City of Oakland

OAKLAND PUBLIC WORKS
250 FRANK H. OGAWA PLAZA
4TH FLOOR • OAKLAND • CALIFORNIA • 94612
(510) 238-3961 FAX (510) 238-7227 TTD (510) 238-3254

Bureau of Engineering & Construction

Storm Drainage Design Standards

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Michael J. Neary, P.E., Assistant Director   10/31/14 Date

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Michael Neary, P.E.
Assistant Director

Gus Amirzehni, P.E.
Principal Civil Engineer

Prepared by:
Kevin Kashi, P.E.

Contributors/Advisors:
Lesley Estes, Watershed & Stormwater Management

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Hydrology and Hydraulics Manual published by the Alameda County Flood Control and Water Conservation District (2003) is a primary reference for the City's Standards.
Other references are listed in the Reference section.

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1. INTRODUCTION

1.1. Purpose

The Storm Drainage Design Standards (**Standards**) provides design criteria, standards, policies, and procedures for storm drainage improvements within the City of Oakland (City). All storm drainage facilities shall be designed in accordance with these Standards, accepted engineering principles, and state and federal water quality regulations.

Oakland has guidelines to protect natural creek segments, creek vegetation, controlling bank erosion and natural character of creeks. For storm drainage improvements and projects inside or near watercourses and creek, the City has developed the Creek Protection Storm Water Management and Discharge Control Ordinance to comply with the City of Oakland Municipal Code, Chapter 13.16. Information regarding creek related projects may be obtained from the City of Oakland Creeks and Watershed Improvement Program website. In addition, information on storm water pollution prevention and methods to keep the creeks and the Bay clean and healthy may be obtained from the Alameda Countywide Clean Water Program website.

Procedures in this manual apply to the design of typical facilities. Where unusual circumstances exist, the engineer should contact the City for additional guidance and approval. The City's Standards will be updated on a continual basis to reflect changes in City practices.

1.2. Legal Authority

Refer to the City of Oakland Municipal Code (OMC) amendment to Chapter 13, Section 13.14 (Ordinance No. 12916 C.M.S.) for the authority to develop, implement, and enforce Storm Drainage Design Standards as part of the OMC.

1.3. Description of Oakland's Storm Drainage System

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, over 100 miles of open creeks, and 15,000 structures (mostly inlets, manholes, and catch basins). These facilities are both publicly and privately owned.

City-owned storm drainage facilities are typically located within easements and right-of-ways. Privately owned facilities typically occur within private properties and include above-ground drainage systems, creeks, and watercourses; most of these privately owned facilities are not 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.

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

1. Major Facilities
Major 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 owned and maintained by the Alameda County Flood Control District.

2. Primary Facilities
Primary Facilities are waterways and drainage facilities with tributary areas more than 50 acres and less than 25 square miles. These facilities mostly consist of creeks and larger improved waterways or drainage facilities. Most of these facilities are owned and maintained by the Alameda County Flood Control District.

3. Secondary Facilities
Secondary Facilities include waterways or drainage facilities with tributary areas equal or less than 50 acres. Most of the City's drainage facilities are under this category, including pipes, conduits, and drainage structures that are owned and maintained by the City.

For Major and Primary Facilities that belong to the Alameda County Flood Control District, refer to Zone 12 Map.

2. STORM DRAINAGE SUBMITTAL REQUIREMENTS AND STANDARDS

Refer to Appendix A of these Standards for Storm Drainage Impact Review Checklist. This includes a list of essential submittals needed for a drainage feasibility study. For projects abutting creeks, watercourses, lakes, and the estuary, refer to the following sources for information and requirements:

Oakland Creeks & Watershed Improvement Program

A Creek and Watershed Map is available and published by the Oakland Museum of California.

CEDA Planning and Zoning Creek Protection Permit Application

In general, storm drainage impact reports shall consist of hydrological, hydraulic, and geomorphologic studies with calculations supporting damage assessments and mitigation plans. The drainage report shall include watershed maps, elevation contour, hydrological elements, creek or pipe plan/profile, starting water surface elevations, energy grade lines, hydraulic grade lines, flow line elevations, surface elevations, flows, velocities with flow regime, freeboard, system loses, etc.

Prior to design and during feasibility studies, refer to the City's Sewer Sheets showing the storm drainage facilities. Note that verification and accuracy of the information shown in the City's records is the sole responsibility of the applicants.

The project applicant shall verify the ownership of the storm drainage facilities by referring to the City of Oakland's Sewer Sheets located at the City's Permit Counter located on the 2nd floor of 250 Frank Ogawa Plaza, Oakland, CA 94612. If the ownership of the facility is not clear, the applicant may proceed with the following sources of information:

a) Zone 12 Map for the Alameda County Flood Control District (ACFCD) facilities. Information on this map must be verified with the ACFCD offices located at 399 Elmhurst Street, Hayward, CA 94544.

b) Port of Oakland's Storm Drainage System for the Port's drainage facilities. The information on this map must be verified with the Port of Oakland offices located at 530 Water St # 3, Oakland, CA 94607

c) State of California Department of Transportation (Caltrans) District 4 offices located at 111 Grand Avenue, Oakland, CA,

For the City's storm drainage structures standard details, refer to the following: City Standard Details.

3. GENERAL DESIGN PRACTICES

This section presents the standards to which all City storm drain designs shall conform.

3.1. General Standards

All storm drain facilities are to be independent of sanitary sewer systems. In all newly developed areas and/or in all existing areas where new storm drainage facilities are required, the designs shall ensure that the system size and capacity will adequately accommodate the ultimate anticipated conditions and shall comply with the requirements stated in these Standards.

The City's storm drainage system is aged and the physical conditions of the conduits and structures continue to change. In order to reduce the impact of storm water on the City's storm drainage system, these Standards require a reduction of up to twenty-five (25) percent in the peak flow, to the extent possible, prior to discharge into City's drainage system.

Commonly utilized storm water quality control measures shown in the Alameda Countywide Clean Water Program (C.3 Requirements) such as pervious ground cover, grassy swales, tree wells, rain gardens, infiltration, retention, and other methods acceptable by the City, may be utilized in attenuation runoff and reducing impervious ground cover.

3.2. 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. Flow cross-sectional areas of pipes may not decrease in diameter in downstream directions.

Minimum bottom width for open channels with established vegetated bottoms and sides shall be no less than four (4) feet. For these channels, the ratio of side slope shall be no steeper than two-and-one-half (2.5) 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 and structures for underground storm drainage conduits shall not exceed 400 feet on center. Inlets may be considered access structures if installed directly above the storm drainpipe with unobstructed access.

3.3. Easement Widths and Setback Requirements

For all new drainage facilities, the City requires a minimum 10-foot wide easement and wide enough such that a 1:1 slope may be projected from the bottom outer limits of the storm drain conduit to the surface. Installation of permanent structures over easements shall be avoided.

Easements may have to be wider than 10 feet to accommodate safe access to the underground facility and prevent damage to any adjacent underground utilities and above ground structures. Structures located over easements must be removed, reinstalled, or relocated as needed by the property owners and at the property owners' expense for the maintenance of the City's storm drainage system by City forces or City contractors.

3.4. Debris and Sediment Basins

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

The National Pollutant Discharge Elimination System (NPDES) permit and the Alameda Countywide Clean Water Program prohibit unauthorized discharge of pollutants into U.S. waters. Erosion control measures must be designed and maintained to prevent excessive amounts of sediments and debris from being carried into the storm water system. If sedimentation basins are required, installation and maintenance of such facilities are the responsibilities of the project owners.

3.5. Retention Facilities

Retention facilities do not have surface outflow and rely instead on percolation and/or evaporation to dispose 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)

One foot of freeboard is required for all retention basins.

3.6. Detention Facilities

The City's drainage system may not have the required capacity to handle additional storm water. In most cases, and to the extent possible, the City requires that developments shall 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 limiting the discharge rate. Private parties such as developments or project owners are responsible for detention facilities. The following suggested development features may be utilized as detention basins:

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 (4) inches.

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. Care should be taken to prevent siltation problems, and allowances must be made for minimum capacity at maximum silt buildup.

4. Multi-purpose facilities can be used as detention facilities such as park areas, tennis courts, parking areas, and landscaped areas. Existing ponds and wetland areas may not be suitable to receive additional storm water or change in the flow of storm water due to existing ecological balance. Additional studies may be needed to add storm water to an existing pond or wetland.

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.

3.6.1- Design Procedure

Detention basins shall be designed to delay the flow of urban runoff from the development site such that post-project discharge rate would not exceed the pre-project flow rate. In addition, to the maximum extent possible, the existing peak discharge may be reduced by a factor of 25%. This goal may be achieved by including commonly used post construction and best management practice features that are proposed in the Alameda County-wide Clean Water Program (C.3 Requirements) and other resources listed in the Reference section of these Standards.

Procedures stated below are common in planning and designing detention systems and shall incorporate the aforementioned post construction best management practices for water quality control, pervious ground cover, and runoff attenuation.

1. For single-family homes and single lot improvements, builders and developers are encouraged to employ concepts of bio-retention, swales, pervious pavers, rain barrels, cisterns, tree wells, and other commonly used features listed in the Reference section of these Standards to treat the storm water and reduce the peak flow. The Modified Triangular Hydrograph Method with the method described in Section 6.1 of these Standards can be used to calculate the detentions volume.

2. For commercial and multi-unit development projects 50 acres and less, use the Modified Triangular Hydrograph Method with the method described in Section 6.1 of these Standards to calculate the detentions volume.

3. For development projects, grading, and alterations to ground cover exceeding 50 acres but less than 640 acres, calculate the existing 15- and 100-year peak discharges using the methods described in Section 6.1 of these Standards to establish a baseline (existing conditions) hydrograph. Create a new hydrograph using the same Standards to represent the development or changes in the ground cover and topography.

   Compare the new 100-year hydrograph with the baseline 100-year hydrograph for any increase in the flow. A continuous base flow of 7 cubic feet per second for the 15-year hydrograph and 10 cubic feet per second for the 100-year hydrograph may be considered. Detention volume shall be calculated by subtracting the baseline 100-year hydrograph from the new 100-year hydrograph multiplied by a factor of three.

Size the discharge outlet using the 15-year baseline flow in a manner that at least two feet of freeboard is provided from the water level in the detention facility to the crest of the overflow spillway. Design an overflow spillway to pass the 100-year baseline flow in a manner that at least two feet of freeboard is provided from the water surface over the spillway to the top of the dike protecting the detention facility. Be certain that the detention basin returns to the starting elevation within 24 hours of the end of the 100-year storm event.

4. TAILWATER ELEVATIONS IN FLOOD ZONES & DESIGN DISCHARGE FREQUENCIES

Primary facilities that flow into or may be located within Federal Emergency Management Agency (FEMA) study areas are subject to FEMA 100-year water surface elevation using the FEMA criteria. FEMA public flood and flood insurance maps may be accessed using the following link: http://msc.fema.gov/portal/search?AddressQuery=oakland%2C%20ca

Tutorial to understand FEMA flood maps may be accessed using the following link: http://www.fema.gov/media/fhm/firm/ot_firm.htm.

When a facility is located in the 100-year flood zone, it 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 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 Attachments 1, 2, and 3.

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.

In general, the design tailwater elevation for secondary and primary facilities shall be the water surface elevation of the receiving waterway.

5. ELEVATION DATUM

City of Oakland datum shall be used for all projects in the City.

The engineer shall assure that all elevations used for tides, cross sections, tailwater elevations, and bridge geometry are on the same vertical datum systems. Refer to Attachment 4. The City of Oakland's datum plane is 3.0 feet above the National Geodetic Vertical Datum of 1929 while the Port of Oakland's datum plane is 3.2 feet below.

6. RUNOFF DETERMINATION METHODS

Modified Rational Method and Synthetic Unit Hydrograph Method are accepted computation techniques to estimate the storm water runoff in watersheds. Rational Method is suitable for drainage areas less than 320 acres or half square mile. For drainage areas equal or greater than 320 acres, the Synthetic Unit Hydrograph Method is acceptable.

For projects that may alter the existing ground cover or the intensity of the storm water runoff, the project engineer shall submit documentation for review and comment as indicated in Appendix A of these Standards.

6.1. Modified Rational Method

Data used in Rational Method may be entered into a Data Table shown in Attachment 5 to show peak runoff net-effect analysis. The overall watershed shall be broken down into smaller contributing boundaries. The boundaries shall be established based on local topography 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)

6.1.1- 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.

Note: Post-construction best management practices and methods selected to comply with the C.3 Guidance shall be considered in calculations and shown on the Site Plan as indicated in the Storm Drainage Impact Review Checklist.

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 drainage system or the upstream end of a defined ditch or swale or other attenuating devices.

$$\text{For undeveloped watersheds, Initial Tc} = L / (60\ V) \qquad \textit{(2)}$$

Where:

Tc = Time of concentration in minutes

L = Overland flow length in feet

V = Overland flow velocity in feet per second from Attachment 6. Velocities in engineered swales and other facilities for water quality compliance are addresses in the C.3 Guidance. Commonly used post construction and best management practice features shall be include in these calculations for cumulative time of concentration.

For urbanized watersheds, the initial time of concentration shall be taken as the roof-to-gutter time including bio-retention, swales, retention tanks, and the time required for the water to flow from the street gutter to the first inlet of the storm drainage system.

For residential areas, five minutes shall be used for roof-to-gutter time. Any addition to this time of concentration must be supported by calculations routing the storm water through swales, rain gardens, basins, and other post construction best management storm water features.

Conduit Time is the time frame 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 drainage system.

Stream Time is the time frame for the water to flow in a natural stream. The velocity may be determined using Attachment 7.

6.1.2- 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 (in hours or the time of concentration in hours)

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

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

Table 1. Frequency factor of selected recurrence intervals

The MAP (mean annual precipitation) of a drainage area can be determined using Attachment 8. 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.

6.1.3- 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:

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

Attachment 9 or the 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 the 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	Runoff Coefficient, C 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 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 Attachment 10 or Equation 3

6.1.4- 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 stream downstream of the tributary. The higher of the two peak discharges is the governing peak discharge in the main stem downstream of the tributary.

6.2. 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 analysis of drainage network for watersheds larger than 320 acres. Synthetic hydrograph method is commonly used for the design of detention/retention basins.

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 as follows:

- Initial Abstraction. This is the total amount of water that is intercepted and absorbed into the ground before any runoff or ponding begins.
- Uniform Abstraction. This is the amount of water that continues to be absorbed into the ground as ponding or runoff from the site begins. The soil continues to absorb water known as uniform abstraction.

Uniform abstraction follows the initial abstraction and accounts for additional water retained in the watershed. This depends on the hydrologic soil group and the type of ground cover.

6.2.1- Initial Abstraction (Ia)

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 Abstraction (inches)
6	0.8*

Table 3. I_a, Initial Abstraction

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

6.2.2- Uniform Abstraction (Fa)

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 (Oakland)	0.05	0.07	0.09

Table 4. F_a, Uniform abstractions for soil group and coverage type

6.2.3- Watershed Runoff Computation

The site topographical maps shall be field verified to identify the appropriate catchment boundaries. The less developed areas may conform to the natural earth patterns where the developed or the urban areas are better defined by improvements and facilities in the area. The City's Sewer Sheets showing the existing storm drainage and sanitary sewer lines and the City's contour maps are available at the City's Permit Counter. Note: Accuracy of these maps are never guaranteed, they are intended to help the hydrologists with the planning phase of the project, and all information shown on the City's record drawings shall be verified.

http://www.oaklandnet.com/maproom/GPZ/HTML/CEDA_Disclaimer.html

City of Oakland GIS Application

6.2.3.1- Precipitation

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 Attachment 8.
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 5 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

Table 5. Frequency factor of selected recurrence intervals

6.2.3.2- 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:

Multiply the time-step values for the 6-hour design storm shown in Attachment 11 by the total rainfall amount (P) obtained from equation (7). The result is a rainfall hyetograph for each ¼ hour increment; known 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.

6.2.3.3- 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)

P_e = 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.

6.2.3.4- 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)

P_e = Excess precipitation (inches)

P = Total Precipitation (inches)

I_a = 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.

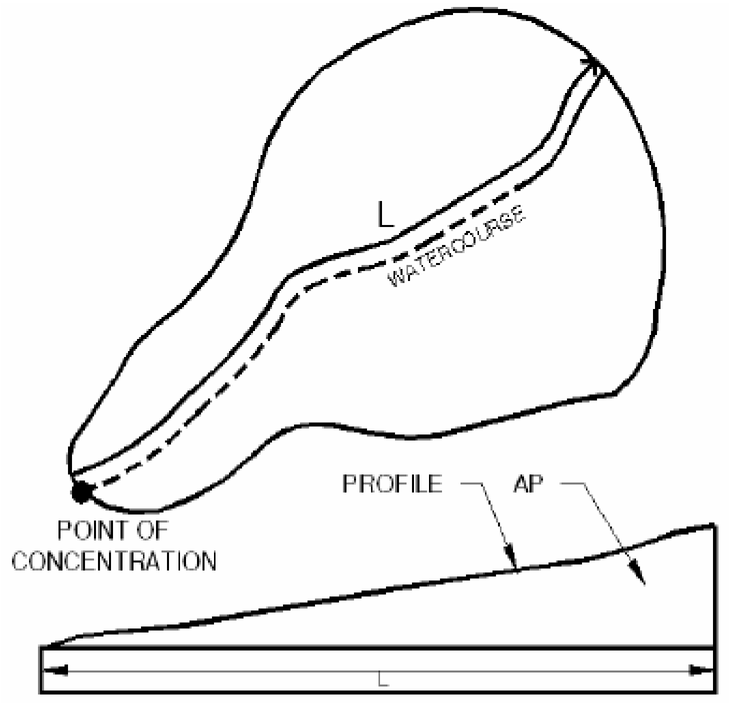

Figure 1. Hydraulic Length, L

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

Where:

T_{Lag} = Lag time (minutes)

L = Hydraulic length (feet)

CN = curve number (dimensionless)

Y = Average catchment slope (percent)

7. HYDRAULIC DESIGN

The City generally uses Manning's equation to calculate friction losses and determine water surface profiles and hydraulic grade lines.

7.1- 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	2.0	Top of conduit or provide 100-year design storm capacity

7.2- Hydraulic Profile

7.2.1- Friction Losses

The Manning's 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-trowel, 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 manufacturer
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. 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 increased as follows:

$$+n = \frac{0.29}{R} \tag{14}$$

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.

7.2.2- Junction Losses

At points of change in the hydraulic parameters, the hydraulic grade line (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:

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

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.

7.2.3- 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 the following:

- Scouring and maintenance
- Uneven flow conditions and sudden rise in the HGL

Design shall include provisions for the conditions when HGL rises to level that may cause flooding, pressure flow, and maintenance issues. 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	

7.2.4- 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.

Flows that may result in slug-flow or roll-wave during high velocity flows shall be avoided. Facilities on steep slopes shall be designed to avoid roll waves, created when the normal depth of flow is within ten percent of the critical depth for the section, and slug flow or pulsating flow which tend to amplify. When critical flows cannot be avoided, closed or open conduits shall be sized such that normal depth does not exceed at least one-half the depth of the conduit.

7.2.5- 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.

7.2.6- 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. Normal depth shall be increased to include the rise in the water level.

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

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

7.2.7- Gutter flow

Design the roadway and drainage facilities with adequate inlets and structures to restrict the width of water flow, also known as width of spread, from exceeding one-half of the parking lane. If no parking lane is made available, width of spread shall not exceed the lip of gutter.

Where parking lanes end and turn lanes begin or where bike lanes are designed, adequate number of inlets and structures shall be designed to prevent the storm water from reaching the travel lanes.

Storm water shall be collected upstream of all intersections, pedestrian walkways, wheelchair ramps, and other public access features. Gutter flow is prohibited across all public access features along the roadway.

APPENDIX A: STORM WATER IMPACT REVIEW CHECKLIST

Applicant/Project Name: ____

Project No. _________________________ Checked by ________________________

Date Received ______________________ Accounting Codes ___________________

Yes No N/A

_ _ _ 1) Has the project met a reduction of up to twenty-five percent in the peak flow to the extent possible (Section 3.1)?

_ _ _ 2) Can the City's storm drainage system handle any additional flows (Section 3.6)?

_ _ _ 3) Are detention facilities proposed for the development (Section 3.6)?

_ _ _ 4) Have the physical requirements for underground pipes, open channels sections, structures, manholes, and easements been met (Section 3.2 & 3.3)?

_ _ _ 5) Has the sediment flow prevention requirement been met (Section 3.4)?

_ _ _ 6) Is the project in the Federal Emergency Management Agency (FEMA) study areas and did the project meet the minimum design requirements (Section 4)?

_ _ _ 7) Has the design met the Alameda County Clean Water Program requirements stated in the C.3 Technical Guidance (Section 6)?

_ _ _ 8) Applicable design calculations addressing the clean water compliance stated in the C.3 Technical Guidance (Section 6)?

_ _ _ 9) Did the project follow the City's Storm Drainage Design Standards?

_ _ _ 10) Did the project meet the hydraulic design criteria for freeboard, velocities, and energy grade lines (Section 7.1 and 7.2)?

_ _ _ 11) Did the design calculation show adequate drainage structures to keep the gutter from exceeding its limits (Section 7.2.7)?

_ _ _ 12) Were the plan/profile prepared using the requirements shown in the Plan & Profile Guidelines?

8. REFERENCES

Hydrology and Hydraulics Manual (2003) published by the Alameda County Flood Control and Water Conservation District (County Flood Control District).

The following links are provided to serve professionals and developers with ideas and useful information to attenuate, detain, retain, and treat storm water runoff:

https://library.municode.com/index.aspx?clientId=16308&stateId=5&stateName=California
Oakland Municipal Code Chapter 13.16 - Creek Protection, Storm Water Management and Discharge Control

http://www.cleanwaterprogram.org/
Clean Water Program

Attachments/C3%20Technical%20Guidance%20June%202013.pdf C.3 Guidance (Alameda Countywide Clean Water Program)

http://www.cleanwaterprogram.org/uploads/ACCWP_Site_Design_Guidebook_final.pdf
Post-Construction BMP Examples (Alameda Countywide Clean Water Program)

http://strawberrycreek.berkeley.edu/pdfs/Start@Source/%20usingstartatthesource.pdf
Using Site Design Techniques to Meet Development Standards for Storm Water Quality (Bay Area Storm Water Management Agencies Association)

http://www.acgov.org/pwa/documents/brochure_9_05_final.pdf
Storm Water Quality Control Requirements & Information for Developers, Builders and Project Proponents (Alameda Countywide Clean Water Program)

https://www.casqa.org/resources/bmp-handbooks
Storm Water Best Management Practice Handbooks (California Storm Water Quality Association)