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HYDROLOGIC DESIGN (G 200)

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G 200 HYDROLOGIC DESIGN

The term hydrology, as used in the design of storm drains, is the study and application of past rainfall records and runoff data to a method of predicting future rainfall and runoff. There are three methods of calculating runoff in official use by the City of Los Angeles. They are the Peak Rate Method, the Method of Summing Hydrographs, and the Method for Undeveloped Mountain Areas. All three methods are given herein.

The most commonly used method is the Peak Rate Method. Many years of research, experiment, and analysis were required in its development. Involved in the study were the determination of the rainfall curves for various runoff frequencies and locations, storm patterns, reduction in rainfall intensity with the increase in the size of the drainage area, degree of soil saturation, amount of infiltration, quantity of runoff held in temporary storage or detention, rate of flow across various surfaces, and the basic features of flow in gutters and conduits. From these experiments and analysis were developed the data which were incorporated in the Peak Rate Method.

The theory and development of this method is explained in A Method of Computing Urban Runoff, by Mr. W. I. Hicks, Civil Engineer, of the Storm Drain Design Division, and the discussions thereof in Transactions, American Society of Civil Engineers, Vol. No. 109, March, 1945, and in the paper Surface Runoff Determination from Rainfall Without Using Coefficients, by *Mr. W. W. Horner and Mr. S. W. Jens, Transactions No. 107 (1942), Page 1039 AS.C.E*. The Method of Summing Hydrographs and the Peak Rate Method are taken from the Storm Drain Design Division Office Standard No. 71, 1939 Runoff Instructions.

G 201 HYDROLOGICAL ABBREVIATIONS AND DEFINITIONS

Iso –Isohyetal--WA line representing points of equal rainfall intensity (inches per hour)

Frequency –Return Period--The average amount of time (in years) between occurrences of equal rainfall intensity.

***Hydrograph*-**A graph representing the amount or rate of rainfall or runoff with respect to time.

***Clock Time*-**The time (in minutes) on that portion of a typical 24-hour (1440 minute) runoff hydrograph which covers the period of intense runoff. Zero clock time of that period is equal to 1062 minutes after runoff has started.

***Distribution Factor*-**A factor, varying with the time of concentration, applied to the rainfall rate as the size of the drainage area increases.

A **-**Drainage Area (Acres)

A_e **–**Equivalent Area (Acres)-A theoretical area of 100% imperviousness which is equal in runoff yield to an actual area of less imperviousness.

t_C **-**Time of Concentration-The shortest time required for runoff to flow from the most distant upstream point of a drainage area to the point in question. For a storm drain system, the time of concentration includes the inlet time (t_i) and the flow time in the conduit (t_f).

I_d -The percentage of imperviousness of an area.

BPRR **-**Base Peak Runoff Rate-The peak rate of runoff from a 100% impervious area on a 133 Isohyetal during a 10-year frequency storm at a specified time of concentration (cfs per acre).

F_{RO} -Runoff Factor-A factor which reflects the rainfall intensity, area classification, and soil type.

q_C -Runoff per acre (cfs) or the runoff rate (inches per hour).

q_C = BPRR x F_{RO}

Q **-**Quantity of runoff flow (cfs)

$Q = q_C \times A_e$

t_f -Time of Flow-The time of flow of peak runoff from one point to another.

t_{CX} -Adjusted Time of Concentration- The computed time of concentration below a major junction point obtained by adjusting the combined peaks of two or more flows. It is used as the time of concentration in runoff computations downstream of the junction.

G 210 RAINFALL FREQUENCIES FROM RAIN GAGES

Rainfall is usually measured by automatic recording rain gages which make a continuous chart record of the weight of water collected and the duration of rainfall. With this data, the storm pattern and frequencies can be determined for a specific locality. The City of Los Angeles maintains these gages at key locations throughout the City. For rainfall data in areas not covered by City gages, refer to the LACFCD Biennial Report on Hydrologic Data, available at the Drainage Systems Engineering Division. The point values in the example shown are from a single gage. Determination from a series of gages, each evaluated as shown, is necessary to tell the storm pattern and frequencies.

G 211 METHOD

This method of determining rainfall frequencies from rain gages was developed by Mr. Floyd Js Doran, P.E., City of Los Angeles. The method consists of the following steps:

First, from the automatic rain gage graph (Figure G 212), the rainfall depth is converted for the desired time increments to rate in inches per hour as follows:

a. The largest 5-minute increment of rainfall depth is found and converted to rate in inches per hour by multiplying the factor by 60/5.

b. The largest 10-minute increment is found and converted by multiplying the factor by 60/10.

c. Other time periods are found and converted in a similar fashion.

Second, this rainfall rate is converted to the U.S.W.B. base rate by multiplying said rate by 1.33 isohyetal and dividing by the isohyetal value at the gage.

Third, the storm frequency is determined from Figure G 212A (U.S.W.B. at Los Angeles, 18971y944, Isohyetal = 1.33) as follows:

a. Enter Figure G 212A with the adjusted rate, proceed horizontally to the rainfall curve of the time increment being processed, and then vertically read % Value.

b. 100/S Value = Storm frequency in years.

G 212 SAMPLE PROBLEM

The 30 min., 60 min., 120 min., and 180 min. rainfall frequency is determined at Sta. 16-A, Upper Stone Canyon (Figure G 212), for the storm of January 5, 1959. Isohyetal at rain gage = 1.85.

Step 1. From Automatic Rain Gage Chart, the Maximum rainfall depth is read for largest:

30 min. increment		= 0.70
60	" "	= 1.00
120	" "	= 1.50
180	" "	= 2.25

Step 2. The maximum rainfall depths are converted to rate in inches per hour.

Time Increment	Rainfall Depth	Conversion Factor	Rate in Inches Per Hour
30 min	0.70	x 60/30	= 1.40
60 min.	1.00	x 60/60	= 1.00
120 min.	1.50	x 60/120	= 0.75
l80 min.	2.25	x 60/180	= 0.75

Step 3. The rate from 1.85 isohyetal is converted to U.S.W.B. 1.33 isohyetal base rate.

Rate from 1.85 lsohyetal		Conversion Faetor	Adjusted Rate (1.33 Isohyetal)
1.40	x	1.33/1.85	1.01
1.00	x	"	0.72
0.75	x	"	0.54
0.75	x	"	0 .54

Step 4. The storm frequencies are determined from Figure G 212A as follows:

a. 30 min. time increment; adjusted rate = 1.01. Enter Figure at rate = 1.01, proceed horizontally to 30 min. curve, thence vertically and read 609 Storm Frequency = 100/60 = 1.66 years.

b. 60 min. time increment; adjusted rate = 0.72. Enter Figure at rate = 0.72, proceed horizontally to 60 min. curve, thence vertically and read 45% Storm Frequency = 100/45 = 2.22 years.

c. 120 min. time increment; adjusted rate = 0.54. Enter Figure at rate = 0.54, proceed horizontally to 120 min. curve, thence vertically and read 4l% Storm Frequency = 100/41 = 2.44 years.

d. 180 min. time increment; adjusted rate = 0.54. Enter Figure at rate = 0.54, proceed horizontally to 180 min. curve, thence vertically and read 19% Storm Frequency = 100/19 = 5.27 years.

The storm frequency used is the highest one of those determined from the different time increments selected. As previously stated, the highest rainfall rate is chosen for each increment of time so the resulting frequency is the highest for that increment. The storm frequency determined must therefore be the highest for that particular storm whether the storm be of short duration at high intensity or long duration at low intensity. However, the time increment chosen on the graph must result with a rainfall rate (in./hr) within the range of that time increment curve as shown on the storm frequency chart.

A Rainfall vs. Duration graph included in Figure G 212 may also be used to determine storm frequency.

G 220 GENERAL CRITERIA

Storm drain facilities shall be designed in accordance with the policies and practices stated in Sections G 110 and G 120. The design frequencies shall be chosen as indicated in Section G 222. The various storm frequencies are proportional to the 50-year storm as shown on the Isohyetal Map (Figure G 241.1A). Land development used in runoff determination shall be based on the Master Plan of Zoning.

G 221 PATTERN STORM

The pattern storm (illustrated on Figure G 221) utilized in the Los Angeles area is a 24-hour rainfall producing 6 inches of rain. This storm is preceded by 3 days of rainfall whose peak rates are 10%, 40%, and 35% of the above peak rainfall rate. The pattern storm was developed from many rainfall records by placing the 5-minute period of highest intensity at a common time-90 minutes clock time (or 1152 minutes elapsed time). Eighty percent of the total rainfall time of the storm occurs before the period of highest intensity. (80% x 24 hrs. x 60 min. = 1152 min.).

G 222 DESIGN FREQUENCIES

In general, the frequencies selected for storm drain design in areas zoned for urban residential or more impervious types of improvement are as follows:

1. A 10-year storm frequency for areas without sumps.
2. A 50-year storm frequency for sump areas.
3. A 10-year storm frequency for closed conduits in natural watercourses if the watercourse is maintained in place. The combined capacity of. watercourse and conduit must contain a storm of 50-year frequency.
4. A 10-year runoff frequency for open channels in natural watercourses with freeboard to contain a storm of 50-year frequency.
5. A 50-year storm frequency for any storm drain in a natural watercourse if the watercourse is eliminated.
6. Flow from a 50-year storm frequency shall not do any damage to private property, such as flows which overflow the curb on hillside streets.

Certain conditions require special consideration and should be discussed with the supervisor.

They are:

A. Combinations of storm frequencies in laterals and main lines.

B. Situations where overflows of a 10-year storm drain would cause excessive or serious damage during a 50-year storm. The overflow street capacity under these conditions should not be limited to the flow momentum requirements.

C. Situations where street overflow is not allowed onto private property, e.g. on the downhill side in hillside areas.

When new drains are to empty into existing drainage systems which have capacities less than that required for the frequencies indicated above, the design should be based upon the above criteria only if the existing drains are to be relieved in the near future. Otherwise, the new drains should be proportioned to the capacity of the existing drain. Under special conditions, sections of the new drain may be designed for standard frequency if upstream conditions (street grades, sumps, type of development, and other designing factors) of an existing drainage system so warrant. The water in excess of the capacity of the existing drain may be discharged onto the street surface at appropriate locations if care is taken to assure that no diversion of flow occurs. In those specific cases where a deviation from the standard design frequency is warranted, specific approval of the supervisor must be obtained.

G 230 SURFACE FLOW

The analysis of street flow required for tabling runoff mainly involves a block-by-block determination of velocity, time of concentration, and flow splits at intersections.

The analysis of flow in natural watercourses varies with its location. In undeveloped mountain areas, the flow can be determined by the Method for Undeveloped Mountain Areas (Section G 260). In flat terrain or developed hillside areas, the flow is computed by the Peak Rate Method (Section G 240), determining the channel shape, size, and slope from maps or surveys, and assuming an "n" value from those given in Section G 262.

G 231 FLOW DISTRIBUTION AT INTERSECTIONS

In routing street flow, the proper distribution of flow at intersections may be a major source of error where the flow split is a major factor in obtaining a runoff quantity for design. The bound charts and graphs Flow Distribution in Street Intersections, developed by the Hydraulic Laboratory for high flow values, should be used to determine flow splits. For flow quantities or street conditions different from those used in the charts, the designer must improvise a solution, using the charts as a guide.

Factors which affect flow splits at intersections are as follows:

a. the street width, pavement width, crown height, crossfall, and street grade,

b. the number, direction, and quantity of street flows into the intersection,

c. the type and angle of intersection: Y,T, +, etc.. and

d. the existence of cross-gutters, culverts, islands, driveways, etc. at the intersection.

When surface flow includes flow splits at intersections, the isohyetal for the entire contributing watershed area shall be averaged in lieu of the individual areas. as shown on Figure G 241.1C.

G 232 STREET FLOW

To table runoff for a storm drain, the depth and the velocity of street flow must be determined to compute the increment time of flow and quantity of flow. The flow is then computed as conduit flow when the conditions in Section G 111 are met. To determine catch basin size, the street flow must be analyzed independently of the main line flow. The charts from Office Standard No. 94 may be used for flow determination only for the exact street configuration shown thereon. Where the charts are not representative of the street cross-section, a Street Flow Computation must be made from survey data as shown on Figure G 232.

The charts in Office Standard No. 118 provide depth vs. Q values for present street width standards for slopes of 2% or over.

For the charts and computations in Office Standard No. 94, the street flow is assumed to be confined by vertical surfaces within the street property lines. For depths above the property line, the vertical plane of water at the property line is not included in the street wetted perimeter. This condition is assumed because obstructions along the property line constrict the flow, thereby increasing the depth.

G 240 PEAK RATE METHOD

The basic method of computing runoff used by the City is the Peak Rate Method. It is essentially the same as the rational method (Q = CIA) except that the peak rates of runoff are used rather than the average rates of rainfall intensity. The runoff equation for the Peak Rate Method is as follows:

$Q = BPRR \times F_{RO} \times A$

where Q = Runoff quantity (cfs)
BPRR = Base Peak Runoff Rate
F_{RO} = Runoff factor
A = Area (acres)

Numerically, the rainfall rate (inches per hour) and the runoff rate (cfs per acre) would be equal if there were no losses. These losses, infiltration, surface detention, and others (Figure G 221) are based on previous storms and are assumed constant. The Base Peak Runoff Rate and the Runoff Factor include all losses between the gross rainfall and the net runoff.

The Base Peak Runoff Rate is a function of the time of concentration. Base Runoff Hydrographs for times of concentration from 5 minutes to 120 minutes were developed from a study of the Mass Gross Rainfall Curve of the pattern storm and from experimental tests. The features of the pattern storm used to develop the BPRR are a 10-year frequency storm of 24-hour duration with an isohyetal of 1.33 inches per hour. The experimental tests consisted of infiltration studies on small test plots of open soils varying from sand to clay to determine the amount of runoff to be expected.

The Runoff Factor is a function of rainfall intensity, soil type, and type of development. Both the one-hour maximum rainfall rate and the relative one-hour maximum rainfall factor for the storm frequency selected are determined from the Isohyetal Map. When more than one isohyetal line passes through a drainage area, a weighted average rainfall rate must be computed. This, in turn, will change the F_{RO}. When the summation of the areas being tabled exceeds 100 acres, the rainfall rate must be modified by applying a Distribution Factor in all further downstream tabling.

The computation of runoff by the Peak Rate Method is accomplished in two steps. First, the drainage area map is prepared, and second, from this data, the peak rate of runoff is computed.

G 241 PREPARING DRAINAGE AREA MAP
To prepare a drainage area map (scale: 1" = 400'), an up-to-date print of a Drainage Map containing the area should be used. When no City drainage map exists because the watershed boundary is outside the City, the data from the County Surveyor's Maps on file in the County Engineer's Office may be used. Map data should be verified in the field and from County Street and Storm Drain plans and profiles. Watershed boundaries outside the City limits should be verified with the responsible agency.

G 241.1 Procedure: The procedure for the preparation of a drainage area map is as follows:

1. The drainage area is determined by outlining the watershed ridgeline on the map (Figure G 241.1). For trunk lines, finger-like projections at the head of the drainage area must be excluded.

2. The isohyetal lines (from Figure G 241.1A) are drawn on the map.

3. The zoning and types of soils are classified as shown on Figure G 241.2.

4. All existing storm drains and any appropriate proposed storm drains are shown on the map.

5. The drainage area is divided into local drainage areas called sub-areas. The sub-areas are planimetered and their acreage (Figure G 241.1B), soil, and zoning classifications are recorded on the map. The sizes of the sub-areas are controlled by the location of the tabling points along the alignment. To a large extent, the tabling points are selected by the points of inflow or outflow, usually at each intersection.

6. The sub-areas are colored for easy identification and numbered as a cross-reference to the tabling sheets (Figures G 241.1C and G 242).

7. The sub-area classification acreage can be converted to an equivalent 100% (or other predominant classification) impervious acreage (4) as shown in Subsection G 241.5 and is listed on the map.

8. The average isohyetal for sub-areas is determined as shown on Figure G 241.1C. (Also see last paragraph, Section G 231.)

9. The sub-areas contributing to a sump are converted to a larger area which will produce the same runoff under a 10-year frequency as the actual area would produce on a 50-year frequency as shown in Subsection G 241.5. The resultant Amended A_e is listed on the map.

G 241.2 Soil Types: The U.S.G.S. 2000-scale maps in each office show soil types as surveyed and classified by the U.S. Department of Agriculture. They show the boundaries of any given soil type except certain mountain areas where soil and rock formations occur erratically in small areas which are indicated as R-Rough broken land. Some of the soil names and classifications on said maps are shown on Figure G 241.2. Due to the differences in terminology, the numerical soil classification should always be used rather than the soil name.

G 241.3 Classification of Areas: Runoff varies considerably with differing types of development or land use because of the increased imperviousness in areas where more intensive use of the land is made. The Master Plan of Zoning must be used to determine the proposed land use or zoning of each drainage area. These areas are classified to reflect this differing development or land use in accordance with Figure G 241.3, Development Classification. The current summary of Zoning Regulations (Figures G 241.3A and B) will help in the classification.

Drainage area classification for development or land use (zoning) and soil type is indicated by use of the Id value and the numerical soil classification. For example, 40-2 indicates an area having a value of 40% imperviousness and a loam type of soil.

When storm drain construction is being planned for agricultural and mixed agricultural and urban areas, careful analysis should be made of probable trends of future development in the areas.

G 241.4 Steep Hillside or Mountainous Areas: Many of the steep hillside or mountainous areas of the City of Los Angeles as designated by Ordinance No. 129,885 and Figure G 241.4 are presently being, or have been, subdivided and developed. In order that adequate facilities may be provided to dispose of storm water runoff, the amount of runoff from an area must be computed for the ultimate or most intensive use that can be reasonably expected of the land. Also, it is often necessary to compute runoff from an area in its natural or undeveloped state in order to determine the increased runoff that may be expected from a proposed development. (See Section G 260.)

The following items must be considered in computing the runoff from a developed hillside or mountainous area:

1. The entire filled area of a building site (in lieu of the building pad only) is usually compacted to 90% relative compaction, which results in increased runoff rates. Figure G 241.3, Development Classifications, reflects these increased Id values. The ultimate land use to be expected from the area must be determined from up-to-date zoning maps.

2. Soil classifications in the mountainous areas are frequently listed on the soils maps as R - Rough broken land. A loam type soil (2) should be used in all mountainous areas unless the soil maps or tests show a definite classification for the area.

3. If a street pattern has not been completely laid out for the area, a pattern with maximum street slopes of 15% should be assumed.

4. Drainage areas which are in Parks or National Forests and which will probably not be subdivided in the foreseeable future generally should be considered as areas in their natural or undeveloped state. Runoff from these areas should be based on observable phenomena and the engineer's good judgment. A limited examination of runoff records from these areas with brush and forest cover shows a tendency of this cover to lower and broaden the normal sharp peak of runoff, thus resulting in a lower but more prolonged peak rate. Runoff from these areas, however, will be greater than that from park areas with dense grass cover and flat slope.

Some rainfall in steep hillside areas percolates into the soil to a depth of 6 to 8 inches, flows underground down the slopes, and reappears in depressed areas at the bottom of steep slopes and in cut slopes. This is known as quick subsurface return. Consideration of this phenomenon in newly graded hillside areas should be given in runoff calculations.

When the natural cover and litter of a mountainous area are removed by fire, the runoff is sharp and heavy and frequently laden with silt and debris. This debris may choke drains and leave deposits on streets and private property near the mouths of canyons and at the feet of steep hillside slopes. While the combination of a burn-off and a 50-year frequency rainfall is a rare occurrence, a burn-off followed by a rainfall of lesser frequency may produce the runoff equivalent to that of a 50-year or greater frequency rainfall.

Retarding basins may be required in order to retard peak flows, reduce peak runoff, and minimize downstream flooding. Debris basins may be built immediately after a burn-off to control excessive deposition of silt and debris.

G 241.5 Area Conversions: To reduce the number of individual computations involved when tabling large or complicated areas or even smaller areas on a combined frequency basis, acreages may be converted to any common basis which will produce the same quantity of runoff.

Generally, the most useful common acreage basis is 1009 imperviousness (A.). To convert the different area classifications to one of 100% imperviousness, the formula used is:

$$A_e = \frac{A\ (\text{Actual})\ \times\ F_{RO}\ (\text{class})}{F_{RO}\ (100\%\ \text{Imperviousness})}$$

In tabling runoff through a sump area, it is usually necessary to table the area tributary to the sump on a 50-year frequency basis, and the balance of the drainage area on a 10-year frequency basis. In combining the computed runoff from the sump with that of a 10-year frequency basis downstream of the sump, the area must be converted to an area yielding the same total runoff using the F_{RO} for a 10-year frequency. The formula for this conversion is:

$$A_{10} = A_{50} \times \frac{F_{RO50}\,(\text{class})}{F_{RO10}\,(\text{calss})}$$

where the F_{RO}'s are taken from Figures G 242.2F to K.

G 242 TABLING THE RUNOFF

After completion of the drainage area map, the quantity of runoff is determined by the use of the tabling sheet (Figure G 242) and the instructions given in Subsection G 242.2. The storm drain conduit is designed for peak flow, which varies with the time of concentration and the contributing area at any point. Knowing the contributing areas from the drainage area map, the time of concentration and the quantity of flow are determined by routing the flow on the surface and computing the runoff in the conduit

G 242.1 Routing the Flow: In general, the initial time of concentration, t_C, (the time of the peak runoff for all initial drainage areas) is determined from empirically derived charts (Inlet Time Charts). Using the F_{RO} and the initial t_C a quantity of runoff (Q) is determined. If the initial point of concentration is a street intersection, the distribution of flow in each street must be determined (as shown in Section G 231). Generally, the street having the largest outflow is then followed to the next intersection (or point of concentration) by calculating the flow velocity and thereby the time of flow. This increment of flow time is added to the initial t_c; all new tributary areas are added, and a second Q is determined (as shown in the tabling). Again the flow distribution at the intersection is determined. This procedure is repeated, point by point, along the street until a storm drain conduit is considered necessary.

In flat land, there may be parallel street flows in the initial area. The above procedure must be applied to each street flow for the entire area at the head of a drain.

When the street flow exceeds the conditions stated in Section G 111, the flow is considered as taken into an underground conduit, even iI the conduit is not built as part of the project. This provides the proper conduit size downstream to allow for a future extension. The tabling from this point on is calculated along the main line alignment predetermined for the storm drain system. The minimum pipe size for main line or lateral is 24". Laterals to the main line are considered necessary when catch basin connector pipes exceed 100 feet. The tributary areas for laterals are tabled as discussed in the first paragraph above.

G 242.2 Computing the Runoff: Runoff computations are made on a Runoff Tabling Sheet (Figure G 242). The runoff computation procedure is discussed in the item numbers below, and the sequence of tabling is illustrated by the same numbers on the figure. The nomograph (Figure G 242.2) is presented as a short-cut to the computations; The drainage area map shown on Figure G 241.1 is used as an example in this computation.

1. The location of the point of concentration, the sub-area numbers, their acreage (A), and their converted acreage (A.) tributary to this point are taken from the drainage area map.

2. The F_{RO} is determined as follows:

(a) The isohyetal for the sub-area is taken from the drainage area map, or, as in this case, the Average Isohyetal (1.55) is taken from Figure G 241.1C. It is reduced to the rate desired for design (10-year frequency) by applying the frequency factor (0.762) shown on Figure G 241.1A. This gives the Frequency 1-hour Rainfall Rate (RFR) of 1.18.

(b) When the sum of the areas (SA, Column 2) exceeds 100 acres, the Distribution Factor (in this case 1.00) is taken from Figure G 242.2E. Then the Frequency 1-hour RFR (1.18) x Distribution Factor (1.00) = Amended 1-hour RFR (1.18) .

(c) The F_{RO} is obtained from Figures G 242.2F to K for the specified area classification. The proper 50-year isohyetal value is followed until it intersects with the Amended 1-hour RFR vertical line. This point is extended horizontally to read the F_{RO}. In this case, since the area has been converted to a 100% impervious area (A_e), the 100% line intersects the Amended 1-hour RFR (1.18) at an F_{RO} value of 1.17. (Also see Figure G242.2Q).

3. The time of concentration (t_c) is equal to the initial time at the first point of concentration plus the travel time from point to point. The initial time to the first point of concentration (8 minutes) and the resulting BPRR (2.76) are determined as follows:

a. Determine the longest flow route to the first point of concentration (A Street and 3rd Street) and the average slope of the streets.

b. The initial t_c should be determined using the true classification of the initial block and not equivalent 100'S (A_e). Therefore, using the "F_{RO}" (1.03) for the area classification (70-2) and the average gutter slope (.012), enter the Initial or Inlet Time Chart (Figure G 242.2A) and find the t_c for the first 700-foot length (6.7 min.).

c. If the gutter length is other than 700 feet, adjust the initial t_c as follows:

$$t_c = t_{c_{700}} \times \left(\frac{\text{Actual length}}{700} \right)^{1/2}$$

$$= 6.7 \left(\frac{1000}{700} \right)^{1/2} = 8.0 \text{ min.}$$

The Base Peak Runoff Rate changes rapidly when the t_c is small. Therefore, the initial time for areas of short t_c's should be based on the average initial time of the first few sub-areas at the head of each trunk or lateral line.

When the upper end of a drainage area is slow concentrating and of small runoff productivity (such as park land or a long and narrow drainage corridor), and the major area downstream is fast concentrating and of high runoff productivity, the initial time should be adjusted between the two areas by taking a weighted average of the two areas as follows:

$$t_c = \frac{(t_{c_A} \times A_A) + (t_{c_B} \times A_B)}{A_A + A_B}$$

Possibly a peak rate higher than the weighted average may be obtained if the upper area is deleted. When a fast concentrating, high productivity area downstream is several times larger than a slow concentrating, low productivity area upstream, the initial t_c should be based on the downstream area only.

The above weighted average formula is to determine initial t_c only. For an adjusted t_c other than initial, the formula for t_{cx} in item 7 should be used.

d. With the initial t_c (8 minutes), the BPRR (2.76) is obtained from Figure G 242.2L. Even if the t_c is less than 5 minutes, the value of the BPRR must not exceed 3.40.

4. q = F_{RO} (1.17) x BPRR (2.76) = 3.23 cfs/acre.

Q = q (3.23) x A_e (6.4) = 20.6 cfs

For surface flow, the Q is listed in the column of direction of flow (N,S,E,W). For conduit flow, the Q is listed in the Storm Drain column. The Flow Routing columns are most useful to simplify the computations when flow is split (G 231) from the main runoff path. The amount contributing to the drain is marked (*) and added to the next area.

5. Items 1 to 4 above give the quantity and direction of runoff. Items 5 and 6 travel the flow to the next concentration point. The average street slope (S) to the next concentration point is determined. For street or channel flow, S (Figure G 242.2P) is used as the friction slope. For pipe or box flow, 80%S is used as the friction slope. 20% of the available head is allotted to other losses (manholes, junctions, etc.). (See Figure G 242.2N for a more accurate estimate of the conduit friction slope.) With a known street size (100') and pavement width (70'), the cross-sectional area and depth of flow are determined from the charts on Figure G 184A, Office Standard No. 94, or Office Standard No. 118, whichever applies.

For flow in pipe conduits, the conduit conveyance factor ($Q/S^{1/2}$) is calculated. A pipe size equal to or larger than the conveyance factor is selected from Figure G 242.2M, which also gives the pipe conveyance factor and the pipe area. Minimum pipe size for main line and lateral is 24".

6. Velocity = Q/A (4.2 ft./sec.). With a known distance (350') to the next point of concentration, the travel time is calculated as follows:

$$t_t = \frac{350}{4.2 \times 60} = 1.4 \text{ minutes}$$

The t_c at the next point of concentration is then 8 + 1.4 = 9.4 minutes.

7. Repeat above items (1 to 6) for each point of concentration and continue tabling to project outlet.

At station 4 + 00 (Line 16) a lateral joins the trunk line, and the time of concentration is adjusted by the weighted average method described below:

$$t_{cx} = \frac{(t_{C_A} \times Q_A) + (t_{C_B} \times Q_B)}{Q_A + Q_B}$$

here t_{cx} = t_c of Combined peak flows
t_{CA} = t_C of Trunk (longer time)
t_{CB} = t_C of Lateral (shorter time)
Q_A = Q of Trunk
Q_B = Q of Lateral

The values for the peak Q's and their t_C's, as shown in the formula above, are taken at the point of junction. The runoff for both the trunk line and the lateral must therefore be tabled to the point of junction. When the small Q at a junction is less than 10% of the large Q, the t_C of the large Q is used in lieu of the t_{CX}.

G 243 LIMITATIONS OF PEAK RATE METHOD

The Peak Rate Method of computing runoff is subject to the following limitations:

1. Theoretically, adjustment of the time of concentration at the junction of a main line and a lateral should be done by the method of Summing Hydrographs when the difference in the two times of concentration is greater than 60 minutes. In actual practice, however, these limitations are impractical because the difference in accuracy between the weighted average t_{CX} and the method of summing hydrographs is negligible in most cases. In addition, the method of summing hydrographs is very time-consuming.

2. When the t_{CX} exceeds 60 minutes, the engineer should also theoretically change to the method of summing hydrographs. However, the basic hydrograph is generally distorted to such a large extent by outflow streets, existing storm drains, and other conditions that the Peak Rate Method may be extended, at the discretion of the supervisor, beyond this limit. It generally gives results as acceptable as the method of Summing Hydrographs.

3. When the tabled runoff quantity enters a retarding reservoir, it is necessary to make a special calculation using hydrographs to determine the outflow from the reservoir.

G 250 METHOD OF SUMMING HYDROGRAPHS

The summing of hydrographs is a graphical solution of combining flows of two or more laterals and a trunk line at a junction point. The respective hydrographs of the inflows to the junction are plotted and combined into one hydrograph, which is adjusted to determine the peak Q and t_C outflowing from the junction. These values of peak Q and t,. leaving the junction are required to continue the tabling of runoff downstream of the junction.

Because this graphical solution is laborious and time-consuming, the Method of Summing Hydrographs should not be used if the Peak Rate Method can be applied. The Method of Summing Hydrographs is particularly difficult to apply when a City street system with all its flow splits and confluences is to be tabled. The designer should review the *Limitations of Peak Rate Method* (Section G 243), and should consult his supervisor before making use of this method.

G 25 l COMB I N I NG THE FLOWS

The application of the Method of Summing Hydrographs with additional background and theoretical information is briefly outlined in this section. Eight steps in series are used to complete the summation of flows at any one junction point. The flow between junction points and the results of the summed hydrographs at the junction point are tabled for the entire storm drain system to its outlet. A sample problem is shown to further explain and illustrate each step of this method in order that the designer may have a more thorough understanding of the procedures involved.

In this outline and in the sample problem, the hydrographs must be identified with the junction point and with the source of the flow. To this end, the following nomenclature is used. On Figure G 252C, hydrograph A is the hydrograph at point A (Figure G 252). Hydrograph B_1 is hydrograph A traveled to point B, and it is a balanced hydrograph. Hydrograph B_2 is the hydrograph of the lateral at point B. Hydrograph B2 is the summed hydrograph of B1 and B_2. Hydrograph B is the adjusted hydrograph of B2 which flows out of junction point B. The nomenclature for points C, D, etc., are similar to point B above.

STEP 1: Tabulate Runoff from Tributary Areas

Each area contributing to a junction point (A, B, or C on Figure G 252) is tabled individually by the Peak Rate Method or determined from previously summed hydrographs. These flows are tabulated only to the point of concentration immediately preceding the junction point. In most areas to be tabled, one of the flows may readily be determined to be a main line flow. However, if there are two approximately equal flows, one must be assumed to be the main line and the other a lateral. On Figure G 252, the runoff from the area tributary to point A is the main line flow, and the runoff from areas tributary to points B and C are lateral flows.

STEP 2: Plot Hydrograph A

The controlling points which determine the shape of the Hydrograph are those at 80%, 90%, and 100% of peak runoff. The clock times for these rates are determined from Time Elements of the Hydrograph (Figure G 252E). Additional points required to complete the hydrograph are calculated from the Table for Base Runoff Hydrographs (Figure G 252F) based on the time of concentration (t_C).

STEP 3: Calculate Main Line Time of Flow

Assuming a preliminary design of a closed conduit, open channel, or natural stream flow, calculate the time of flow (t_f) of the main line from point A to point B.

STEP 4: Reduce Peak Flow

The peak runoff rate is reduced for conduit detention as the main line flows from a point immediately preceding junction point A to junction point B. If the flow is in a non-pressure system, the Peak Reduction Factor for Conduit Detention (Figure G 252B) is applied. If the flow is in a pressure system, this factor is not applied, because the conduit is usually full before peak flow is reached.

The peak Q to point B is further reduced because of the smaller value of the Distribution Factor (Figure G 242.2E) for the increased total area tributary to point B. The ratio of the Distribution Factors may be applied directly to the Q of a tributary area to make this reduction. The distribution factor is normally used to reduce the intensity of rainfall over an area as the size of the area increases. Since the F_{RO} curves are, in effect, straight lines within the range of variations between the 1-hour rainfall rate and the amended 1-hour rainfall rate, the quantity of runoff varies directly as the Distribution Factor.

STEP 5: Transfer and Balance Hydrograph

The expression Transfer and Balance Hydrograph means transfer Hydrograph A (traveled from A to B) and balance the area of the Hydrographs. Hydrograph B1 is thus plotted and balanced so that its area under the curve is equal to the area under the curve of Hydrograph A (as shown on Figure G 252D).

Since the total volume of water in the main line leaving point A and entering point B is the same, the areas under the hydrographs (which represent volume) must be equal. However, the clock time of peak flow will be different due to the time of flow from point A to point B. Also, the peak rate will be lower due to the reductions made in step 4. The clock time at point B is determined by adding the time of flow (t_f) to the clock time of point A.

STEP 6: Plot Lateral Hydrograph B_L

The tabulated flow for each lateral entering the main line at point B is reduced by the ratio of the Distribution Factors (see step 4) of the combined areas to point B and the lateral area. Hydrograph B_L for the lateral is plotted as shown in step 2 at the clock time for the lateral time of concentration.

STEP 7: Combine Hydrographs

All the flows into a junction must be combined to determine the peak Q, clock time, and t_C required to continue tabling the flow downstream from the junction. The combination or sum of the main line hydrograph B_1 and the lateral hydrograph B_L results in hydrograph B_2 as shown on Figure G 252C. Since the area under each hydrograph represents a volume of water entering the junction, the area under the combined hydrograph must be equal to the sum of the areas under each hydrograph. The summed area then represents the volume of water leaving the junction.

STEP 8: Adjust the Hydrograph

The summed hydrograph (B_2) is always distorted. This usually results in a shortening of the duration of the high flows and consequently an increase in the value of the peak rate. This distortion must be adjusted to the standard hydrograph shape before the peak values of Q, clock time, and t_C are determined. However, the adjustment cannot be accomplished as was the balancing in step 5 (although the same principles apply), because the peak Q and clock time are not known.

The hydrograph (B_2) must be adjusted by trial and error; therefore, it is necessary to assume a peak Q, to plot a trial curve, and to compare the areas under the two hydrographs (B_2 and B). This assures that equal volume of flow is maintained. Although the shape of the adjusted hydrograph (B) will influence the area under the curve, minor variations in shape are not significant in determining the peak flow. If the adjusted peak flow were assumed to be 2100 or 2200 cfs (instead of 2150 cfs) in the following example, the error would be less than 2%%; this percentage is within the limits of accuracy in any runoff calculation.

G 252 SAMPLE PROBLEM

The application of the eight steps required in the Method of Summing Hydrographs is illustrated in this sample problem based on the Drainage Area Map shown on Figure G 252. Sample computations are made; hydrographs are combined and adjusted as illustrated in Figures G 252C and D; and the results are tabulated on a Tabling Sheet as shown on Figure G 252A.

STEP 1: *Tabulate Runoff from Tributary Areas A, B, and C*

The runoff from the area tributary to point "A" (700 Acres) is tabled by the Peak Rate Method. Point A is the first point on the main (or trunk) line shown on Figure G 252.

$$Q = \text{Area} \times F_{RO} \times \text{BPRR} \times \text{Distribution Factor}$$
$$Q = 700 \times 1.80 \times 1.37 \times 0.966 = 1668 \text{ cfs}$$
$$t_C \text{ at A} = 40 \text{ minutes}$$

The values from the tabling for this area are listed in columns 3 to 10, line 1 of Figure G 252A.

Flows from areas tributary to points B and C are also tabled to those points by the Peak Rate Method, but these flows are lateral flows to the trunk line at those points. Their values from the tabling are listed in columns 3 to 10, lines 4 and 7. Columns 1 and 2 are self-explanatory.

STEP 2: Plot Hydrograph A

The main line flow at point A is plotted as Hydrograph A (Figure G 252C). The points required to plot this curve are calculated for 80%, 90%, and 100% of peak runoff, and the clock times for these values are taken from Figure G 252E for the t_C of 40 minutes as follows:

100% peak runoff = 1668 cfs at 103.0 min.
90% peak runoff = 1501 cfs at 97.2 and 110.9 min.
80% peak runoff = 1334 cfs at 96.1 and 113.2 min.

These points control the shape of the curve at the peak of the hydrograph.

Additional points on the hydrograph are calculated by multiplying the Runoff Rate at Clock Time (for the t_C of 40 minutes from Figure G 252F) with the multiplying factor "MF" (Q/BPRR = 1668/1.37) as follows:

Clock Time			**R.O.R. @ C.T.**		**MF**		**Runoff Rate**
60	min.	-	0.40	x	1218	=	487 cfs
80	"	-	0.54	x	"	=	658 cfs
90	"	-	0.72	x	"	=	877 cfs
120	"	-	0.76	x	"	=	926 cfs
140	"	-	0.31	x	"	=	378 cfs
160	"	-	0.20	x	"	=	244 cfs

Reference to Hydrograph A is listed in column 23.

STEP 3: Calculate Main Line Time of Flow

Assuming an open channel (non-pressure system), the time of flow for the trunk line from point A to point B must be calculated from the known values of Q, flow area, and traveled distance. (Velocity = Q/A and Time = Distance/ Velocity.) The values are listed in columns 16 to 22 on line 2. The 2-minute time of flow is added to the t_C of 40 minutes, thus finding the trunk line t. (42 minutes) entering point B (column 5, line 3).

STEP 4: Reduce Peak Flow

The main line peak flow entering junction point B is reduced for conduit detention because the flow is in a non-pressure system. The peak Q is reduced for the 2-minute time of flow by the use of the table in Figure G 252B (column B). For a t_C of 40 minutes, 1.24%, and for 41 minutes, 1.21% (interpolated), gives a total reduction of 2.45%, which is listed as 0.9775 = (1.0-.0245) in column 9, line 3. Then Q = 1668 x .9755 = 1627 cfs, as listed in column 10. line 3.

The peak Q's for the trunk line and lateral at B are reduced because of the increase in tributary area by the ratio of Distribution Factors (1000 Acres = .954, 700 Acres = .966, and 300 Acres = .983) from Figure G 242.2E as follows:

$$\text{Trunk A: } 1627 \times \left(\frac{.954}{.966}\right) = 1607 \text{ cfs}$$

$$\text{Lateral B: } 654 \times \left(\frac{.954}{.983}\right) = 634 \text{ cfs}$$

These values are listed in columns 11 and 12, lines 3 and 4.

STEP 5: Transfer and Balance Hydrograph

Hydrograph A illustrates the peak flow (Q = 1668) at point A for a t_C of 40 minutes. Hydrograph B1 represents the same volume of flow as Hydrograph A, but traveled from point A to point B. This increases the t_C to 42 minutes. The transfer and balance of hydrographs is complete if Hydrograph B1 reflects the reduced peak flow (Q = 1607) because of the increased t_C , and if it is aligned to the proper peak clock time for t_C of 42 minutes, and if it has the same area (or volume) under the curve as Hydrograph A. This transfer and balance is accomplished as follows:

a. The clock time of peak flow for Hydrograph B1 (1607 cfs) is determined by adding the time of flow (t_f) to the clock time of peak flow for Hydrograph A.

103.0 min. + 2 min. = 105.0 minutes.

b. The clock time 105.0 is located on the 1009 curve on Figure G 252E (upper half), then down vertically to intersect the t_p and t_{pb} curves (lower half) to read the clock time for 90% to 90% peak (t_p = 15.5 min.) and for 90% to l00% peak (t_{pb} = 6.7 min.)

c. 90% of peak Q is calculated thus: 1607 x 90% = 1446 cfs.

d. A trial hydrograph B_1 is plotted using above Q's and clock times and other points as shown on Figure G 252D. Using a planimeter, the increments of area under Hydrographs A and B_l are checked and the values of t_p and t_{pb} and other points on Hydrograph B_1 are changed until the areas under the curves are substantially the same.

STEP 6: Plot Lateral Hydrograph B_L

The peak Q of lateral B_L reduced by the Distribution factor in step 4 is 634 cfs at a t_C of 25 minutes. Entering Figure G 252E with the t_C, the clock time for the peak flow is found (99.1 minutes). The curve values for Hydrograph B_L are determined and a hydrograph is plotted in the same manner as Hydrograph A in step 2.

STEP 7: Combine Hydrographs B_l and B_L

Hydrographs B1 and BL are combined by adding the runoff rate ordinates of the curve at their respective clock times. The resulting sums are then plotted as hydrograph B2, as illustrated in Figure G 252C. The combined peak Q is 2240 cfs at a clock time of 102 minutes.

STEP 8: Adjust Hydrograph B,

With the aid of Time Elements of the Hydrograph (Figure G 252E), the summed hydrograph (B2) is adjusted by trial and error. The clock time of the peak for the adjusted hydrograph is estimated by measuring the duration of time (16.5 minutes) between the points of 809 peak (2240 x 80% = 1792 cfs) of the distorted hydrograph (B2) This time is scaled vertically to match the two 80% peak curves on the upper portion of Figure

G 252E, and the clock time for 100% peak (102.6 minutes) is read. After examining the narrow peak of the distorted hydrograph (B_2) and knowing that the adjusted curve should approximate the standard shape (as indicated by the values shown on Figure G 252E), the peak Q is estimated slightly below 2200 cfs, say 2150 cfs. A trial curve for the adjusted hydrograph B is plotted between the points of 80% of the assumed peak rate (2150 x 809 = 1720 cfs). The curve for the adjusted hydrograph B will usually join the distorted hydrograph B_2 at approximately 80% of the assumed peak value.

To check the trial curve, the areas under the two hydrographs B and B_2 are compared. If the areas are approximately equal, the assumed peak flow may be used for design and for tabling downstream of the junction. In comparing areas, only the areas above a line through the 80% peak points must be planimetered. The areas below this line are equal.

After the hydrograph has been adjusted, the new peak of 2150 cfs at a t_C of 38 minutes is listed in columns 13 and 14, line 4, Figure G 252A. The combined flows are then "traveled" from point B to point C as shown in step 3 above. The flow is again reduced (step 4); hydrograph B is transferred and balanced to hydrograph C_l (step 5); the lateral hydrograph CL is plotted (step 6); hydrographs C_1 and C_L are summed to hydrograph C_2 (step 7); and hydrograph C_2 is adjusted to hydrograph C (step 8). This procedure is followed for each junction and tabled to the outlet of the drain.

G 260 METHOD FOR UNDEVELOPED MOUNTAIN AREAS

This method of computing runoff is specifically applicable to steep hillside or mountainous areas in their natural or undeveloped state. It is most commonly used for parks or natural forests likely to remain undeveloped in the foreseeable future, and for determining the increase of runoff of an area from its natural state to its developed state. It is most useful to determine the time of concentration (t_C) of an area as a single calculation (without tabling from the first small sub-area) with reasonable accuracy to the first major junction of main line and lateral.

The nomograph (Figure G 261) is used to determine the t_C to the first main line junction. The use of this nomograph is intended for natural basins with well defined channels, for overland flow on bare earth, and for mowed grass roadside channels. The slope correction curves (Figure G 262A) are used to convert the ground slope (scaled from maps or taken from surveys) to the effective slope of a natural channel used in runoff calculations. The runoff tabling sheet (Figure G 242) may be modified for use in these computations.

G 261 CRITERIA

Runoff computed by this method shall comply with the following criteria:

1. The drainage area must be unimproved and, with the exception of small. finger-like projec- tions, must have a single watercourse or ravine. It must also be confined (that is, have no other outflows except at the point under consideration).

2. The impervious classification (Id) shall be 35% and the soil classification shall be loam (2) unless soil maps or tests show a definite classification.

3. The Isohyetal shall be taken from Figure G 241.1A and the runoff shall be computed downstream from the first junction by the Peak Rate Method.

4. The time of concentration of a drainage area to a selected concentration point or the first junction of main line and lateral shall be from either of the following:

a. Kirpich's nomograph (Figure G 261), or
b. Pickering's formula:

$$t_c = 60\left(\frac{11.9 \times L^3}{H}\right)^{.385}$$

where
L = length in miles from upper boundary of the drainage area to the first major junction of main line and lateral
H = fall in feet for length L above.
t_C = time of concentration (minutes)

If there are no major laterals, and the nomograph or formula is used to determine the t_C for the entire drainage area, a minimum t_C of 5 minutes is used for areas of 5 acres or larger. For less than 5 acres, use a t_C determined from the nomograph (or formula), but use the maximum value of the BPRR of 3.40 as indicated in Figure G 242.2L.

G 262 COMPUTING THE FLOW

The drainage area map is prepared as shown in Subsection G 241.1. The runoff is computed by the Peak Rate Formula:

$Q = A \times F_{RO} \times \text{BPRR} \times \text{Distribution Factor}$

The area is taken from the drainage area map; the Runoff Factor is taken from Figures G 242.2F to K; the Base Peak Runoff Rate is taken from Figure G 242.2L; and the Distribution factor is taken from Figure G 242.2E.

If the watershed consists of several separate drainage areas and is not applicable to this method as a whole, it may be applicable to the individual areas by combining the t_C 's of main line and laterals at a common point by the weighted average formula shown in Subsection G 242.2, No. 7, and by tabling the flow from junction point to junction point similar to the tabling shown in Figure G 242. The cross section of the natural channel for this flow is assumed as trapezoidal and is determined from map contours, survey elevations, or field inspection. Its base width (b) and side slopes (Z) are averaged for each reach. The ground slope of the channel is also determined from maps or surveys and is averaged for every reach. However, the effective slope from Figure G 262A is used in lieu of the ground slope for time of flow calculations from junction point to junction point. The "n" value of the channel is selected from the following:

Regular section with earth or short grass = .03
Irregular section with earth or short grass = .04

Regular section with tall grass or some brush = .05

Irregular section with tall grass or some brush = .06

Irregular section with heavy brush or erosion

= .08

(Values averaged from tables in Appendix "A," U.S. Bureau of Public Roads, Hyd. Eng. Circ. No. 6, Apr. '62)

G 270 HYDROLOGY FOR RESERVOIRS

The hydrology required for the solution of reservoir problems is included in the chapter Storage Basin and Pumping Plant Design (G 500).