9. FLOW IN OPEN CHANNELS.

a. For our purposes, an open channel is defied as any conduit in which water flows with a free-water surface such as an open ditch, creek, river, or canal. Although pipe drains act as open channels when flowing partially full, they are considered separately in paragraph 8 and elsewhere in this publication.

b. It is recommended that maximum use be made of open channels because of the relatively low cost thereof as compared with capacity attained. Obviously, open channels need to be consistent with safety and operational requirements of the airport. Channels shall be of adequate capacity and free from excessive maintenance which could result from erosion, silting, or steep back slopes. In the latter connection, it is noted that mowers cannot normally be used on slopes steeper than 2.5 to 1. Slopes steeper than 2.5 to 1 will usually need to be cut by hand or cut by mowers restrained by other equipment at the top of the slope. Some areas of the country, where soil is generally noncohesive and vegetation is likely to be sparse, use 3 to 1 as maximum.

c. The Manning formula may be applied directly to hard surfaced channels of concrete, corrugated metal, etc., and to earth channels using coefficient of roughness values for "n" as shown in Table II. That table also lists maximum permissible velocities for such channels.

d. A nomographic solution of the Manning formula can be made by using Figure 13. The channel-dimension diagrams for trapezoidal, triangular, and parabolic shaped channels, Figures 14 through 19 permit rapid solution of many ordinary channel problems.

e. The Manning formula is Q = AV where:

Q = rate of discharge or flow in cubic feet per second.

A = cross sectional area of the flow in square feet.

$V = \frac{1.486 R^{2/3} S^{1/2}}{n}$ = velocity of flow in feet per second.

n = coefficient of roughness.

R = hydraulic radius in feet or cross sectional area A divided by the wetted perimeter.

S = slope of energy gradient in feet per foot, considered equal to slope of channel bed for most situations.

f. The following indicates the use of the channel-dimension diagrams in Figures 14 to 19.

(1) For example, given Q = 100 cubic feet per second and S = 0.0002 feet per foot.

Find: The bottom width, depth of flow, and velocity of a drainage channel with 2.5 to 1 side slope. Assume channel will be maintained regularly and kept free of excessive vegetal growth. Estimate "n" to be 0.04. (See Table II for open channel "n" values).

Solution: Requires one or more trial solutions. The steps are: (a) select a velocity, (b) compute the area required from Q/V, (c) determine the required R from the nomograph of Figure 13, and (d) determine the bottom width and depth from the 2.5 to 1 dimension diagram of Figure 14.

Item	*Trial Solution* 1	2	3
V=feet per second (selected)	1.0	0.9	0.95
A=square feet (from Q/V)	100	111.1	105.6
R=feet (from Figure 13)	2.60	2.20	2.40
b=bottom width in feet (from Figure 14)	20	36	28
d=depth of flow (from Figure 14)	3.6	2.6	3.1

The solution chosen will depend upon depth and botom width of channel desired. If still not satisfactory, change *V* slightly and redetermine dimensions. Final selection of channel dimensions must consider addition of reasonable freeboard to depth of flow.

(2) For example, given Q = 50 c.f.s. and trapezoidal channel with side slopes of 3 to 1 and bottom width of 10 feet.

Find: The bed slope required to maintain a velocity of 2.5 f.p.s. Use "n" = 0.04.

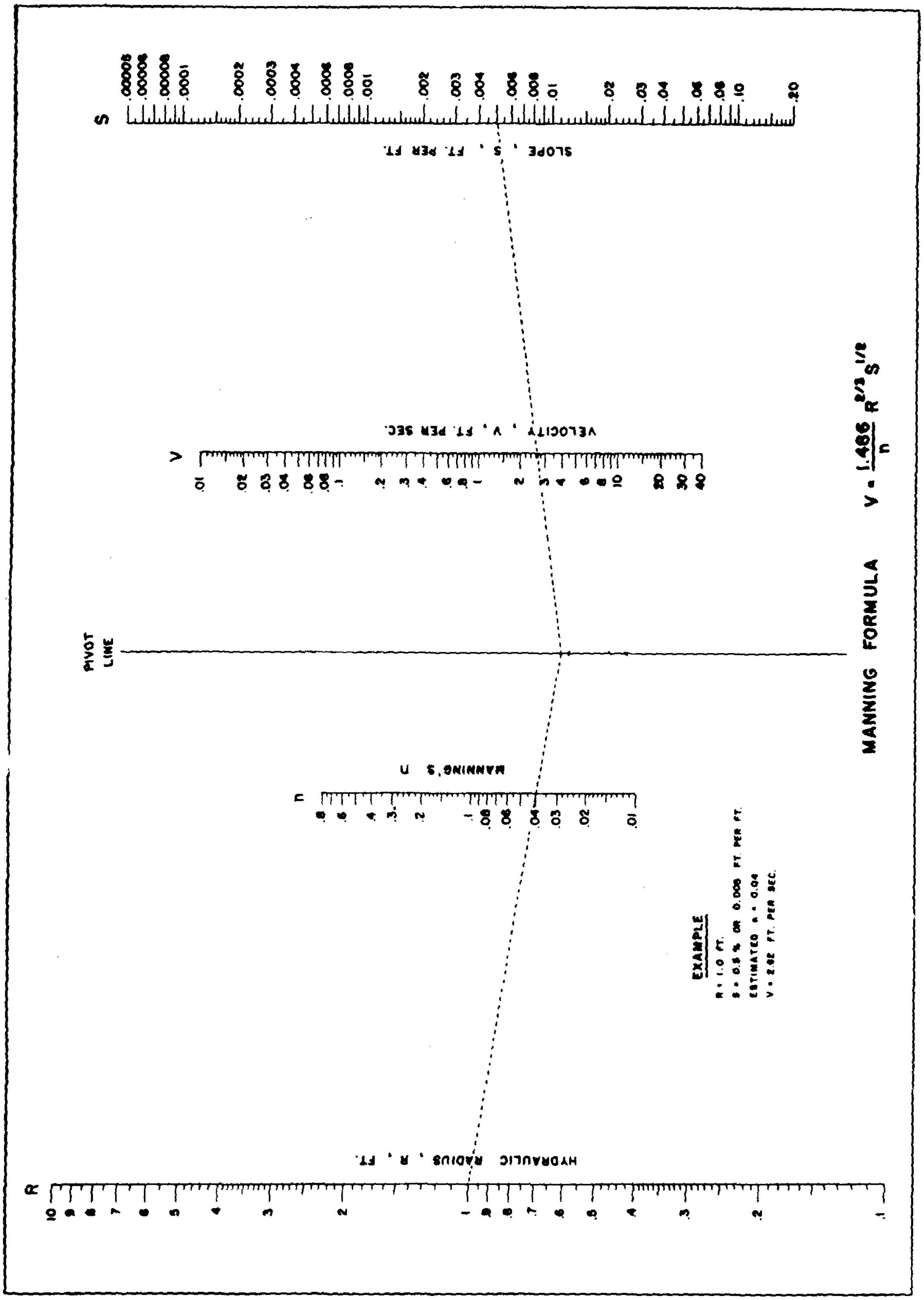


FIGURE 13. Solution of the Manning formula.

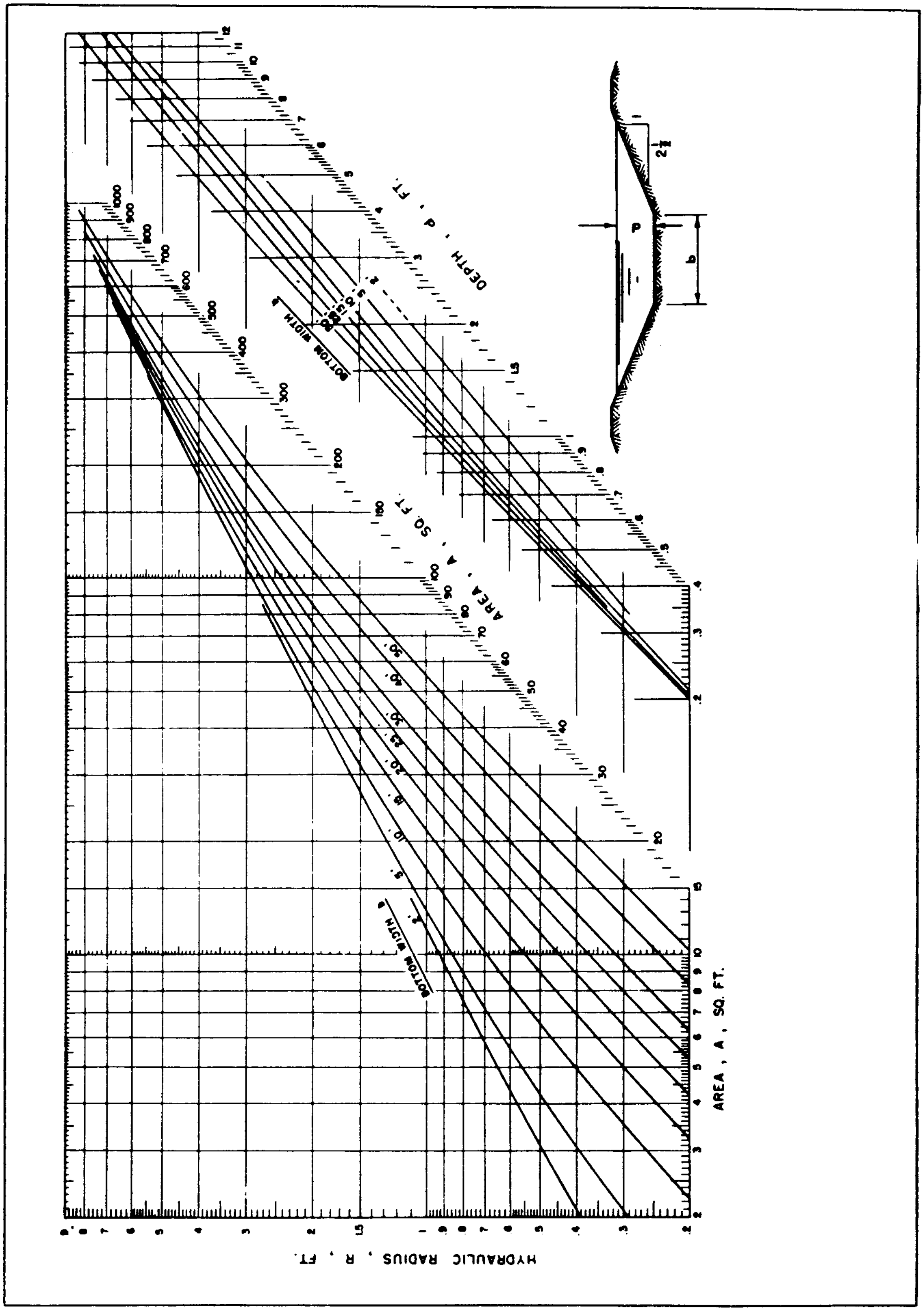


FIGURE 14. Dimensions of trapezoidal channels with 2½ to 1 side slope.

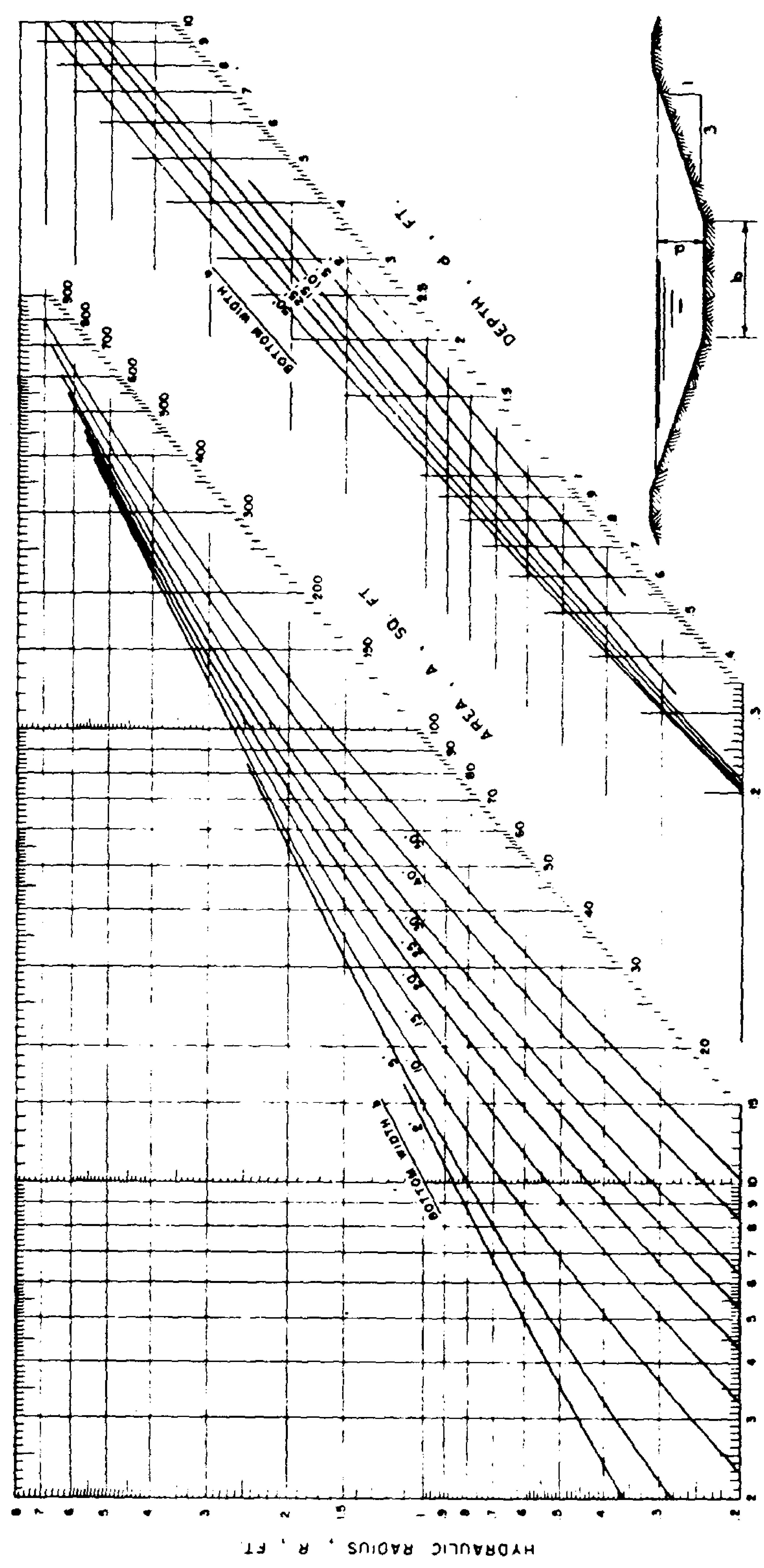


FIGURE 15. Dimensions of trapezoidal channels with 3 to 1 side slopes.

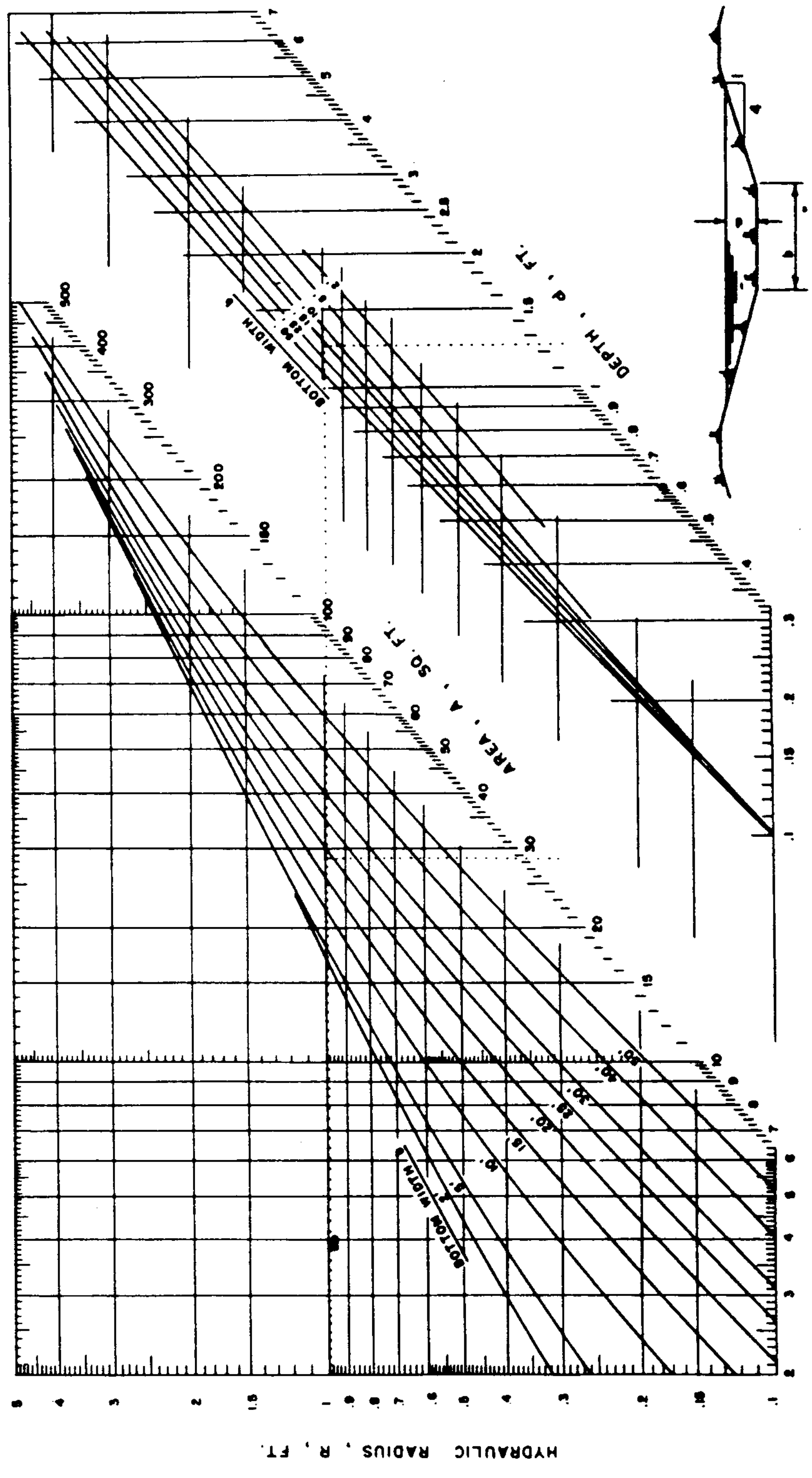


FIGURE 16. Dimensions of trapezoidal channels with 4 to 1 side slopes.

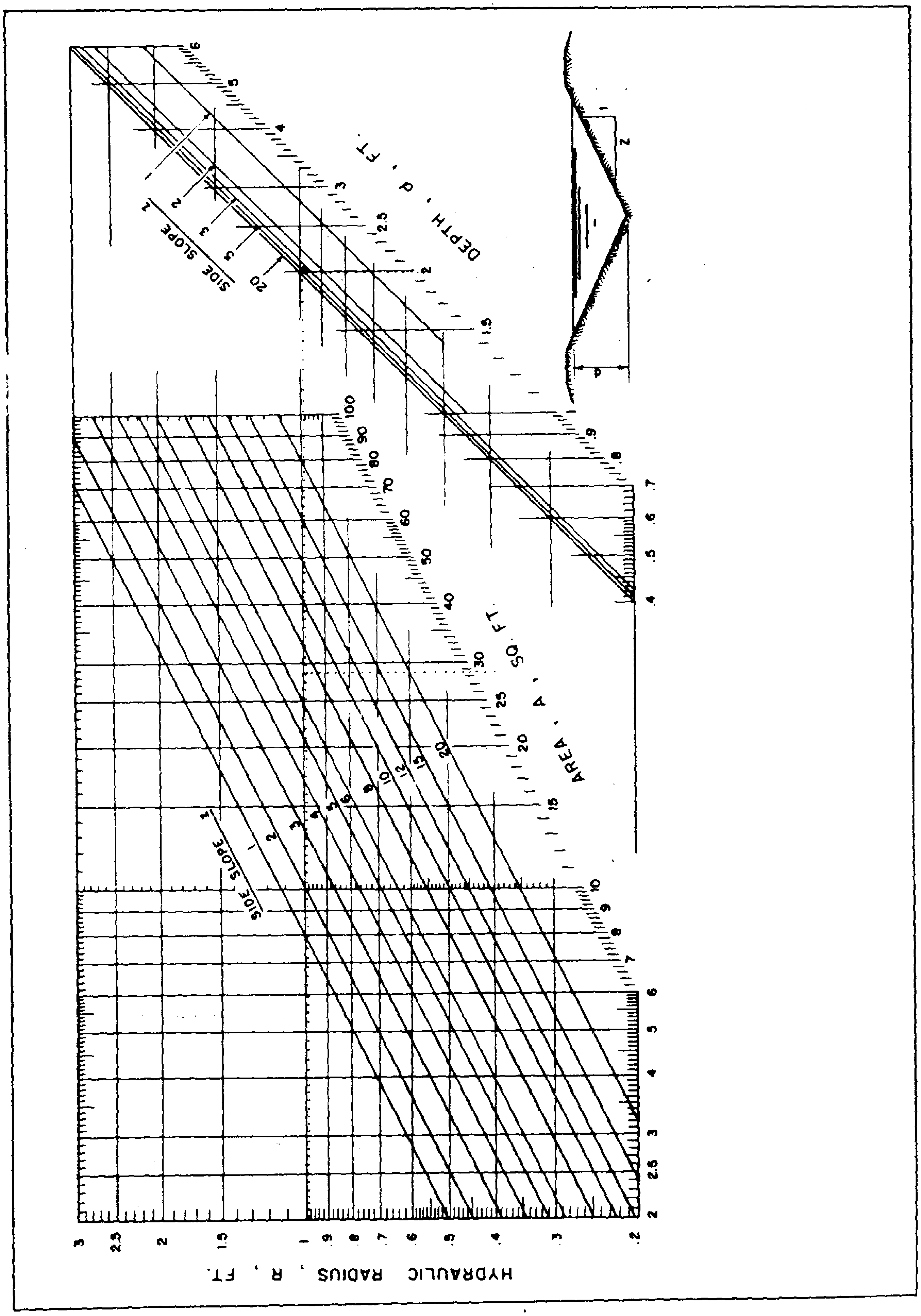


FIGURE 17. Dimensions of triangular channels.

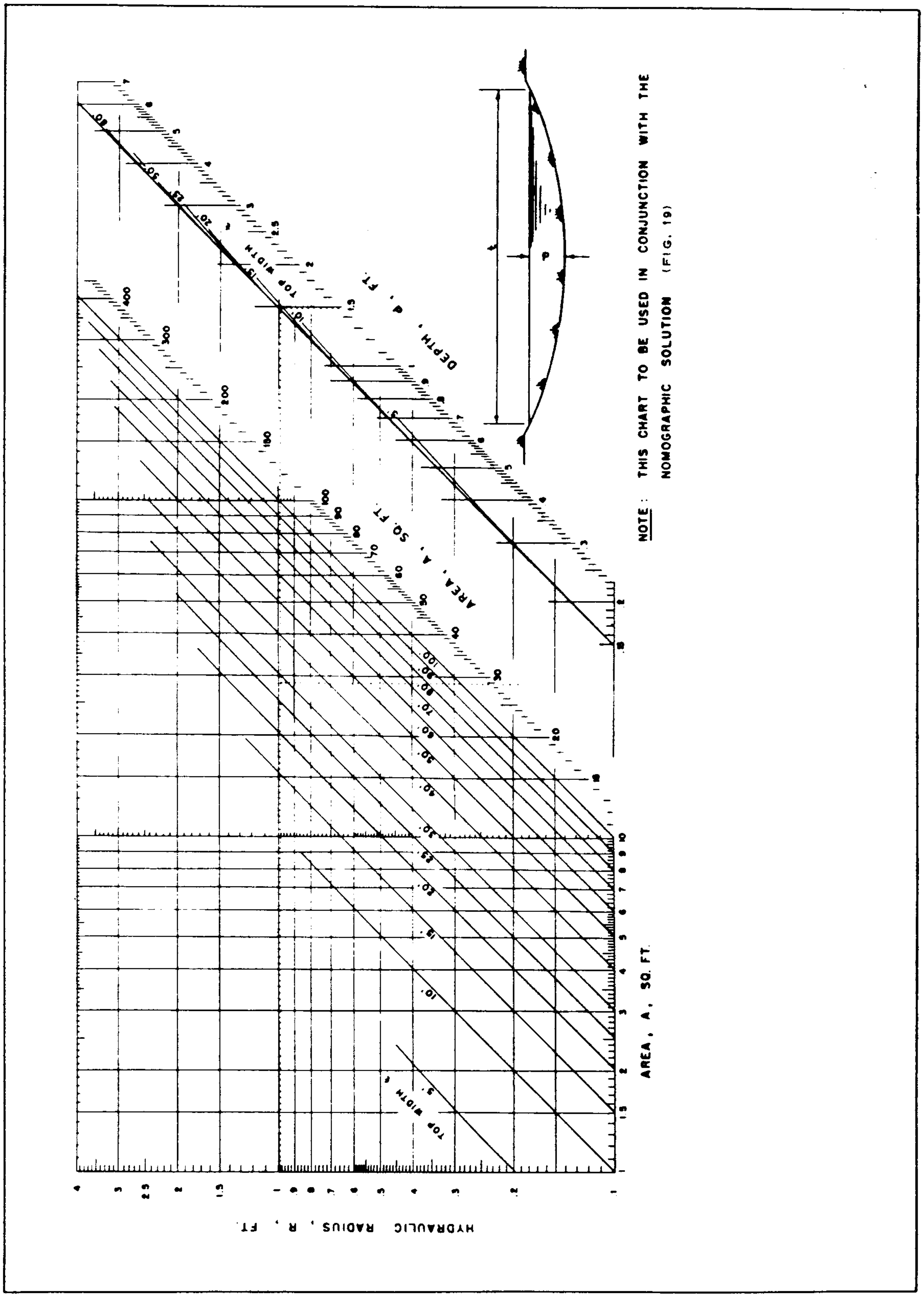


FIGURE 18. Dimensions of parabolic channels.

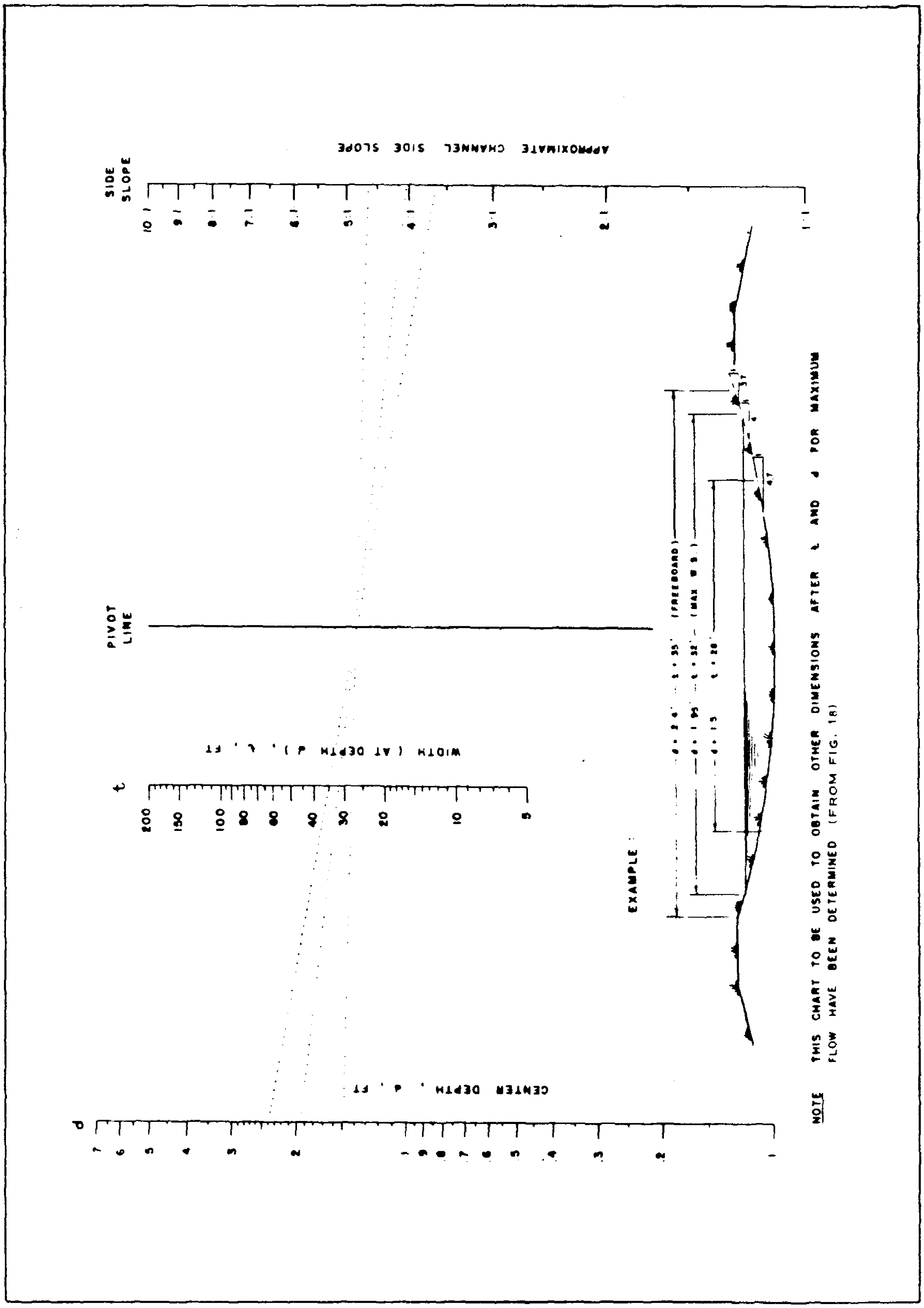
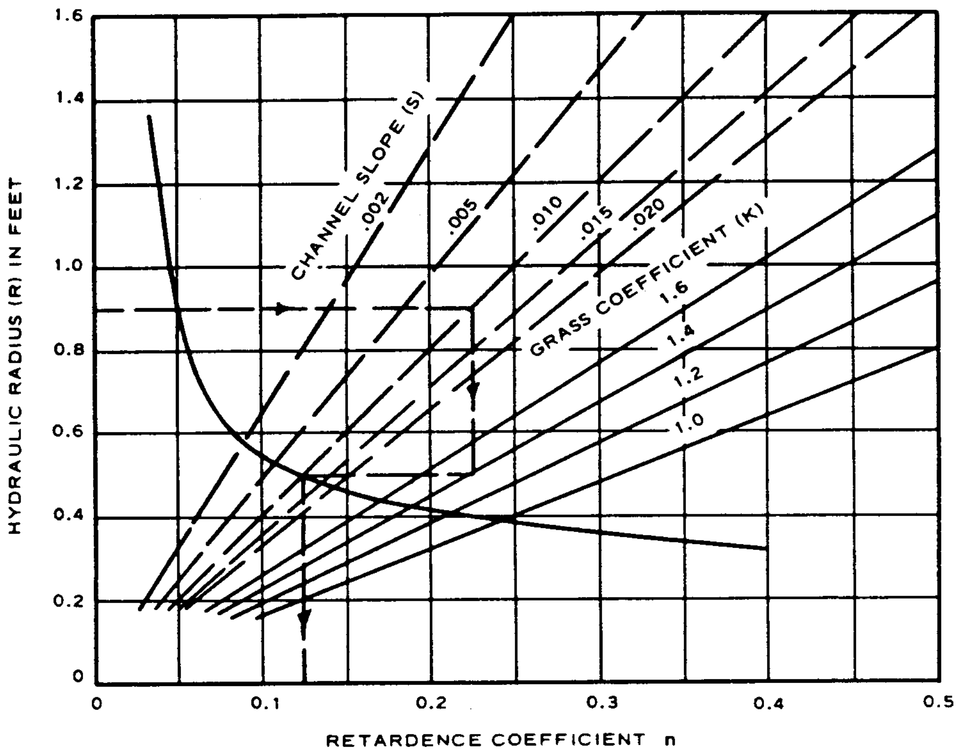


FIGURE 19. Solution for dimensions of parabolic channels.



GRASS COEFFICIENTS (K) FOR DENSE AIRFIELD TURF

GRASS SPECIES	AVG LENGTH OF GRASS IN INCHES		
	<6	6-12	>12
BUFFALO	1.6	--	--
BLUE GRAMMA	1.5	1.4	1.3
BLUE GRASS	1.4	1.3	1.2
BERMUDA	1.4	1.3	1.2
LESPEDEZA SERICEA	1.3	1.2	1.1

EXAMPLE:

DETERMINE n FOR 4-INCH BERMUDA GRASS CHANNEL WITH R = 0.9 AND S = .010.

FROM TABLE K = 1.4 AND FROM GRAPH, FOLLOWING DASHED LINE, n IS EQUAL TO 0.125.

FIGURE 20. Retardence coefficients for flow in turfed channels.

Solution: The area required is Q/V or 20 sq. ft. Enter Figure 15 with A = and b = 10, and find that R = 1.08 feet. Then in the nomograph, Figure 13, extend a line from R = 1.08 through n = 0.04 to the pivot line, extend a line through the velocity scale at 2.5 to the slope line. The slope required is found to be 0.004 feet per foot. Depth of flow from Figure 15 is found to be 1.45 feet.

(3) For example, given an existing channel with side slopes of 3:1, bottom width 12 feet, depth 3.5 feet, bed slope 0.0008 feet per foot. Channel has not been well maintained and some willows and other vegetal growth exists, therefore, estimate "n" to be 0.06.

Find: Present capacity allowing a 0.5-foot freeboard.

Solution: Depth of flow will be 3.5–0.5 or 3.0 feet. From Figure 15, R = 2.05 feet and A = 63 square feet. From Figure 13, V = 1.15 feet per second. Then Q, as channel capacity, = 63 × 1.15 = 72.45 c.f.s.

g. When the channel is to be lined with vegetation, the design problem is complicated by a vegetal retardence element, which is a function of both the turf characteristics and the depth and velocity of flow. This retardal element or coefficient of roughness varies with VR, the product of velocity and hydraulic radius. The degree of vegetal retardence depends on the height and density of cover, particularly the height.

Most drainage references include 4 or 5 degrees of retardence, which are related to type and height of grass in the channel. As shown by Figure 20, having a known hydraulic radius, slope of channel, and selecting a grass coefficient, one can obtain a retardence or roughness coefficient.

10. STRUCTURES.

a. The structures usually built in connection with airport drainage are quite similar to those used in municipal construction. Generally speaking, the standard types are adequate, but occasionally a special type of structure will be needed. Structures located in the usable areas on airports should be so designed that they do not extend above the ground level. The tops of such structures should be one or two-tenths of a foot below the ground line to allow for possible settlement around the structure, to permit unobstructed use of the area by equipment, and to facilitate entrance of surface water.

b. The structures most generally used are inlets, manholes, combination manholes and inlets, catch basins, lampholes, and headwalls. Some of these structures will be covered with a grate when it is necessary to admit the surface water into the system. The grates may be of cast iron, steel, or ductile iron. Several suggested designs of grates and inlets are shown in Figures 21 and 22. For suggested headwall details, see Figure 23. Precast and manufactured headwalls are available.

c. The general designs of drainage structures used by the municipalities are quite alike; however, almost every large city has its own special standards which vary in details according to the desires and ideas of the design engineer. These structures all vary as to the design load they will support and should be thoroughly checked for load-carrying capacities.

d. In aircraft traffic areas, grates and frames should usually be specified to support loads from aircraft which will use the facility as these loads normally exceed those imposed by maintenance equipment or other traffic. Although roadway type grates used by the municipality or highway department will certainly be adequate for all single gear and some dual gear aircraft, it is apparent that most dual and dual tandem gear aircraft weigh far more than highway vehicles and have higher tire pressures. Representative takeoff weights and tire pressure are:

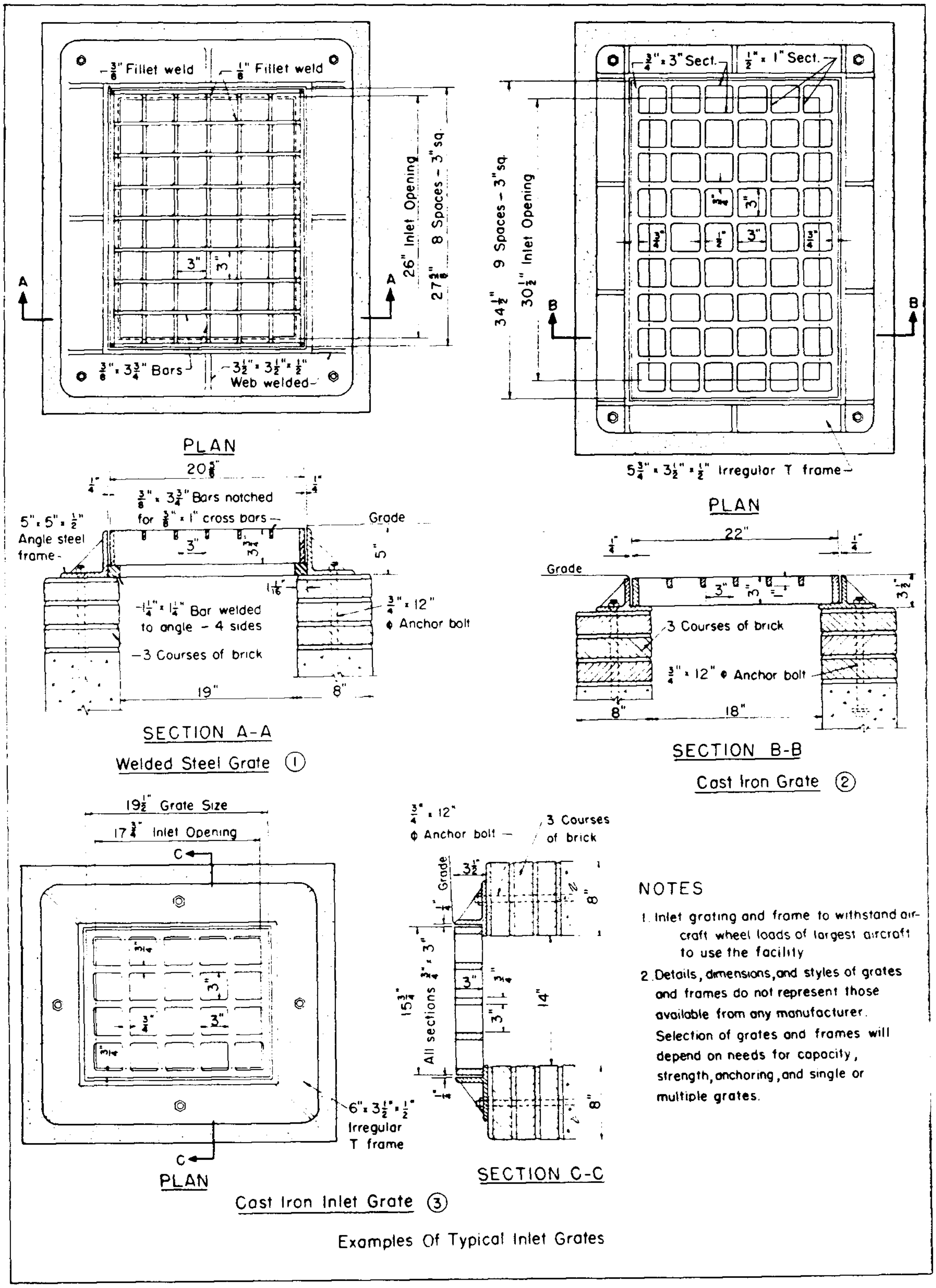


FIGURE 21. Examples of typical inlet grates.

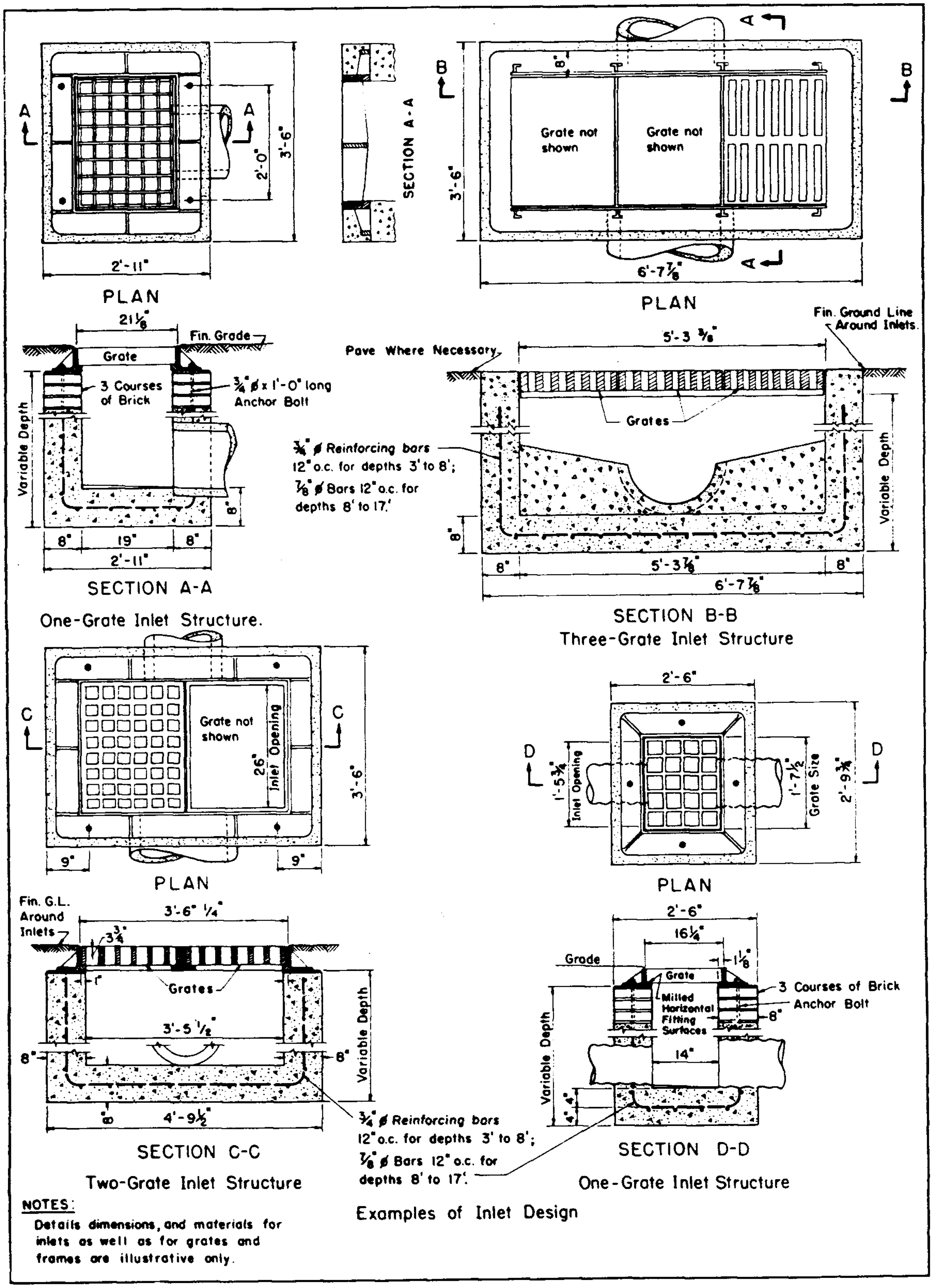
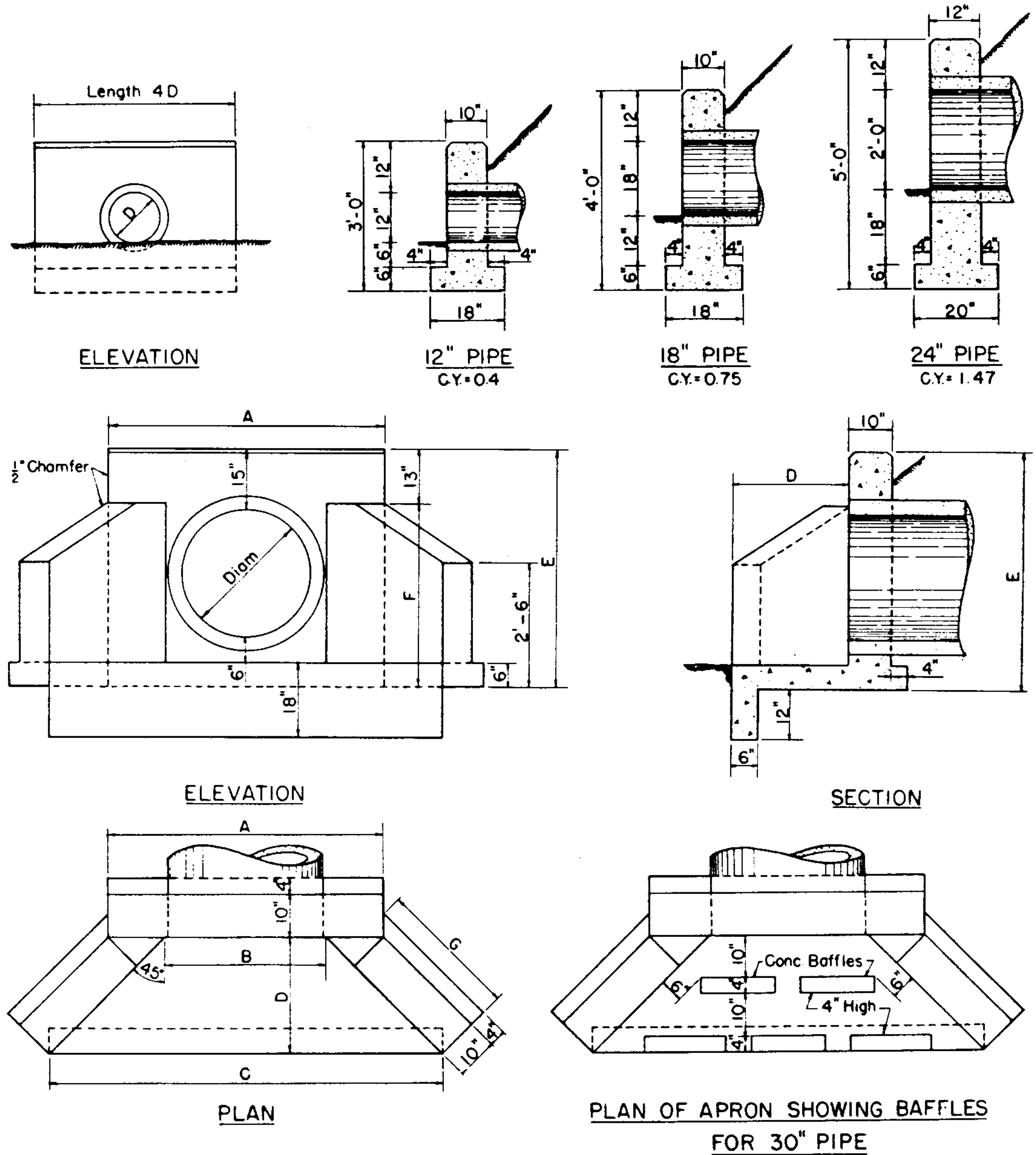


FIGURE 22. Examples of inlet design.



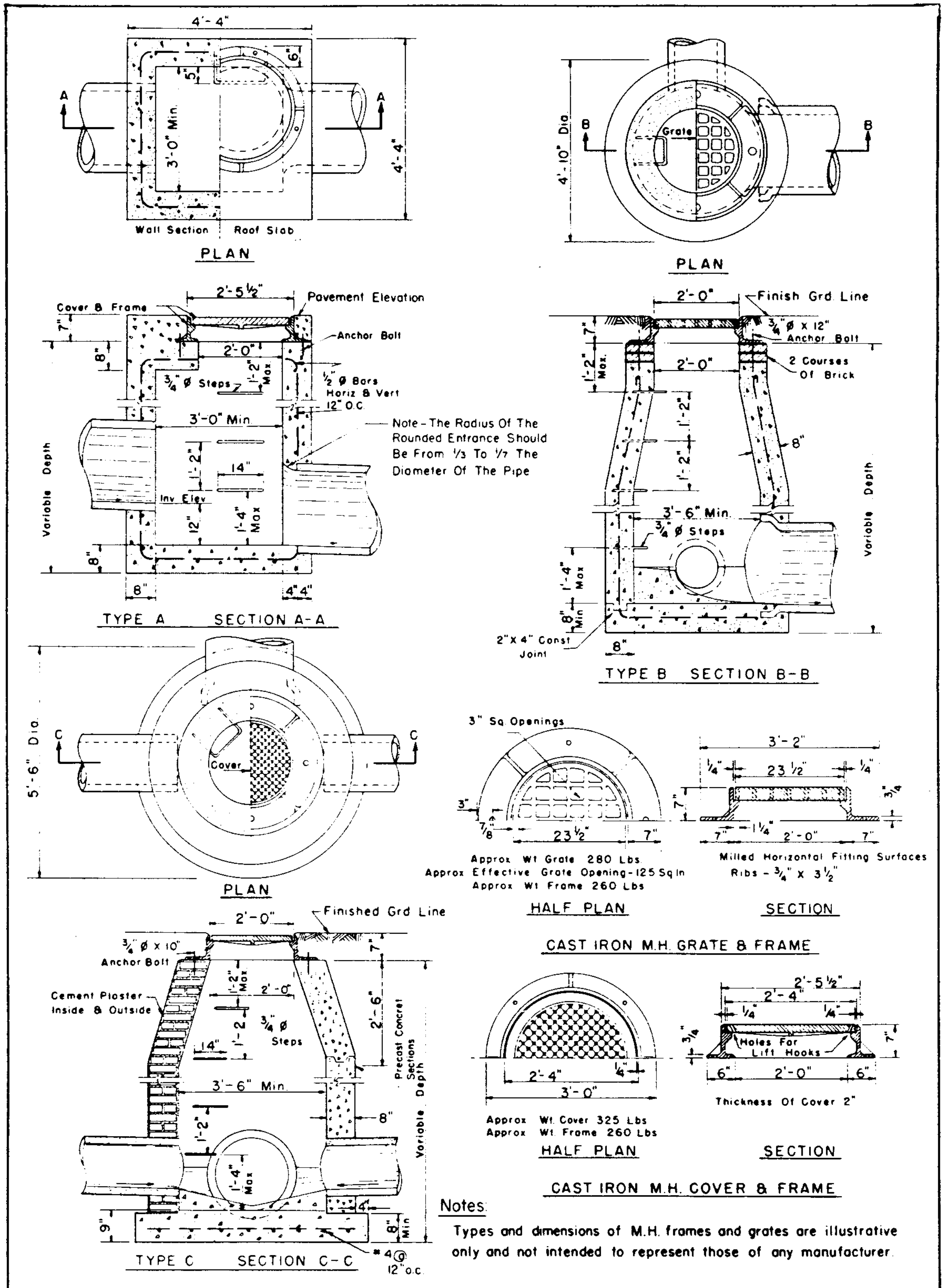
Diam. of Pipe	C.Y Conc. Headwall	A	B	C	D	E	F	G
30"	1.67	5'-6"	3'-2"	7'-10"	2'-4"	4'-9"	3'-8"	2'-5 3/4"
36"	2.24	6'-0"	3'-8"	9'-10"	3'-1"	5'-3"	4'-2"	3'-6 1/2"
42"	2.88	7'-0"	4'-2"	11'-10"	3'-10"	5'-9"	4'-8"	4'-5 1/2"
48"	3.59	7'-6"	4'-8"	13'-10"	4'-7"	6'-3"	5'-2"	5'-6 3/4"

NOTES

1. Reinforcing material may be installed in headwalls whenever necessary.
2. Baffles may be installed in headwall aprons to break up excess velocity of water.

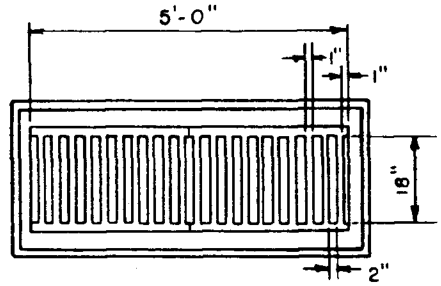
Suggested Types of Headwalls

FIGURE 23. Suggested types of headwalls.



Suggested Manhole Design

FIGURE 24. Suggested manhole design.



TYPICAL PLAN OF DOUBLE INLET GRATING

WATERWAY OPENING = 5.0 SQ. FT. (DOUBLE GRATING)

ASSUME GRATING IS PLACED SO THAT FLOW WILL OCCUR FROM ALL SIDES OF INLET. FOR LOW HEADS DISCHARGE WILL CONFORM WITH GENERAL WEIR EQUATION.

$Q = CLH^{3/2}$

WHERE

C = 3.0

L = 13.0 FT. GROSS PERIMETER OF GRATE OPENING (OMITTING BARS) FOR GRATE ILLUSTRATED

H = HEAD IN FEET

FOR HIGH HEADS DISCHARGE WILL CONFORM WITH ORIFICE FORMULA:

$Q = CA\sqrt{2gH}$

WHERE

C = 0.6

A = 5.0 SQ. FT.

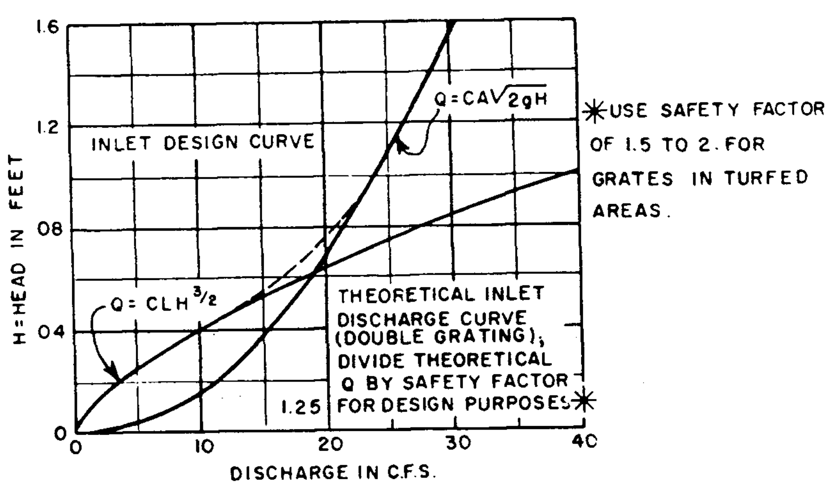
g = ACCELERATION OF GRAVITY IN FEET PER SECOND²

H = HEAD IN FEET

THEORETICAL DISCHARGE RELATION TO BE MODIFIED BY 1.25 SAFETY FACTOR

COEFFICIENTS BASED ON MODEL TEST OF SIMILAR GRATES WITH RATIO:

NET WIDTH OF GRATE OPENING TO GROSS WIDTH = 2:3



DETERMINATION OF TYPICAL INLET GRATING DISCHARGE CURVE

FIGURE 25. Determination of typical inlet grating discharge curve.

Aircraft type	Maximum takeoff weight (lb.)	Tire pressure (p.s.i.)	Gear type
Martin 202, 404; F-27, FH-227; Convairs 240, 340, 440	39,900 to 53,200	60 to 95	Dual
Viscount; BAC-111; DC-4	64,500 to 87,000	75 to 120	Dual
Constellations; DC-9; B-737; DC-7; B-727	90,700 to 170,000	95 to 168	Dual
Caravelle	110,200	124	Dual tandem
Convair 880, 990; B-720, B-707	184,500 to 312,000	120 to 172	Dual tandem
DC-8	273,000 to 355,000	148 to 168	Dual tandem
Concorde	367,000	189	Dual tandem
DC-10; L-1011	411,000 to 518,000	170 to175	DC-10=Dual tandem +2 L-1011=Dual tandem
B-SST	750,000	220	12 Wheel
B-747; L-500	713,000 to 861,500	185 to 210	Dual, dual tandem
Future heavy aircraft	1,500,000	250	(See App. 2 of AC 150/5320-6A)

Grates and frames for Utility and Basic Transport airports may be those used by the municipality or for highway loadings. Those for other airports should be selected to accept aircraft then using or expected to use the airport in the future.

Some manufacturers of grates and frames produce types specifically designed for airport loadings such as 100, 150, 200, and 250 p.s.i. Hold-down bolts, hooks, or other devices should be provided for the grates to prevent displacement by traffic. The engineer should specify the design load to be carried by the grates and frames and require the vendor to certify that the items furnished have that capacity.

e. The inlet structures may be constructed of reinforced concrete, brick, concrete block, precast concrete, or rubble masonry. They should be strong enough to withstand the loads to which they will be subjected.

f. Catch basins for airport drainage are not usually considered necessary particularly when drainage lines are laid on self-cleaning grades. Under certain conditions, catch basins are needed to prevent solids and debris from washing into the system. They should be cleaned out frequently and involve an additional maintenance problem.

g. Manholes are more or less standardized as to type and can be round, oval, square, or rectangular design. They are usually made of reinforced concrete, brick, concrete block, precast concrete, corrugated metal, or precast pipe sections (Figure 24). The design will depend on the stresses to which they will be subjected. Adequate unobstructed space must be provided within the manhole to enable workmen to clean out the line when necessary. Inside barrel dimensions equivalent to a diameter of 3-½ feet and a height of 4 feet are usually considered sufficient, but they can be varied to suit particular situations.

h. A gutter is not permissible along a runway or a taxiway as it could be hazardous to aircraft operations and would interrupt the runoff which should flow unobstructed transversely off the pavement and across the safety area to the field inlets. In wide apron areas, the inlets should be placed in the valley of the pavement at proper intervals to collect runoff.

i. Inlets in paved areas frequently cause trouble due to differential settlement of the drainage structure and the adjacent pavement. The resulting depressions around the inlets may result in pavement failure through softening of the subgrade. Particular care in compaction of backfill around inlets will prevent settlement. In rigid pavement, the structure is normally protected by expansion joints placed around the inlet frame. Also, construction joints are installed at a distance from the structure which either matches or is equal to the normal spacing for joints. The slab thus formed around the structure should include steel reinforcement to prevent cracking outward from each corner of the inlet.

j. The number and capacity of grates selected will be dictated by the quantity of runoff to be handled as well as by the depth of head at the grate. In low head situations, capacity (discharge) of the grate(s) will conform to the general weir formula, while in higher head situations the orifice formula is applicable. These formulas and the transition between them are described in Figure 25.

Medium or high head ponding will be unacceptable to personnel servicing aircraft with baggage, fuel, food, etc. Moreover, such ponding could obscure pavement markings and thus inhibit parking of aircraft at gate positions. In some cases, flooding of refueling pits would occur. Although inlets should be located beyond gate positions, controlling elevations and apron configuration may result in location of inlets fairly close to the field side of the airplane. For these reasons, the weir formula and low head is appropriate for use at some locations.

In general, aprons serving instrument or all weather air carrier operations warrant a head limitation of 0.4 feet. Also, airports where continuing operations are regularly conducted in severe weather conditions would warrant use of a similar limitation. This presumes that apron widths and configurations at other locations will allow runoff to flow directly to turfed areas or that the comparatively light traffic in severe weather conditions could adapt to deeper ponding. It is recommended that apron areas drain away from buildings, whenever possible.

The orifice formula portion of Figure 25 is applicable to grates in turfed areas, except for the unique case where ponding must be severely limited, such as small turfed areas between aprons and closeby taxiways. The safety factor of 1.25 mentioned in the diagram in Figure 25 is intended for use in paved areas. That factor should be increased to 1.5 to 2.0 for grates in turfed areas to compensate for grass cuttings. Replacement of drainage lines or structures in or under pavement would be expensive and would disrupt traffic, therefore, the drainage system should be designed with a capacity sufficient for the ultimate pavement configuration. Terminal apron drainage design should be coordinated with those responsible for the fueling methods and systems to be employed, as well as with those responsible for fire and rescue services.

In large paved areas, such as aprons, sometimes it is feasible to design several segments of the system without ridges between adjacent inlets, thus avoiding grade changes and attendant inhibitions to traffic. In these cases, the efficiency of the grate and its placement should be considered. For such efficiency, all rectangular bars should be parallel with the flow and the openings should cover at least 50 percent of the width of the grate. Multiple grates should be placed normal to the direction of the flow.

11. CULVERTS.

A drainage culvert is designed as a structure (other than a bridge) to convey water through or under a roadway, runway, taxiway, or other obstruction. The choice of circular, oval, elliptical, arch, or box cross section and single or multiple installation will depend on capacity, headroom, economy and occasionally be dictated by local rules or requirements.

The development of a new airport site, or the interruption of natural stream channels by airport facilities, or the increase in runoff caused by airport development may require the design of culverts or consideration of the capacity of existing culverts. In some cases, the airport storm sewer system will need to accept off-site runoff and culverts under roads on the airport perimeter could indicate the volume of runoff to be accommodated.

Local highway departments or drainage districts normally have jurisdiction over design and construction of culverts and channels. Indeed, such governmental units may insist on use of certain design criteria. Accordingly, proposed modification or improvement of drainage facilities under roadways or off-site should be reviewed and approved by the responsible government agency.

The objective in the design of an economical culvert is to provide a waterway opening adequate for the passage of floods, and selection of a culvert meeting that objective is based on the culvert's hydraulic capacity and site conditions.

Many formulas have been developed over the years to estimate runoff/flood discharge and to determine the size of a culvert. The magnitude of the design flood is a function of its frequency, therefore, it is important to use the appropriate frequency.

Because of the potential risk to adjacent property and to automobile traffic, the design flood frequency used for culverts under roads is for a greater recurrence interval than that used for airport storm sewer systems. The local highway department or drainage district may establish the flood frequency to be used: common practice is to design culverts for minor roads for 10- to 25-year flood frequency and for major highways for a recurrence interval of 50 years or more.

The rate of discharge or flow through a culvert involves either of two major types of culvert flow; (1) flow with inlet control or (2) flow with outlet control. Under inlet control, the cross sectional area of the culvert, the inlet geometry, and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness, and length of the culvert barrel.

Field inspection and evaluation of downstream controls or obstructions would indicate the potential depth of tailwater and thus indicate when outlet control will be applicable. Usually channels are wide as compared to the culvert and the depth of water in the channel is considerably less than critical depth and thus inlet control is applicable.

It has been demonstrated that the capacity of a culvert in inlet control can be increased by providing a rounded, bevelled, or tapered entrance. In other words, the capacity of a thin edge projecting metal pipe can be enhanced by the addition of an attachment or the building of a rounded, bevelled, or tapered entrance into a headwall. Similarly, the capacity of other types of pipe or box culverts can be increased at little cost by incorporating a bevel into the headwall.

The procedure for selection of the culvert size and type for most design conditions is well covered in several publications. These publications include charts or nomographs allowing comparison of the capacities of various types and sizes of culverts, therefore, the most economical choice will be apparent.

Current issues of several, widely used publications are listed in the bibliography as references 9, 14, 15, 17, 18, 21, and 24.

Obviously, the foregoing is not intended to be a complete treatment of culvert design, but is included to emphasize some of the fundamentals and to recognize that the airport designer may need to consider and include culverts in the total design.