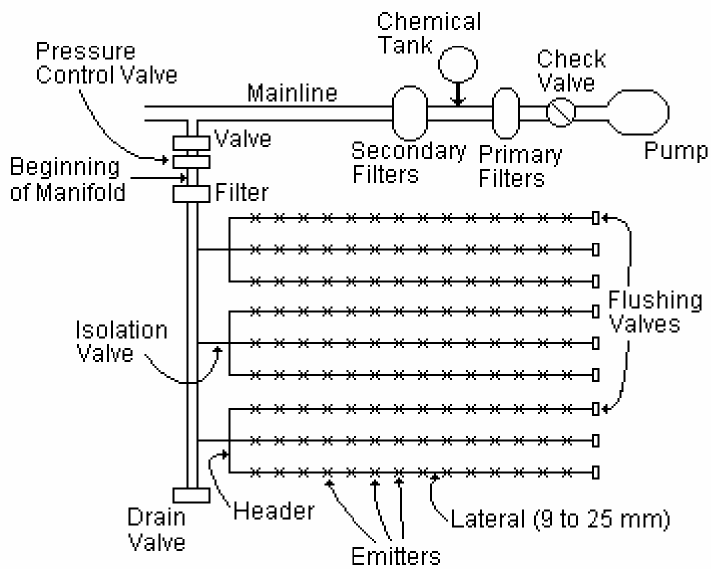
- These are like small sprinklers, but may not overlap enough to wet the entire ground surface
- Sometimes referred to as "spitters"
- A "gray area" between micro and sprinkle irrigation – they have both precise application and aerial + soil distribution of water
- Larger wetted area per applicator, compared to non-spray emitters
- Less susceptible to clogging than most emitters

V. Types of Pipe Materials

- Laterals are usually polyethylene (PE)
- Laterals may be made from polybutylene (PB) or PVC
- Metal pipes are not used because of corrosion problems from injected chemicals
- Non-buried lateral tubing should be black to discourage growth of algae and other organic contaminants (don't allow light to enter the tubing)
- Non-buried pipes preferably have ultraviolet light (UV) protection to prevent rapid deterioration from exposure to sunlight

VI. Typical Trickle System Layouts

- Chemical injection should be upstream of filters to prevent system clogging
- Chemical tank and valves are often plastic to avoid corrosion and freeze-up
- A check valve will help prevent contamination of the water supply
- Manifolds are often the basic operational subunits
- Valves for flushing at ends of manifolds and laterals are often manually operated
- Various mainline-manifold-header arrangements are possible



VII. Emitter Flow Rate versus Pressure

- The discharge equation for emitters is similar to that used for sprinkler nozzles, but the exponent on the head or pressure term is variable
- An exponent of ½ corresponds to orifice flow, which is how some, but not all, emitters are designed
- The general emitter equation is:

$$q = K_d H^x \tag{329}$$

where q is the volumetric flow rate; K_d is the discharge coefficient; H is the head (or pressure); and x is the exponent

- Note that the value of K_d is different depending on whether units of head or pressure are used in the equation
- The exponent in the above equation is a function of the emitter design

- For purely turbulent orifice flow the exponent is ½, assuming the pressure head is fully converted to velocity head
- Pressure compensating emitters have $x \leq$ ½. A fully pressure compensating emitter would have x = 0 (but these don't really exist)
- Long-path, laminar-flow emitters typically have $x \approx 0.7$ (if the flow were completely laminar, the exponent would be 1.0)
- Vortex-type emitters typically have $x \approx 0.4$

VIII. Emitter Design Objectives

- The two basic emitter design objectives, other than energy dissipation, are:

  1. Have a low value of the exponent, x
  2. Have flushing properties to prevent clogging

- The first objective provides for pressure compensating features, if the exponent is less than 0.5 (i.e. compensates better than a simple orifice)
- The first and second objectives tend to be conflicting, because the more pressure compensating the less ability to flush particles

IX. Field-Measured Uniformity

- To measure emission uniformity in the field you can use an equation equivalent to the Distribution Uniformity (DU), as applied to sprinkler systems:

$$EU' = 100\left(\frac{q_n'}{q_a}\right) \tag{330}$$

  where EU' is the field test emission uniformity; q_n' is the average discharge of the low ¼ emitters from the sampling; and q_a is the average discharge of all emitters sampled

- EU' should be at least 95% for properly designed and properly maintained trickle irrigation systems
- Note that it is impossible to calculate EU' based on field measurements if the system is being designed (hasn't been installed yet) – in this case there are other equations to approximate EU (recall the *design efficiency* for sprinkler systems)
- Most nonuniformity in micro irrigation systems is caused by: (1) emitter plugging, wear, and manufacturing variations; and, (2) nonuniform pressure distribution in pipes and hoses

X. Manufacturer's Coefficient of Variation

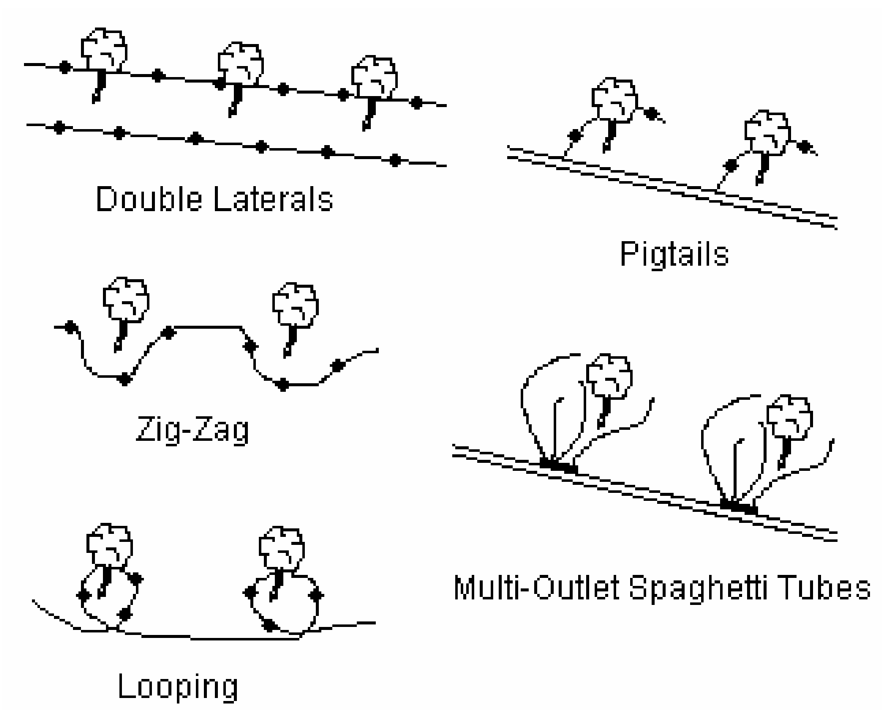
- Emitters of the same type and manufacture have variations in discharge (at the same operating pressure) due to small differences from manufacturing tolerances. Some variation is allowed in the interest of cost savings.
- The *manufacturer's coefficient of variation* is defined as:

$$\nu = \frac{s}{q_a} \tag{331}$$

where the standard deviation, s, is calculated from at least 50 samples (all at the same pressure) of the same emitter type and model; and the denominator is the average discharge of all emitters

XI. Emitter Layouts

- There are a number of configurations designed to increase the percent wetted area, and still be economical
- There are tradeoffs between flow per lateral and total length of pipe and tubing
- Below are some common emitter layouts



XII. Valves & Automation

- Valve automation can be accomplished with individual timers/devices, or with a central controller

1. *Volumetric Valves*

    - Manually turned on
    - Automatically turn off

2. *Sequential Operation*

    - Manually turned on
    - Automatic sequencing from low to high elevation

3. *Fully Automatic*

    - Time-based: soil moisture sensors determine whether to turn the system on at a given time each day, then the system runs for fixed duration in each subunit
    - Volume-based: uses a flow meter to measure a specified volume delivered to a subunit, then turns water off to that subunit
    - Soil moisture-based: uses tensiometers, resistance blocks, or other device to determine when to irrigate and for how long

- Note that time-based systems may give varying application depths over time if the system flow rate changes due to clogging of filters
- This can be partially corrected by using pressure compensating emitters
- However, the use of a volume-based system with a flow meter may be best because the flow rate measurement also gives an indication about filter clogging

Lecture 17

Filtration for Trickle Irrigation Systems

I. Introduction

- Water for trickle irrigation systems can come from open reservoirs, canals, rivers, groundwater, municipal systems, and other sources
- Solid contaminants can include both organic and inorganic matter
- Examples of inorganic matter are sand, silt and clay (soil particles), and trash floating in the water
- Examples of organic matter are bacteria, algae, moss, weeds & weed seeds, small fish, insect larvae, snails, and others (see the figure below)

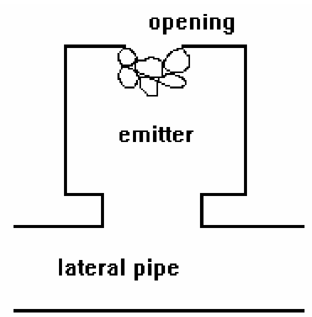
- Solid contaminants need to be removed from trickle irrigation systems because they:

  1. Cause clogging of emitters, which can lead to serious water deficits and non-uniformity of the applied water
  2. May cause wear on pump impellers, emitter outlets, and other hardware

3. Can provide nutrients which support the growth of bacteria in the pipes
4. Can accumulate at the ends of pipelines and clog valves
5. Can contain weed seeds which aggravate weed control in the irrigated area
6. Cost the farmer money

- All of the above problems translate to direct costs to the farmer

- These filters do not remove salts from the irrigation water (unless reverse osmosis membranes are used, which are very uncommon in irrigation systems and are not covered here)



- Groundwater usually requires less filtration than surface water, but even groundwater should be filtered
- The maximum allowable particle size in trickle irrigation water is usually between 0.075 mm and 0.2 mm, so the water must be quite clean
- Filtration is almost always complemented by the injection of various chemicals into the water to help prevent clogging due to bacterial growth and precipitation of solids from the water
- Solid particles smaller than emitter outlets can cause clogging when they bridge at the opening (see the figure above)
- Some consultants recommend the removal of all particles larger than 1/10 of the minimum outlet diameter for drip emitters, or about 1/7 of the minimum outlet diameter for spitters, misters, and microsprayers
- Larger particles may be allowed with spitters, misters, and microsprayers because of "shorter pathways" and sometimes larger openings

II. Types of Filtration

- The basic types of filtration used in trickle systems are:

1. Reservoirs (settling ponds)
2. Pre-screening devices
3. Sand separators
4. Sand (*media*) tanks
5. Gravity overflow screens
6. Tubular screens
7. Disc filters

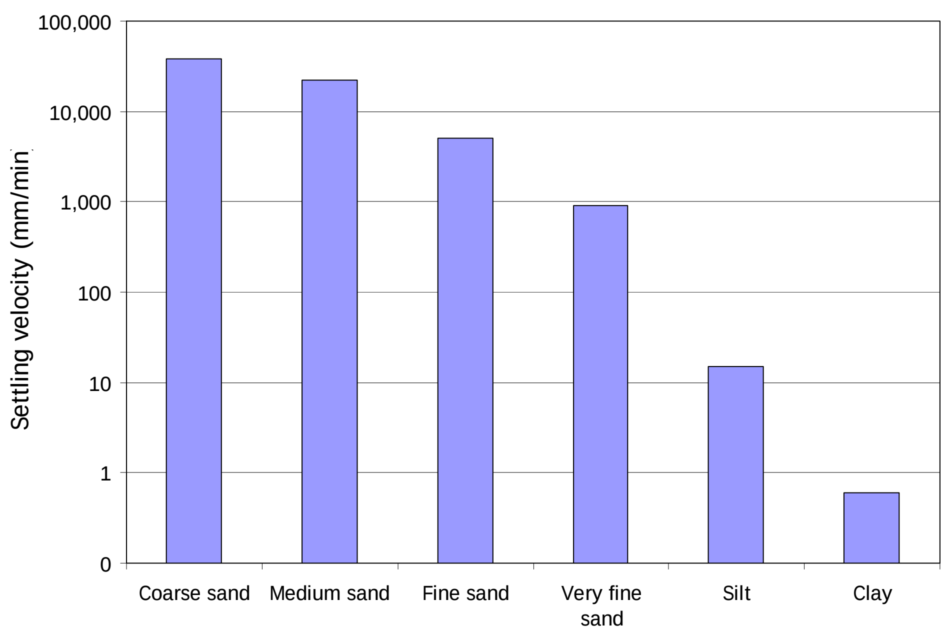
III. Use of Reservoirs in pre-Filtration

- Some of the benefits of an open reservoir upstream of the pumps in a trickle irrigation system are:

1. To buffer differences in supply and demand rates. The supply from a canal or well seldom coincides exactly with the system requirements (flow rate and duration), and the system requirements can change due to different numbers of stations in operation, “down time”, and duration of sets.
2. To allow for settling of some of the suspended particles. In these cases the reservoir serves as a “settling basin”. Precipitated sediment can be periodically removed from the reservoir with equipment or manual labor.

Soil Texture	Particle Size (microns)	Vertical Settling Velocity (mm/min)
Coarse sand	> 500	38,000 (1½ mph)
Medium sand	250 - 500	22,000
Fine sand	100 - 250	5,000
Very fine sand	50 - 100	900
Silt	2 - 50	15
Clay	< 2	0.6 (very slow!!)

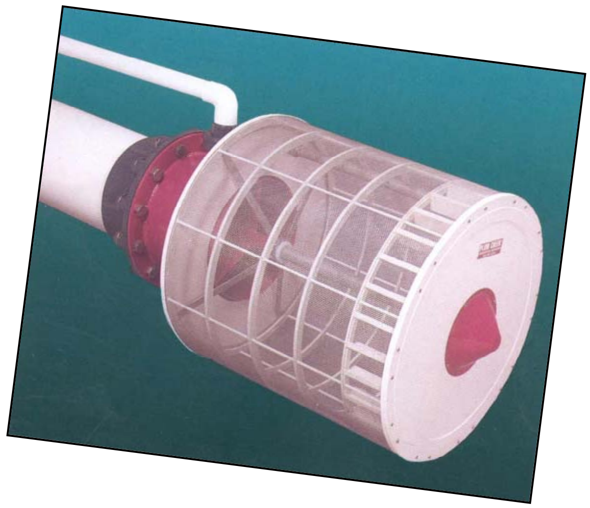
The barchart below has a logarithmic scale on the ordinate:



Longer settling basins will allow more time for suspended particles to fall to the bottom before arriving at the pump intake.

3. To aerate water pumped from wells, thereby oxidizing and precipitating manganese and iron out of the water (some groundwater has manganese and iron, and these can cause plugging of emitters). Only 1.5 ppm of either manganese or iron can cause severe clogging problems in trickle laterals and emitters (see Table 18.1 in the textbook).
4. To allow for air to escape when the water comes from a "cascading" well, in which air becomes entrained into the water. Air in pipelines can dampen the effects of water hammer, but also causes surges and blockages of flow.
5. To allow oils to collect on the water surface. Oils can cause rapid clogging of most types of filters, requiring special cleaning with solvents and possible replacement of sand media. When pumping from a reservoir the inlet is below the water, and oil does not enter.

IV. Pre-Screening Devices

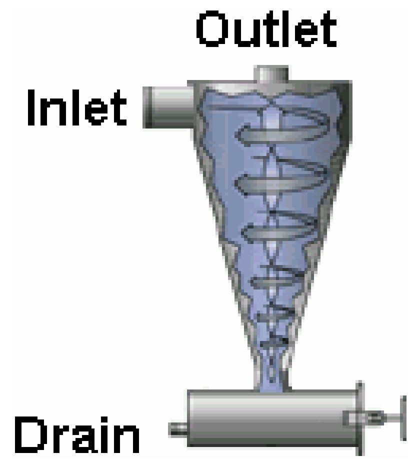


- These screens are intended to prevent fish, large debris and trash from entering the pipe system, upstream of the other filtration devices
- Pre-screening is not necessary for groundwater or municipal water supplies
- Pre-screening devices often have "self-cleaning" features, otherwise they can clog up rapidly
    - Horizontal grills
    - Screen plates
    - Rotating and self-cleaning screens
    - Gravity screen filters
- If the inlet line upstream of a pre-screening device is pressurized, you will lose all of the pressure and have to repressurize downstream of the screens – this is a major disadvantage to pre-screening in such cases

V. Sand Separators

- Sand separators are used to remove sand (but not organic matter) from the water
- Most work by spinning the water in an enclosed column (or cone) to remove sand through a centrifuge-type action
- There are no moving parts
- Solid particles with a density of approximately 1.5 g/cm^3 can be removed by these devices (most sand has a density of about 2.65 gm/cm^3)

- Can remove from 70 to 95% of dense particles
- Periodic purging of accumulated sand (manual or automatic) is necessary to maintain performance
- Must have the correct flow rate through the sand separator for proper operation, otherwise less sand will be removed from the water
- Most sand separators have a pressure loss of between 5 and 12 psi, from inlet to outlet. This pressure loss does not change with time, only with flow rate.
- Some sand separators are designed to fit down into wells to protect the impellers and pump bowls, but they are not as efficient as above-ground sand separators



- Sand separators cannot remove all of the sand, and may pass large amounts when the system is starting or stopping
- Therefore, screen filters should be installed downstream
- Sand separators are available but are not used as much as they were in the past because people are using media tanks and other filters instead
- When taking water from a deep well, an alternative to using a sand separator is to properly develop the well and use a good quality well screen

VI. Sand Media Filters

1. Introduction

- This type of filter is filled with sand, or some other particles such as crushed (makes it angular, traps debris better) granite, crushed silica material (e.g. garnet)
- Some designers go by a uniformity coefficient for the media, defined as the ratio of 40% retained size (larger) to the 90% retained size (smaller) from a sieve analysis
- This uniformity should usually be between 1.0 (perfect!) and 1.5
- In most media filters the water passes through vertically, from top to bottom
- A drain screen (of which there are various types) at the bottom allows clean water to exit, but prevents the media from leaving the filter tank.
- Many media filters have a layer of gravel around the bottom drain
- The inside top of the tank usually has some kind of “diffuser” to help spread the dirty water evenly over the media surface
- Standard tank sizes are 24, 36 and 48 inches in diameter, with a media depth of about 30 to 40 cm
- Tank walls are usually 3 - 5 mm thick, depending on the brand

- The tanks are often made out of carbon steel, type 304 stainless steel, or type 316 stainless steel (if water has high salt content)

- Every installation should have at least two tanks so that back-flushing can occur during operation, but many designers recommend at least three tanks in which only one is back-flushed at a time
- New media should be rinsed with clean water before placing it in the tanks because it may have dust and other particles in it
- Some tanks have not performed well when the installers failed to rinse the media first (resulting in fine particles passing into the irrigation system when the tanks are first put into use)

2. Applicability of Media Filters

- These filters are very good for removing relatively large amounts of organic and inorganic matter, but some pre-screening is usually necessary with surface water supplies
- High volume filtration at 20 to 30 gpm/ft^2 (1.3 to 2.0 cm/s)
- Some silt and clay particles can also be removed by sand media filters, but not by most screen-type filters. However, much silt and clay can pass through a media filter too.
- Large volumes of particle contaminants can be collected in the sand media before the media must be cleaned, or “back-flushed”
- In some cases the water must be pre-cleaned before entering the sand tanks to prevent rapid accumulation of particle contaminants
- Media filters can also remove some sand from the supply water, but this sand cannot always be effectively back-flushed from the media -- for large amounts of sand, there should be a sand separator upstream of the media tanks
- Industrial media filters are often five feet deep (or more), but have smaller flow rates and less frequent back-flushing than agricultural media filters, which may be only 14 inches deep
- Many of the particles captured by agricultural media filters stay within the upper few inches of the sand because they are back-flushed frequently

3. Back-Flushing the Tanks

- Back-flushing is required to clean the tanks
- Back-flushing can be performed manually or automatically, based on elapsed time and or on a pressure differential limit across the tanks
- Typical pressure differential triggers are 5 to 10 psi (35 to 70 kPa) greater than the pressure differential when the media is clean (clean media typically has a pressure differential of 3 to 5 psi, or 20 to 35 kPa)
- Often, a timer is set to back-flush at least one time per day, even if the pressure differential criteria (for flushing) is not met

- Automatic back-flushing is recommendable, because labor is not always reliable
- Some installations have view ports on the back-flush pipes so that an operator can see if the water is clean (to know if the duration of the back-flush is sufficient) and to see if any of the sand media is escaping during back-flush
- The pressure differential during a backflush operation should be 7 - 10 psi (50 - 70 kPa) – if it is greater than 10 psi, the flow rate is too high

VII. Secondary Filters

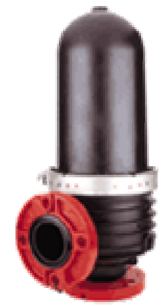
1. Tubular Screen Filters

- These are conventional screen filters, with two-dimensional surfaces and little capacity to accumulate debris
- There are many different kinds and variations of these filters
- Primarily used as backup (safety) filters downstream of the primary filters
- If the screen becomes dirty and is not cleaned, the pressure differential can become great enough to burst the screen. Or, the screen may stretch until the openings expand enough to pass some of the debris (which is not desirable)

- Flow through the filter is usually from inside to outside (debris is trapped on the inside surface during operation) to prevent collapse of the screens
- Cleaning can be manual or automatic, and there are many varieties of automatic cleaning methods

- Some filter designs have a rotating suction mechanism to clean the dirty (inner) side of the screen element
- Manually-cleaned filters can have slow or quick release cover latches -- the slow release latches are preferred because the quick release version can “explode” if opened while the system is at operating pressure (dangerous to personnel)

2. Disc Filters



- Similar to a tubular screen filter, but using tightly packed plastic disks for the filter media, with a deeper filter area
- Holds more contaminants than a regular screen filter without clogging
- Often installed in banks (several filters in parallel)
- Often have automatic back-flushing features, requiring higher pressure (about 45 psi minimum pressure) than normally available for system operation, so there is a special booster pump for cleaning
- Cleaning is often performed when the pressure differential across the filter reaches 6 psi
- The pressure differential for a clean filter should be about 1 to 4 psi (unless too much water is being pumped through the filter)
- These filters are not designed to remove sand from the water (sand gets stuck in the grooves)
- These filters can have clogging problems with some kinds of stringy algae

VIII. Chemical Injection for System Maintenance

- It is usually necessary to use chemicals to maintain a trickle irrigation system -- if not, the system will eventually become clogged
- Some chemicals are for the addition of plant nutrients, or fertilizers; this is called “fertigation”
- Different chemicals may be used for pest control: herbicides, insecticides, fungicides
- Other chemicals are used to kill bacteria and other organic contaminants, and maintain a sufficiently low pH
- The uniformity at which chemicals are applied to the field can be assumed to be equal to the emission uniformity of the emitters (assuming the chemicals are water soluble, which they should be if injected into the irrigation system)
- Following are guidelines for clogging hazard of irrigation water in trickle systems with emitter flow rates of 2 to 8 lph (after Bucks and Nakayama 1980):

Kind of Problem	Hazard Level		
	low	**moderate**	**severe**
suspended solids	50 ppm	50-100 ppm	> 100 ppm
pH	7.0	7.0-8.0	> 8.0
salts	500 ppm	500-2,000 ppm	> 2,000 ppm
bicarbonate	--	100 ppm	--
manganese	0.1 ppm	0.1-1.5 ppm	> 1.5 ppm
total iron	0.2 ppm	0.2-1.5 ppm	> 1.5 ppm
hydrogen sulfide	0.2 ppm	0.2-2.0 ppm	> 2.0 ppm
bacteria count	10,000/liter	10,000-50,000/liter	> 50,000/liter

- Types of clogging in trickle systems that can be managed through the injection of chemicals:

1. Slimy bacteria
   These can grow inside pipes and inside emitters. The chemicals used to kill this bacteria are chlorine, ozone, and acids.

2. Iron and manganese oxides
   Some kinds of bacteria can oxidize iron and manganese. Only small amounts of iron or manganese are necessary to support bacterial growth in the water. This can be treated with chlorine injection, acid and chlorine injection, linear phosphate injection, and aeration in a pond.

3. Iron and manganese sulfides
   These are problematic with some groundwater. Dissolved iron and manganese form a black, insoluble material. Manganese is toxic to most plants in small concentrations, and this may become a problem before clogging occurs. This can be treated by aeration, chlorination, and acid injection.

4. Precipitation of calcium and magnesium carbonates
   These are typically quantified by SAR (or adjusted SAR). You can use a dropper to put some hydrochloric or muriatic acid on selected drip emitters to test for the presence of carbonates and bicarbonates. If present, the acid will react and cause a “fizzing” sound and bubbles. One method to control precipitation of carbonates is to continuously inject carbon dioxide gas into the system, creating carbonic acid and lowering the pH. Sulfuric and phosphoric acids are also used, as are SO_2 generators. All of these are designed to maintain the pH near (or slightly below) 7.0.

5. <u>Plant root entry into underground emitters</u>
   This is mainly a problem in permanent (several years) buried trickle irrigation laterals. Can use acid injection at end of each season for perennials to kill roots that are in the buried drip tubing. Or, use herbicides to kill roots in the tubing without damaging the plants. Some emitters and plastic drainage pipe have herbicide in the plastic to discourage roots from entering.

- It is dangerous to experiment with chemical mixtures in trickle systems, because some mixtures can cause clogging of the emitters
- Test the mixture in a glass container first, checking it after several hours or more, and determining whether the chemical mixture is water soluble

- Injection of chemicals should be after the system starts, and stopping before the system is turned off
- As a rule of thumb, one can assume an average pipe flow velocity of 1 fps, or 0.3 m/s, divide this into the longest pipe distance in the system (from pump to farthest emitter), and determine the time
- This is the time to wait after starting the pump, and the time to allow for flushing before turning the pump off

For example, if the farthest emitter is 600 ft from the pump, the travel time can be estimated at 600 s, or 10 min. Thus, you should wait 10 min before beginning chemical injection, and discontinue chemical injection 10 min before stopping the system, or before changing stations.

- Chemicals should be injected on a mass basis per set, not time. Thus, one would want to apply a certain number of lbs or kg of a chemical in an irrigation set, and it does not matter that it is all applied quickly or over a long time (provided that the starting and stopping delay discussed above is adhered to)
- The <u>minimum</u> <u>injection</u> <u>rate</u> can be put into equation form:

$$q_c = \frac{F_r A}{c t_r T_a} = \frac{(\mathrm{kg/ha})(\mathrm{ha})}{(\mathrm{kg/liter})(0.8)(\mathrm{hrs/set})} = \mathrm{lph} \tag{332}$$

where F_r is the mass application rate per unit area; A is the area irrigated per set; c is the concentration of the chemical; t_r is some kind of uniformity ratio, taken to be 0.8; and T_a is the hours per set, or hours of chemical injection, if shorter than the set time

Lecture 18

Trickle Irrigation Planning Factors

I. Soil Wetted Area

- Trickle irrigation systems typically apply small amounts of water on a frequent basis, maintaining soil water near field capacity
- But, usually not all of the soil surface is wetted, and much of the root zone is not wetted (at least not by design) by the system
- Recall that the system is applying water to each individual plant using one or more emission points per plant

Widely-Spaced Crops

- These include orchards and vineyards, for example
- According to Keller & Bliesner, for widely-spaced crops, the percent wetted area, P_w, should normally be between 33% and 67%
- The value of P_w from the irrigation system can fall below 33% if there is enough rainfall to supplement the water applied through the trickle system
- Lower values of P_w can decrease the irrigation system cost because less emitters per unit area are required
- Lower values of P_w can allow more convenient access (manual labor & machinery) for cultural practices during irrigation
- Lower values of P_w can also help control weed growth in arid and semi-arid regions, and reduce soil surface evaporation
- Lower values of P_w carry the danger that the soil will dry to dangerously low levels more quickly in the event the irrigation system goes "off-line" for any reason (power failure, broken pipe, pump problems, labor shortage, etc.)
- With lower values of P_w, there is less storage of applied water in the root zone, especially with light-textured soils (sandy soils)
- With tree crops, low values of P_w can lead to "root anchorage" problems, in which root extension is insufficient to support the trees during winds

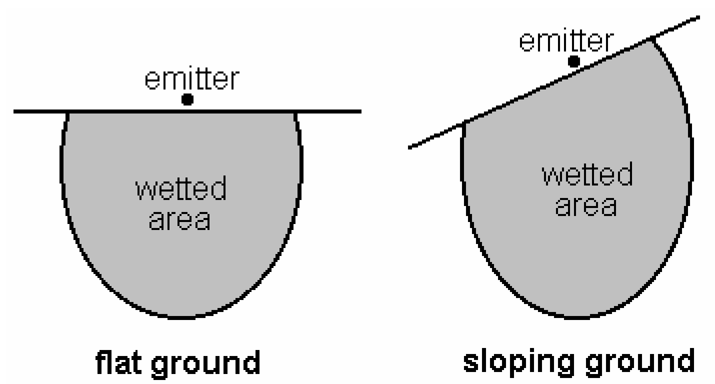
Closely-Spaced Crops

- These include most row crops
- Actual P_w values may be near or at 100% with row crops and subsurface drip irrigation systems (in the USA rows are typically spaced from 30 inches to 60 inches)
- Larger values of P_w usually mean more extensive root development, and enhanced ability for the plant to make use of any rain water that may come
- Figure 19.1 in the textbook shows a generalized relationship between P_w, amount of rainfall, and crop production level – the figure implies that maximum crop yield may be higher under a trickle irrigation system than with other methods

- Figure 19.1 indicates that 100% crop yield might be obtained, in general, with $P_w \geq 33\%$

<u>Wetted Soil Area, A_w</u>

- The wetted soil area, A_w, is not measured at the soil surface, but from a horizontal plane about 30 cm below the soil surface (actually, it depends on root depth and soil type)
- The same is true for P_w
- The reason we are interested in P_w is to calculate the application depth "d_x,” as discussed in the following lecture
- This wetted area is distorted for sloping terrain, but the distortion is uniform for uniform slopes (all other factors being the same)



- Wetted soil area can be estimated from empirical relationships and tables (Table 19.1 in the textbook), but it is best to have site-specific field data in which potential emitters are operated in the design area
- That is, test the emitter(s) and spacings in the field before completing the irrigation system design
- Calculate percent wetted area, P_w, as follows:

$$P_w = 100\left(\frac{N_p S_e w}{S_p S_r P_d}\right),\ \text{for } S_e < 0.8w \tag{333}$$

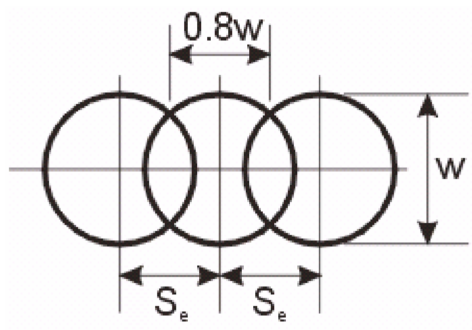
where N_p is the number of emission points (emitters) per plant; S_e is the spacing of emitters along a lateral; w is the wetted width along the lateral; S_p is the spacing of plants along a row; S_r is the spacing between rows; and P_d is the fraction (not percent) of area shaded (see Lecture 19)

- Note that the numerator of Eq. 333 is wetted area, and the denominator is actual plant area
- Note also that some emitters have multiple emission points

- S_e is the spacing between emitters on the lateral; however, if S_e is greater than 0.8w, then use 0.8w instead:

$$P_w = 100\left(\frac{0.8 N_p w^2}{S_p S_r P_d}\right), \text{ for } S_e \geq 0.8w \tag{334}$$

- Note that w is a function of the soil type
- S_e' is the "optimal" emitter spacing, defined as 0.8w
- There are practical limitations to the value of S_e with respect to S_p, otherwise there may not be enough emitters per plant (perhaps less than one)



Continuous wetted strip

- <u>Sample calculation</u>:

    - Suppose $S_r = S_p = 3.0$ m, P_d = 80%, and w = 1.1 m
    - Determine N_p for $P_w \geq 33\%$

$$S_e' = 0.8w = 0.8(1.1) = 0.88 \text{ m} \tag{335}$$

$$0.33 = \frac{N_p (0.88)(1.1)}{(3.0)(3.0)(0.80)} \tag{336}$$

whereby N_p = 2.45. Then,

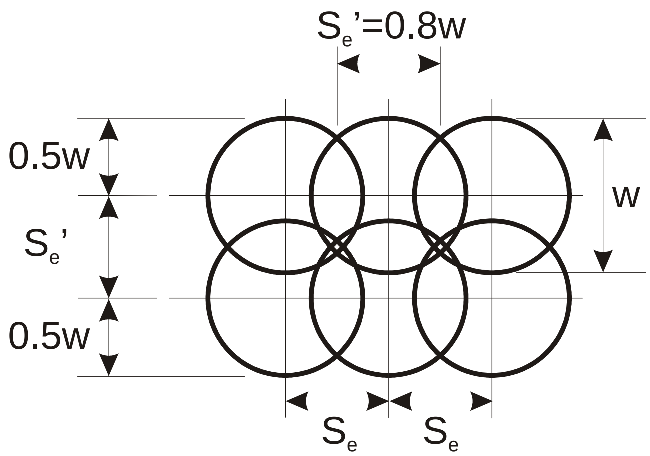
$$P_w = \frac{3(0.88)(1.1)}{(3.0)(3.0)(0.80)} = 0.40 \tag{337}$$

- For double-lateral trickle systems, spaced S_e' apart, P_w is calculated as follows (see Eq. 19.4):

$$P_w = 100\left(\frac{N_p S'_e \left(S'_e + w\right)}{2P_d\left(S_p S_r\right)}\right), \text{ for } S_e \leq 0.8w \tag{338}$$

or,

$$P_w = 100\left(\frac{1.44 w^2 N_p}{2P_d\left(S_p S_r\right)}\right) = \frac{72 w^2 N_p}{P_d\left(S_p S_r\right)}, \text{ for } S_e \leq 0.8w \tag{339}$$



Double laterals

- As in the previous equation, if $S_e > S_e'$, use S_e' instead of S_e in the above equation for double laterals
- In the above equation, the denominator has a "2" because N_p for double lateral systems is always at least 2
- For micro-spray emitters, the wetted area is greater than that measured at the surface (because it is measured below the surface):

$$P_w = 100\left[\frac{N_p\left(A_s + (PS)\frac{S_e}{2}\right)}{S_p S_r P_d}\right], \text{ for } S_e \leq 0.8w \tag{340}$$

where A_s is the surface area wetted by the sprayer; and PS is the perimeter (circumference) of the wetted surface area

- In the above equation for P_w, the term in the inner parenthesis is:

$$A_s + (PS)\frac{S_e}{2} = \frac{\pi w^2}{4} + \frac{\pi w S_e}{2} = \frac{\pi w}{2}\left(\frac{w}{2} + S_e\right) \tag{341}$$

where w is the diameter corresponding to A_s, assuming a circular area

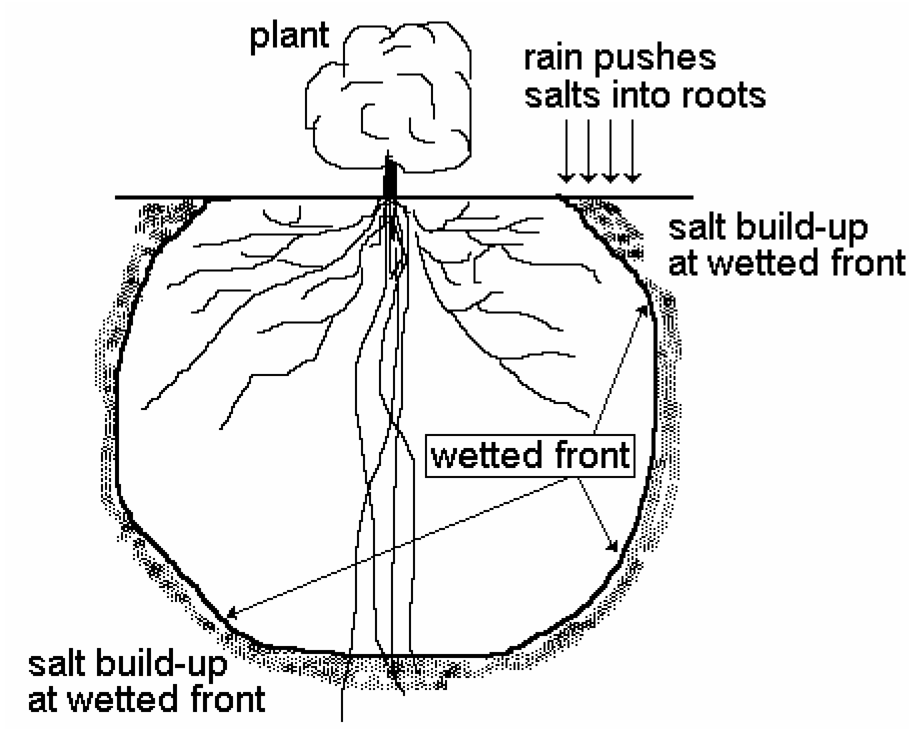
- See Fig. 19.4 on sprayers

Salinity in Trickle Irrigation

I. Salinity in Trickle Systems

- Salinity control is specialized with trickle irrigation because (usually) less than 100% of the area is wetted, and because water movement in the soil has significant horizontal components
- Irrigation water always contains salts, and fertilizers add salt to the crop root zones -- salinity management in the crop root zone is a long-term management consideration with trickle systems, as it is with any other irrigation method
- Salts tend to accumulate, or “build up”, at the periphery of the wetted bulb shape under the soil surface

    1. Rain can push salts near the surface down into the crop root area (but a heavy rain can push them all the way through the root zone)
    2. If and when the irrigation system is not operated for a few days, there can be pressure gradients in the soil that pulls salts from the periphery up into the root zone



- The crop is depending on frequent irrigations (perhaps daily) to keep salt build-ups from moving into the root mass
- It may be necessary to operate the trickle system immediately following a light rain to keeps salts away from roots (even if the soil is at field capacity)

- Annual leaching with surface irrigation or sprinklers (on a trickle-irrigated field) may be necessary to clean salts out of the root zone, unless there is a rainy period that provides enough precipitation to leach the soil
- If the irrigation water has high salinity, trickle systems can provide for higher crop production because the frequent irrigations maintain the soil salinity nearer to the EC_w (this is often not the case with sprinklers and surface irrigation systems - salinity concentrates due to ET processes between water applications)

II. Yield Effects of Salinity

- According to Keller, the relative crop yield can be estimated as (Eq. 19.6):

$$Y_r = \frac{Y_{actual}}{Y_{potential}} = \frac{(EC_e)_{max} - EC_w}{(EC_e)_{max} - (EC_e)_{min}} \tag{342}$$

- This is the relative crop yield (or production) in terms of soil water salinity only
- EC_w is the electrical conductivity of the irrigation water
- $(EC_e)_{max}$ is the zero yield point, and $(EC_e)_{min}$ is the 100% yield threshold value
- $(EC_e)_{max}$ may be as high as 32, and $(EC_e)_{min}$ can be as low as 0.9
- This is based on the linear relationship between relative yield and salinity as adopted years ago by FAO and other organizations
- Of course, calculated Y_r values must be between 0 and 1

- Salinity of the soil extract, EC_e, is measured by taking a soil sample to the laboratory, adding pure water until the soil is saturated, then measuring the electrical conductivity -- most published crop tolerance and yield relationships are based on the EC_e as a standard reference
- Crops don’t instantly die when the salinity approaches $(EC_e)_{max}$; the osmotic potential increases and roots cannot extract the water that is there
- There can also be specific toxicity problems with minerals at high salinity levels
- According to Allen, the relative yield will be near 100% for EC_w less than about $2(EC_e)_{min}$, provided that frequent irrigations are applied (maintaining salinity concentrations in root zone)

III. Leaching Requirement

- According to Keller & Bliesner, the leaching requirement under a trickle system in an arid or semi-arid region does not consider effective rainfall (arid regions often have more serious salinity problems, but tropical regions are also subject to salinity in low areas)

- Look at Eq. 19.7:

$$LR_t = \frac{EC_w}{EC_{dw}} \tag{343}$$

where LR_t is the leaching requirement under trickle irrigation (fraction); and EC_{dw} is the electrical conductivity of the “drainage water”, which means the water that moves downward past the root zone

- EC_{dw} can be replaced by $2(EC_e)_{max}$ for daily or every-other-day irrigations (keep water moving through the root zone), still obtaining $Y_r = 1.0$

$$LR_t = \frac{EC_w}{2\left(EC_e\right)_{max}} \tag{344}$$

IV. Allen’s Equation for LR_t

- R.G. Allen suggests a more conservative equation for calculating the leaching requirement under trickle irrigation:

1. For continuous trickle system operation (daily or once every two days), the soil water in the root zone is maintained near field capacity, which can be taken as approximately 50% saturation (θ_v) for many soils. Thus,

$$EC_e = 0.5EC_{soil} \tag{345}$$

(recall that EC_e is measured after adding distilled water to the soil sample until it is saturated)

2. Suppose the average EC_{soil} is taken as $(0.667EC_w + 0.333EC_{dw})$. Then, for 100% relative yield at field capacity:

$$\left(EC_e\right)_{min} = 0.5\left(0.667EC_w + 0.333EC_{dw}\right) \tag{346}$$

solving for EC_{dw},

$$EC_{dw} = 6\left(EC_e\right)_{min} - 2EC_w \tag{347}$$

3. Substitute this last equation into Eq. 19.7 from the textbook to obtain:

$$LR_t = \frac{EC_w}{6\left(EC_e\right)_{min} - 2EC_w} \tag{348}$$

this is similar to the leaching requirement as calculated for sprinkler irrigation in Eq. 3.3 (coefficients 5 and 1 instead of 6 and 2), except that $(EC_e)_{min}$ is for 100% yield rather than 10% reduction in yield

Lecture 19

Water Requirements in Trickle Irrigation

I. Trickle Irrigation Requirements

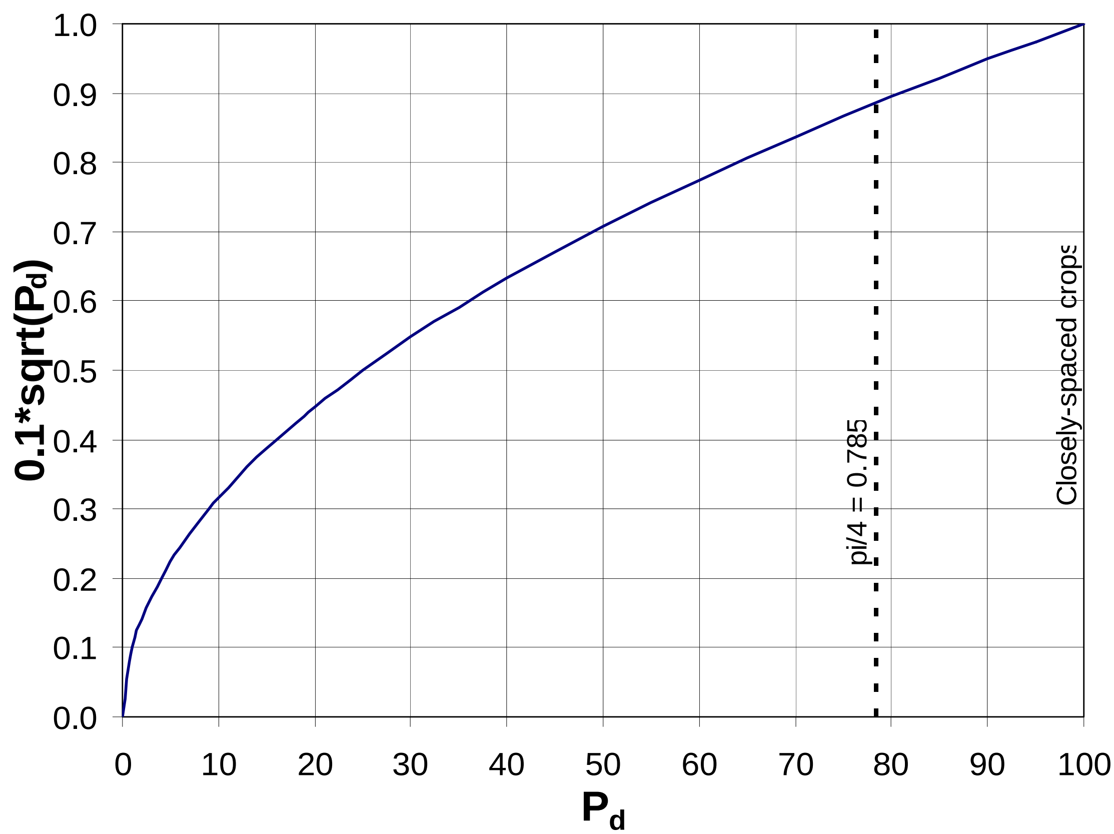
1. Daily Use Rate

- The daily transpiration rate under a trickle system is based on U_d and the percent area shaded (covered) by the plant leaves. Eq. 19.9:

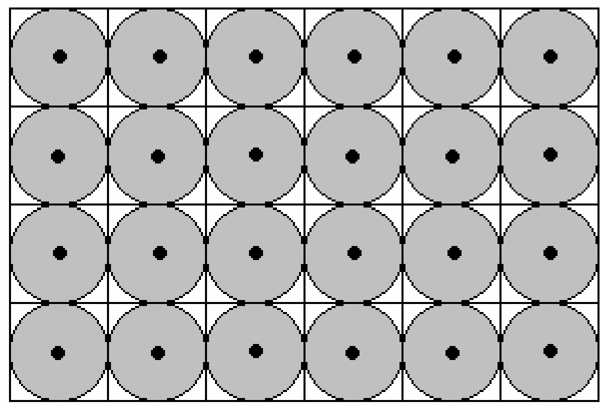
$$T_d = 0.1 U_d \sqrt{P_d} \tag{349}$$

where U_d is as previously defined and P_d is the percent (0 to 100) shaded area when the sun is overhead (or most nearly overhead, in temperate zones)

- Note that when $P_d = 0$, $T_d = 0$
- Note that when $P_d = 100\%$, $T_d = U_d$
- Note that T_d is called "transpiration," but it really includes evaporation too



- The reduction from U_d is justified by considering the typical reduction in wet soil evaporation with trickle irrigation
- The maximum P_d for a mature orchard is usually about $\pi/4$ (0.785), which is the ratio of the area of a square and the circle it encloses:



- Tree spacing is generally such that the trees do not compete for sunlight, and the area of each tree is equal to the square of the spacing between them (for a square spacing)

2. Seasonal Water Use

- This is calculated as for the peak daily use in Eq. 19.9:

$$T_s = 0.1\,U\sqrt{P_d} \tag{350}$$

3. Seasonal Water Deficit

- To determine the seasonal water deficit, to be supplied from the irrigation system, consider effective rainfall and initial soil moisture, in addition to percent shaded area:

$$D_n = \left(U - P_e - M_s\right)\left(0.1\sqrt{P_d}\right) \tag{351}$$

where U is used instead of T_s because P_e (effective precipitation) and M_s (initial soil water content) are over the entire surface area

4. Net Depth per Irrigation

- This is the same as for sprinkle irrigation (or surface irrigation), but with an adjustment for percent wetted area. Eq. 19.12 is:

$$d_x = \frac{MAD}{100}\frac{P_w}{100}W_a Z \tag{352}$$

- Essentially, the same net volume of water is applied as with other irrigation methods, but on a smaller area of the surface (and subsurface)
- Then, the maximum irrigation interval is:

$$f_x = \frac{d_x}{T_d} \tag{353}$$

and f' (round down from f_x to get whole number of days) is less than or equal to f_x, but often assumed to be 1 day for trickle system design purposes. Then,

$$d_n = T_d f' \tag{354}$$

II. Gross Irrigation Requirements

- The transmission ratio (peak use period) takes into account the two-dimensional infiltration pattern, or bulb shape, under trickle irrigation
- Even if the net depth is exactly right, there will almost always be some deep percolation (more than that which may be required for leaching purposes)
- The transmission ratio, T_r, is used as a factor to increase required gross application depth from d_n
- The transmission ratio is equivalent to the inverse of the distribution efficiency, DE_{pa}, as given in Chapter 6 of the textbook
- The transmission ratio is lower for heavy-textured ("fine") soils because there is more lateral water movement in the soil, and the bulb shape is flatter; thus, potentially less deep percolation losses
- Table 19.3 gives approximate values of T_r for different soil textures and root depths ($1.0 < T_r < 1.1$) – obtain more representative values from the field, if possible
- Then, for $LR_t < 0.1$, or $T_r > 1/(1-LR_t)$, Eq. 19.15a:

$$d = 100\left(\frac{d_n T_r}{EU}\right) \tag{355}$$

where EU is the *emission uniformity* (%), which can be taken as a field-measured value for existing trickle systems, or as an assumed design value

- EU takes into account pressure variations due to friction loss and elevation change, and the manufacturer's variability in emitter production
- If f' = 1 day, then d_n can be replaced by T_d in Eq. 19.15a

- For $LR_t > 0.1$, or $T_r < 1/(1-LR_t)$, Eq. 19.15c:

$$d = \frac{100\, d_n}{EU\left(1.0 - LR_t\right)} \tag{356}$$

- The difference in the above two equations is in whether LR_t or T_r dominates
- If one dominates, it is assumed that the other is "taken care of" automatically

<u>Gross Volume of Water per Plant per Day</u>

- Equation 19.16:

$$G = \frac{d}{f'} S_p S_r \tag{357}$$

with d in mm; S_p and S_r in m; and G in liters/day

- Note that millimeters multiplied by square meters equals liters
- This equation does not use P_w because d is calculated for the entire surface area, and each plant occupies an S_pS_r area
- Other versions of this equation are given in the textbook for gross seasonal volume of water to apply

<u>Required Application Time During Peak-Use Period</u>

- Equation 20.11:

$$T_a = \frac{G}{N_p q_a} \tag{358}$$

where T_a is the required application (irrigation) time during the peak-use period (hr/day), with G in litres/day, and q_a in litres/hr

III. Coefficient of Variation

- This is a statistical index to quantify discharge variations in emitters, at the same operating pressure, due to differences in the emitter construction
- The coefficient of variation is important in trickle system design and evaluation because it can significantly affect the adequacy of the system to irrigate the least watered areas of a field
- For statistical significance, there should be at least 50 measurements of discharge from 50 individual emitters of the same design and manufacture

$$\nu = \frac{\sqrt{\sum_{i=1}^{n}\left(q_i^2\right) - \frac{1}{n}\left(\sum_{i=1}^{n} q_i\right)^2}}{\sqrt{n-1}\left(\frac{1}{n}\sum_{i=1}^{n} q_i\right)} = \frac{\sigma}{q_{avg}} \tag{359}$$

or,

$$\nu = \frac{1}{q_{avg}}\sqrt{\frac{\sum_{i=1}^{n}\left(q_i - q_{avg}\right)^2}{n-1}} \tag{360}$$

where n is the number of samples; σ is the standard deviation; q_i are the individual discharge values; and q_{avg} is the mean discharge value of all samples

- Standard classifications as to the interpretation of ν have been developed (Soloman 1979):

Classification	**Drip & Spray Emitters**	**Line-Source Tubing**
Excellent	$\nu < 0.05$	$\nu < 0.1$
Average	$0.05 < \nu < 0.07$	$0.1 < \nu < 0.2$
Marginal	$0.07 < \nu < 0.11$	---
Poor	$0.11 < \nu < 0.15$	$0.2 < \nu < 0.3$
Unacceptable	$0.15 < \nu$	$0.3 < \nu$

- For a large sample (n > 50) the data will usually be normally distributed (symmetrical “bell-shaped” curve) and,

68% of the discharge values are within............... $(1\pm\nu)q_{avg}$
95% of the discharge values are within............. $(1\pm2\nu)q_{avg}$
99.75% of the discharge values are within........ $(1\pm3\nu)q_{avg}$

IV. System Coefficient of Variation

- The system coefficient of variation takes into account the probability that the use of more than one emitter per plant will cause an effective decrease in the combined discharge variability per plant due to differences in the emitters (not due to pressure variability due to pipe friction losses and elevation changes)
- On the average, discharge variability due to manufacturer tolerances will tend to balance out with more emitters per plant

$$\nu_s = \frac{\nu}{\sqrt{N_p'}} \tag{361}$$

where N_p' is the minimum number of emitters from which each plant receives water (see page 493 of the textbook)

- For a single line of laterals per row of plants,

$$L_w = w + (N-1)S_e \tag{362}$$

where L_w is the length of the wetted strip; and N is the number of emitters (assumed to be evenly spaced). Then,

$$N = 1 + \left(\frac{L_w - w}{S_e}\right) \tag{363}$$

or,

$$N_p' \approx 1 + \left(\frac{S_p - w}{S_e}\right) \tag{364}$$

V. Design Emission Uniformity

- In new system designs it is not possible to go out to the field to measure the EU' (Eq. 17.2) – a different approach is required to estimate EU
- The design EU is defined as (Eq. 20.13):

$$EU = 100\left(1 - 1.27\,\nu_s\right)\frac{q_n}{q_a} \tag{365}$$

where q_n is the minimum emitter discharge rate in the system, corresponding to the emitter with the lowest pressure; and q_a is the average emitter discharge rate in the system, corresponding to the location of average pressure in the system

- q_n and q_a are calculated (not measured) in new system designs by knowing the topography, system layout, pipe sizes, and Q_s
- Note that the value in parenthesis in Eq. 20.13 corresponds to the low one-quarter emitter discharge

- EU gives a lower (more conservative) value than EU', and the equation is biased toward lower discharges to help ensure that the least watered areas will receive an adequate application
- Graphical interpretations of these relationships are given in Figs. 20.9 and 20.10

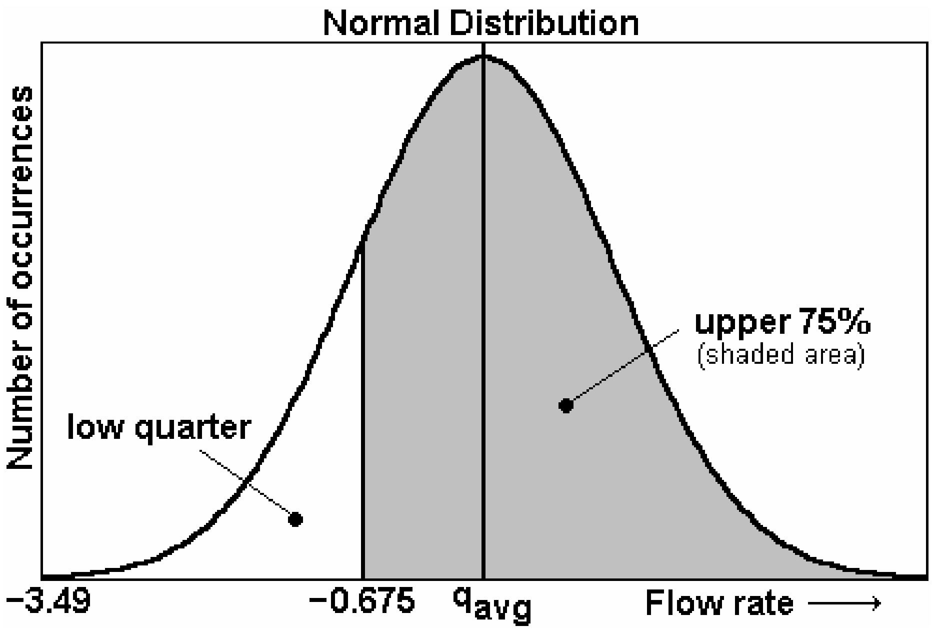
VI. Average of the low ¼

- Note that the inclusion percentages for 1, 2 and 3 standard deviations correspond to any normally distributed data
- Note also that $\nu \, q_{avg} = \sigma$
- The textbook says that for a normal distribution, the average flow rate of the low one-quarter of measured q samples is approximately $(1 - 1.27\,\nu)q_{avg}$
- The 1.27 coefficient can be determined from the equation for the normal distribution and tabular values of the area under the curve
- The equation is:

$$\text{occurrences} = \frac{e^{-\frac{1}{2}\left(\frac{q-q_{avg}}{\sigma}\right)^2}}{\sigma\sqrt{2\pi}} \tag{366}$$

- To use the tabular values of area under the curve (e.g. from a statistics book), it is necessary to use $q_{avg} = 0$ and $\sigma = 1$ (the alternative is to integrate the above equation yourself, which can also be done)
- Actually, q_{avg} never equals zero, but for the determination of the 1.27 coefficient it will not matter
- In the tables, for area = **75%**, the abscissa value (q, in our case) is about 0.675
- The same tables usually go up to a maximum abscissa of 3.49 (recall that 99.75% of the values are within $\pm 3\sigma$, so 3.49 is usually far enough)
- Anyway, for 3.49, the area is about **99.98%**, and that is from $-\infty$ to +3.49 (for $q_{avg} = 0$ and $\sigma = 1$), for the *high* ¼
- For the *low* ¼, take the opposite, changing to q = -0.675 and q = -3.49
- In this case ($q_{avg} = 0$ and $\sigma = 1$), the equation reduces to:

$$\text{occurrences} = \frac{e^{-0.5q^2}}{\sqrt{2\pi}} \tag{367}$$



- For q = 0.675, occurrences = 0.31766718
- For q = 3.49, occurrences = 0.0.00090372
- Finally,

$$\frac{0.31766718 - 0.00090372}{0.9998 - 0.7500} = 1.268 \qquad (368)$$

- Therefore, (1 - 1.268 ν)q_{avg} is the average of the lowest 25% of measured discharge values, for any given values of ν & q_{avg}, and given normally-distributed data

V. System Capacity

- The system capacity of a trickle system can be calculated by Eq. 20.15:

$$Q_s = 2.78 \frac{A}{N_s} \frac{N_p q_a}{S_p S_r} \qquad (369)$$

where N_s is the number of stations (sets); and A is the total net irrigated area. Or,

$$Q_s = 2.78 \frac{A}{N_s} \frac{q_a}{S_e S_l} \qquad (370)$$

where the coefficient "2.78" is for Q_s in lps; A in ha; q_a in lph; and S_p, S_r, S_e, and S_l in m (10,000 m^2/ha divided by 3,600 s/hr = 2.78)

VI. Operating Hours per Season

- The approximate number of hours the system must operate per irrigation season (or per year, in many cases) is equal to the required gross seasonal application volume, divided by the system flow rate:

$$O_t = K \frac{V_s}{Q_s} \tag{371}$$

where K = 2,778 for V_s in ha-m; and Q_s in lps; and V_s is gross seasonal volume of irrigation water

Lecture 20

Emitter Selection & Design

I. Introduction

- There are hundreds of models, sizes and types of emitters, sprayers, bubblers, and others, available from dozens of manufacturers
- Prices of emitters can change frequently
- Some emitters have longer life than others, but cost more
- Some emitters have better pressure compensating features, but cost more
- Some emitters have better flushing capabilities, but cost more
- It is very difficult to know which is the "correct" emitter for a particular design, and usually there are a number of emitters that could work and would be acceptable for a given system
- Thus, the selection of an emitter involves knowledge of the different types, their prices, their availability, and their performance
- Experience on the designer's part is valuable, and emitter selection will often involve a process of elimination

II. Long-Path Emitters

- So-called "spaghetti" tubing is a typical example of a long-path emitter
- Long-path emitters also come in spiral configurations (Fig. 20.1 of the textbook)
- These can be represented by an equation used for capillary flow under laminar conditions:

$$\ell_c = \frac{g\,\pi\,D^4 H}{\nu q K} \tag{372}$$

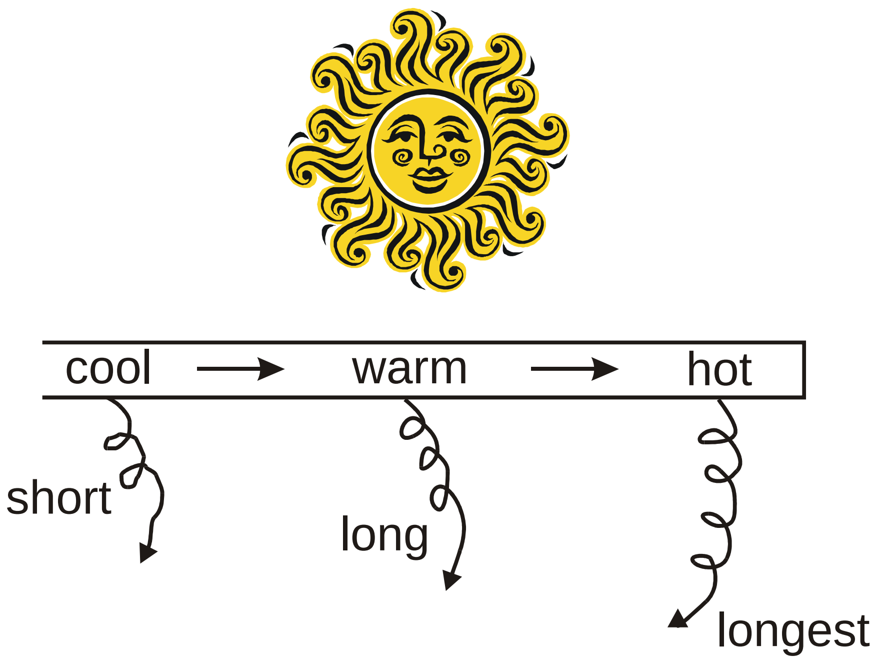
where l_c is the length of the flow path; D is the inside diameter; H is the pressure head; ν is the kinematic viscosity (a function of water temperature); q is the flow rate; K is for units conversion; and g is the ratio of force to mass

- The above equation is only approximately correct for long-path emitters
- The above equation is based on circular cross-sections, which is typical
- The above equation assumes laminar flow, which may not be the case
- Note that the flow rate is proportional to the fourth power of the diameter, so the diameter is a very important dimension
- Note also that the flow rate is inversely proportional to the length (double the length and get half the flow rate)

- When is it valid to assume laminar flow? Consider that a Reynolds number of 4,000 is probably as high as you can go without transitioning from laminar to turbulent flow:

$$\frac{VD}{\nu} = \frac{4Q}{\pi \nu D} < 4{,}000 \qquad (373)$$

or, Q < 15D @ 10°C, with Q in lph and D in mm



- In black PE lateral hose, sunlight warms the water significantly as the velocity slows down, and water viscosity decreases
- Long-path emitters would ideally be progressively longer along the lateral to compensate and provide a more uniform discharge along the lateral

III. Tortuous- and Short-Path Emitters

- Tortuous-path emitters also have long paths, but not laminar flow. This is because the path has many sharp bends, and is in the form of a maze
- Tortuous-path emitters tend to behave hydraulically like orifices, and so do many short-path emitters
- Flow rate is nearly independent of the viscosity, at least over typical ranges in viscosity
- Many short-path emitters have pressure compensating features

IV. Orifice Emitters

- Many drip emitters and sprayers behave as orifices
- The orifice(s) are designed to dissipate energy and reduce the flow rate to an acceptable value
- Flow rate is approximately proportional to the square root of the pressure

V. Line Source Tubing

- Single-chamber tubing provides less uniformity than dual-chamber tubing
- In dual-chamber tubing, much of the head loss occurs through the orifices between the two chambers. The outer chamber is somewhat analogous to a manifold or header.
- The flow rate equation for dual-chamber tubing can be expressed as:

$$q = a'K\sqrt{\frac{2gHn_o^2}{\left(1+n_o^2\right)}} \qquad (374)$$

where a' is the area of the outer orifice; K is an empirical coefficient; H is the pressure head; and n_o is the number of outer orifices per inner orifice (n_o > 1.0)

- See Fig. 20.2 in the textbook

VI. Vortex and Sprayer Emitters

- Vortex emitters have a whirlpool effect in which the water must exit through the center of the whirlpool
- Energy is dissipated by the friction from spinning in a chamber, and from exiting through an orifice in the center
- As mentioned in a previous lecture, the exponent on the pressure head is approximately equal to 0.4 (in the discharge equation). Thus, these can usually be considered to be (partially) pressure compensating

VII. Pressure Compensating Emitters

- Pressure compensating emitters usually have some flexible or moving parts
- These types of emitters tend to need replacement or repair more often than most of the simpler emitter designs, therefore incurring higher maintenance cost
- Figure 20.3 of the textbook shows one design approach for a pressure compensating emitter

- As defined previously, pressure compensating emitters always have a pressure head exponent of less than 0.5 (otherwise they aren't considered to be pressure compensating)

VIII.Self-Flushing Emitters

- In this category there are *continuous* and *periodic* flushing emitters
- Periodic flushing emitters perform their self cleaning when the lateral is filled (before it reaches full operating pressure), and when the lateral is emptied. In other words, they typically flush once per day.
- Continuous-flushing emitters have flexible parts that can stretch to allow solid particles to pass through
- Fig. 20.4 in the textbook shows an example of one such design
- These can be sensitive to temperature changes and are not normally pressure compensating

IX. Calculating the Discharge Exponent

- You can calculate the exponent, x, based on a pair of measured flow rates and pressure heads
- Recall a rule of logarithms: $\log(a^x) = x(\log a)$
- The solution can be obtained graphically, but is more quickly accomplished with calculators and electronic spreadsheets
- If you have more than two pairs of q and H, then you can take the logarithmic transformation of the equation and perform linear regression; however, the regression will be mathematically biased toward the smaller values

Design Approach & Example

I. Review of Example Designs

- We will review example designs in Chapter 21 of the textbook, and discuss design alternatives and parameters affecting efficiency, etc

II. Summarized Trickle Irrigation Design Process

- These are 15 basic steps, following the material presented in Chapters 17-24 of the textbook, that can be followed for the design of many trickle systems
- These are basic steps and represent a summary of the generalized design process, but remember that each design situation will have some unique features

1. Collect data on the crop, climate, soil, topography, and irrigation water quality, field shape & size, water availability.

2. Select an emitter and determine an emission point layout such that 33% < P_w < 67%. This will determine the number of emitters per plant, N_p. Emitter selection may involve field testing to determine the wetted width (or diameter), w.

3. Calculate d_x, f_x, and T_d. Note that f_x will almost always be greater than 1.0.

4. Select a target value for EU (usually 70-95%; see Table 20.3) and estimate the peak-use transmission ratio, T_r (usually 1.00-1.10; see Table 19.3).

5. Calculate the leaching requirement, LR_t, based on crop type and irrigation water quality.

6. Let f = 1 day (usually), then $d_n = T_d$. Calculate the gross application depth, d.

7. Calculate the gross volume of water required per plant per day, G.

$$G = K\left(\frac{dS_pS_r}{f}\right) \tag{375}$$

8. Calculate the daily hours of operation, T_a, (per station, or subunit) during the peak-use period.

$$T_a = \frac{G}{N_p q_a} \tag{376}$$

9. Determine the number of operation stations based on T_a (with more stations, the system capacity is lower).

   If T_a = 24 hrs, then N_s = 1
   If T_a = 12 hrs, then N_s = 1 or 2
   If T_a = 8 hrs, then N_s = 2 or 3, and so on

10. Adjust N_p and q_a so that T_aN_s is equal to, or slightly less than, 90%(24 hrs/day) = 21.6 hrs/day. First, try adjusting q_a because this is usually less expensive than increasing N_p. If the emitter is pressure compensating, or if q_a must be greatly altered, you may need to change N_p (or you may need to select a different emitter).

11. Having determined the value of q_a, calculate the minimum allowable emitter discharge, q_n

$$q_n = \frac{q_a\ EU}{100\left(1.0 - 1.27\,\nu_s\right)} \tag{377}$$

Note that if EU is high and ν_s is high, it could be that $q_n > q_a$ (but this would not be a reasonable calculation result!)

12. Calculate the average (nominal) and minimum lateral pressure heads

$$h = \left(\frac{q}{K_d}\right)^{1/x} \tag{378}$$

$$h_n = h_a\left(\frac{q_n}{q_a}\right)^{1/x} \tag{379}$$

13. Calculate the allowable change in pressure head in an operating station

$$\Delta H_s = 2.5\left(h_a - h_n\right) \tag{380}$$

14. Calculate Q_s, V_s, and O_t.

15. Finally, size the laterals, headers, manifolds and mainline(s) according to hydraulic design criteria.

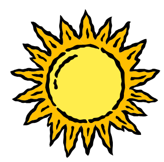
Lecture 21
Pipe Specifications & Trickle Lateral Design

I. Plastic Pipe Specifications

- Trickle and sprinkle irrigation systems are commonly built with plastic pipe, of which there are various types and specifications
- It is important to understand how the technical specifications affect design decisions (pipe sizing)
- Standards for the design and operation of pipelines are available from various professional organizations such as ASAE (American Society of Agricultural Engineers) and AWWA (American Water Works Association)
- Some of the material below is taken from ASAE standard S376.1 OCT92
- ASAE standard S435 pertains to the use of PE pipe for microirrigation laterals

- Plastic pipe is now commonly used in irrigation and other pipelines
- Some of the most common types are PVC (polyvinyl chloride), ABS (acrylonitrile-butadiene-styrene), and PE (polyethylene)
- PVC pipes are usually white, while ABS and PE are usually black
- ABS pipes are often used for buried drains and drainage pipes

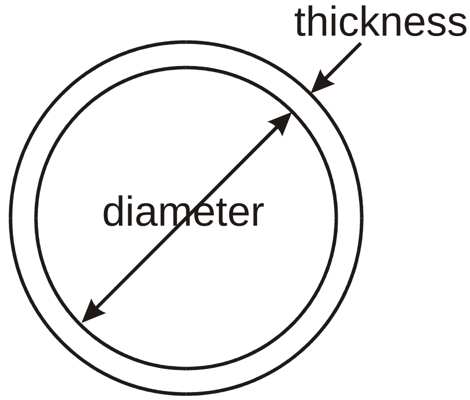
- All of these pipe materials are called "thermoplastic" because the material can be repeatedly softened by increasing the temperature, and hardened by a decrease in temperature
- The pressure rating of plastic pipe (especially PVC) decreases rapidly with increasing temperature of the pipe and or water
- For example, at about 43°C (109°F) the PVC pressure rating drops to one-half of the nominal value at 23°C (73°F), and almost the same amount for PE
- PE pipe temperature can easily reach 43°C on a sunny day



- Unlike most metal pipes, these plastic pipe materials are immune to almost all types of corrosion, whether chemical or electrochemical
- The resistance to corrosion is a significant benefit when chemigation is practiced in a pressurized irrigation system

- The *dimension ratio* (DR) of a plastic pipe is the ratio of average diameter (ID or OD) to wall thickness
- PVC, ABS and some PE are OD-based, while other PE pipe is ID-based

- Plastic pipe is currently manufactured up to a maximum diameter of 54 inches
- There are several *standard dimension ratios* (SDR) for several values, each with its own pressure rating (at 23°C)
- Different types of PVC, ABS and PE compounds exist, some of which are stronger than others
- Some plastic pipe is manufactured with non-standard dimension ratios; in these cases the ratio is called “DR” rather than “SDR”



- Some pipe sizes are correspond to *iron pipe size* (IPS), *plastic irrigation pipe* (PIP), and others
- These are different standards for indirectly specifying pipe dimension ratios and pressure ratings

- The relationship between SDR, hydrostatic design stress (S in psi), and pressure rating (PR in psi) for OD-based pipe is defined by ISO standard 161/1-1978
- The pressure rating (PR), which is the maximum recommended operating pressure, is determined by the following equations:

$$PR = \frac{2S}{SDR - 1} = \frac{2S}{\left(\frac{OD}{t} - 1\right)} \qquad \textit{(OD-based)} \qquad (381)$$

$$PR = \frac{2S}{SDR + 1} = \frac{2S}{\left(\frac{ID}{t} + 1\right)} \qquad \textit{(ID-based)} \qquad (382)$$

where t is the pipe wall thickness

- Values of S can be obtained from published tables, as can values of PR for given SDR and pipe material (plastic compound)
- Values of S vary from 6900 to 13,800 kPa for PVC, and from 3400 to 5500 kPa for PE

- Common terms used in the industry for PVC pipe include *Class 160*, *Class 200*, *Schedule 40*, *Schedule 80* and *Schedule 120* (in increasing strength and decreasing SDR)
- With the “schedule” classification, the higher the schedule, the thicker the walls, for a given nominal pipe diameter

- The maximum allowable operating pressure is approximately equal to:

$$P = \frac{(\text{schedule})SE}{1000} \tag{383}$$

where P is the operating pressure (psi); S is the allowable stress in the pipe material (psi); E is the “joint efficiency”; and “schedule” is the schedule number (e.g. 40, 80, 120, etc.)

- Joint efficiency (or “joint quality factor”) for PVC is approximately 1.00, due to the fact that it is seamless

- Class 160 and 200 refer to 160 psi and 200 psi ratings, respectively
- The Schedule 40 and 80 specifications have carried over from classifications used in iron pipes
- Schedule 80 is seldom used in irrigation because its pressure rating is much higher than the maximum pressures found in most irrigation systems
- Schedule 40 is commonly used in irrigation

- Some specifications for the design and protection of pipelines depend on whether the pressure is “low” or “high”
- Low pressure pipelines are generally considered to have operating pressures less than about 80 psi
- The maximum working pressure in a plastic pipe should normally be about 70% of the pipe’s pressure rating, unless special care is taken in design and operation such that surges and excessive pressure fluctuations will not occur

- Manufacturers and testing centers provide data on minimum *bursting pressures*
- Depending on the SDR value, the *minimum* burst pressure for plastic pipes should be between about 900 and 1800 kPa (130 and 260 psi), otherwise the pipe does not meet standard specifications
- Below is a glossary of common pipe abbreviations and terms:

Abbreviation	Meaning
ABS	Acrylonitrile-Butadiene-Styrene
DR	Dimension Ratio
ID	Inside Diameter
IPS	Iron Pipe Size
ISO	International Organization for Standardization
OD	Outside Diameter
PE	Polyethylene
PIP	Plastic Irrigation Pipe
PR	Pressure Rating
PVC	Polyvinyl Chloride
SDR	Standard Dimension Ratio

II. Trickle Irrigation Laterals

- Laterals are often above ground, but may be buried
- Drip "tape", single- and dual-chamber laterals are usually buried a few centimeters below the ground surface
- Above-ground laterals may be on the ground surface, or suspended above the surface (e.g. in vineyards)
- Black polyethylene (PE) plastic pipe (or "hose") is usually used for trickle irrigation laterals
- Lateral pipes are typically about 0.5 or 1.0 inches in diameter
- Standard PE sizes are usually ID based and come in *standard dimension ratio* (SDR) values of 15, 11.5, 9, 7 and 5.3
- Nominal PE pipe sizes for laterals are ½-inch, ¾-inch, 1-inch, and 1¼-inch (all *iron pipe size*, or IPS)

- Laterals are usually single-diameter, but can be dual-sized
- Dual-sized lateral hydraulic analysis is essentially the same as previously discussed for dual-sized sprinkler laterals

- To start a new system design, Keller & Bliesner recommend limiting the lateral pressure variation to $0.5\Delta H_s$, where ΔH_s is calculated from Eq. 20.14
- Then, $0.5\Delta H_s$ remains for the manifolds (if manifolds are subunits, or "stations")

- In lateral designs, the pipe diameter is usually chosen (not calculated), and if the pressure variation or loss is "out of range", then a different size can be selected
- There are usually only a few lateral diameters to choose from

III. Trickle Lateral Hydraulics

- Friction loss gradients in plastic lateral pipe can be approximated by combining the Darcy-Weisbach and Blasius equations (Eq. 8.7a):

$$J = 7.83(10)^7 \frac{Q^{1.75}}{D^{4.75}} \tag{384}$$

for J in m/100 m; Q in lps; and D in mm

- The Blasius equation estimates the D-W "f" factor for smooth pipes
- If you want to calculate relative roughness, use $\varepsilon = 1.5(10)^{-6}$ m

- It may be necessary to increase the J value because of emitter losses *within* the lateral hose (barb, etc.) (see Fig. 20.8)
- Equation 22.1 is:

$$J' = J\left(\frac{S_e + f_e}{S_e}\right) \tag{385}$$

where f_e is an equivalent length of lateral hose for each emitter, spaced evenly at a distance of S_e

- The f_e pipe length is one way that minor hydraulic losses are calculated in pipes
- From Eq. 8.7a, a dimensionless friction loss equation can be developed (see Fig. 8.2), which is useful in semi-graphical hydraulic design work for trickle irrigation laterals
- This is discussed in detail in the following lectures

- For a given lateral pipe size, lateral length, emitter spacing, and nominal discharge per emitter, the lateral inlet pressure must be determined such that the average lateral pressure is "correct"
- Then, the manifold can be designed to provide this lateral inlet pressure with as little variation (with distance) as possible

- Figure 22.1 shows four different hydraulic cases for single lateral designs
- The design of pairs of laterals is essentially a compound single lateral problem, with the added criterion that the minimum pressure be the same in both laterals

- Not including riser height, the required lateral inlet pressure is (Eq. 22.6):

$$H_l = H_a + k h_f + 0.5 \Delta h_e \tag{386}$$

where k is 0.75 for single pipe size laterals, or 0.63 for dual pipe size laterals (as in the design of sprinkler laterals); and Δh_e is positive for laterals running uphill

- The minimum pressure in a lateral is given by Eq. 22.7:

$$
\begin{aligned}
H_n' &= H_l - (h_f + \Delta h_e) - \Delta H_c \\
H_n' &= H_c - \Delta H_c
\end{aligned}
\tag{387}
$$

where H_c is the pressure head at the closed end of the lateral

IV. References (plastic pipe)

http://www.uni-bell.org/lit.cfm
http://www.dpcpipe.com/ag/pipirrig.html
ASAE Standards (1997). American Soc. of Agric. Engineers, St. Joseph, MI.
Handbook of PVC Pipe. (1979). Uni-Bell Plastic Pipe Association, Dallas, TX.

Trickle Manifold Location

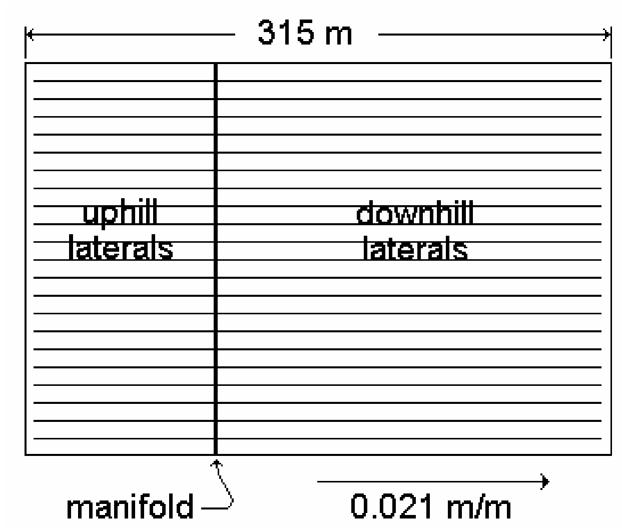
I. Optimal Manifold Location

- If the ground slope along the direction of the laterals is less than 3% or so, it is usually recommendable to run laterals off both sides (uphill and downhill) of each manifold
- If the ground slope along the direction of the laterals is more than 3%, it may be best to run the laterals only in the downhill direction
- The design objective for a pair of laterals is to have equal values of minimum pressure, H_n’, in uphill and downhill laterals
- This means that the downhill lateral will always be longer for laterals of equal pipe size on sloping ground
- The manifold should be located in-between rows of plants (trees), not over a row
- For laterals on flat ground, the manifold goes in the center of the field (the *trivial solution*)

II. Sample Graphical Solution for Manifold Location

- Use the dimensionless friction loss curves (Fig. 8.2) to locate the optimal manifold position in a sloping field
- The laterals run along the 0.021 m/m slope
- The combined uphill + downhill lateral length is 315 m

- The spacing of plants (trees) is S_p = 4.5 m
- The spacing of emitters is S_e = 1.5 m (thus, N_p = 3 for single line)
- The equivalent emitter loss is f_e = 0.12 m
- The nominal emitter discharge is q_a = 3.5 lph at 10 m head (68.9 kPa)
- The lateral pipe ID is 14.7 mm



Solution for Manifold Location:

1. Number of emitters for the pair of laterals is:

$$\frac{315\text{ m}}{1.5\text{ m/emitter}} = 210\text{ emitters} \tag{388}$$

2. Total nominal discharge for the pair of laterals is:

$$Q_{pair} = \frac{(210\text{ emitters})(3.5\text{ lph/emitter})}{(60\text{ min/hr})} = 12.25\text{ lpm} \tag{389}$$

3. From Table 8.2 (page 141), J ≅ 13.3 m/100m. The adjusted J is:

$$J' = J\left(\frac{S_e + f_e}{S_e}\right) = 13.3\left(\frac{1.5 + 0.12}{1.5}\right) = 14.4\text{ m/100 m} \tag{390}$$

4. Multiple outlet factor, F = 0.36 for 210 outlets

5. Friction loss for the pair of laterals:

$$(h_f)_{pair} = \frac{J'FL}{100} = \frac{(14.4)(0.36)(315)}{100} = 16.3\text{ m} \tag{391}$$

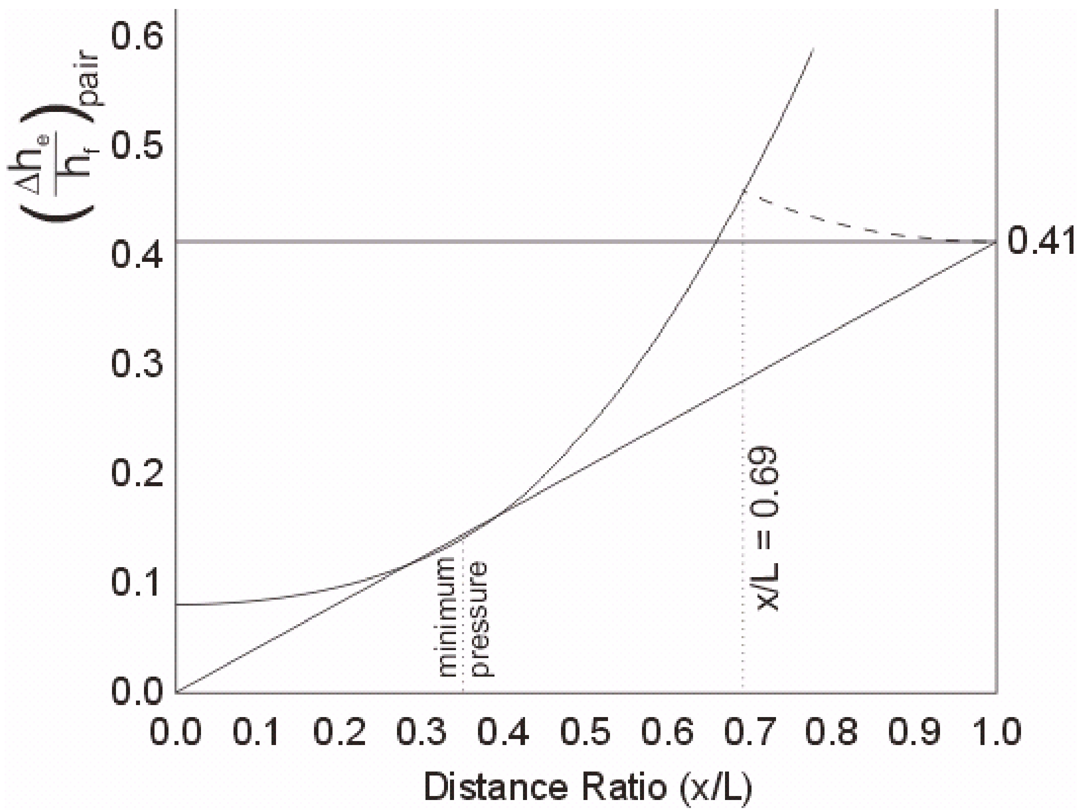
6. Elevation change for the pair of laterals:

$$(\Delta h_e)_{pair} = (315\text{ m})(0.021) = 6.62\text{ m} \tag{392}$$

7. Ratio of elevation change to friction loss for the pair:

$$\left(\frac{\Delta h_e}{h_f}\right)_{pair} = \frac{6.62}{16.3} = 0.41 \qquad (393)$$

8. From the nondimensional graphical solution (Fig. 8.2): x/L = 0.69. Then, x = (0.69)(315 m) = 217 m. Look at the figure below:



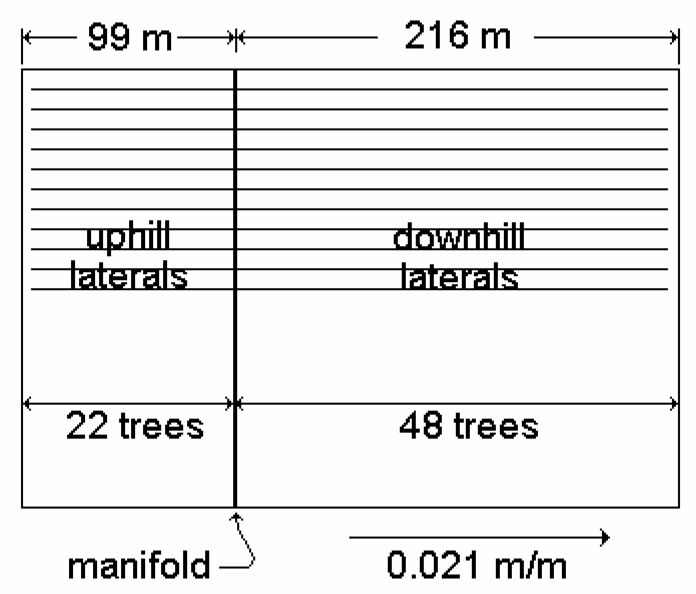
How was this done?

- Looking at the above figure, a straight line was drawn from the origin (0, 0) to (1.0, 0.41), where 0.41 is the ratio calculated above
- The nondimensional curve was overlapped and shifted vertically so that the curve was tangent to the same straight line, then traced onto the graph
- The nondimensional curve was then shifted vertically even more so that the inverse half-curve (dashed) intersected the (1.0, 0.41) point, also tracing it onto the graph
- The intersection of the two traced curve segments gave an abscissa value of about 0.69, which is the distance ratio

9. Finally, adjust x for tree spacing,

$$(217 \text{ m})/(4.5 \text{ m/tree}) = 48.2 \text{ trees}$$

- Therefore, round to 48 trees
- Then, x = (48 trees)(4.5 m/tree) = 216 m
- This way, the manifold lays buried halfway between two rows of trees, not on top of a row



- This manifold position give the same minimum pressure in both the uphill and downhill laterals
- Minimum pressure in the downhill lateral is located approximately (0.35)(315 m) = 110 m from the closed end, or 216 - 110 = 106 m from the manifold.

- This graphical solution could have been obtained numerically
- But the graphical solution is useful because it is *didactic*
- If you like computer programming, you can set up the equations to solve for the lateral hydraulics based on non-uniform emitter discharge (due to pressure variations in the laterals), non-uniform ground slope, etc.

- Note that this procedure could also be used for sprinklers, but it would probably only be feasible for solid-set, fixed systems

Lecture 22

Numerical Solution for Manifold Location

I. Introduction

- In the previous lecture it was seen how the optimal manifold location can be determined semi-graphically using a set of non-dimensional curves for the uphill and downhill laterals
- This location can also be determined numerically
- In the following, equations are developed to solve for the unknown length of the uphill lateral, x_u, without resorting to a graphical solution

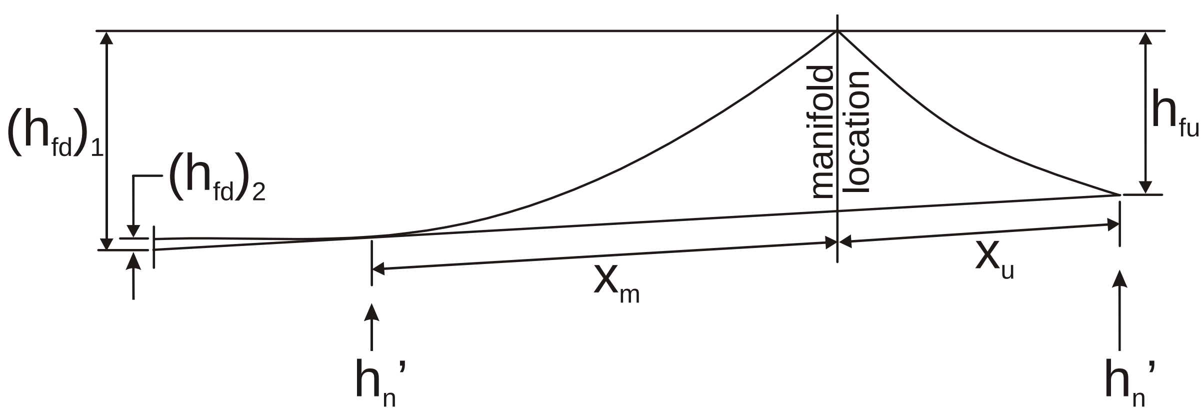
II. Definition of Minimum Lateral Head

- In the uphill lateral, the minimum head is at the closed end of the lateral (furthest uphill location in the subunit)
- This minimum head is equal to:

$$h_n' = h_l - h_{fu} - x_u S \tag{394}$$

where h_n' is the minimum head (m); h_l is the lateral inlet head (m); h_{fu} is the total friction loss in the uphill lateral (m); x_u is the length of the uphill lateral (m); and S is the slope of the ground surface (m/m)

- Note that S must be a positive value



- In the downhill lateral, the minimum head may be anywhere from the inlet to the outlet, depending on the lateral hydraulics and the ground slope
- The minimum head in the downhill lateral is equal to:

$$h_n' = h_l - (h_{fd})_1 + (h_{fd})_2 + x_m S \tag{395}$$

where $(h_{fu})_1$ is the total friction loss in the downhill lateral (m); $(h_{fu})_2$ is the friction loss from the closed end of the downhill lateral to the location of minimum head (m); and x_m is the distance from the manifold (lateral inlet) to the location of minimum head in the downhill lateral (m)

- Combining Eqs. 1 and 2:

$$h_{fu} + S\left(x_u + x_m\right) - \left(h_{fd}\right)_1 + \left(h_{fd}\right)_2 = 0 \tag{396}$$

III. Location of Minimum Head in Downhill Lateral

- The location of minimum head is where the slope of the ground surface, S, equals the friction loss gradient, J':

$$S = J' \tag{397}$$

where both S and J' are in m/m, and S is positive (you can take the absolute value of S)

- Using the Hazen-Williams equation, the friction loss gradient in the downhill lateral (at the location where S = J') is:

$$J' = \left[\frac{S_e + f_e}{S_e}\right]\left[1.212(10)^{10}\left(\frac{q_a\left(L - x_u - x_m\right)}{3{,}600\,S_e\,C}\right)^{1.852} D^{-4.87}\right] \tag{398}$$

where: J' is the friction loss gradient (m/m);
S_e is the emitter spacing on the laterals (m);
f_e is the equivalent lateral length for emitter head loss (m);
q_a is the nominal emitter discharge (lph);
L is the sum of the lengths of the uphill and downhill laterals (m);
x_u is the length of the uphill lateral (m);
x_m is the distance from the manifold to the location of minimum head in
the downhill lateral (m);
C is approximately equal to 150 for plastic pipe; and
D is the lateral inside diameter (mm);

- The value of 3,600 is to convert q_a units from lph to lps
- Note that $x_d = L - x_u$, where x_d is the length of the downhill lateral
- Note that $q_a(L - x_u - x_m)/(3{,}600\ S_e)$ is the flow rate in the lateral, in lps, at the location of minimum head, x_m meters downhill from the manifold
- Combining the above two equations, and solving for x_m:

$$x_m = L - x_u - \left(\frac{0.0129\,S_e C D^{2.63}}{q_a}\right)\left(\frac{S_e S}{S_e + f_e}\right)^{0.54} \quad (399)$$

where the permissible values of x_m are: $0 \le x_m \le x_d$

- Combine Eqs. 396 & 399, and solve for x_u by iteration
- Alternatively, based on Eq. 8.7a from the textbook, x_m can be defined as:

$$x_m = L - x_u - \left[\frac{3{,}600\,S_e}{q_a}\right]\left[\left(\frac{S D^{4.75}}{7.89(10)^5}\right)\left(\frac{S_e}{S_e + f_e}\right)\right]^{0.571} \quad (400)$$

IV. Definition of Head Loss Gradients

- In the uphill lateral, the head loss is:

$$h_{fu} = J_u' F_u x_u \quad (401)$$

- In the downhill lateral, the head losses are:

$$\left(h_{fd}\right)_1 = J_{d1}' F_{d1}\left(L - x_u\right) \quad (402)$$

and,

$$\left(h_{fd}\right)_2 = J_{d2}' F_{d2}\left(L - x_u - x_m\right) \quad (403)$$

- The above three “F” values are as defined by Eq. 8.9 in the textbook
- The friction loss gradients (in m/m) are:

$$J_u' = K_J\left(x_u\right)^{1.852} \quad (404)$$

$$J_{d1}' = K_J\left(L - x_u\right)^{1.852} \quad (405)$$

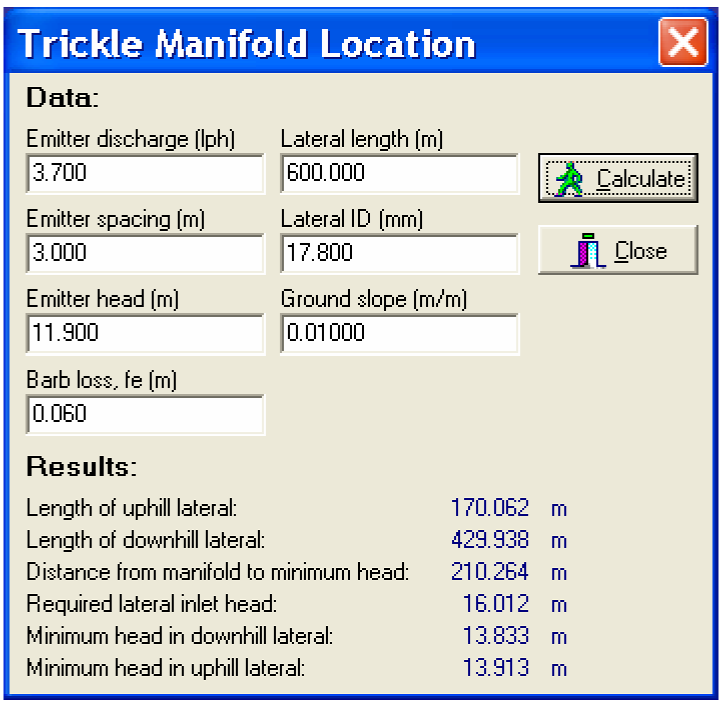
$$J_{d2}' = K_J\left(L - x_u - x_m\right)^{1.852} \quad (406)$$

where, for the Hazen-Williams equation,

$$K_J = \left(\frac{S_e + f_e}{S_e}\right)\left[1.212(10)^{10} D^{-4.87}\left(\frac{q_a}{3,600\, S_e C}\right)^{1.852}\right] \qquad (407)$$

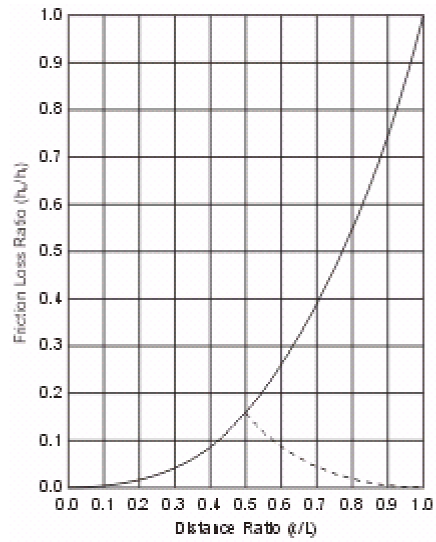
V. Solving for Optimal Manifold Location

- Using the definitions above, solve for the length of the uphill lateral, x_u
- Then, $x_d = L - x_u$
- Note that you might prefer to use the Darcy-Weisbach and Blasius equations for the manifold calculations; they may be more accurate than Hazen-Williams
- The “OptManifold” computer program uses the Darcy-Weisbach & Blasius equations



Where do these Equations Come From?

I. Derivation of Nondimensional Friction Loss Curves



- The nondimensional friction loss curves are actually one curve, with the lower half laterally inverted and shown as a dashed line (Fig. 8.2)
- The dashed line is simply for flow in the opposite direction, which for our purposes is in the uphill direction
- We know from the previous lectures and from intuition that the uphill segment of lateral pipe will not be more than ½ the total length, because it is equal to ½ for the case where the ground slope is zero

- Following is the derivation for Eq. 8.10b, from which Fig. 8.2 was plotted

- Darcy-Weisbach equation for circular pipes:

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \tag{408}$$

- Blasius equation, for estimating f for small diameter (D < 125 mm) "smooth pipes" (e.g. PE & PVC), and based on more complete equations that are used to plot the Moody diagram

$$f \cong 0.32 N_R^{-0.25} \tag{409}$$

where N_R is the Reynolds number, which for circular pipes is:

$$N_R = \frac{VD}{\nu} = \frac{4Q}{\nu \pi D} \tag{410}$$

- The kinematic viscosity, ν, is equal to about $1.003(10)^{-6}$ m^2/s for water at 20°C
- Then, for this kinematic viscosity,

$$f \cong 0.32 \left(\frac{4Q}{\nu \pi D} \right)^{-0.25} \approx 0.0095 \left(\frac{Q}{D} \right)^{-0.25} \tag{411}$$

- Putting the above into the Darcy-Weisbach equation:

$$h_f = 0.0095\left(\frac{Q}{D}\right)^{-0.25}\frac{L}{D}\frac{V^2}{2g} \tag{412}$$

or:

$$h_f \cong 0.00079 L \frac{Q^{1.75}}{D^{4.75}} \tag{413}$$

where h_f is in m; L is in m; Q is in m^3/s; and D is in m

- Eq. 8.7a is obtained by having Q in lps, and D in mm, whereby the above coefficient changes to $7.9(10)^7$
- Finally, in the above, use L(x/L) instead of L, and Q(x/L) instead of Q, and call it "h_{fx}":

$$h_{fx} \cong 0.00079 L(x/L)\frac{\left[Q(x/L)\right]^{1.75}}{D^{4.75}} \tag{414}$$

- Then,

$$\frac{h_{fx}}{h_f} = (x/L)(x/L)^{1.75} = (x/L)^{2.75} \tag{415}$$

which is Eq. 8.10b and the basis for the nondimensional friction loss curves, valid for plastic pipes with D < 125 mm

II. Derivation of Equation for ΔH_c

- The difference between the minimum pressure head and the pressure head at the closed end of a lateral, ΔH_c, is used to calculate the minimum head in the lateral, H_n'
- This is because the pressure head at the end of the lateral is easily calculated as:

$$H_c = H_l - h_f - \Delta h_e \tag{416}$$

where Δh_e is negative for downhill slopes

- But the minimum pressure head does not necessarily occur at the end of the lateral when the lateral runs downhill
- Thus, in general,

$$H'_n = H_c - \Delta H_c \tag{417}$$

- The above is from Eq. 22.7 in the textbook
- These concepts can also be interpreted graphically as in Fig. 22.1
- Following is a derivation of an equation for ΔHc (based on Keller and Rodrigo 1979)

1. The minimum pressure in the lateral occurs where the ground slope (for a uniform slope) equals the slope of the friction loss curve. The dimensionless friction loss curve is defined as (Eq. 8.10b or Eq. 22.3b):

$$\left(\frac{h_{fx}}{h_f}\right)_{pair} = \left(\frac{x}{L}\right)^{2.75} \tag{418}$$

2. The slope of this friction loss curve is:

$$\frac{d\left(\frac{h_{fx}}{h_f}\right)_{pair}}{d\left(\frac{x}{L}\right)} = 2.75\left(\frac{x}{L}\right)^{1.75} \tag{419}$$

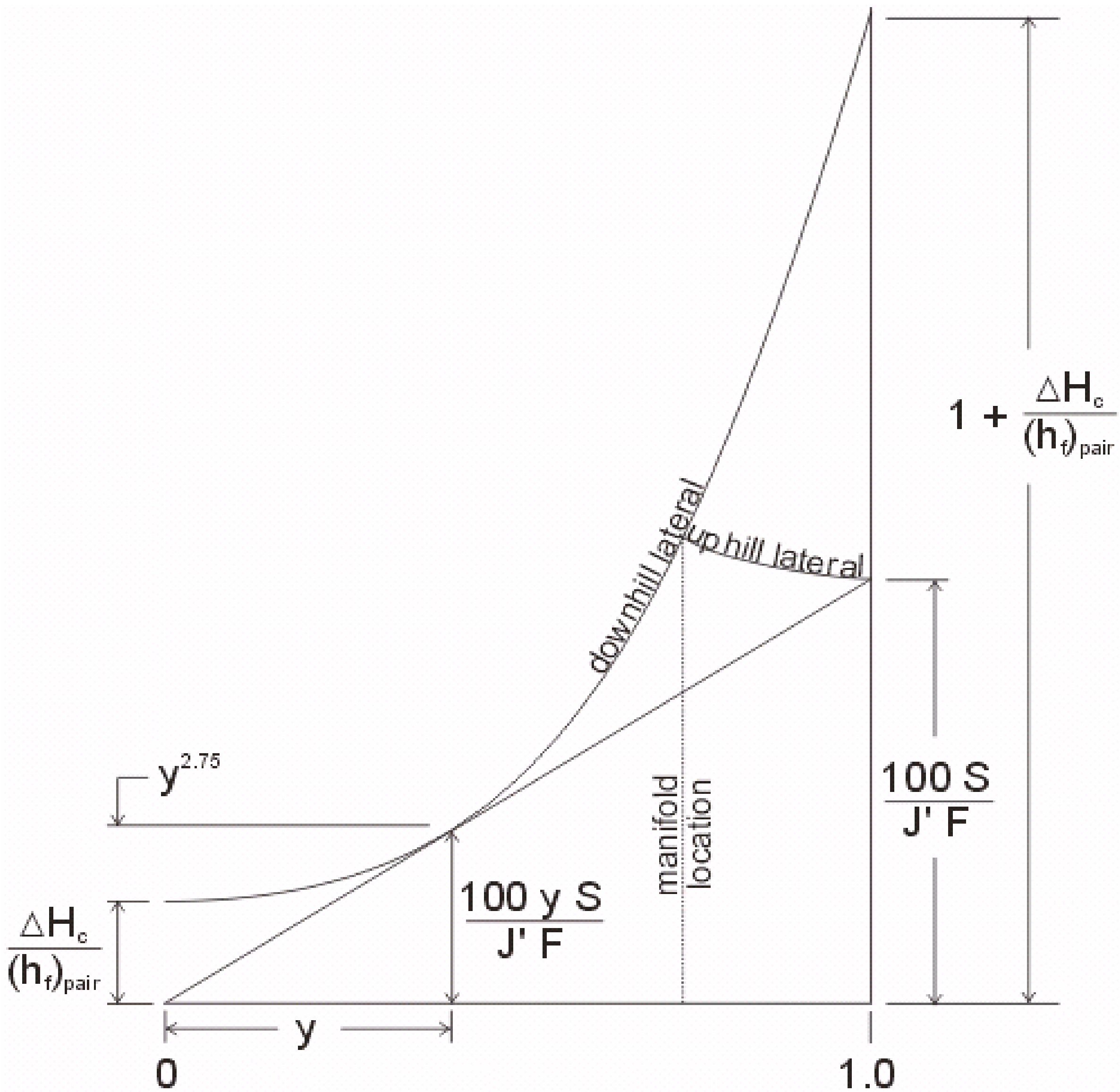
3. The uniform ground slope on the dimensionless graph is:

$$\left(\frac{\Delta h_e}{h_f}\right)_{pair} = \frac{SL}{\left(\frac{J'FL}{100}\right)} = \frac{100\,S}{J'F} \tag{420}$$

4. Then,

$$\frac{100\,S}{J'F} = 2.75(y)^{1.75} \tag{421}$$

in which y is the value of x/L where the minimum pressure occurs ($0 \le y \le 1$); S is the ground slope (m/m); J' is the friction loss gradient for the flow rate in the pair of laterals (m/100 m); and F is the reduction coefficient for multiple outlet pipes (usually about 0.36)



5. Solve for y:

$$y = \left(\frac{100\,S}{2.75\,J'F} \right)^{1/1.75} \tag{422}$$

or,

$$y \approx \left(\frac{100\,S}{J'} \right)^{1/1.75} \tag{423}$$

where $F \approx 0.36$

6. Referring to the figure on the previous page, the following equality can be written:

$$y^{2.75} + \frac{\Delta H_c}{(h_f)_{pair}} = \frac{100\,y\,S}{J'F} \tag{424}$$

solving for ΔH_c,

$$\Delta H_c = (h_f)_{pair}\left(\frac{100\,y\,S}{J'F} - y^{2.75}\right) \tag{425}$$

where,

$$(h_f)_{pair} = \frac{J'FL}{100} \tag{426}$$

and y can be approximated as in step 5 above (for F = 0.36)

7. After manipulating the equation a bit, the following expression is obtained:

$$\Delta H_c = 8.9\,L\,S^{1.57}\,(J')^{-0.57} \tag{427}$$

for ΔH_c in m; L in m; S in m/m; and J' in m/100 m. Note that J' and L are for the <u>pair</u> of laterals, not only uphill or only downhill

III. Derivation of Equation for α

- The parameter α is used in the calculation of inlet pressure for a pair of laterals on sloping ground where (Eq. 22.17):

$$H_l = H_a + \alpha (h_f)_{pair} + \left(\frac{x}{L} - 0.5\right)(\Delta h_e)_{pair} \tag{428}$$

with,

$$H_a = \left(\frac{q_a}{K_d}\right)^{1/x} \tag{429}$$

$$(h_f)_{pair} = \frac{J'FL}{100} \tag{430}$$

$$\left(\Delta h_e\right)_{pair} = \frac{100\,S}{J'F}\left(h_f\right)_{pair} = SL \tag{431}$$

- Note that $(\Delta h_e)_{pair}$ must be a negative number
- The ratio x/L is the distance to the manifold, where L is the length of the <u>pair of laterals</u>
- The following derivation is based on equations presented by Keller and Rodrigo (1979):

   1. Given that for a single lateral approximately ¾ of the friction loss occurs from the inlet to the point where the average pressure occurs (multiple outlets, uniform outlet spacing, constant discharge from outlets, single lateral pipe size) we have the following:

$$\alpha\left(h_f\right)_{pair} = \frac{3}{4}\left(h_f\right)_{downhill}\left(\frac{x}{L}\right) + \frac{3}{4}\left(h_f\right)_{uphill}\left(1-\frac{x}{L}\right) \tag{432}$$

The above equation is *a weighted average* because the uphill lateral is shorter than the downhill lateral

2. Recall that,

$$\left(\frac{h_{fx}}{h_f}\right)_{pair} = \left(\frac{x}{L}\right)^{2.75} \tag{433}$$

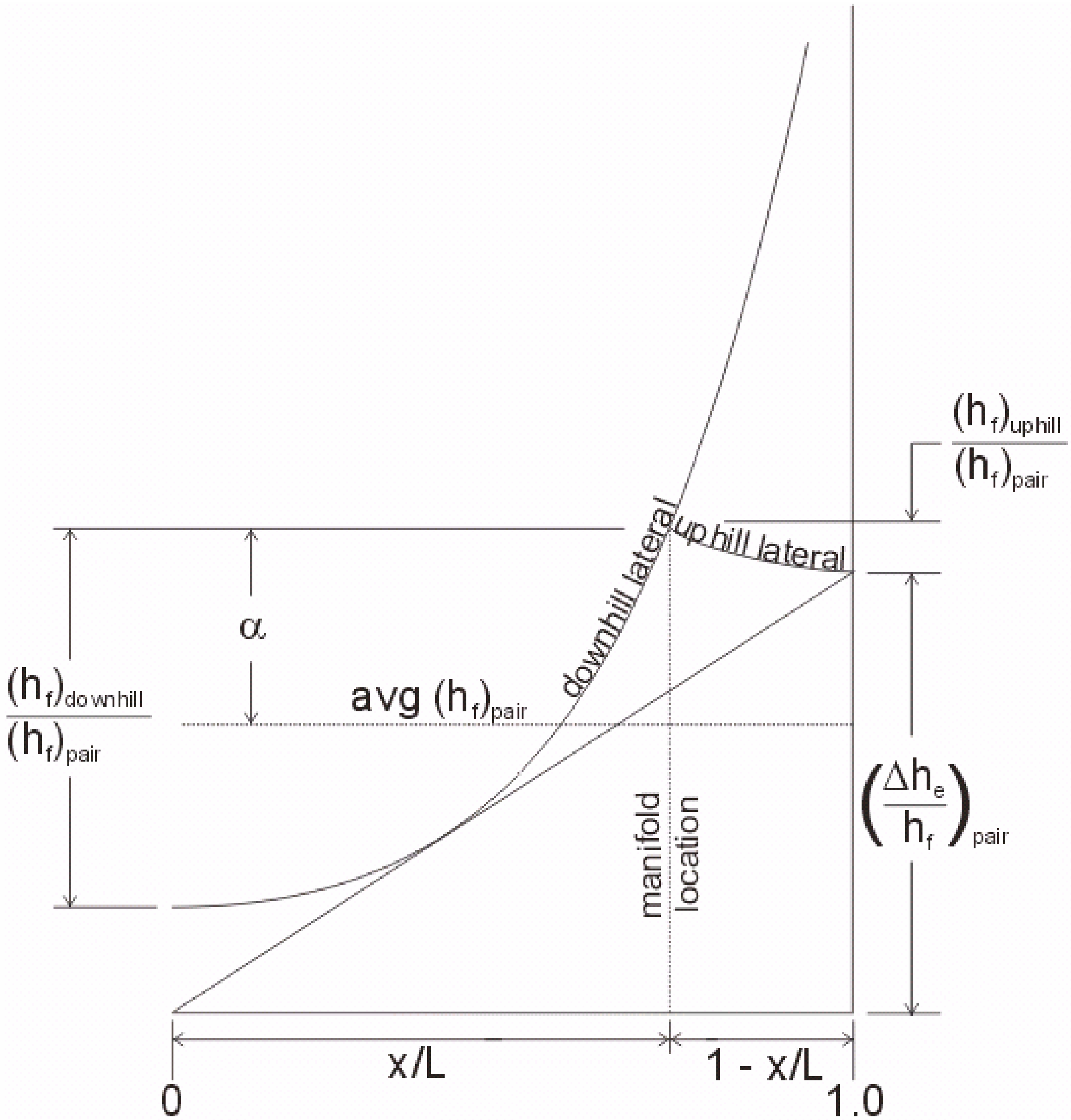
Then,

$$\begin{aligned}\left(h_f\right)_{downhill} &= \left(\frac{x}{L}\right)^{2.75}\left(h_f\right)_{pair}\\ \left(h_f\right)_{uphill} &= \left(1-\frac{x}{L}\right)^{2.75}\left(h_f\right)_{pair}\end{aligned} \tag{434}$$

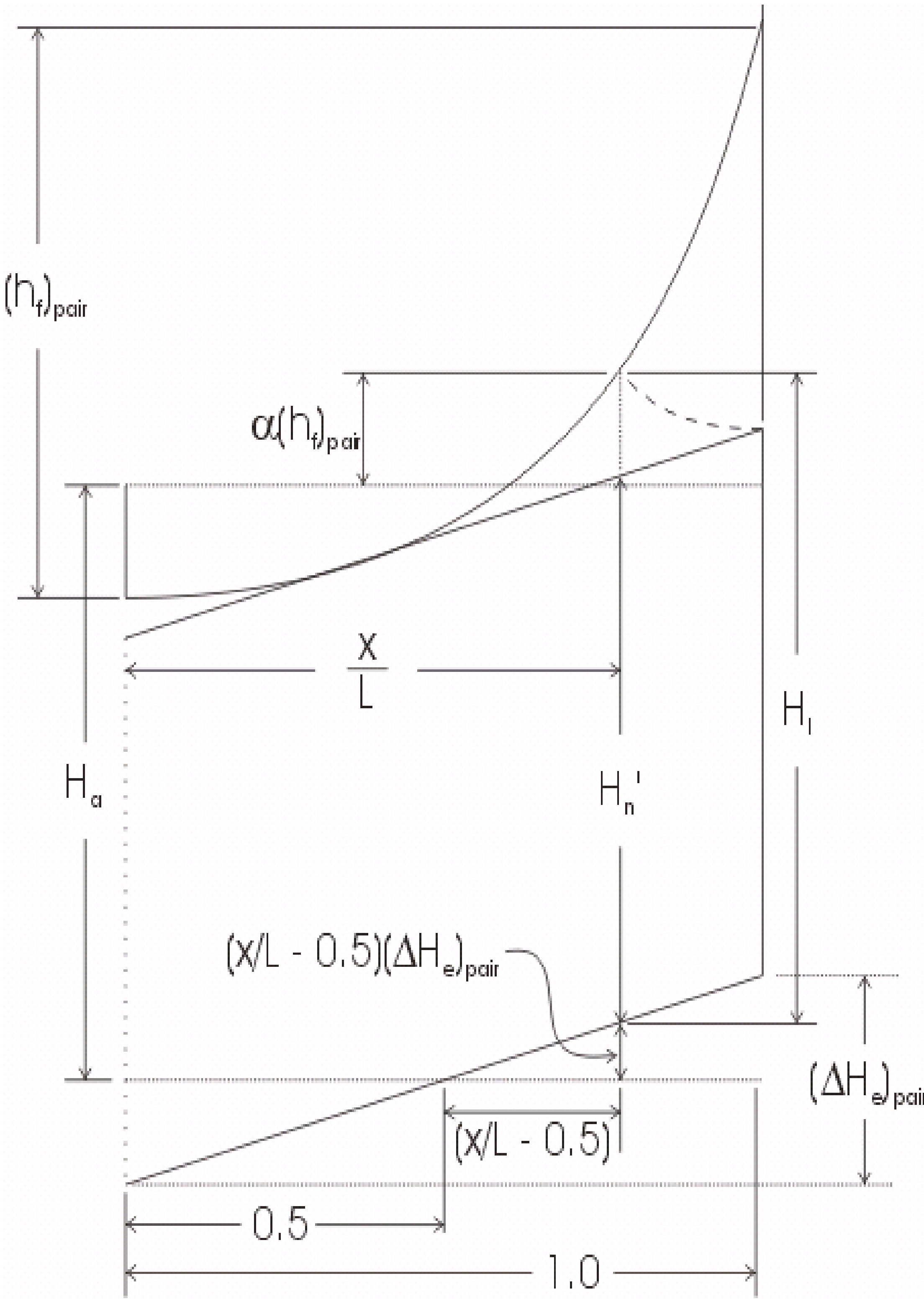
3. Combining equations:

$$\alpha\left(h_f\right)_{pair} = \frac{3}{4}\left(h_f\right)_{pair}\left[\left(\frac{x}{L}\right)\left(\frac{x}{L}\right)^{2.75} + \left(1-\frac{x}{L}\right)\left(1-\frac{x}{L}\right)^{2.75}\right] \tag{435}$$

$$\alpha = \frac{3}{4}\left[\left(\frac{x}{L}\right)^{3.75} + \left(1-\frac{x}{L}\right)^{3.75}\right] \tag{436}$$



- This last equation for α is Eq. 22.25 from the textbook
- See the figure below



$(h_f)_{pair}$
$\alpha(h_f)_{pair}$
$\frac{X}{L}$
H_l
H_a
H_n'
$(X/L - 0.5)(\Delta H_e)_{pair}$
$(\Delta H_e)_{pair}$
$(X/L - 0.5)$
0.5
1.0

Lecture 23

Manifold Hydraulic Design

I. Introduction

- Manifolds in trickle irrigation systems often have multiple pipe sizes to:

  1. reduce pipe costs
  2. reduce pressure variations

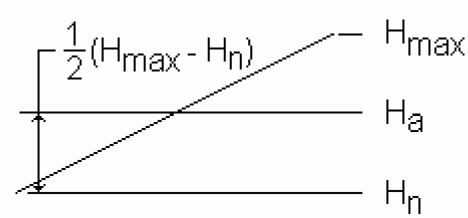
- In small irrigation systems the reduction in pipe cost may not be significant, not to mention that it is also easier to install a system with fewer pipe sizes
- Manifold design is normally subsequent to lateral design, but it can be part of an iterative process (i.e. design the laterals, design the manifold, adjust the lateral design, etc.)
- The allowable head variation in the manifold, for manifolds as subunits, is given by the allowable subunit head variation (Eq. 20.14) and the calculated lateral head variation, ΔH_l
- This simple relationship is given in Eq. 23.1:

$$\left(\Delta H_m\right)_a = \Delta H_s - \Delta H_l \tag{437}$$

- Eq. 23.1 simply says that the allowable subunit head variation is shared by the laterals and manifold
- Recall that a starting design point can be to have $\Delta H_l = ½\Delta H_s$, and $\Delta H_m = ½\Delta H_s$, but this *half and half* proportion can be adjusted during the design iterations
- The lateral pressure variation, ΔH_l, is equal to the maximum pressure minus the minimum pressure, which is true for single-direction laterals and uphill+downhill pairs, if H_n' is the same both uphill and downhill

II. Allowable Head Variation

- Equation 20.14 (page 502 in the textbook) gives the allowable pressure head variation in a "subunit"
- This equation is an approximation of the true allowable head variation, because this equation is applied before the laterals and manifold are designed
- After designing the laterals and manifold, the actual head variation and expected EU can be recalculated



- Consider a linear friction loss gradient (no multiple outlets) on flat ground:

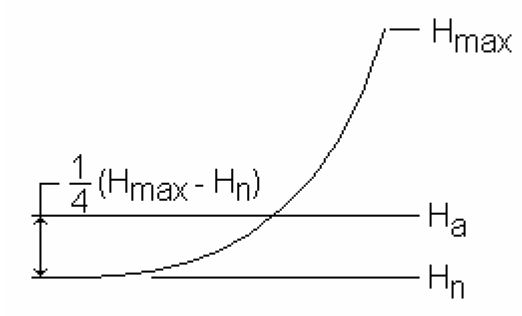
  In this case, the average head is

equal to H_n plus half the difference in the maximum and minimum heads:

$$H_{max} - H_n = 2\left(H_a - H_n\right) \tag{438}$$

- Consider a sloping friction loss gradient (multiple outlets) on flat ground:

  In this case, the average head occurs after about ¾ of the total head loss (due to friction) occurs, beginning from the lateral inlet. Then,



$$H_{max} - H_n = 4\left(H_a - H_n\right) \tag{439}$$

- For a sloping friction loss gradient (multiple outlets) on flat ground with dual pipe sizes, about 63% of the friction head loss occurs from the lateral inlet to the location of average pressure. Then 100/(100-63) = 2.7 and,

$$H_{max} - H_n = 2.7\left(H_a - H_n\right) \tag{440}$$

- In summary, an averaging is performed to skew the coefficient toward the minimum value of 2, recognizing that the maximum is about 4, and that for dual-size laterals (or manifolds), the coefficient might be approximately 2.7
- The value of 2.5 used in Eq. 20.14 is such a weighted average
- With three or four pipe sizes the friction loss gradient in the manifold will approach the slope of the ground, which may be linear
- Thus, as an initial estimate for determining allowable subunit pressure variation for a given design value of EU, Eq. 20.14 is written as follows:

$$\Delta H_s = 2.5\left(H_a - H_n\right) \tag{441}$$

- After the design process, the final value of ΔH_s may be different, but if it is much different the deviation should be somehow justified

III. Pipe Sizing in Manifolds

- Ideally, a manifold design considers all of the following criteria:

    1. economic balance between pipe cost (present) and pumping costs (future)
    2. allowable pressure variation in the manifold and subunit
    3. pipe flow velocity limits (about 1.5 - 2.0 m/s)

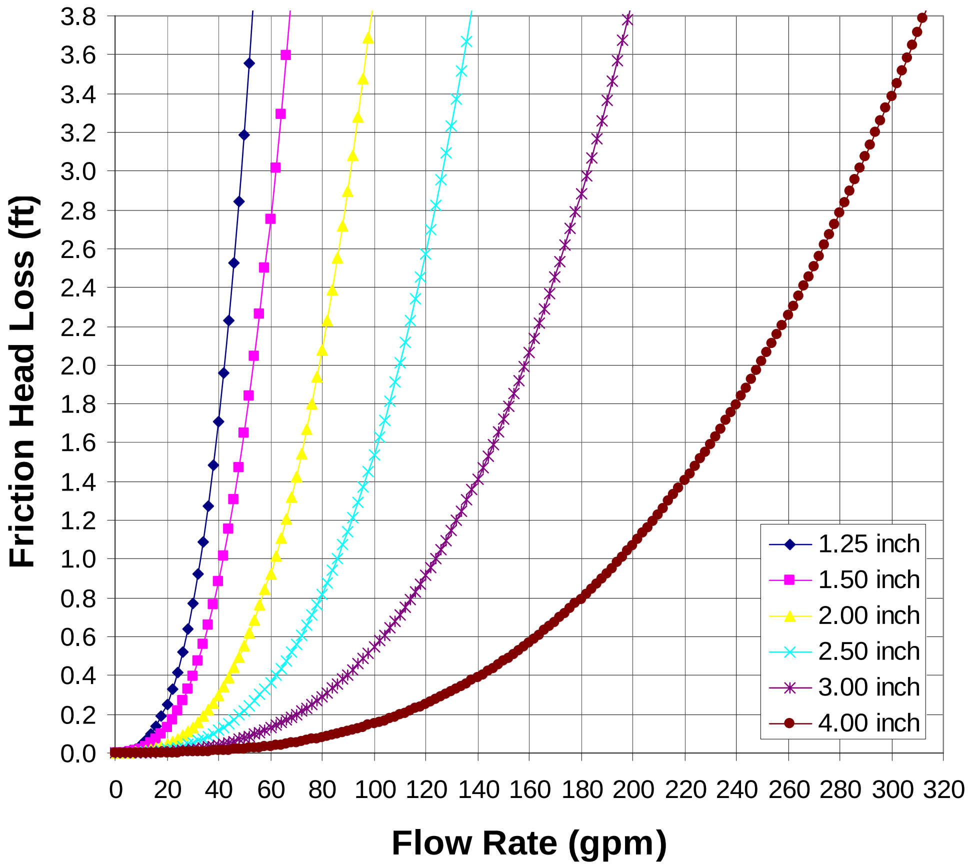
- From sprinkler system design, we already know of various pipe sizing methods
- These methods can also be applied to the design of manifolds
- However, the difference with trickle manifolds is that instead of one or two pipe sizes, we may be using three or four sizes
- The manifold design procedures described in the textbook are:

    1. Semi-graphical
    2. Hydraulic grade line (HGL)
    3. Economic pipe sizing (as in Chapter 8 of the textbook)

Semi-Graphical Design Procedure

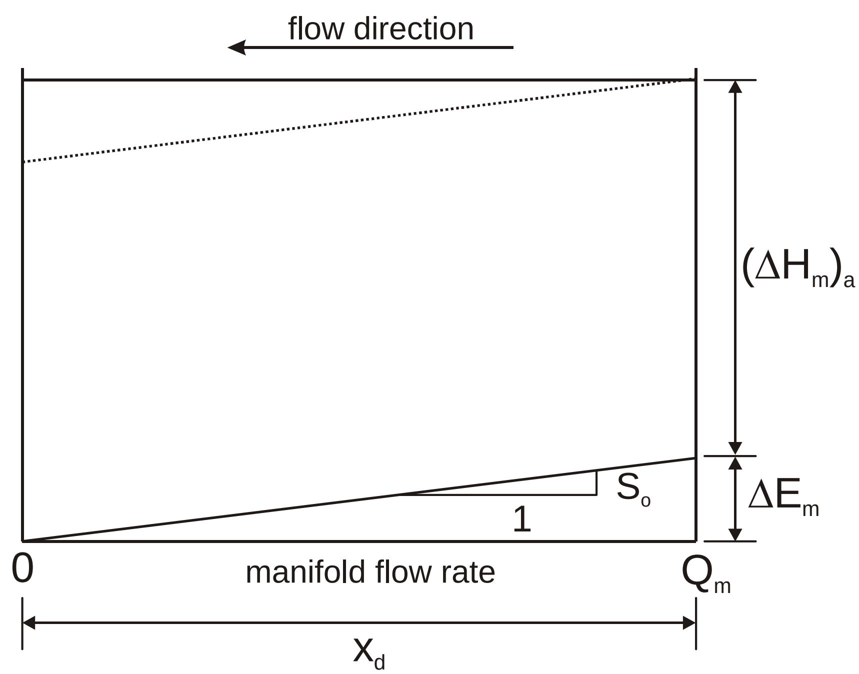
- The graphical method uses "standard" head loss curves for different pipe sizes and different flow rates with equally-spaced multiple outlets, each outlet with the same discharge
- The curves all intersect at the origin (corresponding to the downstream closed end of a pipe)
- Below is a sample of the kind of curves given in Fig. 23.2 of the textbook

- Instead of the standard curves, specific curves for each design case could be custom developed and plotted as necessary in spreadsheets
- The steps to complete a graphical design are outlined in the textbook
- The graphical procedure is helpful in understanding the hydraulic design of multiple pipe size manifolds, but may not be as expedient as fully numerical procedures

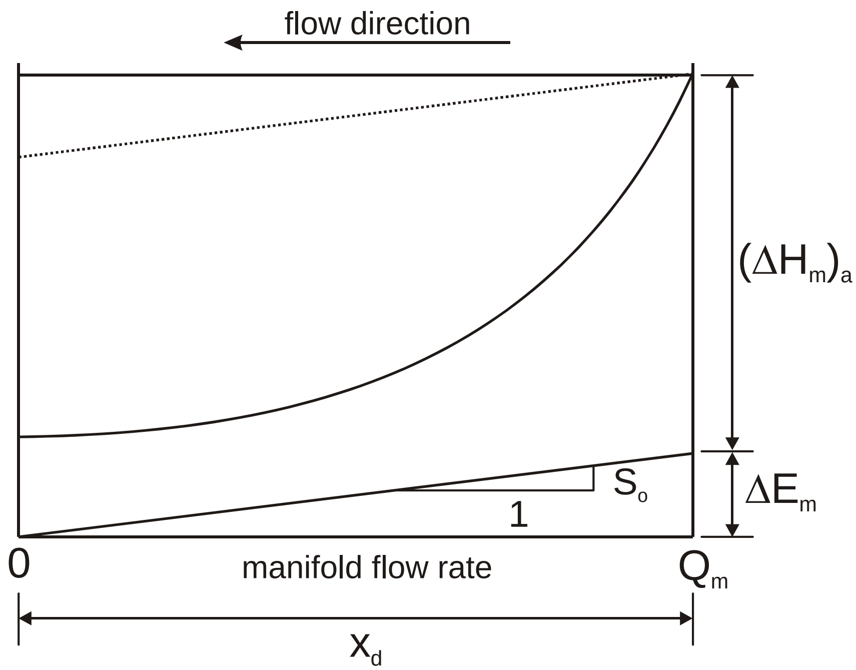


- The following steps illustrate the graphical design procedure:

Step 1:

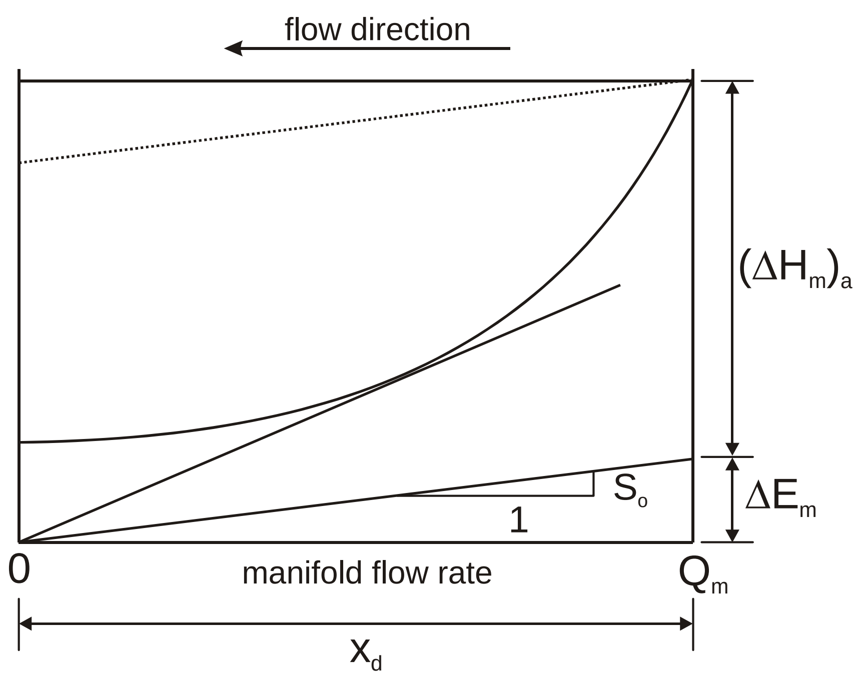


Step 2:



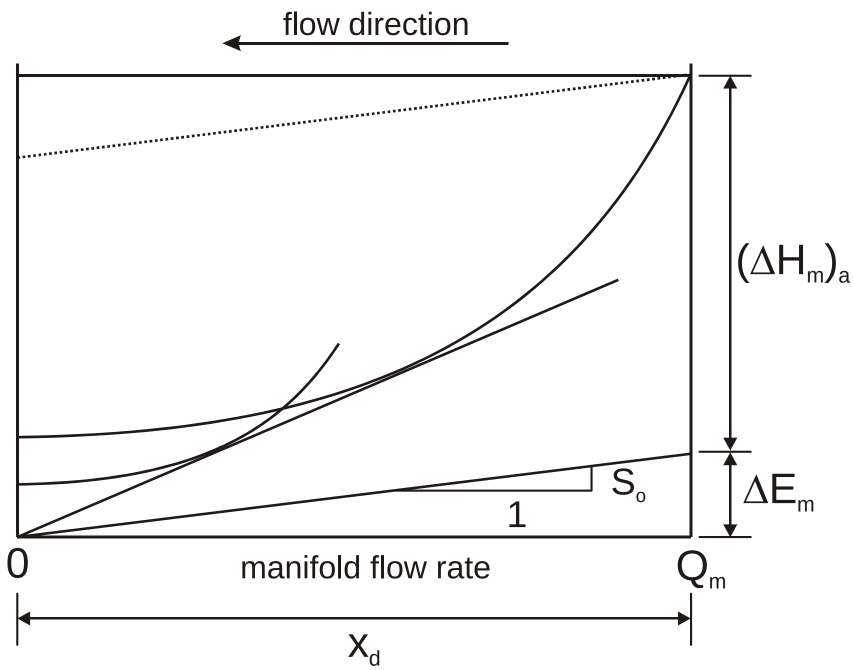
flow direction
$(\Delta H_m)_a$
S_o
1
ΔE_m
0
manifold flow rate
Q_m
x_d

Step 3:



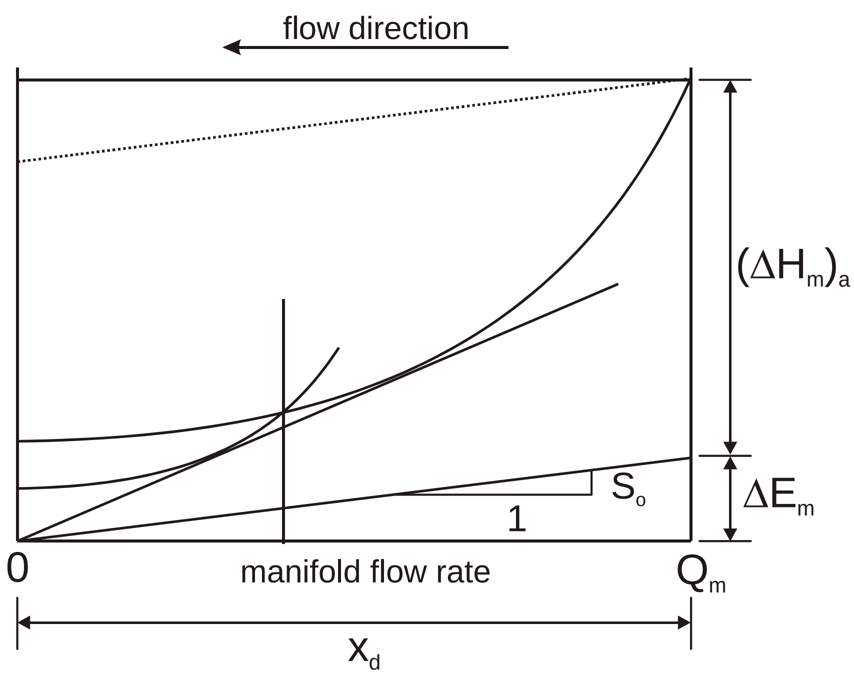
flow direction
$(\Delta H_m)_a$
S_o
1
ΔE_m
0
manifold flow rate
Q_m
x_d

Step 4:



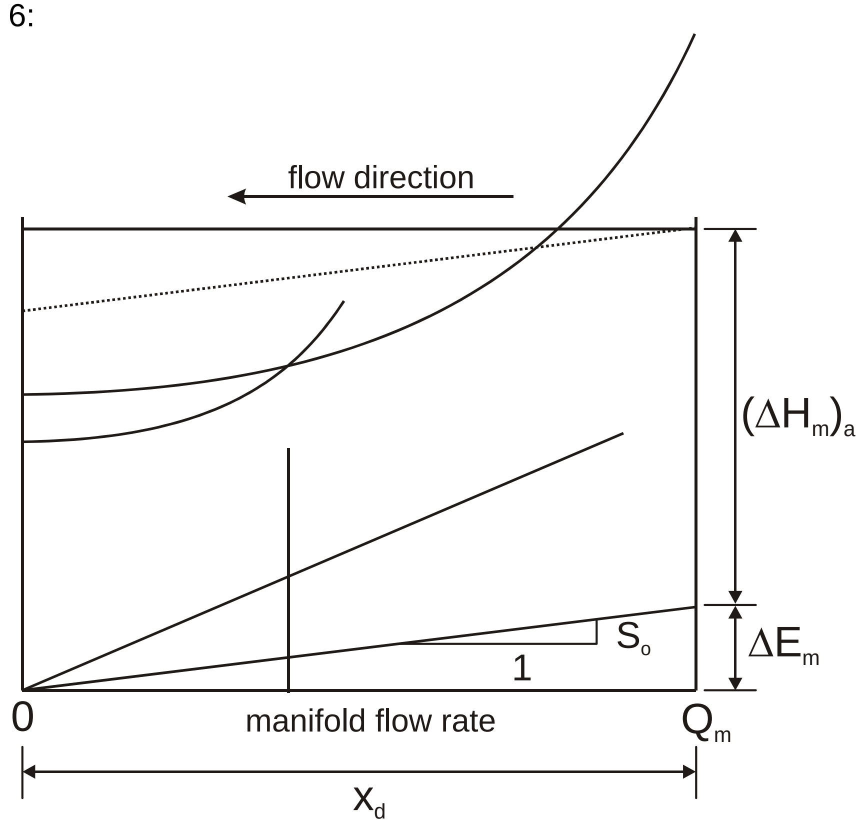
flow direction
$(\Delta H_m)_a$
S_o
1
ΔE_m
0
manifold flow rate
Q_m
x_d

Step 5:



flow direction
$(\Delta H_m)_a$
S_o
1
ΔE_m
0
manifold flow rate
Q_m
x_d

Step 6:



HGL Design Procedure

- The HGL procedure is very similar to the graphical procedure, except that it is applied numerically, without the need for graphs
- Nevertheless, it is useful to graph the resulting hydraulic curves to check for errors or infeasibilities
- The first (upstream) head loss curve starts from a fixed point: maximum discharge in the manifold and upper limit on head variation
- Equations for friction loss curves of different pipe diameters are known (e.g. Darcy-Weisbach, Hazen-Williams), and these can be equated to each other to determine intersection points, that is, points at which the pipe size would change in the manifold design
- But, before equating head loss equations, the curves must be vertically shifted so they just intersect with the ground slope curve (or the tangent to the first, upstream, curve, emanating from the origin)
- The vertical shifting can be done graphically or numerically

Economic Design Procedure

- The economic design procedure is essentially the same as that given in Chapter 8 of the textbook

- The manifold has multiple outlets (laterals or headers), and the "section flow rate" changes between each outlet
- The "system flow rate" would be the flow rate entering the manifold

IV. Manifold Inlet Pressure Head

- After completing the manifold pipe sizing, the required manifold inlet pressure head can be determined (Eq. 23.4):

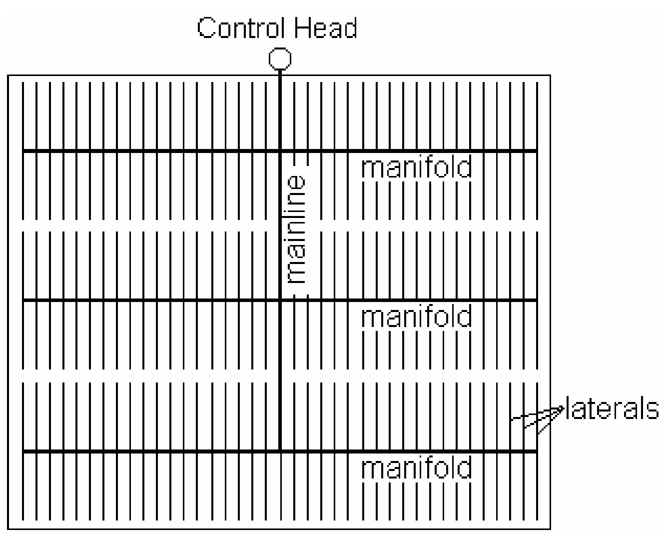
$$H_m = H_l + k\,h_f + 0.5\Delta E_m \tag{442}$$

where k = 0.75 for single-diameter manifolds; k = 0.63 for dual pipe size laterals; or k ≈ 0.5 for three or more pipe sizes (tapered manifolds); and ΔE_l is negative for downward-sloping manifolds

- As with lateral design, the friction loss curves must be shifted up to provide for the required average pressure
- In the case of manifolds, we would like the average pressure to be equal to the calculated lateral inlet head, H_l
- The parameter ΔE_l is the elevation difference along one portion of the manifold (either uphill or downhill), with positive values for uphill slopes and negative values for downhill slopes

V. Manifold Design

- Manifolds should usually extend both ways from the mainline to reduce the system cost, provided that the ground slope in the direction of the manifolds is less than about 3% (same as for laterals, as in the previous lectures)
- As shown in the sample layout (plan view) below, manifolds are typically orthogonal to the mainline, and laterals are orthogonal to the manifolds



- Manifolds usually are made up of 2 to 4 pipe diameters, tapered (telescoping) down toward the downstream end
- For tapered manifolds, the smallest of the pipe diameters (at the downstream end) should be greater than about ½ the largest diameter (at the upstream end) to help avoid clogging during flushing of the manifold

- The maximum average flow velocity in each pipe segment should be less than about 2 m/s
- Water hammer is not much of a concern, primarily because the manifold has multiple outlets (which rapidly attenuates a high- or low-pressure wave), but the friction loss increases exponentially with flow velocity

VI. Trickle Mainline Location

- The objective is the same as for pairs of laterals: make $(H_n)_{uphill}$ equal to $(H_n)_{downhill}$
- If average friction loss slopes are equal for both uphill and downhill manifold branches (assuming similar diameters will carry similar flow rates):

Downhill side:

$$\left(\Delta H_m\right)_a = h_{fd} - \Delta E\left(\frac{x}{L}\right) = h_{fd} - Y\Delta E \tag{443}$$

Uphill side:

$$\left(\Delta H_m\right)_a = h_{fu} + \Delta E\left(\frac{L-x}{L}\right) = h_{fu} + (1-Y)\Delta E \tag{444}$$

where x is the length of downhill manifold (m or ft); L is the total length of the manifold (m or ft); Y equals x/L; and ΔE is the absolute elevation difference of the uphill and downhill portions of the manifold (m or ft)

- Note that in the above, ΔE is an absolute value (always positive)
- Then, the average uphill and downhill friction loss slopes are equal:

$$\begin{gathered}\bar{J}_{uphill} = \bar{J}_{downhill} \\ \frac{h_{fu}}{L-x} = \frac{h_{fd}}{x}\end{gathered} \tag{445}$$

where J-bar is the average friction loss gradient from the mainline to the end of the manifold (J-bar is essentially the same as JF)

Then,

$$h_{fd} = \bar{J}x$$
$$h_{fu} = \bar{J}(L - x) \tag{446}$$

and,

$$(\Delta H_m)_a = \bar{J}x - Y\Delta E$$
$$(\Delta H_m)_a = \bar{J}(L - x) + (1 - Y)\Delta E \tag{447}$$

then,

$$\frac{(\Delta H_m)_a + Y\Delta E}{x} = \bar{J}$$
$$\frac{(\Delta H_m)_a - (1 - Y)\Delta E}{L - x} = \bar{J} \tag{448}$$

- Equating both J-bar values,

$$\frac{(\Delta H_m)_a + Y\Delta E}{x} = \frac{(\Delta H_m)_a - (1 - Y)\Delta E}{L - x} \tag{449}$$

- Dividing by L and rearranging (to get Eq. 23.3),

$$\frac{(\Delta H_m)_a + Y\Delta E}{Y} = \frac{(\Delta H_m)_a - (1 - Y)\Delta E}{1 - Y} \tag{450}$$

or,

$$\frac{\Delta E}{(\Delta H_m)_a} = \frac{2Y - 1}{2Y(1 - Y)} \tag{451}$$

- Equation 23.3 is used to determine the lengths of the uphill and downhill portions of the manifold
- You can solve for Y (and x), given ΔE and $(\Delta H_m)_a = \Delta H_s - \Delta H_l$
- Remember that $\Delta H_s \approx 2.5(H_a - H_n)$, where H_a is for the average emitter and H_n is for the desired EU and ν_s
- Equation 23.3 can be solved by isolating one of the values for Y on the left hand side, such that:

$$Y = 1 - \left(\frac{2Y - 1}{2Y}\right)\left(\frac{(\Delta H_m)_a}{\Delta E}\right) \tag{452}$$

and assuming an initial value for Y (e.g. Y = 0.6), plugging it into the right side of the equation, then iterating to arrive at a solution

- Note that $0 \le Y \le 1$, so the solution is already well-bracketed
- Note that in the trivial case where $\Delta E = 0$, then $Y = 0.5$ (don't apply the above equation, just use your intuition!)
- A numerical method (e.g. Newton-Raphson) can also be used to solve the equation for Y

VII. Selection of Manifold Pipe Sizes

The selection of manifold pipe sizes is a function of:

1. Economics, where pipe costs are balanced with energy costs
2. Balancing h_f, ΔE, and $(\Delta H_m)_a$ to obtain the desired EU
3. Velocity constraints

VIII. Manifold Pipe Sizing by Economic Selection Method

- This method is similar to that used for mainlines of sprinkler systems
- Given the manifold spacing, S_m, and the manifold length, do the following:

(a) Construct an economic pipe size table where $Q_s = Q_m$
(b) Select appropriate pipe diameters and corresponding Q values at locations where the diameters will change
(c) Determine the lengths of each diameter of pipe (where the Q in the manifold section equals a breakeven Q from the *Economic Pipe Size Table* (EPST)

$$L_D = L\left(\frac{Q_{beg} - Q_{end}}{Q_m}\right) \tag{453}$$

where Q_{beg} is the flow rate at the beginning of diameter "D" in the EPST (lps or gpm); Q_{end} is the flow rate at the end of diameter "D" in the EPST, which is the breakeven flow rate of the next larger pipe size) (lps or gpm); L is the total length of the manifold (m or ft); and Q_m is the manifold inflow rate (lps or gpm). (see Eq. 23.7)

(d) Determine the total friction loss along the manifold (see Eq. 23.8a):

$$h_f = \frac{FLK}{100\,Q_m}\left(\frac{Q_1^a}{D_1^c} + \frac{Q_2^a - Q_1^a}{D_2^c} + \frac{Q_3^a - Q_2^a}{D_3^c} + \frac{Q_4^a - Q_3^a}{D_4^c}\right) \tag{454}$$

where,

a = b+1 (for the Blasius equation, a = 2.75)
c = 4.75 for the Blasius equation (as seen previously)

Q_1 = Q at the beginning of the smallest pipe diameter
Q_2 = Q at the beginning of the next larger pipe diameter
Q_3 = Q at the beginning of the third largest pipe diameter in the manifold
Q_4 = Q at the beginning of the largest pipe diameter in the manifold
F = multiple outlet pipe loss factor

- For the Hazen-Williams equation, F equals 1/(1.852+1) = 0.35
- For the Darcy-Weisbach equation, F equals 1/(2+1) = 0.33

L = the total length of the manifold
D = inside diameter of the pipe
K = $7.89(10)^7$ for D in mm, Q in lps, and length in m
K = 0.133 for D in inches, Q in gpm, and length in ft
h_f = friction head loss

- The above equation is for four pipe sizes; if there are less than four sizes, the extra terms are eliminated from the equation
- An alternative would be to use Eq. 23.8b (for known pipe lengths), or evaluate the friction loss using a computer program or a spreadsheet to calculate the losses section by section along the manifold
- Eq. 23.8b is written for manifold design as follows:

$$h_f = \frac{FKQ_m^{a-1}}{100L^{a-1}}\left(\frac{x_1^a}{D_1^c} + \frac{x_2^a - x_1^a}{D_2^c} + \frac{x_3^a - x_2^a}{D_3^c} + \frac{x_4^a - x_3^a}{D_4^c}\right) \qquad (455)$$

where, x_1 = length of the smallest pipe size
x_2 = length of the next smaller pipe size
x_3 = length of the third largest pipe size
x_4 = length of the largest pipe size

- Again, there may be up to four different pipe sizes in the manifold, but in many cases there will be less than four sizes

(e) For $s \geq 0$ (uphill branch of the manifold),

$$\Delta H_m = h_f + S x_u \qquad (456)$$

For $s < 0$ (downhill branch of the manifold),

$$\Delta H_m = h_f + S\left(1 - \frac{0.36}{n}\right)x_d \qquad (457)$$

where n is the number of different pipe sizes used in the branch; and S is the ground slope in the direction of the manifold (m/m)

- The above equation estimates the location of minimum pressure in the downhill part of the manifold

(f) if ΔH_m < 1.1 $(\Delta H_m)_a$, then the pipe sizing is all right. Go to step (g) of this procedure. Otherwise, do one or more of the following eight adjustments:

(1) Increase the pipe diameters selected for the manifold

- Do this proportionately by reselecting diameters from the EPST using a larger Q_s (to increase the energy "penalty" and recompute a new EPST). This will artificially increase the break-even flow rates in the table (chart).
- The new flow rates to use in re-doing the EPST can be estimated for s > 0 as follows:

$$Q_s^{new} = Q_s^{old}\left(\frac{h_f}{(\Delta H_m)_a - \Delta E_m}\right)^{1/b} \tag{458}$$

and for s < 0 as:

$$Q_s^{new} = Q_s^{old}\left(\frac{h_f}{(\Delta H_m)_a - \Delta E_l\left(1 - \frac{0.36}{n}\right)}\right)^{1/b} \tag{459}$$

- The above two equations are used to change the flow rates to compute the EPST
- The value of Q_m remains the same
- The elevation change along each manifold (uphill or downhill branches) is $\Delta El = sL/100$

(2) Decrease S_m

- This will make the laterals shorter, Q_m will decrease, and ΔH_l may decrease
- This alternative may or may not help in the design process

(3) Reduce the target EU

- This will increase ΔH_s

(4) Decrease ΔH_l (use larger pipe sizes)

- This will increase the cost of the pipes

(5) Increase H_a

- This will increase ΔH_s
- This alternative will cost money and or energy

(6) Reduce the manufacturer's coefficient of variation

- This will require more expensive emitters and raise the system cost

(7) Increase the number of emitters per tree (N_p)

- This will reduce the value of ν_s

(8) If $N_s > 1$, increase T_a per station

- Try operating two or more stations simultaneously

- Now go back to Step (b) and repeat the process.

(g) Compute the manifold inlet head,

$$H_m = H_l + k h_f + 0.5\Delta E_m \tag{460}$$

where, k = 0.75 for a single size of manifold pipe
k = 0.63 for two pipe sizes
k = 0.50 for three or more sizes

- For non-critical manifolds, or where $\Delta H_m < (\Delta H_m)_a$, decrease Q_s (or just design using another sizing method) in the Economic Pipe Selection Table to dissipate excess head
- For non-rectangular subunits, adjust F using a shape factor:

$$F_s = 0.38 S_f^{1.25} + 0.62 \tag{461}$$

where $S_f = Q_{lc}/Q_{la}$; Q_{lc} is the lateral discharge at the end of the manifold and Q_{la} is the average lateral discharge along the manifold. Then,

$$h_f = F_s F\left(\frac{JL}{100}\right) \tag{462}$$

IX. Manifold Pipe Sizing by the "HGL" Method

- This is the "Hydraulic Grade Line" method
- Same as the semi-graphical method, but performed numerically

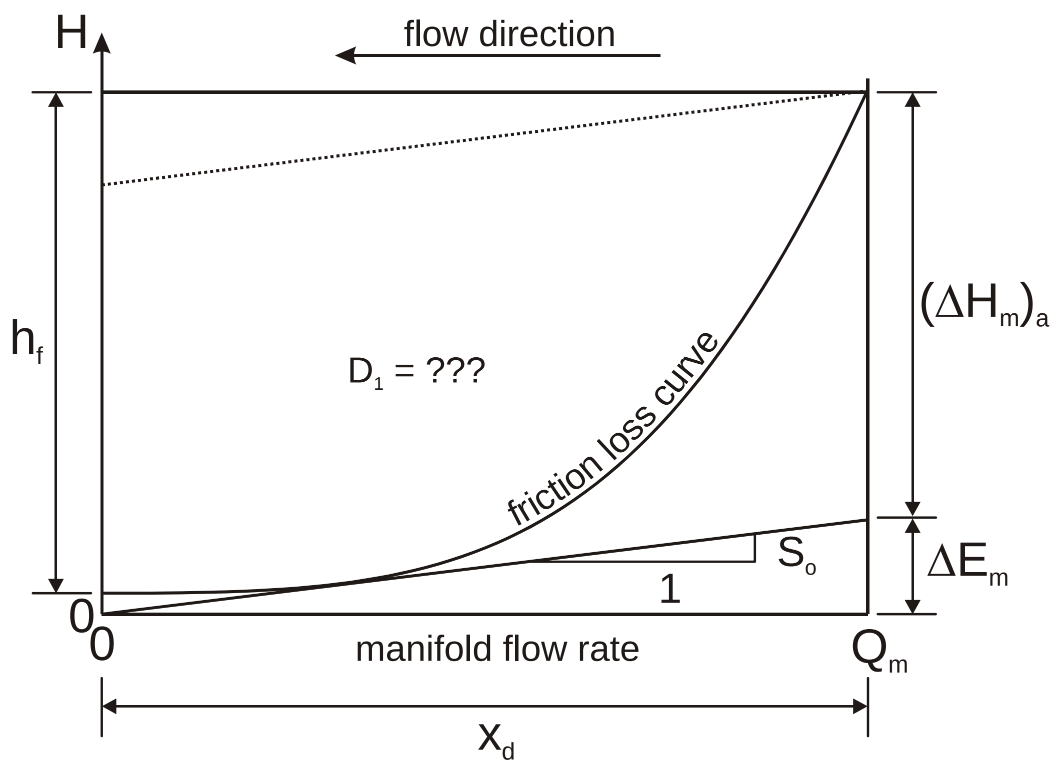
(a) Uphill Side of the Manifold

- Get the smallest allowable pipe diameter and use only the one diameter for this part of the manifold

(b) Downhill Side of the Manifold

Largest Pipe Size, D_1

- First, determine the minimum pipe diameter for the first pipe in the downhill side of the manifold, which of course will be the largest of the pipe sizes that will be used
- This can be accomplished by finding the inside pipe diameter, D, that will give a friction loss curve tangent to the ground slope
- To do this, it is necessary to: (1) have the slope of the friction loss curve equal to S_o; and, (2) have the H values equal at this location (make them just touch at a point)
- These two requirements can be satisfied by applying two equations, whereby the two unknowns will be Q and D_1
- Assume that Q_l is constant along the manifold…
- See the following figure, based on the length of the downstream part of the manifold, x_d
- Some manifolds will only have a downhill part – others will have both uphill and downhill parts



- For the above figure, where the right side is the mainline location and the left side is the downstream closed end of the manifold, the friction loss curve is defined as:

$$H = \left(\Delta H_m\right)_a + \Delta E_m - h_f + \frac{JFL}{100} \tag{463}$$

where, using the Hazen-Williams equation,

$$J = K\left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \quad \text{for} \quad 0 \le Q \le Q_m \tag{464}$$

$$F = \frac{1}{2.852} + \frac{1}{2N} + \frac{\sqrt{0.852}}{6N^2} \tag{465}$$

$$N = \left(\frac{x_d}{S_l}\right)\left(\frac{Q}{Q_m}\right) \quad \text{for} \quad N > 0 \tag{466}$$

where N is the number of outlets (laterals) from the location of "Q" in the manifold to the closed end

$$L = x_d\left(\frac{Q}{Q_m}\right) \tag{467}$$

For Q in lps and D in cm, $K = 16.42(10)^6$

- The total head loss in the downhill side of the manifold is:

$$h_f = \frac{J_{hf} F_{hf} x_d}{100} = 0.01K\left(\frac{Q_m}{C}\right)^{1.852} D^{-4.87} F_{hf} x_d \tag{468}$$

where F_{hf} is defined as F above, except with $N = x_d/S_l$.

- The slope of the friction loss curve is:

$$\frac{dH}{dQ} = \frac{1}{100}\left(FL\frac{dJ}{dQ} + JL\frac{dF}{dQ} + JF\frac{dL}{dQ}\right) \tag{469}$$

where,

$$\frac{dJ}{dQ} = \frac{1.852 K Q^{0.852}}{C^{1.852} D^{4.87}} \tag{470}$$

$$\frac{dF}{dQ} = -\frac{x_d}{S_l Q_m N^2}\left(\frac{1}{2} + \frac{\sqrt{0.852}}{3N}\right) \tag{471}$$

$$\frac{dL}{dQ} = \frac{x_d}{Q_m} \tag{472}$$

- Note that $dH/dQ \neq J$
- The ground surface (assuming a constant slope, S_o) is defined by:

$$H = S_o L = S_o x_d \left(\frac{Q}{Q_m}\right) \tag{473}$$

and,

$$\frac{dH}{dQ} = \frac{S_o x_d}{Q_m} \tag{474}$$

- Combine the two equations defining H (this makes the friction loss curve just touch the ground surface):

$$S_o x_d \left(\frac{Q}{Q_m}\right) = \left(\Delta H_m\right)_a + \Delta E_m - h_f + \frac{JFL}{100} \tag{475}$$

- Solve the above equation for the inside diameter, D:

$$D = \left[\frac{100\, C^{1.852}\left(\frac{S_o x_d Q}{Q_m} - \left(\Delta H_m\right)_a - \Delta E_m\right)}{K\left(Q^{1.852} FL - Q_m^{1.852} F_{hf} x_d\right)}\right]^{-0.205} \tag{476}$$

- Set the slope of the friction loss curve equal to $S_o x_d / Q_m$,

$$\frac{S_o x_d}{Q_m} = \frac{1}{100}\left(FL\frac{dJ}{dQ} + JL\frac{dF}{dQ} + JF\frac{dL}{dQ}\right) \tag{477}$$

- Combine the above two equations so that the only unknown is Q (note: D appears in the J & dJ/dQ terms of the above equation)
- Solve for Q by iteration; the pipe inside diameter, D, will be known as part of the solution for Q
- The calculated value of D is the minimum inside pipe diameter, so find the nearest available pipe size that is larger than or equal to D:

$$D_1 \geq D \quad \& \quad \text{minimize}(D_1 - D) \tag{478}$$

Slope of the Tangent Line

- Now calculate the equation of the line through the origin and tangent to the friction loss curve for D_1
- Let S_t be the slope of the tangent line

$$H = S_t L = S_t x_d \left(\frac{Q}{Q_m} \right) \tag{479}$$

then,

$$S_t x_d \left(\frac{Q}{Q_m} \right) = \left(\Delta H_m \right)_a + \Delta E_l - h_f + \frac{JFL}{100} \tag{480}$$

- Set the slope of the friction loss curve equal to $S_t x_d / Q_m$,

$$\frac{S_t x_d}{Q_m} = \frac{1}{100} \left(FL \frac{dJ}{dQ} + JL \frac{dF}{dQ} + JF \frac{dL}{dQ} \right) \tag{481}$$

- Combine the above two equations to eliminate S_t, and solve for Q (which is different than the Q in Eq. 476)
- Calculate the slope, S_t, directly

Smaller (Downstream) Pipe Sizes

- Then take the next smaller pipe size, D_2, and make its friction loss curve tangent to the same line (slope = S_t);

$$H = H_0 + \frac{JFL}{100} \tag{482}$$

where H_0 is a vertical offset to make the friction loss curve tangent to the S_t line, emanating from the origin

- Equating heads and solving for H_0,

$$H_0 = S_t x_d \left(\frac{Q}{Q_m} \right) - \frac{JFL}{100} \tag{483}$$

- Again, set the slope of the friction loss curve equal to S_t,

$$\frac{S_t x_d}{Q_m} = \frac{1}{100} \left(FL \frac{dJ}{dQ} + JL \frac{dF}{dQ} + JF \frac{dL}{dQ} \right) \tag{484}$$

- Solve the above equation for Q, then solve directly for H_0
- Now you have the equation for the next friction loss curve
- Determine the intersection with the D_1 friction loss curve to set the length for size D_1; this is done by equating the H values for the respective equations and solving for Q at the intersection:

$$H_{big} - H_{small} + \frac{FLK}{100} \left(\frac{Q}{C} \right)^{1.852} \left(D_{big}^{-4.87} - D_{small}^{-4.87} \right) = 0 \tag{485}$$

where, for the first pipe size (D_1):

$$H_{big} = \left(\Delta H_m \right)_a + \Delta E_l - h_f \tag{486}$$

and for the second pipe size (D_2):

$$H_{small} = H_0 \tag{487}$$

and F & L are as defined in Eqs. 437 to 439.

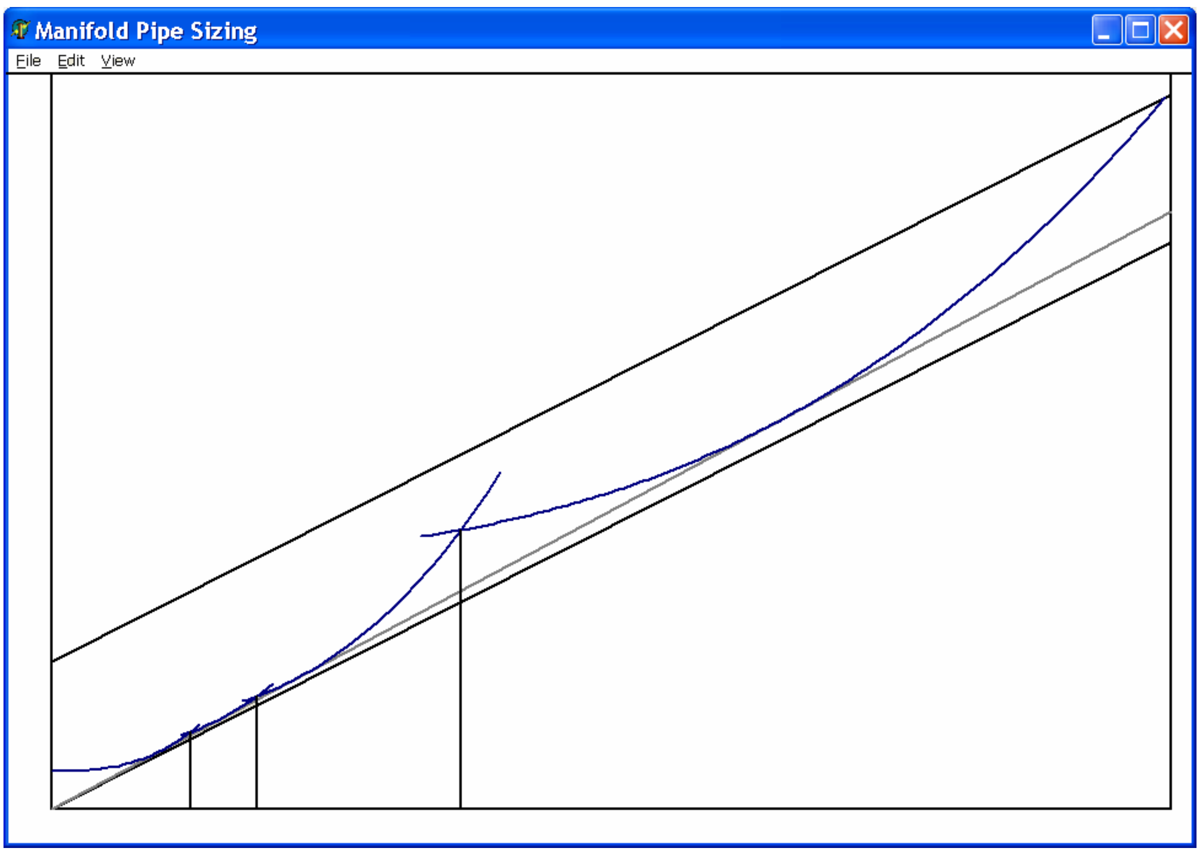
- Then, the length of pipe D_1 is equal to:

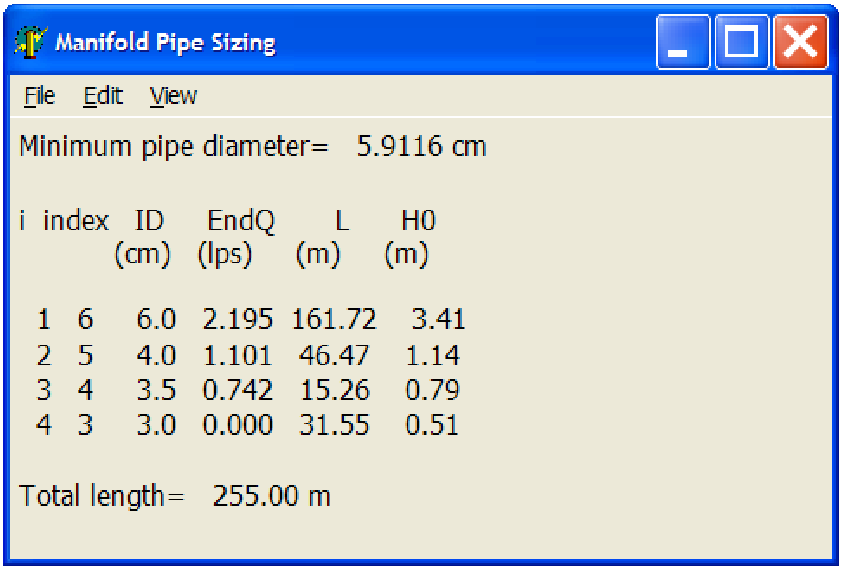
$$L_{D1} = x_d \left(1 - \frac{Q}{Q_m} \right) \tag{488}$$

- Continue this process until you have three or four pipe sizes, or until you get to a pipe size that has $D < ½D_1$

Comments about the HGL Method

- The above equation development could also be done using the Darcy-Weisbach equation
- Specify a minimum length for each pipe size in the manifold so that the design is not something ridiculous (i.e. don't just blindly perform calculations, but look at what you have)
- For example, the minimum allowable pipe length might be something like $5S_l$
- Note that the friction loss curves must be shifted vertically upward to provide the correct average (or minimum, if pressure regulators are used) pressure head in the manifold; this shifting process determines the required manifold inlet pressure head, H_m
- Below is a screen shot from a computer program that uses the HGL method for manifold pipe sizing




Manifold Pipe Sizing

File Edit View

Minimum pipe diameter= 5.9116 cm

i	index	ID (cm)	EndQ (lps)	L (m)	H0 (m)
1	6	6.0	2.195	161.72	3.41
2	5	4.0	1.101	46.47	1.14
3	4	3.5	0.742	15.26	0.79
4	3	3.0	0.000	31.55	0.51

Total length= 255.00 m

Lecture 24

Hydraulic Design of Mainline & Supply Line

I. Introduction

- Chapter 24 of the textbook contains a good summary and discussion of the design process for trickle irrigation systems
- Keller & Bliesner divide the trickle irrigation design process into three phases:

    1. *Phase 1*: Planning factors, gross application depth, preliminary system capacity, filtration requirements, etc.

    2. *Phase 2*: Hydraulic design of laterals, manifolds, and subunit layout.

    3. *Phase 3*: Hydraulic design of main and supply lines, control head, and pumping plant.

II. Factors Affecting the EU

- Solomon (1985) studied the various factors affecting emission uniformity and ranked them according to importance using model studies and field surveys. This is his ranking:

    1. Plugging of emitters and other system components
    2. Number of emitters per plant, N_p
    3. Emitter coefficient of variation, ν
    4. Emitter exponent, x
    5. Emitter discharge variation with temperature
    6. Pressure head variations in subunits
    7. Coefficient of variation on pressure regulators
    8. Friction loss from manifolds to laterals
    9. Number of pipe sizes in manifolds

- In general, plugging (or the lack thereof) has the greatest influence on EU, except in cases where the system design is very poor
- Plugging cannot be prevented by design only -- it is up to the management to continuously maintain the system as necessary
- The above ranking implies that a poorly designed system might perform better (in terms of application uniformity) than a well designed system, provided that the system operation and maintenance is given due attention and effort

III. Mainline, Supply, and Control Head Design

- After designing laterals and manifolds, the mainline, supply line (if necessary), control head, and pump must be designed/selected. This involves calculating the TDH at the pump.
- Maximum system capacity would have already been determined in the preceding design steps, based on the number of subunits, their sizes, emitter discharge and spacing, and number of emitters per plant.
- Subunits should be designed to have nearly the same discharge so that the pump is not wasting energy in lower capacity subunits; but this is not always possible.
- Mainline design is the same as in sprinkle systems, and various pipe selection methods are available (including the economic pipe selection method).
- The control head includes the filters, sand separators, chemical injection equipment, flow rate and or volumetric meters, timers, pressure gauges, and/or other hardware. Most of this equipment is located in the same place in a trickle irrigation system. However, other filters, screens, and gauges may be installed at downstream locations in some cases.
- Sand media and other filters may have a combined head loss of 10-20 psi when "dirty", and 3-10 psi when clean. However, if too much flow is forced through the filters (i.e. not enough filter capacity) the head loss can be higher, even when clean.
- Flow meters and chemical injection equipment may have1-5 psi loss, and valves can have up to several psi loss (even when fully open).
- The pressure changes from the control head to the subunits are due to a combination of friction loss and elevation change: if the subunit inlet is at a higher elevation than the control head, both friction and elevation change contribute to a higher required pressure at the control head, otherwise they tend to cancel out (partially or completely).
- Mainline friction losses include minor (local) losses at bends, through valves, and through screens.
- Losses at reductions in pipe diameter (in series) are usually small, unless the reduction is sharp and abrupt. Diameter transitions for PVC are often smooth.
- A "critical" subunit can be defined by calculating the combined friction loss and elevation change to each subunit, and taking the highest value plus the required inlet pressure head to the manifold. The textbook calls this $(H_m + H_{fe})_c$.

- The critical subunit will define the “worst case” for which the pumping unit should be designed. It may then be possible to reduce pipe sizes in other subunits if they will have excess head available.

IV. Trickle System Mainline Design

- Trickle mainlines can be sized using the same approach as is used for sprinkle irrigation systems.
- Manifolds can be considered as stationary sprinkler laterals.
- You can use the EPST approach, where O_t depends (in part) on the number of stations, N_s

Example calculation:

- Size sections A-B, B-C and C-D for a trickle irrigation mainline having three manifolds
- First, decide on the number of stations, N_s

This could be 1, 3 or 6:

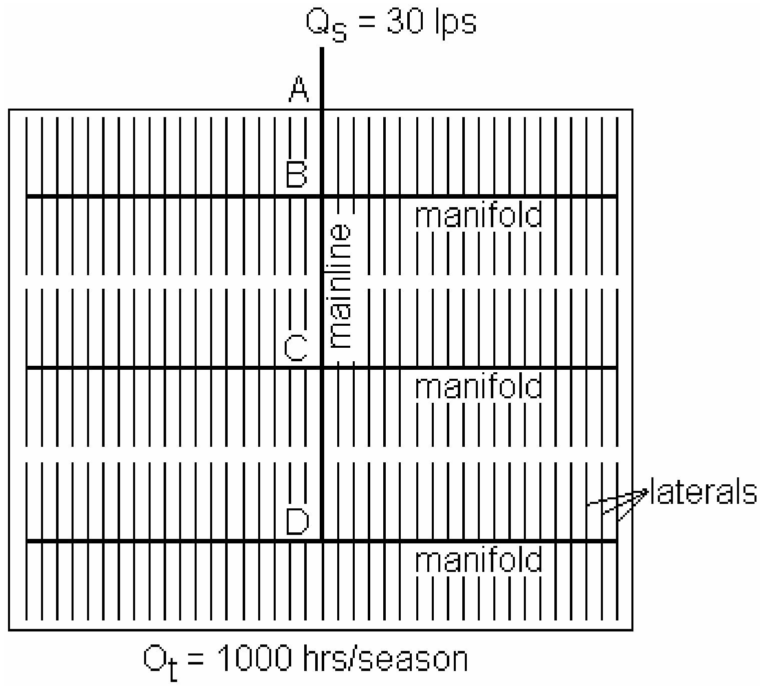
1. All laterals at the same time, or
2. Only laterals on one manifold at a time, or
3. Only laterals on one half of a manifold (on one side of the mainline) at a time

- Set up a table to see the effect of N_s on the flow rates and O_t per station (also see the figure below):

	Number of Stations		
	1	**2**	**3**
Q_s (lps)	30	30	30
O_t/station (hrs)	1000	330	165
$O_{t, CD}$	1000	330	330
Q_{CD}	10	30	30
$O_{t, BC}$	1000	660	660
Q_{BC}	20	30	30
$O_{t, AB}$	1000	1000	1000
Q_{AB}	30	30	30

- $O_{t, CD}$, for example, is the number of hours that water is flowing in section CD per season
- This is the O_t that would be used in creating an economic pipe sizing table to determine the pipe size of section CD
- O_t varies with the number of stations and with the location of the mainline segment in the system

- Therefore, the EPST must be developed several times (this is easy in a spreadsheet, as you have seen)



- Notice that the product of Q and O_t is a constant, as this is proportional to the irrigation water requirement, which is the same for each station and location in the field
- Pipe diameters are selected using Q_{AB}, Q_{BC}, and Q_{CD} as the break-even flow rates in the EPST; Q_s remains constant

The TDH is the sum of:

1. static lift
2. well losses (if applicable)
3. supply line losses (elevation and friction)
4. control head pressure losses
5. losses to the critical subunit plus inlet pressure, $(H_m + H_{fe})_c$
6. screen and valve losses (if applicable) at subunit inlets

- As in many hydraulic designs for pressurized pipe systems, it is often recommendable to add a “safety factor” to the losses, because losses are not precisely known, and they will probably increase with time
- A common safety factor is to add 10% to the friction losses for the calculation of TDH

- A separate safety factor can be added, in some cases, to the TDH for compensation for emitter plugging and degradation
- See sample calculations of TDH in Chapter 24

VI. We Live in an Imperfect World

- In many trickle systems, the ground slope changes significantly between subunits, or within subunits
- This complicates the system design and may require more pressure regulation and larger pipe sizes than would otherwise be necessary

- In many locations and countries the available pipes, fittings, emitters, filters, and other hardware are very limited
- Therefore, innovation, resourcefulness, and improvisation may be very important
- If hardware is very limited, it may be best to consider another type of irrigation system

- The system capacity for a micro-irrigation system can have a safety factor added on to account for the possibility of:

    1. slow, partial clogging of emitters and laterals
    2. changes in crop type
    3. inaccurate estimations of peak crop ET
    4. more system "down time" than originally anticipated
    5. various other factors

- It is desirable to minimize the hardware cost of an irrigation system, but the cost of having an insufficient system capacity may be many times higher than the marginal cost of larger pipes, filters and valves
- When in doubt, it may be a good idea to increase the calculated system capacity by 10 or 20%

- See Chapter 25 of the textbook for a thoughtful discussion of factors in selecting a sprinkler or trickle system, or for selecting another type of irrigation system

Sprinkle and Trickle Irrigation Text Errata

September 1, 2004

p. 17: Second line add comma between wetted and may to read: --- "but lower leaves, if wetter, may" -----.

p. 32: Equation 3.1. In definitions of terms, change "W_A" to "W_a"; and for depth Z change metric dimension from "mm" to "m".

p. 38: In full sentence above Table 3.5, pluralize value to read: ---"the EC_e values presented in Table 3.5" ----.

p. 39 End of fifth line from top should be changed from $LR < 0.1$ to $LR > 0.1$.

p. 41: Add an another "e" in Hargr(e)aves in second line and in References.

p. 46: Third line from bottom of third paragraph, change "as" to "an" to read: --- "making too sharp an "S" turn"----.

p. 65: In item 6, change "form" to "from".

p. 70: Fig. 5.6. Two errors: a) the wind speed should be "5 m/s" not "1 m/s"; and b) the sprinkler location "0" on the right hand (vertical) scale is not offset to reflect wind, it should be shifted to where the lower "6" is located.

p. 83: Table 5.4. Change last number in first column from "(0.40)" to "(0.15)".

p. 91: Fig. 6.2. Replace "Pivot" with "Pitot" in figure title.

p. 99: Table 6.2. For CU = 94% and pa = 80% the value given should be "94".

p. 131: Sample Calculation 7.1. In last line of GIVEN: change "5-ft" to "50-ft".

p.134: The value for the constant "K " below Equation 8.1 for metric units would be slightly more accurate if changed from 1.212 to 1.217×10^{12}.

p. 135: On third line above Equation (8.2), change "13-m" to "13-mm".

p.138: The value for the constant "K " below Equation 8.7a for metric units would be slightly more accurate if changed from 7.89 to 7.88×10^{7}.

p. 141: In the Flow rate column for (gpm): the 6.0 should be followed by "6.1" rather than "0.1".

p. 146: Two typos in the text one line above and two lines below Equation (8.9a), change "form" to "from".

p. 147: Sample Calculation 8.1. At end of last line, change "7.07" to "70.7".

p. 164: Table 8.12. The heading for the first column should simply be "Method", strike out "(or size for C-D)".

p. 167: Equation 8.19. Change "Table 8.11" to "Table 8.9" in definition for EAE(e).

p. 170: Fig. 8.8. The long FIXED PLUS OPERATING COSTS arrow should have had its arrowhead at and terminated at the point where it crossed the solid line.

p. 196: In last line, change "H_a" to "P_a".

p. 197: Fig. 9.7. In the left hand caption unit line, change "5 = 5%" to "S = 5%".

p. 197: Sample Calculation 9.8. In third line of CALCULATIONS: change "two-end outlets" to "two end-outlets".

p. 209: Sample Calculation 10.2. In the line above the fourth equation from the bottom of the page, the sentence should read, "The final h_{f2} with all 6-in. pipe is:"

p. 213: Equation 10.5. Change "L_S" to "L_s".

p. 218: Sample Calculation 10.5. In GIVEN: under Economic relationships: change "$\mathrm{C_p}$" to "C_p".

p. 219: Sample Calculation 10.5. In CALCULATION: Step 2, change the "0.0001" in the equation to "0.001".

p. 220: Sample Calculation 10.5. Clarification, in calculating L_4, $J_s = J_4 = 3.17$ m/100m and $J_b = J_6 = 0.52$ m/100m were interpolated from Table 8.5.

p. 222: Change "g = accumulation…" to "g = acceleration".

p. 223: Table 11.2. Change the bottom Equation for "sudden contractions" to:

$$\mathrm{K_r} = 0.7\left[1-(\mathrm{D_r})^2\right]^2$$

p. 226: Sample Calculation 11.1. In CALCULATIONS: in first line change "K" to "K_r" and in fifth line from bottom, change "interests" to "intersects".

p. 230: The fifth text line should read: "The static head ----- and C is:"

p. 231: Equation 11.4a. Change the "Q_g" in the middle term on the right side to "Q_s".

p. 242: Near bottom, change “H = 1197 ft” to “H = 197 ft”

p. 256: Table 12.2. Change caption to read “.hp / 100 ft…” rather than “.hp / 1000 ft…”

p. 292: Below Equation 13.4. In the definition of **T**, change “180° $ **T** # 360°” to “180° # **T** # 360°”.

p. 313: Table 14.1. Change the values in the “Irrigation interval” columns for both 5 and 6 days for Pasture - Peak from “0.02” to “1.02”.

p. 322: In first line, change “(0.3 in. / hr)” to “(0.4 in. / hr)”.

p. 324: Table 14.2. This table is out of date. The Irrigation Association’s Center Pivot manual has an updated equivalent.

p. 343: Sample Calculation 14.6. Change the subscript of *R’* from “R'_e” to “R'_n” in two places a) in the sixth line from top of page; and b) in the line above the last equation.

p. 361: Error in four places, should read “…for the 168-mm (6 5/8-in.)…” (i.e., change all occurrences of “6-in.” to “6 5/8-in.”).

p. 362: Error in eight places, change occurrence of “6-in.” to “6 5/8-in.”.

p. 363: Error in one place, change occurrence of “6-in.” to “6 5/8-in.”.

p. 363: Second equation should read:

$$(h_f - h_{fj})_6 = (h_f - h_{fj})_8 \; J_6 / J_8 = 1.16 \text{ x } 5.97/2.31 = 3.0 \text{ m}$$

In other words, change the “+” separating h_f and h_{fj} to a “-“

Note that in the equation for $(h_f)_{8,6}$ that one should also subtract out for the h_f due to the end gun; however, this turns out to be small (0.04 m).

p. 395: In the line below Equation (15.5), change “*LET*” to “LET” and “indix” “index”.

p. 407: In eleventh line from bottom, change “drop” to “crop”; and in sixth line from bottom, change “precipitaton” to “precipitation”.

p. 436: Fig. 17.5. In Part C, add value so ($P_w \cong 50\%$).

p. 443: Table 18.2. In first column change “Very find sand” to “Very fine sand”.

p. 456: Sample Calculation 19.1. In first equation "w" under CALCULATIONS: change the ".15" to "1.5", but the calculation is OK.

p. 457: Item 5 should read: "A 100-ml graduated cylinder;"

p. 462: Equation 19.3, 19.4, and 19.5. A "P_d" should be added to the denominator to allow for applications to sparse plantings. As is, it applies only to dense mature orchards. Otherwise, for sparse plantings, the canopy area should be used rather than S_p x S_r, but this could become confusing. Therefore, the denominators should read: "S_p x S_r x $P_d/100$".

p. 492: At the end of last bullet near top of page, change "(1 + ν)" to "(1 " ν)".

p. 496: Fig. 20.8. The three curves should be labeled from top to bottom: "LARGE ON-LINE"; STANDARD ON-LINE"; AND "SMALL ON-LINE".

p. 497: Errors in <u>two</u> places about mid-page, change "L / s" to "L / h".

p. 512: In fourth line from top, change "d^n" to "d_n".

p. 512 The *G* calculated using Equation 19.16a be changed from 9.3 to 93.3 gal/day.

p. 513: In line above Equation 20.13a', change "n_p" to "N_p".

p. 516: Near bottom of page in reference to "*Seasonal Irrigation Efficiency*" should refer to Table 19.4 (rather than 19.3).

p. 521: In the second equation on the page for using Equation 19.11, change "[0.1 + $(75)^{0.5}$]" to "[0.1 $(75)^{0.5}$]", but calculation is OK.

p. 521: In line above last equation, change "O^t" to "O_t".

p. 535: <u>Comment</u>: rearranged, Equation 22.11 becomes:

$$\Delta H_c = \frac{L\,(1-F)}{100}\, S^{a/b}\; J^{-1/b}$$

(see Equation 23.14b). This form is simpler and it might be more didactic to present both forms. (S is absolute slope).

p. 537: Fig. 22.5. Insert arrows to show where "Av. h_{fp}" refers to (left-hand side, middle of figure). "Av. h_{fp}" should refer to the vertical distance between the horizontal dashed line and the lower end of the h_{fp} "curve" (i.e., the vertical height of the hatched area that is beneath the dashed line).

p. 538: Equation 22.19, 22.20. Add "-x/L_p ΔE_p" to each equation so that they read as:

$$(H)_d = h_{fp}\left(\frac{x}{L_p}\right)^{2.75} + \Delta H_c + H_n^{'} - \frac{x}{L_p}\Delta E_p \qquad (22.19)$$

$$(H)_u = h_{fp}\left(1 - \frac{x}{L_p}\right)^{2.75} + \left(1 - \frac{x}{L_p}\right)\Delta E_p + H_n^{'} \qquad (22.20)$$

p. 549: Equation 22.17. Change "(y – 0.5)" to "(Y – 0.5)".

p. 566: For clarification the bottom paragraph should read:
"**Basic Equations.** The elevation *(relative to the datum at the minimum pressure head but at the end of the pipe)* of the hydraulic grade at any point x ….."

Furthermore, in the legend following Equation 23.14a, change explanation for H_x to: "H_x = hydraulic *head (minimum)* at point *x* along a pipe-friction curve that is tangent to the HGL, m (ft). (*this is contrasted against H_x on page 577, which is a friction loss)*"

p. 567: Equation 23.14b. Change the exponent on (x/L) to "a" instead of "1/a".

p. 570: The numerical values in the second equation should be: 640/177 = 3.6 (not 640/17).

p. 581: Change referenced publication in next to last line to read: "*Trickle Irrigation for Crop Production* ".

p. 598: Table 24.6. An "=" sign is missing in the equation in the middle of the table, it should read: "($H_m + H_{fe}$) = (50.2 + 7.4) =".

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester 2004 – Exam #1

Include units in all results. Indicate any assumptions that you might make.

Name__________________________________

1. Preliminary Design Calculations (25 pts)

   Almonds (deciduous trees, with no cover crop) have been planted in an 80-acre orchard and will be irrigated with a set sprinkler system. The climate can be characterized as "moderate." The topsoil is 2.7 ft of silty clay loam, and below that are 3.1 ft of clay loam soil. The soil intake rate is 0.5 inch/hr.

   Use an MAD value of 30%. EC_w is 0.47 dS/m and the estimated water application efficiency will be E_a = 85%. There is no effective rain during the peak-use period. For these preliminary calculations, and based on the spacing of the trees, use a sprinkler spacing of 40 ft x 50 ft.

   a) Obtain the values for average W_a, average Z, U_d, U, and EC_e from the tables in Chapter 3 of the textbook.
   b) Calculate the maximum net application depth per irrigation, d_x.
   c) Calculate the maximum irrigation interval, nominal irrigation interval (whole number of days), and the net application depth per irrigation, d_n.
   d) What is the gross depth to apply per irrigation?
   e) What is the irrigation set time, S_{to}, in hours?

2. Economic Pipe Size Selection (25 pts)

   Suppose you applied the economic pipe sizing method. What if you were using the Hazen-Williams equation and based all your calculations on a system capacity of 100 lps, but now you realize a calculation mistake was made, and the system capacity should really be 115 lps.

   A section flow rate, q, (threshold between two adjacent pipe sizes) was 50 lps, but now it needs to be adjusted for the new system capacity of 115 lps.

   What is the new section flow rate for this system capacity?

3. Set Sprinkler Lateral Design (25 pts)

A fixed sprinkler system with buried IPS-PVC (*thermoplastic pipe*) laterals is to be designed. The 304-m long laterals will be a -0.394% (downhill) slope. The nominal sprinkler flow rate is 10 lpm at a pressure of 280 kPa, and the sprinkler spacing on the laterals is 8.0 m. Riser height is 1.0 m. Let the allowable lateral Δh be equal to 20% of h_a.

a) How many sprinklers will operate on the lateral if the first sprinkler is spaced S_e from the lateral inlet?
b) For a dual pipe size lateral, what is the allowable friction loss gradient, J_a?
c) What two adjacent IPS-PVC thermoplastic pipe sizes would you recommend? Specify the nominal diameters in inches according to Table 8.3.
d) What are the respective lengths (x_1 and x_2) of the two pipe sizes? Round the lengths to a multiple of S_e for each size.
e) What is the required lateral inlet pressure head, h_l?
f) What is the pressure at the downstream end of the lateral during operation?
g) Is the mean velocity at the lateral inlet too high?

4. (25 pts) A portable aluminum sprinkler lateral has a nominal diameter of 4 inches and goes downhill at a uniform slope of -0.5%. The inlet flow rate is 260 gpm, the lateral length is 840 ft, and the sprinkler spacing along the lateral is 30 ft. The lateral inlet pressure head is h_l = 30.0 m.

a) What is the location of minimum pressure in the lateral pipe during operation?
b) What is the minimum pressure in the lateral pipe during operation?
c) Is the mean velocity at the lateral inlet too high?

Solutions:

1. Preliminary Design Calculations (25 pts)

   a) Obtain the values for average W_a, average Z, U_d, U, and EC_e from the tables in Chapter 3 of the textbook.

   - From Table 3.1, W_a range is the same for the topsoil & subsoil. Use W_a = ½(145 + 208) = 176.5 mm/m.
   - From Table 3.2, for almonds, Z = ½(0.6 + 1.2) = 0.9 m.
   - From Table 3.3, for deciduous orchard w/o cover crop in a "moderate" climate, U_d = 4.8 mm/day, and U = 533 mm/season.
   - From Table 3.5, EC_e = 2.0 dS/m for almonds.

   b) Calculate the maximum net application depth per irrigation, d_x.

$$d_x = MAD(W_a)(Z) = 0.30(176.5)(0.9) = 47.7 \text{ mm/irrig}$$

   c) Calculate the maximum irrigation interval, nominal irrigation interval (whole number of days), and the net application depth per irrigation, d_n.

$$f_x = \frac{d_x}{U_d} = \frac{47.7 \text{ mm/irrig}}{4.8 \text{ mm/day}} = 9.94 \text{ days/irrig}$$

Rounding down, f = 9 days. This is conservative because f_x is nearly 10 days. Then,

$$d_n = (9 \text{ days/irrig})(4.8 \text{ mm/day}) = 43.2 \text{ mm/irrig}$$

   d) What is the gross depth to apply per irrigation?

$$LR = \frac{EC_w}{5EC_e - EC_w} = \frac{0.47}{5(2.0)\text{-}0.47} = 0.049$$

LR < 0.1, so,

$$d = \frac{d_n}{E_a} = \frac{43.2 \text{ mm/irrig}}{0.85} = 50.8 \text{ mm/irrig}$$

   e) What is the irrigation set time, S_{to}, in hours?

The soil intake rate is given, at 0.5 inch/hr, which equals 12.7 mm/hr. Then, the minimum set time is:

$$\left(S_{to}\right)_{min} = \frac{50.8 \text{ mm/set}}{12.7 \text{ mm/hr}} = 4.0 \text{ hrs/set}$$

2. Economic Pipe Size Selection (25 pts)

Note the graph on page 67 of the lecture notes. The lines separating the adjacent pipe sizes do not change because, in this problem, none of the economic parameters have changed. The relationship between Q and q is fixed along the slope of 2:1 on the log-log plot, or 1.852:1 in our case (with Hazen-Williams, as specified).

We have:

$$Q_s = K\left(q^{-1.852}\right)$$

Then,

$$\frac{100 = K\left(50^{-1.852}\right)}{115 = K\left(q_{new}{}^{-1.852}\right)}$$

or,

$$q_{new} = 50\left(\frac{115}{100}\right)^{1/-1.852} = 46.6 \text{ lps}$$

3. Set Sprinkler Lateral Design (25 pts)

a) How many sprinklers will operate on the lateral if the first sprinkler is spaced S_e from the lateral inlet?

$$\frac{304 \text{ m/lat}}{8 \text{ m/sprink}} = 38 \text{ sprink/lat}$$

b) For a dual pipe size lateral, what is the allowable friction loss gradient, J_a?

Since there are more than 30 outlets along the lateral pipe, F = 0.36. The nominal sprinkler pressure head is 280 kPa/9.81 = 28.5 m. Then,

$$J_a = 100\left(\frac{0.20h_a - \Delta h_e}{FL}\right) = 100\left(\frac{0.20(28.5) - (304)(-0.00394)}{(0.36)(304)}\right) = 6.3 \text{ m/100m}$$

c) What two adjacent IPS-PVC thermoplastic pipe sizes would you recommend? Specify the nominal diameters in inches according to Table 8.3.

The lateral inflow rate is:

$$Q_\ell = (38\text{ sprink/lat})(10\text{ lpm/sprink}) = 380\text{ lpm/lat }(6.33\text{ lps/lat})$$

With J_a = 6.3 m/100 m, Table 8.3 gives the following two adjacent PVC pipe sizes: 2" and 2½." Note that the respective pipe IDs are 55.7 mm and 67.4 mm.

d) What are the respective lengths (x_1 and x_2) of the two pipe sizes? Round the lengths to a multiple of S_e for each size.

This problem requires a few iterations. Use C = 150 for PVC with Hazen-Williams. Use units of lps and mm. From the lecture notes, we have:

$$\alpha_1 = \frac{1.217(10)^{12}}{100(150)^{1.852}} = 1.135(10)^6$$

$$\alpha_2 = \left[\frac{(10/60)(304)}{8}\right]^{1.852}(67.4)^{-4.87}(0.36)(304) = 4.152(10)^{-6}$$

$$\alpha_3 = \left(67.4^{-4.87} - 55.7^{-4.87}\right)\left(\frac{10/60}{8}\right)^{1.852} = -1.464(10)^{-12}$$

Note that F_1 = 0.36 due to the 38 sprinklers over the length L. F_2 will depend on x_1. Make a table of values, arbitrarily choosing an initial x_1 value of 150 m, and searching for $f(x_1) = 0$:

x_1 (m)	N	F_2	$f(x_1)$
150	19	0.38	-1.10
100	26	0.37	0.19
125	22	0.37	-0.55
112	24	0.37	-0.19
105	25	0.37	0.03

Rounding, let x_1 = 104 m, whereby $x_2 = L - x_1$ = 200 m.

e) What is the required lateral inlet pressure head, h_l?

Calculate h_f using F, as described in the lecture notes:

$$Q_2 = \left(\frac{x_2}{S_e}\right)q_a = \left(\frac{200}{8}\right)(10/60) = 4.17\text{ lps}$$

$$J_1 = 1.217(10)^{12}\left(\frac{6.33}{150}\right)^{1.852}(67.4)^{-4.87} = 4.30 \text{ m/100m}$$

$$J_2 = 1.217(10)^{12}\left(\frac{4.17}{150}\right)^{1.852}(67.4)^{-4.87} = 1.99 \text{ m/100m}$$

$$J_3 = 1.217(10)^{12}\left(\frac{4.17}{150}\right)^{1.852}(55.7)^{-4.87} = 5.03 \text{ m/100m}$$

where $F_1 = 0.36$ and $F_2 = 0.37$. Then,

$$h_f = \frac{471 - 147 + 372}{100} = 7.0 \text{ m}$$

Finally,

$$h_l = h_a + \frac{5}{8}h_f + \frac{1}{2}\Delta h_e + h_r$$

$$h_l = 28.5 + \frac{5}{8}(7.0) + \frac{1}{2}(-1.2) + 1.0 \cong 33 \text{ m (320 kPa)}$$

f) What is the pressure at the downstream end of the lateral during operation?

$$\begin{aligned} h_{end} &= h_l - h_f - \Delta h_e \\ &= 33 - 7 + 1.2 \\ &\cong 27 \text{ m} \end{aligned}$$

or, $27 - h_r = 27 - 1 = 26$ m head at the last sprinkler.

g) Is the mean velocity at the lateral inlet too high?

$$V = \frac{Q}{A} = \frac{4(6.33 \text{ lps})}{\pi(0.0674 \text{ m})^2(1{,}000 \text{ l/m}^3)} = 1.77 \text{ m/s}$$

Then, $V < 2.0$ m/s, so it is probably all right.

4. (25 pts) Portable aluminum sprinkler lateral.

- From Table 8.1, 4-inch pipe has ID = 3.9" (99.1 mm).
- Q_l = 260 gpm, or 16.4 lps.
- S_e = (30)(0.3048) = 9.14 m.
- N = 840/30 = 28 sprinklers.
- q_a = 16.4/28 = 0.586 lps.
- For aluminum lateral pipe, C = 130.

a) What is the location of minimum pressure in the lateral pipe during operation?

From the lecture notes, page 74:

$$x = \left(\frac{9.14}{0.586}\right)\left[16.4 - 3(10)^{-7}\left(130(0.5)^{0.54}(99.1)^{2.63}\right)\right] = 181\,\mathrm{m}$$

which is x = 595 ft.

b) What is the minimum pressure in the lateral pipe during operation?

- L = 840 ft, or 256 m.
- F_1 = 0.37 for 28 outlets (Table 8.7).
- Outlets beyond x = 181 m: N_2 = (256 – 181)/9.14 ≈ 8.
- F_2 = 0.42 for 8 outlets (Table 8.7).

$$J_1 = 1.217(10)^{12}\left(\frac{16.4}{130}\right)^{1.852}(99.1)^{-4.87} = 5.00\text{ m/100 m}$$

$$J_2 = 1.217(10)^{12}\left(\frac{(8)(0.586)}{130}\right)^{1.852}(99.1)^{-4.87} = 0.49\text{ m/100 m}$$

The friction head loss from inlet to x = 595 ft (181 m) is:

$$\left(h_f\right)_{x=181m} = \frac{J_1F_1L}{100} - \frac{J_2F_2(L-x)}{100}$$

$$\left(h_f\right)_{x=181m} = \frac{(5.00)(0.37)(256)}{100} - \frac{(0.49)(0.42)(75)}{100} = 4.6\text{ m}$$

or, 15 ft of head loss. Note that h_l is given in m, while all other values are given in English units. Finally,

$$\begin{aligned} h_{x=181\,m} &= h_l - \left(h_f\right)_{x=181\,m} - \Delta h_e \\ &= 30.0 - 4.6 - 181(-0.005) \\ &\cong 26\ \text{m}\ \ (86\ \text{ft, or } 37\ \text{psi}) \end{aligned}$$

c) Is the mean velocity at the lateral inlet too high?

$$V = \frac{Q}{A} = \frac{4(16.4\ \text{lps})}{\pi(0.0991\,\text{m})^2(1{,}000\ \text{l/m}^3)} = 2.1\,\text{m/s}$$

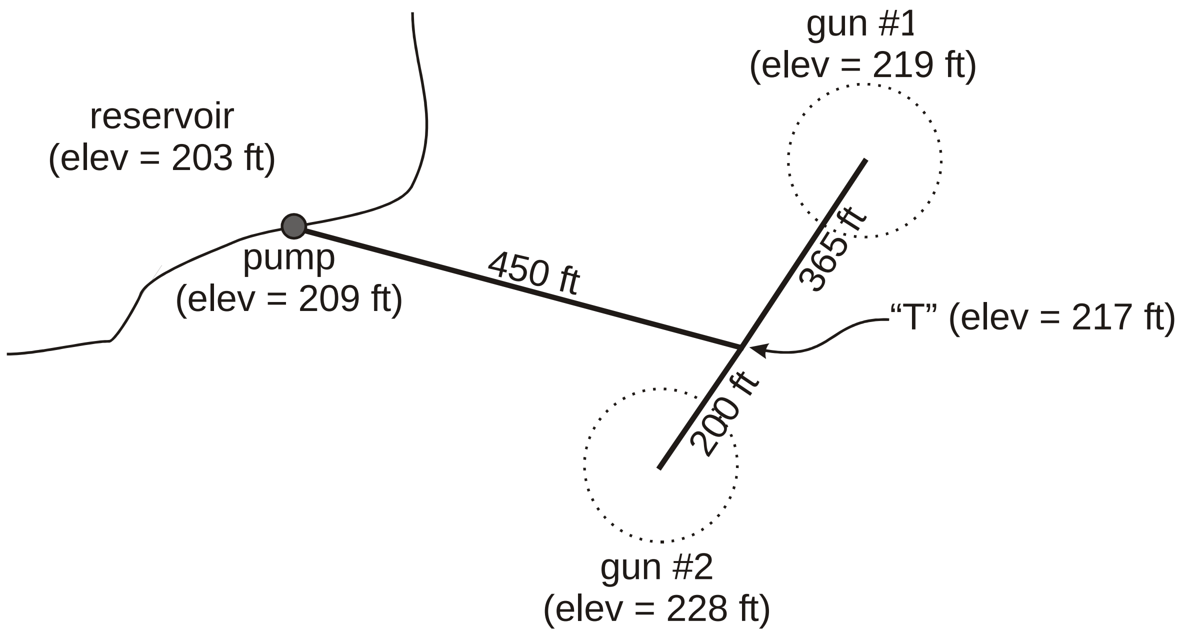
Whereby V > 2.0 m/s, so the entrance velocity in the lateral is higher than what we might nominally allow. This suggests consideration of a larger pipe size.

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester 2004 – Exam #2

Include units in all results. Indicate any assumptions that you might make.
Don't show more than three significant digits in the results.

Name___________________________________

1. (60 pts) Two gun sprinklers are supplied water from a pump at an open reservoir (water surface at 203 ft above msl), as shown in the figure below:



There is a common 4-inch (ID = 4.280") PVC supply line from the pump to a "T". The supply line is 450 ft long.

A 3-inch (ID = 3.284") PVC pipe goes 365 ft from the "T" to gun #1, and another 3-inch pipe goes 200 ft from the "T" to gun #2. The sprinkler height above the buried lateral pipe is h_r = 8.00 ft for both guns.

The flow rate vs. pressure data for the gun sprinkler give the following relationship:

$$q = 11.7P^{0.49}$$

for q in gpm; and P in psi.

The suction side of the pump has the same 4-inch PVC pipe as the supply line, 12 ft in length, with two 45-degree long-radius, flanged elbows, a basket strainer, and a foot valve.

a) Develop one point on the system curve using the Hazen-Williams equation (with C = 150) for friction losses. Use a flow rate of 80 gpm for gun sprinkler #1.
b) Calculate the flow rate for gun sprinkler #2.
c) Calculate the total system flow rate, Q_s.
d) Calculate TDH for this flow rate. Show your calculations for minor losses.

2. (20 pts) For the B2TPM Berkeley™ pump and 6-½" impeller, and the same system, suppose now that the desired operating point is for Q = 150 gpm.

   a) From the pump curves, determine $NPSH_r$.
   b) Calculate $NPSH_a$ (water temperature is 12°C).
   c) Determine whether the pump is expected to cavitate at the operating point.

3. (20 pts) For the same B2TPM Berkeley™ pump and 6-½" impeller, suppose that the desired operating point is 150 gpm at a TDH of 150 ft. If the nominal pump speed is 3,600 RPM, what is the required speed for the desired operating point?

4. (5 bonus pts) Which of the following are a function of a center pivot's radial speed? (check all that apply)

   ☐ wetted width, w
   ☐ net application depth, d_n
   ☐ average application rate, AR_{avg}
   ☐ maximum application rate, AR_x
   ☐ friction loss in the lateral pipe, h_f

Solutions:

1. (60 pts) Two gun sprinklers are supplied water from a pump at an open reservoir (water surface at 203 ft above msl), as shown in the figure below:

Move along the pipes from sprinkler #1 to the "T," then to sprinkler #2 to determine the flow rate there, then get the system flow rate ($Q_s = Q_1 + Q_2$), and finally move to the pump to determine P_{pump}.

<u>I. Flow rate at sprinkler #2:</u>

Pressure at gun sprinkler #1:

$$P_1 = \left(\frac{80}{11.7}\right)^{1/0.49} = 50.6 \text{ psi}$$

Pressure head at gun sprinkler #1:

$$h_1 = (50.6 \text{ psi})(2.31 \text{ ft/psi}) = 117 \text{ ft}$$

Pressure head at the "T":

$$h_T = h_1 + h_r + \Delta h_e + h_f$$
$$h_T = 117 + 8 + (219 - 217) + 10.5(365)\left(\frac{80}{150}\right)^{1.852}(3.284)^{-4.87}$$
$$h_T = 127 + 0.00109(80)^{1.852}$$
$$h_T = 131 \text{ ft}$$

Pressure head at gun sprinkler #2:

$$h_2 = h_T + \Delta h_e - h_f - h_r$$
$$h_2 = 131 + (217 - 228) - 10.5(200)\left(\frac{Q_2}{150}\right)^{1.852}(3.284)^{-4.87} - 8$$
$$h_2 = 131 - 11 - 0.000599\,Q_2^{1.852} - 8$$
$$h_2 = 112 - 0.000599\,Q_2^{1.852}$$

Flow rate at gun sprinkler #2:

$$Q_2 = 11.7\left(\frac{h_2}{2.31}\right)^{0.49} = 11.7\left(\frac{112 - 0.000599\,Q_2^{1.852}}{2.31}\right)^{0.49}$$

giving Q_2 = 77.7 gpm.

II. System flow rate:

$$Q_s = Q_1 + Q_2 = 80 + 77.7 \cong 158 \text{ gpm}$$

III. Pressure at pump outlet:

Pressure head at pump outlet:

$$h_{pump} = h_T + \Delta h_e + h_f$$
$$h_{pump} = 131 + (217 - 209) + 0.000371 Q_s^{1.852}$$
$$h_{pump} = 139 + 4.38 = 143 \text{ ft}$$

Pressure at pump outlet:

$$P_{pump} = \frac{h_{pump}}{2.31} = \frac{143 \text{ ft}}{2.31} = 61.9 \text{ psi}$$

IV. Suction side of the pump:

From Table 11.2, for a 4-inch pipe:

Item	Count	K_r	Total
Foot valve	1	0.80	0.80
Basket strainer	1	1.05	1.05
45-deg elbow	2	0.18	0.36
		Total:	**2.21**

Velocity head:

$$\frac{V^2}{2g} = \frac{8Q^2}{g\pi^2 D^4} = \frac{8\left(Q/448.86\right)^2}{32.2\pi^2(4.280/12)^4} = 7.72(10)^{-6}\,Q_s^2$$

Minor losses (see the above table):

$$\left(h_{f,minor}\right)_{suction} = 2.21\left(\frac{V^2}{2g}\right)$$

Pipe friction loss:

$$\left(h_f\right)_{suction} = 10.5(12)\left(\frac{Q_s}{150}\right)^{1.852}(4.280)^{-4.87} = 9.89(10)^{-6}Q_s^{1.852}$$

Static lift:

$$\left(h_{lift}\right)_{suction} = 209 - 203 = 6 \text{ ft}$$

V. Total dynamic head (TDH):

$$TDH = \frac{P_{pump}}{\gamma} + \left(h_f\right)_{suction} + \left(h_{f,minor}\right)_{suction} + \left(h_{lift}\right)_{suction} + \frac{V^2}{2g}$$

where P_{pump} is the pressure at the pump outlet. Simplifying,

$$TDH = h_{pump} + 9.89(10)^{-6}Q_s^{1.852} + 2.48(10)^{-5}Q_s^2 + 6$$

then,

$$TDH = 143 + 9.89(10)^{-6}(158)^{1.852} + 2.48(10)^{-5}(158)^2 + 6$$
$$TDH = 143 + 0.117 + 0.619 + 6$$
$$TDH = 150 \text{ ft}$$

VI. System curve point:

TDH = 150 ft at Q_s = 158 gpm

2. (20 pts) For the B2TPM Berkeley™ pump and 6-½” impeller, and the same system, suppose now that the desired operating point is for Q = 150 gpm.

   a) From the pump curves, determine $NPSH_r$.
   b) Calculate $NPSH_a$ (water temperature is 12°C).
   c) Determine whether the pump is expected to cavitate at the operating point.

From the pump curves, at 150 gpm, $NPSH_r \approx 9$ ft.

Mean atmospheric pressure head:

$$h_{atm} = \frac{10.3 - 0.00105(203 * 0.3048)}{0.3048} = 33.6 \text{ ft}$$

For water at 12°C,

$$h_{vapor} = 0.0623 \exp\left(\frac{17.27(12)}{12 + 237.3}\right) = 0.143 \text{ m } (0.469 \text{ ft})$$

Velocity head:

$$\frac{V^2}{2g} = 7.72(10)^{-6} Q_s^2 = 7.72(10)^{-6}(150)^2 = 0.174 \text{ ft}$$

Minor losses:

$$\left(h_{f,minor}\right)_{suction} = 2.21\left(\frac{V^2}{2g}\right) = 2.21(0.174) = 0.384 \text{ ft}$$

Pipe friction loss:

$$\left(h_f\right)_{suction} = 10.5(12)\left(\frac{150}{150}\right)^{1.852}(4.280)^{-4.87} = 0.106 \text{ ft}$$

Static lift:

$$\left(h_{lift}\right)_{suction} = 209 - 203 = 6 \text{ ft}$$

Available NPSH:

$$NPSH_a = h_{atm} - h_{vapor} - \left(h_f\right)_{total} - h_{lift} - \frac{V^2}{2g}$$

$$NPSH_a = 33.6 - 0.469 - 0.382 - 0.106 - 6 - 0.174$$

$$NPSH_a = 26.5 \text{ ft}$$

The pump is not expected to cavitate because $NPSH_a >> NPSH_r$.

3. (20 pts) For the same B2TPM Berkeley™ pump and 6-½" impeller, suppose that the desired operating point is 150 gpm at a TDH of 150 ft. If the nominal pump speed is 3,600 RPM, what is the required speed for the desired operating point?

Follow the steps in the lecture notes. Make a table for the equal efficiency curve, using Q_2 = 150 gpm, and H_2 = 150 ft:

Q_1 (gpm)	H_1 (ft)
100	66.67
120	96.00
140	130.67
160	170.67
180	216.00

Plot the equal efficiency curve and look for the intersection with the pump characteristic curve, defining point (Q_3,H_3). From the graph, $Q_3 \approx$ 158 gpm, and $H_3 \approx$ 167 ft. Then,

$$N_{new} = N_{old}\left(\frac{Q_2}{Q_3}\right) = 3{,}600\left(\frac{150}{158}\right) = 3{,}418 \text{ RPM}$$

4. (5 bonus pts) Which of the following are a function of a center pivot's radial speed? (check all that apply)

- ☐ wetted width, w
- ☑ net application depth, d_n
- ☐ average application rate, AR_{avg}
- ☐ maximum application rate, AR_x
- ☐ friction loss in the lateral pipe, h_f

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester 2004 – Final Exam

Include units in all results. Indicate any assumptions that you might make.
Don't show more than three significant digits in any of the results.

Name________________________________

1. (35 pts) A mature citrus orchard will be drip-irrigated (drip emitters) using a single lateral per row of trees in a 132-ha field area. Other information:

- Tree spacing is 6.0 x 6.0 m.
- Peak daily ET is U_d = 5.1 mm/day.
- Seasonal water requirement: U = 660 mm.
- Effective rain, peak-use period: 1.5 mm/day (average, w/ 90% probability)
- Residual soil water in the spring: assume zero.
- Soil water holding capacity is 175 mm/m (medium texture).
- Irrigation water quality: EC_w = 0.89 dS/m.
- Root zone depth is 1.5 m.
- Shaded area is 78%.
- Emitter equation:

$$q = 0.28P^{0.481}$$

for q in lph; and P in kPa.

- Nominal emitter flow rate: q_a = 3.85 lph.
- Manufacturer's emitter coefficient of variation: 0.0487.
- Average wetted width at 3.85 lph: w = 2.01 m.
- Outlets per emitter: one.
- Use an MAD of 20%.

What you need to do:

1. Select an appropriate emitter spacing, S_e.
2. Determine the number of emitters per tree, N_p.
3. Calculate percent wetted area, P_w.
4. Calculate maximum net depth to apply per irrigation, d_x.
5. Calculate the average peak daily "transpiration" rate, T_d.
6. Calculate the maximum irrigation interval, f_x. If $f_x \geq 1$ day, use f' = 1day.
7. Calculate the net depth per irrigation, d_n.
8. Select a reasonable target EU value.
9. Determine $(EC_e)_{max}$.
10. Determine the transmission ratio, T_r.

11. Calculate the leaching requirement, LR_t.
12. Calculate the gross depth to apply per irrigation, d.
13. Calculate the gross volume of water per tree per day, G.
14. Calculate h_a, corresponding to q_a = 3.85 lph, in m of water head.
15. Calculate the water application time, T_a.
16. Select the number of stations, N_s.
17. Determine the minimum number of emitters per tree, N_p'.
18. Calculate the system coefficient of variation, ν_s.
19. Calculate the minimum allowable emitter flow rate, q_n.
20. Calculate the allowable subunit pressure head variation, ΔH_s.
21. Calculate the system capacity, Q_s.
22. Calculate the total gross seasonal depth to apply, D_g.
23. Calculate the gross seasonal volume of irrigation water, V_s.
24. Calculate the required number of operating hours per season, O_t.

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2. (30 pts) A rectangular field of strawberries will be trickle irrigated. The laterals are 380-m long in the direction of the 17.8-mm inside diameter PE laterals. Nominal emitter flow rate is 2.75 lph at a pressure head of 11.5 m. The emitters are in-line, without any barbs, spaced at 0.4 m along the lateral hose, which lies along a uniform ground slope of 0.761%. The strawberries are spaced at 0.5 m in the field rows. The emitter exponent is x = 0.544, and the system flow rate is 8.05 lps.

What you need to do:

1. Determine the optimal manifold location.
2. Determine the location of the minimum downhill lateral pressure.
3. Calculate the required lateral inlet pressure head, H_l.
4. Calculate the minimum uphill lateral pressure head, $(H_n')_{uphill}$.
5. Do your work neatly for full credit on this problem.

---

3. (35 pts) A trickle irrigation system with a manifold inflow rate of 8.4 lps has an allowable subunit pressure head variation of ΔH_s = 4.72 m. The calculated pressure variation along the lateral pipes is ΔH_l = 2.44 m, and the total length of the manifold will be 290 m. There is a uniform ground slope of 1.73% in the manifold direction. The following PVC pipe sizes are available:

Size (inches)	**I.D. (inches)**
1.5	1.610
2.0	2.067
2.5	2.469
3.0	3.068
4.0	4.000
6.0	6.000

What you need to do:

1. Design the manifold, using up to four different pipe diameters.
2. Use the attached friction loss curves for the six available pipe sizes.
3. Determine appropriate manifold pipe sizes and lengths.
4. Do not allow the maximum velocity in each pipe size to exceed 2.0 m/s.
5. Do your work neatly for full credit on this problem.

Solutions:

1. Trickle Design Calculations

1. Select an appropriate emitter spacing, S_e.

$$S'_e = 0.8w = 0.8(2.01) = 1.61\,\text{m}$$

2. Determine the number of emitters per tree, N_p.

$$N_p = \frac{S_p}{S_e} = \frac{6.0}{1.61} = 3.73$$

3. Calculate percent wetted area, P_w.

$$P_w = 100\left(\frac{N_p S_e w}{S_p S_r P_d}\right) = 100\left(\frac{(3.73)(1.61)(2.01)}{(6.0)(6.0)(0.78)}\right) = 43.0\%$$

which is within acceptable limits.

4. Calculate maximum net depth to apply per irrigation, d_x.

Table 3.2: Z_{avg} (for citrus) = ½(0.9 + 1.5) = 1.2 m.

$$d_x = \frac{MAD}{100}\frac{P_w}{100}W_a Z = (0.2)(0.43)(175)(1.2) = 18.1\,\text{mm}$$

5. Calculate the average peak daily “transpiration” rate, T_d.

$$T_d = 0.1U_d\sqrt{P_d} = 0.1(5.1)\sqrt{78} = 4.50\text{ mm/day}$$

6. Calculate the maximum irrigation interval, f_x. If $f_x \geq 1$ day, use f’ = 1day.

$$f_x = \frac{d_x}{T_d} = \frac{18.1}{4.50} = 4.0 \text{ days}$$

Then, let f' = 1 day (for design).

7. Calculate the net depth per irrigation, d_n.

$$d_n = T_d f' = (4.5)(1) = 4.5 \text{ mm/day}$$

8. Select a reasonable target EU value.

   Table 20.3: "point-source" water applicators with $N_p > 3$ gives recommended EU range of 90 to 95%. Choose EU = 92%.

9. Determine $(EC_e)_{max}$.

   From Table 19.2, for a citrus (e.g. orange) crop, $(EC_e)_{max}$ = 8 dS/m.

10. Determine the transmission ratio, T_r.

    From Table 19.3, for a "deep-rooted" (Z > 1.5 m) crop and a "medium-textured" soil: T_r = 1.00.

11. Calculate the leaching requirement, LR_t.

$$LR_t = \frac{EC_w}{2\left(EC_e\right)_{max}} = \frac{0.89}{2(8)} = 0.056$$

12. Calculate the gross depth to apply per irrigation, d.

    For $LR_t < 0.1$, the following equation is applied:

$$d = 100\left(\frac{d_n T_r}{EU}\right) = 100\left(\frac{(4.50)(1.00)}{92\%}\right) = 4.89 \text{ mm/day}$$

13. Calculate the gross volume of water per tree per day, G.

$$G = \frac{d}{f'} S_p S_r = \frac{4.89}{1}(6.0)(6.0) = 176 \text{ liter/day/tree}$$

14. Calculate h_a, corresponding to q_a = 3.85 lph, in m of water head.

    Apply the given emitter equation, and use 9.81 kPa/m:

$$h_a = \left(\frac{1}{9.81}\right)\left(\frac{3.85}{0.28}\right)^{1/0.481} = 23.7 \text{ m}$$

15. Calculate the water application time, T_a.

$$T_a = \frac{G}{N_p q_a} = \frac{176}{(3.73)(3.85)} = 12.3 \text{ hrs/day}$$

16. Select the number of stations, N_s.

    Two stations would require 2(12.3) = 24.6 hrs/day. Thus, there can be only one station (N_s = 1) in this design.

17. Determine the minimum number of emitters per tree, N_p'.

    Here is a new equation for N_p':

$$N'_p = 2\left[\text{trunc}\left(1 + \frac{S_p + w - 2S_e}{2S_e}\right)\right] + 1$$

In this problem,

$$\begin{aligned} N'_p \quad &= 2\left[\text{trunc}\left(1 + \frac{6.0 + 2.01 - 2(1.61)}{2(1.61)}\right)\right] + 1 \\ &= 2\left[\text{trunc}(2.49)\right] + 1 \\ &= 5 \text{ emitters} \end{aligned}$$

Note that N_p' must be an integer value.

18. Calculate the system coefficient of variation, ν_s.

$$\nu_s = \frac{\nu}{\sqrt{N'_p}} = \frac{0.0487}{\sqrt{5}} = 0.022$$

19. Calculate the minimum allowable emitter flow rate, q_n.

$$q_n = \frac{q_a EU}{100(1 - 1.27\nu_s)} = \frac{(3.85)(92)}{100(1 - 1.27(0.022))} = 3.64 \text{ lph}$$

which corresponds to a head of:

$$h_n = \left(\frac{1}{9.81}\right)\left(\frac{3.64}{0.28}\right)^{1/0.481} = 21.1\,\text{m}$$

20. Calculate the allowable subunit pressure head variation, ΔH_S.

$$\Delta H_s = 2.5\left(h_a - h_n\right) = 2.5\left(23.7 - 21.1\right) = 6.50\ \text{m}$$

21. Calculate the system capacity, Q_S.

$$Q_s = 2.78\frac{A N_p q_a}{N_s S_p S_r} = 2.78\frac{(132)(3.73)(3.85)}{(1)(6.0)(6.0)} = 146\ \text{lps}$$

22. Calculate the total gross seasonal depth to apply, D_g.

Assume a T_R value of 1.00 (Table 19.4). Then, E_s = EU = 92%. Seasonal effective rain is assumed to be zero because the value is not given. Residual soil moisture is given to be zero. Thus,

$$D_n = U\left(0.1\sqrt{P_d}\right) = 660\left(0.1\sqrt{78}\right) = 583\ \text{mm}$$

Then, gross seasonal depth is:

$$D_g = \frac{100\,D_n}{E_s\left(1 - LR_t\right)} = \frac{100(583)}{92(1 - 0.056)} = 671\,\text{mm}$$

23. Calculate the gross seasonal volume of irrigation water, V_s.

$$V_s = \frac{D_g A}{1000} = \frac{(671)(132)}{1000} = 88.6\ \text{ha-m}$$

24. Calculate the required number of operating hours per season, O_t.

$$O_t = 2778\frac{V_s}{Q_s} = 2778\left(\frac{88.6}{146}\right) \approx 1{,}690\ \text{hrs/season}$$

2. Optimal Manifold Location

Number of emitters for the pair of laterals:

$$\frac{380 \text{ m}}{0.4 \text{ m/emitter}} = 950 \text{ emitters}$$

Total nominal discharge for the pair of laterals:

$$Q_{pair} = \frac{(950 \text{ emitters})(2.75 \text{ lph/emitter})}{3600 \text{ s/hr}} = 0.726 \text{ lps}$$

Friction loss gradient (Eq. 8.7a):

$$J = 7.83(10)^7 \frac{(0.726)^{1.75}}{(17.8)^{4.75}} = 51.4 \text{ m/100 m}$$

Note that this is a high value for J, and is beyond the values given in Table 8.2. The multiple-outlet factor, F, is 0.36 for 950 outlets. Take $f_e = 0$ since the emitters are in-line.

Friction loss for the pair of laterals:

$$\left(h_f\right)_{pair} = \frac{(51.4)(0.36)(380)}{100} = 70.3 \text{ m}$$

which is equal to about 100 psi.

Elevation change for the pair of laterals:

$$\left(\Delta h_e\right)_{pair} = (0.00761)(380) = 2.89 \text{ m}$$

Ratio of elevation change to friction loss for the pair of laterals:

$$\left(\frac{\Delta h_e}{h_f}\right)_{pair} = \frac{2.89}{70.3} = 0.041$$

Plot the above value (0.041) on the ordinate of the dimensionless friction loss graph (Fig. 8.2) at x/L = 1.0, then draw a straight line from the origin to this point. Slide the dimensionless curve for the downhill portion of the laterals until it is tangent to the line, then trace it. Slide the curves up further so that the uphill curve passes through the plotted point. Determine the intersection of the two curves:

$$x/L \approx 0.52$$

Then, the downhill portion of the pair of laterals will have a length of:

$$L_d = (380)(0.52) \approx 198 \text{ m}$$

The uphill portion of the pair of laterals will have a length of:

$$L_u = 380 - 198 = 182 \text{ m}$$

Graphically, the location of minimum pressure in the downhill lateral would be:

$$L_{min} = 0.1(380) = 38 \text{ m}$$

as measured from the end of the downhill lateral, or 198 – 38 = 160 m from the manifold location.

The required lateral inlet head is:

$$\alpha = \frac{3}{4}\left[(0.52)^{3.75} + (1 - 0.52)^{3.75}\right] = 0.112$$

and,

$$h_l = h_a + \alpha\left(h_f\right)_{pair} + \left(\frac{x}{L} - 0.5\right)\left(\Delta h_e\right)_{pair}$$

$$h_l = 11.5 + 0.112\left(70.3\right) + \left(0.52 - 0.5\right)\left(2.89\right) = 19.5 \text{ m}$$

which is 191 kPa, or 28 psi.

The minimum pressure head in the uphill lateral is:

Friction loss gradient (Eq. 8.7a):

$$J = 7.83(10)^7 \frac{\left(\frac{(182)(2.75)}{(0.4)(3600)}\right)^{1.75}}{(17.8)^{4.75}} = 14.2 \text{ m/100 m}$$

Friction loss:

$$\left(h_f\right)_{up} = \frac{(14.2)(0.36)(182)}{100} = 9.30 \text{ m}$$

Minimum pressure head:

$$\begin{aligned}\left(h_n'\right)_{up} &= h_a - \left(h_f\right)_{up} - \left(\Delta h_e\right)_{up} \\ &= 19.5 - 9.30 - (182)(0.00761) \\ &= 8.81\,\text{m}\end{aligned}$$

3. Manifold Design

The elevation change over the manifold length is:

$$\Delta E_m = (290)(0.0173) = 5.02 \text{ m}$$

The allowable head variation in the manifold is:

$$\left(\Delta h_m\right)_a = 4.72 - 2.44 = 2.28 \text{ m}$$

The sum of elevation change and allowable head variation is: 5.02 + 2.28 = 7.30 m.

Draw a line from the origin to (8.4, 5.02). Draw a parallel line from (0.0, 2.28) to (8.4, 7.30). These two lines define the band within which the friction loss curves should be contained.

Look at the six friction loss curves on the attached graph. The smallest four sizes are too small to fit within the band. The largest (6 inches) is too large because the friction loss curve goes outside the band. Start with the 4-inch pipe size, but not at the point (8.4, 7.30); start at (8.4, 5.02) such that the curve can fit within the band. Trace the curve for the 4-inch pipe on the graph.

Entrance velocity would be:

$$V_{4-\text{inch}} = \frac{4Q}{\pi D^2} = \frac{4(0.0084)}{\pi\left[\left(\frac{4}{12}\right)(0.3048)\right]^2} = 1.04 \text{ m/s}$$

which is within acceptable limits.

Note that the 4-inch pipe could be used for the whole length of the manifold and still accommodate the allowable pressure head variation. But part of the manifold should be 3-inch pipe, thereby reducing the head variation further, and also decreasing the pipe cost.

Trace the 3-inch friction loss curve on the graph, making it tangent to the lower line (ground slope line). This curve intersects the 4-inch curve at a flow rate of about 3.45 lps. The entrance velocity to the 3-inch pipe would be:

$$V_{3-\text{inch}} = \frac{4Q}{\pi D^2} = \frac{4(0.00345)}{\pi\left[\left(\frac{3.068}{12}\right)(0.3048)\right]^2} = 0.72 \text{ m/s}$$

which is also within acceptable limits.

Then, use two pipe diameters (4- and 3-inch) with the following lengths of pipe in the manifold:

$$L_{3-\text{inch}} = 290\left(\frac{3.45}{8.4}\right) = 119 \text{ m}$$

$$L_{4-\text{inch}} = 290 - 119 = 171 \text{ m}$$

where, of course, the upstream end has the 4-inch pipe. These lengths could be rounded as necessary to accommodate the lateral spacing.

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester, 2004

Assignment #1 (100 pts)
Due: 15 Sep 04

Given:

Data for a field in Cache Valley, Utah:

Crop=	Sweet corn
Topsoil=	Sandy loam
Topsoil depth (m)=	0.5
Subsoil=	Silt Loam
Subsoil depth (m)=	1.4
Field area (ha)=	25
MAD (%)=	35
Irrigation water salinity, EC_w (dS/m)=	1.02
Application efficiency=	88%
Soil intake rate (mm/hr)=	14
Time to change sets (hrs)=	0.5
Lateral length (m)=	180
Lateral spacing, S_l (m)=	12

Weather data:

1. Go to http://climate.usu.edu/
2. Click on "Utah Climate Center Data (Use Microsoft Explorer)"
3. Look at the instructions for selecting a region
4. Select a region which includes Utah
5. Click on the "Update Station List" button
6. At the upper left you see a list of stations
7. Click on the list and type "L"
8. Scroll down further to "logan usu exp stn"
9. Under "Element," select "Maximum Air Temperature"
10. Check the "Full period of record" box
11. Click on the "Export Data" button at the top
12. Click on the "Update Output" button at the lower left
13. Type "Ctrl-A" to select all, then "Ctrl-C" to copy
14. Paste it into Notepad or Word
15. Clean up the data and import to Excel
16. Go back and select “Minimum Air Temperature" and get that data
17. Go back and select “Total Precipitation" and get that data, too

Note that values with “-99999,M” are missing.

Required:

- Perform calculations to answer all of the questions as shown in the table format on the next page
- Show your steps in logical order, and write down your assumptions (if any) in determining the respective values
- Do you work neatly

Notes:

1. Many years of weather data for Logan, Utah (Experiment Station site), are given on the web site.
2. Determine the mean monthly values (Jan – Dec) of maximum daily air temperature for the entire period of record (about 34 years). To do this in Excel, you may want to use functions like COUNTIF and SUMIF.
3. Plot the mean monthly values of maximum air temperature and determine which month is the warmest; this month will be used below as the peak-use (peak ET) month.
4. Use the precipitation data to calculate the 75% rainfall probability value for the peak-use month to determine the net crop ET requirement during that month. This means you need to calculate the total rainfall (inches) for the peak-use month for the 34 years of record. You can consider that the 75% rainfall value is all "effective" rainfall for that particular month.
5. You may notice some problems with the data sets from the Utah Climate Center web site. Document these problems and describe how you have dealt with them.
6. Assume that there will be only six days of irrigation per week (one day off), even during the peak-use period.
7. Obtain needed soil, root depth, EC and ET information from tables in the text (Chapter 3), or from another source (if so, name that source). Use average values where max-min ranges are given in the tables.
8. Use Eq. 3.1 to calculate the maximum net application depth per irrigation.
9. Use Eq. 3.2 to calculate the maximum irrigation interval, then to calculate the net application depth.
10. Use Eq. 3.3 to calculate the leaching requirement.
11. Use Eqs. 5.3a and 5.3b to determine the gross application depth.
12. Use Eq. 5.4 to calculate system flow capacity.

Table Format for BIE 5110/6110 Assignment #1 (Fall 2004)

Given Values:

Crop: ....
Topsoil depth (m):..
Subsoil depth (m): ....
Location ....
Field area (ha): ....
MAD (%): ....
Irrigation water salinity, EC_w (dS/m):........................
Application efficiency (%): ....
Soil intake rate (mm/hr) ....
Time to change sets (hrs): ....
Lateral length (m) ....
Lateral spacing (m)..

Obtained from Tables and Weather Data:

Average W_a of topsoil (mm/m): ....
Average W_a of subsoil (mm/m): ....
Average root depth, Z (m): ....
Seasonal effective rainfall at 75% prob. (mm): ....
Peak ET (mm/day): ....
Seasonal ET (mm): ....
Salinity of soil extract, EC_e (dS/m): ....

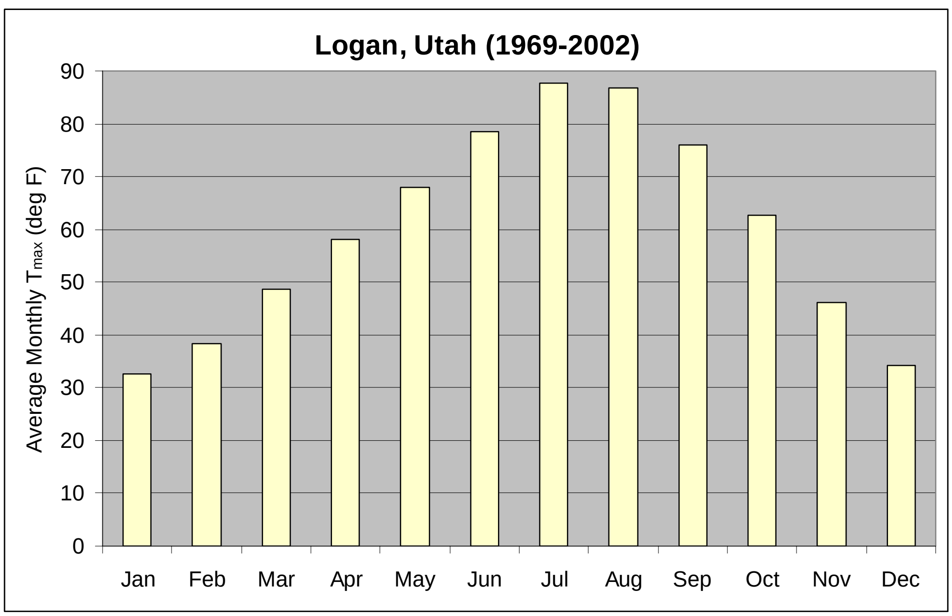
Calculated Values:

Average W_a of root zone (mm/m): ....
Maximum net depth per irrigation (mm): ....
Maximum irrigation interval (days):.........................
Nominal irrigation interval (days): ....
Net depth per irrigation (mm): ....
Days off in each irrigation:.......................................
Operating time per irrigation (days): ....
Leaching requirement: ....
Gross application depth per irrigation (mm): ....
Minimum set operating time (hrs) ....
Nominal set operating time (hrs).............................
Number of sets per day:..
Area per 200-m lateral per irrigation (ha): ....
Number of 200-m laterals required:.........................
Approximate number of irrigations per year: ....
System flow capacity (lps):.......................................

Solution:

To determine the mean monthly values of maximum daily air temperature, the SUMIF function was used in Excel to key on the column with the month names for the 34 years of record. The COUNTIF function was used in the same way, for each month, to determine the number of records in each month. Finally, for each month, the sum of temperature values was divided by the corresponding record count to arrive at an average monthly value:

Month	Count	Sum	Avg Temp
Jan	32	1,041	32.5
Feb	32	1,228	38.4
Mar	33	1,605	48.6
Apr	33	1,914	58.0
May	32	2,173	67.9
Jun	33	2,588	78.4
Jul	33	2,892	87.6
Aug	34	2,954	86.9
Sep	31	2,358	76.1
Oct	32	2,006	62.7
Nov	32	1,480	46.2
Dec	32	1,096	34.3



It is seen that the month of July has the highest maximum monthly temperature for the 34 years of record, and is closely followed by August. This will be the month for which peak ET will occur in Logan, Utah.

To determine the rainfall probability for the month of July, the total daily rainfall values for that month were summed up for each year of record. Following are the tabulated results:

Year	July Rain (inch)	Year	July Rain (inch)
1969	0.34	1986	1.79
1970	0.67	1987	1.65
1971	0.08	1988	0.00
1972	0.21	1989	0.11
1973	0.98	1990	0.31
1974	0.11	1991	0.16
1975	1.46	1992	0.99
1976	0.83	1993	3.21
1977	0.87	1994	0.02
1978	0.11	1995	0.72
1979	0.58	1996	1.44
1980	1.18	1997	2.30
1981	0.49	1998	0.12
1982	1.40	1999	0.57
1983	1.52	2000	0.07
1984	1.59	2001	*missing*
1985	1.50	2002	0.38

Rain (inch)		Frequency	Relative Frequency	Up	Down
From	To				
0.00	0.25	10	0.303	1.000	0.303
0.26	0.50	4	0.121	0.697	0.424
0.51	0.75	4	0.121	0.576	0.545
0.76	1.00	4	0.121	0.455	0.667
1.01	1.25	1	0.030	0.333	0.697
1.26	1.50	4	0.121	0.303	0.818
1.51	1.75	3	0.091	0.182	0.909
1.76	2.00	1	0.030	0.091	0.939
2.01	2.25	0	0.000	0.061	0.939
2.26	2.50	1	0.030	0.061	0.970
2.51	2.75	0	0.000	0.030	0.970
2.76	3.00	0	0.000	0.030	0.970
3.01	3.25	1	0.030	0.030	1.000
	Totals:	**33**	**1.000**		

As seen in the table above, based on the 34 years of record, there is a 69.7% probability that the total July rainfall in Logan, Utah, will be 0.26 inches or more (column "Up"). There are really not enough data points to determine the 75% level of confidence, so use 0.26 inches of rain in July (the peak ET month) with an approximately 70% level of exceedance. This is equivalent to 6.6 mm.

Given Values:

Crop: .... Sweet corn
Topsoil depth (m):... 0.5 m
Subsoil depth (m): .... 1.4 m
Location .... Logan, Utah
Field area (ha): .... 25 ha
MAD (%): 35%
Irrigation water salinity, EC_w (dS/m):........................ 1.02 dS/m
Application efficiency (%): 88%
Soil intake rate (mm/hr)... 14 mm/hr
Time to change sets (hrs): .... 0.5 hrs
Lateral length (m) .... 180 m
Lateral spacing (m).. 12 m

Obtained from Tables and Weather Data:

Average W_a of topsoil (mm/m):................................ 125 mm/m (Table 3.1, average for sandy loam)
Average W_a of subsoil (mm/m): .... 167 mm/m (Table 3.1, average for silt loam)
Average root depth, Z (m):....................................... 0.5 m (Table 3.2, average for sweet corn)
Seasonal effective rainfall at 70% prob. (mm): .... 6.6 mm (for month of July only, not season)
Peak ET (mm/day): .... 6.4 mm/day (Table 3.3, corn in “moderate” climate)
Seasonal ET (mm): .... 559 mm (Table 3.3, corn in “moderate” climate)
Salinity of soil extract, EC_e (dS/m):.......................... 2.5 dS/m (Table 3.5, sweet corn)

Calculated Values:

Note: most of the following is specifically for the peak-use period and does not dictate what the irrigation scheduling might be throughout the growing season. These are calculations leading to system design.

Average W_a of root zone (mm/m):

The topsoil depth is given as 0.5 m, and we have 0.5 m for the average effective root depth of sweet corn, so the subsoil W_a is not considered herein.

Use W_a = 125 mm/m

Maximum net depth per irrigation (mm):

$$d_x = \frac{MAD}{100} W_a Z = \left(\frac{35}{100}\right)(125)(0.5) = 21.9 \text{ mm}$$

Maximum irrigation interval (days):

For July, we have determined that there is a 70% probability of a monthly total of 0.26 inches (6.6 mm), or more, of rain. This comes to an average of 6.6/31 = 0.21 mm/day, which is very little rain. This fact, together with the realization that the rain might not fall during the peak-use period, may lead us to conclude the safer choice is to assume <u>zero effective rainfall</u> during the peak-use period.

$$f_x = \frac{d_x}{U_d} = \frac{21.9 \text{ mm}}{6.4 \text{ mm/day}} = 3.42 \text{ days}$$

Nominal irrigation interval (days):

$$f' = \text{trunc}\left(f_x\right) = 3 \text{ days}$$

Net depth per irrigation (mm):

$$d_n = f'U_d = \left(3 \text{ days}\right)\left(6.4 \text{ mm/day}\right) = 19.2 \text{ mm}$$

Days off & operating time per irrigation:

The specification in this case is for one day off per week, but with f' = 3 days, we can assume that the one day off will not fall within the three-day interval during the peak-use period, which might involve only two or three irrigations. Thus, let f = f' = 3 days.

Leaching requirement:

$$LR = \frac{EC_w}{5EC_e - EC_w} = \frac{1.02}{5\left(2.5\right) - 1.02} = 0.075$$

LR < 0.1; therefore, use Eq. 5.3a...

Gross application depth per irrigation (mm):

$$d = \frac{d_n}{\left(E_a / 100\right)} = \frac{19.2}{0.88} = 21.8 \text{ mm}$$

Minimum set operating time (hrs):

With 21.8 mm to apply and a soil intake rate of 14 mm/hr, this gives 1.56 hrs <u>minimum set time</u> (so as not to exceed the soil intake rate).

Nominal set operating time (hrs)

Make the nominal set time equal to <u>2.0 hours</u> for convenience. With 0.5 hrs to move each set, there are a total of <u>2.5 hrs/set</u>.

Number of sets per day:

With 24 hrs per day, there can be 24/2.5 = 9.6 sets/day. Round this down to a whole number: <u>8 sets per day</u>, giving a total daily operations time of 8(2.5) = 20 hrs.

Area per 200-m lateral per irrigation (ha):

(3 days/irrigation)(8 sets/day) = <u>24 sets/irrigation</u>

Lateral spacing on mainline is S_l = 12 m. Lateral length is 180 m. The area per lateral is:

(12 m/set)(24 sets)(180 m/lateral) = <u>5.18 ha/lateral</u>

Number of 180-m laterals required:

$$\frac{25\text{ ha}}{5.18\text{ ha/lateral}} = 4.83\text{ laterals}$$

Round up to a whole number to obtain <u>5 laterals</u> for the system. Alternatively, round up to 6 laterals if laterals will operate on both sides of the mainline, thereby balancing the number of laterals on each side.

Approximate number of irrigations per year:

Assuming zero effective rainfall during the growing season:

$$\frac{U - P_e}{d_n} = \frac{559\text{ mm - 0 mm}}{19.2\text{ mm/irrig}} = 29.1\text{ irrigations}$$

We could do a seasonal analysis of the probability of rainfall exceedance, but we can already surmise that most of the rain will fall during times other than the peak-use period. So, to be conservative in our design, we assume no seasonal effective rainfall. On the other hand, it is very unlikely that a farmer in Logan, Utah would irrigate a corn field 29 times in a season, which could indicate a nonzero contribution from rain and or the possibility that the effective root depth for sweet corn is somewhat more than 0.5 m. Also, there may be a significant residual water content (from snowmelt and rain) in the soil at the beginning of the growing season.

System flow capacity (lps):

At 8 sets/day and 2.0 hours set time, there are 16 hours of system operation per day:

$$Q_s = 2.78\frac{Ad}{fT} = 2.78\frac{(25\text{ ha})(21.8\text{ mm})}{(3\text{ days})(16\text{ hrs/day})} = 31.6\text{ lps}$$

This gives a capacity of 31.6/25 = 1.26 lps per ha, which is a reasonable value.

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester, 2004

Assignment #2 (100 pts)
Due: 29 Sep 04

Given:

A solution for the economic selection of pipe sizes is needed. The solution is to be based on the following data:

- interest rate, i = 2.4%
- inflation rate, e = 0.1%
- useful system life, n = 22 years
- area irrigated, A = 58 ha
- gross annual depth, d = 890 mm/year
- maximum system capacity, Q = 210 lps
- motor efficiency, E_m = 94%
- pump efficiency, E_p = 88%
- avg cost of electricity for pumping = 0.06104 $/kWh
- purchase cost of PVC pipe:

Size (inches)	O.D. (inches)	I.D. (inches)	Wall (inch)	Price ($/20 ft)
6	6.625	6.031	0.297	35.10
8	8.625	7.943	0.341	53.22
10	10.750	9.976	0.387	75.81
12	12.750	11.890	0.430	98.08
14	14.000	13.072	0.464	116.95
16	16.000	14.940	0.530	154.33
18	18.000	16.809	0.595	198.39

- In the above table: “O.D.” is outside diameter; “I.D.” is inside diameter; and “Wall” is the pipe wall thickness

Required:

- Determine the cutoff flow rate values for each adjacent pair of pipe sizes (you are not required to graph the pipe selection chart)
- The solution is to include the PVC pipe with nominal sizes 6, 8, 10, 12, 14, 16, and 18 inches, as shown above.
- You can use either the Hazen-Williams or Darcy-Weisbach equations.
- For PVC pipe, use a Hazen-Williams “roughness” factor of C = 150.
- Or, for Darcy-Weisbach, use the Blasius equation for smooth pipe.
- Assume that MAC is negligible.

Solution:

BIE 5110/6110
Economic Pipe Selection Method
Assignment #2, Fall 2004

Given data:

useful life:	22	years	inflation:	0.001	per year	motor efficiency:	0.94	
interest rate:	0.024	per year	area:	58	ha	pump efficiency:	0.88	
			depth:	0.89	m/year	electricity:	0.06104	$/kWh
			capacity:	210	lps	Hazen-Williams:	150	C factor

Pipe purchase prices:

Size (inches)	O.D. (inches)	I.D. (inches)	Wall (inch)	Price ($/20 ft)
6	6.625	6.031	0.297	35.10
8	8.625	7.943	0.341	53.22
10	10.750	9.976	0.387	75.81
12	12.750	11.890	0.430	98.08
14	14.000	13.072	0.464	116.95
16	16.000	14.940	0.530	154.33
18	18.000	16.809	0.595	198.39

Capital Recovery Factor: CRF= 0.0590

Uniform annual pipe cost:

Size	UAC	
(inches)	($/20 ft/yr)	($/100 ft/yr)
6	2.07	10.36
8	3.14	15.71
10	4.48	22.38
12	5.79	28.95
14	6.90	34.52
16	9.11	45.56
18	11.71	58.56

Operating hours per year:

Ot= 683 hrs/year

Present annual energy cost:

E= 50.38 $/kW/yr *(MAC said to be negligible)*

Equivalent annual energy cost:

EAE= 1.0096
E'= 50.87 $/kW/yr

Difference in WHP between adjacent pipe sizes:

Sizes (inches)	delta WHP kW/100 ft
6 and 8	0.10515
8 and 10	0.13109
10 and 12	0.12923
12 and 14	0.10950
14 and 16	0.21691
16 and 18	0.25567

Difference in J between adjacent pipe sizes:

Sizes (inches)	delta J m/100 m
6 and 8	0.16756
8 and 10	0.20889
10 and 12	0.20593
12 and 14	0.17449
14 and 16	0.34566
16 and 18	0.40743

(note: m/100 m is equivalent to ft/100 ft)
(however, must use 102 for lps or 3,960 for gpm)

Threshold flow rate between adjacent pipe sizes:

Sizes	Section flow rate	
(inches)	lps	gpm
6 and 8	11.2	177.4
8 and 10	27.4	434.3
10 and 12	53.8	852.8
12 and 14	99.1	1,570.0
14 and 16	160.0	2,535.6
16 and 18	260.9	4,134.8

(note: Hazen-Williams equation)

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester, 2004

Assignment #3 (100 pts)
Set Sprinkler Lateral Design

Given:

- Sprinkler spacing, S_e, of 12 m
- Lateral length of 252 m
- Riser height of 1.30 m
- Lateral will run downhill along a ground slope of 0.38%
- Nominal sprinkler discharge of 22 lpm at 2.08 atm pressure
- Nominal aluminum pipes sizes of 2, 3, 4 and 5 inches are available
- Assume a constant value of q_a for each sprinkler along the lateral
- See Table 8.1 in the textbook (Keller & Bliesner) for pipe inside diameters

Required:

- Calculate the required aluminum lateral pipe size
- Round up to the nearest available pipe size
- Calculate the required lateral inlet pressure
- Calculate the location of minimum pressure in the pipe
- Calculate the pressure in the lateral pipe at the downstream end
- Calculate the percent pressure variation in the lateral pipe (%)

Solution:

If Hazen-Williams equation is used, the C value will be 130.
The number of sprinklers on the lateral is:

$$N_n = \frac{L}{S_e} = \frac{252 \text{ m}}{12 \text{ m}} = 21$$

There will be 21 sprinklers per lateral. The multiple-outlet friction loss factor for 21 sprinklers is:

$$F = 0.351 + \frac{1}{2(21)}\left[1 + \frac{4}{13(21)}\right] = 0.38$$

The lateral inflow rate is:

$$Q_l = N_n q_a = \frac{(21)(22 \text{ lpm})}{60 \text{ s/min}} = 7.70 \text{ lps}$$

The elevation change along the length of the lateral is:

$$\Delta h_e = \left(\frac{-0.38}{100}\right)(252 \text{ m}) = -0.958 \text{ m}$$

The nominal sprinkler pressure is ha = 2.08 atm, or,

$$h_a = (2.08 \text{ atm})(10.34 \text{ m/atm}) = 21.5 \text{ m}$$

The allowable friction loss gradient is:

$$\begin{aligned} J_a &= 100\left(\frac{0.20h_a - \Delta h_e}{FL}\right) \\ &= 100\left(\frac{0.2(21.5 \text{ m}) - (-0.958 \text{ m})}{(0.38)(252 \text{ m})}\right) \\ &= 5.49 \text{ m/100m} \end{aligned}$$

The minimum pipe ID is:

$$D = \left[\frac{16.42(10)^6}{5.49} \left(\frac{7.70 \text{ lps}}{130} \right)^{1.852} \right]^{0.205} = 7.27 \text{ cm}$$

or 7.27 cm/2.54 = 2.86 inches.

From Table 8.1 (page 140) in the textbook, the nominal size of the aluminum lateral pipe is the outside diameter, and the wall thickness is 0.05 inches. The 2" size has ID = 1.9", the 3" size has ID = 2.9", and so forth. From our calculated minimum diameter, we must **round up to the 3" nominal size**. Then, the inside diameter is (2.9 inch)(2.54) = 7.37 cm.

The actual friction loss along the lateral will be:

$$J = 16.42(10)^6 \left(\frac{7.70 \text{ lps}}{130} \right)^{1.852} (7.37 \text{ cm})^{-4.87} = 5.22 \text{ m}/100 \text{m}$$

$$h_f = \frac{JFL}{100} = \frac{(5.22)(0.38)(252 \text{ m})}{100} = 5.00 \text{ m}$$

The required lateral inlet pressure is:

$$\begin{aligned} h_l &= h_a + 0.75 h_f + 0.5 \Delta h_e + h_r \\ &= 21.5 \text{ m} + 0.75(5.00 \text{ m}) + 0.5(-0.958 \text{ m}) + 1.3 \text{ m} \\ &= 26.1 \text{ m } \left(2.52 \text{ atm} \right) \end{aligned}$$

The minimum pressure in the lateral pipe is located at a distance x from the inlet (q_a is 22 lpm/60 = 0.367 lps; D is 73.7 mm):

$$\begin{aligned} x &= \frac{S_e}{q_a} \left[Q_l - 3(10)^{-7} \left(C(-S)^{0.54} D^{2.63} \right) \right] \\ &= \frac{12}{0.367} \left[7.70 - 3(10)^{-7} \left(130(0.38)^{0.54} (73.7)^{2.63} \right) \right] \\ &= 190 \text{ m} \end{aligned}$$

according to the equations in the lecture notes on pages 64 and 65. This "x" value (distance) is confirmed (approximately) by assuming an inlet pressure head of 26.1 m, a C value of 130, and so forth, calculating the head loss segment-by-segment in a spreadsheet:

Sprinkler Position	Distance (m)	Q (lps)	hf (m)	head (m)
0	0	7.70	0.0000	26.10
1	12	7.33	0.6262	25.52
2	24	6.97	0.5721	24.99
3	36	6.60	0.5203	24.52
4	48	6.23	0.4707	24.09
5	60	5.87	0.4234	23.72
6	72	5.50	0.3785	23.38
7	84	5.13	0.3358	23.09
8	96	4.77	0.2955	22.84
9	108	4.40	0.2576	22.63
10	120	4.03	0.2221	22.45
11	132	3.67	0.1891	22.31
12	144	3.30	0.1585	22.20
13	156	2.93	0.1304	22.11
14	168	2.57	0.1048	22.05
15	180	2.20	0.0819	22.02
16	192	1.83	0.0615	22.00
17	204	1.47	0.0439	22.00
18	216	1.10	0.0290	22.02
19	228	0.73	0.0170	22.05
20	240	0.37	0.0080	22.09
21	252	0.00	0.0022	22.13

whereby the minimum head is found somewhere between 192 and 204 m from the lateral inlet. The difference in the calculations are due to the F-factor for multiple outlets. It is also seen that the maximum pressure head is at the lateral inlet (h_{max} = 26.1 m), which might have been expected because the ground slope is very small.

Thus, the minimum pressure is at a distance of approximately 190 m from the lateral inlet, which is nearest sprinkler number 190/12 ≈ 16.

The pressure in the pipe at the downstream end will be:

$$\begin{aligned} h_{end} &= h_l - h_f - (-\Delta h_e) \\ &= 26.1 - 5.0 + 0.958 = 22.1\,\text{m} \end{aligned}$$

The head loss due to friction from sprinkler 16 to the end of the lateral is estimated as (from Table 8.7, F = 0.46 for 21 – 16 = 5 outlets):

$$J_{x-end} = 16.42(10)^6 \left(\frac{(0.367)(5)\text{ lps}}{130} \right)^{1.852} (7.37\text{ cm})^{-4.87} = 0.366\text{ m/100m}$$

where the pipe length is 5(12 m) = 60 m,

$$\left(h_f\right)_{x-end} = \frac{JFL}{100} = \frac{(0.366)(0.46)(60\text{ m})}{100} = 0.101\text{ m}$$

The friction loss from the inlet to distance x = 190 m is, then, 5.00 – 0.101 m = 4.90 m. The elevation change from the inlet to 190 m is (-0.0038)(12)(16) = -0.730 m. Finally, the minimum pressure head in the lateral pipe is:

$$h_{min} = 26.1 - 4.90 - (-0.730) = 21.9\text{ m}$$

The pressure (or head) variation is:

$$\Delta P = \frac{h_{max} - h_{min}}{h_a} = \frac{26.1 - 21.9}{21.5} = 0.20$$

which is 20%. This just meets our design criterion.

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester, 2004

Assignment #4 (100 pts)
Mainline Design
Due: 13 Oct 04

Given:

- A large rectangular field, 1,200 m long and 1,000 m wide
- Periodic-move sprinkler laterals with buried mainline pipe
- Four sprinkler laterals operate, each covering ¼ of the field area
- Two laterals are on one side of the mainline, and two on the other side
- All four laterals move in the same direction when changing sets
- The mainline will run down the middle of the field, 1,200 m long
- The mainline will run uphill at a uniform slope of 0.387%
- System capacity is Q_s = 135 lps
- PVC pipe sizes given in Table 8.5 are the available sizes
- Pressure available at the upstream end of the mainline is 389 kPa
- Required lateral inlet pressure is 275 kPa
- Laterals connect to mainline through hydrant valves
- Hydrant hydraulic loss to from mainline to lateral inlet is 25 kPa

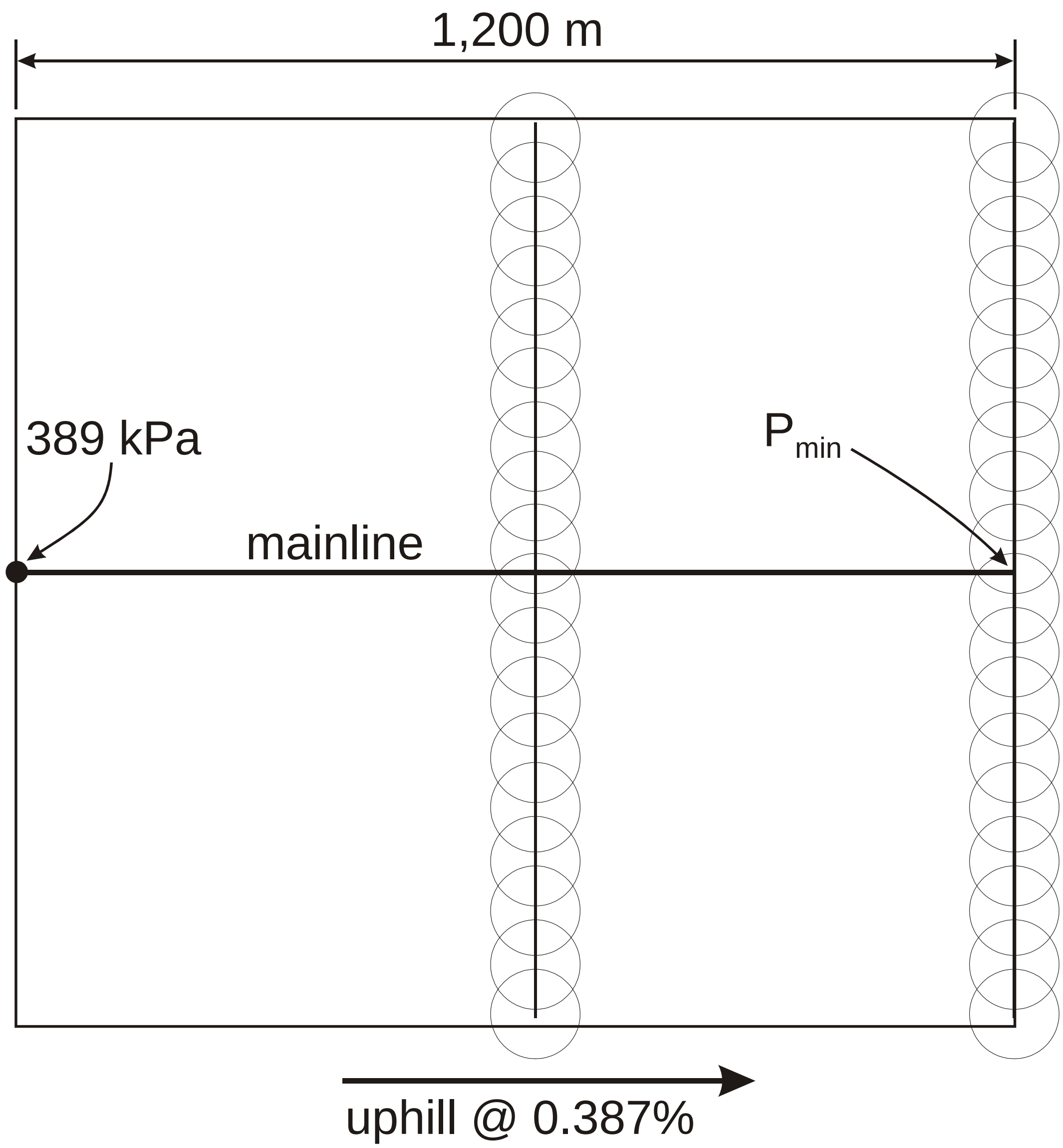
Required:

- Design the mainline to consume all of the available head
- Use the basic procedures from Lecture 9 and Chapter 10
- Consider the critical lateral positions
- Determine mainline pipe diameters and respective lengths of each size
- Check operational velocity limits in the mainline pipe
- Do all calculations in metric units (lengths in m, flow in lps, and diameter in mm)
- Do your work neatly and logically – make it understandable to another engineer
- Include brief comments, as necessary, about design details and decisions

Solution:

(1) Extreme Lateral Position

- Recognize that the extreme lateral position is when two laterals are at the mid-point of the mainline, and the other two at the uphill end of the mainline
- This is because the mainline runs uphill, so pressure must decrease monotonically from upstream to downstream along the mainline pipe
- Also, the critical point is the uphill end of the mainline, because that must be the location of minimum pressure in the mainline
- Thus, for the extreme lateral position, the two laterals at the mid-point will have more than enough pressure if the last two laterals have just the required pressure
- Mainline design, then, should focus on providing P_{min} = 275 + 25 kPa in the pipe
- These facts should be obvious



(2) Allowable Loss due to Friction

- The allowable friction loss is the available pressure at the mainline inlet, minus the elevation change, hydrant loss, and required lateral inlet pressure
- As in the example mainline design in the lecture notes, do not consider minor losses due to flow past closed hydrants along the mainline
- And, as in the example problems in the lecture notes, it may help to look at this problem using a schematic diagram
- First, convert pressures to heads and determine the elevation change along the length of the mainline:

Mainline inlet head: 389 kPa/9.81 = 39.65 m

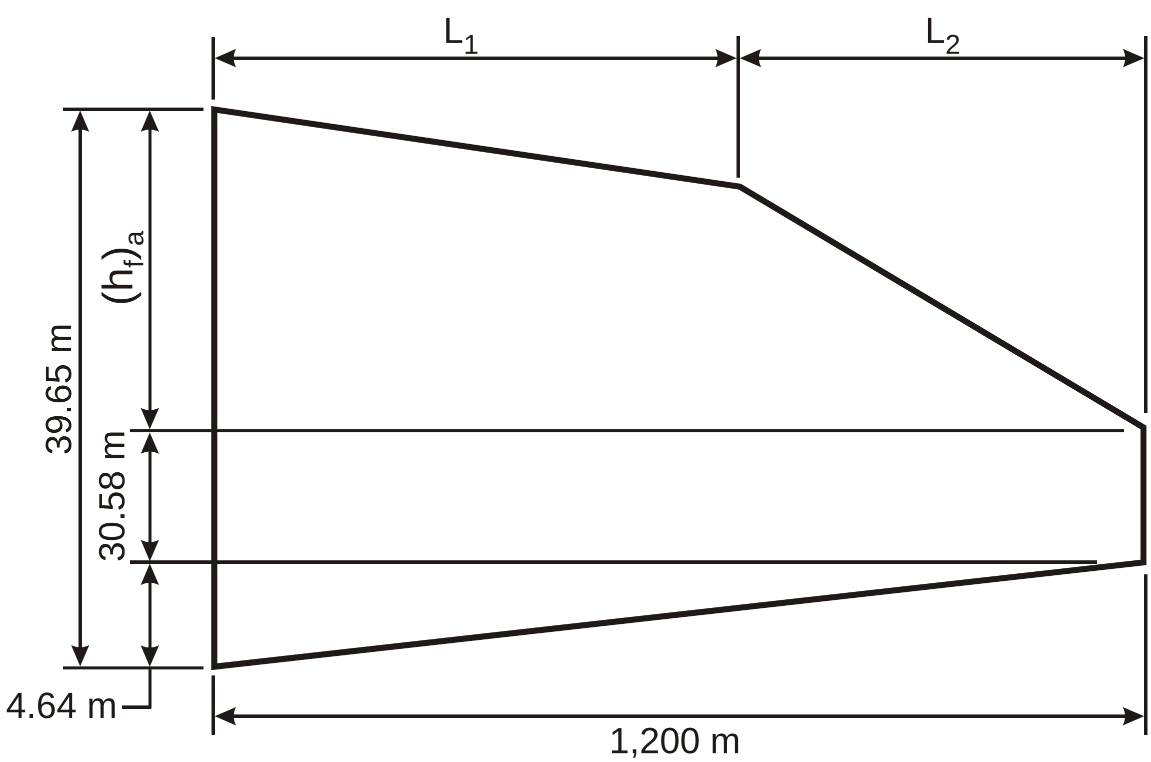
Elevation change:

$$\Delta h_e = 0.00387(1{,}200) = 4.64 \text{ m}$$

Lateral inlet pressure plus hydrant loss:

$$h_l + h_{hydrant} = \frac{275 + 25}{9.81} = 30.58 \text{ m}$$

- Let L_1 be the length of pipe diameter D_1, and L_2 the length for diameter D_2



- Allowable loss due to friction along the entire length of the mainline pipe:

$$(h_f)_a = 39.65 - 30.58 - 4.64 = 4.43 \text{ m}$$

(3) Selection of Mainline Pipe Diameters

- Calculate the required mainline pipe inside diameter assuming only one pipe size
- Recognize that at the extreme lateral position, the full system flow rate goes from the beginning of the mainline to the mid-point, where only half the system flow rate continues to the end of the mainline
- Make the allowable friction loss equal to the actual friction loss

$$(h_f)_a = \frac{J_1(L/2)}{100} + \frac{J_2(L/2)}{100}$$

where L = 1,200 m; and $(h_f)_a$ = 4.43 m

$$4.43 = 6\left[\frac{1.217(10)^{12} D^{-4.87}}{C^{1.852}}\right]\left[Q_s^{1.852} + \left(\frac{Q_s}{2}\right)^{1.852}\right]$$

where Q_s = 135 lps; and C = 150 (plastic pipe).

- Solving the above equation, D = 326 mm (12.8 inches).
- This is slightly larger than the 12" pipe (ID = 308.1 mm) in Table 8.5.
- However, the velocity in the 12" pipe at 135 lps would be:

$$V = \frac{4Q}{\pi D^2} = \frac{4(0.135)}{\pi(0.3081)^2} = 1.81 \text{ m/s}$$

which is not too high, but as seen above, the friction loss would be too high

- It might also be noted that 135 lps is more than 2,000 gpm, a flow rate for which an irrigation system would almost always use 12" or 15" nominal pipe size
- Based on the preceding, try 15" pipe (Table 8.6) for the first half of the mainline:

$$(h_f)_{15"} = 1.217(10)^{10}\left(\frac{Q_s}{C}\right)^{1.852} D^{-4.87}\left(\frac{L}{2}\right)$$

$$(h_f)_{15"} = 1.217(10)^{10}\left(\frac{135}{150}\right)^{1.852}(369.7)^{-4.87}\left(\frac{1{,}200}{2}\right) = 1.88 \text{ m}$$

- This leaves $(h_f)_a - (h_f)_{15"}$ = 4.43 – 1.88 = 2.55 m allowable head loss in the second half of the mainline
- The allowable friction loss gradient in the second half of the mainline is:

$$J_a = 100\left(\frac{2.55}{600}\right) = 0.425 \text{ m}$$

- At ½ Q_s = 67.5 lps, this is very close to the J value for the 10" pipe in Table 8.5
- Using that 10" pipe,

$$\left(h_f\right)_{10"} = 1.217(10)^{10}\left(\frac{67.5}{150}\right)^{1.852}(259.7)^{-4.87}\left(600\right) = 2.90 \text{ m}$$

which is greater than the allowable loss of 2.55 m, but close

- Using the 15" pipe on the first half of the mainline, and 10" pipe on the second half, the total friction loss would be: $(h_f)_{total}$ = 1.88 + 2.90 = 4.78 m
- Then, the pressure at the last pair of laterals would be:

$$P_{min} = 389 - 25 - 9.81(4.64 + 4.78) = 272 \text{ kPa}$$

which is very close to the required 275 kPa at the lateral inlets

(4) Design Summary

- The design could involve three pipe sizes along the mainline
- But, in this case it works out well to use two sizes: 15" SDR 41 PIP for the first half of the mainline, and 10" SDR 41 IPS for the second half of the mainline
- Thus, there will be 600 m of 15" mainline, and 600 m of 10" mainline
- The pressure will be just about right at the end of the mainline, only 3 kPa below that which is required
- The pressure at other lateral positions will be more than enough

(5) Notes

- Tables 8.5 and 8.6 do not use the same friction loss equation
- Table 8.6 is based on Hazen-Williams with C = 155
- Minor losses past closed hydrants were not considered – if they were, it might be necessary to use a combination of 12" and 10" pipe in the second half of the mainline
- It might also be necessary to use a combination of 12" and 10" pipe in the second half of the mainline if we include a safety factor for uncertainties

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Sprinkle & Trickle Irrigation
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Assignment #5 (100 pts)
Minor Losses & Pumps
Due: 13 Oct 04

Given:

- A rectangular field to be sprinkler irrigated
- There will be four laterals, two on each side of the mainline
- All of the periodic-move laterals will move in the same direction
- At the beginning of an irrigation, two laterals are at the upstream end of the mainline (first hydrant) and the other two are at the mid-point of the mainline
- The nominal hydrant diameter, and the inside diameter, is 4 inches
- Each lateral irrigates ¼ of the field area, and each is 600 ft in length
- Required lateral inlet pressure head is P_l = 44 psi
- The mainline will be 5-inch aluminum (see Table 8.4), with hydrant values spaced at S_l = 30 ft along the 1,320-ft length of the mainline
- The first hydrant is located 30 ft from the beginning of the mainline, and the last hydrant is 1,320 ft from the beginning of the mainline
- The sprinkler system will operate 12 hrs/day, and the irrigation interval during the peak-use period is 7 days
- The gross depth to apply per irrigation is 1.9 inches
- The mainline slopes downhill at a uniform slope of 0.18%
- See the table in Chapter 8 for the roughness height of aluminum pipe
- See Table 11.1 for minor loss coefficients, K_r

Required:

- What is the field area, in acres?
- What is the system capacity, in gpm?
- Will the velocity in the mainline acceptable, or too high?
- Use the Darcy-Weisbach equation for pipe friction loss
- Make a graph of required (minimum) pressure at the upstream end of the mainline as a function of lateral position, such that the minimum lateral inlet pressure (for each lateral position) is exactly 44 psi
- This means that one pair of laterals will have exactly 44 psi inlet pressure, while the other pair will have a slightly higher inlet pressure, and this will be the case for each lateral position
- Note that there will be 1,320/(2*30) = 22 different positions lateral for each pair of laterals
- On the graph, you can call the lateral positions “1,” “2,” “3,” … “21,” and “22.”
- Make note of any assumptions and of references which you use to obtain data

Solution:

1. Field Area:

The field is given to be rectangular. Note that there are 43,560 ft^2/acre. The irrigated area is the length of the mainline (1,320 ft) multiplied by twice the length of one lateral (2 x 600 ft):

$$\frac{(1{,}320)(1{,}200)}{43{,}560} = 36.4 \text{ acres}$$

2. System Capacity:

Use Eq. 5.4 and the given data:

$$Q_s = 453\frac{Ad}{fT} = 453\frac{(36.4)(1.9)}{(7)(12)} = 373 \text{ gpm}$$

3. Velocity Checks:

Table 8.4: 5-inch aluminum pipe has an inside diameter of 4.900 inches (0.408 ft). Note that the maximum recommended velocity, in general, for sprinkler systems is 5 to 7 fps.

3(a). Full system capacity in the mainline:

$$V_{Q_s} = \frac{Q_s}{A} = \frac{4(373 \text{ gpm})}{\pi(60 \text{ s/min})(7.481 \text{ gal/ft}^3)(0.408 \text{ ft})^2} = 6.36 \text{ fps}$$

3(b). Half system capacity in the mainline:

$$V_{Q_s/2} = \frac{Q_s}{2A} = \frac{2(373 \text{ gpm})}{\pi(60 \text{ s/min})(7.481 \text{ gal/ft}^3)(0.408 \text{ ft})^2} = 3.18 \text{ fps}$$

3(c). Half system capacity through a hydrant valve:

$$V_{hydrant} = \frac{Q_s}{2A} = \frac{2(373 \text{ gpm})}{\pi(60 \text{ s/min})(7.481 \text{ gal/ft}^3)(0.333 \text{ ft})^2} = 4.77 \text{ fps}$$

All of the above velocities are below 7 fps, so they are found to be acceptable.

4. Reynolds Numbers and Darcy-Weisbach f:

The Reynolds number for a circular pipe is defined as:

$$R_e = \frac{VD}{\nu} = \frac{4Q}{\pi D \nu}$$

Assume a water temperature of 10°C. From the table on page 126 of the lecture notes (or from any other reference), the kinematic viscosity at this temperature is ν = 1.306(10)$^{-6}$ m^2/s.

4(a). Full system capacity in the mainline:

$$\left(R_e\right)_{Q_s} = \frac{4(373\text{ gpm})(0.3048\text{ m/ft})^2}{\pi(448.86\text{ gpm/cfs})(0.408\text{ ft})(1.306\text{E-6 m}^2\text{/s})} \approx 184{,}000$$

4(b). Half system capacity in the mainline:

$$\left(R_e\right)_{Q_s/2} = \frac{2(373\text{ gpm})(0.3048\text{ m/ft})^2}{\pi(448.86\text{ gpm/cfs})(0.408\text{ ft})(1.306\text{E-6 m}^2\text{/s})} \approx 92{,}000$$

From the table on page 138 (Chapter 8) of the textbook, the roughness height of aluminum pipe (with couplers as an equivalent length of pipe) is 0.005 ft. Then, from the Swamee-Jain equation:

$$f_{Q_s} = 0.0213$$

and,

$$f_{Q_s/2} = 0.0225$$

5. Velocity Heads:

There are three different velocity heads to be considered, based on the three velocities given in 3(a) – 3(c) above. These are:

5(a). Full system capacity in the mainline:

$$\frac{V_{Q_s}^2}{2g} = \frac{(6.36\text{ fps})^2}{2(32.2\text{ ft/s}^2)} = 0.628\text{ ft}$$

5(b). Half system capacity in the mainline:

$$\frac{V_{Q_s/2}^2}{2g} = \frac{(3.18\ \text{fps})^2}{2(32.2\ \text{ft/s}^2)} = 0.157\ \text{ft}$$

5(c). Half system capacity through a hydrant valve:

$$\frac{V_{\text{hydrant}}^2}{2g} = \frac{(4.77\ \text{fps})^2}{2(32.2\ \text{ft/s}^2)} = 0.353\ \text{ft}$$

6. Minor Loss Coefficients:

From Table 11.2 for a 4-inch aluminum hydrant valve:

Flow Path	**K_r**
Past closed hydrant	0.5
Past open hydrant	0.6
Through open hydrant	7.5

7. Calculating Hydraulic Losses:

At the start of an irrigation, one pair of laterals is at the first hydrant (#1), which is 30 ft from the beginning of the mainline. The second pair of laterals is at a distance of 660 ft from the first pair, at hydrant #23. At each subsequent set, the laterals move S_l = 30 ft down the mainline.

The second set will find the first pair of laterals at hydrant #2, and the second pair at hydrant #24. Finally, the last set of the irrigation will have the first pair at hydrant #22, and the second pair at hydrant #44 (the last one on the mainline).

The minimum pressure head required in the mainline pipe at a open hydrant is the required lateral inlet pressure head of (44 psi)(2.31 ft/psi) = 102 ft, plus the head loss through the hydrant valve, which is:

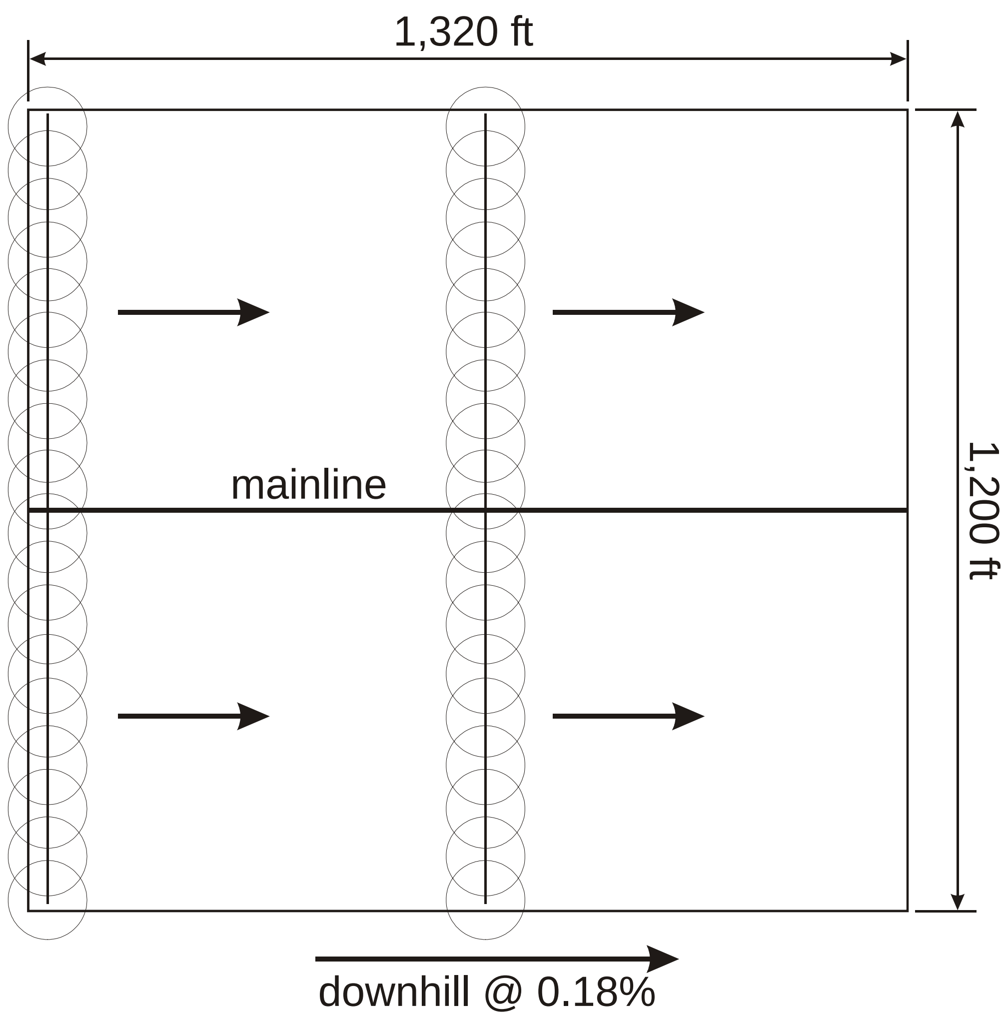
$$\left(h_f\right)_{\text{hydrant}} = K_r \frac{V^2}{2g} = 7.5\left(\frac{4.77^2}{2(32.2)}\right) = 2.65\ \text{ft}$$

Then, the minimum pressure head required in the mainline pipe at a open hydrant is:

$$h_{\text{main}} = 102 + 2.65 \approx 105\ \text{ft}$$

which is constant for any lateral position.

The following figure gives a schematic plan view of the field area:



Note that for every one of the 22 lateral positions, the second pair of laterals is always 660 ft downstream of the first pair of laterals.

7(a). Considering the First Pair of Laterals

Make a table of lateral positions in which the number of upstream closed laterals increases by one for each new lateral position (because the pair of laterals moves further from the upstream end of the mainline). Thus, the pipe friction loss and the minor losses due to flow past a closed hydrant increase with each lateral position. On the other hand, the change in elevation partially offsets these friction losses. Note that from the upstream end of the mainline to the first pair of laterals, the discharge is equal to the entire system flow rate. Consider the following table:

First Pair of Laterals						
Lateral Position	Distance to 1st Pair (ft)	Elev Change (ft)	Pipe h_f (ft)	Number of US hydrants	$(h_f)_{minor}$ (ft)	Req'd at Mainline Inlet (ft)
1	30	-0.054	0.98	0	0.00	105.9
2	60	-0.108	1.97	1	0.31	107.2
3	90	-0.162	2.95	2	0.63	108.4
4	120	-0.216	3.93	3	0.94	109.7
5	150	-0.270	4.92	4	1.26	110.9
6	180	-0.324	5.90	5	1.57	112.1
7	210	-0.378	6.88	6	1.88	113.4
8	240	-0.432	7.87	7	2.20	114.6
9	270	-0.486	8.85	8	2.51	115.9
10	300	-0.540	9.84	9	2.83	117.1
11	330	-0.594	10.82	10	3.14	118.4
12	360	-0.648	11.80	11	3.45	119.6
13	390	-0.702	12.79	12	3.77	120.9
14	420	-0.756	13.77	13	4.08	122.1
15	450	-0.810	14.75	14	4.40	123.3
16	480	-0.864	15.74	15	4.71	124.6
17	510	-0.918	16.72	16	5.02	125.8
18	540	-0.972	17.70	17	5.34	127.1
19	570	-1.026	18.69	18	5.65	128.3
20	600	-1.080	19.67	19	5.97	129.6
21	630	-1.134	20.65	20	6.28	130.8
22	660	-1.188	21.64	21	6.59	132.0

7(b). Considering the Second Pair of Laterals

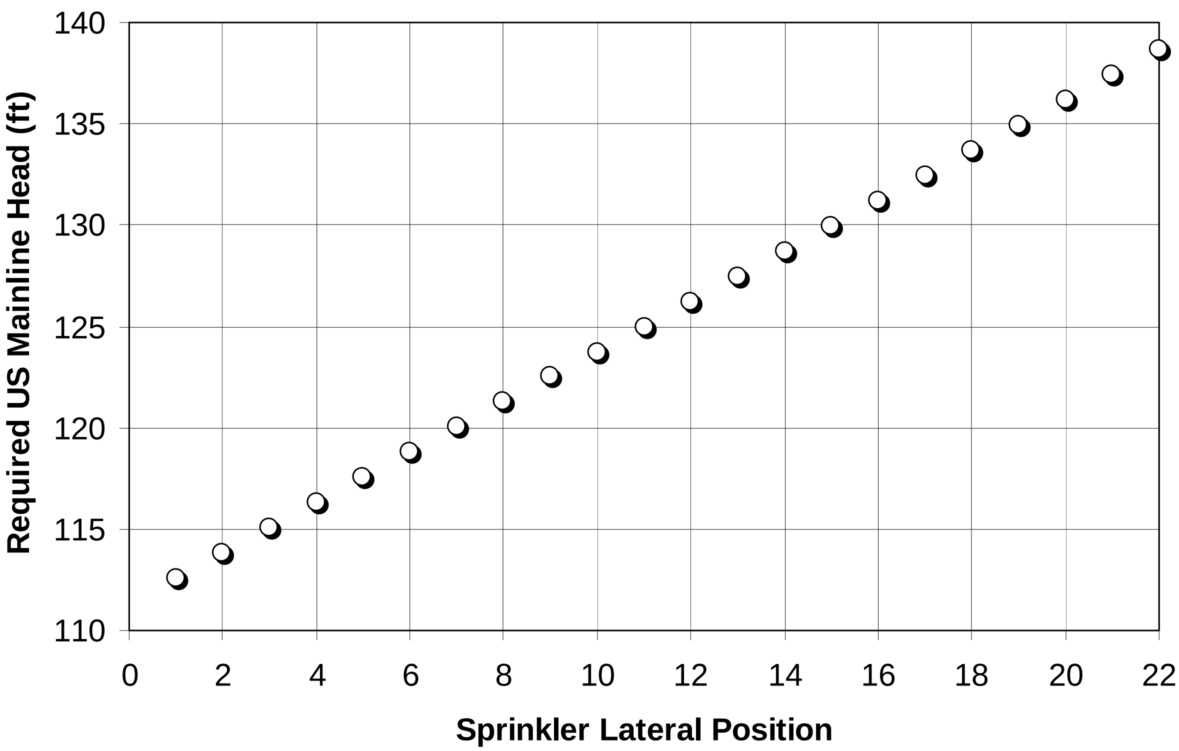
Do the same thing as for the first pair of laterals, but considering that part of the mainline has the full system flow rate, and part has only half of the system flow rate. Also, the minor loss due to “line flow” past one open hydrant (location of the first pair of laterals) must be added to the head losses.

The losses from the second pair of laterals to the upstream end of the mainline must be added to the 105-ft head requirement (see above) in the mainline at the location of the second pair of laterals. These losses include pipe friction and minor losses. Consider the following table (next page):

7(c). Extreme Position

It is seen that for each of the 22 lateral positions, the second pair of laterals require a higher pressure head at the upstream end of the mainline. This is because the downhill slope of the mainline is very small, so the friction losses dominate the pressure variation along the mainline. Thus, the following graph is for the required pressure head at the upstream end of the mainline from the perspective of the second (downstream) pair of laterals for each position, thereby giving more than enough pressure in the mainline at the location of the first pair of laterals.

Second Pair of Laterals							
Position	Distance to 1st Pair (ft)	Distance to 2nd Pair (ft)	Elev Change (ft)	Pipe hf (ft)	Number of closed US hydrants	$(h_f)_{minor}$ (ft)	Req'd at Mainline Inlet (ft)
1	30	690	-1.242	6.70	21	2.10	112.6
2	60	720	-1.296	7.68	22	2.42	113.8
3	90	750	-1.350	8.67	23	2.73	115.0
4	120	780	-1.404	9.65	24	3.05	116.3
5	150	810	-1.458	10.63	25	3.36	117.5
6	180	840	-1.512	11.62	26	3.67	118.8
7	210	870	-1.566	12.60	27	3.99	120.0
8	240	900	-1.620	13.58	28	4.30	121.3
9	270	930	-1.674	14.57	29	4.62	122.5
10	300	960	-1.728	15.55	30	4.93	123.8
11	330	990	-1.782	16.53	31	5.24	125.0
12	360	1,020	-1.836	17.52	32	5.56	126.2
13	390	1,050	-1.890	18.50	33	5.87	127.5
14	420	1,080	-1.944	19.48	34	6.19	128.7
15	450	1,110	-1.998	20.47	35	6.50	130.0
16	480	1,140	-2.052	21.45	36	6.81	131.2
17	510	1,170	-2.106	22.43	37	7.13	132.5
18	540	1,200	-2.160	23.42	38	7.44	133.7
19	570	1,230	-2.214	24.40	39	7.76	134.9
20	600	1,260	-2.268	25.39	40	8.07	136.2
21	630	1,290	-2.322	26.37	41	8.38	137.4
22	660	1,320	-2.376	27.35	42	8.70	138.7



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Sprinkle & Trickle Irrigation
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Assignment #6 (100 pts)
Pump Characteristic Curves
Due: 20 Oct 04

Given:

- A graph of pump characteristic curves for Cornell Model 4HS
- Notice the abscissa scale and the note saying that the pump curves are "flat" from Q = 0 to Q = 100 gpm
- The desired operating point on your system curve is Q = 550 gpm at TDH = 72 ft
- The pump will take water from a large concrete-lined canal with water surface which can vary from 1,330 to 1,333 ft above mean sea level
- The suction side of the pump has a screen, 8.0 ft of 6-inch SDR41 IPS PVC pipe, and one 45-degree elbow
- The pump itself is located at an elevation of 1,338 ft above mean sea level
- The crop to be grown in the irrigated field is alfalfa

Required:

1. What is the nominal pump speed (RPM)?
2. What is the pump efficiency at the desired operating point?
3. What is the WHP at the desired operating point?
4. What is the calculated BHP at the desired operating point? Does it match the BHP value given by the manufacturer on the graph?
5. Select an impeller diameter (inches) from the options given on the graph
6. What is the required pump speed (RPM) such that the desired operating point is the actual operating point?
7. What is the required impeller trim (reduction in D) if the pump speed cannot be changed, such that the desired operating point is the actual operating point?
8. What is the $NPSH_r$ at the desired operating point?
9. What is the $NPSH_a$ at the desired operating point?
10. Do you expect the pump to cavitate at the desired operating point?

Solutions:

1. The nominal pump speed is listed on the Cornell sheet as 1200 RPM. The exact speed for the pump curves is 1175 RPM.
2. From the graph, at 550 gpm and 72 ft head, the efficiency is approximately 75%, to the nearest whole number; or more generally, the efficiency is between 74 and 75%.
3. At the desired operating point, the WHP is:

$$WHP = \frac{QH}{3956} = \frac{(550)(72)}{3956} = 10.0 \text{ HP}$$

   or, 7.46 kW.

4. The calculated BHP is:

$$BHP = \frac{WHP}{E_{pump}} = \frac{10.0}{0.75} = 13.3 \text{ HP}$$

   or, 9.94 kW. Interpolating on the graph, the BHP appears to be approximately 14 HP, or perhaps slightly less than 14 HP. Thus, the calculated value agrees fairly well with the graphical value, given the need for interpolation “by eye.” Also, the 14 HP estimate is sufficient to determine a power unit (motor) to drive this pump.

5. Select the nearest pump curve which is above the desired operating point (unless a pump curve is only slightly below the desired operating point). In this case, choose the curve for the 14” nominal diameter.

6. For this, apply the procedure given in the lecture notes. Develop some points for the “equal efficiency” curve, and make them near the desired operating point such that the curve will intersect with the pump curve.

Q_1 (gpm)	H_1 (ft)
560	74.6
570	77.3
580	80.1
590	82.9
600	85.7

   After graphing the points from the above table, the intersection with the 14” pump characteristic curve is approximately:

   H_3 = 77 ft
   Q_3 = 568 gpm

Finally, reduce the pump speed as follows:

$$N_{new} = 1175\left(\frac{550}{568}\right) = 1138 \text{ RPM}$$

Note that the intersection (Q_3, H_3) is usually close to the actual operating point without a change in speed, N, and the value of Q_3 is not much different than Q_2. In this case, the adjustment in speed is slight and it might not be worth the expense to gear down to N_{new}; instead, it might be better to accept the actual operating point without changing speeds.

To know the actual operating point without changing speeds, we would need sufficient information to develop the system curve.

7. The intersection (Q_3, H_3) is already known from the previous calculations, so calculate the required impeller diameter straight away as follows:

$$D_{new} = 14\left(\frac{550}{568}\right) = 13.6 \text{ inches}$$

where it is assumed that the nominal diameter is the actual standard diameter. The table below the graph shows maximum impeller diameters, but to apply them it would be necessary to consult the manufacturer to be sure.

Note that the above impeller trim (0.4”) is very slight.

8. Extrapolating in the manufacturer’s curves, at the desired operating point, $NPSH_r \approx 5$ ft.

9. Determine $NPSH_a$ as given in the lecture notes:

Maximum static lift is given as:

$$(h_{lift})_{max} = 1338 – 1330 = 8 \text{ ft}$$

Average atmospheric pressure head:

$$(h_{atm})_{avg} = 10.3 – 0.00105\ (1{,}338)(0.3048) = 9.87 \text{ m } (32.4 \text{ ft})$$

Assuming a water temperature of 10°C:

$$h_{vapor} = 0.0623\exp\left(\frac{17.27(10)}{10+237.3}\right) = 0.125 \text{ m } (0.411 \text{ ft})$$

From Table 8.5, the suction pipe has an ID of 6.301 inches. Then, pipe area is:

$$A_{pipe} = \frac{\pi(6.301/12)^2}{4} = 0.217 \text{ ft}^2$$

The flow rate is: Q = 550/448.86 = 1.23 cfs. And, the velocity head is:

$$\frac{V^2}{2g} = \frac{(1.23)^2}{2(32.2)(0.217)^2} = 0.500 \text{ ft}$$

Friction loss in the suction pipe, using Hazen-Williams:

$$h_f = 10.5(8.0)\left(\frac{550}{150}\right)^{1.852}(6.301)^{-4.87} = 0.12 \text{ ft}$$

Minor losses on suction side of pump: From Table 11.2 for a flanged “long-radius” 45-degree elbow, 6” nominal size, K_r = 0.17. Also from Table 11.2, for a “basket strainer,” K_r = 0.85. Then,

$$(h_f)_{minor} = (0.17 + 0.85)(0.500) = 0.51 \text{ ft}$$

Notice that the minor losses are greater than the pipe friction losses (the pipe is only 8-ft long, and it is PVC, so it’s smooth) . Note that the average velocity in the suction pipe is 5.67 fps, which is OK. Note also that the static lift will have a much greater influence on $NPSH_a$ than friction losses, and that the velocity head is on the same order of magnitude as the friction losses. Finally,

$$NPSH_a = 32.4 - 0.12 - 0.51 - 8.0 - 0.50 = 23.3 \text{ ft}$$

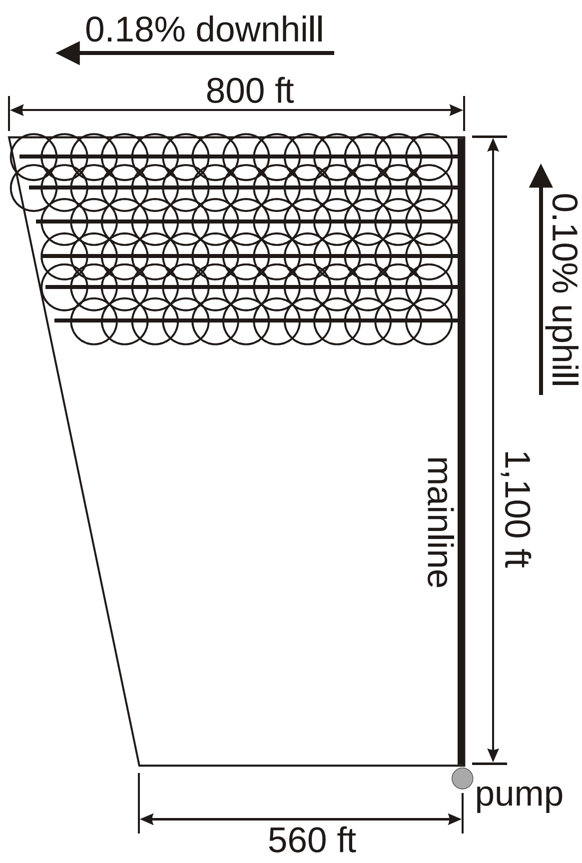
10. This pump installation will not be expected to cavitate because $NPSH_a$ >> $NPSH_r$.

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester, 2004

Assignment #7 (200 pts)
Operating Point for Fixed Sprinkler System
Due: 29 Oct 04

Given:

- A fixed sprinkler system (all sprinklers operate simultaneously) in an orchard.
- Buried IPS-PVC 1-½" lateral pipes (see Table 8.3 of the textbook).
- Rainbird® under-tree M20VH impact sprinklers with 5/64-inch SBN-1 nozzle (see www.rainbird.com for technical specifications).
- Lateral spacing: S_l = 40.00 ft.
- Sprinkler spacing: S_e = 40.00 ft.
- The field is trapezoidal in shape, as shown below:



- In the above figure, only some of the laterals and sprinklers are shown, but remember that the field area is covered with sprinklers in a fixed, permanent system.
- There are 27 laterals along the mainline.

- Note the field slopes along mainline and lateral directions.
- The suction (upstream) side of the pump has a 4-ft static lift from a pond and 10 ft of 8" PVC pipe (ID = 8.205") with one 90-degree elbow and a strainer screen at the inlet.
- The mainline is 8" PVC pipe (ID = 8.205"), and is 1,100-ft long.
- Sprinkler riser height is: h_r = 3.0 ft.
- A Berkeley model 4GQH pump curve as shown on the following page. You will use the characteristic curve for 1600 RPM.
- Ignore minor losses along the mainline and laterals.

Required:

1. Determine the irrigated area (acres).
2. Develop an equation for sprinkler flow rate (q) as a function of pressure (P), whereby $q = K_d P^x$ (you determine K_d and x based on manufacturer's data). Note that for a straight-bore nozzle, you would expect x to be very close to 0.5, as in Eq. 5.1a.
3. Determine the number of sprinklers along each lateral, where lateral #1 is the closest to the pump, and lateral #27 is the furthest from the pump.
4. Develop at least five points on the system curve and present those points numerically in a table. You should write a macro (Excel) or computer program to do this (don't attempt to do it by hand with only a calculator).
5. Plot the system curve points on the attached pump curve graph, and draw a smooth curve through the points.
6. Determine the operating point (Q in gpm & TDH in ft) for this system.
7. Determine the average application rate for the whole field area.

Do you work in an organized, neat way. Make comments about assumptions and other technical issues as appropriate. Turn in all your work, including the code for your macro or computer program.

Solution:

1. The irrigated area is approximately:

$$A = \frac{0.5(800+560)(1100)}{43{,}560} = 17.2 \text{ acres}$$

   Due to the trapezoidal field shape, and the fact that there are 27 laterals at S_l = 40 ft, the effective irrigated area is slightly less than 17.2 acres.

2. A linear regression is performed on logarithms of the manufacturer's data for P and q (Rainbird® M20VH with 5/64-inch SBN-1 nozzle):

Pressure (psi)	Flow (gpm)	ln(P)	ln(q)
25	0.88	3.2189	-0.1278
30	0.97	3.4012	-0.0305
35	1.05	3.5553	0.0488
40	1.12	3.6889	0.1133
45	1.19	3.8067	0.1740
50	1.25	3.9120	0.2231

   given an R_2 of 0.9996, and the following equation:

$$q = 0.173 P^{0.506}$$

   for q in gpm; and P in psi. Note that the exponent is close to 0.500, which is expected for a straight-bore nozzle. But notice also that allowing for the flexibility in the exponent, x, gives a better mathematical fit to the manufacturer's data.

3. Let the first lateral be located at ½S_l from the lower edge of the field (where the pump is located). Calculate the number of sprinklers per lateral by rounding the potential lateral length by S_e. The potential lateral length is calculated by linear interpolation along the left side of the field area (see the figure given above). The equation is given below, and the calculation results are shown in the following table.

$$L = 800 - (1{,}100 - y)\left(\frac{800-560}{1{,}100}\right)$$

   where y is the distance along the mainline (ft); and L is the potential lateral length (ft).

Lateral	Distance (ft)	Length (ft)	No. of Sprinklers
1	20	564.4	14
2	60	573.1	14
3	100	581.8	15
4	140	590.5	15
5	180	599.3	15
6	220	608.0	15
7	260	616.7	15
8	300	625.5	16
9	340	634.2	16
10	380	642.9	16
11	420	651.6	16
12	460	660.4	17
13	500	669.1	17
14	540	677.8	17
15	580	686.5	17
16	620	695.3	17
17	660	704.0	18
18	700	712.7	18
19	740	721.5	18
20	780	730.2	18
21	820	738.9	18
22	860	747.6	19
23	900	756.4	19
24	940	765.1	19
25	980	773.8	19
26	1,020	782.5	20
27	1,060	791.3	20

4. Develop a computer program to start with a given pressure at the furthest downstream sprinkler on lateral #27, calculate the flow rate at that sprinkler, calculate the pressure at the next upstream sprinkler, then the flow rate at that sprinkler, and so on, until reaching the mainline. Calculate the pressure in the mainline at the location of lateral #26, then iterate along lateral #26 to get the same pressure in the mainline at that location. Repeat for all other laterals, moving in the upstream direction, until a pressure is obtained for the upstream end of the mainline. The system flow rate is known from these calculations (sum of all individual sprinklers). Assume pipe leakage is zero.

   Knowing the system flow rate, determine the losses in the suction side of the pipe, and determine TDH by adding the velocity head at the beginning of the mainline, the pressure head at the beginning of the mainline, the static lift on the suction side, and the hydraulic losses on the suction side of the pump.

Preliminary calculations and assumptions:

Assume a water temperature of 10°C, giving a kinematic viscosity of: $1.306(10)^{-6}$ m^2/s, or $1.406(10)^{-5}$ ft^2/s.

Use the Swamee-Jain equation with $\varepsilon = 1.5(10)^{-6}$ m, or $4.92(10)^{-6}$ ft for PVC to obtain the Darcy-Weisbach friction factor, f.

From Table 8.3, the ID of the lateral pipe is 1.754 inches (0.1462 ft). The ID of the mainline pipe is 8.205 inches (0.6838 ft).

Once the pressure at the upstream end of the mainline is calculated, the additional TDH values include static lift, velocity head, and losses in the suction side of the pump, plus the riser height. Assume the pump outlet is at the same elevation as the upstream end of the mainline (you don't know if the mainline or laterals are buried, nor how deep they might be, but this information could be used to develop a somewhat more specific design).

$$\mathrm{TDH} = 2.308\,P_{main} + h_{lift} + h_r + \left(h_f\right)_{suction} + \frac{V^2}{2g}$$

where TDH is in ft; P_{main} is the pressure at the upstream end of the mainline (psi); h_{lift} is given as 4.0 ft; h_r is given as 3.0 ft; $(h_f)_{suction}$ are the hydraulic losses in the suction pipe (ft); and the last term is the velocity head (ft).

The iterative part is to determine P_{main}; the rest of the TDH terms are easy to calculate directly.

From Table 11.2 (minor loss coefficients):

8-inch "basket strainer": $K_r = 0.75$
8-inch "regular 90-deg elbow": $K_r = 0.26$

Velocity head in suction pipe:

$$\frac{V^2}{2g} = \frac{8Q^2}{g\pi^2 D^4} = 0.1151 Q^2$$

Friction loss in suction pipe:

$$h_f = f\frac{L}{D}\frac{V^2}{2g} = 1.684\,f\,Q^2$$

Putting it all together:

$$TDH = 2.308 P_{main} + 4.0 + 3.0 + 1.684\, f\, Q^2 + 0.1151 Q^2 \left(1 + 0.75 + 0.26\right)$$

or,

$$TDH = 2.308 P_{main} + 7.0 + Q^2 \left[1.684\, f + 0.2314\right]$$

The results are given in the table below:

P (psi)	**Q_s (gpm)**	**P_{main} (psi)**	**R_e**	**f**	**TDH (ft)**
20	367.2	21.8	108,347	0.01761	57.50
25	411.2	27.2	121,318	0.01721	70.09
30	451.0	32.7	133,061	0.01689	82.66
35	487.6	38.1	143,853	0.01663	95.21
40	521.6	43.5	153,902	0.01641	107.74
45	553.6	48.9	163,344	0.01622	120.26
50	583.9	54.3	172,277	0.01605	132.77
55	612.7	59.7	180,777	0.01590	145.28
60	640.2	65.1	188,901	0.01577	157.77

where P is the pressure at the furthest downstream sprinkler on lateral #27; Q_s is the total system flow rate; P_{main} is the pressure at the upstream end of the mainline (just downstream of the pump); R_e is the Reynold's number in the suction pipe; f is the value from Swamee-Jain; and TDH is the total dynamic head.

5. The system curve (Qs versus TDH) is superimposed upon the pump manufacturer's curves, as shown in the figure below.

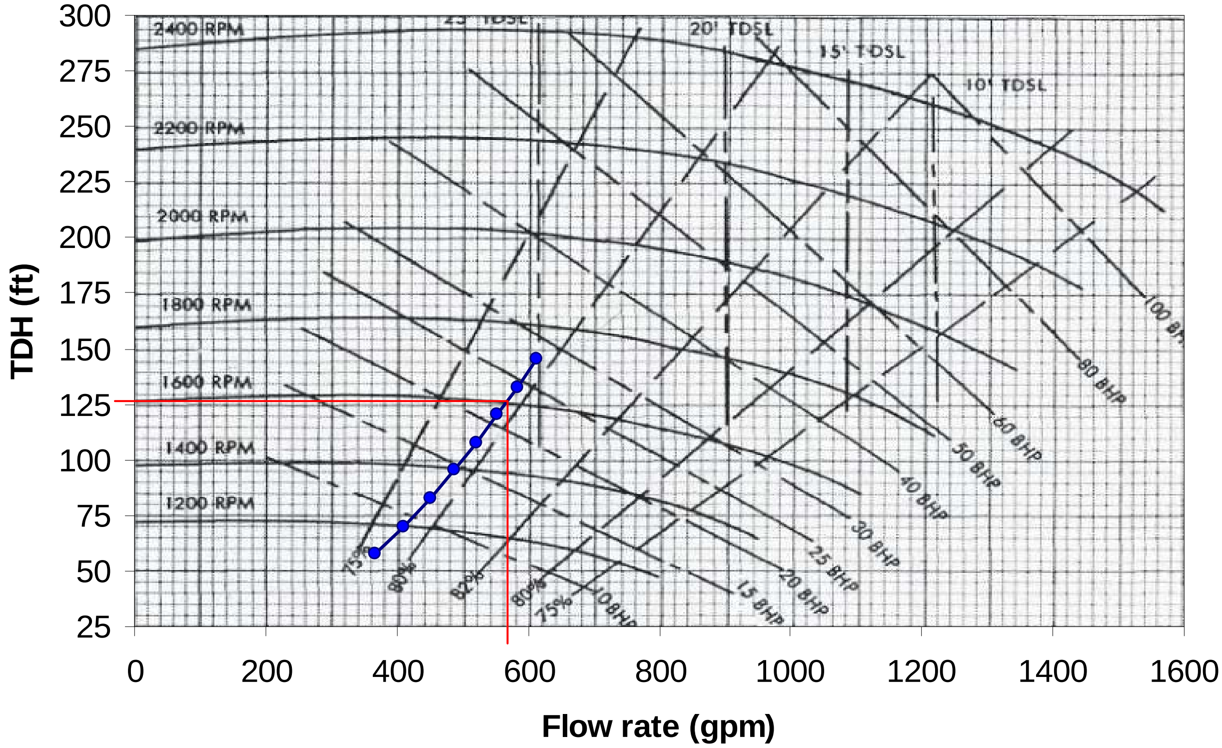
6. The operating point (intersection of the system curve and the 1600 RPM pump curve) is seen to be approximately:

Q_s = 568 gpm
TDH = 126 ft

7. The average application rate at this operating point is approximately:

$$AR_{avg} = \frac{(568)(12)(3600)}{(458)(40)(40)(448.86)} = 0.075 \text{ inch/hr}$$

or 1.9 mm/hr.



The MS Excel VBA listing follows:

```
Const Pi = 3.141593
Const Se = 40#          'ft
Const Sl = 40#          'ft
Const LatCount = 27     'number of laterals
Const Dlat = 0.1462     'ft
Const Dmain = 0.6838    'ft
Const SoLat = -0.0018   'ft/ft
Const SoMain = 0.001    'ft/ft

Function hf(ByVal Q As Double, ByVal D As Double, ByVal L As Double) As Double
'-------------------------------------------------------------------------------
' Returns friction loss in feet of water head (Darcy-Weisbach).
' If turbulent, uses the Swamee-Jain equation for the friction factor, f.
'-------------------------------------------------------------------------------

  Dim Re As Double, RelRough As Double, f As Double

  viscosity = 0.00001406  'ft2/s

  RelRough = 0.00000492 / D
  Re = 4 * Q / (viscosity * Pi * D)

  If Re > 4000 Then
    f = RelRough / 3.75 + 5.74 / Re ^ 0.9   'Turbulent
    f = WorksheetFunction.Log10(f)
    f = 0.25 / f ^ 2
  Else
    f = 64 / Re                              'Laminar
  End If
```

```
  hf = 8# * f * L * (Q / Pi) ^ 2 / (32.2 * D ^ 5)

End Function

Function Lateral(ByVal P As Double, n As Integer, Qlat As Double) As Double
'-------------------------------------------------------------------------------
' Calculates lateral inlet pressure head for "n" sprinklers.
' Pressure, P, is in psi.
'-------------------------------------------------------------------------------

  Dim Qs As Double, h As Double

  Qlat = 0# 'cfs

  For i = 1 To n
    Qs = 0.173 * P ^ 0.506                                'gpm
    Qlat = Qlat + Qs / 448.86                             'cfs
    h = P * 2.308 + hf(Qlat, Dlat, Se) + SoLat * Se     'ft
    P = h / 2.308                                         'psi
  Next

  Lateral = h

End Function

Function ParabolaFit(x, y, target As Double) As Double
'-------------------------------------------------------------------------------
' Determines constants a, b, and c for a parabola through three points.
' Equation is: y = ax^2 + bx + c.  If parabola impossible, uses bisection.
' Returns the x-value (P) which matches the specified target y-value (hmain).
' P is the pressure at the furthest DS sprinkler in the lateral.
'-------------------------------------------------------------------------------

  Dim Bisection As Boolean
  Dim foo As Double, boo As Double
  Dim a As Double, b As Double, c As Double

  ParabolaFit = 0
  foo = x(2) - x(1)
  Bisection = Abs(foo) < 0.0000000001

  If Not Bisection Then

    C0 = x(1) ^ 2
    c1 = (x(3) - x(1)) / foo
    c2 = x(2) ^ 2 - C0

    boo = x(3) ^ 2 - C0 - c1 * c2
    Bisection = Abs(boo) < 0.0000000001

    If Not Bisection Then

      '-------------------------
      ' Parabolic interpolation
      '-------------------------

      a = ((y(1) - y(2)) * c1 - y(1) + y(3)) / boo
      b = (y(2) - y(1) - a * c2) / foo
      c = y(2) - x(2) * (a * x(2) + b)

      c = c - target
      ParabolaFit = (-b + Sqr(b * b - 4 * a * c)) / (2 * a)
      Exit Function
    End If
  End If
```

```
  If Bisection Then

    '--------------
    ' Use bisection
    '--------------

    If y(2) < target Then
      ParabolaFit = (y(2) + y(3)) / 2
    Else
      ParabolaFit = (y(2) + y(1)) / 2
    End If
  End If

End Function

Function QsPmain(ByVal P As Double, Flow As Boolean) As Double
'-------------------------------------------------------------------------------
' Iterates to determine the system flow rate for a given starting pressure.
' Returns either flow rate (Qs) or pressure (Pmain) at US end of the mainline.
'-------------------------------------------------------------------------------

  Dim n As Integer
  Dim lat As Integer
  Dim LatHead(1 To 3) As Double
  Dim Pressure(1 To 3) As Double
  Dim Qlat As Double, x As Double
  Dim Qmain As Double, L As Double, NewP As Double, hmain As Double

  hmain = 0
  Qmain = 0

  For lat = LatCount To 1 Step -1

    '--------------------------------------------
    ' Determine number of sprinklers this lateral
    '--------------------------------------------

    x = 20 + (lat - 1) * Sl
    L = 800# - (1100# - x) * 0.2181818
    n = Round(L / Se, 0)

    '--------------------------------------
    ' Calculate lateral inlet pressure head
    '--------------------------------------

    If lat = LatCount Then

      '------------------------------------
      ' No need to iterate for last lateral
      '------------------------------------

      LatHead(2) = Lateral(P, n, Qlat)
    Else

      '------------------------------------------------
      ' Iterate to match lateral inlet & mainline heads
      '------------------------------------------------

      Pressure(1) = P / 4
      Pressure(3) = 3 * P
      Pressure(2) = (Pressure(1) + Pressure(3)) / 2

      LatHead(1) = Lateral(Pressure(1), n, Qlat)
      LatHead(3) = Lateral(Pressure(3), n, Qlat)
      LatHead(2) = Lateral(Pressure(2), n, Qlat)
```

```
      If (LatHead(1) > hmain) Or (LatHead(3) < hmain) Then

        '-------------------------------
        ' Failed to bracket the solution
        '-------------------------------

        QsPmain = -100
        Exit Function
      End If

      For i = 1 To 50

        '----------------------------------
        ' Search by parabolic interpolation
        '----------------------------------

        NewP = ParabolaFit(Pressure, LatHead, hmain)

        If NewP < Pressure(2) Then
          Pressure(3) = Pressure(2)
          LatHead(3) = LatHead(2)
        Else
          Pressure(1) = Pressure(2)
          LatHead(1) = LatHead(2)
        End If

        Pressure(2) = NewP
        LatHead(2) = Lateral(NewP, n, Qlat)

        If Abs(LatHead(2) - hmain) < 0.001 Then

          '-------------------
          ' Solution converged
          '-------------------

          Exit For
        End If
      Next

    End If

    '-----------------------------------------
    ' Move upstream one hydrant along mainline
    '-----------------------------------------

    Qmain = Qmain + Qlat
    hmain = LatHead(2) + hf(Qmain, Dmain, Sl) + SoMain * Sl
  Next

  '----------------------------
  ' Return the system flow rate
  ' or the mainline pressure
  '----------------------------

  If Flow Then
    QsPmain = Qmain * 448.86  'gpm
  Else
    QsPmain = hmain / 2.308   'psi
  End If

End Function
```

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester, 2004

Assignment #8 (100 pts)
Trickle System Design Calculations
Due: 01 Dec 04

Do you work in an organized, neat way. Write down any assumptions you make.

Given:

- A mature walnut orchard will be drip-irrigated.
- Use a single lateral per row of trees.
- Irrigated area is 44 ha.
- Tree spacing is 6.2 x 6.2 m.
- Peak daily ET is U_d = 4.9 mm/day.
- Seasonal water requirement: U = 541 mm.
- Effective rain, peak-use period: assume zero.
- Residual soil water in the spring: assume zero.
- Use an MAD of 25%.
- Soil water holding capacity is 178 mm/m.
- Water source: deep well with maximum discharge of 125 lps.
- Irrigation water quality: EC_w = 0.61 dS/m.
- Root zone depth is 2.0 m.
- Shaded area is 75%.
- Emitter equation:

$$q = 0.32P^{0.53}$$

for q in lph; and P in kPa.

- Nominal emitter flow rate: q_a = 4 lph.
- Manufacturer coefficient of variation: 0.062.
- Average wetted width at 4 lph: w = 2.33 m.
- Outlets per emitter: one.

Required:

1. Use metric units in your calculations.
2. Select an appropriate emitter spacing, S_e (m).
3. Determine the number of emitters per tree, N_p.
4. Calculate percent wetted area, P_w. Use the equation from Lecture 18 (includes P_d in the denominator). Make sure P_w is between 33% and 67%; if not, increase N_p as necessary.
5. Calculate maximum net depth to apply per irrigation, d_x (mm).
6. Calculate the average peak daily "transpiration" rate, T_d (mm/day).
7. Calculate the maximum irrigation interval, f_x. If $f_x \geq 1$ day, then use f' = 1day.
8. Calculate the net depth per irrigation, d_n (mm).
9. Select a reasonable target EU value (Table 20.3).
10. Determine $(EC_e)_{max}$ (Table 19.2).
11. Determine the transmission ratio, T_r (Table 19.3).
12. Calculate the leaching requirement, LR_t.
13. Calculate the gross depth to apply per irrigation, d (mm).
14. Calculate the gross volume of water per tree per day, G (liter/tree/day).
15. Calculate h_a, corresponding to q_a = 4 lph, in m of water head (not kPa).
16. Calculate the water application time, T_a (hrs).
17. If T_a > 21.6 hrs/day, recalculate q_a such that T_a = 21.6 hrs/day, then calculate h_a corresponding to the new q_a value.
18. Determine the number of stations, N_s.
19. Determine the minimum number of emitters per tree, N_p'.
20. Calculate the system coefficient of variation, ν_s.
21. Calculate the minimum allowable emitter flow rate, q_n (lph).
22. Calculate the allowable subunit pressure head variation, ΔH_s (m).
23. Calculate the system capacity, Q_s (lps). Is this less than or equal to the well capacity of 125 lps?
24. Calculate the total gross seasonal depth to apply, D_g (mm).
25. Calculate the gross seasonal volume of irrigation water, V_s (m^3).
26. Calculate the required number of operating hours per season, O_t (hrs/season). Make sure it is not more than 8,760 hrs!

Required:

I. Emitter spacing

Use the "optimal" spacing: S_e = 0.8w = 0.8(2.33) = 1.86 m

II. Emitters per tree

$$N_p = \frac{S_p}{S_e} = \frac{6.2}{1.86} = 3.33$$

III. Percent wetted area

$$P_w = 100\left(\frac{N_p S_e w}{S_p S_r P_d}\right) = 100\left(\frac{(3.33)(1.86)(2.33)}{(6.2)(6.2)(0.75)}\right) = 50.1\%$$

IV. Maximum net application depth

$$d_x = \frac{MAD}{100}\frac{P_w}{100}W_a Z = (0.25)(0.501)(178)(2.0) = 44.6 \text{ mm}$$

V. Average peak daily transpiration rate

$$T_d = 0.1U_d\sqrt{P_d} = 0.1(4.9)\sqrt{75} = 4.24 \text{ mm/day}$$

VI. Maximum irrigation interval

$$f_x = \frac{d_x}{T_d} = \frac{44.6}{4.24} = 10.5 \text{ days}$$

Then, use f' = 1 day (for design purposes).

VII. Net depth per irrigation

$$d_n = T_d f' = (4.24)(1) = 4.24 \text{ mm/day}$$

VIII. Target EU

Table 20.3: "point-source" water applicators with $N_p > 3$ gives recommended EU range of 90 to 95%. In this design iteration, choose EU = 92%.

IX. Maximum EC_e

From Table 19.2, for a walnut crop, $(EC_e)_{max}$ = 8 dS/m.

X. Transmission ratio

From Table 19.3, for a “deep-rooted” (Z > 1.5 m) crop and a “medium-textured” (see Wa above) soil: T_r = 1.00.

XI. Leaching requirement

$$LR_t = \frac{EC_w}{2\left(EC_e\right)_{max}} = \frac{0.61}{2(8)} = 0.038$$

XII. Gross application depth

For $LR_t < 0.1$, the following equation is applied:

$$d = 100\left(\frac{d_n T_r}{EU}\right) = 100\left(\frac{(4.24)(1.00)}{92\%}\right) = 4.61 \text{ mm/day}$$

XIII. Gross volume of water per tree

$$G = \frac{d}{f'} S_p S_r = \frac{4.61}{1}(6.2)(6.2) = 177 \text{ liter/day/tree}$$

XIV. Nominal emitter pressure head

Apply the given emitter equation, and use 9.81 kPa/m:

$$h_a = \left(\frac{1}{9.81}\right)\left(\frac{4}{0.32}\right)^{1/0.53} = 12.0 \text{ m}$$

XV. Water application time per irrigation

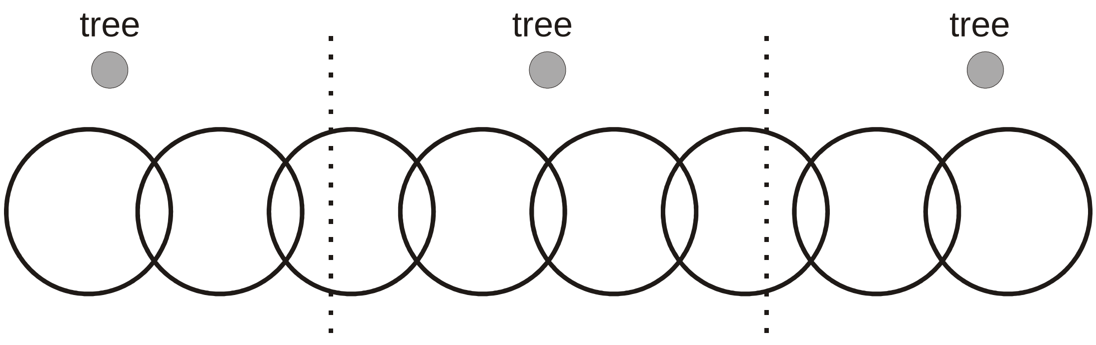
$$T_a = \frac{G}{N_p q_a} = \frac{177}{(3.33)(4)} = 13.3 \text{ hrs/day}$$

XVI. Number of stations

Two stations would require 2(13.3) = 26.6 hrs/day. Thus, there can be only one station (N_s = 1) in this design.

XVII. Minimum number of emitters per tree

See the figure below, showing a tree spacing of 6.2 m, and emitter spacing of 1.86 m, and a wetted width of 2.33 m:



It is seen that, on average, four emitters contribute some irrigation water to each tree. Alternatively,

$$N'_p = \text{trunc}\left(\frac{5 \text{ m/tree}}{1.86 \text{ m/emitter}} + 2\right) = 4$$

where "trunc" means to truncate (round down) to the nearest whole number.

XVIII. System coefficient of variation

$$\nu_s = \frac{\nu}{\sqrt{N'_p}} = \frac{0.062}{\sqrt{4}} = 0.031$$

XIX. Minimum allowable emitter flow

$$q_n = \frac{q_a EU}{100\left(1 - 1.27\nu_s\right)} = \frac{(4)(92)}{100\left(1 - 1.27(0.031)\right)} = 3.83 \text{ lph}$$

which corresponds to a head of:

$$h_n = \left(\frac{1}{9.81}\right)\left(\frac{3.83}{0.32}\right)^{1/0.53} = 11.0 \text{ m}$$

XX. Allowable subunit head variation

$$\Delta H_s = 2.5\left(h_a - h_n\right) = 2.5\left(12.0 - 11.0\right) = 2.5 \text{ m}$$

XXI. System capacity

$$Q_s = 2.78\frac{A N_p q_a}{N_s S_p S_r} = 2.78\frac{(44)(3.33)(4)}{(1)(6.2)(6.2)} = 42.4 \text{ lps}$$

which is less than the well capacity of 125 lps. Thus, the well has sufficient flow rate to accommodate this design.

XXII. Gross season water application depth

Assume a T_R value of 1.00 (Table 19.4). Then, E_s = EU = 92%. Effective rain and residual soil moisture are given to be zero. Thus,

$$D_n = U\left(0.1\sqrt{P_d}\right) = 541\left(0.1\sqrt{75}\right) = 469 \text{ mm}$$

Then, gross seasonal depth is:

$$D_g = \frac{100 D_n}{E_s\left(1 - LR_t\right)} = \frac{100(469)}{92(1 - 0.038)} = 530 \text{ mm}$$

XXIII. Gross season application volume

$$V_s = \frac{D_g A}{1000} = \frac{(530)(44)}{1000} = 23.3 \text{ ha-m}$$

XXIV. Operating hours per season

$$O_t = 2778\frac{V_s}{Q_s} = 2778\left(\frac{23.3}{42.4}\right) \approx 1{,}530 \text{ hrs/season}$$

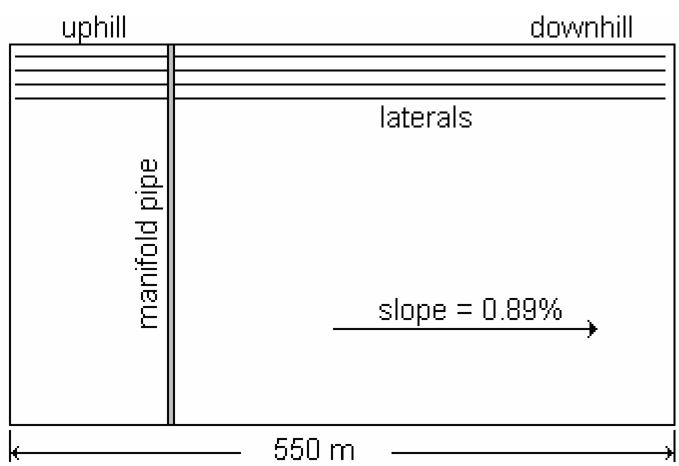
BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester, 2004

Assignment #9 (100 pts)
Trickle Manifold Location
Due: 3 Dec 04

Given:

A rectangular field of orchard trees, 550-m long in the direction of the PE laterals. The preliminary design data are as follows:

lateral ID	= 17.8 mm	q_a	= 3.95 lph
S_e	= 2.5 m	f_e	= 0.10 m
S_p	= 5.0 m	H_a	= 13.0 m
x	= 0.55	Q_s	=9.77 lps



Required:

Determine the following using either: (1) the semi-graphical, non-dimensional, design procedure, or (2) a completely numerical design procedure:

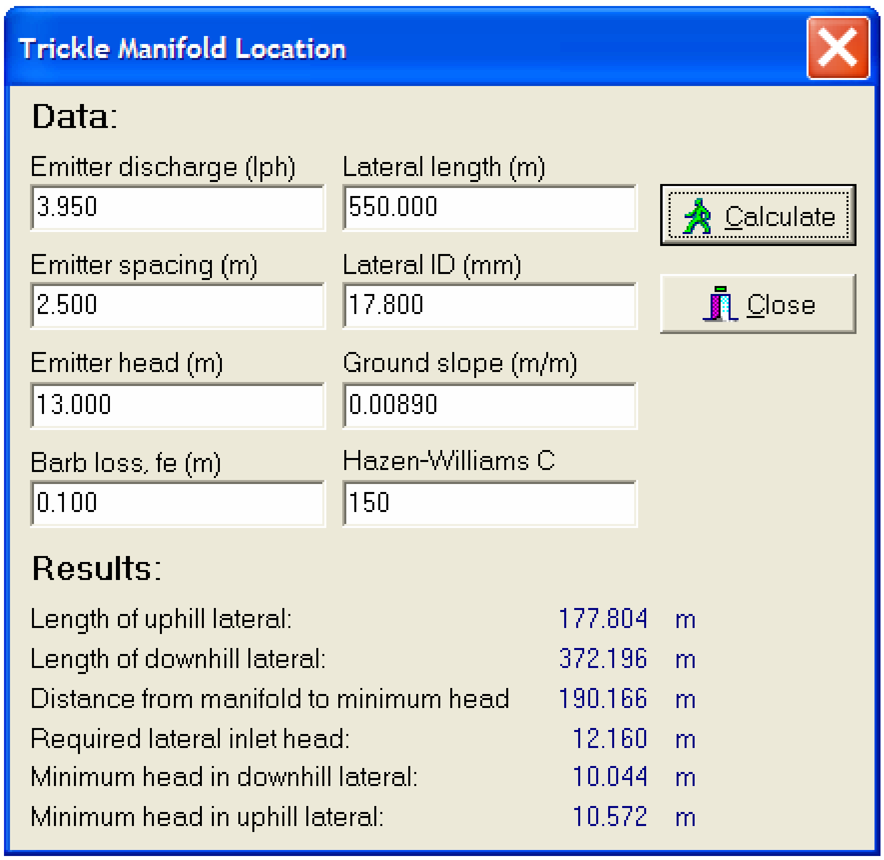
1. Optimal manifold location
2. Required lateral inlet pressure head, H_l
3. Minimum lateral pressure head, H_n'

Show all of your work neatly, step by step. Adjust the manifold location uphill by as much as $0.75(S_p)$, or downhill by as much as $0.25(S_p)$ so that is is positioned midway between two plant rows.

1. How many trees on the uphill side?
2. How many trees on the downhill side?
3. Is H_n' the same on the uphill & downhill sides?

Solution:

- The solution can be obtained by different methods, as explained in class.
- The quickest and easiest solution is to use the *New and Improved* “OptManifold.exe” computer program, as shown below:



- The *New and Improved* version of the program uses Hazen-Williams.
- Due to approximations and simplifying assumptions in the equations, the calculated value of H_n’ is not exactly equal in the uphill and downhill parts of the lateral.
- The tree spacing is given as S_p = 5.0 m. Make the downhill lateral 375-m long, and the uphill lateral will be 175 m in length (a slight adjustment on the calculation results).
- This gives 175/5 = 35 trees on the uphill side of the manifold, and 375/5 = 75 trees on the downhill side.

BIE 5110/6110
Sprinkle & Trickle Irrigation
Fall Semester, 2004

Assignment #10 (100 pts)
Trickle Manifold Pipe Sizing
Due: 13 Dec 04

Given:

- A trickle irrigation system with a manifold inflow rate of 5.00 lps.
- An allowable subunit pressure head variation of ΔH_s = 4.2 m.
- A lateral pressure variation of ΔH_l = 2.3 m.
- A uniform ground slope of 1.04% in the manifold direction.
- A total manifold length of 200 m.
- A lateral spacing of 2.0 m.

- The following PVC pipe sizes are available:

Size	I.D.	
(inches)	(inches)	(mm)
0.5	0.622	15.8
0.75	0.824	20.9
1	1.049	26.6
1.25	1.380	35.1
1.5	1.610	40.9
2	2.067	52.5
2.5	2.469	62.7
3	3.068	77.9
4	4.000	101.6
6	6.000	152.4
8	8.000	203.2
10	10.000	254.0

Required:

1. Design the manifold, using up to four different pipe diameters.
2. Use the Darcy-Weisbach and Blasius equations for friction loss.
3. Do not use any pipe diameter which is less than ½ the largest diameter.
4. Determine appropriate manifold pipe sizes and lengths.
5. Use either the semi-graphical method, or develop a computer program.

Solution:

- Assume a water temperature of 10°C, giving a kinematic viscosity of:

$$\nu = 1.306(10)^{-6}\ m^2/s.$$

- Then, the Reynolds number is:

$$N_R = \frac{4Q}{\nu\pi D} = 9.75(10)^5\left(\frac{Q}{D}\right)$$

- Plugging the above into the Blasius equation:

$$f = 0.0102\left(\frac{Q}{D}\right)^{-0.25}$$

- Darcy-Weisbach:

$$h_f = 0.169\left(\frac{Q^{1.75}}{D^{4.75}}\right)$$

for L= 200 m; h_f in m; Q in m^3/s; and D in m.

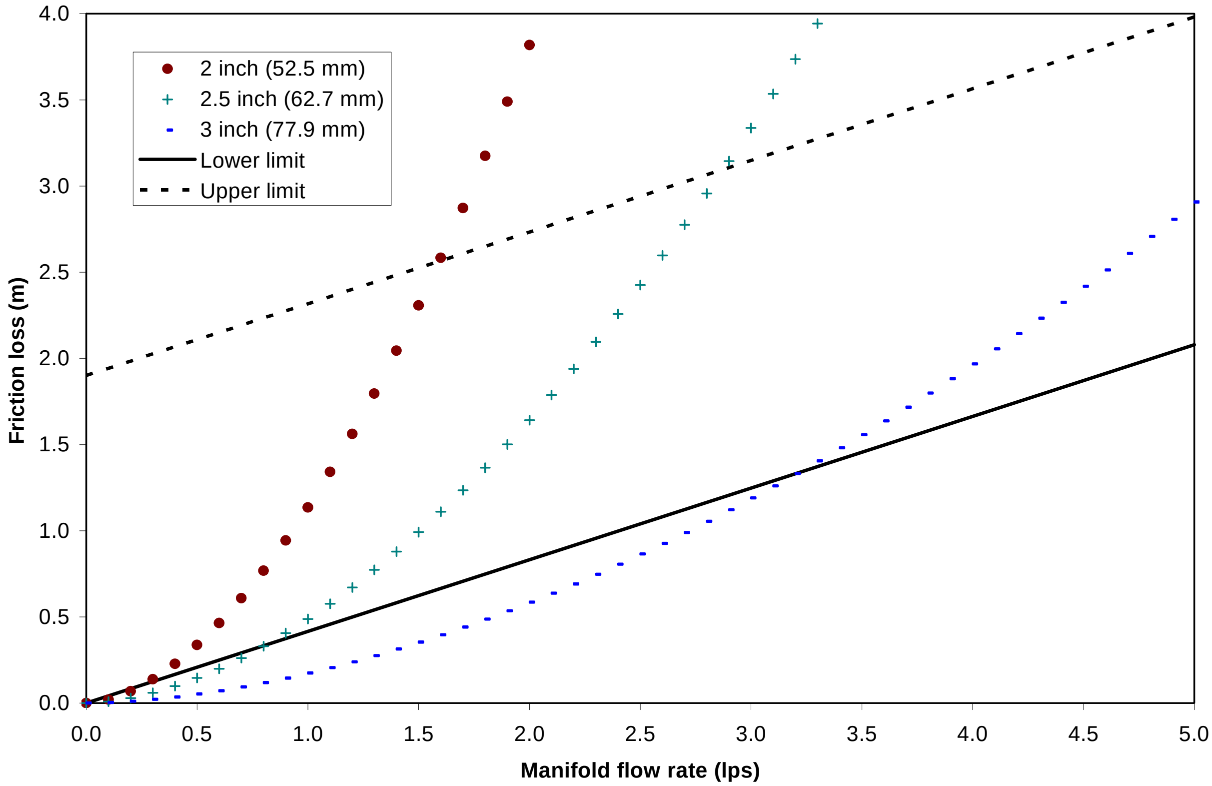
- Elevation change over 200-m manifold length:

$$\Delta h_e = (200)(0.0104) = 2.08\ m$$

- Allowable manifold pressure head variation:

$$\left(\Delta h_m\right)_a = \Delta H_s - \Delta h_l = 4.2 - 2.3 = 1.9\ m$$

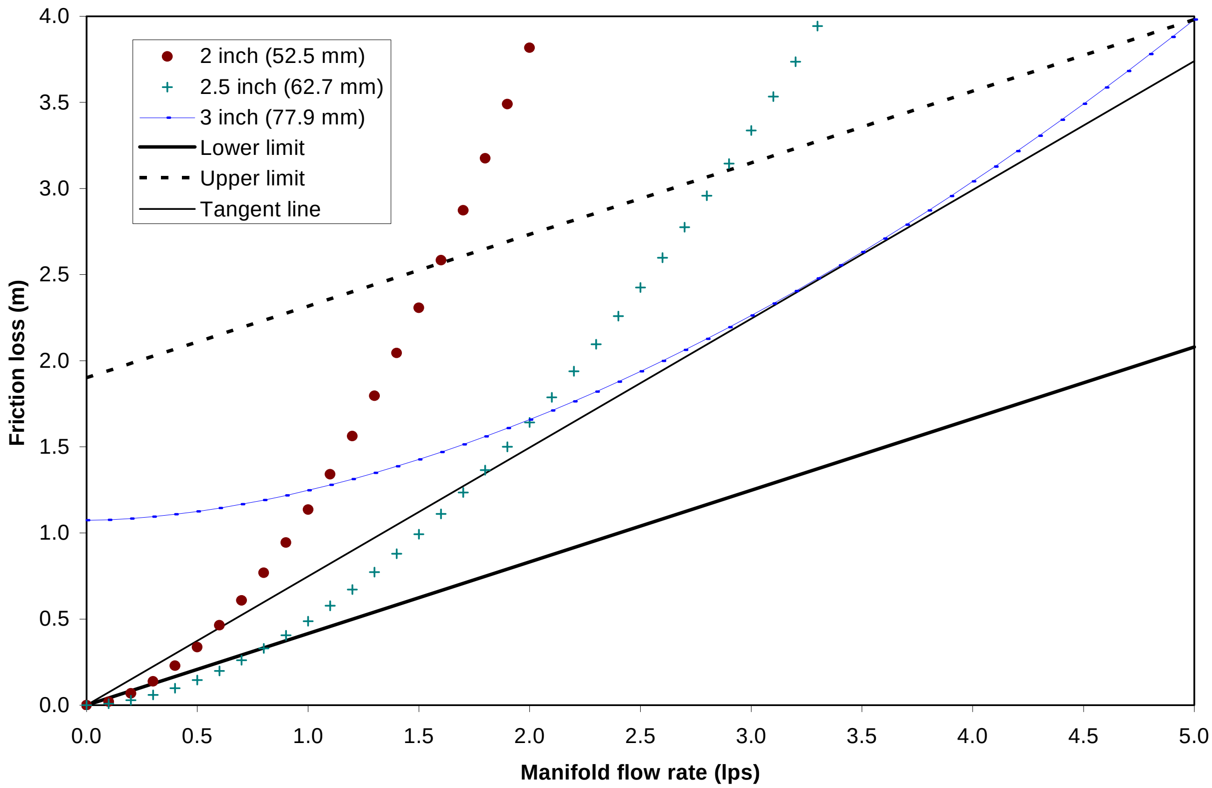
- In a spreadsheet, make a graph of h_f versus Q (0 to 5 lps) with a separate curve for each of the available pipe sizes (diameters).
- It is seen that the smallest five pipe sizes (1.5 inches and lower) have curves which are obviously too steep for the range $0 < h_f < (2.08 + 1.9)$, so these are omitted from the graph.
- Plot straight lines which define the limits of the allowable manifold pressure head variation. The lower line begins at the origin (0,0) and goes to (5,2.08). The upper line goes from (0,1.9) to (5,2.08+1.9) = (5,3.98).
- Now it is seen that the largest four pipe sizes have head loss curves which are too flat. These are eliminated from the graph. The graph has three pipe diameters:



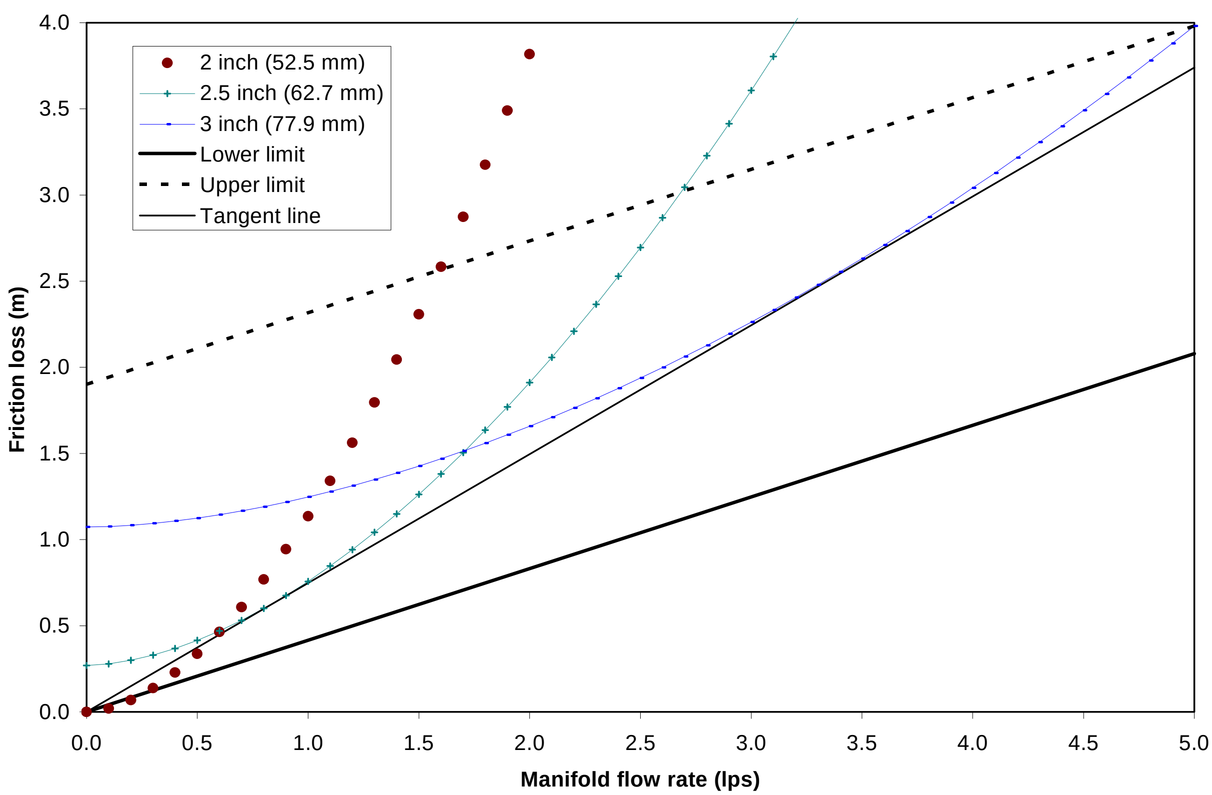
- Make the head loss curve for the largest of the three diameters (3 inch) pass through the point (5,3.98). Do this by observing that at 5 lps, the head loss from the 3-inch curve is 2.907 m. Add 3.98 – 2.907 = 1.073 m to all points on the curve for the 3-inch pipe size, thereby shifting it vertically as needed, then draw a straight line through the origin, tangent to this shifted curve (see the figure below).
- Next, vertically shift the curve for the 2.5-inch pipe so that it is tangent to the tangent line, as shown in the second figure on the next page.
- Draw a vertical line at the intersection between these two curves, defining the length of the 3-inch pipe size. The break point is at 1.71 lps. This gives the following length for the 3-inch pipe size (assuming a linear change in flow rate along the manifold):

$$L_{3''} = 200\left(\frac{1.71 - 5.00}{-5.00}\right) \approx 132 \text{ m}$$

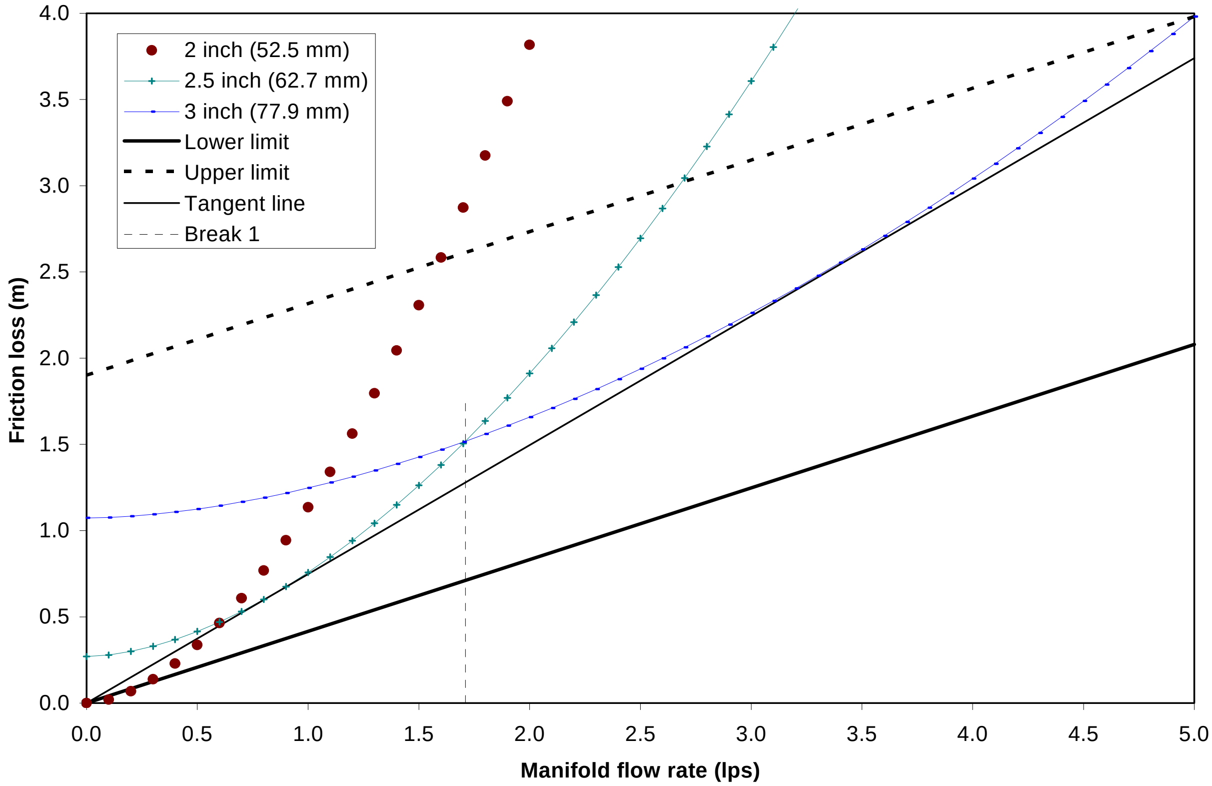
- Finally, make the 2-inch curve tangent to the tangent line (by vertical shifting), then draw a vertical line at the intersection of this curve with the 2.5-inch curve. See the final graph below.



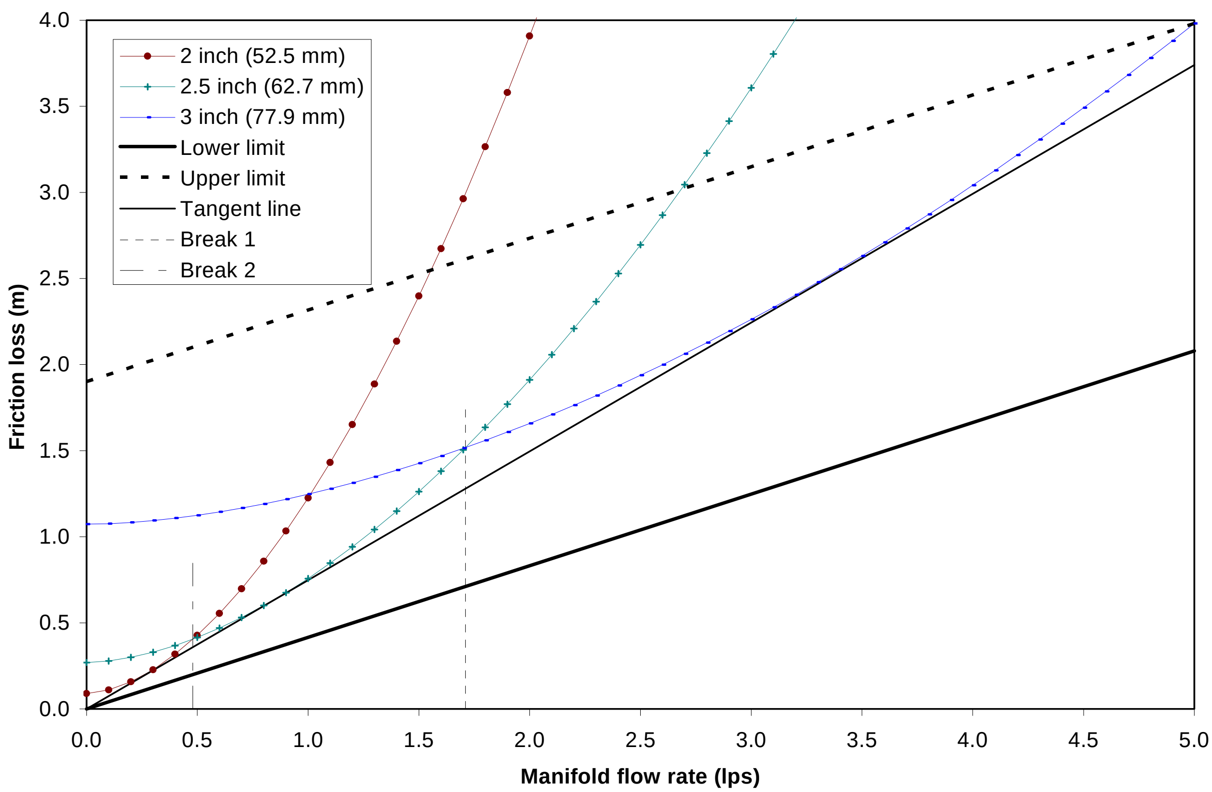
2 inch (52.5 mm)
2.5 inch (62.7 mm)
3 inch (77.9 mm)
Lower limit
Upper limit
Tangent line
Friction loss (m)
Manifold flow rate (lps)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0



2 inch (52.5 mm)
2.5 inch (62.7 mm)
3 inch (77.9 mm)
Lower limit
Upper limit
Tangent line
Friction loss (m)
Manifold flow rate (lps)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0



2 inch (52.5 mm)
2.5 inch (62.7 mm)
3 inch (77.9 mm)
Lower limit
Upper limit
Tangent line
Break 1
Friction loss (m)
Manifold flow rate (lps)



2 inch (52.5 mm)
2.5 inch (62.7 mm)
3 inch (77.9 mm)
Lower limit
Upper limit
Tangent line
Break 1
Break 2
Friction loss (m)
Manifold flow rate (lps)

- The break point between the 2.5 and 2.0 curves is at 0.48 lps. This gives the following length for the 2.5-inch pipe size:

$$L_{2"} = 200\left(\frac{0.48}{5.00}\right) \approx 19 \text{ m}$$

- Then, the 2.5-inch pipe size would have a length of 200 – 19 – 132 = 49 m.
- In summary:

Size (inches)	**Length (m)**	**V_{max} (m/s)**
3	132	1.05
2.5	49	0.55
2	19	0.22
Total:	**200**	

- Note that the design would also be acceptable (within allowable limits) if only two sizes were used: 3-inch and 2.5-inch. This would also simplify installation and would not increase the cost significantly.
- Note also that the maximum average velocity in each of the pipe sizes is well within acceptable limits.
- Finally, observe that the smallest pipe diameter (2 inches) is greater than ½ the largest pipe diameter (3 inches), as desired for the manifold design.
- Note that if a different kinematic viscosity is applied, a different pipe sizing solution might be obtained.

BIE 5300/6300 Assignment #4
Broad-Crested Weir Calibrations

28 Sep 04 (due 05 Oct 04)

Show your calculations in an organized and neat format. Indicate any assumptions or relevant comments. You can use ***ACA*** *or* ***WinFlume*** *if you like, or you can do calculations in a spreadsheet or other program.*

I. You have to design a BCW for a concrete-lined trapezoidal canal with a bottom width of 2.0 m, inverse sides slopes of 1.25:1 (H:V), lining depth of 2.7 m, and maximum discharge of 12 m^3/s. The Manning roughness is estimated to be 0.013, and the longitudinal bed slope is 0.000123 m/m. Make sure the BCW will operate under free-flow conditions up to Q_{max} = 12 m^3/s.

(a) Give the design dimensions for the BCW, and provide any relevant comments about the design and your assumptions.
(b) What is the minimum flow rate which can be accurately measured with your BCW design?
(c) Will the upstream canal walls need to be raised after installing the BCW when operating at Q_{max} = 12 m^3/s?
(d) Would you recommend including a DS ramp on the BCW? Why or why not?
(e) Suppose you include two 1-inch diameter PVC drainage pipes at the base of the BCW. What is the estimated discharge through the pipes at the 12 m^3/s capacity of the BCW? Is this a significant fraction of Q_{max}?

II. Use a spreadsheet or your own custom computer program to check the calibration of your BCW design from the problem above, but based only on energy-balance from upstream to the location of critical flow on the sill. In this case, you will assume free-flow conditions at the BCW.

(a) Do the comparison for the full flow range of the BCW.
(b) Make a graph (two curves) of h_u versus Q for the full calibration (from the problem above) and for the simpler energy-balance calibration.
(c) Are the two calibrations significantly different in this case?

III. Suppose the Parshall flume at the location (Logan Canyon) of our field exercises is getting badly deteriorated and needs to be replaced. A decision is made to install a BCW instead of the Parshall flume, at approximately the same location. Based on your lab data, and a Q_{max} of 40 cfs, what BCW design dimensions and features would you propose?