Appendix E1: Rules Relating to Storm Drainage Standards, 2012

RULES RELATING TO STORM DRAINAGE STANDARDS

JANUARY 2000

Department of Planning and Permitting
City and County of Honolulu
Honolulu, Hawaii

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Department of Planning and Permitting
City and County of Honolulu

RULES RELATING TO

STORM DRAINAGE STANDARDS

Adopted October 4, 1999
Effective January 1, 2000

Amended Section 1-4 and
Plates 1, 2, and 6
November 27, 2010
Effective May 1, 2011

Amended Page ii,
Sections 1-1, 1-2, 1-3 and
Section 1-5
December 12, 2012
Effective June 1, 2013

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DEPARTMENT OF PLANNING AND PERMITTING
CITY AND COUNTY OF HONOLULU

RULES RELATING TO STORM DRAINAGE STANDARDS

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RULES OF THE DEPARTMENT OF PLANNING AND PERMITTING RELATING TO STORM DRAINAGE STANDARDS

§1-1 PURPOSE

These Rules address requirements for both storm runoff quantities for flood control as well as storm runoff quality and reflect the [most] recent changes to Federal, State, and County requirements related to the quality of storm water discharges. By establishing criteria to address water quality, the City and County of Honolulu continues its efforts in complying with Federal Regulatory requirements to control the discharge of pollutants in storm water as specified in the Clean Water Act as amended by the Water Quality Act of 1987.

These standards are not intended to limit the initiative and resourcefulness [of the engineer] in developing drainage plans, or be viewed as maximum limits in design criteria. More stringent criteria should be used where reasonable.

[Eff: June 1, 2013] (*Auth: Sec 14-12.31, ROH) (Imp: Sec14-12.31, ROH)*

§1-2 MODIFICATIONS

A. The Director may modify provisions of these rules whenever:

   1. Full conformance to these rules is not achievable because of the size and shape, location or geological or topographical conditions, or land uses.

   2. The project provides for adequate storm water controls to mitigate adverse downstream impacts related to runoff flows and water quality; complies with Subdivision Rules and Regulations and the Land Use Ordinance; and covenants or other legal provisions are provided as needed, to ensure continued conformity to and achievement of mitigation measures; and

   3. The modification is reasonably necessary and not contrary to the intent and purpose of these rules.

B. Modification requests must be in writing and substantiated by facts presented with the request.

C. Before granting any modification, the Director may consult with the Departments of Design and Construction, Environmental Services, Facilities Maintenance, Parks and Recreation, Transportation Services, Board of Water Supply or any other appropriate agency for review and recommendation.

[Eff: June 1, 2013] (Auth: Sec 14-12.31, ROH) (Imp: Sec14-12.31, ROH)

§1-3 DEFINITIONS

As used in these Rules, the following definitions shall apply unless the context indicates otherwise:

"Best Management Practices" or "BMPs" means pollution control measures, applied to nonpoint sources, on-site or off-site, to control erosion and the transport of sediments and other pollutants, which have an adverse impact on waters of the state. BMPs may include a schedule of activities, the prohibition of practices, maintenance procedures, treatment requirements, operating procedures, and practices to control site runoff, spillage or leaks, or drainage from raw material storage.

"Biofiltration" means the simultaneous process of filtration, adsorption and biological uptake of pollutants in stormwater that takes place when shallow-depth runoff flows slowly over and through vegetated areas.

"City" means the City and County of Honolulu.

"Department" means the Department of Planning and Permitting, City and County of Honolulu.

"Department of Health" or "DOH" means the Clean Water Branch, Department of Health, State of Hawaii, the water pollution regulatory agency of the state.

"Design Engineer" means a licensed civil engineer in the State of Hawaii.

"Development" means land which is being developed or developed lands.

"Director" means the Director of the Department of Planning and Permitting.

"Disturbed Area" means the area of the project that is expected to undergo any disturbance, including, but not limited to excavation, grading, clearing, demolition, uprooting of vegetation, equipment staging, and storage areas. Areas which are cleared, graded, and/or excavated for the sole purpose of landscape renovation or growing crops are not included in the disturbed area quantity. This exemption does not extend to the construction of buildings and roads of agriculture-related operations that disturb one (1) acre or more.

"Engineering Control Facility" means any drainage device such as a basin, well, pond, ditch, dam, or excavation used for the temporary or permanent storage of storm water by means of detention, retention, divergence, or infiltration for the purpose of reducing storm water volume and/or peak storm discharge flows, and which may provide gravity settling of particulate pollutants. It includes, but is not limited to, detention ponds, retention ponds, infiltration wells or ditches, holding tanks, diversion ditches or swales, drainpipes, check dams, and debris basins.

"EPA" means United States Environmental Protection Agency.

"Evapotranspiration" means the combined loss of water into the atmosphere by evaporation (water changing from a liquid to a vapor from soil, water, or plant surfaces) and transpiration (water that is taken up by plant roots and transpired through plant tissue and leaves).

"Flood" or "flooding" means the inundation to a depth of three inches or more of any property not ordinarily covered by water. The terms do not apply to inundation caused by tsunami wave action.

“Impervious Surface” means a surface covering or pavement of a developed parcel of land that prevents the land’s natural ability to absorb and infiltrate rainfall/storm water.

“Infiltration” means the downward migration of surface water (i.e., runoff) through the planting soil (if present) and into the surrounding *in situ* soils and ultimately into groundwater.

“Low Impact Development, or LID” means a storm water management strategy that seeks to maintain or restore the natural hydrologic character of the site, reduce off-site runoff, improve water quality, provide groundwater recharge, and mitigate the impacts of increased runoff and storm water pollution. LID comprises a set of site design approaches and integrated management techniques that promote the use of natural systems for infiltration, evapotranspiration, treatment, and use of rainwater.

“Maximum Extent Practicable” or "MEP" means economically achievable measures for the control of the addition of pollutants from existing and new categories of nonpoint sources of pollution, which reflect the greatest degree of pollutant reduction achievable through the application of the best available nonpoint source pollution control practices, technologies, processes, siting criteria, operating methods or other alternatives.

“National Pollutant Discharge Elimination System permit” or “NPDES permit” means the permit issued to the City pursuant to *Title 40, Code of Federal Regulations, Part 122,* Subpart B, Section 122.26(a) (1) (iii), for storm water discharge from the City’s separate storm sewer systems; or the permit issued to a person or property owner for a storm water discharge associated with industrial activity pursuant to *Title 40, Code of Federal Regulations, Part 122*, Subpart B, Section 122.26(a) (1) (ii), or other applicable section of Part 122; or the permit issued to a person or property owner for the discharge of any pollutant from a point source into the state waters through the City's separate storm sewer system pursuant to *Hawaii Administrative Rules*, Chapter 11-55, "Water Pollution Control".

“New Development” means land disturbing activities; structural development, including construction or installation of a building or structure, the creation of impervious surfaces; and land subdivision.

“Redevelopment” means development that would create or add impervious surface area on an already developed site.

“Site Design Strategies” means LID design techniques that are intended to maintain or restore the site’s hydrologic and hydraulic functions with the intent of minimizing runoff volume and preserving existing flow paths.

“Source Control BMPs” means low-technology practices designed to prevent pollutants from contacting storm water runoff or to prevent discharge of contaminated runoff to the storm drainage system.

"Storm water" means storm water runoff, surface runoff, street wash, or drainage and may include discharges from fire fighting activities.

“Treatment Control BMPs” means engineered technologies designed to remove pollutants from storm water runoff prior to discharge to the storm drain system or receiving waters.

[Eff: June 1, 2013] (Auth: Sec 14-12.31, ROH) (Imp: Sec14-12.31, ROH)

§1-4 SECTION I - STANDARDS FOR FLOOD CONTROL

Standards and regulations for flood control are adopted to protect life and property during intense storms. Small storms that occur frequently usually do not cause significant property damages or loss of life, therefore, peak runoff from large storms are regulated for flood control.

The data from *85* U.S. Geological Survey (USGS) *stream flow gauges on the Island of Oahu* form the basis for Plate 6, "Design Curves for Peak Discharge vs. Drainage Area". The rainfall data on Plates 1 and 2 are from the *National Oceanic and Atmospheric Administration (NOAA), National Weather Service, Silver Spring, Maryland, 2009*. Rainfall data on Plates 1, 2 and 6 will be updated periodically and such updates will automatically be incorporated into these rules when the updates are adopted by the Department. *[Eff:* APR 0 8 2011 *] (Auth: Sec 14-12.31,ROH) (Imp: Sec 14-12.31.ROH)*

§1-4.1 PART I - HYDROLOGIC CRITERIA

A. RECURRENCE INTERVAL

1. For drainage areas of 100 acres or less, Tm (recurrence interval) = 10 years, unless otherwise specified.

2. For drainage areas of 100 acres or less with sump, or tailwater effect and for the design of roadway culverts and bridges, Tm (recurrence interval) = 50 years.

3. For drainage areas greater than 100 acres and all streams, design curves based upon the U.S. Geological Survey data on flood magnitude and frequency, Tm (recurrence interval) = 100 years.

4. Interim measures for areas where downstream facilities are inadequate shall be reviewed on a case-by-case basis.

B. RUNOFF QUANTITY

1. For drainage areas of 100 acres or less, the rational method shall be used.

2. For drainage areas greater than 100 acres:

   a. Plate 6 titled, "Design Curves for Peak Discharge vs. Drainage Area" should be used to determine the 100-year peak discharge.

   b. Modifications from the Plate 6 peak discharge values may be used if the Design Engineer can justify more acceptable values and it is approved by the Director.

   [Eff: APR 0 8 2011 *] (Auth: Sec 14-12.31,ROH) (Imp: Sec 14-12.31.ROH)*

3. For drainage areas where downstream capacities are inadequate to accommodate runoff quantity identified above, runoff shall be limited to pre-development conditions or as specified in the General Conditions.

C. RATIONAL METHOD

The formula $Q = CIA$ shall be used to determine quantities of flow rate, in which

- Q = flow rate in cubic feet per second;
- C = runoff coefficient;
- I = rainfall intensity in inches per hour for a duration equal to the time of concentration; and
- A = drainage area in acres.

1. RUNOFF COEFFICIENT

The runoff coefficient shall be determined from Table 1 for agricultural and open areas and from Table 2 for built-up areas. It shall be based on the ultimate use of the project drainage area. For distinctive composite drainage areas, a weighted value of runoff coefficient shall be used.

For interim drainage measures, existing upstream land use conditions may be used to size interim measures as long as ultimate drainage requirements can be met when downstream restrictions are removed.

2. TIME OF CONCENTRATION

a. Determine overland flow time from Plate 3 generally for paved, bare soil and grassed areas.

b. Determine flow time over small agricultural areas with well-defined divides and drainage channels from Plate 5.

1) Use upper curve for well-forested areas, representing

$$Tc = 0.0136\ K^{0.77}$$

2) Use lower curve for areas with little or no cover, representing

$$Tc = 0.0078\ K^{0.77}$$

c. In case of uncertainty, check the time of concentration by dividing the estimated longest route of runoff by the appropriate runoff velocity from Table 3.

3. RAINFALL INTENSITY

The design rainfall intensity of a drainage area shall be determined by the following procedure:

a. Select the appropriate 1-hour rainfall value from Plate 1 or Plate 2 for the design recurrence interval.

b. Enter Plate 4 with the rainfall intensity duration equal to the required time of concentration, select the corresponding correction factor, and multiply the 1-hour rainfall value by the factor to obtain the design rainfall intensity.

D. HYDROLOGIC STUDIES

Since 1959, the City and County of Honolulu and the U.S. Geological Survey have participated in a cooperative program for the collection of special stream flow data. This program included the installation of additional stream gaging stations and crest-stage gages. With the additional hydrologic data supplementing the data from the existing gaging stations, it was anticipated that more would be known of the effects of exposure, altitude, basin slope, basin shape and degree of urbanization on stream runoff on Oahu.

The U.S. Geological Survey developed flood-frequency curves for 74 gaging stations on Oahu by using the log Pearson Type III distribution (U.S. Water Resources Council, 1977, Bulletin 17A). The length of record for the individual stations ranged from 10 to 60 years. In order to furnish data at ungaged sites, they attempted to regionalize the available data by the use of multiple-regression techniques. In the study, a regional analysis was made by using these techniques to relate floodflows to basin and climatic characteristics. The results are contained in the *U.S. Geological Survey Water-Resources Investigations 80-45 Report, An Analysis of the Magnitude and Frequency of Floods on Oahu, Hawaii,* dated June 1980. The results were subsequently updated.

The U.S. Geological Survey and City have further extended the data to facilitate the determination of peak discharge values for the design of drainage facilities by developing Plate 6, Design Curves for Peak Discharge vs. Drainage Area. The curves are based upon the 100-year recurrence interval data. For clarification, the boundaries between the groups shown on Plate 6 are as follows:

Group Boundary	**Location**
A and B	Between Oio Stream and Malaekahana Stream and along Koolau Range Ridge.
B and C	Between Honouliuli Stream and Waikele Stream, along Waianae Range Ridge, and between Makaleha Stream and Kaukonahua Stream.

The rainfall data shown on Plates 1 and 2 have been updated from the *Rainfall Frequency Study for Oahu*, Department of Land and Natural Resources, Division of Water and Land Development, State of Hawaii, dated 1984.

§1-4.2 PART II - DESIGN STANDARDS

A. GENERAL CONDITIONS

The design and capacity of a drainage system shall be predicated on the following conditions:

1. On the basis of the runoff resulting from the selected design storm, the system shall dispose of surface runoff and subsurface water without damage to street facilities, structures or ground and cause no serious interruption of normal vehicular traffic.

2. Runoff exceeding the design storm must be disposed of with the least amount of interruption to normal traffic and minimum amount of damage to surrounding property.

3. System must have maximum reliability of operation with minimum maintenance and upkeep requirements.

4. System must be adaptable to future expansion, if necessary, with minimum additional cost.

5. Where sump conditions exist, a safety measure such as an overflow swale shall be provided to prevent flooding of adjacent lots in the event the design capacity of the closed conduit is exceeded. Floor levels of homes adjoining sumps shall be a minimum of 3 feet above the low point on roadway.

6. Lots abutting streams and open channels may be graded to drain towards the waterway.

7. In general, natural gullies, waterways, streams and tributaries shall not be replaced with a closed system except at roadway crossings.

8. Roadway culverts and bridges shall be designed to pass the design flow under open channel hydraulic analysis with a minimum freeboard as specified in the attached freeboard chart. Multiple span road crossings shall have minimum clear spans of 30 feet, unless otherwise permitted by the Director. Where possible, the roadway shall be designed to form a sag vertical curve with a low point at the waterway crossing with minimum grades to confine and control overflow at the crossing. Whenever the difference in elevations of the roadway and water surface is such that there could be a deep fill, the roadway culvert or bridge shall be designed to include available headroom up to five feet from the water surface to the soffit of the culvert or bridge. After this headroom requirement is fulfilled, fill material may be used to meet roadway elevations.

9. Outlets for enclosed drains emptying into open channels shall be designed to point downstream at an angle of 45 degrees.

10. Where groundwater is encountered, or may be present during wet weather, subsurface drains shall be installed wherever recommended by the Design Engineer, or the Director.

11. New developments shall provide adequate drainage capacity to accommodate the offsite design storm entering the development site.

12. When downstream drainage systems cannot accommodate peak runoff rates from design storms, runoff rates discharged downstream from new developments will be limited to predevelopment values unless improvements to the downstream system are made.

13. Runoff volume from the design storm shall be limited to predevelopment values unless it can be shown that the runoff can be safely conveyed through existing or planned conveyances, the increased volume would not have adverse impacts downstream, and provided further that the final receiving waters are open coastal waters.

B. DESIGN COMPUTATIONS

The following data shall be submitted to the Director by the Design Engineer.

1. HYDRAULIC DESIGN DATA

a. Computations for runoff, conduit and channel sizes, slopes, losses, hydraulic gradient and other hydraulic characteristics and information pertinent to the system. Computations shall be properly arranged and presented in such a manner that they may be readily checked.

b. The following data shall be shown on the construction plans.

1) Design flow (Q), watershed area (A), roughness coefficient (n), and velocity (v), for all conduits and channels.

2) Hydraulic grade lines, including water surface elevation at each manhole and catch basin.

3) Building setback lines, where required.

4) Floodway/flood fringe boundary, as applicable.

c. When interim drainage measures are required due to restrictions in the downstream drainage systems, the following additional data shall also be provided:

1) Runoff rate using the design storm for existing upstream land use conditions.

2) Runoff volume using the design storm for existing upstream land use conditions.

3) Detention volume and discharge rate.

4) If necessary, capacity of downstream drainage systems.

2. STRUCTURAL DESIGN DATA

a. Structural design computations for all drainage structures other than pipes used within the limits of current loading tables and structures shown in the *"Standard Details for Public Works Construction"* for the City and County of Honolulu.

b. Information pertinent to the design, such as boring data, soils report, etc.

c. Upon the completion of construction of major structures, submit pertinent data such as pile driving logs, pile tip elevations, etc.

C. CLOSED CONDUITS

1. SIZES AND GRADIENTS

a. The size and gradient will be determined by the Manning Formula:

$$Q = A \frac{1.486}{n} R^{2/3} S^{1/2}$$

Q = flow, in cfs
A = area, in sq. ft.
R = hydraulic radius in ft.
S = slope, in ft./ft.
n = roughness coefficient (Manning)

Charts enabling direct solution of Manning formula are found on Plates 8 to 16.

b. The following limitations apply -

1) Minimum size pipe: 18 inches inside diameter

2) Minimum velocity: 2-1/2 feet per second

3). Pipe sizes should not decrease in the direction of the flow.

2. MATERIALS AND "n" VALUES

The following pipes are acceptable for storm drain construction together with the roughness coefficient to be used in the solution of the Manning Formula.

Materials	n
Concrete	0.013
Cast Iron	0.013
Corrugated metal pipe (CMP) *	
Unpaved	0.024
25% paved invert	0.021
Lower 50% paved	0.018
100% paved	0.013
High Density Polyethylene (HDPE) *	0.015

*Use of CMP or HDPE shall be permitted only when specifically approved for an installation by the Director in writing.

3. LOADING

a. Reinforced Concrete Pipes: Reinforced concrete pipes shall be constructed to American Society for Testing and Materials (ASTM) Specifications.

1) Minimum pipe cover in roadways, driveways and other areas with vehicular traffic shall be two feet.

Should there be a need for a pipe cover of less than 2 feet or should the design or construction method deviate from the Standards of the Department of Planning and Permitting, City and County of Honolulu, the Design Engineer shall submit a structural design for review and approval. The decision to allow such design will be made by the Director.

2) Minimum pipe cover in easement areas without vehicular traffic shall be 1'- 0".

3) Maximum permissible pipe cover will be determined from current loading tables in pipe handbooks for the respective pipes, using 120 lbs. per cu. ft. as the weight of earth.

4) All pipes shall be installed using a first class bedding trench condition. Proper foundations shall be provided for pipes. Pipes on unstable ground or fresh fill shall be supported by a method acceptable to the Director.

5) Drain pipes installed along the longitudinal axis of the roadway shall be located in the pavement area between curbs.

b. Other Closed Conduits. There shall be no minimum cover or maximum permissible depth requirements for closed conduits other than pipes except that such structures, shall be designed to support all loads that it shall be subjected to.

4. MANHOLES AND INLETS

a. Manholes:

1) Location. Manholes shall be located at all changes in pipe size and changes in alignment or grade and at all junction points.

2) Spacing. Maximum manhole spacing shall be 250 feet for pipes 36 inches or less in diameter, or box drains with the smallest dimension less than 36 inches. Maximum manhole spacing for larger pipes and box drains shall be 500 feet.

3) Special Details. Bottoms of manholes and inlets serving as manholes shall be shaped to channelize flow and sloped with slope of pipe as shown in the "Standard Details" of the Department of Planning and Permitting, City and County of Honolulu.

b. Inlets (Catch Basins):

1) Location. Inlets shall be located at the upstream side of intersections, in sumps and where required by quantity of flow.

2) Spacing. Maximum spacing shall be 500 feet.

3) Types. For gutter grades up to 4 percent, standard 10-foot curb inlets with a depressed gutter shall be used. For grades 4 percent and greater, 10-foot long deflector inlets shall be used.

4) Capacity. Inlet capacities as follows, are acceptable:

	Type	Gutter Grade	cfs
a.	Std. depressed	0.4%	6
	gutter inlet	4.0%	4
		sump	10
b.	Deflector inlet	4.0%	4.5
		12.0%	5.5
	Greater than	12.0%	6 max

5) Gutter Flow. The gutter flow shall not exceed a width of 8 feet.

5. PIPE SYSTEM ANALYSIS

Generally speaking, the pipe system shall be analyzed by sections, that is, outlet-to-manhole, manhole-to-manhole or manhole-to-inlet. The analysis shall start at the lowest point of flow and continued upstream. The design flow shall be used in determining whether the pipe will flow full or partially full. Full consideration of the tailwater, entrance and critical flow conditions shall be made.

a. Pipe Flowing Full. If the conditions show that the pipe section will flow full, the principles of flow of water in closed conduits shall be used. The water surface elevation of the upstream manhole is determined by adding the pipe friction and manhole losses to the water surface elevation of the downstream manhole or the beginning elevation as previously stated.

b. Pipe Flowing Partially Full. If the conditions show that the pipe section will flow partially full, the principles of flow water in open channels shall be used. The pipe partially full condition may be determined from the *Pipe Flow Charts* on Plates 8 to 16. The tailwater condition must also be considered in this determination.

c. Manhole Losses.

1. For junction conditions such as drop manholes, or where the outflow line deflects more than 10 degrees with any inflow line, the hydraulic grade shall be determined by applying the "Entrance Control loss" and "C & D losses" (where applicable), or "A, B, C & D losses", whichever is greater.

2. For junction conditions where the outflow line deflects 10 degrees or less with the inflow line, the hydraulic grade shall be determined by applying the "A, B, C & D losses".

6. HYDRAULIC GRADIENT COMPUTATIONS

The hydraulic gradient is: (1) a line connecting points to which water will rise in manholes and inlets throughout the system during the design flow; or (2) the level of flowing water at any point along an open channel.

It shall be determined starting at the downstream end of the proposed drainage system and proceeding upstream by adding the friction losses and manhole losses of the system.

The hydraulic gradient for the design flow shall be at least 1 foot below the top of the manhole cover, or 1 foot below the invert of catch basin inlet opening.

a. Beginning Elevation. The elevation of the hydraulic gradient at the downstream end shall be selected according to the following conditions:

1) Connection to existing drainage system - determined from the hydraulic gradient computations of the existing drain;

2) Discharge into a stream - determined from the flow conditions of the stream;

3) Submerged tailwater condition - begin at the tailwater elevation; and

4) Freefall condition (conduit) - begin at the crown of the proposed drain.

b. Friction Loss.

$h_f = S_f(L)$, where:

h_f = head loss due to friction

S_f = friction slope from Manning Formula

$$= \frac{(nv)^2}{2.208\ R^{4/3}},$$

L = length of pipe or channel

The friction loss shall be calculated for the condition of the design flow, that is, pipe flowing full or partially full.

c. Manhole Losses.

Manhole losses shall be as shown on the charts, *Head Losses in Manholes* (Plates 17 and 18). The losses shall be evaluated with pipes flowing full in the vicinity of the manholes; and therefore the velocity shall be for the pipe flowing full. The curves on the charts show the various losses:

1) A curve - loss due to entrance and exit

2) B curve - velocity head

a. Where the downstream velocity exceeds the upstream velocity, the head loss shall be difference in velocity heads.

b. Where the downstream velocity is less than the upstream velocity, the velocity head loss shall be zero.

3) C curve - loss due to change in direction, taking the worst case for branches at a manhole.

4) D curve - loss due to incoming volume.

7. SPECIAL DETAILS

The following structures shall be installed where required:

a. Headwalls, aprons and cut-off walls at drain inlets and outlets.

b. Energy dissipators at outlets.

c. Debris and boulder control structures.

d. Guard rails or fences on channel walls, headwalls and inlets, where they present a hazard to vehicular traffic or pedestrians.

D. OPEN CHANNELS

1. CHANNEL SIZE

Use the Manning Formula to determine the required waterway areas where uniform flow can be assumed.

$$Q = AV \text{ and } V = \frac{1.486\, R^{2/3}\, S^{1/2}}{n}$$

A= area of flow, in square feet
V= velocity, in feet per second
n = roughness coefficient (Manning)
R= hydraulic radius, in feet
S= slope of the energy gradient, in feet per feet

The channel depth shall include design water depth and minimum freeboard allowances. Design water depth shall include rise in water surface caused by curves and junctions.

2. CHANNEL RIGHT-OF-WAY

The channel width shall be sufficient to provide the required waterway area for the design storm as determined by these standards. The total right-of-way shall include a 15-foot wide maintenance road along both banks where the top width of channel exceeds 50 feet, and along one bank where the top width is 50 feet or less. The maintenance road along the channel shall be topped with 6 inches of Asphalt Treated Basecourse (ATB) or Asphalt Concrete (AC). In lieu of a maintenance road, for normally dry channels, access ramps or other suitable alternative measures to facilitate maintenance may be provided.

3. PERMISSIBLE VELOCITIES AND "n" VALUES

Following is a list of "n" values for open channels and maximum permissible velocities. Maximum velocities shall be based upon design flow quantities.

Unlined Channel	Manning "n"	Maximum Velocity (fps)
Rock	0.030	10
Ledge coral or limestone	0.025	10
Earth with vegetation (grassed)	0.035	5
Lined Channels		
Conc. (trowel finish)	0.013	No limitation
Conc. (smooth wood forms)	0.015	No limitation
Gunite	0.020	20
Grouted Rip-rap & CRM (Cement Rubble Masonry)	0.025	20
Asphaltic Concrete	0.015	20
Corrugated Metal Flumes (Part-circle Sections)	0.021	25

Note: Use of CMP shall be permitted only when specifically approved for an installation by the Director in writing.

a. Maximum design velocity for channels cut in earth shall not exceed 5 feet per second. The velocity shall be determined by using the natural existing slope of the waterway without utilizing grade transition structures to control the maximum slope for a given unlined channel cross-section and design flow.

b. Velocities between 5 feet per second and 10 feet per second will be permitted in materials, such as cemented gravel, hard pan, or mud rock, depending upon its hardness and resistance to scouring. Borings and samples shall be submitted for evaluation before velocities exceeding 5 feet per second will be permitted.

4. CHANNEL LINING

a. Earth channel shall be fully lined when velocities exceed 5 feet per second, unless otherwise permitted as previously noted above in Section D.3.b of §1-4.2 Part II - Design Standards.

b. All fill sections shall be lined. This lining shall be a complete lining including side slopes and invert with appropriate cut-off walls. If the invert of the channel is in a cut section, the invert slab may be omitted and appropriate cut-off walls provided at the toe of the side slope lining.

c. Where linings are required or used, the linings, shall be continuous. Lining of fill sections without continuing the lining out through cut sections in a channel will not be allowed unless adequate provisions are made to reduce the velocity from the lined section to meet the allowable velocity for the unlined section.

d. Total depth of channel lining will include design water depth and freeboard.

e. Attention shall be given to construction details of linings, such as thickness, reinforcement, expansion and construction joints, cut-off walls, watertight joints, and placement of reinforcement, etc. Where the channel discharges into streams or other channels outside of the limits of a development, velocity reducing and transition structures shall be constructed to minimize erosion and overtopping of banks and subsequent flooding of downstream areas.

f. Where velocities are supercritical, rectangular channels shall be used, unless otherwise permitted by the Director.

g. Earth channels shall be planted with vegetation, such as grass of a species not susceptible to rank growth.

5. FREEBOARD

In designing open channels, freeboard must be provided to allow for surface roughness, wave action, air bulking, and splash and spray. These phenomena depend on the energy content of flow. For water flowing at velocity **v** and depth **d,** the energy per foot of width per second is equal to $\mathbf{(wvd)\ (v^2/2g) = wdv^3/2g}$, where **w** is the unit weight of water.

Thus, this kinetic energy can be converted to potential energy to lift the water surface when flow is stopped or changing direction as a function of depth and velocity of flow. The U.S. Bureau of Reclamation has developed an empirical expression to express a reasonable indication of desirable freeboard in terms of depth and velocity as follows:

$$\text{Freeboard in feet} = 2.0 + 0.025\ v\ (d)^{1/3}$$

where **v** is the velocity in feet per second and **d** is the depth of flow in feet. The velocity of flow can be computed by dividing the design discharge by the cross-sectional area of flow. For convenience of application, the above expression is shown graphically in Plate 7.

6. JUNCTIONS

Junctions shall be designed to channel both flows as nearly parallel as possible to reduce velocity and momentum components, deposition of debris and erosion of banks.

7. BENDS AND SUPERELEVATIONS

Changes in the direction of flow shall be made with smoothly curved channel walls allowing for superelevation in water surface. Curves will nearly always require additional depth. Trapezoidal channels for supercritical velocities are not permitted. Curve radii should be sufficiently great to limit superelevation of the water surface to one foot above computed depth of flow or 10 percent of water surface width, whichever is the least. The amount of superelevation for simple curves may be determined as follows:

a. Trapezoidal Channels:

Subcritical velocity:

$$e = \frac{V^2(b + 2zd)}{(gR - 2zV^2)}$$

b. Rectangular Channel:

Subcritical velocity:

$$e = \frac{V^2 b}{gR}$$

Supercritical velocity:

$$e = \frac{2V^2 b}{gR}$$

Supercritical velocity - compound curve:

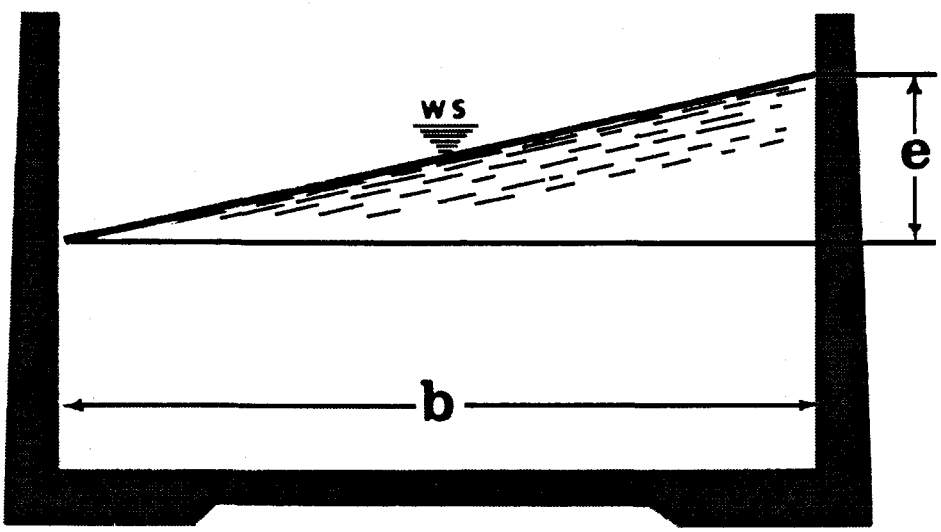
$$e = \frac{V^2 b}{gR}$$

The compound curve is a simple curve of radius R preceded and followed by a section of simple curve with radius of 2R and length of

$$\frac{b}{\tan \beta}, \text{ where } \sin \beta = \frac{(gd_m)^{1/2}}{V}$$

Where,

b	=	channel bottom width (ft)
d	=	normal depth (ft)
d_m	=	mean depth (ft)
e	=	maximum difference in elevation of water surface between channel sides (ft)
g	=	acceleration due to gravity (fps^2)
R	=	radius of curve to centerline (ft)
V	=	normal velocity (fps)
z	=	co-tangent of bank slopes



Water Surface Superelevation Showing, "e"

8. TRANSITIONS

a. The maximum angle between channel centerline and transition walls should be 12.5 degrees.

b. Sharp angles in alignment of transition structures should be avoided.

9. DEBRIS BARRIERS

Debris barriers should be provided upstream of the intake to prevent clogging.

10. DEBRIS BASINS

Where required by the Director, debris basins shall be provided upstream of the debris barrier. Debris basins shall also be provided at the intake of a drainage system when the upstream drainage area is undeveloped.

The volume of debris to be impounded shall be estimated based on the existing upstream land uses.

The basin design shall include an access ramp to the bottom of the basin for maintenance purposes.

11. ENERGY DISSIPATORS

Energy dissipators shall be used to dissipate energy where necessary, and to transition the flow from a lined channel to a normal flow in an unlined channel.

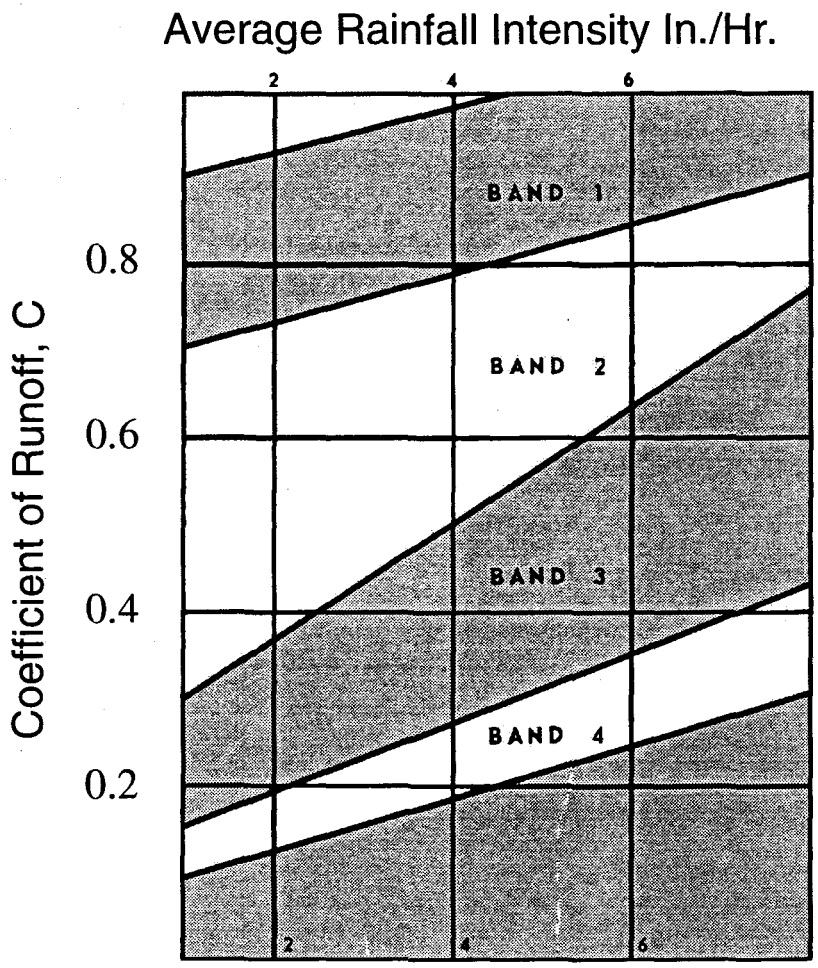
Energy dissipators may be any of the following types, such as the SAF basin, baffled chute, dentated sills, buckets, impact, hydraulic jump, or other approved designs.

[Eff: JAN 01 2000 *] (Auth: Sec 14-12.31, ROH) (Imp: Sec14-12.31, ROH)*

§1-4.3 DESIGN CHARTS

Table 1

RUNOFF COEFFICIENT FOR AGRICULTRUAL AND OPEN AREAS



Band 1	Steep, barren, impervious surfaces
Band 2	Rolling barren in upper band values, flat barren in lower part of band, steep forested and steep grass meadows
Band 3	Timber lands of moderate to steep slopes, mountainous, farming
Band 4	Flat pervious surface, flat farmlands, wooded areas and meadows

Table 2

MINIMUM RUNOFF COEFFICIENTS FOR BUILT-UP AREAS

RESIDENTIAL AREAS:	C	= 0.55 to 0.70
HOTEL-APARTMENT AREAS:	C	= 0.70 to 0.90
BUSINESS AREAS:	C	= 0.80 to 0.90
INDUSTRIAL AREAS:	C	= 0.80 to 0.90

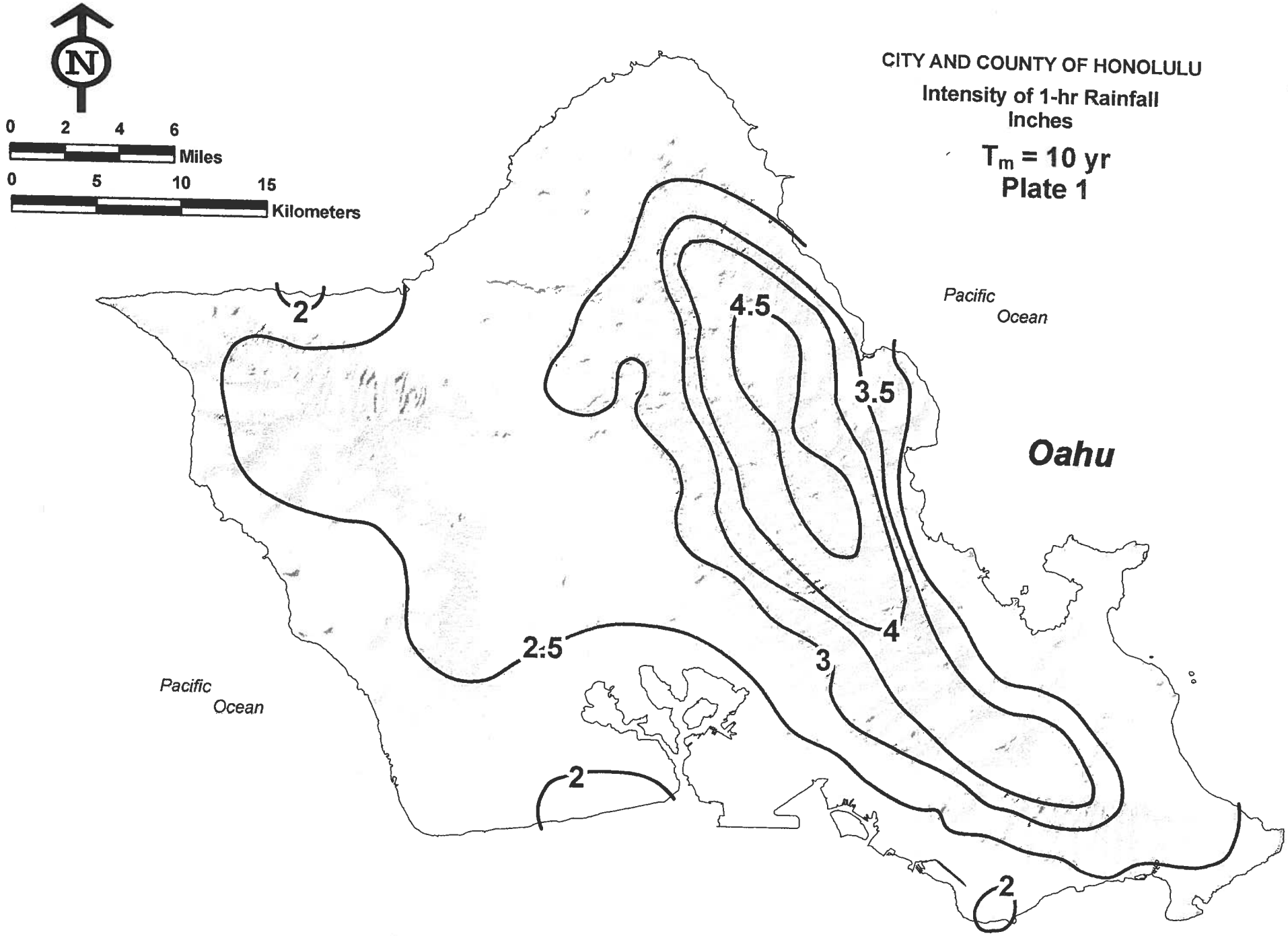
The type of soil, the type of open space, and ground cover and the slope of the ground shall be considered in arriving at reasonable and acceptable runoff coefficients.

Table 3

APPROXIMATE AVERAGE VELOCITIES OF RUNOFF FOR CALCULATING TIME OF CONCENTRATION

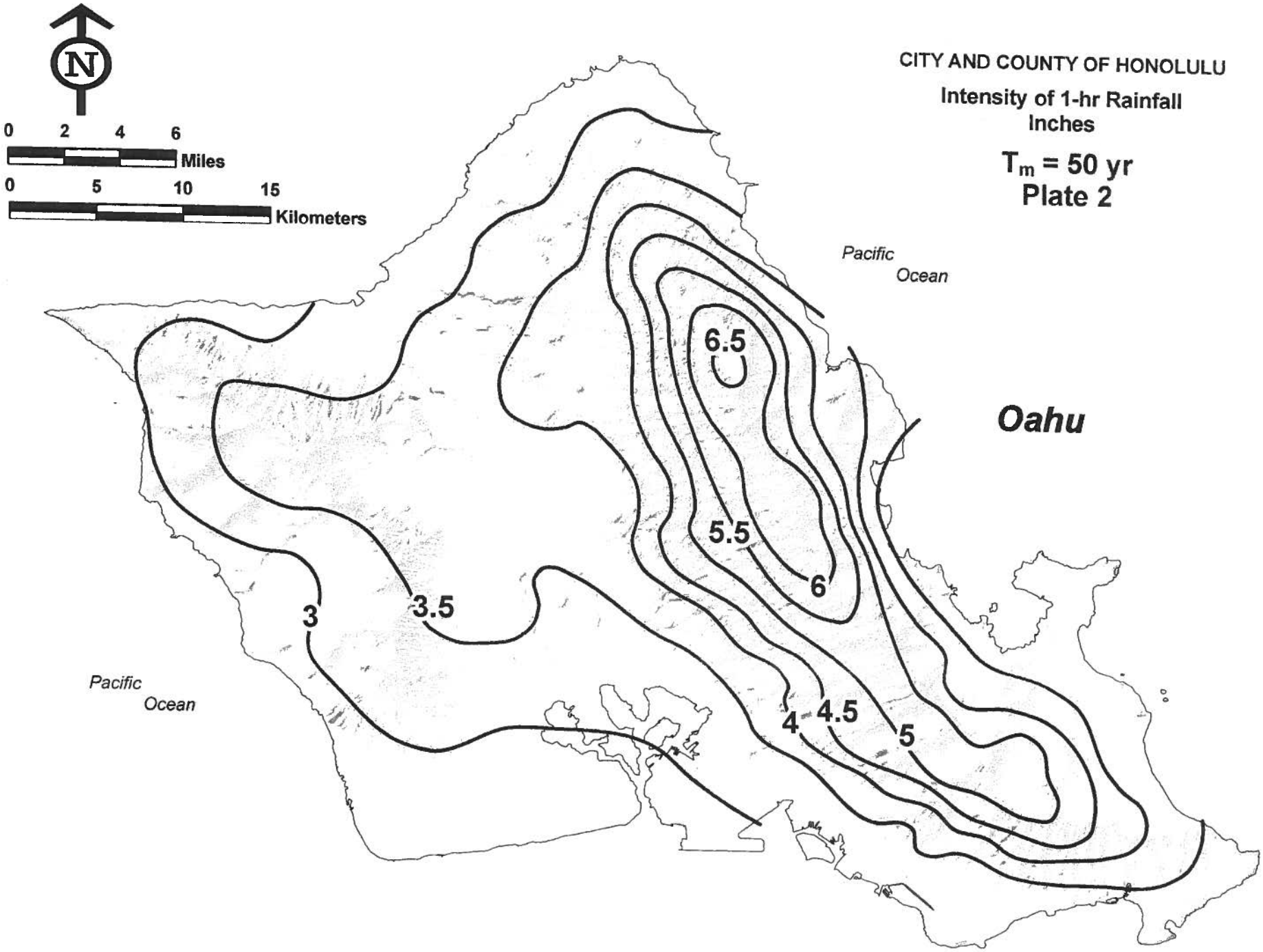
TYPE OF FLOW	VELOCITY IN fps FOR SLOPES (in percent) INDICATED			
OVERLAND FLOW:	0-3%	4-7%	8-11%	12-15%
Woodlands	1.0	2.0	3.0	3.5
Pastures	1.5	3.0	4.0	4.5
Cultivated	2.0	4.0	5.0	6.0
Pavements	5.0	12.0	15.0	18.0
OPEN CHANNEL FLOW:				
Improved Channels	Determine Velocity by Manning Formula			
Natural Channel* (not well defined)	1.0	3.0	5.0	8.0

* *These values vary with the channel size and other conditions so that the ones given are averages of a wide range. Wherever possible, more accurate determinations should be made for particular conditions by Manning Formula or from Plate 5.*



[Eff: APR 08 2011] (Auth: Sec 14-12.31, ROH) (Imp: Sec 14-12.31, ROH)

Source: National Oceanic and Atmospheric Administration (NOAA), National Weather Service, Silver Spring, Maryland, 2009

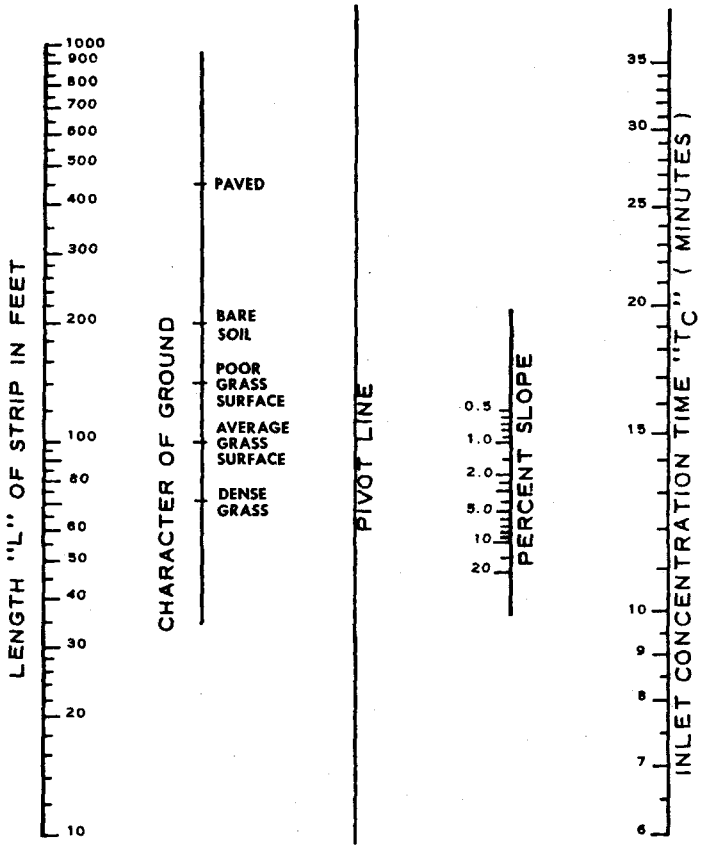


[Eff: APR 0 8 2011] (Auth: Sec 14-12.31, ROH) (Imp: Sec 14-12.31, ROH)

Source: National Oceanic and Atmospheric Administration (NOAA), National Weather Service, Silver Spring, Maryland, 2009

Plate 3

Overland Flow Chart



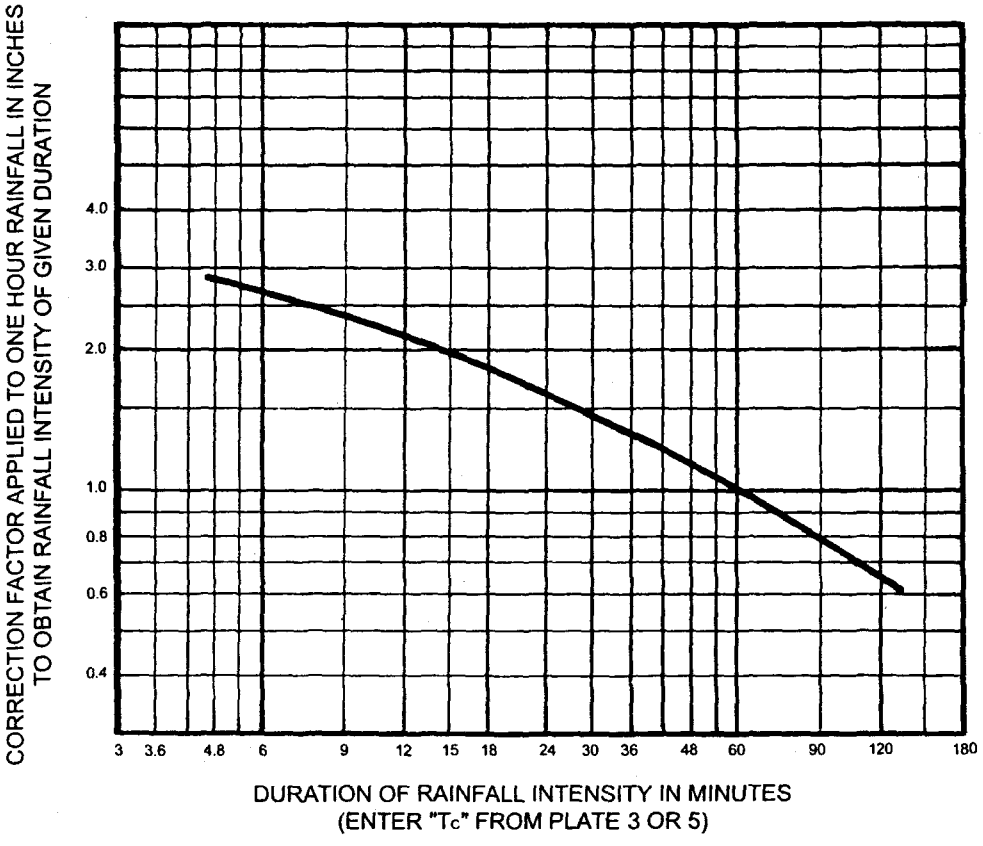


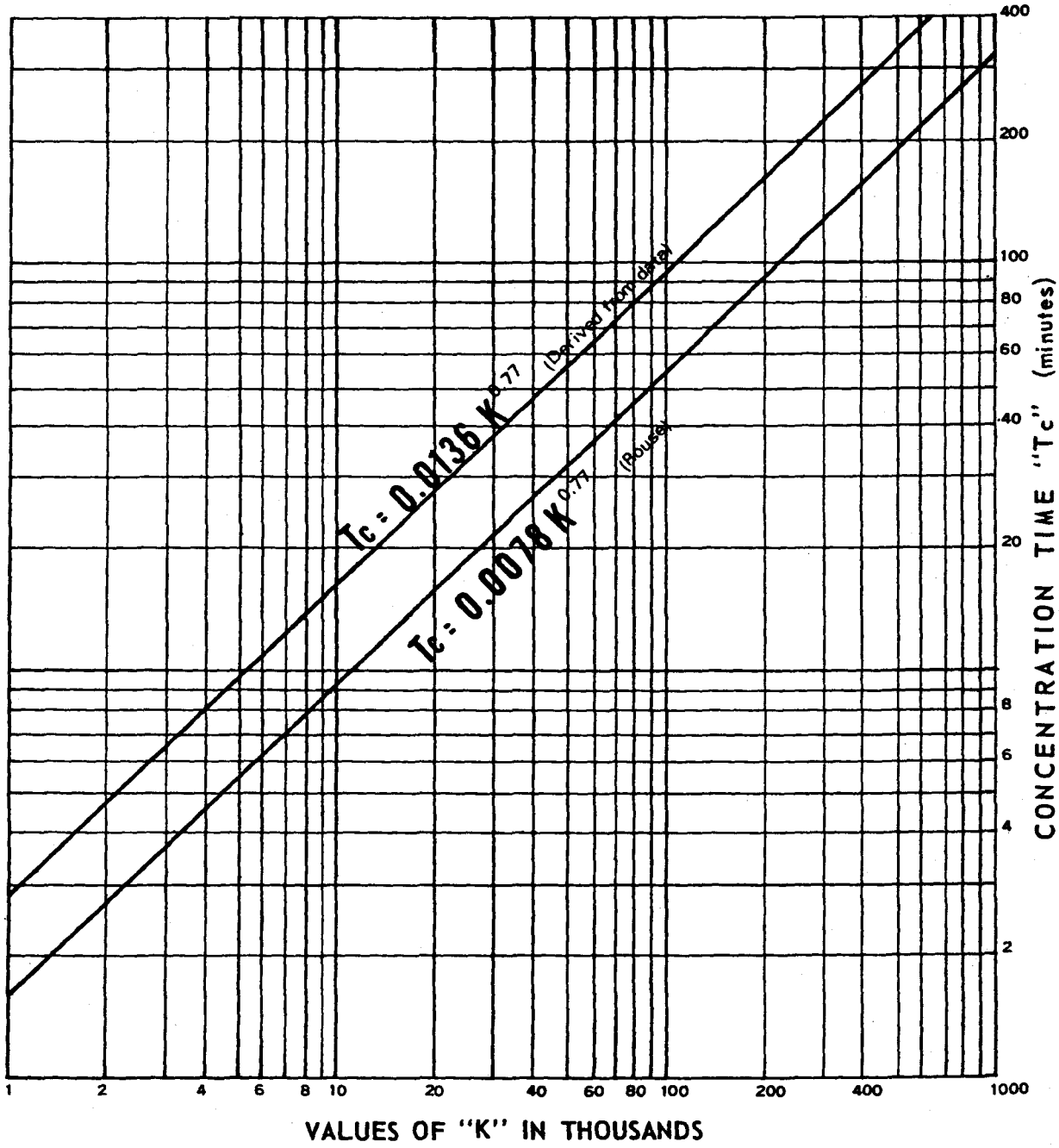
Plate 4

CORRECTION FACTOR

FOR CONVERTING 1 HR. RAINFALL TO RAINFALL INTENSITY OF VARIOUS DURATIONS

TO BE USED FOR AREA LESS THAN 100 ACRES

(See Plate 6 for area more than 100 acres)



L = Maximum length of travel in feet

H = Difference in elevation between most remote point and outlet in feet.

S = Slope H/L

$$K = \frac{L}{\sqrt{S}} = \sqrt{\frac{L^3}{H}}$$

Use upper curve for well forested areas

Use lower curve for areas with little or no cover.

SOURCE: CITY PLANNING COMMISSION
graph from Hunter Rouse "Engineering Hydraulics."

NOTE: Use 5 minutes if Tc is 5 minutes or less.

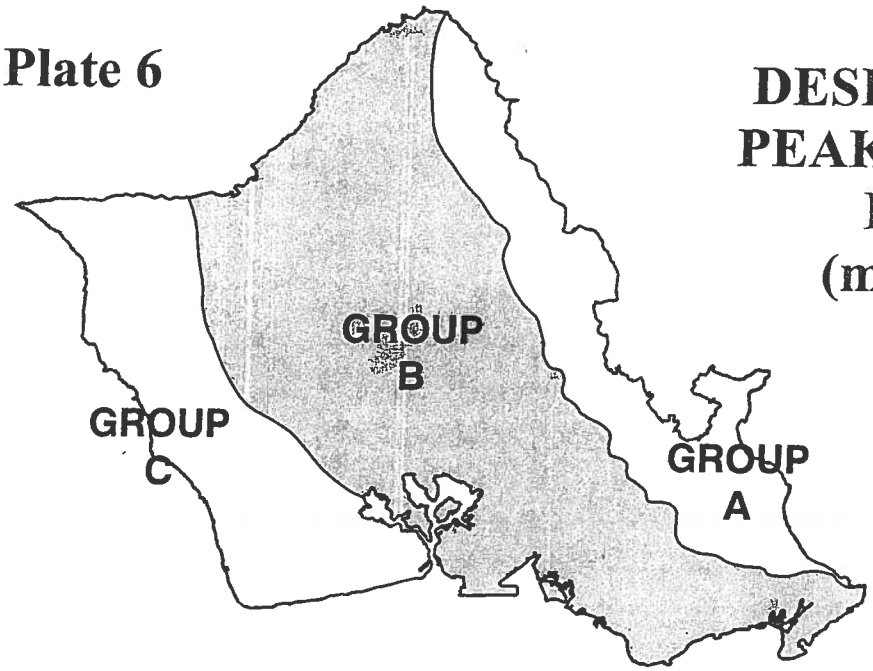
Plate 5

Time of Concentration

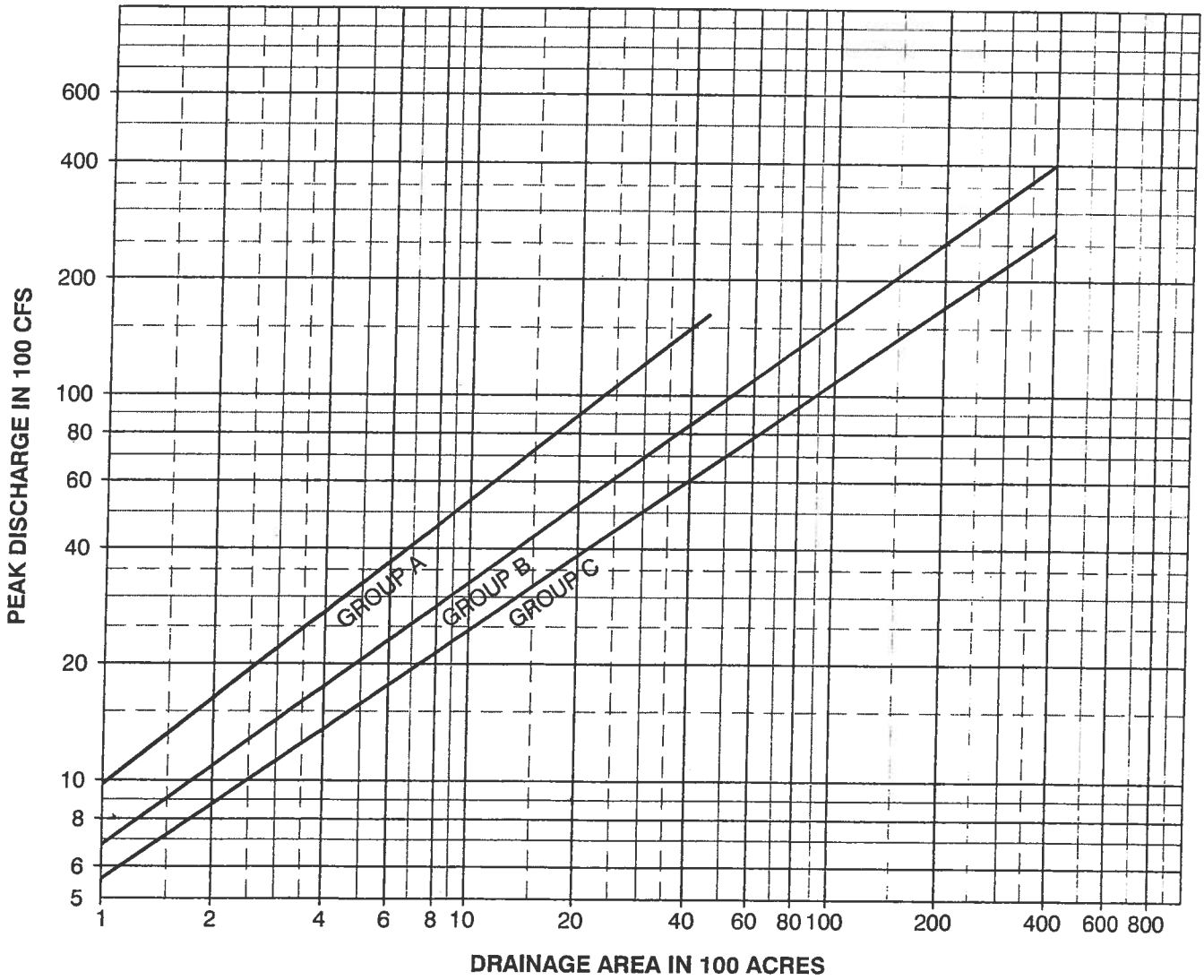
(OF SMALL AGRICULTURAL DRAINAGE BASIN)

Plate 6

DESIGN CURVES FOR PEAK DISCHARGE VS. DRAINAGE AREA (more than 100 acres)



● CURVES ARE FOR STREAM CHANNELS AND DRAINAGE STRUCTURES.



SOURCE: DATA FROM U.S. GEOLOGICAL SURVEY REV. FEB 2003.

jsb - D:\PROJECTS\26537218_000B1\DC-DA.DWG - Mon, 21 Jun 2010

[Eff: APR 08 2011] (Auth: Sec 14-12.31, ROH) (Imp: Sec 14-12.31, ROH)

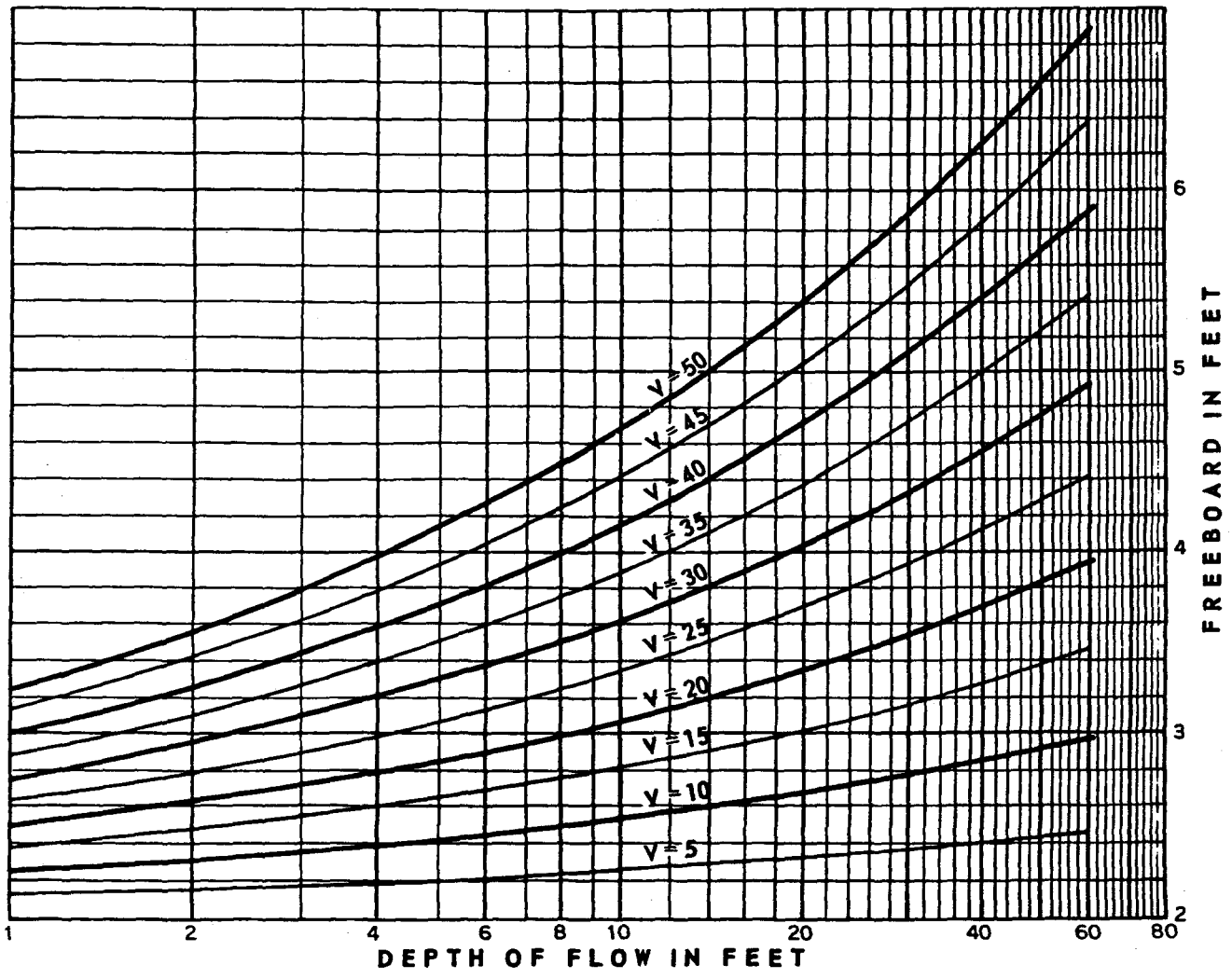
FREEBOARD ALLOWANCES Plate 7

FREEBOARD IN FEET:

$$2.0 + 0.025\, V \sqrt[3]{d}$$

Where V = Velocity, in feet per second

d = Depth of flow, in feet



Pipe Flow Charts

The following pipe flow charts have been derived by the *U.S. Public Roads Administration, Division Two, Washington, D.C.* These charts are designed to enable direct solution of the Manning formula for circular pipes flowing full and for uniform part-full flow in circular pipes. The "n" scales of 0.013 and 0.024 have been inserted to facilitate the use of these charts for storm drainage systems in Honolulu. The following examples help explain the use of the pipe flow charts.

EXAMPLES

A. Determine the depth and velocity of flow in a long 30-inch pipe, n = 0.013, on a 0.5 percent slope ($S_o = 0.005$) discharging 25 cfs. Enter the 30-inch diameter chart at Q = 25 on n = 0.013 scale, follow up to intersection with the line for slope $S_o = 0.005$, and read normal depth $d_n = 1.8$ feet and normal velocity V = 6.7 fps.

To find critical depth, enter chart Q = 25 and n = 0.015 scale , and read critical depth $d_c = 1.7$ feet at intersection with dotted critical curve. Also critical velocity $V_c = 7.0$ fps. (Note: Critical depth and velocity would be the same, regardless of pipe roughness).

B. Determine friction slope for a 30-inch corrugated metal pipe, n = 0.024, on a slope $S_o = 0.008$ ft/ft with a discharge Q = 25 cfs. Enter the 30-inch diameter chart at Q = 25 on n = 0.024 scale. Note that this ordinate falls to the right of the 0.008 slope line, therefore, the pipe will flow full. Read friction slope $S_f = 0.012$ at the line for depth equal to pipe diameter.

$$\text{Note } Q = 25 \times \frac{0.024}{0.015} = 40 \text{ cfs on the Q-scale for } n = 0.015)$$

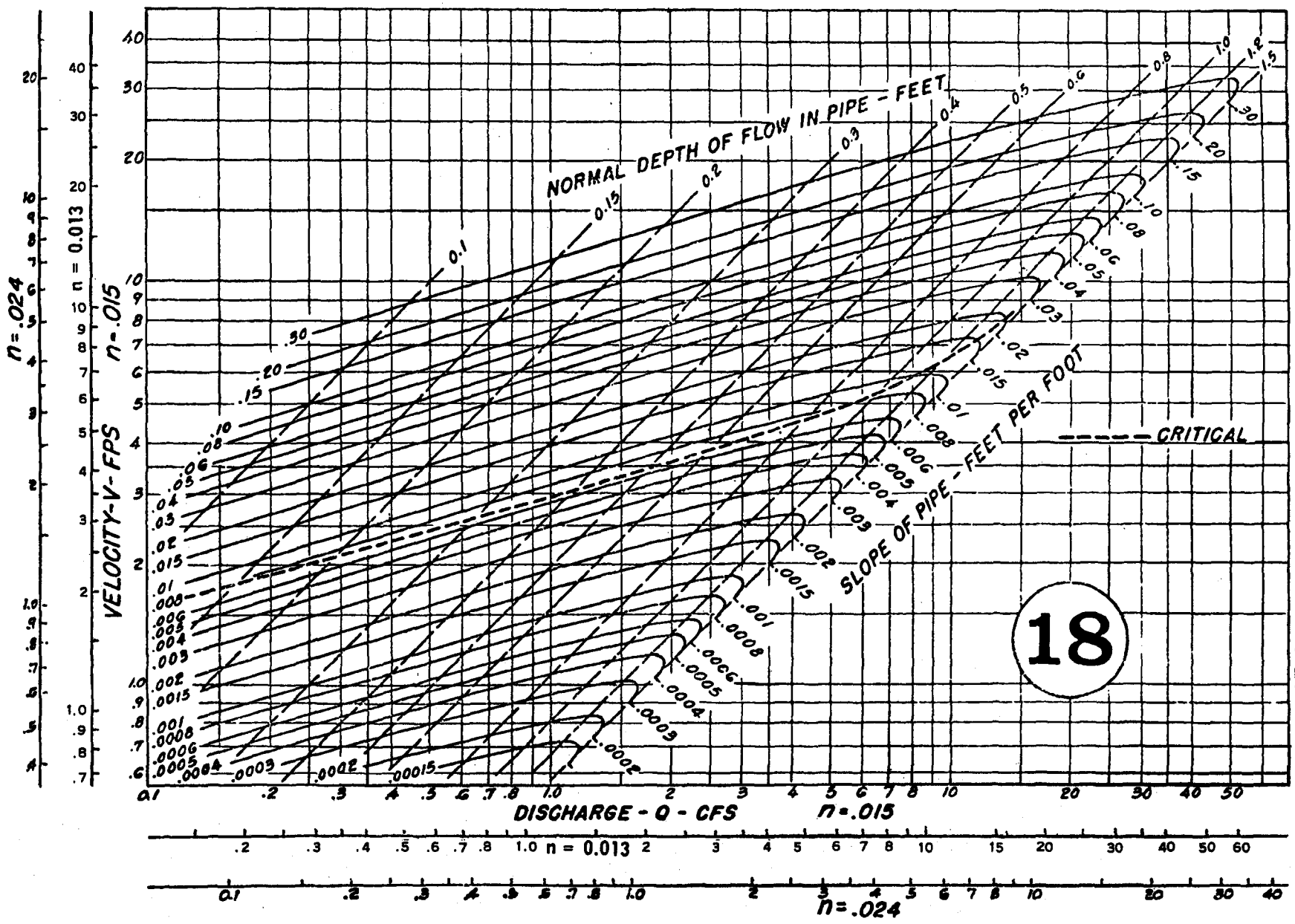
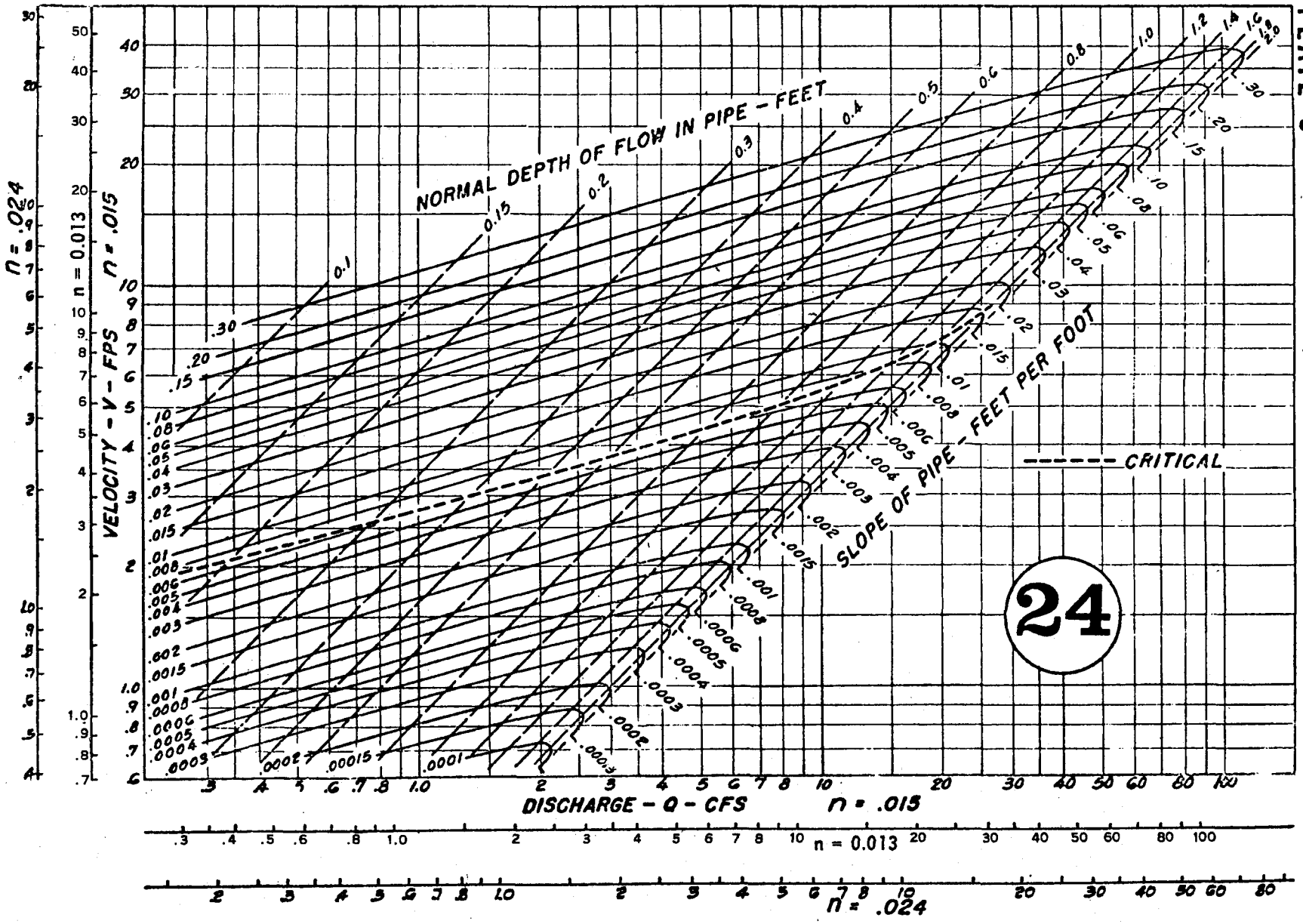


PLATE 8

Pipe Flow Chart 18 inch Diameter

PLATE 9



Pipe Flow Chart 24 inch Diameter

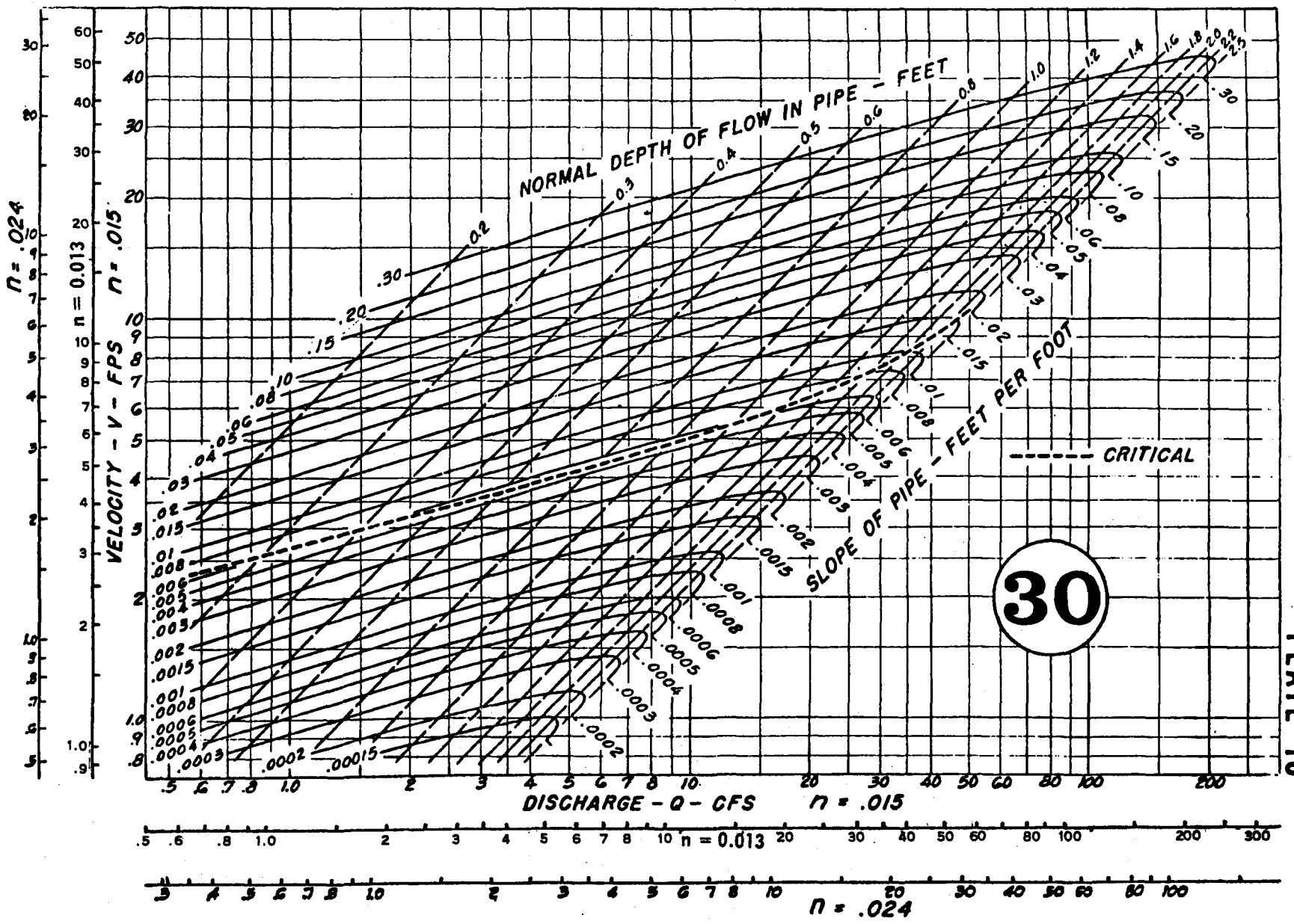
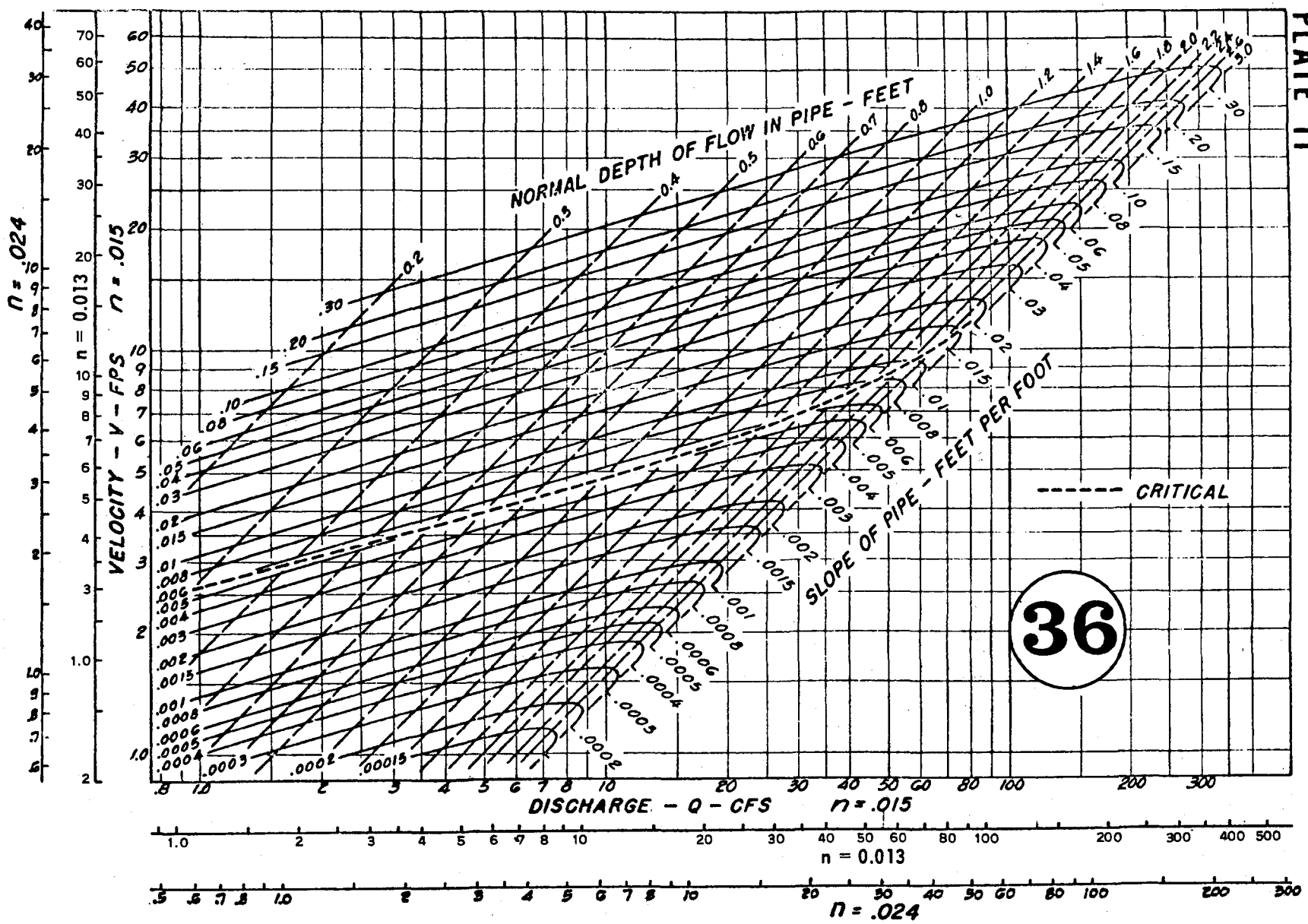


PLATE 10

Pipe Flow Chart 30 inch Diameter

PLATE 11



Pipe Flow Chart 36 inch Diameter

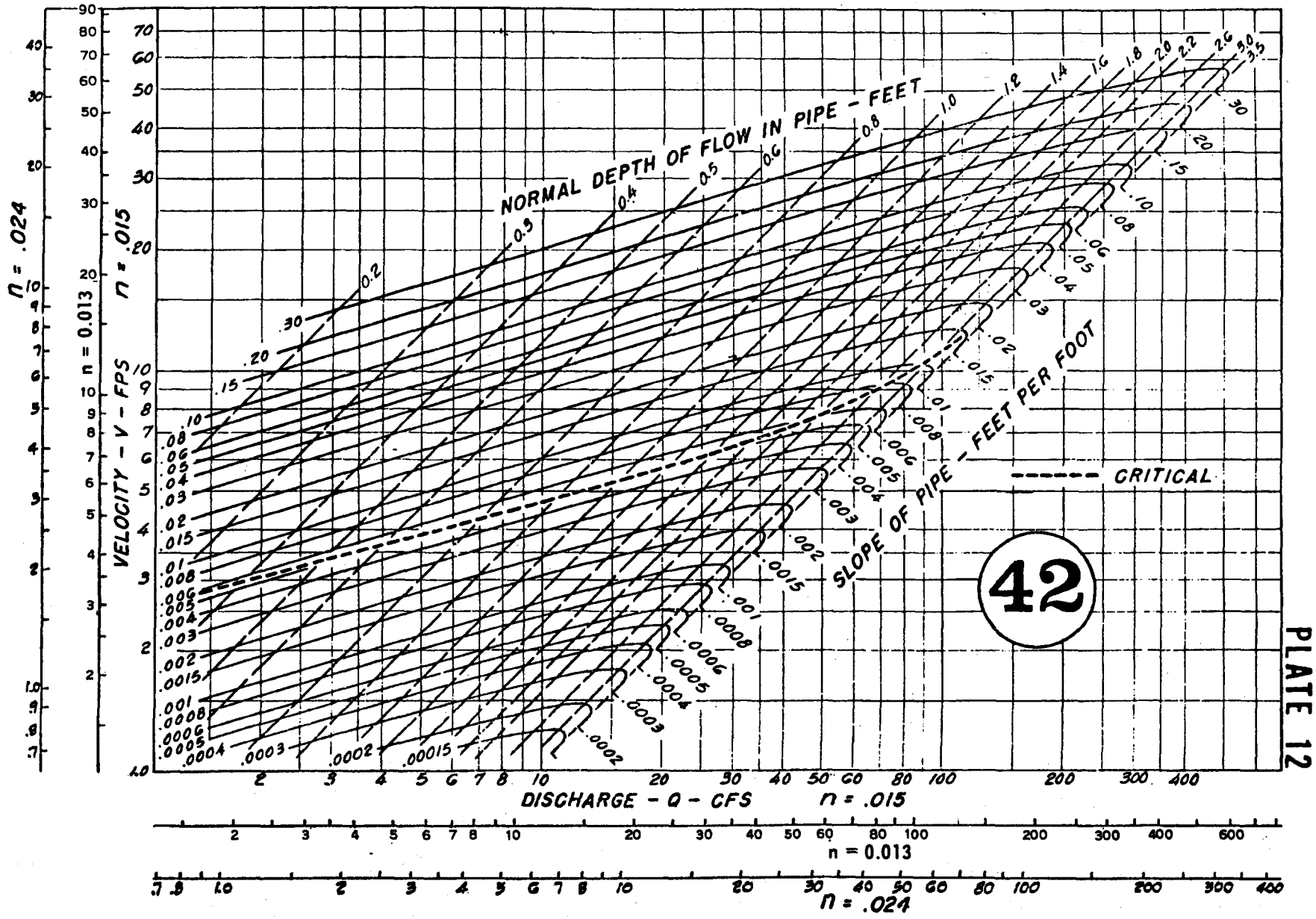
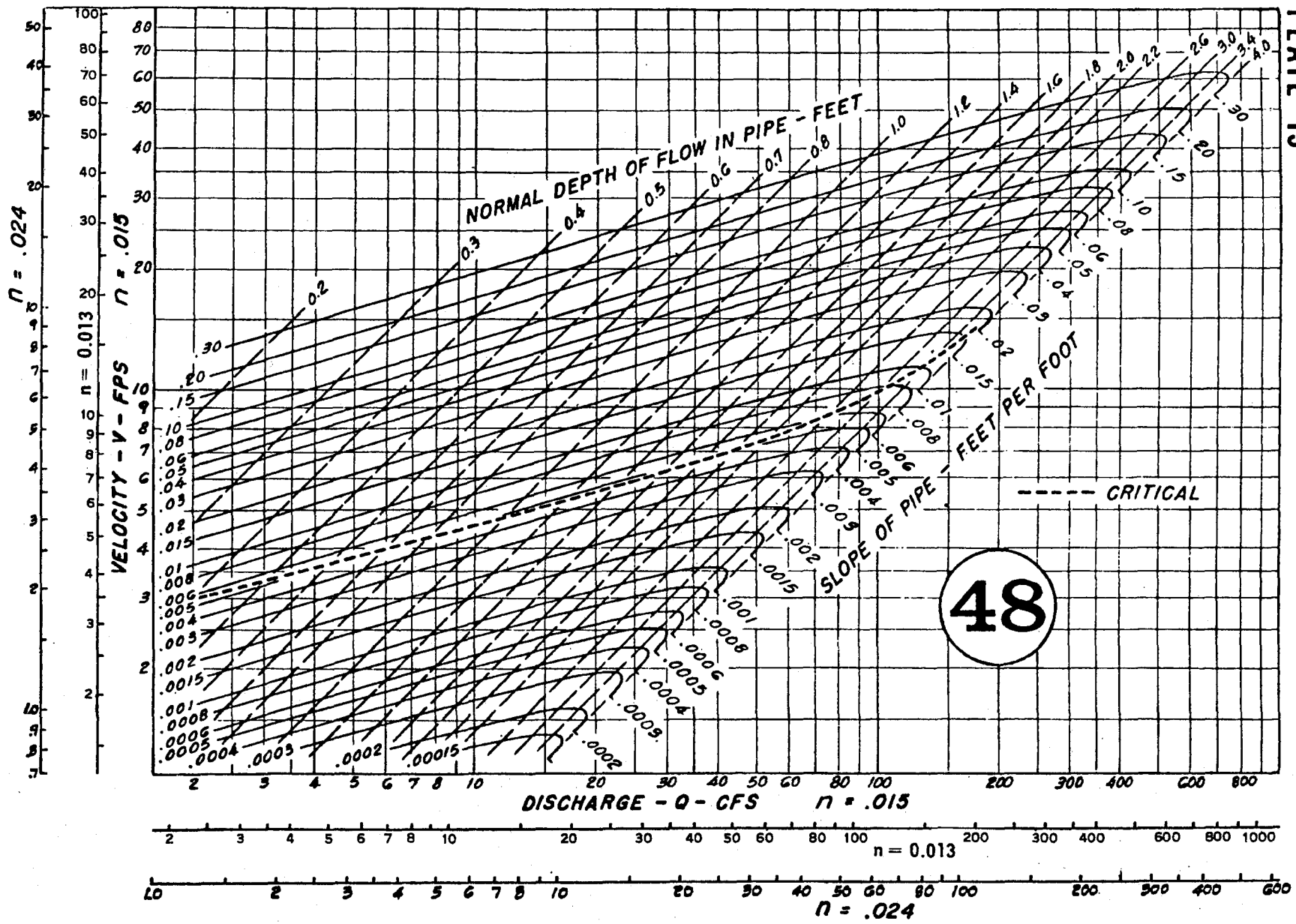


PLATE 12

Pipe Flow Chart 42 inch Diameter

PLATE 13



Pipe Flow Chart 48 inch Diameter

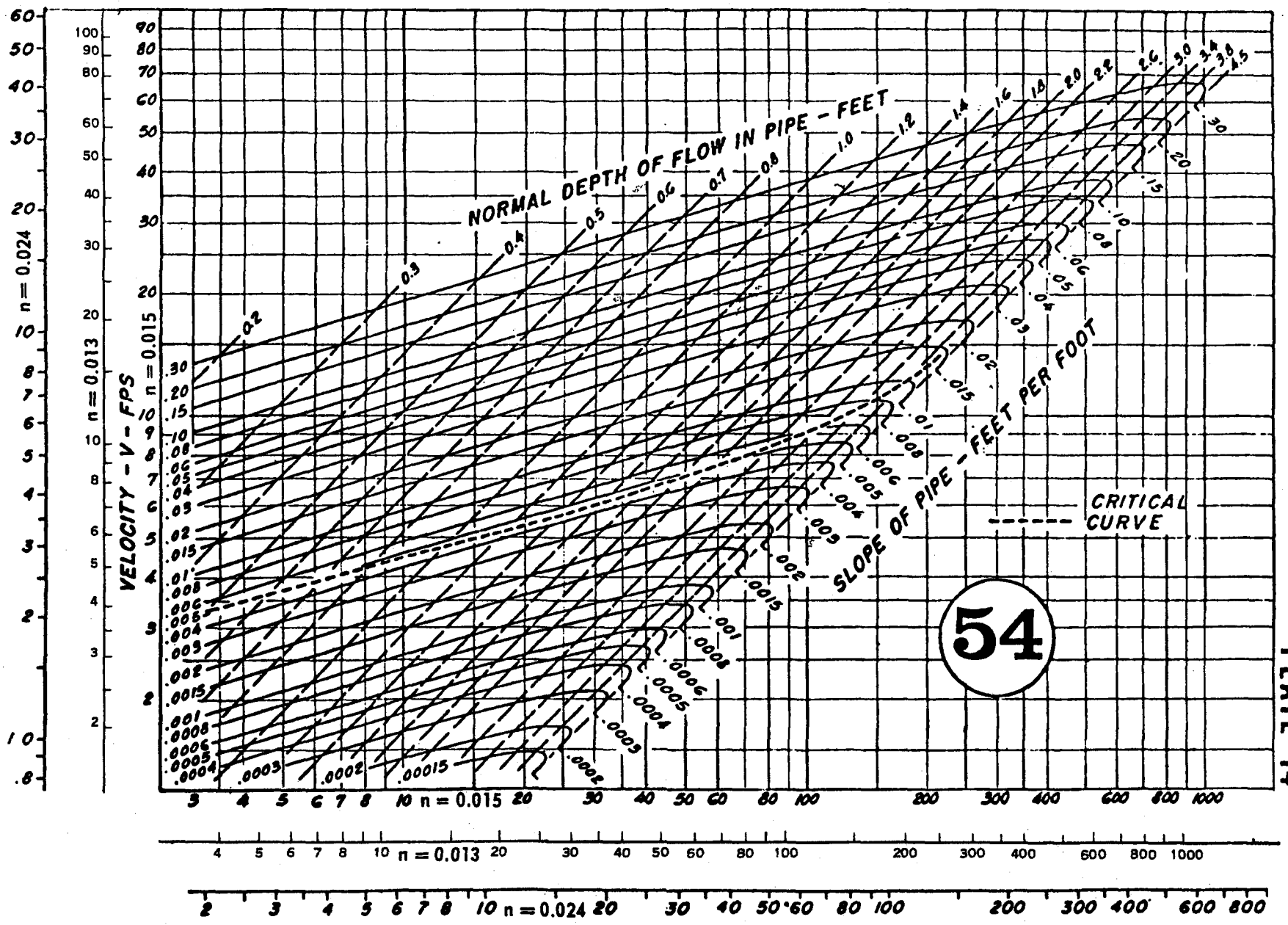
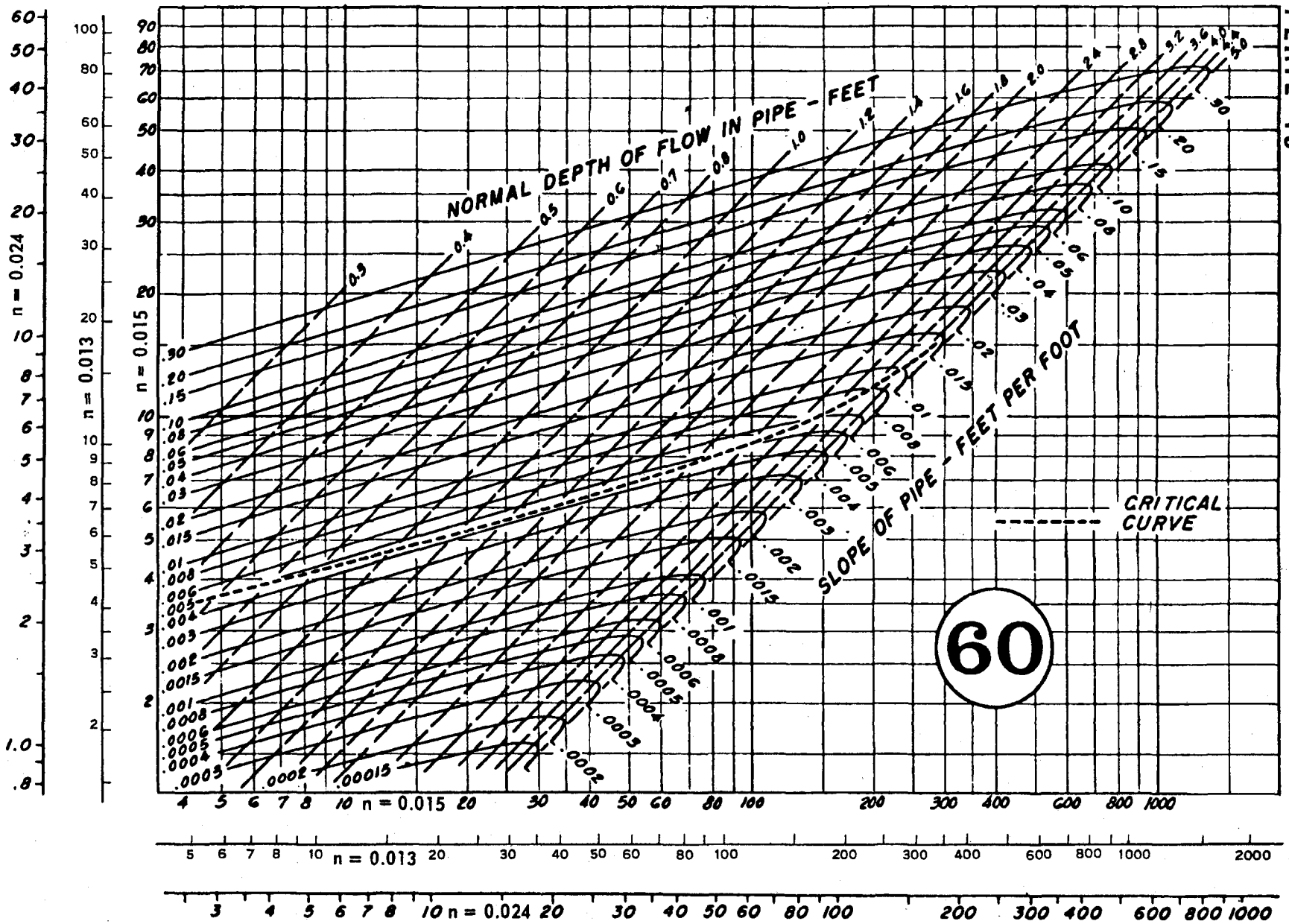


PLATE 14

Pipe Flow Chart 54 inch Diameter



Pipe Flow Chart 60 inch Diameter

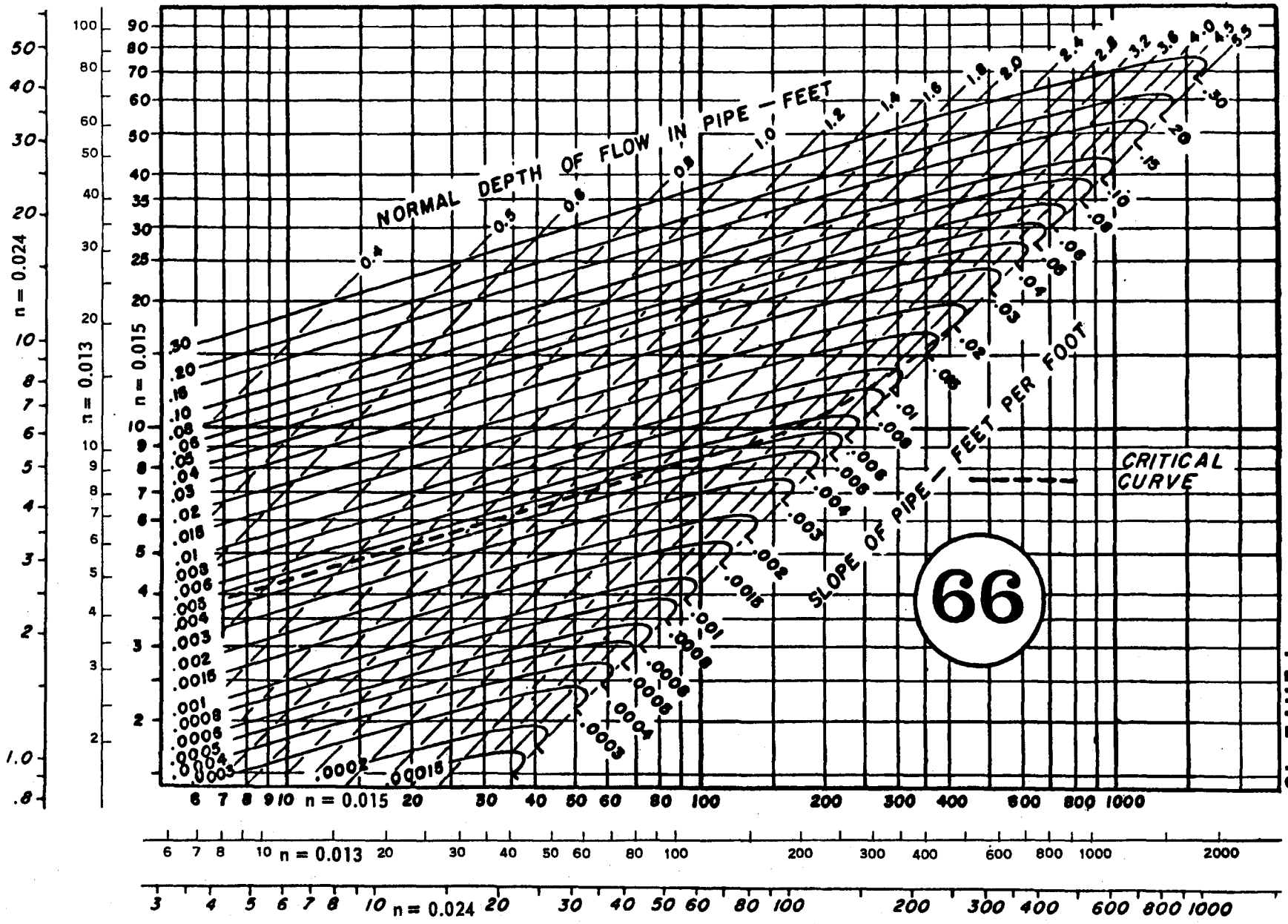
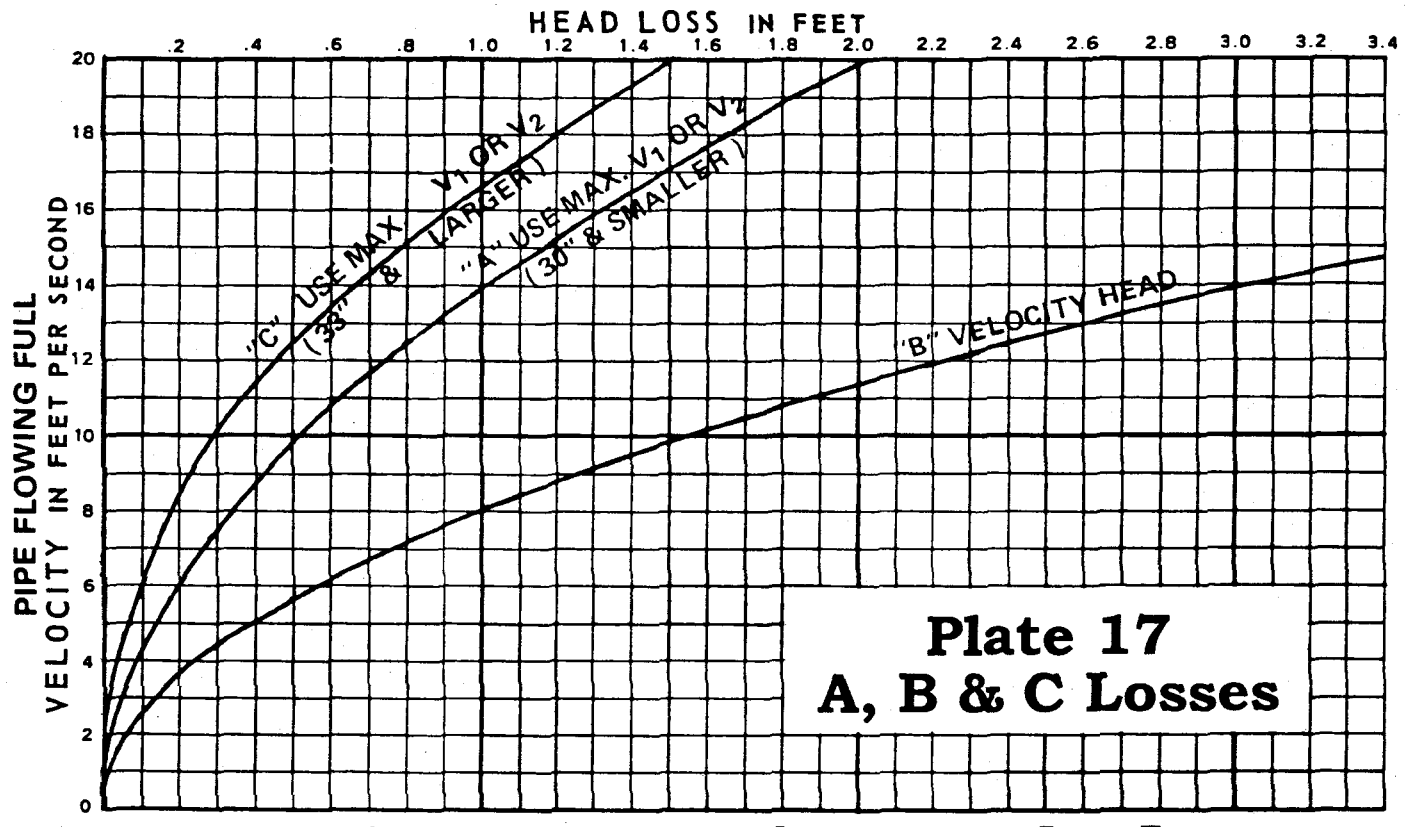
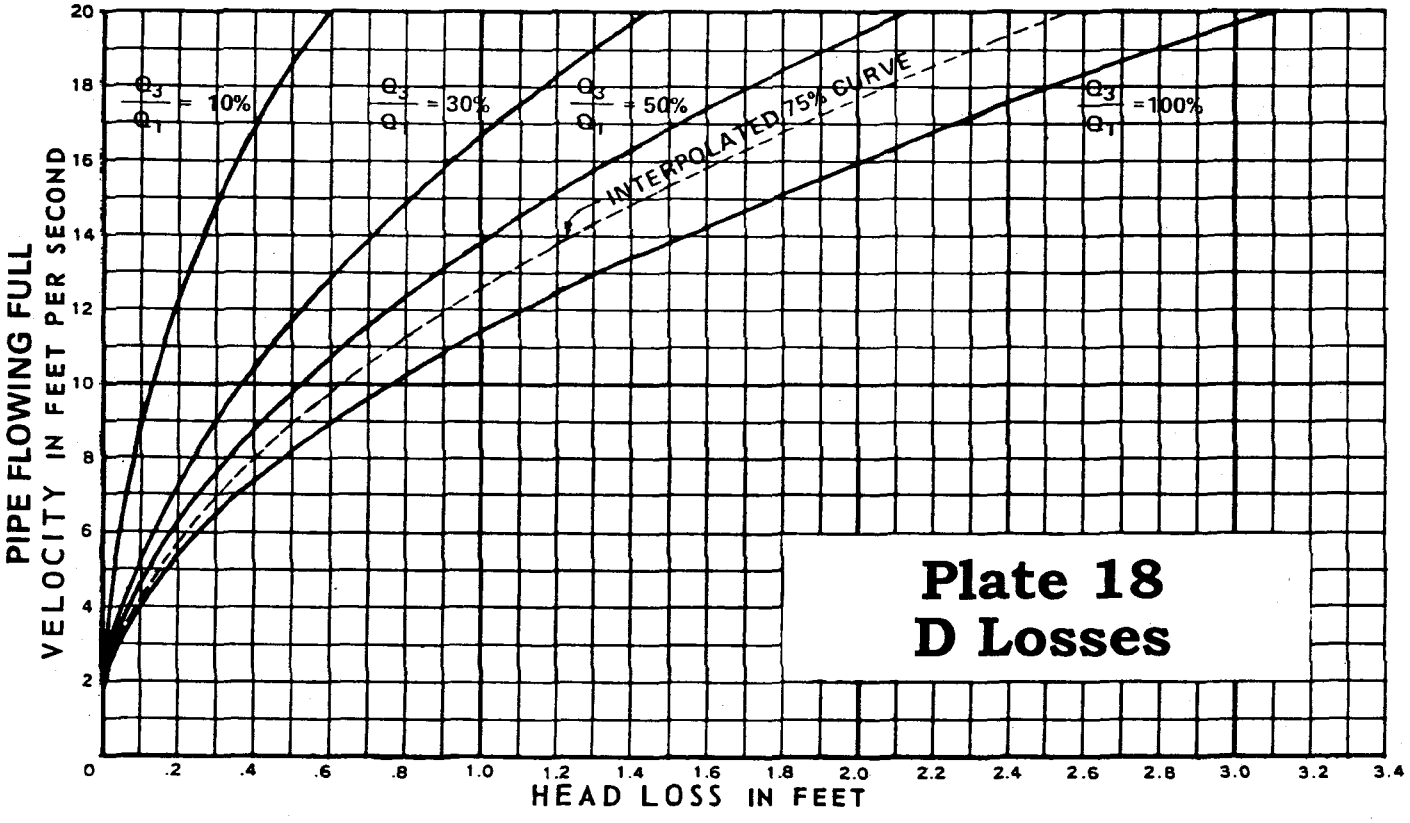


PLATE 16

Pipe Flow Chart 66 inch Diameter



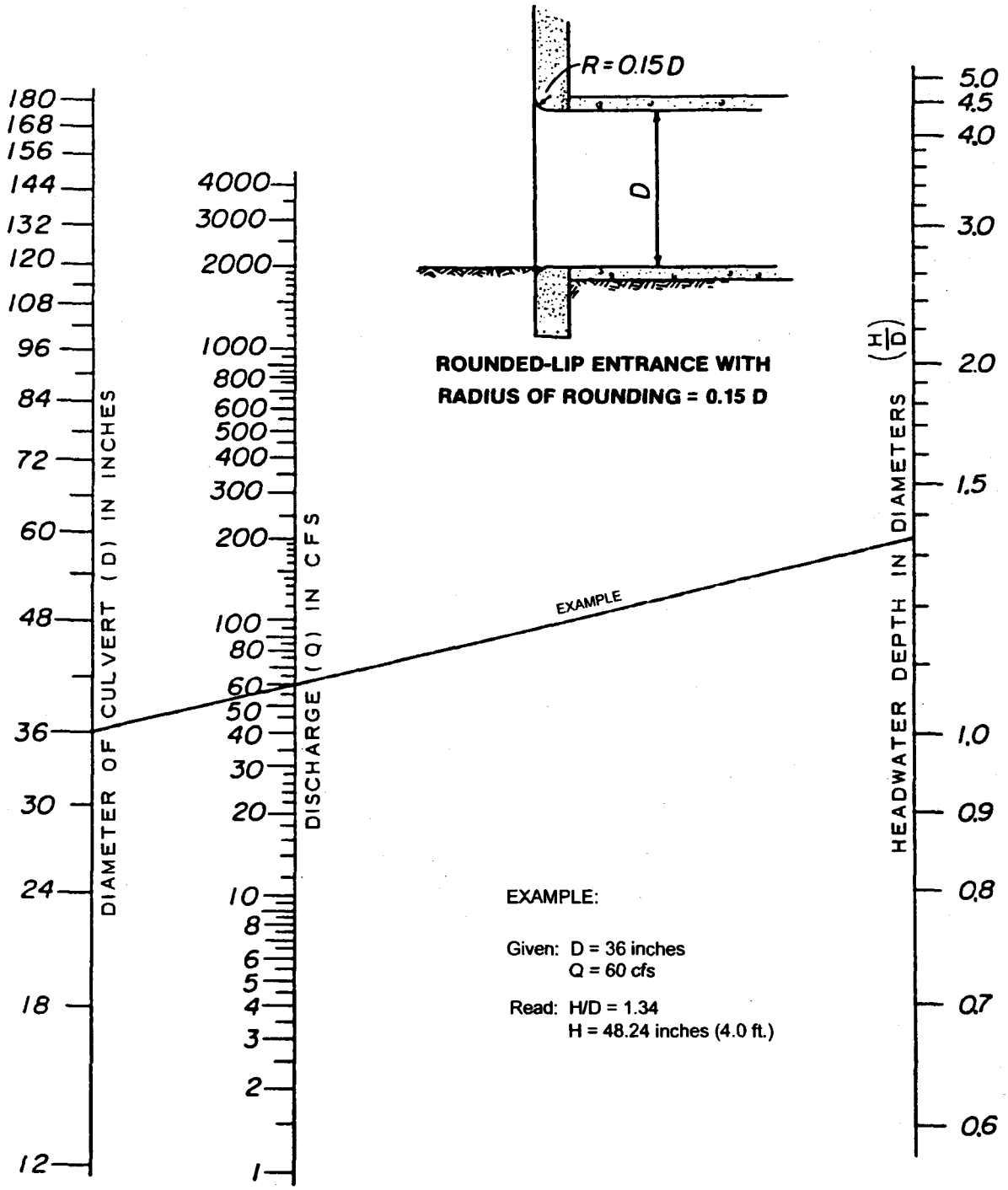
Head Losses in Manholes



SOURCE: BALTIMORE COUNTY DEPARTMENT OF PUBLIC WORKS

NOMOGRAPH FOR PIPE CULVERTS WITH ENTRANCE CONTROL

Plate 19



EXAMPLE:

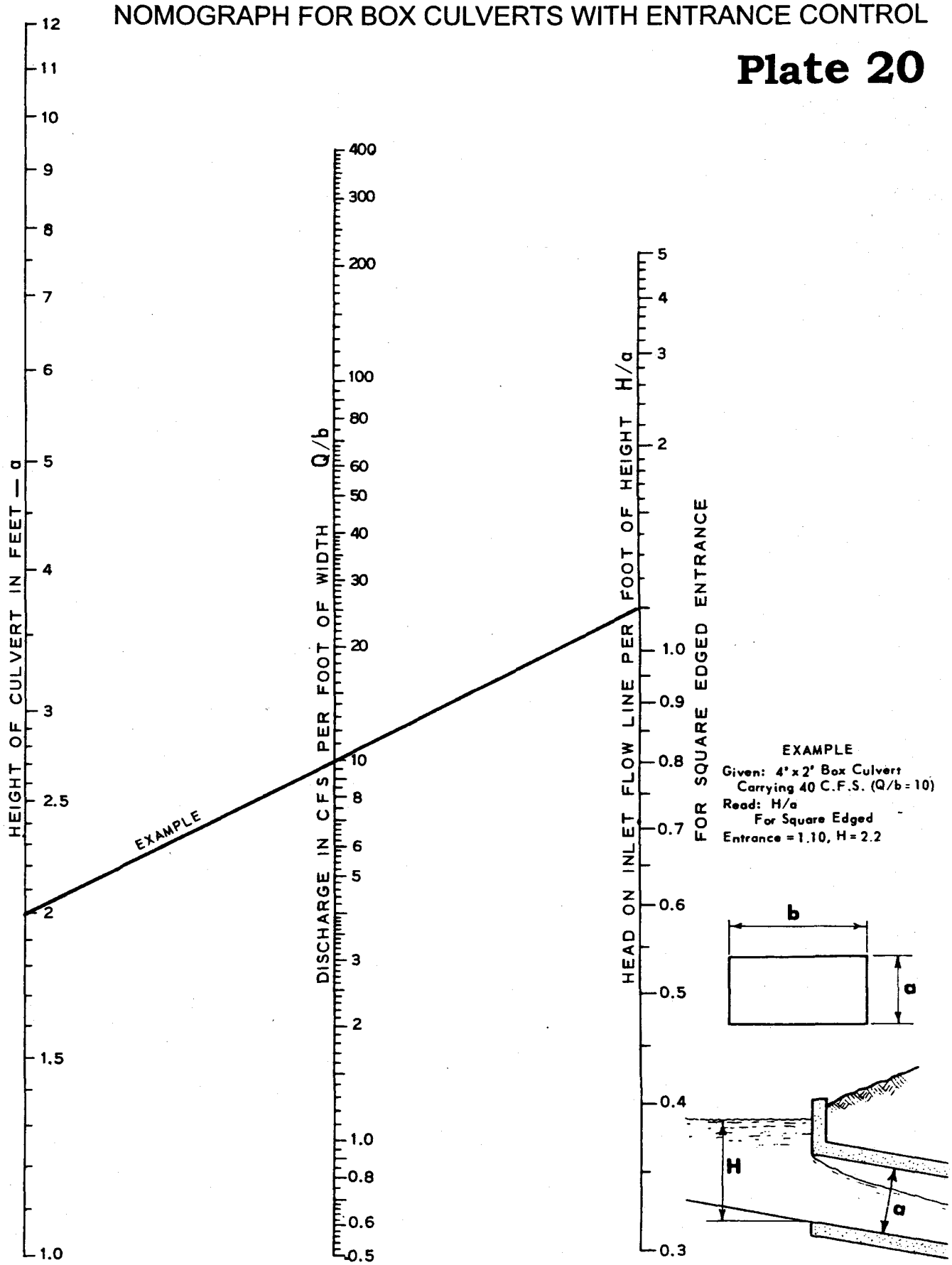
Given: D = 36 inches
Q = 60 cfs

Read: H/D = 1.34
H = 48.24 inches (4.0 ft.)

SOURCE: HANDBOOK OF CONCRETE CULVERT PIPE HYDRAULICS

NOMOGRAPH FOR BOX CULVERTS WITH ENTRANCE CONTROL

Plate 20



§1-5 SECTION II – STANDARDS FOR STORM WATER QUALITY

In response to the requirements of the City's NPDES permit, the City Council passed Ordinance 96-34 addressing the need to regulate storm runoff design criteria for flood control and water quality. This includes establishing controls on the timing and rate of discharge of storm water runoff to reduce storm water runoff pollution to the maximum extent practicable through the implementation of best management practices and engineering control facilities designed to reduce the generation of pollutants.

Long-term water quality is impacted by the volume and frequency of discharged pollutants.

Water quality is also impacted by the modification of a stream's hydrograph caused by increases in flows and durations that result when land is developed (e.g., made more impervious). This phenomenon known as hydromodification, effectively reduces stream base-flow (groundwater flow into streams) and increases overland or storm-flow which causes reduced groundwater recharge and increased peak discharge rates into streams. Hydromodification may result in stream channel instability, streambank or shoreline erosion, loss of habitat, increased sediment transport and deposition, and increased flooding. Consequently, water quality measures for a development should also be designed to include LID BMPs to manage and control hydromodification.

§1-5.1 PART I - WATER QUALITY CRITERIA

A. OBJECTIVES OF WATER QUALITY CRITERIA

The purpose of the water quality criteria is to reduce the pollution associated with storm water runoff from new development and redevelopment. By establishing these criteria, the City and County of Honolulu is satisfying Federal regulatory requirements to control the discharge of pollutants in storm water as specified in the Clean Water Act Amendments of 1987 and its NPDES permit for discharges from the Municipally Owned and Operated Separate Storm Sewer System issued by the Hawaii Department of Health (DOH) under the authority by the United States Environmental Protection Agency (EPA). Under the NPDES program, the City is required to reduce the discharge of pollutants to receiving waters to the "maximum extent practicable" (MEP).

B. REQUIREMENT APPLICABILITY

1. DEVELOPMENT AND REDEVELOPMENT INCLUDED

Applicable new development and redevelopment projects as defined in B.2a of §1-5.1 Part I Water Quality Criteria must address storm water quality to the MEP through the use of Low Impact Development (LID) Site Design Strategies, Source Control Best Management Practices (BMPs), LID Post-Construction Treatment Control BMPs, and Other Post-Construction Treatment Control BMPs.

For redevelopment projects, the requirements presented in B.6 of §1-5.1 Part I Water Quality Criteria apply only to the addition, and not to the entire development. Redevelopment includes, but is not limited to expansion of a building footprint; addition to or replacement of a structure; replacement of an impervious surface that is

not part of a routine maintenance activity; land disturbing activities related to structural or impervious surfaces. Redevelopment does not include routine maintenance activities that are conducted to maintain original hydraulic capacity, original purpose of facility or emergency redevelopment activity required to protect public health and safety. Impervious surface replacement, such as the reconstruction of parking lots and roadways which does not disturb additional area is considered a routine maintenance activity. Redevelopment does not include the repaving of existing roads.

Projects cannot be subdivided or phased to avoid complying with these requirements. Development and redevelopment of the same or adjacent property (ies) permitted within 5 years may be considered together for purposes of assessing the above criteria. The sizing of water quality facilities and drainageways shall be based upon the ultimate use of the drainage area, unless the water quality feature will be re-built/sized during subsequent phases of construction.

2. REGULATED PROJECTS

For purposes of meeting the objectives presented in A of §1-5.1 Part I Water Quality Criteria, projects shall be regulated as follows[1]:

a. Priority A Projects. New development and redevelopment projects that disturb at least 1 acre of land and that are not required to obtain a separate industrial NPDES storm water permit from DOH for long term storm water discharges. Projects at least 5 acres in size are classified as A1, and all others are classified as A2.

b. Priority B Projects. New development and redevelopment projects that do not meet the criteria of a Priority A project but meet any of the following criteria:

   1) Retail Gasoline Outlet with at least 10,000 square feet of total impervious surface area;

   2) Automotive Repair Shop with at least 10,000 square feet of total impervious surface area;

   3) Restaurant with at least 10,000 square feet of total impervious surface area;

   4) Parking lot with at least 10,000 square feet of total impervious surface area

   Impervious surfaces include, but are not limited to, rooftops; walkways; patios; driveways; parking lots; storage areas; impervious concrete and asphalt; and any other continuous watertight pavement or covering. Landscaped soil and pervious pavement, underlain with pervious soil or pervious storage material, are not impervious surfaces.

3. PROJECT APPLICABILITY

These rules shall be effective as of June 1, 2013. The Director may exempt projects from the application of these rules if projects are determined to have submitted

[1] **Criteria for Regulated Projects may be revised as necessary by the Department (as described in B.7 of §1-5.1 Part I Water Quality Criteria)**

completed construction drawings and completed site-specific drainage reports prior to June 1, 2013.

4. OFF-SITE RUNOFF APPLICABILITY

   These criteria are required to be applied to runoff arising from a site and not from off-site runoff, unless the off-site runoff is entering the site as overland flow, and/or will not be separated from on-site runoff. If off-site runoff is to be conveyed through a water quality facility, then the facility must be designed to meet the requirements as described below for the combined on-site and off-site runoff volumes and/or rates.

5. JURISDICTIONAL APPLICABILITY

   These requirements apply to projects that drain to City and County drainage facilities and all natural drainage ways that the City and County has ownership and/or responsibility for. Developments that are located in areas that do not drain to the above facilities may be required to meet other DOH requirements.

6. MANAGEMENT PRACTICES TO MEET CRITERIA

   a. Priority A1 Projects

      The criteria shall be met for Priority A1 projects as follows:

      i. Incorporate appropriate LID Site Design Strategies to the MEP.

      ii. Incorporate appropriate Source Control BMPs to the MEP.

      iii. Unless determined to be infeasible, retain on-site by infiltration or evapotranspiration, the Water Quality Volume or "WQV" with appropriate LID Retention Post-Construction Treatment Control BMPs. The WQV is defined in A of §1-5.2 Part II, Water Quality Design Standards.

      iv. Unless determined to be infeasible, biofilter any portion of the Water Quality Volume that is not retained on-site with appropriate LID Biofiltration Post-Construction Treatment Control BMPs.

      "Infeasible" means conditions at the site make the implementation of a specific Low Impact Development Best Management Practice technically infeasible. Infeasibility criteria are defined in E of §1-5.2 Part II, Water Quality Design Standards. If it is demonstrated to be infeasible to retain and/or biofilter the Water Quality Volume, one of the following alternative compliance measures is required:

      - Either harvest/reuse, or treat (by detention, filtration, settling, or vortex separation) and discharge with appropriate Other Post-Construction Treatment Control BMPs, any portion of the Water Quality Volume that is not retained on-site or biofiltered.

      - Retain or biofilter at an offsite location, the volume of runoff equivalent to the difference between the project's WQV and the amount retained on-site or biofiltered. Offsite mitigation projects must be submitted for City approval.

b. Priority A2 Projects

The criteria shall be met for Priority A2 projects as follows:

i. Incorporate appropriate LID Site Design Strategies to the MEP.

ii. Incorporate appropriate Source Control BMPs to the MEP.

iii. Unless determined to be infeasible, either retain on-site by infiltration or evapotranspiration, the Water Quality Volume with appropriate LID Retention Post-Construction Treatment Control BMPs, or biofilter the Water Quality Volume with appropriate LID Biofiltration Post-Construction Treatment Control BMPs, or a combination of the two.

Infeasibility criteria are defined in E of §1-5.2 Part II, Water Quality Design Standards. If it is demonstrated to be infeasible to retain and/or biofilter the Water Quality Volume, one of the following alternative compliance measures is required:

- Either harvest/reuse, or treat (by detention, filtration, settling, or vortex separation) and discharge with appropriate Other Post-Construction Treatment Control BMPs, any portion of the Water Quality Volume that is not retained on-site or biofiltered.
- Retain or biofilter at an offsite location, the volume of runoff equivalent to the difference between the project's WQV and the amount retained on-site or biofiltered. Offsite mitigation projects must be submitted for City approval.

c. Priority B Projects

The criteria shall be met for Priority B projects as follows:

i. Consider appropriate LID Site Design Strategies.

ii. Incorporate appropriate Source Control BMPs to the MEP.

Documents providing details and recommendations on LID Site Design Strategies, Source Control BMPs, and Treatment Control BMPs may be found on the City's website.

7. ADDITIONAL REQUIREMENTS

The criteria identified in B.6 of §1-5.1 Part I, Water Quality Criteria are minimum requirements. If the department determines that additional controls and/or lower thresholds for developments are required to meet the specific water quality needs in watersheds that drain to sensitive receiving waters (as defined by the Hawaii State Department of Health Water Quality Limited Segments [WQLS], or Class 1 Inland Waters, or Class AA Marine Waters), additional requirements may be imposed. These may include design requirements that result in larger facilities as well as additional types of structural or non-structural controls. The design solution will be contingent upon the pollutants that are found to be impacting such water bodies and the regulatory status of the water body.

8. DEDICATION OF FACILITIES TO CITY AND COUNTY

Water Quality facilities may be dedicated to the City. Application for dedication to the City must be approved prior to preparing subdivision maps and construction plans.

9. WATER QUALITY FACILITIES WITHIN PARKS

Parks may be utilized to satisfy water quality facility requirements, with concurrence of the appropriate City agencies, if such parks meet the intent and requirements of the park dedication ordinance and rules.

10. STORM WATER QUALITY FACILITIES REVIEW

The incorporation of storm water quality considerations is encouraged early in the development process as early design considerations will likely lead to more cost-effective projects. Storm water quality management strategies for Priority A1 projects shall be documented in a Storm Water Quality Report (SWQR). Storm water quality management strategies for Priority A2 and Priority B projects shall be documented in a Storm Water Quality Checklist (SWQC). A Storm Water Quality Report Preparation Manual, Storm Water Quality Checklist Preparation Manual, Storm Water Quality Report Template, and Storm Water Quality Checklist Templates may be found on the City's website to assist with and facilitate the preparation of SWQRs and SWQCs.

a. Submittal Requirements

Storm Water Quality Reports or Storm Water Quality Checklists shall be submitted for City review as follows:

1) For Priority A1 and Priority A2 projects, the project's Storm Water Quality Report or Storm Water Quality Checklist shall accompany construction plan approvals.

2) For Priority B projects, the project's Storm Water Quality Checklist shall accompany applications for applicable building and grading permits.

A narrative explaining the project's water quality management strategy must be included in the project's Master Plan, discretionary land use permit, or Environmental Assessment/Environmental Impact Statement.

Storm Water Quality Reports and Storm Water Quality Checklists shall be signed by the owner/developer certifying that the management practices will be implemented and maintained, and signed and stamped by a Professional Engineer licensed and registered to practice in the state of Hawaii, stating that the management practices are in accordance with these Rules and are consistent with the information presented in the construction plans.

11. MAINTENANCE

All storm water quality facilities, including those constructed offsite per B.6 of §1-5.1 Part I, will require regular maintenance by the owner/developer or authorized representative to ensure they operate as designed and to prevent resuspension of previously captured particles. Necessary information, such as inspection/maintenance

frequencies, activities, and responsible individuals shall be documented in the Storm Water Quality Report or Storm Water Quality Checklist as applicable. In addition to regular maintenance, annual inspections must be performed for all Post-Construction BMPs by the owner/developer or authorized representative, including inspection and performance of any required maintenance in the late summer/early fall, prior to the start of the rainy season. A log of inspection and maintenance activities must be kept for a minimum of 5 (five) years and be made available to the City upon request.

For facilities that will be dedicated to the City, the City reserves the right to alter the maintenance plan to conform to its practices.

§1-5.2 PART II - WATER QUALITY DESIGN STANDARDS

A. VOLUME BASED STORM WATER QUALITY CONTROL FACILITIES

Volume based storm water quality facilities include Infiltration Basins, Infiltration Trenches, Subsurface Infiltration Systems, Dry Wells, Bioretention Basins, Permeable Pavement, Green Roofs, Vegetated Bio-Filters, Enhanced Swales, Detention Basins, and Sand Filters.

Volume based storm water quality facilities shall be sized as determined in B.6 of §1-5.1 Part I, Water Quality Criteria. The WQV is calculated as follows:

$$\boldsymbol{WQV = PCA \times 3630}$$

Where:
- WQV = water quality volume (cubic feet)
- P = design storm runoff depth (inches)
- C = volumetric runoff coefficient
- A = total drainage area (acres)

A design storm runoff depth of 1 inch shall be used. The volumetric runoff coefficient shall be calculated using the following equation as developed by EPA for smaller storms in urban areas:

$$\boldsymbol{C = 0.05 + 0.009I}$$

Where:
- C = volumetric runoff coefficient
- I = percent of impervious cover, expressed as a percentage

Infiltration Basin. An infiltration basin is a shallow impoundment with no outlet, where storm water runoff is stored and infiltrates through the basin invert and into the soil matrix. Infiltration Basins shall have a flat invert, interior side slopes (length per unit height) no steeper than 3:1 unless approved by a licensed professional engineer with geotechnical expertise, and at least 3 feet from the basin invert to the seasonally high groundwater table. The soil infiltration rate below the basin invert shall be at least 0.5 inches per hour, and drain completely in 48 hours.

Infiltration Trench. An infiltration trench is a rock-filled trench with no outlet, where storm water runoff is stored in the void space between the rocks and infiltrates through the bottom and into the soil matrix. Infiltration Trenches shall have no more than 6 inches of a top backfill layer, no more than 12 inches of a bottom sand layer, and 1.5-3.0 inch diameter trench rock. The soil infiltration rate below the trench invert shall be at least 0.5 inches per hour, the depth from the trench invert to the seasonally high groundwater table shall be at least 3 feet, and the trench shall completely drain in 48 hours. The depth of the infiltration trench shall not exceed the greater of the trench width and trench length to avoid classification as a Class V injection well.

Subsurface Infiltration System. A subsurface infiltration system is a rock (or alternative pre-manufactured material) storage bed below other surfaces such as parking lots, lawns and playfields for temporary storage and infiltration of runoff. In addition to applicable manufacturer's guidelines, the soil infiltration rate below the system invert shall be at least 0.5 inches per hour, the depth from the system invert to the seasonally high groundwater table shall be at least 3 feet, and the system shall completely drain in 48 hours. The depth of the subsurface infiltration system storage bed shall not exceed the greater of the storage bed's width and storage bed's length to avoid classification as a Class V injection well.

Dry Well. A dry well is a subsurface aggregate-filled or prefabricated perforated storage facility, where roof runoff is stored and infiltrates into the soil matrix. The soil infiltration rate below the dry well invert shall be at least 0.5 inches per hour, the depth from the dry well invert to the seasonally high groundwater table shall be at least 3 feet, and the dry well shall completely drain in 48 hours. If the dry well is aggregate-filled, 1.0-3.0 inch aggregate shall be used unless an alternative is approved by a licensed professional engineer with geotechnical expertise. The depth of the dry well shall not exceed the diameter to avoid classification as a Class V injection well.

Bioretention Basin. Sometimes referred to as a Rain Garden, a Bioretention Basin is an engineered shallow depression that collects and filters storm water runoff using conditioned planting soil beds and vegetation. The filtered runoff infiltrates through the basin invert and into the soil matrix. Bioretention Basins shall have a flat invert, interior side slopes (length per unit height) no steeper than 1:1 for single family residential installations and no steeper than 3:1 for all other installations unless approved by a licensed professional engineer with geotechnical expertise, and at least 3 feet from the basin invert to the seasonally high groundwater table. The ponding depth shall be no greater than 12 inches, the mulch thickness shall be 2-4 inches, and the planting soil depth shall be 2-4 feet. The soil infiltration rate below the basin invert shall be at least 0.5 inches per hour, and the basin shall drain completely in 48 hours.

Permeable Pavement. Sometimes referred to as pervious pavement or porous pavement, permeable pavement refers to any porous, load-bearing surface that allows for temporary rainwater storage in an underlying aggregate layer until it infiltrates into the soil matrix. It includes pervious concrete, porous asphalt, interlocking paver blocks, and reinforced turf and gravel filled grids. Permeable pavement shall have a reservoir layer no thicker than 3 feet and have at least 3 feet from the reservoir invert to the seasonally high groundwater table. The soil beneath the reservoir layer invert shall have an infiltration

rate of at least 0.5 inches per hour, and the reservoir layer shall drain completely in 48 hours.

Green Roof. Sometimes referred to as a Vegetated Roof or Eco-roof, a green roof is a roof that is entirely or partially covered with vegetation and soils for the purpose of filtering, absorbing, evapotranspirating, and retaining/ detaining the rain that falls upon it. Green roofs shall have a slope no greater than 20 percent, at least 2 inches of soil media, and at least 2 inches of drainage layer.

Vegetated Bio-Filter. Sometimes referred to as a Bioretention Filter, Stormwater Curb Extension, or Planter Box, a Vegetated Bio-Filter is an engineered shallow depression that collects and filters storm water runoff using conditioned planting soil beds and vegetation. The filtered runoff discharges through an underdrain system. Vegetated Bio-Filters shall have a relatively flat invert, the ponding depth shall be no greater than 12 inches, the mulch thickness shall be 2-4 inches, and the planting soil depth shall be 2-4 feet. The planting soil shall have a coefficient of permeability equal to at least 1.0 foot per day, and the WQV shall drain completely in 48 hours.

Enhanced Swale. Sometimes referred to as a Bioretention Swale or Dry Swale, an Enhanced Swale is a shallow linear channel with a planting bed and covered with turf or other surface material (other than mulch or plants). Runoff filters through a planting bed, is collected in an underdrain system, and discharged at the downstream end of the swale. Enhanced Swales shall have interior side slopes (length per unit height) no steeper than 3:1 unless approved by a licensed professional engineer with geotechnical expertise, a bottom width between 2-8 feet, and a longitudinal slope no greater than 2 percent without check dams or 5 percent with check dams. If used, check dams shall be no higher than 12 inches. The maximum ponding depth is 18 inches and the minimum media depth is 18 inches.

Detention Basin. Sometimes referred to as a Dry Extended Detention Basin, a detention basin is a shallow man-made impoundment intended to provide for the temporary storage of storm water runoff to allow particles to settle. It does not have a permanent pool and is designed to drain between storm events. Detention Basins shall have an invert sloped between 1-2 percent, interior side slopes (length per unit height) no steeper than 3:1 unless approved by a licensed professional engineer with geotechnical expertise, a minimum length to width ratio of 2 to 1, and a maximum depth of 8 feet. With outlets no smaller than 4 inches in diameter, the basin shall drain completely in 48 hours when full and 24-36 hours when half full.

Sand Filter. A sand filter is an open chambered structure that captures, temporarily stores, and treats storm water runoff by passing it through an engineered media (e.g., sand). Sand filter beds shall have at least 18 inches of sand with a coefficient of permeability of at least 3.5 feet per day, and shall drain completely in 48 hours.

B. FLOW BASED STORM WATER QUALITY CONTROL FACILITIES

Flow-through based storm water quality facilities include Vegetated Swales, Vegetated Filter Strips, and Manufactured Treatment Devices.

Flow-through based storm water quality facilities shall be sized for the Water Quality Flow Rate (WQF), which is calculated using the Rational Formula as follows:

$$\boldsymbol{WQF = CiA}$$

Where:
- WQF = water quality flow rate (cubic feet per second)
- C = runoff coefficient
- i = peak rainfall intensity (inches per hour)
- A = total drainage area (acres)

A peak rainfall intensity of 0.4 inches per hour shall be used. The runoff coefficient shall be determined from Table 4. The runoff coefficient shall be, at a minimum, the midpoint of the given range of values. The higher value shall be used if soil conditions indicate that pervious areas will have little infiltration/interception potential.

For drainage areas containing multiple land uses the following formula may be used to compute a composite weighted runoff coefficient:

$$\boldsymbol{C_c = \left(\sum_{i=1}^{n} C_i A_i\right) \Big/ A_t}$$

Where:
- C_c = composite weighted runoff coefficient
- $C_{1,2,\ldots n}$ = runoff coefficient for each land use cover type
- $A_{1,2,\ldots n}$ = drainage area of each land use cover type (acres)
- A_t = total drainage area (acres)

The calculated WQF for Vegetated Swales and Vegetated Filter Strips may be reduced by 25% if the soil beneath the BMP is classified as Hydrologic Soils Group (HSG) "A" or "B", as reported by the USDA Natural Resources Conservation Service (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm), or if the soil beneath the BMP is amended by incorporating 6 inches of compost/amendments and tilled up to 8 inches.

Vegetated Swale. Sometimes referred to as a Grass Swale, Grass Channel, or Biofiltration Swale, a vegetated swale is a broad shallow earthen channel vegetated with erosion resistant and flood tolerant grasses. Runoff typically enters the swale at one end and exits at the other end. Vegetated Swales shall have interior side slopes (length per unit height) no steeper than 3:1 unless approved by a licensed professional engineer with geotechnical expertise, a bottom width no greater than 10 feet, and a water depth no greater than 4 inches. The velocity in the swale shall not exceed 1 foot per second, and the hydraulic residence time shall be at least 7 minutes.

Vegetated Buffer Strip. Sometimes referred to as a Vegetated Filter Strip or Biofiltration Strip, a vegetated buffer strip is a grassy slope vegetated with turf grass that is designed to accommodate sheet flow. They may remove pollutants by vegetative filtration. Vegetated Buffer Strips shall have a length (in the direction of flow) no less than 15 feet, the depth of flow shall not exceed 1 inch, and the velocity shall not exceed 1 foot per

second. The flow length of the tributary area discharging onto the strip shall not exceed 75 feet.

Manufactured Treatment Device. Sometimes referred to as hydrodynamic or vortex separators, a manufactured treatment device is a proprietary water quality structure utilizing settling, filtration, adsorptive/absorptive materials, vortex separation, vegetative components, or other appropriate technology to remove pollutants from storm water runoff. These devices must provide a TSS removal rate of 80%, verified by a Technology Acceptance and Reciprocity Partnership (TARP) state or other third party testing organization, provided that such verification is conducted in accordance with the protocol "Stormwater Best Management Practices Demonstration Tier II Protocol for Interstate Reciprocity" (which may be found at http://www.dep.state.pa.us/dep/deputate/pollprev/techservices/tarp/).

C. AREA BASED STORM WATER QUALITY CONTROL FACILITIES

Area based storm water quality facilities include Downspout Disconnection.

Downspout Disconnection. Sometimes referred to as Rooftop Disconnection or Downspout Dispersion, is the redirection of roof runoff to a vegetated area in a dispersed manner. Downspout disconnection facilities shall be sized such that the size of the vegetated area receiving the roof runoff is at least 10% of the size of the roof area that drains to the downspout, or the flow path of the vegetated area receiving the roof runoff is at least as long as the flow path of the roof area that drains to the downspout.

D. DEMAND BASED STORM WATER QUALITY CONTROL FACILITIES

Demand based storm water quality facilities include Harvesting / Reuse.

Harvesting/Reuse. Sometimes referred to as Capture/Reuse or Rainwater Harvesting, is the collection and temporary storage of roof runoff in rain barrels or cisterns for subsequent non-potable outdoor use (landscape irrigation, vehicle washing). Harvesting / Reuse facilities shall be sized such that at least 80% of the total annual runoff is captured, and at least 80% of the total annual reuse demand is met.

E. INFEASIBILITY CRITERIA

Table 5 lists exemption criteria for Low Impact Development (LID).

[Eff: June 1, 2013] (Auth: Sec 14-12.31, ROH) (Imp: Sec14-12.31, ROH)

TABLE 4: RUNOFF COEFFICIENTS FOR WATER QUALITY FLOW CALCULATIONS

Type of Drainage Area	Runoff Coefficient
Business	
Downtown areas	0.70 – 0.95
Neighborhood areas	0.50 – 0.70
Residential	
Single-family areas	0.30 – 0.50
Multi-units, detached	0.40 – 0.60
Multi-units, attached	0.60 – 0.75
Suburban	0.25 – 0.40
Apartment dwelling areas	0.50 – 0.70
Industrial	
Light areas	0.50 – 0.80
Heavy areas	0.60 – 0.90
Parks, cemeteries	0.10 – 0.25
Playgrounds	0.20 – 0.40
Railroad yards	0.20 – 0.35
Unimproved areas	0.10 – 0.30
Lawns	
Sandy soil, flat, ≤ 2%	0.05 – 0.10
Sandy soil, average 2-7%	0.10 – 0.15
Sandy soil, steep ≥ 7%	0.15 – 0.20
Heavy soil, flat, ≤ 2%	0.13 – 0.17
Heavy soil, average 2-7%	0.18 – 0.22
Heavy soil, steep ≥ 7%	0.25 – 0.35
Streets	
Asphaltic	0.70 – 0.95
Concrete	0.70 – 0.95
Brick	0.75 – 0.85
Drives and walks	0.75 – 0.95
Roofs	0.75 – 0.95

TABLE 5: EXEMPTION CRITERIA FOR LOW IMPACT DEVELOPMENT

Exemption Criteria	Infiltration Basin	Infiltration Trench	Subsurface Infiltration	Dry Well	Bioretention Basin	Permeable Pavement
Soils beneath basin invert have measured infiltration rates less than 0.5 in/hr	●	●	●	●	●	●
Unable to maintain a distance of at least 3 ft from BMP invert to seasonally high groundwater table	●	●	●	●	●	●
Site has known man-made plumes or contaminated soils	●	●	●	●	●	●
Site has high potential for concentrated pollutant/chemical spills	●	●	●	●	●	●
Site is up-gradient of ephemeral streams (i.e. habitat type change downstream)	●	●	●	●	●	●
Site is up-gradient of known shallow landslide-prone area	●	●	●	●	●	●
Unable to maintain a distance of at least 50 ft to the nearest groundwater well used for drinking water	●	●	●	●	●	●
Unable to maintain a distance of at least 35 ft to the nearest septic system	●	●	●	●	●	●
Unable to maintain a distance of at least 20 ft to the nearest building foundation	●	●	●		●	
Unable to maintain a distance of at least 10 ft to the nearest building foundation				●		
Unable to maintain a distance of at least 100 ft to the nearest down-gradient building foundation	●	●	●	●	●	
Unable to maintain a distance of at least 10 ft to the nearest property line	●	●	●	●	●	
Unable to divert flows in excess of WQDS around BMP, and unable to create safe overflow mechanism for flows in excess of WQDS	●	●	●		●	
Excavation would disturb iwi kupuna or other archaeological resources	●	●	●		●	
Site has high potential for oil and/or grease spills						●
Site has high potential to receive sand and/or sediment loads						●
Unable to maintain a pavement slope no greater than 5%						●
Pavement would be above a utility vault						●
Pavement is expected to receive more than 1,000 average daily trips						●
Other justification for an exemption proposed by the developer/agent and is acceptable to the City	●	●	●	●	●	●

TABLE 5: EXEMPTION CRITERIA FOR LOW IMPACT DEVELOPMENT (continued)

Exemption Criteria	Vegetated Bio-Filter	Green Roof	Enhanced Swale	Downspout Disconnect	Vegetated Swale	Vegetated Filter Strip	Tree Box Filter
Unable to divert flows in excess of WQDS around BMP, and unable to create safe overflow mechanism for flows in excess of WQDS	●		●		●	●	
Excavation would disturb iwi kupuna or other archaeological resources	●		●		●	●	●
Invert of underdrain layer is below seasonally high groundwater table	●		●				
Site does not receive enough sunlight to support vegetation	●				●	●	
Site lacks sufficient hydraulic head to support BMP operation by gravity	●		●				●
Roof is for a single family residential dwelling		●					
Space is unavailable due to renewable energy, electrical, and mechanical systems		●					
Slope on roof exceeds 20% (11 degrees)		●					
Slope of receiving vegetated area exceeds 5%				●			
Diverted runoff drains within 10 feet of a retaining wall				●			
Diverted runoff drains within 10 feet of property line				●			
Concentrated flow cannot be established naturally					●		
Sheet flow cannot be established naturally						●	
Entrance at surface not possible							●
Residential and no planting strip							●
No curb and gutter							●
Other justification for an exemption proposed by the developer/agent and is acceptable to the City	●	●	●	●	●	●	●

● denotes that the BMP is considered infeasible if the exemption criteria is applicable

[Eff: June 1, 2013] (Auth: Sec 14-12.31, ROH) (Imp: Sec14-12.31, ROH)

§1-6 REPEAL

The City and County of Honolulu's Storm Drainage Standards, revised printing dated May 1988, is repealed in its entirety.

[Eff: JAN 01 2000 *] (Auth: Sec 14-12.31, ROH) (Imp: Sec14-12.31, ROH)*

DEPARTMENT OF PLANNING AND PERMITTING
CITY AND COUNTY OF HONOLULU

These rules were adopted on October 4, 1999, following public hearing held on July 23, 1999, after public notice was given on June 21, 1999, in the Hawaii State and County Public Notices, Honolulu City and County.

These rules shall take effect on January 1, 2000.

JAN NAOE SULLIVAN
Director
Department of Planning and Permitting

APPROVED:

JEREMY HARRIS ACTING MAYOR
Mayor
City and County of Honolulu

Dated: October 18, 1999

APPROVED AS TO FORM
AND LEGALITY:

Deputy Corporation Counsel

FILED:

Given unto my hand and affixed with the Seal of the City and County of Honolulu this 19th day of October, 1999.

GENEVIEVE G. WONG, City Clerk

DEPARTMENT OF PLANNING AND PERMITTING
CITY AND COUNTY OF HONOLULU

These amendments to the rules were adopted on November 27, 2010, following a public hearing held on November 17, 2010, after public notice was given on October 14, 2010, in the Hawaii State and County Public Notices, Honolulu City and County.

These amendments to the rules shall take effect on May 1, 2011.

DAVID K. TANOUE
Director
Department of Planning and Permitting

APPROVED:

PETER B. CARLISLE
Mayor
City and County of Honolulu

Dated: MAR 16 2011

APPROVED AS TO FORM
AND LEGALITY:

Corporation Counsel

FILED

Given unto my hand and affixed with the
Seal of the City and County of Honolulu this
29 day of March, 2011.

BERNICE K. N. MAU, City Clerk

These amendments to the rules were adopted on December 12, 2012, following a public hearing held on November 27, 2012, after public notice was given on October 26, 2012, in the Hawaii State and County Public Notices, Honolulu City and County.

These amendments to the rules shall take effect on June 1, 2013.

JIRO A. SUMADA
Acting Director
Department of Planning and Permitting

APPROVED:

PETER B. CARLISLE
Mayor
City and County of Honolulu

Dated: 12/28/12

APPROVED AS TO FORM
AND LEGALITY:

Deputy Corporation Counsel

FILED:

Given unto my hand and affixed with the
Seal of the City and County of Honolulu this
02 day of January, 2013.

BERNICE K.N. MAU, City Clerk

Examples Illustrating Applications of Rules Relating to Storm Drainage Standards

FINAL

December 2012

By:
City and County of Honolulu
Department of Planning and Permitting

E Mālama I Ka Wai Ola
Protect our waters...
FOR LIFE

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A. Introduction

This booklet contains examples, which are intended to illustrate some of the more important aspects of the application of the "Rules Relating to Storm Drainage Standards" of the Department of Planning and Permitting, City and County of Honolulu. They are not intended to be complete examples of what must be submitted to the City and County of Honolulu. All designs must be completed per City and County requirements.

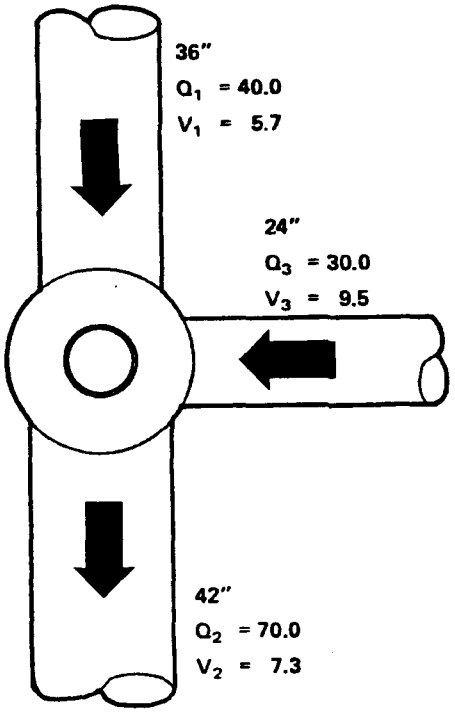
The information is brief and subject to change. The user is encouraged and invited to consult with the appropriate staff of the Department of Planning and Permitting for discussions on site-specific best management practices ("BMPs") and to consult with design guidance that has been developed by other agencies on the design of BMPs. For a list of other design guidance manuals, please consult the City and County of Honolulu, Department of Planning and Permitting.

B. Flood Control Design

Example #1: Analysis & Solution for Manhole Losses

NOTE: in lieu of the following analysis, an analysis based upon the Bernoulli's Energy Theorem, such as the pressure-momentum method, will be acceptable.

GIVEN: Pipe size, Q, pipe flowing full, velocity and direction of flow.



LEGEND

Q_1 = Upstream Volume, cfs
Q_2 = Downstream Volume, cfs
Q_3 = Incoming Volume, cfs
V_1 = Upstream Velocity, fps
V_2 = Downstream Velocity, fps
V_3 = Upstream Branch Velocity, fps
h = Head Loss, in ft.

SOLUTION

"A" LOSS (ENTRANCE & EXIT LOSS)

1. Determine higher velocity between **V_1** and **V_2**
2. Use Curve **"A"** or **"C"** depending on pipe size and determine **h_A** (Ex. Prob. **h_A** = 0.15)

"B" LOSS (VELOCITY HEAD LOSS)

1. Use Curve **"B"** and determine h_V for V_1 and V_2

   a. If V_2 is lower than V_1, then h_B shall be **0**
   b. If V_2 is higher than V_1, then h_B shall be $h_{B_2} - h_{B_1}$

   Ex. Prob. $h_{B_2} = 0.83$ and $h_{B_1} = 0.50$
   $h_B = 0.83 - 0.50 = 0.33$

"C" LOSS (DIRECTIONAL CHANGE LOSS)

1. Use worst case and determine degree of bend.
2. With higher V_1 or V_2, use Curve **"C"** and determine head loss (**h**).

   a. For **0°** to **22½°** bends, h_C shall be **0.67** times **h.**
   b. For **22½°** to **45°** bends, h_C shall be **1.00** times **h.**
   c. For **45°** to **90°** bends, h_C shall be **2.00** times **h.**

   Ex. Prob. $h = 0.15$
   $h_C = 2 \times 0.15 = 0.30$

"D" LOSS (LOSS DUE TO INCOMING VOLUME)

1. Add total branch volume and determine ratio of branch volume to upstream volume.
2. Use appropriate curve and determine h_D with higher V_1 or V_3.

   Ex. Prob. $Q_3/Q_1 = 30/40 = 75\%$,
   $h_D = 0.56$

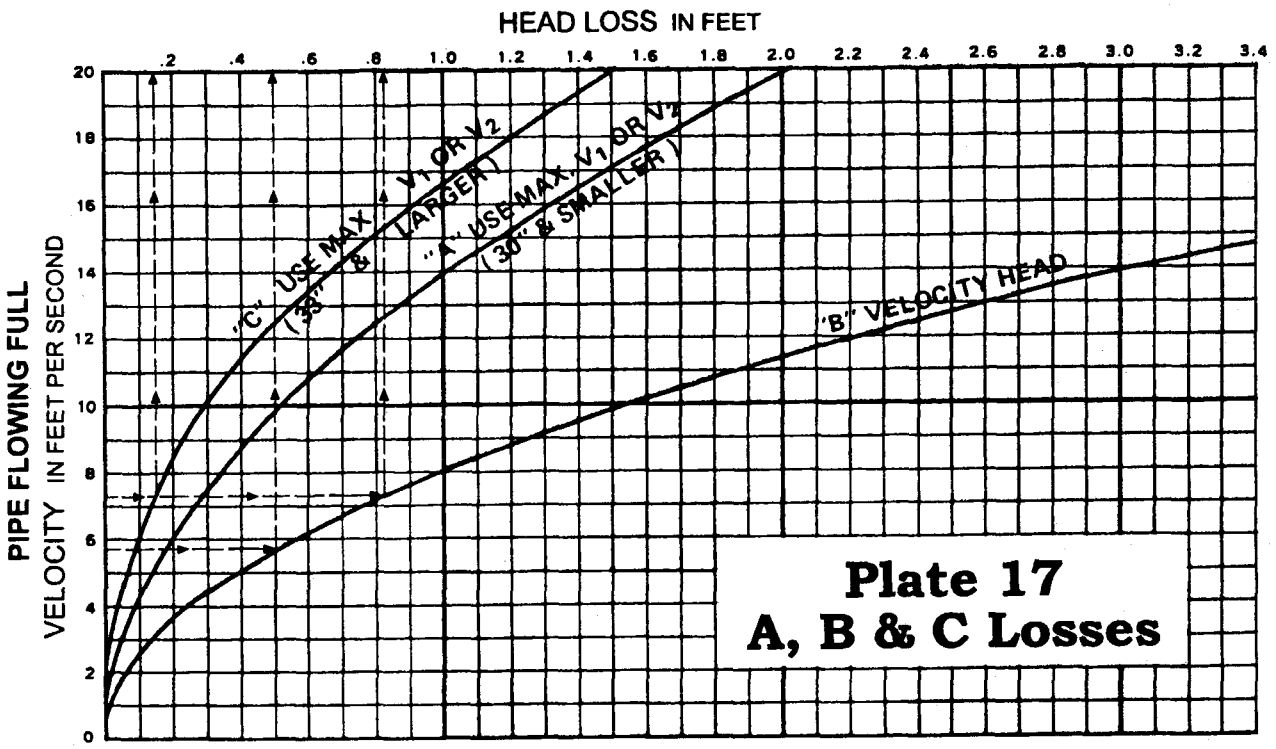
TOTAL LOSS:

1. Add h_A, h_B, h_C, and h_D

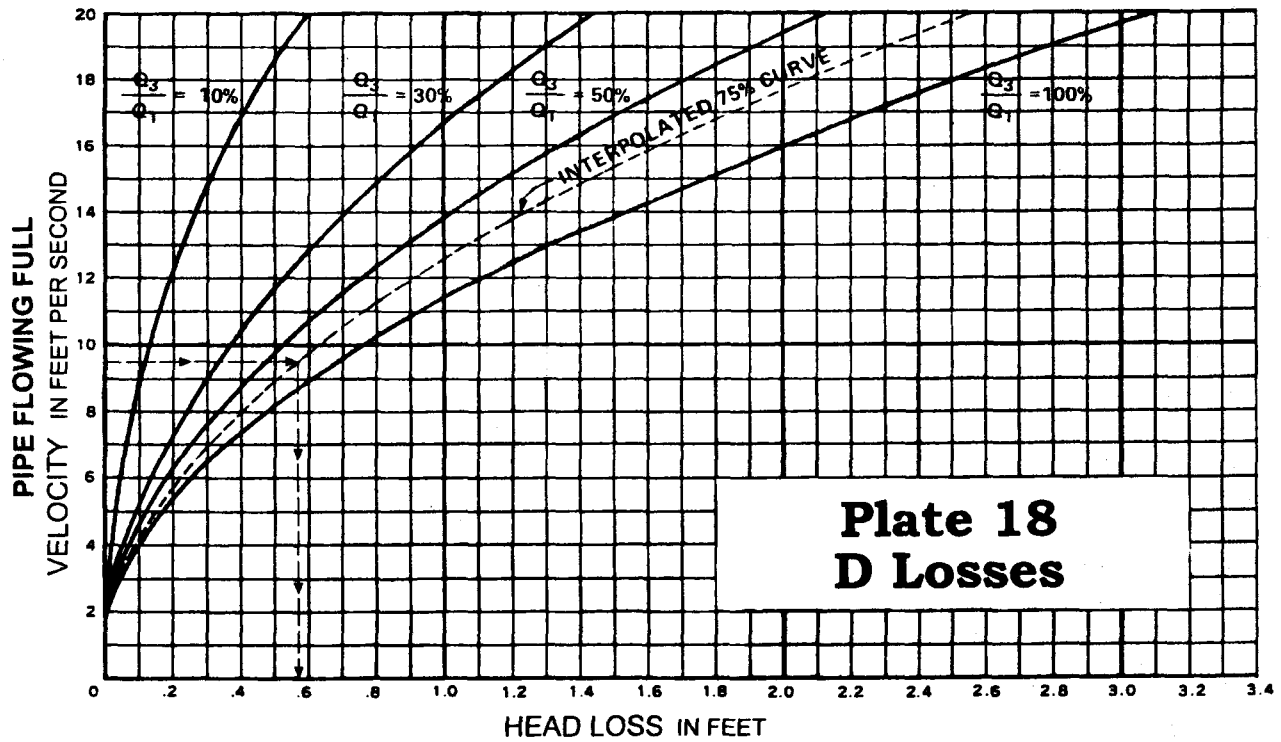
   Ex. Prob. $h_T = 0.15 + 0.33 + 0.30 + 0.56$
   $h_T = 1.34$

Losses

A		= 0.15
B	0.83 – 0.50	= 0.33
C	2(0.15)	= 0.30
D		= 0.56
Total Loss		= **1.34 ft.**



Head Losses in Manholes



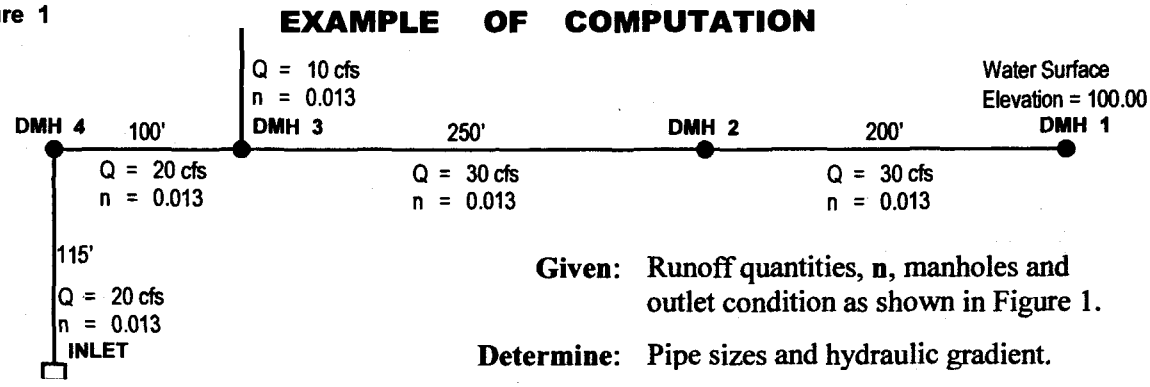
SOURCE: BALTIMORE COUNTY DEPARTMENT OF PUBLIC WORKS

C. Flood Control Design

Example #2: Pipe System Analysis

Figure 1

EXAMPLE OF COMPUTATION



Given: Runoff quantities, **n**, manholes and outlet condition as shown in Figure 1.

Determine: Pipe sizes and hydraulic gradient.

SOLUTION

USE PLATES 8 TO 20 AS AID TO ANALYSIS.

Make preliminary determination of pipe sizes for the data given using pipe flow charts. This is shown in Figure 2.

Using the pipe sizes and slopes of pipes as determined above, compute hydraulic gradient for the system. This is shown in Figure 3.

1. Controlling grade at DMH1 is 100.00 as shown in Figure 3.

   Study conditions of flow between manholes or inlets to determine if entrance control or losses govern hydraulic gradient.

2. With the selected pipe size between DMH 1 and DMH 2, 24" diameter pipe at S = 0.010, compute the head loss in the pipe by the formula $h = SL$ or $h_f = S_fL$, whichever controls.

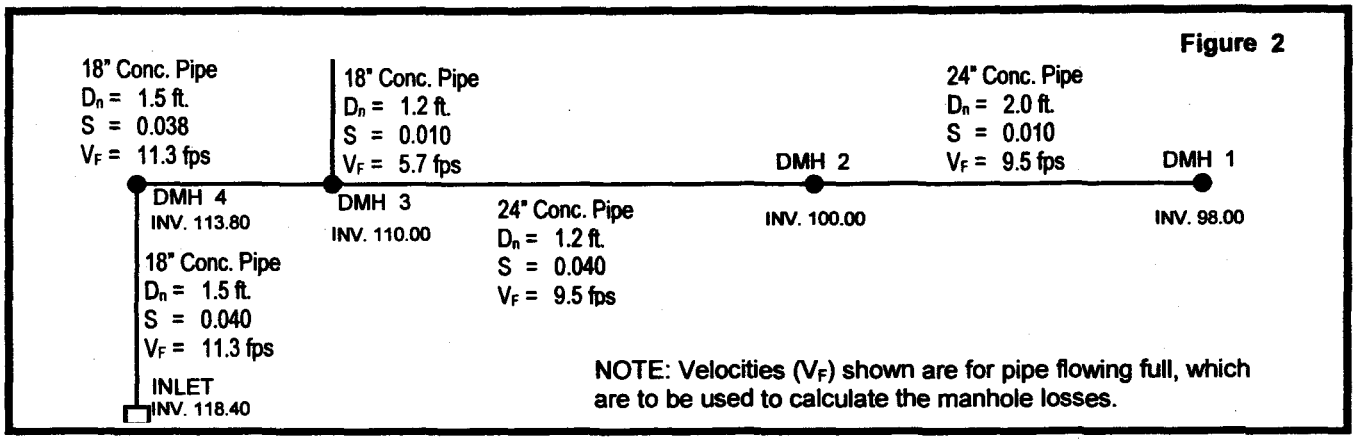
   h = elevation head loss
   h_f = friction head loss
   S = slope of pipe
   S_f = friction slope (used when pipe flowing full)
   L = length of the pipe or channel

   Since the pipe is flowing full, as determined by the pipe flow chart using 24" diameter, the friction slope 0.018 must be used. The head loss in the pipe is:

   $$h_f = S_fL$$
   $$h_f = (0.018)\,(200) = 3.60 \text{ feet}$$

The downstream hydraulic gradient at DMH 2 is equal to the controlling grade at DMH 1 plus the head loss or

100.00 + 3.60 = 103.60

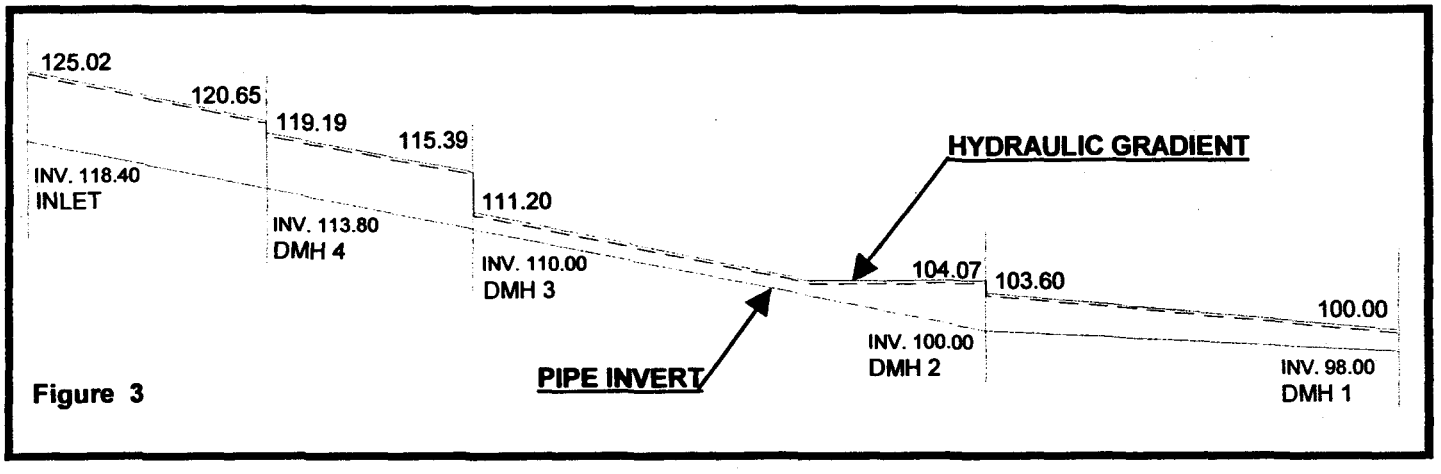


3. Since the pipe is flowing full, and there are no bends or drops, compute the upstream hydraulic gradient at DMH 2 by adding the manhole losses to the downstream hydraulic gradient at DMH 2. These values are obtained from charts on manhole losses. From the charts:

A = 0.47
B = 0.00 (since the velocities are equal)
C = 0.00
D = 0.00
0.47 ft. (Total DMH losses)

The upstream hydraulic gradient at DMH 2 is:

103.60 + 0.47 = 104.07



4. With the selected pipe size between DMH 2 and DMH 3, 24" diameter pipe at S = 0.040, compute the head loss elevation in the pipe:

   h = SL
   h = (0.040) (250) = 10.00 feet

   Since the pipe is not flowing full as determined by the pipe flow chart, the elevation head loss and the normal depth must be added to the invert of DMH 2. Therefore, the downstream hydraulic gradient at DMH 3 is

   100.00 + 10.00 + 1.20 = 111.20

5. Compute the upstream hydraulic gradient at DMH 3 by adding to the invert elevation the manhole losses and entrance control losses for open channel flow. Only manhole losses "C" and "D" need be considered.

   From the charts:

   C = 2(0.40) = 0.80 (90° Bend)
   D = 0.69
   1.49 ft. (Total DMH losses)

   Entrance control loss for Q = 30 cfs, D = 24" is:

   H/D = 1.95, From Plate 19
   H = 3.90 feet

   The upstream hydraulic gradient at DMH 3 is:

   110.00 + 1.49 + 3.90 = 115.39

6. With selected pipe size between DMH 3 and DMH 4, 18" diameter pipe at S = 0.038, compute the head loss in the pipe:

   $h_f = S_f L$
   $h_f = (0.038)(100) = 3.80$ **feet**

   The downstream hydraulic gradient at DMH 4 is:

   115.39 + 3.80 = 119.19

   since the tailwater condition of the pipe is submerged.

7. Since there is a bend greater than 10° at DMH 4, compare losses and use the higher HGL.

A = 0.66	**C = 0.80**
B = 0.00	**D = 0.00**
C = 0.80 (0.40 x 2)	**0.80**
D = 0.00	
1.46	

Entrance control loss for Q = 20 cfs, D = 18" is:

H/D = 3.0 From Plate 19
H = 4.50
119.19 + 1.46 = 120.65 > 113.80 + 4.50 + 0.80 = 119.10

The upstream hydraulic gradient at DMH 4 is 120.65.

8. Since the pipe is flowing full, the downstream hydraulic gradient at the inlet is determined by friction loss in the length of pipe.

$h_f = S_f L$
$h_f = (0.038)(115) = 4.37$ **feet**
120.65 + 4.37 = 125.02

Entrance control loss at the inlet for Q = 20 cfs, D = 18" is:

H/D = 3.0 From Plate 19
H = 4.50
118.40 + 4.50 = 122.90

Since the hydraulic gradient is higher, the top of headwall must be at least 125.02 + 1 foot = 126.02.

Adjust pipe sizes if warranted by the hydraulic gradient as computed above.

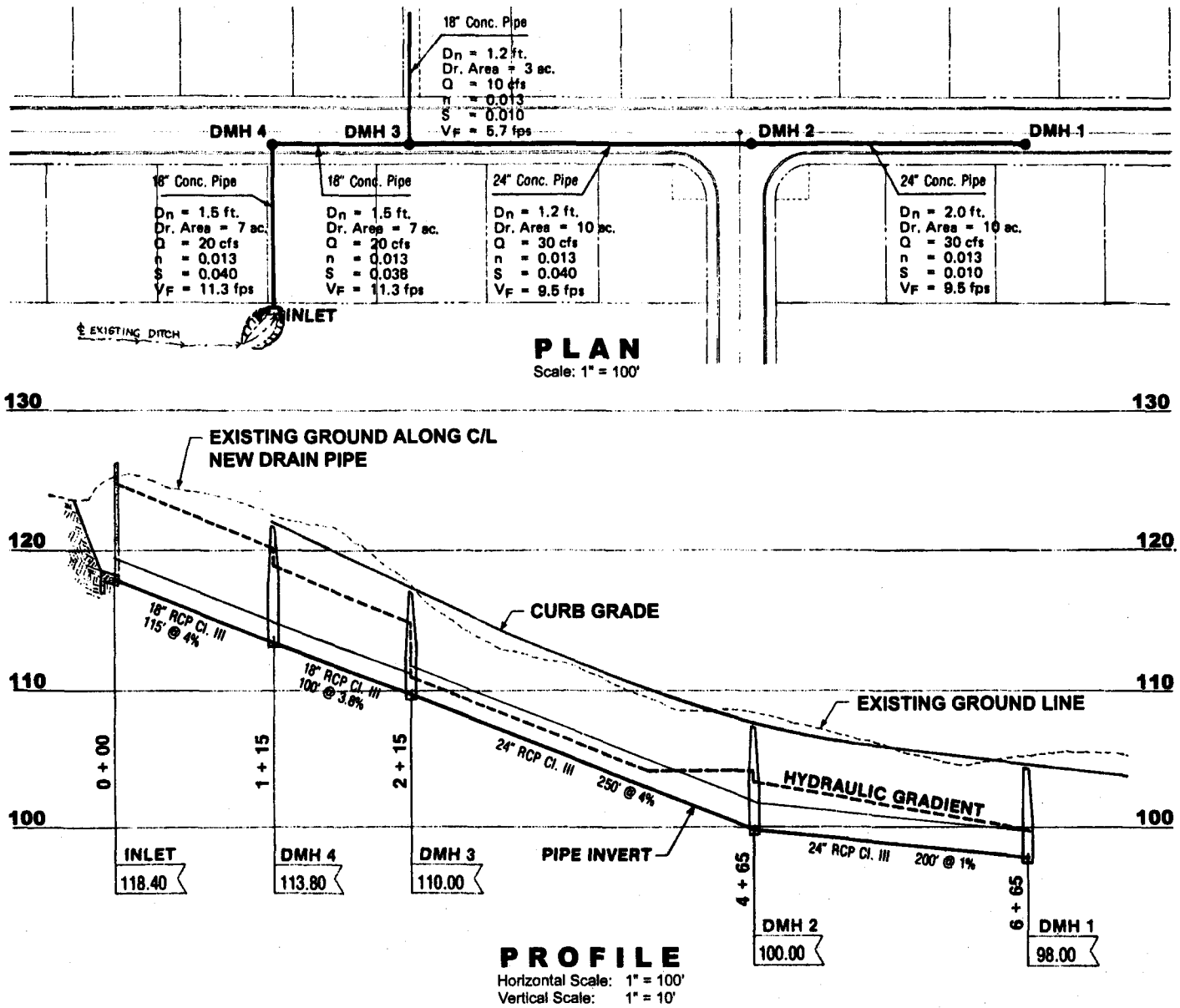
PROJECT: XYZ Subdivision
DRAIN LINE A

DRAINAGE DESIGN DATA

DATE: 10/28/85
COMPUTED BY: TM
CHECKED BY: LW

Inlet	T_c	C	I / i	Drain Area	Q Runoff / Accum. Q	d_n	V_F	Size of Pipe	Length of Seg.	S_f	S	Invert Elevation	H_f	Manhole Losses A	Manhole Losses B	Manhole Losses C	Manhole Losses D	Ent. Cont.	Hydraulic Grade Elev. Down	Hydraulic Grade Elev. Up	Finished Grade Elevation	Remark
DMH 1												98.00								100.00		
				10	30	2.0	9.5	24	200	.018	.010		3.60						103.60			
DMH2												100.00		.47	0	0	0	3.80		104.07		
				10	30	1.2	9.5	24	250	.018	.040		4.50						111.20			
DMH3												110.00		.66	0	.80	.69	3.90		115.39		
				7	20	1.5	11.3	18	100	.038	.038		3.80						119.19			
DMH 4												113.80		.66	0	.80	0	4.35		120.65		
				7	20	1.5	11.3	18	115	.038	.040		4.37						125.02			
INLET												118.40										

Suggested layout of tabulated computation form for DRAINAGE DESIGN DATA to be submitted for approval



Drainage Plan Showing Design Data to be Submitted on Drawing

EXAMPLE 1, 6.22 ACRE RESIDENTIAL DEVELOPMENT

A 6.22-acre site will be developed for the future construction of 22 single family residential houses. The site is divided into two drainage areas, as shown in Figure 1.

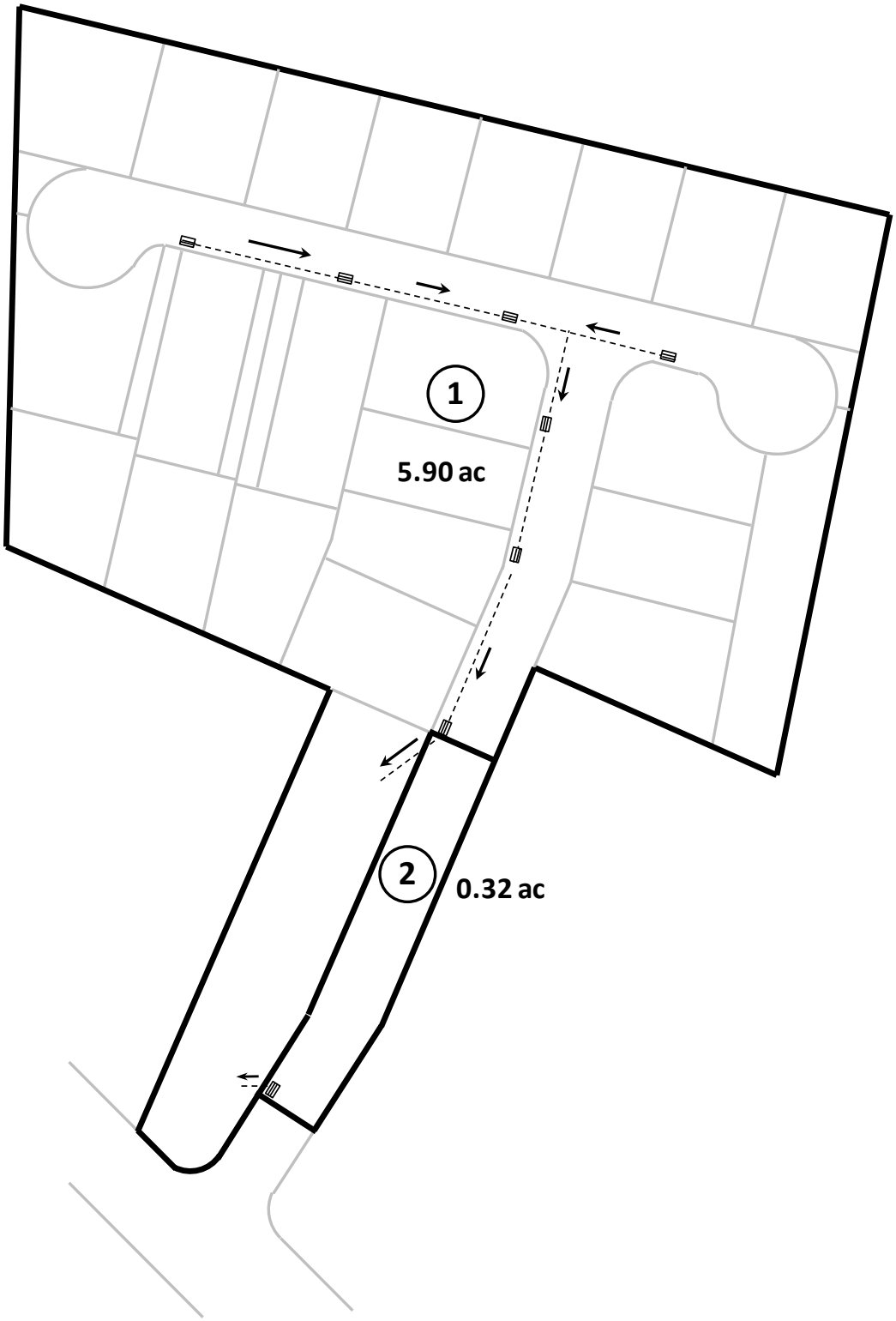


FIGURE1: EXAMPLE 1 DEVELOPMENT PLAN

Based on the size, the project meets the criteria of a Priority A1 new development project and requires preparation of a storm water quality report (SWQR). As specified in Section B.6 of §1-5.1 Part I, water quality criteria shall be achieved with LID Site Design, Source Control, LID Retention, LID Biofiltration (if LID Retention is infeasible), and Alternative Compliance (if LID Retention and LID Biofiltration are infeasible). Implementation of each of these five elements for this example is presented below.

A. LID SITE DESIGN

The *City and County of Honolulu Storm Water BMP Guide* (*BMP Guide*) identifies 5 LID Site Design Strategies for new development and redevelopment projects. Those that are applicable to this project will be implemented.

B. SOURCE CONTROL

The *BMP Guide* identifies 12 Source Control BMPs for new development and redevelopment projects. Those that are applicable to this project will be implemented.

C. LID RETENTION

Assume for purposes of this example that retention is feasible. Although many, if not all, of the 7 LID Retention BMPs identified in the *BMP Guide* may be implemented, details for an Infiltration Basin are presented below.

Assume that a single Infiltration Basin will be used for both drainage areas (see Figure 2). The sizing of the Infiltration Basin is accomplished using the City's BMP sizing worksheet, which is consistent with the Step-by-Step Sizing Procedure provided in the *BMP Guide*. The sizing worksheet is presented in Figure 3, and the calculations are summarized as follows:

1. Water Quality Volume (WQV). For purposes of this example, assume 70% impervious cover. Using a 1-inch design storm depth, the WQV for 6.22 acres is calculated to be 15,353 cubic feet.
2. Maximum allowable storage depth. For purposes of this example, assume a soil infiltration rate of 1.5 in/hr and an infiltration rate safety factor of 2. Using a drawdown time of 48 hours, the maximum allowable storage depth is calculated to be 3.0 feet.
3. Design storage depth. The ponding depth is set to 3 feet, which is equal to or less than the maximum allowable storage depth.
4. Basin invert footprint. Using a basin fill time of 2 hours (industry accepted practice), the required invert surface area is calculated to be 4,913 square feet.
5. BMP area requirements. The invert width is set to 25 feet, and the invert length is calculated to be 196.5 feet. Using a side slope of 3:1 and a freeboard depth of 1 foot (actual free board requirements must be determined for flood design storm), the top width and top length are calculated to be 49 feet and 220.5 feet, respectively. The total area, excluding pretreatment, is calculated to be 10,806 square feet.

The BMP layout and BMP details are presented in Figures 2 and 4, respectively. The Infiltration Basin will have two inlet points; one at the eastern end capturing 5.90 acres of roadway and residential lot runoff, and one at the western end capturing 0.32 acres of roadway runoff.

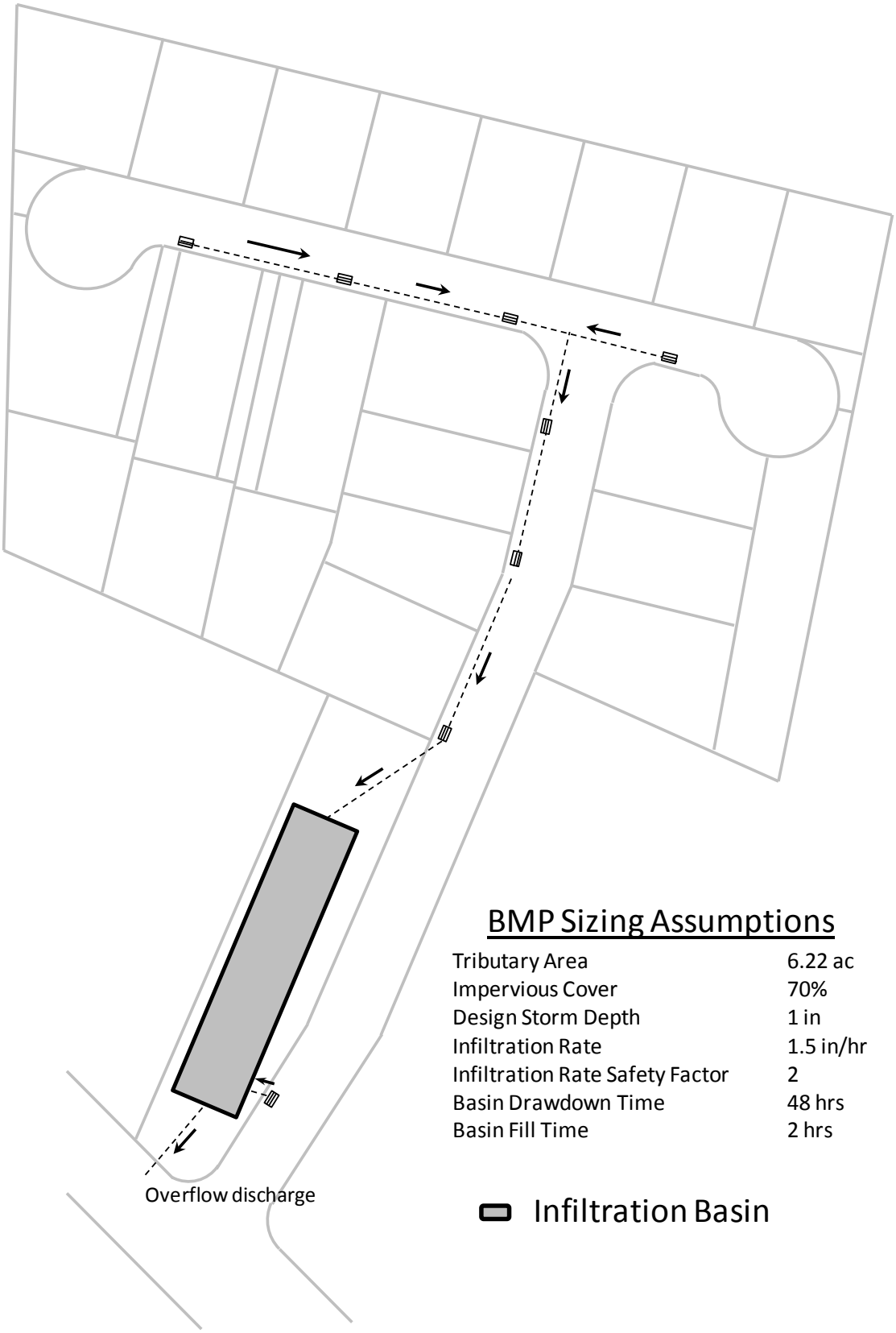


FIGURE 2: BMP LAYOUT, RETENTION

BMP Sizing Worksheet: Infiltration Basin

Project: 6.22 ac Residential Development **Date:** July 2012

1. Water Quality Volume		
a. BMP Tributary Drainage Area, **A**	6.22	ac
b. % Impervious Area, **I**	70	%
c. Water Quality Design Storm Depth, **P**	1.0	in
d. Volumetric Runoff Coefficient, **C**	0.68	
e. Water Quality Volume, **WQV**	15,353	cu-ft
2. Maximum Storage Depth		
a. Soil Infiltration Rate, **k** (0.5 min)	1.5	in/hr
b. Infiltration Rate Safety Factor (2 - 5), $\mathbf{F_s}$	2	
c. Drawdown Time, **t**	48	hrs
d. Max. Storage Depth, $\mathbf{d_{max}}$	3.0	ft
3. Design Storage Depth		
a. Ponding Depth, $\mathbf{d_p}$	3.00	ft
4. Basin Invert Footprint		
b. Reservoir Fill Time, **T**	2	hrs
c. Min. Bottom Surface Area, $\mathbf{A_b}$	4,913	sq-ft
5. BMP Area Requirements		
a. Side Slopes (length per unit height), **z** (3.0 min)	3	
b. Freeboard, **f** (1.0 min)	1	ft
c. Invert Width, $\mathbf{w_b}$	25.0	ft
d. Invert Length, $\mathbf{l_b}$	196.5	ft
e. Top Width, $\mathbf{w_t}$	49.0	ft
f. Top Length, $\mathbf{l_t}$	220.5	ft
g. Min. Top Surface Area excluding pretreatment, $\mathbf{A_{BMP}}$	10,806	sq-ft

FIGURE 3: INFILTRATION BASIN SIZING WORKSHEET

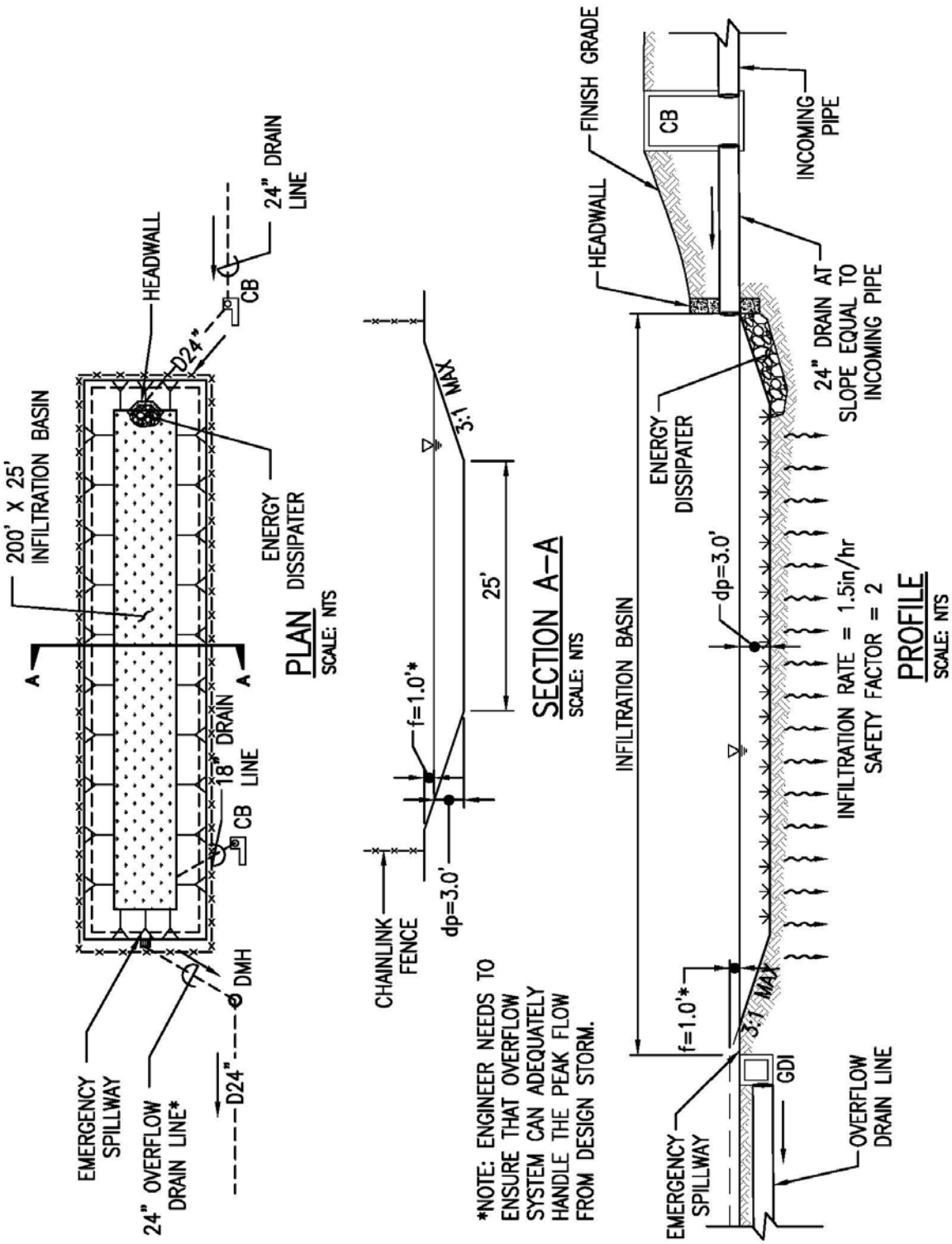


FIGURE 4: BMP DETAILS, RETENTION

D. LID BIOFILTRATION

For purposes of this example, assume that retention is determined to be infeasible, but biofiltration is feasible. Of the 7 LID Biofiltration BMPs identified in the *BMP Guide*, the water quality criteria may be met by biofiltering the runoff from both drainage areas with a Vegetated Swale. See Figure 5 for the proposed layout.

The sizing of the Vegetated Swale is accomplished using the City's BMP sizing worksheet, which is consistent with the Step-by-Step Sizing Procedure provided in the *BMP Guide*. The sizing worksheet is presented in Figure 6, and the calculations are summarized as follows:

1. Water Quality Flow Rate (WQF). For purposes of this example, assume a weighted runoff coefficient of 0.7. Using a rainfall intensity of 0.4 in/hr, the WQF for 6.22 acres is calculated to be 1.74 cfs.
2. Swale geometry. The bottom width, depth of flow, side slope, longitudinal slope, and Manning's roughness coefficient are set to 7 feet, 4 inches, 3, 4%, and 0.20, respectively.
3. Swale hydraulic capacity. Using the dimensions from step 2, the cross sectional area, wetted perimeter, and hydraulic radius are calculated to be 2.67 sq-ft, 9.11 ft, and 0.29 ft. The design flow rate is calculated to be 1.75 cfs, which is equal to or greater than the WQF.
4. Design flow velocity. The design flow velocity is calculated to be 0.66 feet per second, which is less than the minimum allowed velocity of 1 foot per second.
5. Swale length. Using a hydraulic residence time of 7 minutes, the swale length is calculated to be 276 feet.
6. BMP area requirements. Using 6 inches of freeboard, the required total surface area is calculated to be 3,310 sq-ft. At 276 feet long, this equates to a top width of 12 feet.

The vegetated swale does not require pretreatment, as specified in the *BMP Guide*. The BMP details for the vegetated swale are presented in Figure 7.

E. ALTERNATIVE COMPLIANCE

If LID Retention and LID Biofiltration are determined to be infeasible, alternative compliance would be required. Options include treating the runoff with Other Treatment Control BMPs identified in the BMP Guide (detention basin, manufactured treatment device, sand filter, etc.).

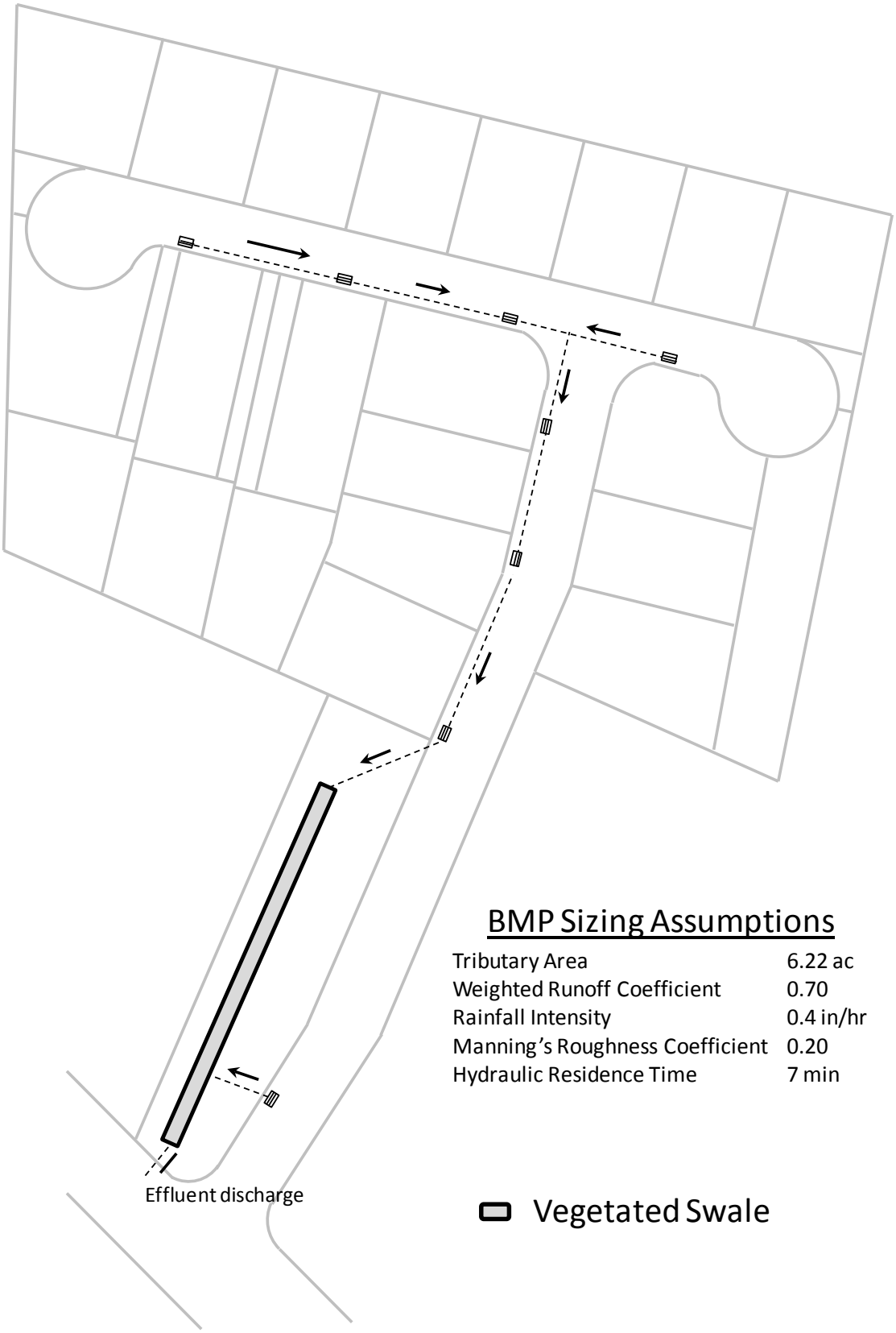


FIGURE 5: BMP LAYOUT, BIOFILTRATION

BMP Sizing Worksheet: Vegetated Swale

Project: 6.22 ac Residential Development **Date:** July 2012

1. Water Quality Flow Rate		
a. BMP Tributary Drainage Area, **A**	6.22	ac
b. Weighted Runoff Coefficient, **C**	0.7	
c. Rainfall Intensity, **i**	0.4	in/hr
d. Water Quality Flow Rate, **WQF**	1.74	cfs
2. Swale Geometry		
a. Bottom Width, **b** (10.0 ft max)	7.00	ft
b. Flow Depth, **y** (4.0 in max)	4.0	in
c. Side Slopes (length per unit height), **z** (3.0 max)	3	ft/ft
d. Longitudinal Slope, **s**	4.0	%
e. Manning's Roughness Coefficient, **n**	0.20	
3. Swale Hydraulic Capacity		
a. Cross-sectional Area @ Flow Depth, **A**	2.67	sq-ft
b. Wetted Perimeter, **WP**	9.11	ft
c. Hydraulic Radius, **R**	0.29	ft
d. Calculated Flow Rate, **Q**	1.75	cfs
4. Design Flow Velocity		
a. Design Flow Velocity, **V** (1.0 fps max)	0.66	fps
5. Swale Length		
a. Hydraulic Residence Time, **T** (7.0 min)	7	min
b. Minimum Length, **L**	276	ft
6. BMP Area Requirements		
a. Freeboard, **f** (6 min)	6	in
b. Embankment Top Surface Area, $\mathbf{A_{BMP}}$	3,310	sq-ft

FIGURE 6: VEGETATED SWALE SIZING WORKSHEET

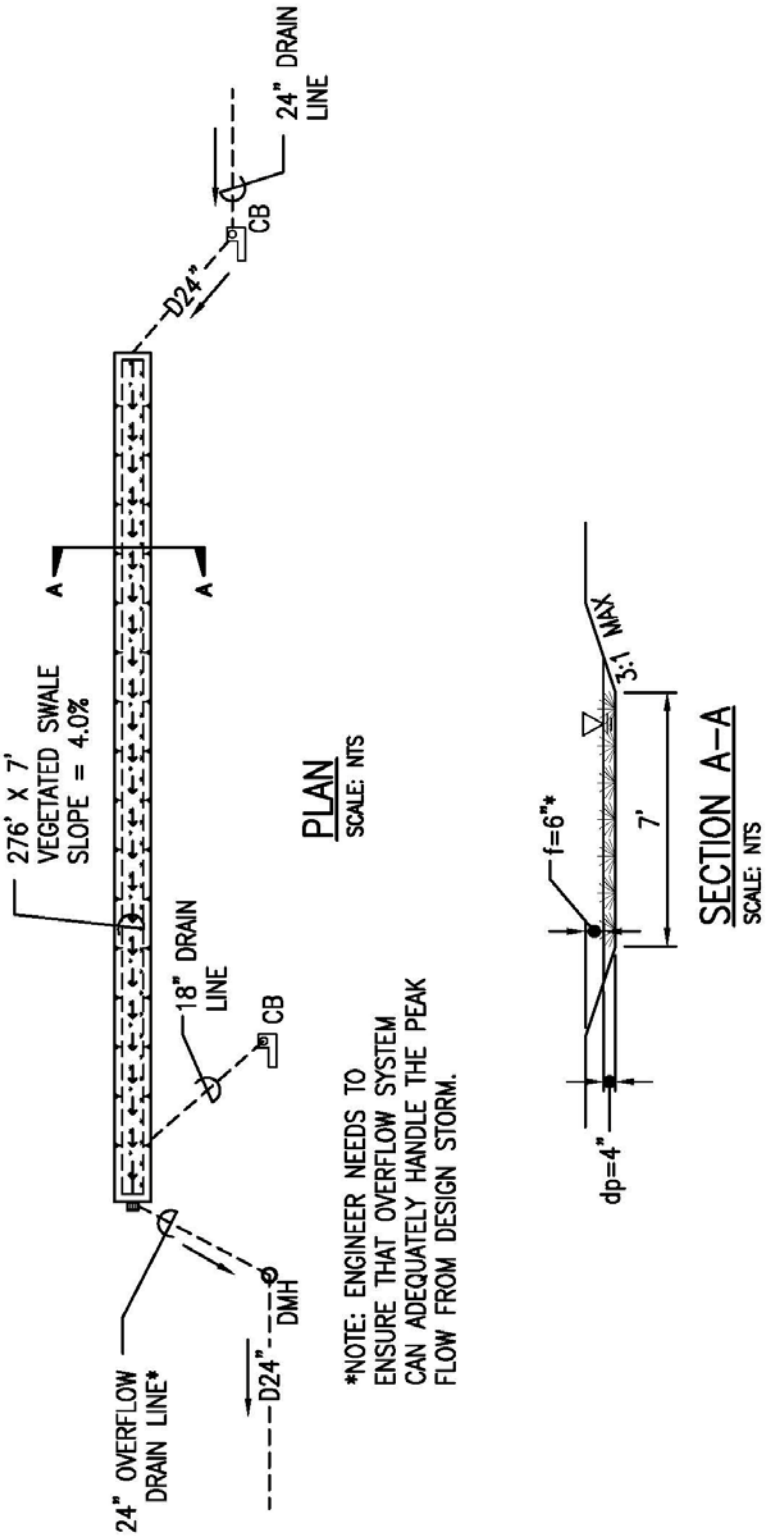


FIGURE 7: BMP DETAILS, BIOFILTRATION

EXAMPLE 2, 3.44 ACRE COMMERCIAL DEVELOPMENT

A 3.44-acre site will be developed for the construction of a commercial development of 4 retail establishments. The site is divided into 7 drainage areas, as shown in Figure 1.

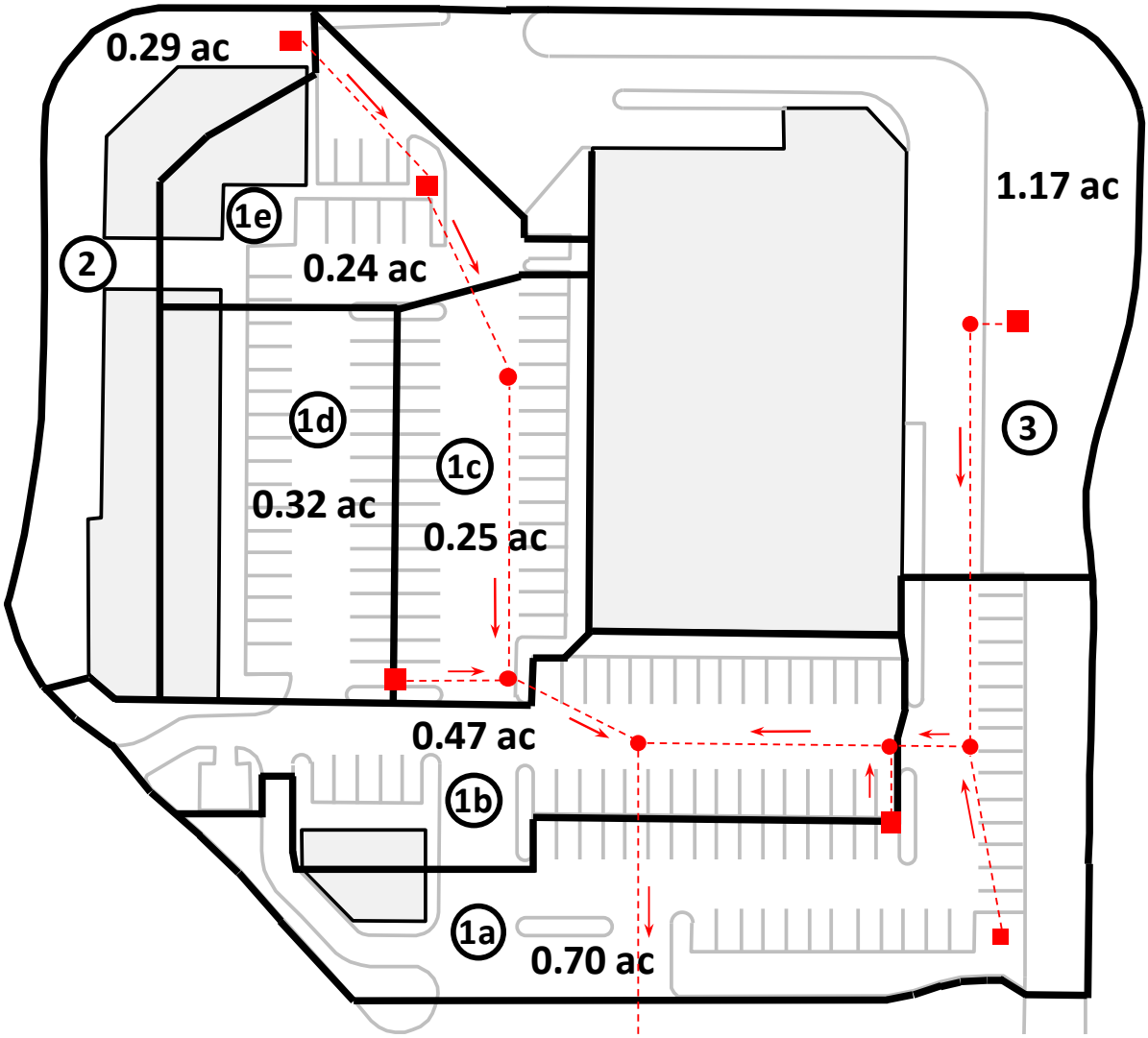


FIGURE 1: EXAMPLE 2 DEVELOPMENT PLAN

Based on the size, the project meets the criteria of a Priority A2 new development project and requires preparation of a storm water quality checklist (SWQC). As specified in Section B.6 of §1-5.1 Part I, water quality criteria shall be achieved with LID Site Design, Source Control, LID Retention or LID Biofiltration, and Alternative Compliance (if necessary). Implementation of each of these four elements for this example is presented below. References to specific proprietary products should not be interpreted as an endorsement of those products.

A. LID SITE DESIGN

The *City and County of Honolulu Storm Water BMP Guide* (*BMP Guide*) identifies 5 LID Site Design Strategies for new development and redevelopment projects. Those that are applicable to this project will be implemented.

B. SOURCE CONTROL

The *BMP Guide* identifies 12 Source Control BMPs for new development and redevelopment projects. Those that are applicable to this project will be implemented.

C. LID RETENTION OR BIOFILTRATION

Retention or biofiltration requirements are presented for each drainage area individually, as each drainage area has its own characteristics and options.

C.1 Drainage Area 1

Drainage Area 1 is composed of 5 subareas as shown in Figure 1. The runoff from each subarea may be retained with permeable pavement if retention is feasible (see Figure 2), or biofiltered with a vegetated bio-filter (see Figure 3). The sizing of both BMPs is accomplished using the City's BMP sizing worksheets, which are consistent with the Step-by-Step Sizing Procedures provided in the *BMP Guide*.

Permeable Pavement. The BMP sizing worksheet for Drainage Area 1a is presented in Figure 4, and the calculations are summarized as follows:

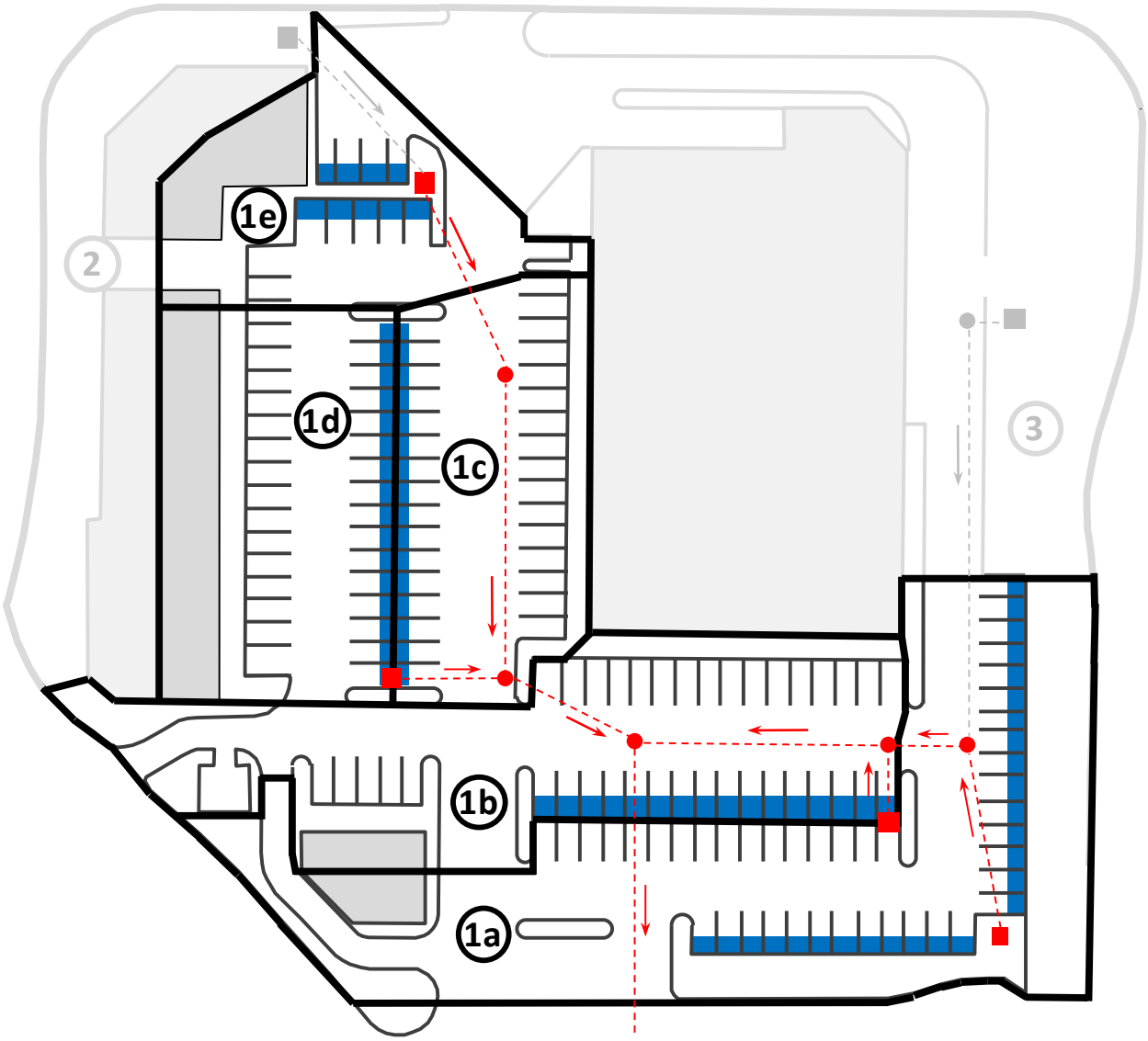
1. Water Quality Volume (WQV). Assuming 70% impervious cover, and using a 1-inch design storm depth, the WQV is calculated to be 1,957 cubic feet.
2. Maximum allowable storage depth. For illustration purposes, assume a soil infiltration rate of 1.5 in/hr and an infiltration rate factor of safety of 2. Using a drawdown time of 48 hours, the maximum allowable storage depth is calculated to be 3.0 feet.
3. Design depths. The pavement course and reservoir course are set to 7 inches and 36 inches, respectively. Using a pavement course porosity of 0.15 and a reservoir course porosity of 0.35, the total effective water storage depth is calculated to be 1.14 feet, which is equal to or less than the maximum allowable storage depth.
4. Pavement surface area. Using a fill time of 2 hours, the required surface area is calculated to be 1,550 square feet.

The length of the permeable pavement is set to the length of the corresponding parking area, and the width is then calculated using the required surface area. This results in a width of 6.48 feet. The necessary length will achieved with two sections, one for each parking area.

The calculations for the other 4 subareas follow the same steps using the same design parameters for constructability and consistency. A summary is as follows:

Subarea ID	Drainage Area (ac)	% Impervious Cover	WQV (cu-ft)	Surface Area (sq-ft)	Length (ft)	Width (ft)
1a	0.70	70%	1,957	1,550	239	6.48
1b	0.47	95%	1,544	1,223	140	8.74
1c	0.25	95%	821	651	140	4.65
1d	0.32	95%	1,051	833	140	5.95
1e	0.24	90%	749	593	87	6.82

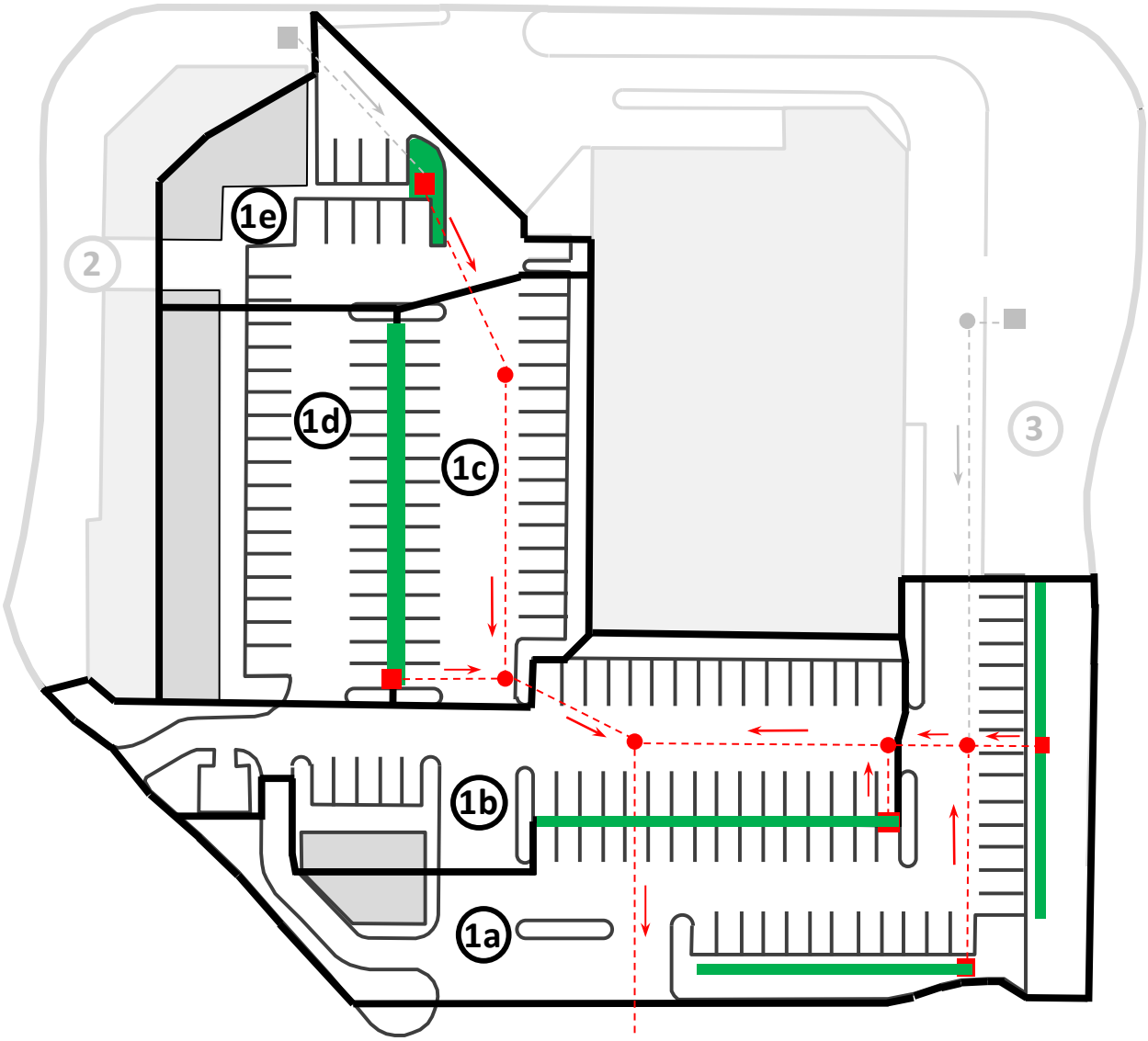
BMP Details are presented in Figure 5.



Permeable Pavement Design Parameters

Design Storm Depth	1 in
Infiltration Rate	1.5 in/hr
Infiltration Rate Safety Factor	2
Drawdown Time	48 hrs
Pavement Course Thickness	7 in
Reservoir Course Thickness	36 in

FIGURE 2: BMP LAYOUT 1, DRAINAGE AREA 1



Vegetated Bio-Filter Design Parameters

Design Storm Depth	1 in
Planting Media Thickness	2 ft
Maximum Ponding Depth	4 in
Filter Bed Drain Time	48 hrs

FIGURE 3: BMP LAYOUT 2, DRAINAGE AREA 1

1. Water Quality Volume		
a. BMP Tributary Drainage Area, **A**	0.70	ac
b. % Impervious Area, **I**	80	%
c. Water Quality Design Storm Depth, **P**	1.0	in
d. Volumetric Runoff Coefficient, **C**	0.77	
e. Water Quality Volume, **WQV**	1,957	cu-ft
2. Maximum Storage Depth		
a. Soil Infiltration Rate, **k** (0.5 min)	1.5	in/hr
b. Infiltration Rate Safety Factor (2 - 5), $\mathbf{F_s}$	2	
c. Drawdown Time, **t**	48	hrs
d. Max. Storage Depth, $\mathbf{d_{max}}$	3.0	ft
3. Design Storage Depths		
a. Pavement Course Thickness, $\mathbf{l_p}$	7.0	in
b. Reservoir Course Thickness, $\mathbf{l_r}$	36.0	in
c. Pavement Course Porosity, $\mathbf{n_p}$	0.15	
d. Reservoir Course Porosity, $\mathbf{n_r}$	0.35	
e. Total Effective Storage Depth, $\mathbf{d_t}$	1.14	ft
4. BMP Area Requirements		
a. Fill Time, **T**	2	hrs
b. Min. Surface Area, $\mathbf{A_{BMP}}$	1,550	sq-ft

FIGURE 4: PERMEABLE PAVEMENT SIZING WORKSHEET (DA 1A)

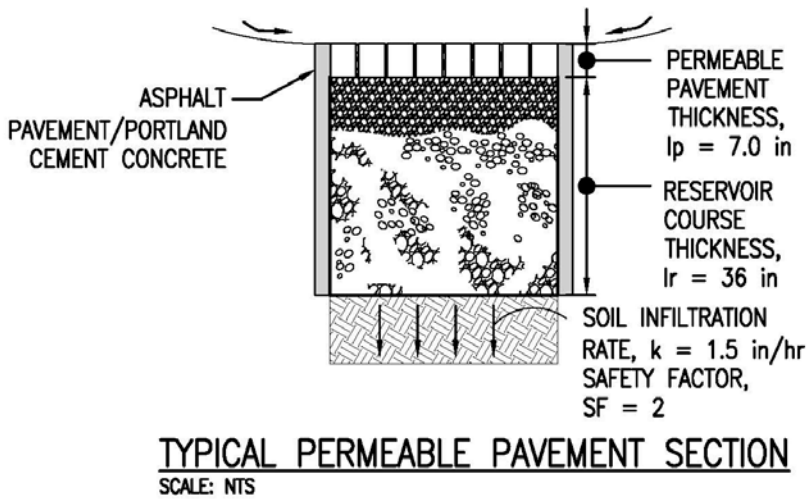


FIGURE 5: PERMEABLE PAVEMENT DETAILS

Vegetated Bio-Filter. The BMP sizing worksheet for Drainage Area 1a is presented in Figure 6, and the calculations are summarized as follows:

1. Water Quality Volume (WQV). The WQV is the same as that given above for the permeable pavement option.
2. Design depths. The planting media depth and maximum ponding depth are set to 2 feet and 4 inches, respectively.
3. Filter bed surface area. Using a planting media permeability coefficient of 1 ft/day and a filter bed drain time of 48 hours, the required filter bed surface area, excluding pretreatment, is calculated to be 903 square feet.
4. Filter bed dimensions. The width of the filter bed is set such that the resulting length occupies the length of the respective parking areas. Using this approach, the filter bed width is set to 3.75 feet.
5. Total Area. Using an embankment side slope of 0 and 3 inches of freeboard, the total BMP areas are the same as the filter bed surface area. Similarly to the retention option, the Vegetated Bio-Filter will be divided into two sections.

The calculations for the other 4 subareas follow the same steps using the same design parameters for constructability and consistency. A summary of all 5 subareas is as follows:

Subarea ID	Drainage Area (ac)	% Impervious Cover	WQV (cu-ft)	Surface Area (sq-ft)	Length (ft)	Width (ft)
1a	0.70	70%	1,957	903	239	3.78
1b	0.47	95%	1,544	713	140	5.09
1c	0.25	95%	821	379	140	2.71
1d	0.32	95%	1,051	485	140	3.47
1e	0.24	90%	749	346	37	13

Not that for Drainage Area 1e, the length and width given above represent maximum values since the BMP area is an irregular shape (i.e., not rectangular). For this reason, the length times the width does not equal the required surface area. BMP details are presented in Figure 7.

1. Water Quality Volume		
a. BMP Tributary Drainage Area, **A**	0.70	ac
b. % Impervious Area, **I**	80	%
c. Water Quality Design Storm Depth, **P**	1.0	in
d. Volumetric Runoff Coefficient, **C**	0.77	
e. Water Quality Volume, **WQV**	1,957	cu-ft
2. Filter Bed Surface Area		
a. Planting Media Depth, $\mathbf{l_m}$ (2.0 - 5.0 ft)	2.0	ft
b. Maximum Ponding Depth, $\mathbf{d_p}$ (12 in)	4.0	in
c. Planting Media Coefficient of Permeability, **k**	1	ft/day
d. Filter Bed Drain Time, **t**	48	hrs
e. Filter Bed Surface Area, $\mathbf{A_{BMP}}$	903	sq-ft
3. BMP Area		
a. Side Slopes (length per unit height), **z**	0	
b. Freeboard, **f**	0.25	ft
c. Filter Bed Width, $\mathbf{w_b}$	3.78	ft
d. Filter Bed Length, $\mathbf{l_b}$	239	ft
e. Top Width, $\mathbf{w_t}$	3.78	ft
f. Top Length, $\mathbf{l_t}$	239	ft
g. Min. Top Surface Area excluding pretreatment, $\mathbf{A_{BMP}}$	903	sq-ft

FIGURE 6: VEGETATED BIO-FILTER SIZING WORKSHEET (DA 1A)

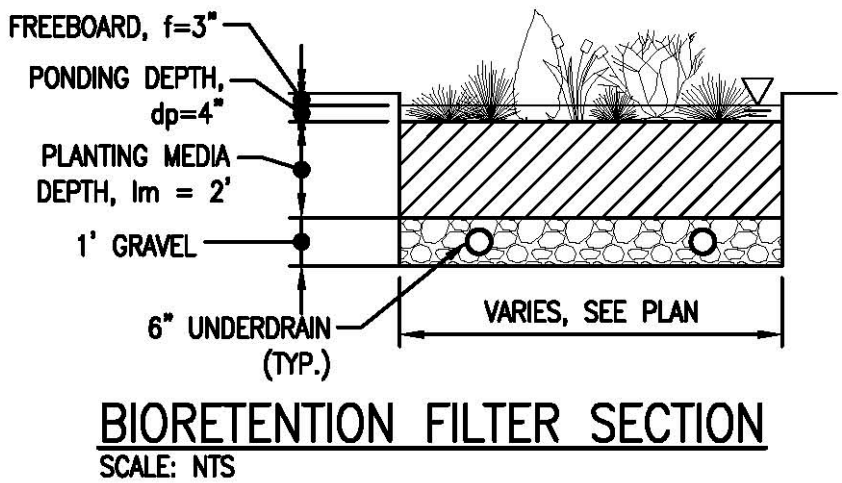


FIGURE 7: VEGETATED BIO-FILTER DETAILS

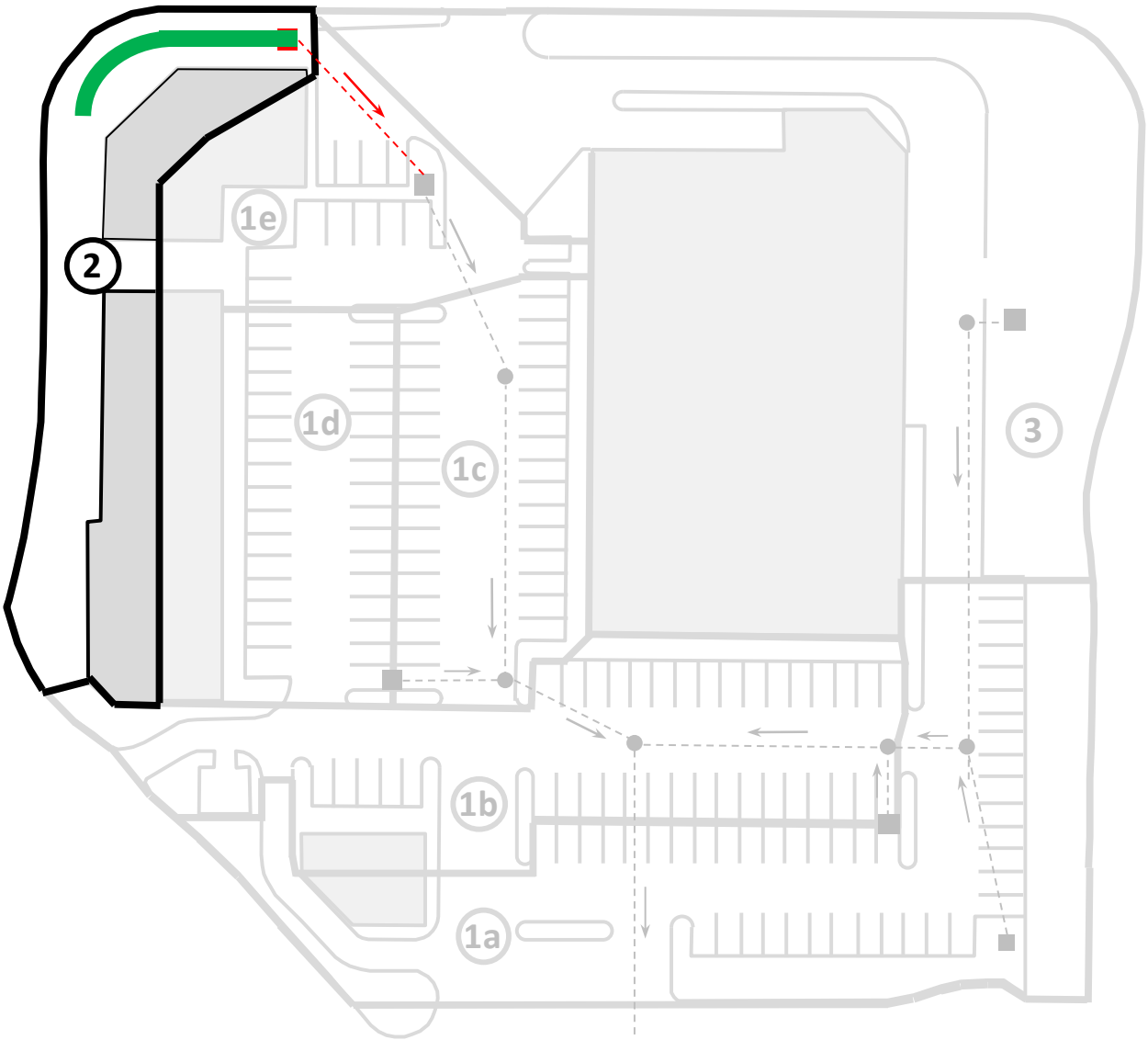
C.2 Drainage Area 2

The 0.29 acres of runoff from Drainage Area 2 may not be retained because there is not enough space to meet the building setback criterion (20 feet). It may be biofiltered with either a vegetated swale (see Figure 8) or vegetated bio-filter (see Figure 9). The sizing of the BMPs is accomplished using the City's BMP sizing worksheets, which are consistent with the Step-by-Step Sizing Procedure provided in the *BMP Guide*.

Vegetated Swale. The BMP sizing worksheet is presented in Figure 10, and the calculations are summarized as follows:

1. Water Quality Flow Rate (WQF). Assuming a weighted runoff coefficient of 0.50, and using a rainfall intensity of 0.4 in/hr, the WQF is calculated to be 0.058 cfs.
2. Swale geometry. The bottom width, depth of flow, side slope, longitudinal slope, and Manning's roughness coefficient are set to 3 feet (to minimize the length), 1.25 inches, 3:1, 2%, and 0.20, respectively.
3. Swale hydraulic capacity. The cross sectional area, wetted perimeter, and hydraulic radius are calculated to be 0.35 sq-ft, 3.66 ft, and 0.09 ft, respectively. The design flow rate is calculated to be 0.075 cfs, which is equal to or greater than the WQF.
4. Design flow velocity. The design flow velocity is calculated to be 0.22 feet per second, which is less than the maximum allowed velocity of 1 foot per second.
5. Swale length. Using a hydraulic residence time of 7 minutes, the swale length is calculated to be 92 feet.
6. BMP area requirements. Using 6 inches of freeboard, the required total surface area is calculated to be 607 sq-ft. At 92 feet long, this equates to a top width of 6.6 feet.

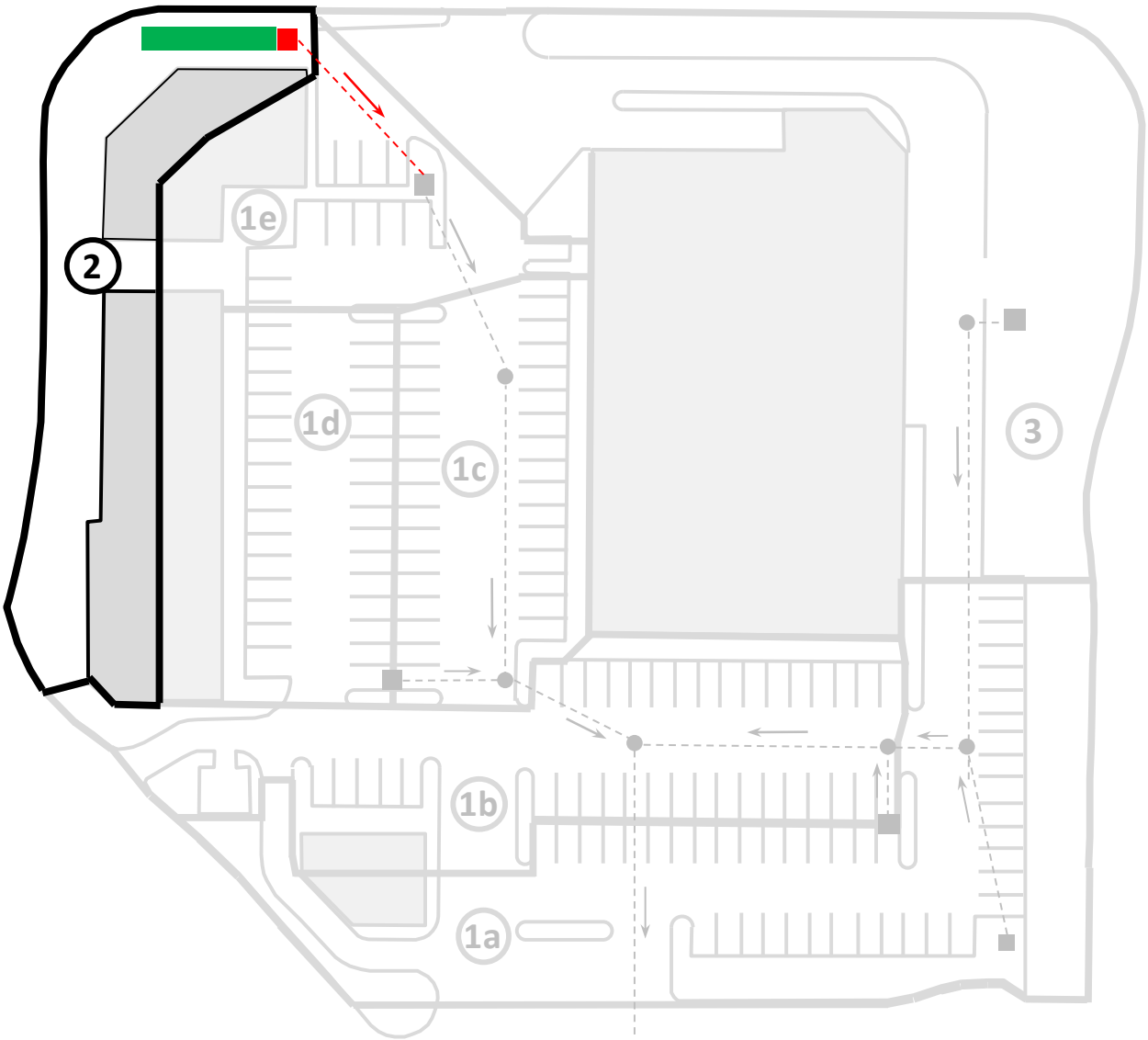
BMP details are presented in Figure 11.



Vegetated Swale Design Parameters

Weighted Runoff Coefficient	50%
Rainfall Intensity	0.4 in/hr
Manning's Roughness Coeff.	0.20
Hydraulic Residence Time	7 min

FIGURE 8: BMP LAYOUT 1, DRAINAGE AREA 2



Vegetated Bio-Filter Design Parameters

Design Storm Depth	1 in
Planting Media Thickness	2 ft
Maximum Ponding Depth	4 in
Filter Bed Drain Time	48 hrs

FIGURE 9: BMP LAYOUT 2, DRAINAGE AREA 2

1. Water Quality Flow Rate		
a. BMP Tributary Drainage Area, **A**	0.29	ac
b. Weighted Runoff Coefficient, **C**	0.5	
c. Rainfall Intensity, **i**	0.4	in/hr
d. Water Quality Flow Rate, **WQF**	0.058	cfs
2. Swale Geometry		
a. Bottom Width, **b** (10.0 ft max)	3.00	ft
b. Flow Depth, **y** (4.0 in max)	1.25	in
c. Side Slopes (length per unit height), **z** (3.0 max)	3	ft/ft
d. Longitudinal Slope, **s**	2.0	%
e. Manning's Roughness Coefficient, **n**	0.20	
3. Swale Hydraulic Capacity		
a. Cross-sectional Area @ Flow Depth, **A**	0.35	sq-ft
b. Wetted Perimeter, **WP**	3.66	ft
c. Hydraulic Radius, **R**	0.09	ft
d. Calculated Flow Rate, **Q**	0.075	cfs
4. Design Flow Velocity		
a. Design Flow Velocity, **V** (1.0 fps max)	0.22	fps
5. Swale Length		
a. Hydraulic Residence Time, **T** (7.0 min)	7	min
b. Minimum Length, **L**	92	ft
6. BMP Area Requirements		
a. Freeboard, **f** (6 min)	6	in
b. Embankment Top Surface Area, $\mathbf{A_{BMP}}$	607	sq-ft

FIGURE 10: VEGETATED SWALE SIZING WORKSHEET (DA 2)

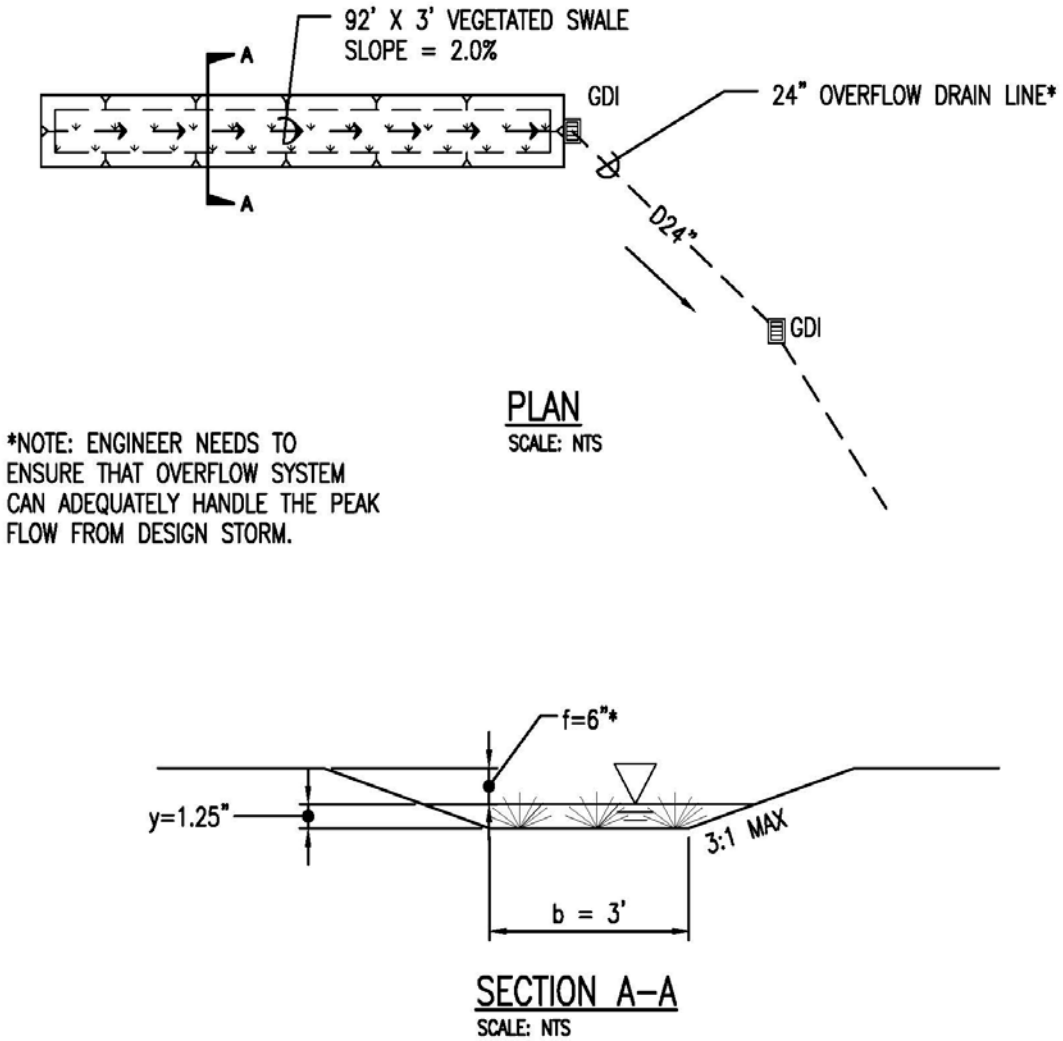


FIGURE 11: VEGETATED SWALE DETAILS

Vegetated Bio-Filter. The BMP sizing worksheet is presented in Figure 12, and the calculations are summarized as follows:

1. Water Quality Volume (WQV). Assuming 50% impervious cover, and using a 1-inch design storm depth, the WQV is calculated to be 526 cubic feet.
2. Design depths. For constructability and consistency, the planting media depth and maximum ponding depth used for Drainage Area 1 are used here.
3. Filter bed surface area. Using the same parameters as those presented for Drainage Area 1, the required filter bed surface area is calculated to be 243 square feet.
4. Filter bed dimensions. The width and length of the filter bed are set to 5 feet and 49 feet, respectively, which are based on available space and drainage area characteristics.

5. Total Area. Using an embankment side slope of 3:1 and 3 inches of freeboard, the top width, top length, and total BMP area are calculated to be 8.5 feet, 52.1 feet, and 443 square feet, respectively.

BMP details are presented in Figure 7.

1. Water Quality Volume		
a. BMP Tributary Drainage Area, **A**	0.29	ac
b. % Impervious Area, **I**	50	%
c. Water Quality Design Storm Depth, **P**	1.0	in
d. Volumetric Runoff Coefficient, **C**	0.5	
e. Water Quality Volume, **WQV**	526	cu-ft
2. Filter Bed Surface Area		
a. Planting Media Depth, $\mathbf{l_m}$ (2.0 - 5.0 ft)	2.0	ft
b. Maximum Ponding Depth, $\mathbf{d_p}$ (12 in)	4.0	in
c. Planting Media Coefficient of Permeability, **k**	1	ft/day
d. Filter Bed Drain Time, **t**	48	hrs
e. Filter Bed Surface Area, $\mathbf{A_{BMP}}$	243	sq-ft
3. BMP Area		
a. Side Slopes (length per unit height), **z**	3	
b. Freeboard, **f**	0.25	ft
c. Filter Bed Width, $\mathbf{w_b}$	5.0	ft
d. Filter Bed Length, $\mathbf{l_b}$	49	ft
e. Top Width, $\mathbf{w_t}$	8.5	ft
f. Top Length, $\mathbf{l_t}$	52.1	ft
g. Min. Top Surface Area excluding pretreatment, $\mathbf{A_{BMP}}$	443	sq-ft

FIGURE 12: VEGETATED BIO-FILTER SIZING WORKSHEET (DA 2)

C.3 Drainage Area 3

The 1.17 acres of runoff from Drainage Area 3 may be retained with an infiltration basin if retention is feasible (See Figure 13), or biofiltered with a vegetated bio-filter (see Figure 14). The sizing of both BMPs is accomplished using the City's BMP sizing worksheets, which are consistent with the Step-by-Step Sizing Procedure provided in the *BMP Guide*.

Infiltration Basin. The BMP sizing worksheet is presented in Figure 15, and the calculations are summarized as follows:

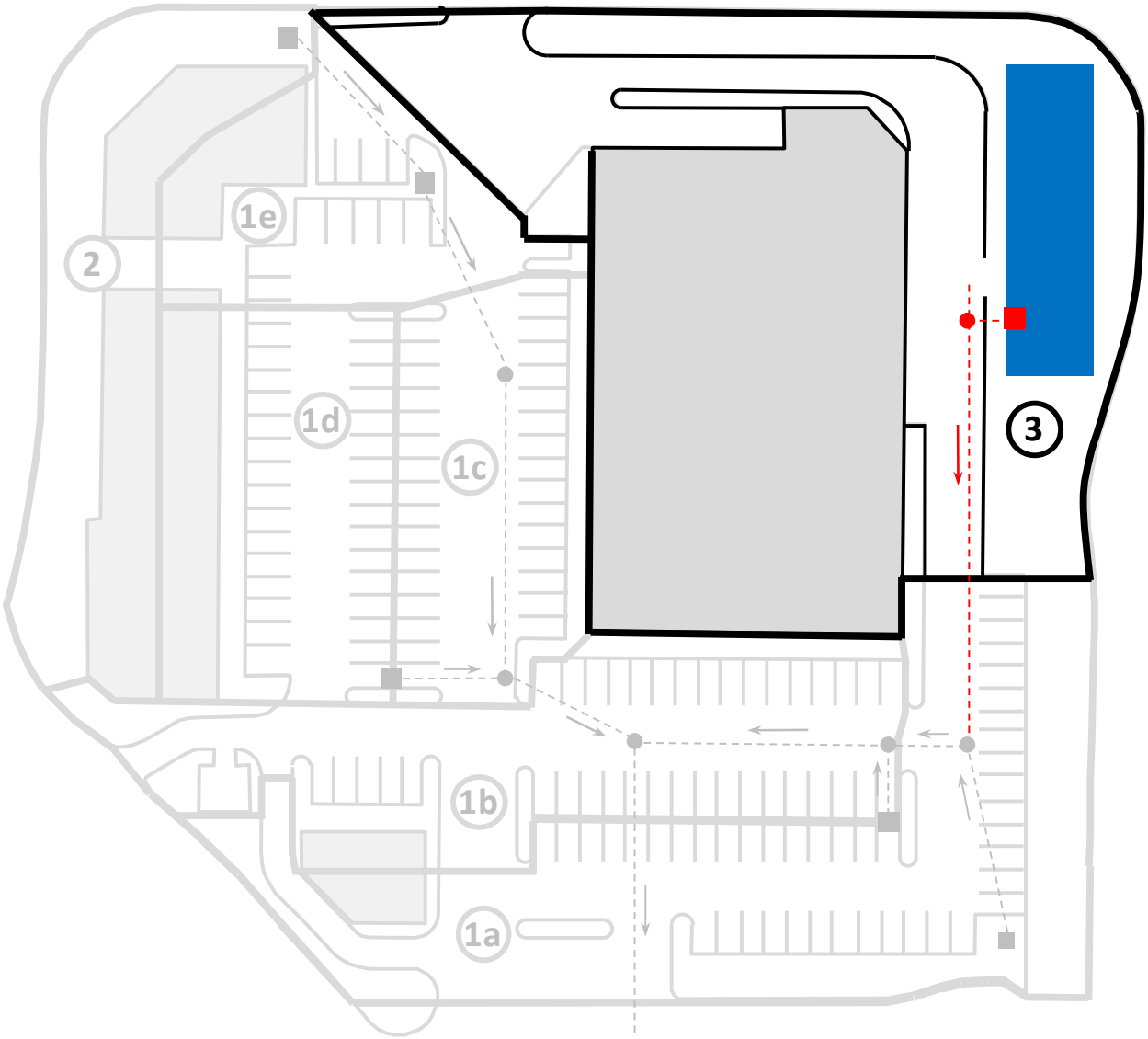
1. Water Quality Volume (WQV). Assuming 75% impervious cover, and using a 1-inch design storm depth, the WQV is calculated to be 3,079 cubic feet.
2. Maximum allowable storage depth. Using the same parameters as those presented for Drainage Area 1, the maximum allowable storage depth is calculated to be 3.0 feet.
3. Design storage depth. The ponding depth is set to 3 feet, which is equal to or less than the maximum allowable storage depth.
4. Basin invert footprint. Using a basin fill time of 2 hours (industry accepted practice), the required invert surface area is calculated to be 985 square feet.
5. BMP area requirements. Based on available space, the invert width is set to 10 feet, and the invert length is calculated to be 98.5 feet. Using a side slope of 3:1 and a freeboard depth of 1 foot (actual free board requirements must be determined for flood design storm), the top width and top length are calculated to be 34 feet and 122.53 feet, respectively. The total footprint is calculated to be 4,166 square feet.

BMP details are presented in Figure 16.

Vegetated Bio-Filter. The BMP sizing worksheet is presented in Figure 17, and the calculations are summarized as follows:

1. Water Quality Volume (WQV). The WQV is the same as that given above for the infiltration basin (3,079 cubic feet).
2. Design depths. For constructability and consistency, the planting media depth and maximum ponding depth used for Drainage Area 1 are used here.
3. Filter bed surface area. Using the same parameters as those presented for Drainage Area 1, the required filter bed surface area is calculated to be 1,421 square feet.
4. Filter bed dimensions. Based on available space, the invert width is set to 20 feet, and the invert length is calculated to be 71 feet.
5. Total Area. Using an embankment side slope of 3:1 and 3 inches of freeboard, the top dimensions are calculated to be 24 feet by 75 feet, and the total BMP area is calculated to be 1,752 square feet.

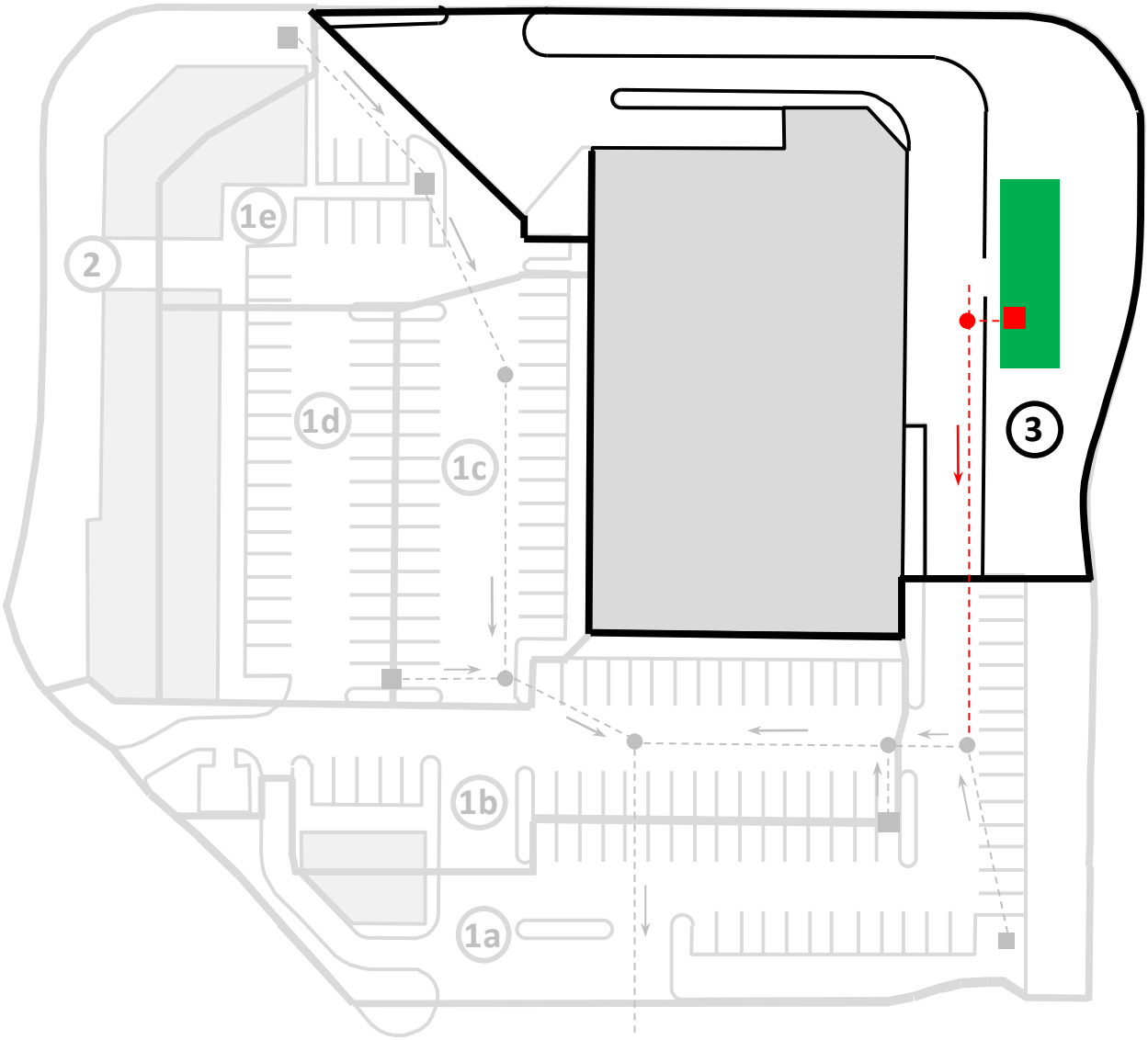
BMP details are presented in Figure 7.



Infiltration Basin Design Parameters

Design Storm Depth	1 in
Infiltration Rate	1.5 in/hr
Infiltration Rate Safety Factor	2
Drawdown Time	48 hrs
Basin Fill Time	2 hrs

FIGURE 13: BMP LAYOUT 1, DRAINAGE AREA 3



Vegetated Bio-Filter Design Parameters

Design Storm Depth	1 in
Planting Media Thickness	2 ft
Maximum Ponding Depth	4 in
Filter Bed Drain Time	48 hrs

FIGURE 14: BMP LAYOUT 2, DRAINAGE AREA 3

1. Water Quality Volume		
a. BMP Tributary Drainage Area, **A**	1.17	ac
b. % Impervious Area, **I**	75	%
c. Water Quality Design Storm Depth, **P**	1.0	in
d. Volumetric Runoff Coefficient, **C**	0.725	
e. Water Quality Volume, **WQV**	3,079	cu-ft
2. Maximum Storage Depth		
a. Soil Infiltration Rate, **k** (0.5 min)	1.5	in/hr
b. Infiltration Rate Safety Factor (2 - 5), $\mathbf{F_s}$	2	
c. Drawdown Time, **t**	48	hrs
d. Max. Storage Depth, $\mathbf{d_{max}}$	3.0	ft
3. Design Storage Depth		
a. Ponding Depth, $\mathbf{d_p}$	3.00	ft
4. Basin Invert Footprint		
b. Reservoir Fill Time, **T**	2	hrs
c. Min. Bottom Surface Area, $\mathbf{A_b}$	985	sq-ft
5. BMP Area Requirements		
a. Side Slopes (length per unit height), **z** (3.0 min)	3	
b. Freeboard, **f** (1.0 min)	1	ft
c. Invert Width, $\mathbf{w_b}$	10.0	ft
d. Invert Length, $\mathbf{l_b}$	98.5	ft
e. Top Width, $\mathbf{w_t}$	34.00	ft
f. Top Length, $\mathbf{l_t}$	122.53	ft
g. Min. Top Surface Area excluding pretreatment, $\mathbf{A_{BMP}}$	4,166	sq-ft

FIGURE 15: INFILTRATION BASIN SIZING WORKSHEET (DA 3)

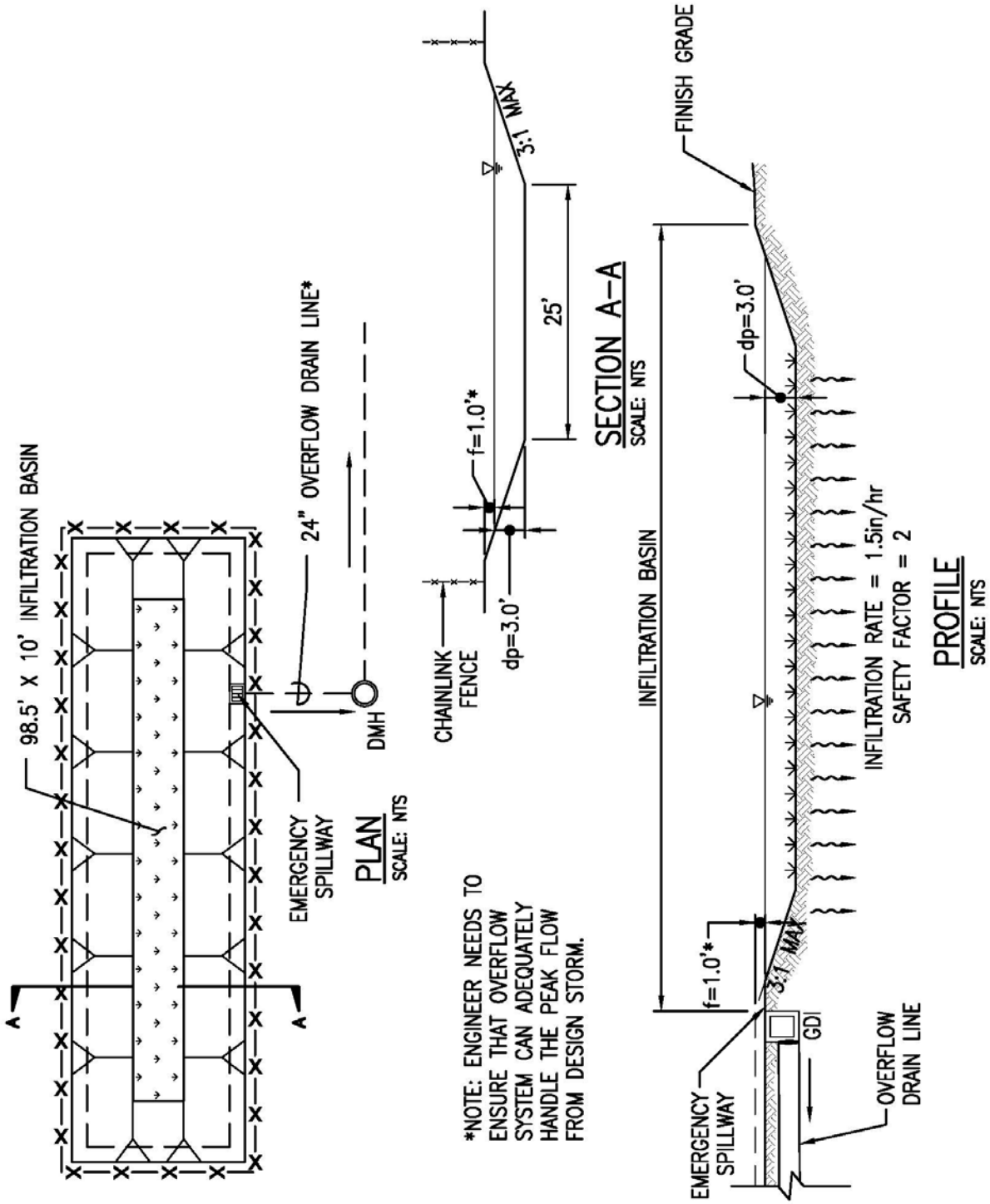


FIGURE 16: INFILTRATION BASIN DETAILS

1. Water Quality Volume		
a. BMP Tributary Drainage Area, **A**	1.17	ac
b. % Impervious Area, **I**	75	%
c. Water Quality Design Storm Depth, **P**	1.0	in
d. Volumetric Runoff Coefficient, **C**	0.725	
e. Water Quality Volume, **WQV**	3,079	cu-ft
2. Filter Bed Surface Area		
a. Planting Media Depth, $\mathbf{l_m}$ (2.0 - 5.0 ft)	2.0	ft
b. Maximum Ponding Depth, $\mathbf{d_p}$ (12 in)	4.0	in
c. Planting Media Coefficient of Permeability, **k**	1	ft/day
d. Filter Bed Drain Time, **t**	48	hrs
e. Filter Bed Surface Area, $\mathbf{A_{BMP}}$	1,421	sq-ft
3. BMP Area		
a. Side Slopes (length per unit height), **z**	3	
b. Freeboard, **f**	0.25	ft
c. Filter Bed Width, $\mathbf{w_b}$	20	ft
d. Filter Bed Length, $\mathbf{l_b}$	71.06	ft
e. Top Width, $\mathbf{w_t}$	23.50	ft
f. Top Length, $\mathbf{l_t}$	74.56	ft
g. Min. Top Surface Area excluding pretreatment, $\mathbf{A_{BMP}}$	1,752	sq-ft

FIGURE 17: VEGETATED BIO-FILTER SIZING WORKSHEET (DA 3)

D. ALTERNATIVE COMPLIANCE

If LID Retention and LID Biofiltration are determined to be infeasible, alternative compliance would be required. Options include treating the runoff with Other Treatment Control BMPs identified in the BMP Guide (detention basin, manufactured treatment device, sand filter, etc.).

EXAMPLE 3, 0.74 ACRE COMMERCIAL DEVELOPMENT

A restaurant is being constructed on a 0.74-acre site. The layout is presented in Figure 1, and includes more than 10,000 square feet of impervious surface.

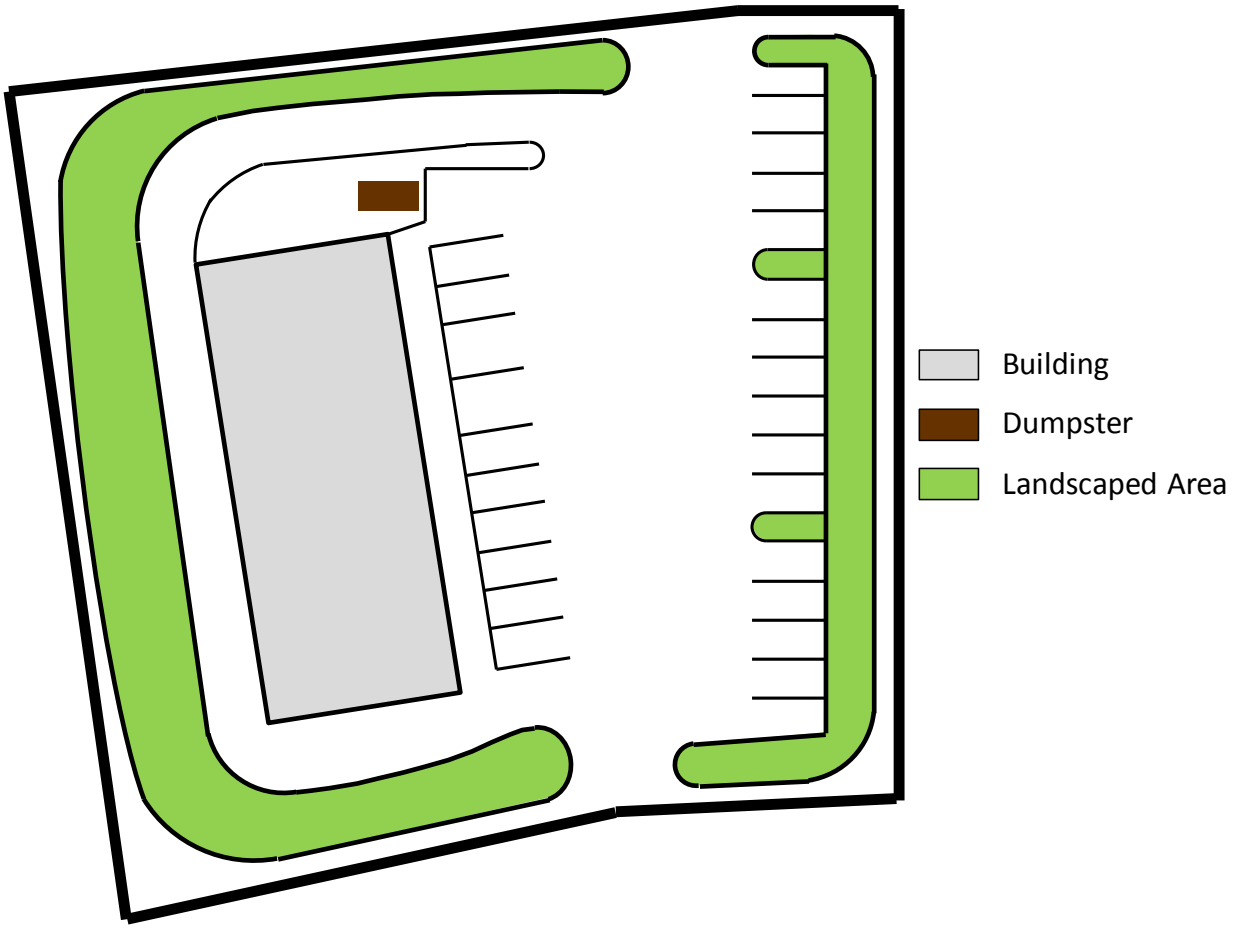


FIGURE1: EXAMPLE 3 DEVELOPMENT PLAN

The project meets the criteria of a Priority B new development project and requires the preparation of a Storm Water Quality Checklist (SWQC). As specified in Section B.6 of §1-5.1 Part I, water quality criteria shall be achieved by considering LID Site Design and implementing appropriate Source Control. Each of these elements is presented below.

A. LID SITE DESIGN

All 5 LID Site Design Strategies specified in the *City and County of Honolulu Storm Water BMP Guide* (*BMP Guide*) were considered. The strategy "Direct Runoff to Landscaped Areas" will be implemented by directing all pavement runoff to the surrounding landscaped areas, and directing roof runoff to the adjacent landscaped area using roof drains. Details are shown in Figure 2.

B. SOURCE CONTROL

The *BMP Guide* identifies 12 Source Control BMPs for new development and redevelopment projects. The following ones will be implemented:

- Automatic Irrigation Systems
  - Irrigation system will be designed to each landscape area's specific water requirements.
  - Irrigation system will be designed to minimize the runoff of excess irrigation water into the storm water drainage system.
  - Plants with similar water requirements will be grouped together in order to reduce excess irrigation runoff and promote surface filtration.
- Storm Drain Inlets
  - All storm drain inlets and catch basins within the project area will be stenciled with appropriate signage.
- Outdoor Trash Storage
  - Dumpster area will be graded towards vegetated/landscaped area.
  - Drip pans will be placed underneath dumpster to reduce/prevent leaking of liquid wastes.
  - Dumpster with attached lids will be used to prevent rainfall from entering container.
  - Dumpster area will be paved with an impervious surface to mitigate spills.
  - Signs will be posted indicating that hazardous material are not to be disposed of therein.
- Parking Areas
  - Pavement runoff will be directed towards vegetated/landscaped areas.

Details are shown in Figure 2.

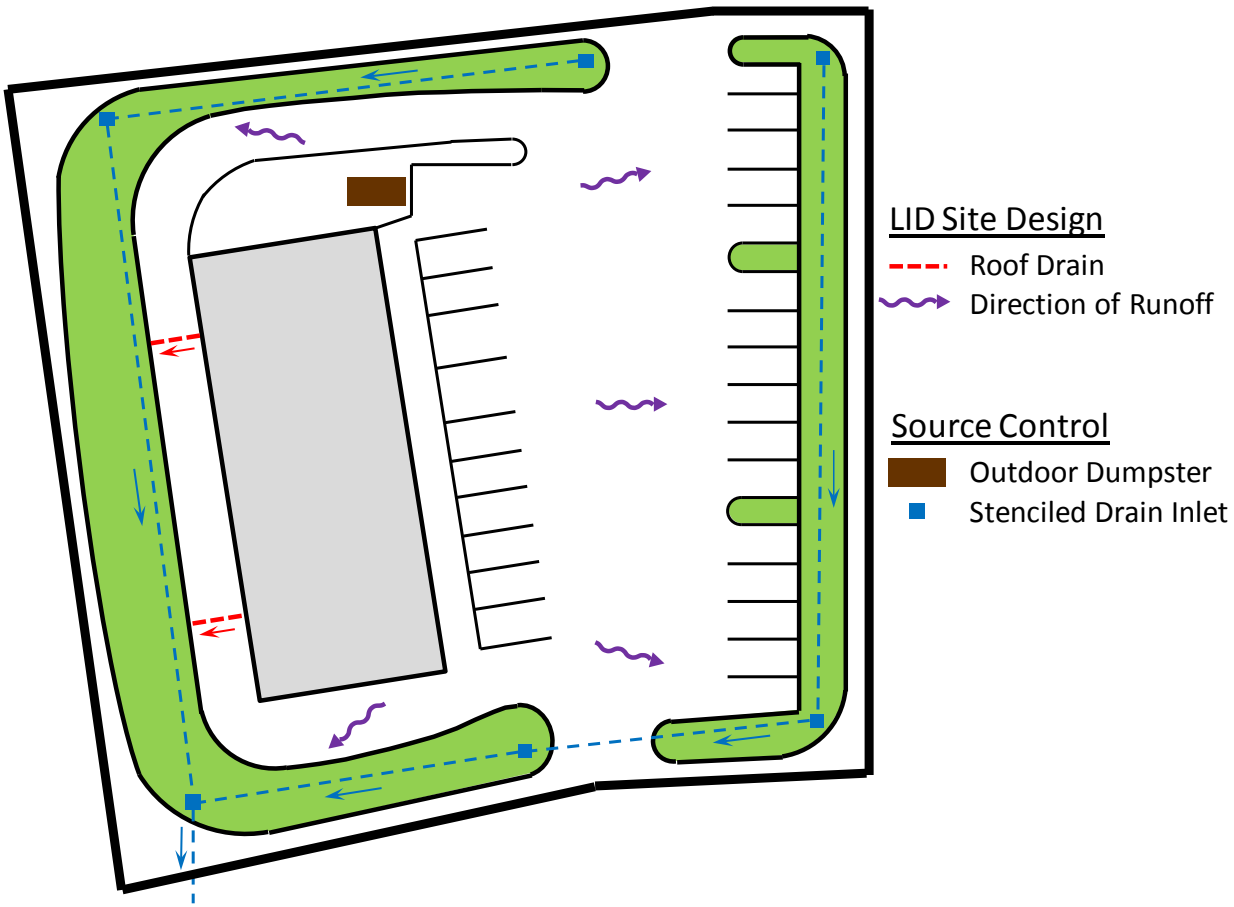


FIGURE2: LID SITE DESIGN AND SOURCE CONTROL PLAN