11 Open Channel Flow

Open channels can range from small irrigation furrows to huge irrigation canals that are hundreds of kilometers long and supply billions of cubic meters per year for irrigation, industry, and domestic purposes. Agricultural canal categories include the irrigation district main, secondary and tertiary canals, laterals, on-farm irrigation ditches, and drainage channels. This chapter covers the structures and principles that are related to open channel delivery of water to agriculture: water diversion structures, conveyance efficiency, siphons, canal hydraulics (uniform flow, energy drop structures, and gradually varied flow), and flow measurement. The government has encouraged farmers to conserve water by lining irrigation ditches with concrete (Fig. 11.1); however, concrete channels can develop cracks and gaps that have excessive water loss. Economic analysis can determine whether lining a canal is worth the cost. Manning's equation calculates the head loss along a canal based on slope, roughness, and channel geometry. Energy dissipation structures use supercritical flow and hydraulic jumps to dissipate energy. The Froude number determines the relationship between subcritical and supercritical flow. A finite difference solution calculates water depth changes along a canal with gradually varied flow.

Water Diversion

Properly designed diversion structures are an important part of irrigation canal systems. Water is backed up behind a diversion structure such as a dam and then flows by gravity to a different field or region. Many river valleys have canal systems that take water out of the river at the upper end of the valley, and the canal, having a shallower grade than the river, is able to irrigate farms along the sides of the valley.

Many irrigation districts begin with large diversion structures that direct it into their main canal. For example, the Imperial Diversion Dam (Fig. 11.2) in southeastern California diverts water from the Colorado River into the All-American Canal, which delivers water to the Imperial Irrigation District and to San Diego. The dam increases the water surface elevation (head) and diverts water into the diversion canal. The diversion has large desilting basins, without which the canal system and farm furrows would rapidly fill with silt.

Diversions within canals generally have downstream control structures to control the upstream water level (Fig. 11.3).

The same technique is used to divert water from small streams to on-farm irrigation systems. A small dam is constructed, sometimes with a fish ladder, just downstream from the turnout (Fig. 11.4). There are many methods to divert water from irrigation ditches into fields, such as slide gates over small tubes (Fig. 11.1), slide gate turnouts or alfalfa valves over spiles (Fig. 11.5) or siphon tubes over the canal bank (Fig. 11.6).

Water also needs to be collected from the ends of fields and carried away from the field. Drainage channels and tailwater recovery channels (Fig. 11.6) are normally installed below grade in order to remove drainage water from the field by gravity flow (Fig. 11.7).

Conveyance Efficiency

Irrigation canals lose water through seepage and evaporation, which reduces conveyance efficiency. The conveyance efficiency is the amount of water delivered by a canal divided by the amount of water diverted into the canal. The water duty is the percent of water lost from the canal.

P. Waller, M. Yitayew, *Irrigation and Drainage Engineering*, DOI 10.1007/978-3-319-05699-9_11

Fig. 11.1 Square and trapezoidal irrigation ditches (Credit NRCS)

Fig. 11.2 Imperial diversion dam (Courtesy of United States Bureau of Reclamation)

Example 11.1 An irrigation district must divert 20 cm (8 in) depth of water into a lateral in order to deliver 15 cm (6 in) depth of water to a field. What are the efficiency and water duty?

$$\text{Efficiency} = 15/20(100\%) = 75\%$$
$$\text{Water duty} = 5/20(100\%) = 25\%$$

Evaporation from canals and reservoirs is approximately the same as reference ET_0. The volume of water lost per day is the product of canal water surface area and ET_0.

Fig. 11.3 Headgate on irrigation district lateral canal and downstream control structure

Fig. 11.4 Irrigation turnout upstream from dam in stream

Example 11.2 Reference evapotranspiration is 12 mm/day in summer and 4 m/year. Calculate the evaporation per day in summer and per year from a 200 m long, 1 m wide canal.

$$V_{\text{Evap/day}} = (200\,\text{m})\ \ (1\,\text{m})\ \ (0.012\,\text{m/day}) = 1.2\ \text{m}^3/\text{day}/1,233 = 0.001\,\text{ac} - \text{ft/day}.$$
$$V_{\text{Evap/yr}} = (200\,\text{m})\ \ (1\,\text{m})\ \ (4\,\text{m/year}) = 800\ \text{m}^3/\text{year}/1,233 = 0.65\ \ \text{ac} - \text{ft/day}.$$

Calculate the evaporation per day in summer and per year for a 540 km long, 25 m wide canal. What is the water duty if the canal delivers 1.85 billion m^3/year, and seepage losses are 3 %?

Fig. 11.5 Slide gate in farm irrigation ditch bank with energy dissipation blocks to prevent erosion (*left*) and spile (*right*)

Fig. 11.6 Siphon tubes over irrigation ditch bank (Credit NRCS Jeff Vanuga)

Fig. 11.7 Tailwater recovery channel (Credit NRCS Jeff Vanuga)

$$V_{Evap/day} = (540{,}000\,m)\ (25\,m)\ (0.012\,m/day) = 162{,}000\,m^3/day/1{,}233 = 131ac - ft/day$$
$$V_{Evap/yr} = (540{,}000\,m)\ (25\,m)\ (4\,m/yr) = 54{,}000{,}000\,m^3/year/1{,}233 = 44{,}000\ ac - ft/year$$

The percent evaporation is (54,000,000/1.85 × 10^9) (100 %) = 3 %.

Total water duty = % seepage losses + % evaporation losses = 3 % + 3 % = 6 %.

Canals may be lined in order to reduce seepage. Lining canals is especially important in permeable (sandy) soils. The *NRCS NEH 7* describes canal lining materials as follows:

> *Ditch lining materials include compacted soil, high expanding colloidal clay (bentonite), hand formed nonreinforced or reinforced concrete, slip formed nonreinforced concrete, pneumatic applied concrete mortar (gunnite), cold spray-applied membrane, and flexible membranes of plastic, elastomeric, or butyl rubber. Flexible membranes should be protected from physical damage and ultraviolet light by covering with aggregate or soil. Flexible membranes with concrete or aggregate protection can be installed underwater if the water velocity is less than 5 feet per second.*

Lining canals may have political ramifications if seepage to groundwater is reduced. One person's lost seepage water is another person's irrigation water. For example, the Imperial Irrigation District in California estimates that lining a 23-mile section of the All-American Canal that passes through sand dunes will save 70,000 acre-ft/yr (86 million m^3/yr). The Metropolitan Water District in Los Angeles will pay the Imperial Irrigation District to line the canal and divert the saved water for urban use. The problem with lining the canal, however, is that Mexican farmers just across the border depend on the seepage water from the All-American canal to recharge aquifers from which they pump irrigation water. They claim that this water is rightfully theirs because the canal has been seeping since it was completed in 1942, and they have developed farms that depend on the canal seepage. However, the seepage volume is not mentioned in the treaty signed in 1944 between the

Table 11.1 Canal seepage rates reported in published studies (After Leigh and Fipps (2009))

Lining/soil type	Seepage rate (gal/ft^2/day)	Seepage rate (L/m^2/day)
Unlined[a]	2.21–26.4	90–1,076
Portland cement[b]	0.52	21
Compacted earth[b]	0.52	21
Brick masonry lined[c]	2.23	91
Earthen unlined[c]	11.34	462
Concrete[d]	0.74–4.0	30–163
Plastic[d]	0.08–3.74	3.2–152
Concrete[d]	0.06–3.22	2.4–131
Gunite[d]	0.06–0.94	2.4–38
Compacted earth[d]	0.07–0.6	2.9–24
Clay[d]	0.37–2.99	15–122
Loam[d]	4.49–7.48	183–305
Sand[d]	4.0–19.45	163–792
Clay[e]	1.5	61
Silty clay loam[e]	2.24	91
Clay loam[e]	2.99	122
Silt loam earth[e]	4.49	183
Loam[e]	7.48	305
Fine sandy loam[e]	9.35	381
Sandy loam[e]	11.22	457

[a]DeMaggio (1990)
[b]U.S. Bureau of Reclamation (1963)
[c]Nayak et al. (1996)
[d]Nofziger (1979)
[e]Texas Board of Water Engineers (1946)

United States and Mexico, which guaranteed 1.5 MAF (million acre-ft) of Colorado River water per year to Mexico.

Leigh and Fipps (2009) did a literature study and compiled a table with typical canal seepage rates as a function of lining and soils (Table 11.1).

Canal seepage can be measured by closing the ends of a canal and measuring the rate of decline of the canal water surface (the best method), by measuring the inflow and outflow to a canal, or, in an earthen canal, by measuring seepage rates with an infiltrometer.

Canal seepage rates in lined canals can be higher than seepage rates in unlined canals because nonreinforced concrete lined canals crack (Fig. 11.8).

In studies in Texas conducted by Fipps, seepage losses in old concrete lined small to medium width (1–3 m wide) canals were in the range of 15–24 gal/ft^2/day (600–960 L/m^2/day); larger canals, with better construction techniques (reinforced concrete) generally had much lower seepage losses – in the range of 1–4 gal/ft^2/day (40–160 L/m^2/day). In the same study, earthen canals had seepage rates less than 2 gal/ft^2/day (80 L/m^2/day); to be fair, it should be noted that these unlined canals were constructed in less permeable soils. Another advantage to the unlined canals was that

Fig. 11.8 Cracked concrete lined canal (Credit NRCS, Ron Nichols)

silting of the canal bottoms had decreased the rate of infiltration into the soil (Fipps G, 2005, Personal communication). In the same study by Fipps (2005, Personal communication), irrigation district conveyance efficiencies in an old irrigation district ranged from 60 % to 75 %.

Lining canals in order to prevent infiltration is expensive. Lining a 75 cm deep canal with a 30 cm wide bottom can cost $60,000/km, and lining a major canal can cost $600,000/km.

Total water requirement for an irrigation district is the farm water requirement divided by the total district efficiency. The total efficiency of an irrigation district includes the storage efficiency of the reservoir, the conveyance efficiency of the canal system, flexibility factors (percent extra water) required for multiple farm delivery, irrigation delivery waste at turnouts due to lack of timing of water use and water delivery, and on-farm irrigation efficiency.

Example 11.3 Calculate the conveyance efficiency to field 1 in Fig. 11.5 from the point of water diversion to the irrigation district. The conveyance efficiency of the irrigation district up to the farm turnout is 70 %. The main concrete canal on the farm has a conveyance efficiency of 90 % and the earth-lined canal has a conveyance efficiency of 80 %

The total conveyance efficiency from the point of diversion to the irrigation district to field 1 is (Fig. 11.9)

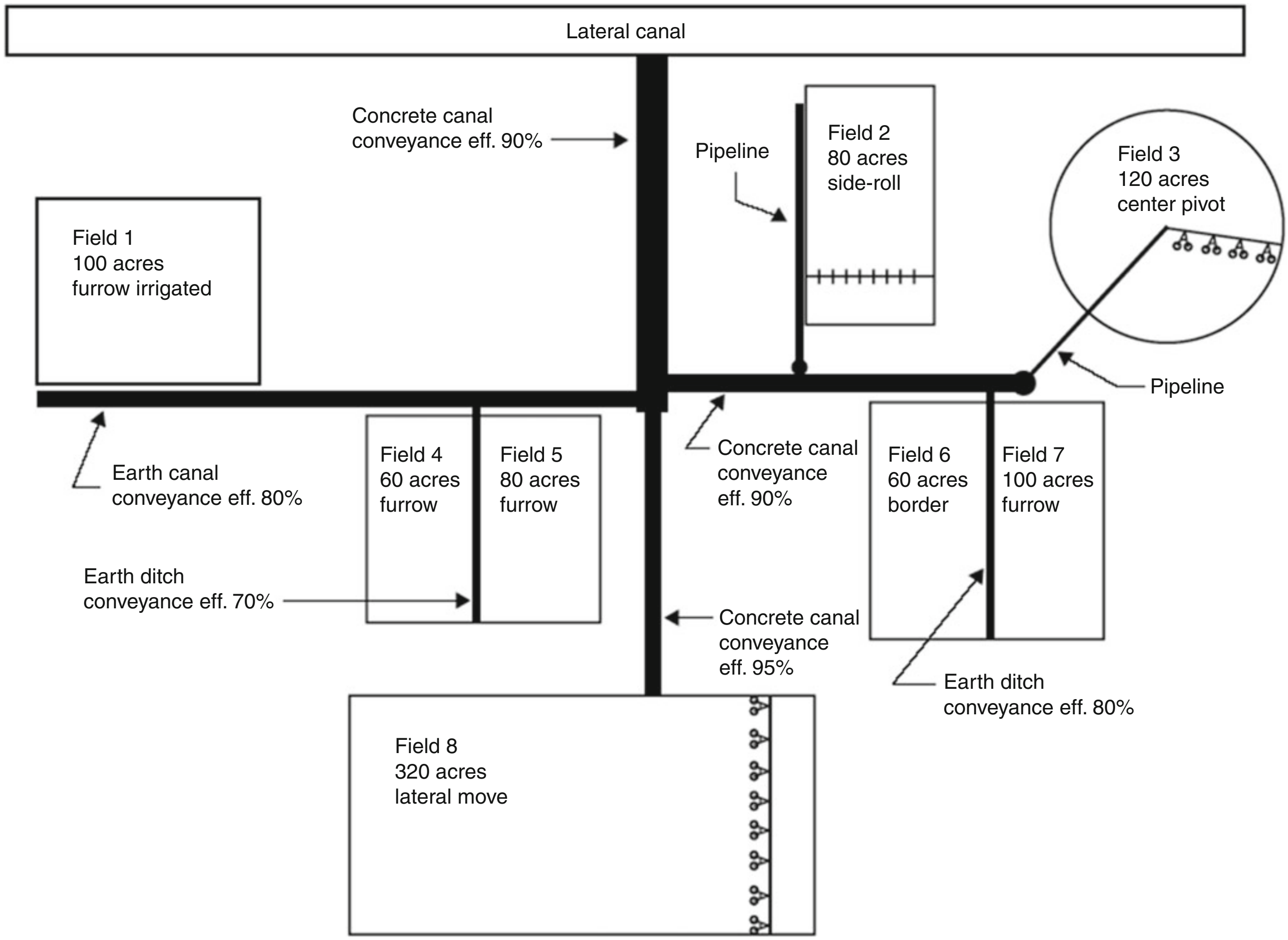


Fig. 11.9 Farm water conveyance system (Credit NRCS. National Irrigation Guide, Part 652, Chap. 7. http://www.irrigationtoolbox.com/NEH/Part652_NationalIrrigationGuide/ch7.pdf)

$$\text{Eff} = (0.7)\ (0.9)\ (0.8)\ (100\%) = 50\%$$

The wetted perimeter of a canal determines the area of infiltration (seepage). Most canals have a trapezoidal shape (Fig. 11.10). The wetted perimeter of a trapezoidal canal is

$$P = b + 2s = b + 2y\left(1 + z^2\right)^{0.5} \qquad (11.1)$$

where

P = wetted perimeter, m
z = side slope (run over rise)
b = channel bottom width, m
y = canal water depth, m.

Example 11.4 A farm irrigation ditch has a trapezoidal shape and has a 30 cm wide bottom, 2:1 side slopes (2 in the horizontal direction and one in the vertical direction, z = 2) and is 75 cm deep. It is filled with water for 60 days per year, and it is located in a silt loam soil. The irrigation ditch is 1 km long. The cost of irrigation water is \$3.27/ha-cm. The project life is 20 years, and the required rate of return is 8 %. The cost of lining the ditch is \$60,000/km. Determine whether to line the irrigation ditch.

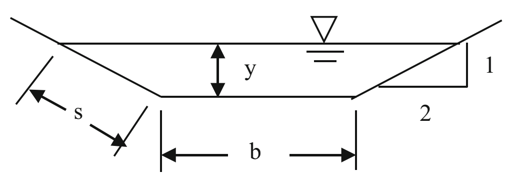


Fig. 11.10 Trapezoidal canal

The wetted perimeter is

$$P = b + 2{*}y{*}\left(1 + z^2\right)^{0.5} = 0.3 + 2{*}0.75{*}\left(1 + 2^2\right)^{0.5} = 3.65\ \text{m}$$

In a silt loam soil, the expected infiltration (seepage) rate is 183 L/m^2/day (Table 11.1)

$$V_{Seepage/day} = 3.65 \text{ m}*1,000 \text{ m}*183 \text{ L/m}^2\text{/day} = 660,000 \text{ L/day/km}$$
$$V_{Seepage/yr} = 660,000 \text{ L/day}*60 \text{ days}*0.001 \text{ m}^3\text{/L}*0.01\text{ha} - \text{cm/m}^3 = 400 \text{ ha} - \text{cm/yr}$$
$$\$_{Seepage/yr} = 400 \text{ ha} - \text{cm/yr}*\$3.27\text{/ha} - \text{cm} = \$1,300\text{/year.}$$

The present value of $1,300/yr for a 20 yr project at 8 % interest is approximately $13,000. Therefore, it is not profitable to line the irrigation ditch. However, if the ditch was always filled, then the present value of the cost of seepage would be 6 * $13,000 = $78,000, greater than the lining cost.

Pipes and Siphons in Canal Systems

Pipes can be used to get past obstructions or canyons and yet maintain the same head (elevation). For example, the Roza Irrigation District in the lower Yakima Valley in Washington State delivers water from the Roza dam on the Yakima River in the upper Yakima Valley. The canal is kept at a high elevation along the Rattlesnake Hills. In order to maintain this elevation, five siphons, having a total length of 7 km and averaging 5 m in diameter, bring the Roza irrigation water under five rivers (valleys) and preserve the elevation energy in the canal. Because the canal is at a high elevation above many of the farms it serves, the district is able to deliver water to farms through pipes and maintain a high enough pressure (>210 kPa) for farmers to run sprinklers with low pressure nozzles without a pump.

A partially completed concrete siphon on the Salt River Project Canal in Phoenix, Arizona is shown in Fig. 11.11.

Fig. 11.11 Partially completed concrete siphon on Salt River Project (Credit USBR)

Fig. 11.12 Culvert across drainage ditch

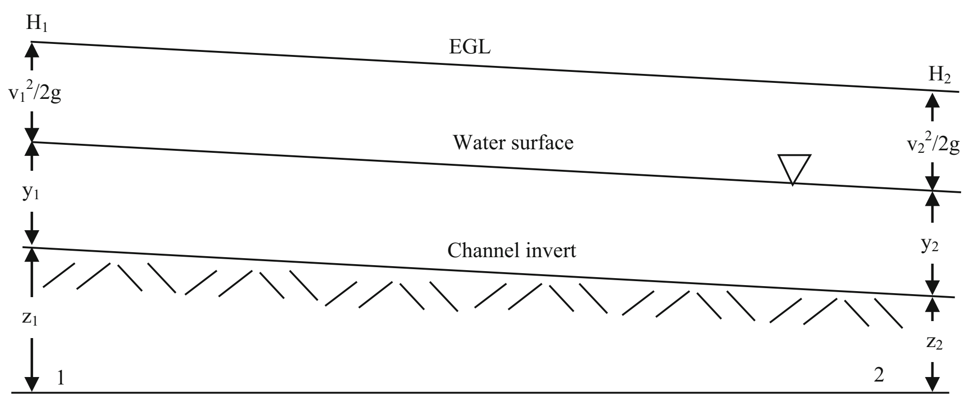


Fig. 11.13 Uniform open channel flow

Smaller siphons are also needed to transfer water past obstructions in smaller irrigation canals: under or over rivers, drainage channels, canals, and roads. Smaller canals can use a corrugated metal pipe to traverse drainage channels or roads (Fig. 11.12).

Canal Hydraulics: Steady-State, Uniform Flow

Canal flow is characterized as uniform, non-uniform, steady, and/or unsteady flow. Uniform flow is characterized by uniform depth along a channel ($y_1 = y_2$ in Fig. 11.13). Steady-state refers to the fact that flow does not change with time. Unsteady state flow is not covered.

The energy in open channel flow is the sum of the elevation of the channel, z, the depth of water flow, y (can be converted to pressure in Bernoulli's equation for a given depth in the channel), and the velocity head, $v^2/2$ g (Fig. 11.13). The energy grade line (EGL) is the sum of kinetic and elevation energy (Fig. 11.13).

$$H = y + \frac{v^2}{2g} + z \tag{11.2}$$

where

y = depth of flow in channel, m
v = average velocity of water flow in channel, m/sec
z = elevation of channel invert (bottom of channel), m.

Open channel flow is driven by inertial forces (momentum) and gravitational forces (differences in elevation of the water surface). For steady state (constant in time) and uniform (constant in space) flow in a channel, the water surface slope is the same as the channel slope (Fig. 11.13) and momentum is constant in time and space. Thus, the slope of the energy grade line (EGL) is equal to the slope of the channel. For uniform and steady-state flow in a channel, the flow velocity is determined by Manning's equation, which is a function of the channel roughness, energy grade line (equal to slope, S), and channel dimension (hydraulic radius).

$$\mathrm{v} = \frac{R^{2/3}S_0^{0.5}}{n} \quad Q = \frac{AR^{2/3}S_0^{0.5}}{n} \tag{11.3}$$

where

S_0 = channel slope, positive downward in direction of flow, m/m
n = Manning's roughness coefficient
v = velocity of flow in the channel, m/sec.
Q = flow in channel, m^3/sec
A = cross-sectional area of channel.

The hydraulic radius, R, is representative of the length dimension of the channel perpendicular to the direction of flow. It is calculated as the cross-sectional area of the channel divided by the wetted perimeter.

$$\mathrm{R = A/P} \tag{11.4}$$

The slope is the difference in elevation between two points (Fig. 11.13) divided by the distance between the points. For uniform flow, the difference in elevation is also equal to the friction loss, H_f.

$$S_0 = \frac{z_2 - z_1}{L\cos\theta} = \frac{H_f}{L} \tag{11.5}$$

where

z_1 = Elevation energy at upper end of channel, m
z_2 = Elevation energy at lower end of channel, m
L = Length of channel, m
θ = channel slope, degrees.

Equation 11.3 can be rearranged to solve for slope.

$$S_0^{0.5} = \frac{\mathrm{v}n}{R^{2/3}} \quad S_0 = \frac{\mathrm{v}^2n^2}{R^{4/3}} \tag{11.6}$$

Substitute Eqs. 11.5 into 11.6

$$H_f = L\frac{\mathrm{v}^2n^2}{R^{4/3}} = L\frac{\left(\frac{Q}{A}\right)^2n^2}{R^{4/3}} = L\frac{Q^2n^2}{A^2R^{4/3}} \tag{11.7}$$

Channel area is a function of the square of average channel dimension, D^2, and R is a function of D, thus

$$H_f = L\frac{Q^2n^2}{A^2R^{4/3}} = kL\frac{Q^2n^2}{D^{5.3}} \tag{11.8}$$

where

k = constant

The Manning's equation as written in Eq. 11.7 is similar to the to the Hazen-Williams equation.

Minimum, normal (design), and maximum values of the roughness coefficient, n, are tabulated in Table 11.2. The roughness coefficient increases surface roughness.

Geometric elements of typical channel cross sections are presented in Table 11.3.

When Manning's equation is solved in order to find required channel dimensions for a given flow rate, it is normally written as in Eq. 11.9. The right side of Eq. 11.9 is calculated first, and then an iterative solution is used to find geometric parameters on the left side (section factor) of the equation. After the design depth of flow is found, 25 % freeboard elevation is normally added to the channel depth.

$$AR^{2/3} = \frac{Qn}{S_0^{0.5}} \tag{11.9}$$

Example 11.5 A concrete lined irrigation ditch (Fig. 11.14) has a slope of 0.2 % = 0.002 m/m. Flow rate in the channel is 100 L/sec, the Manning's roughness coefficient, n, of the channel is 0.013. Calculate the depth of flow in the channel. The channel is a trapezoidal channel with bottom width of 0.3 m and side slopes of 1:1 (z = 1).

Solve for left side of Eq. 11.9. The following procedure is in the *Canal Depth* Worksheet.

$$\frac{Qn}{S_0^{0.5}} = \frac{0.1*0.013}{0.002^{0.5}} = 0.029$$

Use iteration to find y. First try y = 18 cm = 0.18 m.

$$\begin{aligned} AR^{2/3} &= (b+zy)y\left(\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}\right)^{2/3} \\ &= (0.3+(1)(0.18))(0.18)\left(\frac{(0.3+(1)(0.18))(0.18)}{0.3+(2)(0.18)\sqrt{1+1^2}}\right)^{2/3} \\ &= 0.01945 \end{aligned}$$

Adjust y as follows (this is a fast iteration procedure).

$$y_2 = y_1\left(\frac{\left(AR^{2/3}\right)_{actual}}{\left(AR^{2/3}\right)}\right)^{0.5} = 0.18\left(\frac{0.029}{0.01945}\right)^{0.5} = 0.2201 \ m$$

Table 11.2 Values of manning's n for various channel and agricultural surfaces (Chow 1959)

Type of surface	Minimum	Maximum	Normal
a. Corrugated metal culvert flowing partly full	0.021	0.030	0.024
b. Lined channels			
1. Metal smooth surface – unpainted	0.011	0.014	0.012
2. Metal smooth surface – painted	0.012	0.017	0.013
3. Concrete – trowel finish	0.011	0.015	0.013
4. Concrete – float finish	0.013	0.016	0.015
5. Concrete – unfinished	0.014	0.020	0.017
6. Concrete – finished with gravel on bottom	0.015	0.020	0.017
7. Corrugated metal			
c. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.020	0.018
2. Clean, after weathering	0.018	0.025	0.022
3. Gravel, uniform section, clean	0.022	0.030	0.025
4. With short grass, few weeds	0.022	0.033	0.027
d. Earth, winding and sluggish			
1. No vegetation	0.023	0.030	0.025
2. Grass, some weeds	0.025	0.033	0.030
3. Dense weeds or aquatic plants in deep channels	0.030	0.040	0.035
4. Earth bottom and rubble sides	0.025	0.035	0.030
5. Stony bottom and weedy sides	0.025	0.045	0.035
6. Cobble bottom and clean sides	0.030	0.050	0.040
e. Dragline-excavated or dredged			
1. No vegetation	0.025	0.033	0.028
2. Light brush on banks	0.035	0.060	0.050
f. Rock cuts			
1. Smooth and uniform	0.025	0.040	0.035
2. Jagged and irregular	0.035	0.050	0.040
g. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.120	0.080
2. Clean bottom, brush on sides	0.040	0.080	0.050
3. Same, highest stage of flow	0.045	0.110	0.070
4. Dense brush, high stage	0.080	0.140	0.100
h. Agricultural surfaces			
1. Smooth, bare soil surfaces and furrows, SCS (1974, 1984)			0.04
2. Small-grain crops with drill rows in the direction of water flow (SCS)			0.10
3. Alfalfa, broadcast small grains, SCS (1974)			0.15
4. Dense alfalfa or alfalfa on long fields with no ditches, Clemmens (1991)			0.20
5. Dense sod crops and small grains drilled perpendicular to the flow (SCS)			0.25

Another iteration results in 0.2224. Thus, final answer is 0.22 m = 22 cm water depth.

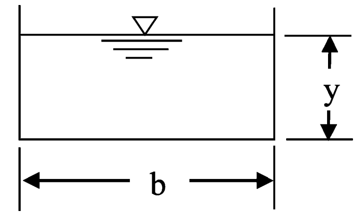
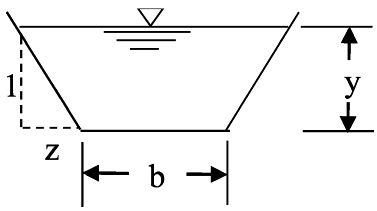
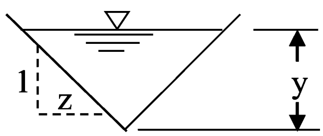
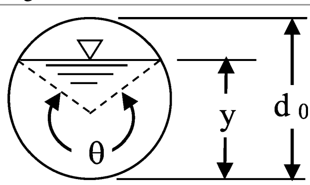
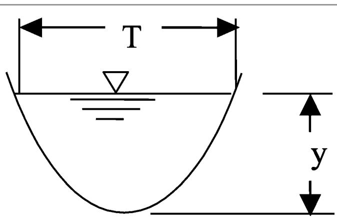
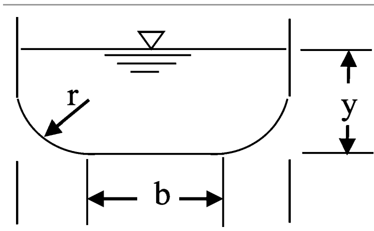
Add 25 % freeboard elevation.

$$0.22*1.25 = 0.28 \text{ m}$$

Canal Hydraulics: Energy Dissipation

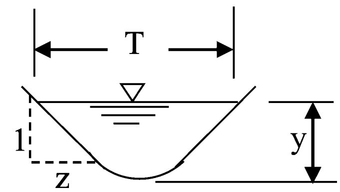
Irrigation canals and channels generally have long stretches of uniform subcritical flow in almost level canals. However, if the natural land slope is greater than the maximum subcritical flow slope, then periodic energy dissipation structures are needed. Energy dissipation structures may also be needed downstream from a diversion dam. Hydraulic jumps often take place at the bottom of energy dissipation structures as the flow transitions from supercritical to subcritical flow. This is explained by the fact that the energy in open channel flow is the sum of the elevation potential energy (y = water surface elevation) and kinetic energy (v). Flow can either have high elevation energy (y_2 in Fig. 11.15) or high kinetic energy (y_1 in Fig. 11.15). The Froude diagram shows the relationship between depth and energy (Fig. 11.15).

Table 11.3 Geometric elements of channel sections (After Cuenca (1989) from Chow (1959))

Section	Area A	Wetted perimeter, P	Hydraulic radius R	Top widthT
 Rectangle	by	b + 2y	$\frac{by}{b+2y}$	b
 Trapezoid	(b + zy)y	$b+2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	b + 2zy
 Triangle	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2y\sqrt{1+z^2}}$	2zy
 Culvert	$\frac{1}{8}(\theta-\sin\theta)d_0^2$	$\frac{1}{2}(\theta)d_0$	$\frac{1}{4}\left(1-\frac{\sin\theta}{\theta}\right)d_0$	$\left(\sin\frac{\theta}{2}\right)d_0$
 Parabola	$\frac{2}{3}Ty$	$T+\frac{8}{3}\frac{y^2}{T}$ Satisfactory for $0 < x < 1$ where $x = 4y/T$	$\frac{2T^2y}{3T^2+8y^2}$	$\frac{3A}{2y}$
 Round-cornered rectangle	$\left(\frac{\pi}{2}-2\right)r^2+(b+2r)y$	$(\pi-2)r+b+2y$	$\frac{\left(\left(\frac{\pi}{2}-2\right)r^2+(b+2r)y\right)}{\left((\pi-2)r+b+2y\right)}$	b + 2r

(continued)

Table 11.3 (continued)

Section	Area A	Wetted perimeter, P	Hydraulic radius R	Top widthT
 Round-bottomed triangle	$\frac{T^2}{4z} - \frac{r^2}{z}*\left(1 - z \ \cot^{-1} z\right)$	$\frac{T}{z}\sqrt{1+z^2} - \frac{2r}{z}\left(1 - z \ \cot^{-1} z\right)$	A/P	$2\left[z(y-r) + r*\sqrt{1+z^2}\right]$

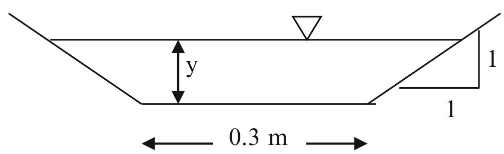


Fig. 11.14 Trapezoidal channel cross-section

$$H = y + \frac{v^2}{2g} \quad (11.10)$$

A hydraulic jump takes place when flow jumps from high kinetic energy to high elevation energy. For low slopes with low velocity and high depth (the norm in open channels), the elevation energy is greater; however, for steep slopes with low depth and high velocity, the kinetic energy is greater. If velocity is high and depth is low, then flow is called supercritical and is in the lower range of Fig. 11.15. If velocity is low and depth is high, then flow is called subcritical, and is in the upper range of Fig. 11.15.

The Froude number, Fr, is the dimensionless ratio of a momentum (velocity) force to the gravitational force. Flow is said to be critical if the Froude number is equal to one, subcritical if the Froude number is less than one, and supercritical if the Froude number is greater than one. The general form of the Froude number, which is applicable for channels with vertical sides is as follows:

$$F = \frac{\text{v}}{(\alpha y g)^{0.5}} \quad (11.11)$$

where

α = velocity distribution coefficient, dimensionless.

The α term is necessary because total energy in a channel is not a linear function of velocity because of the variation in flow with distance from the channel wall. The velocity distribution coefficient in Eq. 11.11, α, is the ratio between the actual kinetic energy in a channel and kinetic energy calculated with average velocity. It is assumed to be one in irrigation channels with turbulent flow.

The Froude number can be written as follows for all open channels (non-vertical sides):

$$Fr^2 = \frac{Q^2 T}{A^3 g} \quad Fr = \frac{\text{v}}{\left(A/Tg\right)^{0.5}} \quad (11.12)$$

where

Q = total flow in channel, m^3/s
T = channel top width, m
A = channel cross-sectional area, m^2.

If the slope of the channel is steep in the direction of flow, then velocity is high with respect to the depth, the Froude number is greater than 1, and flow is supercritical; thus, canals must be designed with a small slope so that the Froude number is less than 1, and flow is subcritical. Irrigation canals and ditches are designed to operate at subcritical flow for two reasons. In earth-lined canals and ditches, flow must be subcritical in order to prevent erosion. In concrete lined irrigation canals and ditches, it would be impossible to have energy control structures and diversions if the channel operated at supercritical flow because flow would change from supercritical to subcritical flow if the water was slowed at diversion structures.

Energy drop structures are classified as vertical (hard) drops or sloping drops (Fig. 11.16). Five energy drop structures used in irrigation canals are shown in Fig. 11.17. Problems with vertical drop structures include sedimentation and maintenance of the impact basin, and erosion

Fig. 11.15 Froude diagram (Wikipedia)

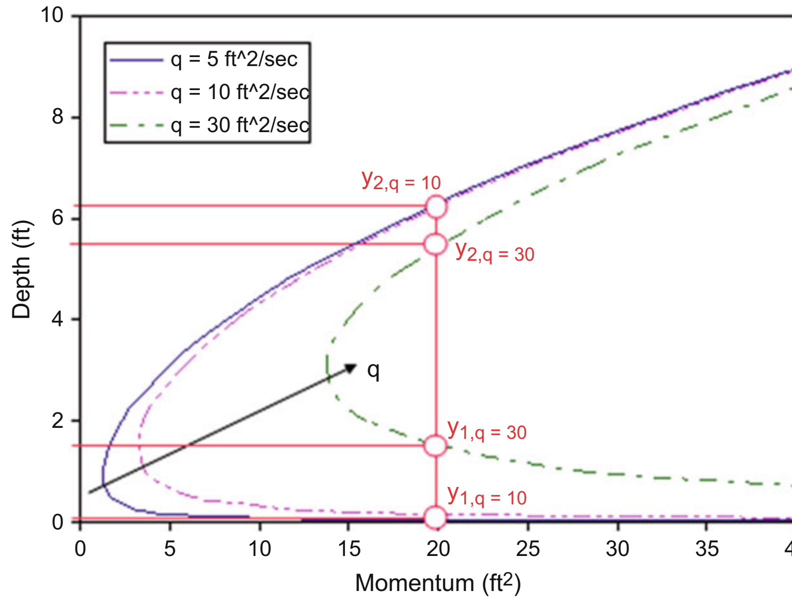


Fig. 11.16 Baffle chute energy drop structure (Wikipedia)

downstream from the impact basin. Sloped drop structures either dissipate energy with rip rap or concrete blocks along the slope or at the base of the sloped section with a hydraulic jump.

If energy is dissipated with a hydraulic jump (Fig. 11.18) at the base of a sloped drop structure, then the structure must be carefully designed. Energy is lost in the turbulence of a hydraulic jump; however, momentum is conserved. Thus, the depth of flow and velocity of flow after a hydraulic jump can be found with the conservation of momentum equation. If there is no change in flow direction, then the summation of forces before and after the hydraulic jump is the same.

$$\rho Q_1 v_1 + P_1 = \rho Q_2 v_2 + P_2 \tag{11.13}$$

$$\begin{aligned}\rho Q_1 \frac{Q_1}{A_1} + \gamma \bar{y}_1 A_1 &= \rho Q_2 \frac{Q_2}{A_2} + \gamma \bar{y}_2 A_2 \\ \rho \frac{Q^2}{gA_1} + \bar{y}_1 A_1 &= \rho \frac{Q^2}{gA_2} + \bar{y}_2 A_2\end{aligned} \tag{11.14}$$

For a rectangular channel, Eq. 11.14 can be written in terms of flow per unit width, q.

$$\rho \frac{q^2}{gA_1} + \bar{y}_1 A_1 = \rho \frac{q^2}{gA_2} + \bar{y}_2 A_2 \tag{11.15}$$

Equation 11.15 can be rearranged and written in terms of the Froude number for a rectangular channel.

$$\frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1 + 8Fr_1^2}\right) \tag{11.16}$$

$$Fr_1 = \frac{q}{y_1 \sqrt{g y_1}} \tag{11.17}$$

where

Fr_1 = upstream Froude number for a rectangular channel.

The amount of energy lost in the hydraulic jump depends on the upstream Froude number. A higher Froude number leads to a greater energy loss. An upstream Froude number between 4.5 and 9.0 maintains a stable hydraulic jump and high energy loss so this is the desired range of Froude number for upstream flow. Lower Froude numbers create

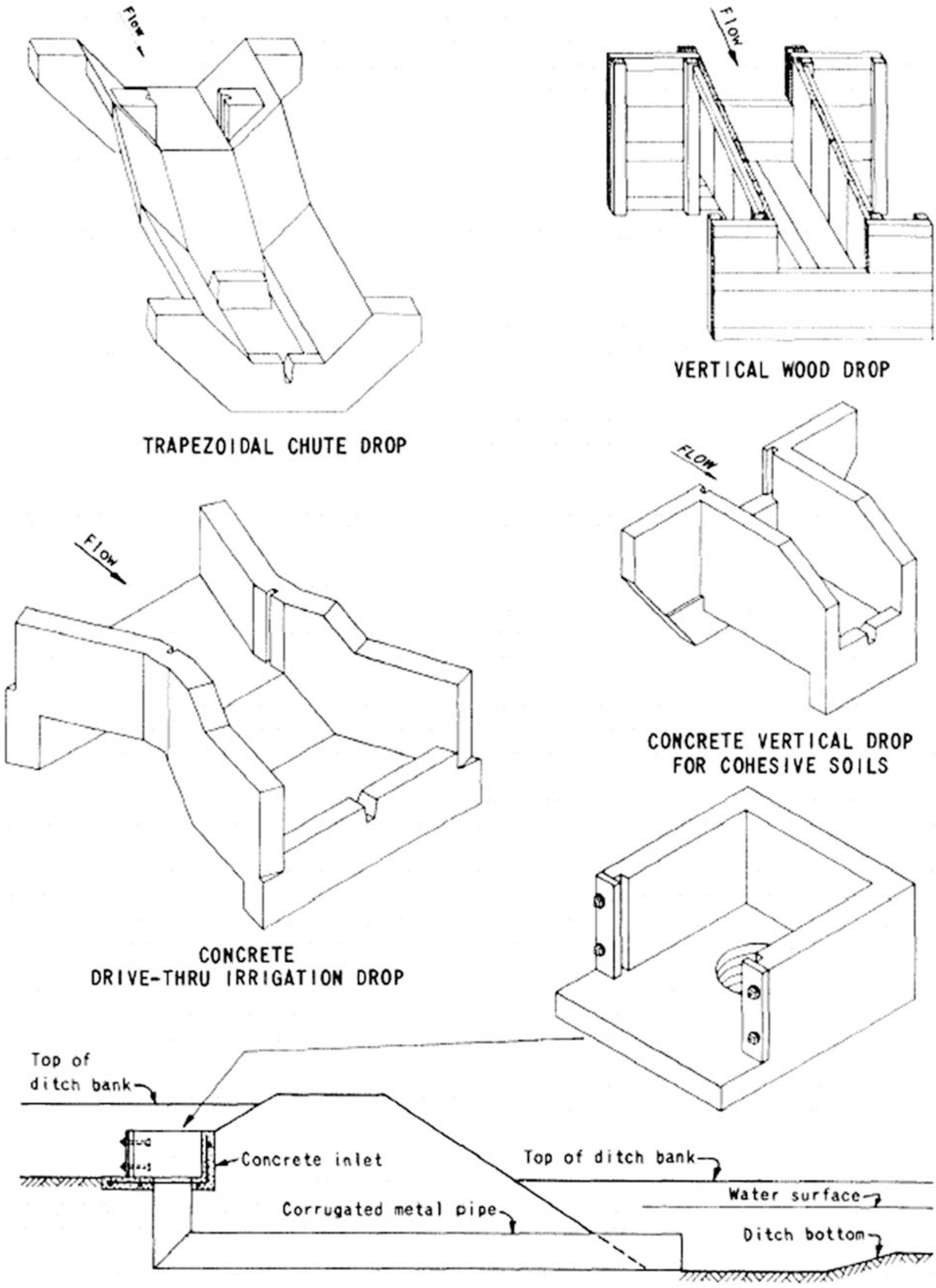


Fig. 11.17 Five energy dissipation structures (Credit NRCS)

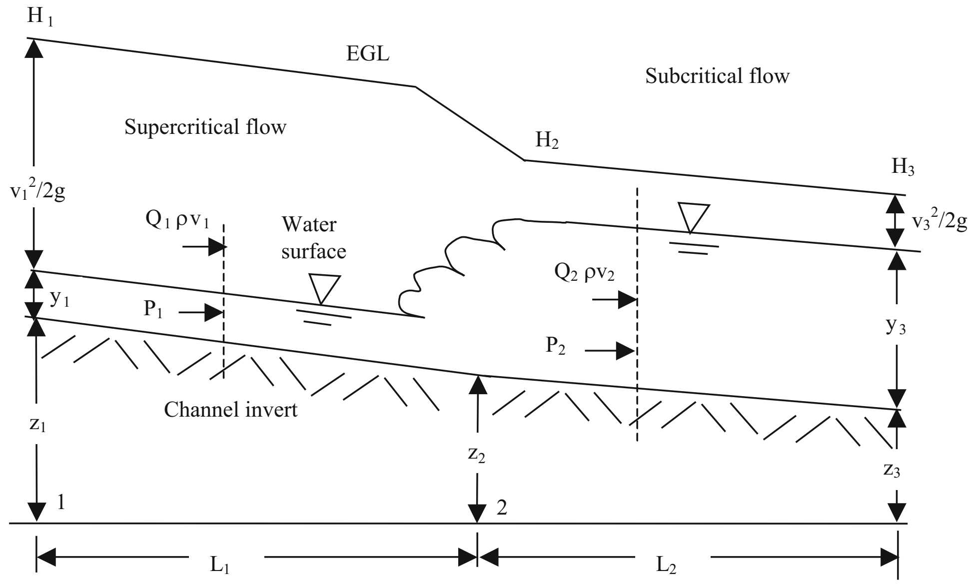


Fig. 11.18 Hydraulic jump

Table 11.4 Hydraulic jump characteristic versus upstream froude number (After Chin (2000))

Fr_1	Jump characteristic
1–1.7	Standing wave (undular jump); kinetic-energy loss (as a percentage of the upstream kinetic energy, $V_1^2/2\,g$) < 5 %
1.7–2.5	Smooth surface rise (weak jump); kinetic energy loss 5–15 %
2.5–4.5	Unstable oscillating jump, where each irregular pulsation creates a large wave that can travel far downstream, damaging earth banks and other structures; kinetic energy loss 45–70 %
4.5–9.0	Stable jump, best performance and action, and insensitive to downstream conditions (steady jump); kinetic energy loss 45–70 %
>9.0	Rough, somewhat intermittent (strong jump); kinetic energy loss 70–85 %

downstream instability. Table 11.4 lists energy loss and hydraulic jump wave behavior vs. Froude number.

Example 11.6 A sloped drop structure is to be placed along a channel in order to dissipate energy. The channel cross section is b = 0.3 and z = 2. Manning's n is 0.013. The flow is 100 L/s. Find the slope of the drop structure required to develop a Froude number equal to 6.0 before the hydraulic jump.

The equation for top width of a trapezoidal channel is b + 2 z y (Table 11.3). The equation for channel area is (b + z y) y. Substitute into Eq. 11.12 with Fr = 6.

Iterate to solve for y: 0.059 m.

Now find the slope. $Q = \frac{AR^{2/3}S_0^{0.5}}{n} \Rightarrow S_0 = \left(\frac{Qn}{AR^{2/3}}\right)^2 = \left(\frac{(0.10)\quad(0.013)}{(0.025)\quad(0.044)^{2/3}}\right)^2 = 0.178$

Thus, the bottom slope should be 17.8 %

One of the main reasons to carefully characterize the downstream depth of a hydraulic jump is to ensure that the channel walls are high enough to contain the jump. Equation 11.13 can be used to find downstream depth for rectangular channels as is shown in the following example.

Example 11.7 Repeat Example 11.6 for a rectangular channel with 0.3 m width. Find the downstream depth after the hydraulic jump.

$$Fr^2 = \frac{Q^2T}{A^3g}$$

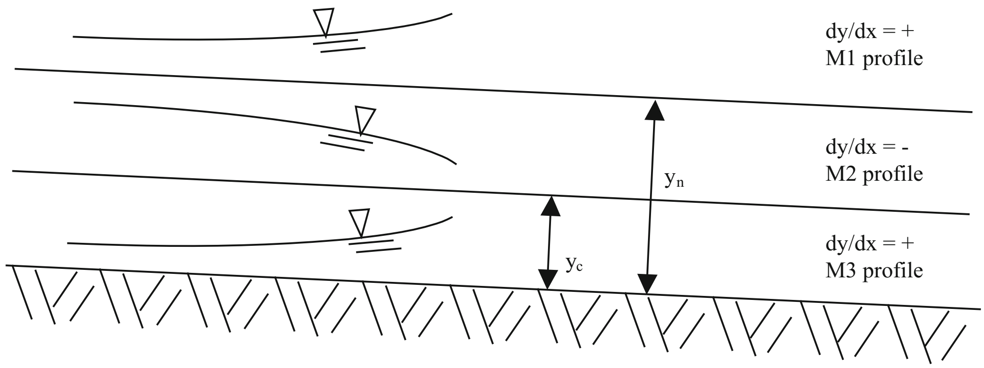
$$6^2 = 36 = \frac{Q^2(b)}{(by)^3g} \Rightarrow y = \sqrt[3]{\frac{Q^2b}{36b^3g}} = \sqrt[3]{\frac{0.1^2(0.3)}{36{*}0.3^3\ {*}9.8}}$$
$$= 0.068\ m$$

The same answer is found with Eq. 11.14.

$$q = Q/b = 0.1\ m^3/s/0.3\ m = 0.333\ m^2/\sec$$

$Fr_1 = \frac{q}{y_1\sqrt{gy_1}} = 6 = \frac{0.333}{y_1\sqrt{9.8y_1}}$ y_1 is found by iteration = 0.068 m

Fig. 11.19 Mild (M) channel



Find the slope

$$v = q/y = 0.333\ m^2/sec\ /\ 0.068\ m = 4.90\ m/sec$$

$$v = \frac{R^{2/3}S_0^{0.5}}{n} \Rightarrow S_0 = \left(\frac{vn}{\left(\frac{by}{b+2y}\right)^{2/3}}\right)^2 = \left(\frac{4.90*0.013}{\left(\frac{0.3*0.068}{0.3+2*0.068}\right)^{2/3}}\right)^2 = 0.245$$

$$Q = \frac{AR^{2/3}S_0^{0.5}}{n} \Rightarrow S_0 = \left(\frac{Qn}{AR^{2/3}}\right)^2 = \left(\frac{0.1*0.013}{(0.3*0.068)*0.0468^{2/3}}\right)^2 = 0.241$$

In this case, the bottom slope is 24 %, which is 6 % higher than for the trapezoidal channel in Example 11.6. Thus, channel geometry has an influence on the required slope needed to generate a sufficiently high Froude number.

The downstream water surface depth is found with Eq. 11.13. for a rectangular channel.

$$y_2 = y_1\frac{1}{2}\left(-1+\sqrt{1+8Fr_1^2}\right) = 0.068\frac{1}{2}\left(-1+\sqrt{1+8*6^2}\right) = 0.544\ m$$

Canal Hydraulics: Gradually Varied Flow

Canal velocity and depth can change gradually, rather than a hydraulic jump, upstream or downstream from an obstruction. This section gives an introduction to the calculation of gradually varying depth in a canal.

$$S_0 - S_f = \frac{\Delta\left(y + \alpha\frac{v^2}{2g}\right)}{\Delta x} \tag{11.18}$$

where

S_f = friction loss calculated with Manning's equation, also slope of EGL, m/m

Δx = distance down the channel, m.

For a rectangular channel, Eq. 11.11 can be substituted into Eq. 11.18 with Q /A substituted for v and written as follows:

$$\frac{dy}{dx} = \frac{y_2 - y_1}{x_2 - x_1} = \frac{S_0 - S_f}{1 - \alpha Fr^2} \tag{11.19}$$

Channel slopes are classified as mild (M), steep (S), critical (C), horizontal (H), and adverse (A). A mild slope is shown in Fig. 11.19. The normal depth is the depth of uniform and steady state flow. A mild slope is classified as a slope in which the normal depth is greater than the critical depth. Thus, the flow is subcritical in an M channel, which is the slope used for irrigation canals and ditches. There are three conditions for flow on a mild slope. If the depth of flow is greater than the normal depth, then the flow is categorized as M1. This would occur if there was a downstream obstruction and water was backed up in the channel. In this case, the depth of flow is increasing. If flow depth is between normal and critical depth (M2), then there is a discharge downstream and flow depth will decrease. If the flow depth is less than critical (supercritical, M3), then the upstream flow is approaching the channel from a steep slope and will increase in depth or possibly undergo a hydraulic jump.

Example 11.8 For the channel section described in Example 11.5 and an initial depth of 0.4 m, calculate the depth of flow 20-, 40-, 60-, 80-, and 100-m downstream.

Calculate the critical depth at a flow rate of 100 L/s = 0.1 m^3/sec.

$Fr^2 = 1 = \frac{Q^2T}{A^3g} = \frac{Q^2(b+2zy)}{((b+zy)y)^3g} = \frac{0.1^2(0.3+2(1)y)}{((0.3+1y)y)^3 9.8}$ By iteration $y_c = 0.182$.

Because the normal depth is greater than the critical depth, the slope is subcritical and the channel is classified as a mild slope (M) as shown in Fig. 11.16.

As calculated in Example 11.5, the normal depth, y_n, is 0.25 m. The actual depth, y = 0.4 m, is greater than the normal depth so the flow condition is M1 (Fig. 11.16). Thus, the flow depth is increasing with distance down the channel (there is a downstream obstruction such as a weir).

The section factor is calculated as follows:

$$AR^{2/3} = (b+zy)y\left(\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}\right)^{2/3}$$
$$= (0.3+(1)(0.4))(0.4)\left(\frac{(0.3+(1)(0.4))0.4}{0.3+(2)(0.4)\sqrt{1+1^2}}\right)^{2/3}$$
$$= 0.094$$

The friction slope is calculated with Eq. 11.19.

$$S_f = \left(\frac{nQ}{AR^{2/3}}\right)^2 = \left(\frac{0.013*0.1}{0.094}\right)^2 = 0.00019 \text{m/m}$$

Calculate the Froude number

$$\text{T} = \text{b} + 2\,\text{zy} = 0.3 + 2*1*0.4 = 1.1\text{ m}$$

$$\text{A} = (\text{b}+\text{zy})\text{y} = (0.3+0.4)0.4 = 0.28\text{ m}^2$$
$$Fr^2 = \frac{Q^2T}{A^3g} = \frac{0.1^2*1.1}{0.28^3*9.8} = 0.0511$$

The downstream depth at 20 m, y_{20}, is

$$y_2 = y_1 + (x_2 - x_1)\frac{S_0 - S_f}{1-\alpha Fr^2}$$
$$= 0.4 + (20)*\frac{0.002 - 0.00019}{1 - 1*(0.0511)} = 0.438\ \ m$$

Calculate the downstream depth at 40 m. First calculate the upstream section factor.

$$AR^{2/3} = (b+zy)y\left(\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}\right)^{2/3}$$
$$= (0.3+1*0.438)*0.438\left(\frac{(0.3+1*0.438)0.438}{0.3+2*0.438\sqrt{1+1^2}}\right)^{2/3}$$
$$= 0.11$$

The friction slope is calculated with Eq. 11.19.

$$S_f = \left(\frac{nQ}{AR^{2/3}}\right)^2 = \left(\frac{0.013*0.1}{0.11}\right)^2 = 0.00013\text{ m/m}$$

Calculate the Froude number

$$\text{T} = \text{b} + 2\,\text{zy} = 0.3 + 2*1*0.438 = 1.17\,\text{m}$$

$$\text{A} = (\text{b}+\text{zy})\text{y} = (0.3+0.438)0.438 = 0.321\,\text{m}^2$$
$$Fr^2 = \frac{Q^2T}{A^3g} = \frac{0.1^2*1.17}{0.321^3*9.8} = 0.035$$

The downstream depth at 40 m, y_{40}, is

$$y_2 = y_1 + (x_2 - x_1)\frac{S_0 - S_f}{1-\alpha Fr^2} = 0.43 + (20)*\frac{0.002 - 0.00013}{1 - 1*(0.035)}$$
$$= 0.477\,\text{m}$$

Flow depth at 60 m downstream is calculated similarly. Flow depths and distances are as follows:

Distance (m)	Depth, y (m)	Difference (m)
0	0.4	
20	0.438	0.0381
40	0.477	0.0388
60	0.516	0.0392
80	0.555	0.0394
100	0.595	0.0396

The flow depth increases with distance down the channel, as predicted by the M1 flow profile in Fig. 11.19. The flow profile is also concave upward as with the M1 profile. This is because the difference between flow depths increases with distance down the channel.

Iteration within each step would have improved the accuracy of the solution by using an estimate of average channel depth between x_2 and x_1 to calculate S_f. However, there is not a large difference between y_2 and y_1 in this example so the answer would not change significantly.

Flow Measurement

Flow measurement is based on the concept that volumetric flow rate is equal to the product of flow velocity and cross-sectional area. The discharge from a flow measurement device is a function of the upstream depth of water.

$$\text{Q} = \text{vA} \tag{11.20}$$

Flow from a submerged orifice is described by the orifice equation. The contraction coefficient is 1.0 for orifices with no contraction (gradual entrance into orifice) and 0.61 for sharp edged orifices (m or ft).

$$Q = C_cA\sqrt{2gh} \tag{11.21}$$

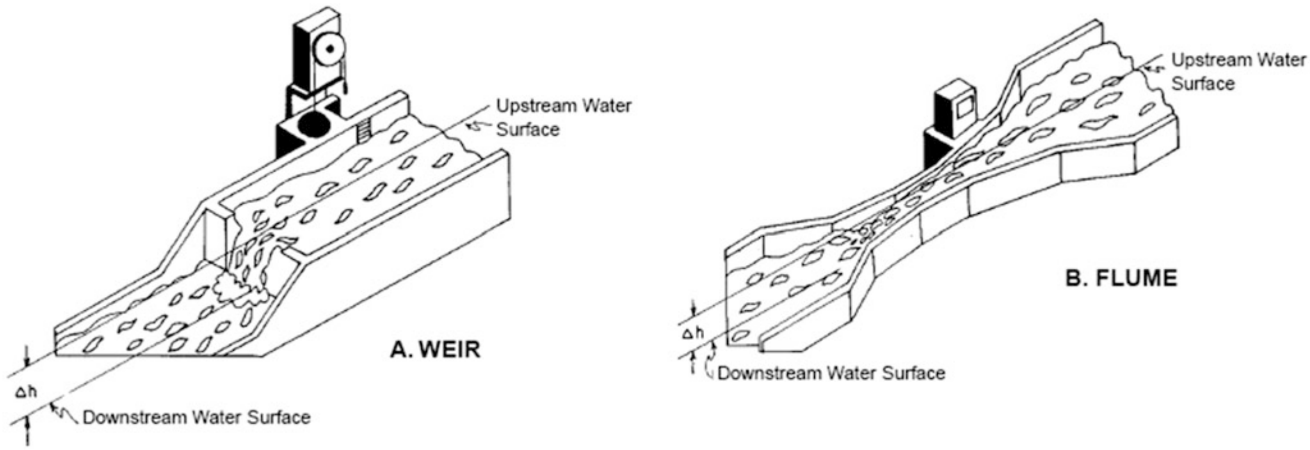
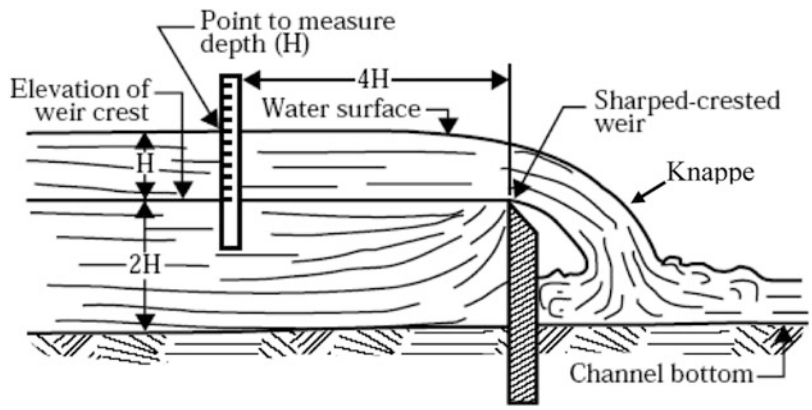


Fig. 11.20 Broad crested weir and long throated flume (Credit NRCS)

Fig. 11.21 Profile of sharp-crested weir (Credit NRCS)



where

C_c = contraction coefficient
h = depth of center of orifice below upstream water surface or pressure in pipe, m.

Sharp-crested weirs or flumes (Fig. 11.20) are used to measure subcritical flow in channels.

Sharp-crested weirs (Fig. 11.21) have a sharp edged weir blade (7–20 mm width). Discharge calculation accuracy over a sharp crested weir is within ± 2 % under best flow conditions.

For sharp-crested weirs, water approach velocity should be less than 0.15 m/sec, and tranquil flow conditions should extend 15–20 times H upstream from the weir blade. The face of the weir should be vertical and perpendicular to the direction of water flow and the upstream edge should be sharp. The depth of water over the crest should be at least 5 cm for accurate measurements. The minimum flow should be more than 2 % of the maximum flow for accurate measurement of low flows. Suppressed weirs extend completely across a channel and contracted weirs do not. The crest and edges of contracted weirs should be at least 2H away from the channel sides.

The following equation is a general equation for flow over a sharp crested weir (Fig. 11.21), where flow rate is a function of the upsteam depth of flow, H:

$$Q = C_d \frac{2}{3} \sqrt{2g}\, L_b H^{3/2} \qquad (11.22)$$

where

C_d = discharge coefficient, determined by analysis and calibration tests
H = upstream elevation of water surface above weir (Fig. 11.20).

There are five types of sharp-crested contracted weirs: rectangular (Fig. 11.22), V-notch (Fig. 11.23), Cipoletti (Fig. 11.24), partially contracted rectangular weirs, and partially contracted 90-degree V-notch weirs.

The Francis equation is used to calculate flow rate over a standard contracted rectangular weir. The last term, 0.2 H, accounts for the fact that the edges of the flow contract from the sides of the weir.

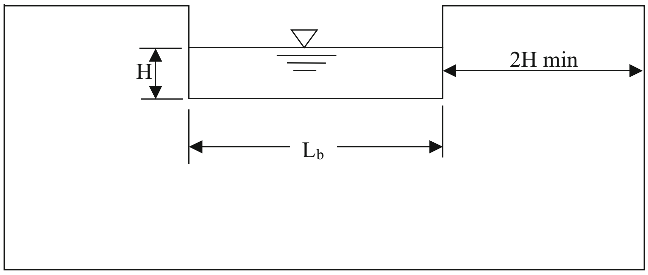


Fig. 11.22 Contracted rectangular sharp-crested weir

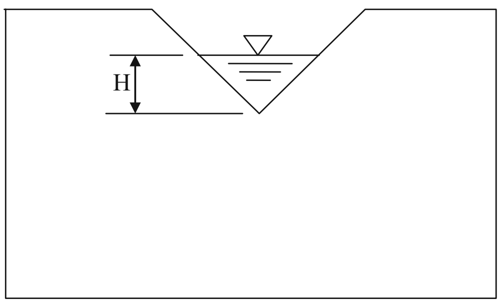


Fig. 11.23 90-degree V-notched weir

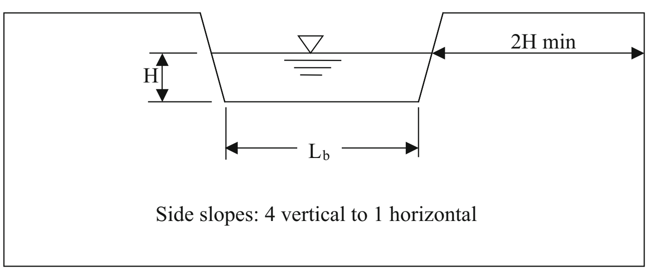


Fig. 11.24 Standard cipolletti weir

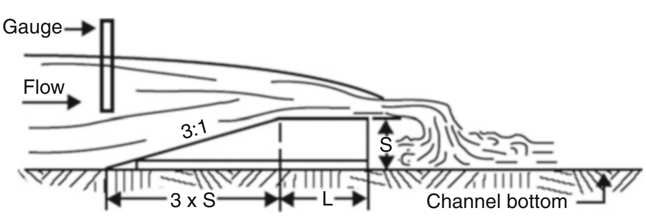


Fig. 11.25 Profile of long throated flume (Credit NRCS)

$$Q = 3.33H^{3/2}(L_b - 0.2H)\left(\text{ft and ft}^3/\text{sec}\right)$$
$$Q = 1.74H^{3/2}(L_b - 0.2H)\left(\text{m and m}^3/\text{sec}\right) \quad (11.23)$$

where

Q = discharge flow rate neglecting velocity of approach, ft^3/sec or m^3/sec
L_b = length of weir, ft or m
H = head on weir, ft or m.

The constant discharge coefficient, 3.33, is valid as long as the head over the weir, H, is not more than 1/3 the length of the weir, L_B. If the head over a rectangular weir is greater than 1/3 the length, then tables must be used to find the constant discharge coefficient. The USBR Water Measurement Manual (WMM) has an appendix with tables of flows over contracted weirs and contains information on proper weir design.

Equation 11.25 can be used to calculate flow over a 90-degree V-notch weir (WMM). Equations for calculation of flow over V-notch weirs with other angles as well as tables of calculations are given in the WMM.

$$Q = 2.49H^{2.48}\left(\text{ft and ft}^3/\text{sec}\right)$$
$$Q = 1.35H^{2.48}\left(\text{m and m}^3/\text{sec}\right) \quad (11.24)$$

In-class Exercise 11.1 Of the three weirs, rectangular, V-notch, and Cipolletti, which is best for small flows?

Which weir provides the highest sensitivity to changes in flow rate?

Which weir is best for large flow rates?

The following equation can be used to find the flow rate over a standard Cipolletti weir (Fig. 11.24). The equation is valid as long as H is no greater than 0.33 L_b.

$$Q = 3.367L_bH^{3/2}\left(\text{ft and ft}^3/\text{sec}\right).$$
$$Q = 1.86L_bH^{3/2}\left(\text{m and m}^3/\text{sec}\right). \quad (11.25)$$

Example 11.9 If the head, H, over a standard Cipolletti weir is 0.15 m and the weir blade length, L_b, is 0.5 m, then what is the flow rate?

$$Q = 1.86L_bH^{3/2} = 1.86{*}0.3{*}0.1^{3/2} = 0.018\ m^3/\text{sec}.$$

A standard suppressed rectangular weir extends across the full width of a channel. This type of weir has no flow contraction from the sides. An air vent is needed below the knappe to the atmosphere in order to ensure that air pressure below the knappe (Fig. 11.21) remains the same as atmospheric pressure. The crest height above the channel bottom should be at least 3H. The equation for a suppressed rectangular weir is the same as for the contracted rectangular weir except that the 0.2H value for side contraction is left out.

$$Q = 3.33H^{3/2}(L_b)\left(\text{ft and ft}^3/\text{sec}\right)$$
$$Q = 1.74H^{3/2}(L_b)\left(\text{m and m}^3/\text{sec}\right) \quad (11.26)$$

Broad-crested weirs and long-throated flumes (Figs. 11.20 and 11.25) have long flat approaches. Broad-crested weirs are simply long throated flumes with no side contractions.

Fig. 11.26 Large long-throated flume under construction (Credit U.S. Water Conservation Laboratory, Phoenix, Arizona)

Flumes increase water velocity and change water flow from subcritical to supercritical as the flow is accelerated. Acceleration occurs by increasing the bottom depth (Fig. 11.26) and/or converging the sidewalls (Fig. 11.25). Flumes include a measurement of upstream flow depth, which is directly correlated with flow rate because the upstream profile is unique for each flow rate if the water passes through critical depth. A major advantage of flumes over weirs is that flumes only dissipate 25 % of the energy that is dissipated in the weir. Thus, a flume may be advantageous on shallow slopes where water energy must be conserved. Flumes are also more difficult to tamper with and alter flow readings than weirs; this prevents users from obtaining an unfair allocation of water. A disadvantage of flumes is that they cost more than weirs.

Long throated flumes are easier to install (Figs. 11.26 and 11.27) than other flume geometries. A concrete ramp is simply placed in the channel bottom. In the past, Parshall flumes were installed in many channels; however, because of difficulty of installation, they have decreased in popularity. However, in some regions, local regulations still mandate the use of Parshall flumes.

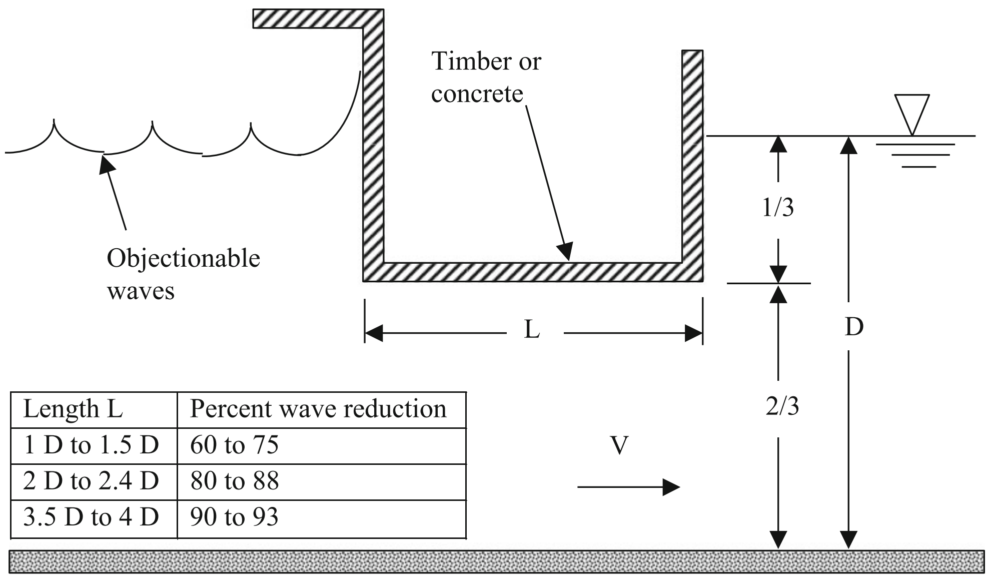
Calibration of weirs and flumes may be necessary if the geometry or upstream flow conditions are not as specified in design manuals. Water measurement manuals should be consulted for nonstandard conditions.

If upstream flow is choppy, then readings will be inaccurate. In this case, an energy dissipation structure can be installed upstream from a weir or flume as shown in Fig. 11.28. As the structure forces water under a flat surface, waves are dissipated.

When no water measurement structure is installed, water flow rate can be measured in canals or surface irrigation ditches by measuring the length of time that it takes for a floating ball or other buoyant device to travel a 30 m length of the channel. Repeat the measurement five or ten times and take the average. The water at the surface of the channel travels faster than the average velocity; thus, the calculated velocity should be multiplied by a coefficient (Table 11.5). The corrected velocity is then multiplied by the cross-sectional area of the channel in order to find the flow rate.

The velocity at the channel water surface also can be found by placing a thin and wide meter stick in the center of the channel and observing the difference in elevation of

Fig. 11.27 Large long-throated flume under operation for left to right flow (Credit U.S. Water Conservation Laboratory, Phoenix, Arizona)



Length L	Percent wave reduction
1 D to 1.5 D	60 to 75
2 D to 2.4 D	80 to 88
3.5 D to 4 D	90 to 93

Fig. 11.28 Underpass wave suppressor section (USBR Water Measurement Manual)

water with the sharp edge of the meter stick parallel to the flow and turned 90°. The difference in elevation is equal to the velocity head in meters at the surface of the ditch.

Example 11.10 A channel has the cross sectional area calculated in Example 11.5. The difference in head on the meter stick is 6 cm between the parallel and perpendicular to the flow positions. Calculate the flow velocity at the surface and the flow rate in the channel.

$$\mathrm{v} = (2\mathrm{gh})^{0.5} = (2*9.8*0.06)^{0.5} = 1.08\ \mathrm{m/sec}\,.$$

$$\mathrm{v}_{\mathrm{ad\,j}} = 1.08*0.66 = 71\ \mathrm{m/sec}\ \ (\text{Table } 11.5)$$

$$A = (b + zy)y = (0.3 + 1*0.25)*0.25 = 0.14\ \ m^2$$

$$\mathrm{Q} = \mathrm{vA} = 0.71*0.14 = 0.1\ \mathrm{m/sec}$$

Table 11.5 Velocity coefficient versus depth of water in the ditch

Average depth (m)	Coefficient
0.3	0.66
0.6	0.68
0.9	0.70
1.2	0.72
1.5	0.74
1.8	0.76
2.7	0.77
3.6	0.78
4.5	0.79
6	0.80

Questions

1. Describe the different types of canals.
2. How is water diverted from a main canal to a lateral canal?
3. How is water diverted from a lateral canal to a field?
4. What is the reason for energy dissipation structures in canals and in canal outlets?
5. Calculate the conveyance efficiency and water duty for a canal that is 20 km long, has a wetted top width = 20 m, wetted perimeter = 26 m, and cross-sectional area = 100 m^2? Average canal flow velocity is 1 m/sec. Reference ET is 10 mm/day. Average seepage rate is 5 mm/day. In addition to reporting the water duty and efficiency, convert the seepage rate to L/m^2/day.
6. Calculate the conveyance efficiency to field 1 in Fig. 11.9 from the point of water diversion to the irrigation district. The conveyance efficiency of the irrigation district up to the farm turnout is 80 %. The main concrete canal on the farm has a conveyance efficiency of 80 %, and the earth-lined canal has a conveyance efficiency of 80 %.
7. Redo Example 11.4, but assume the ditch is in loam and sandy loam soil (Fig. 11.8) with a seepage rate of 1.5 m^3/m^2/day.
8. Describe the method used to run canal water past a road, drainage ditch, or river valley.
9. A concrete lined trapezoidal channel (Fig. 11.14) has a slope of 0.3 % = 0.003 m/m. Flow rate in the channel is 300 L/sec, and the Manning's roughness coefficient, n, of the channel is 0.015. Calculate the depth of flow in the channel. The bottom width is 1 m and side slope z = 1.5.
10. Calculate the Froude number for the previous problem, and determine whether the channel has supercritical or subcritical flow.
11. If a canal was laid on a relatively steep slope, what strategy could be used to prevent supercritical flow in the canal?
12. For the canal dimensions described in question 9, calculate the chute slope required to have a stable hydraulic jump with Froude number = 6.
13. Describe why a Froude number 6 is desirable before a hydraulic jump.
14. Calculate the flow over a standard contracted rectangular weir if the head over the weir is 0.13 m and the wier blade is 0.6 m across.

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