
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

This material has been duplicated by special permission of the copyright holders. It is not to be duplicated or used for purposes other than learning resource support for Utah State University. Any violation of this agreement is punishable under existing copyright laws and may include severe fines and or imprisonment.

© Copyright 1990 to 2004

These lecture notes are formatted 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

No.	Lecture Title	Page
Sprinkle Irrigation		
1	Course Introduction.....	9
2	Types of Sprinkler Systems; Soil-Water-Plant Relationships; Planning Factors	11
3	Sprinkler Characteristics; Application Rates	25
4	Set Sprinkler Uniformity & Efficiency.....	39
5	Layout of Laterals & Mainline for Set Sprinklers; Pipe Hydraulics.....	51
6	Economic Pipe Selection Method.....	59
7	Set Sprinkler Lateral Design	71
8	Set Sprinkler Lateral Design & Analysis.....	85
9	Mainline Pipe Design	101
10	Minor Losses, Pressure Requirements & Pumps.....	111
11	Pumps & System Curves; Affinity Laws & Cavitation.....	123
12	Center Pivot Design & Operation	145
13	Center Pivot Nozzling & Hydraulic Analysis.....	155
14	Center Pivot Uniformity Evaluation; Linear Move Systems	169
15	Maximizing Linear Move Field Length; Design Example	179
Trickle Irrigation		
16	Components & Layout; Pressure Control & Flow Regulation.....	189
17	Filtration for Trickle Irrigation Systems.....	197
18	Trickle Irrigation Planning Factors; Salinity in Trickle Irrigation	207
19	Water Requirements; Coefficient of Variation & System Capacity	215
20	Emitter Selection & Design; Design Approach & Example.....	225
21	Pipe Specifications & Lateral Design; Manifold Location	231
22	Numerical Solution for Manifold Location; Derivations.....	241
23	Manifold Hydraulic Design	253
24	Hydraulic Design of Mainline & Supply Line.....	275

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/ft³
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/ft³, or 1000 kg/m³ (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/cm²
0.0419 MJ/m² per cal/cm²

0.3048 m/ft
1.609 km/mile
2.471 acre/ha
43,560 ft²/acre
1,233 m³/acre-ft

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

$^{\circ}\text{C} = (^{\circ}\text{F} - 32)/1.8$
 $^{\circ}\text{F} = 1.8(^{\circ}\text{C}) + 32$

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

PVC = Polyvinyl chloride
PE = Polyethylene
ABS = Acrylonitrile-Butadiene-Styrene

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 your own work
- 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

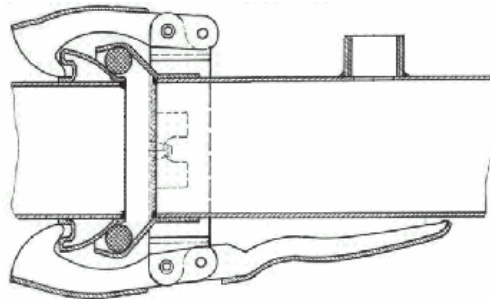
I. Sprinkler System Categories

- 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 valves (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 than 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

- *Low Energy Precision Application* (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

$$d_x = \frac{\text{MAD}}{100} W_a Z \quad (1)$$

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:

$$f_x = \frac{d_x}{U_d} \quad (2)$$

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

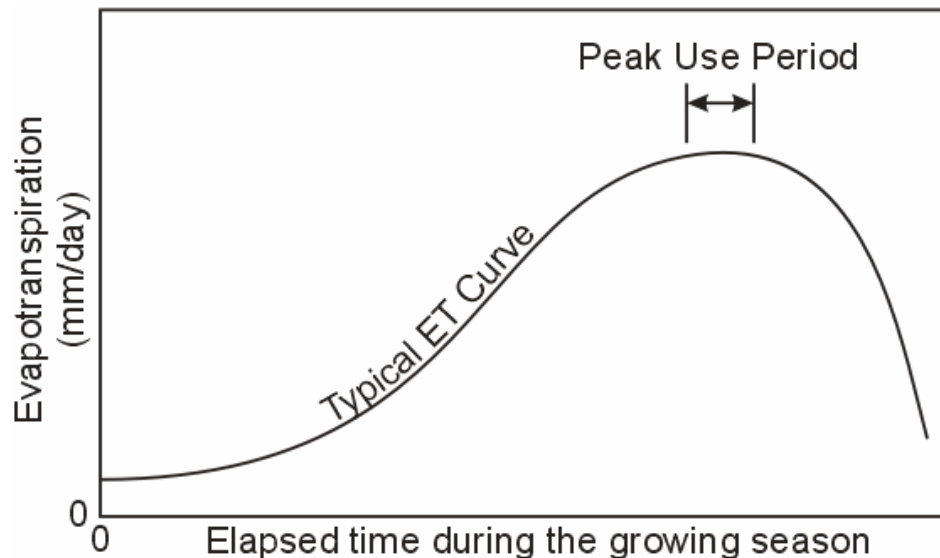
- The range of f_x values for agricultural crops is usually:

$$0.25 < f_x < 80 \text{ days} \quad (3)$$

- 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, $d_n \leq d_x$, 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:

$$LR = \frac{EC_w}{5EC_e - EC_w} \quad (4)$$

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:

$$LR \leq 0.1: \quad d = \frac{d_n}{E_a} \quad (5)$$

$$LR > 0.1 \quad d = \frac{0.9d_n}{(1-LR)E_a} \quad (6)$$

- 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,

$$LR = \frac{EC_w}{5EC_e - EC_w} = \frac{2.1}{5(2.5) - 2.1} = 0.20 \quad (7)$$

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:

$$d = d_n + (LR)d = \frac{d_n}{1 - LR} \quad (8)$$

and,

$$d = \frac{d_n}{1 - 0.20} = 1.25d_n \quad (9)$$

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...

$$W_a = \frac{1.0(167) + (1.5 - 1.0)(183)}{1.5} = 172.3 \text{ mm/m} \quad (10)$$

2. Max net application depth (Eq. 3.1):

$$d_x = \frac{MAD}{100} W_a Z = \left(\frac{50}{100} \right) (172.3)(1.5) = 129.2 \text{ mm} \quad (11)$$

3. Maximum irrigation interval (Eq. 3.2):

$$f_x = \frac{d_x}{U_d} = \frac{129.2 \text{ mm}}{7.6 \text{ mm/day}} = 17.0 \text{ days} \quad (12)$$

4. Nominal irrigation interval (round down, or truncate):

$$f' = \text{trunc}(f_x) = 17 \text{ days} \quad (13)$$

5. Net application depth:

$$d_n = f' U_d = (17 \text{ days})(7.6 \text{ mm/day}) = 129.2 \text{ mm} \quad (14)$$

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):

$$LR = \frac{EC_w}{5EC_e - EC_w} = \frac{2.0}{5(3.4) - 2.0} = 0.13 \quad (15)$$

$LR > 0.1$; therefore, use Eq. 5.3 b...

8. Gross application depth (Eq. 5.3b):

$$d = \frac{0.9d_n}{(1-LR)(E_a/100)} = \frac{0.9(129.2)}{(1-0.13)(0.8)} = 167.1 \text{ mm} \quad (16)$$

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:

$$(14 \text{ days/irrigation})(2 \text{ sets/day}) = 28 \text{ sets}$$

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:

$$(15 \text{ m/set})(28 \text{ sets})(400 \text{ m/lateral}) = 16.8 \text{ ha/lateral}$$

13. Number of laterals needed:

$$\frac{35 \text{ ha}}{16.8 \text{ ha/lateral}} = 2.08 \text{ laterals} \quad (17)$$

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:

$$\frac{U - P_e}{d_n} = \frac{914 \text{ mm} - 190 \text{ mm}}{129.2 \text{ mm/irrig}} = 5.6 \text{ irrigations} \quad (18)$$

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...

$$Q_s = 2.78 \frac{Ad}{fT} = 2.78 \frac{(35 \text{ ha})(167.1 \text{ mm})}{(14 \text{ days})(23 \text{ hrs/day})} = 50.5 \text{ lps (800 gpm)} \quad (19)$$

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)

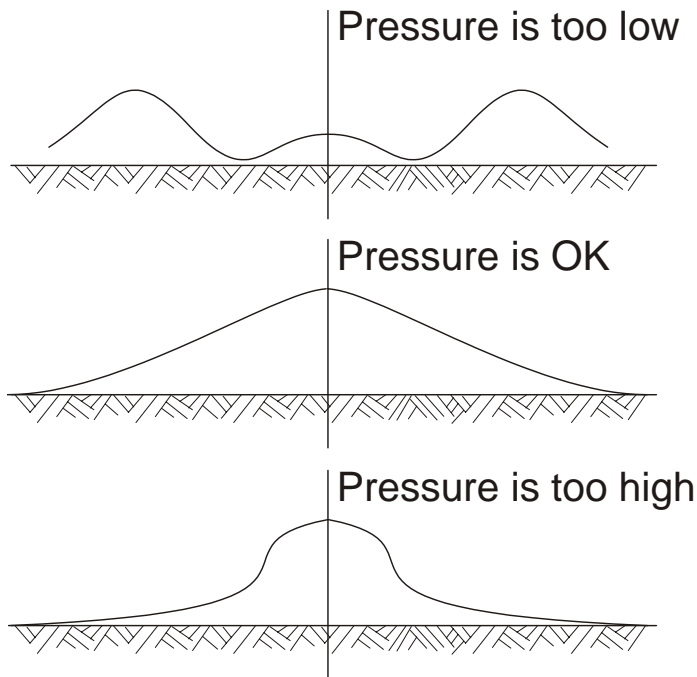
- Agricultural sprinklers typically have flow rates from 4 to 45 lpm (1 to 12 gpm), at nozzle pressures of 135 to 700 kPa (20 to 100 psi)
- “Gun” sprinklers may have flow rates up to 2,000 lpm (500 gpm; 33 lps) or more, at pressures up to 750 kPa (110 psi) or more
- Sprinklers with higher manufacturer *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

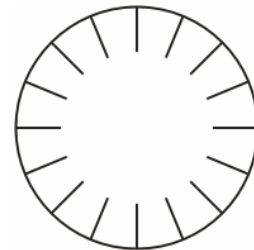
III. Precipitation Profiles

- Typical examples of low, correct, and high sprinkler pressures (see Fig 5.5).

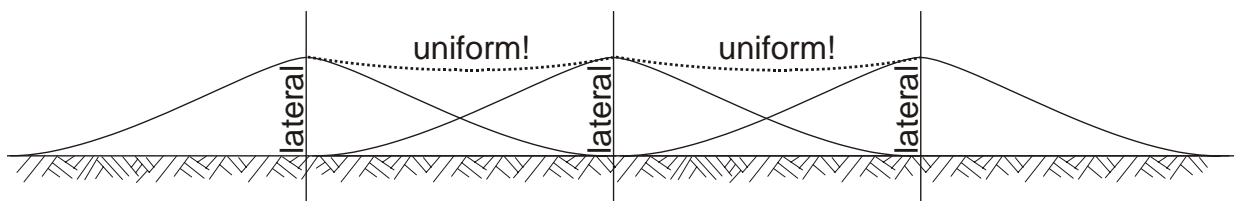


- The precipitation profile (and uniformity) is a function of many factors:

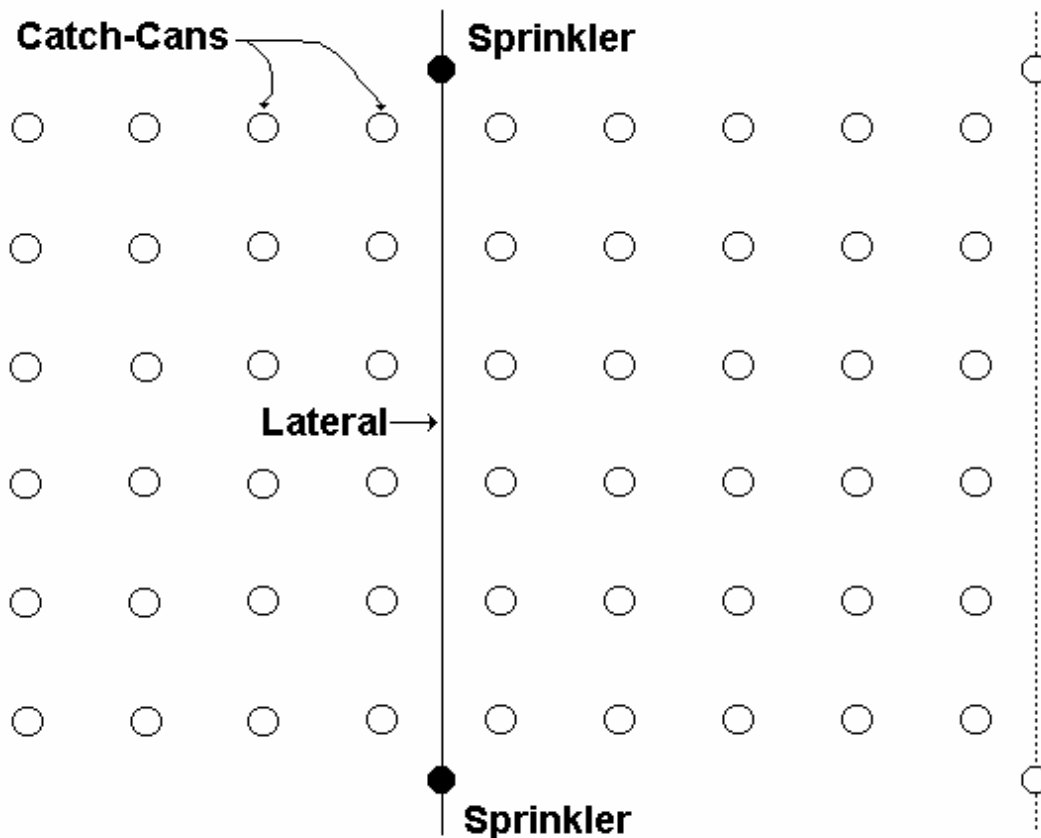
1. nozzle pressure
2. nozzle shape & size
3. sprinkler head design
4. presence of straightening vanes
5. sprinkler rotation speed
6. trajectory angle
7. riser height
8. wind



- 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
- the objective 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 ($\frac{1}{2}S_l$) 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 increase uniformity, as the randomness of wind turbulence and gusts helps to smooth out the precipitation profile

Wind effects on the diameter of throw:

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):

$$\text{diam} = 0.9 \text{diam}_{\text{manuf}} \quad (20)$$

For > 3 mph (> 1.34 m/s):

$$\text{diam} = \text{diam}_{\text{manuf}} \left[0.9 - 0.025(\text{wind}_{\text{mph}} - 3) \right] \quad (21)$$

or,

$$\text{diam} = \text{diam}_{\text{manuf}} \left[0.9 - 0.056(\text{wind}_{\text{m/s}} - 1.34) \right] \quad (22)$$

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?

$$80 \text{ ft} - (0.10)(0.80) = 72 \text{ ft} \quad (23)$$

$$72 \text{ ft} - (5 \text{ mph} - 3 \text{ mph})(0.025)(72 \text{ ft}) = 68 \text{ ft} \quad (24)$$

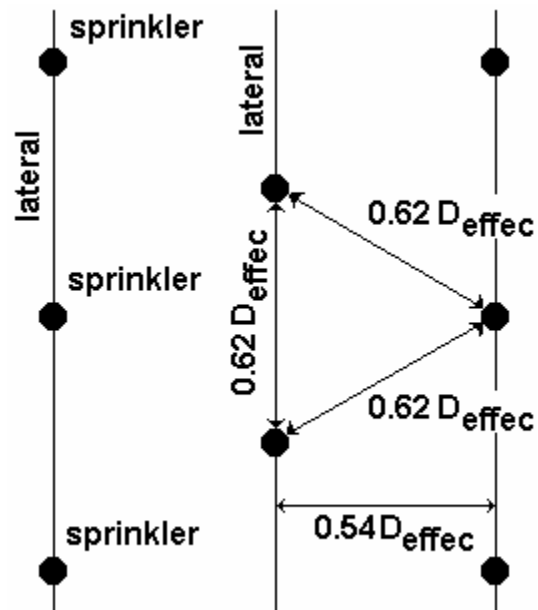
or,

$$\text{diam} = 80(0.9 - 0.025(5 - 3)) = 68 \text{ ft} \quad (25)$$

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 $0.62 \cos(60^\circ/2) = 0.54$, or 54% of the effective diameter, D_{effec}]



- See Fig. 5.8 about profiles and spacings

VIII. Pressure-Discharge Relationship

- Equation 5.1:

$$q = K_d \sqrt{P} \quad (26)$$

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:

$$\frac{P}{\gamma} = \frac{V^2}{2g} = \frac{q^2}{2gA^2} \quad (27)$$

$$\sqrt{\frac{2gA^2P}{\rho g}} = K_d \sqrt{P} = q \quad (28)$$

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 :

$$q = K_o A \sqrt{P} \quad (29)$$

whereby,

$$K_o = \sqrt{2/\rho} \quad (30)$$

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 (64ths 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

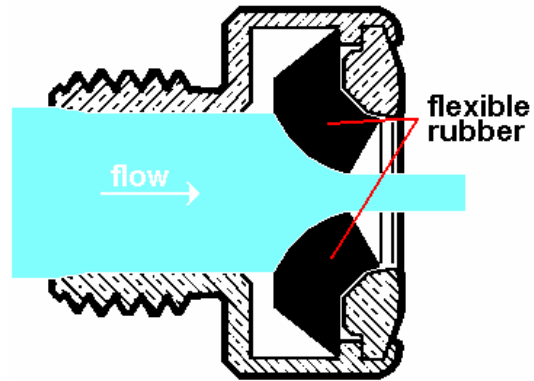
Straight Bore Nozzle (SBN-4) with Plug (Stream Height: 8 ft.)

PSI @ Nozzle	NOZZLE SIZE													
	7/32"		1/4"		9/32"		15/16"		11/32"		3/8"		13/32"	
	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

- 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 $\pm 5\%$ of q ; don't use FCNs unless pressure variation is greater than about 10% (along lateral and for different lateral positions)



Flow Control Nozzle

$$\sqrt{1.10P} \approx 1.05\sqrt{P} \quad (32)$$

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

$$d = \frac{d_n}{E_{pa}}, \quad \text{for } LR \leq 0.1 \quad (33)$$

where E_{pa} is the “designer” application efficiency (decimal; Eq. 6.9). And,

$$d = \frac{0.9d_n}{(1-LR)E_{pa}}, \quad \text{for } LR > 0.1 \quad (34)$$

- 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 ($EC_w \leq \frac{1}{2}EC_e$), 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):

$$Q_s = K \frac{Ad}{fT} \quad (35)$$

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 ($d_{\text{applied}} < d$): 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):

$$I = \frac{3600qR_e}{S_e S_l} \quad (36)$$

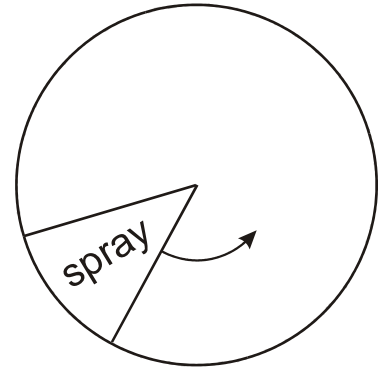
where I is the application rate (mm/hr); q is the flow rate (lps); S_e is the sprinkler spacing (m); S_l is the lateral spacing (m); and R_e is the fraction of water emitted by the nozzle that reaches the soil (takes into account the evaporative/wind loss)

- R_e is defined in Chapter 6 of the textbook
- The instantaneous application rate for a rotating sprinkler (after Eq. 5.6):

$$I_i = \frac{3600 q R_e}{\pi R_j^2 \left(\frac{S_a}{360} \right)} \quad (37)$$

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:



$$\frac{(1.5 \text{ m/s})(60 \text{ s/min})}{(1 \text{ rev/min})(2\pi \text{ rad/rev})} = 14.3 \text{ m (radius)} \quad (38)$$

- 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

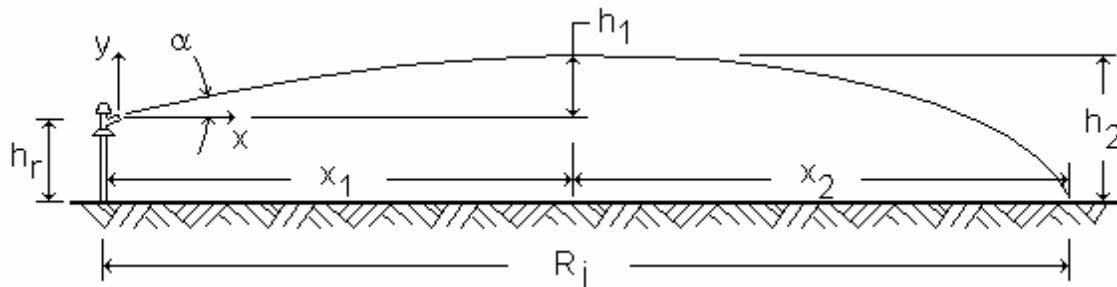
VI. Intake & Optimum Application Rates

- 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,

$$(V_y)_{t_1} = V_0 \sin \alpha - gt_1 = 0 \quad (39)$$

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 $V_0 \sin \alpha$ 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:

$$t_1 = \frac{V_0 \sin \alpha}{g} \quad (40)$$

then,

$$h_1 = V_0 \sin \alpha t_1 - \frac{gt_1^2}{2} = \frac{V_0^2 \sin^2 \alpha}{2g} \quad (41)$$

- Assuming no acceleration in the horizontal direction,

$$x_1 = V_0 \cos \alpha t_1 \quad (42)$$

solving for h_2 ,

$$h_2 = h_r + h_1 = V_y t_2 + \frac{gt_2^2}{2} \quad (43)$$

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:

$$x_2 = V_0 \cos \alpha t_2 = V_0 \cos \alpha \sqrt{\frac{2(h_r + h_1)}{g}} \quad (44)$$

And, the approximate wetted radius of the sprinkler is:

$$R_j = x_1 + x_2 \quad (45)$$

- 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

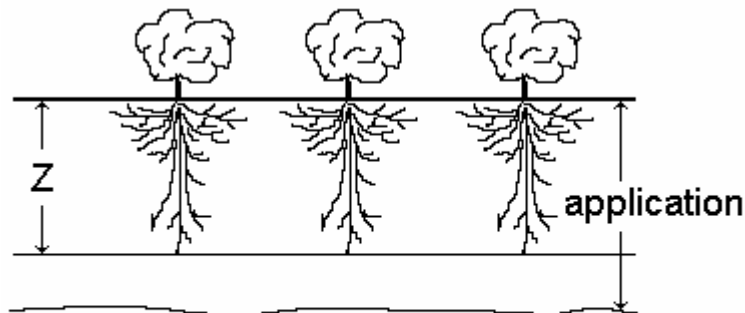
- And, q_a is a function of P

Lecture 4

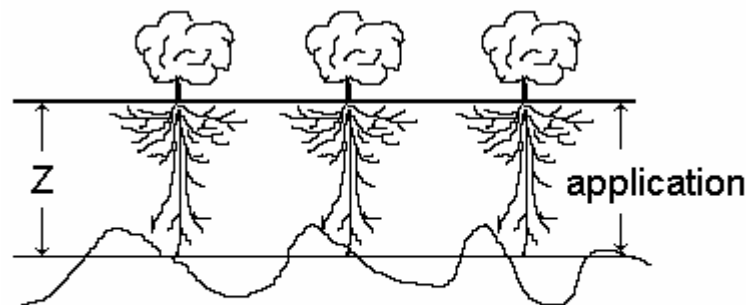
Set Sprinkler Uniformity & Efficiency

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):

$$DU = 100 \left(\frac{\text{avg depth of low quarter}}{\text{avg depth}} \right) \quad (46)$$

- 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 $\frac{1}{4}$ 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):

$$CU = 100 \left(1.0 - \frac{\sum_{j=1}^n \text{abs}(z_j - m)}{\sum_{j=1}^n z_j} \right) \quad (47)$$

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,

$$CU \approx 100 \left(\frac{\text{avg depth of low half}}{\text{avg depth}} \right) \quad (48)$$

another way to define CU is through the standard deviation of the values,

$$CU = 100 \left(1.0 - \frac{\sigma}{m} \sqrt{\frac{2}{\pi}} \right) \quad (49)$$

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:

$$\sum |z - m| = n\sigma\sqrt{2/\pi} \quad (50)$$

- 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:

$$CU \approx 100 - 0.63(100 - DU) \quad (51)$$

or,

$$DU \approx 100 - 1.59(100 - CU) \quad (52)$$

- 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 ($\frac{1}{2}S_i$)
- This is usually practiced by placing laterals halfway between the positions from the previous irrigation, alternating each time
- The relationship is:

$$\begin{aligned} CU_a &\approx 10\sqrt{CU} \\ DU_a &\approx 10\sqrt{DU} \end{aligned} \quad (53)$$

- The above are also valid for “double” alternate sets ($S_i/3$)
- Use of alternate sets is a good management practice for periodic-move systems
- The use of alternate sets approaches an S_i 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.

$$\text{system CU} \approx \text{CU} \left[\frac{1}{2} \left(1 + \sqrt{P_n / P_a} \right) \right] \quad (54)$$

$$\text{system DU} \approx \text{DU} \left[\frac{1}{4} \left(1 + 3\sqrt{P_n / P_a} \right) \right] \quad (55)$$

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.

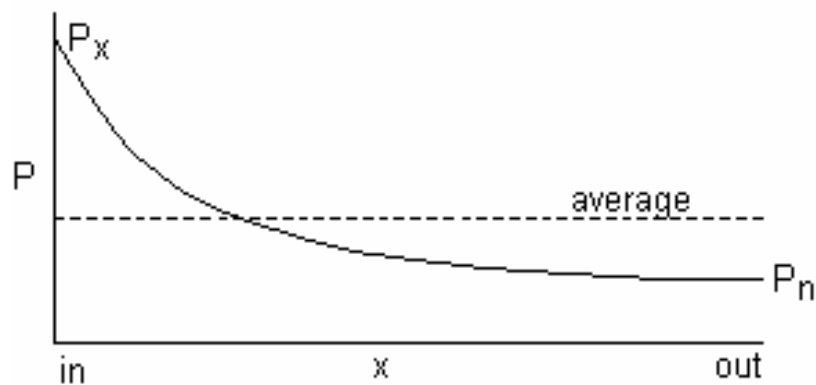
- These equations can be used in design and evaluation
- Note that when $P_n = P_a$ (no pressure variation) the system CU equals the CU
- If pressure regulators are used at each sprinkler, the system CU is approximately equal to 0.95CU (same for DU)

- 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 :

$$P_a = \frac{2P_n + P_x}{3} \quad (56)$$

where P_x is the maximum nozzle pressure in the system

Due to parabolic head loss vs. flow rate relation, the average is closer to P_n



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

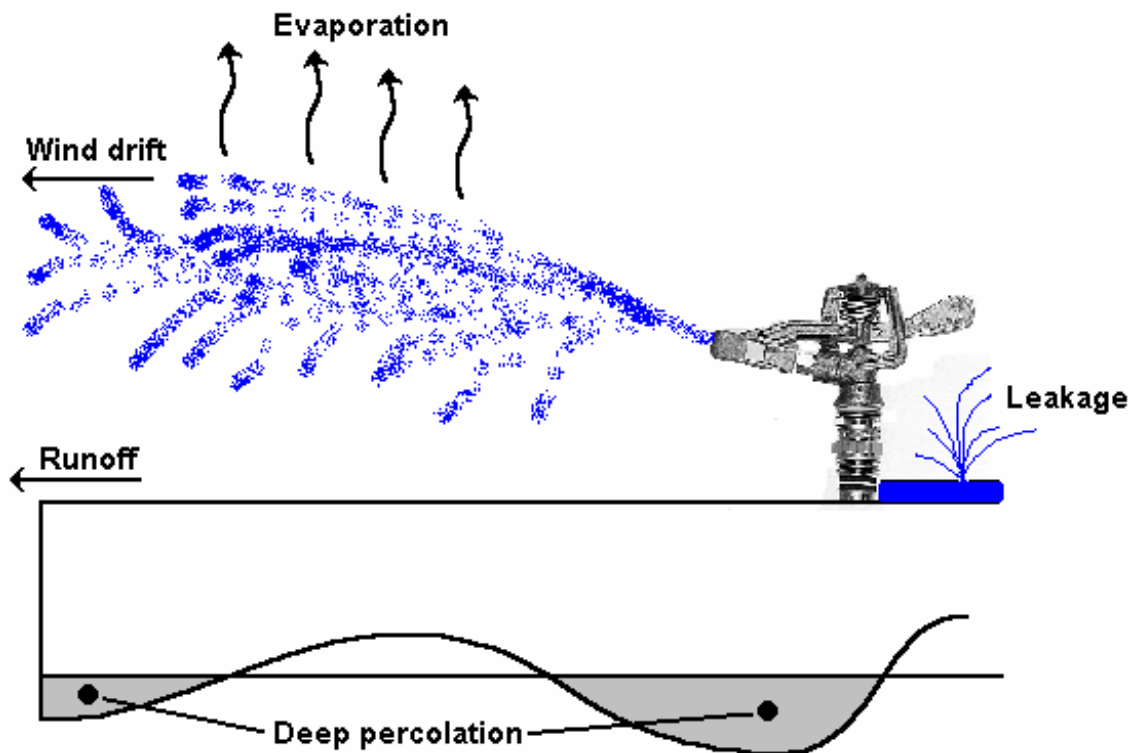
The following material leads up to the development of a general sprinkle application efficiency term (Eq. 6.9) as follows:

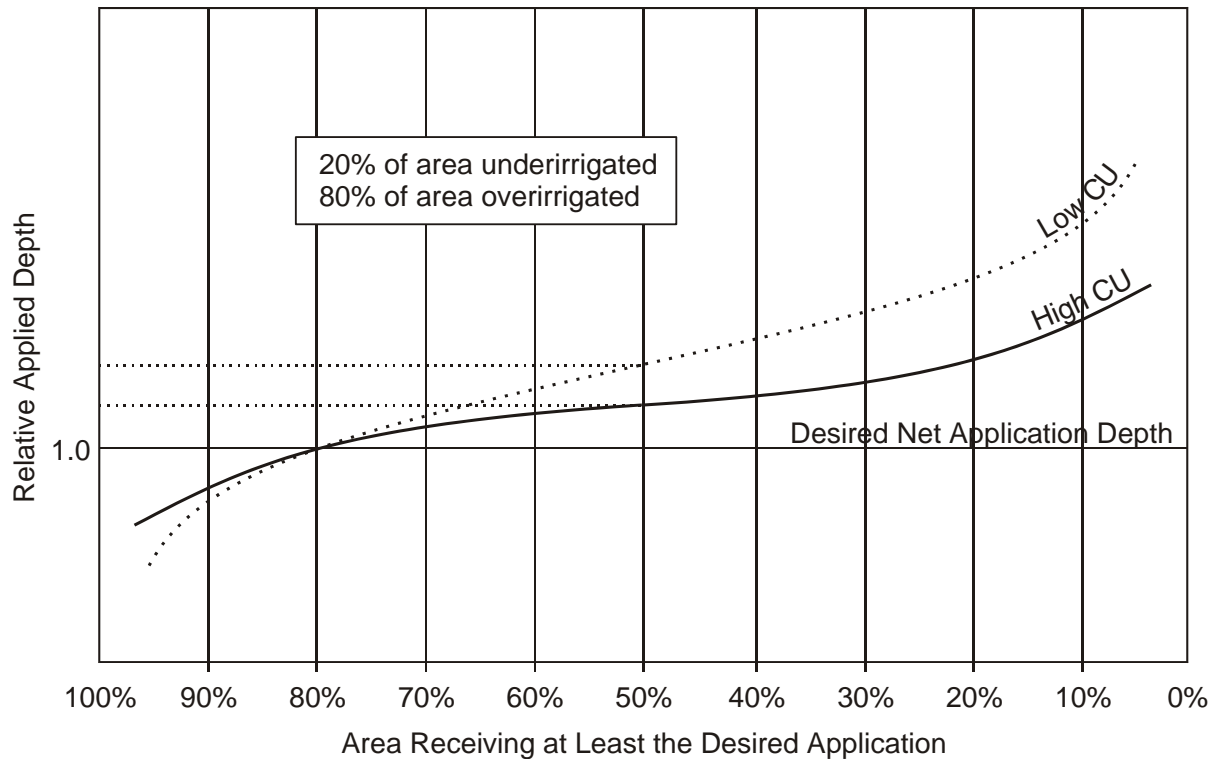
Design Efficiency:

$$E_{pa} = DE_{pa}R_eO_e \quad (57)$$

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):

$$CI = 0.032 \left(\frac{P^{1.3}}{B} \right) \quad (58)$$

where P is the nozzle pressure (kPa) and B is the nozzle diameter (mm)

$CI > 17$	$17 \geq CI \geq 7$	$CI < 7$
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 $(R_e)_{\text{fine}}$ and $(R_e)_{\text{coarse}}$ (Eq. 6.8):

$$R_e = \frac{(CI - 7)}{10} (R_e)_{\text{fine}} + \frac{(17 - CI)}{10} (R_e)_{\text{coarse}} \quad (59)$$

- Allen and Fisher (1988) developed a regression equation to fit the curves in Fig. 6.8:

$$R_e = 0.976 + 0.005ET_o - 0.00017ET_o^2 + 0.0012W \\ - 0.00043(CI)(ET_o) - 0.00018(CI)(W) \\ - 0.000016(CI)(ET_o)(W) \quad (60)$$

where ET_o is the reference ET in mm/day (grass-based); CI is the coarseness index ($7 \leq CI \leq 17$); 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:

$$E_{pa} = DE_{pa} R_e O_e \quad (61)$$

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 researchers 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

$$q_a = \frac{I(S_e S_l)}{3600} = \frac{d_n S_e S_l O_e}{3600 E_{pa} S_{to}} \quad (62)$$

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:

$$N_n = \frac{Q_s}{q_a} \quad (63)$$

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

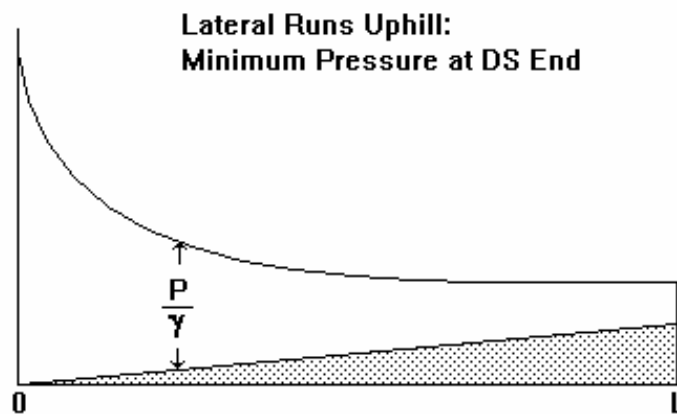
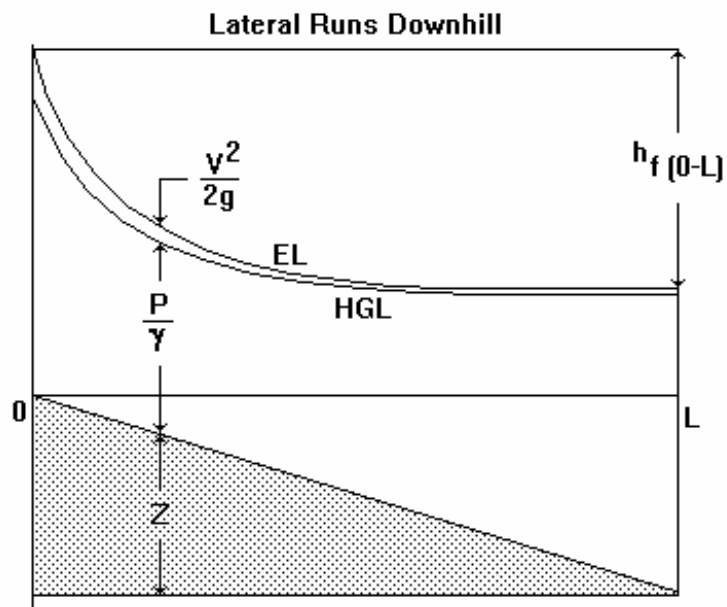
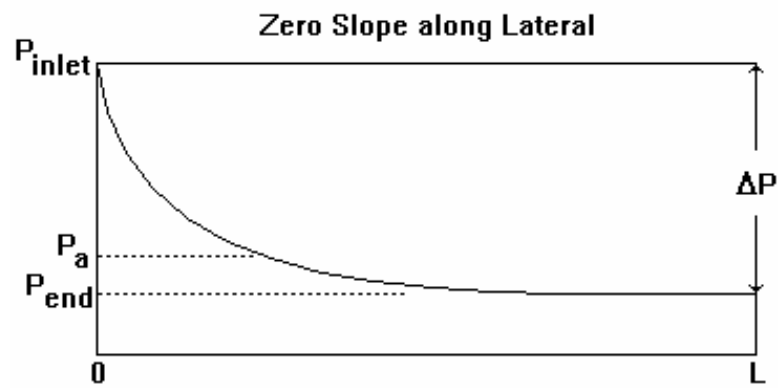
- Lateral pressure varies from inlet to extreme end due to:
 1. friction loss
 2. elevation change
- The fundamental basis upon which sprinkler laterals are designed is:

“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.



- 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 any 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 ($\frac{1}{2}$ 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:

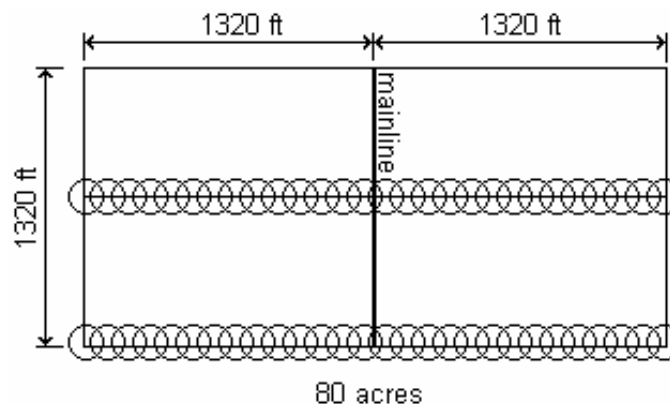
$$N_s = \frac{Q_s}{q_a} = \frac{532}{4.78} = 111 \text{ sprinklers} \quad (65)$$

- 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 4 laterals



- 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 $2 \times 27 = 54$ 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:

$$f = \frac{14 \text{ sets}}{2 \text{ sets/day}} = 7 \text{ days} \quad (69)$$

- 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:

$$Q_s = (4 \text{ laterals})(33 \text{ sprinklers/lateral})(4.78 \text{ gpm/sprinkler}) \\ = 631 \text{ gpm} \quad (70)$$

- Another way to adjust the system capacity:

$$Q_s = \left(\frac{8 \text{ days}}{7 \text{ days}} \right) (532 \text{ gpm}) = 608 \text{ gpm} \quad (71)$$

- 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 $2 \times 13 + 2 \times 14 = 54$ sets, but the last 2 sets have only 2 laterals. So, $(52/54) \times 631 = 608$ gpm, as calculated above.
- Which is correct?
- There are $(52/54) \times (4 \text{ laterals}) = 3.85$ 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:

$$f = \frac{0.25}{\left[\log_{10} \left(\frac{\varepsilon}{3.75D} + \frac{5.74}{N_R^{0.9}} \right) \right]^2} \quad (72)$$

which is valid for turbulent flow in the range: $4,000 \leq N_R \leq 1.0(10)^8$. The ratio ε/D is called “relative roughness.” The roughness height, ε , varies widely

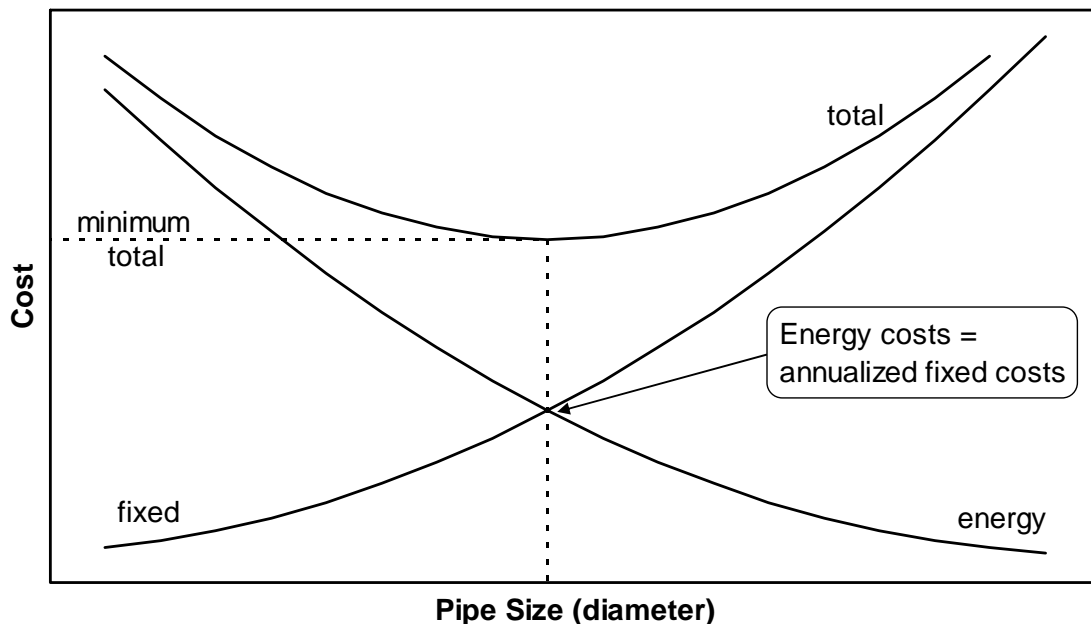
- We will also use the Blasius equation (Eq. 8.6) to determine the value of “f,” in some cases, for “smooth pipes”

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 $\frac{3}{4}$ 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

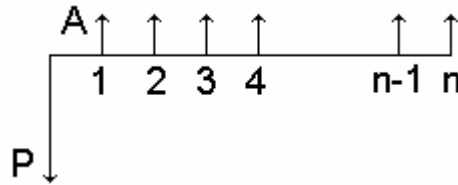
1. Select a period of time over which comparisons will be made between fixed and annual costs. This will be called the *useful life* of the system, n , in years.
 - The “useful life” is a subjective value, subject to opinion and financial amortization conditions
 - This value could alternatively be specified 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

$$UAC = P(\text{CRF}) \quad (73)$$

$$CRF = \frac{i(1+i)^n}{(1+i)^n - 1} \quad (74)$$

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 :

$$O_t = \frac{(\text{irrigated area})(\text{gross annual depth})}{(\text{system capacity})} = \text{hrs / year} \quad (75)$$

- 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}

$$E_p = E_{\text{pump}}E_{\text{motor}} \quad (76)$$

- This is equal to the ratio of “water horsepower”, WHP, to “brake horsepower”, BHP ($E_{\text{pump}} \equiv \text{WHP/BHP}$)

- 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:

$$WHP = \frac{QH}{102} \quad (77)$$

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 $1,000/g = 1,000/9.81 \approx 102$, 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:

$$E = \frac{O_t C_f}{E_p} \quad (78)$$

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 deliver 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:

$$MAC = \frac{CRF(\$_{big} - \$_{small})}{O_t(kW_{big} - kW_{small})} \quad (79)$$

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:

$$BHP = \frac{Q_s H_{pump}}{102 E_p} \quad (80)$$

- 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:

$$EAE = \left[\frac{(1+e)^n - (1+i)^n}{e-i} \right] \left[\frac{i}{(1+i)^n - 1} \right] \quad (81)$$

where e is the annual inflation rate (fraction), i is the annual interest rate (fraction), and n is in years

- Notice that for $e = 0$, $EAE = \text{unity}$ (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:

$$E' = (EAE)(E) \quad (82)$$

- 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:

$$E'(HP_{s1}) + UAC_{s1} = E'(HP_{s2}) + UAC_{s2} \quad (83)$$

or,

$$\Delta WHP_{s1-s2} = \frac{(UAC_{s2} - UAC_{s1})}{E'} \quad (84)$$

- 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:

$$\Delta J_{s1-s2} = 102 \left(\frac{\Delta WHP_{s1-s2}}{Q_s} \right) \quad (85)$$

- 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:

$$\Delta J = J_{s1} - J_{s2} = 16.42(10)^6 \left(\frac{q}{C} \right)^{1.852} \left(D_{s1}^{-4.87} - D_{s2}^{-4.87} \right) \quad (86)$$

where q is in lps, and D is the inside diameter of the pipe in cm

- Or, using the Darcy-Weisbach equation:

$$\Delta J = \frac{800 f q^2}{g \pi^2} \left(D_{s1}^{-5} - D_{s2}^{-5} \right) \quad (87)$$

- Solve for the flow rate, q (with q in lps; D in cm):

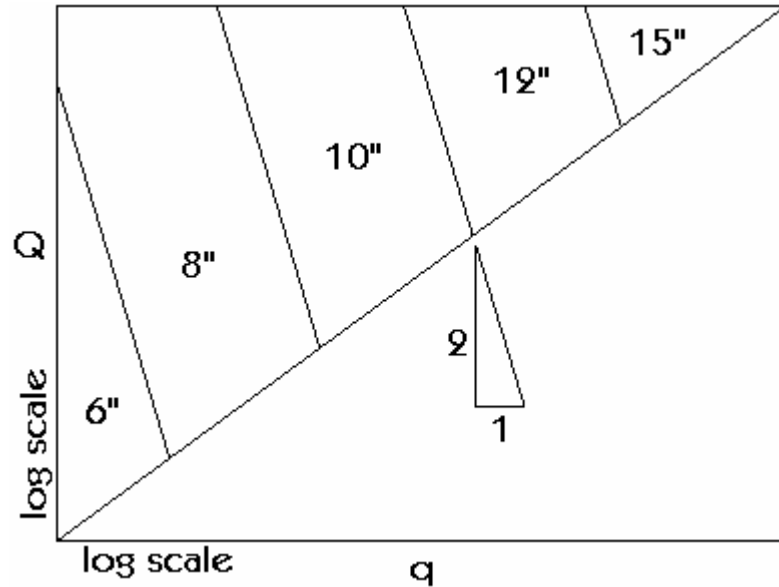
$$q = C \left[\frac{\Delta J}{16.42(10)^6 \left(D_{s1}^{-4.87} - D_{s2}^{-4.87} \right)} \right]^{0.54} \quad (88)$$

- 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



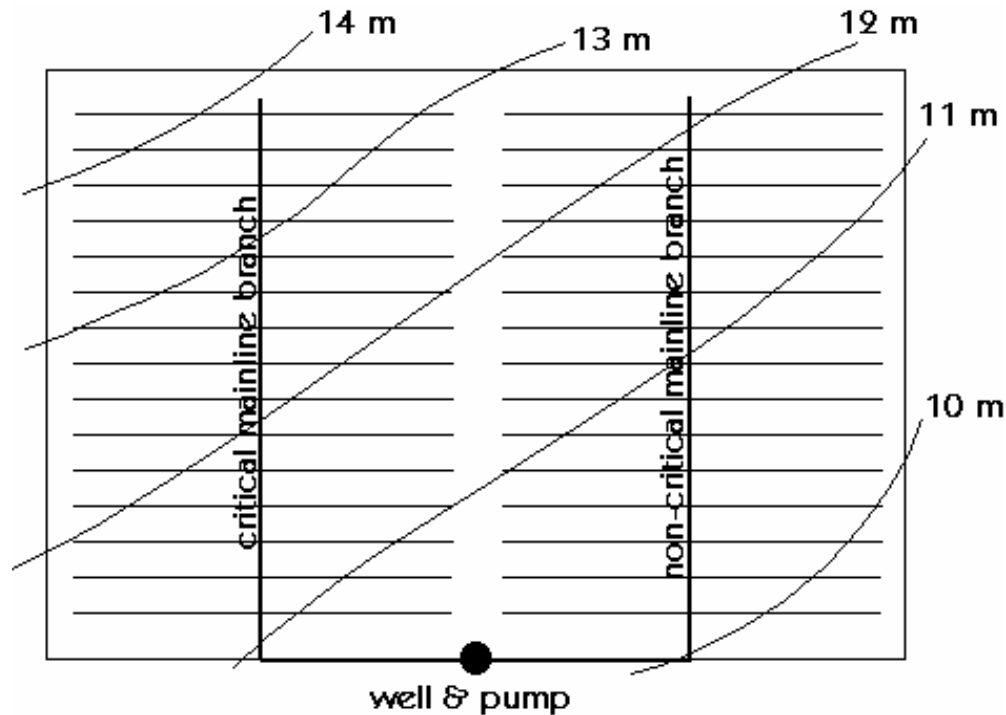
14. Applying the graph.

- 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

- 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).



- 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.
- 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).
- 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:

$$D = \sqrt{\frac{4Q}{\pi V}} \quad (89)$$

- The following tables show maximum flow rates for specified average velocity limits and different pipe inside diameters

Gallons per Minute			
D (inch)	A (ft2)	Velocity Limit	
		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			
D (ft)	A (ft2)	Velocity Limit	
		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			
D (mm)	A (m2)	Velocity Limit	
		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

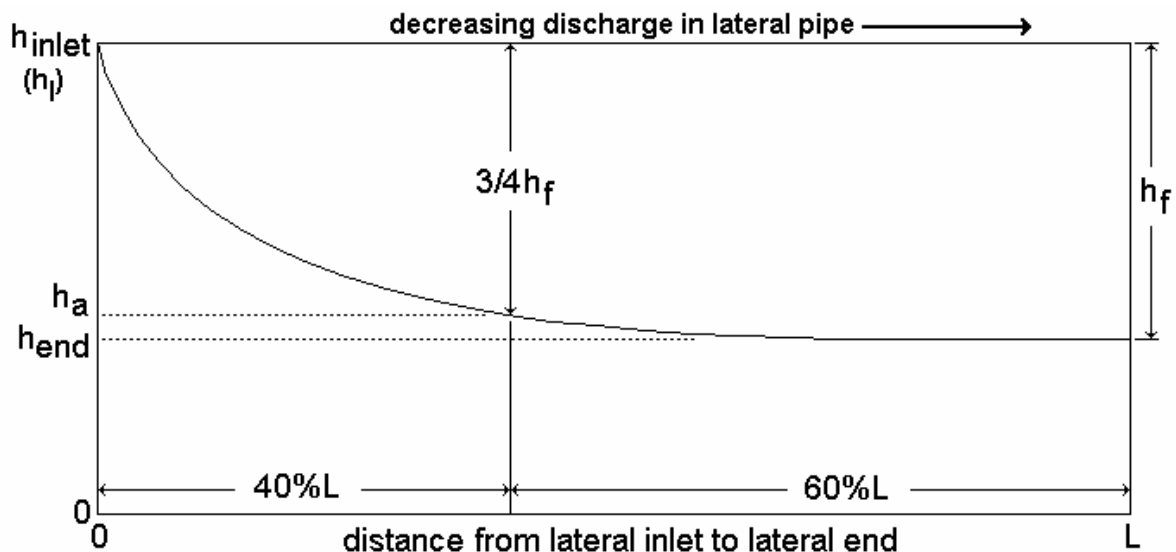
Set Sprinkler Lateral Design

I. Basic Design Criterion

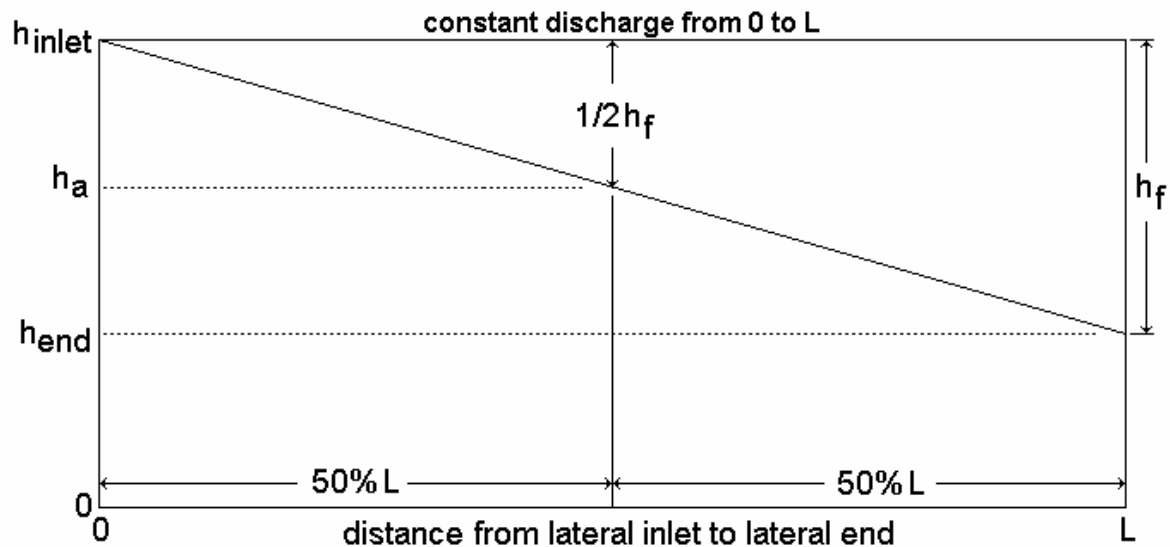
- The basic design criterion is to size lateral pipes so that pressure variation along the length of the lateral does not exceed *20% of the nominal design pressure* for the sprinklers
- This criterion is a compromise between cost of the lateral pipe and application uniformity in the direction of the 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:

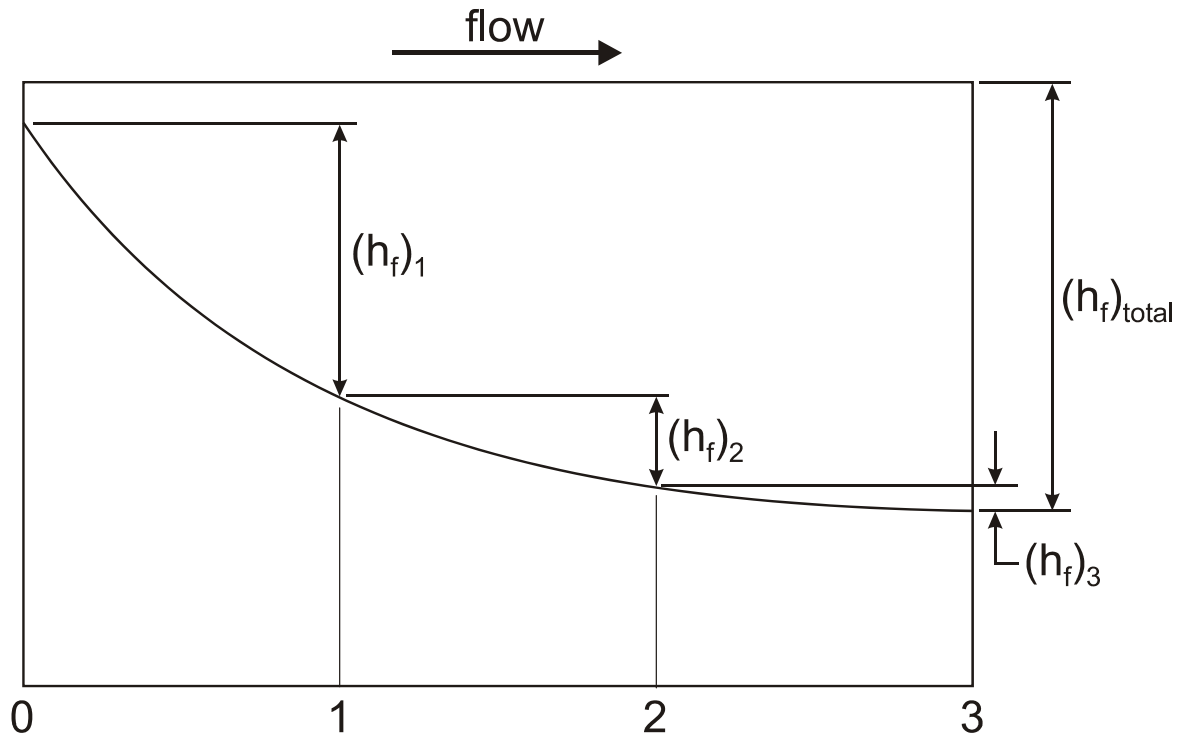
$$(h_f)_{\text{total}} = \sum_{i=1}^n (h_f)_i \quad (90)$$

Friction head loss to location of h_a :

$$(h_f)_a = \frac{\sum_{i=1}^n \sum_{j=1}^i (h_f)_j}{n+1} \quad (91)$$

where n is the number of sprinklers; $(h_f)_{\text{total}}$ 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 $(h_f)_a$, so the denominator is $(n+1)$, not n



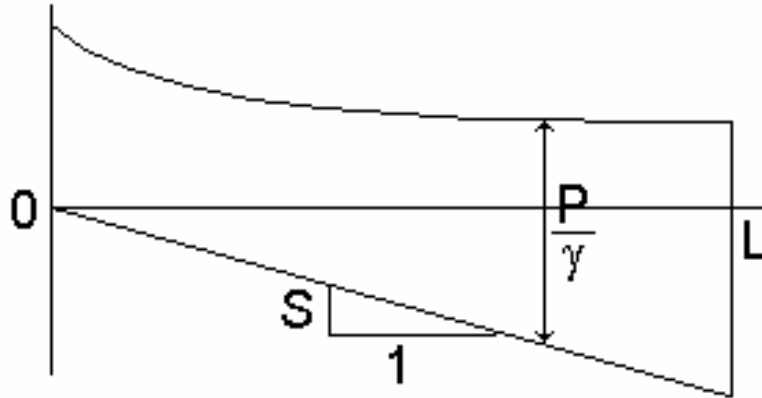
- In applying these equations with sample data, the following result can be found:

$$\frac{(h_f)_a}{(h_f)_{total}} \approx 0.73 \quad (92)$$

- This supports the above claim that approximately $\frac{3}{4}$ 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:

$$J = 1.21(10)^{12} \left(\frac{Q}{C} \right)^{1.852} D^{-4.87} \quad (93)$$

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:

$$Q = Q_l - \left(\frac{q_a}{S_e} \right) x \quad (94)$$

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,

$$x = \frac{S_e}{q_a} \left[Q_l - 3(10)^{-7} \left(C(-S)^{0.54} D^{2.63} \right) \right] \quad (95)$$

where x is the distance, in m, from the lateral inlet to the minimum pressure

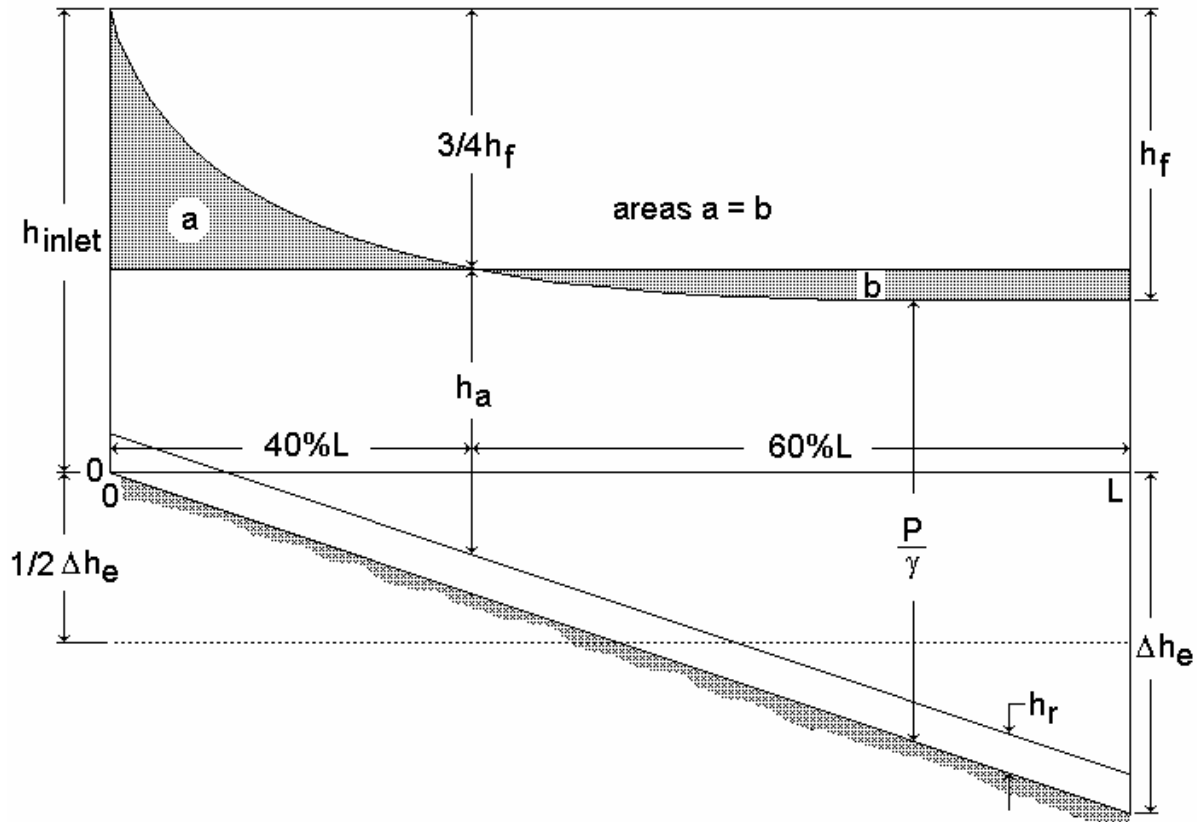
- S is in percent; S_e and x are in m; D is in mm; and Q_i and q_a are in lps
- Note that the valid range of x is: $0 \leq x \leq L$, and that you won't necessarily get $J = S$ over this range of x values:
 - If you get $x < 0$ then the minimum pressure is at the inlet
 - If you get $x > L$ then the 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:

$$h_l = h_a + \frac{3}{4}h_f + \frac{1}{2}\Delta h_e \quad (96)$$

- h_l is the required pressure head at the lateral inlet
- Strictly speaking, we should take approximately $0.4\Delta h_e$ 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

$$h_f = -\Delta h_e \quad (97)$$

- 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)...

$$-\Delta h_e > 0.3h_a \quad (98)$$

- 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:

$$h_f = 0.20h_a \quad (\text{for } S = 0 \text{ only}) \quad (99)$$

- 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):

$$h_l = h_a + \frac{3}{4}h_f + \frac{1}{2}\Delta h_e + h_r \quad (100)$$

or, in terms of pressure...

$$P_l = P_a + \frac{3}{4}P_f + \frac{1}{2}\Delta P_e + P_r \quad (101)$$

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:

$$h_f = \frac{JFL}{100} \quad (102)$$

where F is from Eq. 8.9a

$$F = \frac{1}{b+1} + \frac{1}{2N} + \frac{\sqrt{b-1}}{6N^2} \quad (103)$$

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

$$F(\alpha) = \frac{NF - (1 - \alpha)}{N - (1 - \alpha)} \quad (104)$$

where $0 < \alpha \leq 1$

- Note that when $\alpha = 1$, $F(\alpha) = F$
- Many sprinkler systems have the first sprinkler at a distance of $\frac{1}{2}S_e$ 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:

$$\Delta h = h_f + \Delta h_e \quad (105)$$

If we allow $\Delta h = 0.20 h_a$, then

$$0.20h_a = h_f + \Delta h_e \quad (106)$$

$$0.20h_a - \Delta h_e = \frac{J_a FL}{100} \quad (107)$$

and,

$$J_a = 100 \left(\frac{0.20h_a - \Delta h_e}{FL} \right) \quad (108)$$

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:

$$h_f = -\Delta h_e \quad (109)$$

$$J_a = 100 \left(\frac{-\Delta h_e}{FL} \right) \quad (110)$$

- 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:

$$D = \left[\frac{K}{J_a} \left(\frac{Q_l}{C} \right)^{1.852} \right]^{0.205} \quad (111)$$

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 rounded up 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:

$$\begin{aligned}N_n &= 396/12 = 33 \text{ sprinklers} \\F &= 0.36 \\Q_l &= (0.315)(33) = 10.4 \text{ lps} \\\Delta h_e &= SL = (-0.0253)(396) = -10.0 \text{ m}\end{aligned}$$

$$\begin{aligned}(P_f)_a &= 0.20Pa - \Delta h_e = 0.20(320 \text{ kPa}) - 9.81(-10.0 \text{ m}) = 162 \text{ kPa} \\(h_f)_a &= 162/9.81 = 16.5 \text{ m}\end{aligned}$$

VI.3. Calculations leading to required lateral pipe inside diameter:

$$\begin{aligned}0.3P_a &= 0.3(320 \text{ kPa}) = 96.0 \text{ kPa} \\0.3h_a &= 96.0/9.81 = 9.79 \text{ m}\end{aligned}$$

Now, $0.3h_a < -\Delta h_e$ (steep downhill). Therefore, may want to use $h_f = -\Delta h_e$. Then, J_a is:

$$J_a = 100 \left(\frac{-\Delta h_e}{FL} \right) = 100 \left(\frac{-(-10.0\text{m})}{(0.36)(396)} \right) = 7.01\text{m}/100\text{m} \quad (112)$$

However, if $0.3h_a > -\Delta h_e$, J_a would be calculated as:

$$J_a = 100 \left(\frac{0.20h_a - \Delta h_e}{FL} \right) = 100 \left(\frac{16.5}{(0.36)(396)} \right) = 11.6\text{m}/100\text{m} \quad (113)$$

For now, let's use $J_a = 7.01 \text{ m}/100 \text{ m}$. Then, the minimum pipe inside diameter is ($C \approx 130$ for aluminum):

$$D = \left[\frac{1.21\text{E}12 \left(\frac{10.4}{130} \right)^{1.852}}{7.01} \right]^{0.205} = 77.7\text{mm} \quad (114)$$

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 $J_a = 11.6$, $D = 70.0 \text{ mm}$).

VI.4. Check the design with the choices made thus far

The real friction loss will be:

$$J = 1.21E12 \left(\frac{10.4}{130} \right)^{1.852} (99.1 \text{ mm})^{-4.87} = 2.14 \text{ m}/100 \text{ m} \quad (115)$$

$$h_f = \frac{JFL}{100} = \frac{(2.14)(0.36)(396)}{100} = 3.06 \text{ m} \quad (116)$$

The required lateral inlet pressure head is:

$$\begin{aligned} h_l &= h_a + 0.75h_f + 0.5\Delta h_e + h_r \\ h_l &= 320/9.81 + 0.75(3.06) + 0.5(-10.0) + 1.0 = 30.9 \text{ m} \end{aligned} \quad (117)$$

Thus, P_l is $(30.9)(9.81) = 303 \text{ kPa}$, 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

$$h_{\text{end}} = h_l - h_f - \Delta h_e = 30.9 - 3.06 - (-10.0) = 37.8 \text{ m} \quad (118)$$

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 $37.8 \text{ m} - 1.0 \text{ m} = 36.8 \text{ m}$ (361 kPa)

VI.6. Calculate the location of minimum pressure in the lateral pipe

$$\begin{aligned} x &= \frac{S_e}{q_a} \left[Q_l - 3(10)^{-7} \left(C(-S)^{0.54} D^{2.63} \right) \right] \\ x &= \frac{12}{0.315} \left[10.4 - 3(10)^{-7} \left(130(2.53)^{0.54} (99.1)^{2.63} \right) \right] = -39.6 \text{ m} \end{aligned} \quad (119)$$

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 $h_l - h_r = 30.9 - 1.0 = 29.9 \text{ m}$, 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:

$$\Delta P = \frac{P_{\max} - P_{\min}}{P_a} = \frac{361 - 293}{320} = 0.21\% \quad (120)$$

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:

$$x = \frac{12}{0.315} \left[10.4 - 3(10)^{-7} \left(130(2.53)^{0.54} (73.7)^{2.63} \right) \right] = 196 \text{ m} \quad (121)$$

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:

$$J_{x\text{-end}} = 1.21E12 \left(\frac{(17)(0.315)}{130} \right)^{1.852} (73.7)^{-4.87} = 2.65 \text{ m}/100 \text{ m} \quad (122)$$

$$(h_f)_{x\text{-end}} = \frac{(2.65)(0.38)(396 - 196)}{100} = 2.01 \text{ m} \quad (123)$$

Friction loss from the inlet to the end is:

$$J_{\text{inlet-end}} = 1.21E12 \left(\frac{10.4}{130} \right)^{1.852} (73.7)^{-4.87} = 9.05 \text{ m}/100 \text{ m} \quad (124)$$

$$(h_f)_{\text{inlet-end}} = \frac{(9.05)(0.36)(396)}{100} = 12.9 \text{ m} \quad (125)$$

Then, friction loss from inlet to x is:

$$(h_f)_{\text{inlet-x}} = 12.9 - 2.01 = 10.9 \text{ m} \quad (126)$$

The required lateral pipe inlet head is:

$$\begin{aligned} h_l &= h_a + 0.75h_f + 0.5\Delta h_e + h_r \\ h_l &= 320/9.81 + 0.75(12.9) + 0.5(-10.0) + 1.0 = 38.3 \text{ m} \end{aligned} \quad (127)$$

giving a P_l of $(38.3)(9.81) = 376 \text{ kPa}$, which is higher than P_l for the 4" pipe

The minimum pressure head (at distance $x = 196 \text{ m}$) is:

$$\begin{aligned} h_x &= h_l - (h_f)_{\text{inlet-x}} - (\Delta h_e)_{\text{inlet-x}} \\ h_x &= 38.3 - 10.9 - (-0.0253)(196) = 32.4 \text{ m} \end{aligned} \quad (128)$$

giving a P_x of $(32.4)(9.81) = 318 \text{ kPa}$, which is very near P_a .

The pressure head at the end of the lateral pipe is:

$$h_{\text{end}} = h_l - h_f - \Delta h_e = 38.3 - 12.9 + 10.0 = 35.4 \text{ m} \quad (129)$$

giving P_{end} of $(35.4)(9.81) = 347 \text{ kPa}$, 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 $P_{\text{max}} = 376 - (1.0)(9.81) = 366 \text{ kPa}$, and $P_{\text{min}} = 318 - (1.0)(9.81) = 308 \text{ kPa}$:

$$\% \Delta P = \frac{P_{\text{max}} - P_{\text{min}}}{P_a} = \frac{366 - 308}{320} = 0.18 \quad (130)$$

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:

$$\begin{aligned} J_a &= 100 \left(\frac{0.20h_a - \Delta h_e}{FL} \right) \\ &= 100 \left(\frac{0.2(320/9.81) - 10.0}{(0.36)(396)} \right) = -2.44\text{m}/100\text{m} \end{aligned} \quad (131)$$

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).

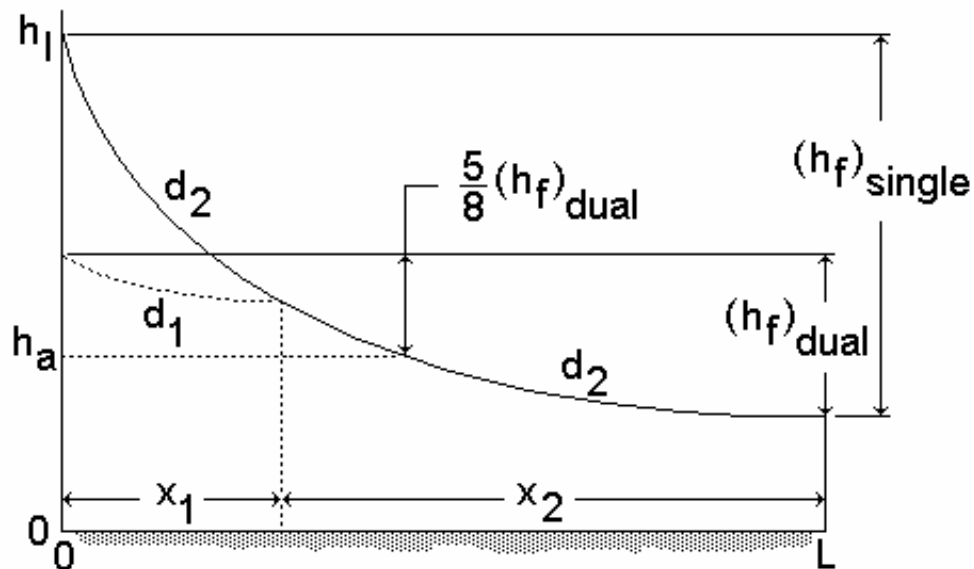
Most sprinkler laterals are laid on slopes less than 2.5%, in fact, most are on fields with less than 1% slope.

Lecture 8

Set Sprinkler Lateral Design

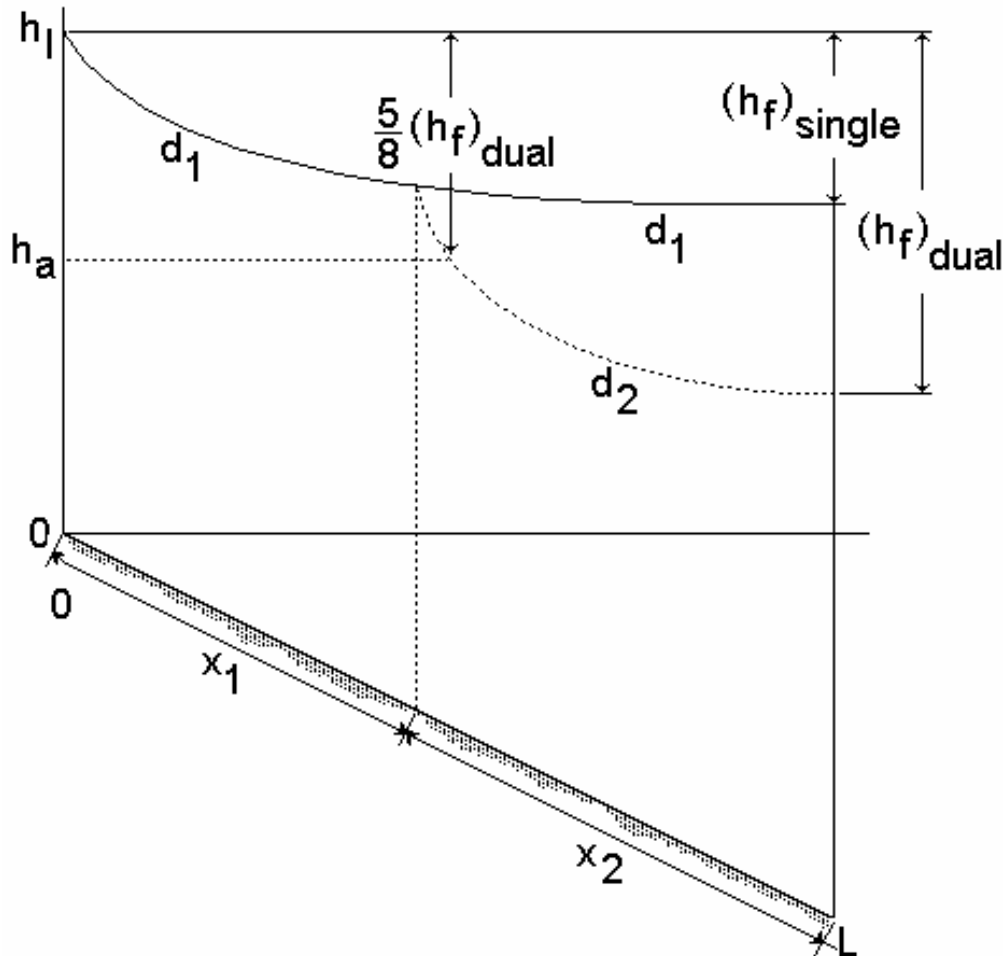
I. Dual Pipe Size Laterals

- 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 $(h_f)_{\text{single}}$ is much larger than $(h_f)_{\text{dual}}$

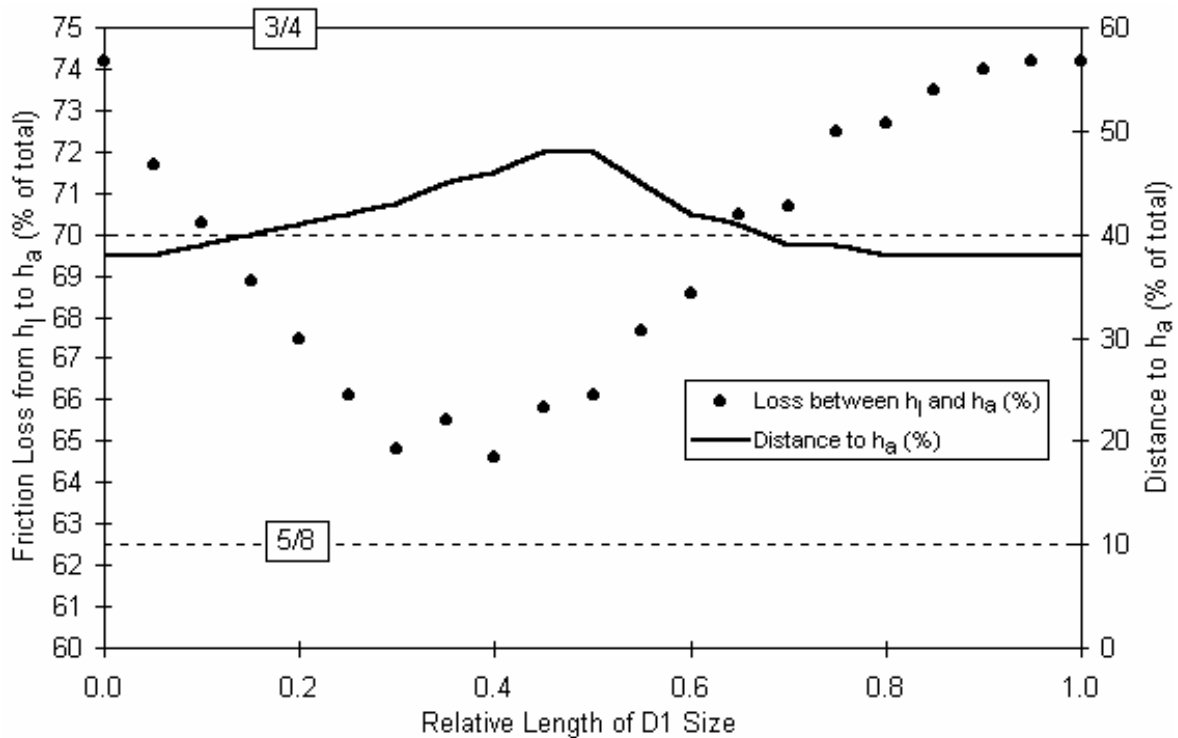
- **Case 2:** a lateral running downhill where one pipe size is too big, but the next smaller size is too small...



- 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 $\frac{5}{8}(h_f)_{dual}$ occurs between h_1 and h_a ?
- Consider the analysis shown graphically below ($\frac{5}{8} = 0.625$)
- 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 $\frac{3}{4}$ value is on the left-hand ordinate
- Notice that the head loss from h_1 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_1 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 h_a (%)	$h_f/(h_f)_{total}$
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:

$$h_f = \left(\frac{J_1 F_1 L}{100} - \frac{J_2 F_2 x_2}{100} \right) + \left(\frac{J_3 F_2 x_2}{100} \right) \quad (132)$$

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

$$\Delta h = h_f + \Delta h_e = 20\%h_a \quad (133)$$

and,

$$h_f = 20\%h_a - \Delta h_e \quad (134)$$

- Find d_1 and d_2 in tables (or by calculation) using Q_{inlet} and...

$$(J)_{d1} \leq J_a \leq (J)_{d2} \quad (135)$$

for,

$$J_a = 100 \left(\frac{20\%h_a - \Delta h_e}{FL} \right) \quad (136)$$

- 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:

$$F_1 \approx 0.351 + \frac{1}{2N_1} \left(1 + \frac{4}{13N_1} \right) \quad (137)$$

$$F_2 \approx 0.351 + \frac{1}{2N_2} \left(1 + \frac{4}{13N_2} \right) \quad (138)$$

where

$$N_1 = \frac{L}{S_e} \quad (139)$$

$$N_2 = \frac{L - x_1}{S_e} \quad (140)$$

- The three friction loss gradients are:

$$J_1 = K \left(\frac{Q_1}{C} \right)^{1.852} D_1^{-4.87} \quad (141)$$

$$J_2 = K \left(\frac{Q_2}{C} \right)^{1.852} D_1^{-4.87} \quad (142)$$

$$J_3 = K \left(\frac{Q_2}{C} \right)^{1.852} D_2^{-4.87} \quad (143)$$

where

$$Q_1 = \left(\frac{L}{S_e} \right) q_a \quad (144)$$

$$Q_2 = \left(\frac{L - x_1}{S_e} \right) q_a \quad (145)$$

- 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:

$$f(x_1) = \alpha_1 \left[\alpha_2 - \alpha_3 (L - x_1)^{2.852} F_2 \right] - 0.2h_a + \Delta h_e = 0 \quad (146)$$

where

$$\alpha_1 = \frac{K}{100C^{1.852}} \quad (147)$$

$$\alpha_2 = \left(\frac{q_a L}{S_e} \right)^{1.852} D_1^{-4.87} F_1 L \quad (148)$$

$$\alpha_3 = \left(D_1^{-4.87} - D_2^{-4.87} \right) \left(\frac{q_a}{S_e} \right)^{1.852} \quad (149)$$

- 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

$$\begin{aligned} \frac{df(x_1)}{dx_1} &= \\ &= \alpha_1 \alpha_3 \left[2.852 F_2 (L - x_1)^{1.852} - \frac{S_e (L - x_1)^{0.852}}{2} \left(1 + \frac{8 S_e}{13(L - x_1)} \right) \right] \quad (150) \end{aligned}$$

- 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

Input	Value
Number of sprinklers	30
Sprinkler spacing (m)	9.00
Sprinkler head, ha (m)	28.093
Sprinkler flow rate, qa (lps)	0.230
Hazen-Williams C	130.0
Ground slope (neg downhill)	-0.0010000
Pipe IDs (cm)	4.826, 7.366, 9.906, 12.466, 14.945, 19.954, 24.938, 30.018
Solution	D1 = 7.366 cm, D2 = 4.826 cm, L1 = 174.05 m, L2 = 95.95 m

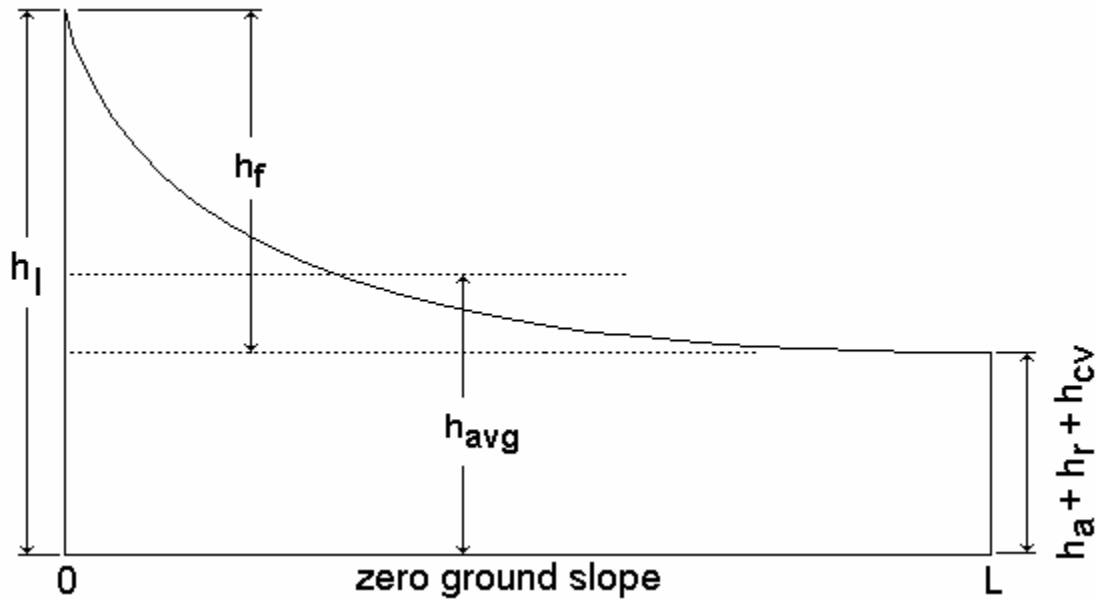
V. Inlet Pressure for Dual Pipe Size Laterals

$$h_l = h_a + \frac{5}{8}h_f + \frac{1}{2}\Delta h_e + h_r \quad (151)$$

- 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 opposing

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...

$$h_l = h_a + h_f + \Delta h_e + h_r + h_{cv} \quad (152)$$

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

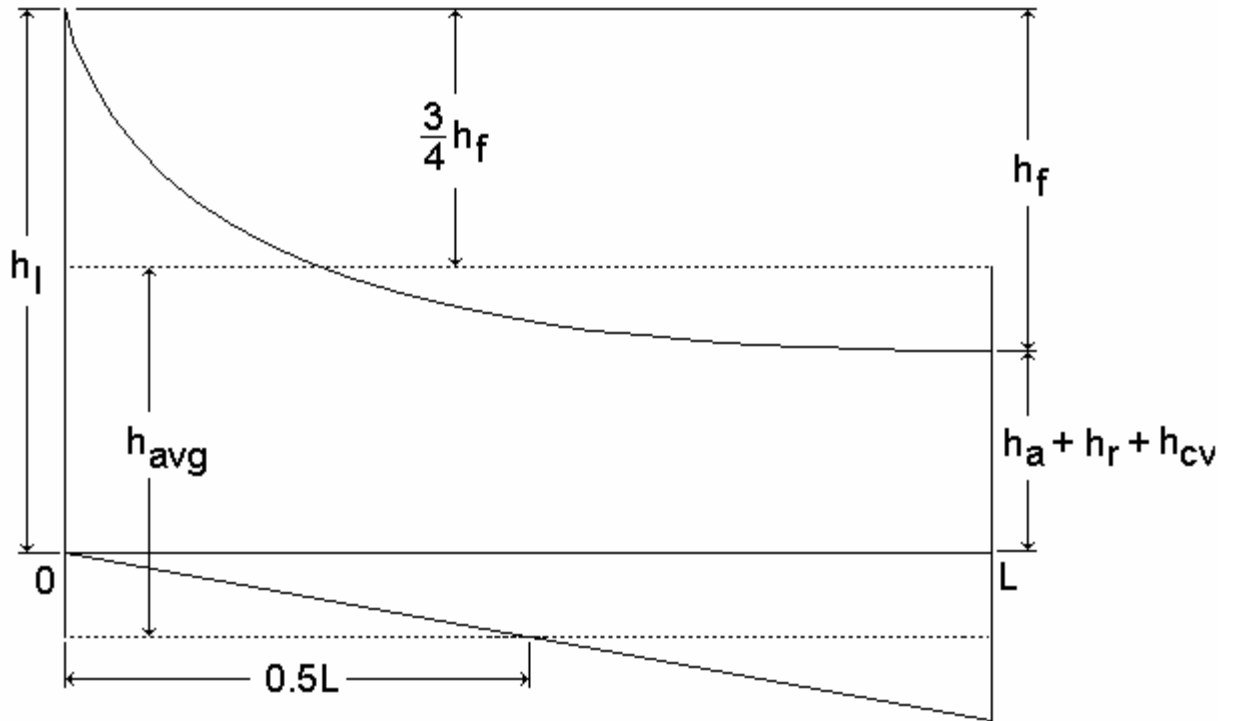
$$h_{avg} \neq h_a \quad (153)$$

- If the pressure in the lateral is enough everywhere, then

$$h_a = \left(\frac{q_a}{K_d} \right)^2 \quad (154)$$

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



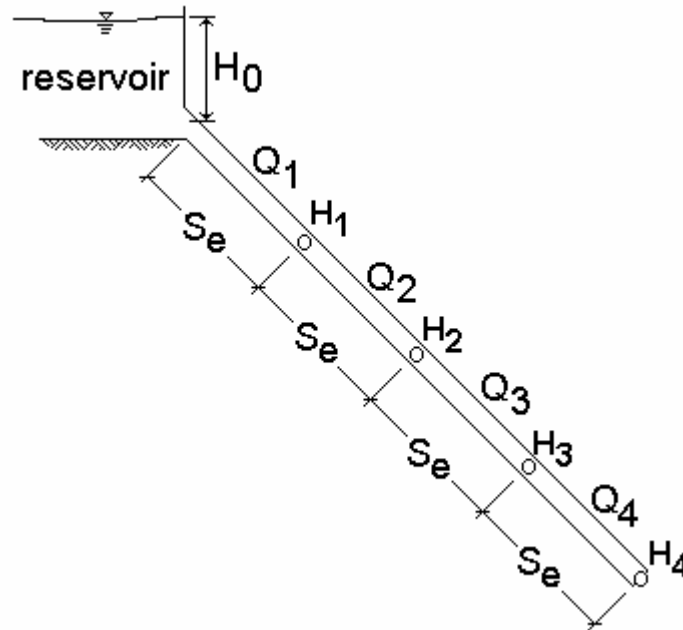
VII. Anti-Drain Valves

- 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), what is the flow rate through each sprinkler?
- 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:

$$h_f = \frac{JL}{100} \quad (155)$$

$$J = 16.42(10)^6 \left(\frac{Q}{C} \right)^{1.852} D^{-4.87} \quad (156)$$

for Q in lps; D in cm; J in m/100 m; L in m; and h_f in m.

- Between two sprinklers,

$$h_f = \frac{JS_e}{100} = 16.42(10)^4 S_e \left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \quad (157)$$

or,

$$h_f = h_w Q^{1.852} \quad (158)$$

where Q is the flow rate in the lateral pipe between two sprinklers, and

$$h_w = 16.42(10)^4 S_e C^{-1.852} D^{-4.87} \quad (159)$$

III. Sprinkler Discharge

typically,

$$q = K_d \sqrt{h} \quad (160)$$

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

$$q_1 = K_d \sqrt{H_1 - h_r} \rightarrow H_1 = h_r + \left(\frac{q_1}{K_d}\right)^2 = h_r + \frac{(Q_1 - Q_2)^2}{K_d^2} \quad (161)$$

$$q_2 = K_d \sqrt{H_2 - h_r} \rightarrow H_2 = h_r + \left(\frac{q_2}{K_d}\right)^2 = h_r + \frac{(Q_2 - Q_3)^2}{K_d^2} \quad (162)$$

$$q_3 = K_d \sqrt{H_3 - h_r} \rightarrow H_3 = h_r + \left(\frac{q_3}{K_d}\right)^2 = h_r + \frac{(Q_3 - Q_4)^2}{K_d^2} \quad (163)$$

$$q_4 = K_d \sqrt{H_4 - h_r} \rightarrow H_4 = h_r + \left(\frac{q_4}{K_d}\right)^2 = h_r + \frac{Q_4^2}{K_d^2} \quad (164)$$

- Pressure heads can also be defined independently in terms of friction loss along the lateral pipe

$$H_1 = H_0 - h_w Q_1^{1.852} - \Delta h_e \quad (165)$$

$$H_2 = H_1 - h_w Q_2^{1.852} - \Delta h_e \quad (166)$$

$$H_3 = H_2 - h_w Q_3^{1.852} - \Delta h_e \quad (167)$$

$$H_4 = H_3 - h_w Q_4^{1.852} - \Delta h_e \quad (168)$$

where,

$$\Delta h_e = \frac{S_e}{\sqrt{S_o^{-2} + 1}} \quad (169)$$

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:

$$\frac{(Q_1 - Q_2)^2}{K_d^2} = H_0 - h_w Q_1^{1.852} - \Delta h_e - h_r \quad (170)$$

$$\frac{(Q_2 - Q_3)^2}{K_d^2} = H_1 - h_w Q_2^{1.852} - \Delta h_e - h_r \quad (171)$$

$$\frac{(Q_3 - Q_4)^2}{K_d^2} = H_2 - h_w Q_3^{1.852} - \Delta h_e - h_r \quad (172)$$

$$\frac{Q_4^2}{K_d^2} = H_3 - h_w Q_4^{1.852} - \Delta h_e - h_r \quad (173)$$

- Setting the equations equal to zero:

$$f_1 = H_0 - \frac{(Q_1 - Q_2)^2}{K_d^2} - h_w Q_1^{1.852} - \Delta h_e - h_r = 0 \quad (174)$$

$$f_2 = \frac{(Q_1 - Q_2)^2}{K_d^2} - \frac{(Q_2 - Q_3)^2}{K_d^2} - h_w Q_2^{1.852} - \Delta h_e - h_r = 0 \quad (175)$$

$$f_3 = \frac{(Q_2 - Q_3)^2}{K_d^2} - \frac{(Q_3 - Q_4)^2}{K_d^2} - h_w Q_3^{1.852} - \Delta h_e - h_r = 0 \quad (176)$$

$$f_4 = \frac{(Q_3 - Q_4)^2}{K_d^2} - \frac{Q_4^2}{K_d^2} - h_w Q_4^{1.852} - \Delta h_e - h_r = 0 \quad (177)$$

The system of equations can be put into matrix form as follows:

$$\begin{bmatrix} \frac{\partial f_1}{\partial Q_1} & \frac{\partial f_1}{\partial Q_2} & & & \\ \frac{\partial f_2}{\partial Q_1} & \frac{\partial f_2}{\partial Q_2} & \frac{\partial f_2}{\partial Q_3} & & \\ & \frac{\partial f_3}{\partial Q_2} & \frac{\partial f_3}{\partial Q_3} & \frac{\partial f_3}{\partial Q_4} & \\ & & \frac{\partial f_4}{\partial Q_3} & \frac{\partial f_4}{\partial Q_4} & \end{bmatrix} \begin{bmatrix} \delta Q_1 \\ \delta Q_2 \\ \delta Q_3 \\ \delta Q_4 \end{bmatrix} = \begin{bmatrix} f_1 \\ f_2 \\ f_3 \\ f_4 \end{bmatrix} \quad (178)$$

where the two values in the first row of the square matrix are:

$$\frac{\partial f_1}{\partial Q_1} = \frac{-2(Q_1 - Q_2)}{K_d^2} - 1.852 h_w Q_1^{0.852} \quad (179)$$

$$\frac{\partial f_1}{\partial Q_2} = \frac{2(Q_1 - Q_2)}{K_d^2} \quad (180)$$

The two values in the last row of the square matrix, for n sprinklers, are:

$$\frac{\partial f_n}{\partial Q_{n-1}} = \frac{2(Q_{n-1} - Q_n)}{K_d^2} \quad (181)$$

$$\frac{\partial f_n}{\partial Q_n} = \frac{-2Q_{n-1}}{K_d^2} - 1.852h_w Q_n^{0.852} \quad (182)$$

and the three values in each intermediate row of the matrix are:

$$\frac{\partial f_i}{\partial Q_{i-1}} = \frac{2(Q_{i-1} - Q_i)}{K_d^2} \quad (183)$$

$$\frac{\partial f_i}{\partial Q_i} = \frac{2(Q_{i+1} - Q_{i-1})}{K_d^2} - 1.852h_w Q_i^{0.852} \quad (184)$$

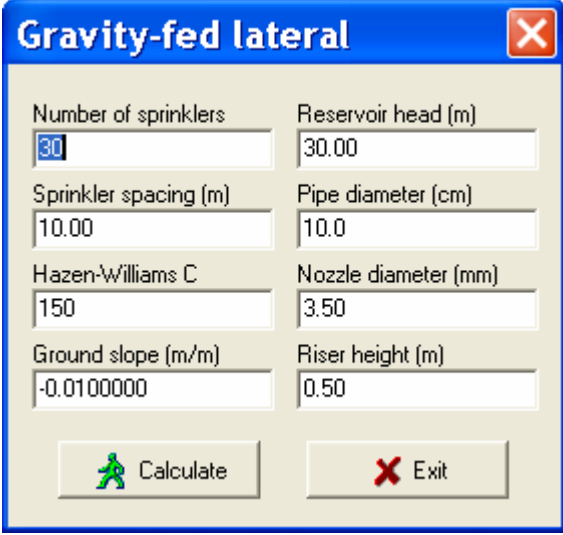
$$\frac{\partial f_i}{\partial Q_{i+1}} = \frac{2(Q_i - Q_{i+1})}{K_d^2} \quad (185)$$

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



The screenshot shows a software window titled "Gravity-fed lateral" with a close button (X) in the top right corner. The window contains several input fields arranged in two columns:

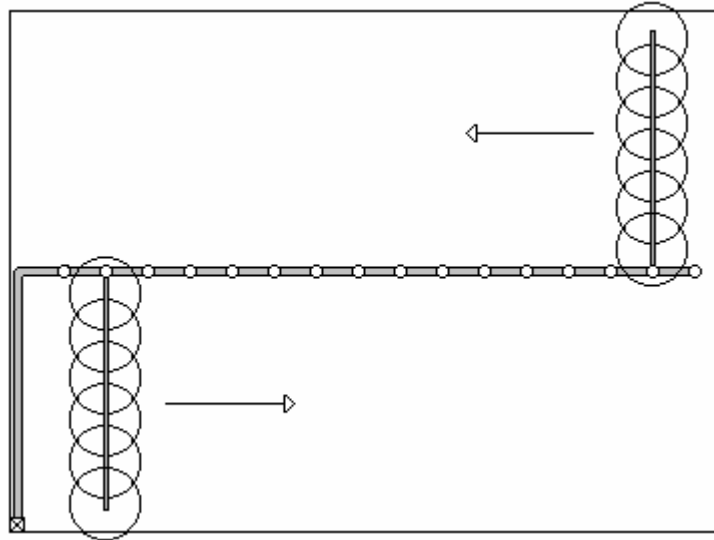
Number of sprinklers 30	Reservoir head (m) 30.00
Sprinkler spacing (m) 10.00	Pipe diameter (cm) 10.0
Hazen-Williams C 150	Nozzle diameter (mm) 3.50
Ground slope (m/m) -0.0100000	Riser height (m) 0.50

At the bottom of the window, there are two buttons: "Calculate" (with a green person icon) and "Exit" (with a red X icon).

Lecture 9

Mainline Pipe Design

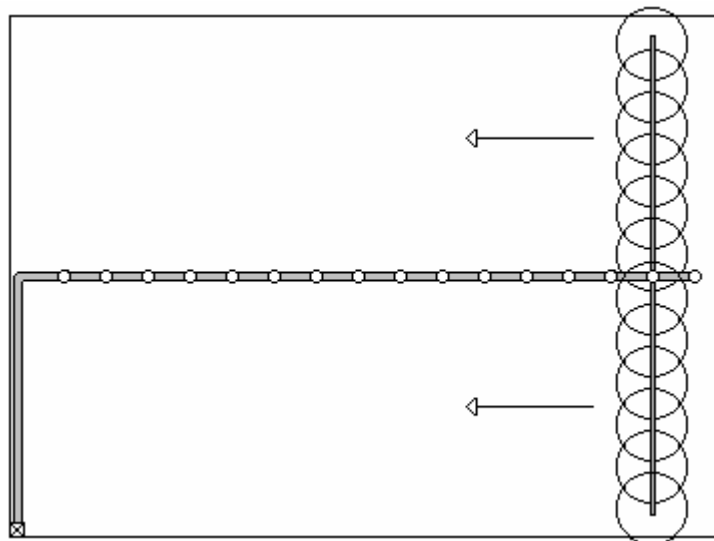
I. Split-Line Laterals



Laterals are usually distributed evenly along a mainline because:

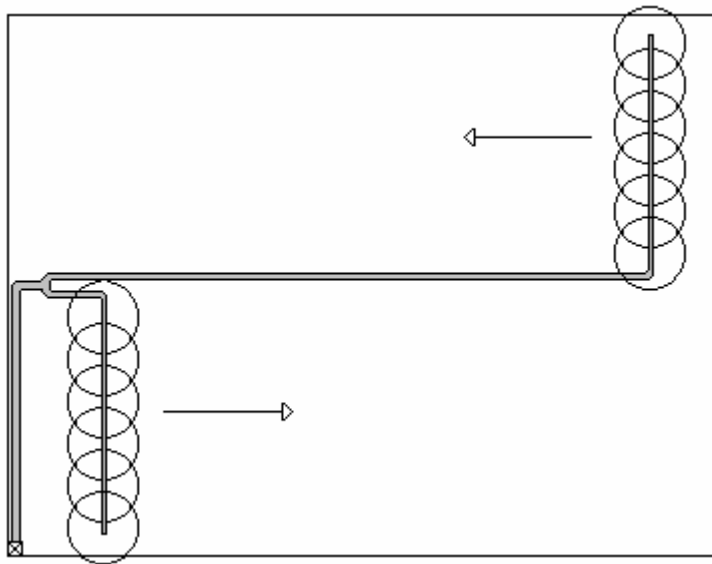
- More equal pump load at different lateral positions
- Reduced mainline cost
- Don't need to "dead-head" back when finished (cross over to other side)
- But, split-line laterals may interfere with cultural operations (wet areas at both ends of field)

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

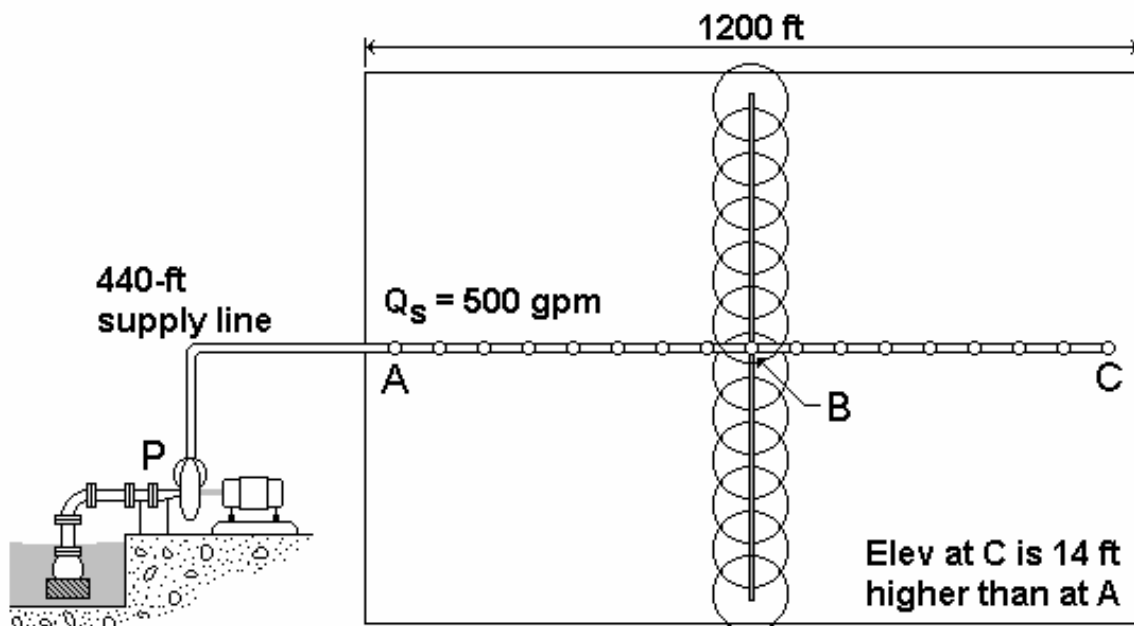
- Look at extreme operating conditions for the mainline by varying lateral positions (this can be more complicated for irregular field shapes and non-uniform field slopes)
- Can use economic pipe selection method, but don't make a big sacrifice in terms of pressure uniformity along the mainline to save 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

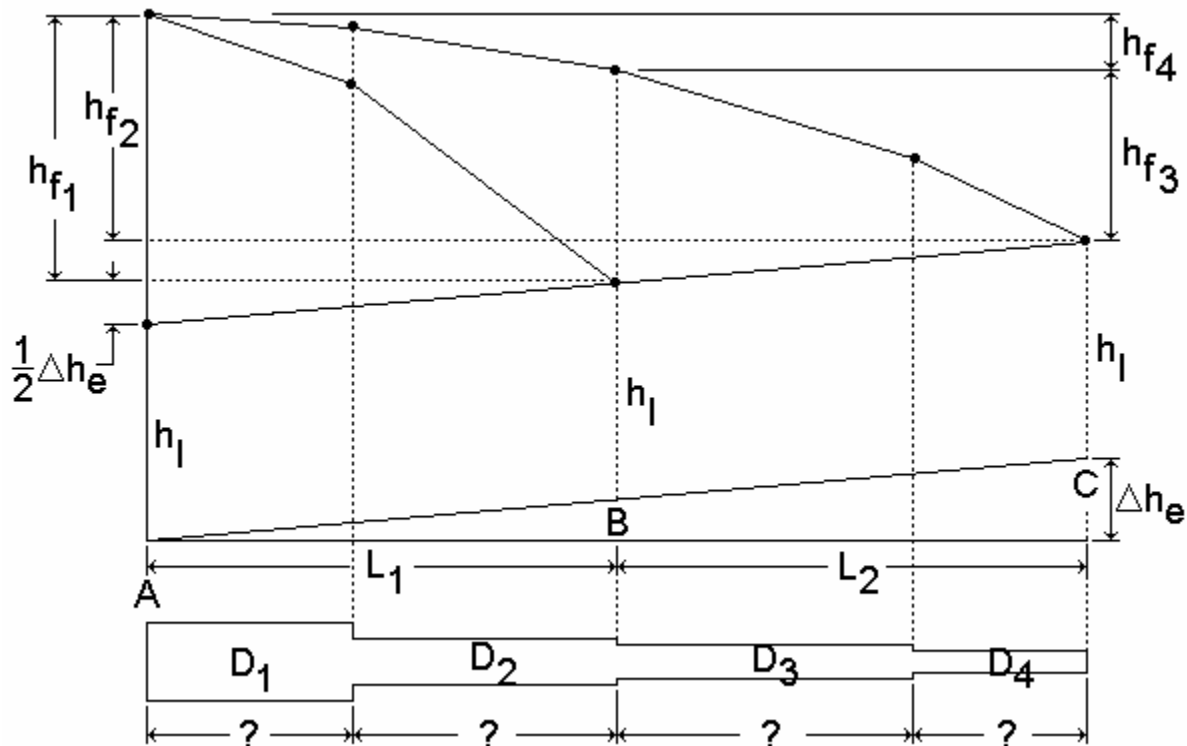
Uphill Split-Line Mainline Design

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:

$$D = \sqrt{\frac{4Q}{\pi V}} = \sqrt{\frac{4(1.11 \text{ cfs})}{\pi(5 \text{ fps})}} = 0.53 \text{ ft} \quad (186)$$

- 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,

$$(h_f)_{P-A} = (2.27 \text{ ft}/100 \text{ ft}) \left(\frac{440 \text{ ft}}{100} \right) = 10.0 \text{ ft} \quad (187)$$

- 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:

$$h_{f1} = 162 \text{ ft} - h_l + 0.5 \Delta h_e \quad (188)$$

$$h_{f1} = 162 \text{ ft} - 125 \text{ ft} - 7 \text{ ft} = 30 \text{ ft} \quad (189)$$

- The allowable loss gradient in section L_1 for both laterals operating at B is

$$(J_a)_{L1} = 100 \left(\frac{30 \text{ ft}}{600 \text{ ft}} \right) = 5 \text{ ft per } 100 \text{ ft} \quad (190)$$

- 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...

$$L_{D1}(2.27) + (600 - L_{D1})(5.54) = 100(30 \text{ ft}) \quad (191)$$

- Solving the above, $L_{D1} = 99.1 \text{ ft}$
- Using 30-ft pipe lengths, we adjust the length to...

$$L_{D1} = 90 \text{ ft of } 6'' \text{ pipe (3 sections)} \quad (192)$$

$$L_{D2} = 510 \text{ ft of } 5'' \text{ pipe (17 sections)} \quad (193)$$

- 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 ...

$$h_{f4} = \frac{(90 \text{ ft})(0.63) + (510 \text{ ft})(1.53)}{100} = 8.37 \text{ ft} \quad (194)$$

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_1 (we know that the pressure at A is 162 ft - it is more than enough)...

$$h_{f2} = h_{f1} + 0.5 \Delta h_e = 23.0 \text{ ft} \quad (195)$$

$$h_{f3} = h_{f2} - h_{f4} = 23.0 \text{ ft} - 8.37 \text{ ft} = 14.6 \text{ ft} \quad (196)$$

- The allowable loss gradient in section L_2 for laterals at A and C is

$$(J_a)_{L2} = 100 \left(\frac{14.6 \text{ ft}}{600 \text{ ft}} \right) = 2.43 \text{ ft per } 100 \text{ ft} \quad (197)$$

- 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...

$$L_{D3}(1.53) + (600 - L_{D3})(4.66) = 100(14.6 \text{ ft}) \quad (198)$$

- Solving the above, $L_{D3} = 427$ ft
- Using 30-ft pipe lengths, we adjust the length to...

$$L_{D3} = 420 \text{ ft of } 5" \text{ pipe (14 sections)} \quad (199)$$

$$L_{D4} = 180 \text{ ft of } 4" \text{ pipe (6 sections)} \quad (200)$$

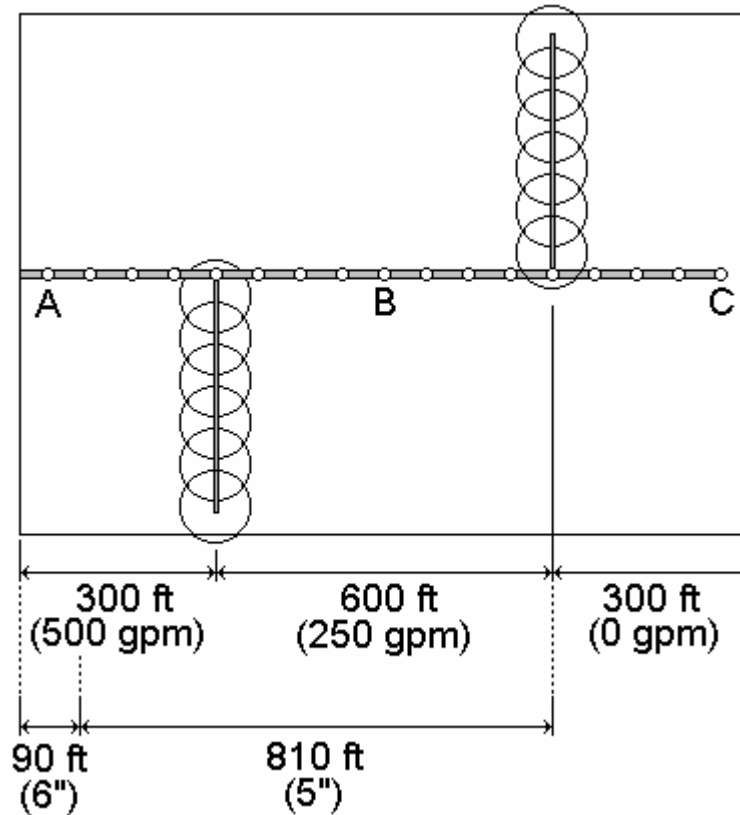
- With the adjusted lengths, we will get 14.8 ft of head loss over section L_2 for 250 gpm (this is close enough to the allowable 14.6 ft)

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 $(h_f)_2 + \frac{1}{4}\Delta h_e$, or 23.0 ft + 3.5 ft = 26.5 ft. The actual friction loss would be:

$$h_f = 0.01[(2.27)(90) + (5.54)(210) + (1.53)(600)] = 22.9 \text{ ft} \quad (201)$$

- 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:

$$h_{f2} = 162 \text{ ft} - h_l + \Delta h_e \quad (202)$$

$$h_{f2} = 162 \text{ ft} - 125 \text{ ft} - 14 \text{ ft} = 23 \text{ ft} \quad (203)$$

- The allowable loss gradient over the length of the mainline for both laterals operating at C is

$$J_a = 100 \left(\frac{23 \text{ ft}}{1,200 \text{ ft}} \right) = 1.92 \text{ ft per 100 ft} \quad (204)$$

- 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...

$$L_{8"} (0.56) + (1,200 - L_{8"}) (2.27) = 100(23 \text{ ft}) \quad (205)$$

- Solving the above, $L_{8"} = 248 \text{ ft}$
- Using 30-ft pipe lengths, we adjust the length to...

$$L_{8"} = 270 \text{ ft of 8" pipe (9 sections)} \quad (206)$$

$$L_{6"} = 930 \text{ ft of 6" pipe (31 sections)} \quad (207)$$

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:

$$h_{ml} = K_r \frac{V^2}{2g} \quad (208)$$

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:

$$K_r = 0.7(1 - D_r^2)^2 \quad (209)$$

where D_r is the ratio of the small to large inside diameters ($D_{\text{small}}/D_{\text{large}}$)

- Allen (1991) proposed a regression equation for gradual contractions and expansions using data from the *Handbook of Hydraulics* (Brater & King 1976):

$$K_r = K_f(1 - D_r^2)^2 \quad (210)$$

where K_f is defined as:

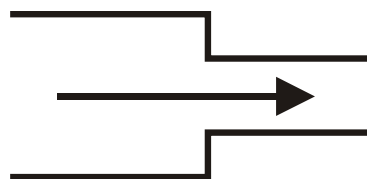
$$K_f = 0.7 - \cos(f) [\cos(f)(3.2 \cos(f) - 3.3) + 0.77] \quad (211)$$

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), $f = 0^\circ$ (whereby $K_f = 0.03$)
- For an abrupt change in pipe diameter (no transition), $f = 90^\circ$ (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

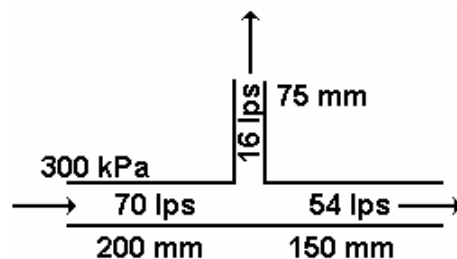
- For a sudden (abrupt) expansion, the head loss can also be approximated as a function of the difference of the mean flow velocities upstream and downstream:

$$h_{ml} = \frac{(V_{us} - V_{ds})^2}{2g} \quad (212)$$

- 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:

$$V_{200} = \frac{0.070 \text{ m}^3/\text{s}}{\left(\frac{\pi(0.200 \text{ m})^2}{4}\right)} = 2.23 \text{ m/s} \quad (213)$$

$$V_{150} = \frac{0.070 - 0.016 \text{ m}^3/\text{s}}{\left(\frac{\pi(0.150 \text{ m})^2}{4}\right)} = 3.06 \text{ m/s} \quad (214)$$

$$V_{hydrant} = \frac{0.016 \text{ m}^3/\text{s}}{\left(\frac{\pi(0.075 \text{ m})^2}{4}\right)} = 3.62 \text{ m/s} \quad (215)$$

- 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,

$$(h_{ml})_{past} = 0.5 \frac{(3.06)^2}{2(9.81)} = 0.24\text{m} \quad (216)$$

- 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:

$$K_r = 0.7 \left[1 - \left(\frac{150}{200} \right)^2 \right]^2 = 0.13 \quad (217)$$

- And, if it were a 45° transition, $K_r = 0.67$, also giving a K_r of 0.13
- Then, the head loss is:

$$(h_{ml})_{contraction} = 0.13 \frac{(3.06)^2}{2(9.81)} = 0.06\text{m} \quad (218)$$

- 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):

$$(h_{ml})_{through} = 8.0 \frac{(3.62)^2}{2(9.81)} = 5.3\text{m} \quad (219)$$

- 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):

$$P_{150} = P_{200} - \gamma(h_{ml})_{past} + \gamma \left(\frac{V_{200}^2 - V_{150}^2}{2g} \right) \quad (220)$$

$$P_{150} = 300 - (9.81)(0.24) + 9.81 \left(\frac{(2.23)^2 - (3.06)^2}{2(9.81)} \right) = 295\text{kPa}$$

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_f 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



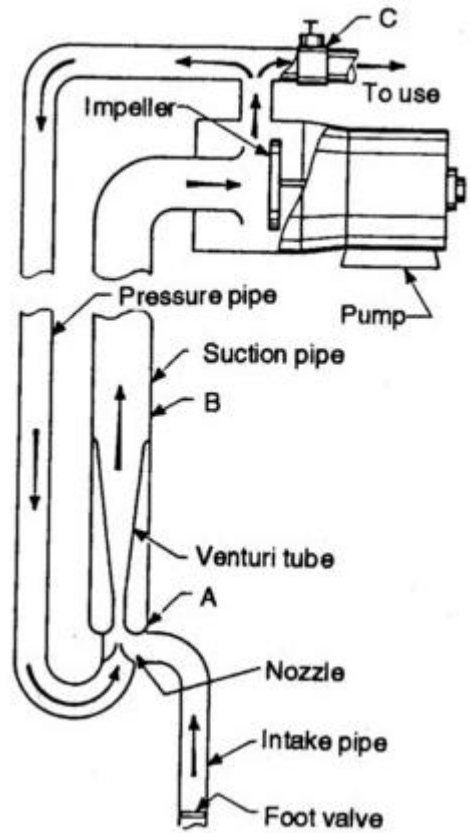
Positive Displacement Pumps



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. Volute Case This is the most common type of irrigation and drainage pump (excluding deep well pumps). Produce relatively high flow rates at low pressures.
 2. Diffuser (Turbine) 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. Mixed Flow Uses a combination of centrifugal and axial flow action. For high capacity at low heads.
 4. Axial Flow 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. Regenerative 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



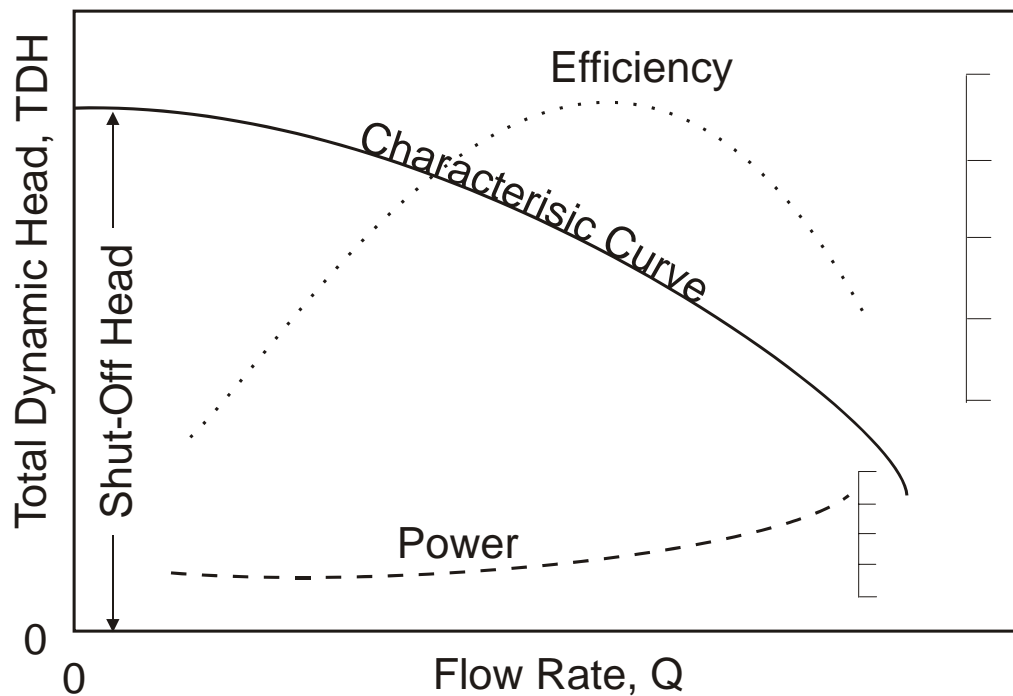
Semi-Open Impeller



Open Impeller

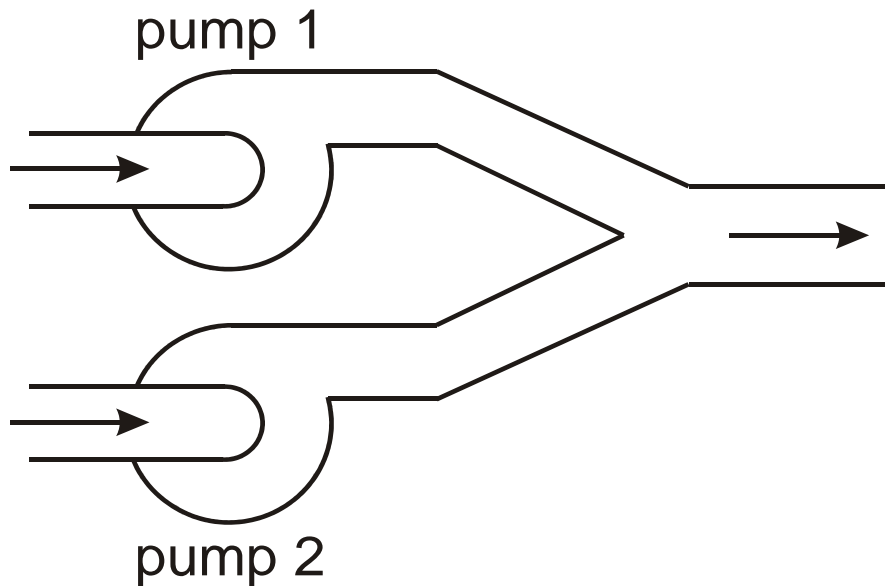
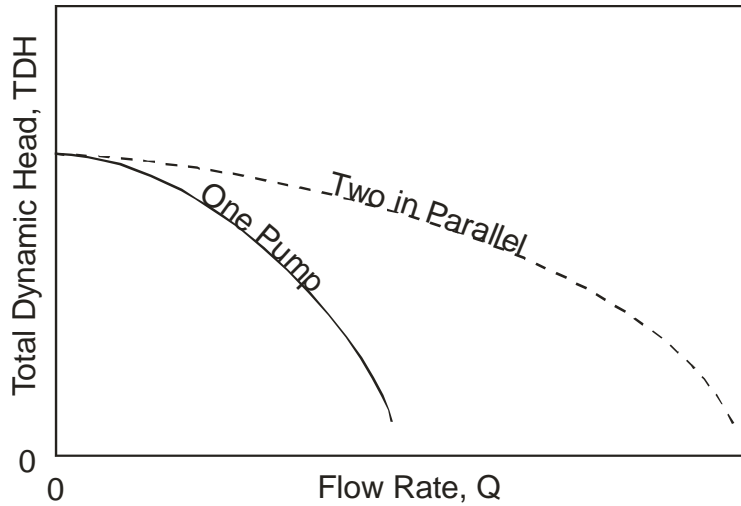
Characteristic Curve

- 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

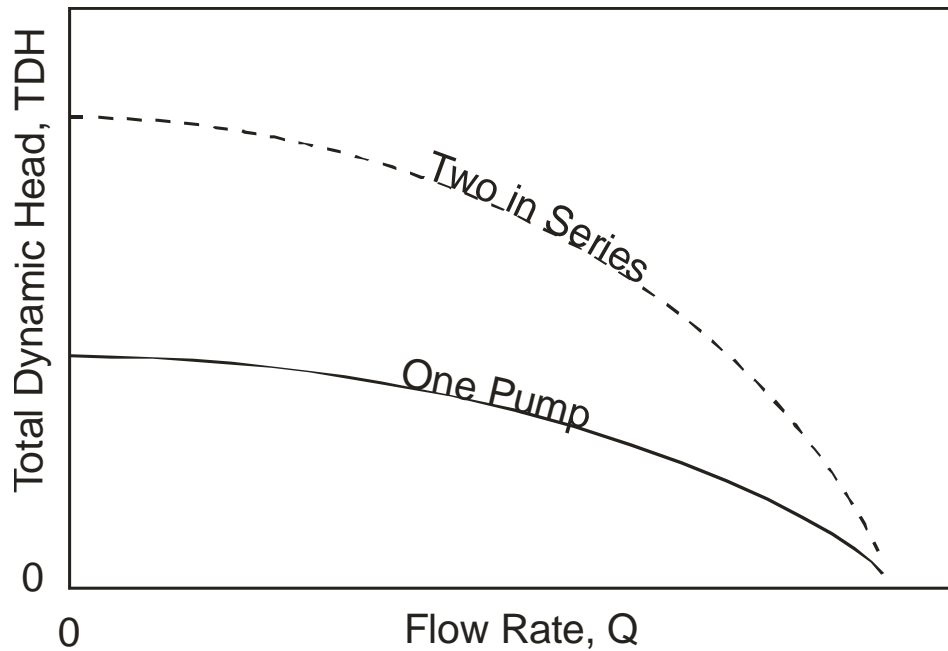
- Pumps in **PARALLEL** means that the total flow is divided into two or more pumps
- Typical installations are for a single inlet pipe, branched into two pumps, with the outlets from the pumps converging to a single discharge pipe
- If only one of the pumps operates, some type of valve may be required so that flow does not flow backwards through the idle pump
- Flow rate is additive in this case



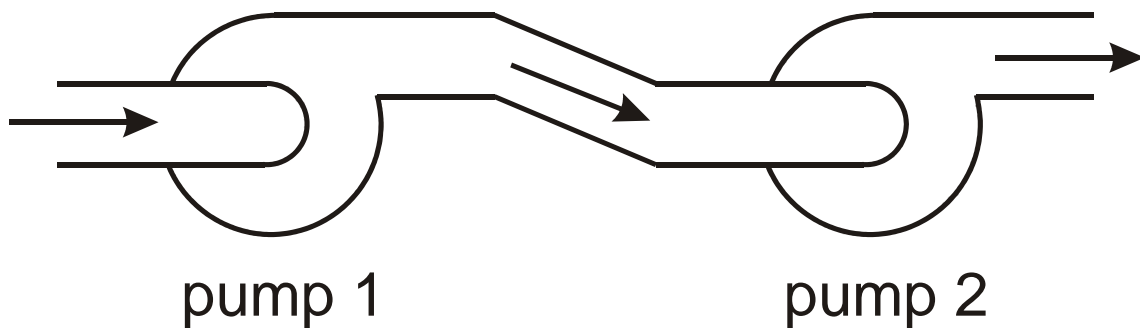
Two Pumps in Parallel

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}

$$E_{\text{pump}} = \frac{\text{water horsepower}}{\text{brake horsepower}} = \frac{\text{WHP}}{\text{BHP}} \quad (221)$$

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:

$$\text{WHP} = \frac{QH}{3956} \quad (222)$$

where WHP is in horsepower; Q in gpm; and H in feet of head. The denominator is derived from:

$$\gamma QH = \frac{(62.4 \text{ lbs/ft}^3)(\text{gal/min})(\text{ft})}{(33,000 \text{ ft-lbs/min-HP})(7.481 \text{ gal/ft}^3)} \approx \frac{QH}{3956} \quad (223)$$

where $\gamma = \rho g$, and ρ is water density. In metric units:

$$\text{WHP} = \rho g QH = \frac{(1000 \text{ kg/m}^3)(9.81 \text{ m/s}^2)(\text{l/s})(\text{m})}{(1000 \text{ l/m}^3)(1000 \text{ W/kW})} = \frac{QH}{102} \quad (224)$$

where WHP is in kW; Q in lps; and H in meters of head

$$1 \text{ HP} = 0.746 \text{ kW} \quad (225)$$

- *Total Dynamic Head*, TDH, is defined as:

$$\text{TDH} = \Delta \text{Elev} + h_f + \frac{P}{\gamma} + \frac{V^2}{2g} \quad (226)$$

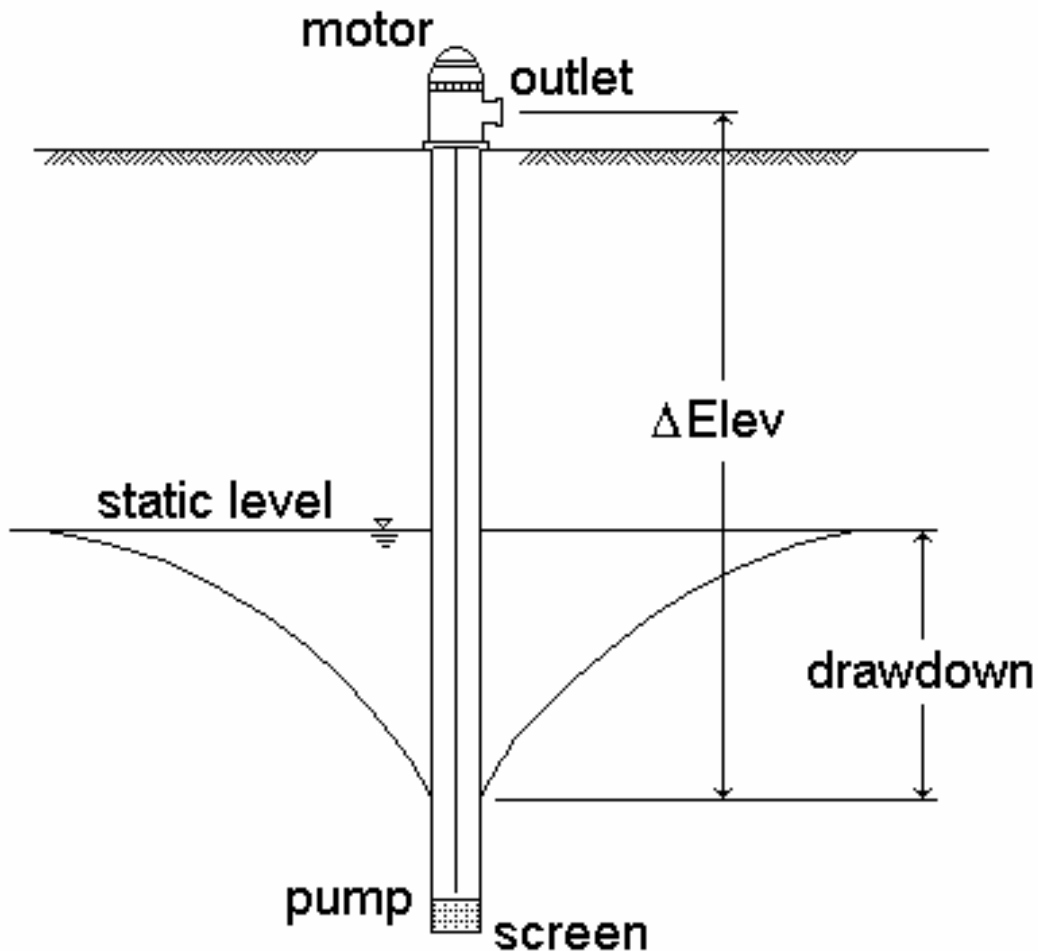
where the pressure, P , and velocity, V , are measured at the pump outlet, and h_f is the total friction loss from the entrance to the exit, including minor losses

- At zero flow, with the pump running,

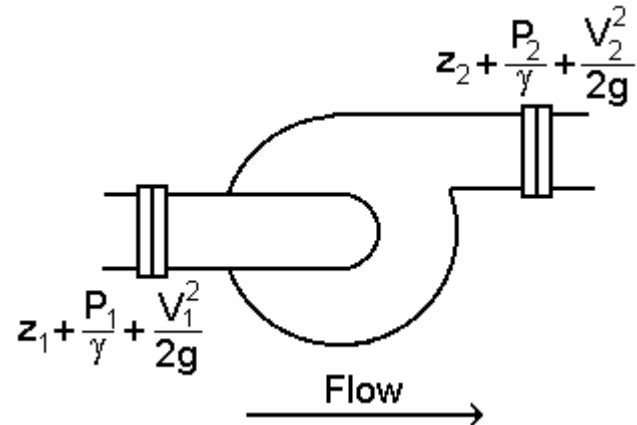
$$\text{TDH} = \Delta\text{Elev} + \frac{P}{\gamma} \quad (227)$$

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:

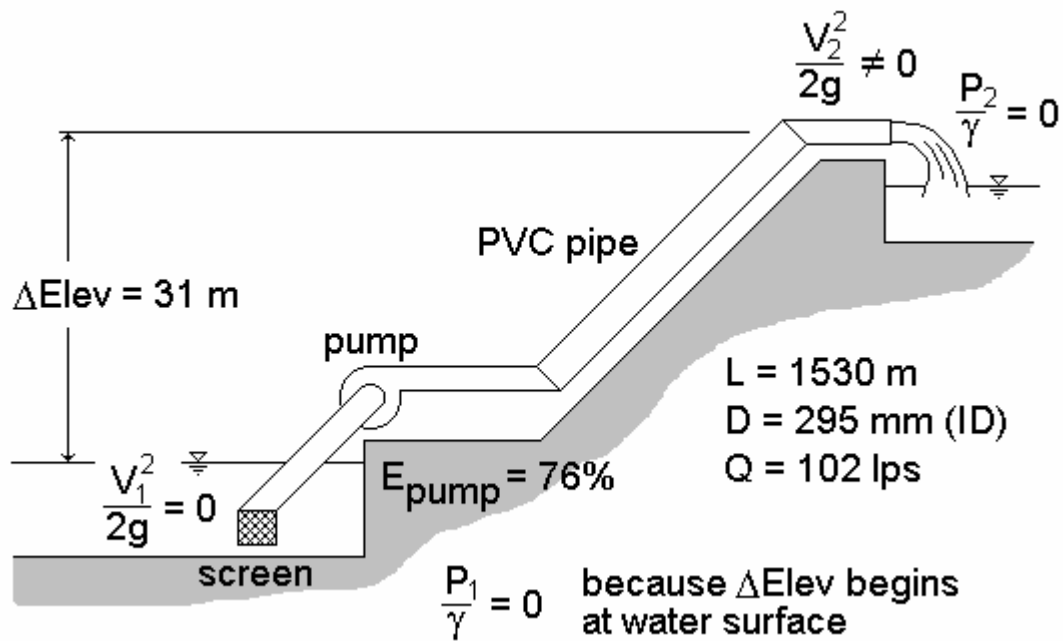


Consider a centrifugal pump:



II. Example TDH & WHP Calculation

- Determine TDH and WHP for a centrifugal pump discharging into the air...



Head loss due to friction:

$$h_f = h_{\text{screen}} + 3h_{\text{elbow}} + h_{\text{pipe}} \quad (228)$$

for PVC, $\varepsilon \approx 1.5(10)^{-6}$ m, relative roughness is:

$$\frac{\varepsilon}{D} = \frac{1.5(10)^{-6}}{0.295} = 0.0000051 \quad (229)$$

Average velocity,

$$V = \frac{Q}{A} = \frac{4(0.102)}{\pi(0.295)^2} = 1.49 \text{ m/s} \quad (230)$$

Reynolds number, for 10°C water:

$$N_R = \frac{VD}{\nu} = \frac{(1.49 \text{ m/s})(0.295 \text{ m})}{1.306(10)^{-6} \text{ m}^2/\text{s}} = 336,600 \quad (231)$$

- 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,

$$h_{\text{pipe}} = f \frac{L}{D} \frac{V^2}{2g} = 0.0141 \left(\frac{1,530}{0.295} \right) \frac{(1.49)^2}{2(9.81)} = 8.27 \text{ m} \quad (232)$$

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:

$$\nu = \left(83.9192T^2 + 20,707.5T + 551,173 \right)^{-1} \quad (233)$$

where T is in °C; and ν is in m²/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

$$h_{\text{elbow}} = K_r \frac{V^2}{2g} = (0.15) \frac{(1.49)^2}{2(9.81)} = (0.15)(0.11) = 0.017 \text{ m} \quad (234)$$

For the screen, assume a 0.2 m loss. Then, the total head loss is:

$$h_f = 0.2 + 3(0.017) + 8.27 = 8.5 \text{ m} \quad (235)$$

With the velocity head of 0.11 m, the total dynamic head is:

$$\text{TDH} = 31 + 8.5 + 0.11 \approx 40 \text{ m} \quad (236)$$

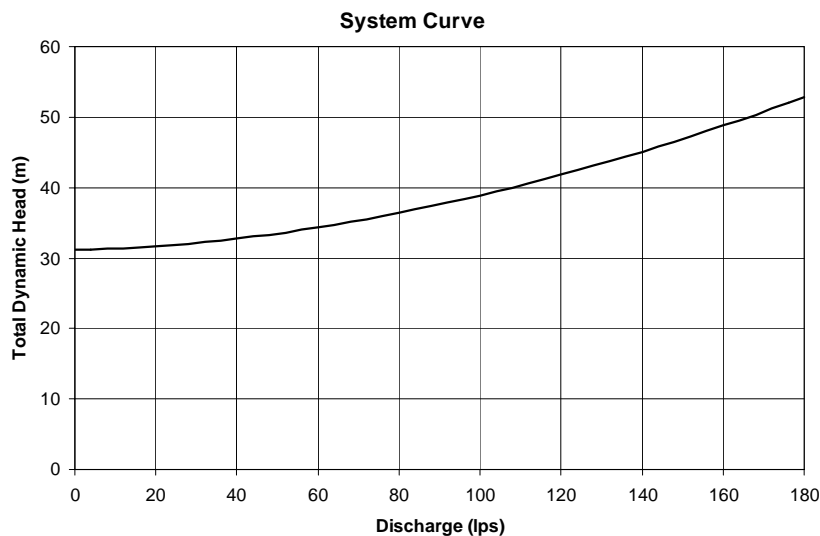
The water horsepower is:

$$\text{WHP} = \frac{QH}{102} = \frac{(102 \text{ lps})(40 \text{ m})}{102} = 40 \text{ kW (54 HP)} \quad (237)$$

The required brake horsepower is:

$$\text{BHP} = \frac{\text{WHP}}{E_{\text{pump}}} = \frac{40 \text{ kW}}{0.76} \approx 53 \text{ kW (71 HP)} \quad (238)$$

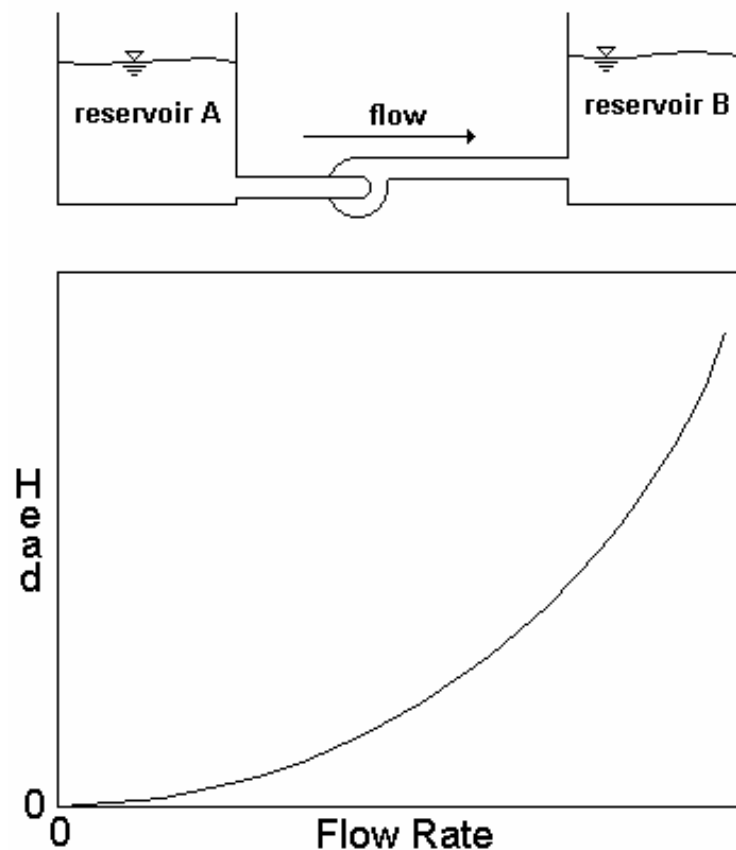
- This BHP value would be used to select a motor for this application
- These calculations give us one point on the system curve (Q and TDH)
- In this simple case, there would be only one system curve:



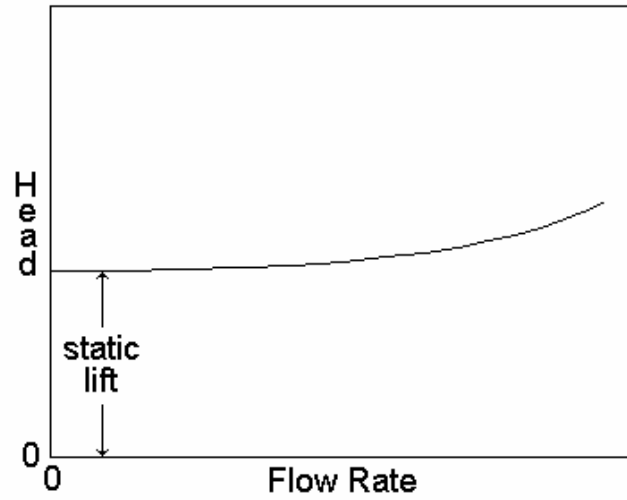
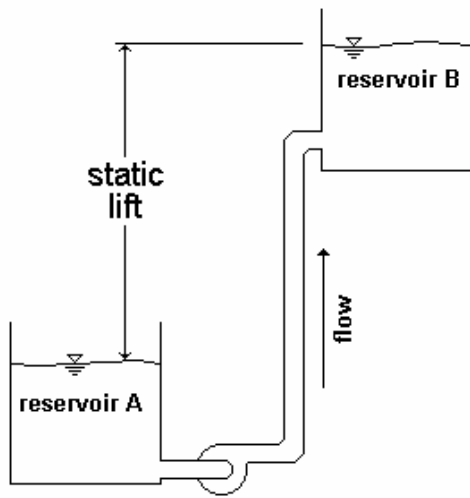
III. System Curves

- 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:

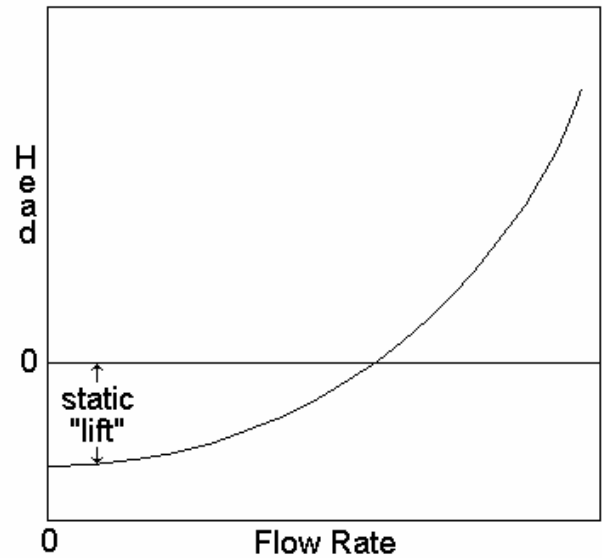
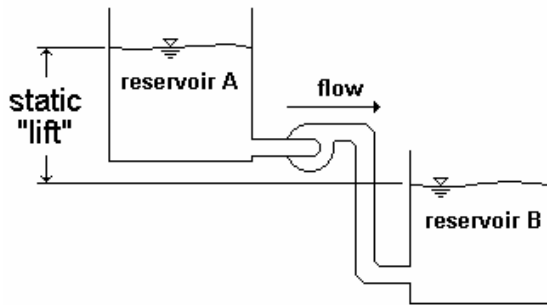
1. All Friction Loss and No Static Lift



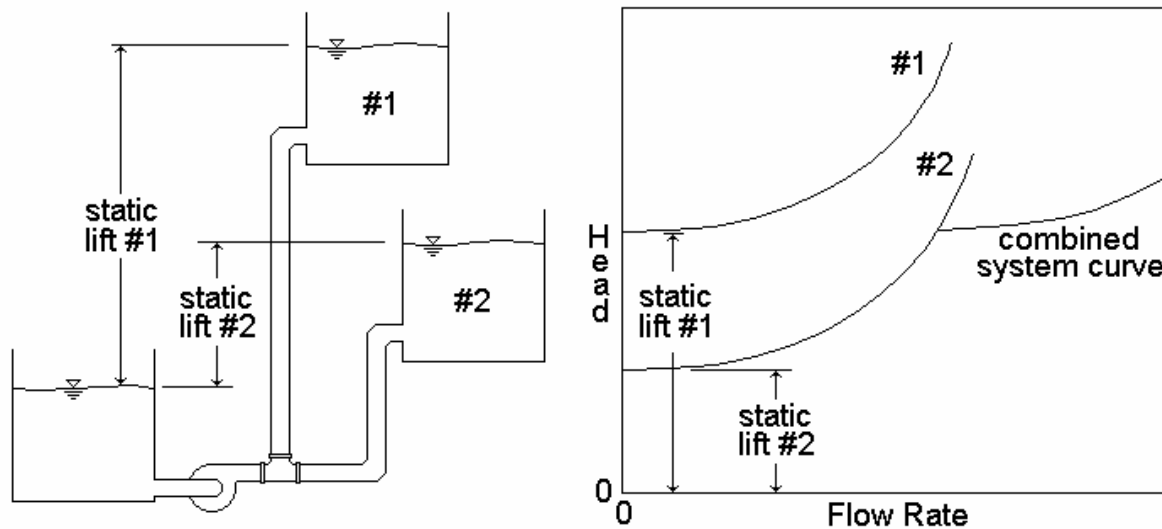
2. Mostly Static Lift, Little Friction Loss



3. Negative Static Lift

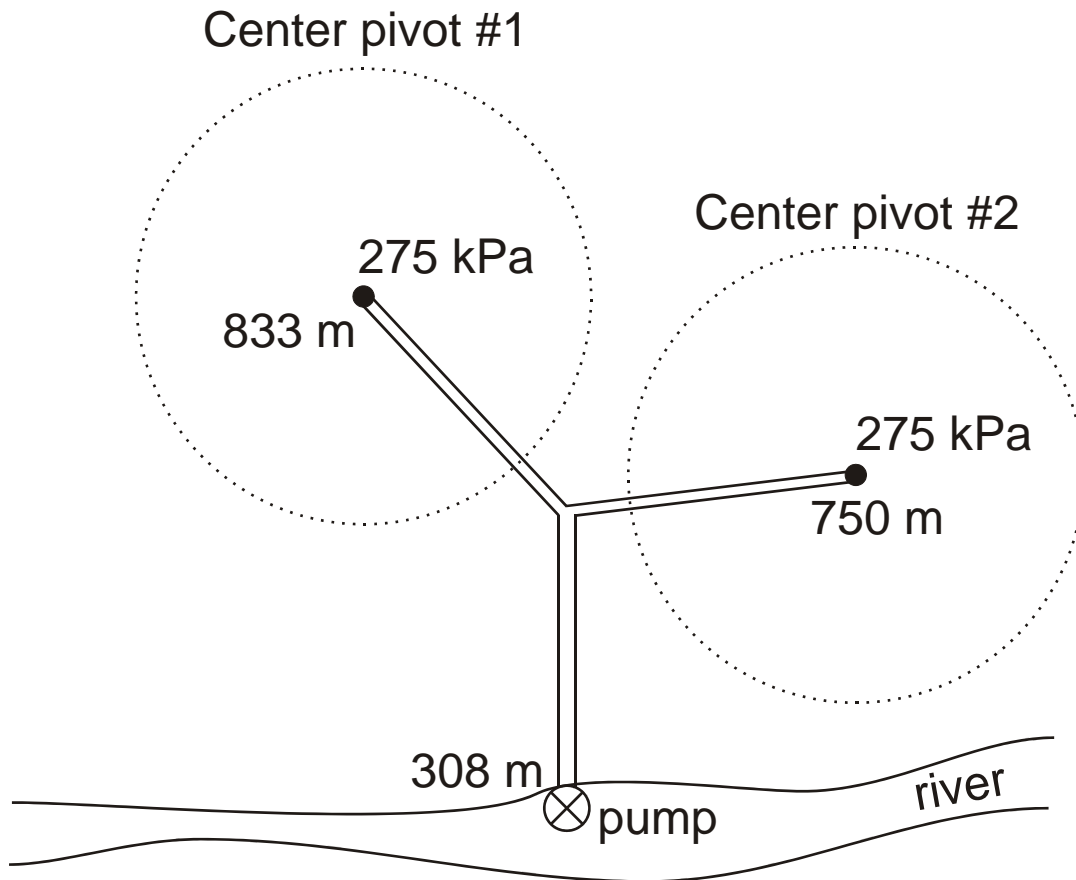


4. Two Different Static Lifts in a Branching Pipe



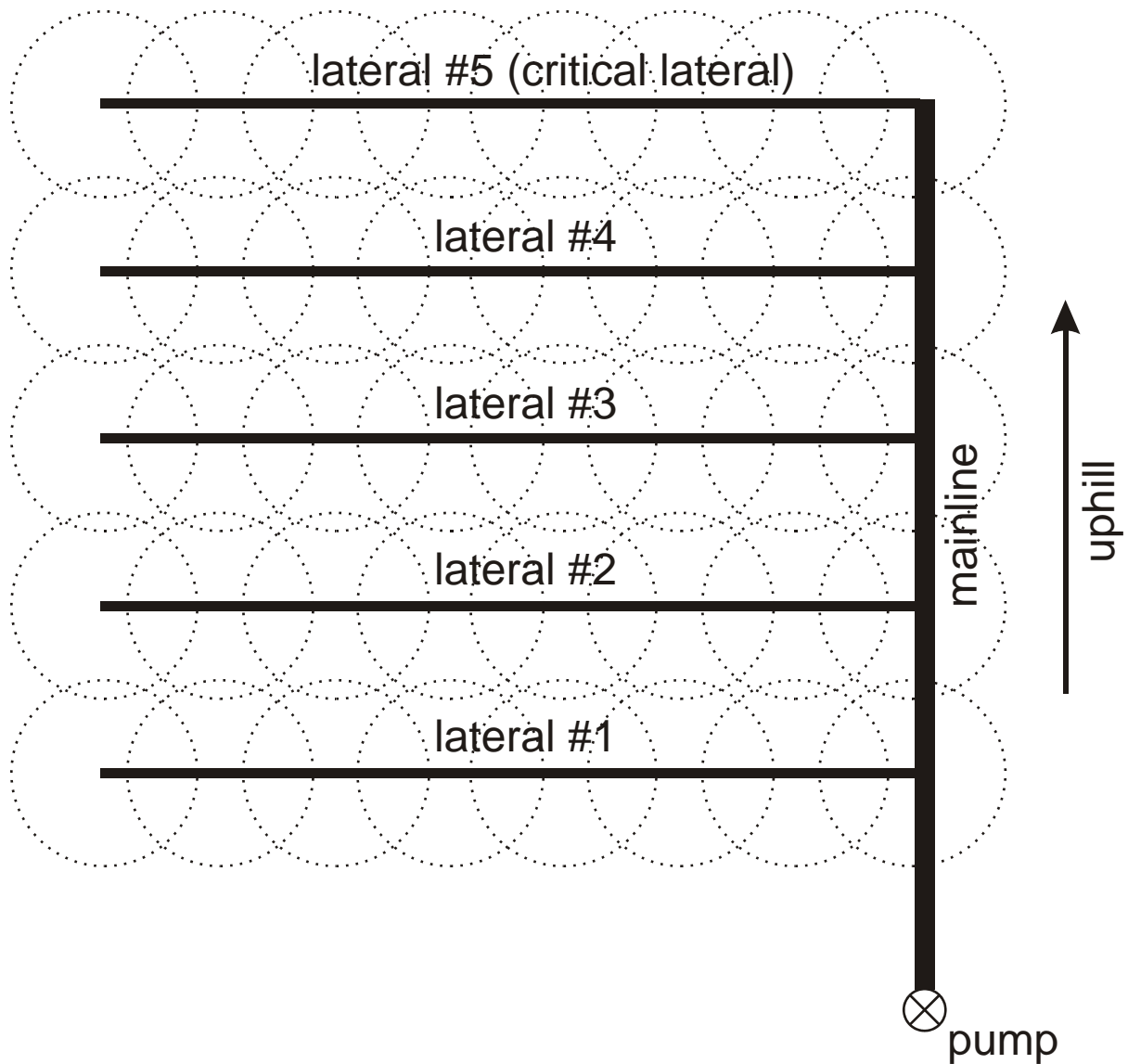
5. Two Center Pivots in a Branching Pipe Layout

- 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



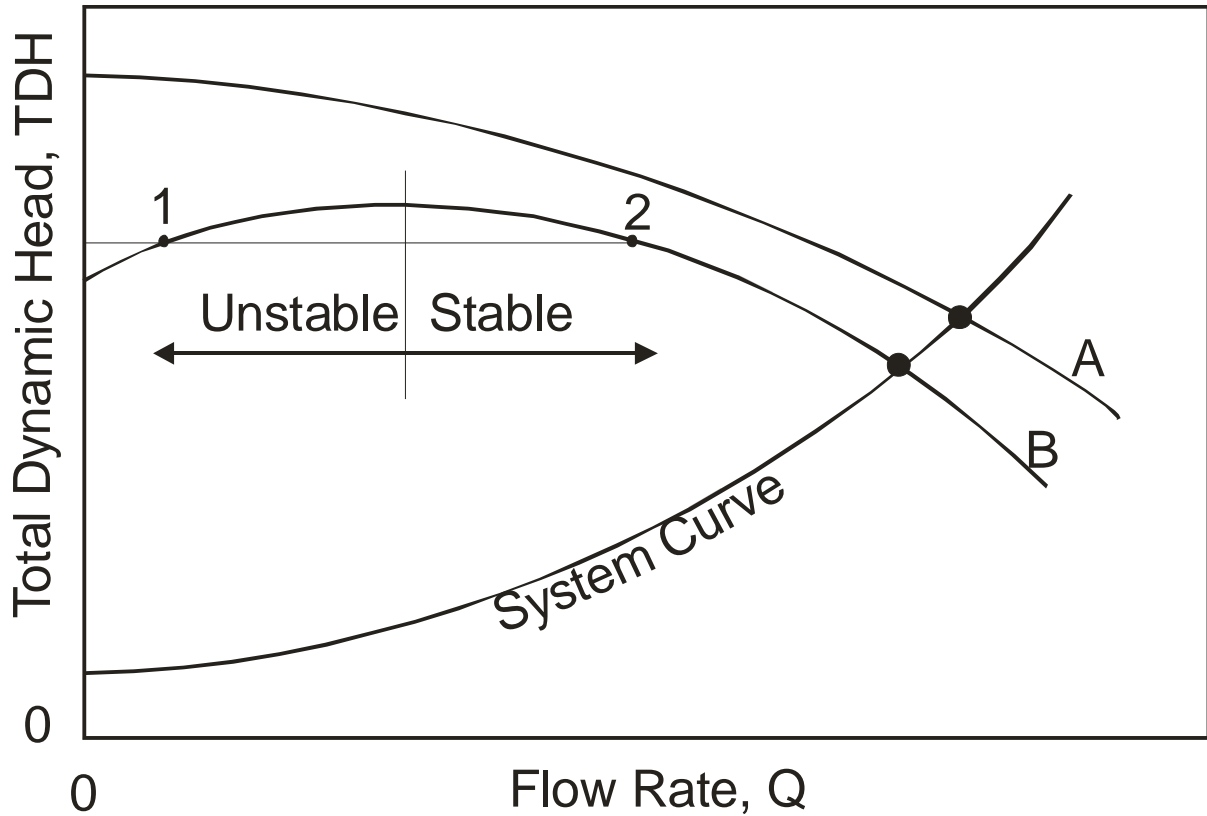
6. A Fixed Sprinkler System with Multiple Operating Laterals

- 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, Do Not Follow Multiple Branches 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:

$$\frac{Q_1}{Q_2} = \frac{N_1}{N_2} \quad \frac{H_1}{H_2} = \left(\frac{N_1}{N_2}\right)^2 \quad \frac{BHP_1}{BHP_2} = \left(\frac{N_1}{N_2}\right)^3 \quad (239)$$

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:

$$\frac{Q_1}{Q_2} = \frac{D_1}{D_2} \quad \frac{H_1}{H_2} = \left(\frac{D_1}{D_2} \right)^2 \quad \frac{BHP_1}{BHP_2} = \left(\frac{D_1}{D_2} \right)^3 \quad (240)$$

- 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:

$$\frac{H_1}{H_2} = \left(\frac{Q_1}{Q_2} \right)^2 \quad (241)$$

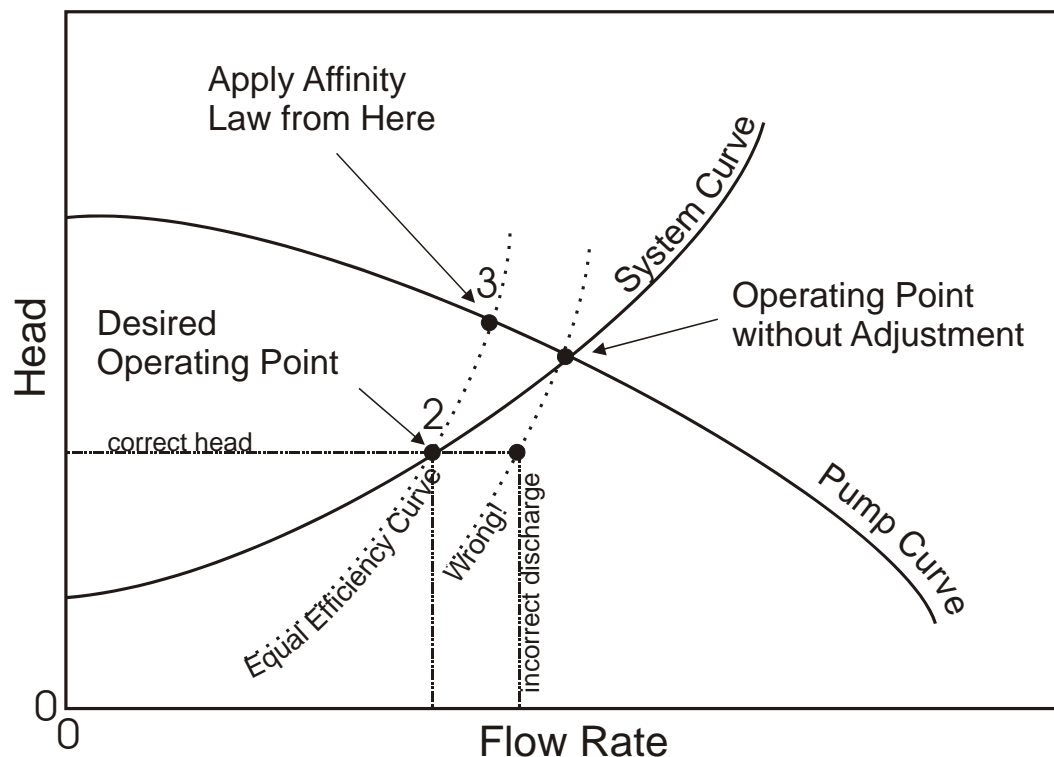
- If this relationship is plotted with the pump characteristic curve and the system curve, it is called the “equal efficiency curve”
- This is because there is typically only a small change in efficiency with a small change in pump speed
- Note that the “equal efficiency curve” will pass through the 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):

$$H_1 = H_2 \left(\frac{Q_1}{Q_2} \right)^2 \quad (242)$$

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):

$$N_{\text{new}} = N_{\text{old}} \left(\frac{Q_2}{Q_3} \right) \quad \text{or,} \quad N_{\text{new}} = N_{\text{old}} \sqrt{\frac{H_2}{H_3}} \quad (243)$$

6. Now your actual operating point will be the desired operating point (at least until the pump wears appreciably or other physical changes occur)
- You cannot directly apply any of the affinity laws 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

<i>Pump Type</i>	<i>Specific Speed</i>
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:

$$N_s = \frac{2\pi N \sqrt{Q}}{(gH)^{0.75}} \quad (244)$$

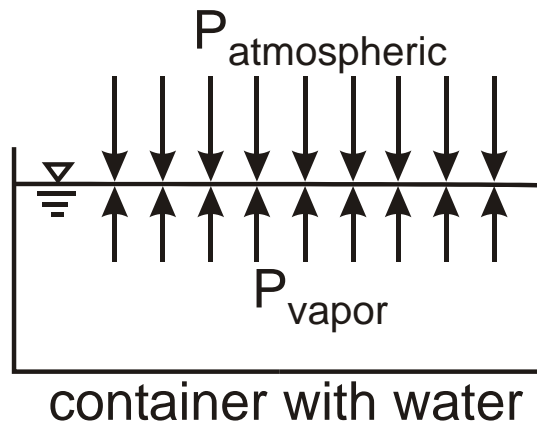
where the 2π is to convert revolutions (dimensional) to radians (dimensionless)

- Example: units could be $N = \text{rev/s}$; $Q = \text{m}^3/\text{s}$; $g = \text{m/s}^2$; and $H = \text{m}$
- 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

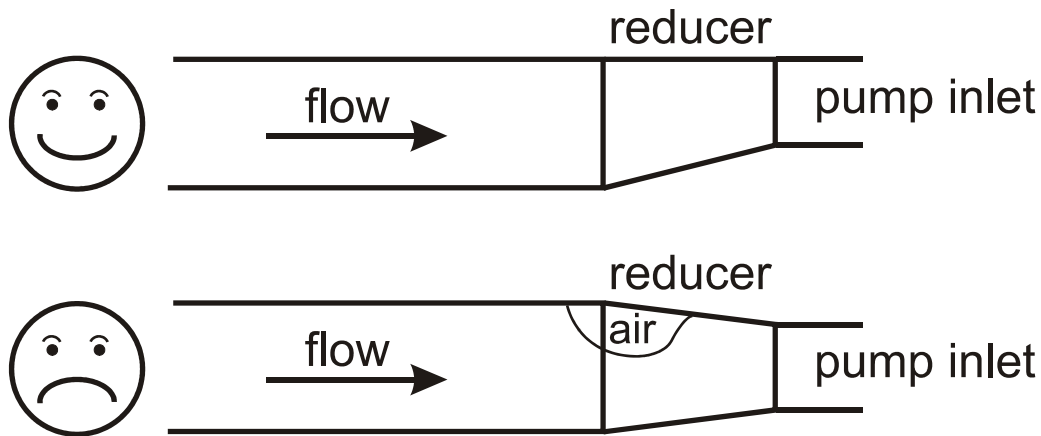
IV. Cavitation

- 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)



- 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 NPSH_a, 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...



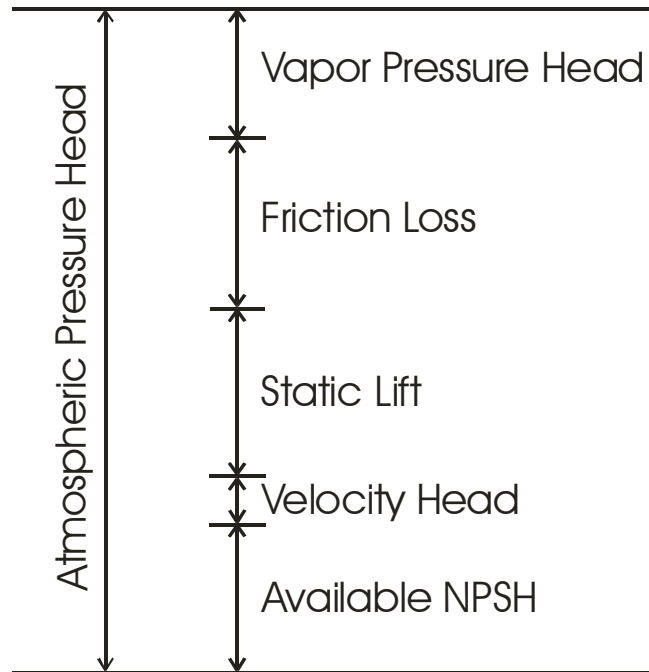
Required NPSH

- 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

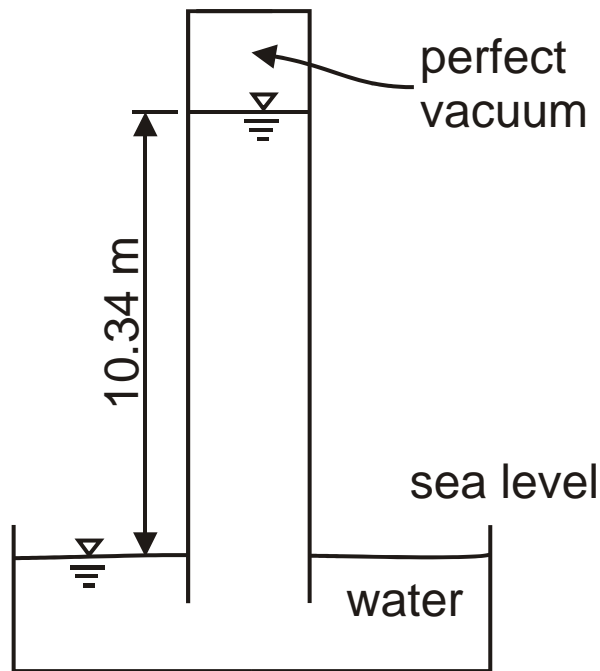
Available NPSH

- 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

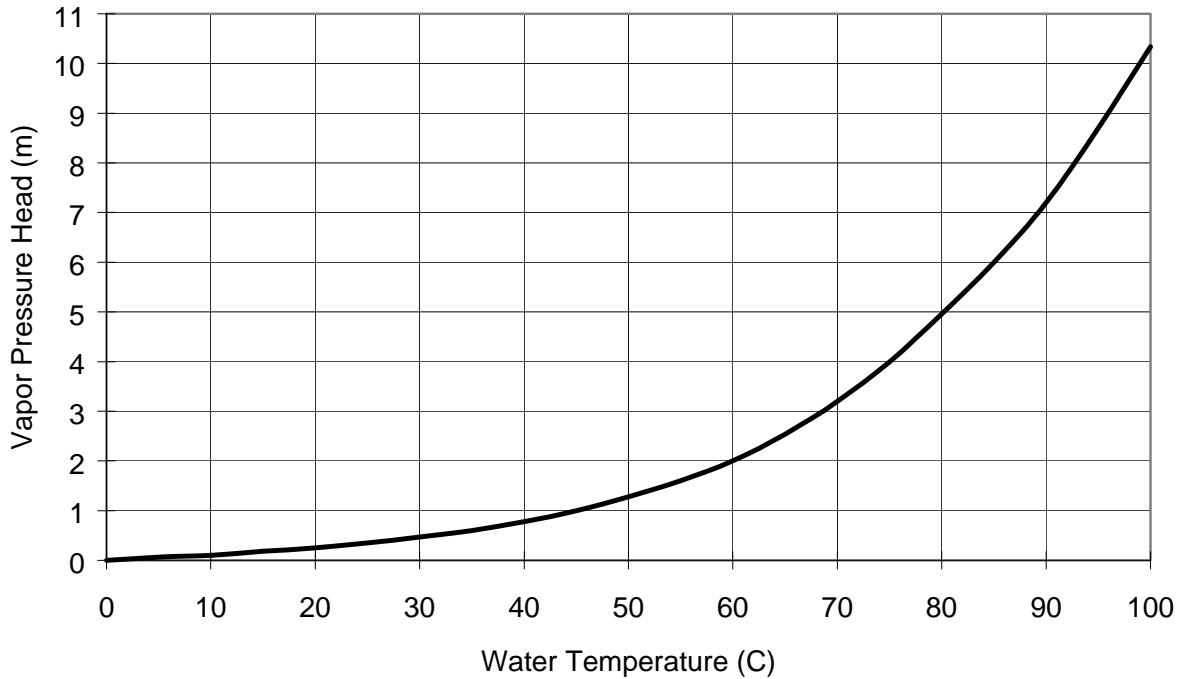
$$NPSH_a = h_{atm} - h_{vapor} - h_f - h_{lift} - \frac{v^2}{2g} \quad (245)$$



- 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:

$$h_{\text{vapor}} = 0.0623 \exp\left(\frac{17.27T}{T + 237.3}\right) \quad (246)$$

for h_{vapor} in m; and T in °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:

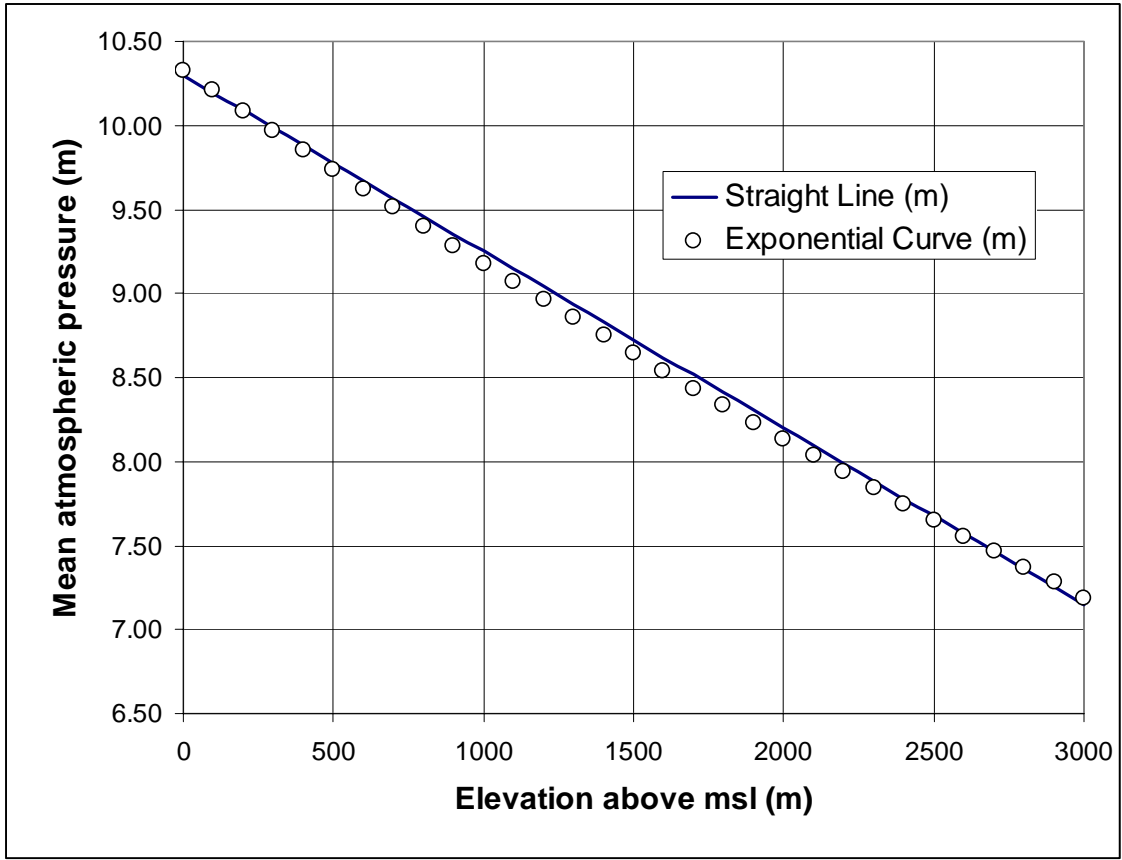
Straight line:

$$h_{\text{atm}} = 10.3 - 0.00105z \quad (247)$$

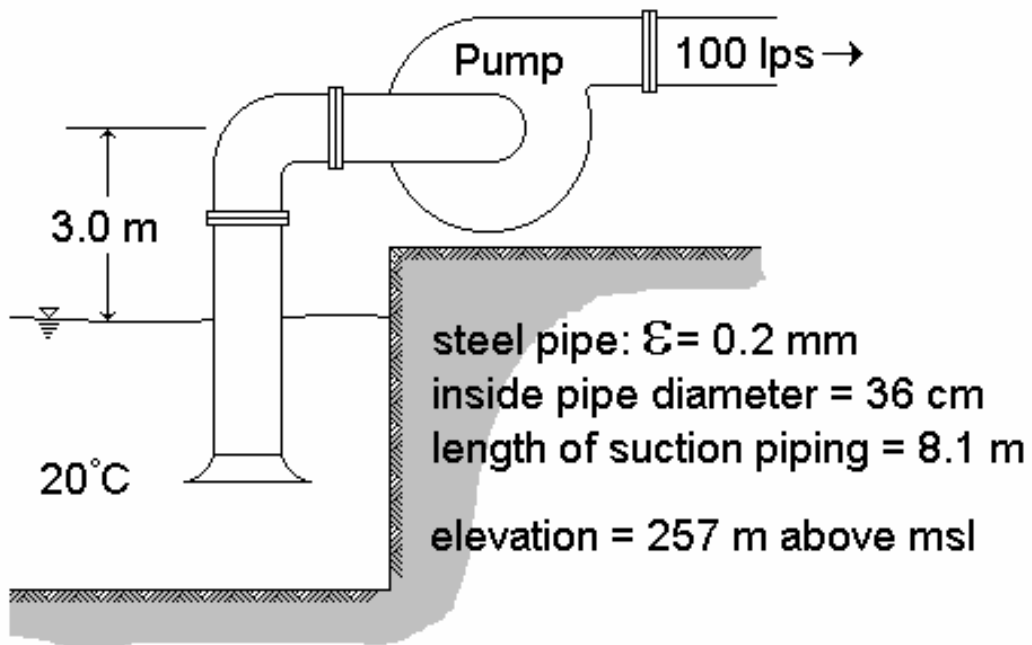
Exponential curve:

$$h_{\text{atm}} = 10.33 \left(\frac{293 - 0.0065z}{293}\right)^{5.26} \quad (248)$$

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

$$\frac{\varepsilon}{D} = \frac{0.2 \text{ mm}}{360 \text{ mm}} = 0.000556 \quad (249)$$

viscosity at 20°C, $\nu = 1.003(10)^{-6} \text{ m}^2/\text{s}$

flow velocity,

$$V = \frac{Q}{A} = \frac{0.100 \text{ m}^3/\text{s}}{\frac{\pi}{4}(0.36)^2} = 0.982 \text{ m/s} \quad (250)$$

Reynold's Number,

$$N_R = \frac{VD}{\nu} = \frac{(0.982)(0.36)}{1.003(10)^{-6}} = 353,000 \quad (251)$$

Darcy-Weisbach friction factor, $f = 0.0184$

velocity head,

$$\frac{V^2}{2g} = \frac{(0.982)^2}{2g} = 0.049 \text{ m} \quad (252)$$

head loss in suction pipe,

$$(h_f)_{\text{pipe}} = f \frac{L}{D} \frac{V^2}{2g} = 0.0184 \left(\frac{8.1}{0.36} \right) (0.049) = 0.0203 \text{ m} \quad (253)$$

local losses, for the bell-shaped entrance, $K_r = 0.04$; for the 90-deg elbow, $K_r = 0.14$. Then,

$$(h_f)_{\text{local}} = (0.04+0.14)(0.049) = 0.0088 \text{ m} \quad (254)$$

finally,

$$(h_f)_{\text{total}} = (h_f)_{\text{pipe}} + (h_f)_{\text{local}} = 0.0203 + 0.0088 = 0.0291 \text{ m} \quad (255)$$

2. Vapor Pressure

for water at 20°C, $h_{\text{vapor}} = 0.25 \text{ m}$

3. Atmospheric Pressure

at 257 m above msl, $h_{\text{atm}} = 10.1 \text{ m}$

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

$$\text{NPSH}_a = h_{\text{atm}} - h_{\text{vapor}} - (h_f)_{\text{total}} - h_{\text{lift}} - \frac{V^2}{2g} \quad (256)$$

$$\text{NPSH}_a = 10.1 - 0.25 - 0.0291 - 3.0 - 0.049 = 6.77 \text{ m}$$

VI. Relationship Between NPSHr and NPSHa

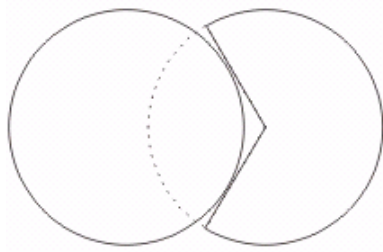
- If $\text{NPSH}_r < \text{NPSH}_a$, there should be no cavitation
- If $\text{NPSH}_r = \text{NPSH}_a$, cavitation is impending
- As the available NPSH drops below the required value, cavitation will become stronger, the pump efficiency will drop, and the flow rate will decrease
- At some point, the pump would “break suction” and the flow rate would go to zero (even with the pump still operating)

Lecture 12

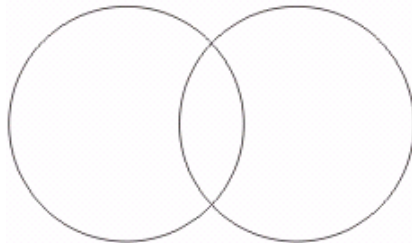
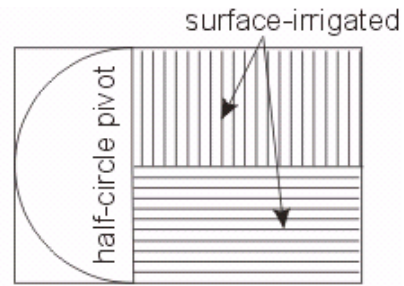
Center Pivot Design & Operation

I. Introduction and General Comments

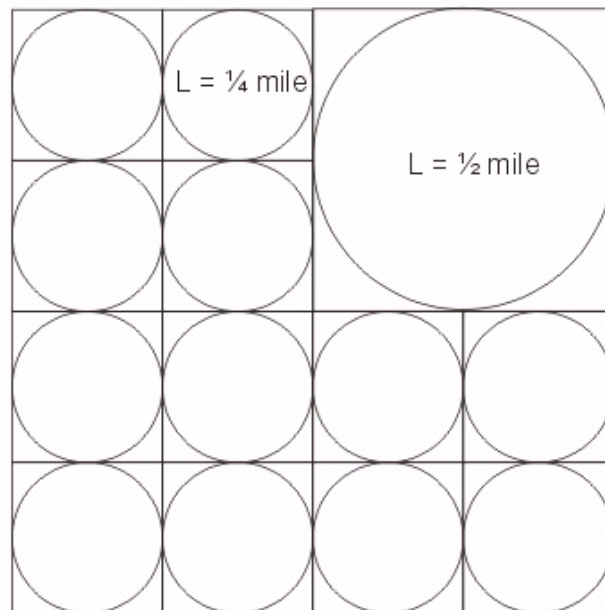
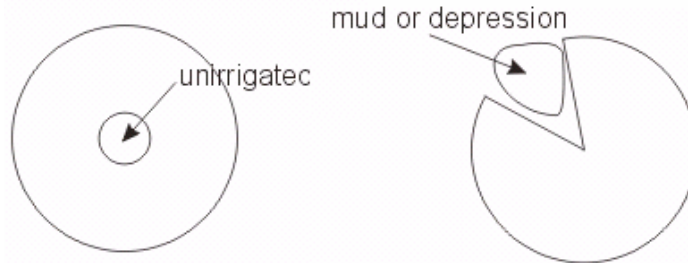
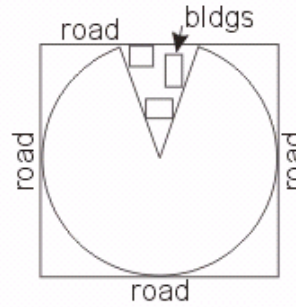
- Center pivots are used on about half of the sprinkler-irrigated land in the USA
- Center pivots are also found in many other countries
- Typical lateral length is 1,320 ft (400 m), or ¼ mile
- The lateral is often about 10 ft above the ground
- Typically, 120 ft pipe span per tower (range: 90 to 250 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





- 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:

$$Q_s = K \frac{Ad}{fT} = \frac{R^2 d}{k_1 f T} = \frac{R^2 U_d k_f}{k_1 T E_{pa}} \quad (257)$$

where,

K is 2.78 for metric units and 453 for English units

k_1 is $(3,600 \text{ s/hr})/\pi = 1,146$ 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

$$d' = \frac{k_f U_d}{E_{pa}} = \frac{k_f U_d}{D E_{pa} R_e O_e} \quad (258)$$

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:

$$d' = \frac{0.9k_f U_d}{(1-LR)DE_{pa}R_eO_e} \quad (259)$$

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:

$$k'_f = (k_f - 1) \frac{(100 - PT')/PT'}{(100 - PT)/PT} + 1.0 \quad (260)$$

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

$$PT = \frac{T}{ET} \quad (261)$$

- 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:

$$a = \pi(r + 0.5)^2 - \pi(r - 0.5)^2 = 2\pi r \quad (262)$$

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:

$$\frac{q}{Q_s} = \frac{a}{A} = \frac{2\pi r}{\pi R^2} = \frac{2r}{R^2} \quad (263)$$

or,

$$q = \frac{2rQ_s}{R^2} \quad (264)$$

where q can be in units of lps per m, or gpm per ft

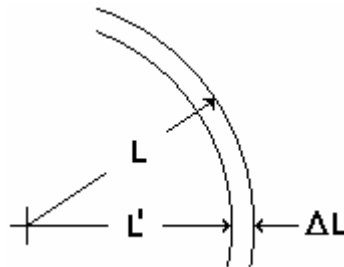
- This gives the amount of water which should be discharging from a specific unit length of lateral at a radial distance r from the pivot point
- The q value at the end of the lateral ($r = R$) per ft or m is:

$$q_{\text{end}} = \frac{2Q_s}{R} \quad (265)$$

- Use q to select the nozzle size, where $q_{\text{nozzle}} = q S_e$

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:

$$\frac{Q_b}{\pi L^2} \approx \frac{q_g}{\Delta L (2\pi L')} \quad (266)$$

or, perhaps more precisely,

$$\frac{Q_b}{\pi L^2} \approx \frac{q_g}{\Delta L (2\pi(L' + \Delta L/2))} \quad (267)$$

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:

$$q_g \cong \frac{2L'\Delta L}{L^2} Q_b; \text{ for } \Delta L < 0.03L \quad (268)$$

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:

$$AR = \frac{k_3 2r Q_s R_e O_e}{R^2 w} = \frac{2\pi r k_f d R_e O_e}{60 f T w} = \frac{2\pi r U'_d R_e O_e}{60 T w} \quad (269)$$

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

$$U'_d = \frac{k_f d}{f} = \frac{k_f (U_d - P_e)}{DE_{pa}} \quad (270)$$

- 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:

$$AR_{r=R} = \frac{2\pi R U'_d R_e O_e}{60 T w} \quad (271)$$

again, where a rectangular application pattern is assumed

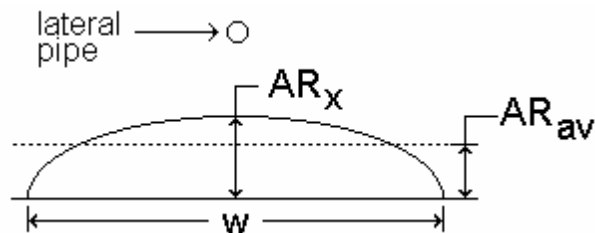
VII. Application Rate with an Elliptical Pattern

- If the application pattern perpendicular to the lateral were elliptical in shape:

$$AR_x = \frac{4}{\pi} \left(\frac{2k_3 r Q_s R_e O_e}{R^2 w} \right) = \frac{4}{\pi} \left(\frac{2\pi r U'_d R_e O_e}{60 T w} \right) = \frac{r U'_d R_e O_e}{7.5 T w} \quad (272)$$

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:

$$AR_x = \frac{(400)(9)(0.95)(1.0)}{(7.5)(22)(30)} = 0.69 \text{ mm/min} \quad (273)$$

- 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 $30 \text{ m}/18 \text{ min} = 1.7 \text{ m/min}$ 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:

$$w_r = \frac{8rU_d}{60TAR_x DE_{pa}} \quad (274)$$

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:

$$AR_r \leq \frac{rU_d R_e O_e}{7.5T w_d} \quad (275)$$

- The term " $R_e O_e$ " 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:

$$(t_t)_r = \frac{4D_f}{\pi AR_r} \quad (276)$$

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' :

$$U_d' = \frac{k_f (U_d - P_e)}{DE_{pa}} = \frac{1.02(8 - 0)}{0.74} = 11.0 \text{ mm/day} \quad (277)$$

- Tangential speed of the pivot at distance “r” along the lateral:

$$S_r = \frac{2\pi r}{t} \quad (278)$$

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:

$$\tau = \frac{w_r}{S_r} \quad (279)$$

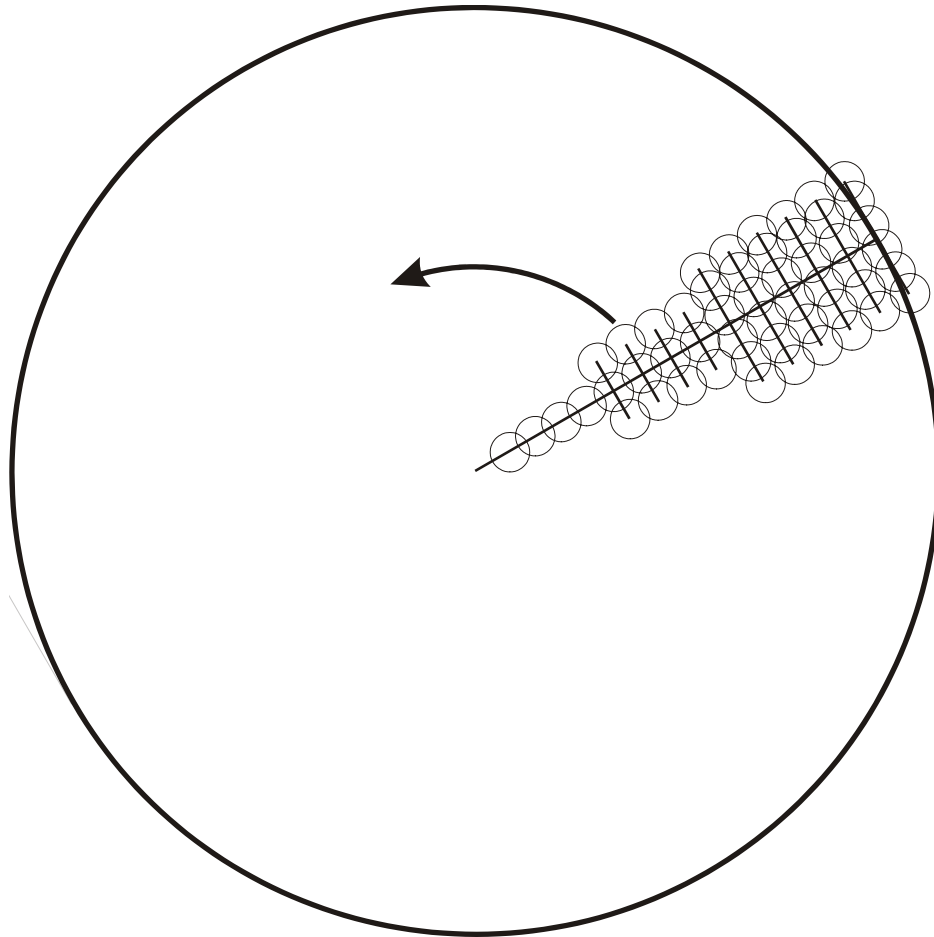
where τ is wetting time in minutes; and w_r is the minimum wetted width in m.

- 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)

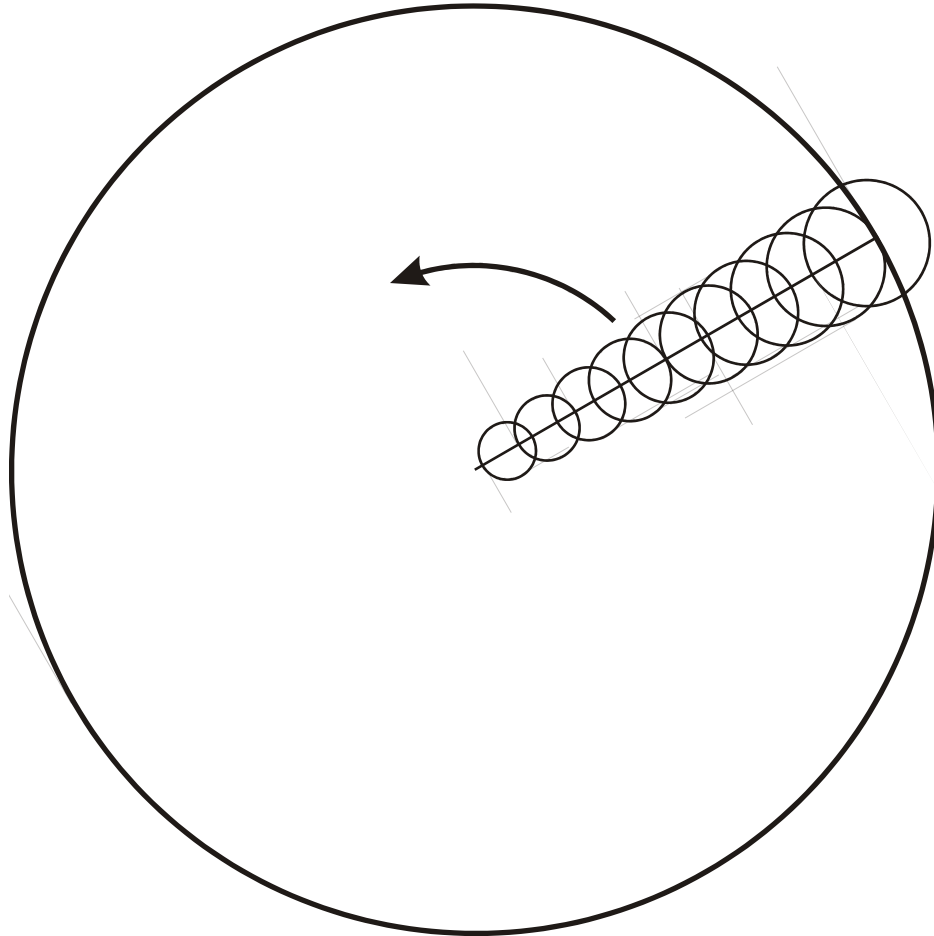


III. Sprinkler/Nozzle Configurations

- 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

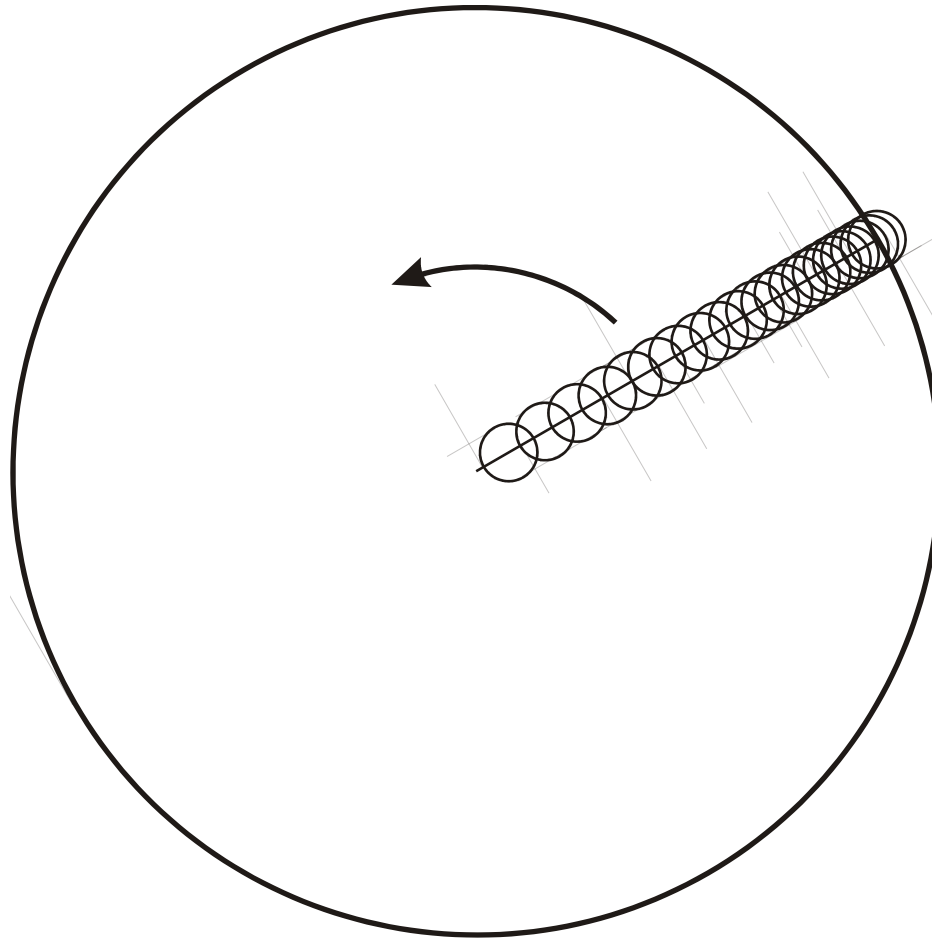


2. **Uniform Sprinkler Size** where the distance between sprinklers decreases with r

- 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):

$$q_r = r S_r \frac{2Q_s}{R^2} \quad (280)$$

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 :

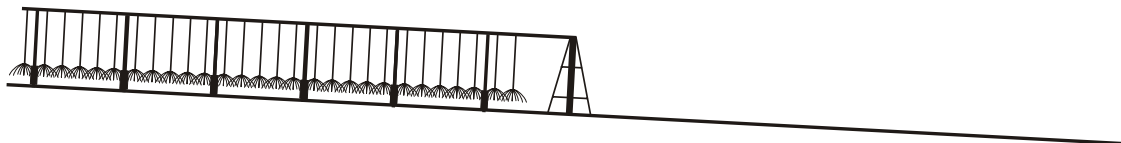
$$P_r = (P_a)_{\text{end}} + (h_{fr})_{\text{end}} + (\Delta H_e)_{\text{end-r}} \quad (281)$$

where $(h_{fr})_{\text{end}}$ is the friction loss from point r to the far (downstream) end of the lateral. Note that $(\Delta H_e)_{\text{end-r}}$ 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):

$$Q_g = \frac{\left(\frac{2S_r Q \left(L + \frac{S_r}{2} \right)}{R^2} \right)}{0.93} \quad (282)$$

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:

$$Q_g \approx Q \left(1 - \frac{L^2}{R^2} \right) \quad (283)$$

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:

$$\begin{aligned} \frac{2(R-L)Q[L + 0.5(R-L)]}{0.93R^2} &= \\ \frac{2QL(R-L) + Q(R-L)^2}{0.93R^2} & \quad (284) \\ &= \frac{Q}{0.93} \left(1 - \frac{L^2}{R^2} \right) \end{aligned}$$

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: $S_{\text{corner}} = S[Q/(Q+Q_g)]$, 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:

$$Q_r = Q_s \left(1 - \frac{r^2}{R^2} \right) \quad (285)$$

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:

$$h_f = k_h F_p L \left(\frac{Q}{C} \right)^{1.852} D^{-4.87} \quad (286)$$

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:

$$h_f = k_h F_p R \left(\frac{Q}{C} \right)^{1.852} D^{-4.87} - k_h F_g (R - L) \left(\frac{Q_g}{C} \right)^{1.852} D^{-4.87} \quad (287)$$

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

$$H_l = H_a + h_f + \Delta H_e + H_r + H_{\text{minor}} \quad (288)$$

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 $JF_p L$ 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

$$H_l = H_{\text{end}} + \sum_{i=0}^n \left[(h_{f_i} + \Delta H_{e_i}) + H_r + H_{\text{minor}} \right] \quad (289)$$

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_{e_i} 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:

$$h_{fi} = \frac{J_i \Delta L_i}{100} \quad (290)$$

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):

$$H_{\text{ew}} = \int_{r=0}^L \int_{\alpha}^{2\pi} \frac{r(H_e)_{r,\alpha}}{2\pi L^2} dr d\alpha \quad (294)$$

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):

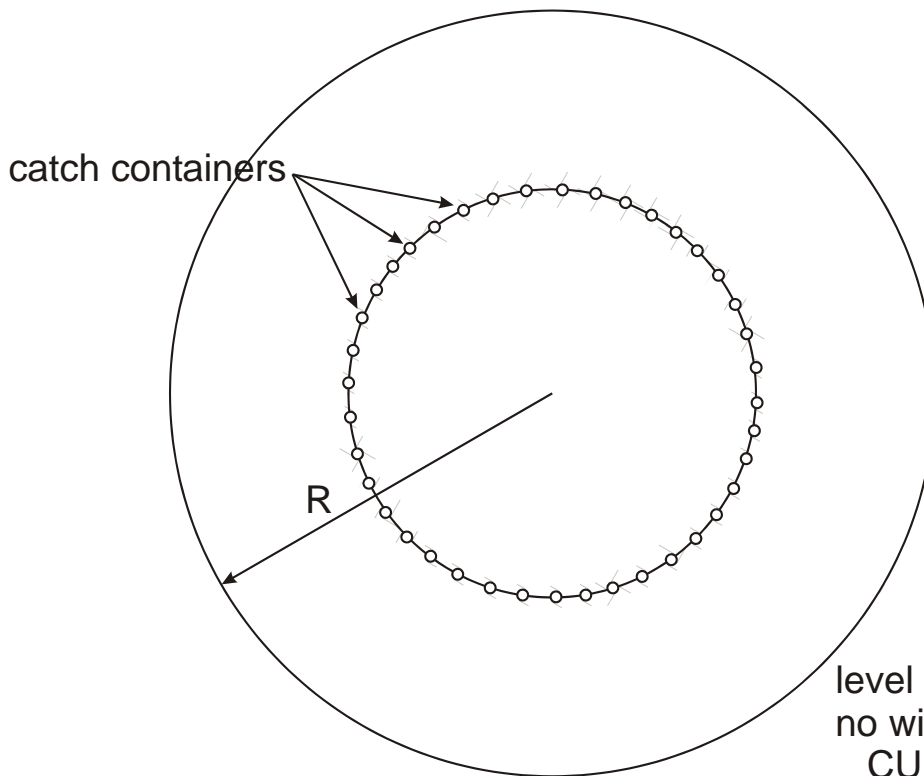
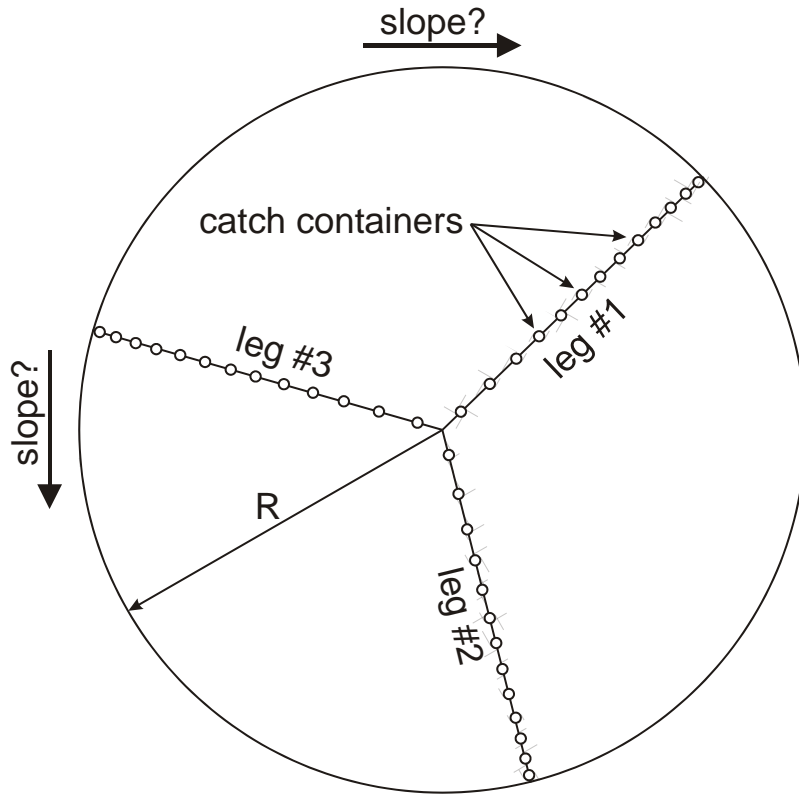
$$CU = 100 \left(1.0 - \frac{\sum_{i=1}^n \left(r_i \left| d_i - \frac{\sum_{i=1}^n (d_i r_i)}{\sum_{i=1}^n r_i} \right| \right)}{\sum_{i=1}^n (d_i r_i)} \right) \quad (295)$$

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:

$$\sum_{i=1}^n r_i \quad \text{and,} \quad \sum_{i=1}^n (d_i r_i) \quad (296)$$

- 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:



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
- Don't 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
- Don't 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):
- 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
		<i>smallest</i>		
		<i>largest</i>		
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
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
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

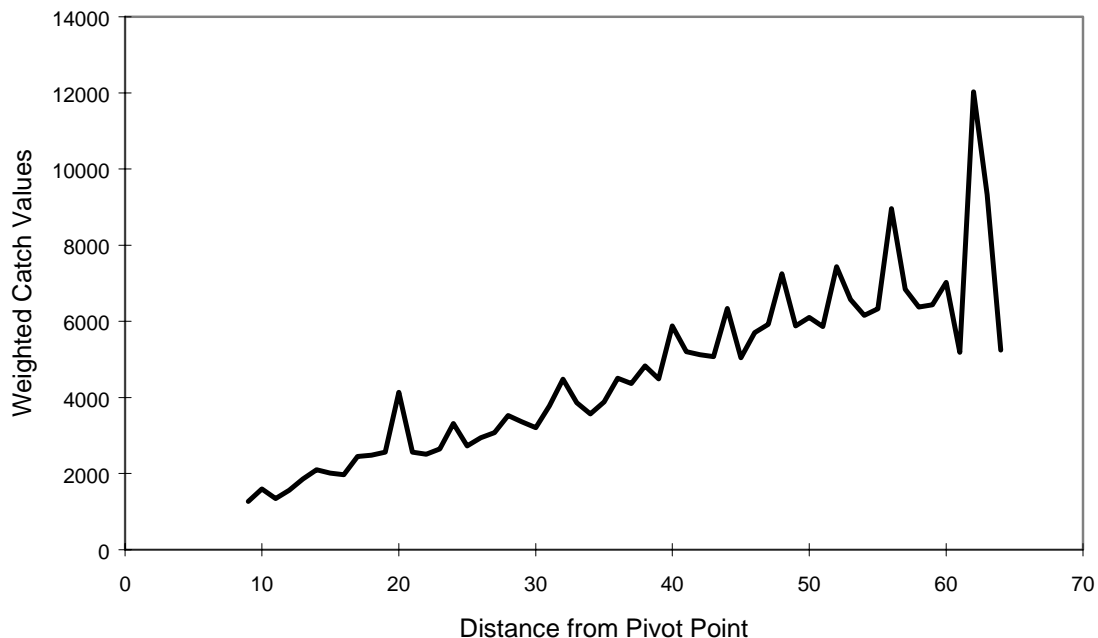
1/4 area (11,055)

1/2 area (22,110)

- 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:

$$DU \cong 100 \left(\frac{1.491}{1.985} \right) \cong 75\% \quad (297)$$

- 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:

Pros

- Low pressure (energy) requirement
- Totally automated system
- More frequent irrigations than hose-fed, since no one needs to be available to move the hose

Cons

- 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

Pros

- 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

Cons

- 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)

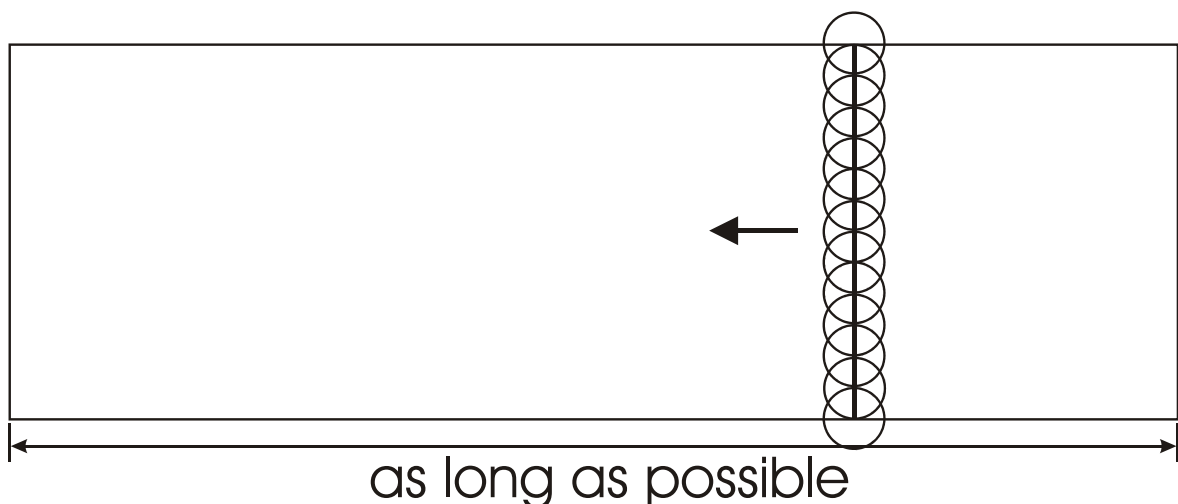
II. System Costs

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 minimum time to finish an irrigation, for a given gross application depth, whereby a maximum 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).

$$H_l = \frac{P_a}{\gamma} + \frac{3}{4}h_f + H_r + \frac{1}{2}\Delta H_e + (h_f)_{\text{minor}} + (h_f)_{\text{hose}} \quad (298)$$

- Or, if using flow control nozzles, with a minimum pressure required at the end (assuming the minimum pressure occurs at the end):

$$H_l = \frac{P_{\text{end}}}{\gamma} + h_f + H_r + \Delta H_e + (h_f)_{\text{minor}} + (h_f)_{\text{hose}} \quad (299)$$

where H_r is the height of the lateral or spray boom above the ground; and, $(h_f)_{\text{minor}}$ are the hydrant coupler and tower connection losses.

- The parameter $(h_f)_{\text{hose}}$ is the loss in the flexible hose connection on a hose-fed system
- Note that $(h_f)_{\text{hose}}$ may be a major loss, since the hose diameter is usually less than 5" or 6".

$$h_f = k_h FL \left(\frac{Q}{C} \right)^{1.852} D^{-4.87} \quad (300)$$

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 ($d_x \leq MAD \cdot Z \cdot W_a$). Note that the maximum application depth may be less than $MAD \cdot Z \cdot W_a$ 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 $(D_f)_{\max}$:

$$d_f = (i/10)(D_f)_{\max} \text{ where } i = 1 \text{ to } 10$$

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):

$$AR_x = \frac{\left(1 - \frac{(SF)(AR_x)}{k}\right) \left(\frac{1}{k^{n+1}} (n+1)^{\frac{n}{n+1}} (D - SS - c)^{\frac{n}{n+1}}\right)}{\sqrt{1.05 - 1.6 \left(\frac{\pi}{2}\right)^2 \left(\frac{D}{d_f} - 0.5\right)^2}} \quad (301)$$

where,

$$D = \frac{\left(\left(1.05 AR_x^2 - 1.6 AR_x^2 \left(\frac{\pi}{2}\right)^2 \left(\frac{D}{d_f} - 0.5\right)^2 \right)^{-0.5} \left(-1.6 AR_x^2 \left(\frac{\pi}{2}\right)^2 \left(\frac{D}{d_f} - 0.5\right) \right) \right)^{-n-1}}{d_f n \left(1 - \frac{(SF)(AR_x)}{k}\right) \left((n+1)^{\frac{-1}{n+1}}\right) \left(\frac{1}{k^{n+1}}\right)} + SS + c \quad (302)$$

and AR_x is the peak application per pass (mm/min); D is the applied depth at time $t = \int (AR) dt$ (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:

$$t_i = \frac{d_f}{\frac{\pi}{4}(AR_x)} \quad (303)$$

7. Compute the speed of the system for the required t_i :

$$S = w/t_i \text{ (m/min)} \quad (w \text{ is for a specific nozzle type})$$

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:

$$t_i = \frac{w}{S_{\max}} \quad (304)$$

$$AR_x = \frac{4d_f}{\pi t_i} \quad (305)$$

Thus,

$$S = S_{\max} \quad (306)$$

8. Calculate maximum field length, X :

8(a). For irrigation in one direction, only (dry return, or deadheading):

$$X = \frac{60fT - 2t_{\text{reset}}}{\left(\frac{1}{S_{\text{wet}}} + \frac{1}{S_{\text{dry}}} + \frac{t_{\text{hose}}}{100} \right)} \quad (307)$$

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)

$$\text{labor} = \frac{2t_{\text{reset}} + 0.01X(t_{\text{hose}} + 2t_{\text{super}})}{60f} \quad (308)$$

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):

$$X = \frac{60fT - 2t_{\text{reset}}}{2\left(\frac{1}{S_{\text{wet}}} + \frac{t_{\text{hose}}}{100}\right)} \quad (309)$$

and labor is calculated as above in 8(a)

9. Calculate the irrigated area:

$$\text{Area}_{\text{max}} = \frac{XL}{10,000} \quad (310)$$

where Area_{max} is in ha; and L is the total lateral length (m)

10. Labor per hectare per irrigation, L_{ha} :

$$L_{\text{ha}} = \frac{\text{labor}}{\text{Area}_{\text{max}}} \quad (311)$$

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 $f \uparrow$, $\theta \downarrow$, so that the Kostikov coefficients will change. Therefore, the procedure (and field ring infiltration tests) should be repeated using coefficients which represent the Kostikov 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 width of 10 m (33 ft)
- The soil infiltration characteristics are defined for the Kostiakov-Lewis equation as:

$$Z = 5.43\tau^{0.49} \quad (314)$$

with Z in mm of cumulative infiltrated depth; and τ is intake opportunity time in minutes. Other design parameters:

$$U_d = 7.7 \text{ mm/day}$$

$$\text{MAD} = 50\%$$

$$Z = 0.9 \text{ m}$$

$$W_a = 125 \text{ mm/m}$$

$$O_e = 1.00$$

$$R_e = 0.94$$

$$E_{pa} = 85\%$$

- 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 $w = 10 \text{ m}$
- 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 [$d_x = \text{MAD}(z)(W_a)$, or less]

$$d_x = (0.5)(0.9)(125) = 56 \text{ mm} \quad (315)$$

$$f' = \frac{d_x}{U_d} = \frac{56}{7.7} = 7.3 \Rightarrow f' = 7 \text{ days} \quad (316)$$

2. Net and gross application depths:

$$d_n = f U_d = (7)(7.7) = 54 \text{ mm} \quad (317)$$

$$d = \frac{d_n}{E_{pa}} = \frac{54}{0.85} = 64 \text{ mm} \quad (318)$$

3. Infiltrated depth at each irrigation:

$$D_f = d R_e = (64)(0.94) = 60 \text{ mm} \quad (319)$$

4. For a series of 10 infiltration values, calculate d_f , beginning with $d_f = D_f / 10$:

$$d_f = D_f \left(\frac{i}{10} \right) \quad (320)$$

where $i = 1$ to 10. For this example, let $i = 4$ and, $d_f = (0.4)(60 \text{ mm}) = 24 \text{ mm}$. Then,

$$f' = \frac{d_f D E_{pa}}{U_d} = \frac{(24)(0.85/0.94)}{7.7} = 2.8 \text{ days} \quad (321)$$

Assume no days off (no down time during the peak use period)

$$f = f' - \text{days off} = 2.8 - 0 = 2.8 \text{ days} \quad (322)$$

5. Determine the maximum AR_x for the particular d_f depth:

From Eq. 282:

$$AR_x = 0.97 \text{ mm/min}$$

$$AR_x \text{ reaching the soil surface} = 0.97 (R_e) = 0.91 \text{ mm/min}$$

6. Compute the total wetting time, t_i , in minutes:

$$t_i = \frac{4 d_f}{\pi AR_x} = \frac{4(24)}{\pi(0.91)} = 34 \text{ min} \quad (323)$$

7. Compute the speed of the system for the required t_i :

$$S = \frac{w}{t_i} = \frac{10}{34} = 0.3 \text{ m/min} \quad (324)$$

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):

$$\begin{aligned} X &= \frac{60fT - 2t_{\text{reset}}}{\left(\frac{1}{S_{\text{wet}}} + \frac{1}{S_{\text{dry}}} + \frac{t_{\text{hose}}}{100} \right)} = \\ &= \frac{60(2.8)(22) - 2(30)}{\left(\frac{1}{0.3} + \frac{1}{3.5} + \frac{10}{100} \right)} = 970 \text{ m} \end{aligned} \quad (325)$$

and, the labor requirements are:

$$\begin{aligned} \frac{2t_{\text{reset}} + 0.01(t_{\text{hose}} + 2t_{\text{super}})X}{60f} &= \\ &= \frac{2(30) + 0.01[10 + 2(5)][970]}{60(2.8)} = 1.5 \text{ hrs/day} \end{aligned} \quad (326)$$

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:

$$\text{Area}_{\max} = XL/10000 = 970(400)/10000 = 38.8 \text{ ha}$$

which is only about half of the actual field area!

10. Labor per ha per irrigation, L/ha:

$$L/\text{ha} = (\text{labor/area})_{\max} = 1.5/38.8 = 0.039 \text{ hr/ha/day}$$

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 \ll 64 ha, which is the size of the field, (970 m \ll 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 both 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:

$$Q_s = \frac{\pi AR_x wL}{4k_3R_e} = \frac{\pi(0.91)(10)(400)}{4(60)(0.94)} = 51 \text{ lps (809 gpm)} \quad (327)$$

alternatively,

$$Q_s = \frac{d_f wL}{t_i k_3 R_e} = \frac{(24)(10)(400)}{(33.6)(60)(0.94)} = 51 \text{ lps (809 gpm)} \quad (328)$$

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} ,

$$Q_s = 2.78 \frac{Ad}{fT} = 2.78 \frac{(38.3)(24)}{(2.8)(22)(0.94)} = 44 \text{ lps (700 gpm)} \quad (16)$$

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

Advantages

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

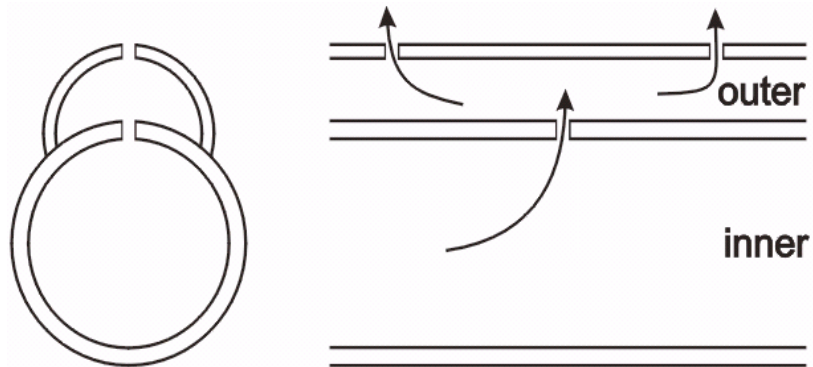
- Emitters are the typical water application device in many systems
- Can be “on-line” (usually with barb) or “in-line” (inserted into tubing by the tubing manufacturer)
- Various approaches are used to provide energy dissipation: long-path (“spaghetti tubing”), tortuous path (labyrinth), orifice, vortex,
- 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

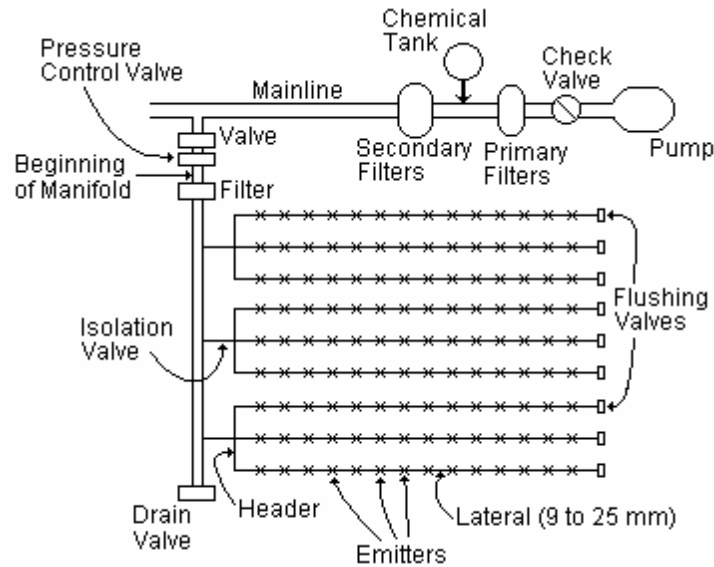


V. Types of Pipe Materials

- 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 $\frac{1}{2}$ corresponds to orifice flow, which is how some, but not all, emitters are designed
- The general emitter equation is:

$$q = K_d H^x \quad (329)$$

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 $\frac{1}{2}$, assuming the pressure head is fully converted to velocity head
- Pressure compensating emitters have $x \leq \frac{1}{2}$. 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:

$$EU' = 100 \left(\frac{q_n'}{q_a} \right) \quad (330)$$

where EU' is the field test emission uniformity; q_n' is the average discharge of the low $\frac{1}{4}$ 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

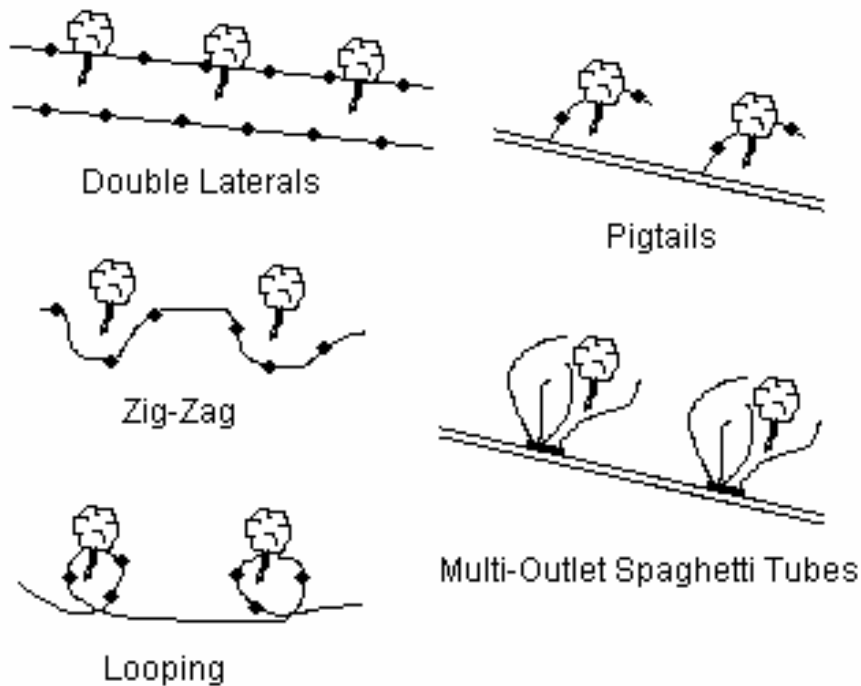
- Emitters of the same type and manufacture have variations in discharge (at the same operating pressure) due to small differences from manufacturing tolerances. Some variation is allowed in the interest of cost savings.
- The *manufacturer's coefficient of variation* is defined as:

$$v = \frac{s}{q_a} \quad (331)$$

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

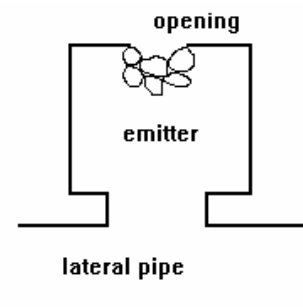
I. Introduction

- 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

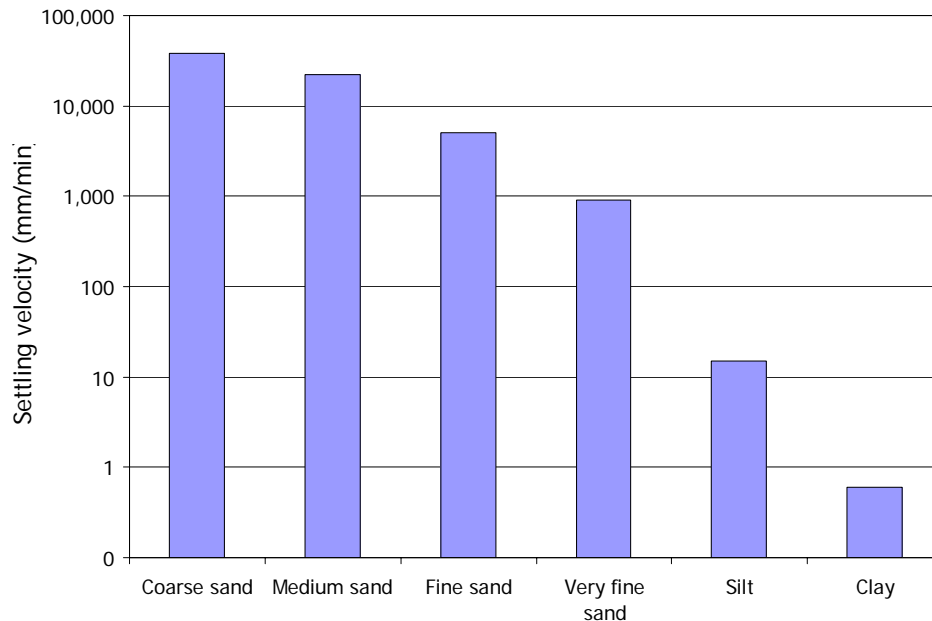
- The basic types of filtration used in trickle systems are:
 1. Reservoirs (settling ponds)
 2. Pre-screening devices
 3. Sand separators
 4. Sand (*media*) tanks
 5. Gravity overflow screens
 6. Tubular screens
 7. Disc filters

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:

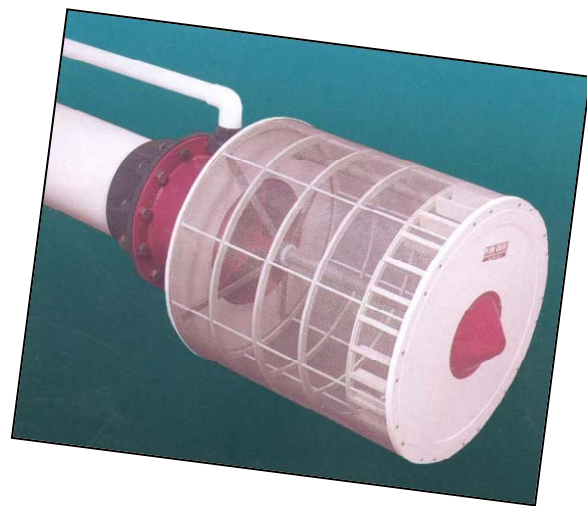


Longer settling basins will allow more time for suspended particles to fall to the bottom before arriving at the pump intake.

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

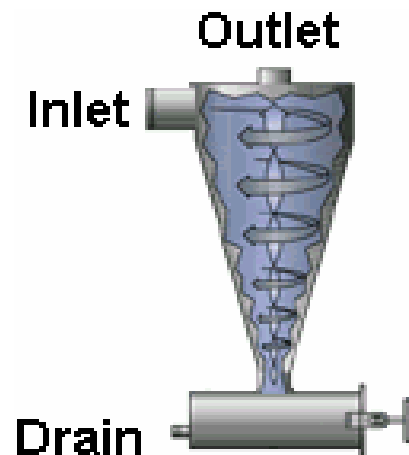


- Horizontal grills
 - Screen plates
 - Rotating and self-cleaning screens
 - Gravity screen filters
- If the inlet line upstream of a pre-screening device is pressurized, you will lose all of the pressure and have to repressurize downstream of the screens – this is a major disadvantage to pre-screening 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/ft² (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:

- Slimy bacteria

These can grow inside pipes and inside emitters. The chemicals used to kill this bacteria are chlorine, ozone, and acids.



- 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.

- 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.

- 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 SO₂ generators. All of these are designed to maintain the pH near (or slightly below) 7.0.

5. Plant root entry into underground emitters

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 minimum injection rate can be put into equation form:

$$q_c = \frac{F_r A}{c t_r T_a} = \frac{(\text{kg/ha})(\text{ha})}{(\text{kg/liter})(0.8)(\text{hrs/set})} = \text{lph} \quad (332)$$

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

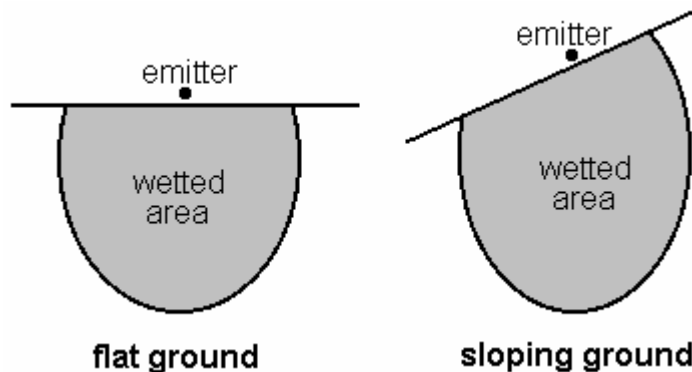
Closely-Spaced Crops

- 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 $P_w \geq 33\%$

Wetted Soil Area, A_w

- 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:

$$P_w = 100 \left(\frac{N_p S_e w}{S_p S_r P_d} \right), \text{ for } S_e < 0.8w \quad (333)$$

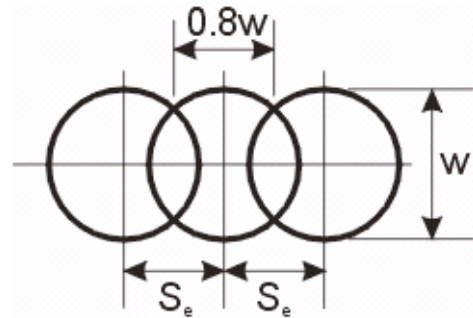
where N_p is the number of emission points (emitters) per plant; S_e is the spacing of emitters along a lateral; w is the wetted width along the lateral; S_p is the spacing of plants along a row; S_r is the spacing between rows; and P_d is the fraction (not percent) of area shaded (see Lecture 19)

- 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:

$$P_w = 100 \left(\frac{0.8N_p w^2}{S_p S_r P_d} \right), \text{ for } S_e \geq 0.8w \quad (334)$$

- 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

- Sample calculation:

- Suppose $S_r = S_p = 3.0$ m, $P_d = 80\%$, and $w = 1.1$ m
- Determine N_p for $P_w \geq 33\%$

$$S_e' = 0.8w = 0.8(1.1) = 0.88 \text{ m} \quad (335)$$

$$0.33 = \frac{N_p (0.88)(1.1)}{(3.0)(3.0)(0.80)} \quad (336)$$

whereby $N_p = 2.45$. Then,

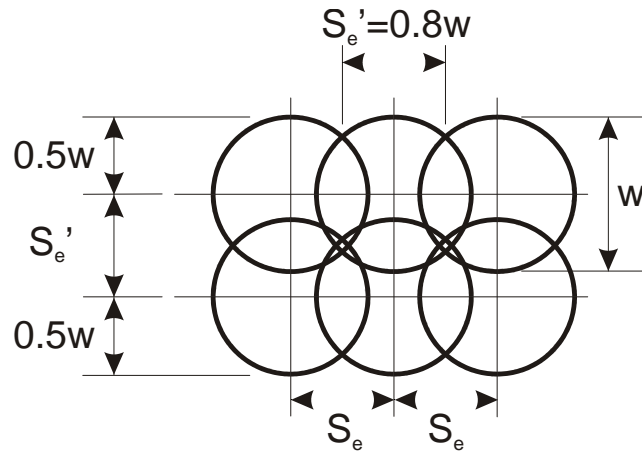
$$P_w = \frac{3(0.88)(1.1)}{(3.0)(3.0)(0.80)} = 0.40 \quad (337)$$

- For double-lateral trickle systems, spaced S_e' apart, P_w is calculated as follows (see Eq. 19.4):

$$P_w = 100 \left(\frac{N_p S_e' (S_e' + w)}{2P_d (S_p S_r)} \right), \text{ for } S_e \leq 0.8w \quad (338)$$

or,

$$P_w = 100 \left(\frac{1.44 w^2 N_p}{2P_d (S_p S_r)} \right) = \frac{72 w^2 N_p}{P_d (S_p S_r)}, \text{ for } S_e \leq 0.8w \quad (339)$$



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):

$$P_w = 100 \left[\frac{N_p \left(A_s + (PS) \frac{S_e}{2} \right)}{S_p S_r P_d} \right], \text{ for } S_e \leq 0.8w \quad (340)$$

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:

$$A_s + (PS) \frac{S_e}{2} = \frac{\pi w^2}{4} + \frac{\pi w S_e}{2} = \frac{\pi w}{2} \left(\frac{w}{2} + S_e \right) \quad (341)$$

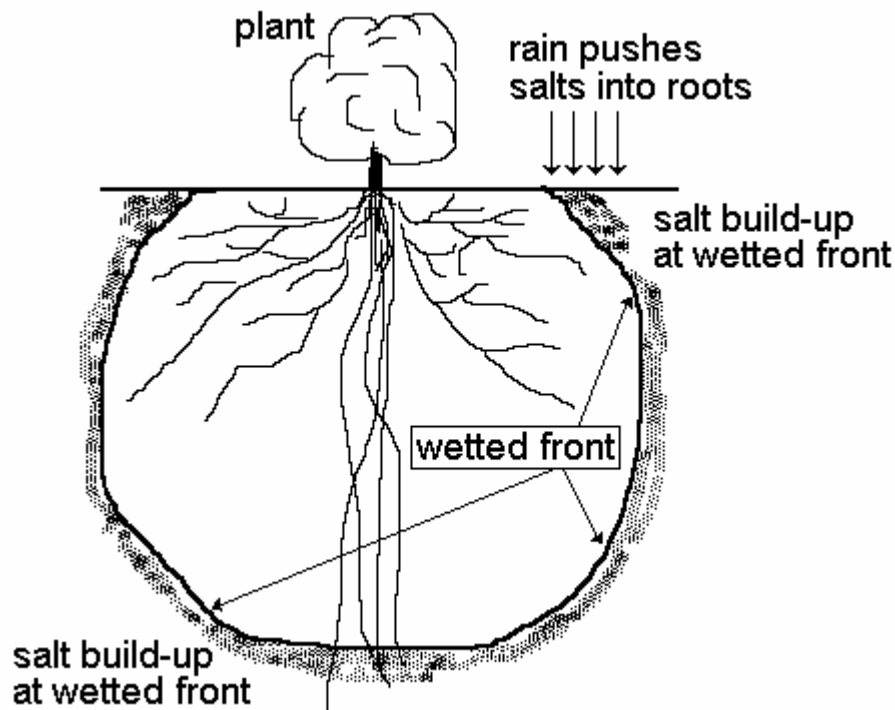
where w is the diameter corresponding to A_s , assuming a circular area

- 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 keep 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):

$$Y_r = \frac{Y_{\text{actual}}}{Y_{\text{potential}}} = \frac{(EC_e)_{\text{max}} - EC_w}{(EC_e)_{\text{max}} - (EC_e)_{\text{min}}} \quad (342)$$

- 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
- $(EC_e)_{\text{max}}$ is the zero yield point, and $(EC_e)_{\text{min}}$ is the 100% yield threshold value
- $(EC_e)_{\text{max}}$ may be as high as 32, and $(EC_e)_{\text{min}}$ 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 $(EC_e)_{\text{max}}$; 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 $2(EC_e)_{\text{min}}$, 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:

$$LR_t = \frac{EC_w}{EC_{dw}} \quad (343)$$

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 $Y_r = 1.0$

$$LR_t = \frac{EC_w}{2(EC_e)_{max}} \quad (344)$$

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,

$$EC_e = 0.5EC_{soil} \quad (345)$$

(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:

$$(EC_e)_{min} = 0.5(0.667EC_w + 0.333EC_{dw}) \quad (346)$$

solving for EC_{dw} ,

$$EC_{dw} = 6(EC_e)_{min} - 2EC_w \quad (347)$$

3. Substitute this last equation into Eq. 19.7 from the textbook to obtain:

$$LR_t = \frac{EC_w}{6(EC_e)_{min} - 2EC_w} \quad (348)$$

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

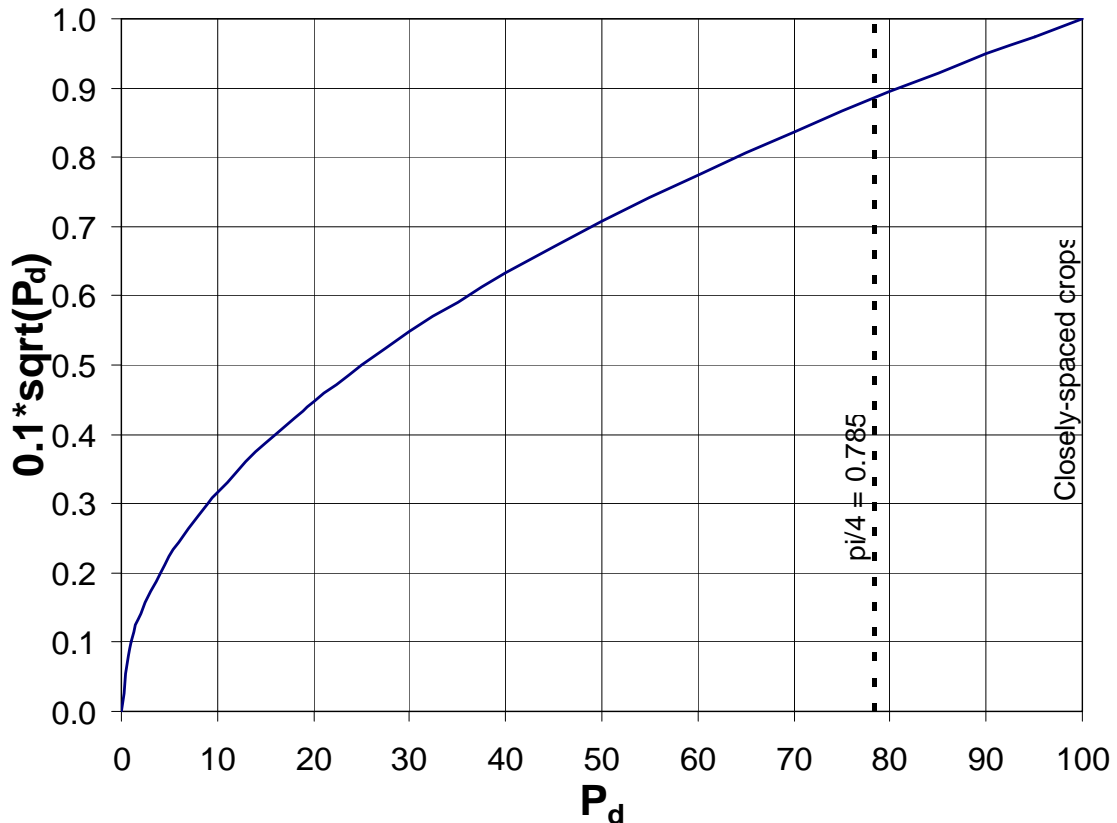
1. Daily Use Rate

- 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:

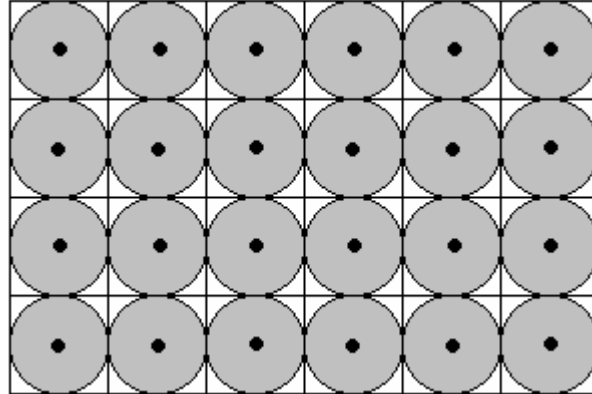
$$T_d = 0.1 U_d \sqrt{P_d} \quad (349)$$

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:

$$T_s = 0.1 U \sqrt{P_d} \quad (350)$$

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:

$$D_n = (U - P_e - M_s) (0.1 \sqrt{P_d}) \quad (351)$$

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:

$$d_x = \frac{MAD}{100} \frac{P_w}{100} W_a Z \quad (352)$$

- 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:

$$f_x = \frac{d_x}{T_d} \quad (353)$$

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,

$$d_n = T_d f' \quad (354)$$

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:

$$d = 100 \left(\frac{d_n T_r}{EU} \right) \quad (355)$$

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:

$$d = \frac{100d_n}{EU(1.0 - LR_t)} \quad (356)$$

- 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

Gross Volume of Water per Plant per Day

- Equation 19.16:

$$G = \frac{d}{f'} S_p S_r \quad (357)$$

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 $S_p S_r$ area
- Other versions of this equation are given in the textbook for gross seasonal volume of water to apply

Required Application Time During Peak-Use Period

- Equation 20.11:

$$T_a = \frac{G}{N_p q_a} \quad (358)$$

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

$$v = \frac{\sqrt{\sum_{i=1}^n (q_i^2) - \frac{1}{n} \left(\sum_{i=1}^n q_i \right)^2}}{\sqrt{n-1} \left(\frac{1}{n} \sum_{i=1}^n q_i \right)} = \frac{\sigma}{q_{avg}} \quad (359)$$

or,

$$v = \frac{1}{q_{avg}} \sqrt{\frac{\sum_{i=1}^n (q_i - q_{avg})^2}{n-1}} \quad (360)$$

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 v have been developed (Soloman 1979):

Classification	Drip & Spray Emitters	Line-Source Tubing
Excellent	$v < 0.05$	$v < 0.1$
Average	$0.05 < v < 0.07$	$0.1 < v < 0.2$
Marginal	$0.07 < v < 0.11$	---
Poor	$0.11 < v < 0.15$	$0.2 < v < 0.3$
Unacceptable	$0.15 < v$	$0.3 < v$

- 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..... $(1 \pm v)q_{avg}$
 95% of the discharge values are within..... $(1 \pm 2v)q_{avg}$
 99.75% of the discharge values are within..... $(1 \pm 3v)q_{avg}$

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

$$v_s = \frac{v}{\sqrt{N_p'}} \quad (361)$$

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,

$$L_w = w + (N - 1)S_e \quad (362)$$

where L_w is the length of the wetted strip; and N is the number of emitters (assumed to be evenly spaced). Then,

$$N = 1 + \left(\frac{L_w - w}{S_e} \right) \quad (363)$$

or,

$$N_p' \approx 1 + \left(\frac{S_p - w}{S_e} \right) \quad (364)$$

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):

$$EU = 100 \left(1 - 1.27 v_s \right) \frac{q_n}{q_a} \quad (365)$$

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

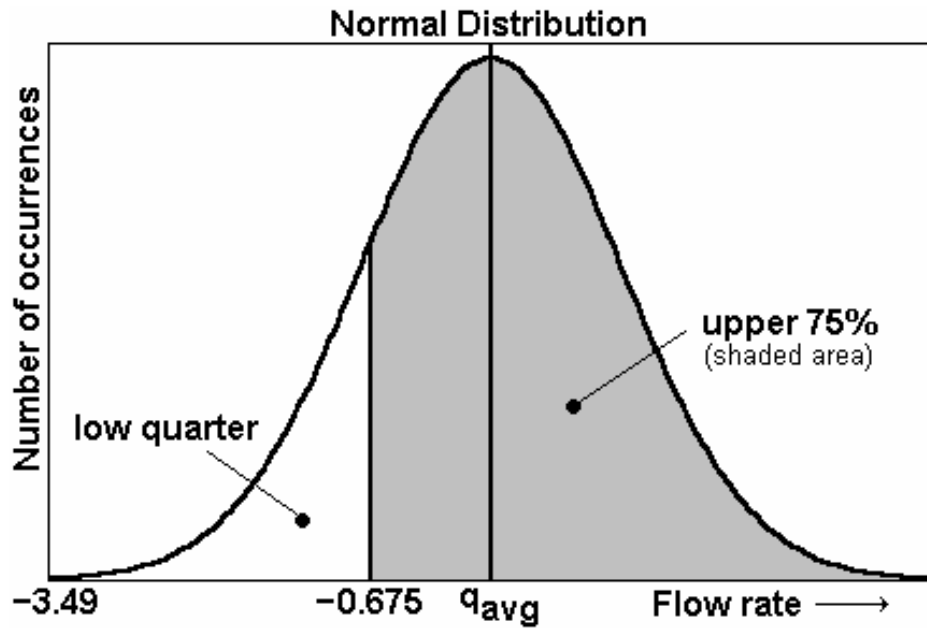
VI. Average of the low ¼

- Note that the inclusion percentages for 1, 2 and 3 standard deviations correspond to any normally distributed data
- Note also that $v q_{avg} = \sigma$
- The textbook says that for a normal distribution, the average flow rate of the low one-quarter of measured q samples is approximately $(1 - 1.27 v)q_{avg}$
- 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:

$$\text{occurrences} = \frac{e^{-\frac{1}{2}\left(\frac{q-q_{avg}}{\sigma}\right)^2}}{\sigma\sqrt{2\pi}} \quad (366)$$

- 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:

$$\text{occurrences} = \frac{e^{-0.5q^2}}{\sqrt{2\pi}} \quad (367)$$



- For $q = 0.675$, occurrences = 0.31766718
- For $q = 3.49$, occurrences = 0.00090372
- Finally,

$$\frac{0.31766718 - 0.00090372}{0.9998 - 0.7500} = 1.268 \quad (368)$$

- Therefore, $(1 - 1.268 v)q_{avg}$ is the average of the lowest 25% of measured discharge values, for any given values of v & q_{avg} , and given normally-distributed data

V. System Capacity

- The system capacity of a trickle system can be calculated by Eq. 20.15:

$$Q_s = 2.78 \frac{A N_p q_a}{N_s S_p S_r} \quad (369)$$

where N_s is the number of stations (sets); and A is the total net irrigated area.
Or,

$$Q_s = 2.78 \frac{A q_a}{N_s S_e S_l} \quad (370)$$

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_i in m (10,000 m²/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:

$$O_t = K \frac{V_s}{Q_s} \quad (371)$$

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:

$$l_c = \frac{g \pi D^4 H}{v q K} \quad (372)$$

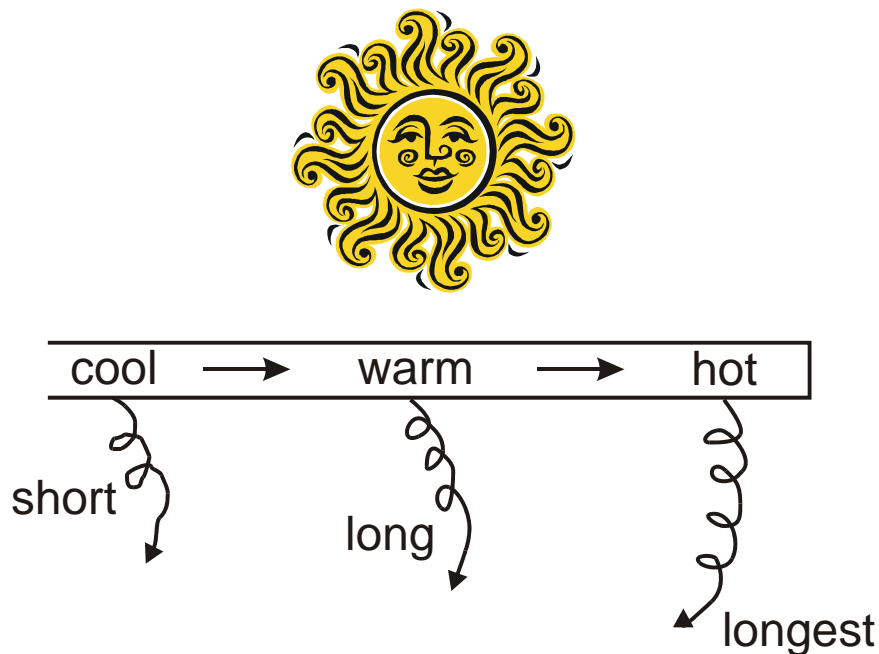
where l_c is the length of the flow path; D is the inside diameter; H is the pressure head; v is the kinematic viscosity (a function of water temperature); q is the flow rate; K is for units conversion; and g is the ratio of force to mass

- 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:

$$\frac{VD}{\nu} = \frac{4Q}{\pi \nu D} < 4,000 \quad (373)$$

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:

$$q = a'K \sqrt{\frac{2gHn_o^2}{(1+n_o^2)}} \quad (374)$$

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 .

$$G = K \left(\frac{d S_p S_r}{f} \right) \quad (375)$$

8. Calculate the daily hours of operation, T_a , (per station, or subunit) during the peak-use period.

$$T_a = \frac{G}{N_p q_a} \quad (376)$$

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 $T_a N_s$ 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

$$q_n = \frac{q_a EU}{100(1.0 - 1.27 v_s)} \quad (377)$$

Note that if EU is high and v_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

$$h = \left(\frac{q}{K_d} \right)^{1/x} \quad (378)$$

$$h_n = h_a \left(\frac{q_n}{q_a} \right)^{1/x} \quad (379)$$

13. Calculate the allowable change in pressure head in an operating station

$$\Delta H_s = 2.5(h_a - h_n) \quad (380)$$



14. Calculate Q_s , V_s , and O_t .

15. Finally, size the laterals, headers, manifolds and mainline(s) according to hydraulic design criteria.

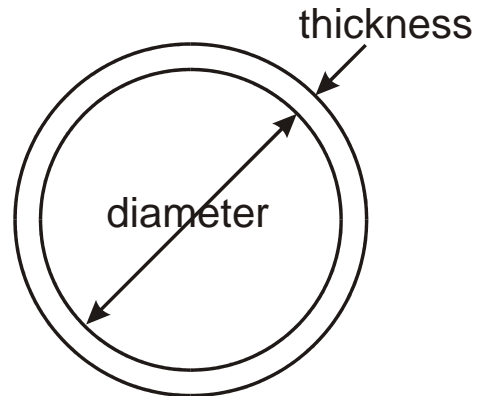
Lecture 21

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 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:



$$PR = \frac{2S}{SDR - 1} = \frac{2S}{\left(\frac{OD}{t} - 1\right)} \quad (OD\text{-based}) \quad (381)$$

$$PR = \frac{2S}{SDR + 1} = \frac{2S}{\left(\frac{ID}{t} + 1\right)} \quad (ID\text{-based}) \quad (382)$$

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:

$$P = \frac{(\text{schedule})SE}{1000} \quad (383)$$

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):

$$J = 7.83(10)^7 \frac{Q^{1.75}}{D^{4.75}} \quad (384)$$

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:

$$J' = J \left(\frac{S_e + f_e}{S_e} \right) \quad (385)$$

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):

$$H_l = H_a + kh_f + 0.5\Delta h_e \quad (386)$$

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:

$$\begin{aligned} H'_n &= H_l - (h_f + \Delta h_e) - \Delta H_c \\ H'_n &= H_c - \Delta H_c \end{aligned} \quad (387)$$

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

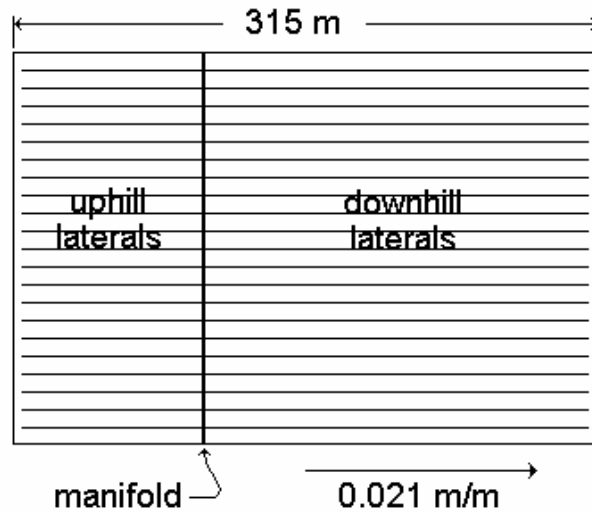
I. Optimal Manifold Location

- 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:

$$\frac{315 \text{ m}}{1.5 \text{ m/emitter}} = 210 \text{ emitters} \quad (388)$$

2. Total nominal discharge for the pair of laterals is:

$$Q_{\text{pair}} = \frac{(210 \text{ emitters})(3.5 \text{ lph/emitter})}{(60 \text{ min/hr})} = 12.25 \text{ lpm} \quad (389)$$

3. From Table 8.2 (page 141), $J \cong 13.3$ m/100m. The adjusted J is:

$$J' = J \left(\frac{S_e + f_e}{S_e} \right) = 13.3 \left(\frac{1.5 + 0.12}{1.5} \right) = 14.4 \text{ m/100 m} \quad (390)$$

4. Multiple outlet factor, $F = 0.36$ for 210 outlets

5. Friction loss for the pair of laterals:

$$(h_f)_{\text{pair}} = \frac{J'FL}{100} = \frac{(14.4)(0.36)(315)}{100} = 16.3 \text{ m} \quad (391)$$

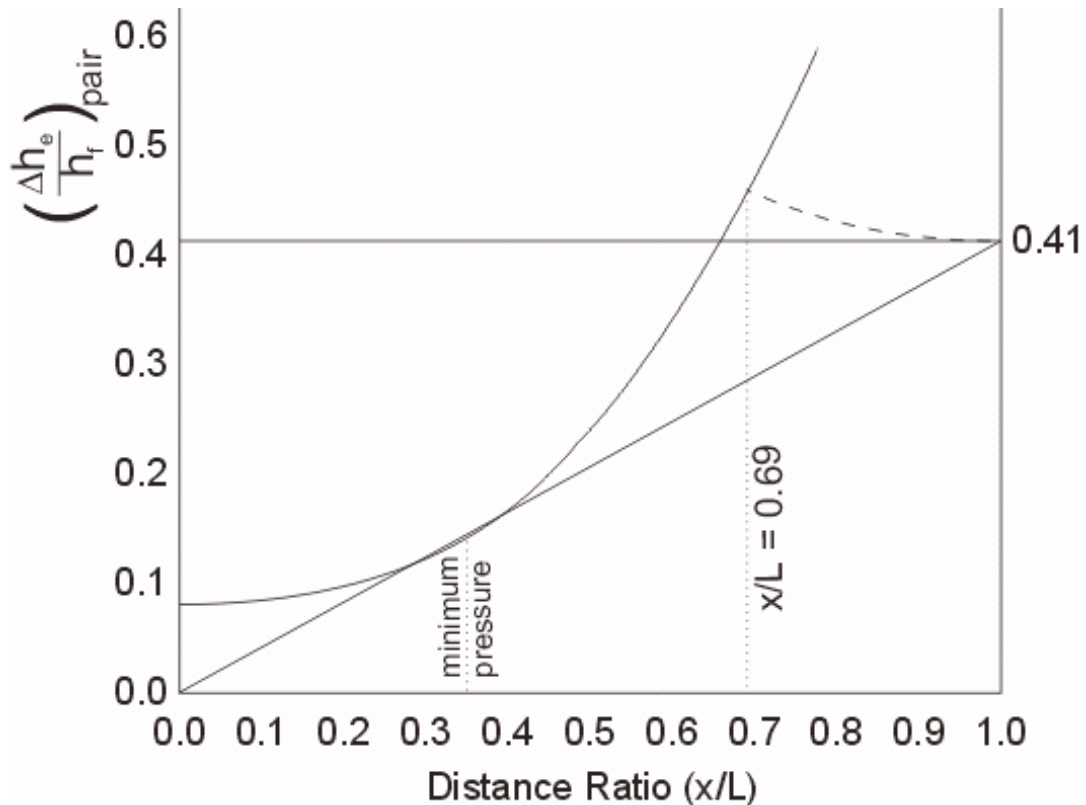
6. Elevation change for the pair of laterals:

$$(\Delta h_e)_{\text{pair}} = (315 \text{ m})(0.021) = 6.62 \text{ m} \quad (392)$$

7. Ratio of elevation change to friction loss for the pair:

$$\left(\frac{\Delta h_e}{h_f}\right)_{\text{pair}} = \frac{6.62}{16.3} = 0.41 \quad (393)$$

8. From the nondimensional graphical solution (Fig. 8.2): $x/L = 0.69$. Then, $x = (0.69)(315 \text{ m}) = 217 \text{ m}$. Look at the figure below:



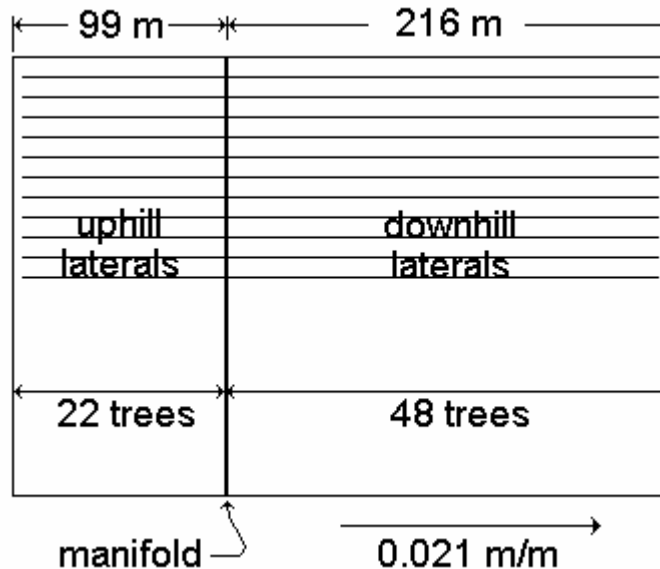
How was this done?

- 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, $x = (48 \text{ trees})(4.5 \text{ m/tree}) = 216 \text{ m}$
- 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 $(0.35)(315 \text{ m}) = 110 \text{ m}$ from the closed end, or $216 - 110 = 106 \text{ m}$ 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

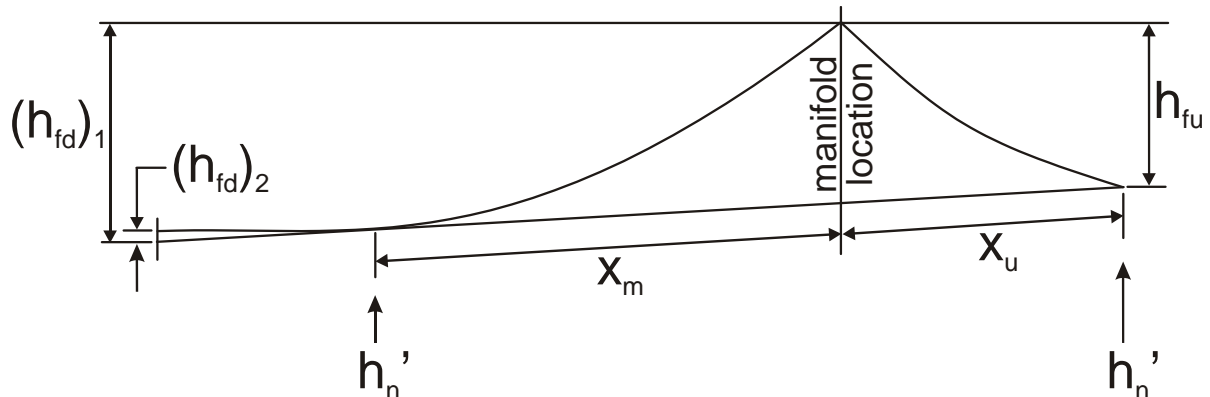
II. Definition of Minimum Lateral Head

- 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:

$$h_n' = h_l - h_{fu} - x_u S \quad (394)$$

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:

$$h_n' = h_l - (h_{fd})_1 + (h_{fd})_2 + x_m S \quad (395)$$

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:

$$h_{fu} + S(x_u + x_m) - (h_{fd})_1 + (h_{fd})_2 = 0 \quad (396)$$

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' :

$$S = J' \quad (397)$$

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:

$$J' = \left[\frac{S_e + f_e}{S_e} \right] \left[1.212(10)^{10} \left(\frac{q_a(L - x_u - x_m)}{3,600 S_e C} \right)^{1.852} D^{-4.87} \right] \quad (398)$$

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: $0 \leq x_m \leq x_d$

- 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:

$$(h_{fd})_1 = J_{d1} ' F_{d1} (L - x_u) \quad (402)$$

and,

$$(h_{fd})_2 = J_{d2} ' F_{d2} (L - x_u - x_m) \quad (403)$$

- The above three “F” values are as defined by Eq. 8.9 in the textbook
- The friction loss gradients (in m/m) are:

$$J_u ' = K_J (x_u)^{1.852} \quad (404)$$

$$J_{d1} ' = K_J (L - x_u)^{1.852} \quad (405)$$

$$J_{d2} ' = K_J (L - x_u - x_m)^{1.852} \quad (406)$$

where, for the Hazen-Williams equation,

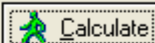
$$K_J = \left(\frac{S_e + f_e}{S_e} \right) \left[1.212(10)^{10} D^{-4.87} \left(\frac{q_a}{3,600 S_e C} \right)^{1.852} \right] \quad (407)$$

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

Trickle Manifold Location ✕

Data:

Emitter discharge (lph) <input style="width: 90%;" type="text" value="3.700"/>	Lateral length (m) <input style="width: 90%;" type="text" value="600.000"/>	
Emitter spacing (m) <input style="width: 90%;" type="text" value="3.000"/>	Lateral ID (mm) <input style="width: 90%;" type="text" value="17.800"/>	
Emitter head (m) <input style="width: 90%;" type="text" value="11.900"/>	Ground slope (m/m) <input style="width: 90%;" type="text" value="0.01000"/>	
Barb loss, f_e (m) <input style="width: 90%;" type="text" value="0.060"/>		

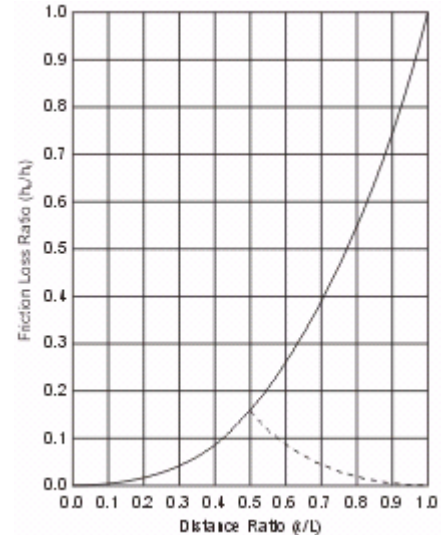
Results:

Length of uphill lateral:	170.062 m
Length of downhill lateral:	429.938 m
Distance from manifold to minimum head:	210.264 m
Required lateral inlet head:	16.012 m
Minimum head in downhill lateral:	13.833 m
Minimum head in uphill lateral:	13.913 m

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 $\frac{1}{2}$ the total length, because it is equal to $\frac{1}{2}$ 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:



$$h_f = f \frac{L}{D} \frac{V^2}{2g} \quad (408)$$

- 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

$$f \cong 0.32 N_R^{-0.25} \quad (409)$$

where N_R is the Reynolds number, which for circular pipes is:

$$N_R = \frac{VD}{\nu} = \frac{4Q}{\nu\pi D} \quad (410)$$

- The kinematic viscosity, ν , is equal to about $1.003(10)^{-6}$ m²/s for water at 20°C
- Then, for this kinematic viscosity,

$$f \cong 0.32 \left(\frac{4Q}{\nu\pi D} \right)^{-0.25} \approx 0.0095 \left(\frac{Q}{D} \right)^{-0.25} \quad (411)$$

- Putting the above into the Darcy-Weisbach equation:

$$h_f = 0.0095 \left(\frac{Q}{D} \right)^{-0.25} \frac{L}{D} \frac{V^2}{2g} \quad (412)$$

or:

$$h_f \cong 0.00079L \frac{Q^{1.75}}{D^{4.75}} \quad (413)$$

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} ":

$$h_{fx} \cong 0.00079L(x/L) \frac{[Q(x/L)]^{1.75}}{D^{4.75}} \quad (414)$$

- Then,

$$\frac{h_{fx}}{h_f} = (x/L)(x/L)^{1.75} = (x/L)^{2.75} \quad (415)$$

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:

$$H_c = H_l - h_f - \Delta h_e \quad (416)$$

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,

$$H'_n = H_c - \Delta H_c \quad (417)$$

- 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 ΔH_c (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):

$$\left(\frac{h_{fx}}{h_f} \right)_{\text{pair}} = \left(\frac{x}{L} \right)^{2.75} \quad (418)$$

2. The slope of this friction loss curve is:

$$\frac{d \left(\frac{h_{fx}}{h_f} \right)_{\text{pair}}}{d \left(\frac{x}{L} \right)} = 2.75 \left(\frac{x}{L} \right)^{1.75} \quad (419)$$

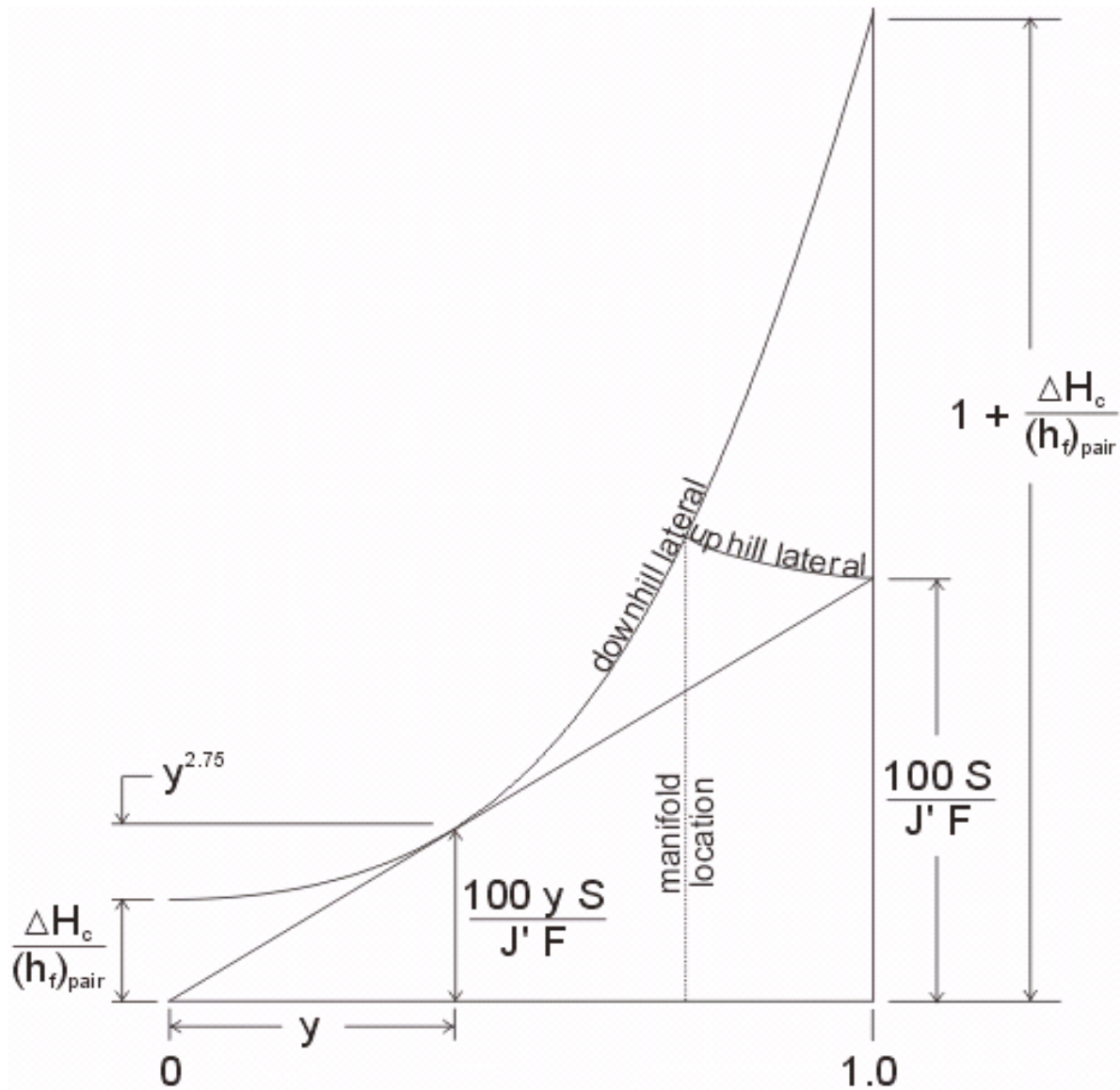
3. The uniform ground slope on the dimensionless graph is:

$$\left(\frac{\Delta h_e}{h_f} \right)_{\text{pair}} = \frac{SL}{\left(\frac{J'FL}{100} \right)} = \frac{100S}{J'F} \quad (420)$$

4. Then,

$$\frac{100S}{J'F} = 2.75(y)^{1.75} \quad (421)$$

in which y is the value of x/L where the minimum pressure occurs ($0 \leq y \leq 1$); 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:

$$y = \left(\frac{100S}{2.75J'F} \right)^{1/1.75} \quad (422)$$

or,

$$y \approx \left(\frac{100S}{J'} \right)^{1/1.75} \quad (423)$$

where $F \approx 0.36$

6. Referring to the figure on the previous page, the following equality can be written:

$$y^{2.75} + \frac{\Delta H_c}{(h_f)_{\text{pair}}} = \frac{100yS}{J'F} \quad (424)$$

solving for ΔH_c ,

$$\Delta H_c = (h_f)_{\text{pair}} \left(\frac{100yS}{J'F} - y^{2.75} \right) \quad (425)$$

where,

$$(h_f)_{\text{pair}} = \frac{J'FL}{100} \quad (426)$$

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:

$$\Delta H_c = 8.9LS^{1.57} (J')^{-0.57} \quad (427)$$

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 pair 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):

$$H_l = H_a + \alpha (h_f)_{\text{pair}} + \left(\frac{x}{L} - 0.5 \right) (\Delta h_e)_{\text{pair}} \quad (428)$$

with,

$$H_a = \left(\frac{q_a}{K_d} \right)^{1/x} \quad (429)$$

$$(h_f)_{\text{pair}} = \frac{J'FL}{100} \quad (430)$$

$$(\Delta h_e)_{\text{pair}} = \frac{100S}{J'F} (h_f)_{\text{pair}} = SL \quad (431)$$

- Note that $(\Delta h_e)_{\text{pair}}$ must be a negative number
- The ratio x/L is the distance to the manifold, where L is the length of the pair of laterals
- The following derivation is based on equations presented by Keller and Rodrigo (1979):

1. Given that for a single lateral approximately $\frac{3}{4}$ 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:

$$\alpha (h_f)_{\text{pair}} = \frac{3}{4} (h_f)_{\text{downhill}} \left(\frac{x}{L} \right) + \frac{3}{4} (h_f)_{\text{uphill}} \left(1 - \frac{x}{L} \right) \quad (432)$$

The above equation is a *weighted average* because the uphill lateral is shorter than the downhill lateral

2. Recall that,

$$\left(\frac{h_{fx}}{h_f} \right)_{\text{pair}} = \left(\frac{x}{L} \right)^{2.75} \quad (433)$$

Then,

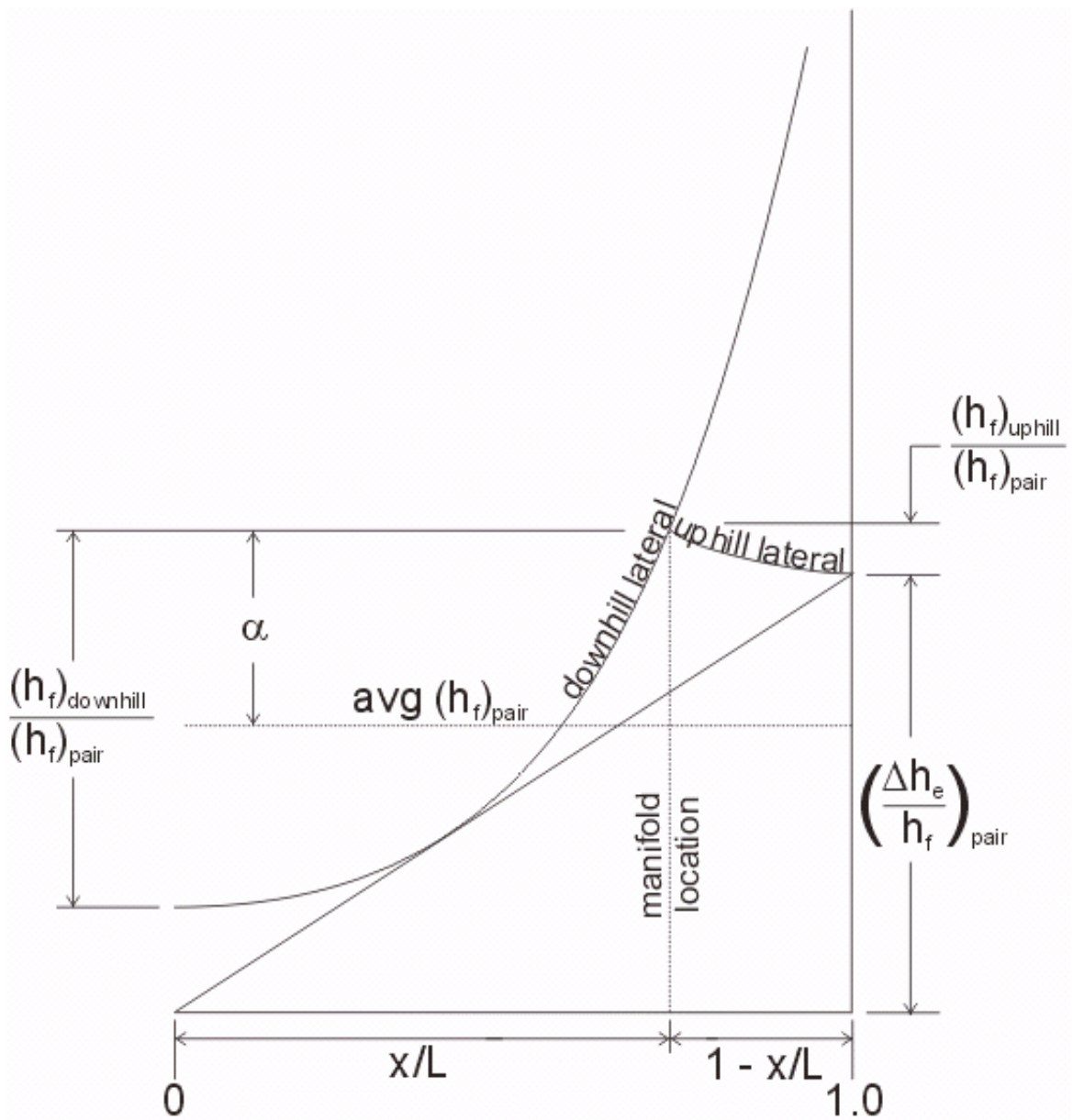
$$(h_f)_{\text{downhill}} = \left(\frac{x}{L} \right)^{2.75} (h_f)_{\text{pair}} \quad (434)$$

$$(h_f)_{\text{uphill}} = \left(1 - \frac{x}{L} \right)^{2.75} (h_f)_{\text{pair}}$$

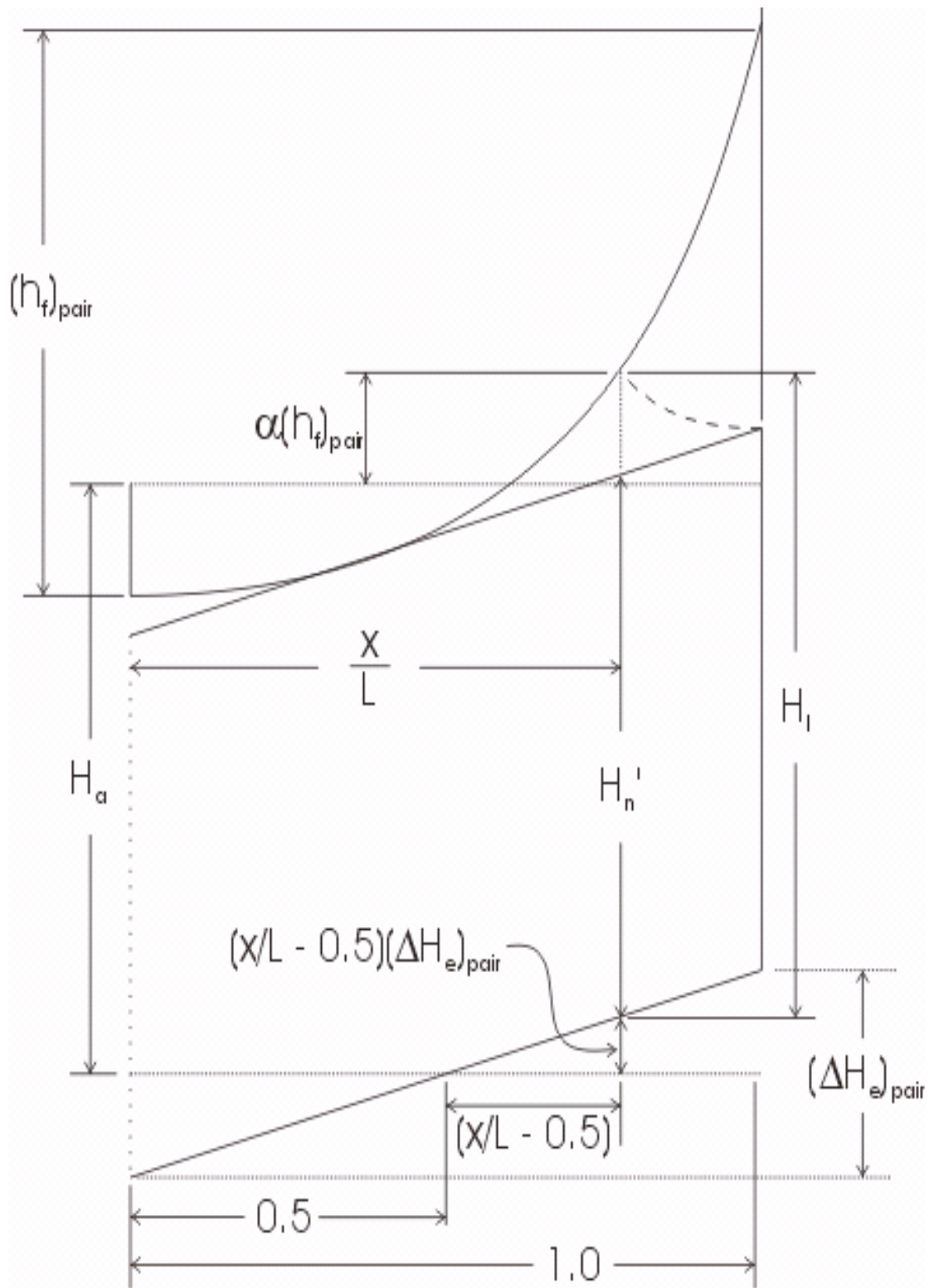
3. Combining equations:

$$\alpha (h_f)_{\text{pair}} = \frac{3}{4} (h_f)_{\text{pair}} \left[\left(\frac{x}{L} \right) \left(\frac{x}{L} \right)^{2.75} + \left(1 - \frac{x}{L} \right) \left(1 - \frac{x}{L} \right)^{2.75} \right] \quad (435)$$

$$\alpha = \frac{3}{4} \left[\left(\frac{x}{L} \right)^{3.75} + \left(1 - \frac{x}{L} \right)^{3.75} \right] \quad (436)$$



- This last equation for α is Eq. 22.25 from the textbook
- See the figure below



Lecture 23

Manifold Hydraulic Design

I. Introduction

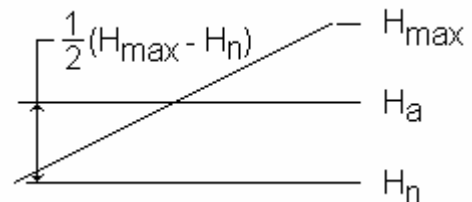
- Manifolds in trickle irrigation systems often have multiple pipe sizes to:
 1. reduce pipe costs
 2. reduce pressure variations
- In small irrigation systems the reduction in pipe cost may not be significant, not to mention that it is also easier to install a system with fewer pipe sizes
- Manifold 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:

$$(\Delta H_m)_a = \Delta H_s - \Delta H_l \quad (437)$$

- 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 $\Delta H_l = \frac{1}{2}\Delta H_s$, and $\Delta H_m = \frac{1}{2}\Delta H_s$, 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

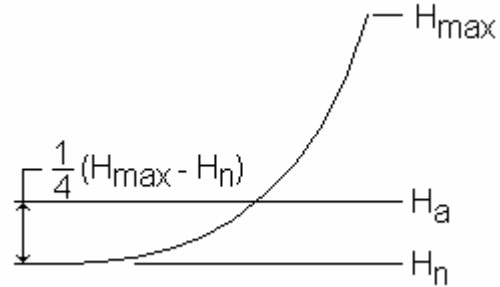


equal to H_n plus half the difference in the maximum and minimum heads:

$$H_{\max} - H_n = 2(H_a - H_n) \quad (438)$$

- Consider a sloping friction loss gradient (multiple outlets) on flat ground:

In this case, the average head occurs after about $\frac{3}{4}$ of the total head loss (due to friction) occurs, beginning from the lateral inlet. Then,



$$H_{\max} - H_n = 4(H_a - H_n) \quad (439)$$

- 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,

$$H_{\max} - H_n = 2.7(H_a - H_n) \quad (440)$$

- 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:

$$\Delta H_s = 2.5(H_a - H_n) \quad (441)$$

- 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

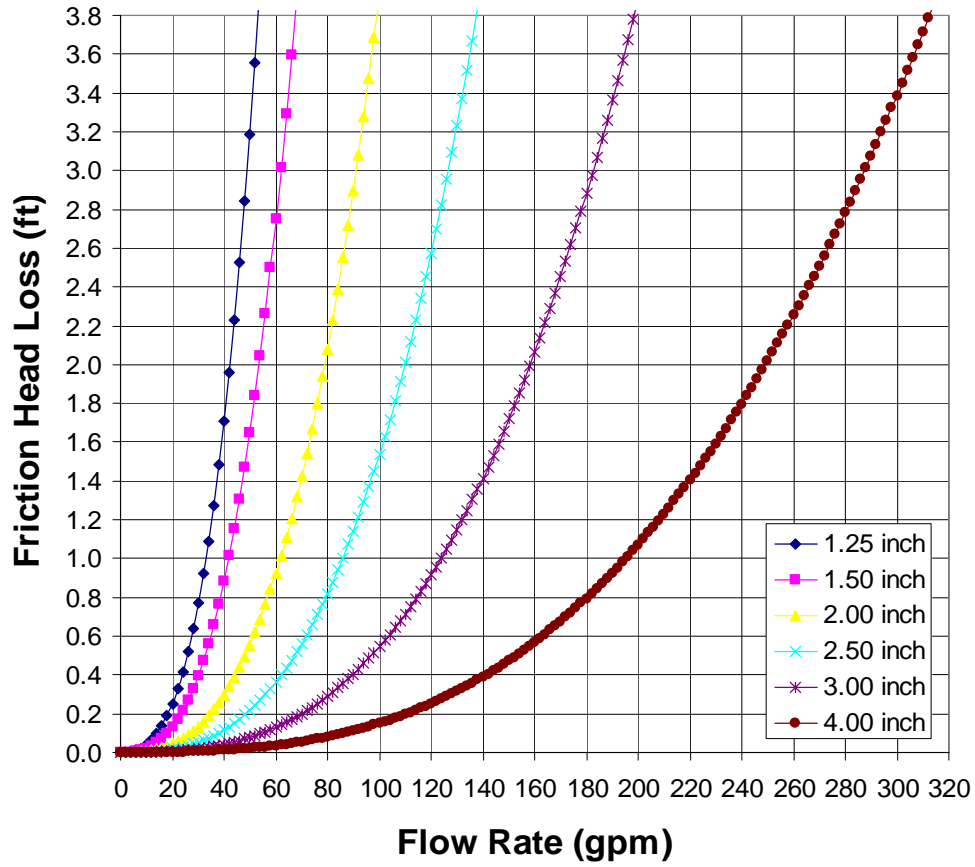
III. Pipe Sizing in Manifolds

- 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

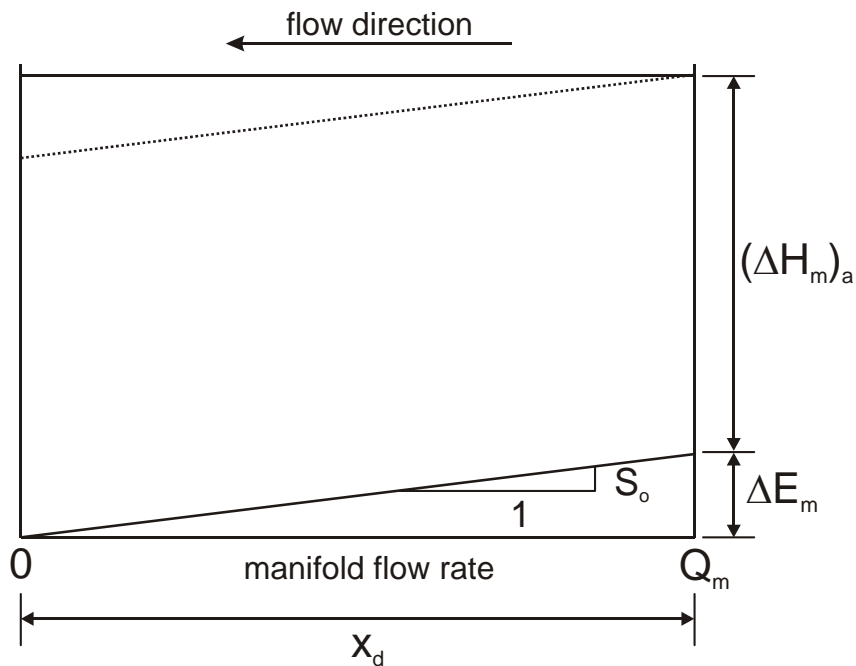
- The graphical method uses “standard” head loss curves for different pipe sizes and different flow rates with equally-spaced multiple outlets, each outlet with the same discharge
- The curves all intersect at the origin (corresponding to the downstream closed end of a pipe)
- Below is a sample of the 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

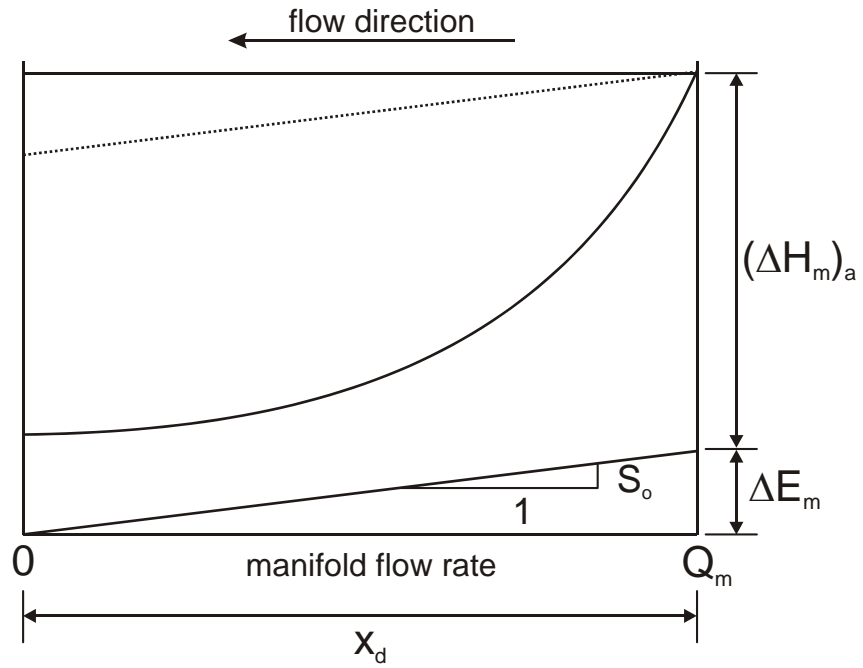


- The following steps illustrate the graphical design procedure:

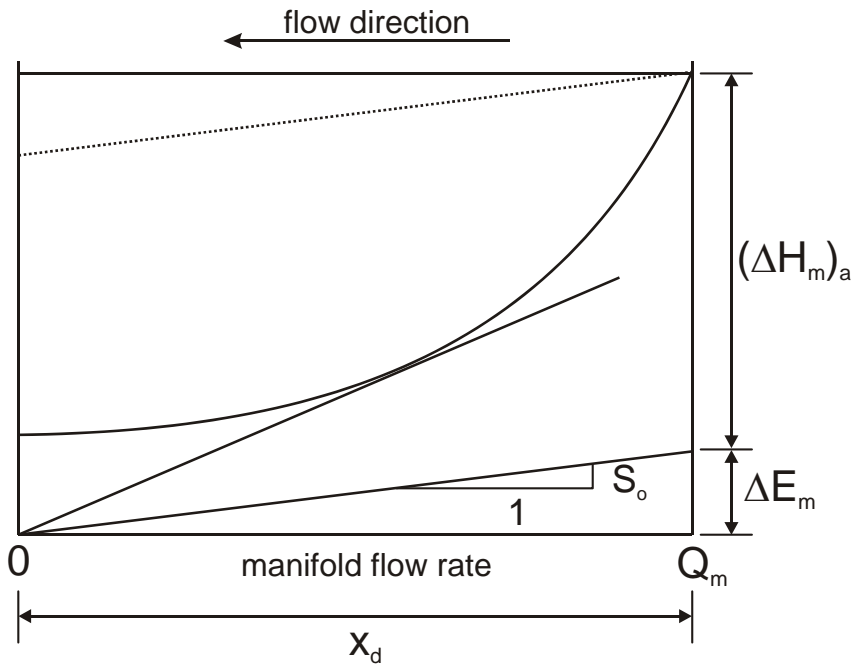
Step 1:



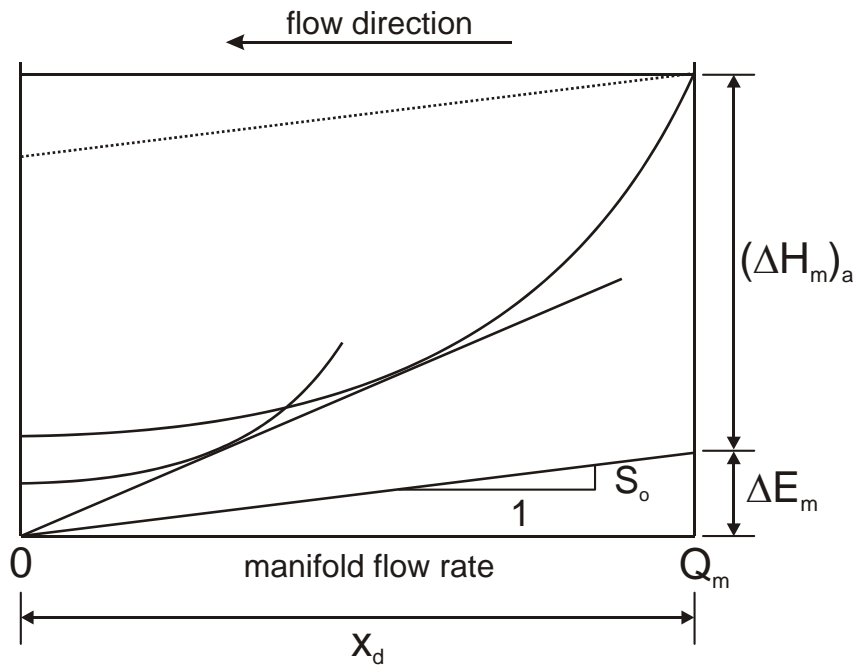
Step 2:



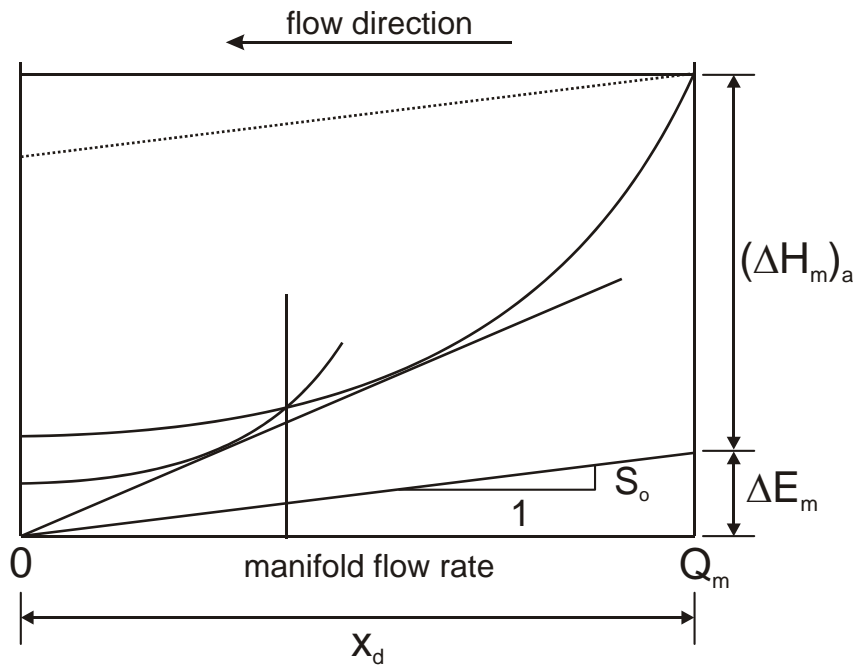
Step 3:



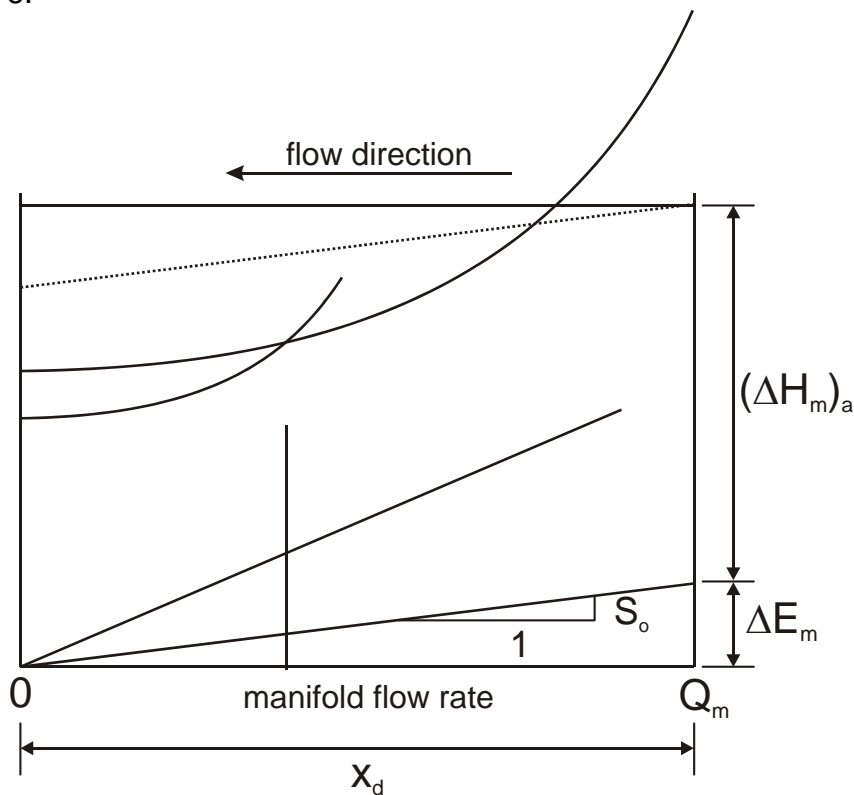
Step 4:



Step 5:



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):

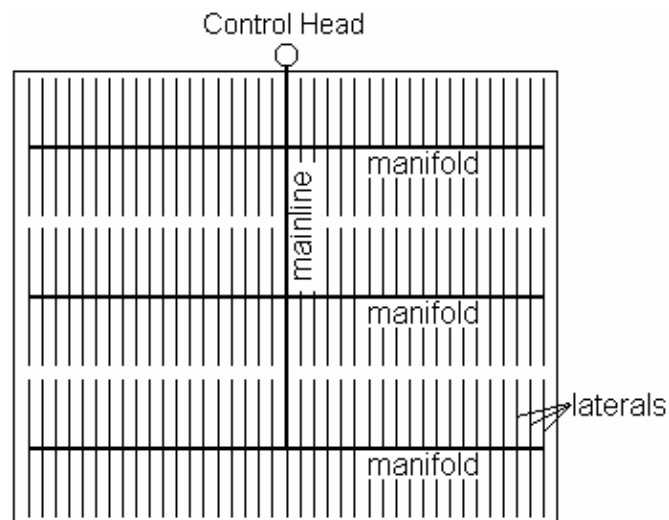
$$H_m = H_l + k h_f + 0.5\Delta E_m \quad (442)$$

where $k = 0.75$ for single-diameter manifolds; $k = 0.63$ for dual pipe size laterals; or $k \approx 0.5$ 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 $(H_n)_{\text{uphill}}$ equal to $(H_n)_{\text{downhill}}$
- If average friction loss slopes are equal for both uphill and downhill manifold branches (assuming similar diameters will carry similar flow rates):

Downhill side:

$$(\Delta H_m)_a = h_{fd} - \Delta E \left(\frac{x}{L} \right) = h_{fd} - Y \Delta E \quad (443)$$

Uphill side:

$$(\Delta H_m)_a = h_{fu} + \Delta E \left(\frac{L-x}{L} \right) = h_{fu} + (1-Y) \Delta E \quad (444)$$

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:

$$\bar{J}_{\text{uphill}} = \bar{J}_{\text{downhill}} \quad (445)$$

$$\frac{h_{fu}}{L-x} = \frac{h_{fd}}{x}$$

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,

$$\begin{aligned} h_{fd} &= \bar{J}x \\ h_{fu} &= \bar{J}(L - x) \end{aligned} \quad (446)$$

and,

$$\begin{aligned} (\Delta H_m)_a &= \bar{J}x - Y\Delta E \\ (\Delta H_m)_a &= \bar{J}(L - x) + (1 - Y)\Delta E \end{aligned} \quad (447)$$

then,

$$\begin{aligned} \frac{(\Delta H_m)_a + Y\Delta E}{x} &= \bar{J} \\ \frac{(\Delta H_m)_a - (1 - Y)\Delta E}{L - x} &= \bar{J} \end{aligned} \quad (448)$$

- Equating both J-bar values,

$$\frac{(\Delta H_m)_a + Y\Delta E}{x} = \frac{(\Delta H_m)_a - (1 - Y)\Delta E}{L - x} \quad (449)$$

- Dividing by L and rearranging (to get Eq. 23.3),

$$\frac{(\Delta H_m)_a + Y\Delta E}{Y} = \frac{(\Delta H_m)_a - (1 - Y)\Delta E}{1 - Y} \quad (450)$$

or,

$$\frac{\Delta E}{(\Delta H_m)_a} = \frac{2Y - 1}{2Y(1 - Y)} \quad (451)$$

- 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 v_s
- Equation 23.3 can be solved by isolating one of the values for Y on the left hand side, such that:

$$Y = 1 - \left(\frac{2Y - 1}{2Y} \right) \left(\frac{(\Delta H_m)_a}{\Delta E} \right) \quad (452)$$

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 $0 \leq Y \leq 1$, 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))

$$L_D = L \left(\frac{Q_{\text{beg}} - Q_{\text{end}}}{Q_m} \right) \quad (453)$$

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):

$$h_f = \frac{\text{FLK}}{100Q_m} \left(\frac{Q_1^a}{D_1^c} + \frac{Q_2^a - Q_1^a}{D_2^c} + \frac{Q_3^a - Q_2^a}{D_3^c} + \frac{Q_4^a - Q_3^a}{D_4^c} \right) \quad (454)$$

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:

$$h_f = \frac{FKQ_m^{a-1}}{100L^{a-1}} \left(\frac{x_1^a}{D_1^c} + \frac{x_2^a - x_1^a}{D_2^c} + \frac{x_3^a - x_2^a}{D_3^c} + \frac{x_4^a - x_3^a}{D_4^c} \right) \quad (455)$$

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),

$$\Delta H_m = h_f + Sx_u \quad (456)$$

For $s < 0$ (downhill branch of the manifold),

$$\Delta H_m = h_f + S \left(1 - \frac{0.36}{n} \right) x_d \quad (457)$$

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 $\Delta H_m < 1.1 (\Delta H_m)_a$, 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:

$$Q_s^{\text{new}} = Q_s^{\text{old}} \left(\frac{h_f}{(\Delta H_m)_a - \Delta E_m} \right)^{1/b} \quad (458)$$

and for $s < 0$ as:

$$Q_s^{\text{new}} = Q_s^{\text{old}} \left(\frac{h_f}{(\Delta H_m)_a - \Delta E_l \left(1 - \frac{0.36}{n} \right)} \right)^{1/b} \quad (459)$$

- 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 $\Delta E_l = sL/100$

(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 v_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,

$$H_m = H_l + kh_f + 0.5\Delta E_m \quad (460)$$

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:

$$F_s = 0.38 S_f^{1.25} + 0.62 \quad (461)$$

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,

$$h_f = F_s F \left(\frac{JL}{100} \right) \quad (462)$$

IX. Manifold Pipe Sizing by the "HGL" Method

- This is the "Hydraulic Grade Line" method
- Same as the semi-graphical method, but performed numerically

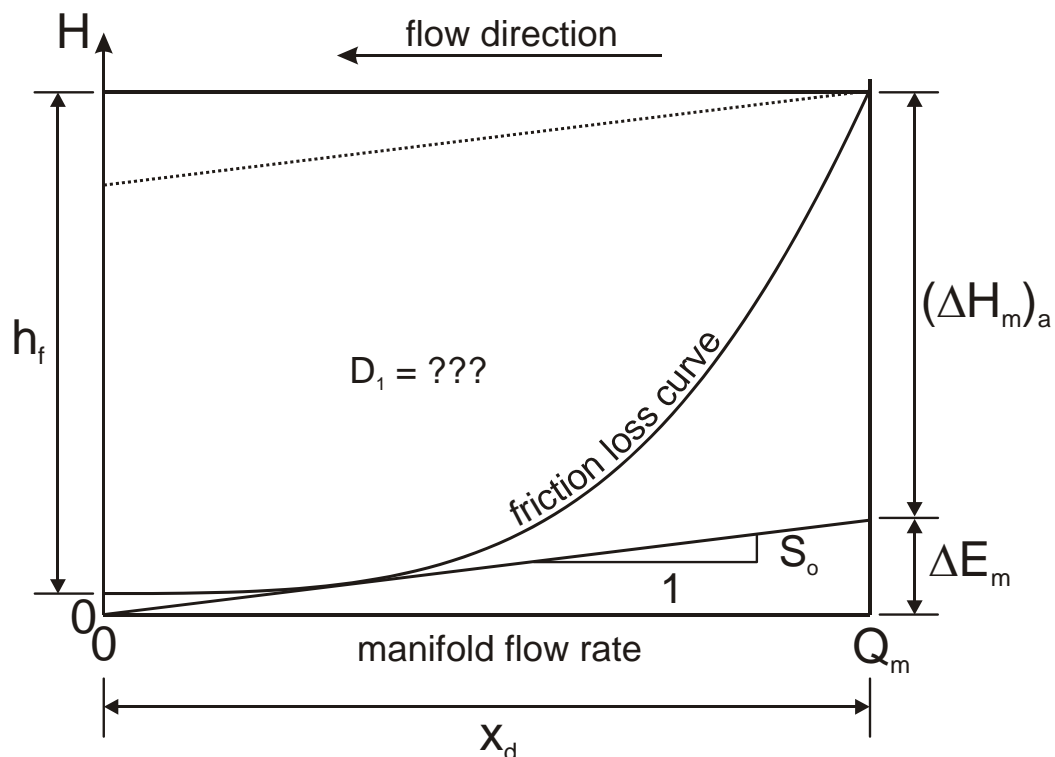
(a) Uphill Side of the Manifold

- 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_1 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:

$$H = (\Delta H_m)_a + \Delta E_m - h_f + \frac{JFL}{100} \quad (463)$$

where, using the Hazen-Williams equation,

$$J = K \left(\frac{Q}{C} \right)^{1.852} D^{-4.87} \quad \text{for } 0 \leq Q \leq Q_m \quad (464)$$

$$F = \frac{1}{2.852} + \frac{1}{2N} + \frac{\sqrt{0.852}}{6N^2} \quad (465)$$

$$N = \left(\frac{x_d}{S_l} \right) \left(\frac{Q}{Q_m} \right) \quad \text{for } N > 0 \quad (466)$$

where N is the number of outlets (laterals) from the location of “Q” in the manifold to the closed end

$$L = x_d \left(\frac{Q}{Q_m} \right) \quad (467)$$

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:

$$h_f = \frac{J_{hf} F_{hf} x_d}{100} = 0.01K \left(\frac{Q_m}{C} \right)^{1.852} D^{-4.87} F_{hf} x_d \quad (468)$$

where F_{hf} is defined as F above, except with $N = x_d/S_l$.

- The slope of the friction loss curve is:

$$\frac{dH}{dQ} = \frac{1}{100} \left(FL \frac{dJ}{dQ} + JL \frac{dF}{dQ} + JF \frac{dL}{dQ} \right) \quad (469)$$

where,

$$\frac{dJ}{dQ} = \frac{1.852KQ^{0.852}}{C^{1.852}D^{4.87}} \quad (470)$$

$$\frac{dF}{dQ} = -\frac{x_d}{S_1Q_m N^2} \left(\frac{1}{2} + \frac{\sqrt{0.852}}{3N} \right) \quad (471)$$

$$\frac{dL}{dQ} = \frac{x_d}{Q_m} \quad (472)$$

- Note that $dH/dQ \neq J$
- The ground surface (assuming a constant slope, S_o) is defined by:

$$H = S_o L = S_o x_d \left(\frac{Q}{Q_m} \right) \quad (473)$$

and,

$$\frac{dH}{dQ} = \frac{S_o x_d}{Q_m} \quad (474)$$

- Combine the two equations defining H (this makes the friction loss curve just touch the ground surface):

$$S_o x_d \left(\frac{Q}{Q_m} \right) = (\Delta H_m)_a + \Delta E_m - h_f + \frac{JFL}{100} \quad (475)$$

- Solve the above equation for the inside diameter, D:

$$D = \left[\frac{100C^{1.852} \left(\frac{S_o x_d Q}{Q_m} - (\Delta H_m)_a - \Delta E_m \right)}{K \left(Q^{1.852} FL - Q_m^{1.852} F_{hf} x_d \right)} \right]^{-0.205} \quad (476)$$

- Set the slope of the friction loss curve equal to $S_o x_d / Q_m$,

$$\frac{S_o x_d}{Q_m} = \frac{1}{100} \left(FL \frac{dJ}{dQ} + JL \frac{dF}{dQ} + JF \frac{dL}{dQ} \right) \quad (477)$$

- 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:

$$D_1 \geq D \quad \& \quad \text{minimize}(D_1 - D) \quad (478)$$

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

$$H = S_t L = S_t x_d \left(\frac{Q}{Q_m} \right) \quad (479)$$

then,

$$S_t x_d \left(\frac{Q}{Q_m} \right) = (\Delta H_m)_a + \Delta E_l - h_f + \frac{JFL}{100} \quad (480)$$

- Set the slope of the friction loss curve equal to $S_t x_d / Q_m$,

$$\frac{S_t x_d}{Q_m} = \frac{1}{100} \left(FL \frac{dJ}{dQ} + JL \frac{dF}{dQ} + JF \frac{dL}{dQ} \right) \quad (481)$$

- 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);

$$H = H_0 + \frac{JFL}{100} \quad (482)$$

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 ,

$$H_0 = S_t x_d \left(\frac{Q}{Q_m} \right) - \frac{JFL}{100} \quad (483)$$

- Again, set the slope of the friction loss curve equal to S_t ,

$$\frac{S_t x_d}{Q_m} = \frac{1}{100} \left(FL \frac{dJ}{dQ} + JL \frac{dF}{dQ} + JF \frac{dL}{dQ} \right) \quad (484)$$

- 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:

$$H_{\text{big}} - H_{\text{small}} + \frac{FLK}{100} \left(\frac{Q}{C} \right)^{1.852} \left(D_{\text{big}}^{-4.87} - D_{\text{small}}^{-4.87} \right) = 0 \quad (485)$$

where, for the first pipe size (D_1):

$$H_{\text{big}} = (\Delta H_m)_a + \Delta E_l - h_f \quad (486)$$

and for the second pipe size (D_2):

$$H_{\text{small}} = H_0 \quad (487)$$

and F & L are as defined in Eqs. 437 to 439.

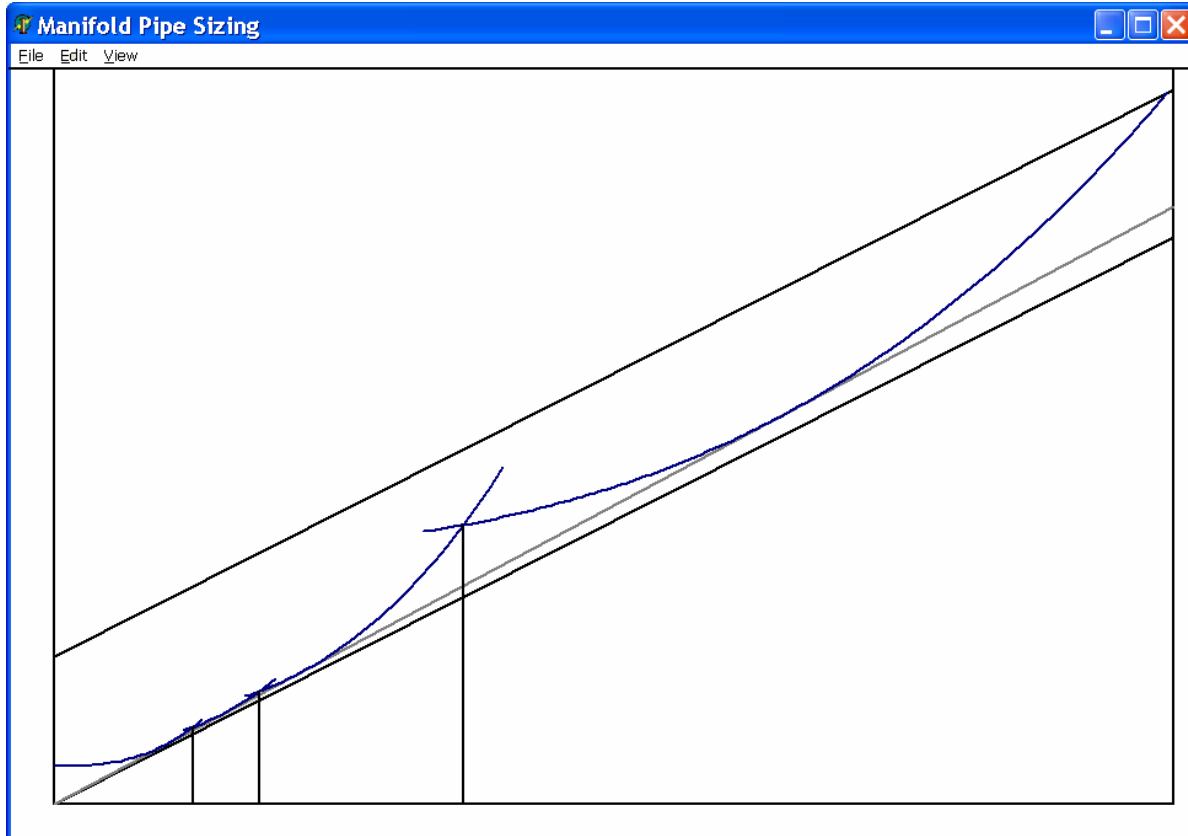
- Then, the length of pipe D_1 is equal to:

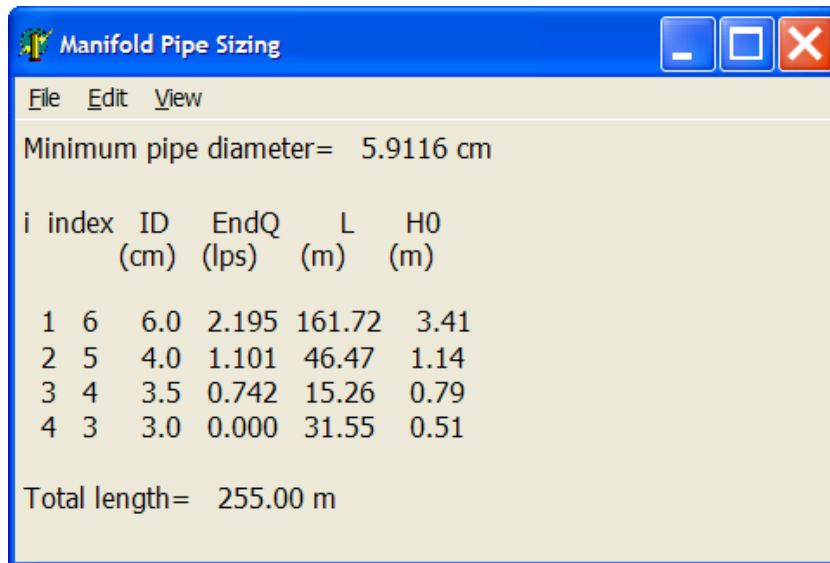
$$L_{D1} = x_d \left(1 - \frac{Q}{Q_m} \right) \quad (488)$$

- Continue this process until you have three or four pipe sizes, or until you get to a pipe size that has $D < \frac{1}{2}D_1$

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 $5S_i$
- 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





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, v
 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 have 1-5 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:

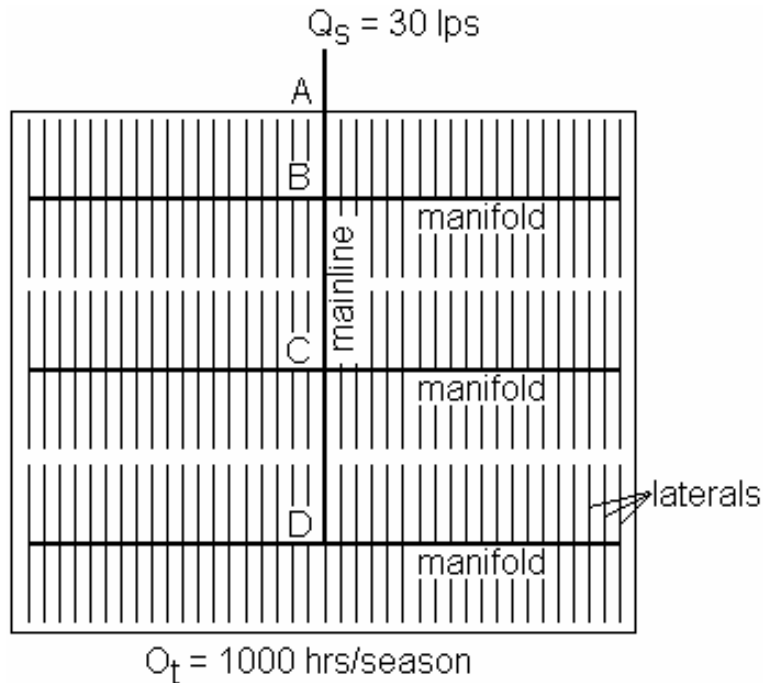
1. All laterals at the same time, or
2. Only laterals on one manifold at a time, or
3. Only laterals on one half of a manifold (on one side of the mainline) at a time

- 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

