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HYDROLOGIC MODELING FOR THE DETERMINATION OF DESIGN DISCHARGES IN UNGAUGED BASINS

E.I. DANIIL*
S.N. MICHAS
L.S. LAZARIDIS

Hydroexigiantiki
Evias 3, 15125 Maroussi, Greece

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*to whom all correspondence should be addressed
Tel.: +(30) 210 8064543; Fax: +(30) 210 8055495
e-mail: edaniil@hydroex.gr

ABSTRACT
All stormwater management projects in Greece are required to get environmental permit before construction. The design return period is often determined by the environmental permit. Determination of design discharges is an important parameter for the design. Design discharge for a given return period is not uniquely defined and may vary considerably depending on the selection of the parameters and methodologies involved. Using a rainfall height distribution that tends to maximize the peak discharge (worst profile distribution) in essence corresponds to a lower probability of occurrence that is not quantified at present. There are publications, based on data from the US and UK that show that center-loaded storms are appropriate for design of stormwater systems.
In this paper a test case study is presented. Possible variation of the estimated flood peak that can result from variation of rainfall distribution of given total height and duration is shown. Comparisons are shown between the rational method, hydrographs based on the ones given in Design of Small Dams and also with modeling with the HEC-HMS system using SCS hydrographs. Results from the HEC-HMS modeling include variation of the rainfall peak location, use of two different idf curves and SCS - Type I storm. It is seen that the same discharge value can be derived from different storm durations or different return periods depending on the rainfall distribution. Another interesting unresolved matter is the size of sub basins that should be included in modeling of hydrographs as the processes involved are not linear and the computed results may vary.
It is proposed that hydrologic modeling is used including a considerable part of the drainage system of a basin and different "scenarios" of rainfall distribution and rainfall duration are used to get a feeling of the possible variations of the T-year return flood that the environmental permit describes, before more strict guidelines are adopted.

KEYWORDS: Design discharge, flood management, ungauged basin, HEC-HMS, rainfall distribution.

1. INTRODUCTION
Stormwater management projects in Greece are presently required to get environmental permit before construction. The natural state of watercourses with open cross sections should be preserved and materials friendly to the environment, such as gabions, should be used in most cases. Existing structures and upstream control should be integrated in the final layout [1, 2]. The design return period is often determined by the environmental permit and T=50 yrs tends to be the standard for most ephemeral streams. Especially in ephemeral streams hydrometric measurements are in most cases completely lacking and even when they exist in many cases the flood peak is probably missed. As a result the calibration of any model for the existing conditions is not possible. If we take into account the consequences of expanding urbanization, the uncertainty of the projected peak

values also increases [3]. It should also be taken into account that there is always a lag between the time of the design and the construction of the training works and the drainage system. Therefore, projections concerning the change in land use should be made for at least 50-yrs in the future.

Recently, programs have developed in Europe, e.g. Ecoflood, Daywater, investigating the possibilities of control at the source and restoration of the floodplains. This is not always feasible in heavily urbanized areas and for high return period floods. It is considered necessary to assure that the final design can handle discharges higher than the design discharge with reduced freeboard and that critical points, such as road and highway crossings are designed to handle higher return period floods with safety. Sensitive points of existing closed sections to pressurized flow should be identified and protected [4].

For the determination of the design discharge, hydrologic modeling of the whole watershed system should be used. For ungauged basins one relies on the geometric and geomorphologic characteristics of the basins and available precipitation data. A design hyetograph and unit hydrograph should be selected as well as a method or formula for the determination of concentration or lag time that has a strong influence on the computed peak discharge. Hydrologic issues related to the design of major hydraulic structures that involve higher return periods have been discussed [5].

Uncertainty of hydrological predictions occurs at two different levels [6]; inherent uncertainty associated with any given model is caused by uncertainties in the estimated parameter values and in the climatic inputs, and uncertainty that arises from the imperfectness in the model that is used to make predictions in a specific basin. Since it is unclear if any of the available models can make accurate predictions in an ungauged basin, model structure uncertainty can only be assessed by comparison with measured data in the catchment of interest, or in similar catchments in the same hydroclimatic zone.

An international program is currently underway by the International Association of Hydrologic Sciences (IAHS) for improving predictions for ungauged basins by examining and improving existing models in terms of their ability to predict in ungauged basins and by developing new, innovative models to capture space and time variability of hydrological processes for making predictions.

In the present paper, choices of parameters and methodologies preferred by the authors for use in practice, are described, that are thought to reduce, or in a way quantify, uncertainty in ungauged basins.

2. HYDROGRAPH DETERMINATION

2.1 Design Hyetographs

Design hyetographs should be based on observed storm events analysis [7]. By analysis of observed storm events, the sequence of precipitation in typical storms can be determined. Such an analysis was performed by Huff [8] for heavy storms on areas ranging up to 400 mi^2 in Illinois. The first quartile 50-percent distribution has been used in the ILLUDAS storm drainage simulation model.

The U.S. Soil Conservation Service (1986) developed synthetic storm hyetographs for use in the US. Types I and IA are for the Pacific climate with wet winters and dry summers. The effect of using SCS Type I, IA for parts of Greece is investigated and compared to distributions obtained from idf curves.

Design hyetographs can also be developed from intensity - duration - frequency curve. A simple way, used also as default in some computer programs is the alternating block method, referred to also as a center-loaded storm, as the peak of the hyetograph is placed at the middle of the storm duration. Levy and McCuen [9] report that actual data from six Maryland watersheds (5<A<135 sq. Km) suggest that center-loaded design storms are appropriate. Packman and Kidd [10] also reported that center-loaded hyetograph was most appropriate, based on the analysis of data from the United Kingdom.

Two intensity – duration – frequency (idf) curves, that give the same 24-hr 50-yr rainfall depth, H=150 mm, were used:

$$i=15.822\ T^{0.25}\ d^{\ -0.59} \tag{A}$$

$$i(d,T)=\frac{51.975\left(T^{0.16}-0.636\right)}{\left(d+0.0679\right)^{0.732}} \tag{B}$$

where d is the rainfall duration in hours, i the rainfall intensity in mm/h and T the return period in years.
Curve A represents a classic method for fitting idf curves, while curve B was constructed based on the methodology described in [11]. An idf curve of the same type has been developed for Athens area [12]. For return periods up to 100 years, the dependence of rainfall intensity on the return period is similar for the two curves. For higher return periods the two curves deviate considerably, as can be seen in Figure 1, where the T-dependent coefficient expresses the variation of the intensity due only to the return period, normalized to T=10 yrs.

$$\text{T-dependent coefficient} = i(d,T)/i(d,10)$$

In figure 2 the profiles produced by the two idf curves and the SCS storm profiles I, IA are compared. The alternating block distribution resulting from the two idf curves is almost identical for the return periods considered and consequently results only for curve (A) are reported. In all computations presented herein the assumption is made that the precipitation is uniform over the entire modeled area.

2.2 Unit Hydrograph

Unit hydrograph is a familiar and convenient method. Many unit hydrographs have been proposed over the years. In the third edition of Design of Small Dams [13] six types of dimensionless hydrographs are presented based on 162 reconstructed flood hydrographs. In order to apply these dimensionless hydrographs, lag time has to be determined, based on the geometric characteristics of the basin and a parameter, Kn, which can be interpreted as a Manning coefficient of the whole basin. This coefficient depends also on the return period, with higher values associated with lower return periods. The hydrographs are parameterized with LD = Lg + 1/2 D, where D is the unit rainfall duration. This parameterization, the determination of Lg based on basin characteristics and classification, is the most important issue in the construction of these dimensionless hydrographs.
In HEC-HMS [14] SCS hydrographs can easily be used. The general rule for application of the SCS hydrographs is taking the lag time to be equal to 0.60 times the concentration time. Using as lag time for the SCS hydrographs the time to peak determined from the Sierra Nevada hydrograph, we get values between those determined based on the Kirpich formula and the SCS lag equation. These were considered appropriate for use [4].

2.3 Lag Time Determination

Lag time is the time from the midpoint of the unit rainfall excess to the time that 50% of the volume of unit runoff from the drainage basin has passed the concentration point [13].

$$L_g = C\left(\frac{L\,L_{ca}}{S^{0,5}}\right)^N$$

Lg is the unit hydrograph time in hours, C, N are constants N = 0.33, C = 26 Kn, L is the longest watercourse from the point of concentration to the boundary of the drainage basin in miles, Lca is the length along the longest watercourse from the point of concentration to a point opposite the centroid of the drainage basin in miles, and S is the overall slope of the longest watercourse (along L) in feet per mile.

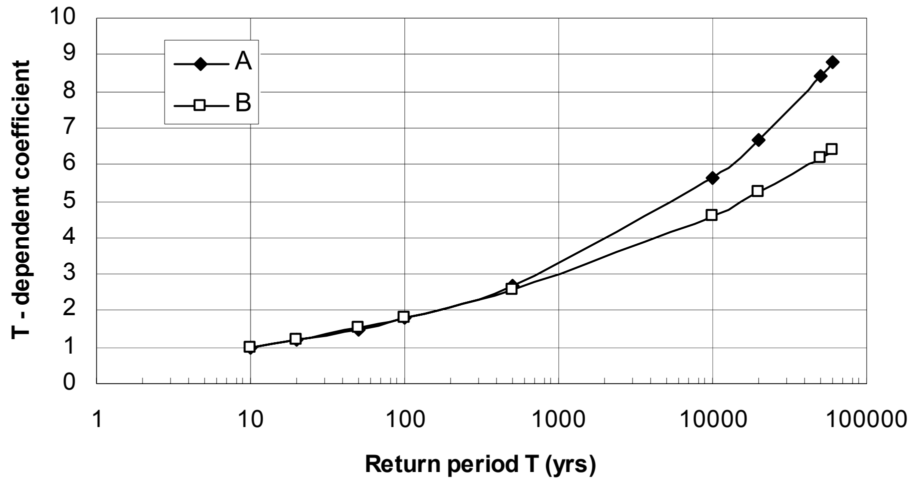

Figure 1. Variation of T-dependent coefficient with return period

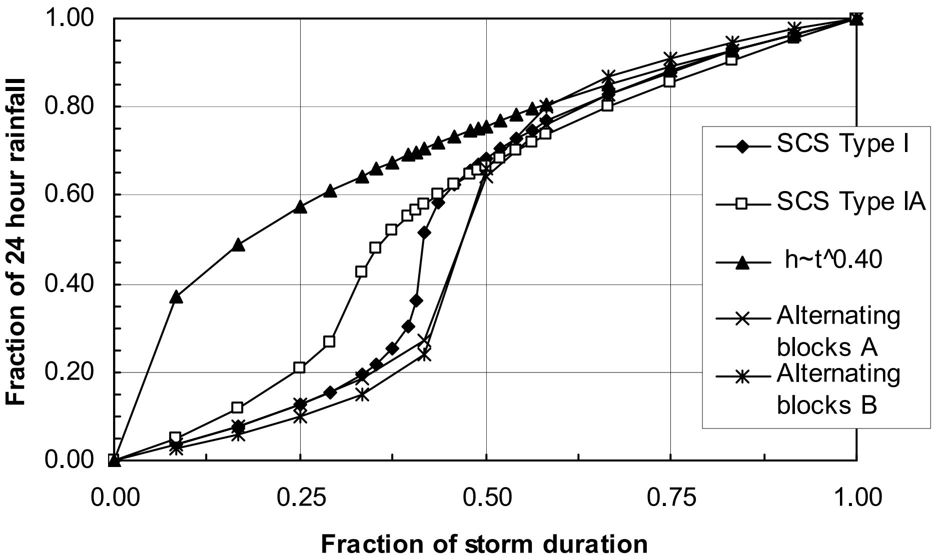

Figure 2. Storm profile comparison

For the Sierra Nevada type hydrograph the time to peak is: $t_p = 0.75 * (L_g + {}^1/_2\ D)$. For small rainfall duration as in the case of unit rainfall duration used in the computer programs that is on the order of couple minutes $t_p \approx 0.75 * L_g$. For the other types of hydrographs the proportionality coefficient ranges from 0.60 to 0.80.

While the most critical parameter in determining the outflow from a drainage basin is time of concentration, formulas in the literature give greatly varying results. The choice of the appropriate formula to be used should always be given serious consideration; especially in ungauged basins the impact on design discharges can be very significant. In Greece the most popular formulas for the determination of the time of concentration are Kirpich and Giandotti formula. Kirpich gives very short times, while Giandotti gives very long times. Comparison to other formulas for time of concentration have been presented [15].

Kirpich formula was developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%). For overland flow on concrete or asphalt surface multiplying tc by 0.40 is suggested.

$$tK = 0.02\ (L/S^{0.5})^{0.77}$$

The Kirpich formula can also be applied in segments as:

$$tKseg = 0.02\sum\left(Li / Si^{0.5}\right)^{0.77}$$

Hotchkiss and McCallum [16] investigated the performance of the nomograph used by the Nebraska Dept. of Roads, which is essentially the Kirpich formula multiplied by 1.5 for agricultural watersheds. They conclude that the SCS lag equation showed poor results on all sites regardless of how the curve number was determined.

The lag time of the unit SCS hydrograph is assumed to be equal to 0.60 times the time of concentration. The time of concentration tp is computed by the following formula:

$$t_p(\text{hours}) = \frac{\ell^{0.8}\ (S+1)^{0.7}}{1900\ y^{0.5}}$$

where ℓ the length of the watercourse to the divide (ft) y the average slope (%), S = 1000/CN – 10, CN curve number used for calculation of losses.

The average slope is computed either as the average watercourse slope or as the average watershed slope. Watershed slope is essentially the hillside slope. It is used in overland flow calculations, where no defined flow path exists, and is also applicable for the determination of the runoff coefficient in the rational method. The two estimations may vary depending on the morphology of the basin and the grid size of the digital elevation model - if one is used.

The issue of resolution effects of digital elevation models on hydrological modeling parameters and peak discharge has been discussed in recent publications [17], [18], [19]. investigate the effect of digital elevation model grid size and source on the average watershed slope. This presents an interest in view of the continuously increasing use of DEM's and computers in hydrologic analysis. A protocol, or appropriate guidelines have still to be developed regarding the appropriate resolution to be used for the evaluation of hydrologic modeling parameters from DEM's.

3. TEST CASE

In this paper a test case study is presented. A hypothetic basin of 10 Km^2 is selected and modeled in HEC-HMS with three different layouts as shown in Fig 3 and Table 1. Sink 1 and Sink 2 give identical results. Another interesting unresolved matter is the size of sub basins that should be included in modeling of hydrographs as the processes involved are not linear and the computed results may vary. This is partly illustrated by comparison of Sink 1 and Sink 3 cases.

In Table 2 comparisons are shown between the rational method, Design of Small Dams hydrographs (0.5 hr time step) and with HEC-HMS modeling using SCS hydrographs and 24-hr storm. Results from the HEC-HMS include variation of the peak location, and SCS - Type I, IA storm.

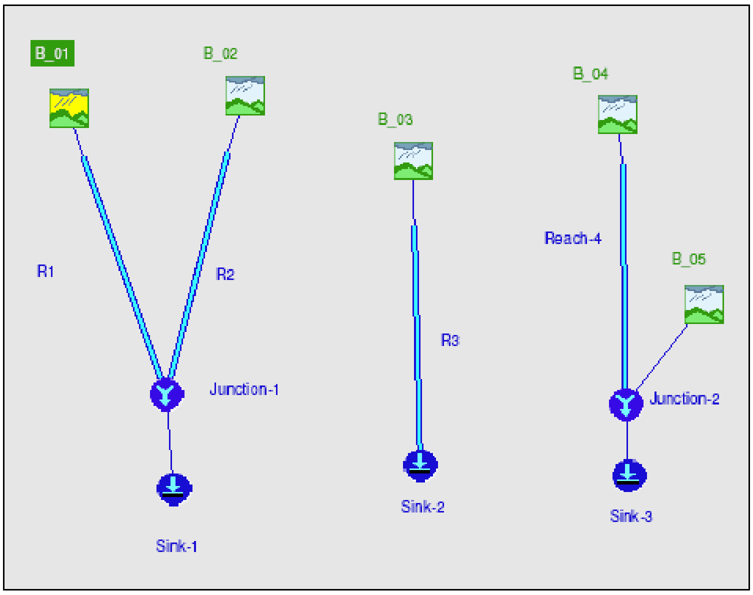

Figure 3. Schematic Layout of test cases

Table 1. Selected parameters and geometric characteristics for test cases

Element	Area (Km^2)	CN	Lag (min)	Length, Manning's n	Geometry b, H:V
B_01	5	75	60		
B_02	5	75	60		
B_03	10	75	60		
B_04	5	75	30		
B_05	5	75	30		
R1, R2, R3				100, 0.016	5, 1:1
Reach 4			30		

Table 2. Discharge ($m^3 s^{-1}$) for 24-hr storm

Rainfall distribution Scenario	Sink 1 T=10yrs	Sink 1 T=50 yrs	Sink 1 T=100 yrs	Sink 3 T=10 yrs	Sink 3 T=50 yrs	Sink 3 T=100 yrs
Peak at 25%	20.6	47.6	64.3	26.0	59.0	79.3
Peak at 50%	28.5	58.7	76.7	35.1	71.5	93.0
Peak at 75%	34.5	66.3	84.7	41.9	80.0	102.4
SCS Type IA		30.6			35.6	
SCS Type I		52.2			64.0	
Rational method c=0.55, tc=1.46 hr		51.2				
DSD hydrograph Kn=0.08, Lg=1.46 hr		52.4				

From Table 2 it is seen that the 100□yr flood with precipitation peak at 25% is almost the same with the 50-yr flood with precipitation peak at 75%. This shows clearly that determining only the return period does not lead to a unique estimation of the design discharge. The SCS Type I storm gives values close to the 25% and 50% peak distributions. In Figs 4, 5, 6, 7 outflow hydrographs for different scenarios and return periods are given. The same discharge value can be derived from different storm durations or different return periods depending on the rainfall distribution. In fig. 4 the variation of hydrograph and peak discharge for five different scenarios for the 50-yrs, 24-hr storm are given.

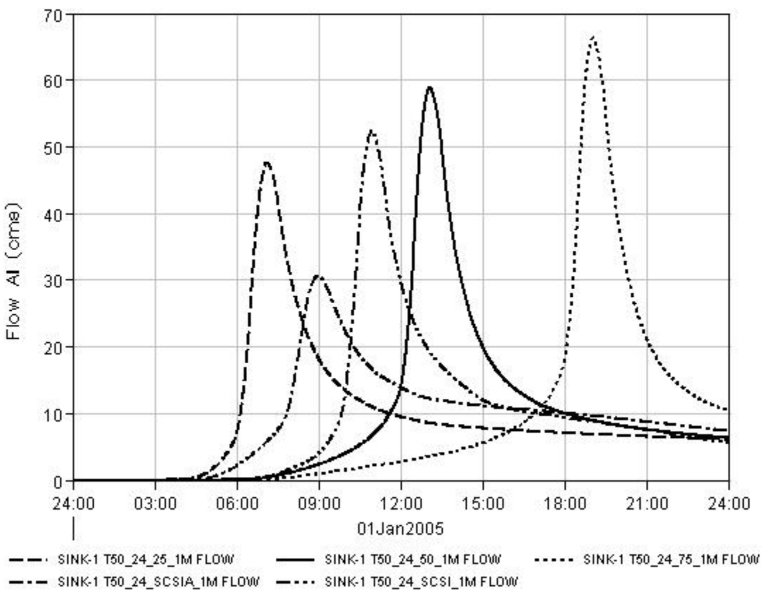

Figure 4. Sink 1 - Outflow Hydrographs for different rainfall distributions, H=150mm (T=50yrs, d=24hrs).

In Fig. 5 the variation of hydrograph and peak discharge for the 12-hr and 24-hr center-loaded storm for return periods of 10, 50 and 100-yrs storm are given for Sink-2 (the results are identical for sink 1).

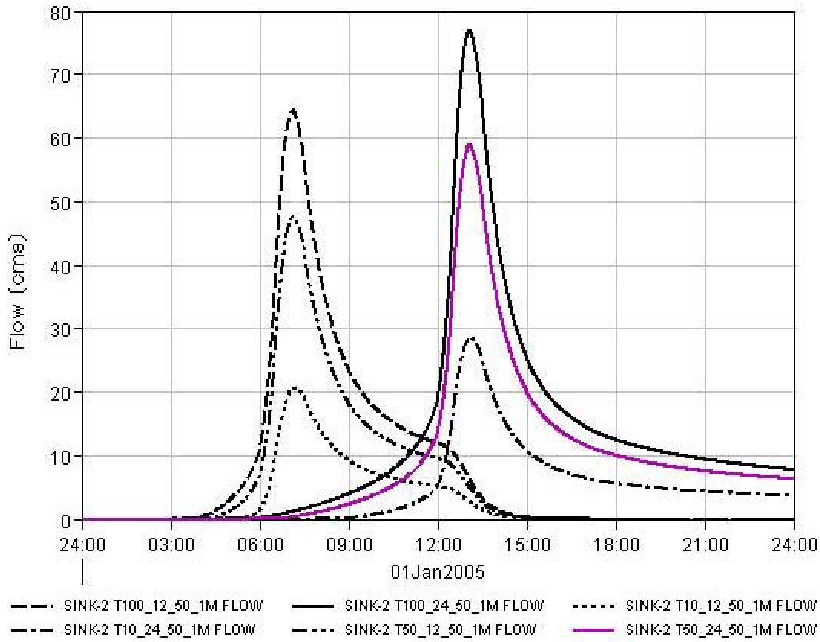

Figure 5. Sink 2 - Hydrographs for T=10, 50, 100 yrs and 12, 24 hrs storm with peak at 50%.

In Fig. 6 Hydrographs for T=50 yrs for 12-hr storm with peak at 50% and 24-hr with peak at 25% are given. The hydrograph is identical up to 12-hr for the two cases as in both cases the same peak is placed at 6-hrs. The difference is in the total volume produced.

In Fig. 7 the Sierra Nevada Hydrograph for T=50 yrs, 24 hrs center-loaded storm is given for comparison.

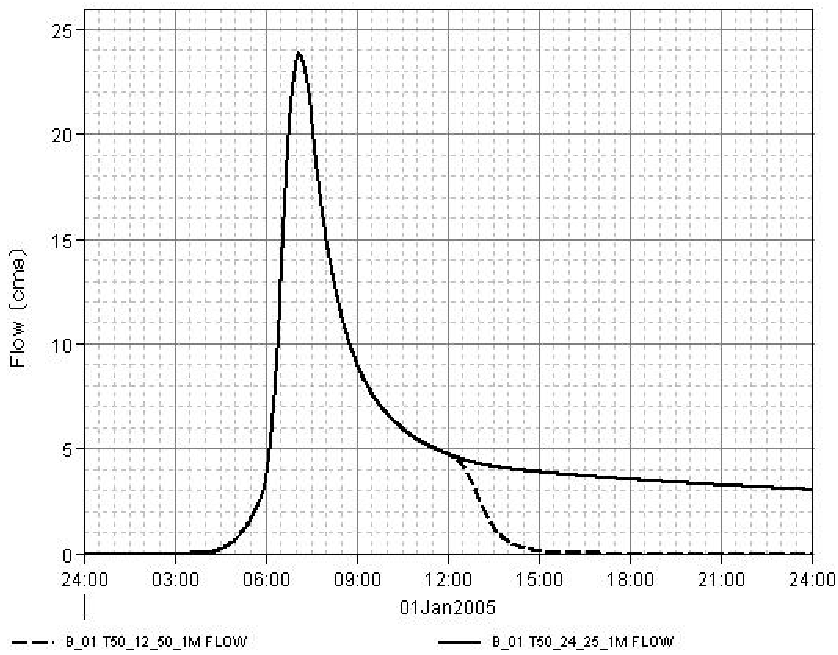

Figure 6. Sink 1 - Hydrographs for T=50 yrs for 12-hr storm with peak at 50% and 24-hr with peak at 25%

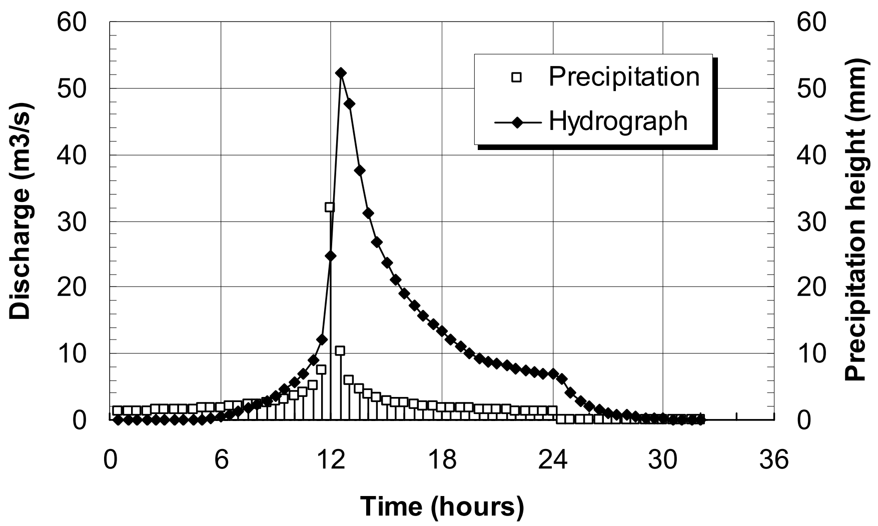

Figure 7. Sink 1 - Sierra Nevada Hydrograph (Kn=0.08, Lg=1.46 hr) for T=50 yrs, 24 hrs storm with peak at 50%.

4. WATER QUALITY MODELING CONSIDERATIONS

Hydrologic modeling is also related to water quality modeling. Calabro [20] mentions that design storms with triangular or Chicago shapes and durations, similar to the time of concentration of the catchment, are in several cases the worst regarding impact on the receiving water body, especially for the longest return periods tested. Antecedent dry period is also involved in the computation and is not always easy to assess, thus a certain degree of uncertainty is involved. Xiong and Melching [21] conclude that although the overall model-fit efficiency of non-linear reservoir routing can be fairly high under some circumstances, the kinematic wave method is a better choice in urban rainfall - runoff studies, especially when water-quality simulation is required. The subject should be pursued further as restrictions on stormwater quality are increasing.

5. CONCLUSIONS

For the determination of design discharges hydrologic modeling should be implemented, including a considerable part of the drainage system of a basin so that not only the peaks, but their time distribution in the drainage system can be estimated.

Different "scenarios" of rainfall distribution and duration to get a feeling of the possible variations of the T-year return flood that the environmental permit prescribes, before more strict guidelines are adopted. Using 6, 12, and 24 hrs storms with peaks at 25 – 50 –75% peak location a range of the expected variation can be obtained. Different duration storms with the peak center placed at the same time five the same peak value but different runoff volumes. This analysis expresses in a way the prediction uncertainty involved related to the rainfall input.

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