DESIGN AND ANALYSIS OF STORMWATER RETENTION PONDS BASED ON WATER QUALITY OBJECTIVES

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الخلاصة :

تعتبر عملية تحويل المناطق الريفية الى مناطق سكنية من الامور المؤثرة على تلوث مياه الامطار الجارية الى الانهار أوالمجاري اذ لوحظ ان احدى الطرق الحديثة في السيطرة على تلوث المياه من جريان الامطار في المناطق السكنية هي بناء مجمع أدخار قبل الاطلاق الى الانهار او المجاري . حيث يتناول هذا البحث الاساسيات المتبعة حاليا في تصميم مثل هذه المجمعات . . . وبناء نموذج لقياس وتحليل كفاءة هذه المجمعات والتصرفات الهايدرولكية واعداد طريقة للتصميم ضمن الاهداف المتعددة الجوانب .

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Synopsis

The change from rural to urban landuse has an affect on the volume, rate and quality of stormwater runoff. The pollutants originating from a wide variety of processes such as soil erosion, litter, dust, de-icing salts, tire wear, etc accumulate on urban land surfaces, and are subsequently washed off by rainfall or snowmelt.

The problem of deteriorating the quality of stormwater runoff has not received as much attention as that of increased flooding. In this paper a brief review is made initially of the alternative methods available for controlling the quality of the runoff, discharging into the river or sewer. It is observed that one of the interesting methods is the construction of natural or artificial retention ponds. In the present study, an attempt is made to define the water quality criteria currently used in the design those ponds. A simulation model is developed to evaluate and analyse the removal efficiency and hydraulic behavior of retention facilities; also a design procedure is constructed. A multi – objective analysis is formulated to illustrate a possible decision – making process for the case when water quality objectives are not specified.

Introduction

The most notable techniques currently used for controlling the quality of water runoff are the use of vegetative filters [1] and retention ponds [2] Street sweeping and its effect on the quality of urban runoff has also been studied [3] In extreme cases, where combined sewer overflows are the cause of water quality deterioration, storm water may be routed to an off-line storage basin and returned to a sewer plant during low flow conditions [4]

Each of the above-mentioned methods has problems associated with it. The vegetative filters are constrained to the season when vegetative growth is limited, also they are effective in significantly reducing the sediment load only if the flow is relatively shallow. Retention ponds require periodic removal of trapped solids, and there is some evidence to suggest problems of algae blooms and insect breeding. Street sweeping does not provide a complete solution to the water quality problem, since pollution from pervious areas may be as significant as that from the impervious areas. [5] The quality of surface water runoff is generally not adverse enough to warrant the cost of sewage treatment.

Of all the above methods, the use of retention ponds has been by far the most popular method employed, [6] as they have more advantages, for example: being relatively inexpensive, requiring minimum maintenance, and having capabilities for both flow control and quality improvement. Most recently [7] McCuen (1980) collected data on 148 acre site in Maryland (USA) and concluded that a detention pond can trap as much as 98 percent of the pollutant in the flow. However, in spite of the popularity of the detention pond, there is still considerable difference of opinion regarding design procedures.

Design Criteria

The design of retention pond facilities for water quality control is not straightforward. The quality of pond effluent is a function of both the influent water quality and the treatment administered to the flow, which is a direct function of the detention time of the water. Both of these parameters vary within the duration of a particular storm event, and it is therefore necessary to identify the naturally occurring combination of these parameters which govern the effluent water quality .

Typical concentrations of the major pollutants found in stormwater are summarized [8] by Waller (1977) .It is observed that stormwater is often equivalent to or better in quality than effluent from secondary sewage treatment plants, with the exception of suspended solids concentrations which far exceed the effluent levels. It is probably for these reasons that most water quality improvement projects concentrate on removal of suspended solids as a water quality objective.The criterion to achieve reducedsuspended solids levels can be either or both the removal rates or the lower limiting particle size. above which all particles must be removed & If the required trapping efficiency of the pond is specified (e.g. from legislative criteria). then the limiting particle size may be determined using a grain size distribution graph of the soil in the watershed.[9] The grain size distribution graph should be modified to exclude abnormaly large particles. if any. since these are not usually part ofthe watershed[10]. The choice ofthe limiting particle size should be determined in conjunction with a design flow rate. The design flow quite often corresponds to the peak runoff rate of a design storm of a given return period. Davis and Bain (1975). and Oscanyan (1975). both used a 10 year storm which is a popular period among water engineers.[11,10] The authors found it is useful to consider also the concentration levels and its variation within the storm in the design of retention ponds.

Having obtained the design flow rate and a maximum allowable particle size. the pond is then sized based on the required removal rate. This technique involves calculating the rate at which a particle of a given size will settle and subsequently enough volume must be provided so that sufficient time is available for this settling to occur at the design flow condition: Quite often this information is contained in a curve of particle diameter versus overflow rate (e.g. Oscanyan, 1975) which may be based on empirical measurements. Alternatively. such removal rates may be theoretically calculated using the work of Camp (1945) and then modified to account for non ideal conditions.[12]

Generation of Suspended Load :

The calculation of expected mass flux , suspended solids concertrations and maximum particedsize throughout rainfall events requires repeated use of specific equations . To reduce computational effect a computer program was formulated for this purpose . The desired water quality parameters are calculated for a finite number of time increments whose duration and length are specified by the user . Other input variables to the program include a rainfall hyetograph, outflow hydrograph, a specified soil loss equation parameter (e.g. the one given by Huber et al .) , area of the watershed and total length of the watercourse .[13]

Using the above-mentioned model. pollutant loads (in particular. suspended solids concentrations and mass fluxes) have been synthesized for storms corresponding to 5.10 and 25 years return periods. Figure I summarizes the results .

Water Quality Simulation Model

To compare the pollutant removal efficiencies of the various retention ponds designed using different procedures, it was necesary to develop a model to simulate retention pond operation. Two basic methods have been developed to perform such simulations. One method is empirical. based on trap efficiency and relates the removal efficiency of a reservoir to both its physical characteristics and the magnitude of the inflow. Such models are based on the analysis of numerous reservoirs for a wide variety of events. One of the better known examples of such a model is that developed by Brune (1953), who related the percentage of sediment trapped to the ratio of reservoir capacity and annual inflow[14].Such empirical methods have only limited utility since they are transferable only to watersheds withalmost identical properties as those for which the model was developed. These methods are also limited since they are quite often developed to obtain an estimate of the overall trapping efficiency and as such are not useful in identifying how the performance of these devices vary during a storm.

To avoid these problems a deterministic model based on the work of Camp (1945) was used. Camp's approach to clarification theory was developed for an ideal basin under quiescent flow conditions .

Most sedimentation theories employ the concept of "plug flow" which assumes delivery of the flow on a first - in - first - out basis and allows no mixing between plugs. The detention time (t^*) of each plug is required for solids removal calculations and so it is imperative that the identity of each plug remains separate and is identifiable. The procedure used in the present program to obtain this can be explained, using Figure 2. Consider the n^{th} time increment of the inflow hydrograph corresponding to t_1. The cumulative inflow at this time can be found by summing the area under the inflow graph. This can be determined using a finite difference approach :

$$I_{cum\,n} = \sum_{i=1}^{n-1} I_i \Delta t + \frac{1}{2} I_n \Delta t \qquad \text{at } t = t_1 \qquad \text{......(1)}$$

From the assumption of plug flow, it follows that the volume of water contained within any time increment must remain in the pond until the qunatity of water corresponding to the previous inflows has been discharged. This implies that as the plug (of time increment n) begins to discharge at some time ($t = t_2$) then :

$$0_{cum} = I_{cum\,n} \qquad \text{at } t = t_2 \qquad \text{......(2)}$$

The detention time can then be easily determined since,

$$t^* = t_2 - t_1 \qquad (3)$$

If the influent suspended solids concentrations, $C_{in\,i}$ is known for a particular time increment, then the effluent concentration can be found by,

$$C_{out\,i} = (1 - X_{Ti})\, C_{in\,i} \qquad \text{......(4)}$$

where $X_{T\,i}$ is the average removal rate for time increment i and is a function of :

$$X_{Ti} = F\,(H_{avg\,i}\, t^*_i\, v_t\, \alpha) \qquad \text{......(5)}$$

where v_t represents the terminal velocity of the particles, H_{avg} is the average depth over the detention time, and α is a correction factor to account for nonidealized conditions, such as short circuiting, resuspension of sediments or the non-spherical nature of the particles. The actual equation used to calculate the removal rate is based on the work of Camp (1945) and is described in the next section. It should also be noted that this simulation model reflects the unsteady nature of the pond's operation. Many other sedimentation models, such as SWMM - Treatment Routine, calculate removals based on steady state conditions.[15]

Since it was found that the variation of particle size distribution with time was small, this parameter was assumed constant throughout the storm. It should also be noted that due to a lack of data no attempt was made to evaluate α and for this study it was assumed to equal unity.

Predication of Particles Removal

The settling behavior of suspended discrete non - flocculating in a laminar flow condition can be expressed by the well known equation (Camp . 1945) :

$$V_t = \frac{g}{18_\mu} (\rho_s - \rho_l)\, d^2$$

where V_t is the terminal velocity . d = particle diameter . g = acceleration due to gravity ρ_s = particle density . ρ_l = fluid density and μ = fluid viscosity .

To utilize equation (6) in predicting the quantities of discrete particles removed in a pond it is necessary to assume that the particles are distributed randomly throughout the cross - section of the pond and that plug flow conditions exist .

For a particular time interval the average depth of water in the pond (H_{avg}) and the average out flow from pond (O_{avg}) is known. It is then possible to calculate the critical settling velocity (V_c) for the interval. The critical settling velocity may be described as the velocity of the smallest particle which will be completely removed during the time increment. This velocity may also be defined as :

$$V_c = \frac{H_{avg}}{t^*} \qquad (7)$$

where t* is the detention time of the time interval and is equal to :

$$t^* = \frac{S}{O_{avg}} \qquad (8)$$

where S is the storage volume in the time increment .

Those particles possessing a terminal velocity greater than the critical settling velocity will be completely removed from the pond. If. however. the terminal velocity of a specific group of particles is less than the critical settling velocity. then only a fraction of the particles in the group will be removed. Mathematically this may be expressed as :

$$X_R = \frac{V_{tj}}{V_c} X_j \qquad \ldots (9)$$

where X_R is the percentage of particles of the group removed and X_j is the total number of particles in the j^{th} group (expressed as a percentage of the total). If the first n groups have a terminal velocity greater than the critical velocity. the fraction of particles removed during the time increment can be given by:

$$X_{Ti} = \sum_{j=1}^{n} X_j + \sum_{j=n+1} \frac{V_{tj}}{V_c} X_j \qquad \ldots (10)$$

Using this equation in conjunction with the calculated concentration levels of the influent. as described in the previous section. the effluent concentration levels may be easily obtained.

Design Procedure

The procedure developed for use in the design of retention ponds to meet specific concentration levels requires prior knowledge of inflow hydrographs and inflow sediment concentrations for the design event being considered. Some information regarding the outflow hydrograph and maximum storage volume is also needed, it is possible to use the assumption of a linearly varying outflow versus time relation to obtain estimates of these quantities.

The next step involves the construction of cumulative inflow and outflow curves using the hydrographs specified above. It is now necessary to make some decisions regarding which areas of the storm govern water quality. This decision is not obvious since maximum pollutant concentrations do not always coincide with the shortest detention time. For the case study, it was initially assumed that the concentration corresponding to peak flow conditions would be the controlling element of effluent water quality. Even if such an assumption was found to be invalid through simulations of the designs operation, such simulations would enable the quality controlling portions of the design storm to be identified . The detention time of the incremental volume of water corresponding to the peak of the inflow hydrograph must then be estimated . This valuemay be obtained from the cumulative inflow and out flow graphs using the procedure out-lined previously.

To estimate the removal rate it is necessary to specify the grain size distribution of the transportable sediment. This distribution is usually obtained by performing a standard sieve analysis on surface material which is subject to erosion. The range of existing grain sizes must be divided into a finite number of groups ,each containing a certain weight fraction of the total load. The terminal velocity must then be calculated for each group. From the influent suspended solids concentrations and the desired effluent levels the percantage of pollutant which must be removed can be established. The critial settling velocity necessary to achieve this level of removal must then be determined. This is not a trivial task since the fractions removed from each group vary as a function of this critical settling velocity. A solution to this problem involes construction of a curve ot per cent removed versus settling velocity. With such a curve and knowledge of the requierd fraction to be removed the critical settling velocity may easily be obtained. The equation for a typical curve is :

$$\% = 1 - 281.5\, v_c^{.484}$$

in which % is the percentage removed

The maximum allowable depth can now be determined using the critical settling velocity and the detention time of the plug of flow under consideration.. Since this depth corresponds to a specific storage level this information can then be used to modify the stage-storage characteristics of the basin so that the required water quality level may be obtained for the design storm.

Multiobjective Analysis.

Most water resource projects try to achieve maximum net benefits. This involves estimating all the costs and benefits for all feasible alternatives and then selecting that project which provides the best return on the capital invested. In the present subject the costs of increased water quality control can easily be identified. They include such items as construcion costs, land costs and maintenance costs all of which can easily be quantified into monetary terms. The benefits of increased water quality control are not only more difficult to identify, but are almost impossible to evaluate in monetary terms. These benefits include such things as increased property values downstream, higher values for aesthetic appreciation and an increase in biological life forms. In severe cases pollutant removal may also decrease the cost of water treatment facilites downstream and cause an increase in water- based recreational activities. (16)

Al though it is difficult to put a monetary value on these benefits, they do conrtibute significantly to the overall economic structure and cannot be ignored. The procedure used for decision making must therefore incorporate these values into the process. One methodology useful for this purpose is multiple objective planning.

Multiple objective planning techniques were developed using economic production theory concepts. Its purpose is to identify optimal projects as a function of numerous relevant objectives. Consider for example, the case study in which the optimum level of water quality treatment is at issue. There are two main objectives : to minimize the cost of the structure and to maximize the removal of suspended solids. The first step of the multiobjective analysis is the construction of a transformation curve. This involves evaluating a number of possible alternatives and plotting the points as a function of the objectives. The boundary of this technologically feasible set is the net benefit transformation curve. The transformation curve developed for a typical case can be seen in Figure 3a. The cost (on thevertical axis) consists of the construction and land costs in excess of those required to meet peak flow constraints. The suspended solids concentration (on the horizontal axis) is the highest concentration of the events considered. Figure 3b shows the range of the transformation curve controlled by each storm.

The next step in the use of multiple objective planning involves the development of preference functions. These functions represent the attitudes of the various decision-making bodies such as the developer, governmental authority and the public. Any two points on a preference function line are viewed as equal. i. e. The decision - maker would be equally content with either result (for more information on preference functions see reference 17)
The function describes a specific level of benefit which maybe attained through various combinations of the objectives. The point of tangency between the transformation curve and the preference function indicates which level of operation is desired by that group. The construction of these curves entails the interviewing of the various groups to obtain an indication of their perception of the trade - off between the objectives. Such a detailed analysis is beyond the scope of this paper, however, and for this reason the curves shown in Figure 3a are hypothetical. These curves illustrate the different attitudes of the various groups. The developer, for example, is not willing to sacrifice much money for additional suspended solids removal and at some point he is unwilling to sacrifice any more money regardless of the level of suspended solids removal. This attitude is characterised by the horizontal nature of the preference function for low suspended solids concentrations and may be attributed to a minimum profit level . At some point the additional money spent on water quality improvement will reduce the return on the developer's investment to such an extent that the development will represent a bad investment. The horizontal portion of the preference function, therefore, represents the division between projects that the developer is willing to undertake, and those he will forego.

The public preference function usually places a much higher value on improved water quality. Usually such a curve has a maximum limiting value above which residents are nuwilling to allow further degradation of the water body .
This value could correspond to the concentration level at which severe ecological damage (e. g. fish kills) occurs. The other limiting case (that occurs when the public is unwilling to pay additional costs for water quality improvement) may correspond to a level when the pollutant or its effects are no longer visibly noticeable .

The preference function of the governmental authority may usually be found somewhere between that of the public and that of the developer. If governmental guidelines concerning the quality of urban runoff exist then this curve will also possess an upper bound for the quality . of urban runoff.The presence of an upper bound on the money spent to achieve improved water quality may or may not exist depending on current governmental policies. For example, the government authority may consider the benefits of the development to be well worth the additional deterioration of water quality.

The multiple objective approach illustrates the position and attitudes of the decision - making bodies. This information can then be used to arrive at a suitable compromise between the various groups.

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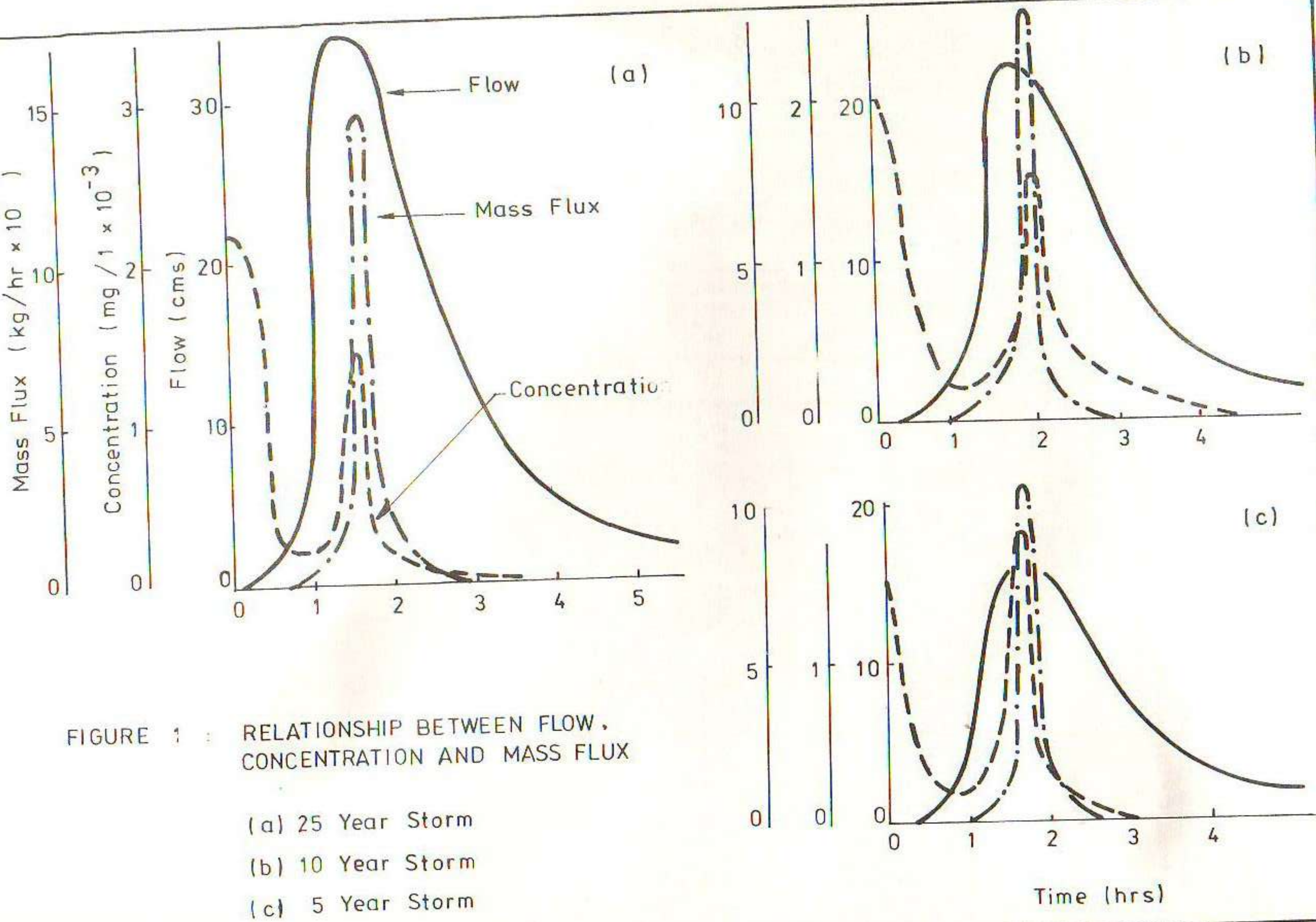

FIGURE 1 : RELATIONSHIP BETWEEN FLOW, CONCENTRATION AND MASS FLUX

(a) 25 Year Storm

(b) 10 Year Storm

(c) 5 Year Storm

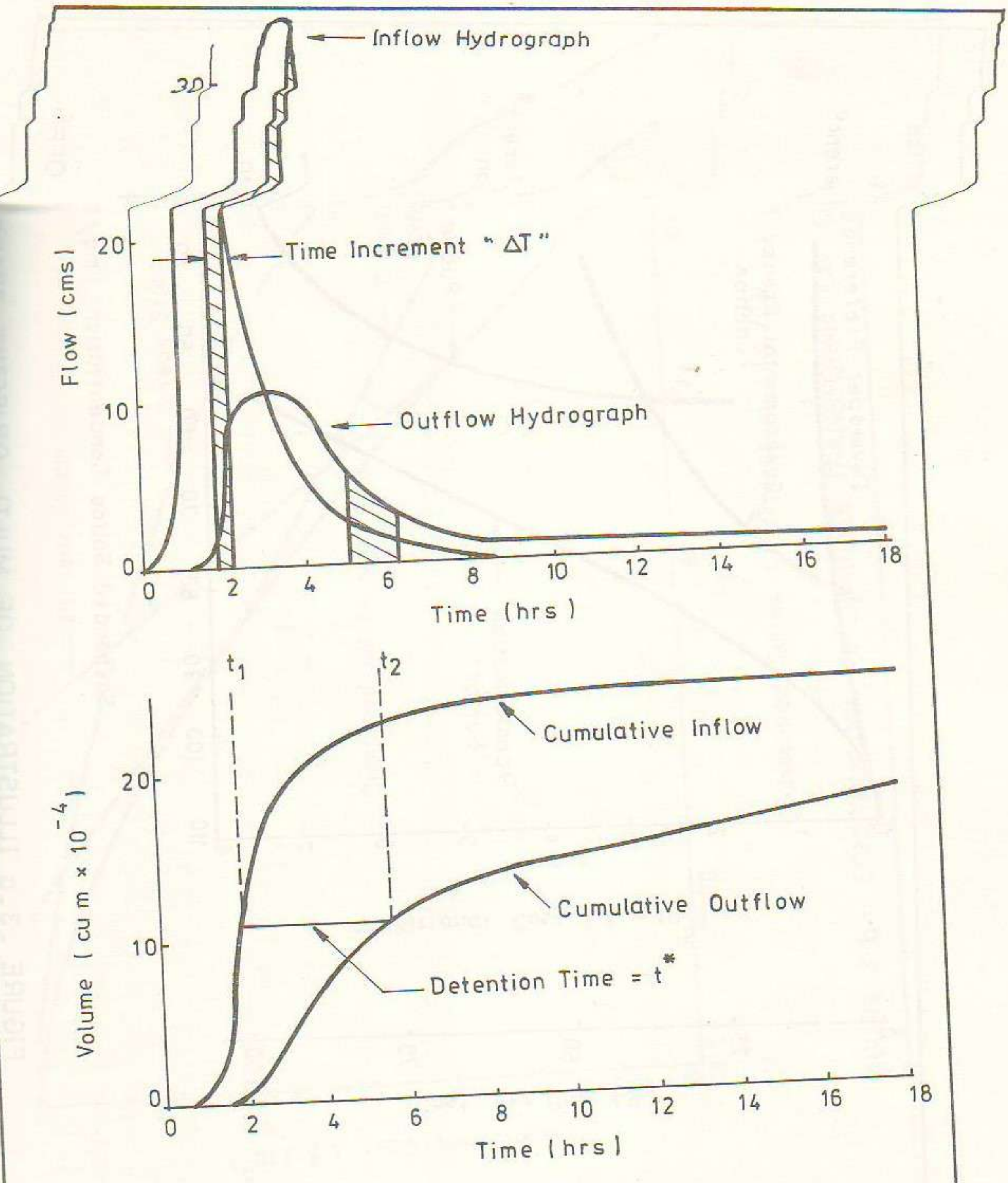

FIGURE 2 : METHOD OF ESTABLISHING DETENTION TIME

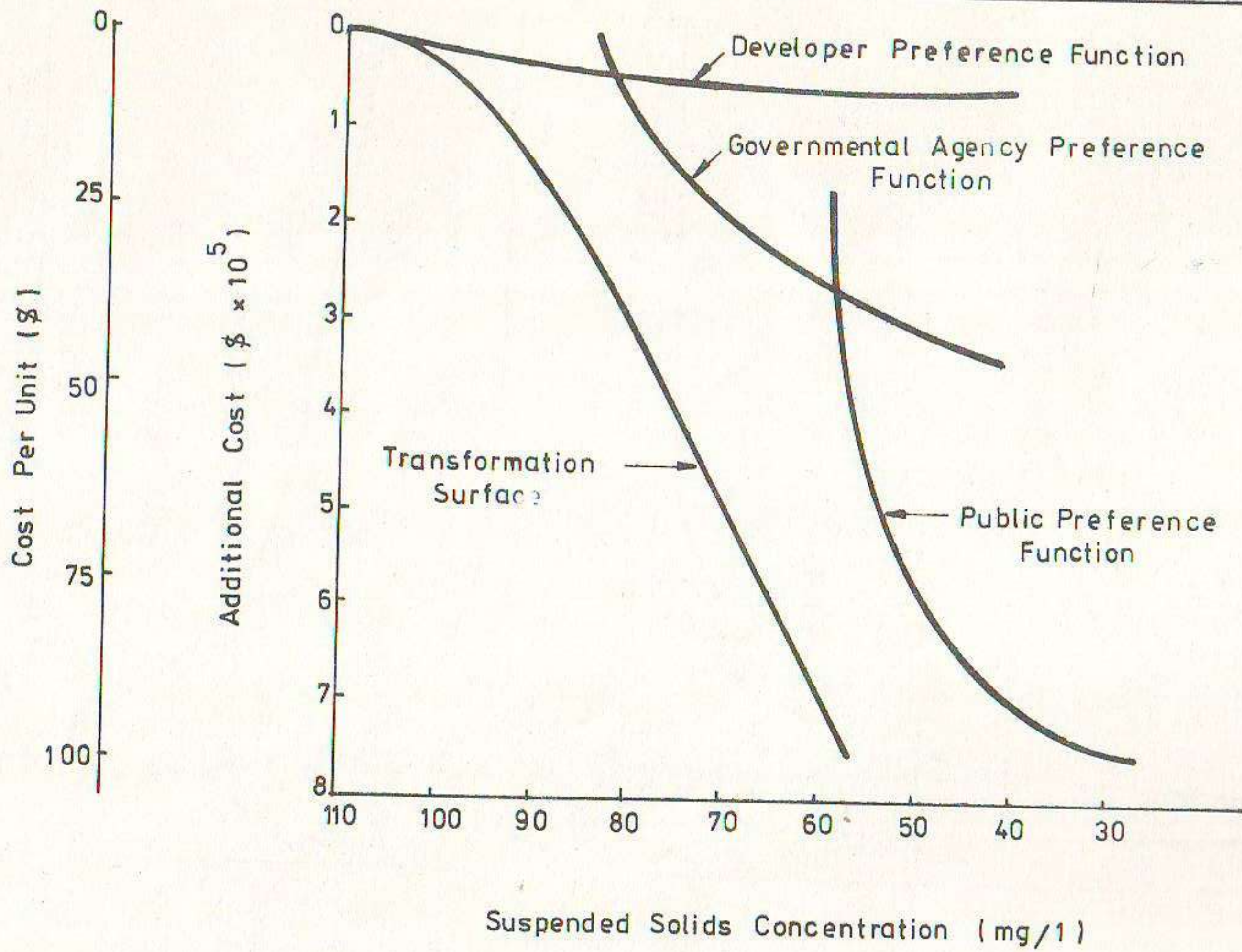

FIGURE _3_a ILLUSTRATION OF MULTI -OBJECTIVE TRADE - OFFS

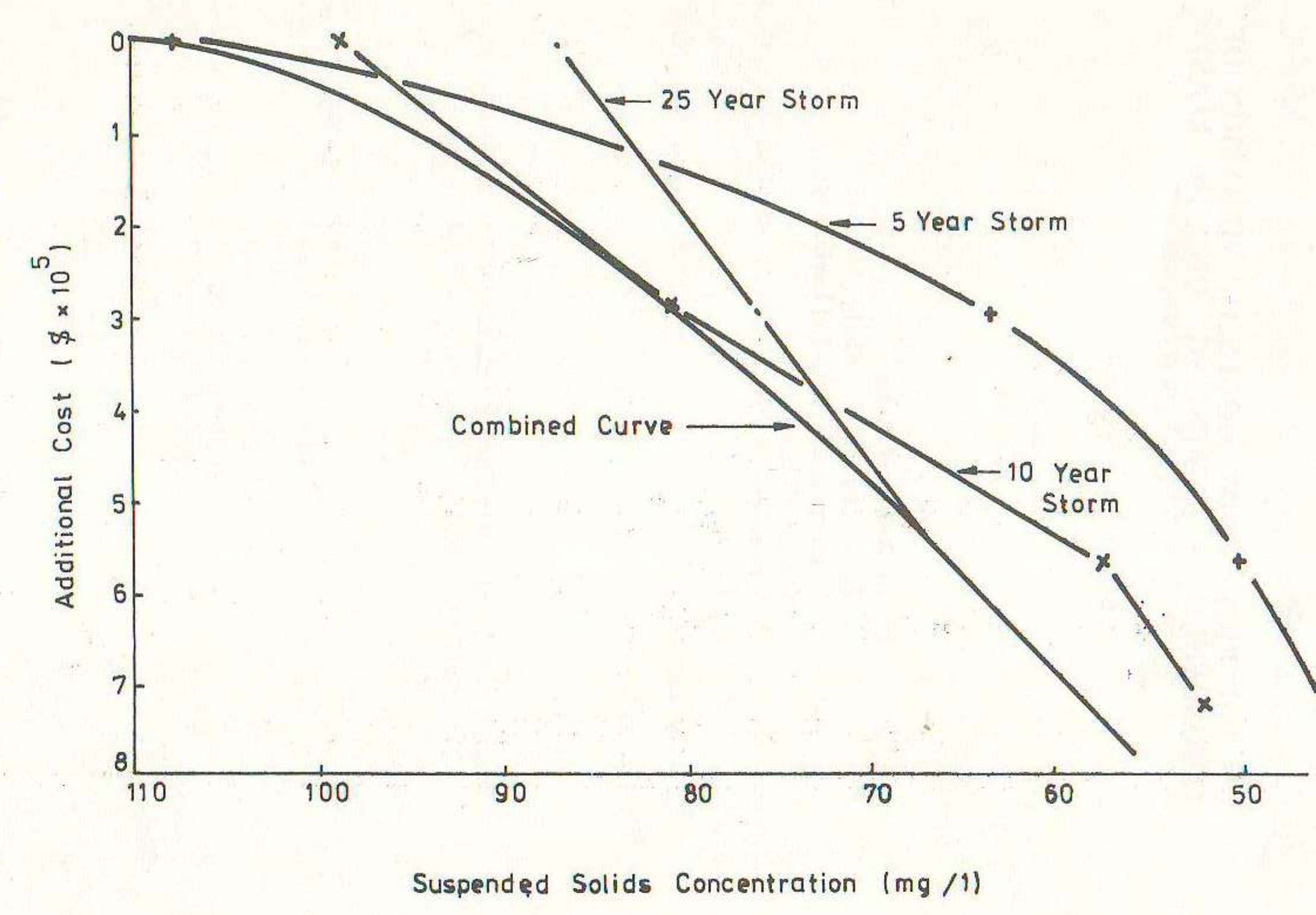

FIGURE 3-b : CONSTRUCTION OF COMBINED TRANSFORMATION CURVE