CHAPTER 6
DRAINAGE, FLOOD CONTROL AND EROSION CONTROL

This chapter presents the standards established for Drainage, Flood Control and Erosion Control within the City of Albuquerque. Detailed requirements r to facilitate the planning, design, construction and operation of both public and private drainage control, flood control, stormwater quality and erosion control facilities are covered as follows:

The standards, guidelines and criteria presented herein are provided in order to facilitate the planning, design, construction and operation of both public and private drainage control, flood control, stormwater quality and erosion control facilities within the community.

The criteria are not intended as a substitute for good engineering judgment; imagination and ingenuity are encouraged. The thrust of these criteria is toward generalization in order to provide guidance for a large majority of design circumstances, but it must be understood that situations will arise in which these criteria are not appropriate.

The City Engineer or the AMAFCA Executive Engineer, as appropriate, may, in specific cases, require more stringent criteria or allow relaxation of these criteria based on his judgment and sound engineering practice.

The DRB representative from the City Engineer's office acts as the designee of the AMAFCA Executive Engineer except in review of proposals involving major arroyos or platting outside the City Limits where there is no immediately pending proposed annexation.

GOVERNING REGULATIONS

The planning, design, construction and operation of both public and private drainage control, flood control, stormwater quality and erosion control facilities must be prepared according to the ordinances and policies listed in the *Drainage, Flood Control and Erosion Control Governing Regulations Summary,* found on the City of Albuquerque website. Some development plans will involve coordination with and approval by jurisdictions in addition to the City of Albuquerque, because the site drains to, or may impact, property in their jurisdiction, these agencies are also listed on the summary,

6-1 HYDROLOGY

AHYMO has been the primary method for hydrology calculations in Albuquerque since the DPM update in 1993, and it continues to be the basis for hydrology calculations. Other methods are allowed only if they agree closely with the AHYMO method. The "Procedures for 40 acres and Smaller Basins" is calibrated to exactly match AHYMO. In 1993, AHYMO replaced a rational method that had been derived from the SCS Curve Number method. One very specific version of the SCS Curve Number method is being allowed with this 2018 update because it agrees closely with AHYMO results.

The methods in the 1993 DPM were based on precipitation data from the NOAA Atlas 2 which has been superseded by NOAA Atlas 14. Atlas 14 Volume 1 Version 1 was published in 2001 Volume 4 in 2006 and the current Version 5 was published in 2011, and more revisions are expected as new data is collected. AHYMO- 93 and AHYMO-97 used the precipitation distributions from Atlas 2 and AHYMO- S4, released in 2009, uses precipitation distribution based on Atlas 14. The methods, graphs, and tables which follow will be used by the City of Albuquerque staff in the review and evaluation of development plans and drainage management plans.

Two basic methods of analysis are presented herein:

- Section 6-1(A) - describes a simplified procedure for smaller watersheds based on the Rational Method and initial abstraction/uniform infiltration precipitation losses. The procedure is applicable to watersheds up to 40 acres in size, but the procedure may be extended to include larger watersheds with some limitations
- Section 6-1(C) - describes two unit hydro graph procedures which are accomplished using computer programs. One method is the AHYMO method and the other method is the SCS Curve Number method. The AHYMO-S4 program is used for the AHYMO method and TR-20 and HEC-HMS are two of the programs that can be used for the SCS CN method and the Atlas 14 precipitation distribution. These procedures are applicable for small and large watersheds.

Section 6-1(B) describes the computation of time of concentration, lag time, and time to peak which are used in 6-1(A) and 6-1(C).

Section 6-1(D) contains a tabulated list of definitions of symbols used in this Section of the D.P.M. and a bibliography.

6-1(A) PROCEDURE FOR 40 ACRE AND SMALLER BASINS

A simplified procedure for projects with sub-basins smaller than 40 acres has been developed based on initial abstraction/uniform infiltration precipitation losses and Rational Method procedures. For this procedure, Bernalillo County has been divided into four (4) Precipitation Zones.

6-1(A)(1) PRECIPITATION ZONES

Albuquerque's four precipitation zones are indicated in TABLE 6.1 and on FIGURE A-1, and the corresponding precipitation values are in Table A-2. When modeling the storm, the standard practice is to set the peak intensity 1.5 hours into the storm when using AHYMO losses and 12 hours into the storm when using the SCS Curve Number losses Atlas 14 precipitation distributions must be used. Do not smooth the distribution and do not use the SCS precipitation distribution. The storm duration must be 24 hours and the calculation increment should be set to 5 minutes for the distribution used with the SCS method. The unit hydrograph time increment must be 0.01 hours or less. NOAA Atlas 14, available on the internet, can be used for several other frequency events, and it can be used to obtain a more precise precipitation depth for a particular location than the depths listed in Table A-2.

TABLE 6.1 PRECIPITATION ZONES

Zone	Location
1	West of the Rio Grande
2	Between the Rio Grande and San Mateo
3	Between San Mateo and Eubank, North of Interstate 40; and between San Mateo and the East boundary of Range 4 East, South of Interstate 40
4	East of Eubank, North of Interstate 40; and East of the East boundary of Range 4 East, South of Interstate 40 Not including the Cibola National Forest

Figure 6.1 **Precipitation Zones**

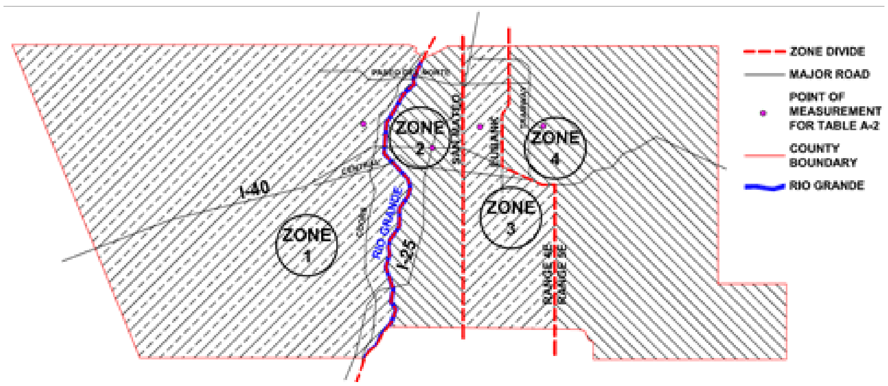


TABLE 6.2 PARTICIPATION FOR ZONES 1-4									
Partial Duration		500 year		100 year		10 year		2 year	
		Depth (in)	Intensity in/hr	Depth (in)	Intensity in/hr	Depth (in)	Intensity in/hr	Depth (in)	Intensity in/hr
ZONE 1									
5	min.	0.701	8.41	0.538	6.46	0.335	4.02	0.207	2.48
10	min.	1.070	6.42	0.819	4.91	0.511	3.07	0.315	1.89
12	min.	-	5.96	-	4.58	-	2.85	-	1.76
15	min.	1.320	5.28	1.020	4.08	0.633	2.53	0.390	1.56
30	min.	1.780	3.56	1.370	2.74	0.852	1.70	0.525	1.05
60	min.	2.200	2.20	1.690	1.69	1.060	1.06	0.650	0.65
2	min.	2.530	1.27	1.920	0.96	1.190	0.60	0.746	0.37
3	min.	2.760	0.92	2.000	0.67	1.250	0.42	0.800	0.27
6	min.	2.780	0.46	2.170	0.36	1.400	0.23	0.920	0.15
24	min.	3.090	0.13	2.490	0.10	1.680	0.07	1.160	0.05
4	day	3.780	0.04	3.120	0.03	2.190	0.02	1.560	0.02
10	day	4.680	0.02	3.900	0.02	2.760	0.01	1.970	0.01
ZONE 2									
5	min.	0.731	8.77	0.565	6.78	0.355	4.26	0.220	2.64
10	min.	1.110	6.66	0.860	5.16	0.540	3.24	0.335	2.01
12	min.	-	6.20	-	4.81	-	3.01	-	1.87
15	min.	1.380	5.52	1.070	4.28	0.669	2.68	0.415	1.66
30	min.	1.860	3.72	1.440	2.88	0.901	1.80	0.559	1.12
60	min.	2.300	2.30	1.780	1.78	1.120	1.12	0.692	0.69
2	min.	2.660	1.33	2.030	1.02	1.260	0.63	0.797	0.40
3	min.	2.730	0.91	2.100	0.70	1.320	0.44	0.844	0.28
6	min.	2.980	0.50	2.290	0.38	1.480	0.25	0.977	0.16
24	min.	3.210	0.13	2.590	0.11	1.760	0.07	1.220	0.05
4	day	3.590	0.04	2.960	0.03	2.070	0.02	1.470	0.02
10	day	4.330	0.02	3.620	0.02	2.560	0.01	1.830	0.01

TABLE 6.2 PARTICIPATION FOR ZONES 1-4

Partial Duration		500 year		100 year		10 year		2 year	
		Depth (in)	Intensity in/hr	Depth (in)	Intensity in/hr	Depth (in)	Intensity in/hr	Depth (in)	Intensity in/hr
ZONE 3									
5	min.	0.753	9.04	0.584	7.01	0.368	4.42	0.228	2.74
10	min.	1.150	6.90	0.889	5.33	0.560	3.36	0.348	2.09
12	min.	-	6.41	-	4.96	-	3.12	-	1.94
15	min.	1.420	5.68	1.100	4.40	0.693	2.77	0.431	1.72
30	min.	1.910	3.82	1.480	2.96	0.934	1.87	0.580	1.16
60	min.	2.370	2.37	1.840	1.84	1.160	1.16	0.718	0.72
2	min.	2.810	1.41	2.150	1.08		0.67	0.845	0.42
3	min.	2.890	0.96	2.220	0.74	1.400	0.47	0.895	0.30
6	min.	3.090	0.52	2.430	0.41	1.570	0.26	1.010	0.17
24	min.	3.570	0.15	2.840	0.12	1.900	0.08	1.300	0.05
4	day	4.000	0.04	3.290	0.03	2.290	0.02	1.620	0.02
10	day	4.940	0.02	4.100	0.02	2.890	0.01	2.060	0.01
ZONE 4									
5	min.	0.798	9.58	0.624	7.49	0.398	4.78	0.249	2.99
10	min.	1.210	7.26	0.950	5.70	0.606	3.64	0.380	2.28
12	min.	-	6.77	-	5.31	-	3.38	-	2.12
15	min.	1.510	6.04	1.180	4.72	0.751	3.00	0.471	1.88
30	min.	2.030	4.06	1.590	3.18	1.010	2.02	0.634	1.27
60	min.	2.510	2.51	1.960	1.96	1.250	1.25	0.784	0.78
2	min.	3.010	1.51	2.330	1.17	1.470	0.74	0.933	0.47
3	min.	3.120	1.04	2.420	0.81	1.530	0.51	0.991	0.33
6	min.	3.340	0.56	2.640	0.44	1.730	0.29	1.150	0.19
24	min.	4.490	0.19	3.600	0.15	2.400	0.10	1.640	0.07
4	day	5.910	0.06	4.750	0.05	3.200	0.03	2.200	0.02
10	day	7.760	0.03	6.270	0.03	4.260	0.02	2.950	0.01

The principal design storm is the 100-year event defined by the NOAA Atlas 14 Volume 1 Version 5, and subsequent updates. Tables A-2, A-8, and A-9 will be updated when NOAA Atlas 14 precipitation depths are updated. For certain applications (e.g., street drainage, low flow channels and sediment transport) storms of greater frequency than the 100-year storm must be considered and the 500-year storm is used for some floodplains.

6-1(A)(2) LAND TREATMENTS

All land areas are described by one of four basic land treatments or by a combination of the four land treatments. Land treatments are given in TABLE 6.3.

TABLE 6.3 LAND TREATMENTS

Treatment	Land Condition
A (CN=77)	Soil uncompacted by human activity with 0 to 10 percent slopes. Native grasses, weeds and shrubs in typical densities with minimal disturbance to grading, ground cover and infiltration capacity.
B (CN=79)	Irrigated lawns, parks and golf courses with 0 to 10 percent slopes. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes greater than 10 percent and less than 20 percent.
C (CN=86)	Soil compacted by human activity. Minimal vegetation. Unpaved parking, roads, trails. Most vacant lots. Gravel or rock (desert landscaping). Irrigated lawns and parks with slopes greater than 10 percent. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes at 20 percent or greater. Native grass, weed and shrub areas with clay or clay loam soils and other soils of very low permeability as classified by SCS Hydrologic Soil Group D.
D (CN=98)	Impervious areas, pavement and roofs. Ponds, channels and wetlands, even if seasonally dry.

Most watersheds contain a mix of land treatments. To determine proportional treatments, measure respective subareas. For large developed basins the areal percentages in "TABLE 6.4 Percent Treatment D (Impervious)"

TABLE 6.4 PERCENT TREATMENT D (IMPERVIOUS)

Land Use	Percent
Commercial*	90
Single Family Residential N=units/acre, N 6	$7 \sqrt{(N^2)} + (5N)$
Multiple Unit Residential Detached* Attached*	 60 70
Industrial Light* Heavy*	 70 80
Parks, Cemeteries	7
Playgrounds	13
Schools	50
Collector & Arterial Streets	90

**Includes local streets*

"TABLE 6.4 Percent Treatment D (Impervious)" does not provide areal percentages for land treatments A, B and C. Use of _ will require additional analysis to determine the appropriate areal percentages of these land treatments.

6-1(A)(3) ABSTRACTIONS

Initial abstraction is the precipitation depth which must be exceeded before direct runoff begins. Initial abstraction may be intercepted by vegetation, retained in surface depressions, or absorbed on the watershed surface. Initial abstractions are shown in TABLE 6.5.

TABLE 6.5 INITIAL ABSTRACTION

Treatment	Initial Abstraction (inches)
A	0.65
B	0.50
C	0.35
D	0.10

Infiltration is the only significant abstraction after the initial abstraction. After initial abstraction is satisfied, treat infiltration as a constant loss rate as specified in TABLE 6.6.

TABLE 6.6 INFILTRATION (INF)

Treatment	Loss Rate (inches/hour)
A	1.67
B	1.25
C	0.83
D	0.04*

**Treatment D infiltration rate is applicable from 0 to 3 hours; use uniform reduction from 3 to 6 hours, with no infiltration after 6 hours.*

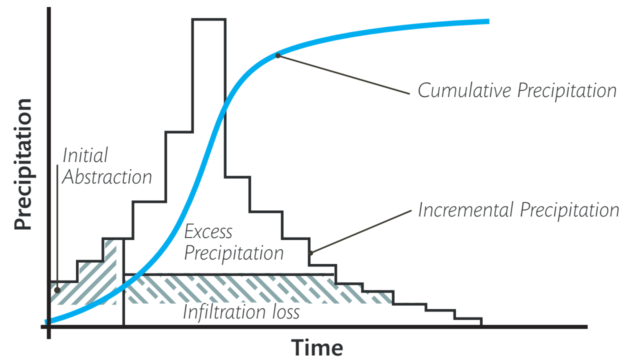
Runoff from a previous event can saturate a channel bed or pond bottom, rendering it minimally pervious for several days. Do not anticipate additional bed losses for design purposes.

6-1(A)(4) EXCESS PRECIPITATION & VOLUMETRIC RUNOFF

Excess precipitation, E, is the depth of precipitation remaining after abstractions are removed. Excess precipitation does not depend on watershed area.

Excess precipitation is determined by subtracting the initial abstraction and infiltration from the design storm hydro graph. Figure 6.2 illustrates the development of excess precipitation.

Figure 6.2 Precipitation and Time



The 6 hour excess precipitation, E, by zone and treatment is summarized in TABLE 6.7.

TABLE 6.7 8 - 6 HOUR EXCESS, 'E'

Zone	Land Treatment			
	A	B	C	D
100 YEAR EXCESS PARTICIPATION, E (IN)				
1	0.55	0.73	0.95	2.24
2	0.62	0.80	1.03	2.33
3	0.67	0.86	1.09	2.58
4	0.76	0.95	1.20	3.34
2 YEAR EXCESS PARTICIPATION, E (IN)				
1	0.00	0.01	0.13	0.92
2	0.00	0.02	0.16	0.98
3	0.00	0.05	0.19	1.05
4	0.00	0.28	0.87	1.39
10 YEAR EXCESS PARTICIPATION, E (IN)				
1	0.11	0.26	0.43	1.43
2	0.15	0.30	0.48	1.51
3	0.18	0.34	0.52	1.64
4	0.25	0.41	0.59	2.15

To determine the volume of runoff:

1. Determine the area in each treatment, A_A, A_B, A_C, A_D
2. Compute the weighted excess precipitation, E

EQUATION 6.1

$$\textbf{Weighted E} = \frac{E_A A_A + E_B A_B + E_C A_C + E_D A_D}{A_A + A_B + A_C + A_D}$$

3. Multiply the weighted E by the watershed area.

EQUATION 6.2

$$\mathbf{V_{360}\ (as\ volume)} = \text{weighted } E^* (A_A + A_B + A_C + A_D)$$

EXAMPLE 1

Find the 100-year V_{360} for 30 acres in zone 1. Eight acres are treatment A, 10 acres are treatment B, 5 acres are treatment C, and 7 acres are treatment D.

Weighted E = ((8 * 0.55) + (10 * 0.73) + (5 * 0.95) + (7 * 2.24)) / 30
= 1.071 inches

Volume = (0.965 * 30) / 12 = 2.68 acre-ft. = V_{360}

For ponds which hold water for longer than 6 hours, longer duration storms are required to establish runoff volumes. Since the additional precipitation is assumed to occur over a long period, the additional volume is based on the runoff from the impervious areas only.

For 24-hour storms:
EQUATION 6.3 $\mathbf{V}_{\mathbf{1440}} = V_{360} + A_D * (P_{1440} - P_{360}) / 12$ in/ft

For 4-day storms:
EQUATION 6.4 $\mathbf{V}_{\mathbf{4DAYS}} = V_{360} + A_D * (P_{4DAYS} - P_{360}) / 12$ in/ft

For 10-day storms:
EQUATION 6.5 $\mathbf{V}_{\mathbf{10DAYS}} = V_{360} + A_D * (P_{10DAYS} - P_{360}) / 12$ in/ft

EXAMPLE 2

Find the 100-year 24-hour and 4-day runoff volume, V_{1440} and V_{4days}, for the area in *EXAMPLE 1*.

$\mathbf{V}_{\mathbf{360}}$ = 2.68 acre-feet

$\mathbf{V}_{\mathbf{1440}}$ = 2.68 + 7 ac * (2.49 - 2.17) / 12 = 2.87 acre-feet

$\mathbf{V}_{\mathbf{4DAYS}}$ = 2.68 + 7 ac * (3.12 - 2.17) / 12 = 3.23 acre-feet

6-1(A)(5) PEAK DISCHARGE RATE FOR SMALL WATERSHEDS

The peak discharge rate is given in Table A-9 for small watersheds, less than or equal to 40 acres, where the time of concentration is assumed to be 12 minutes.

TABLE 6.8 PEAK DISCHARGE

Zone	Land Treatment			
	A	B	C	D
100 YEAR PEAK DISCHARGE (CSF/ACRE)				
1	1.54	2.16	2.87	4.12
2	1.71	2.36	3.05	4.34
3	1.84	2.49	3.17	4.49
4	2.09	2.73	3.41	4.78
2 YEAR PEAK DISCHARGE (CSF/ACRE)				
1	0.00	0.02	0.50	1.56
2	0.00	0.08	0.61	1.66
3	0.00	0.15	0.71	1.73
4	0.00	0.28	0.87	1.88
10 YEAR PEAK DISCHARGE (CSF/ACRE)				
1	0.30	0.81	1.46	2.57
2	0.41	0.95	1.59	2.71
3	0.51	1.07	1.69	2.81
4	0.70	1.28	1.89	3.04

To determine the peak rate of discharge,

1. Determine the area in each treatment, A_A, A_B, A_C, A_D
2. Multiply the peak rate for each treatment by the respective areas and sum to compute the total Q_P.

EQUATION 6.6 **Total Q_P** $= Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$

EXAMPLE 3

Find 100-year Q_P for 14 acres in zone 1. The four land treatments are: 3 acres in treatment A, 5 acres in treatment B, 2 acres in treatment C and 4 acres in treatment D.

Total $Q_P = (1.54 * 3) + (2.16 * 5) + (2.87 * 2) + (4.12 * 4) = 37.64$ cfs

Approximately the same results can be achieved by a Rational Method solution. The 0.2-hour (12-minute) peak intensities, I, are given in TABLE 6.2 and Rational Method coefficients, C, are given in TABLE 6.9.

EQUATION 6.7 **Total Q_P =** $(C_A * I * A_A) + (C_B * I * A_B) + (C_C * I * A_C) + (C_D * I * A_D)$

TABLE 6.9 COEFFICIENT C

Zone	Land Treatment			
	A	B	C	D
100 YEAR PEAK DISCHARGE (CSF/ACRE)				
1	0.34	0.47	0.63	0.90
2	0.36	0.49	0.63	0.90
3	0.37	0.50	0.64	0.91
4	0.39	0.51	0.64	0.90
2 YEAR PEAK DISCHARGE (CSF/ACRE)				
1	0.00	0.01	0.28	0.89
2	0.00	0.04	0.33	0.89
3	0.00	0.08	0.37	0.89
4	0.00	0.13	0.41	0.89
10 YEAR PEAK DISCHARGE (CSF/ACRE)				
1	0.11	0.28	0.51	0.90
2	0.14	0.32	0.53	0.90
3	0.16	0.34	0.54	0.90
4	0.21	0.38	0.56	0.90

Note the quote from the ASCE Manual and Report on Engineering Practice No. 37 (1969): The commonly reported Rational C values "are applicable for storms to 5- to 10-yr. frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff." Thus higher C's realized under heavy precipitation might be expected.)

EXAMPLE 4 Recompute "EXAMPLE 3" using the Rational Method.

Q = C I A
= (0.27 * 4.02 *3) + (0.43 * 4.02 * 5) + (0.61 * 4.02* 2) + (0.93 * 4.02 * 4)
= 37.13 cfs

6-1(A)(6) USE OF RATIONAL METHOD FOR WATERSHEDS LARGER THAN 40 ACRES

Peak rates of discharge may be computed for watersheds larger than 40 acres by using the Rational Method Coefficients (C's) from Table 6.11 and modifying the Intensity (in/hr) for a larger time of concentration (t_c). This method may be used to establish peak flow rates for off-site flow areas when sizing channels, pipes and road crossings. On-site areas should be divided into 40 acre or smaller sub- basins and should not use this procedure. For watersheds larger than 40 acres, the rational method should not be used to establish allowable historic flow rates since it will tend to give somewhat larger values than those computed by unit hydro graph procedures.

The procedures outlined in 6-1(D) should be used to compute the time of concentration (t_C).

Then compute the Intensity (in/hour), using the time of concentration, t_C and linear interpolation between the intensities given in TABLE 6.2 to get the intensity corresponding to the tc calculated using the procedures in section 6-1(B).

Do not use this formula for t_c larger than 2.0 hours.

EXAMPLE 5

Find Q_P for a 100-year storm at a 120 acre watershed in zone 3, with a 2600 feet shallow concentrated flow upper subreach at 0.015 ft/ft slope and 1200 feet natural channel lower subreach at 0.02 ft/ft slope. The watershed is 50 percent treatment A, 20 percent treatment B, 10 percent treatment C and 20 percent treatment D.

Compute the time of concentration using TABLE 6.10 from Section 6-1(B) as follows:

With a reach length longer than 2000 feet, use K = 3 for the portion below the first 2000 feet.

Since total reach length (2600 + 1200) is less than 4000 feet use equations b-1 and b-2 from Section 6-4.

$\mathbf{t_c}$ = ((2000 / (10 * 2 * (0.015$^{0.5}$))) + (600 / (10 * 3 (0.015$^{0.5}$))) + (1200 / (10 * 3 * (0.02$^{0.5}$)))) / 60 = 21 min.

Compute the Intensity, I, using linear interpolation between the 15 min and 30 min 100 year intensities of 4.40 and 2.96 in/hr from TABLE 6.2 as follows:

I = 4.40-[(21-15)/(30-15)*(4.40-2.96)] = 3.82 inches/hour

Using equation EQUATION 6.7 and the percentage of treatment types:

When:

$\mathbf{A_A}$ = 120 * 0.50 = 60 acres
$\mathbf{A_B}$ = 120 * 0.20 = 24 acres
$\mathbf{A_C}$ = 120 * 0.10 = 12 inches
$\mathbf{A_D}$ = 120 * 0.20 = 24 acres

$\mathbf{Q_P}$ = (0.37 * 3.82* 60) + (0.50 * 3.82* 24) + (0.64 * 3.82 * 12) + (0.91 * 3.82 * 24) = 243.41 cfs

6-1(A)(7) HYDROGRAPH FOR SMALL WATERSHED

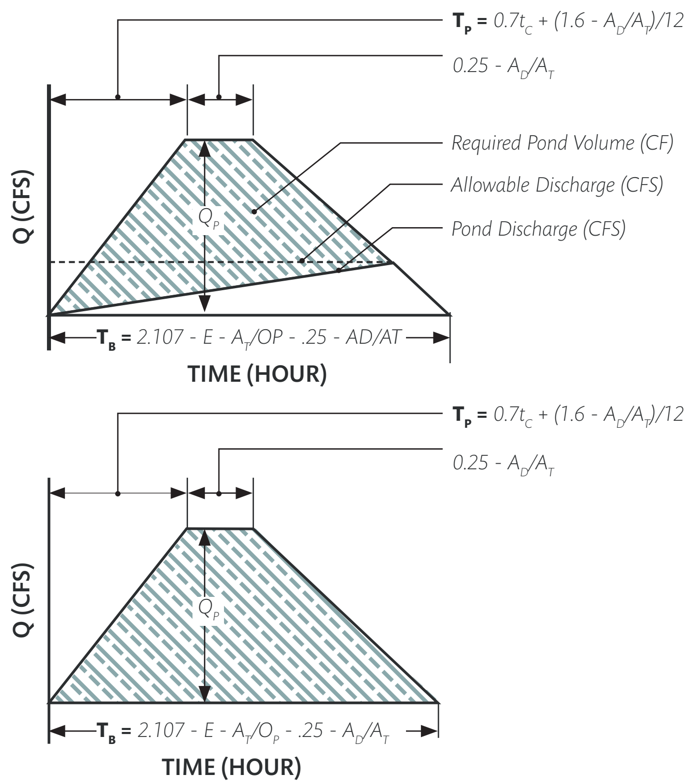
Base time, t_B, for a small watershed hydrograph is,

EQUATION 6.8 $\mathbf{t_B} = (2.107 * E * A_T / Q_P) - (0.25 * A_D / A_T)$

Where t_B is in hours, E is the excess precipitation in inches (from *TABLE 6.7*), Q_P is the peak flow in cfs, A_D is the area in treatment D, and A_T is the total area in acres. Using the time of concentration, t_c (hours), the time to peak in hours is:

EQUATION 6.9 $\mathbf{t_P} = (0.7 * t_C) + ((1.6 - (A_D / A_T)) / 12)$

Figure 6.3 **Time to Peak in Hours**



6-1(B) TIME OF CONCENTRATION, LAG TIME, AND TIME TO PEAK

There is a delay, after a brief heavy rain over a watershed, before the runoff reaches its maximum. The length of time it takes for runoff from a watershed to reach an analysis point affects the peak runoff rate, with shorter times producing higher peak flow for a constant runoff volume. The velocity at which water can flow through a watershed and the length of flow path are used to determine the time factors. Time of concentration, lag time, and time to peak are three related watershed parameters that are used to determine peak rates of runoff.

6-1(B)(1) DEFINITIONS

The three time parameters used are defined as follows:

1. **Time of concentration (t_c)** = time it takes for runoff to travel from the hydraulically most distant part of the watershed basin to the basin outlet or point of analysis
2. **Lag time (L_G)** = time from the center of unit rainfall excess to the time of the peak flow of the unit runoff hydrograph.
3. **Time to peak (t_p)** = time from the beginning of unit rainfall excess to the time of the peak flow of the unit runoff hydrograph.

The three time parameters can be computed using the procedures identified in this section. The peak discharge rates and intensity factors identified in *Table 6.9* and *Table 6.10* were computed using a time of concentration (t_c) of 0.2 hour. The procedures in *Section 6-1(D)* require the computation of time to peak (t_p) as specified herein.

6-1(B)(2) COMPUTATION OF TIME OF CONCENTRATION

Two different equations are used to compute time of concentration (tc) for larger watersheds. For subbasin reach lengths shorter than 4000 feet the SCS Upland Method is used. A transition equation is used for subbasin reach lengths between 4000 and 12000 feet. For subbasin reach lengths longer than 12000 feet, divide the subbasin into smaller subbasins. Use of the USDI Bureau of Reclamation lag time equation is not recommended, instead the subbasins should be routed.

Consideration should be given to splitting large watersheds into smaller subbasins with reach lengths less than 4000 feet. Smaller subbasins will allow more accurate modeling of channels and basin topography, and should provide for greater modeling accuracy.

1. For subbasin reach lengths less than 4000 feet compute time of concentration, t_c (hours), for the entire (pervious and impervious) watershed by the SCS Upland Method, the sum of the travel times in the subreaches comprising the longest flow path to the watershed outlet.

EQUATION 6.10 $$t_c = (L_1 / V_1 + L_2 / V_2 + \dots + L_n / V_n) / 3600$$

and,

$$(L_1 + L_2 + \dots + L_n) < 4000 \text{ feet}$$

where:

t_c = time of concentration for the subbasin, in hours. If t_c is computed to be less than 0.2 hours, use t_c = 0.2 hours.
L_x = the subreach length for the n^{th} subreach in feet
V_n = subreach velocity for the n^{th} subreach, in feet per second

The subreach velocity V_n, is as determined by the following equation:

EQUATION 6.11 $$v_n = K (s \cdot 100)^{0.5} = 10K (s)^{0.5}$$

where:

K = conveyance factor, per *TABLE 6.10*, unitless
s = slope, in feet/feet

TABLE 6.10 CONVEYANCE FACTORS	
K	Conveyance Condition
0.7	Turf, landscaped areas and undisturbed natural areas (sheet flow* only).
1	Bare or disturbed soil areas and paved areas (sheet flow* only).
2	Shallow concentrated flow (paved or unpaved).
3	Street flow, storm drains less than 48" diameter, natural channels, and that portion of subbasins (without constructed channels) below the upper 2000 feet for subbasins longer than 2000 feet.
4	Constructed channels (for example: riprap, soil cement or concrete lined channels).
7	Storm drains 48" diameter and larger.
** Sheet flow is flow over plane surfaces, with flow depths up to 0.1 feet. Sheet flow applies only to the upper 400 feet (maximum) of a subbasin.*	

For composite reaches where the basin slope is not uniform, the composite basin conveyance condition, K, can be computed using the following equation:

EQUATION 6.12

$$\mathbf{K} = (L / \sqrt{s}) / (L_1 / (K_1 * \sqrt{s})) + (L / \sqrt{s}) / (L_2 / K_2 * \sqrt{s})) + \ldots + (L / \sqrt{s}) / (L_n / K_n \sqrt{s}))$$

where:

$\mathbf{L} = L_1 + L_2 + \ldots + L_n$

and,

$\mathbf{L} = L_1 + L_2 + \ldots + L_n$

2. For subbasin reach lengths between 4000 and 12000 feet compute the time of concentration, t_C (hours), for the entire watershed using the following equation:

EQUATION 6.13

$$\mathbf{t_C} = ((12000 - L) / (72000 * K * s^{0.5})) + ((L - 4000) * K_N * (L_{CA} / L)^{0.33} / (552.2 * s^{0.165}))$$

where:

K = Conveyance factor from TABLE 6.10. For composite reaches, K is computed using the equations for EQUATION 6.11 and EQUATION 6.12.

L = distance of longest watercourse, in feet.

$\mathbf{L_{CA}}$ = distance along L from point of concentration to a point opposite centroid of drainage basin, in feet.

s = overall slope of L, in foot per foot. For composite reaches s is computed using the equation for _.

$\mathbf{K_N}$ = a basin factor based on an estimate of the weighted, by stream length, average Manning's n value for the principal watercourses in the drainage basin. For the Albuquerque area, values of K_N may be estimated from TABLE 6.11.

TABLE 6.11 LAG EQUATION BASIN FACTORS

K_N	Basin Condition
0.042	Mountain Brush and Juniper
0.033	Desert Terrain (Desert Brush)
0.025	Low Density Urban (Minimum improvements to watershed channels)
0.021	Medium Density Urban (Flow in streets, storm sewers and improved channels)
0.016	High Density Urban (Concrete and rip-rap lined channels)

6-1(B)(3) COMPUTATION OF TIME TO PEAK

For the procedures outlined in 6-1(C), the time to peak (t_p) is assumed to be a constant ratio of the time of concentration (t_C). The following equation is used to compute time to peak:

EQUATION 6.14 $L_G = 0.6t_c$

EQUATION 6.15 $t_P = (2 / 3) * t_C$

EXAMPLE 6

Find the time of concentration (t_c) for a watershed with a 4000 feet desert terrain upper reach (shallow concentrated flow) at 0.015 ft/ft slope and a 3000 feet low density urban lower reach (streets and natural channels) at 0.02 ft/ft slope. The distance to the centroid point is 60% of the total reach length.

$\mathbf{L} = 4000 + 3000 = 7000$ ft

$\mathbf{L_{CA} / L} = 0.60$

$\mathbf{s} = (0.015 * 4000 + 0.02 * 3000) / 7000 = 0.01714$ foot per foot

$\mathbf{K_N} = (0.033 * 4000 + 0.025 * 3000) / 7000 = 0.030$

from

$\mathbf{K} = (7000 / (0.01714^{0.5})) / ((2000 / (2 * (0.015^{0.5}))) + (2000 / (3 * (0.015^{0.5}))) + (3000 / (3 * (0.02^{0.5})))) = 2.59$

$\mathbf{t_c} = ((12000 - 7000) / (72000 * 2.59 * 0.01714^{0.5})) + ((7000 - 4000)* 0.030 * 0.60^{0.33} / (552.2 * 0.01714^{0.165}))$
$= 0.2048 + 0.2694 = 0.4742$ hours

EXAMPLE 7

Find the time of concentration (t_c), lag time (L_G) and time to peak (t_p) for a watershed with a 8000 feet desert terrain upper reach at 0.015 ft/ft slope and a 6000 feet low density urban lower reach at 0.02 ft/ft slope. The distance to the centroid point is 60% of the total reach length.

L = 8000 + 6000 = 14000 feet

$\mathbf{L_{CA}}$ = 0.60 * 14000 = 8400 feet

use

s = (0.015 * 8000 + 0.02 * 6000) / 14000 = 0.01714 ft/ft
$\mathbf{K_N}$ = (0.033 * 8000 + 0.025 * 6000) / 14000 = 0.030

use

$\mathbf{L_G}$ **=** 26 * 0.030 * ((14000 * 8400 / (52802 * (0.01714 * 5280)0.5))0.33) = 0.596 hours
$\mathbf{t_c}$ = (4/3) * 0.596 = 0.795 hours
$\mathbf{t_p}$ = (2/3) * 0.795 = 0.530 hours

6-1(B)(4) TIME OF CONCENTRATION FOR STEEP SLOPES AND NATURAL CHANNELS

The procedures used to compute time of concentration (t_c) as described in *Section 6-1(B)(2)* may compute values that are too small to be sustained for natural channel conditions. In natural channels, flows become unstable when a Froude Number of 1.0 is approached. The procedures identified in *Section 6-1(B)(2)* may compute flow velocities for steep slopes that indicate supercritical flow conditions, even though such supercritical flows cannot be sustained for natural channels.

For steep slopes, natural channels will likely experience chute and pool conditions with a hydraulic jump occurring at the downstream end of chute areas; or will experience a series of cascading flows with very steep drops interspersed with flatter channel sections.

For the purposes of this section, steep slopes are defined as those greater than 0.04 foot per foot. The procedures outlined in this section should not be used for the following conditions:

1. Slopes flatter than 0.04 foot per foot.
2. Channels with irrigated grass, riprap, soil cement, gabion, or concrete lining which cannot be clearly identified as natural or naturalistic.
3. The hydraulic design of channels or channel elements. The purpose this section is to define procedures for hydrologic analysis only. The design of facilities adjacent to or within channels with chute and pool conditions cannot be analyzed with the simplified procedures identified herein. It may be necessary to design such facilities for the supercritical flows of chutes (for sediment transport, local scour, stable material size) and for the hydraulic jump of pool conditions (for maximum water surface elevation and flood protection).

The slope of steep natural watercourses should be adjusted to account for the effective slope that can be sustained. The slope adjustment procedures identified in the Denver - Urban Drainage and Flood Control District (UDFCD) Urban Storm Drainage Criteria Manual (Figure 4-1, Runoff chapter, 1990) are applicable for the slope adjustment identified herein. In addition, channel conveyance factors (K) should be checked to make sure that appropriate equivalent Froude Numbers are maintained. The UDFCD Figure 4-1 can be approximated by the following equation:

EQUATION 6.16

$$s' = 0.052467 + (0.063627 * s) - 0.18197 * e^{(-62.375 * s)}$$

where:

s = measured slope (foot per foot)
s' = adjusted slope (foot per foot)

The conveyance factors (K) for the upland method should be checked to make sure that appropriate Froude Numbers are maintained. To accomplish this, it is necessary to estimate the peak flow rate from the watershed. Using estimated conveyance factors (K) from Table B-1 and the procedures outlined in Part A, an estimated peak flow rate for the basin (Q_p) can be computed. The following formulas are then used to compute conveyance factor adjustment:

EQUATION 6.17 $K' = 0.302 * s'^{(-0.5)} * Q_p\,(0.18)$

EQUATION 6.18 $K'' = 0.207 * s'^{(-0.5)} * Q_p\,(0.18)$

An adjusted conveyance factor (K) is then obtained based on the following:

if $K > K'$	then $K = K'$
if $K' \geq K \geq K''$	then $K = K$ (no adjustment)
if $K < K''$	then $K = K''$

Recompute Q_p based on the revised conveyance factor (K) using the procedures in Section 6-1(D) or Section 6-1(C) as appropriate. If the recomputed Q_p is within 10 percent of the Q_p used to compute K' and K'', the estimate is sufficiently accurate. If the recomputed Q_p is more than 10 percent from the Q_p used to compute K' and K'', repeat the process using the revised $Q_{P.}$

The Lag Equation Basin Factors, K_N, from, TABLE 6.11 remain applicable when using equations EQUATION 6.8 and EQUATION 6.9 with the adjusted slope computed by the equation shown in EQUATION 6.12.

EXAMPLE 8

Compute the time of concentration (t_c) for a natural basin having a length of 4,000 feet and a uniform slope of 0.12 foot per foot. The basin is estimated to have a peak flow of 600 cfs using the procedures in Section 6-1(D).

s = 0.12 foot per foot
$\mathbf{Q_P}$ = 600 cfs

Compute the adjusted slope using EQUATION 6.16.

s' = 0.052467+ 0.063627 * 0.12 - 0.18197 * ($e^{(-62.375 * 0.12)}$)
= 0.052764 + 0.007635 - 0.000102 = 0.0603 ft/ft

Compute conveyance factors from TABLE 6.10 and EQUATION 6.9.

K = 4000 / (300 / 0.7 + 1700 / 2.0 + 2000 / 3.0) = 2.056

From EQUATION 6.17 and EQUATION 6.18.

K' = 0.302 * $(.0603)^{(-0.5)}$ * $(600)^{0.18}$ = 3.89
K" = 0.207 * $(.0603)^{(-0.5)}$ * $(600)^{0.18}$ = 2.66

Since K < K" then use K = 2.66

From EQUATION 6.6 and b-2.

V = 10 * 2.66 * ($0.0603^{0.5}$) = 6.53 ft/sec
$\mathbf{t_C}$ = (4000 / 6.53) / 3600 = 0.170 hour (Use 0.200 hour min.)

The Q_P should then be recomputed using the revised t_c and the procedures in Section 6-1(A) or Section 6-1(C).

6-1(B)(5) CHANNEL ROUTING FOR STEEP SLOPES AND NATURAL CHANNELS

The procedures outlined to compute time of concentration for steep natural channels in Section 6-1(B)(4) are also applicable for hydrologic routing of hydrographs through channel segments. The restrictions which limit the procedure only to natural channels with slopes steeper than 0.04 foot per foot are also applicable here. The procedures are not applicable to the hydraulic design of channel structures.

EQUATION 6.12 can be used to obtain an adjusted slope for the channel segment. The Manning's roughness (n) for the channel should be checked to make sure that appropriate Froude Numbers are maintained. It is necessary to estimate the peak flow rate (Q_P) for the watershed channel segment to perform this check. An analysis without a Manning's roughness adjustment may be used for the initial estimate. The following formula is then used to compute the Manning's roughness adjustment:

EQUATION 6.19 **n'** = 0.122 * $s'^{(0.5)}$ * $Q_P^{(0.06)}$

An adjusted Manning's roughness (n) is then obtained based on the following:

if n < n' then n = n'
if n $\geq$ n' then n = n (no adjustment)

Recompute the Q_p based on the revised Manning's roughness (n). If the recomputed Q_p is within 30 percent of the Q_p used to compute n', the estimate is sufficiently accurate. If the recomputed Q_p is more than 30 percent from the Q_p used to compute n", repeat the process using the revised Q_p.

EXAMPLE 9

A channel segment immediately downstream of the basin in *EXAMPLE 8* has a slope of 0.08 foot per foot. The channel has an apparent Manning's roughness of 0.035. Compute the equivalent channel slope and Manning's roughness for use in hydrologic routing.

s = 0.08 foot per foot
$\mathbf{Q_p}$ = 600 cfs

s' = 0.052467 + 0.063627 * 0.08 - 0.18197 * e(-62.375*0.08)
= 0.052467 + 0.005090 - 0.001238 = 0.0563 ft/ft

Use **equivalent slope** = 0.0563 ft/ft (from *EQUATION 6.19*)

n' = 0.122 * (.0563)0.5 * (600)0.06 = 0.0425

Since n < n', then use n = 0.0425

6-1(C) PROCEDURE FOR SMALL AND LARGE WATERSHEDS

A unit hydrograph procedure is used for major drainage area analysis and for sub-basins larger than 40 acres. The *6-1(C)* procedure may also be utilized for small watersheds (40 acres or less) in place of the procedures specified in *6-1(A)*. AHYMO is the primary method of hydrograph computation using losses described in *TABLE 6.5* and *TABLE 6.6* for Land Treatments as described in *TABLE 6.3* and a rainfall distribution with peak intensity 1.5hours after the beginning of the storm. The SCS Curve Number method is also allowed using Curve Numbers listed in *TABLE 6.3* with a 24 hour rainfall distribution based on Atlas 14 (smoothing should not be applied to the Atlas 14 data points) with the peak intensity at 12 hours. The unit hydrograph calculation increment is to be 0.01 hours or less for both the AHYMO and the SCS methods.

6-1(C)(1) COMPUTER PROGRAM

The unit hydrograph calculations must be accomplished using computer programs that are acceptable to the City of Albuquerque. Consult the User's Manual for direction on how to use each program. Program data files must be included with applications to hydrology. A list of acceptable programs is available on the Hydrology web page of the City of Albuquerque along with requirements for procedures to be used and the format of the printout to be contained in the application for each program.

6-1(C)(2) ZONES

The unit hydrograph procedure should not utilize precipitation zones from TABLE 6.1 or Figure 6.1 of Section 6-1(D). The precipitation amounts are obtained for a specific location near the center of the watershed being analyzed from the NOAA Atlas 14. The Latitude and Longitude and Elevation of the "Point Precipitation Frequency Estimates and map showing the location of the point should be included in the documentation. Program parameters are obtained based on basin characteristics and precipitation quantities.

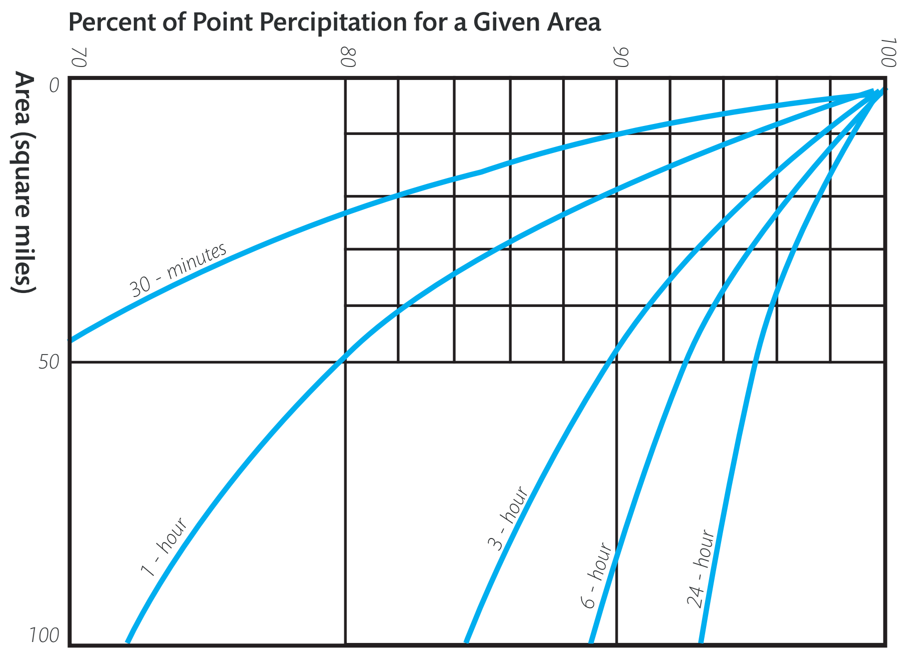
6-1(C)(3) DESIGN STORM

The principal design storm for peak flow determination is the 100-year 24-hour event defined by the NOAA Atlas 14, Precipitation - Frequency Atlas of the United States, Vol. 1 Version 5 Semiarid Southwest or the most current version. Storms of other frequencies or durations are required for design or analysis of volume sensitive facilities, when examining sediment transport, and for complex routing conditions. The following statement from the Federal Emergency Management Agency (FEMA) should be used to provide guidance when selecting storm duration:

"FEMA's position regarding the duration of rainfall is that the storm must extend for a period long enough to include all rainfall excess when the volume of the runoff hydrograph is an important consideration. This includes conditions when detention storage is involved, when sediment processes are a significant factor, and when combining and routing subbasin hydrographs to obtain watershed runoff.

When evaluating uncontrolled watersheds larger than five (5) square miles, the precipitation amounts may be reduced by multiplying the precipitation amounts by the "Percent of Point Precipitation" obtained from Figure 6.4. Uncontrolled watersheds mean those areas not controlled by dams, ponds or partial diversions.

Figure 6.4 Depth - Area Curves



6-1(D) SYMBOLS AND BIBLIOGRAPHY

6-1(D)(1) DEFINITIONS OF SYMBOLS

When evaluation equations use the following order of precedence: 1) parentheses, 2) functions (i.e., SIN or LOG), 3) power or square root, 4) multiplication or division, 5) addition or subtraction.

TABLE 6.12 DEFINITIONS OF SYMBOLS

Symbol	Definition
A_A	area in land treatment A
A_B	area in land treatment B
A_C	area in land treatment C
A_D	area in land treatment D
A_T	total area in sub-basin
Ac	Ft acre feet
C	Rational Method coefficient
C_A	Rational Method coefficient for treatment A
C_B	Rational Method coefficient for treatment B
C_C	Rational Method coefficient for treatment C
C_D	Rational Method coefficient for treatment D
cfs	cubic feet per second
CN	SCS Curve Number
D	duration in days
e	base of natural logarithm system = 2.71828
E	excess precipitation
E_A	excess precipitation for treatment A
E_B	excess precipitation for treatment B
E_C	excess precipitation for treatment C
E_D	excess precipitation for treatment D
EA	elevation Adjustment factor for PMP60
Elev	elevation (feet)
Ft	feet
hr	hour
I	Rational Method intensity (inches/hour)
IA	initial abstraction (inches)
INF	infiltration (inches/hour)
K	conveyance factor for SCS Upland Method
k	recession coefficient for HYMO program
K_N	basin factor for lag time equation
K_X	conveyance factor for watershed subreach
k/t_{pA}	k divided by tp for treatment A

TABLE 6.12 DEFINITIONS OF SYMBOLS

Symbol	Definition
k/t_{pB}	k divided by tp for treatment B
k/t_{pC}	k divided by tp for treatment C
k/t_{pD}	k divided by tp for treatment D
k/t_{P40}	k divided by tp for 40 acres or smaller area
k/t_{P200}	k divided by tp for 200 acres or larger area
L	length of subreach (feet)
L_{CA}	distance to centroid of drainage basin (feet)
L_G	lag time (hours)
L_X	length of watershed subreach
In	natural logarithm (base e)
log_{10}	base 10 logarithm
mi^2	square mile(s)
n	Manning's roughness coefficient
P_{12}	12-minute precipitation
P_{60}	60-minute precipitation at 100-year storm
P_{60-2}	60-minute precipitation at 2-year storm
$P_{60-year}$	60-minute precipitation at "year" storm
P_{360}	360-minute precipitation at 100-year storm
P_{360-2}	360-minute precipitation at 2-year storm
P_{360-10}	360-minute precipitation at 10-year storm
P_{1440}	1440-minute (24-hr) precipitation, 100-year storm
P_{1440-2}	1440-minute (24-hr) precipitation at 2-year storm
P_D	precipitation for "D"-days duration
P_{N-100}	"n"-minute precipitation at 100-year storm
P_{N-YEAR}	"n"-minute precipitation at "year" storm
P_T	precipitation at any time, t
Q_P	peak discharge (cfs)
Q_{PA}	peak discharge rate (cfs/acre) for treatment A
Q_{PB}	peak discharge rate (cfs/acre) for treatment B
Q_{PC}	peak discharge rate (cfs/acre) for treatment C
Q_{PD}	peak discharge rate (cfs/acre) for treatment D
s	slope of subreach in foot per foot
t	time in minutes
t_B	base time for small watershed hydrograph
t_C	time of concentration (hours)
t_p	time to peak (hours)
v	velocity of flow in watershed (feet/sec)
v_x	velocity of flow in watershed subreach
V_{360}	runoff volume for 360-minute storm

TABLE 6.12 DEFINITIONS OF SYMBOLS

Symbol	Definition
V_{1440}	runoff volume for 1440-minute storm
V_{4days}	runoff volume for 4-day storm
V_{10days}	runoff volume for 10-day storm
y^x	y to the x power
+	addition operator
-	subtraction operator
*	multiplication operator
/	division operator
√	square root operator

6-1(D)(2) BIBLIOGRAPHY

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6-2 SITE DEVELOPMENT

It is beneficial to consider the below listed items when beginning to develop a site:

1. Flood Zone-May affect finished floor elevation, locations of structures and increase permit requirements-*Section 6-5*
2. Downstream Capacity-may require onsite ponding – *Section 6-6*
3. Offsite flows- in general, are accepted and conveyed through the site – *Section 6-6*
4. Applicable approved drainage reports and plans-provides previous approvals for downstream capacity and offsite flows – *Section 6-6*
5. Current Topography-accurate depiction of existing conditions
6. Encumbrances-utility corridors and easements may restrict development
7. Water Quality- Design standard volume and construction runoff - *Section 6-11*

6-3 GRADING AND EROSION CRITERIA

6-3(A) SLOPE CRITERIA

Earth slopes shall confirm to the following criteria:

1. For slopes 3.0 feet high or less, maximum slope should not exceed 2:1 (horizontal to vertical)
2. For slopes greater than 3.0 feet high, maximum slope should not exceed 3:1 (horizontal to vertical) unless stabilized from slope failure through City Engineer approved means. Steeper slopes may be approved subject to a geotechnical recommendation and City Engineer concurrence.
3. All slopes shall be protected from erosion, especially when subjected to upland flows.

6-3(B) GRADING NEAR THE PROPERTY LINE

Particular attention must be given to grading (either cut or fill) near property lines. Care should be taken to ensure that existing foundations, retaining walls, stable slopes or other structures are not endangered and that the adjacent property is not damaged or its use constrained due to grading at or near the property line.

6-3(C) GRADING IN AND ADJACENT TO MAJOR FACILITIES

No grading, excavation, or fill may take place in or adjacent to any watercourse defined as a major facility (30 cfs for arroyos and 2 acre-ft for detention basins) without an approved grading and drainage plan.

Construction activities within major facilities shall provide for the safe passage of the 10-year design flow during the months of July, August and September.

6-3(D) GRADING IN AND ADJACENT TO MAJOR PUBLIC OPEN SPACE

1. Width disturbance to slopes and vegetation, and cut and fill, shall be minimized and balanced against the need to provide for bikeways or other amenities within the right-of-way.
2. Materials that blend with the adjacent landscape of the Major Public Open Space in color and texture shall be used. Natural materials are generally preferable to man-made materials.
3. No grading is permitted within Major Public Open Space areas with nine percent or greater slopes except as required for roads, trails, and utilities.

a. *Temporary construction barricades, or 20-foot construction setback, are required from Major Public Open Space areas with 9 percent or greater slopes.*
b. *If damage due to construction occurs on the Major Public Open Space side of the property line, it shall be mitigated at the expense of the property owner.*

4. Corridors for construction projects shall be located to avoid impacts and destruction of petroglyphs or other archaeological sites and environmentally sensitive areas previously identified.
5. Areas that are damaged or altered shall be restored through replacement of boulders to approximate the original location, angle and surface exposure. Revegetation to approximate original cover with appropriate native or naturalized plants is required within 90 days of project completion.
6. The City shall be responsible for restoring existing damaged areas that lie within Major Public Open Space. The property owner shall be responsible for restoring damaged areas on lands accepted by the City to meet open space requirements; this shall occur prior to title transfer if the land is to be deeded to the City and shall be an ongoing responsibility of the property owner if the land remains private open space.

6-3(E) MEANS OF EROSION CONTROL

The means of erosion control shall be specified on the grading plan. Steeper slopes require a larger rock. Please refer to the table below for recommended erosion control. Recommendations are for slopes without upland flows:

1. 3:1 to 4:1 -3/4" or larger rocks
2. 2.5:1 to 3:1- 1.5" angular rock
3. 2:1 to 2.5:1- 4" minimum angular hand-placed with no landscape fabric
4. 1.5:1 to 2:1- 6" or larger angular stone hand placed with no landscape fabric.

Slopes steeper than 1.5:1 may be allowed with a design acceptable to the City Engineer.

For slopes steeper than 5:1with upland flows, the velocity and flow rate should be considered when designing the erosion protection for the slope.

6-3(F) LEVEES AND BERMS

6-3(F)(1) DEFINITIONS

1. **A levee-**FEMA defines a levee as a man-made structure, usually an earthen embankment, designed and constructed in accordance with sound engineering practices to contain, control or divert the flow of water. Levees in general are used to contain flows from the river or major water course where the grade outside the levee is lower than the 100 year 6-hour water surface elevation.
2. **A berm** is a linear earth structure designed to direct or retain/detain storm water. The height is measured from the uphill side. See the section on Ponds for berms used to retain/ detain stormwater.

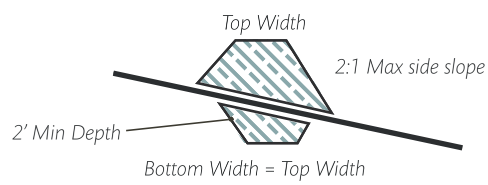
6-3(F)(2) DESIGN CRITERIA

All levees shall be designed to standards published by the Army Corp of Engineers and meet FEMA freeboard requirements. Any berm or levee whose purpose is to divert or convey runoff in a major arroyo (30 cfs or greater) shall be specially designed on a case-by-case basis and shall meet or exceed the guidelines listed herein.

6-3(F)(2)(I) CROSS SECTION

1. The top width should have a minimum width equal to the height of the berm. Construction and maintenance equipment should be considered when considering the top width. The minimum top width is 4 feet.
2. Berms 4 feet and higher must be provided with a structural keyway with bottom width equal to the top width and depth equal to at least half the height, but not less than 2 feet and side slopes not steeper than 2:1 (horizontal to vertical)

Figure 6.5



3. Unarmored faces of berms must have side slopes not steeper than 3:1 (horizontal to vertical).
4. Safety issues should always be considered when designing slopes.
5. For high velocity
   a. *For velocities 5 fps or greater an engineered means of erosion protection is required for bank protection.*
   b. *Erosion protection may be required for velocities less than 5 fps.*
   c. *Rip-rap protected side slopes shall not be steeper than 2:1 (horizontal to vertical). Method of rip-rap installation per engineer.*
   d. *Concrete faced berms may be used on side slopes greater than 2:1 (horizontal to vertical).*

6-3(F)(2)(II) FREEBOARD

Berms and levees must be provided with freeboard for the 100-year design storm based on the following guidelines:
1. For flow depths less than 2.0 feet; minimum freeboard is 1.0 feet.
2. For flow depths greater than 2.0 feet; minimum freeboard is 2.0 feet

6-3(F)(3) EARTHWORK FOR BERMS

All earthen berms and levees shall be constructed of high quality fill material free of debris, organic matter, frozen matter and stones larger than 6 inches in any dimension. The key trench shall be scarified to a depth of 6 inches to ensure bonding with the fill material. Lifts shall not exceed 12 inches of loose material before compaction. The material in each lift shall contain optimum moisture content (-1% to +3%) or per Geotechnical Report and shall be compacted to at least 95% density as determined by ASTM D 1557.

6-3(F)(4) CERTIFICATION

All berms 4 feet and higher shall be inspected during construction and certified by a New Mexico Professional Engineer as to their substantial compliance to the approved plans and specifications.

6-4 VALLEY DRAINAGE CRITERIA

Special considerations are appropriate in the valley due to the flatness of the area and limited storm drain capacity. The valley is defined as the area bounded by: Broadway Blvd/Edith Blvd on the east, the Rio Grande River on the West, and the City limits on the North and South.

6-4(A) SINGLE LOT RESIDENTIAL DEVELOPMENT AND ADDITIONS

For lots less than one acre, water harvesting on the lot is required. The water harvesting volume goal is to capture a ½ inch of runoff from impervious areas on the site.

1. Roof flows should be directed to the water harvesting area(s).
2. Runoff should not adversely impact adjacent properties.
3. The finished pad elevation is recommended to be a minimum of 18 inches above the edge of pavement or roadway.

6-4(B) RESIDENTIAL SUBDIVISIONS

Property that will be subdivided may require a drainage submittal for DRB approval. The drainage submittal shall categorize the downstream capacity per the following:

1. Discharge from the site will be limited to proven downstream capacity.
2. If the site has limited downstream capacity, the site shall retain the runoff from the 100-yr 6-hour storm on lots or in a subdivision pond.
3. If the site has no downstream capacity, the subdivision shall retain the 100-yr 10-day storm.

6-4(C) NON-SINGLE FAMILY RESIDENTIAL DEVELOPMENT AND COMMERCIAL PROPERTIES

These development types will be subject to the following allowable stormwater discharge rates:

1. 2.75 cfs/acre or
2. The site must retain the first ½" of runoff or the design standard volume as defined in the MS-4 permit, whichever is greater. See *Section 6-11*.
3. If downstream capacity is known to be more limited, the allowable discharge may be less.

6-4(D) FLAT GRADING SCHEME FOR RESIDENTIAL SUBDIVISIONS

A flat grading scheme is considered a ponding condition and may be allowed in flat areas such as the Valley region of the City and under the following conditions:

1. There is no outfall or insufficient downstream conveyance for the site.
2. The site must be flat or graded flat.

3. The maximum percent impervious of the lot and the contributing area may not be greater than 45%.
4. Finished pad elevation shall be a minimum of one (1) foot above the 100 year 10-day storm water surface elevation.
5. The flow between the front yard and back yard cannot be obstructed. The storm water must be allowed to equalize to the same level between the front yard and back yard.
6. A permanent perimeter wall or barrier around the development is required to contain the 100 year 10-day storm developed runoff.
7. The high point of all internal streets must be four inches above the 100 year 10-day storm water surface elevation.

6-5 DEVELOPING IN OR ADJACENT TO A FLOOD ZONE

The City of Albuquerque participates in the National Flood Insurance Program (NFIP) and therefore development and construction activities in mapped flood zones must follow the requirements of the NFIP and the Code of Federal Regulations 44 CFR Parts 59, 60, 65 and 70.

6-5(A) GRADING

Grading will not be allowed within a FEMA Special Flood Hazard Area (flood zone) without an approved grading and drainage plan and a Floodplain Development Permit.

A letter of Map Revision will be required when development changes a mapped flood zone. The City Engineer may waive the LOMR requirement for projects involving one acre or less.

6-5(B) COMPENSATORY VOLUME

1. Compensatory volume (AH and AE Zones) is a volume that is provided in the proposed condition that mitigates the displaced volume associated with development. This is most important in AH zones (areas of ponding 1 to 3 feet deep) and ponding AE zones.
2. In an AH or ponding AE Zone the drainage plan is to state the amount of displaced volume in the mapped flood zone and show where this volume is to be accommodated in the proposed condition.

6-5(C) AO ZONE

When developing adjacent to or in an AO Zone, the cross-sectional area of the flow path is to be preserved. This is to be demonstrated in the drainage plan.

6-5(C)(1) DETERMINATION OF BASE FLOOD ELEVATION (BFE) IN AN AO ZONE:

If flooding is conveyed by the street, provide the highest top of curb or crown along the property line and add the AO Zone depth (e.g AO 1) to the higher of the two elevations; top of curb or crown.

If the entire property is inundated and the flow is not conveyed by the street, calculate an average grade for the site and add the AO zone depth to the average grade.

If the property is partially inundated and the street does not convey the flow, add the AO Zone depth to the lowest lot elevation.

6-5(D) A ZONE

For developing adjacent or in an unnumbered A Zone, the base flood elevation will be determined by best available data or if no data is available the BFE is 2 feet above the highest adjacent grade.

6-5(E) FLOODPLAIN DEVELOPMENT PERMIT

A *Floodplain Development Permit* is required for any construction in a mapped flood zone as provided by FEMA. This requirement may be waived if the work is minor (e.g. drivepad) and will not result in a change to the water surface elevation or flow path.

6-5(F) LETTERS OF MAP CHANGE (LOMC)

Map changes come in the form of *Letters of Map Revision (LOMR)*, *Letters of Map Amendment (LOMA)*, *Letters of Map Amendment based on Fill (LOMR-F)* and conditional LOMR and LOMR-F (CLOMR, CLOMR-F)

1. A LOMR, if approved by FEMA, will change/remove the mapped flood zone from the Flood Insurance Rate Map (FIRM).
2. A LOMA, if approved by FEMA, will not change the FIRM, but will remove the structure or property from the flood zone for insurance purposes.
3. A LOMR-F, if approved by FEMA, will not change the FIRM, but will remove the structure or property from the flood zone for insurance purposes. If fill was imported to raise the structure above the Base Flood Elevation (BFE), the LOMR-F and not the LOMA is to be submitted to FEMA.
4. A conditional map change (CLOMR, CLOMR-F) is submitted to FEMA prior to grading/building to obtain their approval or receive comments on the proposed project. A conditional map change is always recommended as it shortens the review time upon the completion of the project and minimizes unexpected review responses from FEMA. CLOMR and CLOMR-F's must demonstrate compliance with the Endangered Species Act.

For more information on the above mentioned letters of map change, refer to *FEMA's website*.

6-5(G) PROJECT REQUIREMENTS

1. If the project proposes any grading in a regulated floodway, an approved CLOMR is required prior to beginning grading operations or receiving project approval at the Development Review Board or prior to Building Permit approval.
2. The lowest finished floor elevation is to be a minimum of 1 foot above the Base Flood Elevation (BFE).
3. An elevation certificate is required to be submitted to the Floodplain Administrator and deemed acceptable prior to obtaining a Certificate of Occupancy for the building. It is advised to follow-up with a LOMR-F or LOMA to remove the building from the flood zone.

6-6 DOWNSTREAM CAPACITY AND OFFSITE FLOWS

Downstream capacity and offsite flows are the most important elements of a successful drainage report/plan. The engineer is expected to research adjacent projects, as-built storm drain construction plans and Drainage Master Plans to correctly identify downstream capacity. See the Valley Drainage Criteria section if the project is in the valley.

Previously approved Drainage Masterplans can be relied upon as long as the basin conditions have not changed.

The engineer is also expected to perform a site visit, review topography and review adjacent drainage reports/plans to accurately identify offsite flows.

6-6(A) DOWNSTREAM CAPACITY

The drainage report/plan shall accurately state allowable downstream capacity. In the case, where the project is a small redevelopment project (less than 0.5 acres) and not in the valley, proposed flows not to exceed historic flows is most likely acceptable. Some small sites may have a history in which proposed flows will have to be less than historic flows.

6-6(B) OFFSITE FLOWS

The drainage report/plan is to show the location and quantify offsite flows. In general, sites are to accept offsite flows and convey them safely to an acceptable outfall. A site may not have to accept offsite flows if a previously approved plan shows the outfall adjacent to the site and flows can be safely conveyed to an acceptable outfall.

6-6(C) HISTORIC FLOW PATH THROUGH ADJACENT PRIVATE PROPERTY

If the only reasonable outfall for a proposed development is a historic flow path through an adjacent private property, the historic flow characteristics and path must be maintained.

6-7 ENGINEERED CHANNELS AND NATURAL ARROYOS

6-7(A) GENERAL HYDRAULIC CRITERIA

In general, all open channels should be designed with the tops of the walls or levees at or below the adjacent ground to allow for interception of surface flows. If it is unavoidable to construct the channel without creating a pocket, a means of draining the pocket must be provided on the drawings. All local drainage should be completely controlled. External flows must enter the channel at designated locations and through designated inlets unless specifically otherwise authorized by the City Engineer.

6-7(B) SHARP CURVES

In making preliminary layouts for the routing of proposed channels, it is desirable to avoid sharp curvatures, reversed curvatures, and closely-spaced series of curves. If this is unavoidable, the design considerations below must be followed to reduce superelevations and to eliminate initial and compounded wave disturbances.

6-7(C) MAXIMUM FROUDE NUMBER

It is generally desirable to design a channel for a Froude number of just under 2.0. In areas within the City of Albuquerque this is not always possible because of steep terrain. If the Froude number exceeds 2.0, any small disturbance to the water surface is amplified in the course of time and the flow tends to proceed as a series of "roll waves". Reference is made to sections below for criteria when designing a channel with a Froude number that exceeds 2.0.

In the design of a channel, if the depth is found to produce a Froude number between 0.7 and 1.3 for any significant length of reach, the shape or slope of the channel should be altered to secure a stable flow condition. All analyses should be performed for the 10-year and 100-year design discharges.

6-7(D) WATER SURFACE PROFILE CALCULATIONS

Water surface profile calculations must be calculated using the Bernoulli energy equation combined with the momentum equation for analyzing confluences and functions.

6-7(D)(1) DETERMINATION OF CONTROLLING WATER SURFACE ELEVATION

The following are generally control points for the calculation of the water surface profile:

1. Where the channel slope changes from mild to steep or critical, the depth at the grade break is critical depth.
2. Where the channel slope changes from critical to steep, the depth at the grade break is critical depth.
3. Where a discharging channel or conduit is on a mild slope, the water surface is generally controlled by the outlet.
4. When a channel on a steep slope discharges into a facility that has a water surface depth greater than the normal depth of the channel, calculate pressure plus momentum for normal depth and compare it to the pressure plus momentum for the water surface depth at the outlet according to the equation, $P_n + M_n \sim P_o + M_o$.
   a. *If $P_n + M_n > P_o + M_o$, this indicates upstream control with a hydraulic jump at the outlet.*
   b. *If $P_n + M_n < P_o + M_o$, this indicates outlet control with a hydraulic jump probably occurring upstream.*
   c. *Where the water surface of the outlet is below the water surface in the channel or conduit, control is upstream and the outflow will have the form of a hydraulic drop.*

When there is a series of control points, the one located farthest upstream is used as a starting point for water surface calculation.

6-7(D)(2) DIRECTION OF CALCULATION

Calculations proceed upstream when the depth of flow is greater than critical depth and proceed downstream when the depth of flow is less than critical depth.

6-7(D)(3) HEAD LOSSES

6-7(D)(3)(I) FRICTION LOSS

Friction losses or open channels shall be calculated by an accepted form of the Manning equation. The Manning equation is commonly expressed as follows:

EQUATION 6.20 $\mathbf{Q} = (1.486/n)\ A\ R^{2/3}\ Sf^{1/2}$

where:
Q = Flow rate, in c.f.s.
n = Roughness coefficient
A = Area of water normal to flow, in ft.2
R = Hydraulic radius
$\mathbf{S_f}$ = Friction slope

when arranged into a more useful form:

$$\mathbf{S_f} = (2gn^2/2.21((V^2/2g)/R^{4/3})$$

the loss of head due to friction throughout the length of reach involved (L) is calculated by:

$$\mathbf{h_f} = S_f\ L$$

Refer to the appendix for values of "n" for different materials and corresponding values of: ($2gn^2/2.21$)

6-7(D)(3)(II) JUNCTION LOSS

Junction losses will be evaluated by the pressure plus momentum equation and must conform to closed conduit angle of confluence criteria, Section 6-1(B)(5). Refer to Miscellaneous Hydraulic Calculations later in this section.

6-7(D)(4) CHANNEL INLETS

6-7(D)(4)(I) SIDE CHANNELS

Flow rates of 25% or more of the main channel flow must be introduced to the main channel by a side channel hydraulically similar to the main channel. The centerline radius of the side channel may not be less than the quantity (QV/l00) in feet.

Velocity and depth of the flows in the side channel when introduced into the main channel must be matched to within 1 foot of velocity head and to within 20% of the flow depth for both the 10-year and l00-year design discharges and the four combinations of side inlet and main channel flows which result. Energy and momentum balance type calculations must be provided to support all designs involving side channels.

6-7(D)(4)(II) SURFACE INLETS

When the main channel is relatively narrow and when the peak discharge of side inflow is in the range between 3 and 6 percent of the main channel discharge, high waves are usually produced by the side inflow and are reflected downstream for a long distance, thus requiring additional wall height to preclude overtopping of the channel walls. This condition is amplified when the side inflow is at a greater velocity than the main channel. To eliminate these wave disturbances, the Los Angeles District of the Corps of Engineers has developed a side channel spillway inlet. The City or AMAFCA may require this type of structure when outletting into one of their facilities, and its use should be considered for city channels if high waves above the normal water surface cannot be tolerated. See Subsection "f" below titled "Transitions" for the Corp's procedure and criteria.

Surface-type inlets shall be constructed of concrete having a minimum thickness of 6 inches and shall be reinforced with the same steel as 6" concrete lining. The upstream end of the surface inlet shall be provided with a concrete cutoff wall having a minimum depth of three feet and the downstream end of the inlet shall be connected to the channel lining by an isolation joint. Side slopes of a surface inlet shall be constructed at slopes no greater than l vertical to 10 horizontal to allow vehicular passage across the inlet where a service road is required.

Drainage ditches or swales immediately upstream of a surface inlet shall be provided with erosion protection consisting of concrete lining, rock riprap or other non-erosive material.

Surface inlets shall enter the channel at a maximum of 90° to the channel centerline, i.e., they may not point upstream.

6-7(D)(4)(III) DIRECT PIPE TO CHANNEL

Junctions involving direct pipe connection to a channel must conform to the criteria listed in the fifth section of the closed conduit criteria. Additionally, pipe and box culvert inlets to channels shall be isolated by expansion joints. Continuously reinforced channels shall be designed to accommodate any extra stress resulting from these discontinuities. Paragraph 18(h), Corps of Engineers EM 1110-2-1061 has additional design criteria.

6-7(D)(4)(III)(A)TRANSITIONS

1. Subcritical Flow
   a. *For subcritical velocities less than 12 f.p.s., the angle of convergence or divergence between the center line of the channel and the wall must not exceed 12º 30'. The length of the transition (L) is determined from the following equation:*

      EQUATION 6.21 $\mathbf{L} \geq 2.5\ \Delta\ B$

   b. *For subcritical velocities equal to or greater than 12 f.p.s., the angle of convergence or divergence between the center line of the channel and the wall must not exceed 5º 45'. The length (L) is determined from the following equation:*

      EQUATION 6.22 $\mathbf{L} \geq 5.0\ \Delta\ B$

   c. *Head losses for transitions with converging walls in subcritical flow conditions can be determined by using either the P + M method or the Thompson equation, both of which are shown in Section 6-8. For transitions, both methods are applicable in all cases and will give the same results.*

2. Supercritical Flow
   a. *Divergent Walls*
      i The angle of divergence between the center line of the channel and the wall must not exceed 5º 45' or tan-1(F/3) whichever is smaller. The length of the transition (L) is the longest length determined from the following equations:

      EQUATION 6.23 $\mathbf{L} \geq 5.0\ \Delta\ B$
      EQUATION 6.24 $\mathbf{L} \geq 1.5\ \Delta\ B^*F$

      where:
      F = Upstream Froude number based on depth of flow
      Δ B = The difference in channel width at the water surface

   b. *Convergent Walls*
      i Converging walls should be avoided when designing channels in supercritical flow; however, if this is impractical, the converging transition will be designed to minimize wave action. The walls of the transition should be straight lines.

3. Transitions Between Channel Treatment Types
   a. *Earth Channel to Concrete Lining Transition*
      i The mouth of the transition should match the earth channel section as closely as practicable. Wing dikes and/or other structures must be provided to positively direct all flows to the transition entrance.
      ii The upstream end of the concrete lined transition will be provided with a cutoff wall having a depth of 1.5 times the design flow depth, but at least 3.0 feet and extending the full width of the concrete section. Erosion protection directly upstream of the concrete transition consisting of grouted or dumped rock riprap at least 12 feet in length and extending full width of the channel section must be provided. Grouted riprap must be at least 12 inches thick and tied to the concrete lining and cutoff wall. Dumped riprap must be properly sized, graded and projected with gravel filter blankets.
      iii The maximum allowable rate of bottom width transition is 1 to 7.5 maximum. Grout, dumped, or wire-tied material may also be used if approved on a case-by-case basis by the City Engineer. Grouted and wire-tied material require gravel filters as well.
   b. *Concrete Lining to Earth Channel Transition*
      i The transition from concrete lined channels to earth channels will include an energy dissipator as necessary to release the designed flows to the earth channel at a relatively non-erosive condition.
      ii Since energy dissipator structures are dependent on individual site and hydraulic conditions, detailed criteria for their design is included in the section Criteria for Hydraulic Design of Closed Conduits. Minimum requirements are included herein for the concrete to earth channel transition.
      iii On this basis, the following minimum standards govern the design of concrete to earth channel transitions. The maximum rate of bottom width transitions are:

Water Velocity	
0 -15 f.p.s	1:10
16 - 30 f.p.s.	1:15
31- 40 f.p.s.	1:20

      iv The downstream end of the concrete transition structure will be provided with a cutoff wall having a minimum depth of 4 feet and extending the full width of the concrete section.
      v Directly downstream of the concrete transition structure erosion protection consisting of rough, exposed surface, grouted rock riprap and extending full width of the channel section shall be provided. The grouted rock riprap should be a minimum of 12 inches thick and tied to concrete structure and the cutoff wall. Grout, dumped, or wire-tied material may also be used if approved on a case-by-case basis by the City Engineer. Grouted and wire-tied material require gravel filters as well. Riprap design criteria is presented in the ninth section.

6-7(D)(5) BANK PROTECTION [1]

All berms and levees expected to convey or divert 30 cfs or more in the event of the 100-year design discharge must be provided with bank protection according to the following guidelines:

1. Bank protection must be provided wherever design velocities exceed 5 feet/sec.
2. Bank protection must be provided on the outside of curves from the beginning of curvature, through the curve and for a distance equal to 5 times the flow velocity in feet downstream from the point of tangency.
3. When required, bank protection must be provided to two feet above the design flow depth plus additional depth as required (e.g. superelevation, waves at confluences, hydraulic jumps, etc.).
4. Bank protection must extend downward on a projection of the bank slope, to a minimum depth equal to 1.5 times the design flow depth, but never less than 3.0 feet. Bank protection for major arroyos shall be accompanied by a City Engineer approved sediment transport analysis.

6-7(D)(6) PIERS

The effect of piers on open channel design must be considered at bridge crossings and where an open channel or box conduit not flowing full discharges into a length of multi-barreled box. This effect is especially important when flow is supercritical and when transported debris impinges on the piers.

The total pier width includes an added width for design purposes to account for debris. Inasmuch as the debris width to be used in design will vary with each particular situation, the City Engineer will be contacted during the preliminary design stages of a project for a determination of the appropriate width. Streamline piers should be used when heavy debris flow is anticipated. Refer to Section 6-8 for design data regarding streamline piers.

The water surface elevations at the upstream end of the piers is determined by equating pressure plus momentum. The water surface profile within the pier reach is determined by the Bernoulli equation. The water surface elevations at the downstream end of the piers may be determined by applying either the pressure plus momentum equation or the Bernoulli equation.

6-7(D)(7) CURVING ALIGNMENTS

6-7(D)(7)(I) SUPERELEVATION

Superelevation is the maximum rise in water surface at the outer wall above the mean depth of flow in an equivalent straight reach, caused by centrifugal force in a curving alignment.

6-7(D)(7)(I)(A)RECTANGULAR CHANNELS

For subcritical velocity, or for supercritical velocity where a stable transverse slope has been attained by an upstream easement curve, the superelevation (S) can be calculated from the following equation:

EQUATION 6.25 $$\mathbf{S} = \frac{V^2 b}{2g\, r}$$

[1] *Berms, dams, levees, and diversions of certain magnitudes and nature may fall within the jurisdiction of the State Engineer of the State of New Mexico. The design professional is expected to be aware of and comply with regulations promulgated by that jurisdiction.*

For supercritical velocity in the absence of an upstream easement curve, the superelevation (S) is given by the following equation:

EQUATION 6.26 $S = \frac{V^2 b}{2g\,r}$

where:
V = velocity of the flow cross section, in f.p.s.
b = Width of the channel, in ft.
g = Acceleration due to gravity
r = Radius of channel center line curve, in ft.
X = Distance from start of circular curve to point of first S in ft.
D = Depth of flow for an equivalent straight reach
B = Wave front angle

where:
X = $(\pi bV)/((12gD)^{0.5})$

"S" will not be uniform around the bend but will have maximum and minimum zones which persist for a considerable distance into the downstream tangent.

6-7(D)(7)(I)(B)TRAPEZOIDAL CHANNELS

For subcritical velocity, the superelevation (S) can be calculated from the following equation:

EQUATION 6.27 $S = 1.15V^2 (b + 2\,z\,D) / 2\,g\,r$

where:
z = cotangent of bank slope
b = channel bottom width, in ft.

For supercritical velocity, curving alignments shall have easement curves with a superelevation (S) given by the following equation:

EQUATION 6.28 $S = 1.3V^2 (b + 2\,z\,D) / 2\,g\,r$

6-7(D)(7)(I)(C) UNLINED CHANNELS

Unlined channels will be considered trapezoidal insofar as superelevation calculations are concerned. However, this does not apply to calculations of stream or channel cross-sectional areas.

6-7(D)(7)(II) EASEMENT CURVES

Easement curves are alignment transition curves, employed upstream and downstream of circular curves, when supercritical flow exists in open channels. The purpose of the easement curve is to alter the transverse slope of the water surface and keep the water prism in constant static equilibrium against centrifugal force throughout the entire length of the easement curve and central circular curves, thus achieving minimum heights of superelevation with avoidance of cross-wave disturbances.

Circular easement curves are recommended in lieu of spiral transition curves for each of design and construction. Also very little hydraulic advantage is gained by the use of the spiral. The circular easement curve consists of curved sections upstream and downstream of the main curve having a radius (2R), twice the main curve radius (R).

6-7(D)(7)(II)(A)CONDITIONS REQUIRING EASEMENT CURVES

1. When the freeboard, above superelevated water surface (as calculated without an easement curve), is less than two feet.
2. In reverse curves or on alignments where curves follow one another closely.
3. For any case where elimination of cross-wave disturbances is required. (If easement curves are not used, additional freeboard downstream of the curve may be necessary).
4. In trapezoidal channels for all cases of supercritical velocity.

6-7(D)(7)(II)(B)LENGTH OF EASEMENT CURVE

For rectangular channels, the length of easement curve (L_E) is given by the following equation:

EQUATION 6.29 $\mathbf{L_E} = 2X = 0.32bVD^{0.5}$

For trapezoidal and associated channel types, the length of easement curve (L_E) can be calculated as follows:

EQUATION 6.30 $\mathbf{L_E} = 0.32\,(b + 2zD)\,VD^{0.5}$

Refer to the section on superelevation above for the definition of terms.

6-7(D)(8) FREEBOARD

Freeboard is the additional wall height applied to a calculated water surface.

6-7(D)(8)(I) RECTANGULAR CHANNELS [2]

1. For flow depths of 1.0 feet or less and average flow velocities less than 35 f.p.s., add l.0 feet.
2. For flow depths of 1.0 feet or less and average flow velocities greater than 35 f.p.s., add 1.5 feet.
3. For flow depths of greater than l.0 feet and average flow velocities of less than 35 f.p.s., add 2.0 feet.
4. For flow depths of greater than 1.0 feet and average flow velocities of greater than 35 f.p.s., add 3.0 feet.
5. For supercritical flow where the depth is between DC and 0.80 DC, the wall height must be equal to the sequent depth, but not less than the heights required above. This condition should be avoided.
6. Freeboard requirements for concrete drainage easement channels shall be established by the City Engineer on a case-by-case basis.

2 *Not used except with City Engineer approval.*

6-7(D)(8)(II) TRAPEZOIDAL CHANNELS AND ASSOCIATED TYPES

Adequate channel freeboard above the designed water surface must be provided and will not be less than determined by the following:

1. For flow rates of less than 100 c.f.s. and average flow velocity of less than 35 f.p.s.:

   EQUATION 6.31 **Freeboard (Feet)** $= 1.0 + 0.025Vd^{1/3}$

2. For flow rates of 100 c.f.s. or greater and average flow velocity of 35 f.p.s. or greater:

   EQUATION 6.32 **Freeboard (Feet)** $= 0.7\ (2.0 + 0.025Vd^{1/3})$

Freeboard will be in addition to any superelevation of the water surface, standing waves and/or other water surface disturbances. When the total expected height of disturbances is less than 0.5 feet, disregard their contribution.

Unlined portions of the drainage way may not be considered as freeboard unless specifically approved by the City Engineer.

For supercritical flow where the specific energy is equal to or less than 1.2 of the specific energy at D_c, the wall height will be equal to the sequent depth, but not less than-the heights required above. This condition should be avoided.

6-7(D)(8)(III) ROLL WAVES

Roll waves, sometimes known as slug flow, are intermittent surges on steep slopes that will occur when the Froude Number (F) is greater than 2.0 and the channel invert slope (S0) is greater than the quotient, twelve divided by the Reynolds Number. When they do occur, it is important to know the maximum wave height at all points along the channel so that appropriate wall heights may be determined based on the experimental results of roll waves by Richard R. Brock, the maximum wave height can be estimated.

6-7(E) CHANNEL DESIGN CRITERIA

6-7(E)(1) UNLINED CHANNELS

After full consideration has been given to the soil type, velocity of flow, desired life of the channel, economics, availability of materials, maintenance and any other pertinent factors, an unlined earth channel may be approved for use. Generally, its use is acceptable where erosion is not a factor and where mean velocity does not exceed 3 f.p.s. Old and well-seasoned channels will stand higher velocities than new ones; and with other conditions the same, deeper channels will convey water at a higher nonerodible velocity than shallower ones. Additional information is provided in Section 6-8.

Maximum side slopes are determined pursuant to an analysis of soil reports. However, in general, slopes should be 3:1 or flatter.

6-7(E)(2) COMPOSITE LININGS
In case part of the channel cross section is unlined or the linings are composed of different materials, a weighted coefficient must be determined using the roughness factors for the materials.

6-7(E)(3) MAXIMUM SIDEWALL SLOPES
The following sidewall slopes are generally the maximum values used for channels on at least one side of the concrete lined channel. The road should be sloped away from the channel, and roadway runoff carried in a controlled manner to the channel.

Lining Material	Maximum Slope
Soil Cement	2:1
Portland Cement Concrete Vertical	2:1 (trapezoidal)
Grouted Rock Rip-Rap	2:1
Dumped Rock Rip-Rap	2:1
Earth Lined	3:1
Grass Lined (sodded)	4:1

6-7(E)(4) CHANNEL MAINTENANCE AND ACCESS ROAD
A maintenance and access road having a minimum of 12 feet top width shall be provided on at least one side of improved channels. In some cases the City Engineer may require additional width. Channel maintenance and access roads shall, at a minimum, be surfaced with gravel base course. The thickness of said base course shall be 6 inches west of the Rio Grande, 4 inches east of the Rio Grande.

Turnouts will be provided at no more than ½ mile intervals and turnarounds must be provided at all access road dead ends.

Ingress and egress from public right-of-way and/or easements to the channel maintenance and access road must be provided.

6-7(E)(5) CHANNEL ACCESS RAMPS
Channel access ramps for vehicular use will be provided as necessary for complete access to the channel throughout its entire length with the maximum length of channel between ramps being one-half mile.

Ramps shall be constructed of 8" thick reinforced concrete and will not have slopes greater than 17% and ramps shall not enter the channel at angles greater than 15o from a line parallel to the channel centerline.

Ramps will be constructed on the same side of the channel as the maintenance and access road. The maintenance and access road shall be offset around the ramp to provide for continuity of the road full length of the channel.

The downhill direction of the ramp should be oriented downstream.

6-7(E)(6) STREET CROSSINGS

Street crossing or other drainage structures over the concrete lined channel should be of the all weather type, i.e., bridges or concrete box culverts. Crossing structures should conform to the channel shape in order that they disturb the flow as little as possible.

It is preferred that the channel section be continuous through crossing structures. However, when this is not practicable, hydraulic disturbance shall be minimized, and crossing structures should be suitably isolated from the channel lining with appropriate joints.

Street crossing structures shall be capable of passing the l00 year frequency design storm flows.

Channel lining transitions at bridges and box culverts should conform to the provisions for transitions hereinafter provided. Drainage structures having a minimum clear height of 8 feet and being of sufficient width to pass maintenance vehicles may result in minimizing the number of required channel access ramps. Unless otherwise specifically authorized by the City Engineer, all crossing structures must have at least 6.0 feet of clear height.

6-7(E)(7) SUBDRAINAGE

Concrete lined channels to be constructed in areas where the ground water table is greater than two feet below the channel invert, weep holes or other subdrainage systems are not required.

Areas where the ground water table is within two feet or less of the channel bottom, there shall be provided, special subdrainage systems as necessary to relieve water pressures from behind the channel lining.

6-7(E)(8) CHANNEL BED WIDTH

The minimum channel (soft or hard bottomed channels) bed width is 10 feet for publicly maintained channels.

6-7(F) MISCELLANEOUS HYDRAULIC CALCULATIONS

6-7(F)(1) HYDRAULIC JUMP

6-7(F)(1)(I) LOCATION

If the water surface from a downstream control is computed until critical depth is reached, and similarly the water surface from an upstream control is computed until critical depth is reached, a hydraulic jump will occur between these controls and the top of the jump will be located at the point where pressure plus momentum, calculated for upper and lower stages, are equal.

6-7(F)(1)(II) LENGTH

The length of a jump is defined as the distance between the point where roller turbulence begins and water becomes white and foamy due to air entrainment, and the point downstream where no return flow is observable.

1. For rectangular channels, the length of jump (L) for the range of Froude Numbers between two and twenty, based on flow depth, is given by the following equation:

   EQUATION 6.33 $\mathbf{L} = 6.9\,(D_2 - D_1)$

   where:
   D_1 and D_2 are the sequent depths.

2. For trapezoidal channels, the length of jump (L) is given by the following equation:

   EQUATION 6.34 $\mathbf{L} = 5D_2(1+4\,(t_2-t_1/t_1)^{0.5})$

   where:
   $\mathbf{t_1}$ = width of water before jump
   $\mathbf{t_2}$ = width of water after jump

Side Slope	$L/(D_2-D_1)$
2:1	44.2
1:1	33.5
1/2:1	22.9
Vertical	6.9

6-7(F)(2) TRASH RACK HEAD LOSS

The head loss through a trash rack is commonly determined from the following equation:

EQUATION 6.35 $\mathbf{h_{TR}} = K_{TR}\,(V_n/2_g)$

EQUATION 6.36 $\mathbf{K_{TR}} = 1.45 - 0.45\,(A_n/A_g) - (A_n/A_g)^2$

where:
$\mathbf{K_{TR}}$ = Trash rack coefficient
$\mathbf{A_n}$ = Net area through bars, in ft.^{2}
$\mathbf{A_g}$ = Gross area of trash rack and supports (water area without trash rack in place), in ft.^{2}
$\mathbf{V_n}$ = Average velocity through the rack openings (A/A_n), f.p.s.

For maximum head loss, assume that the rack is clogged, thereby reducing the value of A_n by 50%.

6-7(F)(3) SIDE CHANNEL WEIRS:

The City or AMAFCA may require a side channel spillway inlet for drains outletting into their facilities. The Corps' procedure for designing a side channel spillway is as follows:

1. Set the top of that part of the main channel wall at the location of the proposed spillway about 6 inches above the computed water surface level in the main channel.
2. Determine the length of spillway (L) required to discharge the design inflow of the side inlet by the following equation, in which the maximum value of H is not greater than one and one-half feet.

EQUATION 6.37 $L = \frac{Q}{CH^{3/2}}$

where:
Q = discharge of side inlet, in c.f.s.
C = weir coefficient
H = depth of water over the crest of the side inlet in feet

3. Determine the depth of flow in the approach side channel at the upstream end of the spillway.
4. Set the side channel invert elevation at the upstream end of the spillway at an elevation below the spillway crest a distance equal to the water depth as determined in c., above, minus the assumed head on the spillway.
5. Set the side channel invert slope equal to the spillway and the main channel water-surface slopes.
6. By trial, determine the width of the side channel required to maintain a constant depth of flow at several points downstream from the upstream end of the spillway. The discharge at each of these points is assumed to be the difference between the initial discharge less the amount spilled over that part of the spillway as computed by $CLH^{3/2}$, in which C is 3.087 and H is equal to the critical depth over the crest (neglecting the velocity of approach).
7. Plot the widths thus determined for the side channel on the channel plan and approximate a straight or curved line through them to locate the point of intersection of this line and the main channel wall.
8. If the length between the assumed point at the upstream end of the spillway and this intersection point is equal to the length determined in 2., above, the angle at the intersection indicates the required convergence for the side channel.
9. From the final layout determine the width and recompute the water surface in the side channel for the final design. The discharge over each portion of the spillway is calculated by using the average head between the two sections considered.

6-7(G) CHANNEL TREATMENT SELECTION GUIDELINES

6-7(G)(1) GENERAL

The selection of a treatment type or of a combination of treatment types for a channel within the Albuquerque area should be based on a rational assessment of the needs of the community as they relate to:

6-7(G)(2) FLOOD CONTROL

The magnitude of the flood control requirements and the consequences of a system failure should be considered foremost in the treatment selection process.

6-7(G)(3) DRAINAGE

The existing and future land uses, the specific on- and off-site drainage treatments, and watershed topography should each be evaluated in terms of their impacts on the channel system. The unmitigated hydrologic effects of urbanization generally include higher peak runoff rates from small frequent storms, more frequent runoff events, cleaner runoff (with respect to sediment), and increased annual runoff volumes.

6-7(G)(4) MAINTENANCE

The selection of a channel treatment type should include analyses of both short and long term maintenance. While maintenance efforts will vary between treatment types, all facilities should be able to function through one runoff event with no maintenance, through one flood season with very little maintenance and from season to season with regular, but minimal maintenance requirements.

6-7(G)(5) RIGHTS-OF-WAY AND EASEMENTS

The cost of land and the availability of rights-of-way or easements should be considered in the channel treatment selection process. Rights-of-way and easements should be appropriately located, aligned and sized for the particular treatment type. Some treatment types may require significant construction easements, but much smaller permanent rights-of-way or easements. The likelihood of replacement or reconstruction should be considered when channel treatment selection is balanced against the configuration of permanent rights-of-way and easements.

6-7(G)(6) SAFETY AND FENCE REQUIREMENTS

The selection of a channel treatment type should be based on any special safety considerations dictated by adjacent or nearby land uses. Whenever a required channel treatment is not compatible with adjacent land uses, adequate safety hazard mitigation measures should be incorporated into the design and construction of the facilities. Channels with vertical walls of 30 inches or greater will require a barrier or fence. Minimum fence or barrier height shall be 42 inches.

6-7(G)(7) UPSTREAM AND DOWNSTREAM CHANNEL TREATMENTS

The treatment selection process for each channel reach should include an analysis of the impacts of existing and planned upstream and downstream treatment types on a proposed treatment type and in turn the effects of the proposed treatment on existing and planned upstream and downstream treatments.

6-7(G)(8) INITIAL COST AND LIFE EXPECTANCY

The initial construction costs of various channel treatment types is, and will always be, one of the most heavily weighted factors in the selection process. However, when viewed on a larger scale, maintenance and replacement costs can be more important to the total costs of providing adequate levels of protection over time, and therefore must be considered in the planning, design and construction of channel treatment measures.

6-7(G)(9) JOINT USE POSSIBILITIES

The opportunities for including other uses such as transportation and utility corridors, open space or recreation in the design should be considered when selecting a treatment type and when establishing rights-of-way and easements. The inclusion of any other uses must be self-supporting financially and in no way impair or delay the implementation of the drainage and flood control function of the facilities.

6-7(G)(10) SEDIMENT TRANSPORT AND CHANNEL STABILITY

Movig water has the ability to transport sediment. The amount of sediment per unit of water that can be transported is related to flow depth, velocity, temperature, vertical and horizontal channel alignment, the amount of sediment available, the size and density of the sediment available and many other minor but sometimes important parameters. A channel's stability can be defined in terms of its ability to function properly during flood event without serious aggradation and/or degradation and that its continued operation can be relied upon without extraordinary maintenance and repairs. While channel stability problems are largely associated with earth and flexibly lined channels, concrete lined, supercritical channels are not immune. Any time a downstream channel reach has a lower sediment capacity than some upstream reach, there is a potential for sediment accumulation. The following worksheets can be used to make qualitative determinations with regard to channel stability.

Detailed qualitative analyses must be performed for any design requiring construction in a major arroyo. Methods found in items C.7 and C.8 in the Bibliography at the end of Section 6-1(D)(2) shall be used in sediment transport analyses.

6-7(G)(11) CHANNEL STABILITY

A stable earth-lined channel is defined for the purposes of design as one in which neither degradation or aggradation is occurring at such a rate that it causes a continuous and serious maintenance problem. Channel degradation can cause extensive damage to bridges and other crossing structures due to the undermining of their foundations. Channel aggradation, on the other hand, results in reduced channel and crossing structure capacities and, therefore, in increased frequency of flooding.

TABLE 6.13 CHANNEL STABILITY CHANGES

An increase or decrease in:	Will have the following effect in the channel:	
	Increase	Decrease
Flow Rate	Degradation	Aggradation
Flow Velocity	Degradation	Aggradation
Flow Frequency	Degradation	Aggradation
Flow Duration	Degradation	Aggradation
Flow Depth	Degradation	Aggradation
Sediment Reaching the Channel	Aggradation	Degradation
Sediment Particle Size	Aggradation	Degradation
Streambed Material Size	Aggradation	Degradation
Channel Vegetation	Aggradation	Degradation

6-7(G)(12) CHANNEL CONSTRUCTION DETAILS

6-7(G)(12)(I) EARTHWORK

The following shall be compacted to at least 90% of maximum density as determined by ASTM D-1557 (modified Proctor):

1. The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).
2. Top 12 inches of maintenance road. (either as subgrade or finished roadway if unsurfaced).
3. Top 12 inches of earth surface within 10 feet of concrete channel lip. It is particularly important to compact earth immediately adjacent to concrete lip. This area is sometimes overlooked when forms are removed.
4. All fill material.

6-7(G)(12)(II) CONCRETE MATERIALS

1. Cement type: ILA or I-IILA
2. Minimum cement content: 5.5 sacks/c.y.
3. Maximum water-cement ratio: 0.53 (6 gals. per sack)
4. Maximum aggregate size: 1 ½ inches
5. Air content range: 4-7%
6. Maximum slump: 3 inches
7. Minimum compressive strength (f_c): 3000 psi at 28 days
8. Class F Flyash meeting the requirements of ASTM C618 shall be proportioned in the mix at a 1:4 ratio of flyash to cement weight.
9. Steel reinforcement shall be grade 60 deformed bars. Wire mesh shall not be used.

6-7(G)(12)(III) CONCRETE LINING

1. Bottom width - 10 feet minimum
2. Side Slopes - 1 vertical to 2 horizontal maximum slope
3. Concrete lining thickness

All concrete lining shall have a minimum thickness of 6 inches. The lining shall be thickened to 7 inches on the channel bottom and lower 18 inches of the side slope. When design velocity exceeds 30 feet per second, the bottom section shall be thickened to 8 inches.

6-7(G)(12)(IV) CONCRETE FINISH

The surface of the concrete lining shall be provided with a wood float finish. Precautions shall be taken to guard against excessive working or wetting of finish.

6-7(G)(12)(V) CONCRETE CURING

All concrete shall be cured by the application of liquid membrane-forming curing compound (white pigmented) immediately upon completion of the concrete finish.

6-7(G)(12)(VI) STEPS

Ladder-type steps shall be installed at locations suitable for rescue operations along the channel but not farther than 700 ft. apart on both sides of the channel. Bottom rung shall be placed approximately 12 inches vertically above channel invert.

6-7(G)(12)(VII) JOINTS

1. Insofar as feasible, channels shall be continuously reinforced without transverse joints. However, expansion joints may be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.
2. The preferred design avoids longitudinal joints. However, if included, longitudinal joints should be on side slope at least one foot vertically above channel invert.
3. All joints shall be designed to prevent differential displacement and shall be watertight.
4. Construction joints are normally appropriate at the end of a day's run, where lining thickness changes, and any time concrete placement stops for more than 45 minutes.

6-7(G)(12)(VIII) REINFORCING STEEL FOR CONTINUOUSLY REINFORCED CHANNELS

1. Ratio of longitudinal steel area to concrete area

$$A_{s\ long} / A_{c\ long.} > 0.005$$

2. Ratio of transverse steel area to concrete area [3]

$$A_{s\ transv.} / A_{c\ transv.} > 0.0025$$

3. For steel Placement the temperature and shrinkage steel shall be placed so as to be in the top of the middle third of the slab, but at least 3" from the bottom of the slab. Longitudinal steel shall be on tip of the transverse steel. [4]

3 *In (1) and (2) above As = crossectional area of steel in the direction indicated; Ac = crossectional area of concrete in the direction indicated. Longitudinal = long.; transverse = transv.*
4 *Inspectors must insure this requirement is not violated by contractors during pouring operations.*

6-8 STREET HYDRAULICS

A secondary use of the street network is the conveyance of stormwater runoff. This secondary use must always be subsidiary to the primary function of streets which is the safe conveyance of people and vehicles. The goals of street hydraulic design are therefore:

1. To provide an economical means of transporting stormwater runoff.
2. To ensure that the safety and convenience of the public are preserved.
3. To prevent stormwater runoff, once collected by the street system, from leaving the street right-of-way except at specially designated locations.

6-8(A) STREET HYDRAULIC DESIGN CRITERIA

Street hydraulic design critical are as follows

1. Manning's roughness coefficient is 0.017.
2. The calculated HGL for the 100-year design discharge may not exceed curb height and the calculated EGL shall be contained within the street right-of-way.
3. For a sump condition, the HGL for the 100-year storm may extend to the street right-of-way.
4. For storm events less than or equal to the 10-year design discharge one lane free of flowing or standing water in each traffic direction must be preserved on arterial streets.
5. The product of depth times velocity shall not exceed 6.5 in any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet.)
6. Gutter pan slope should be accommodated in the street cross-section.
7. The street cross section should be shown graphically. T-intersections, radical slope changes and intersections are potential locations for hydraulic jumps when upstream slopes are steeper than critical slope.
8. The assumption of equal flow distribution between gutters on undivided streets and between street sections on divided streets is only valid where its validity can be demonstrated.

6-8(B) OPTION TO DRAIN THE STREET TO THE MEDIAN

For arterial streets with a median, the street cross-section may be changed to drain the street in the median rather than to the outside edges of the roadway.

6-8(C) EFFECTS OF HYDRAULIC JUMP OR SUPERELEVATION

When conditions indicate that a hydraulic jump or that the effects of superelevation will allow runoff to exceed street hydraulic design criteria, provisions must be made for treatment of the problem. The warping of street sections and the construction of deflector walls for these purposes is prohibited unless specifically authorized by the City Engineer.

6-8(D) INTERSECTION

Intersections and other radical changes in street cross section and slope require special consideration whenever the flow depth/street slope relationship results in flows occurring in the supercritical flow regime. The critical slope line shown on the street rating curves is used to determine on which side of critical depth the flow occurs and if slope or cross section changes will allow the flow to cross through critical depth from supercritical. If flow is likely to cross into the subcritical flow range, the height and length of hydraulic jump must be demonstrated in the drainage report.

6-8(E) DRAINAGE DESIGN CRITERIA IN STREET DESIGN.

1. Nuisance flows will not be conveyed across arterial or collector streets on the surface by valley gutters or other means. Valley gutters conveyance of nuisance flows across major local streets is discouraged. Provisions for storm drainage inlets to meet this requirement must be included at all intersections of major streets (collector or above) as defined by the Long Range Roadway System Plan.
2. The use of quarter point crown (i.e. high point of crown at mid-lane on high side of street) is preferred over the use of full side-hill street configuration to prevent sheet flow across pavement surfaces.
3. Transitional pavement surface approaches to intersections must be designed to contain nuisance flows within gutter lines; valley gutters must be provided to accommodate flows across intersections suitably, parallel to the major traffic carrying street.
4. Arterial, collector and sole access streets to subdivisions may not employ at-grade or dip section crossings of arroyos. Specific criteria for design of these crossings is given in Chapter 6.
5. For undesignated roadways, valley gutters will be required to convey flows across the roadway.
6. Dip or overflow sections will only be permitted on local streets with the approval of the Traffic Engineer and the City Engineer.
   a. *Dip or overflow sections may only be used where the depth of flow times the velocity of flow over the roadway including sidewalks will not exceed 6.5 for that portion of the 10-year storm runoff crossing over the street. Velocity is to be calculated as the velocity measured in feet per second and the flow depth is to be measured in feet at the upstream edge of the roadway including sidewalk.*
   b. *If dip sections are permitted, vertical alignment must satisfy the requirements in Chapter 7 for sight distances considering the design speed of the street in question.*

6-8(F) INLET PLACEMENT AND DESIGN CRITERIA

Inlets should be placed to meet the street flow criteria discussed above. Size and type of inlets should be determined by physical requirements and by grate and flow capacities given in Plates 22.8 D-1 and 22.8 D-2, inclusive. Criteria used, if other than those recommended in this section, must be cited and accompanied by appropriate calculations. Inlet spacing should be per Plate 22.8 D-3.

6-8(F)(1) STANDARD INLETS

The selection of type, number, and spacing of inlets should be based on Plates 22.8 D-I through 22.8 D-4 and the following instructions. A bicycle safe grate should be used with "Type A and C inlets".

City standard inlets "Type A, Type B and C" are combination inlets with both curb opening and grates. Inlet "Type D" is a grate only inlet. Inlet gratings tend to accumulate debris and clog. The curb opening both limits debris accumulation and offsets lost capacity due to clogging of the grating. Except for certain valley applications, combination inlets should be used. Due to main line clogging, grating only inlets should be used in valley applications where main line pipe diameters are 24" or less or where quarter full pipe velocities are less than 2.5 f.p.s.

"Type A" inlets should be used for single basin applications and as the first basin in a battery of basins. The "Type A" inlet performs the function of sweeping debris of the street upstream of the grating and minimizing clogging. "Type A" inlets are used with standard 8" curb and gutter. The capacity is shown in Plates 22.8 D-.1

"Type B" inlets are generally placed downstream of and/or in conjunction with "Type A" inlets on streets other than arterials and collectors. This inlet type has potential to collect substantial runoff when the grating is clean. If "Type B" inlets are used alone, without a "Type A" within 150 feet upstream, the capacity shown in Plate 22.8 D-3 should be reduced by 15% due to clogging. "Type B" inlets are used with standard 8" curb and gutter. A bicycle safe grate shall be used with a "Type B" inlet.

"Type C" inlets are generally placed downstream of and/or in conjunction with "Type A" inlets. If "Type C" inlets are used without a "Type A" within 150 feet upstream, the capacity shown in Plates 22.8 D-1 and 22.8 D-2 should be reduced 15% for clogging. "Type C" inlets are used with standard 8" curb and gutter.

"Type D" inlets are generally used on streets with slope greater than 5%, in driveways and in certain valley areas as described above. "Type D" inlets can be used with either standard 8" curb and gutter or with mountable curb. The capacity shown in Plates 22.8 D-1 and 22.8 D-2 should be reduced 15% for clogging.

The number of inlets to be connected in series should not exceed two. If the connection of more than two catch basins in series is unavoidable, consideration should be given to designing a lateral drain.

The capacity of the lateral storm drain is to be considered when placing inlets as the grate capacity may be limited by the lateral storm drain.
If there is a conflict with an existing Type "A" or "C" inlet with a proposed plan the following criteria should apply:

1. The conversions of type A's, or C's to Type D inlets will be permitted if a throated inlet is within 150 feet upstream.
2. If there is not a throated inlet within 150 feet upstream, the conversions of Type A's, or C's to Type D inlets will be permitted if a throated inlet is added within 150 feet upstream.

3. Or the inlet shall be removed and replaced with an inlet outside the conflict zone.
4. The inlet can be removed and replaced with a Type Double-D inlet

The engineer should verify there is adequate clearance for proposed driveways near inlets. If an apparent conflict exists the proposed drive-ways near inlets should be shown on the grading plan and shall be shown on the DRC construction plans.

If there is a conflict with a Type "B" or cattle guard inlet, the inlet is to be removed and replaced outside the conflict zone.

Figure 6.6 Grating Capacities for Type "A", "C", and "D"

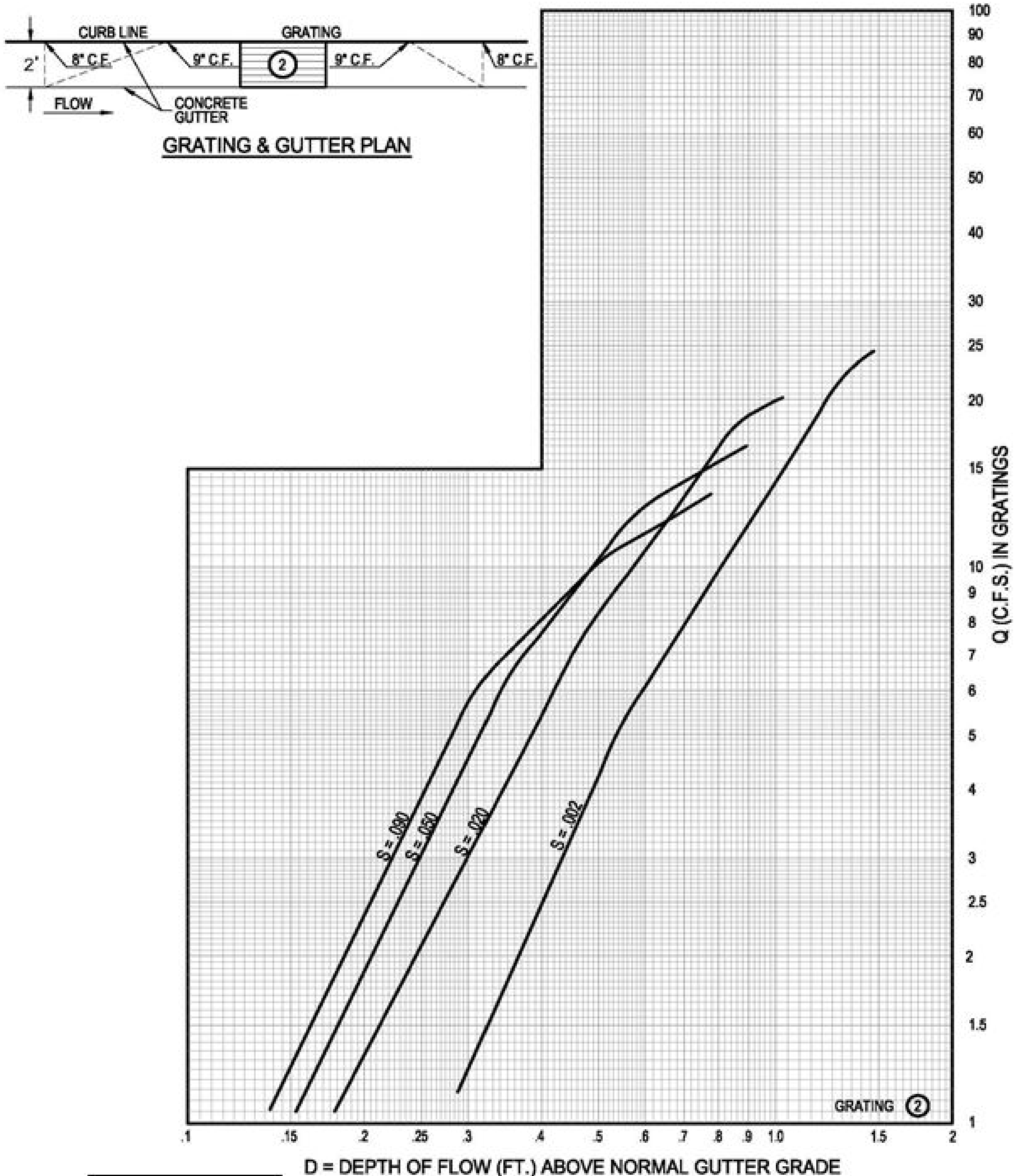


Plate 22.8 D-1

Figure 6.7 Grating Capacities for Type Double "C". "D", and "A"

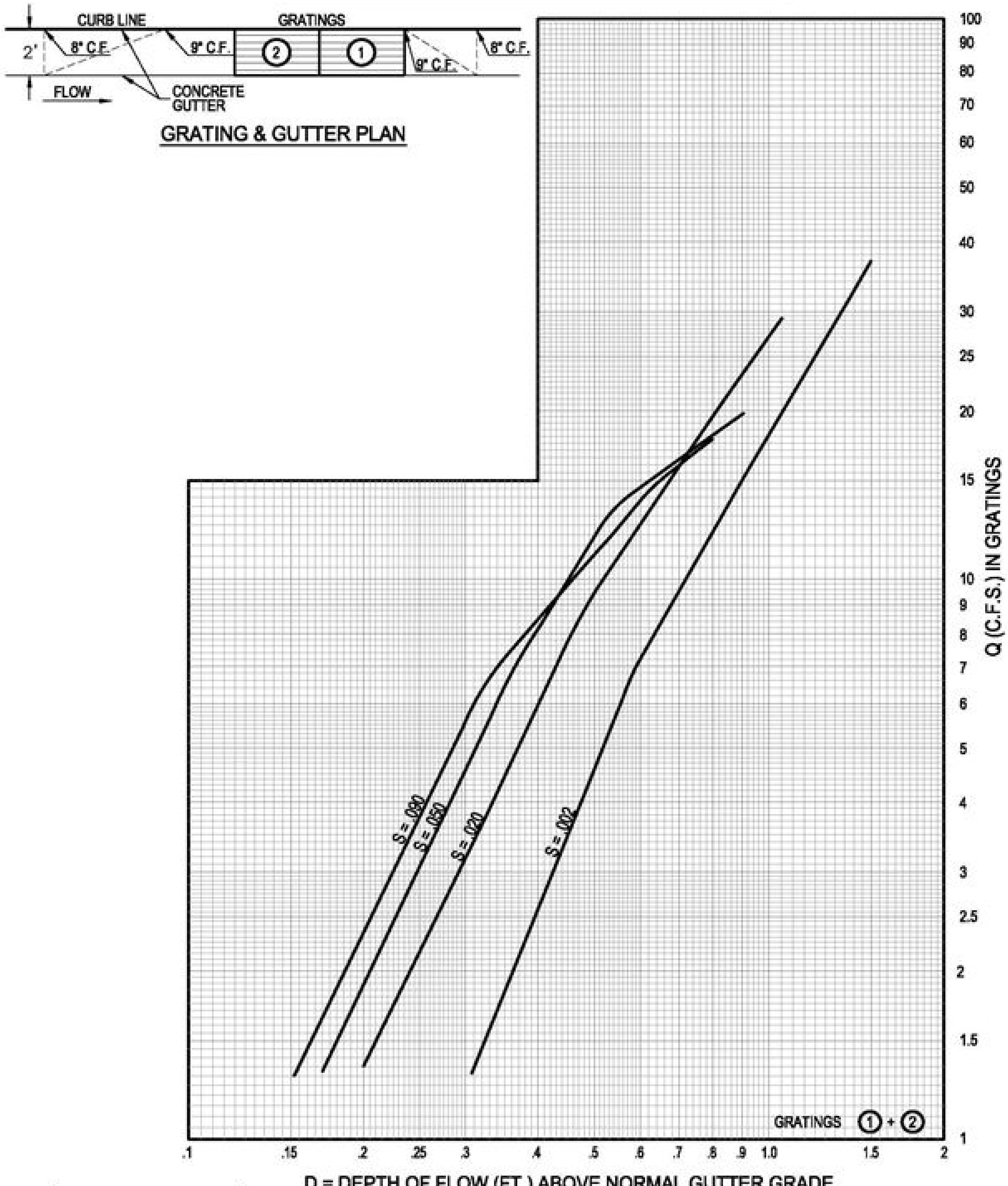


Plate 22.8 D-2

Figure 6.8 Grating Capacities for Type B

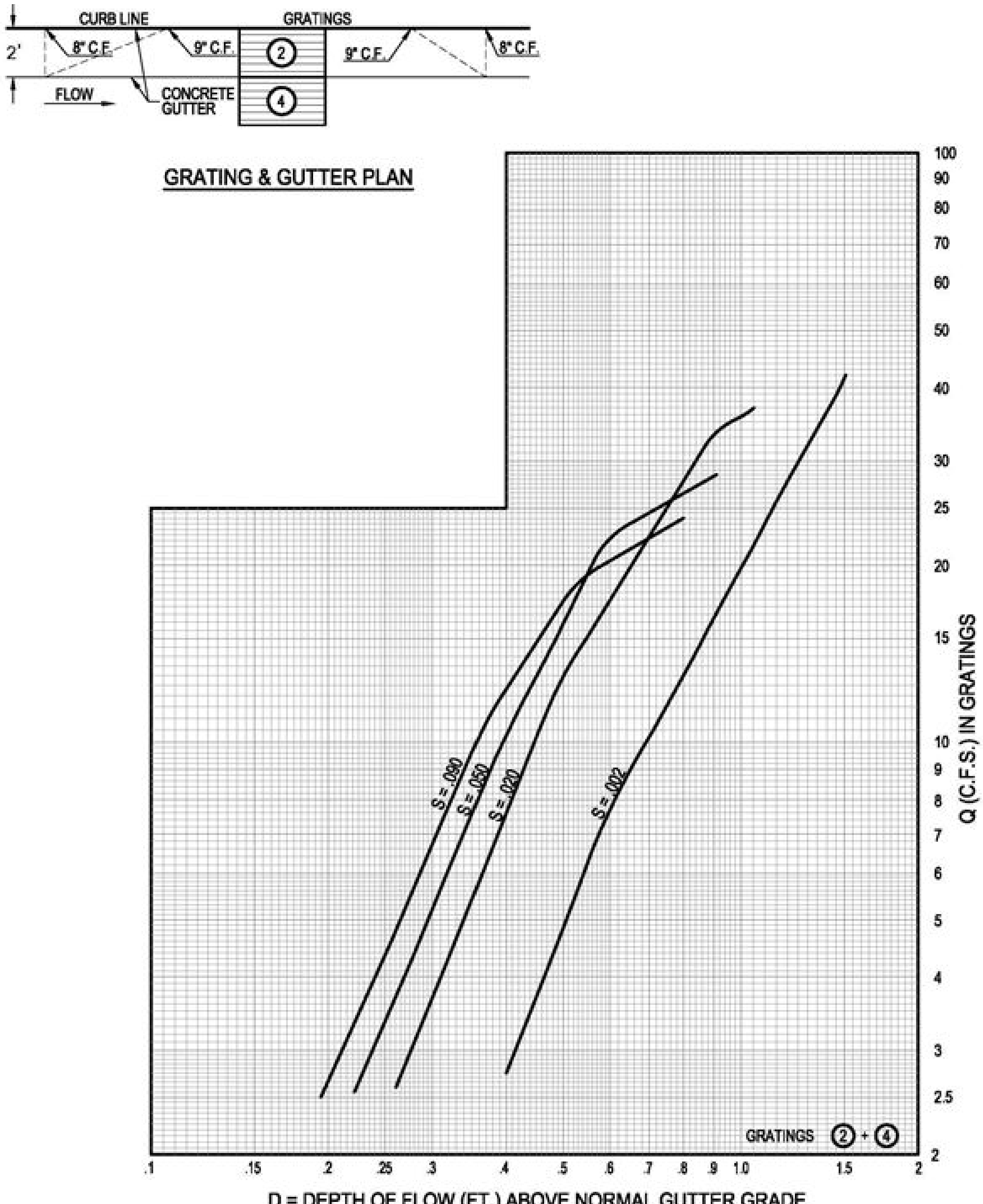


Plate 22.8 D-3

Figure 6.9 **Optimum Spacing of Catch Basins on a Continious Grade**

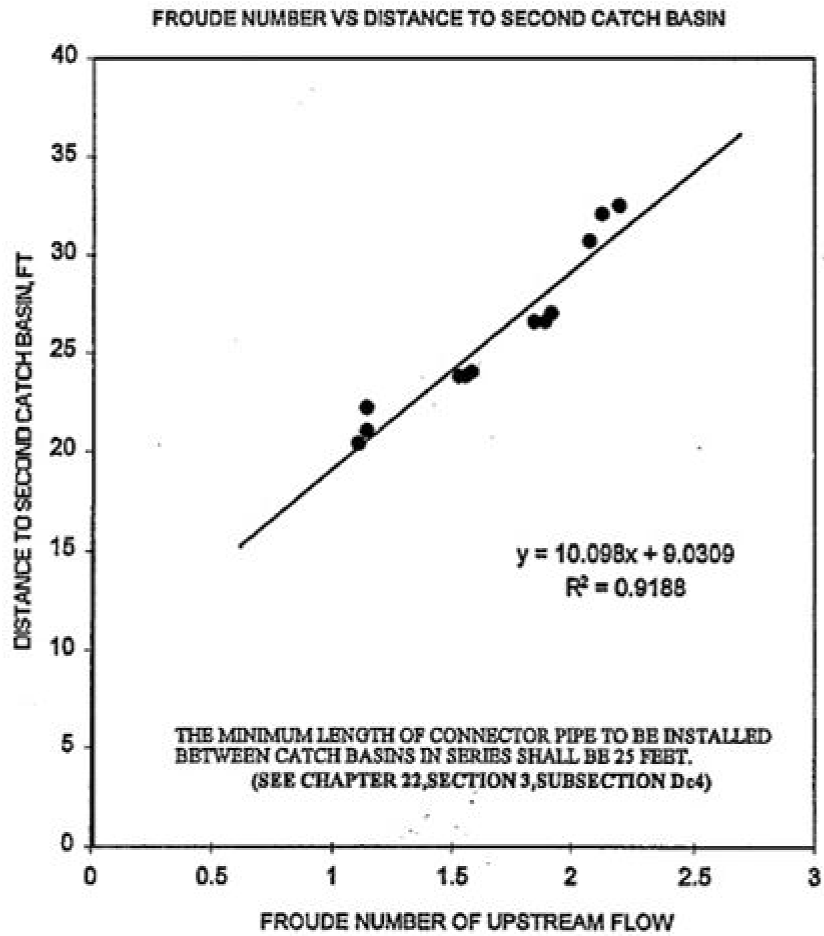


Plate 22.8 D-4

6-8(F)(2) CATTLE-GUARD AND MEDIAN INLETS

Standard drawings are available for cattle-guard and median inlets. Plates presented earlier in this section were for the capacity of Type A, C and D inlets. The engineer shall provide calculations for capacity when proposing cattle-guard and median inlets. A bicycle safe grate shall be used with a cattle- guard inlet.

6-8(F)(3) PUBLICLY MAINTAINED INLETS TO BE LOCATED WITHIN STREET RIGHTS-OF-WAY

Inlets will be located within street rights-of-way unless otherwise approved by the City Engineer. Inlets located outside of Right-of-way require an easement with beneficiary and maintenance responsibilities defined.

Construction of inlets that will be located outside constructed streets to accommodate future street widenings is discouraged. However, the lateral storm drain stub shall be constructed past the permanent pavement section.

Inlets to be constructed off the paved portion of the roadway but within the street property lines must be made operable by grading the roadway to permit storm water to flow to the inlet. The area around the inlet shall be adequately protected from erosion and sedimentation.

6-8(F)(4) INLETS IN A SUMP CONDITION

Sump designs for should normally be limited to local streets and only those situations where terrain or grading considerations warrant their use. When specifying a sump inlet(s) the designer shall ensure that surrounding properties are protected from the occurrence of inlet and lateral clogging by demonstrating that one of the following emergency backup conditions exist:

1. The design storm peak flow rate will release to either a public R.O.W. or public easement without rising above any adjacent structure pad elevations.
    a. *When relying on public easements across private property for this option, the easement language creating the encumbrance shall specify that said easement is a Public Drainage Easement and no structural improvements which would interfere with conveyance or storage of water shall be allowed. Any surface modification within the drainage easement will require an encroachment agreement from the City.*
    b. If the subdivision or street network design does not lend itself to releasing the drainage as stated above, it is acceptable to double the number of sump inlets. The additional inlet(s) are an emergency overflow in case the inlet(s) required to carry the peak flow are clogged.
2. Sufficient storage is available within a combination of public R.O.W., public easement, to hold 100% of the design event volume, without inflicting damage to structures.

6-8(G) INLET LATERAL AND CONNECTOR PIPE CAPACITY

When designing inlets to capture stormwater from the street, the capacity of the lateral (pipe connecting inlet to main line) pipe and the capacity of connector (inlet to inlet) pipes must be determined. Calculations are to be included in the drainage report or plan.

The capacity can be shown with gravity flow using manning's equation or by pressure flow using an acceptable modelling program. The program must meet the following criteria to be accepted:

1. Be able to produce an illustration of the HGL and EGL.
2. Have the ability to include major and minor losses.
3. Meet technical requirements of this chapter.
4. If requested by the City Engineer, the design engineer shall provide a description of how the model meets the requirements of this chapter.

6-8(G)(1) CONNECTOR AND LATERAL PIPE CRITERIA

1. The minimum diameter of connector and lateral pipes is 18 inches.
2. The horizontal alignment of lateral and connector pipes must not contain angle points or bends, unless approved by the City Engineer.
3. Lateral connections to the main line are preferred at manholes or junction structures. Exceptions to this criterion must be approved by the City Engineer. Lateral pipes connecting to a main line from both sides of a street (not using a manhole) should be offset 8 feet or more at the main line and require City Engineer approval.
4. The inlet spacing shall be a minimum of 30 feet center of downstream grate to center of upstream grate.
5. Catch basin connector pipes shall outlet at the downstream end of the catch basins, unless prevented by field conditions. Downstream, in this paragraph, refers to the directions of the gutter slope at the catch basin in question.

6-8(G)(2) CONSIDERATION OF EXISTING DRAINAGE SYSTEMS DURING CONSTRUCTION

Existing drainage systems which are not required to carry any portion of the design Q of a proposed system may be designated to be abandoned in place upon completion of the proposed drain. Such existing drainage systems should not be sealed or removed before completion of the proposed system, if needed to carry off storm water during the construction period. It is the designer's responsibility to ascertain the necessity of maintaining existing drainage systems in place.

Existing street or sidewalk culverts may be designated to have the interfering portions removed and the inlets sealed, or the culverts may be kept in operation and connected to the storm drain or to the back of a proposed catch basin. If the culvert is to be connected, a structural detail should be provided. Refer to the City Engineer for instructions.

Existing street or sidewalk culverts that do not interfere with construction should be maintained in place.

If the existing culvert is located in, or its required to drain a sump, the designer should make every effort to avoid removal of the culvert, especially in instances where the capacity of the proposed drain is less than that required for the correct design frequency.

6-9 CRITERIA FOR HYDRAULIC DESIGN OF CLOSED CONDUITS

6-9(A) GENERAL HYDRAULIC CRITERIA

Closed conduit sections (pipe, box or arch sections) will be designed as flowing full and, whenever possible, under pressure except when the following conditions exist:

1. In some areas of high sediment potential, there is a possibility of stoppage occurring in drains. In situations where sediment may be expected, the City Engineer must be consulted for a determination of the appropriate bulking factor.
2. In certain situations, open channel sections upstream of the proposed closed conduit may be adversely affected by backwater.

If the proposed conduit is to be designed for pressure conditions, the hydraulic grade line shall not be higher than the ground or street surface, or encroach on the same in a reach where interception of surface flow is necessary. However, in those reaches where no surface flow will be intercepted, a hydraulic grade line which encroaches on or is slightly higher than the ground or street surface will be acceptable provided that pressure manholes exist or will be constructed.

6-9(B) HYDRAULIC GRADE LINE CALCULATIONS

6-9(B)(1) DETERMINATION OF CONTROL WATER SURFACE ELEVATION

A conduit to be designed for pressure conditions may discharge into one of the following:

1. A body of water such as a detention reservoir
2. A natural watercourse or arroyo
3. An open channel, either improved or unimproved
4. Another closed conduit

The controlling water surface elevation at the point of discharge is commonly referred to as the control and, for pressure flow, is generally located at the downstream end of the conduit. If flow becomes unsealed, the control may be at the first grade break upstream of the point where unsealing occurs or, under certain conditions, may be farther upstream.

Two general types of controls are possible for a conduit on a mild slope, which is a physical requirement for pressure flow in discharging conduits.

1. Control elevation above the soffit elevation. In such situations, the control must conform to the following criteria:
   a. *In the case of a conduit discharging into a detention facility, the control is the 10-year water surface reservoir elevation.*
   b. *In the case of a conduit discharging into an open channel, the control is the 10-year design water surface elevation of the channel.*

c. *In the case of a conduit discharging into another conduit, the control is the design hydraulic grade line elevation of the outlet conduit immediately upstream of the confluence.*
   i Whenever case (a) or (b) above is used, the possibility of having flow out of manholes or inlets due to discharge elevations at the 100-year level must be investigated and appropriate steps taken to prevent its occurrence.

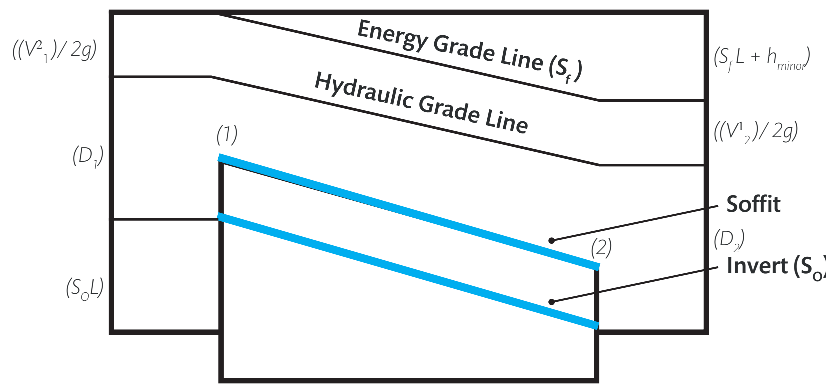
2. Control elevation at or below the soffit elevation. The control is the soffit elevation at the point of discharge. This condition may occur in any one of the four situations described above in 2a.

6-9(C) INSTRUCTIONS FOR HYDRAULIC CALCULATIONS

Most procedures for calculating hydraulic grade line profiles are based on the Bernoulli equation. This equation can be expressed as follows:

Figure 6.10 **Hydraulic Grade**

$$\frac{V_1^2}{2g} + D_1 + S_O L = \frac{V_2^1}{2g} + D_2 + S_f L + h_{minor}$$



where:
D = Vertical distance from invert to H.G.L.
$\mathbf{S_o}$ = Invert slope
L = Horizontal projected length of conduit
$\mathbf{S_g}$ = Average friction slope between Sections 1 and 2
V = Average velocity (g/A)
$\mathbf{h_{minor}}$ = Minor head losses

Minor losses have been included in the Bernoulli equation because of their importance in calculating hydraulic grade line profiles and are assumed to be uniformly distributed in the above figure.

When specific energy (E) is substituted for the quantity (V2/2g + D) in the above equation and the result rearranged,

EQUATION 6.38 $$\mathbf{L} = \frac{E2 - E1}{S_o - S_f}$$

The above is a simplification of a more complex equation and is convenient for locating the approximate point where pressure flow may become unsealed.

6-9(C)(1) HEAD LOSSES

6-9(C)(1)(I) FRICTION LOSS

Friction losses for closed conduits carrying storm water, including pump station discharge lines, will be calculated from the Manning equation or a derivation thereof. The Manning equation is commonly expressed as follows:

EQUATION 6.39 $$\mathbf{Q} = \frac{1.486\ AR^{2/3}\ S_f^{1/2}}{n}$$

where:
Q = Discharge, in c.f.s.
n = Roughness coefficient
A = Area of water normal to flow in ft.2
R = Hydraulic radius
$\mathbf{S_f}$ = Friction slope

when rearranged into a more useful form:

$$\mathbf{S_f} = [Q_n/1.486AR^{2/3}]^2 = [Q/K]^2$$

where:

$$\mathbf{K} = \frac{1.486\ AR^{2/3}}{n}$$

The loss of head due to friction throughout the length of reach (L) is calculated as follows:

$$\mathbf{h_r} = S_rL = [Q/K]^2L$$

The value of K is dependent upon only two factors: the geometrical shape of the flow cross section as expressed by the quantity ($AR^{2/3}$), and the roughness coefficient (n). The values of n are shown in Section 16.

6-9(C)(1)(II) TRANSITION LOSS

Transition losses will be calculated from the equations shown below.

For a Contraction (increasing velocity):

EQUATION 6.40 $$\mathbf{H_f} = K_e/2(V_2-V_1)^2/2g$$

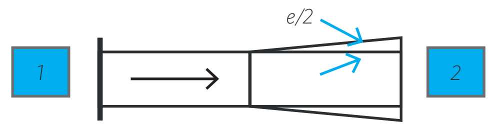
For an Expansion (decreasing velocity):

EQUATION 6.41 $$\mathbf{H_f=K_e(V_2-V_1)^2/2g}$$

where:
$\mathbf{K_e}$=3.50(tan ø/2)$^{1.22}$

These equations are applicable when no change in Q occurs and where the horizontal angle of divergence or convergence (ø /2) between the two sections does not exceed 5 degrees 45 minutes.

Figure 6.11



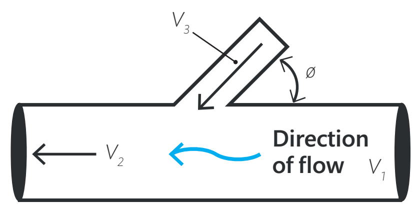
Deviations from the above criteria must be approved by the City Engineer. When such situations occur, the angle of divergence or convergence (ø/2) may be greater than 5 degrees 45 minutes. However, when it is increased beyond 5 degrees 45 minutes, the above equation will give results for hf that are too small, and the use of more accurate methods, such as the Gibson method shown, wherein K_e=3.50(tan ø/2)$^{1.22}$.

6-9(C)(1)(III) JUNCTION LOSSES

In general, junction losses are calculated by equating pressure plus momentum through the confluences under consideration. This can be done by using either the P + M method or the Thompson equation, both of which are shown in the Section . Both methods are applicable in all cases for pressure flow and will give the same results.

For the special case of pressure flow with A_1 = A_2 and friction neglected,

Figure 6.12 **Junction Losses**



6-9(C)(1)(IV) MANHOLE LOSS

Manhole losses will be calculated from the equation shown below. Where a change in pipe size and/or change in Q occurs, the head loss will be calculated in accordance with Sections 6-7 and 6-8, preceding.

EQUATION 6.42 $H_{mh}=0.05(V^2/2g)$

6-9(C)(1)(V) BEND LOSS

Bend losses should be included for all closed conduits, those flowing partially full as well as those flowing full. Bend losses will be calculated from the following equation:

EQUATION 6.43 $H_b=K_b(V^2/2g)$

where:

K_b= 0.20(ø/90°)$^{0.5}$

where:

ø = central angle of bend in degrees

6-9(C)(1)(VI) EXIT LOSS

Exit loss is the loss when storm drains daylight into a pond or channel, the loss associated with this condition is:

EQUATION 6.44 $h_{exit}=0.25(V^2/2g)$

6-9(C)(1)(VII) TRANSITION TO SMALLER PIPE SIZE
As a general rule, storm drains will be designed with sizes increasing in the downstream direction. However, when studies indicate it may be advisable to decrease the size of a downstream section, the conduit may be decreased in size with the approval from the City Engineer.

6-9(D) DESIGN REQUIREMENTS FOR MAINTENANCE AND ACCESS

6-9(D)(1) MANHOLES

6-9(D)(1)(I) SPACING
Manholes should be spaced at intervals of approximately 450 feet. Where the proposed conduit is less than 30 inches in diameter and the horizontal alignment has numerous bends or angle points, the manhole spacing should be reduced to approximately 300 feet.

The spacing requirements shown above apply regardless of design velocities. Deviations from the above criteria are subject to City Engineer approval.

6-9(D)(1)(II) LOCATION
Manholes should not be located in street intersections, especially when one or more streets are heavily traveled. In situations where the proposed conduit is to be aligned both in easement and in street right-of-way, manholes should be located in street right-of-way, wherever possible.

Manholes should be located as close to changes in grade as feasible when the following conditions exist:

1. When the upstream conduit has a steeper slope than the downstream conduit and the change in grade is greater than 10 percent, sediment tends to deposit at the point where the change in grade occurs.
2. When transitioning to a smaller downstream conduit due to an abruptly steeper slope downstream, sediment tends to accumulate at the point of transition.
3. When the design flow in a pipe flowing full has a velocity of 20 f.p.s. or greater, or is supercritical in a partially full pipe, the total horizontal angle of divergence or convergence between the walls of the manhole and its center line should not exceed 5°45'.

6-9(D)(2) PRESSURE MANHOLES
A pressure manhole shaft and a pressure frame and cover will be installed in a pipe or box storm drain whenever the design water surface is more than 0.2 feet above the ground surface. Pressure manholes should only be used when a non-pressure manhole solution is unavoidable.

6-9(D)(3) SPECIAL MANHOLES
Special 36-inch diameter manholes or vehicular access structures will be provided when required. The need for access structures will be determined by the City Engineer during the review of preliminary plans.

6-9(D)(4) DEEP MANHOLES

A manhole shaft safety ledge will be provided in all instances when the manhole shaft is 20 feet or greater in depth. Installation will be in accordance with City Engineer requirements.

6-9(E) CLOSED CONDUIT PIPE SIZE AND SLOPE

6-9(E)(1) MINIMUM PIPE SIZE

In cases where the conduit may carry significant amounts of sediment (greater than 8%), the minimum diameter of main line conduit will be 36 inches. In situations where sediment may be expected, the City Engineer will be consulted to determine the applicability of sediment criteria.

6-9(E)(2) MINIMUM SLOPE

Unless otherwise approved by the City Engineer, the minimum slope for main line conduit will be .004 (.40 percent). Minimum flow velocity for the 10-year design flow will be 3 f.p.s.

6-9(F) EARTHEN CHANNELS TO STORM DRAIN STRUCTURES

An inlet structure will be provided for storm drains located in natural channels. The structure should generally consist of a headwall, wingwalls to protect the adjacent banks from erosion, and a paved inlet apron or rip-rap. The apron slope should be limited to a maximum of 2:1. Wall heights should conform to the height of the water upstream of the inlet, and be adequate to protect both the fill over the drain and the embankments. Headwall and wingwall fencing and a protection barrier to prevent public entry will be provided.

If trash and debris are prevalent, barriers consisting of vertical 3-inch or 4-inch diameter steel pipe at 24 inches to 36 inches on centers should be embedded in concrete immediately upstream of the inlet apron. Trash rack designs must have City Engineer approval.

6-9(G) STORM DRAIN OUTLETS TO PUBLIC EARTHEN ARROYOS AND PONDS

When a storm drain outlets into an earthen arroyo, an outlet structure will be provided which prevents erosion and property damage. Fencing and a protection barrier will be provided where deemed necessary by the City Engineer.

The outlet structure shall have an end treatment and design that minimizes erosion. The following design criteria was adopted from "*Urban Storm Drainage Criteria Manual Volume 2*" from the Urban Drainage and Flood Control District, Denver, Colorado, June 2001, revised April 2008.

6-9(G)(1) INCORPORATION OF "URBAN STORM DRAINAGE CRITERIA MANUAL VOLUME 2" FROM THE URBAN DRAINAGE AND FLOOD CONTROL DISTRICT, DENVER, COLORADO

Energy dissipation or stilling basin structures are required to minimize scour damage caused by high exit velocities and turbulence at conduit outlets. Similarly, culverts nearly always require special consideration at their outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Rock protection at conduit outlets is appropriate where moderate outlet conditions exist; however, there are many situations where rock basins are impractical. Reinforced concrete outlet structures are suitable for a wide variety of site conditions. In some cases, they are more economical than larger rock basins, particularly when long-term costs are considered.

Any outlet structure must be designed to match the receiving stream conditions. The following steps include an analysis of the probable range of tailwater and bed conditions that can be anticipated including degradation, aggradation, and local scour.

Hydraulic concepts and design criteria are provided in this section for an impact stilling basin and adaptation of a baffle chute to conduit outlets. Use of concrete is often more economical due to structure size or local availability of materials. Initial design selection should include consideration of a conduit outlet structure if any of the following situations exist:

1. high-energy dissipation efficiency is required where hydraulic conditions approach or exceed the limits for alternate designs;
2. low tailwater control is anticipated; or
3. site conditions, such as public use areas, where plunge pools and standing water are unacceptable because of safety and appearance, or at locations where space limitations direct the use of a concrete structure.

Longer conduits with large cross-sectional areas are designed for significant discharges and often with high velocities requiring special hydraulic design at their outlets. Here, dam outlet and spillway terminal structure technology is appropriate (USBR 1987). Type II, III or IV stilling basins, submerged bucket with plunge basin energy dissipators and slotted-grating dissipators can be considered when appropriate to the site conditions. For instance, a plunge basin may have applicability where discharge is to a wet detention or retention pond.

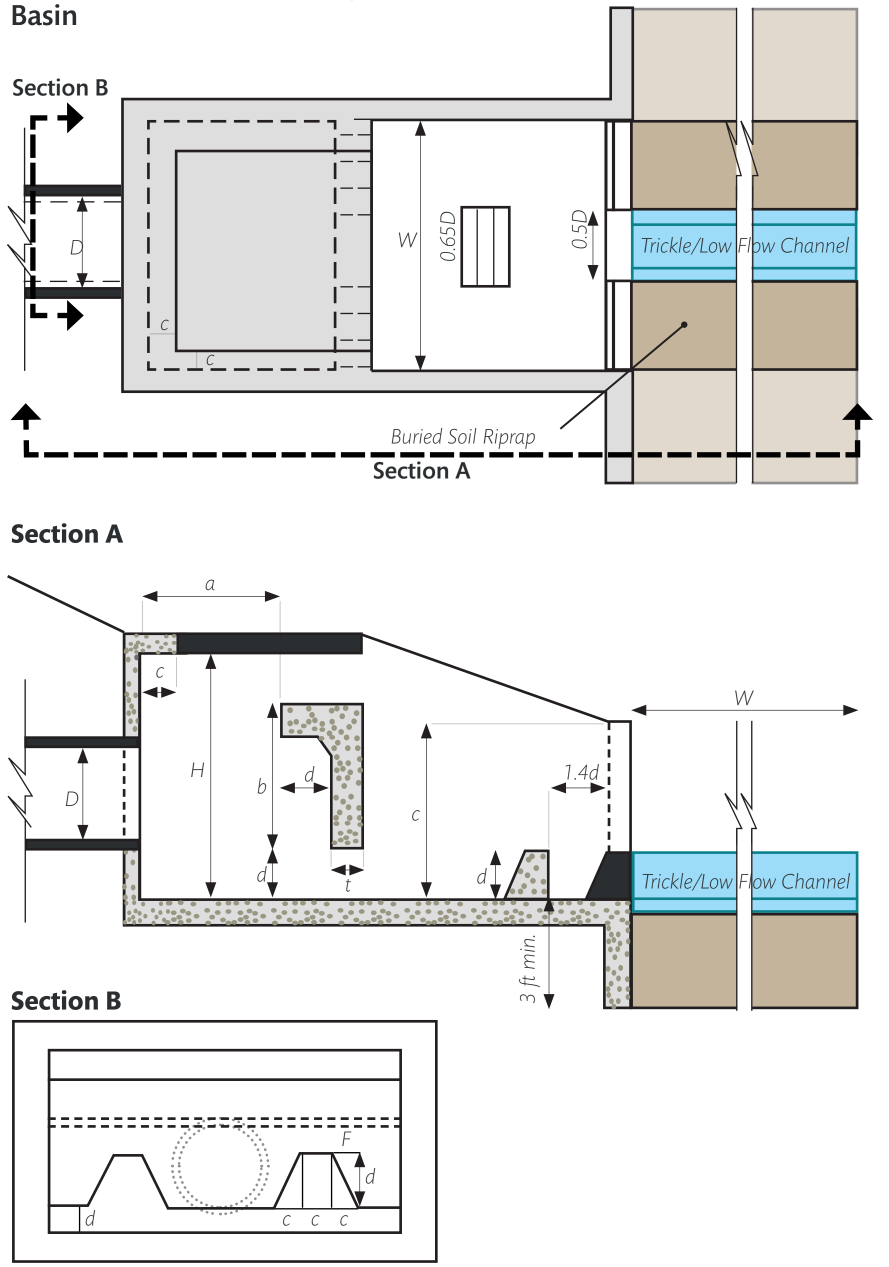
6-9(G)(1)(I) IMPACT STILLING BASINS

Most design standards for an impact stilling basin are based on the USBR Type VI basin, often called "impact dissipator" or conduit "outlet stilling basin". This basin is a relatively small structure that is very efficient in dissipating energy without the need of tailwater. The original hydraulic design reference by Biechly (1971) is based on model studies. Additional structural design details are provided by Aisenbrey, et al. (1974) and Peterka (1984). The type VI basin was originally designed to operate continuously at the design flow rate. However, it is applicable for use under the varied flow conditions of stormwater runoff. The use of this outlet basin is limited only by structural and economic considerations.

Energy dissipation is accomplished through the turbulence created by the loss of momentum as flow entering the basin impacts a large overhanging baffle. Art high flow, further dissipation is produced as water builds up behind the baffle to form a highly turbulent backwater zone. Flow is then redirected under the baffle to the open basin and out to the receiving channel. A check at the basin end reduces exit velocities by breaking up the flow across the basin floor and improves the stilling action at low to moderate flow rates.

The generalized, slightly modified, USBR Type IV Impact Basin design configuration is shown in Figure 6.13 (Figure HS-14 in USDCM), which consist of an open concrete box attached directly to the conduit outlet.

Figure 6.13 General Design Dimensions for USBR Type VI Impact Sill Basin



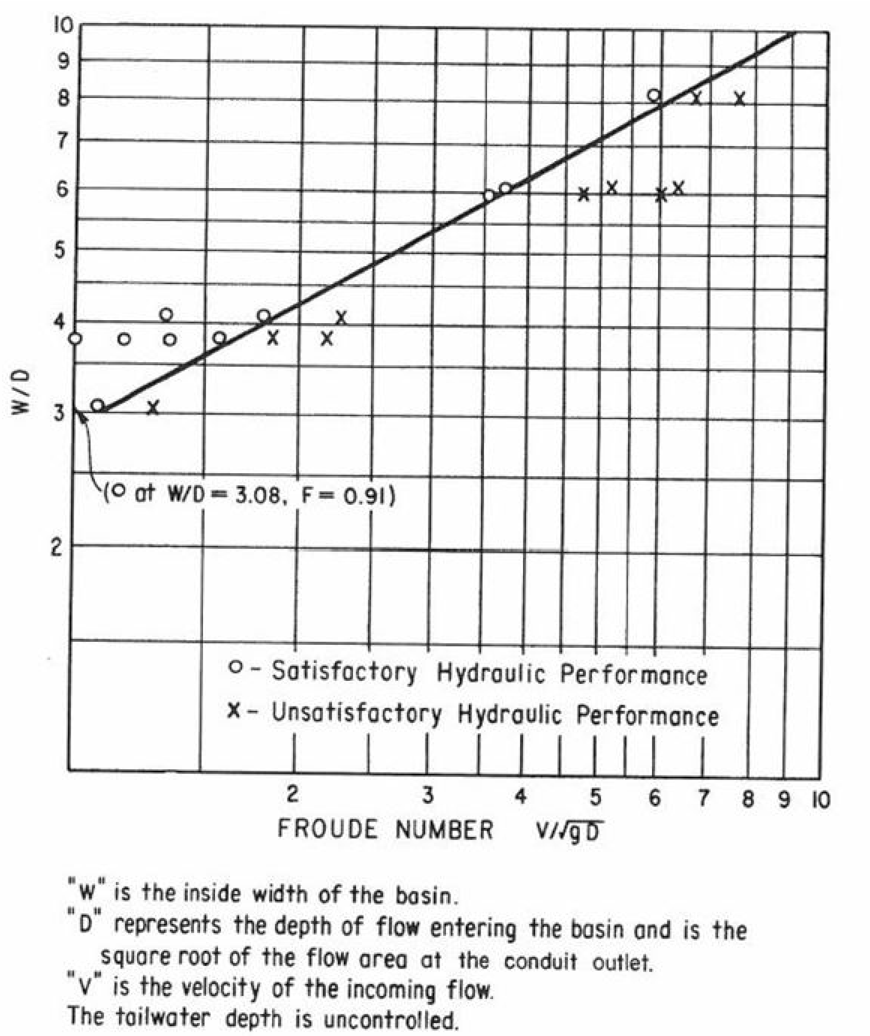
Equations:

$W = 2.94\,[V/gD]^{0.556}$
V = Velocity, ft/s
$D = \sqrt{A}$
A = Area flow in sq. ft
$H = 3/4W$
$L = 4/3W$
$a = 1/2W$
$b = 3/8W$
$c = 1/2W$
$t = 1/12W$
$d = 1/6W$
$f = 1/8W$

The width, W, is a function of the Froude number and can be determined using Figure 6.14 (Figure HS-15 in USDCM). The sidewalls are high enough to contain most of the splashing during high flows and slope down to form a transition to the receiving channel. The inlet pipe is vertically aligned with an overhanging L-shaped baffle such that the pipe invert is not lower than the bottom of the baffle. The end check height is equal to the height under the baffle to produce tailwater in the basin. The alternate end transition (at 45 degrees) is recommended for grass-lined channels to reduce the downstream scour potential.

Figure 6.14 Basin Width Diagram for USBR Type VI Impact Sill Basin

NOTE: *Diagram provided by the Urban Drainage and Flood Control District, HS-15.*

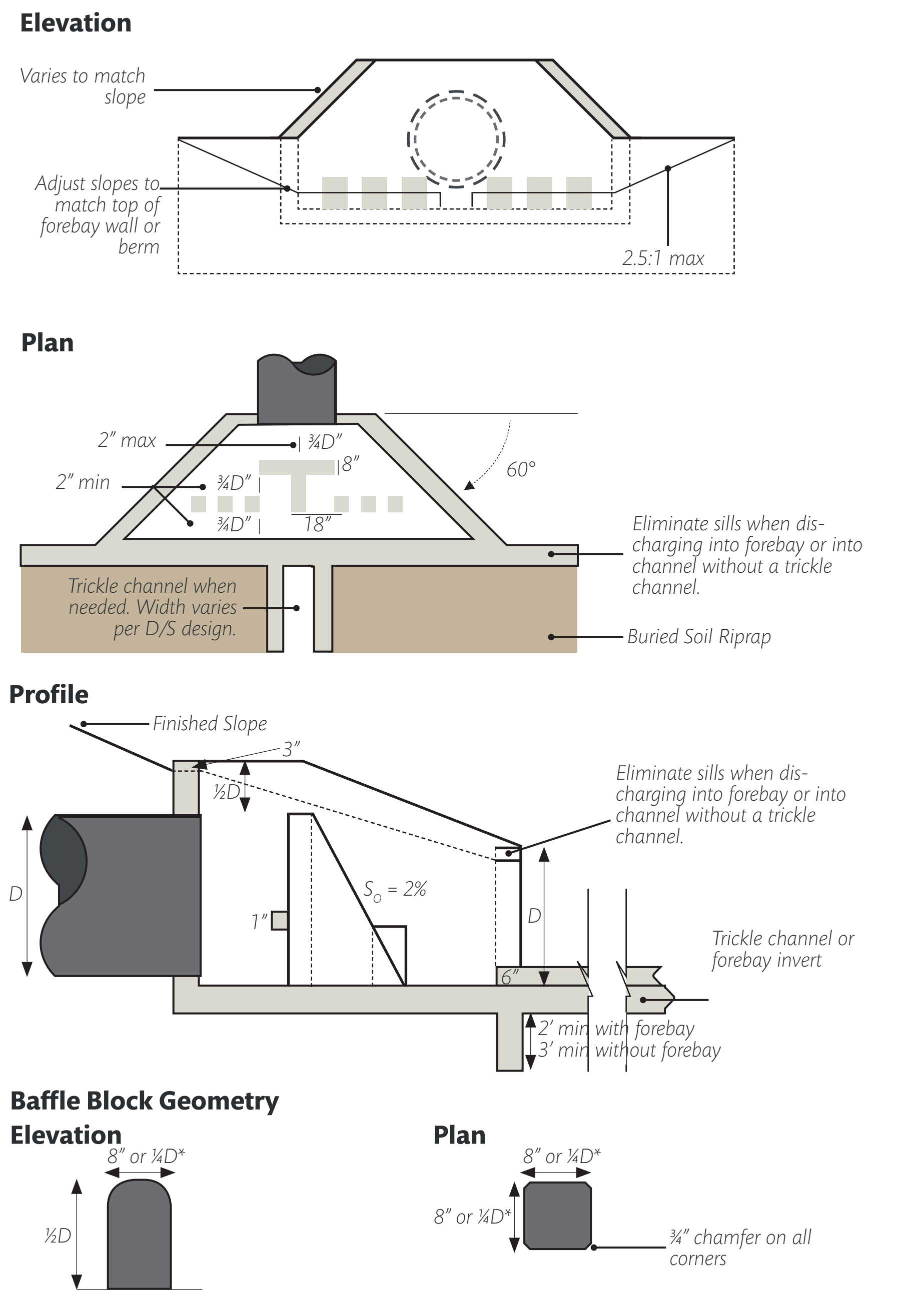


The impact basin can also be adapted to multiple pipe installations. Such modifications are discussed later in 6-9(G)(1)(i)(b), but it should be noted that modifications to the design may affect the hydraulic performance of the structure. Model testing of designs that vary significantly from the standard is recommended.

6-9(G)(1)(I)(A)MODIFIED IMPACT BASINS FOR SMALLER OUT-LETS

For smaller pipe outlets a modified version of the USBR Type IV Impact Basin is suggested in this Manual. Figure 6.15 (Figure HS-16a in USDCM) provides a design layout for circular outlets ranging in size from 18-inches to 48-inches in diameter and Figure 6.16 (Figure HS-16b in USDCM) for pipes 18-inches in diameter and smaller. The latter was added for primary use as an outlet energy dissipator upstream of forebays of small extended detention basins, sand filters and other structural best management practices requiring energy dissipation at the end of the pipe delivering water to the BMP facility.

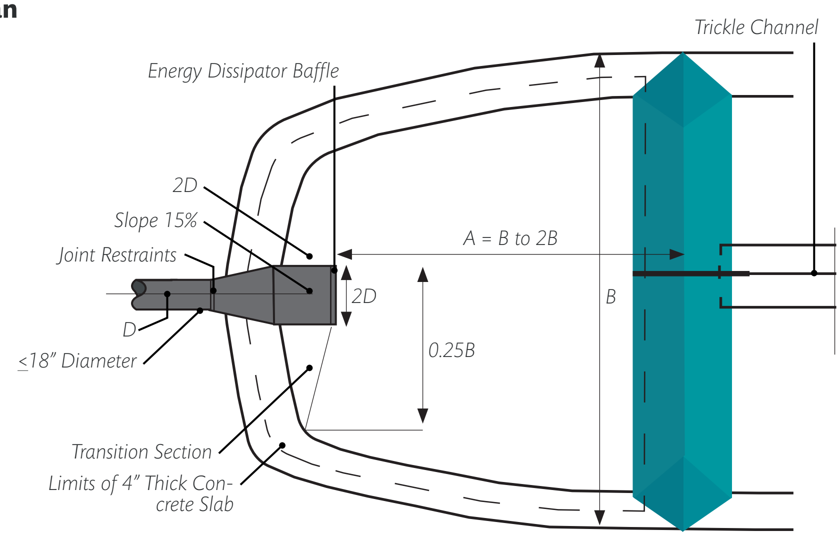
Figure 6.15 Modified Impact Sill Basin for Conduits 18" to 48" in Diameter



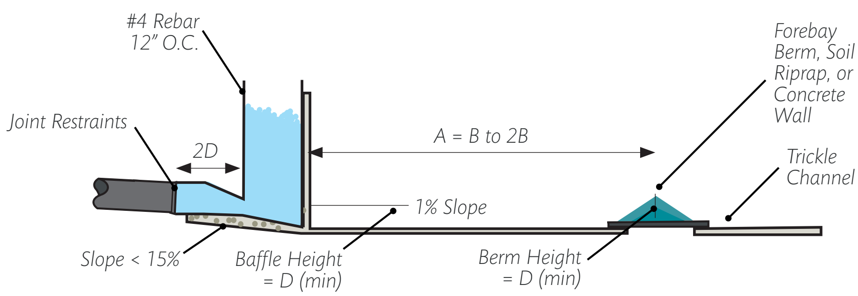
NOTE:
1. Design of reinforcing steel is he responsibility of the design engineer. Reinforce to withstand water earth pressures.
2. When discharging into channel and not forebay install Type M buried soil riprap for a distance of 3'D downstream of structure.

Figure 6.16 Impact Stiling Basin for Pipes Smaller than 18" in Diameter Upstream of Forebays

Plan



Profile



Unlike the Type IV Impact Basin, the modified basins do not require sizing for flow under normal stormwater discharge velocities recommended for storm sewers in this manual. However, their use is limited to exit velocities of 18 feet per second or less. For larger conduits and higher exit velocities, it is recommended that the standard Type IV Impact Basin be used instead.

6-9(G)(1)(I)(B)MULTIPLE CONDUIT INSTALLATIONS

Where two or more conduits of different sizes outlet in proximity, a composite structure can be constructed to eliminate common walls. This can be somewhat awkward since each basin "cell" must be designed as an individual basin with different height, width, etc. Where possible, a more economical approach is to combine storm sewers underground, at a manhole or vault, and bring a single combined pipe to the outlet structure.

When using a Type IV impact basin shown in Figure 6.13 (Figure HS-14 in USDCM) for two side-by-side pipes of the same size, the two pipes may discharge into a single basin. If the basin's design width of each pipe is W, the combined basin width for two pipes would be 1.5W. When the flow is different for the two conduits, the design width W is based on the pipe carrying the higher flow. For the modified impact basin shown in Figure 6.15 (Figure HS-16a in USDCM), add 1/2D space between the pipes and to each outside pipe edge when two pipes discharge into the basin to determine the width of the headwall and extent the width of the impact wall to match the outside edges of the two pipes. The effect of mixing and turbulence of the combined flows in the basin has not been model tested to date.

Remaining structure dimensions are based on the design width of a separate basin W. If the two pipes have different flow, the combined structure is based on the higher Froude number when designing the Type IV basins. Use of a handrail is suggested around the open basin areas where safety is a concern. Access control screens or grating where necessary are a separate design consideration. A hinged rack is also an alternative.

6-9(G)(1)(I)(C)GENERAL DESIGN PROCEDURE FOR TYPE IV IMPACT BASIN

1. Determine the design hydraulic cross-sectional area just inside the pipe, at the outlet. Determine the effective flow velocity, V, at the same location in the pipe. Assume $D=(A_{sect})^{0.5}$ and compute the Froude number=$V/(gD)^{0.5}$.
2. The entrance pipe should be turned horizontally at least one pipe diameter equivalent length upstream from the outlet. For pipe slopes greater than 15 degrees, the horizontal length should be a minimum of two pipe diameters.
3. Determine the basin width, W, by entering the Froude number and effective flow depth into *Figure 6.14* (Figure HS-15 in USDCM). The remaining dimensions are proportional to the basin width according to *Figure 6.13* (Figure HS-14 in USDCM). The basin width should not be oversized since the basin is inherently oversized for less than design flows. Larger basins become less effective as the inflow can pass under the baffle.
4. Structure wall thickness, steel reinforcement, and anchor walls (underneath the flow) should be designed using accepted structural engineering methods. Note that the baffle thickness, tb, is a suggested minimum. It is not a hydraulic parameter and is not a substitute for structural analysis. Hydraulic forces on the overhanging baffle may be approximated by determination of the hydraulic jet force at the outlet:

   EQUATION 6.45 $\mathbf{F_j} = 1.94 V_{out} Q_{des}$ (force in pounds)

   where:
   $\mathbf{Q_{des}}$ = maximum design discharge (cfs)
   $\mathbf{V_{out}}$ = velocity of the outlet jet (ft/sec)

5. Type "M" rock riprap should be provided in the receiving channel from the end check to a minimum distance equal to the basin width. The depth of rock should be equal to the check height or at least 2.0 feet. Rock may be buried to finished grades and planted as desired.
6. The alternate end check and wingwall shown in *Figure 6.13* (Figure HS-14 in USDCM) are recommended for all grass-lined/earthern channel applications to reduce the scour potential below the check wall.
7. Ideally, the low-flow invert matches the floor invert at the basin end and the main channel elevation is equal to the top of the check. For large basins where the check height, d, becomes greater than the low-flow depth, dimension d in *Figure 6.13* (Figure HS-14 in USDCM) may be reduced by no more than one-third. It should not be reduced to less

than 2 feet. This implies that a deeper low-flow channel (1.5 to 2.0 feet) will be advantageous for these installations. The alternate when the check height, d, exceeds the trickle flor depth is that the basin area will not drain completely.

8. A check section should be constructed directly in front of the low-flow notech to break up bottom flow velocities. The length of this check section should overlap the with of the low flow and its dimension is shown in Figure 6.13 (Figure HS-14 in USDCM).

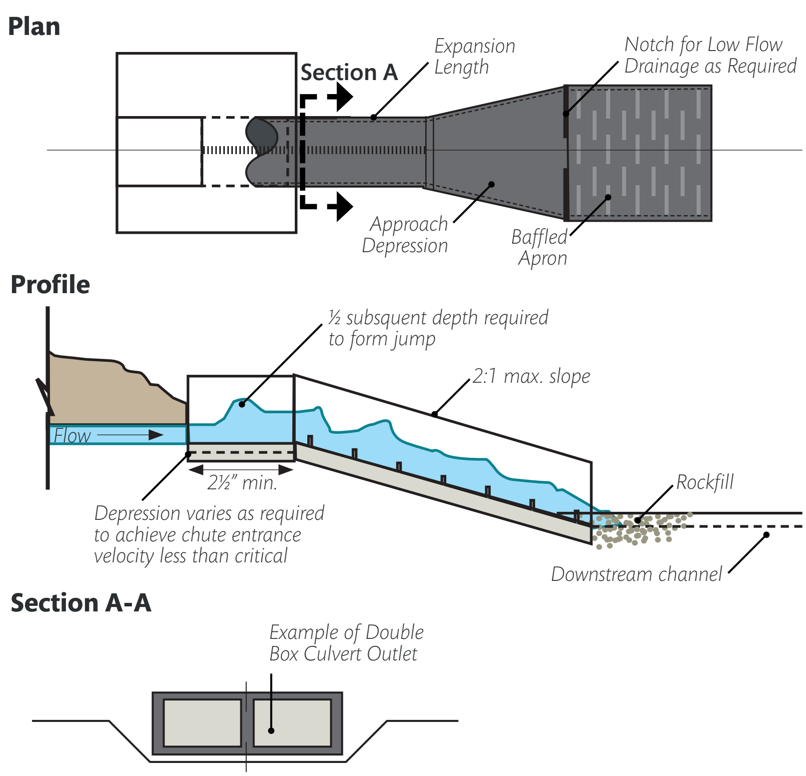
6-9(G)(1)(II) PIPE OUTLET RUNDOWNS

6-9(G)(1)(II)(A)BAFFLE CHUTE RUNDOWN

The baffle chute developed by the USBR (1958) has also been adapted to use at pipe outlets. This structure is well suited to situations with large conduit outfalls and at outfalls to channels in which some future degradation is anticipated. As mentioned previously, tha apron can be extended at a later time to account for channel degradation. This type of structure is only cost effective if a grade drop is necessary below the outfall elevation.

Figure 6.17 (Figure HS-17 in USDCM) illustrates a general configuration for a baffled outlet application for a double box culvert outlet. In this case, an expansion zone occurs just upstream of the approach depression. The depression depth is designed as required to reduce the flow velocity at the chute entrance. The remaining hydraulic design is the same as for a standard baffle chute using conditions at the crest to establish the design. The same crest modifications are applicable to allow drainage of the approach depression, to reduce the upstream backwater effects of the baffles, and to reduce the problems of debris accumulation and standing water at the upstream row of baffles.

Figure 6.17 **Baffle Chute Pipe Outlet**



NOTE: Section A *Possible expansion as required for pipe hydralics and for unit discharge.*

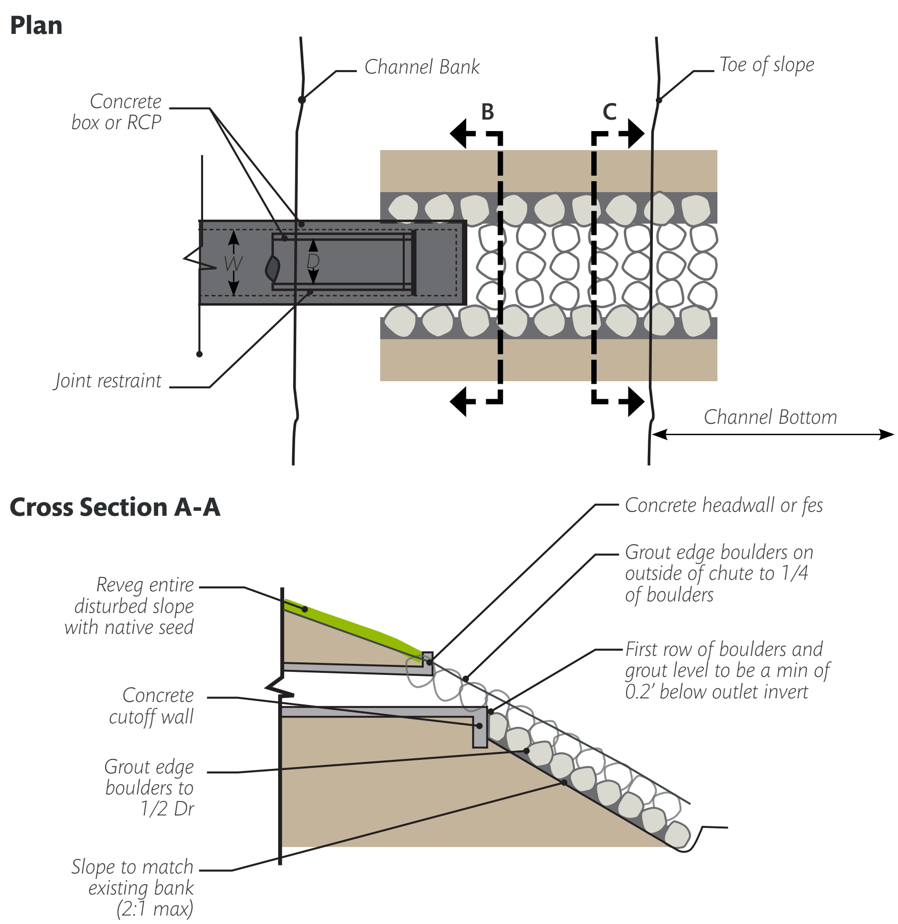
Flow entering the chute should be well distributed laterally across the width of the chute. The velocity should be below critical velocity at the crest of the chute. To insure low velocities at the upstream end, it may be necessary to provide a short energy dissipating pool. The sequent or conjugate depth in the approach basin should be sized to prevent jump sweep-out, but the basin length may be considerably less than a conventional hydraulic jump basin since its primary purpose is only to reduce the average entrance velocity. A basin length of twice the sequent depth will usually provide ample basin length. The end check of the pool may be used as the crest of the chute as shown in Figure 6.17 (Figure HS-17 in USDCM).

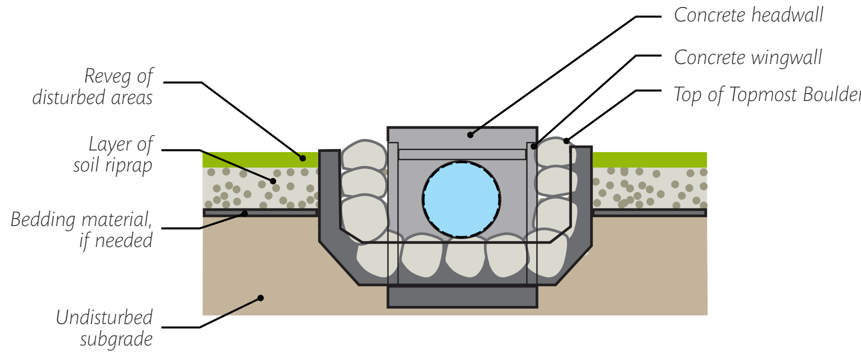
6-9(G)(1)(II)(B)GROUTED BOULDER CHUTE RUNDOWN

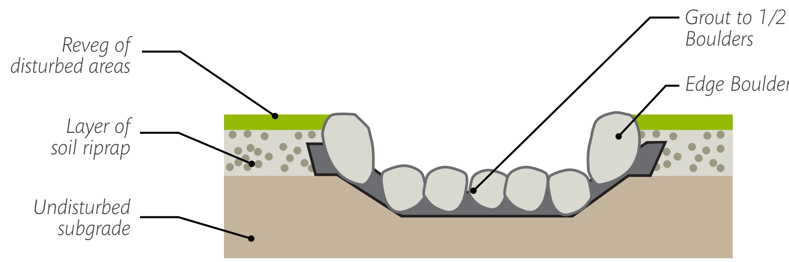
Another option for rundowns at outlets of larger pipes is to use a grouted boulder rundown illustrated in Figure 6.18 (Figure HS-18 in USDCM). This type of rundown has been used successfully for several large storm sewers entering the South Platte River. It is critical that the details shown in Figure 6.18 (Figure HS-18 in USDCM) be strictly followed and the grout and the actual filling of spaces between the boulders with grout closely adhere to the recommendations for grouted boulders.

If the exit velocities of the pipe exceeds 12 feet per second, an approach cute for the baffle chute rundown described above should be considered and provided.

Figure 6.18 **Grouted Boulder Rundown**







6-9(G)(1)(II)(C)LOW TAILWATER RIPRAP BASINS AT PIPE OUTLETS

The design of low tailwater riprap basins for storm sewer pipe outlets and at some culvert outlets is necessary when the receiving or downstream channel may have little or no flow or tailwater at time when the pipe or culvert is in operation. Design criteria are provided in Figure 6.21 (Figure HS-19a in USDCM) through Figure 6.24 (Figure HS-20c in USDCM).

By providing a low tailwater basin at the end of a storm sewer conduit or culvert, the kinetic energy of the discharge is dissipated under controlled conditions without casuing scour at the channel bottom. Figure 6.19 (Photograph HS-12 in USDCM) shows a fairly large low tailwater basin.

Figure 6.19 Upstream and Downstream Views of a Low tailwater Basin in Douglas County

NOTE: *Photographs provided by the Urban Drainage and Flood Control District.*

Low tailwater basin design is described below. Low tailwater is defined as being equal to or less than 1/3 of the height of the storm sewer, that is:

EQUATION 6.46 $\mathbf{Y_t} \leq D/3$ or $\mathbf{Y_t} \leq H/3$

where:
$\mathbf{Y_t}$ = tailwater depth at design
D = diameter of circular pipe (ft)
H = height of rectangular pipe (ft)

1. Finding Flow Depth and Velocity of Storm Sewer Outlet Pipe
   a. *The first step in the design of a scour protection basin at the outlet of a storm sewer is to find the depth and velocity of flow at the outlet. Pipe-full flow can be found using Manning's equation. See* Section 6-12(D).
   b. *Then the pipe-full velocity can be found using the continuity equation.*

   EQUATION 6.47 $\mathbf{V_{full}}=Q_{full}/A_{full}$

   c. *The normal depth of flow, d, and the velocity in a conduit can be found with the aid of* Figure 6.22 (Figure HS-20a in USDCM) *and* Figure 6.23 (Figure HS-20b in USDCM). *Using the known design discharge, Q, and the calculated pipe-full discharge, Q_{full} enter* Figure 6.22 (Figure HS-20a in USDCM) *with the value of Q/Qf_{ull} and find d/D for a circular pipe of d/H for a rectangular pipe.*
   d. *Compare the value of d/D (or d/H) with the one obtained from* Figure 6.23 (Figure HS-20b in USDCM) *using the Froude parameter, $Q/D^{2.5}$ or $Q/(wH^{1/5})$*
   e. *Choose the smaller of the two (d/D or d/H) ratios to calculate the flow depth at the end of pipe.*

   EQUATION 6.48 **D**=D(d/D) or **d**=H(d/H)

   f. *Again, enter* Figure 6.21 (Figure HS-19a in USDCM) *using the smaller d/D (or d/H) ratio to find the A/A_{full} ratio. Then,*

   EQUATION 6.49 $\mathbf{A}=(A/A_{full})A_{full}$

   g. *Finally,* [5]

   EQUATION 6.50 **V**=Q/A

   where:
   $\mathbf{A_{full}}$= cross sectional area of the pipe (ft^2)
   A= area of the design flow in the end of the pipe (ft^2)

5 *See Section 22.16 for definitions of the Manning's equation.*

2. Riprap Size
   a. *For the design velocity, use* Figure 6.24 (Figure HS-20c in USDCM) *to find the size and type of the riprap to use in the scour protection basin downstream of the pipe outlet (e.g. B18, H, M, or L). First calculate the riprap sizing design parameter, P_d, namely,*

EQUATION 6.51 $\mathbf{P_d} = (V^2 + gd)^{0.5}$

where:
V = design flow velocity at pipe outlet (ft/sec)
g - acceleration due to gravity = 32.2 ft/sec^2
d = design depth of flow at pipe outlet (ft)

i When the riprap sizing design parameter indicates conditions that place the design above the Type H riprap line in Figure 6.22 (Figure HS-20 in USDCM), use B18, or larger, grouted boulders. An alternate to a grouted boulder or lose riprap basin is to use the standard USBR Impact Basin VI or one of its modified versions, described earlier in this Section.

   b. *After the riprap size has been selected, the minimum thickness of the riprap layer, T, in feet, in the basin is set at:*

EQUATION 6.52 $\mathbf{T} = 1.75D_{50}$

where:
$\mathbf{D_{50}}$ = the median size of the riprap

Riprap Type	D50- Median Rock Size (inches)
L	9
M	12
H	18
B18	18 (minimum dimension of grouted boulders)

3. Basin Length
   a. *The minimum length of the basin, L, in* Figure 6.20 (Figure HS-20 in USDCM), *is defined as being the greater of the following:*

EQUATION 6.53 **For circular pipe:** $\mathbf{L} = 4D$ or $\mathbf{L} = (D)^{0.5}(V/2)$
EQUATION 6.54 **For rectangular pipe:** $\mathbf{L} = 4H$ or $\mathbf{L} = (H)^{0.5}(V/2)$

where:
L = basin length
H = height of rectangular conduit
V = design flow velocity at outlet
D = diameter of circular conduit

4. Basin Width
   a. *The minimum width, W, of the basin downstream of the pipes flared end section is set as follows:*

EQUATION 6.55 **For circular pipe: W** = 4D
EQUATION 6.56 **For rectangular pipe: W** = w+4H

where:
W = basin width
D = diameter of circular conduit
W = width of rectangular conduit

5. Other Design Requirements
   a. *All slopes in the pre-shaped riprapped basin are 2H to 1V.*
   b. *Provide pipe joint fasteners and a structural concrete cutoff wall at the end of the flared end section for a circular pipe or a headwall with wingwalls and a paved bottom between the walls, both with a cutoff wall that extends down to a depth of:*

EQUATION 6.57 **B** = D/2 + T or **B** = H/2 + T
when:
B = cutoff wall depth
D = diameter of circular conduit
T = $1.75D_{50}$

The riprap must be extended up the outlet embankment's slope to the mid-pipe level.

Figure 6.20 **Low Trailwater Riprap Basins for Storm Sewer Piper Outlets (HS-19)**

NOTE: *For rectangular conduits use a standard design for a headwall with wingwalls, paved bottom between the wingwalls, with an end cutoff wall extending to a minimum depth equal to B.*

Plan

Profile

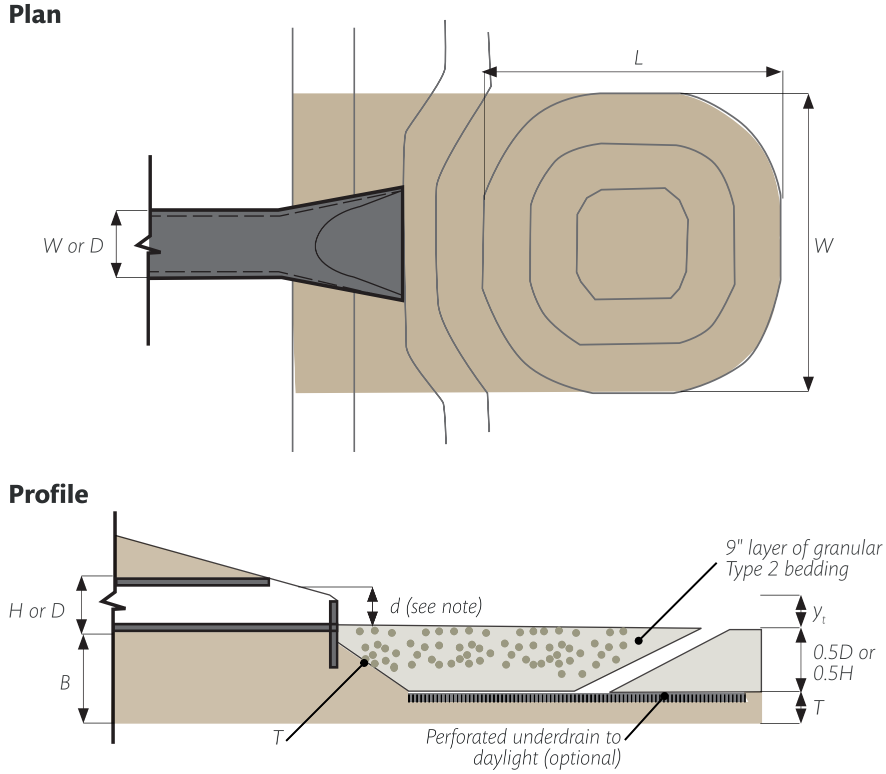


Figure 6.21 Concrete Flared End Section with Cutoff Wall for all Pipe Outlets

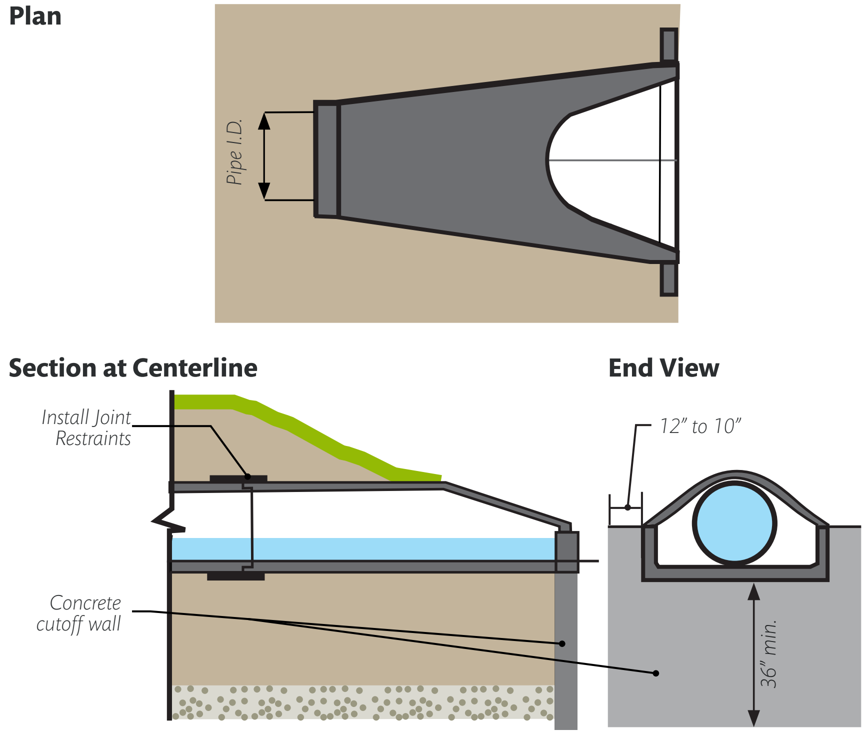


Figure 6.22 Low Trailwater Riprap Basins for Storm Sewer Pipe Outlets - Discharge and Flow Area Relationships for Circular and Rectangular Pipes

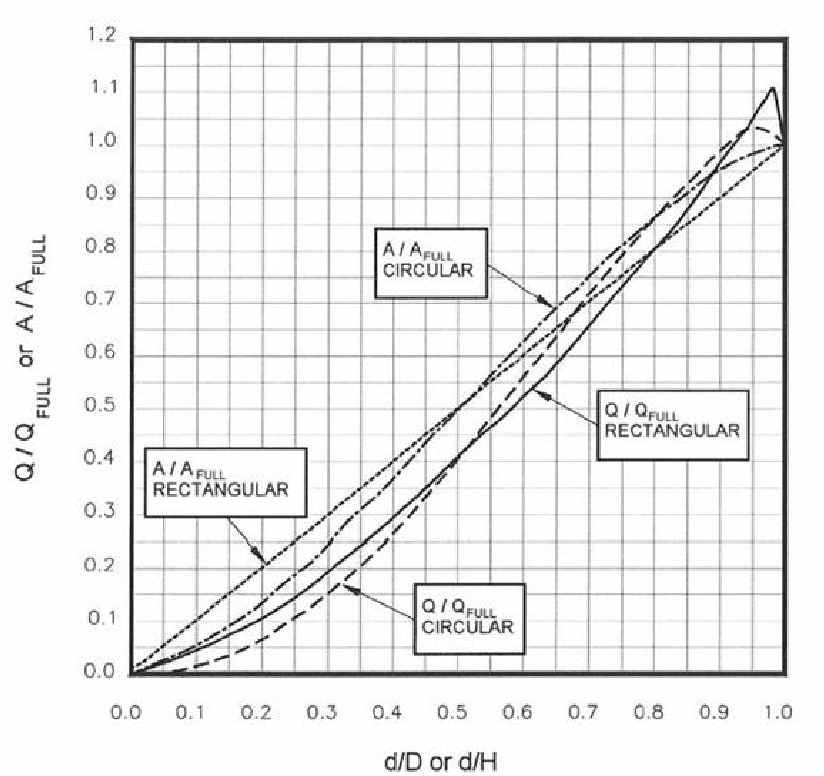


Figure 6.23 **Low Trailwater Riprap Basins for Storm Sewer Pipe Outlets - Brink Depth Horizontal Pipe Outlet**

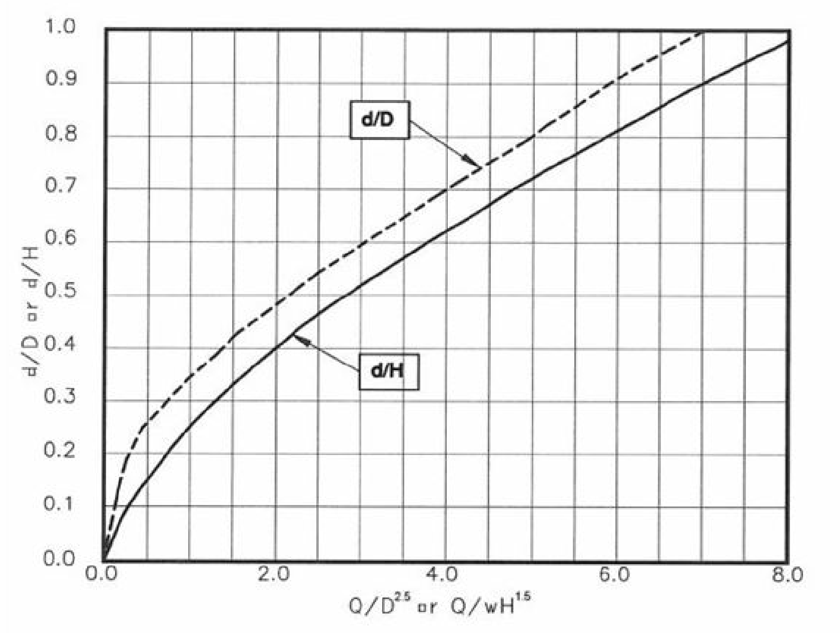
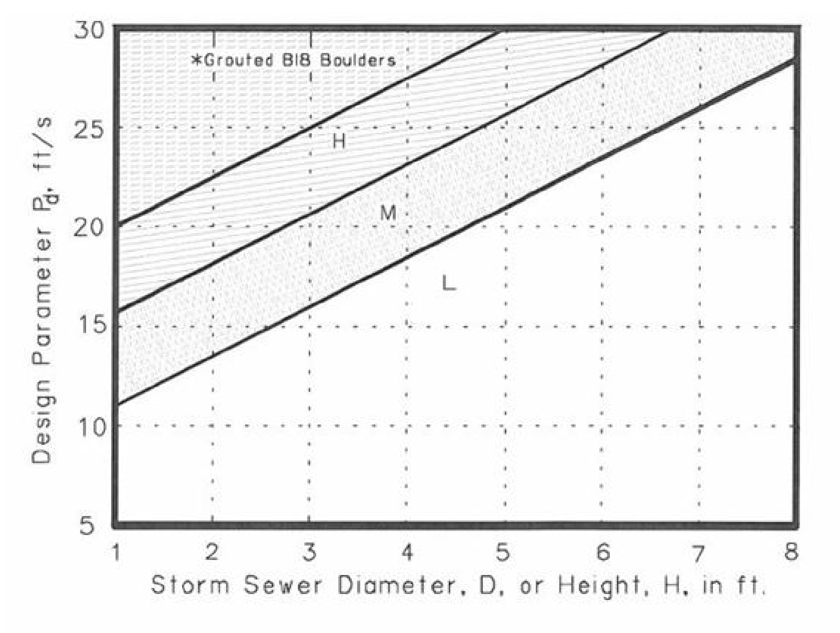


Figure 6.24 **Low Trailwater Riprap Basins for Storm Sewer Pipe Outlets - Riprap Selection Chart for Low Tailwater Basin at Pipe Outlet**



6-9(G)(1)(III) CULVERT OUTLETS

Culvert outlets represent a persistent problem because of concentrated discharges and turbulence that are not fully controlled prior to the flow reaching the standard downstream channel configuration. Too often the designer's efforts are focused on the culvert inlet and its sizing with outlet hydraulics receiving only passing attention. Culvert design is not complete until adequate attention is paid to the outlet hydraulics and proper stilling of the discharge flows.

Culvert outlet energy dissipater and flow spreading may require special structures downstream of the culvert outlet to limit local scour, general stream degradation, and troublesome head cutting. Some of the techniques described earlier in this section may be applied at culvert outlets, as well if the downstream channel and/or tailwater conditions so indicate.

Local scour is typified by a scour hole at the pipe's outlet. High exit velocities cause this, and the effects extend only a limited distance downstream. Coarse material scoured from the hole is deposited immediately downstream, often forming a low bar. Finer material is transported further downstream. The dimensions of the scour hole change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the flow when there is minimal tailwater depth at the outlet and not necessarily when the flow is highest. Methods for predicting scour hole dimensions are found in *HEC No. 14* (Corry, et al. 1975) and need to be applied using a range of possible tailwater depth conditions during different design storms or flows.
General storm degradation, or head cutting, is a phenomenon independent of culvert performance. Natural causes produce a lowering of the streambed over time. The identification of a degrading stream is an essential part of the original site investigation. However, high-energy discharges from a culvert can often cause stream degradation for a limited distance downstream. Both scour and stream degradation can occur simultaneously at a culvert outlet.

Various measures described in HEC No. 14 and listed below need to be considered to protect the downstream channel or stream and control culvert outlet flow. It is beyond the scope of the manual to provide detailed information about all available controls in HEC No. 14, but the City encourages the proper application and design as appropriate for the specific site.

Figure 6.25 **Unprotected Culvert Outlets Cause Downstream Erosion**

NOTE: *Photographs provided by the Urban Drainage and Flood Control District.*

6-9(G)(2) PUBLIC FACILITIES ADDITIONAL EROSION PROTECTION CRITERIA

The facility is considered Public if it maintained publicly or has a Public Drainage Easement upon it.

1. A filter fabric or gravel is to be used in all cases under the riprap.
2. The velocity leaving the energy dissipator/ erosion protection shall be 5 ft/sec or less unless justified.

6-9(G)(3) PRIVATE STORM DRAIN OUTLETS TO ONSITE BASINS OR SWALES

1. The following criteria is acceptable for privately maintained facilities where the storm drain is less than 18 inches in diameter. For private storm drains 18 inches and greater, refer to the sections for public facilities.
2. Criteria:
    a. *Provide erosion control for velocities 5 ft/sec or greater.*
    b. *The pipe invert should be at or close to the invert of the receiving basin or swale.*

6-9(H) PROTECTION AND DEBRIS BARRIERS

6-9(H)(1) PROTECTION BARRIERS

A protection barrier is a means of preventing people from entering storm drains. Protection barriers will be provided wherever necessary to prevent unauthorized access to storm drains. In some cases the barrier may be one of the breakaway type. In other cases the barrier may be a special design. It will be the designer's responsibility to provide a protection barrier appropriate to each situation and to provide details of such on the construction drawings.

6-9(H)(2) DEBRIS BARRIERS

A debris barrier or deflector is a means of preventing large debris or trash, such as tree limbs, logs, boulders, weeds, and refuse, from entering a storm drain and possibly plugging the conduit. The debris barrier should have openings wide enough to allow as much small debris as possible to pass through and yet narrow enough to protect the smallest conduit in the system downstream of the barrier.

One type that has been used effectively in the past is the debris rack. This type of debris barrier is usually formed by a line of posts, such as steel pipe filled with concrete or steel rails, across the line of flow to the inlet. It will be the designer's responsibility to provide a debris barrier or deflector appropriate to the situation and acceptable to the City Engineer.

6-9(H)(3) DEBRIS BASINS

Debris basins, check dams and similar structures are a means of preventing mud, boulders and debris held in suspension and carried along by storm runoff from depositing in storm drains. Debris basins constructed upstream of storm drain conduits, usually in arroyos, trap such material before it reaches the conduit. Debris basins must be cleaned out on a regular basis, however, if they are to continue to function effectively. Refer to the City Engineer and State Engineer regarding the criteria to be used in designing these structures.

6-9(I) CLOSED CONDUIT ANGLE OF CONFLUENCE

Connector pipe may be joined to main line pipe at angles greater than 45 degrees up to a maximum of 90 degrees provided none of the above conditions exist. Under high velocity and high flow conditions it is preferable for the angle of confluence to be 45 degrees or less.

In general, the angle of confluence between main line and lateral must not exceed 45 degrees and, as an additional requirement, must not exceed 30 degrees under any of the following conditions:

1. Where the peak flow (Q) in the proposed lateral exceeds 10 percent of the main line peak flow.
2. Where the velocity of the peak flow in the proposed lateral is 20 f.p.s. or greater.
3. Where the size of the proposed lateral is 60 inches or greater.
4. Where hydraulic calculations indicate excessive head losses may occur in the main line due to the confluence.

If, in any specific situation, one or more of the above conditions does apply, the angle of confluence for connector pipes may not exceed 30 degrees. Connections must not be made to main line pipe which may create conditions of adverse flow in the connector pipes without prior approval from the City Engineer.

The above requirements may be waived only if calculations are submitted to the City Engineer showing that the use of a confluence angle larger than 30 degrees will not unduly increase head losses in the main line.

6-9(J) FLAPGATES

A flapgate must be installed in all laterals outletting into a main line storm drain whenever the potential water surface level of the main line is higher than the surrounding area drained by the lateral.

The flapgate must be set back from the main line drain so that it will open freely and not interfere with the main line flow. A junction structure will be constructed for this purpose in accordance with City Engineer standards.

6-9(K) RUBBER-GASKETED PIPE

Rubber-gasketed pipe will be used in all storm drain construction unless otherwise approved by the City Engineer.

6-9(L) JUNCTIONS INTO EXISTING STORM DRAIN MAIN LINES

Junctions will only be permitted on mains storm drain lines that are >42 inches. Junction locations cannot be more that 24' from the downstream manhole. The maximum lateral size is 24". The City Engineer's approval will be required for variances.

6-9(M) SUBMITTAL REQUIREMENTS

6-9(M)(1) HYDRAULIC MODEL

1. If a Letter of Map Change is to be submitted to FEMA, the model is to be on the approved FEMA models list at the time of submittal. Approved models are shown on FEMA's website.
2. Electronic hydraulic models must meet the following criteria to be accepted:
   a. *Be able to produce an illustration of the HGL and EGL.*
   b. *Have the ability to include major and minor losses.*
   c. *Meet technical requirements of this chapter.*
   d. *The engineer shall include a description of how the model meets the requirements of this chapter and should describe how losses were taken into account.*
3. For the purposes of generating an infrastructure list, in lieu of submitting the results of an electronic model, the engineer may submit pipe capacity calculations based on gravity flow using Manning's equation.
4. An electronic model is required to design the storm drain for the construction plans.

6-9(M)(2) CULVERTS

1. The City has adopted the *Federal Highway Administration, Hydraulic Design Series Number 5*, method for culvert design.
2. If a proprietary model is used to design a culvert, the engineer shall include a description of how the model is in compliance with the *FHWA* method.

6-10 POND REQUIREMENTS

6-10(A) DESIGN REQUIREMENTS

Some sites may require ponding due to limited downstream capacity. The downstream capacity will be identified in a previously approved drainage plan/report or identified in the drainage submittal. Ponds are of the following types:

6-10(A)(1) DETENTION PONDS

A detention pond has an outfall pipe with an outflow rate less than the inflow rate. All detention ponds must be evacuated in twenty four (24) hours or less, except for the stormwater quality volume. See Section 6-11 for stormwater quality volume. The discharge from some ponds may be more limited by downstream constraints and take longer to evacuate. In these cases, approval of an evacuation time greater than 24 hours is required by the City Engineer. Ponds that take more than six (6) hours to drain will be designed for a design storm equal to or exceeding the evacuation time.

Within a detention pond you can have a water quality pond. The water quality volume is excluded from the evacuation criteria as this volume is to infiltrate.

There are numerous software packages that can be used to calculate the pond volume. The input and output parameters and definitions are to be included with the drainage submittal.

The pond volume can also be calculated manually by discretizing the inflow hydrograph then subtracting the outflow hydrograph.

The minimum outfall size shall be 4 inches in diameter, width or depth. An outlet less than 4 inches in diameter, width or depth may be utilized if accompanied by a maintenance schedule on the City approved drainage submittal.
Detention ponds shall have a designated overflow point that indicates the flow direction if the pond overtops.

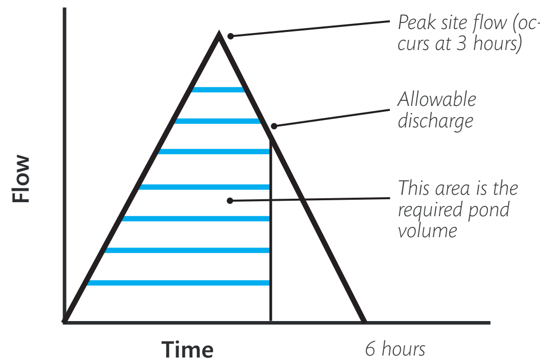
6-10(A)(2) RETENTION PONDS

A retention pond retains stormwater to be infiltrated for the specified design storm. Depending on soil characteristics, the soil on the pond bottom may have to be amended or an infiltration system designed to evacuate the pond within 96 hours.

1. For sites that do not have an outfall, the pond volume will be based on a 100 year 10-day storm.
2. The retention ponds listed below are for sites that have limited downstream capacity, and should have a detention pond, but a detention pond cannot be designed since the outfall pipe could not daylight. Volumes listed below are in addition to the stormwater quality volume.
    a. *For sites that drain to adjoining private property historically, wherein, the adjoining private property does not have an outfall; the pond shall be sized for the 100yr-24hr storm. The adjoining property should not see a change in peak flow or total volume.*

  b. *For sites that drain to adjoining private property historically, wherein the adjoining private property has an outfall, the pond shall be sized for half the runoff from the 100yr-24 hour storm. The adjoining property should not see a change in the peak flow.*
  c. *For sites that drain to a public facility, but have limited capacity, the pond shall be sized via the graphical method shown below.*
3. Retention ponds shall have a designated overflow point that indicates the flow direction if the pond overtops.

Figure 6.26 **Peak Site Flow**



6-10(A)(3) SURGE PONDS

A surge pond functions by ponding the flow in excess of the storm drain capacity. Therefore, lower flows by-pass the pond in the storm drain. Since stormwater quality cannot be addressed in a surge pond, its use is limited to a multi-use facility (e.g. park). Stormwater quality is to be addressed upstream or downstream of the pond prior to discharge to the Rio Grande River.

6-10(A)(4) STORMWATER QUALITY PONDS

Water quality ponds are addressed in Section 6-11.

6-10(B) INFILTRATION RATE

If infiltration rate credit is to be used, it must be supported by a Double-ring Infiltrometer test per ASTM D3385 at the proposed pond bottom. The test results are to be certified by a licensed engineer. In lieu of the double-ring Infiltrometer test, the infiltration rate shall not exceed the rates specified in TABLE 6.7 per the soil type as described in TABLE 6.3 of Section 6-1(A).

6-10(C) PONDS IN PARKING AREAS

Unless otherwise approved by the City Engineer, all ponds in parking lots that affect parking areas must be detention ponds and the depth is not to exceed 8" in any portion of the parking space or parking stall.

6-10(D) FENCING AROUND PONDS

Fencing or similar barricade that will prevent entry is required for private and public ponds where the water depth is 18 inches or greater unless side slopes are 3:1 (H:V) or flatter and the pond drains in 96 hours or less. Fence or barricade minimum height is to be 42 inches.

6-10(E) PRIVATE PONDS TO BE BUILT TO PUBLIC POND SPECIFICATIONS

Private ponds (no Public water or Public Drainage Easement) are to be maintained by the property owner or the party specified on the plat or easement document. If the City finds the pond is not being maintained to the specifications in the drainage report/plan, the City may take over maintenance responsibility of the pond.

Since at a later date, maintenance of the pond may be taken over by the City, Private ponds 2.0 ac-ft and larger are required to be built per Public Pond specifications set forth later in this chapter.

The owner or party specified on the plat or easement document may be financially responsible to the City per *§14-5-2-14* of the Drainage Control Ordinance.

6-10(F) ROCK VOID SPACE FOR POND VOLUME

For underground storage systems the pore void spaces between the aggregate is available to store water. The allowed volume in the aggregate pore void space is 30%. The aggregate is to be natural or uncrushed and be protected from silt and sediment.

There is no pore space volume allowed for surface installations.

6-10(G) PRIVATELY MAINTAINED PONDS WITH A PUBLIC DRAINAGE EASEMENT

Privately maintained ponds which will detain or retain public water must have a Public Drainage Easement and an Agreement and Covenant and be built to City of Albuquerque standards presented later in this section. Ponds exclusively constructed to meet the requirements of *Section 6-11* are excluded.

6-10(H) CITY MAINTAINED PONDS

6-10(H)(1) ACCESS

Access shall be required for all city maintained ponds. Access shall be opposite the outlet if possible with a minimum width of 12 feet. Maximum access slope shall be 10:1 (6:1 if hard surfaced with soil cement or concrete treated base). Standard design tube or pipe gates shall be installed to restrict vehicle access. Gates shall be set back 50 feet from arterial or collector streets so equipment does not have to park in the street.

6-10(H)(2) SPILLWAYS
Emergency spillways shall always be provided, be erosion resistant, and discharge to a public right-of-way, public drainage easement and/or historic flow path. An emergency spillway must safely convey the 100 year design flow entering the pond.

6-10(H)(3) OUTLETS
1. Outlet structures shall be gravity flow, whenever feasible, and be located in a corner or accessible edge of the pond. Outlets shall be opposite of the pond access point if possible. Outlet pipe shall be a minimum of 12 inches in diameter with a slope such that when flowing at 1/4 full, velocity is 3 fps or greater.
2. The outlet should be surrounded by a stabilized grade pad appropriately sized for maintenance.
3. The invert of the pond outlet shall be above the required water quality volume as demonstrated in the drainage report. The pond outlet shall also provide a means to remove floatables and debris.

6-10(H)(4) POND BOTTOMS
1. Pond bottoms shall be designed to convey flows from the inlet to a storm water pollution prevention feature (such as a pervious bottom area for infiltration) prior to discharging to the outlet.
2. Ease of maintenance shall be a consideration in all dams/detention basins.

6-10(H)(5) SIDE SLOPE AND BOTTOM TREATMENTS
1. Vegetation will be accepted if seeded per the *City of Albuquerque Standard Specifications for Public Works Construction*.
2. Aggregate or riprap may be used as an erosion control mulch for 3:1 and steeper slopes.
3. A geotechnical investigation and report may be required at the discretion of the City Engineer.

6-10(H)(6) MINIMUM POND SIZE
In order for a pond to be publicly maintained, it must be a minimum of two (2) acre-feet.

6-10(H)(7) FENCING
1. Ponds 18 inches or greater in depth will require fencing unless side slopes are 3:1 or flatter and the pond drains in 96 hours or less.
2. If fencing is required, the minimum height is 42 inches. All fencing shall conform with the City of Albuquerque Standard Specifications for Public Works Construction.

6-10(I) TEMPORARY PUBLIC PONDS

1. Interim or temporary facilities shall be protected by a Public Drainage Easement and have an Agreement and Covenant for maintenance. These public drainage easements may cover a tract of land larger than that needed for the final permanent facility in lieu of financial guarantees. An agreement and covenant by the developer will be required due to the temporary nature of the facility.
2. Retention pond volume will be based on a 100 year 10-day storm with no percolation credit given for volume reduction.
3. An emergency spillway must be provided that will safely convey the 100 year design flow entering the pond.

6-10(J) POND EVACUATION TIME

All ponds are to be evacuated within 96 hours to comply with State Engineer water rights and to minimize the habitat for mosquitoes.

If soil conditions or bedrock extend the evacuation time to greater than 96 hours, the property owner is to consult with the City Engineer and provide the results of this consultation to the City.

6-10(K) INFILTRATION SYSTEM DESIGN

An infiltration system design should have the width or length dimensions, in plan view, greater than the depth dimension otherwise it is considered an injection well and a permit from the New Mexico Environment Department is required.

The infiltration system design should include a filter material to prevent fine material from entering the system.

6-11 STORMWATER QUALITY AND LOW IMPACT DEVELOPMENT

New development and redevelopment sites are required to capture and infiltrate the stormwater quality volume. The stormwater quality volume is the stormwater runoff from small storms and the initial portion of runoff from larger storms. Stormwater quality requirements shall be satisfied either onsite, offsite or by making a fee-in-lieu, (See Section 6-11(D)).

The stormwater quality volume new development sites are required to manage is the runoff from a 0.62 inch storm. The stormwater quality volume redevelopment sites are required to manage is the runoff from a 0.48 inch storm.

The methodology used in the EPA Report, *Estimating Predevelopment Hydrology in the Middle Rio Grande Watershed*, New Mexico, TetraTech, April 2014, EPA Publication Number 832-R-14-007, yields runoff values of 0.42 inches for the 90th percentile storm and using the same methodology but generated from HEC-HMS, 0.26 inches for the 80th percentile storm.

Therefore, to calculate the required stormwater quality volume to be captured and infiltrated; multiply the impervious area by 0.42 inches for new development sites and 0.26 inches for redevelopment sites.

A site is defined as a redevelopment site if the land was occupied by an artificial surface or by any structure intended for human occupation, including structures intended for commercial enterprise.

For Single Family Subdivisions, stormwater quality ponds will not be allowed on individual lots. Instead a centralized stormwater quality pond for the entire subdivision must be constructed for the entire impervious areas to include the houses, patios, sidewalks, driveways, and public or private streets or a fee-in-lieu can be paid. Alternatively, to determine the amount of impervious area for Single Family Subdivision can use the following equation:

EQUATION 6.58 **Impervious percentage** = $7*\sqrt{((N*N) = (5*N))}$

where:
N = units/acre

For all developments, a combination of on-site ponding and fee-in-lieu is allowed. For these cases, a Fee-in-Lieu waiver may be granted for the impervious areas not treated on-site by adequately sized stormwater quality facility.

6-11(A) LOW IMPACT DEVELOPMENT STRATEGIES

This section outlines principles to apply Low Impact Development strategies to effectively design stormwater quality features to treat the stormwater quality volume as part of the development process.

1. **Consider stormwater quality needs early in the design process.** This will provide for stormwater capture and treatment throughout the site rather than "shoe-horning" the facility resulting in a forced, constrained approach.
2. **Take advantage of the entire site when planning for stormwater treatment.** Spreading the runoff over a larger portion of the site can help to avoid less desirable treatment strategies that rely on underground capture and deep basins that can be difficult to maintain.
3. **Reduce runoff.** Drain impervious areas to landscape areas and minimize directly connected impervious areas. Reduce the amount of impervious areas (e.g. use porous pavement or gravel for low-use or emergency access) and select treatment techniques that promote infiltration.
4. **Integrate stormwater quality management and flood control, when practical.** If the site is required to detain runoff for flood control purposes, the facility used for flood control can be modified for stormwater quality by establishing the overflow elevation above the design standard volume.
5. **Landscape stormwater management facilities.** A stormwater management facility can be an attractive addition to the site, rather than just an unimproved dirt area. In addition, landscaping will minimize the potential for erosion and therefore minimize the amount of required maintenance.
6. **Consider surface conveyance** as an alternative to pipes.
7. **Design facilities for easier maintenance.** Fine soils may clog void spaces with time. The designer should consider a capture area for fine soils where stormwater enters the facility that can be easily replaced or maintained.
8. **Amend the soil** to allow for improved infiltration.

6-11(B) EFFECTIVE STRATEGIES FOR STORMWATER TREATMENT

There is a variety of methods to improve stormwater quality. Not all methods are appropriate for all development types. See TABLE 6.14 for development types.

TABLE 6.14 DEVELOPMENT TYPES

Development Type	Percentage Landscaping	Percentage Parking/Paving	Building Footprint	Parking
Dense Urban	0-5%	0-5%	90-100%	On-Street Structure
High Density Mixed Use	0-10%	0-15%	80-90%	On-Street Structure and Surface
Commercial/ Industrial	5-15%	40-60%	25-50%	Surface
Low Density Mixed Use	10-25%	30-50%	25-60%	Surface
Residential	30-70%	5-20%	30-70%	Surface
Educational/ Institutional	15-60%	10-25%	25-60%	Surface
Parks/Open Space	80-95%	5-15%	0-10%	Surface

The following methods can be used to improve stormwater quality:

6-11(B)(1) LANDSCAPE CATEGORY

1. Depressed parking islands or planters with curb cut(s)

Figure 6.27 Depressed Parking Island

NOTE: *If perforated pipe is used, the pipe is to be wrapped in landscape fabric.*

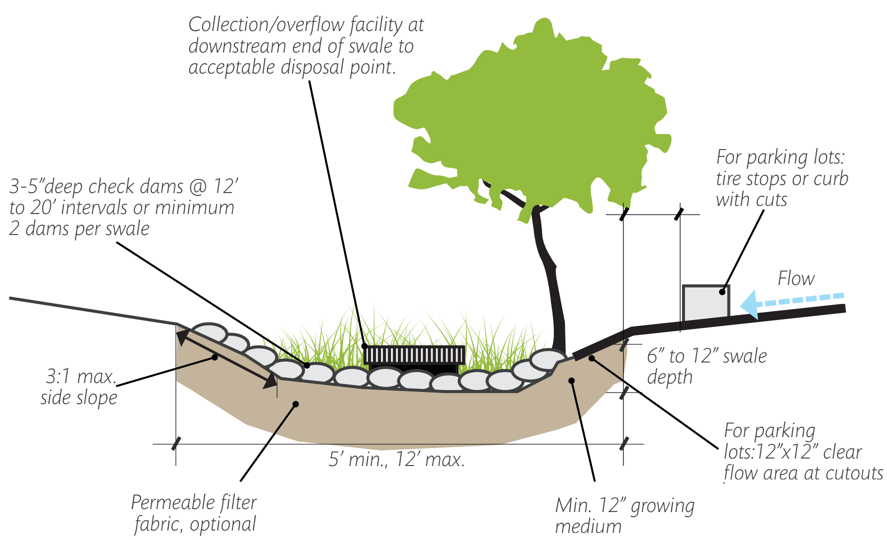
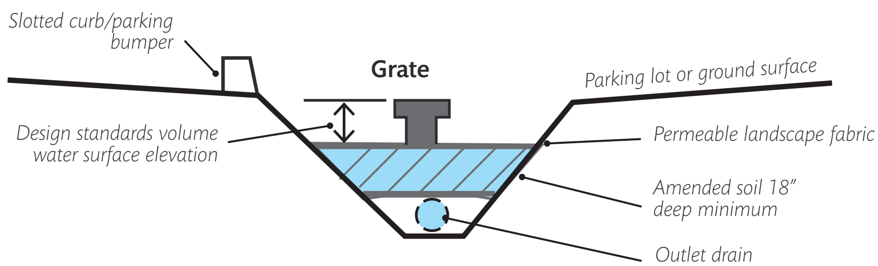
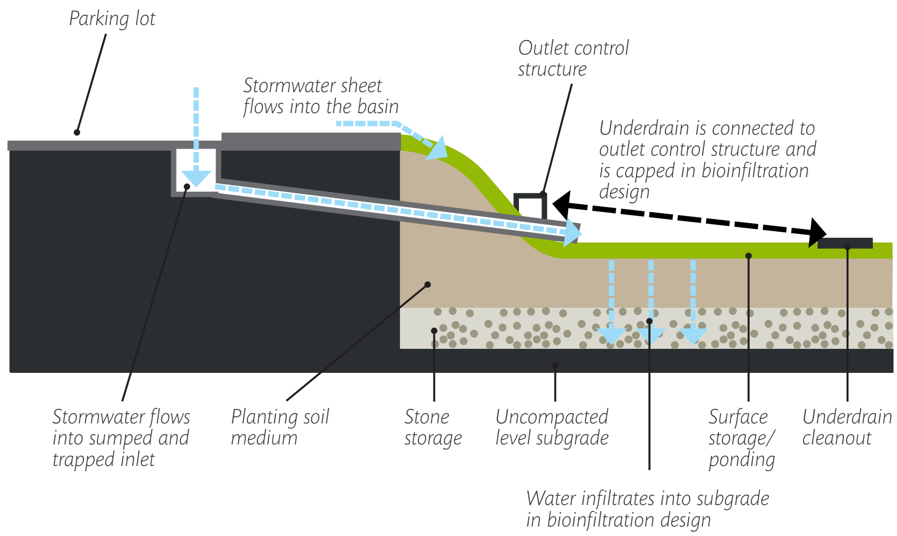


Figure 6.28 **Depressed Parking Island Diagram**



2. Depressed landscape/bioretention areas

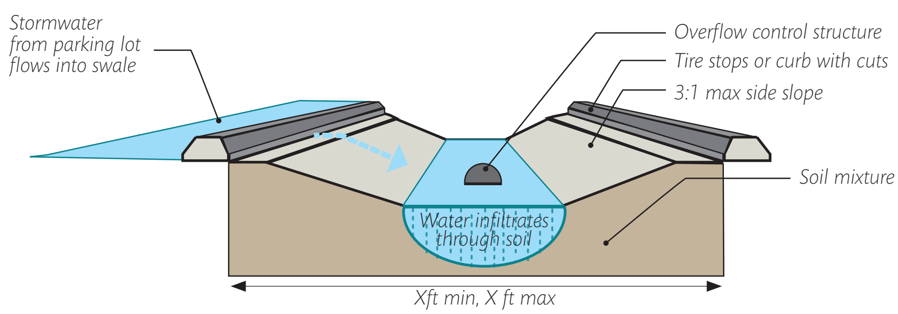
Figure 6.29 **Depressed Bioretention Areas**



3. Landscape Conveyance-Bioswale

Figure 6.30 **Landscape Conveyance-Bioswale**

NOTE: *Plants filter and transpire water while enhancing the parking lot.*

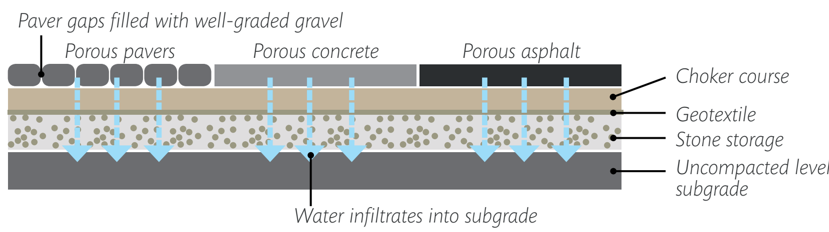


4. Infiltration Trench
   a. *An infiltration trench is an effective means of capturing the design standard volume underground in the void space of the media (e.g. sand, rock). Maximum porosity (void space) to be used is 30%. A replaceable filter material (e.g pea gravel,) shall be used to prevent the build-up of fine material in the trench.*
   b. *The length or width dimension must be greater than the depth dimension so that the trench is not considered an injection well.*

6-11(B)(2) PAVING CATEGORY

1. Pervious pavers, concrete or asphalt
2. Open-cell structure with gravel
3. Gravel parking lots
4. Underground cisterns

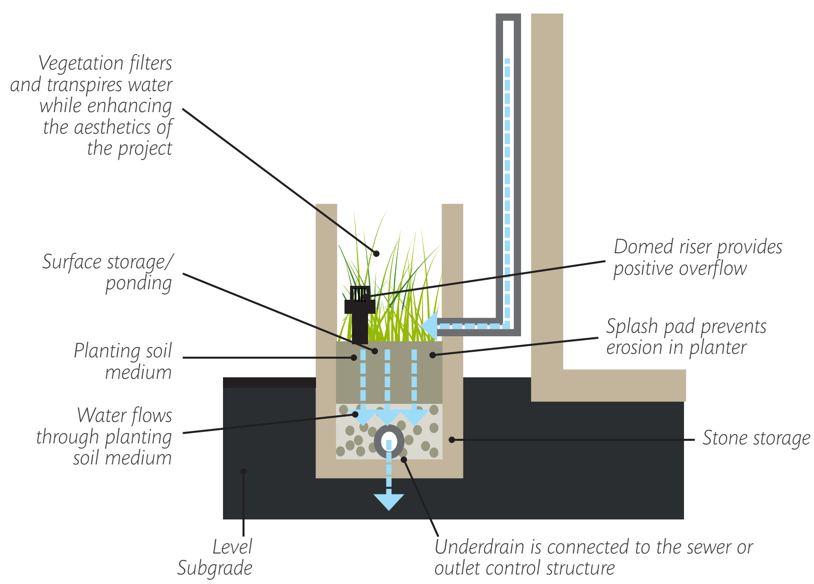
Figure 6.31 Porous Pavement with Typical Features



6-11(B)(3) ELEVATED CATEGORY

1. Planter boxes
2. Cisterns
3. Green/brown roofs

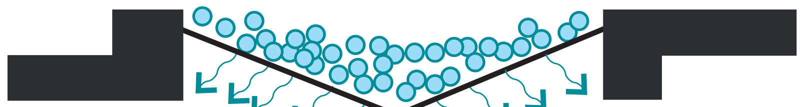
Figure 6.32 Planter Boxes Diagram



6-11(B)(4) STREETSCAPE CATEGORY

1. The landscape area between the sidewalk and back of curb is to be depressed and covered in rock to prevent erosion. See City Of Albuquerque Standard Drawings for construction details.

Figure 6.33 Swale Diagram



2. Street medians
   a. *On arterial streets, the designer may choose to drain the street into the median. Since this is a change from the City standard drawings, approval from the City Engineer is required. The minimum median width is 8 feet. Check dams will be required in the median on streets with slopes greater than 2.5% to reduce velocity.*
   b. *The grate elevation is to be perched to allow the runoff from smaller storms to infiltrate.*

6-11(B)(5) FLOOD CONTROL CATEGORY

The stormwater quality volume can be incorporated into a flood control facility by elevating the discharge point above the water surface elevation of the stormwater quality volume. In addition to managing the stormwater quality volume, flood control facilities shall remove trash and debris.

6-11(B)(6) OFFSITE MITIGATION CATEGORY

Constructing stormwater quality improvements outside the project boundaries is only available for projects that qualify for a Variance as discussed in *Section 6-11(C)*.

6-11(B)(7) PAYMENT-IN-LIEU

After the alternative Variance criteria discussed later in this chapter is demonstrated, payment-in-lieu may be approved by the City Engineer. See *Section 6-11(C)*.

All development types are to manage the stormwater quality volume with one or more of the methods listed in *Section 6-11(A)*. *TABLE 6.14* shows the development types and which categories of methods are most appropriate.

TABLE 6.15 RECOMMENDED IMPLEMENTATION MATRIX

Development Type	Landscape Category	Paving Category	Elevated Category	Streetscape Category	Floor Control Category	Off Site Mitigation Category	Payment-in-Lieu Category
Dense Urban		X	X	X		X	X
High Density Mixed Use	X	X	X	X		X	X
Commercial/Industrial	X	X	X	X	X		
Low Density Mixed Use	X	X		X	X		
Residential	X			X	X		
Educational/Institutional	X	X		X	X		
Parks/Open Space	X	X		X	X		

6-11(C) VARIANCE FROM ON-SITE STORMWATER QUALITY VOLUME REQUIREMENTS

A waiver from constructing the stormwater quality facility required by this section maybe granted when authorized by the City Engineer, on a case-by-case basis. If the City Engineer grants a waiver, a fee in lieu will be paid by the applicant as outlined in Section 6-11(C)(1). Requests to waive the on-site stormwater quality facility requirements shall be submitted to Development & Review Services Division for Hydrology Section staff approval.
The requirements for on-site stormwater quality facility may be waived upon written request of the applicant, provided that the following condition applies:

1. A project may be eligible for a waiver from constructing the on-site stormwater quality facility if the applicant can demonstrate that:
   a. *The proposed project will have no significant adverse impact on the receiving natural waterway or downstream properties; or*
   b. *It can be demonstrated that the proposed development is not likely to impair attainment of the objectives of this ordinance; or*
   c. *Alternative requirements for on-site stormwater quality facility have been established in a stormwater management plan that has been approved by the Development & Review Services Division Hydrology Section.*
2. Justification and a written request, including the following statement: "the increased flows will not have a significant adverse impact on the downstream/adjacent properties" must be submitted with the request.
3. The project's Professional Engineer must sign the waiver request and a variance application must be submitted to the Development & Review Services Division for Hydrology Section staff approval.
4. In order to receive a waiver, the applicant must demonstrate to the satisfaction of the Development & Review Services Division for Hydrology Section that the waiver will not lead to the degradation of aquatic ecosystem or habitat condition downstream.

6-11(C)(1) FEE IN LIEU

The fee is based on the total impervious area calculated then the use of either _ for New Development (Multi-family & Commercial), _ for Redevelopment (Multi-family & Commercial), Table 22.11.3.3 for New Development (Single Family), or Table 22.11.3.4 for Redevelopment (Single Family).
An applicant will not have the option to pay a fee in lieu of constructing a stormwater quality facility if, in the opinion of the city engineer, undetained runoff from the development may materially adversely exacerbate an existing problem.

6-11(C)(2) ANNUAL ADJUSTMENT OF FEE

The fees in Minimum Stormwater Quality Fee Table shall be adjusted upward on every July 1 by multiplying the rates in effect on the prior July 1 by 100 percent of the percentage increase in the Consumer Price Index (CPI) for the 12-month period ending the preceding April. The fees shall remain the same in the event the CPI indicates a decrease. If the index ceases to be published on a monthly basis, the adjustment shall be based on the CPI for the most recent 12-month period. The CPI to be used shall be the Consumer Price Index – All Urban Consumers as published by the United States Department of Labor for the Albuquerque Metropolitan area.

6-11(C)(3) PAYMENT OF FEE

Payment of the fee shall be made based on the following:

1. Multi-Family Development. Prior to the issuance of a building permit; or
2. Commercial Development. Prior to the issuance of a building permit; or
3. Single Family Subdivision. Prior to recording the final plat.

TABLE 6.16 NEW DEVELOPMENT

Multi-Family and Commercial Minimum Stormwater Quality Fee	
Impervious Surface Area	Fee
Less than 2,000 sq. ft.	$600.00
2,000 – 4,000 sq. ft.	$1,200.00
4,000 – 6,000 sq. ft.	$1,800.00
6,000 – 8,000 sq. ft	$2,400.00
8,000 –10,000 sq. ft	$3,000.00
10,000 – 15,000 sq. ft.	$4,537.50
15,000 – 20,000 sq. ft.	$6,050.00
20,000 – 25,000 sq. ft.	$7,562.50
25,000 – 30,000 sq. ft.	$9,075.00
30,000 – 35,000 sq. ft.	$10,675.00
35,000 – 40,000 sq. ft.	$12,200.00
40,000 – 45,000 sq. ft.	$13,725.00
45,000 – 50,000 sq. ft.	$15,250.00
50,000 – 55,000 sq. ft.	$16,912.50
55,000 – 60,000 sq. ft.	$18,450.00
60,000 – 65,000 sq. ft.	$19,987.50
65,000 – 70,000 sq. ft.	$21,525.00
70,000 – 75,000 sq. ft.	$23,250.00
75,000 – 80,000 sq. ft.	$24,800.00
80,000 – 85,000 sq. ft.	$26,350.00
85,000 – 90,000 sq. ft.	$27,900.00
90,000 – 95,000 sq. ft.	$29,687.50
95,000 – 100,000 sq. ft.	$31,250.00
100,000 – 150,000 sq. ft.	$46,875.00
150,000 – 200,000 sq. ft.	$63,000.00
200,000 – 250,000 sq. ft.	$78,750.00
250,000 – 300,000 sq. ft.	$95,250.00
300,000 – 350,000 sq. ft.	$111,125.00
350,000 – 400,000 sq. ft.	$128,000.00
400,000 – 450,000 sq. ft.	$144,000.00
450,000 – 500,000 sq. ft.	$161,250.00
500,000 – 550,000 sq. ft.	$177,375.00
550,000 – 600,000 sq. ft.	$195,000.00
600,000 – 650,000 sq. ft.	$211,250.00
650,000 – 700,000 sq. ft.	$229,250.00
700,000 – 750,000 sq. ft.	$245,625.00
750,000 – 800,000 sq. ft.	$264,000.00
800,000 – 850,000 sq. ft.	$280,500.00
850,000 – 900,000 sq. ft.	$299,250.00
900,000 – 950,000 sq. ft.	$315,875.00
950,000 – 1,000,000 sq. ft.	$335,000.00
Greater than 1,000,000 sq. ft.	$0.335 per sq.ft.

TABLE 6.17 REDEVELOPMENT	
Multi-Family and Commercial Minimum Stormwater Quality Fee	
Impervious Surface Area	Fee
Less than 2,000 sq. ft.	$400.00
2,000 – 4,000 sq. ft.	$800.00
4,000 – 6,000 sq. ft.	$1,200.00
6,000 – 8,000 sq. ft	$1,600.00
8,000 –10,000 sq. ft	$2,000.00
10,000 – 15,000 sq. ft.	$3,037.50
15,000 – 20,000 sq. ft.	$4,050.00
20,000 – 25,000 sq. ft.	$5,062.50
25,000 – 30,000 sq. ft.	$6,075.00
30,000 – 35,000 sq. ft.	$7,175.00
35,000 – 40,000 sq. ft.	$8,200.00
40,000 – 45,000 sq. ft.	$9,225.00
45,000 – 50,000 sq. ft.	$10,250.00
50,000 – 55,000 sq. ft.	$11,412.50
55,000 – 60,000 sq. ft.	$12,450.00
60,000 – 65,000 sq. ft.	$13,487.50
65,000 – 70,000 sq. ft.	$14,525.00
70,000 – 75,000 sq. ft.	$15,750.00
75,000 – 80,000 sq. ft.	$16,800.00
80,000 – 85,000 sq. ft.	$17,850.00
85,000 – 90,000 sq. ft.	$18,900.00
90,000 – 95,000 sq. ft.	$20,187.50
95,000 – 100,000 sq. ft.	$21,250.00
100,000 – 150,000 sq. ft.	$31,875.00
150,000 – 200,000 sq. ft.	$43,000.00
200,000 – 250,000 sq. ft.	$53,750.00
250,000 – 300,000 sq. ft.	$65,250.00
300,000 – 350,000 sq. ft.	$76,125.00
350,000 – 400,000 sq. ft.	$88,000.00
400,000 – 450,000 sq. ft.	$99,000.00
450,000 – 500,000 sq. ft.	$111,250.00
500,000 – 550,000 sq. ft.	$122,375.00
550,000 – 600,000 sq. ft.	$135,000.00
600,000 – 650,000 sq. ft.	$146,250.00
650,000 – 700,000 sq. ft.	$159,250.00
700,000 – 750,000 sq. ft.	$170,625.00
750,000 – 800,000 sq. ft.	$184,000.00
800,000 – 850,000 sq. ft.	$195,500.00
850,000 – 900,000 sq. ft.	$209,250.00
900,000 – 950,000 sq. ft.	$220,875.00
950,000 – 1,000,000 sq. ft.	$235,000.00
Greater than 1,000,000 sq. ft.	$0.235 per sq.ft.

6-11(D) LAND USES THAT REQUIRE ADDITIONAL STORMWATER CONTROLS

6-11(D)(1) AUTOMOTIVE REPAIR AND PARTS SHOPS

These land uses include shops that repair any portion of a vehicle (e.g. automotive body shops, general automotive repair) and retail automotive parts stores that have parking for customers. The exterior impervious area of these land uses shall drain to a surface stormwater quality facility that will remove pollutants from the stormwater prior to discharge into the street or drainage facility.

6-11(D)(2) RESTAURANTS AND COMMERCIAL FOOD PROCESSING

These land uses shall provide a drain in the trash enclosure that drains to the sanitary sewer after passing through a grease trap.

6-11(D)(3) GAS STATIONS/FUELING FACILITIES

These land uses shall provide treatment for the area at the gas pumps, which is usually the same area as the canopy. The drainage/wash water from this area shall enter an area inlet(s), then be treated by a sand filter or similar prior to discharge into the street or drainage facility.

6-11(E) POST-CONSTRUCTION MAINTENANCE AND RESPONSIBILITIES

The following Post-Construction Maintenance and Responsibilities shall be performed in perpetuity:

Private Stormwater Facilities shall be maintained and inspected per the City approved drainage submittal by the facilities' (property) owner or responsible party. Stormwater quality facilities shall be identified on the City approved drainage submittal and recorded by standard form Covenant with the County Clerk.

The Covenant is required prior to issuance of Permanent Certificate of occupancy for commercial projects and prior to building permit approval for single family residential projects as identified on the City approved drainage submittal.

The property owner may choose to document the stormwater quality facility requirements on the plat including benefit and maintenance responsibility language as identified on the City approved drainage submittal.

6-11(E)(1) POST-CONSTRUCTION INSPECTIONS

The City will conduct post construction site inspections to ensure the stormwater quality features of a site are being maintained in accordance with the approved drainage submittal.

6-11(F) STORMWATER CONTROL PERMIT FOR EROSION AND SEDIMENT CONTROL

All grading within the City of Albuquerque must be performed in a manner which prevents the movement of significant and damaging amounts of sediment onto adjacent property and public facilities by both water and wind, and minimizes the impacts to stormwater runoff quality.

To conform with EPA stormwater regulations, the property owner and general contractor must file an eNOI with the EPA for sites disturbing 1 acre or more of land, or is part of a larger common plan of development that will disturb greater than one acre of land,14 days prior to commencing earth disturbing activities.

In addition, a City issued Stormwater Control Permit for Erosion and Sediment Control (ESC) is required prior to earth disturbance or construction on projects that disturb 1 acre or greater of land or the following:
1. The site is part of a larger common plan of development that will disturb greater than one acre of land.
2. The site is identified as having a significant potential for erosion, based on observation or site characteristics including very steep (8% or greater) topography.
3. The site is known to contain contaminated soils.
4. The site lies in a Priority Area as defined by the City Engineer and posted on the City's website.

The ESC Permit is to be approved prior to The City approving a Building Permit(s) for the project.
1. The ESC Permit can be issued for earth disturbance and for Building Permit individually or together. The ESC Permit is the responsibility of the property owner.
2. The following approvals are required in advance of City approval of the ESC Permit:
   a. *Grading and Drainage Plan,*
   b. *Erosion and Sediment Control Plan*
   c. *Floodplain Development Permit, if construction activities will occur in a mapped floodplain.*
3. BMPs identified on the ESC plan are to be in place prior to earth disturbance/construction. If the ESC plan is implemented in phases, the BMPs identified for that phase are to be in place prior to earth disturbance/construction for that phase.
4. A permit application is available on line or at the City Engineer's office.
5. For sites that are part of a larger common plan of development, the last lot or pad site in the development will not need an ESC Plan if it is less than 0.45 acres.

6-11(G) CONSTRUCTION SITE MAINTENANCE AND INSPECTIONS

1. Self-inspections by permittee. At a minimum a routine compliance self- inspection is required to review vegetation, erosion and sediment control measures, and other protective measures identified by the Erosion and Sediment Control Plan and the associated SWPPP. Sites must be maintained per the EPA National Pollutant Discharge Elimination System (NPDES) Construction General Permit and the City Drainage Control Ordinance.
2. The City will conduct inspections of construction sites for compliance with the EPA NPDES Construction General Permit and the Drainage Control Ordinance.
3. Sites located in priority areas will be inspected by the City more frequently. A site is located in a priority area if the site drains to a Waters of the U.S. without passing through a public detention or retention facility that removes sediment, debris and floatables between and the Rio Grande river.

6-12 DRAINAGE RIGHT-OF-WAY AND EASEMENTS

6-12(A) RIGHTS-OF-WAY

Whenever no beneficial use can be derived by an owner from continued retention of that land necessary for permanent drainage, flood control or erosion control facilities or when the facilities involve a major arroyo, the land required for the operation and maintenance of the facilities must be dedicated to AMAFCA or the City. Maintenance responsibility of the facilities must be clearly defined.

6-12(B) EASEMENTS

Easements for drainage, flood control and erosion control facilities are acceptable (except where prohibited in Subsection 22.12.1 above) as long as a clear agreement exists as to other acceptable uses and that no other permanent facilities (e.g. non-drainage facilities) are constructed within them (including masonry fences and retaining walls but excluding pavement) without an agreement between the owner and the City, governing the permitted uses. Maintenance responsibility of the facilities must be clearly defined. Easements can be shown on a plat or be provided by a paper easement. Paper easements are processed through the Design Review and Construction Services section.

6-12(C) CONFIGURATION

Rights-of-way and permanent easements required for drainage, flood control and erosion control facilities will conform to the following criteria:

6-12(C)(1) SURFACE FACILITIES:

The dedicated area should contain the entire facility including any slopes, maintenance roads, turn arounds or other necessary appurtenances. Easement width shall be sufficient to allow for maintenance activities. Public Easements must be a minimum of 10 feet wide

6-12(C)(2) PUBLIC UNDERGROUND FACILITIES:

Dedicated areas for Public underground facilities shall not be narrower than 20 feet for any drainage facility and must conform to the following formula, unless otherwise approved by the City Engineer:

EQUATION 6.59 $\mathbf{W} = 2 \times D_s$ + pipe diameter or box culvert width+ 4 feet

where:
W = dedicated width in feet
$\mathbf{D_s}$ = depth to bottom of the structure (invert + thickness of the structure)

Outside dimensions must be used for pipe diameter and box culvert width. Other utilities shall not be permitted within the trench prism of the drainage facility

6-12(D) DRAINAGE RIGHT-OF-WAY AND PUBLIC DRAINAGE EASEMENT ACCESS

All newly constructed surface drainage facilities within a public right-of-way or Public Drainage Easement must be blocked off at both ends to prevent unauthorized vehicular access with City Standard Tube Gate or removable bollards.

6-12(E) PRIVATE STORM DRAIN IMPROVEMENTS WITHIN CITY RIGHT-OF-WAY AND/OR EASEMENTS

Frequently, a drainage plan developed for a particular property involves either discharge directly into a public facility or across a portion of a public right-of-way to a public facility.

Examples include connections to the back of an existing storm inlet, construction of sidewalk culverts or a connection to a storm drain manhole or a channel. When such solutions are employed the construction of private storm drain improvements within the City Right-of-Way must comply with the following requirements:

1. The proposed improvement must be incorporated on the grading and drainage plan. This plan must include the design or City standards to be used and the location of the proposed construction in the City Right-of-Way.
2. An excavation/construction permit will be required before beginning any work within the City's Right-of-Way. An approved copy of the grading and drainage plan must accompany the excavation/construction permit request.
3. All work to be performed within the public Right-of-Way or easement shall be constructed in accordance with City of Albuquerque Standard Specifications for Public Works Construction.
4. Prior to construction, the contractor shall excavate and verify the horizontal and vertical locations of all constructions to identify a conflict. Should a conflict exist, the contractor shall notify the engineer so that the conflict can be resolved with a minimum of delay.
5. Backfill compaction shall be according to City of Albuquerque Standard Specifications for Public Works Construction.
6. The facility is to be inspected and accepted by the City prior to obtaining a Permanent Certificate of Occupancy.
7. Maintenance of these facilities shall be the responsibility of the owner of the property served.
8. Notes 1 through 7 listed above are to be placed on the grading and drainage plan for approval by the Hydrology Section of the Planning Department..

6-13 DRAINAGE AND STORMWATER QUALITY SUBMITTALS

6-13(A) INTRODUCTION

A drainage and stormwater quality submittal is generally in the form of a Conceptual Grading and Drainage Plan, Drainage Report, Grading and Drainage Plan, Erosion and Sediment Control Plan, LOMR, CLOMR or LOMR-F. The following are definitions of these types of submittals:

6-13(A)(1) DRAINAGE REPORT

A Drainage Report is a comprehensive analysis of the drainage management, flood control, erosion control constraints on and impacts resulting from the proposed platting, development or construction of a particular project.

6-13(A)(2) CONCEPTUAL GRADING AND DRAINAGE PLAN

The purposes of this plan are to check the compatibility of the proposed development within grading, drainage, floodplain, erosion control and stormwater quality as dictated by onsite physical features as well as adjacent properties, streets, alleys and channels. Unless otherwise approved by the City Engineer, a Conceptual Grading and Drainage Plan is required for EPC and DRB approval.

6-13(A)(3) GRADING AND DRAINAGE PLAN

A Grading and Drainage Plan is a comparatively short, yet comprehensive, presentation for small, non-complex development submittals. Grading and Drainage Plans address both onsite and offsite drainage management, flood control, erosion control and stormwater quality.

6-13(A)(4) EROSION AND SEDIMENT CONTROL PLAN

An Erosion and Sediment Control (ESC) plan provides necessary information to prevent erosion and sediment deposition in city streets and drainage facilities during the construction phase of a project. Necessary information includes erosion and sediment control Best management Practices (BMPs) as well as keyed notes. Typical BMPS include inlet protection, silt fence, mulch socks or wattles, erosion control mats, tackifier and a stabilized construction entrance (track-out pad).

6-13(A)(5) LOMR, CLOMR AND LOMR-F (LOMC) SUBMITTALS

Documents that are submitted to FEMA to change a mapped flood zone or remove property or a structure from a flood zone are described in Section 5.

6-13(A)(6) ENGINEER CERTIFICATIONS

Engineer Certifications are as-built grading plans and/or as-built grading and drainage plans.

The table below provides a matrix to aid property owners and consultants to determine which form of drainage submittal to submit to the City based upon the approval sought:[6]

TABLE 6.18 DRAINAGE SUBMITTALS FOR APPROVAL SOUGHT

Approval Sought	Conceptual Grading & Drainage Plan	Drainage Report	Grading & Drainage Plan	Engineer Certification	Erosion & Sediment Control Plan
EPC Site Plan	X				
DRB Site Plan					
For Building Permit	X	X[5]			X[7]
DRB Site Plan for Subdivision	X				
Plat: >10 lots or ≥ 5 acres		X	X		
Plat: <10 lots or ≤ 5 acres		X[1]	X[1]		
Building Permit		X[2]	X[3]		X[7]
Private Facility Drainage Permit			X	X	
Drainage Master Plan		X			
Work Order Construction Plans		X[5]	X[1]		X[7]
Release of Financial Guarantee/ Final Plat				X[4]	
Construction in a Flood Zone(8)		X[5]	X	X	
Certificate of Occupancy				X[6]	

1. A grading plan or drainage report may not be required to obtain approval. Schedule a pre-design meeting with Hydrology to determine if a drainage submittal is required.
2. Projects 5 acres or larger shall require a drainage report. Smaller projects in complex drainage basins may also require a drainage report.
3. Some single family residential homes not located in a mass-graded subdivision may require a drainage submittal based on topography, Flood Hazard Zone designation, or site conditions.

6 *An "X" in a box in the table indicates the submittal is required, unless a note as discussed below the table, indicates otherwise.*

4. The requirement to submit an engineer's certification will be noted on the infrastructure list.
5. A drainage report may not be required for non-complex sites. Schedule a pre-design meeting with Hydrology to determine if a drainage report is required.
6. An Engineer's Certification is required if an approved grading plan was required prior to earth disturbance, except for single family residential homes that are part of a mass graded subdivision.
7. See Section 11 for the criteria when an Erosion and Sediment Control Plan is required.
8. All projects in a flood zone require a Floodplain Development Permit and most will require a submittal to FEMA.

6-13(B) DRAINAGE SUBMITTAL CRITERIA

Each submittal shall include the following information:
1. Project Name
2. Name of Engineering Firm
3. Engineer's Seal (signed and dated)
4. Completed Drainage Information Sheet

Information is identified in the outline below: [7]

6-13(B)(1) EXECUTIVE SUMMARY

1. Provide a brief yet comprehensive discussion of the following:
    a. *General project location*
    b. *Development concept for the site*
    c. *Drainage concept for the site (include relevant numbers as appropriate)*
    d. *How offsite flows will be handled*
    e. *How onsite flows will be handled and discharged*
    f. *Downstream capacity and how determined*
    g. *Impacts on or requirements of other jurisdictions*
    h. *How stormwater quality volume will be managed*
2. Identify all approvals being requested in conjunction with this submittal, such as:
    a. *Zone Change*
    b. *Subdivision Plat*
    c. *Site Plan for Subdivision*
    d. *Site Development Plan for Building Permit*
    e. *Building Permit*
    f. *Private Facility Drainage Permit*
    g. *Grading Permit*
    h. *Paving Permit*
    i. *DPM Design Variance*
    j. *CLOMR, LOMR or LOMA*

7 *The following Outline is intended only as a guide for the preparation of Drainage Submittals. Some items may not be applicable, while other items may require a more in-depth treatment A Pre-design Conference is recommended for projects where the scope may be difficult to define, the constraints and conditions somewhat unique, or the drainage solution non-traditional.*

The allowable discharge from a particular project shall be determined based upon available downstream capacity as defined by the Drainage Ordinance. In certain cases, the allowable discharge shall be based upon the value(s) set forth in previously approved and/or adopted Drainage Management Plans, Drainage Plans reports or studies.

6-13(B)(2) INTRODUCTION

1. Narrative description of project scope
   a. *Provide more detail than presented in the Executive Summary (combine with Executive Summary for non-complex projects)*
2. Project requirements
   a. *Discuss and reference required infrastructure and associated infrastructure list*
   b. *Platting and/or easements*
   c. *Approvals by and/or coordination with other Agencies and/or entities*
3. Attachments (when applicable)
   a. *Infrastructure List (draft, preliminary, amended or approved)*
   b. *Preliminary or Final Plat*
   c. *Easement Documents*
   d. *Drainage Covenants*
   e. *Approval Letters*

6-13(B)(3) PROJECT DESCRIPTION

1. Location
   a. *Discuss relationship of the site to the following:*
      i Well known landmarks
      ii Municipal limits
      iii City Zone Atlas page and reference
      iv Other jurisdictional boundaries
      v Previously approved Drainage Management Plans, Drainage Reports, Plans or studies including watersheds, basins, drainage-ways, etc. as defined therein
   b. *Provide copy of Zone Atlas page, or equivalent, with the site location superimposed*
2. Legal Description
   a. *Identify the current legal description(s) of the land which comprises the site*
   b. *Identify the proposed legal description(s), when applicable, of the land which comprises the site*
   c. *Include a copy of existing and/or proposed platting as an attachment in cases where its inclusion will lend clarity or facilitate the review*
3. Flood Hazard Zone
   a. *Identify proximity of site to a designated Flood Hazard Zone*
   b. *Provide reference to the above referenced Flood Hazard Zone*
   c. *Identify whether or not the site drains to or has an adverse impact upon a designated Flood Hazard Zone*
   d. *Include a copy of the relevant FEMA Flood Insurance Rate Map (FIRM) or Flood Boundary and Floodway Map with the site clearly identified along with all affected Flood Zones*
   e. *Identify portion of designated Flood Hazard Zone to be revised or amended when CLOMR, LOMR or LOMA approval requested.*

6-13(B)(4) BACKGROUND DOCUMENTS

1. Planning History
   a. *Reference and discuss relevant Planning and Zoning actions, plans or studies*
   b. *Verify and/or demonstrate compatibility with the above actions, plans and studies*
2. Drainage History and Related Documents

a. *Reference and discuss relevant Drainage Management Plans, Drainage Plans, Reports and Studies*
b. *Reference applicable Hydrology File, PWD (DRC) Project and DRB Project numbers*
c. *Discuss status of above referenced Plans, Reports and Studies*
d. *Describe compatibility with or deviation from the above referenced Plans, Reports and Studies*
e. *Describe the location of site with respect to previously defined watersheds or drainage basins*
f. *Provide copies of pertinent data from above referenced Plans, Reports and/or Studies when applicable*

6-13(B)(5) EXISTING CONDITIONS

1. Site Investigation
    a. *Describe by text or clearly show graphically the following:*
        i onsite drainage patterns
        ii onsite drainage facilities
        iii point(s) of discharge
        iv drainage basin(s) boundaries
        v offsite drainage facilities
        vi offsite drainage patterns including offsite flow conditions
        vii condition and status of adjacent properties (e.g. developed, undeveloped, under construction, etc.)
        viii condition and status of adjacent right-of-way (e.g. developed, undeveloped, under construction, etc.)
        ix presence of any other relevant features
2. Site Evaluation
    a. *Discuss the significance and impacts of the following:*
        i onsite drainage facilities
        ii offsite drainage facilities
        iii point(s) of discharge
        iv drainage basin(s) boundaries
        v offsite flow conditions
        vi proximity to designated flood hazard zone(s)
        vii presence of any other relevant features or conditions which may impact or be impacted by the development of the property or project
    b. *Form of Analysis*
        i Most situations - most submittals require both qualitative and quantitative analyses
        ii Unique situations - for some cases, such as infill sites, a qualitative analysis by itself may be appropriate. Examples of appropriate qualitative analysis criteria are
            (1) a comparison of the runoff generated by the proposed development to that generated by the overall drainage basin with respect to the impacts of the anticipated increase
            (2) impacts on downstream flood plains
            (3) potential offsite problems which may or may not be attributed to this development
            (4) anticipated impact(s) and/or precedent to be set on the development of the remaining infill sites by following the same drainage concept
    c. *Downstream Capacity. Downstream capacity is discussed in Section 6. (The evaluation of downstream capacity shall include, but not be limited to, the following:)*

i Assumptions
(1) fully developed watershed
(2) ability to accept and safely convey runoff generated from the 100-year design storm
ii Hydraulic capacity
(1) channel
(2) crossing structure
(3) storm inlet and/or entrance conditions
(4) storm drain
(5) street and/or alley
iii Storage capacity
(1) Detention pond/reservoir
(2) Retention pond
(3) Flood zone
iv Stability
(1) Channel/arroyo
(2) Natural slope
(3) Cut/fill slope

6-13(B)(6) DEVELOPED CONDITIONS

1. Onsite
   a. *Discuss the following as applicable:*
      i proposed development/construction
      ii impacts on existing drainage patterns
      iii impacts on existing drainage basins
      iv impacts on existing onsite facilities
      v identification of offsite flow conditions
      vi compatibility/compliance with previously approved and/or adopted Plans, Reports and Studies
      vii sediment bulking
      viii aggradation and/or degradation potential
      ix impacts on designated flood hazard zones (A Zones only)
      x required private drainage improvements
      xi required infrastructure
      xii required easements
      xiii phasing and future improvements
      xiv ownership, operation and maintenance responsibilities
      xv stormwater quality basins and corresponding facility
   b. *Evaluate and/or quantify the following:*
      i capacity and freeboard of existing onsite facilities
      ii capacity and freeboard of proposed onsite facilities
      iii impacts on designated flood hazard zones
      iv impacts on existing drainage patterns and drainage basin boundaries
      v impact of offsite flows on the proposed development
      vi erosion potential and erosion setback requirements
      vii phased system capacities and ability to function as a stand alone system
      viii emergency overflow spillway conditions
2. Offsite
   a. *Discuss the following:*
      i impacts on existing drainage basins and/or watersheds
      ii impacts on existing offsite facilities and downstream capacity

iii compatibility/compliance with previously approved and/or adopted Plans, Reports and Studies
iv impacts on designated flood hazard zones
v required improvements
vi required easements
vii right-of way dedications
viii phasing and future improvements
ix ownership, operation and maintenance responsibilities
x concurrence and/or approval from affected property owners for offsite grading or construction activities

b. *Evaluate and/or quantify the following:*
i capacity of existing offsite facilities
ii capacity of proposed offsite facilities
iii impacts on downstream designated flood hazard zones
iv impacts on downstream drainage basins and/or watersheds
v downstream capacity

6-13(B)(7) GRADING PLAN

1. Description
   a. *Reference the Grading Plan when included as an attachment to the Drainage Submittal*
   b. *Describe elements of the Plan and how those elements relate to the Existing and Developed Conditions sections of the submittal discussed above*
   c. *Discuss and reference all other supporting drawings provided in support of the Drainage Submittal*
2. Content
   a. *Refer to Grading Plan Checklist that follows.*

6-13(B)(8) CALCULATIONS

1. Description
   a. *Provide narrative description of the calculations performed to support the analyses and evaluations discussed above*
   b. *Discuss and reference calculations for Existing, Developed and Future hydrology*
   c. *Discuss and reference hydraulic calculations demonstrating capacity and/or adequacy of existing and proposed facilities*
   d. *Provide sample calculations, tables, charts, etc. as necessary to support the calculations and results discussed above*
   e. *Reference computer software, documents, circulars, manuals, etc. used to produce the calculations and results discussed above*

6-13(B)(9) CONCLUSION

1. Summary of proposed drainage management strategy
2. Justification of rationale for discharge of developed runoff from site
3. Summary of proposed drainage improvements
4. Identification of DPM design variances being requested
5. Identification of required Drainage Covenants
6. Identification of ownership, operation and maintenance responsibilities

6-13(C) GRADING PLAN CHECKLIST

The following checklist is intended only as a guide for preparing a Grading Plan to accompany a drainage report or plan. Some items may not be applicable to your particular project; some items may require more detail. A pre-design conference is recommended to define scope and project specific requirements.

6-13(C)(1) GENERAL INFORMATION

1. Professional Engineer's stamp with signature and date.
2. Drafting Standards: (Reference City Standards, DPM V. 2, Chapter 27).
   a. *North Arrow*
   b. *Scales - recommended engineer scales:*
      i 1" = 20' for sites less than 5 acres
      ii 1" = 50' for sites 5 acres or more
   c. *Legend - see D.P.M. Manual, Volume 2, Tables 27.3a - 27.3d for recommended standard symbols*
   d. *Plan drawings size: 24" x 36"*
   e. *Notes defining property line, asphalt paving, sidewalks, planting areas, ponding areas, project limits, and all other areas whose definition would increase clarity*
3. Vicinity Map
4. Benchmark - location, description and elevation
   a. *Albuquerque control survey vertical datum*
   b. *Permanently marked temporary benchmark on or very near site*
5. Flood Hazard Boundary Map (FHBM) or Flood Insurance Rate Map (FIRM)
6. Legal Description

6-13(C)(2) EXISTING CONDITIONS

6-13(C)(2)(I) ON-SITE

1. Existing Contours - vertical intervals for contour maps shall not exceed the following:
   a. *One foot intervals for slopes under 1% with sufficient spot elevations at key points to adequately show the site's topography*
   b. *Two feet for slopes between 1% and 5%*
   c. *Five feet for slopes in excess of 5%*
2. Spot elevations adequately showing conditions on-site.
3. Contours and spot elevations extending a minimum of 25' beyond property line.
4. Identification of all existing structures located on-site or on adjacent property extending a minimum of 25' beyond property line with particular attention to retaining and garden walls.
5. Identification of all existing drainage facilities located on-site or on adjacent property.
6. Pertinent elevation(s) of structures and facilities defined in A, B and C above with NGVD 29 designation. NGVD 29 is the vertical system on which ACS monuments are currently based. In the future, ACS monuments should be field converted to NAVD 88 at which time NAVD shall become "equivalent".
7. Indication of all existing easements and rights-of-way on or adjacent to the site with dimensions and purpose shown.

8. Existing City top of curb and flow line elevations with NGVD 29 designation, or equivalent.
9. The location of Special Flood Hazard Area Boundaries from the latest FEMA maps must be overlaid on the existing site map (enlarged to site plan scale), when applicable.
10. The topographic survey must be performed by a professional surveyor in accordance with the "New Mexico Engineering and Surveying Practice Act" as amended and any standards adopted by the State Board of Registration.

6-13(C)(2)(II) OFF-SITE

1. Contributing Area - delineation of off-site contributing watersheds and/or drainage basins on City of Albuquerque Ortho-Topo Area Maps or equivalent mapping at a preferable scale of 1" =200' or 1" = 500'. Watershed and Basin designations shall match those used in the hydrology calculations.
2. Existing easements and rights-of-way including ownership and purpose.

6-13(C)(3) PROPOSED CONDITIONS

6-13(C)(3)(I) ON-SITE

1. Proposed improvements superimposed onto the existing conditions,
2. Proposed Grades. Proposed grades shall be adequately depicted by contours and/or spot elevations conforming with the following minimum criteria:
   a. *Contours - vertical intervals for contour maps shall not exceed the following:*
      i One foot intervals for slopes under 1% (with supplemental spot elevations as appropriate to adequately illustrate the proposed grading of the site).
      ii Two feet for slopes between 1% and 5%.
      iii Five feet for slopes in excess of 5%.
   b. *Spot Elevations - supply spot elevations at the following:*
      i Key points and grade breaks
      ii Critical locations
      iii Pad elevations
3. Indication of all proposed easements and rights-of-way on or adjacent to the site with dimensions and purpose identified.
4. City Engineer approved street and/or alley grades when site abuts a dedicated unpaved street or alley. In the event that approved grades are not available, provide preliminary street and/or alley grades.
5. Internal contributory drainage areas, including roof areas, outlined on plan.
6. Flow lines defined by arrows and spot elevations with NGVD 29 designation, or equivalent, as appropriate for clarity.
7. Pond(s) 100 year water surface elevation outlined and indicated on plan.
8. Finish building floor elevation(s) or pad elevation(s)with complete NGVD 29 designation, or equivalent, when applicable.
9. Elevations along property lines including relationship to adjacent top of curb.
10. Details of ponds, inverts, rundowns, curb cuts, water blocks, emergency spillways, retaining walls, pond outlets, safety fences, slopes, and all oth-

er significant drainage structures with contours, cross-sections and spot elevations. All cross-sections must be drawn to a standard engineering scale and adequately dimensioned.
11. Phasing,
12. Proposed construction of private storm drain improvements within public right-of-way and/or easement including identification of the public entity having ownership.
13. Proposed contours superimposed over existing contours adequately demonstrating changes in grade especially at the property line
14. Identification of any required offsite grading
15. Specifications for the proposed grading and/or soil compaction

6-13(C)(3)(II) OFF-SITE

1. Definition, location, and configuration of required drainage facilities.
2. Rights-of-way and easements needed to accommodate (A) above.

6-13(D) EROSION AND SEDIMENT CONTROL PLAN CHECKLIST

Use this checklist to prepare an Erosion and Sediment Control (ESC) Plan. There are three types of approvals for an ESC Plan; ESC Permit for grading, ESC Permit for Building Permit, and Work Order Construction plans. A stormwater Quality Information Sheet is to be submitted with each ESC plan submittal.

1. Checklist for ESC plans to obtain an ESC Permit for Grading:
    a. *Site boundary.*
    b. *Disturbed area boundary*
    c. *Vicinity Map*
    d. *New Mexico Professional Engineer stamp and seal.*
    e. *Sediment barrier BMPs*
    f. *Erosion control BMPs*
    g. *Inlet protection*
    h. *Stabilized Construction entrance or exit (not located at drainage outfall unless there is no alternative due to site constraints)*
    i. *Sediment pond/berm for sites larger than 5 acres or steeper than 8%. The pond is to be sized to function for 1 inch of rainfall or less.*
    j. *BMP installation details.*
    k. *Stabilization of tie-slopes and areas that will not be hard-scaped or landscaped within 14 days, excluding building pads.*
    l. *If a project is to be phased, show phasing and applicable BMPs/per phase.*
2. Checklist for ESC plans to obtain an ESC Permit for Building Permit approval:
    a. *Items listed in section A above.*
    b. *Construction Notes:*
        i When doing work in the City ROW (E.g. sidewalk, drive pads, utilities, etc...) prevent dirt from getting into the street. If dirt is present in the street, the street should be swept daily or prior to a rain event or contractor induced water event (e.g. curb cut, water test).
        ii When installing utilities behind the curb, the excavated dirt should not be placed in the street.

iii When cutting the street for utilities include a note that the dirt shall be placed on the uphill side of the street cut and the area swept after the work is complete. A wattle or mulch sock may be placed at the toe of the excavated dirt pile if site constraints do not allow placing the excavated dirt on the uphill side of the street cut.

3. Checklist for ESC plans to be included in Work Order Construction Plans:
   a. *Items listed in section A, above.*
   b. *Plan to show longitudinal street slope and street names.*
   c. *On streets where the longitudinal slope is steeper than 2.5%, wattles/ mulch socks or j-hood silt fence shall be shown in the front yard swale or on the side of the street.*
   d. *Applicable notes from Section B.2, above.*

6-13(E) CONCEPTUAL GRADING AND DRAINAGE PLAN SUBMITTAL CRITERIA

Conceptual Grading and drainage plans require less information than presented earlier in this section as they are not for construction and their function is to check the compatibility of the proposed development.

The following criteria are minimum requirements for this type of submittal. Downstream capacity and how determined.

1. Offsite flows should be quantified if they are significant (greater than 5 cfs).
2. Flood zone status- If the site is in a flood zone, the engineer is to provide enough information on how the project will meet the requirements of the National flood Insurance Program and the Flood Hazard Control Ordinance.
3. Existing and proposed topography on and adjacent to the site.
4. Provide developed flows and volumes.
5. Provide stormwater quality volume to be managed.
6. Plans are to be stamped and clearly identified “Preliminary - Not For Construction.”
7. If Public drainage infrastructure is required, information must be included to allow the City Engineer to evaluate the infrastructure list.

6-13(F) ENGINEER’S CERTIFICATION FOR NON-SUBDIVISIONS

Use this checklist when certifying compliance with an approved drainage report or drainage plan for public, commercial and multi-residential buildings requiring a Certificate of Occupancy building permit or grading and paving projects. Engineer must revise the original drawing as approved with the following information which shall serve as minimum criteria for evaluation. This is merely a guide. The level of detail necessary for presentation and verification is a function of the specific plan being evaluated. The engineer’s certification must be approved prior to the release of the issuance of a Certificate of Occupancy, or acceptance (by the City) of the completed work.

1. Completed Information Sheet - see Information Sheet.
2. Provide as-built finished floor and/or pad
3. Provide as-built spot elevations on the property line and/or limits of phase development (points of significant grade changes) to demonstrate compliance with the approved drainage report or drainage plan.
4. Provide written acknowledgement of completed construction from the appropriate government agencies for construction within their right-of-ways and/or easements.
5. Outline the as-built drainage basin(s) (including roof areas) supported with sufficient spot elevations and roof drain locations.
6. Provide as-built elevations and dimensions for the following structures:
    a. *Pond(s) (include as-built volume calculations)*
    b. *Pipe inlet(s) and outlet(s) (include as-built capacity calculations)*
    c. *Rundown(s) (including the required inlet dimensions)*
    d. *Spillway(s) (including the required outlet dimensions)*
    e. *Channel(s)*
    f. *Flowlines*
    g. *Erosion control and stormwater pollution prevention structure(s)*
    h. *Temporary drainage, erosion control and stormwater pollution prevention facilities required for phased development*
    i. *Retaining and/or garden wall(s)*
    j. *Other features critical to the drainage scheme.*
7. Professional Certification
    a. *Engineer's stamp dated and signed accompanied with a statement indicating substantial compliance with the approved drainage report and/or deficiencies with recommended corrections.*
    b. *The surveying associated with the certification must be performed by a professional engineer and/or surveyor in accordance with the "New Mexico Engineering and Surveying Practice Act" as amended and any standards adopted by the State Board of Registration.*

6-13(G) ENGINEER'S CERTIFICATION FOR SUBDIVISIONS

Use this checklist when certifying compliance with an approved drainage report or drainage plan for subdivisions when required by the Development Review Board (DRB) for the release of financial guarantees associated with an executed Subdivision Improvement Agreement (SIA). Engineer must revise the DRB approved drawing with the following information, which shall serve as minimum criteria for evaluation. This is merely a guide. The level of detail necessary for presentation and verification is a function of the specific plan being evaluated. The engineer's certification must be approved prior to the release of the SIA and/or financial guarantees.

1. Completed Information Sheet - see Information Sheet.
2. As-Built Information:
    a. *Pad elevations*
    b. *Top of Curb Elevations at critical locations*
    c. *Property corner elevations at each lot*
    d. *Horizontal and vertical data for storm drains (public and private)*
    e. *Horizontal and vertical data for retaining walls*
3. As-Built Analysis
    a. *Statement and verification that all grades inside the subdivision do not deviate by more than 18" of the DRB approved grades within 50 feet of the subdivision's perimeter.*

   b. *Statement and verification of street, storm drain and channel hydraulic capacities.*
   c. *Statement and verification of pond capacities.*
   d. *Statement of as-built elevation tolerances with respect to the feature being analyzed.*
4. Provide written acknowledgement of completed construction from the appropriate government agencies for construction within their right-of-ways and/or easements.
5. Clearly State the origin and Date(s) of As-Built Data
6. Supplemental Information
   a. *Provide details as necessary to illustrate as-built conditions for instances in which the as-constructed work materially deviates from the as approved design.*
   b. *Provide calculations to demonstrate and/or verify that all deviations satisfy the intent of the approved design.*
7. Professional Certification
   a. *Engineer's stamp dated and signed accompanied with a statement indicating substantial compliance with the approved drainage report and/or deficiencies with recommended corrections.*
   b. *The surveying associated with the certification must be performed by a professional engineer and/or surveyor in accordance with the "New Mexico Engineering and Surveying Practice Act" as amended and any standards adopted by the State Board of Registration.*

6-13(H) REQUIRED CERTIFICATION LANGUAGE

The following text shall appear on all Engineer Certifications.

DRAINAGE CERTIFICATION

I, ______________, NMPE ___, OF THE FIRM ______________, HEREBY CERTIFY THAT THIS PROJECT HAS BEEN GRADED AND WILL DRAIN IN SUBSTANTIAL COMPLIANCE WITH AND IN ACCORDANCE WITH THE DESIGN INTENT OF THE APPROVED PLAN DATED ______. THE RECORD INFORMATION EDITED ONTO THE ORIGINAL DESIGN DOCUMENT HAS BEEN OBTAINED BY ______________, NMPS ___, OF THE FIRM ______________. I FURTHER CERTIFY THAT I HAVE PERSONALLY VISITED THE PROJECT SITE ON ______ AND HAVE DETERMINED BY VISUAL INSPECTION THAT THE SURVEY DATA PROVIDED IS REPRESENTATIVE OF ACTUAL SITE CONDITIONS AND IS TRUE AND CORRECT TO THE BEST OF MY KNOWLEDGE AND BELIEF. THIS CERTIFICATION IS SUBMITTED IN SUPPORT OF A REQUEST FOR _________________________.
(DESCRIBE ANY EXCEPTIONS AND/OR QUALIFICATIONS HERE IN A SEPARATE PARAGRAPH)
(DESCRIBE ANY DEFICIENCIES AND/OR CORRECTIONS REQUIRED HERE IN A SEPARATE PARAGRAPH)
THE RECORD INFORMATION PRESENTED HEREON IS NOT NECESSARILY COMPLETE AND INTENDED ONLY TO VERIFY SUBSTANTIAL COMPLIANCE OF THE GRADING AND DRAINAGE ASPECTS OF THIS PROJECT. THOSE RELYING ON THIS RECORD DOCUMENT ARE ADVISED TO OBTAIN INDEPENDENT VERIFICATION OF ITS ACCURACY BEFORE USING IT FOR ANY OTHER PURPOSE.

____
XXXXXXXXXXXXXXX, NMPE XXXX
(SEAL)

____
DATE

6-14 MAINTENANCE AND POST-CONSTRUCTION RESPONSIBILITY

All drainage control, flood control and erosion control facilities, both public and private, shall be regularly maintained. Accumulations of silt, trash, litter or stagnant water which create a health or safety hazard or which endanger the design function of the facility are not permitted. Excessive growth or accumulation of woody vegetation in channels and on dams and levees shall not be permitted. Active erosion due to wind or water associated with drainage control, flood control and erosion control facilities shall not be permitted. The City of Albuquerque may conduct inspections to ensure compliance with the City's Drainage Ordinance, Stormwater Quality Ordinance and the EPA MS4 Permit.

All newly constructed drainage facilities within a public right-of-way must be blocked off at both ends to prevent unauthorized vehicular access with City Standard Tube Gate or removable bollards.

6-15 COMMON EQUATIONS

The most commonly used equations in drainage submittals are: weir, orifice and Manning's. They are presented below.

6-15(A) WEIRS

A weir is a barrier in an open channel, over which water flows. A weir with a sharp upstream corner or edge such that the water springs clear of the crest is a "sharp crested weir". All other weirs are classified as "weirs not sharp crested". Weirs are to be evaluated using the following equation:

EQUATION 6.60 $Q = CLH^{3/2}$

where:
Q = Discharge in cfs
C = Discharge coefficient use 2.7. If a discharge coefficient other than 2.7 is to be used, provide justification in drainage submittal.
L = Effective length of crest in feet
H = Depth of flow above elevation of crest in feet (approach velocity shall bedisregarded in most applications)

Weirs are generally used as measuring and hydraulic control devices. Emergency spillways in which critical depth occurs and overflow-type roadway crossings of channels are the most common applications of weirs. Channel drop structures and certain storm drain inlets may also be analyzed as weirs. Special care must be exercised when selecting weir coefficients in the following cases:

1. Submerged weirs
2. Broad crested weirs
3. Weirs with obstructions (i.e., guardrails, piers, etc.)

6-15(B) ORIFICES

An orifice is a submerged opening with a closed perimeter through which water flows. Orifices are analyzed using the following equation:

EQUATION 6.61 $Q = CA\,(2gh)^{1/2}$

when:
Q = Discharge in cfs
C = Discharge coefficient use 0.6. If a discharge coefficient other than 0.6 is to be used, provide justification in the drainage submittal.
A = Area of opening in square feet
g = 32.2 ft/sec^2
h = Depth of water measured from the center of the opening

Approach velocity shall be disregarded in most applications.

Orifices are generally used as measuring and hydraulic control devices. Orifice hydraulics control the function of many "submerged inlet - free outlet" culverts, primary spillways in detention facilities, manholes in conduit flow, and in storm drain catch basins.

6-15(C) MANNING'S EQUATION AND COEFFICIENT

Manning's equation is used to calculate flow, due to gravity, in open channels and conduits. In a conduit, the HGL must be below the soffit. As the Manning's Roughness Coefficient value increases the velocity decreases and the HGL increases. The equation is presented below:

EQUATION 6.62 **Q** = $(1.486AR^{2/3}S^{0.5})/n$

where:
Q - Flow Rate in Cubic Feet per Second
A - Flow Area
R - Hydraulic Radius; R=A/P where A is the flow area and P is the wetted (flow) perimeter
S - Slope
n - Manning's Roughness Coefficient (values to be used in drainage submittals shown below)

TABLE 6.19 VALUES OF MANNING'S "N"

Material	n
Plastic Pipe-Smooth Bore	0.010
Reinforced Concrete Pipe	0.013
Poured Concrete	0.013
No-Joint Cast In Place Concrete Pipe	0.014
Reinforced Concrete Box	0.015
Reinforced Concrete Arch	0.015
Streets	0.017
Flush Grouted Rip-Rap	0.020
Corrugated Metal Pipe	0.025
Grass Lined Channels (Sodded & Irrigated)	0.025
Earth Lined Channels (Smooth)	0.030
Arroyo Channels	0.030
Wire Tied Rip-Rap	0.040
Medium Weight Dumped Riprap	0..045
Grouted Rip-Rap (Exposed Rock)	0.045
Arroyo Overbank	0.045
Jetty Type Rip-Rap ($D_{50 > 24"}$)	0.050

6-16 HISTORY

In August of 2015, two technical subcommittees were convened to update this chapter. One subcommittee was convened to evaluate a new hydrologic model, evaluate hydraulic models and revise the closed conduit and open channel sections of this chapter. The current hydrologic model, AHYMO, was not replaced as the subcommittee decided that further study was required.

Members of this subcommittee are listed below:

Curtis Cherne, PE, CFM
Technical Subcommittee Chair
City of Albuquerque

Daniel Aguirre	Wilson and Company
Rick Beltramo	Galway Construction
Alandren Etlantus	Bohannan Huston Incorporated
Andreas Sanchez	SSAFCA
Gerhard Schoener	SSAFCA
Stephen Scissons	Army Corp of Engineers
Brad Bingham	AMAFCA
Shahab Biazar	City Engineer
Brian Patterson	Titan Development
Rita Harmon	City of Albuquerque
Charles Easterling	Easterling and Associates
Kevin Daggett	City of Albuquerque
Dave Thompson	Thompson Engineering Associates
Don Briggs	Bernalillo County
Hugh Floyd	RESPEC
Pat Stovall	Smith Engineering
Vince Carrica	Tierra West

The second subcommittee convened to evaluate all other sections of the chapter. The chapter was reorganized for easier use and was structured with the approach to help the development community with site development. Some of the larger changes are:

1. Addition of Floodplain Development
2. Addition of Valley Drainage Criteria
3. Emphasis on Downstream Capacity and Offsite Flows
4. Incorporation of erosion control specifications for pipes outletting into ponds and arroyos from "Urban Storm Drainage Criteria Manual Volume 2" from the Urban Drainage and Flood Control District, Denver, Colorado, June 2001, revised April 2008.
5. Addition of Low Impact Development
6. Removal of Probable Maximum Flood/Precipitation and Dam Design

Members of this subcommittee are listed below:

Curtis Cherne, PE, CFM
Technical Subcommittee Chair
City of Albuquerque

Don Briggs	Bernalillo County
Abiel Carrillo	City of Albuquerque
Kevin Daggett	City of Albuquerque
Scott Steffen	Bohannan Huston Incorporated
Ron Hensley	The Group
Diane Hoelzer	Mark Goodwin and Associates
Jeff Mortensen	High Mesa Consulting Group
Graeme Means	High Mesa Consulting Group
Brian Patterson	Titan Development
Kevin Patton	Pulte Homes
David Soule	Rio Grande Engineering
Jeff Wooten	Wooten Engineering
Rita Harmon	City of Albuquerque

The DPM Technical Subcommittee would like to dedicate this revision to Jeff Mortensen P.E., who sadly passed away during the revising of this manual. Jeff was very knowledgeable in all aspects of drainage and he was involved with the creation of Chapter 22 and every revision since its inception.

February 2015, the DPM revision was approved to incorporate requirements from the EPA MS4 Permit for post-construction development and infiltration was acknowledged in the design of ponds.

Section 22.2, Hydrology was first published in March, 1982, as one of the sections in the three-volume Development Process Manual (DPM). The DPM is the result of the effort of a special team of City of Albuquerque staff and Albuquerque Urban Advisory Council members. The Manual was created in response to mutual needs of the private and public sectors in Albuquerque to clarify the development process. The Three volumes of the DPM are: 1 - “Procedures”, 2 - “Design Criteria”. The Third Volume “Policies and Plans” is obsolete.

A major revision to Section 22.2 was adopted with the approval of a “Notice of Emergency Rule” by the City in January, 1986. This revision deleted a procedure which based rational method “C” coefficients on SCS Hydrologic Soil Group, and adopted Rational Method Coefficients based on textbook and handbook references.

The “D.P.M. Subcommittee on Drainage” was established by the City of Albuquerque in January, 1987. The Subcommittee held its first meeting in February, 1987. The Subcommittee consisted of members from City staff, Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) staff and local engineering consultants, and was organized to update and revise the DPM design criteria for Section 22.2, Hydrology. The Bernalillo County Public Works Department later joined the Subcommittee. In January, 1990 the subcommittee changed its name to the DPM “Drainage Design Criteria Committee” to avoid potential confusion with another committee established by the DPM Steering Group.

The Drainage Design Criteria Committee has met on a regular basis to develop a major update of the hydrology section of the DPM. In 1987, a research study to determine local infiltration factors was conducted by Dr. Richard Heggen at the University of New Mexico to supplement the work of the Committee.

A “draft” of the “Revision of Section 22.2, DPM” was distributed for community review in January, 1990. This document recommended use of initial abstraction and uniform infiltration to complete rainfall loss. It also included a procedure for smaller basins based on the Rational Method, and a procedure for large and small watersheds based on the HYMO computer program.

With the adoption of the Bernalillo County Storm Drainage Ordinance (No. 90-6) the County Engineer was responsible for establishing criteria, procedures and standards for the design of flood control, drainage controls, and erosion control improvements. To fulfill this requirement, Bernalillo County adopted “Interim Drainage Design Criteria for Bernalillo County” (April, 1990). This document incorporated Parts A, B, E and F from the January, 1990 draft of Section 22.2, Hydrology.

In January, 1991, a revision of “Section, 22.2,” was distributed to eight (8) Federal and State agencies, and to 26 local engineering firms. A public “Notice of Review” was published in the Albuquerque Journal and Tribune on February 4, 1991. Following incorporation of review comments, the August, 1991 version of Section 22.2, Hydrology was released for use by the Drainage Design Criteria Committee. This version included the placement of the rainfall peak in this second hour of the design storm. Modifications to the Probable Maximum Flood procedures incorporated a “local storm” and a “general storm.” A “Notice of Second Review” was published in the Albuquerque Journal and Tribune on August 31, 1991. The August, 1991 version has been accepted by the City, County and AMAFCA as an allowable procedure for hydrologic analysis and design of flood control structures.

The January, 1993 version of Section 22.2, Hydrology incorporates comments received since August, 1991. The version includes a procedure to evaluate basin hydrology for steep natural slopes, and some text revisions suggested by the USDA Soil Conservation Service. For most applications, there will be no computational differences between the January, 1993 version and the August, 1991 version. The text has been reformatted into seven (7) separately numbered parts to simplify future revision of the document.

The pages which follow replaced all previous pages in the Hydrology Section of the DPM (Section 22.2, pages 2 through 21). Following a public review and comment period, the revised Section 22.2, Hydrology was approved by the City Engineer and the Mayor. In the City of Albuquerque, the revision became effective on April 7, 1993. Bernalillo County also adopted the revision as the standard for design of flood and drainage control, effective April 7, 1993. The revised Section 22.2, Hydrology is to be regarded as the principal reference for hydrologic design in the City of Albuquerque and Bernalillo County.

The Drainage Design Criteria Committee wish to acknowledge the assistance of the many individuals who reviewed the document. In particular we wish to thank Richard Leonard, Brian Burnett and Dwayne Sheppard for their work on the Committee.

The D.P.M. Drainage Design Criteria Committee:

Richard J. Heggen, PE, PH, PhD
Professor of Civil Engineering
University of New Mexico

Howard C Stone, PE
Water Resources Manager
Bohannan-Huston Inc.

Clifford E. Anderson, PE & PS
Drainage Engineer, AMAFCA

Charles M. Easterling, PE
Pres., Easterling & Assoc.

Robert S. Foglesong, PE & PS
Surface Water Hydrologist
Bernalillo County Public Works

Fred Aguirre, PE
Hydrologist, PWD
City of Albuquerque

6-17 REFERENCE

The reference section remains unchanged, as the latest revision used committee members' experience. Their years of experience were a valuable resource. Their names are listed in the History section.

6-17(A) HYDRAULICS

6-17(A)(1) WEIRS AND ORIFICES

1. King and Brater: Handbook of Hydraulics, McGraw Hill Book Company, Inc., New York, Fifth Edition 1963
2. Merritt: Standard Handbook for Civil Engineers, McGraw Hill Book Company, Inc., New York, 1968
3. Streeter: Fluid Mechanics, McGraw Hill Book Company, Inc., New York, Fifth Edition

6-17(A)(2) CLOSED CONDUITS

1. Los Angeles County Flood Control District Design Manual - Hydraulic, P.O. Box 2418 Los Angeles, California 90054 Rev. 1973.

6-17(A)(3) CHANNELS

1. Chow: Open Channel Hydraulics, McGraw Hill Book Company, Inc., New York, 1959
2. U.S. Army Corps of Engineers:- Hydraulic Design of Flood Control Channels EM 1110-2-1601,. Office of the Chief of Engineers, Washington, D.C. 20314, 1970
3. Merritt: Standard Handbook for Civil Engineers, McGraw Hill Book Company, Inc., New York, 1968.
4. Morris and Wiggert: Applied Hydraulics in Engineering, the Ronald Press Company Second Edition, l972
5. U.S. Department of the Interior, Bureau of Reclamation: Hydraulic Design of Stilling Basins and Energy Dissipaters, U.S. Government Printing Office, Washington, Fourth Printing, Revised 1973
6. U.S.D.A Soil Conservation Service: Planning and Design of Open Channels, Technical Release No. 25, Washington, D.C., October, 1971
7. U.S.D.A Soil Conservation Service:. Sedimentation, National Engineering Handbook, Section 3, Chapter 4, Washington, D.C., 1971
8. Simons, Li and Associates: Design Guidelines and Criteria - Channels and Hydraulic Structures on Sandy Soil, P.O. Box 1816 Ft. Collins, Colorado, 80522, 1981
9. Los Angeles County Flood Control Authority, Design Manual Hydraulic P.O. Box 2418 Los Angeles, California 90054 Rev. 1973.
10. Albuquerque Metropolitan Arroyo Flood Control Authority Draft Design guide for Trapezoidal Concrete Flood Control Channels, Rev. April, 1982.

6-17(A)(4) CATCH BASINS

1. Los Angeles County Flood Control Authority, Design Manual - Hydraulic P.O. Box 2418 Los Angeles, California 90054 Rev. 1972.

6-17(A)(5) STREET HYDRAULICS

1. See Reference 6-17(A)(3) 1
2. See Reference 6-17(A)(3) 4

6-17(A)(6) BERMS AND LEVEES

1. See Reference 6-17(A)(3) 6
2. See Reference 6-17(A)(3) 7
3. See Reference 6-17(A)(3) 8