City of Oakland

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Public Works Agency Standards

DRAFT

STORM DRAINAGE DESIGN GUIDELINES

Engineering Design & ROW Management Division

Effective Date: July 2006

Revised July 2006

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ACKNOWLEDGEMENTS

The City of Oakland's Storm Drainage Design Guidelines have been prepared using the Hydrology and Hydraulics Manual published by the Alameda County Flood Control and Water Conservation District as the primary source of information. City's acknowledgement is also forward to the following organizations for guidelines and conceptual solutions to reduce and attenuate storm water runoff:

1. California Stormwater BMP Handbook for New Development and Redevelopment www.cabmphandbooks.com/
2. lowimpactdevelopment.com http://www.lowimpactdevelopment.com/ for natural alternative solutions
3. Interlocking Concrete Pavement Institute http://www.icpi.org/
4. The California Nevada Cement Promotion Council (http://www.cncpc.org/) and Concrete Network (http://concretenetwork.com/), for Pervious Concrete pavement

PURPOSE

The purpose of the Storm Drainage Design Guidelines for the City of Oakland is to provide an easy-to-follow manual for engineers who are familiar with the generally accepted hydrology and hydraulic design practices. This manual is prepared by the City of Oakland (City) using the Hydrology and Hydraulics Manual published by the Alameda County Flood Control and Water Conservation District (County Flood Control District) as the primary source.

This manual provides computational techniques and criteria for the design of storm water runoff and drainage facilities and procedures to determine the required storage volume for detention and retention basins. The design engineer should contact the City to determine if detention is required for a specific site. Procedures in this manual apply to the design of typical facilities. Where unusual circumstances result in requirements beyond the scope of this manual, the engineer should contact the City for additional guidance. Criteria not specifically detailed herein shall be determined in accordance with sound engineering practices with the City's approval.

The guidelines in this manual will be updated on a continual basis to reflect changes in City practices and the revision dates are shown on the bottom left corner of the pages. It is the responsibility of the user to determine that the guidelines are current.

The use of this manual in no way obviates or replaces the individual design engineer's adherence to the profession's "standard of care" in the design. Facilities designed prior to the adoption of this manual are not necessarily subject to the current criteria.

INTRODUCTION

The City of Oakland is part of the Alameda County Flood Control District Zone 12. The storm drainage system in the City consists of more than 300 miles of storm drainpipes and 15,000 structures (mostly inlets, manholes, and catch basins). Storm drainpipes in the City are not connected, but rather scattered throughout the entire City as small networks of private or public systems. City owned drainage systems are improved drainage facilities located within easements and right-of-ways. Other privately improved drainage systems, creeks, and watercourses are part of the

City's drainage network but not necessarily owned and maintained by the City. City maintained drainage facilities include improvements and structures that are constructed through the permit process and dedicated to the City for maintenance. The City is responsible for maintenance and preservation of the dedicated facilities.

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DRAINAGE FACILITIES

Protection and analysis of the City's drainage system are categorized by the tributary areas. The City considers three categories of natural and improved drainage facilities.

1. Major Facilities

Such facilities are waterways with tributary areas equal or larger than 25 Square miles such as the San Leandro Creek and other major waterways that are primarily maintained by the County Flood Control District.

2. Primary Facilities

Such facilities are waterways and drainage facilities with tributary areas between 50 acres and 25 square miles. These facilities mostly consist of creeks and larger improved waterways and drainage facilities. Many of these facilities are owned and maintained by the Alameda County Flood Control District.

3. Secondary Facilities

Such facilities are drainage-facilities or waterways with tributaries areas less than 50 acres. This includes majority of the City's drainage conduits.

TAILWATER ELEVATIONS IN FLOOD ZONES & DESIGN DISCHARGE FREQUENCIES

Primary Facilities that flow into or located within Federal Emergency Management Agency (FEMA) study areas are subject to FEMA 100-year water surface elevation using the FEMA criteria. In addition, facilities must be analyzed for additional rise in the water levels using the following two cases and selecting the higher water surface elevation for design and planning considerations:

- 5-year recurrence interval for peak discharge with starting water elevation at the 100-year tide levels in the bay*.
- 25-year recurrence interval for peak discharge with the Mean Higher High water (MHHW) elevation in the bay*.
- For facilities draining directly to the bay, the 100-year tide levels shall be used as the design tailwater elevation.

  * For areas with the discharge points further than 100 feet away from the bay or where no starting water surface elevation are provided, the starting water surface elevation has to be calculated upstream to the area under consideration for a more accurate starting water elevation. Refer to **County Flood Control District Attachments 14A, 14B, and 15.**

For drainage areas outside the FEMA study zone, the design storm frequencies shall be 10-year for Secondary Facilities and 25-year for Primary Facilities. The design tailwater elevation for secondary and primary facilities shall be the water surface elevation of the receiving waterway.

For Major facilities, contact the Alameda County Flood Control and Water Conservation District for information and a copy of the Alameda County Hydrology and Hydraulics Manual.

ELEVATION DATUM

The engineer must be certain that all elevations used for tides, cross sections, tailwater elevations and bridge geometry are on the same vertical datum systems. Refer to **County Flood Control District Attachment 13**.

RUNOFF DETERMINATION METHODS

Two hydrologic computation techniques are used for runoff determination. Modified Rational Method is used for drainage areas less than half square mile (320 acres) and Synthetic Unit Hydrograph Method is used for drainage areas greater than half square mile and for the design of detention basins.

1- Modified Rational Method

Data used in Rational Method calculations shall be entered into Data Table shown in **County Flood Control District** **Attachment 2** for submittals. The overall watershed shall be broken down into smaller areas that contribute to hydraulically significant points of concentration. The boundaries shall be established based upon local topographic boundaries such as ridges, streets, existing drainage systems, etc., using good engineering practice. The design flow rate shall be calculated using the Modified Rational Formula:

$$Q = i\,(C'A) \qquad (1)$$

where:

Q = design runoff flow rate (cfs)

I = rainfall intensity (in/hr)

C' = runoff coefficient modified by slope and rainfall intensity

A = drainage area (acres)

1.1- Tc (Time of Concentration)

The time of concentration is the time required for the runoff from the most remote region of the watershed to reach the point of concentration at which the flow is to be calculated. This is composed of the Initial Time of Concentration, sometimes referred to as the inlet time, the Conduit and/or the Stream Time.

Initial Time of Concentration (Initial Tc) is that time required for runoff to travel from the most remote point in the drainage area to the first point of concentration. Often, this first point of concentration is the first inlet of the storm water system or the upstream end of a defined ditch or swale. This time will seldom be less than three minutes and shall be determined using the criteria below:

For undeveloped watersheds, Initial $Tc = L / (60\ V)$ *(2)*

where:

Tc = Time of concentration in minutes

L = Overland flow length in feet

V = Overland flow velocity in feet per second from the **County Flood Control District** **Attachment 3**.

For urbanized watersheds, the initial time of concentration shall be taken as the "roof-to-gutter" time plus the time required for the water to flow from the street gutter to the first inlet of the storm water system. For residential areas, five minutes shall be used for roof-to-gutter time. Sound engineering practices shall be used for other than typical residential areas. Roof to gutter flows shall be modified for non-urbanized areas depending on ground slope and the type of facility, See **County Flood Control District** **Attachment 4**. For typical urbanized watersheds, the Initial Tc equals the roof-to-gutter time plus the travel time from the upstream end of the gutter to the first inlet.

Conduit Time is the length of time required for the water to flow from one point of concentration, or inlet, to the next. The calculated average velocity, or weighted incremental velocities, must accurately reflect the hydraulic conditions (i.e. pressure or open channel) within the storm water system.

Stream Time is the flow that takes place in natural streams, the velocity may be determined using **County Flood Control District** **Attachment 5** or other appropriate methods.

1.2- Ij, (Rainfall Intensity)

The rainfall intensity for the appropriate time of concentration and storm recurrence interval is computed from the following equation:

$$Ij = (0.33 + 0.091144 \text{ x } MAP) \text{ x } \left(0.249 + 0.1006 \text{ x } K_j\right) \text{ x } T_i^{-0.56253} \qquad (3)$$

where:

Ij = Rainfall intensity (in/hr) for return frequency, j, and storm duration, I

MAP = Mean Annual Precipitation (inches)

Ti = Storm Duration (hr) - (Tc/60)

Kj = Frequency factor to be determined as shown in Table 1 below

Recurrence Interval (yrs)	5	10	15	25	100
Frequency Factor, Kj	0.719	1.339	1.684	2.108	3.211

Table 1: *Frequency factors of selected recurrence intervals*

The **MAP** (mean annual precipitation) of a drainage area can be determined using the **County Flood Control District's isohyetal map, Attachment 6**. This requires the engineer to locate the center of gravity of the entire drainage area above the point of concentration at which the flow rate is being determined.

1.3- C' (Runoff Coefficient)

The city requires a modified runoff coefficient, C', to be used in the design of drainage facilities. C' is made up of a basic runoff coefficient, C, a ground slope factor, Cs, and a rainfall intensity factor, Ci. The following sections contain procedures to determine C, Cs, and Ci. The formula for calculating the modified runoff coefficient is:

$$C' = C + Cs + Ci \qquad (4)$$

County Flood Control District **Attachment 10** or the formulated procedures below may be used to determine the value for C'.

Basic Runoff Coefficient, C, shall be chosen to reflect the ultimate development of the drainage area. Ultimate development will normally be based on City's general plans. If the general plans are not available, then a reasonable estimate of ultimate land use shall be made. The basic runoff coefficient, C, is a function of the percent of the watershed that is impervious, and the hydrologic soil group (HSG).

The percent impervious is normally based on land use category; however, it may be measured. In order to determine the appropriate C-value for any given drainage area, the applicable land use is overlaid onto the site map for an area-weighted average C-value (refer to Table 2). Two alternative percent impervious values are provided for 1/8 acre and 1/4 acre residential land use. Older residential areas typically generate less runoff than newer developments. This is because runoff from a significant amount of impervious areas in older developments flows over pervious areas before reaching the storm drain system, and is therefore, reduced.

Land Use Description Impervious	Percent Impervious	Oakland Hydrological Soil Group, D
Undeveloped land, Parks Golf Courses	0%	0.30
Older Residential 1/8 Ac. (5000—7900 SF lots)	24%	0.44
1980 and Newer Residential 1/8 Ac.	50%	0.60
Older Residential 1/4 Ac. (8000—11050 SF lots)	22%	0.43
1980 and Newer Residential 1/4 Ac.	40%	0.54
Residential Zero Lot Line 3600 SF lots	75%	0.75
Residential Duets 4500 SF lots	69%	0.71
Commercial / Industrial *	85%	0.81
Townhouse	68%	0.71
Apartment	89%	0.83
Rural Housing	11%	0.37
Freeway	100%	0.90

Table 2: *Runoff coefficients, C, for corresponding impervious area*

** For Industrial land use, 85% impervious applies to industrial areas that are nearly completely covered with structures and pavement. For other types of* *industrial areas, where large areas of bare ground are present, an appropriate runoff coefficient should be calculated based on measured impervious area.*

For conditions not covered by Table 2, the engineer shall calculate an appropriate runoff coefficient based on impervious area determined using aerial photographs and site plans. The runoff coefficient is calculated based on an area weighted average using C=0.9 for all impervious areas.

Slope Adjustment Factor, ***Cs***, is used to adjust for increases in runoff as the average slope of the incremental drainage area increases. An area weighted average slope, S, for the area shall be calculated as a basis for determining *Cs*.

$$Cs = \frac{(0.8 - C)[\ln(S - 1)]S^{0.5}}{56} \quad \text{, for } C \geq 0.8,\ Cs = 0 \qquad (5)$$

where:

Cs = slope adjustment runoff coefficient

S = average slope in percent

C = base weighted runoff coefficient

Rainfall Intensity Factor, *Ci*, is used to account for the decrease in soil permeability that can be expected with an increase in rainfall intensity.

$$Ci = [0.8 - (C + Cs)] \times \left[1 - \frac{1}{\frac{1}{e^{i}} + \ln(i + 1)}\right] \text{, for } C + Cs \geq 0.8,\ Ci = 0 \qquad (6)$$

where:

C*i* = rainfall intensity adjustment factor

C = base weighted runoff coefficient
Cs = slope adjustment runoff coefficient

i = rainfall intensity in inches/hour from **County Flood Control District** **Attachment 9** or Equation 3.

1.4- Peak Discharges

When using the modified Rational Method, the peak discharge of a tributary area must be compared to the peak discharge in the main stem downstream of the tributary. The higher of the two peak discharges is the governing peak discharge in the main stem downstream of the tributary.

2- Synthetic Flow Hydrograph

Synthetic Flow Hydrograph Method transforms a hypothetical rainfall distribution and design rainfall depth into a design runoff hydrograph. This method is suitable for the analysis of drainage network for watersheds larger than 0.5 square mile and is required for the design of detention/retention basins as shown in section 2.1.

Rainfall that does not result in runoff is identified as losses or abstractions. Losses include surface ponding and infiltration.

Two types of abstractions are considered in a typical storm event. First is the initial abstraction or the initial loss which no runoff will occur prior to any ponding of water. This is the total amount of water that is intercepted and absorbed in to ground before runoff begins. Once runoff begins, the soil continues to absorb some infiltrated water known as the uniform abstraction. Uniform abstraction follows the initial abstraction and accounts for the additional water retained in the watershed and depends on the hydrologic soil group and the type of cover.

2.1- Initial Abstraction

The Initial Abstraction (I_a) shown in table 3 for a 6-hour and 24-hour design storm event shall be used. I_a was developed by the Alameda County Flood Control District based on long-term stream flow gage records.

Design Storm (hr)	Initial Losses (inches)
6	0.8*

Table 3. I_a , Initial Abstractions for Design Storm Events (initial Loss Rates).

* Values are not applicable for recurrence intervals less than 5 years.

2.2- Uniform Abstraction

The uniform abstraction (F_a) varies based on coverage type (Rural, New Urban and Existing Urban) as well as hydrologic soil group. Table 4 provides the F_a values for all soil groups. Oakland consists mostly of hydrological soil group D. Soil groups are classified as A (highest infiltration) to D (lowest infiltration).

Hydrologic Soil Group	Rural Coverage	New Urban Coverage	Existing Urban Coverage
A	0.45	0.45	0.45
B	0.35	0.37	0.40
C	0.14	0.19	0.25
D	0.05	0.07	0.09

Table 4. F_a, Uniform Abstractions for Soil Group and Coverage Type

2.3- Watershed Runoff Computation

The City of Oakland has 15 major watershed areas subdivided into catchments for more refined calculations. For catchment areas were contour maps or Watershed Maps cannot resolve, the actual watershed boundaries shall be field verified to identify the appropriate catchment boundaries. The less developed areas may conform more to the natural earth patterns where the developed and urban areas are better defined by improvements and facilities in the area.

2.3.1- Precipitation

Except for the design of drainage detention facilities, the design rainfall depth shall be for a 15 year, 6 hour duration storm ($P_{15\text{-}6}$) as determined by using the following equation:

$$P_{ij} = (0.33 + 0.091144 * MAP) * (0.249 + 0.1006 * K_j) * T_i^{0.43747} \qquad (7)$$

where

P_{ij} or P = Design rainfall depth (inches) for a 15 year, 6 hour duration storm

MAP= Mean Annual Precipitation (inches). Refer to **Alameda County Flood Control Attachment 6.** An interpolated value between isohyetal lines at the drainage area centroid provides the MAP value to be used.

T_i = Storm duration (6 hours)

K_j = Frequency factor. Refer to table 1 for 15-year storm recurrence interval

Design Storm (years)	**5**	**10**	**15**	**25**	**100**
Frequency Factor (K_j)	0.719	1.339	**1.684**	2.108	3.211

2.3.2- Excess Precipitation and Modified Rainfall Runoff Hydrograph

Excess precipitation (P_e), or direct runoff is the rainfall minus the abstractions and is varied over time distribution

$$P_e = P - I_a - F_a \qquad (8)$$

Where:

P_e = Excess precipitation for ¼ hour increments (Inches)

P = Total precipitation for the 15 year, 6 hour storm (Inches)

I_a = Initial abstraction (Inches)

F_a = Uniform abstraction (Inches)

Excess precipitation over time (time-step graph) is calculated for each ¼ hour increment by the following method:

Excess rainfall hyetograph

Multiply the time-step values for the 6-hour design storm shown in the **Alameda County Flood Control District** **Attachment 11** by the total rainfall amount (P) obtained from equation (7). The result is a rainfall hyetograph for each ¼ hour increment, also know as the time-step hyetograph. Subtract uniform abstraction (F_a) shown in Table 4 from each ordinate. Subtract initial abstraction (I_a) shown in Table 3 from the ascending limb of the rainfall hyetograph only.

The result is a modified rainfall runoff hydrograph with each ordinate representing excess precipitation or runoff in inches per ¼ hour increment.

2.3.3- Runoff Flow Hydrograph

Follow procedures listed in equations 9 through 12 for the Drainage Network Runoff Flow Hydrograph for Watersheds Larger than 0.5 Square Mile.

$$Q_{1/4\ hour} = 0.248 \times (P_e \times A) \qquad (9)$$

Where
Q ¼ hour = Flow for each ¼ hour of excess precipitation (Cubic Feet per Second)
Pe = Excess precipitation (Inches per ¼ hour increment)
A = Drainage area (Acres)

With the Pe values defined in Equation (9), calculate the runoff flow hydrograph showing time step flows every ¼ hour.

2.3.4- Routing of Hydrograph & Computer Modeling Criteria

To generate a basin's flow hydrograph, computer programs may be necessary. However, the Engineer must follow the criteria below as the input data.

Calculate the potential maximum retention

$$S = \frac{(P - I_a)^2}{P_e} - P + I_a \qquad (10)$$

Where
S = Potential maximum retention (inches)
Pe = Excess precipitation (inches)

P = Total Precipitation (inches)
Ia = Initial abstraction (inches)

Calculate the curve number (CN)

$$CN = \frac{1000}{S+10} \qquad (11)$$

The dimensionless, calculated CN is for normal antecedent moisture condition (AMC II). To alter for either dry (AMC I) or wet (AMC III), equivalent CNs can by computed by

$$CN(I) = \frac{4.2CN(II)}{10-0.058CN(II)} \qquad (11)$$

$$CN(III) = \frac{23CN(II)}{10+0.13CN(II)} \qquad (12)$$

Calculate the Lag Time for catchments (time elapsed between the occurrence of unit rainfall and the occurrence of unit runoff) using the hydraulic length of the catch basin, the CN, and the average catchment slope.

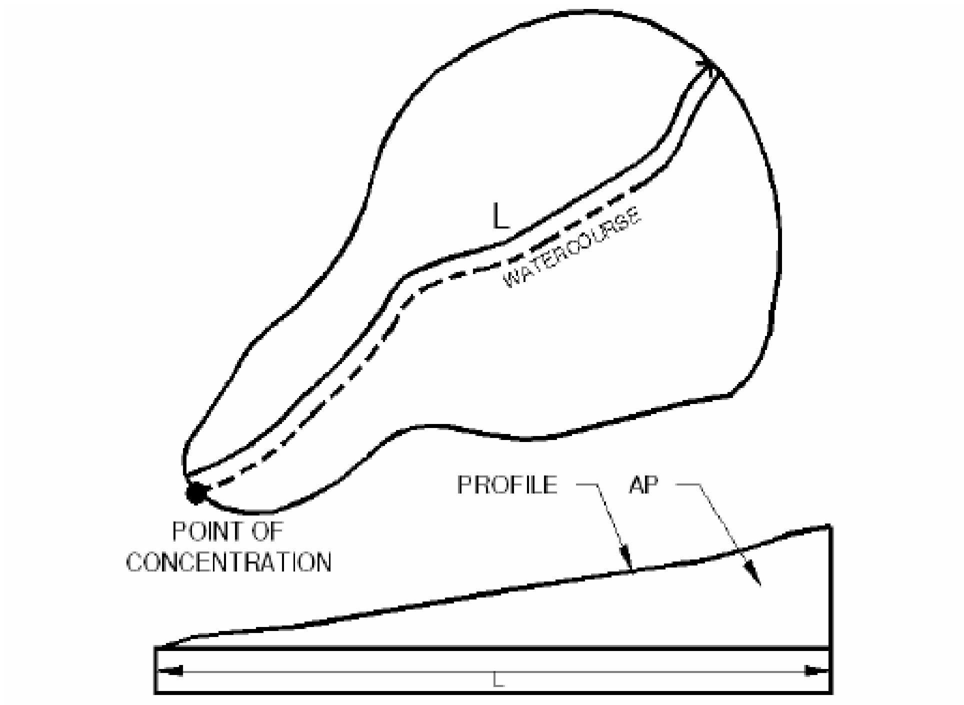

Figure 1. Hydraulic Length, L

$$T_{Lag} = \frac{L^{0.8}\left(\frac{1000}{CN} - 9\right)^{0.7}}{1900Y^{0..5}} \qquad (13)$$

Where
TLag = Lag time (minutes)
L = Hydraulic length (feet)
CN = curve number (dimensionless)
Y = Average catchment slope (percent)

HYDRAULIC DESIGN

The City generally uses Manning's equation to calculate friction losses to determine water surface profiles, hydraulic grade lines, and the conveyance method. For information and data related to Major Facilities, the engineer should contact the Alameda County Flood Control and Water Conservation District for a copy of the Alameda County Hydrology and Hydraulics Manual for the 100-year recurrence interval floods, design floodwater surface elevations, and the affected Primary or Secondary Facilities.

1- Freeboard Requirements

Freeboard for all facilities shall be as follows:

Facility	Freeboard (ft.)	From Design HGL up to:
Closed Conduit	1.25	top of Curb (1)
Non-leveed Channels	1.0	top of bank*
Leveed Channels	1.0	top of Levee*
Street Crossings 100-year	2.0	top of conduit or provide design storm capacity

* For Primary and Facilities in FEMA study areas, refer to the Alameda County Flood Control and Water Conservation District for the Alameda County Hydrology and Hydraulics Manual

(1), * Maximum energy grade lines shall be below the top of bank for open channels and below the top of curb for closed conduits

2- Hydraulic Profile

2.1- Friction Losses

The Mannings Formula shall be used to calculate hydraulic profiles. The friction value “n” is as follows:

Type of Facility	“n”
Reinforced Concrete Pipe	Conduit > 36” diameter 0.012
	Conduit ≤ 36” diameter 0.014
Corrugated Metal Pipe	Annular 0.021 Helical 0.018
Concrete-Lined Channels	Smooth-troweled 0.015
ACPWA Simulated Stone	0.017
Reinforced Concrete Box	Cast-in-Place 0.015 Pre-Cast 0.014
Gabions, mats, and other patented designs	Consult manufacturers specs.
Channels with Natural Surface	Smooth Geometry 0.030 minimum

Other lining materials shall be investigated by the engineer for the proper roughness and geometry. For open channels and closed conduits, the n-value may vary depending upon construction methods, maintenance procedures, and materials involved. As explained in Reference 12, n-values are subject to variation and that variation can cause changes in computed water surface elevations. Engineering assessment of the finished product and alignment is required for the variation in the n-values used for channel or closed conduit. The n-values vary by plus or minus 1/3; and as a result, the sensitivity of the design must be checked for scour in the lower range of n-value and exceeding the freeboard in the higher range.

For curved channels or closed conduits, the “n” value should be increase as follows:

$$+n = \frac{0.29}{R} \qquad (14)$$

where:
+n = adjustment (to be added) to the "n" selected for the facility
R = Radius of curvature at centerline in feet.
Note: For radii less than 20 feet bend losses shall be calculated.

2.2- Junction Losses

At points of change in the hydraulic parameters of flow rate or section, the HGL and Energy Grade Line (EGL) shall be calculated considering velocity heads and losses due to bends, entrances, exits, turbulence, etc. This Pressure-Momentum method should be used to calculate the change in water surface at major junctions and section changes with a corresponding recalculation of the EGL, as follows:

$$\Delta y = \frac{Q_2V_2 - Q_1V_1 \cos \theta_1 - Q_3V_3 \cos \theta_3}{\frac{g\left(A_1 + A_2\right)}{2}} \qquad (15)$$

where:

Δy = Change in hydraulic gradient through the junction (ft).
Q = Flow in cubic feet per second (cfs).
V = Velocity (ft/s).
Q_2 = Exit discharge
Q_1 = Inlet discharge
Q_3 = Lateral discharge ($Q_2 = Q_1 + Q_3$)
θ_1 = Angel of convergene between the center line of the main line and the center line of the lateral (degrees).
θ_1 = Angle of the deflection between the upstream and downstream center lines (degrees).
g = Acceleration due to gravity, 32 ft. per sec^2.
V_1 = Velocity of Inflow (fps)
V_2 = Velocity of Outflow (fps)
V_3 = Velocity of Lateral (fps)
A_1 = Area of Flow (ft^2) of Inlet
A_2 = Area of flow (ft^2) of Outlet

Energy equations should be used to calculate the effect a section change has on the EGL and then compared with the Pressure-Momentum results. The higher of the two is to be used.

2.3- Limiting Flow Velocities, Minimum Invert Slope

Where velocities are greater than 14 ft/sec, special criteria shall be established on a case by case basis to provide for scouring, maintenance or uneven flow conditions. For cases where velocities are outside these ranges, approval is needed from the City.

Facility	Min. Velocity (ft/sec)	Max.Velocity (ft/sec)	Min.Slope (ft/ft)
Earth Channels	2.0	6.0	
Concrete Lined Channels	2.0	14.0	0.007
Closed Conduits	3.0	14.0	

2.4- Hydraulic Jump and High Flow Velocities

Hydraulic jumps occur when the depth of flow changes rapidly from a low stage to a high stage (subcritical to super-critical flows). Where hydraulic jumps are likely to occur, such as where the slope or cross section of the facility changes in supercritical flow, their locations and energy losses shall be determined and considered in the design. See Reference 2, pp. 393-434.

High velocity flows which result from facilities on steep slopes shall consider roll waves and pulsating flows in the design. Roll waves are created when the normal depth of flow is within ten percent of the critical depth for the section. This condition should be avoided. Slug Flow is pulsating flow of waves which tend to amplify.

The Vedernikov Number, or V-No., is a measure of the tendency for supercritical flow stability. Where the V-No. is greater than unity, any wave created in the facility will tend to amplify up to a maximum height of 1.65 times normal depth, given a suitable length of run. Where this condition cannot be avoided, closed conduits shall be sized such that normal depth does not exceed half the depth of the conduit, and open channels shall be lined at least to 1.70 times normal depth. See Reference 2, pp. 210-211.

2.5- Effects of Curvature

In open channels of curved alignment, the rise in the water surface due to super-elevation and cross waves shall be considered. Super-elevation is the rise in water surface around a bend in a channel due to centrifugal force. The rise in the water surface is given by:

$$h = \frac{V^2 b}{2 g r_c} \qquad (16)$$

where:

Δh	=	rise in water surface (ft)	b	=	channel width at the water surface
V	=	velocity (ft/s)	r_c	=	radius of channel centerline's curve
g	=	gravity constant			

Cross waves occur in supercritical flow and should be considered in design. See Ref 2, Page 448.

2.6- Air Entrainment

Velocities above 14 ft/sec entrain air. An increase depth may result, with this depth being related directly to the increase in the volume of water.

$$A_a = 10 \cdot \left[\frac{0.2V^2}{gR} - 1 \right]^{0.5} \qquad (17)$$

where:

A_a	=	increase in flow area attributable to air entrainment (percent)
V	=	velocity at normal depth (ft/s)
R	=	hydraulic radius without air entrainment
g	=	acceleration due to gravity, 32.2 ft per sec^2

See Reference 2, p. 37.

DEBRIS AND SEDIMENT BASINS

Debris and sediment basins may be required in the design of certain drainage control facilities and the need for such structures shall be determined on a site-by-site basis.

The National Pollutant Discharge Elimination System (NPDES) permit program prohibits the unauthorized discharge of pollutants from a point source (pipe, ditch, well, etc.) to U.S. waters. Erosion control measures must be designed and maintained to prevent excessive amounts of sediment and debris carried into the storm water system. If sedimentation basins are required to prevent the flow of debris and sediments, installation and maintenance of such facilities are the responsibilities of the project initiator or the project owners.

CONVEYANCE FACILITIES MINIMUM REQUIREMENTS

Minimum pipe size shall be 12 inches in diameter and the pipe materials for buried pipe installation shall be RCP class III or HDPE SDR 11 minimum. Pipes shall not decrease in flow cross sectional area or diameter in the downstream direction.

Minimum bottom width for open channels with established vegetated bottoms and sides shall be four feet. For these channels, the ratio of side slope shall be no steeper than two and one half (2 1/2) horizontal units to one (1) vertical unit.

Minimum bottom width for improved channels (concrete or paved) shall be no less than two feet and the side slopes shall be no steeper than one (1) horizontal to one (1) vertical for concrete, and two (2) horizontal to one (1) vertical for reinforced earth with vegetation.

Access manholes or access structures for underground storm drainage conduits shall not exceed 400 feet on center. Inlets are considered access structures.

STORM DRAINAGE SUBMITTAL REQUIREMENTS AND STANDARDS

Refer to **Storm Drainage Impact Checklist** for the planning purposes and feasibility studies and refer to **Storm Drainage Project Review Checklist** for preparation of submittals. As applicable, refer to the City's standard details for drainage structure and installation requirements **City of Oakland Storm Drainage Standard Details**

EASEMENT WIDTHS AND SET-BACK REQUIREMENTS

For all new drainage facilities, the City requires 10-foot wide easements. In addition, an influence zone above the pipe shall be kept clear from any permanent structure. The influence zone (or set-back) is defined as an area starting from the edges of the underground facility to the surface having a one-to-one-slope. Permanent structures are defined as foundations, roofs, walls, balconies, and other improvement that may not easily be relocated or removed by the property owners during the repair or maintenance of the dedicated underground facility.

For creek-side properties, refer to the City's Creek Protection Ordinance. http://www.oaklandpw.com/creeks/guide.htm.

For Creek - Design Guidelines for stormwater quality protection, please refer to http://www.oaklandpw.com/Page147.aspx.

RETENTION AND DETENTION FACILITIES

1- Storm Water Retention

Retention facilities are designed to contain approximately 25 percent of the mean annual precipitation regardless of the design storm frequency of the drainage facilities entering the facility.

1.1- Design Procedure

Retention facilities do not have surface outflow and rely instead on percolation and/or evaporation to dispose of runoff. The facility shall be designed such that the water surface returns to its original elevation within 48 hours, after the cessation of a 100-year, 24-hour rainstorm over the contributory watershed. The volume of storm water shall be calculated as follows:

$$V_w = 0.021\,(P)(A) \tag{19}$$

where:

V_w = volume of water to be stored (acre-feet)
P = annual precipitation at the center of gravity of the watershed basin (inches)
A = drainage area (acres)

There must be 1.0 foot of freeboard between the elevation of the storage volume (Vw) and the lowest adjacent ground.

2- Storm Water Detention

The City's drainage system may not have the required capacity and in general, the City's system is aged and may be unable to handle the flows. In most cases, and to the extent possible, the City requires developments to detain storm water.

Detention facilities are those facilities designed to reduce the rate of discharge from a drainage area into a receiving waterway. One of the common uses for a detention facility is to limit the augmented discharge rate from a development site. When such a facility becomes a permanent drainage feature, assurances for the continued maintenance of its design capacity must be provided for; i.e., maintenance by the property owners or a or private party through a maintenance agreement. Several types of detention facilities are acceptable to the City for controlling on-site the augmented storm discharge:

1. Parking lot detention for industrial/business development. Using this method requires the filing of notice with the beneficiaries of the improvement and the City. Parking lots shall provide pedestrian access through the ponded areas. Depths of ponding shall not exceed four inches (4").

2. Conduit storage can be utilized by oversizing the underground drainage facilities. Care should be taken to prevent siltation problems.

3. Channel storage can be utilized by oversizing open channel facilities. Care again should be taken to prevent siltation problems, and allowances must be made for a minimum capacity at a maximum silt buildup.

4. Multi-purpose facilities can be used as detention facilities such as park areas, tennis courts, parking areas, existing ponds and wetland areas, and landscaped areas.

The detention pond shall be designed such that the water surface returns to its base or starting elevation within 24 hours after the cessation of a 24-hour, 100-year storm.

2.1- Design Procedure

Detention basins shall be designed to store urban runoff from sites such that post-project discharge rate is maintained less than or equal to the pre-project peak discharges. In certain cases, a maximum allowable outflow rate may be specified by the City. Otherwise, follow the procedures stated below for the design of detention basis.

1. To the extent possible, for commercial and multi-unit development projects less than 50 acres, the City recommends the Modified Triangular Hydrograph Method with the goal of reducing the peak runoff into the City's storm by 25%.

2. To the extent possible, for commercial and multi-unit development projects larger than 50 acres, use Snyder Unit Hydrograph Method to calculate the detention volume and runoff with the goal of reducing the peak runoff into the City's storm drains by 25%.

3. To the extent possible, for single-family home site development or single lot improvements, the City requires reducing the peak runoff into the City's storm by 25%. Builders and developers are encouraged to employ BMP measures and concepts such as Bio-retention, Swales, Pervious Pavers, Rain Barrels, Cisterns, and Tree Wells as some of the commonly used methods to reduce stormwater peak flows. Refer to Single Family Home Detention/retention Volume Worksheet to calculate the volume of water to be detained at a particular site. As shown on the spreadsheet, the City allows developers to combine any of the

appropriate BMP measures within the development to capture, route, and attenuate stormwater runoff from the site.

4. For drainage areas less than 640 acres, calculate the 15- and 100-year existing conditions peak discharges based on the modified rational formula. For drainage areas greater than 640 acres, calculate the 15-year and 100-year existing conditions peak discharges using the Synthetic Hydrograph Method.

5. Develop future conditions runoff hydrographs for the 15- and 100-year flood events based on the unit hydrograph methodology. Use a Base Flow of 7cfs per square mile of drainage area for the 15-year hydrograph and 10 cfs per square mile of drainage area for the 100-year hydrograph.

6. Calculate the required detention basin storage based on the 100-year hydrograph. Determine the volume in the future conditions runoff hydrograph (2) that exceeds the existing conditions peak discharge (1) and multiply by a factor of 3. This is the first estimate of the required volume in the detention basin. Estimate the dimensions of the base and compute storage-elevation causes.

7. Size the outlet facility such that the existing conditions peak discharges for the 15- and the 100-year events (1) would not be exceeded given the inflow hydrographs (3). Size the outlet pipe(s) to discharge the 15-year existing conditions peak discharge with approximately 1-2 feet between top of pipe and spillway crest. Design an overflow spillway to pass the 100-year existing peak discharge at approximately 2 feet of freeboard above the spillway crest to the lowest adjacent grade.

8. Route the 15-year future land use conditions runoff hydrograph from (2) through the detention basin to verify that the maximum outflow through the storage and outlet pipe combination does not exceed the existing 15-year existing discharge.

9. Route the 100-year future land use conditions hydrograph from (2) through the detention basin to verify that the maximum outflow through the storage outlet pipe and spillway does not exceed the existing 100-year discharge. Be certain that the detention basin returns to its starting elevation used in the analysis within 24 hours of the end of the 100-year 24-hour storm event.

REFERENCES

1. **Hydrology and Hydraulics Manual published by the Alameda County Flood Control and Water Conservation District (County Flood Control District).**
2. U.S. Department of Agriculture Soil Conservation Service, "A Method for Estimating Volume and Rate of Runoff in Small Watersheds, SCS - TP - 149".
3. Chow, Ven Te, Open-Channel Hydraulics, 1959, McGraw-Hill Book Company, New York.
4. Water Resources Council, "A Uniform Technique for Determining Flood Flow Frequencies", December 1967.
5. Department of Water Resources, "Rainfall Analysis for Drainage Design Volume I, II, & III", October 1976.
6. Department of Water Resources, "Rainfall Depth-Duration-Frequency for California".
7. U.S. Weather Bureau and Soil Conservation Service, "Rainfall Intensities for Local Drainage Design in Western United States, Technical Paper No. 28".
8. U.S. Department of Agriculture, Soil Conservation Service, "National Engineering Handbook, Section 4, Hydrology," March, 1985, 210-VI-NEH-4.
9. Barnes, Harry H., Jr., Roughness Coefficients of Natural Channels, U.S. Geological Survey, Water Supply Paper No. 1849, Second Printing, 1977.
10. United States Department of Agriculture, Soil Conservation Service, "Engineering Handbook No. 5, Hydraulics, Supplement B, " 1956.
11. County of Alameda, "Watercourse Protection Ordinance of Alameda County," Ordinance Code of Alameda County, Title 7, Chapter 10.
12. U.S. Army Corps of Engineers, San Francisco District, "San Francisco Bay Tidal Stage vs. Frequency Study," October 1984.
13. U.S. Army Corps of Engineers, Hydrologic Engineering Center, "Accuracy of Computed Water Surface Profiles," December 1986.
14. **The following links are provided to serve development or redevelopment projects in Oakland with conceptual ideas and useful information to attenuate, detain, retain and treat storm water runoff:**

    a. Idea for a Commercial Strip Parking Lot

    b. Idea for a Traffic Circle in Developments

    c. Idea for Low Impact Development single family residents

    d. Idea for Small Subdivision

    e. Idea for Suburban Highrise Apartment

f. Pervious Concrete Analysis & Costs

g. Pervious Concrete Pavements (ConcreteNetwork)

h. Pervious Concrete Sample Spec 2003 revised

i. http://www.lid-stormwater.net/design_img/design_examples.htm for design examples.

j. Interlocking Concrete Pavement Institute PPT presentation on retention/detention, and BMP ideas.

k. Tree-inlet http://www.americastusa.com/filterra.html

15. City of Oakland Storm Drainage Standard Details

16. City of Oakland watershed improvement program http://www.oaklandpw.com/creeks/guide.htm

17. New NPDES stormwater requirements & frequently asked questions concerning provision c.3