TOTAL HYDRAULIC HEAD LOSS IN A WELL (H_w) IS

$$H_w = H_e + H_s + H_r + H_v \qquad (102)$$

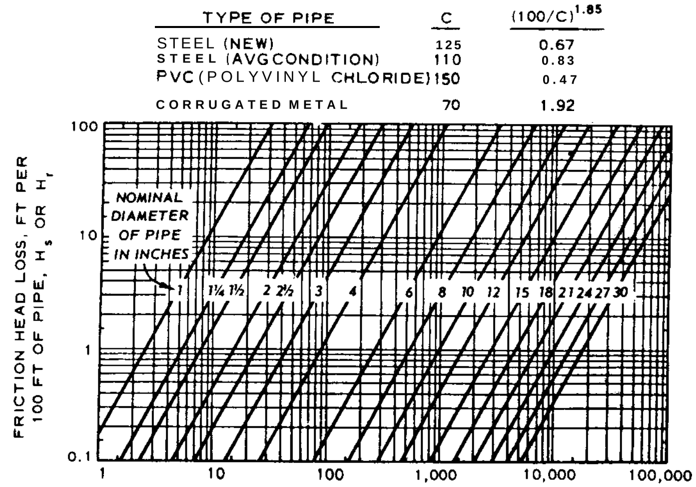
WHERE H_e = ENTRANCE HEAD LOSS (SCREEN AND FILTER) ESTIMATE FROM CURVE a.

H_s = HEAD LOSS IN SCREENED SECTION OF WELL: ESTIMATE FROM CURVE b FOR A DISTANCE OF ONE-HALF THE SCREEN LENGTH.

H_r = HEAD LOSS WITHIN THE RISER AND CONNECTIONS. ESTIMATE FROM CURVE b. (SEE FIG. 4-26 FOR THE EQUIVALENT LENGTH OF STRAIGHT PIPE FOR VARIOUS FITTINGS.)

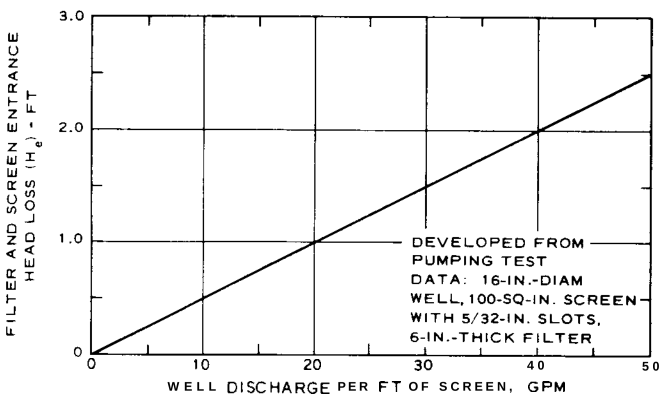
H_v = VELOCITY HEAD LOSS. ESTIMATE FROM CURVE c

THE VALUE OF H_w MUST BE SUBTRACTED FROM THE COMPUTED VALUE OF h_w TO OBTAIN THE LIFT OR WATER LEVEL IN A WELL.

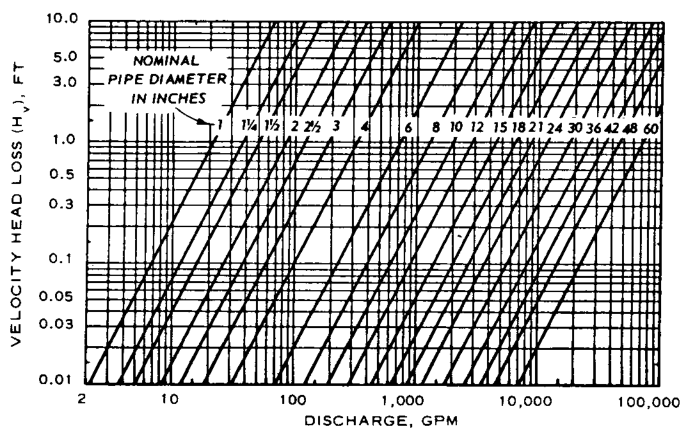


BASED ON HAZEN-WILLIAMS EQUATION WITH C = 100; MULTIPLY LOSSES BY $(100/C)^{1.85}$ FOR VALUES OF C OTHER THAN 100

(b)



(a)



(c)

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Figure 4-24. Hydraulic ***head loss in a well.***

TOTAL HYDRAULIC HEAD LOSS IN A WELLPOINT (H_w) IS

$$H_w = H_e + H_s + H_r + H_v$$

WHERE

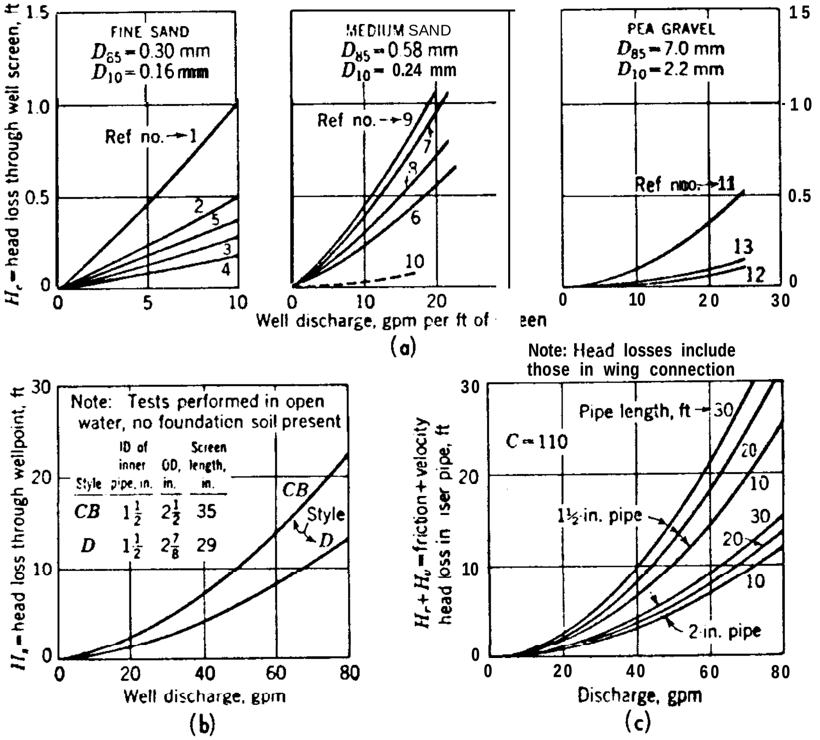
H_e = ENTRANCE HEAD LOSS (WELLPOINT AND FILTER)

H_s = FRICTION HEAD LOSS WITHIN THE WELLPOINT

H_r = FRICTION HEAD LOSS IN RISER, SWING CONNECTION, AND VALVE

H_v = VELOCITY HEAD LOSS IN RISER, SWING CONNECTION, AND VALVE

HYDRAULIC HEAD LOSSES FOR TYPICAL WELLPOINTS AND RISERS CAN BE ESTIMATED FROM THE PLOTS BELOW.



REF NO.	WELLPOINT†	SLOT OR MESH NO.	OPENING IN MM
1	A AND B, GROOVED SLOT	12	0.30
2	C, WIRE WRAP ON PERFORATED PIPE	20	0.51
3	D, WIRE MESH ON PERFORATED PIPE	26	0.59
4	E, WIRE MESH ON PERFORATED PIPE	26	0.59
5	B, GROOVED SLOT	25	0.63
6	A, GROOVED SLOT	25	0.63
7	D, WIRE MESH ON PERFORATED PIPE	26	0.59
8	A, GROOVED SLOT	50	1.27
9	B, GROOVED SLOT	30	0.76
10	F, PERFORATED PIPE WITH 6-IN. PEA-GRAVEL FILTER	5/32 IN.	3.97
11	A, GROOVED SLOT	12	0.30
12	A, GROOVED SLOT	100	2.54
13	E, PERFORATED PIPE	5/32 IN.	3.97
CB	MESHSF, COMMERCIAL BRONZE, SELF-JETTING	40 × 45	0.31 × 0.36
D	MESH E, STAINLESS STEEL STYLE D, SELF-JETTING	12 × 66	0.30 × 1.73

† EXCEPT FOR C, B, AND D, WELLPOINTS ARE PLAIN-TIP, 2-1/2-IN. ID.

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Figure 4-25. Hydraulic head loss in various wellpoints.

(2) *Circular and rectangular slots.* Equations for flow and head or drawdown produced by circular and rectangular slots supplied by a circular seepage source are given in figures 4-6 through 4-9. Equations for flow from a circular seepage source assume that the slot is located in the center of an island of radius R. For many dewatering projects, R is the radius of influence rather than the radius of an island, and procedures for determining the value of R are discussed in a(3) above. Dewatering systems of relatively short length are considered to have a circular source where they are far removed from a line source such as a river or shoreline.

(3) Use *of slots for designing well systems.* Wells can be substituted for a slot; and the flow Q_w, drawdown at the well $(H-h_w)$ neglecting hydraulic head losses at and in the well, and head midway between the wells above that in the wells Δh_m can be computed from the equations given in figures 4-20, 4-21, and 4-22 for a *(single) line source* for artesian and gravity flow for both "fully" and "partially" penetrating wells where the well spacing a is substituted for the length of slot x.

(4) *Partially penetrating slots,* The equations for *gravity* flow *topartially* penetrating slots are only considered valid for relatively high-percent penetrations.

c. Flow to wells.

(1) *Flow to wells from* a *circular source.*

(a) Equations for flow and drawdown produced by a single well supplied by a circular source are given in figures 4-10 through 4-12. It is apparent from figure 4-11 that considerable computation is required to determine the height of the phreatic surface and resulting drawdown in the immediate vicinity of a gravity well (r/h less than 0.3). The drawdown in this zone usually is not of special interest in dewatering systems and seldom needs to be computed. However, it is always necessary to compute the water level in the well for the selection and design of the pumping equipment.

(*b*) The general equations for flow and drawdown produced by pumping a group of wells supplied by a circular source are given in figure 4-13. These equations are based on the fact that the drawdown at any point is the summation of drawdowns produced at that point by each well in the system. The drawdown factors F to be substituted into the general equations in figure 4-13 appear in the equations for both artesian and gravity flow conditions. Consequently, the factors given in figure 4-14 for commonly used well arrays are applicable for either condition.

(c) Flow and drawdown for circular well arrays can also be computed, in a relatively simple manner, by first considering the well system to be a slot, as shown in figure 4-15 or 4-16. However, the piezometric head in the vicinity of the wells (or wellpoints) will not correspond exactly to that determined for the slot due to convergence of flow to the wells. The piezometric head in the vicinity of the well is a function of well flow Q_w; well spacing a; well penetration W; effective well radius r_w; aquifer thickness D, or gravity head H; and aquifer permeability k. The equations given in figures 4-15 and 4-16 consider these variables.

(2) *Flow to wells from a line source,*

(a) Equations given in figures 4-17 through 4-19 for flow and drawdown produced by pumping a single well or group of fully penetrating wells supplied from an infinite line source were developed using the method of image wells. The image well (a recharge well) is located as the mirror image of the real well with respect to the line source and supplies the pervious stratum with the same quantity of water as that being pumped from the real well.

(*b*) The equations given in figures 4-18 and 4-19 for multiple-well systems supplied by a line source are based on the fact that the drawdown at any point is the summation of drawdowns produced at that point by each well in the system. Consequently, the drawdown at a point is the sum of the drawdowns produced by the real wells and the negative drawdowns produced by the image or recharge wells.

(c) Equations are given in figures 4-20 through 4-22 for flow and drawdown produced by pumping an infinite line at wells supplied by a *(single) line source,* The equations are based on the equivalent slot assumption. Where twice the distance to a *single* line source or 2L is greater than the radius of influence R, the value of R as determined from a pumping test or from figure 4-23 should be used in lieu of L unless the excavation is quite large or the tunnel is long, in which case equations for a line source or a flow-net analysis should be used.

(d) Equations for computing the head midway between wells above that in the wells Δh_m are not given in this manual for *two* line sources adjacent to a single line of wells. However, such can be readily determined from (plan) flow-net analyses.

(3) *Limitations on flow to a partially penetrating well.* Theoretical boundaries for a partially penetrating well (for artesian flow) are approximate relations intended to present in a simple form the results of more rigorous but tedious computations. The rigorous computations were made for ratios of R/D = 4.0 and 6.7 and a ratio $R/r_w = 1000$. As a consequence, any agreement between experimental and computed values cannot be expected except for the cases with these particular boundary conditions. In model studies at the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, the flow from a partially penetrating well was based on the formula:

$$Q_{wp} = \frac{2\pi kD(H-h_w)G}{\ln(R/r_w)} \qquad (4\text{-}2)$$

or

$$Q_{wp} = kD(H - h_w)\mathcal{S} \qquad (4\text{-}2a)$$

with

$$\mathcal{S} = \frac{2\pi G}{\ln(R/r_w)}$$

where

G = quantity shown in equation (6), figure 4-10

$\mathcal{S}$ = geometric shape factor

Figure 4-26 shows some of the results obtained at the WES for $\mathcal{S}$ for wells of various penetrations centered inside a circular source. Also presented in figure 4-26 are boundary curves computed for well-screen penetrations of 2 and 50 percent. Comparison of $\mathcal{S}$ computed from WES model data with $\mathcal{S}$ computed from the boundary formulas indicates fairly good agreement for well penetrations > 25 percent and values of R/D between about 5 and 15 where R/r_w 1 200 to 1000. Other empirical formulas for flow from a partially penetrating well suffer from the same limitations.

(4) *Partially penetmting wells.* The equations for *gravity flow* to *partially* penetrating wells are only considered valid for relatively high-percent penetrations.

4-3. Flow-net analyses.

a. Flow nets are valuable where irregular configurations of the source of seepage or of the dewatering system make mathematical analyses complex or impossible. However, considerable practice in drawing and studying good flow nets is required before accurate flow nets can be constructed.

b. A flow net is a graphical representation of flow of water through an aquifer and defines paths of seepage (flow lines) and contours of equal piezometric head (equipotential lines). A flow net may be constructed to represent either a plan or a section view of a seepage pattern. Before a sectional flow net can be constructed, boundary conditions affecting the flow pattern must be delineated and the pervious formation transformed into one where $k_h = k_v$ (app E). In drawing a flow net. the following general rules must be ob served:

(1) Flow lines and equipotential lines intersect at right angles and form curvilinear squares or rectangles.

(2) The flow between any two adjacent flow lines and the head loss between any two adjacent equipotential lines are equal, except where the plan or section cannot be divided conveniently into squares, in which case a row of rectangles will remain with the ratio of the lengths to the sides being constant.

(3) A drainage surface exposed to air is neither an equipotential nor flow line, and the squares at this surface are incomplete; the flow and equipotential lines need not intersect such a boundary at right angles.

(4) For gravity flow, equipotential lines intersect the phreatic surface at equal intervals of elevation, each interval being a constant fraction of the total net head.

c. Flow nets are limited to analysis in two dimensions; the third dimension in each case is assumed infinite in extent. An example of a sectional flow net showing artesian flow from two line sources to a partially penetrating drainage slot is given in figure 4-27a. An example of a plan flow net showing artesian flow from a river to a line of relief wells is shown in figure 4-27b.

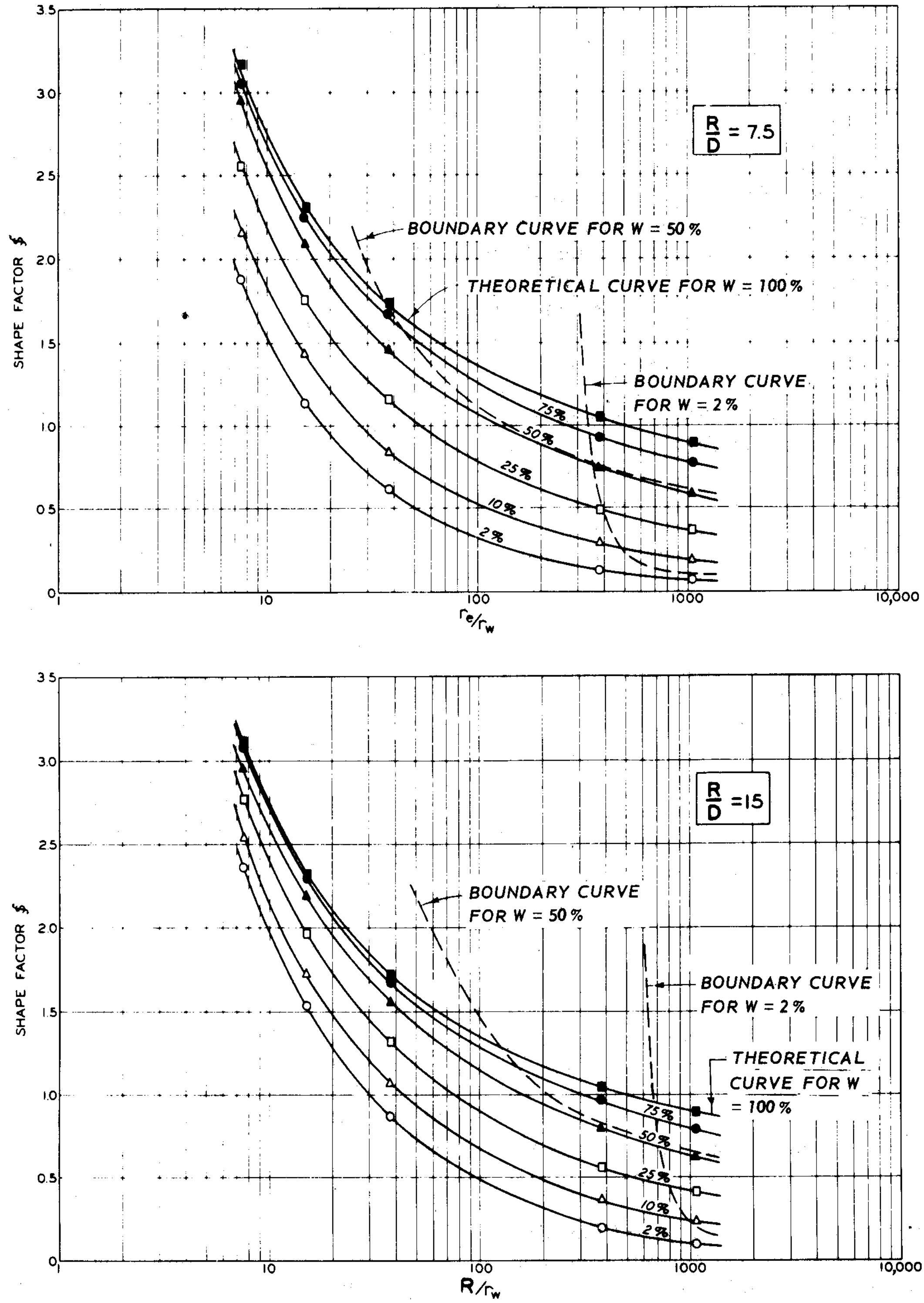
d. The flow per unit length (for sectional flow nets) or depth (for plan flow nets) can be computed by means of equations (1) and (2), and (5) and (6), respectively (fig. 4-27). Drawdowns from either sectional or plan flow nets can be computed from equations (3) and (4) (fig. 4-27). In plan flow nets for artesian flow, the equipotential lines correspond to various values of H—h, whereas for gravity flow, they correspond to $H^2 - h^2$. Since section equipotential lines for gravity flow conditions are curved rather than vertical, plan flow nets for gravity flow conditions give erroneous results for large drawdowns and should always be used with caution.

e. Plan flow nets give erroneous results if used to analyze partially penetrating drainage systems, the error being inversely proportional to the percentage of penetration. They give fairly accurate results if the penetration of the drainage system exceeds 80 percent and if the heads are adjusted as described in the following paragraph.

f. In previous analyses of well systems by means of flow nets, it was assumed that dewatering or drainage wells were spaced sufficiently close to be simulated by a continuous drainage slot and that the drawdown $(H - h_D)$ required to dewater an area equaled the average drawdown at the drainage slot or in the lines of wells $(H - h_e)$. These analyses give the amount of flow Q_T that must be pumped to achieve $H - h_D$ but do not give the drawdown at the wells. The drawdown at the wells required to produce $H - h_D$ downstream or within a ring of wells can be computed (approximately) for artesian flow from plan flow nets by the equations shown in figure 4-28 if the wells have been spaced proportional to the flow lines as shown in figure 4-27. The drawdown at fully penetrating gravity wells can also be computed from equations given in figure 4-28.

4-4. Electrical analogy seepage models.

a. The laws governing flow of fluids through porous media and flow of electricity through pure resistance are mathematically similar. Thus, it is feasible to use electrical models to study seepage flows and pressure



NOTE: BOTTOM OF ALL WELLS SEALED (I.E. INSULATED IN MODEL).

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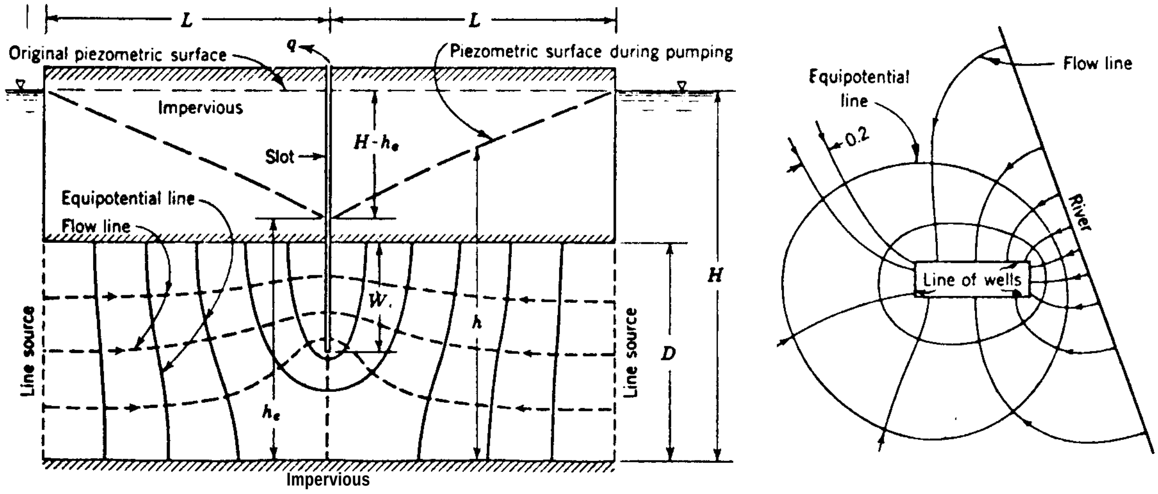
Figure 4-26. Shape factors for wells of various penetrations centered inside a circular source.



SECTIONAL FLOW NET (ARTESIAN)

(a)

PLAN FLOW NET

(b)

SECTIONAL FLOW NET

FLOW

ARTESIAN $q = kH'\$$ (1)

GRAVITY $q = kH''\$$ (2)

DRAWDOWN AT ANY POINT

ARTESIAN $H - h = H'\frac{n_e}{N_e}$ (3)

GRAVITY $H^2 - h^2 = \frac{n_e}{N_e}\left[H^2 - (h_o + h_s)^2\right]$ (4)

PLAN FLOW NET

FLOW

ARTESIAN $Q_T = kDH'\$$ (5)

GRAVITY $Q_T = \frac{kH''\$}{2}$ (6)

DRAWDOWN AT ANY POINT

USE EQ 3 AND 4, FOR ARTESIAN AND GRAVITY FLOW CONDITIONS, RESPECTIVELY.

$H' = H - h_e$ $H'' = H^2 - h_o^2$ $\$ =$ SHAPE FACTOR $= \frac{N_f}{N_e}$ $N_f =$ NUMBER OF FLOW CHANNELS IN NET

$N_e =$ TOTAL NUMBER OF EQUIPOTENTIAL DROPS BETWEEN FULL HEAD, H, AND HEAD AT EXIT, h_e

$n_e =$ NUMBER OF EQUIPOTENTIAL DROPS FROM EXIT TO POINT AT WHICH HEAD, h, IS DESIRED

h_o IS SHOWN IN FIG. 4-1

SEE FIG. 4-29 FOR HEAD CORRECTION FACTORS,

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Figure 4-27. Flow and drawdown to slots computed from flow nets.

distribution for various seepage conditions. Both two- and three-dimensional models can be used to solve seepage problems.

b. Darcy's law for two-dimensional flow of water (previously identified as equation (1) in fig. 4-27) through soil can be expressed for unit length of soil formations as follows:

$$q = kH'\$ \tag{4-3}$$

CONSTRUCT PLAN FLOW NET. SPACE WELLS PROPORTIONAL TO FLOW LINES. COMPUTE TOTAL FLOW TO SYSTEM FROM EQ 5 FOR ARTESIAN FLOW OR EQ 6 FOR GRAVITY FLOW (FIG. 4-28). ASSUME IN EQ 5 $H' = H - h_D$. SEE FIG. 4-20, b, c; 4-22, b; AND 4-26 FOR EXPLANATION OF TERMS.

ARTESIAN FLOW

FLOW TO EACH WELL

$$Q_w = \frac{Q_T}{n} \qquad (1)$$

WHERE n = NUMBER OF WELLS IN THE SYSTEM

DRAWDOWN AT WELLS

FULLY PENETRATING

$$H - h_w = \frac{Q_w}{kD}\left(\frac{n}{\$} + \frac{1}{2\pi}\ln\frac{a}{2\pi r_w}\right) \qquad (2)$$

PARTIALLY PENETRATING

$$H - h_w = \frac{Q_w}{kD}\left(\frac{n}{\$} + \theta_a\right) \qquad (3)$$

WHERE θ_a IS OBTAINED FROM FIG. 4-21 †

HEAD INCREASES MIDWAY BETWEEN AND DOWNSTREAM OF WELLS MAY BE COMPUTED FROM EQUATIONS GIVEN IN FIG. 4-20 AND 4-21.

GRAVITY FLOW

FLOW TO EACH WELL

USE EQ 1

DRAWDOWN AT FULLY PENETRATING WELL

$$H^2 - h_w^2 = \frac{Q_w}{k}\left(\frac{n}{\$} + \frac{1}{\pi}\ln\frac{a\dagger}{\pi r_w}\right) \qquad (4)$$

HEAD INCREASES MIDWAY BETWEEN AND DOWNSTREAM OF WELLS MAY BE COMPUTED FROM EQUATIONS GIVEN IN FIG. 4-22.

† THE AVERAGE WELL SPACING MAY BE USED TO COMPUTE θ_a, θ_m, AND THE DRAWDOWN AT AND BETWEEN WELLS.

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Figure 4-28. *Flow and drawdown to wells computed from flow-net analyses*

where

q = rate of flow
k = coefficient of permeability
H' = differential head
$\$$ = shape factor dependent on the geometry of the system

c. Ohm's law expresses the analogous condition for steady flow of electricity through a medium of pure resistance as follows:

$$I = \frac{E}{\rho} \qquad (4\text{-}4)$$

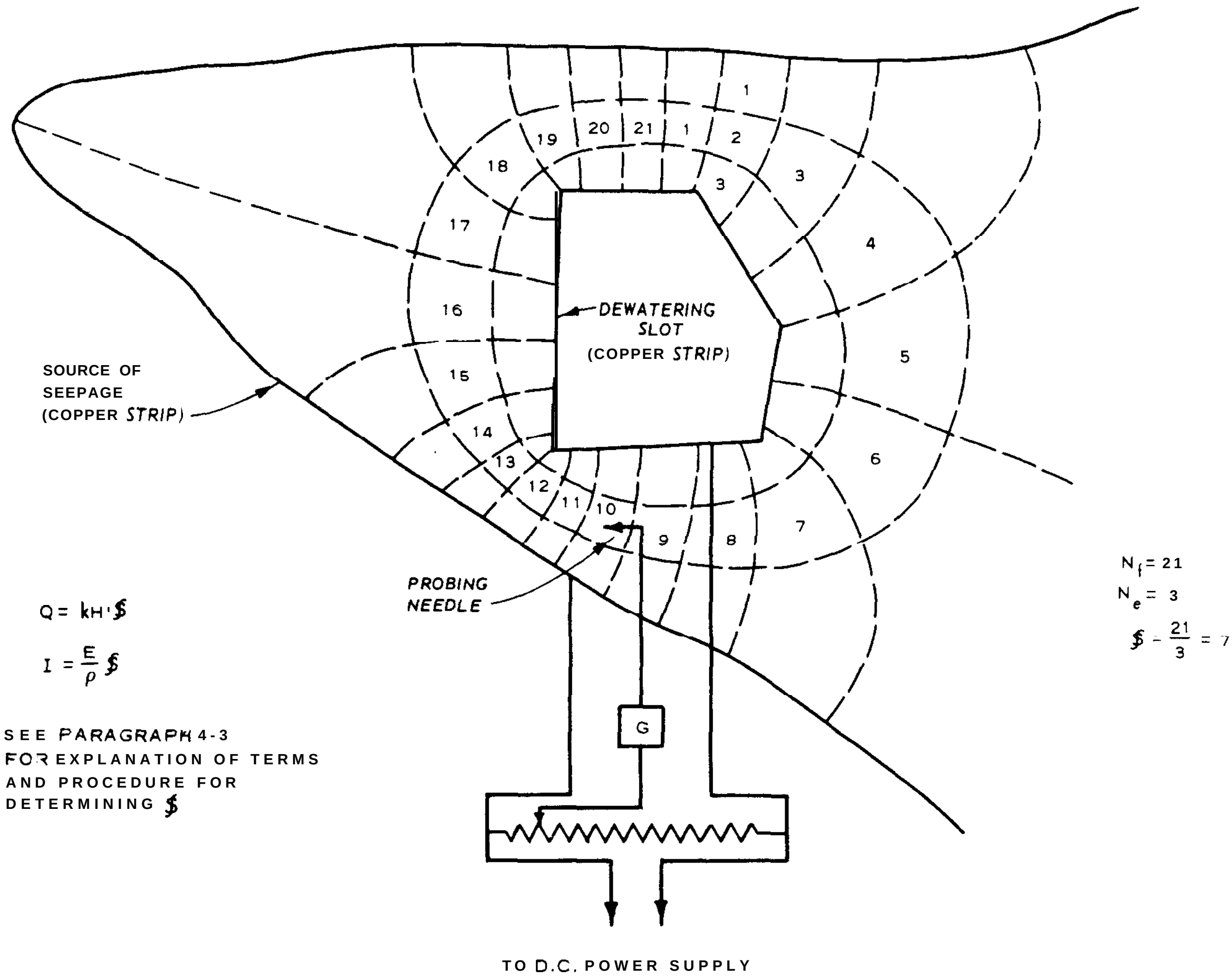
where

I = rate of flow of electricity
E = potential difference or voltage
ρ = specific resistance of electrolyte

Since the permeability in fluid flow is analogous to the reciprocal of the specific resistance for geometrically similar mediums, the shape factors for Darcy's law and Ohm's law are the same.

d. A two-dimensional flow net can be constructed using a scale model of the flow and drainage system made of a conductive material representing the porous media (graphite-treated paper or an electrolytic solution), copper or silver strips for source of seepage and drainage, and nonconductive material representing impervious flow boundaries, The electrical circuit consists of a potential applied across the model and a Wheatstone bridge to control intermediate potentials on the model (fig. 4-29). The flow net is constructed by tracing lines of constant potential on the model, thus establishing the flow-net equipotential lines after which the flow lines are easily added graphically. A flow net constructed using an electrical analogy model may be analyzed in the same manner as one constructed as in paragraph 4-3.

e. Equipment for conducting three-dimensional electrical analogy model studies is available at the WES. The equipment consists basically of a large plexiglass tank filled with diluted copper sulfate solution and having a calibrated, elevated carrier assembly for the accurate positioning of a point electrode probe anywhere in the fluid medium. A prototype is simulated by fabricating appropriately shaped and sealed source and sink configurations and applying an electrical potential across them. The model is particularly useful for analyzing complex boundary conditions that cannot be readily analyzed by two-dimensional techniques.



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(Fruco & Associates, Inc.)

Figure 4-29. Diagrammatic layout of electrical analogy model.

4-5. Numerical analyses.

a. Many complex seepage problems, including such categories as steady confined, steady unconfined, and transient unconfined can be solved using the finite element method. Various computer codes are available at the WES and the NAVFAC program library to handle a variety of two- and three-dimensional seepage problems. The codes can handle most cases of nonhomogeneous and anisotropic media.

b. A general computer code for analyzing partially penetrating random well arrays has been developed based on results of three-dimensional electrical analogy model tests at the WES. The computer code provides a means for rapidly analyzing trial well systems in which the number of wells and their geometric configuration can be varied to determine quantities of seepage and head distributions. Wells of different radii and penetrations can be considered in the analysis.

4-6. **Wellpoints,** wells, and filters.

Wells and wellpoints should be of a type that will prevent infiltration of filter material or foundation sand, offer little resistance to the inflow of water, and resist corrosion by water and soil. Wellpoints must also have sufficient penetration of the principal water-carrying strata to intercept seepage without excessive residual head between the wells or within the dewatered area.

a. *Wellpoints.* Where large flows are anticipated, a high-capacity type of wellpoint should be selected. The inner suction pipe of self-jetting wellpoints should permit inflow of water with a minimum hydraulic head loss. Self-jetting wellpoints should be designed so that most of the jet water will go out the tip of the point, with some backflow to keep the screen flushed clean while jetting the wellpoint in place.

(1) *Wellpoint screens.* Generally, wellpoints are covered with 30- to 60-mesh screen or have an equivalent slot opening (0.010 to 0.025 inch). The mesh should meet filter criteria given in *c* below. Where the soil to be drained is silty or fine sand, the yield of the wellpoint and its efficiency can be greatly improved by placing a relatively uniform, medium sand filter around the wellpoint. The filter should be designed in accordance with criteria subsequently set forth in *c* below. A filter will permit the use of screens or slots with larger openings and provide a more pervious material around the wellpoint, thereby increasing its effective radius (*d* below).

(2) *Wellpoint hydraulics.* The hydraulic head losses in a wellpoint system must be considered in designing a dewatering system. These losses can be estimated from figure 4-25.

b. Wells. Wells for temporary dewatering and permanent drainage systems may have diameters ranging from 4 to 18 inches with a screen 20 to 75 feet long depending on the flow and pump size requirements.

(1) *Well screens.* Screens generally used for dewatering wells are slotted (or perforated) steel pipe, perforated steel pipe wrapped with galvanized wire, galvanized wire wrapped and welded to longitudinal rods, and slotted polyvinyl chloride (PVC) pipe, Riser pipes for most dewatering wells consist of steel or PVC pipe. Screens and riser for permanent wells are usually made of stainless steel or PVC. Good practice dictates the use of a filter around dewatering wells, which permits the use of fairly large slots or perforations, usually 0.025 to 0.100 inch in size. The slots in well screens should be as wide as possible but should meet criteria given in c below.

(2) *Open screen area.* The open area of a well screen should be sufficient to keep the entrance velocity for the design flow low to reduce head losses and to minimize incrustation of the well screen in certain types of water. For temporary dewatering wells installed in nonincrusting groundwater, the entrance velocity should not exceed about 0.15 to 0.20 foot per second; for incrusting groundwater, the entrance velocity should not exceed 0.10 to 0.20 foot per second. For permanent drainage wells, the entrance velocity should not exceed about 0.10 foot per second. As the flow to and length of a well screen is usually dictated by the characteristics of the aquifer and drawdown requirements, the required open screen area can be obtained by using a screen of appropriate diameter with a maximum amount of open screen area.

(3) *Well hydraulics.* Head losses within the well system discussed in paragraph 4-2*a*(5) can be estimated from figure 4-24.

c. *Filters.* Filters are usually 3 to 5 inches thick for wellpoints and 6 to 8 inches thick for large-diameter wells (fig. 4-30). To prevent infiltration of the aquifer materials into the filter and of filter materials into the well or wellpoint, without excessive head losses, filters should meet the following criteria:

Screen-filter criteria

$$\text{Slot or screen openings} \leqq \text{minimum filter } D_{50}$$

Filter-aquifer criteria

$$\frac{\text{Max filter } D_{15}}{\text{Min aquifer } D_{85}} \leqq 5; \quad \frac{\text{Max filter } D_{50}}{\text{Min aquifer } D_{50}} \leqq 25;$$

$$\frac{\text{Min filter } D_{15}}{\text{Max aquifer } D_{15}} \geqq 2 \text{ to } 5$$

If the filter is to be tremied in around the screen for a well or wellpoint, it may be either uniformly or rather widely graded; however, if the filter is not tremied into place, it should be quite uniformly graded ($D_{90}/D_{10} \leqq 3$ to 4) and poured in around the well in a heavy, continuous stream to minimize segregation.

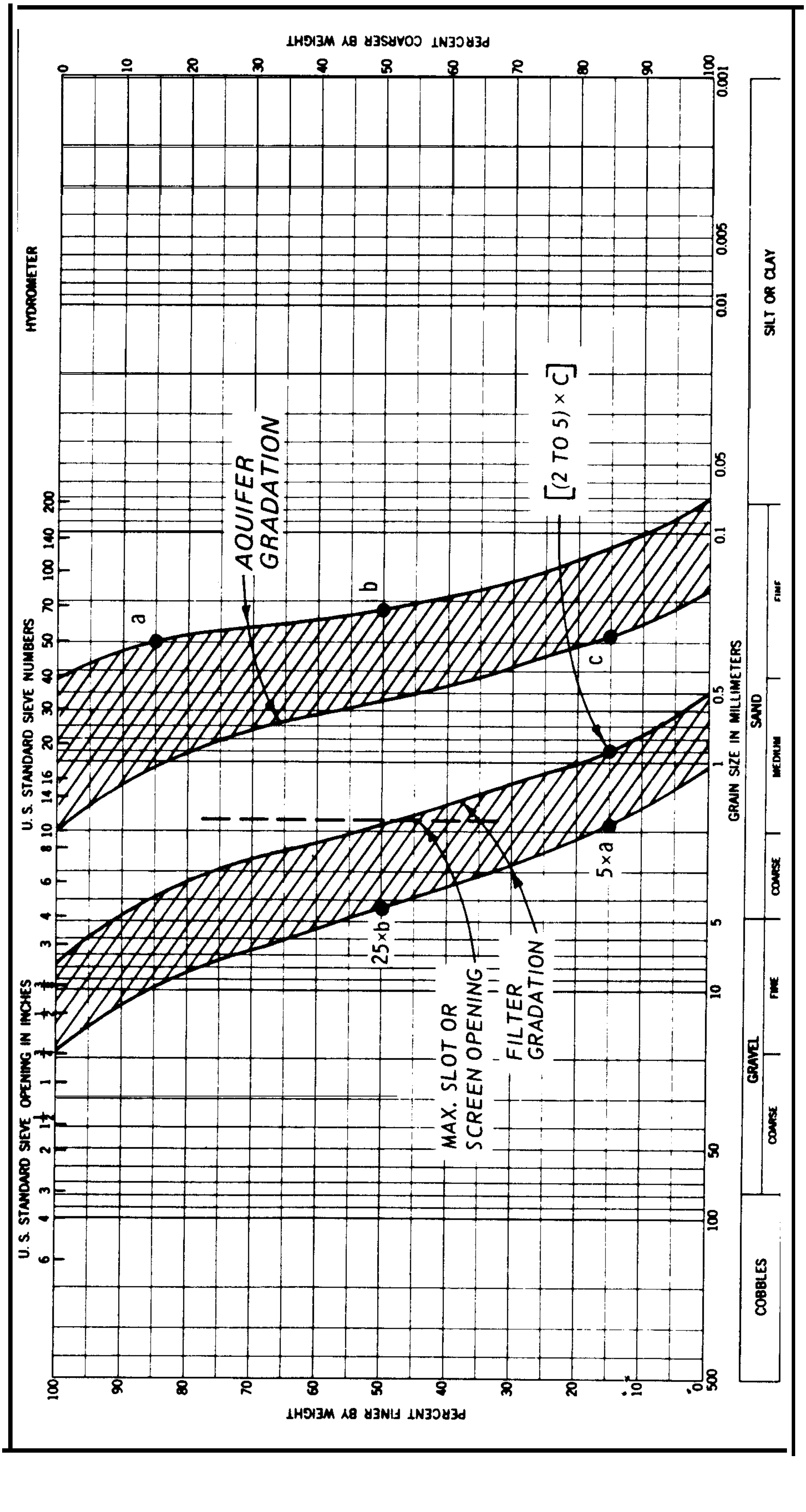

Figure 4-30. Typical design of a filter for a well or wellpoint.

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d. Effective well radius. The "effective" radius r_w of a well is that well radius which would have no hydraulic entrance loss H_w. If well entrance losses are considered separately in the design of a well or system of wells, r_w for a well or wellpoint without a filter may be considered to be one-half the outside diameter of the well screens; where a filter has been placed around a wellpoint or well screen, r_w may generally be considered to be one-half the outside diameter or the radius of the filter.

e. *Well penetration.* In a stratified aquifer, the effective well penetration usually differs from that computed from the ratio of the length of well screen to total thickness of the aquifer. A method for determining the required length of well screen W to achieve an effective penetration $\overline{W}$ in a stratified aquifer is given in appendix E.

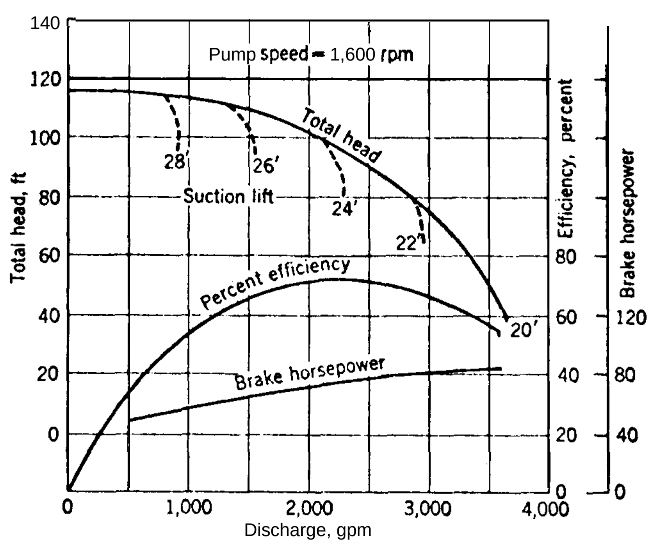
f. Screen length, penetmtion, and diameter. The length and penetration of the screen depends on the thickness and stratification of the strata to be dewatered (para 4-2*a*(6)). The length and diameter of the screen and the area of perforations should be sufficient to permit the inflow of water without exceeding the entrance velocity given in *b*(2) above. The "wetted screen length h_{ws}" (or h_w for each stratum to be dewatered) is equal to or greater than Q_w/q_c (para 4-2*a*(4) and (6)). The diameter of the well screen should be at least 3 to 4 inches larger than the pump bowl or motor.

4-7. **Pumps, headers, and discharge pipes.** The capacity of pumps and piping should allow for possible reduction in efficiency because of incrustation or mechanical wear caused by prolonged operation. This equipment should also be designed with appropriate valves, crossovers, and standby units so that the system can operate continuously, regardless of interruption for routine maintenance or breakdown.

a. *Centrifugal and wellpoin t pumps.*

(1) Centrifugal pumps can be used as sump pumps, jet pumps, or in combination with an auxiliary vacuum pump as a wellpoint pump. The selection of a pump and power unit depends on the discharge, suction lift, hydraulic head losses, including velocity head and discharge head, air-handling requirement, power available, fuel economy, and durability of unit. A wellpoint pump, consisting of a self-priming centrifugal pump with an attached auxiliary vacuum pump, should have adequate air-handling capacity and be capable of producing a vacuum of at least 22 to 25 feet of water in the headers. The suction lift of a wellpoint pump is dependent on the vacuum available at the pump bowl, and the required vacuum must be considered in determining the pumping capacity of the pump. Characteristics of a typical 8-inch wellpoint pump are shown in figure 4-31. Characteristics of a typical wellpoint pump vacuum unit are shown in figure 4-32. Sump pumps of the centrifugal type should be self-priming and capable of developing at least 20 feet of vacuum. Jet pumps are high head pumps; typical characteristics of a typical 6-inch jet pump are shown in figure 4-33.

(2) Each wellpoint pump should be provided with one *connected* standby pump so as to ensure continuity



(Courtesy of Griffin Wellpoint Corp. and "Foundation Engineering," McGraw Hill Book Company)

Figure 4-31. Characteristics of 8-inch Griffin wellpoint pump.

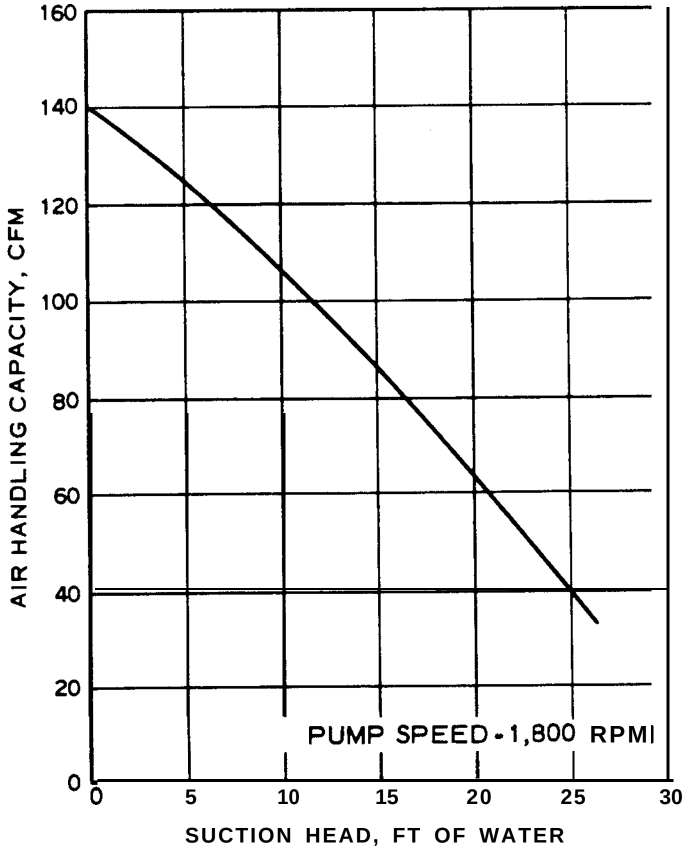


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Figure 4-32. Characteristics of typical vacuum unit for wellpoint pumps

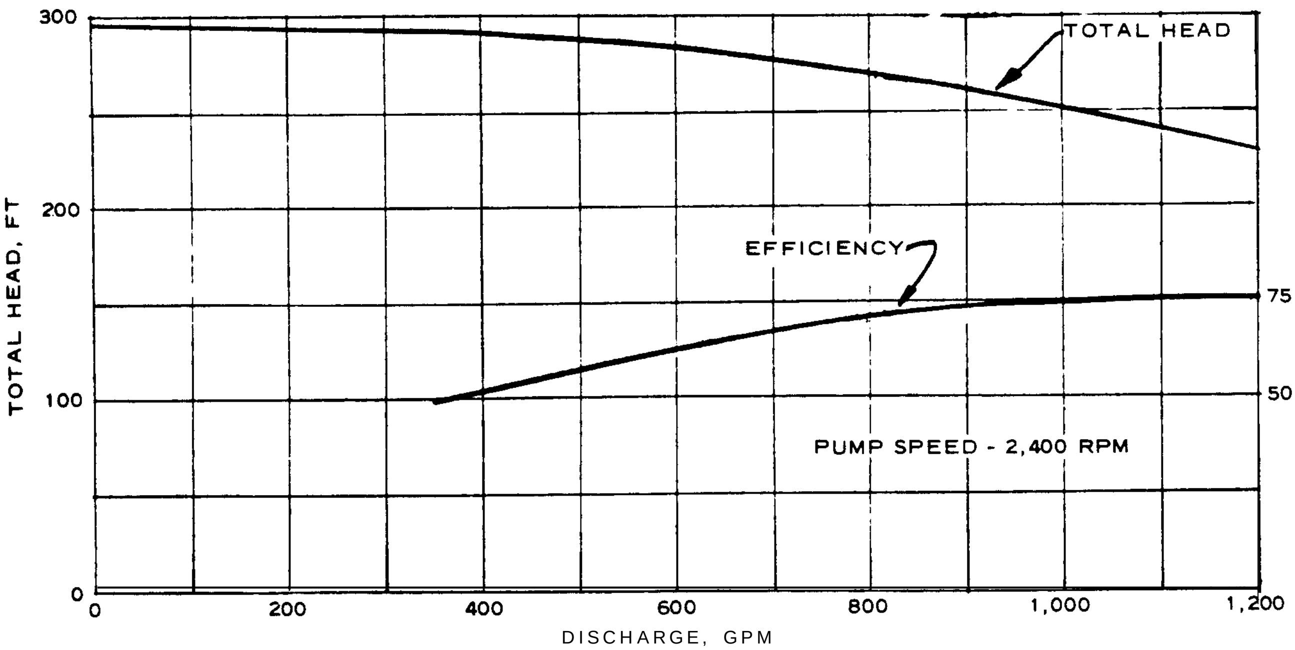

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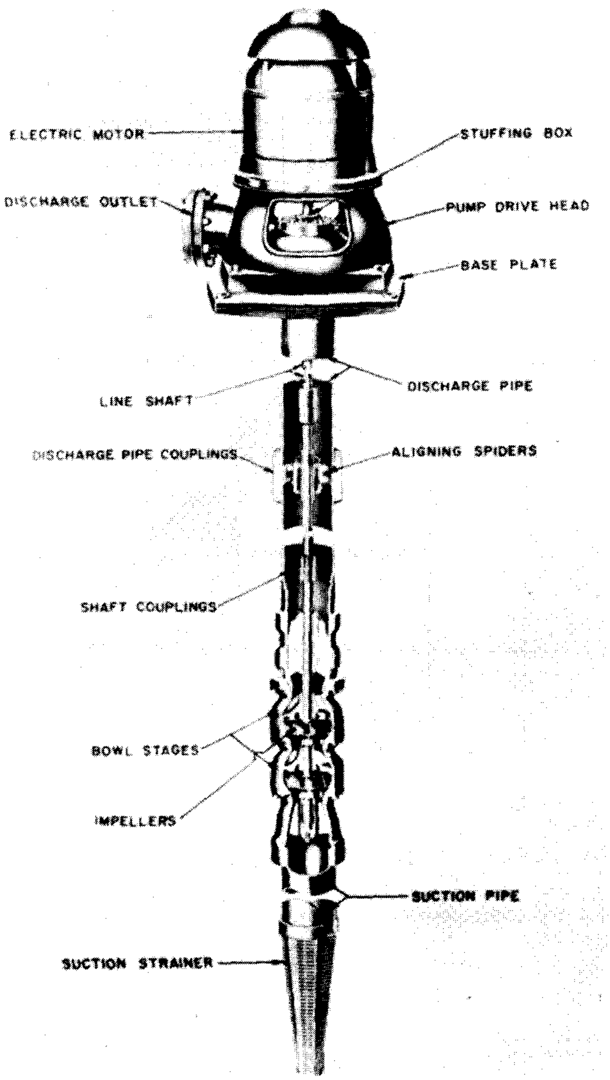
Figure 4-33. Characteristics of 6-inch jet pump.



(Courtesy of Layne & Bowler, Inc., Memphis, Tenn.)

Figure 4-34. Deep-well turbine pump.

of operation in event of pump or engine failure, or for repair or maintenance. By overdesigning the header pipe system and proper placement of valves, it may be possible to install only one standby pump for every two operational pumps. If electric motors are used for powering the normally operating pumps, the standby pumps should be powered with diesel, natural or LP gas, or gasoline engines. The type of power selected will depend on the power facilities at the site and the economics of installation, operation, and maintenance, It is also advisable to have spare power units on site in addition to the standby pumping units. Automatic switches, starters, and valves may be required if failure of the system is critical.

b. Deep-well pumps.

(1) Deep-well turbine or submersible pumps are generally used to pump large-diameter deep wells and consist of one or more stages of impellers on a vertical shaft (fig. 4-34). Turbine pumps can also be used as sump pumps, but adequate stilling basins and trash racks are required to assure that the pumps do not become clogged. Motors of most large-capacity turbine pumps used in deep wells are mounted at the ground surface. Submersible pumps are usually used for pumping deep, low-capacity wells, particularly if a vacuum is required in the well.

(2) In the design of deep-well pumps, consideration must be given to required capacity, size of well

Table 4-2. Capacity of Various Size Submersible and Deep- Well Turbine Pumps

Maximum Pump Bowl or Motor Size inches	Inside Diameter of Well inches	Approximate Maximum Capacity gallons per minute	
		Deep Well	Submersible
4	5-6	90	70
5	6-8	160	--
6	8-10	450	250
8	10-12	600	400
10	12-14	1,200	700
12	14-16	1,800	1,100
14	16-18	2,400	--
16	18-20	3,000	--

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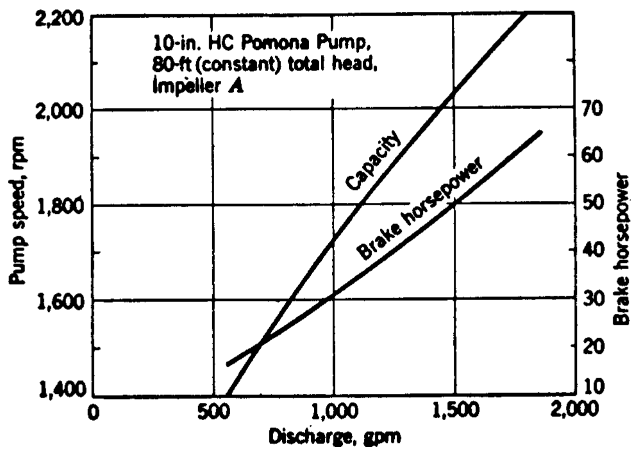
screen and riser pipe, total pumping head, and the lowered elevation of the water in the well. The diameter of the pump bowl must be determined before the wells are installed, as the inside diameter of the well casing should be at least 3 to 4 inches larger in diameter than the pump bowl. Approximate capacities of various turbine pumps are presented in table 4-2. The characteristics of a typical three-stage, lo-inch turbine pump are shown in figure 4-35.

(3) Submersible pumps require either electric power from a commercial source or one or more motor generators. If commercial power is used, 75 to 100 percent of (connected) motor generator power, with automatic starters unless operational personnel are on duty at all times, should be provided as standby for the commercial power. Spare submersible pumps, generally 10 to 20 percent of the number of operating pumps, as well as spare starters, switches, heaters, and fuses, should also be kept at the site.

(4) Deep-well turbine pumps can be powered with either electric motors or diesel engines and gear drives. Where electric motors are used, 50 to 100 percent of the pumps should be equipped with combination gear drives connected to diesel (standby) engines. The number of pumps so equipped would depend upon the criticality of the dewatering or pressure relief needs. Motor generators may also be used as standby for commercial power. For some excavations and subsurface conditions, automatic starters may be required for the diesel engines or motor generators being used as backup for commercial power.

c. Turbovacuum pumps. For some wellpoint systems requiring high pumping rates, it may be desirable



(Courtesy of Fairbanks Morse, Inc., Pump Division)

Figure 4-35. Rating curves for a three-stage 10-inch-high capacity deep-well pump.

to connect the header pipe to a 30- or 36-inch collection tank about 20 or 30 feet deep, sealed at the bottom and top, and pump the flow into the tank with a high-capacity deepwell turbine pump using a separate vacuum pump connected to the top of the tank to produce the necessary vacuum in the header pipe for the wells or wellpoints.

d. Header pipe.

(1) Hydraulic head losses caused by flow through the header pipe, reducers, tees, fittings, and valves should be computed and kept to a minimum (1 to 3 feet) by using large enough pipe. These losses can be computed from equivalent pipe lengths for various fittings and curves.

(2) Wellpoint header pipes should be installed as close as practical to the prevailing groundwater elevation and in accessible locations. Wellpoint pumps should be centrally located so that head losses to the ends of the system are balanced and as low as possible. If suction lift is critical, the pump should be placed low enough so that the pump suction is level with the header, thereby achieving a maximum vacuum in the header and the wellpoints. If construction is to be performed in stages, sufficient valves should be provided in the header to permit addition or removal of portions of the system without interrupting operation of the remainder of the system. Valves should also be located to permit isolation of a portion of the system in case construction operations should break a swing connection or rupture a header.

(3) Discharge lines should be sized so that the head losses do not create excessive back pressure on the pump. Ditches may be used to carry the water from the construction site, but they should be located well back of the excavation and should be reasonably watertight.

4-8. Factors of safety.

a. General. The stability of soil in areas of seepage emergence is critical in the control of seepage. The exit gradient at the toe of a slope or in the bottom of an excavation must not exceed that which will cause surface raveling or sloughing of the slope, piping, or heave of the bottom of the excavation.

b. Uplift. Before attempting to control seepage, an analysis should be made to ensure that the seepage or uplift gradient is equal to or less than that computed from the following equations:

$$i \leq \frac{\gamma'_m}{\gamma_w\ (FS = 1.25 \text{ to } 1.5)} \qquad (4\text{-}5)$$

or

$$Ah = \frac{(\gamma'_m)^T}{\gamma_w\ (FS = 1.25 \text{ to } 1.5)} \qquad (4\text{-}6)$$

where

i = seepage gradient $\Delta h/L$
γ'_m = submerged unit weight of soil
γ_w = unit weight of water
Δh = artesian head above bottom of slope or excavation
T = thickness of less pervious strata overlying a more pervious stratum
L = distance through which Ah acts

In stratified subsurface soils, such as a course-grained pervious stratum overlain by a finer grained stratum of relatively low permeability, most of the head loss through the entire section would probably occur through the finer grained material. Consequently, a factor of safety based on the head loss through the top stratum would probably indicate a more critical condition than if the factor of safety was computed from the total head loss through the entire section. Also, when gradients in anisotropic soils are determined from flow nets, the distance over which the head is lost must be obtained from the true section rather than the transformed section.

c. Piping. Piping cannot be analyzed by any rational method. In a study of'piping beneath hydraulic structures founded on *granular soils,* it was recommended that the (weighted) creep ratio C_w should equal or exceed the values shown in table 4-3 for various types of granular soils,

$$C_w = \frac{\text{I vertical seepage paths} + 1/3 \text{ I horizontal seepage paths}}{H - h_e} \qquad (4\text{-}7)$$

Table *4-3.* ***Minimum Creep Ratios for Various*** Granular ***Soils***

Soil	Creep Ratio
Very fine sand or silt	8.5
Fine sand	7
Medium sand	6
Coarse sand	5
Fine gravel	4
Medium gravel	3.5
Coarse gravel including cobbles	3
Boulders with some cobbles and gravel	2.5

From "Securityfrom Under-Seepage Masonry Dams, "by E.W. Lane, pp. 1235-1272. Transactions, American Society of Civil Engineers, 1935.

where $H - h_e$ represents vertical distance from the groundwater table to the bottom of the excavation. These criteria for piping are probably only applicable to dewatering of sheeted, cellular, or earth-dike cofferdams founded on granular soils. Once piping develops, erosion of the soil may accelerate rapidly. As the length of seepage flow is reduced, the hydraulic gradient and seepage velocity increase, with a resultant acceleration in piping and erosion. Piping can be controlled by lowering the groundwater table in the excavated slopes or bottom of an excavation, or in either less critical situations or emergencies by placement of filters over the seepage exit surface to prevent erosion of the soil but still permit free flow of the seepage. The gradation of the filter material should be such that the permeability is high compared with the aquifer, yet fine enough that aquifer materials will not migrate into or through the filter. The filter should be designed on the basis of criteria given in paragraph 4-6*c*. More than one layer of filter material may be required to stabilize a seeping slope or bottom of an excavation in order to meet these criteria.

d. Dewatering systems. As in the design of any works, the design of a dewatering system should include a factor of safety to cover the variations in characteristics of the subsurface soils, stratification, and groundwater table; the incompleteness of the data and accuracy of the formulations on which the design is based; the reduction in the efficiency of the dewatering system with time; the frailties of machines and operating personnel; and the criticality of failure of the system with regard to safety, economics, and damage to the project. All of these factors should be considered in selecting the factor of safety. The less information on which the design is based and the more critical the dewatering is to the success of the project, the higher the required factor of safety. Suggested factors of safety and design procedures are as follows:

(1) Select or determine the design parameters as accurately as possible from existing information.

(2) Use applicable design procedures and equations set forth in this manual.

(3) Consider the above enumerated factors in selecting a factor of safety.

(4) Evaluate the experience of the designer.

(5) After having considered steps 1-4, the following factors of safety are considered appropriate for modifying computed design values for flow, drawdown, well spacing, and required "wetted screen length."

Factor of Safety for Design (FS = 1.0 + (a+b+c))

		Factor to be Added to 1.0
(a)	*Design Data*	
	Poor	0.25
	Fair	0.20
	Good	0.10
	Excellent	0.05
(b)	*Experience of Designer*	
	Little	0.25
	Some	0.20
	Good	0.10
	Excellent	0.05
(c)	*Criticality*	
	Great	0.25
	Moderate	0.20
	Little	0.15

Application of Factor of Safety to Computed Values or System Design Features

Computed Value System Design Feature	Design Procedure	Remarks
Pump capacity, header, and discharge pipe (Q)	Increase Q to FS	....
Drawdown (Δh)	Decrease Δh by 10 percent	Adjust either drawdown or well spacing, but not both
Well spacing (a)	Decrease a by 10 percent	
Wetted screen length (h_{ws})	h_{ws}	....

Note: In initially computing drawdown, well spacing, and wetted screen, use flow and other parameters unadjusted for factor of safety.

In addition to these factors of safety being applied to design features of the system, the system should be pump-tested to verify its adequacy for the maximum required groundwater lowering and maximum river or groundwater table likely (normally a frequency of occurrence of once in 5 to 10 years for the period of exposure) to occur.

4-9. **Dewatering open excavations.** An excavation can be dewatered or the artesian pressure relieved by one or a combination of methods described in chapter 2. The design of dewatering and groundwater control systems for open excavations, shafts, and tunnels is discussed in the following paragraphs. Examples of design for various types of dewatering and pressure relief systems are given in appendix D.

a. Trenching and sump pumping.

(1) The applicability of trenches and sump pumping for dewatering an open excavation is discussed in chapter 2. Where soil conditions and the depth of an excavation below the water table permit trenching and sump pumping of seepage (fig. 2-1), the rate of flow into the excavation can be estimated from plan and sectional flow analyses (fig, 4-27) or formulas presented in paragraphs 4-2 through 4-5.

(2) Where an excavation extends into rock and there is a substantial inflow of seepage, perimeter drains can be installed at the foundation level outside of the formwork for a structure. The perimeter drainage system should be connected to a sump sealed off from the rest of the area to be concreted, and the seepage water pumped out. After construction, the drainage system should be grouted. Excessive hydrostatic pressures in the rock mass endangering the stability of the excavated face can be relieved by drilling 4-inch-diameter horizontal drain holes into the rock at approximately lo-foot centers. For large seepage inflow, supplementary vertical holes for deep-well pumps at 50- to 100-foot intervals may be desirable for temporary lowering of the groundwater level to provide suitable conditions for concrete placement.

b. Wellpoint system. The design of a line or ring of wellpoints pumped with either a conventional wellpoint pump or jet-eductors is generally based on mathematical or flow-net analysis of flow and drawdown to a continuous slot (para 4-2 through 4-5).

(1) *Conventional wellpoint system.* The drawdown attainable per stage of wellpoints (about 15 feet) is limited by the vacuum that can be developed by the pump, the height of the pump above the header pipe, and hydraulic head losses in the wellpoint and collector system. Where two or more stages of wellpoints are required, it is customary to design each stage so that it is capable of producing the total drawdown required by that stage with none of the upper stages functioning. However, the upper stages are generally left in so that they can be pumped in the event pumping of the bottom stage of wellpoints does not lower the water table below the excavation slope because of stratification, and so that they can be pumped during backfilling operations.

(a) The design of a conventional wellpoint system to dewater an open excavation, as discussed in paragraph 4-2*b*, is outlined below.

Step 1. Select dimensions and groundwater coefficients (H, L, and k) of the formation to be dewatered based on investigations outlined in chapter 3.

Step 2. Determine the drawdown required to dewater the excavation or to dewater down to the next stage of wellpoints, based on the maximum groundwater level expected during the period of operation.

Step 3. Compute the head at the assumed slot (h_e or h_o) to produce the desired residual head h_D in the excavation.

Step 4. Compute the flow per lineal foot of drainage system to the slot Q_p.

Step 5. Assume a wellpoint spacing a and compute the flow per wellpoint, $Q_w = aQ_p$.

Step 6. Calculate the required head at the wellpoint h_w corresponding to Q_w.

Step 7. Check to see if the suction lift that can be produced by the wellpoint pump V will lower the water level in the wellpoint to $h_w(p)$ as follows:

$$V \geqq M - h_w(p) + H_c + H_w \tag{4-8}$$

where

v = vacuum at pump intake, feet of water
M = distance from base of pervious strata to pump intake, feet
H_c = average head loss in header pipe from wellpoint, feet
H_w = head loss in wellpoint, riser pipe, and swing connection to header pipe, feet

Step 8. Set the top of the wellpoint screen at least 1 to 2 feet or more below $h_w—H_w$ to provide adequate submergence of the wellpoint so that air will not be pulled into the system.

(b) An example of the design of a two-stage wellpoint system to dewater an excavation is illustrated in figure D-l, appendix D.

(c) If an excavation extends below an aquifer into an underlying impermeable soil or rock formation, some seepage will pass between the wellpoints at the lower boundary of the aquifer. This seepage may be intercepted with ditches or drains inside the excavation and removed by sump pumps. If the underlying stratum is a clay, the wellpoints may be installed in holes drilled about 1 to 2 feet into the clay and backfilled with filter material, By this procedure, the water level at the wellpoints can be maintained near the bottom of the aquifer, and thus seepage passing between the wellpoints will be minimized. Sometimes these procedures are ineffective, and a small dike in the ex-

cavation just inside the toe of the excavation may be required to prevent seepage from entering the work area. Sump pumping can be used to remove water from within the diked area.

(2) *Jet-eductor (well or) wellpoint systems.* Flow and drawdown to a jet-eductor (well or) wellpoint system can be computed or analyzed as discussed in paragraph 4-2*b*. Jet-eductor dewatering systems can be designed as follows:

Step 1. Assume the line or ring of wells or wellpoints to be a drainage slot.

Step 2. Compute the total flow to the system for the required drawdown and penetration of the well screens.

Step 3. Assume a well or wellpoint spacing that will result in a reasonable flow for the well or wellpoint and jet-eductor pump.

Step 4. Compute the head at the well or wellpoint h_w required to achieve the desired drawdown.

Step 5. Set eductor pump at $M = h_w - H_w$ with some allowance for future loss of well efficiency.

The wells or wellpoints and filters should be selected and designed in accordance with the criteria set forth under paragraph *4-6.*

(*a*) If the soil formation being drained is stratified and an appreciable flow of water must be drained down through the filter around the riser pipe to the wellpoint, the spacing of the wellpoints and the permeability of the filter must be such that the flow from formations above the wellpoints does not exceed

$$Q_w = k_v A i \tag{4-9}$$

where

Q_w = flow from formation above wellpoint
k_v = vertical permeability of filter
A = horizontal area of filter
i = gradient produced by gravity = 1.0

Substitution of small diameter well screens for wellpoints may be indicated for stratified formations. Where a formation is stratified or there is little available submergence for the wellpoints, jet-eductor wellpoints and risers should be provided with a pervious filter, and the wellpoints set at least 10 feet back from the edge of a vertical excavation.

(b) Jet-eductor pumps may be powered with individual small high-pressure centrifugal pumps or with one or two large pumps pumping into a single pressure pipe furnishing water to each eductor with a single return header. With a single-pump setup, the water is usually circulated through a stilling tank with an overflow for the flow from wells or wellpoints (fig. 2-6). Design of jet eductors must consider the static lift from the wells or wellpoints to the water level in the recirculation tank; head loss in the return riser pipe; head loss in the return header; and flow from the wellpoint. The (net) capacity of a jet-eductor pump depends on the pressure head, input flow, and diameter of the jet nozzle in the pump. Generally, a jet-eductor pump requires an input flow of about 2 to 2% times the flow to be pumped depending on the operating pressure and design of the nozzle. Consequently, if flow from the wells or wellpoints is large, a deep-well system will be more appropriate, The pressure header supplying a system of jet eductors must be of such size that a fairly uniform pressure is applied to all of the eductors.

(3) *Vacuum wellpoint system.* Vacuum wellpoint systems for dewatering fine-g-rained soils are similar to conventional wellpoint systems except the wellpoint and riser are surrounded with filter sand that is sealed at the top, and additional vacuum pump capacity is provided to ensure development of the maximum vacuum in the wellpoint and filter regardless of air loss. In order to obtain 8 feet of vacuum in a wellpoint and filter column, with a pump capable of maintaining a 25-foot vacuum in the header, the maximum lift is 25-8 or 17 feet. Where a vacuum type of wellpoint system is required, the pump capacity is small. The capacity of the vacuum pump will depend on the air permeability of the soil, the vacuum to be maintained in the filter, the proximity of the wellpoints to the excavation, the effectiveness of the seal at the top of the filter, and the number of wellpoints being pumped. In very fine-g-rained soils, pumping must be continuous. The flow may be so small that water must be added to the system to cool the pump properly.

c. Electroosmosis

(1) An electroosmotic dewatering system consists of anodes (positive electrodes, usually a pipe or rod) and cathodes (negative electrodes, usually wellpoints or small wells installed with a surrounding filter), across which a d-c voltage is applied. The depth of the electrodes should be at least 5 feet below the bottom of the slope to be stabilized. The spacing and arrangement of the electrodes may vary, depending on the dimensions of the slope to be stabilized and the voltage available at the site. Cathode spacings of 25 to 40 feet have been used, with the anodes installed midway between the cathodes. Electrical gradients of 1.5- to 4-volts-per-foot distance between electrodes have been successful in electroosmotic stabilization. The electrical gradient should be less than about 15 volts per foot of distance between electrodes for long-term installations to prevent loss in efficiency due to heating the ground. Applied voltages of 30 to 100 volts are usually satisfactory; a low voltage is usually sufficient if the groundwater has a high mineral content.

(2) The discharge of a cathode wellpoint may be estimated from the equation

$$Q_e = k_e i_e a z \tag{4-10}$$

where

k_e = coefficient of electroosmotic permeability (assume 0.98 x10^{-4} feet per second per volt per foot)

i_e = electrical gradient between electrodes, volts per foot

a = effective spacing of wellpoints, feet

z = depth of soil being stabilized, feet

Current requirements commonly range between 15 and 30 amperes per well, and power requirements are generally high. However, regardless of the expense of installation and operation of an electroosmotic dewatering system, it may be the only effective means of dewatering and stabilizing certain silts, clayey silts, and clayey silty sands. Electroosmosis may not be applicable to saline soils because of high current requirements, nor to organic soils because of environmentally objectionable effluents, which may be unsightly and have exceptionally high pH values.

d. Deep-well systems

(1) The design and analysis of a deep-well system to dewater an excavation depends upon the configuration of the site dewatered, source of seepage, type of flow (artesian and gravity), penetration of the wells, and the submergence available for the well screens with the required drawdown at the wells. Flow and drawdown to wells can be computed or analyzed as discussed in paragraph 4-2*b*.

(2) Methods are presented in paragraphs 4-2*b* and 4-3 whereby the flow and drawdown to a well system can be computed either by analysis or by a flow net assuming a continuous slot to represent the array of wells, and the drawdown at and between wells computed for the actual well spacing and location. Examples of the design of a deep-well system using these methods and formulas are presented in figures D-2 and D-3.

(3) The submerged length and size of a well screen should be checked to ensure that the design flow per well can be achieved without excessive screen entrance losses or velocities. The pump intake should be set so that adequate submergence (a minimum of 2 to 5 feet) is provided when all wells are being pumped. Where the type of seepage (artesian and gravity) is not well established during the design phase, the pump intake should be set 5 to 10 feet below the design elevation to ensure adequate submergence. Setting the pump bowl below the expected drawdown level will also facilitate drawdown measurements,

e. *Combined systems.*

(1) Well and wellpoint systems. A dewatering system composed of both deep wells and wellpoints may be appropriate where the groundwater table has to be lowered appreciably and near to the top of an impermeable stratum. A wellpoint system alone would require several stages of wellpoints to do the job, and a well system alone would not be capable of lowering the groundwater completely to the bottom of the aquifer. A combination of deep wells and a single stage of wellpoints may permit lowering to the desired level. The advantages of a combined system, in which wells are essentially used in place of the upper stages of wellpoints, are as follows:

(a) The excavation quantity is reduced by the elimination of berms for installation and operation of the upper stages of wellpoints.

(b) The excavation can be started without a delay to install the upper stages of wellpoints.

(c) The deep wells installed at the top of the excavation will serve not only to lower the groundwater to permit installation of the wellpoint system but also to intercept a significant amount of seepage and thus reduce the flow to the single stage of wellpoints. A design example of a combined deep-well and wellpoint system is shown in figure D-4.

(2) *Sand drains with deep wells and wellpoints,* Sand drains can be used to intercept horizontal seepage from stratified deposits and conduct the water vertically downward into a pervious stratum that can be dewatered by means of wells or wellpoints. The limiting feature of dewatering by sand drains is usually the vertical permeability of the sand drains itself, which restricts this method of drainage to soils of low permeability that yield only a small flow of water. Sand drains must be designed so that they wlll intercept the seepage flow and have adequate capacity to allow the seepage to drain downward without any back pressure. To accomplish this, the drains must be spaced, have a diameter, and be filled with filter sand so that

$$Q_D \leq k_D i A_D = k_v A_D \qquad (4\text{-}11)$$

where

Q_D = flow per drain

k_D = vertical permeability of sand filter

i = gradient produced by gravity = 1.0

A_D = area of drain

Generally, sand drains are spaced on 5- to 15-foot centers and have a diameter of 10 to 18 inches. The maximum permeability k_v of a filter that may be used to drain soils for which sand drains are applicable is about 1000 to 3000 × 10^{-4} centimetres per second or 0.20 to 0.60 feet per minute. Thus, the maximum capacity Q_D of a sand drain is about 1 to 3 gallons per minute. An example of a dewatering design, including sand drains, is presented in figure D-5. The capacity of sand drains can be significantly increased by installing a small (1- or 1½-inch) slotted PVC pipe in the drain to conduct seepage into the drain downward into underlying more pervious strata being dewatered.

f. Pressure relief systems.

(1) Temporary relief of artesian pressure beneath an open excavation is required during construction

where the stability of the bottom of the excavation is endangered by artesian pressures in an underlying aquifer. Complete relief of the artesian pressures to a level below the bottom of the excavation is not always required depending on the thickness, uniformity, and permeability of the materials. For uniform tight shales or clays, an upward seepage gradient i as high as 0.5 to 0.6 may be safe, but clay silts or silts generally require lowering the groundwater 5 to 10 feet below the bottom of the excavation to provide a dry, stable work area.

(2) The flow to a pressure relief system is artesian; therefore, such a system may be designed or evaluated on the basis of the methods presented in paragraphs 4-2 and 4-3 for *artesian flow*. The penetration of the wells or wellpoints need be no more than that required to achieve the required drawdown to keep the flow to the system a minimum. If the aquifer is stratified and anisotropic, the penetration required should be determined by computing the effective penetration into the transformed aquifer as described in appendix E. Examples of the design of a wellpoint system and a deep-well system for relieving pressure beneath an open excavation are presented in figures D-6 and D-7.

g. *Cutoffs.* Seepage cutoffs are used as barriers to flow in highly permeable aquifers in which the quantity of seepage would be too great to handle with deep-well or wellpoint dewatering systems alone, *or when pumping costs would be large and a cutoff is more economical.* The cutoff should be located far enough back of the excavation slope to ensure that the hydrostatic pressure behind the cutoff does not endanger the stability of the slope. If possible, a cutoff should penetrate several feet into an underlying impermeable stratum. However, the depth of the aquifer or other conditions may preclude full penetration of the cutoff, in which case seepage beneath the cutoff must be considered. Figure 4-36 illustrates the effectiveness of a partial cutoff for various penetrations into an aquifer. The figure also shows the soils to be homogeneous and isotropic with respect to permeability. If, however, the soils are stratified or anisotropic with respect to permeability, they must be transformed into an isotropic section and the equivalent penetration computed by the method given in appendix E before the curves shown in the figure are applicable.

(1) *Cement and chemical grout curtain.* Pressure injection of grout into a soil or rock may be used to reduce the permeability of the formation in a zone and seal off the flow of water. The purpose of the injection of grout is to fill the void spaces with cement or chemicals and thus form a solid mass through which no water can flow. Portland cement, fly ash, bentonite, and sodium silicate are commonly used as grout materials. Generally, grouting pressures should not exceed about 1 pound per square inch per foot of depth of the injection.

(a) Portland cement is best adapted to filling voids and fractures in rock and has the advantage of appreciably strengthening the formation, but it is ineffective in penetrating the voids of sand with an effective grain size of 1 millimetre or less. To overcome this deficiency, chemical grouts have been developed that have nearly the viscosity of water, when mixed and injected, and later react to form a gel which seals the formation. Chemical grouts can be injected effectively into soils with an effective grain size D_{10} that is less than 0.1 millimetre. Cement grout normally requires a day or two to hydrate and set, whereas chemical grout can be mixed to gel in a few minutes.

(b) Cement grouts are commonly mixed at water-cement ratios of from 5:1 to 10:1 depending on the grain size of the soils. However, the use of a high water-cement ratio will result in greater shrinkage of the cement, so it is desirable to use as little water as practical. Bentonite and screened fly ash may be added to a cement grout to both improve the workability and reduce the shrinkage of the cement. The setting time of a cement grout can be accelerated by using a 1:1 mixture of gypsum-base plaster and cement or by adding not more than 3 percent calcium chloride. High-early-strength cement can be used when a short set time is required.

(c) Chemical grouts, both liquid and powder-based, are diluted with water for injection, with the proportions of the chemicals and admixtures varied to control the gel time.

(d) Injection patterns and techniques vary with grout materials, character of the formation, and geometry of the grout curtains. (Grout holes are generally spaced on 2- to 5-foot centers.) Grout curtains may be formed by successively regrouting an area at reduced spacings until the curtain becomes tight. Grouting is usually done from the top of the formation downward.

(e) The most perplexing problem connected with grouting is the uncertainty about continuity and effectiveness of the seal. Grout injected under pressure will move in the direciton of least resistance. If, for example, a sand deposit contains a layer of gravel, the gravel may take all the grout injected while the sand remains untreated. Injection until the grout take diminishes is not an entirely satisfactory measure of the success of a grouting operation. The grout may block the injection hole or penetrate the formation only a short distance, resulting in a discontinuous and ineffective grout curtain. The success of a grouting operation is difficult to evaluate before the curtain is complete and in operation, and a considerable construction delay can result if the grout curtain is not effective. A single row of grout holes is relatively ineffective for cutoff purposes compared with an effectiveness of 2 or 3 times

that of overlapping grout holes. Detailed information on grouting methods and equipment is contained in TM 5-818-6.

(2) *Slurry walls.* The principal features of design of a slurry cutoff wall include: viscosity of slurry used for excavation; specific gravity or density of slurry; and height of slurry in trench above the groundwater table. The specific gravity of the slurry and its level above the groundwater table must be high enough to ensure that the hydrostatic pressure exerted by the slurry will prevent caving of the sides of the trench and yet not limit operation of the excavating equipment. Neither shall the slurry be so viscous that the backfill will not move down through the slurry mix, Typical values of specific gravity of slurries used range from about 1.1 to 1.3 (70 to 80 pounds per cubic foot)

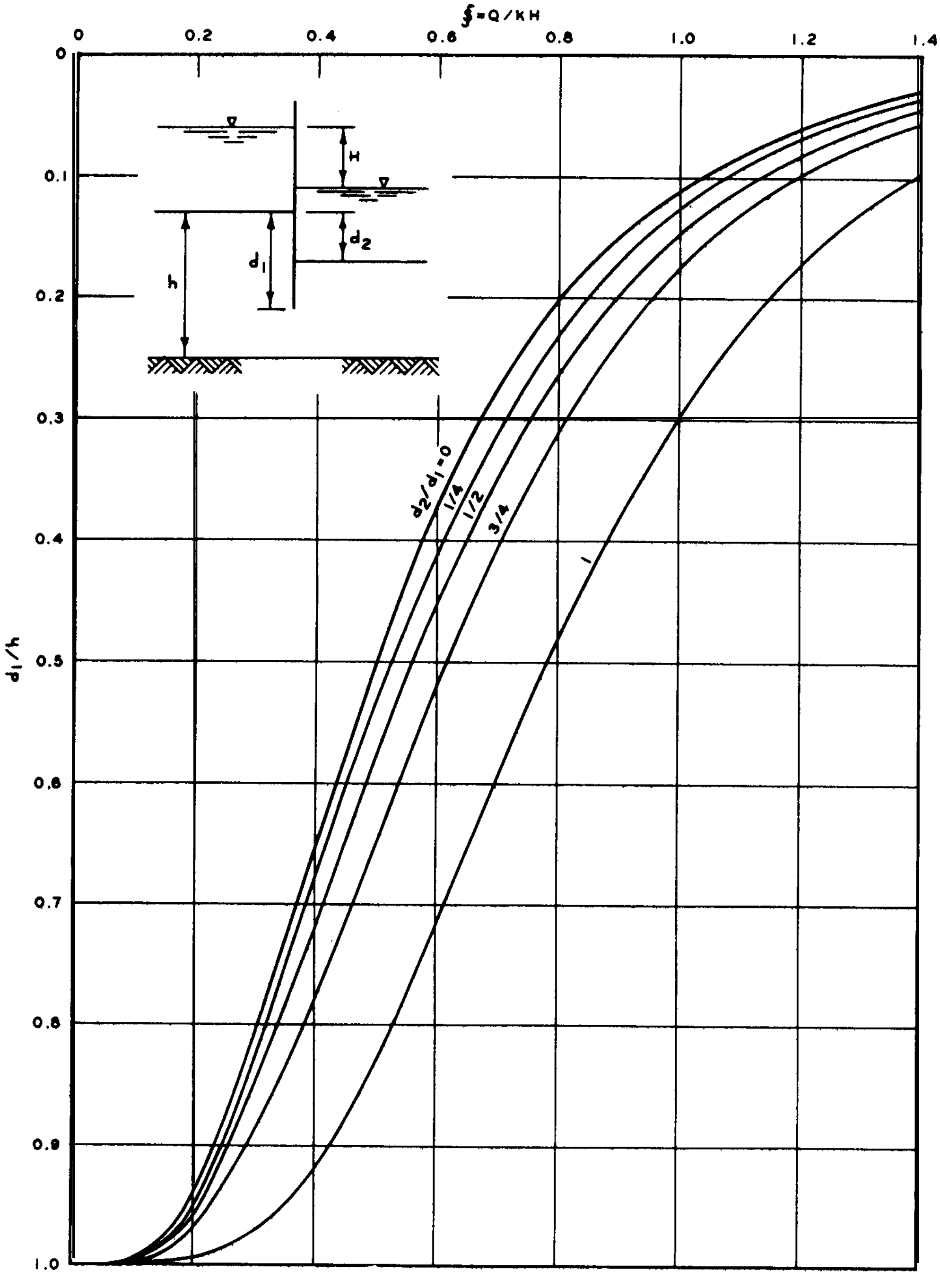


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Figure 4-36. Flow beneath a partially penetrating cutoff wall with sand or weighting material added. The viscosity of the slurry for excavating slurry wall trenches usually ranges from a Marsh funnel reading of 65 to 90 seconds, as required to hold any weighting material added and to prevent any significant loss of slurry into the walls of the trench. The slurry should create a pressure in the trench approximately equal to 1.2 times the active earth pressure of the surrounding soil. Where the soil at the surface is loose or friable, the upper part of the trench is sometimes supported with sand bags or a concrete wall. The backfill usually consists of a mixture of soil (or a graded mix of sand-gravel-clay) and bentonite slurry with a slump of 4 to 6 inches.

(3) *Steel sheet piling.* Seepage cutoffs may be created by driving a sheet pile wall or cells to isolate an excavation in a river or below the water table. Sheet piles have the advantage of being commonly available and readily installed, However, if the soil contains cob bles or boulders, a situation in which a cutoff wall is applicable to dewatering, the driving may be very difficult and full penetration may not be attained. Also, obstructions may cause the interlocks of the piling to split, resulting in only a partial cutoff.

(*a*) Seepage through the sheet pile interlocks should be expected but is difficult to estimate. As an approximation, the seepage through a steel sheet pile wall should-be assumed equal to at least 0.01 gallon per square foot of wall per foot of net head acting on the wall. The efficiency of a sheet pile cutoff is sub stantial for short paths of seepage but is small or negligible for long paths.

(*b*) Sheet pile cutoffs that are installed for long-term operation will usually tighten up with time as the interlocks become clogged with rust and possible incrustation by the groundwater.

(4) *Freezing.* Freezing the water in saturated porous soils or rock to form an ice cutoff to the flow of groundwater may be applicable to control of groundwater for shafts or tunnels where the excavation is small but deep. (See para 4-12 for information on design and operation of freezing systems.)

4-10. Dewatering shafts and tunnels.

a. The requirements and design of systems for dewatering shafts and tunnels in cohesionless, porous soil or rock are similar to those previously described for open excavations, As an excavation for a shaft or tunnel is generally deep, and access is limited, deep-wells or jet-eductor wellpoints are considered the best method for dewatering excavations for such structures where dewatering techniques can be used. Grout curtains, slurry cutoff walls, and freezing may also be used to control groundwater adjacent to shafts or tunnels.

b. Where the soil or rock formation is reasonably homogeneous and isotropic, a well or jet-eductor system can be designed to lower the water table below the tunnel or bottom of the shaft using methods and formulas presented in paragraphs 4-1 through 4-4. If the soil or rock formation is stratified, the wells must be screened and filtered through each pervious stratum, as well as spaced sufficiently close so that the residual head in *each stratum* being drained is not more than 1 or 2 feet. Dewatering stratified soils penetrated by a shaft or tunnel by means of deep wells may be facilitated by sealing the wells and upper part of the riser pipe and applying a vacuum to the top of the well and correspondingly to the filter. Maintenance of a vacuum in the wells and surrounding earth tends to stabilize the earth and prevent the emergence of seepage into the tunnel or shaft.

c. In combined well-vacuum systems, it is necessary to use pumps with a capacity in excess of the maximum design flow so that the vacuum will be effective for the full length of the well screen. Submersible pumps installed in sealed wells must be designed for the static lift plus friction losses in the discharge pipe plus the vacuum to be maintained in the well. The pumps must also be designed so that they will pump water and a certain amount of air without cavitation. The required capacity of the vacuum pump can be estimated from formulas for the flow of air through porous media considering the maximum exposure of the tunnel facing or shaft wall at any one time to be the most pervious formation encountered, *assuming the porous stratum to be fully drained.* The flow of air through a porous medium, assuming an ideal gas flowing under isothermal conditions, is given in the following formula:

$$Q_a = \Delta p(D - h_w)k \frac{\mu_w}{\mu_a} \mathcal{S} \qquad (4\text{-}12)$$

where

Q_a = flow of air at mean pressure of air in flow system $\bar{p}$, cubic feet per minute
Δp = pressure differential $(p_1 - p_2)$ in feet of water
p_1 = absolute atmospheric pressure
p_2 = absolute air pressure at line of vacuum wells
D = thickness of aquifer, feet
h_w = head at well, feet
k = coefficient of permeability for water, feet per minute
μ_w = absolute viscosity of water
μ_a = absolute viscosity of air
$\mathcal{S}$ = geometric seepage shape factor (para 4-3)

The approximate required capacity of vacuum pump is expressed as

$$Q_{a-vp} = Q_a \times \frac{\bar{p}}{\text{absolute atmospheric pressure (feet of water)}} \qquad (4\text{-}13)$$

$$= \frac{Q_a \bar{p}}{34} \quad \text{(cubic feet per minute)}$$

where $\bar{p}$ represents mean absolute air pressure $\left(\frac{P_1 + P_2}{2}\right)$ in feet of water. Wells, with vacuum, on 15- to 20-foot centers have been used to dewater caissons and mine shafts 75 to 250 feet deep. An example of the design of a deep-well system supplemented with vacuum in the well filter and screen to dewater a stratified excavation for a shaft is shown in figure D-8, and an example to dewater a tunnel is shown in figure D-9.

d. In designing a well system to dewater a tunnel or shaft, it should be assumed that any one well or pumping unit may go out of operation. Thus, any combination of the other wells and pumping units must have sufficient capacity to provide the required water table lowering or pressure relief, Where electrical power is used to power the pumps being used to dewater a shaft or tunnel, a standby generator should be connected to the system with automatic starting and transfer equipment or switches.

4-11. Permanent pressure relief systems. Permanent drainage or pressure relief systems can be designed using equations and considerations previously described for various groundwater and flow conditions. The well screen, collector pipes, and filters should be designed for long service and with access provided for inspection and reconditioning during the life of the project. Design of permanent relief or drainage systems should also take into consideration potential encrustation and screen loss. The system should preferably be designed to function as a gravity system without mechanical or electrical pumping and control equipment. Any mechanical equipment for the system should be selected for its simplicity and dependability of operation. If pumping equipment and controls are required, auxiliary pump and power units should be provided. Piezometers and flow measuring devices should be included in the design to provide a means for controlling the operation and evaluating its efficiency,

4-12. Freezing.

a. General.

(1) The construction of a temporary waterstop by artificially freezing the soil surrounding an excavation site is a process that has been used for over a century, not always with success and usually as a last resort when more conventional methods had failed. The method may be costly and is time-consuming. Until recent years far too little engineering design has been used, but nowadays a specialist in frozen-soil engineering, given the site information he needs, can design a freezing system with confidence. However, every job needs care in installation and operation and cannot be left to a general contractor without expert help. A favorable site for artificial freezing is where the water table is high, the soil is, e.g., a running sand, and the water table cannot be drawn down because of possible damage to existing structures of water (in a coarser granular material). The freezing technique may be the best way to control water in some excavations, e.g., deep shafts.

(2) Frozen soil not only is an effective water barrier but also can serve as an excellent cofferdam. An example is the frozen cofferdam for an open excavation 220 feet in diameter and 100 feet deep in rubbishy fill, sands, silts, and decomposed rock. A frozen curtain wall 4000 feet long and 65 feet deep has been successfully made but only after some difficult problems had been solved. Mine shafts 18 feet in diameter and 2000 feet deep have been excavated in artificially frozen soils and rocks where no other method could be used. Any soil or fractured rock can be frozen below the water table to form a watertight curtain provided the freeze-pipes can be installed, but accurate site data are essential for satisfactory design and operation.

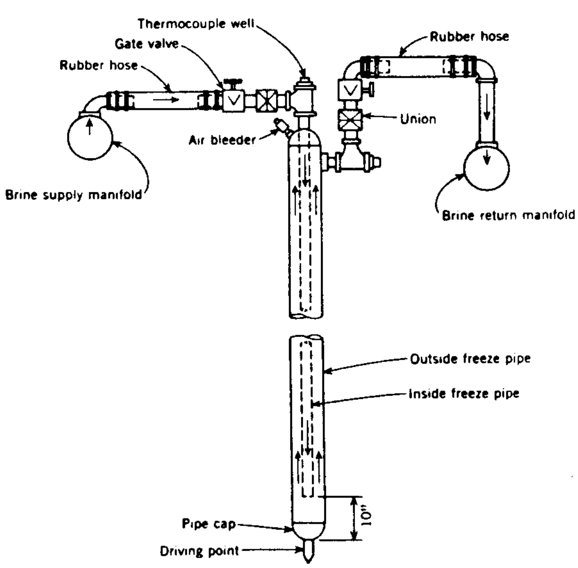
b. Design. As with the design of any system for subsurface water control, a thorough site study must first be made. *Moving water is the factor most likely to cause failure;* a simple sounding-well or piezometer layout (or other means) must be used to check this. If the water moves across the excavation at more than about 4 feet per day, the designer must include extra provisions to reduce the velocity, or a curtain wall may never close. If windows show up in the frozen curtain wall, flooding the excavation and refreezing with added freeze-pipes are nearly always necessary. A knowledge of the creep properties of the frozen soils may be needed; if the frozen soil is used as a cofferdam or earth retaining structure, such can be determined from laboratory tests. Thermal properties of the soils can usually be reliably estimated from published data, using dry unit weight and water content.

c. *Operation.* The ground is frozen by closed-end, steel freeze-pipes (usually vertical, but they can be driven, placed, or jacked at any angle) from 4 to 6 inches in diameter, spaced from 3 to 5 feet in one or more rows to an impervious stratum. If there is no impervious stratum within reach, the soil may be completely frozen as a block in which the excavation is made, or an impervious stratum may be made artificially. In one project, a horizontal disk about 200 feet across and 24 feet thick was frozen at a minimum depth of 150 feet. Then, a cylindrical cofferdam 140 feet in diameter was frozen down to the disk, and the enclosed soil was excavated without any water problem.

(1) Coaxial with each freeze-pipe is a 1% to 2-inch steel, or plastic, supply pipe delivering a chilled liquid (coolant) to the bottom of the closed freeze pipe. The

coolant flows slowly up the annulus between the pipes, pulls heat from the ground, and progressively freezes the soil, (A typical freeze-pipe is shown in fig. 4-37.) After a week or two, the separate cylinders of frozen soil join to form the barrier, which gradually thickens to the designed amount, generally at least 4 feet (walls of 24-foot thickness with two rows of freeze-pipes have been frozen in large and deep excavations in soft organic silts), The total freeze-time varies from 3 to 4 weeks to 6 months or more but is predictable with high accuracy, and by instrumentation and observation the engineer has good control. Sands of low water content freeze fastest; fine-grained soils of high water content take more time and total energy, although the refrigeration horsepower required may be greater than for sands.

(2) The coolant is commonly a chloride brine at zero to -20 degrees Fahrenheit, but lower temperatures are preferable for saving time, reducing the amount of heat to be extracted, and minimizing frost-heave effects (which must be studied beforehand). In recent years, liquid propane at −45°F has been used in large projects, and for small volumes of soil, liquid nitrogen that was allowed to waste has been used. (These cryogenic liquids demand special care-they are dangerous.) Coolant circulation is by headers, commonly 8-inch pipes, connected to a heat-exchanger at the refrigeration plant using freon (in a modern plant) as the refrigerant. The refrigeration equipment is usually rented for the job. A typical plant requires from 50 horsepower and up; 1000 horsepower or more has sometimes been used, Headers should be insulated and are recoverable. Freeze-pipes may be withdrawn but are often wasted in construction; they are sometimes used for thawing the soil back to normal, in which case they could be pulled afterward.



(From "Tunnel Driven Using Subsurface Freezing," by C. P. Gail, pp. 37-40. Civil Engineering, American Society of Civil Engineers, May 1972.)

Figure 4-37. Typical freeze-pipe.

d. Important considerations. The following items must be considered when the freezing technique is to be used:

(1) Water movement in soil.

(2) Location of freeze-pipes. (The spacing of freeze-pipes should not exceed the designed amount by more than 1 foot anywhere along the freeze wall.)

(3) Wall closure. (Freeze-pipes must be accurately located, and the temperature of the soil to be frozen carefully monitored with thermocouples to ensure 100 percent closure of the wall. Relief wells located at the center of a shaft may also be used to check the progress of freezing. By periodically pumping these wells, the effectiveness of the ice wall in sealing off seepage flow can be determined.)

(4) Frost-heave effects-deformations and pressures. (Relief wells may be used to relieve pressures caused by expansion of frozen soil.)

(5) Temperature effects on buried utilities.

(6) Insulation of aboveground piping.

(7) Control of surface water to prevent flow to the freezing region.

(8) Coolant and ground temperatures. (By monitoring coolant and soil temperatures, the efficiency of the freezing process can be improved.)

(9) Scheduling of operations to minimize lost time when freezing has been completed.

(10) Standby plant. (Interruption of coolant circulation may be serious. A standby plant with its own prime movers is desirable so as to prevent any thaw. A continuous advance of the freezing front is not necessary so that standby plant capacity is much less than that normally used.)

4-13. Control of surface water.

a. Runoff of surface water from areas surrounding the excavation should be prevented from entering the excavation by sloping the ground away from the excavation or by the construction of dikes around the top of the excavation. Ditches and dikes can be constructed on the slopes of an excavation to control the runoff of water and reduce surface erosion. Runoff into slope ditches can be removed by pumping from sumps installed in these ditches, or it can be carried in a pipe or lined ditch to a central sump in the bottom of the excavation where it can be pumped out, Dikes at the top of an excavation and on slopes should have at least 1 foot of freeboard above the maximum elevation of water to be impounded and a crown width of 3 to 5 feet with side slopes of 1V on 2-2.5H.

b. In designing a dewatering system, provision must be made for collecting and pumping out surface water so that the dewatering wells and pumps cannot be flooded. Control of surface water within the diked area will not only prevent interruption of the dewatering operation, which might seriously impair the stability of the excavation, but also prevent damage to the construction operations and minimize interruption of work. Surface water may be controlled by dikes, ditches, sumps, and pumps; the excavation slope can be protected by seeding or covering with fabric or asphalt. Items to be considered in the selection and design of a surface water control system include the duration and season of construction, rainfall frequency and intensity, size of the area, and character of surface soils.

c. The magnitude of the rainstorm that should be used for design depends on the geographical location, risk associated with damage to construction or the dewatering system, and probability of occurrence during construction. The common frequency of occurrence used to design surface water control sumps and pumps is a once in 2-to 5-year rainfall. For critical projects, a frequency of occurrence of once in 10 years may be advisable.

d. Impounding runoff on excavation slopes is somewhat risky because any overtopping of the dike could result in overtopoping of all dikes at lower elevations with resultant flooding of the excavation.

e. Ample allowance for silting of ditches should be made to ensure that adequate capacities are available throughout the duration of construction. The grades of ditches should be fairly flat to prevent erosion. Sumps should be designed that will minimize siltation and that can be readily cleaned. Water from sumps should not be pumped into the main dewatering system.

f. The pump and storage requirements for control of surface water within an excavation can be estimated in the following manner:

Step 1. Select frequency of rainstorm for which pumps, ditches, and sumps are to be designed.

Step 2. For selected frequency (e.g., once in 5 years), determine rainfall for 10-, 30-, and 60-minute rainstorms at project site from figure 3-6.

Step 3. Assuming instantaneous runoff, compute volume of runoff V_R (for each assumed rainstorm) into the excavation or from the drainage area into the excavation from the equation

$$V_R = cRA = c\ \frac{R}{12}\ 43{,}560A \text{ (cubic feet)} \qquad (4\text{-}14)$$

where

c = coefficient of runoff
R = rainfall for assumed rainstorm, inches
A = area of excavation plus area of drainage into excavation, acres

(The value of c depends on relative porosity, character, and slope of the surface of the drainage area. For impervious or saturated steep excavations, c values may be assumed to range from 0.8 to 1.0.)

Step 4. Plot values of V_R versus assumed duration of rainstorm.

Step 5. Plot pumpage rate of pump to be installed assuming pump is started at onset of rain.

This method is illustrated by figure D-10.

g. The required ditch and sump storage volume $\overline{V}$ is the (maximum) difference between the accumulated runoff for the various assumed rainstorms and the amount of water that the sump pump (or pumps) will remove during the same elapsed period of rainfall. The capacity and layout of the ditches and sumps can be adjusted to produce the optimum design with respect to the number, capacity, and location of the sumps and pumps.

h. Conversely, the required capacity of the pumps for pumping surface runoff depends upon the volume of storage available in sumps, as well as the rate of runoff (see equation (3-3)). For example, if no storage is available, it would be necessary to pump the runoff at the rate it enters the excavation to prevent flooding. This method usually is not practicable. In large excavations, sumps should be provided where practicable to reduce the required pumping capacity. The volume of sumps and their effect on pump size can be determined graphically (fig, D-10) or can be estimated approximately from the following equation:

$$Q_P = Q - \overline{V}/T \qquad (4\text{-}15)$$

where

Q_P = total pump capacity, cubic feet per second
Q = average rate of runoff, cubic feet per second
$\overline{V}$ = volume of sump storage, cubic feet
T = duration of rainfall, hours