Sprinklerdeck Specifications and Technical Data

Sprinklerdeck has two advantages over wire decking. First, it redistributes water under the material on the deck rather than having runoff drop straight down. Second, it acts as a shield to suppress the “chimney effect” of open decking.

While each Fire Marshall or Township has its own particular ideas, and so does each insurance company, many are guided by the publications of the National Fire Protection Association. However the NFPA has established no very rigid standards for racks with decking. Generally they suggest in-rack sprinklers, perhaps good for fire protection, but the cost is almost unimaginable and they limit or virtually eliminate future rack rearrangement.

The NFPA is the first to admit the subject is still open. On page 44 of Standard 231C “Rack Storage of Materials” in B-5-13.2 they simply recommend that “...sound engineering judgment should be used” (reprint available on request).
Sprinklerdeck has the advantage of acting as a shield which wire deck does not. It passes water down but blocks hot air and flame upward. The advantage of this is clearly shown in B-6-3 (reprint available on request) .

Most monetary loss from fires is water damage rather than burning. It thus becomes highly desirable to confine the fire in as small an area as possible. This was our thought in designing the decking for flow rates of 0.4 to 0.7 gpm/sq.ft., which is more than most sprinkler systems can supply to a large area. Douse it heavily and hold it in one spot is our philosophy.

Design Detailing

In the design detailing for decking limitless configurations of channel size and drainage hole size are possible, provided the following relationships are observed:

1. The channels must be of sufficient size for gravity flow of sprinkler fluid to all holes supplied by the channel.

2. The drainage holes in each channel must be sufficiently restrictive to allow sprinkler fluid to flow back to the center of each deck, under the load. If too large all sprinkler fluid will be drained near the edges of the load above and none will reach back to the center of the deck. And if too small total sprinkler fluid discharge cannot be handled.

Our production design is for sprinkler water discharge of 0.4 to 0.7 gallons per minute per square foot of area protected. The following calculations for this rate and pattern can also serve as a model for any desired sprinkler fluid discharge rate and decking configuration.

For manufacturing convenience the decking cross section shown in figure 1 has been selected together with the 2-1/2" x 6" round hole pattern shown in figure 2. Now the proper hole size to suit this arrangement must be calculated.

Each hole drains an area of 2-1/2" x 6" or .1042 square feet. Hence the flow required per hole at maximum rate of 0.7 gallons per minute:

Q = flow per hole = 0.7 x .1042
= .073 gallons per minute
= 16.8 cubic inches per minute = .281 cubic inches per second

The general equation for steady state fluid flow is (Bernoulli):

$$\frac{p_1}{w} + \frac{v_1^2}{2g} + h_1 = \frac{p_2}{w} + \frac{v_2^2}{2g} + h_2$$

Where p = pressure
w = specific weight, lb. per cubic inch
v = velocity, inches per second
g = acceleration of gravity, 386 inches/sec^2
h = height, inches

Since no external pressure is being applied, p_1 and p_2 are zero. Since the vertical velocity of fluid at open surface above the hole is zero and height of fluid column at hole exit is zero, the equation becomes:

$$h_1 = \frac{v_2^2}{2g} \quad \text{or} \quad v = \sqrt{2gh}$$

Since Q = vA where A is the cross section area of the hole in square inches:

$$v = \frac{Q}{A} = \sqrt{2gh} \quad \text{or} \quad A = \frac{Q}{\sqrt{2gh}}$$

However the holes we manufacture in production are similar in cross section to the "sharp edged orifice" shown on page 3-69 (copy attached) of "Marks Mechanical Engineers Handbook" for which an overall flow coefficient of 0.61 is shown. Hence:

$$.61A = \frac{Q}{\sqrt{2gh}} \quad \text{or} \quad A = \frac{1.64Q}{\sqrt{2gh}}$$

Since this particular design is for sprinkler water discharge of 0.4 to 0.7 gallons per minute per square foot, at maximum flow of 0.7 gpm the channels should be almost full. The overall inside height of our channel is .534", so assuming a liquid column height of .500 above the holes, the hole cross section area becomes:

$$A = \frac{(1.64)(.281)}{\sqrt{(2)(386)(.500)}}$$

$$= \frac{.461}{19.65}$$

= .0235 square inches

The diameter is then $D = \sqrt{\frac{4A}{\pi}}$

= .172 inches

This is the size we have adopted for production.

From the equation of continuity, $Q/V = 360/3.914 = 91.97 = 4(b + 4)$, $b = 18.99$. Subsequent trial-and-error solutions result in a balance at $b = 19.93$ ft (6.075 m).

Specific Energy Specific energy is defined as the energy of the fluid referred to the bottom of the channel as the datum. Thus the specific energy E at any section is given by $E = y + V^2/2g$; from the continuity equation $V = Q/A$ or $E = y + (Q/A)^2/2g$. For a rectangular channel whose width is b, $A = by$; and if $\mathbf{q}$ is defined as the flow rate per unit width, $\mathbf{q} = Q/b$ and $E = y + (\mathbf{q}b/by)^2/2g = y + (\mathbf{q}/y)^2/2g$.

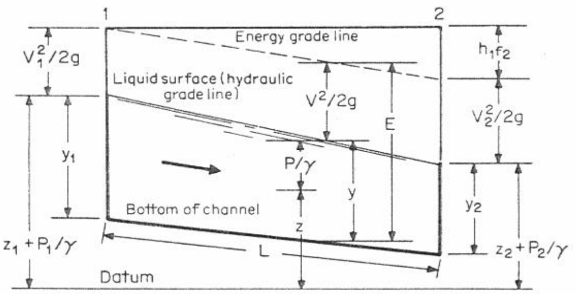


Fig. 33 Notation for open-channel flow.

Critical Values For rectangular channels, if the specific-energy equation is differentiated and set equal to zero, critical values are obtained; thus $dE/dy = d/dy\,[y + (\mathbf{q}/y)^2/2g] = 0 = 1 - \mathbf{q}^2/y^3g$ or $\mathbf{q}_c^2 = y_c^3g$. Substituting in the specific-energy equation, $E = y_c + y_c^3g/2gy_c^2 = 3/2\ y_c$. Figure 34 shows the relation between depth and specific energy for a constant flow rate. If the depth is greater than critical, the flow is *subcritical;* at critical depth it is *critical* and at depths below critical the flow is *supercritical*. For a given specific energy, there is a maximum unit flow rate that can exist.

The **Froude number** $\mathbf{F} = V/\sqrt{gy}$, when substituted in the specific-energy equation, yields $E = y + (\mathbf{F}^2gy)/2g = y(1 + \mathbf{F}^2/2)$ or $E/y = 1 + \mathbf{F}^2/2$. For critical flow, $E_c/y_c = 3/2$. Substituting $E_c/y_c = 3/2 = 1 + \mathbf{F}_c^2/2$, or $\mathbf{F} = 1$,

$\mathbf{F} < 1$	Flow is subcritical
$\mathbf{F} = 1$	Flow is critical
$\mathbf{F} > 1$	Flow is supercritical

It is seen that for open-channel flow the Froude number determines the type of flow in the same manner as Mach number for compressible flow.

EXAMPLE Water flows at a rate of 600 ft³/s in a rectangular channel 10 ft wide at a depth of 4 ft. Determine (1) specific energy and (2) type of flow.

1. From the continuity equation,

$$V = Q/A = 600/(10 \times 4) = 15 \text{ ft/s}$$
$$E = y + V^2/2g = 4 + (15)^2/2(2 \times 32.17) = 7.497 \text{ ft}$$

2. $\mathbf{F} = V/\sqrt{gy} = 15/\sqrt{32.17 \times 4} = 1.322$; $\mathbf{F} > 1$ ∴ flow is supercritical.

FLOW OF LIQUIDS FROM TANK OPENINGS

Steady State Consider the jet whose velocity is V discharging from an open tank through an opening whose area is a, as shown in Fig. 35. The liquid height above the centerline is b, and the cross-sectional area of the tank at b is A. The ideal velocity of the jet is $V_i = \sqrt{2gb}$. The ratio of the actual velocity V to the ideal velocity V_i is the **coefficient of velocity** C_v, or $V = C_vV_i = C_v\sqrt{2gb}$. The ratio of the actual opening a to the minimum area of the jet a_c is the **coefficient of contraction** C_c, or $a = C_ca_c$. The ratio of the actual discharge Q to the ideal discharge Q_i is the **coefficient of discharge** C, or $Q = CQ_i = C_aV_i = C_cC_va\sqrt{2gb}$, and $C = C_cC_v$. Nominal values of coefficients for various openings are given in Fig. 36.

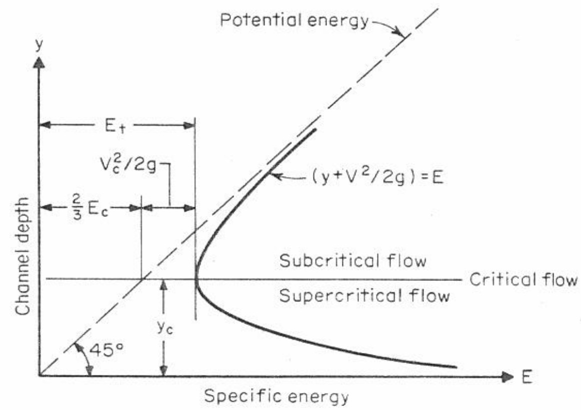


Fig. 34 Specific-energy diagram, constant flow rate.

Unsteady State If the rate of liquid entering the tank Q_{in} is different from that leaving, the level b in the tank will change because of the change in storage. For liquids, the conservation-of-mass equation may be written as $Q_{in} - Q_{out} = Q_{stored}$; for a time interval dt, $(Q_{in} - Q_{out})dt = A\ db$, neglecting fluid acceleration,

Table 17. Values of Roughness Factor n for Use in Manning Equation

Surface	n	Surface	n
Brick	0.015	Earth, with stones and weeds	0.035
Cast iron	0.015	Gravel	0.029
Concrete, finished	0.012	Riveted steel	0.017
Concrete, unfinished	0.015	Rubble	0.025
Brass pipe	0.010	Wood, planed	0.012
Earth	0.025	Wood, unplaned	0.013

Compiled from data given in R. Horton, *Engineering News*, **75**, 373, 1916.

$$Q_{out}\,dt = Ca\sqrt{2gh}\,dt, \text{ or } (Q_{in} - Ca\sqrt{2gh})\,dt$$
$$= A\,dh, \text{ or } \int_{t_1}^{t_2} dt = \int_{h_1}^{h_2} \frac{A\,dh}{Q_{in} - Q_{out}} = \int_{h_1}^{h_2} \frac{A\,dh}{Q_{in} - Ca\sqrt{2gh}}$$

EXAMPLE. An open cylindrical tank is 6 ft in diameter and is filled with water to a depth of 10 ft. A 4-in-diameter sharp-edged orifice is installed on the bottom of the tank. A pipe on the top of the tank

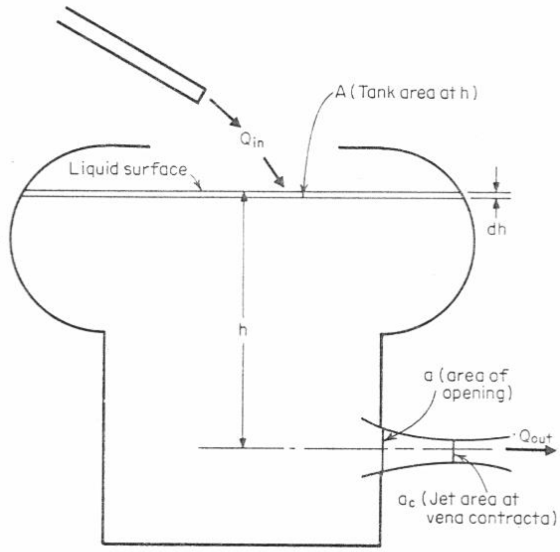


Fig. 35 Notation for tank flow

supplies water at the rate of 1 ft³/s. Estimate (1) the steady-state level of this tank, (2) the time required to reduce the tank level by 2 ft.

1. Steady-state level. From Fig. 36, $C = 0.61$ for a sharp-edged orifice, $a = (\pi/4)d^2 = (\pi/4)(4/12)^2 = 0.08727$ ft². For steady state, $Q_{in} = Q_{out} = Ca\sqrt{2gh} = 1 = (0.61)(0.08727)(2 \times 32.17\,h)^{1/2}$; $h = 5.484$ ft.

2. Time required to lower level 2 ft, $A = (\pi/4)D^2 = (\pi/4)(6)^2 = 28.27$ ft²

$$t_2 - t_1 = \int_{h_1}^{h_2} \frac{A\,dh}{Q_{in} - Ca\sqrt{2gh}}$$

This equation may be integrated by letting $Q = Ca\sqrt{2g}\,h^{1/2}$; then $dh = 2Q\,dQ/(Ca\sqrt{2g})^2$; then

$$t_2 - t_1 = \frac{2A}{(Ca\sqrt{2g})^2}\left[Q_{in}\log_e\left(\frac{Q_{in} - Q_1}{Q_{in} - Q_2}\right) + Q_2 - Q_1\right]$$

At t_1 : $Q_1 = 0.61 \times 0.08727\sqrt{2 \times 32.17 \times 10} = 1.350$ ft³/s

At t_2 : $Q_2 = 0.61 \times 0.08727\sqrt{2 \times 32.17 \times 8} = 1.208$ ft³/s

$$t_2 - t_1 = \frac{2 \times 28.27}{(0.61 \times 0.08727\sqrt{2 \times 32.17})^2} \times \left[(1)\log_e\left(\frac{1 - 1.350}{1 - 1.208}\right) + 1.208 - 1.350\right]$$

$t_2 - t_1 = 117.3$ s

WATER HAMMER

Equations Water hammer is the series of shocks, sounding like hammer blows, produced **by suddenly reducing the flow** of a fluid in a pipe. Consider a fluid flowing frictionlessly in a rigid pipe of uniform area A with a velocity V. The pipe has a length L, and inlet pressure p_1 and a pressure p_2 at L. At length L, there is a valve which can suddenly reduce the velocity at L to $V - \Delta V$. The equivalent mass rate of flow of a pressure wave traveling at sonic velocity c, $\dot{M} = \rho Ac$. From the impulse-momentum equation, $\dot{M}(V_2 - V_1) = p_2A_2 - p_1A_1$; for this application, $(\rho Ac)(V - \Delta V - V) = p_2A - p_1A$, or the increase in pressure $\Delta p = -\rho c\,\Delta V$. When the liquid is flowing in an elastic pipe, the equation for pressure rise must be modified to account for the expansion of the pipe; thus

$$c = \sqrt{\frac{E_s}{\rho[1 + (E_s/E_p)(D_o + D_i)/(D_o - D_i)]}}$$

where E_p is the bulk modulus of elasticity of the pipe material, D_o the outside diameter of the pipe, and D_i the inside diameter.

Time of Closure The time for a pressure wave to travel the length of pipe L and return is $t = 2L/c$. If the time of closure $t_c \leq t$, the approximate pressure rise $\Delta p \approx -2\,\rho V(L/t_c)$. When it is not feasible to close the valve slowly, **air chambers** or **surge tanks** may be used to absorb all or most of the pressure rise.

EXAMPLE. Water flows at 68°F (20°C) in a 3-in steel schedule 40 pipe at a velocity of 10 ft/s. A valve located 200 ft downstream is suddenly closed. Determine (1) the increase in pressure considering pipe to be rigid, (2) the increase considering pipe to be elastic, and (3) the maximum time of valve closure to be considered "sudden." For

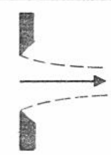
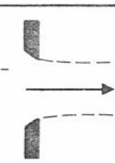
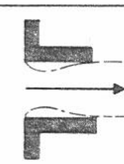
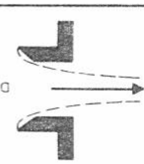
Type		Coefficient		
		C	C_c	C_v
Sharp-edged orifice		0.61	0.62	0.98
Rounded-edged orifice		0.98	1.00	0.98
Short tube		0.80	1.00	0.80
Borda		0.51	0.52	0.98

Fig. 36 Nominal coefficients of orifices.