In Defence of the Rational Method

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The origin and history of the Rational Method are presented along with a discussion of the theory behind it. The relevance and importance of the method are demonstrated in the context of its influence on modern computer models which use various methods of determining runoff from rainfall.

The probabilistic Rational Method and the deterministic Rational Method are explained with reference to Australian Rainfall and Runoff, 1987 (ARR87).

There is a discussion on the guidance by Engineers Australia (EA) on the use of the new Bureau of Meteorology (BOM) IFDs (2013), that:

‘The 2013 IFD design rainfalls should definitely NOT be used in conjunction with the following techniques:

Probabilistic Rational Method

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Other regional flood techniques based on AR&R87 IFD design rainfalls.’

The treatment of the Rational Method in the Queensland Urban Drainage Manual, 2013 (QUDM, 2013) is presented. The use of the Rational method in the Brisbane City Council (BCC) Subdivision and Development Guidelines, 2008 is also presented. These Subdivision and Development Guidelines have now been incorporated in the new City Plan 2014. The transfer of their content to the new City Plan is explained.

The use of the Rational Method by the BCC in Development Assessment (DA) and in drainage planning and design is demonstrated.

The case for the opposition to the proposed limitation on the use of the Rational Method to the design of no more than two allotments in the revision of ARR is presented for discussion.

The main objectives are to:

- promote the use of the Rational Method
- provide information on the origin, history, theory, use and relevance of the Rational Method
- discuss the guidance by EA on the use of the new BOM IFDs
• present information on the use of the Rational Method in: QUDM, the draft new City Plan and the Brisbane City Council

• present the case for the opposition to the proposed limitation on the use of the Rational Method in the revision of ARR.

The paper relies on a literature search and a review of other methods of determining runoff from rainfall, including in computer models.

There is a presentation on the treatment and use of the Rational Method in QUDM, the draft new City Plan, and the BCC in DA and drainage planning and design.

Arguments for the continued use of the Rational Method are presented.

The conclusion is that the Rational Method is still relevant and useful and should be continued to be used.

I. Introduction

This paper is only concerned with the urban Rational Method (uRM).

The publication ‘Australian Rainfall and Runoff (ARR)’ is a national guideline for the estimation of design flood characteristics in Australia. It is published by Engineers Australia and is being revised.

On the basis of the advice in Goyen,(2014), the AR&R revision team proposes to limit the use of the urban Rational Method (uRM) to the design of no more than two allotments.
The implication of this proposal is that computer models will be required to be used for the design of developments and for the design of relief drainage, for all areas larger than two allotments. This would encompass the majority of drainage design in Australia.

The advice is based on an introspective consideration of the history of the use of the uRM in previous editions of AR&R and an 1989 review of the possible effects of differences between the uRM procedures recommended in 1977 ARR and 1987 ARR, undertaken in the ACT.

The advice does not consider the current use of the uRM by urban drainage authorities in Australia except to say:

‘Other authorities restrict the application of the urban Rational Method to urban catchments less than 400 hectares and in the case of some Councils this is further restricted to less than 1 hectare.’

The only reason given for the view of the authors above, that continued use of the Rational Method for urban drainage analysis and design can no longer be justified, is that studies have not been carried out in the last 25 years on a significant number of additional gauged urban catchments. This position cannot be justified.

II. History of the Rational Method

The following has been adapted from Gomez, (2014).

The Rational Method is the most-employed formula of engineering hydrology.

Prior to 1850, investigators, primarily in the British Isles, concluded that the ratio of runoff to rainfall, at least for a particular watershed, might be approximated as a coefficient, typically in the 0.4 - 0.6 range for natural catchments. Such a coefficient is the first principle of what would become known as the Rational Method.

In 1851, Thomas Mulvaney presented the second principle, the role of the runoff’s time of concentration in quantifying the storm event, to the Irish Institution of Civil Engineers. Mulvaney proposed that,

\[ Q = C_{ave}A_{cont} \]  

Eq. (1)
where $Q$ (in Imperial units) is watershed runoff rate in cubic feet per second (cfs), $C$ is a dimensionless runoff-to-rainfall coefficient between 0.00 and 1.00, $i_{ave}$ is rainfall intensity in inches/hour averaged over the time of concentration and $A_{cont}$ is contributing watershed area in acres.

The method is also known as the Kuichling formula in the United States in honour of Emil Kuichling who applied it for sewer design in Rochester, NY, 1877-1888. The method is known as the Lloyd-Davies formula in the United Kingdom in honour of D.E. Lloyd-Davies who wrote about it in 1906.

Most engineering techniques are revised, if not revolutionized, every few generations. Mulvaney's work persists. The method's basis, however, has no more verification than that of one small parking lot study by Johns Hopkins University (ASCE, 1996, p. 582). Mulvaney's work persists because of its ease, not its confirmation.

Equation (1) is unsystematically dimensional, employing feet, seconds, inches, hours and acres. If each right-hand-side variable is taken as 1.0, runoff is 1.0 acre-inch/hour, which converts to 1.00833 cfs, or 1 cfs in any practical sense. Some authors attribute the title “rational” to the closeness of 1.00833 to unity, ie it has been rationalised by not including the coefficient 1.00833 in the formula.

The equation might more appropriately be known as the “Coincidental Imperial Units Method”. It could also be called the “Close Enough for all Practical Purposes Method”.

Equation (1) can be formulated for any dimensional system by an appropriate constant. In SI form, Eq. (1) becomes,

$$Q = C_i A_{cont} \frac{i_{ave}}{360}$$

Eq. (2).

where $Q$ is in cubic metres/second, $i_{ave}$ is in millimetres/hour and $A_{cont}$ is in hectares and 360 is for units conversion.
Tables summarize C for various land treatments, the latter term not necessarily implying human intervention. Many of the slope-unspecified values have been reprinted from one reference to the next for more than 50 years. Check your hydrologic references for “Railroad Yards” in the C table. If you find it, you have this data set.

**III. Limitations of the Rational Method**

The following is from QUDM, (2013):

The Rational Method provides a simple means for the assessment of the peak discharge rate for design storms, but does not provide a reliable basis for the determination of runoff volume, hydrograph shape, or peak discharge rates from historical (real) storms.

Use of the Rational Method is generally not suitable for the following applications:

- analysis of historical storms
- design of detention basins
- catchments of unusual shape—refer to section 4.7
- catchments with significant, isolated areas of vastly different hydrologic characteristics, such as a catchment with an upper forested sub-catchment and a lower urbanised sub-catchment
- catchments with significant floodplain storage, detention basins, or catchments with wide spread use of on-site detention systems
- urban catchments with an area greater than 500 hectares
- catchments with a time of concentration greater than 30 minutes where a high degree of reliability is required in the hydrologic analysis.

In QUDM there are 13 examples of catchments where application of the Rational Method is generally not recommended.

Also:

Technical note 4.6.1

Use of the Rational Method is generally not recommended for urban catchments greater than 500 ha, or rural catchment greater than 25 km².
Hydrologic analysis of urban catchments greater than 500 ha should be performed using a combination of suitable runoff-routing modelling and dynamic hydraulic modelling. Designers should refer to the latest recommendations of Australian Rainfall and Runoff.

Reference should also be made to Rossmiller, (1980), McPherson (1969) and Jones (2006).

The following is from Rossmiller, (1980):

The most outstanding limitation is that the only product of the method is a peak discharge. The method provides only an estimate of a single point on the runoff hydrograph.

Another limitation is that the results are usually not replicable from user to user. There are considerable variations in interpretation and methodology in the use of the formula. The simplistic approach permits and requires a wide latitude of subjective judgment in its application. Each firm or agency has its favorite formula, its favorite table for determining C, its own method for determining the tributary area and its own set of criteria for determining which recurrence interval is to be used in certain situations.

The average rainfall intensities used in the method bear no time sequence relation to the actual rainfall pattern during a storm. The intensity - duration - frequency (I-D-F) curves prepared by the Weather Bureau are not time sequence curves of precipitation. The maximums of the several durations as used in the method are not necessarily in their original sequential order; and the resulting tabulations of maximums ordered by size or duration may bear little resemblance to the original storm pattern. In many, if not most, cases, the intensities on the same frequency curve for various durations are not from the same storm.

The method assumes that the rainfall intensity is uniform over the entire watershed during the "duration" of the storm. This assumption is true only for small watersheds and time periods, thus limiting the use of the rational formula to small watersheds. Whether "small" means 20 acres or 200 acres is still being discussed.

The method also assumes that the runoff rate reaches a maximum at a time equal to tc. This assumption is true only when equilibrium conditions exist, which seldom occur during a thunderstorm, except over small areas, again limiting the usefulness of the rational formula.

The following is from McPherson, (1969):

SOME LIMITATIONS OF THE RATIONAL METHOD

From the examples given in Section 6, it is evident that there are considerable variations in interpretation and methodology in the use of the rational method. The simplistic scope of the method permits and requires a wide latitude of subjective judgment in its application. The ranges of interpretation and methodology might be narrowed by formalizing national or regional "standards." In
In that context, why has no group of civil engineers presented detailed "cookbook" instructions for using the rational method? Why are expositions of procedures for general practice couched in obtuse or indirect language?

To codify is to formalize, and formality tends towards rigidity. The writer, for one, would be extremely loathe to be a party to any codification attempt, because the need is for better methods, not improved consensus on an inadequate status quo. He suspects that this view is shared by the majority in the civil engineering profession who are involved in drainage design.

The great majority of drainage areas in U.S. cities are of small size, for which the estimated time of concentration is relatively short. The steepest portion of intensity duration-frequency curves is for short durations. Design criteria can be seemingly raised by shifting from, say, a 5-year curve to a 10-year curve, but if design inlet time is simultaneously increased the sizes of sewers designed under the "higher standards" may be little different from sizes that would be selected using the original criteria. An equal compensation can be achieved by merely lowering the prevailing schedule of C-values, or by means of a combination of both steps. In confidence, the writer has learned of instances where the C-values of sewer extension programs were adjusted downward until the cost of the planned facilities coincided with available budgets or limits set in bond issue referenda. At any rate, in order to compare the design criteria of one community with another it is necessary to obtain detailed information and sample computations, and even then the task can be exceedingly difficult. Generally speaking, there are no "wrong" storm sewer designs when the rational method is employed.

Improved methods are evolving very slowly because of a dearth of field measurements of rainfall-runoff from which logical, reliable models and quantification of their principal parameters can be developed. No municipality has had the financial and manpower resources to mount a research program suited for national transferability of results, and currently no prospect is in evidence of a consortium of local governments financially prepared to undertake the task.

An outstanding limitation of the rational method is that the product is restricted to a peak flow. By heaping more assumptions on the method a hypothetical hydrograph can be computed, such as by arbitrarily proportioning the contributing area with time, but few designers are willing to elect this option.

The major alternative to expressing collected surface runoff through conduits is to employ various forms of detention storage. Rather than provide the usual drain capacity to remove the increased flows resulting from urbanization, Bauer has suggested that it may be more economical in some instances to provide local detention storage, releasing the flood waters over a larger period of time than the inflow, at a reduced peak and with a consequent reduction in receiving drain capacity. The turf areas separating pavements have been exploited to produce "ponding" of overland flow, with a consequent savings in drain size, at airports, highway interchanges and for shopping centers. Jones proposed "planned integration of permanent water areas in open spaces, with
provision for flood storage.” Daily and Associates of Champaign, Illinois, investigated the possibility of using detention basins in the suburban, growing areas adjacent to the City of Memphis, Tennessee. Consideration included use of shallow sodded detention basins near schools and in parks, to provide the dual purpose of flood mitigation and recreation. Effective design of detention storage facilities requires use of complete runoff hydrographs, eliminating the rational method from consideration. Complete sewer outlet hydrographs are also needed for designing local flood protection works along streams, such as pumping stations for passing local drainage flows over levees and dikes. Because flows from sewer outlets comprise major components of total receiving stream flow at many cities, complete sewer outlet hydrographs are often needed for design of stream and river development works. Here, again, the rational method is of little or no value or utility.

Savini and Kammerer have reported on a review, classification, and preliminary evaluation of the significance of the effects of urbanization on the hydrologic regimen. Quantification of many of these effects is dependent upon the availability of sewer outlet hydrographs, where the rational method is again found wanting, such as for evaluation of the progressive effect of urbanization on receiving stream flood characteristics and the magnitude of volumes of water passed through outlets.

Public concern on urban surface water runoff as a source of pollution is steadily increasing, particularly with regard to overflows from combined sewers and discharges from separate storm drains. However, sanitary engineers have been attempting to indicate the extent of pollution from combined sewer overflows for a number of years. Pollutational loading in units of weight as a function of time must be evaluated as the product of the time-history of contaminants concentration and the runoff hydrograph. The absence of a basis for developing a hydrograph using the rational method precludes its application in pollution loading evaluations; and for the same reasons this applies to sediment transport evaluations as well.

As urban water management problems become increasingly acute, the need for multiple-use of water becomes more evident. In exchanging one use for another, for example using stormwater as a source of water supply, knowledge of the time-history of flows is essential for a reliable design of transfer facilities. It should go without saying that knowledge of the time-history of the water quality is also essential. A hydrograph is needed for both aspects, and the rational method is once again inappropriate.

In Section 7, it was noted that in order to check the rational method in a literal sense using field gaugings, it would be necessary that a rainfall of the design intensity for the design time of concentration occur on the gauged catchment. The probability of this occurrence within a period of several years is quite low. The merits of the rational method can be extolled in endless dialogues, but the simple fact that it is almost impossible to verify its true veracity cannot be circumvented. That use of the method tends to result in marginally acceptable designs is not a credit to the rather crude concepts underlying its logic. Instead, the few gauging results available suggest that collection systems tend to modulate flow peaks so that they spread over only a modest range, compared with natural catchments. Further, systems as designed appear to have more inherent "elasticity" than is generally suspected. For
example: designs are actually for a no-flooding condition; and the joint probability of simultaneous flooding in all tributary areas of a catchment can be much lower than the probability of flooding in some one part of the catchment.

There is a strong temptation to attack the rationale behind the rational method on hydrologic grounds, but there presently being nothing better to offer this would not be constructive criticism. However, one of its features should be considered further. When the rational method is applied literally, it models a process that can be designated as a linear system. The outputs of a linear system of this type are attenuations and prolongations of its inputs. An identity exists between the statistical properties of the modes (maximum values) of the input and output distributions. For example, the 5-year frequency peak flow would result from the 5-year maximum average rainfall intensity, in the literal use of the rational method. Hence, it is mathematically legitimate to speak of the 5-year peak flow, and this is more useful than referring to rainfall intensity because flow rates are relatable to flooding protection.

Kutchling recognized that the true C-value would be affected by antecedent precipitation and soil moisture conditions. Also, the true C-value could be a function of the rate of rainfall and the rate of runoff, in which case the simple system would be non-linear and a 5-year runoff peak would not necessarily coincide with the occurrence of a 5-year rainfall intensity. All this is to say simply that in the usual application of the rational method the existence of a linear system is implicitly assumed and the ascribing of the input frequency to corresponding outputs is consistent and proper under the circumstances. However, it should be evident that elaborations cannot be added to the rational method without risking further uncertainties about the frequency of flows computed thereby.

Lastly, the pure simplicity of the rational method should be considered. There must be thousands of persons engaged in sewer capacity determinations in the U.S. The majority probably has a very limited knowledge of hydrology, and in many offices routine determinations must necessarily be performed by technicians rather than professional engineers. When better methods become available, there will be a large number of elite designers who will use them simply because they are superior, regardless of their complexity or sophistication. Computer utilization for design is becoming commonplace, particularly in the offices of larger cities and consulting firms.

The ASCE Urban Hydrology Research Council fully realizes that the product of urban storm drainage research must be expressed in simple, direct, unambiguous terms that can be applied across the country by the typical designer. Unfortunately, the typical designer is not fully aware of all of the limitations of the rudimentary rational method he is now using, and much less of the research needs. The following plan should be implemented: a) assess research needs as viewed by elite designers, planners and managers of urban water resources; b) obtain precise field data from a number of representative catchments across the nation on the rainfall-runoff-quality process; c) develop mathematical models using the field data; d) from research on metropolitan storms develop suitable inputs for the models; e) synthesize on a regional basis the model and input-output relations in summary form for
the various regions of the country; f) reduce the findings to straightforward and explicit vehicles, such as charts and graphs, for their expeditious use in general practice; and g) promote the use of the more reliable methods for general practice and employment by the elite of the basic, more comprehensive research results.

The ASCE Urban Water Resources Research Program, under the guidance of the ASCE Urban Hydrology Research Council, has undertaken the above item a) with the help and financial support of the USGS, together with a study of instrumentation and data network needs for item b). With the support of a first-year contract with OWRR, research needs on items c) and d) have been defined. Implementation of item b) is absolutely essential, simultaneously with items c) and d), to get to items e), f) and g). Item b) remains the outstanding obstacle to design progress in the urban drainage field.

In closing this chapter, it is appropriate to note that because the rational method is not applicable for catchments smaller than an inlet area, it is not usable in certain urban runoff management, such as the use of roofs and yards for deliberate detention storage, as a part of appropriate land development design.

IV. Bureau of Meteorology IFDs (2013)

The Bureau of Meteorology (BOM) released new Intensity-Frequency-Duration (IFD) data for Australia in July 2013. The BOM website includes information on Engineers Australia guidance as follows:

The aim of the 1987 edition of Australian Rainfall and Runoff (AR&R87) design flood estimation techniques is to achieve Annual Exceedance Probability (AEP) neutrality—where the technique results in a design flood estimate with the same probability of exceedance as the IFD design rainfall estimate. All the other design flood inputs from AR&R87 were developed specifically to achieve this aim. The revised AR&R aims to achieve AEP neutrality, and so updates to the other design flood inputs are needed to ensure new design flood estimates are produced with the same AEP as the new 2013 IFD design rainfalls.

You cannot assume that using the 2013 IFD design rainfalls with AR&R87 techniques and design parameters will deliver a more reliable estimate of the design flood.

In most cases it would be prudent to use the AR&R87 design parameters and conduct sensitivity testing with revised AR&R design parameters (including the 2013 IFD design rainfalls) as they become available. This will allow you to assess the impact of updated information on your decisions.

The 2013 IFD design rainfalls should definitely NOT be used in conjunction with the following techniques:

- Probabilistic Rational Method
• Other regional flood techniques based on AR&R87 IFD design rainfalls.

If you are seeking consistency across a number of flood estimation studies, you should continue with the AR&R87 design parameters and do sensitivity testing with the 2013 IFD design rainfalls until the entire suite of new AR&R techniques and design parameters is available.

If you are undertaking a one-off flood estimation study you may choose, on a case-by-case basis, to use the 2013 IFD design rainfalls and other revised AR&R design parameters as they become available.

No information is provided on the reasons why the 2013 IFD design rainfalls should definitely NOT be used in conjunction with the Probabilistic Rational Method, nor is the Probabilistic Rational Method defined. The ARR revision team has been requested to provide clarification; however, a response has not been received.

V. Rational Method Interpretations

Aitken, (1975), discusses the differences between the two interpretations of the Rational Method:

• the deterministic interpretation, and;

• the statistical interpretation.

The two interpretations are discussed in the following sections.

There are 32 instances of the word ‘probabilistic’ in AR&R87 including in 13 instances of the term ‘probabilistic Rational Method’; however, there is no instance of the term ‘Probabilistic Rational Method’, ie with an upper case ‘P’.

VI. Deterministic Rational Method

The following is from Aitken, (1975):

THE DETERMINISTIC INTERPRETATION

The origin of the Rational Formula has been traced to an Irish engineer called Mulvaney (1850) who first stated the principle of the formula as follows:

'The first matter of importance to be ascertained in the case of a small or mountainy catchment, is the time which a flood requires to attain to its maximum height, during the continuance of a uniform rate of fall of rain. This may be assumed to be
the time necessary for the rain which falls on the most remote portion of the catchment, to travel to the outlet, for it appears to me that the discharge must be greatest when the supply from every portion of the catchment arrives simultaneously at the point of discharge, supposing, as-above promised, the rate of supply to continue constant, and this length of time being ascertained, we may assume that the discharge will be the greatest possible, under the circumstance of a fall of rain occurring, of the maximum uniform rate of fall for that time. This question of time, as regards any catchment, must depend chiefly on the extent, form, and rate of inclination of its surface; and, therefore, one great object for investigation is the relation between these causes and their effect; so that, having ascertained the extent, form and average inclination of any catchment, we may be able to determine, in the first place, the duration of constant rain required to produce a maximum discharge, and consequently to fix upon the maximum rate of rain-fall applicable to the case.’

In modern jargon Mulvaney's formula is referred to as the deterministic interpretation of the Rational Formula (RFDM) as it proposes a deterministic relationship exists between the rainfall on a catchment and the peak discharge resulting from that rainfall. as a mathematical formula we have:

\[ Q = \frac{1}{360} CIA \]  \hspace{1cm} (3.1)

where \( Q \) = peak rate of runoff in cubic metres per second

\( C \) = coefficient of runoff

\( I \) = mean storm rainfall intensity for the catchment time of concentration in millimetres per hour

\( A \) = catchment area in hectares

This formula is commonly used in Australia to estimate peak runoff rates from observed storms and even hydrographs. Watkins (1962) also used the Rational Formula in this manner when comparing it with the results obtained from the Transport and Road Research Laboratory Hydrograph Model (RRLM).

VII. Probabilistic Rational Method

The following is from Aitken, (1975):

THE STATISTICAL INTERPRETATION

In 1936 Horner & Flynt (1936) studied the relation between rainfall and runoff for three small urban catchments in the City of St. Louis, USA. A frequency analysis of the rainfall and runoff for the gauged catchments was carried out and the runoff intensity and the rainfall intensity for the same return periods were substituted into the Rational Formula to determine a value of the coefficient of runoff (C). Again in modern jargon this interpretation is referred to as the statistical model of the Rational Formula (RFSM). Most recent applications of the Rational Formula for both rural and urban catchment studies have used this version. The most
A thorough modern examination of the Rational Formula as a statistical model was given by Schaake et al (1967). In this reference it was presented as follows:

\[ Q = \frac{1}{360} CIA \]  \hspace{1cm} (3.2)

rearranging

\[ \frac{Q}{A} = \frac{1}{360} CI \]  \hspace{1cm} (3.3)

and rewriting

\[ q_p(T) = \frac{1}{360} CI(t_a,T) \]  \hspace{1cm} (3.4)

where \( Q \) = peak runoff in cubic metres per second

\( C \) = coefficient of runoff

\( A \) = catchment area in hectares

\( I \) = rainfall intensity in millimetres per hour for a return period \( T \) and duration \( t_a \)

\( q_p \) = peak runoff rate per unit area for a return period \( T \)

\( t_a \) = rainfall intensity averaging time

\( T \) = return period

The value of \( C \) in the above equations was determined from a frequency analysis of rainfall and runoff. In Schaake's work on urban catchments \( C \) was then related by regression analysis to the impervious area and slope as the independent variables.

In Book IV Section 1 ‘Estimation of peak flows for small to medium sized rural catchments’ of ARR87 it is stated:

1.3.2 Rational Method

(a) The Formula

As used in design, the formula of the Rational Method is

\[ Q_Y = 0.278C_Y I_{t_c, Y} A \]  \hspace{1cm} (1.1)

where

\( Q_Y \) = peak flow rate (m\(^3\)/s) of average recurrence interval (ARI) of \( Y \) years

\( C_Y \) = runoff coefficient (dimensionless) for ARI of \( Y \) years

\( A \) = area of catchment (km\(^2\))

\( I_{t_c, Y} \) = average rainfall intensity (mm/h) for design duration of \( t_c \) hours and ARI of \( Y \) years.
The value of 0.278 (or 1/3.6) is merely a conversion factor to balance the units used. If area is in hectares instead of km², the conversion factor is 0.00278 (or 1/360).

(b) Interpretation of the Rational Method

The method can be interpreted in two quite different ways. Recognition of this difference is important in understanding the method. In the past, it has often been regarded as a deterministic representation of the flood resulting from an individual storm. However, the value of the runoff coefficient $C$ varies widely from storm to storm on a given catchment, and the method is a poor deterministic model. The Rational Method is not used in a deterministic manner to represent floods from individual storms in this document, and generally should not be used for this purpose.

The Rational Method is presented herein as a probabilistic or statistical method for use in estimating design floods. It is used to estimate a peak flow of selected ARI from an average rainfall intensity of the same ARI derived from Book II Section 1. The runoff coefficient represents the ratio of a peak flow and a rainfall rate of selected duration determined for the same ARI from frequency analyses of flood peaks and rainfalls. This is why $Q$, $I$ and $C$ in equation (1.1) are subscripted by $Y$ to represent the ARI. This probabilistic interpretation of the Rational Method and the runoff coefficient exactly fits the way in which the method is used in design practice. Even when it is not recognised, estimation of a design flood from rainfall frequency data such as those in Book II Section 1 involves use of the Rational Method as a probabilistic model.

This probabilistic interpretation of the Rational Method for estimating design floods does not make sense, since even a recorded storm has an associated ARI and therefore, there is no real difference between these probabilistic and deterministic interpretations of the Rational Method.

However, Book IV Section 1 of ARR87 goes on to say:

(c) Values of the Runoff Coefficient

From equation (1.1), the value of the runoff coefficient is given by:

$$C_Y = \frac{Q_Y}{0.278 A I_Y Y}$$  \hspace{1cm} (1.2)

Values of $I_{1C,Y}$ for all of Australia can be found from Book II Section 1. For several regions with adequate streamflow data, flood frequency analyses have been carried out for many small to medium sized catchments. From $Q_Y$ values obtained by these analyses, values of $C_Y$ have been determined, and the resulting design data and methods for these regions are included in the recommended procedures in Section 1.4. The catchment and rainfall characteristics and conditions affecting the relation between $Q_Y$ and $I_Y$ are automatically incorporated in $C_Y$, but not necessarily in a physically realistic fashion. Derived values of $C_Y$ have generally been
found to vary in a reasonably regular or consistent manner over the range of ARI values on a given catchment, and for different catchments over a particular region. Therefore, they provide a suitable basis for design.

For a design procedure, values of $CY$ must be derived from observed flood data and the rainfall data in Book II Section 1. This fulfills the primary requirement for recommendation of procedures noted in Section 1.1, and is in contrast to the rather arbitrary values of $C$ based on experience or judgement that have often been recommended in the past. It is noteworthy that values of $CY$ derived from observed flood data often show much less dependence on variations in catchment characteristics such as slope and soil or vegetation type and condition than is assumed in arbitrary or handbook values of $C$.

For the probabilistic Rational Method, values of $CY$ depend on the rainfall frequency data used, as shown by the term $Itc,Y$ in equation (1.2). If different rainfall data are used, or duration $tc$ is derived in a different fashion, different values of $CY$ are required. The values recommended herein assume use of the rainfall data in Book II Section 1, and determination of the values of $tc$ by the procedure specified for each method.

In some regions of Australia, sufficient streamflow data are not available for derivation of $CY$ values, or the required analysis has not been carried out. For these regions, it has been necessary to recommend rather arbitrary values based on judgement, with some checking against observed data where possible.

The following is from Pilgrim (1989):

**Probabilistic rather than deterministic interpretation**

With methods in which a flood is estimated from a design rainfall, there are two quite different interpretations of the method applying to two different types of problems. In the first, the flood resulting from a particular storm is estimated, and the answer depends on the antecedent precipitation and the prevailing conditions on the basin. This is a deterministic problem and the method is used as a deterministic model.

The second type of problem is where a flood of a selected probability is estimated from a design rainfall of the same probability. The method is here used as a probabilistic model, and this is the interpretation that is almost always required and used in design. Although the computational procedure may be virtually the same in both cases and the differences in the two types of problems are often not recognized, there are three important practical consequences of the differences in the interpretations. Firstly, a particular procedure may be good or satisfactory for one case, but quite unsuitable for the other. As discussed later the Rational Method using the probabilistic interpretation can be a very satisfactory approach for estimating design floods, but is not satisfactory for estimating the flood resulting from a given rainfall. Most of the criticism of the Rational Method has been based on a deterministic interpretation. The second consequence concerns the manner in which values of parameters are derived from observed data, the manner in which the values are applied and the manner in which the method is tested. Many design methods have been tested by
their ability to reproduce particular observed events. This is inappropriate, as illustrated by Hoesein et al. (1989) with the US Soil Conservation Service Method. Thirdly, the interpretation of a method affects the manner in which its parameters are viewed by designers and analysts. For example, the common visualization of the runoff coefficient as the fraction of rainfall that runs off is correct in the very unusual case where the Rational Method is used to estimate an actual flood, but is quite incorrect in the design case, as discussed later.

It is important that flood estimation methods should be developed on a probabilistic basis if they are to be used to estimate design events. This principle is illustrated in a later section with the Rational Method as incorporated in Australian Rainfall and Runoff.

Further:

THE PROBABILISTIC RATIONAL METHOD

As used in design, the Rational Method formula is:

\[ Q(Y) = 0.278 \cdot C(Y) \cdot I(t_c, Y) \cdot A \]  

(3)

where the peak discharge \( Q \), average rainfall intensity \( I \) and area \( A \) have units of \( \text{m}^3\text{s}^{-1}, \text{mmh}^{-1} \) and \( \text{km}^2 \) respectively. The rainfall has a duration of \( t_c \), and \( C \), as well as \( Q \) and \( I \), has an average recurrence interval (ARI) of \( Y \) years. The intention in design is that the formula converts a rainfall with ARI of \( Y \) years derived from the design intensity-frequency-duration data for the region into a peak discharge with the same ARI. It is also the intention that this is the value that would be given for the ARI by a frequency analysis of observed floods if a long and representative record of discharge was available at the site. Both the design rainfall and peak discharge are probabilistic values derived from frequency analyses, and the design formula in (3) really has nothing to do with the runoff in a particular storm.

To derive design values of \( C \), equation (3) is rearranged as:

\[ C(Y) = \frac{Q(Y)}{I(t_c, Y) \cdot 0.278 \cdot A} \]  

(4)

Fig. 2 illustrates the derivation of the value of \( C(Y) \) for a particular ARI of \( Y \) years for a gauged basin. The figure shows frequency curves of recorded floods and of design rainfalls of duration \( t_c \). The rainfall would be from the design intensity-frequency-duration data for the site. With the appropriate scaling of \( (0.278A) \), the value of \( C(Y) \) is given by the ratio of \( Q(Y) \) to \( I(Y) \). The value of \( C(Y) \) will be different for different values of \( Y \). In all of the Australian studies, \( C(Y) \) has varied with ARI in a consistent and predictable manner that could be incorporated into the design procedure for each region. \( C(Y) \) derived in this manner exactly fits the way in which the runoff coefficient is used in design in transforming the \( Y \) year design rainfall into an estimate of the \( Y \) year flood.

Viewed in this way, the probabilistic Rational Method is conceptually a type of regional flood frequency procedure, with rainfall intensity as one of the independent predictor variables. As rainfall intensity is one of the major determinants of the flood
characteristics of a basin, this approach is an efficient form of regional flood frequency analysis, and is simple and familiar to most designers.

With this probabilistic interpretation, time of concentration as a physical measure of maximum travel time is not really relevant. However, (4) shows that $C(Y)$ depends on the duration of rainfall, and some design duration related to basin characteristics must be specified. A typical response time is appropriate, and $t_c$ is a convenient measure. In this context, its accuracy regarding travel time is much less important than the consistency and reproducibility of derived $C(Y)$ values. Also, values of $C(Y)$ cannot be compared unless consistent estimates of $t_c$ are used in their derivation.

The probabilistic interpretation of the Rational Method was developed by Horner & Flynt (1936), but was then neglected until the work of Schaake et al. (1967) with urban drainage basins and French (1967) with rural basins. The latter was extended by French et al. (1974), and the first complete design procedure for Australia was developed by McDermott & Pilgrim (1982) and Pilgrim & McDermott (1982) for eastern New South Wales, covering an area of approximately 1000 by 400 km. As noted earlier, equation (1) for $t_c$ was derived from minimum times of rise on 96 basins. Runoff coefficients were derived for a range of ARIs from observed floods on 308 basins. Possible relationships between $C(10)$ and basin characteristics were investigated, but the most satisfactory procedure was found to be the mapping of $C(10)$ values, and drawing contours or isolines. In this, consideration was given to the shape of isohyets of average annual precipitation and short duration design rainfall intensities, topography, soil type, and relative reliability of gauging station records. Comparison of the deviations between the derived values and values estimated from the contours with the expected errors in the flood frequency curves indicated that the mapped contours extracted virtually all of the available information from the frequency analyses of recorded floods. Real improvements in accuracy would require longer or more accurate records, or greater density of stations. When the new design rainfall data were developed for the 1987 AR&R, the $C(10)$ values had to be re-derived and the contours redrawn, as (4) shows that values of $C$ depend on the design rainfalls that are used. Fig. 3 shows the redrawn $C(10)$ contours for part of south eastern New South Wales. Average frequency factors were also derived for basins above and below 500 m elevation for each of six regions within eastern New South Wales. These enable $C(Y)$ values to be determined from $C(10)$ for a range of $Y$ from 1 to 100 years.

Although Pilgrim discusses the Probabilistic Rational Method as being used to determine the coefficient of runoff from recorded data, he does not use it in this way, but instead uses it in estimating design floods. There is no obvious reason for this use.

There is a different view of the Probabilistic Rational Method elsewhere in the literature, where it is used to determine coefficients of runoff.
Hodgkins, (2007) states the following:

**Probabilistic Rational Method**

In the application of the Rational Method, the estimated peak flow varies with the magnitude of the estimated runoff coefficient, $C$. Design $C$ values are commonly chosen on the basis of soil and land-cover types in a drainage basin (Pilgrim and Cordery, 1993). The Probabilistic Rational Method involves rearranging the Rational Method to

$$CT = \frac{QT}{ITA},$$

(2)

where

- $CT$ is the dimensionless runoff coefficient for a given return period, $T$;
- $QT$ is the observed peak flow for that return period, in cubic feet per second;
- $IT$ is the rainfall intensity in inches per hour estimated from a design rainfall intensity-duration-frequency curve for that location and return period; and
- $A$ is the area, in acres, of the drainage basin.

$CT$ values are developed for as many basins and return periods as regional data permit. Calculated values of $C$ for a selected return period (50 years, for example) can be related to basin characteristics by regression or mapped over a region (Pilgrim and Cordery, 1993).

The Probabilistic Rational Method was first developed by Horner and Flynt (1936) and later used by French and others (1974) on rural basins to derive regional $C$ values for varying return periods. In the Probabilistic Rational Method, $C$ represents the ratio of peak runoff of a given frequency to rainfall of the same frequency and a duration equal to the time of concentration (Dooge, 1973, p. 83). This method has been used extensively in Australia to derive $C$ values for small- and medium-sized basins. In a study of 271 drainage basins in Australia, estimates of peak flow for the basins were made with the Probabilistic Rational Method and with the traditional Rational Method. Values from the traditional Rational Method compared very poorly to values from the Probabilistic Rational Method (Pilgrim, 1989; Pilgrim and others, 1989).

For this study, $C$ values for 2-, 10-, 25-, 50-, and 100-year return periods were developed for the 17 USGS streamflow-gaging stations in Maine with drainage areas of 1.0 mi2 to 10 mi2 and a minimum of 10 years of peak-flow records (table 11). Drainage areas and 2-, 10-, 25-, 50-, and 100-year peak flows were taken from Hodgkins (1999). The peak flows were the best estimate of peak flows at each station (station values weighted with regression-estimate values). Rainfall intensities for the 2-, 10-, 25-, 50-, and 100-year return periods were taken from MaineDOT IDF curves for the nearest of six locations in Maine (State of Maine, 2005).
Schaake (1967) (referred to by Pilgrim, 1989) does not use the term ‘Probabilistic Rational Method’, however he does use it to determine coefficients of runoff from a series of recorded rainfall and runoff values.

Rahman, (2010) states:

In ARR1987, the probabilistic rational method (PRM) was recommended for general use in Victoria and eastern NSW (IEAust, 1987). The PRM in ARR1987 was based on research by Pilgrim (1982), Pilgrim & McDermott (1982) and Adams (1984).

Also:

2.1 Basis of probabilistic rational method

The rational method, introduced by Mulvany (1851), has been widely regarded as a deterministic method for estimating the peak discharge from an individual storm. However in ARR1987 (IEAust, 1987), it was presented as a probabilistic method (referred to as PRM). The rational method has often been recommended for application to only small catchments below some arbitrary limit such as 25 km2. This limited range of applicability reflects the inadequate manner in which the method considers physical factors such as the effects of temporary storage on the catchment, and temporal and spatial variations of rainfall intensity. These physical considerations have little relevance to the probabilistic interpretation of the PRM, where their effects are incorporated in the recorded floods, and hence in the flood frequency statistics and the derived values of the runoff coefficient $CY$. As mentioned in ARR1987, the PRM derived from observed data should be valid for catchment areas and average recurrence intervals (ARIs) up to and somewhat beyond the maximum areas and record lengths used in their derivation (IEAust, 1987).

In ARR1987, the PRM is represented by:

$$Q_Y = 0.278 C_Y I_{tc,Y} A$$  \hspace{1cm} (1)

where $Q_Y$ is the peak flow rate (m$^3$/s) for an ARI of $Y$ years; $C_Y$ is the runoff coefficient (dimensionless) for ARI of $Y$ years; $I_{tc,Y}$ is the average rainfall intensity (mm/h) for a time of concentration $tc$ (hours) and ARI of $Y$ years; and $A$ is the catchment area (km$^2$).

From equation (1), the value of the runoff coefficient is given by:

$$C_Y = Q_Y / \left(0.278 A I_{tc,Y} \right)$$  \hspace{1cm} (2)

The values of $Q_Y$ for a station can be obtained from at-site flood frequency analysis, subject to the availability of reasonably long streamflow records. Values for $I_{tc,Y}$ at a given location can be found from Book II, Section 1 of ARR. The catchment and rainfall characteristics and conditions affecting the relation between $Q_Y$, $A$ and $I_{Y}$ are incorporated in $C_Y$, but not necessarily in a physically realistic fashion.
In the deterministic interpretation of the rational method, the critical rainfall duration is $t_c$, which is considered to be the travel time from the most remote point on the catchment to the outlet, or the time taken from the start of rainfall until all of the catchment is simultaneously contributing flow to the outlet. For the probabilistic interpretation of the rational method, as in the PRM, these physical measures are not really relevant. However, equation (2) shows that the value of $C_Y$ depends on the duration of rainfall, and some design duration related to catchment characteristics must be specified as part of the overall procedure. A typical response time of flood runoff is appropriate, and the “time of concentration” is a convenient measure. In this context, its accuracy regarding travel time is much less important than the consistency and reproducibility of derived $C_Y$ values. Also, values of $C_Y$ cannot be compared unless consistent estimates of $t_c$ are used in their derivation. Pegram (2002) and French (2002) discussed various methods of estimating $t_c$, as well as the strengths and weaknesses of the PRM.

VIII. Criticism of the Rational Method

The following is from Aitken, 1975.

CRITICISM OF THE RATIONAL FORMULA

There has been widespread criticism of the use of the Rational Formula because it is a gross over-simplification of the rainfall-runoff process. This criticism usually takes the following form:

(a) The formula is illogical because it assumes uniform rainfall which rarely occurs.

(b) The formula incorrectly assumes that runoff is a fraction of the rainfall (as implied by $C$) rather than a residual after abstraction of losses.

(c) The formula does not account for storage effects, for example, in pipes.

(d) The determination of the value of $C$ is too difficult and too subjective.

It is important to note that while these criticisms are valid for the RFDM they do not carry the same weight when the formula is applied as a statistical model. The inadequacy of the Rational Formula as a deterministic model has been proved on many occasions. Perhaps the best example of this for an urban study was that by Watkins (1962) who showed the RRLM to be superior to the RFDM.

IX. Assumptions in the use of the Rational Method

The following is from Westphal, (2004).

There are several assumptions inherent to the Rational Method:

- The rainfall intensity is constant over a period that equals the time of concentration of the basin.

• The rainfall intensity is constant throughout the basin.
• The frequency distribution of the event rainfall and the peak runoff rate are identical (this assumption is true for all event-based computations).
• The time of concentration of a basin is constant and is easily determined (this assumption is also shared by other event-based methods).
• Despite the natural temporal and spatial variability of abstractions from rainfall, the percentage of event rainfall that is converted to runoff (the runoff coefficient, C) can be estimated reliably.
• The runoff coefficient is invariant, regardless of season of the year or depth or intensity of rainfall.

X. Misconceptions

It is often stated that the Rational Method gives larger flows than other methods and that this is a reason why it should not be used. This is not the case in South East Queensland. For Brisbane and using the ILSAX method, the following parameters are used:

• Soil type 3
• AMC of 4 (rainfall in the 5 days prior to a storm is likely to exceed 25 mm).

The flows generated are greater than those using the Rational Method estimates (O’Loughlin, 2014). An example is shown in the DRAINS manual.

BCC City Projects Office has recently confirmed this for Brisbane in testing of model hydrology methods. The testing is on-going.

The following is from Rossmiller, (1980):

Assumptions and Misconceptions

Assumptions and misconceptions are grouped together because an assumption used in the rational formula might in itself be a misconception or could be a conclusion based on some misconception. Several assumptions are listed below with each followed by a brief discussion.

The peak rate of runoff at some point is a direct function of the tributary drainage and the average rainfall intensity during the time of concentration to that point. This is the rational formula stated in words and is the basis of Kuichling's 1889 paper. Sufficient data, both rainfall and runoff records, have not been available to either prove or disprove this hypothesis.
The method assumes that the frequency (recurrence interval) of the peak discharge rate is the same as the frequency of the average rainfall intensity. This is not always the case due to watershed related variations. However, this assumption is used in many methodologies for estimating peak flows or runoff hydrographs.

The runoff frequency curve is parallel to the rainfall frequency curve. This implies that the same value of the runoff coefficient C is used for all recurrence intervals. However, work done by Schaake, Geyer and Knapp indicates that the two curves tend so converge at the rarer frequency rainfall events.

Each of the variables (C, i, A) is independent of each other and each is estimated separately. This is one of the major misconceptions. There is some interdependency among the variables. Present procedure is to estimate each variable separately from an equation, graph, map or table. A close look at these aids indicates, in most cases, a lack of recognition of any interdependency between these variables.

The time of concentration tc is the time required for water to flow from the hydraulically most remote point in the watershed to the point of design. Rather than an assumption, the foregoing statement is usually given as the definition of tc. However, Schaake, Geyer and Knapp have stated that there is no known way to determine tc, either from measurements in the field during storms or from records of rainfall and runoff and

"except for steady state conditions, which rarely, if ever, are reached during a thunderstorm, there is no good reason to believe that the time of flow from the farthest point in a drainage area should necessarily be the best rainfall averaging time to use in the Rational Method."

The rainfall intensity remains constant during the time period equal to tc. Based on rainfall records, this assumption is true for short periods of time, such as a few minutes. However, as the time period increases, this assumption becomes less and less realistic.

The above assumption has led to another assumption: the definition of i in the rational formula. A common definition is the rainfall intensity in inches per hour of a storm whose duration is equal to the time of concentration of the basin. This definition evolved from current practice OR current practice evolved from this definition.

"Duration" has been placed in parenthesis because the interpretation placed on "duration": has led to the worst misconception of all.

The common interpretation is that the duration of the storm is 3 equal to tc. This assumption is totally false and misleading. It is, of course, theoretically possible, since rainfall is a random event; however, the much more common case is that the total storm duration is considerably longer than tc. Of equal importance is the concept that tc (rainfall intensity averaging time) can occur during any segment of the total storm duration - at the beginning, before, during or after the middle portion or near the end.
This concept also has implications for the runoff coefficient C and how well the rational formula mirrors the hydrologic cycle. If t occurs at the beginning of the storm, then the antecedent moisture conditions become important. If t_c occurs near the end of a long storm, then the ground may be saturated and the depressions already filled with water when t_c begins.

Another assumption and misconception is that the area to be used is the total area tributary to the point of design. Kuichling recognized this possibility when he stated that

“the conclusion is accordingly irresistible that the rates of rainfall adopted in computing the dimensions of a main sewer must correspond to the time required for the concentration of the drainage waters from the whole tributary area when small, or from so much thereof as will produce an absolute maximum discharge when the area is very large.”

Time of concentration formulas estimate t_c. Unfortunately, many times this assumption is just not true. T_c consists of an inlet time plus flow time. Inlet time consists of the time required for water flowing overland to reach established surface drainage channels, such as ditches and street gutters, plus travel time through them to the point of inlet to a storm sewer. Flow time is the time of flow through the storm sewer to the point of design. Even though many equations purportedly yield t_c, some estimate only overland flow time or inlet time.

The rational method assumes that runoff is linearly related to rainfall. If rainfall is doubled; runoff is doubled. This is not really accurate, for many variables interact.

One last major misconception is that the runoff coefficient C is a constant. C is a variable and during the design of a storm sewer system, it should take on several different values for the various pipe segments, rather than retain a constant value throughout the entire design, even though the land use remains the same.

**XI. Computer Models**

The Rational Method is used in various computer models in the hydrology component as either the only option or as one of a number of options.

XP Solutions in their Hydrology 101 Fact Sheet list the following hydrology methods:

- Rational Method
- Unit Hydrographs – Time Area
- Laurenson Method
- SCS Method
The problem common to all computer models is summed up in McPherson, (1974), as follows:

Over and above the storm definition problem is the inherent difficulty with any runoff model in the necessarily subjective separation of abstractions (infiltration, depression storage, etc.) from total rainfall to resolve rainfall excess (amount and pattern) which is the input from which an equal volume of direct runoff is generated by models of one kind or another. This problem is greatly aggravated by the extreme scarcity of field data for calibrating and verifying models of all types.

XII. Revision of Australian Rainfall and Runoff

The publication ‘Australian Rainfall and Runoff (ARR)’ is a national guideline for the estimation of design flood characteristics in Australia. It is published by Engineers Australia and is being revised.

In Goyen, (2014), it is stated:

Should the Urban Rational Method continue to be included in ARR?

The Rational Method has been included in each of the Australian Rainfall and Runoff documents since the release of the first edition in 1958. The ARR 1987 recommendations for the application of the urban Rational Formula are somewhat vague. They state that appropriate uses include design of small and medium street drainage systems, and large property drainage systems. Other authorities restrict the application of the urban Rational Method to urban catchments less than 400 hectares and in the case of some Councils this is further restricted to less than 1 hectare.

Since the publication of 1987 ARR a number of water authorities as well as Councils have also published their own recommendations on how the Rational Method should be applied to urban catchments in their jurisdiction. Typically these guidelines recommend procedures for estimating runoff coefficient and time of concentration which differ from those recommended in the 1987 ARR. It is unclear if these guidelines are based on a comprehensive study of one or more gauged urban catchments or whether values which are somewhat arbitrary and based on intuitive judgement rather than adequately controlled experiments (as concluded in the 1958 ARR).

There are, however, a number of problems associated with the use of the Rational Method. Most of these problems are associated with the estimation of parameter values such as the time of concentration and the runoff coefficient. As a result, the Rational Method may be easy to implement, but it is difficult to ensure that the predictions adequately represents processes occurring in the catchment.
In 1989 a review the possible effects of differences between the urban Rational Method procedures recommended in 1977 ARR and 1987 ARR was undertaken in the ACT. A key conclusion of this Part I study was that the runoff coefficient and time of concentration relationships are paired i.e. They both need to be derived concurrently using gauged data rather than derived relationships independently.

It was concluded from a preliminary updated analysis of the Giralang catchment in the ACT that

- The 1977 ARR procedures give peak flows which match the peak flows adopted for the composite series based on flood frequency analysis (FFA) except for flows based on EIA only;
- The 1987 procedures give peak flows lower than the peak flows adopted for the composite series based on flood frequency analysis with the estimated 100 yr ARI peak flow comparable to the 10 yr ARI peak flow from the FFA;
- For 10 yr ARI and above the 1987 ARR procedures give similar peak flows to the ARR Project 5 procedures for rural catchments.

The preliminary assessment of gauged urban catchments in Sydney, Melbourne and Darwin disclosed that in general the 1977 ARR Rational Method gives peak flows which better match the peak flows calculated by flood frequency analysis (FFA) than the 1987 ARR Rational Method (refer Appendix C).

Notwithstanding that the 1989 Part I study concluded that the results from the study lent further support to the continued use of the Rational Formula for drainage design in small to medium sized urban catchments this was on the basis that further studies be undertaken to further examine possible modifications to the recommended 1987 ARR procedures to improve the estimation of surface flow times of concentration and corresponding runoff coefficients. In particular, it recommended that further studies should aim to determine appropriate surface roughness values for use in the kinematic wave formulation for overland flow in Australia.

These further studies have not been undertaken in the 24 years since.

Without carrying out similar studies to the Part I study undertaken in the ACT on a significant number of additional gauged urban catchments then it is the view of the authors that continued use of the Rational Method for urban drainage analysis and design can no longer be justified.

On the basis of this advice, the AR&R revision team proposes to limit the use of the urban Rational Method (uRM) to the design of no more than two allotments.
The implication of this proposal is that computer models will be required to be used for the design of developments and for the design of relief drainage, for all areas larger than two allotments. This would encompass the majority of drainage design in Australia.

The advice is based on an introspective consideration of the history of the use of the uRM in previous editions of AR&R and an 1989 review of the possible effects of differences between the uRM procedures recommended in 1977 ARR and 1987 ARR, undertaken in the ACT.

The advice does not consider the current use of the uRM by urban drainage authorities in Australia except to say:

Other authorities restrict the application of the urban Rational Method to urban catchments less than 400 hectares and in the case of some Councils this is further restricted to less than 1 hectare.

The only reason given for the view of the authors above, that continued use of the Rational Method for urban drainage analysis and design can no longer be justified, is that studies have not been carried out in the last 25 years on a significant number of additional gauged urban catchments. This position cannot be justified. If the same argument was used, then the continued use of Manning’s equation in hydraulics would no longer be justified. Clearly, this is not a reasonable proposition.

By the way, Manning’s equation was proposed in 1889 in a paper presented by Robert Manning at a meeting of the Institution of Engineers in Ireland (Yen, 1992).

Also in Goyen, (2014), it is stated:

**Should the Urban Rational Method be used to Calibrate Hydrological Models?**

With the advent of PCs in the 1980s and the improvements in computer speed and capabilities since that time as well as the continued development of urban rainfall runoff catchment simulation models, computer based modelling has almost totally supplanted the role of Rational Method calculations in urban drainage design. Notwithstanding these advances some authorities still require urban hydrological models to be “calibrated” to match peak flows estimated using the 1987 ARR urban Rational Method.
It is the view of the authors that the urban Rational Method should not be used to calibrate urban hydrological models unless it can be demonstrated that:

(i) A detailed Part I study has been undertaken on one or more gauged urban catchments in the relevant city or town which has calibrated and validated relations for the calculation of runoff coefficients and times of concentration; and

(ii) The urban catchment which is being modelled is subject to a similar hydrological regime and has a level of imperviousness comparable to the gauged urban catchment(s) analysed in the Part I study; and

(iii) WSUD measures are not present in the urban catchment which is being modelled.

It is agreed that the urban Rational Method should not be used to calibrate urban hydrological models

XIII. Current use of the urban Rational Method

The uRM is used by most urban drainage authorities and councils and a large number of consultants throughout Australia. A limited search was undertaken of manuals/guidelines produced by some of these authorities to determine the limit of catchment area for urban drainage design. The results are as follows:

- Austroads 100 ha
- Melbourne Water 400 ha
- QUDM 500 ha
- ACT For catchment areas greater than 50 hectares, two recognised flow estimation methods shall be used for comparative purposes.
- ARRB (Argue, 1986) 20 ha
- Darwin City Council All in accordance with AR&R.
- Qld Transport and Main Roads 100 ha (urban creeks)

XIV. Queensland Urban Drainage Manual

The Queensland Urban Drainage Manual (QUDM, 2013) has a strong emphasis on the use of the uRM in Queensland. It is known that QUDM is also used outside Queensland.
QUDM has extensive information and data on the use of the uRM covering approximately 30 pages of information and guidance on the use of the uRM, including many charts and tables.

QUDM has a catchment limit for the use of the Rational Method of 500 hectares (5 km²) for urban catchments, and 25 km² for rural catchments.

XV. Brisbane City Council City Plan 2014

City Plan 2014 includes the following, in Chapter 7 Stormwater drainage, of the Infrastructure design Planning Scheme Policy (PSP):

7.3.2 Flow estimation methods

(1) For guidance to the design of urban drainage systems Council refers the designer to QUDM and Australian Rainfall and Runoff. Council will accept flow estimations using the rational method or from run-off or storage routing models (e.g. DRAINS, ILSAX, XP-RAFTS, WBNM, RORB).

(2) For complex drainage situations (particularly as part of a flood study for setting building development levels) or for sizing stormwater detention systems, a run-off storage routing model must be used to estimate flows and/or analyse the hydraulics of an urban drainage system.

For Hydrology and Hydraulics and Rational Method assumptions in City Plan 2014, refer to Appendix A of this paper.

XVI. Development Assessment in BCC

The Development Assessment Section of Brisbane City Council assesses applications for developments to ensure they are produced in accordance with the requirements of City Plan 2104.

The Development Assessment Section of Brisbane City Council accepts any hydrology/hydraulic method in AR&R and QUDM.

Refer to Section IV for limitations of the Rational Method in QUDM.

A spreadsheet tool was developed to enable the quick calculation of urban runoff to check on submitted results in development applications.
XVII. Drainage Planning and Design in BCC

The BCC Drainage Design Section uses the uRM for drainage investigations, following the receipt from residents of complaints of local flooding. These cases often involve significant catchments of several hectares in area. The use of the uRM allows for simple procedures that can be used by drainage designers and engineers without the need for complex hydrologic/hydraulic modelling by expert modellers.

The BCC Drainage Design Section also uses the uRM for the design of relief drainage schemes. This is done using PC Drain which relies on the use of the uRM exclusively. PC Drain enables the preparation of design drawings, including longitudinal sections and calculation tables. Again the procedures can be used by drainage designers.

Other planning, analysis and design methods are also used, such as the use of programs including 12d, 12d plus 12d Drainage Module, 12d plus 12d Drainage Module plus TUFLOW, TUFLOW, DRAINS, XP Rafts, XP Storm, XP 2D, etc.

XVIII. Information from the Literature

A. Bevan, (2011)

The following is from Bevan, (2011):

The Starting Point: The Rational Method

It is worth remembering that rainfall-runoff modelling has a long history and that the first hydrologists attempting to predict the flows that could be expected from a rainfall event were also thoughtful people who had insight into hydrological processes, even if their methods were limited by the data and computational techniques available to them. We can go back nearly 150 years to the first widely used rainfall-runoff model, that of the Irish engineer Thomas James Mulvaney (1822-1892), published in 1851 (and reproduced in League, 2010). The model was a single simple equation but, even so, manages to illustrate most of the problems that have made life difficult for hydrological modellers ever since. The equation was as follows:

\[(2.1)\]
The Mulvaney equation does not attempt to predict the whole hydrograph but only the hydrograph peak $Q_p$. This is often all an engineering hydrologist might need to design a bridge or culvert capable of carrying the estimated peak discharge. The input variables are the catchment area, $A$, a maximum catchment average rainfall intensity, $R_a$, and an empirical coefficient or parameter, $C$. Thus, this model reflects the way in which discharges are expected to increase with area and rainfall intensity in a rational way.

It has become known as the Rational Method. In fact, variations on Equation (2.1) have been published by a variety of authors based on different empirical data sets (see Dooge (1957) for a summary) and are still in use today (try searching on "peak discharge rational method").

The scaling parameter $C$ reflects the fact that not all the rainfall becomes discharge, but here the method is not quite so rational since it makes no attempt to separate the different effects of runoff generation and runoff routing that will control the relationship between the volume of rainfall falling on the catchment in a storm, effectively $AR$, and the discharge at the hydrograph peak. In addition, the coefficient $C$ is required to take account of the nonlinear relationship between antecedent conditions and the profile of storm rainfall and the resulting runoff generation. Thus $C$ is not a constant parameter, but varies from storm to storm on the same catchment and from catchment to catchment for similar storms. The easiest way to get a value for $C$ is to back-calculate it from observations of rainfall and peak discharge (the very simplest form of model calibration). Predicting the correct value for a different set of conditions, perhaps more extreme than those that have occurred before, or for a catchment that has no observations is a much more difficult task.

Similar difficulties persist to the present day, even in the most sophisticated computer models. It is still difficult to take proper account of the nonlinearities of the runoff production process, particularly in situations where data are very limited. It is still easiest to obtain effective parameter values by backcalculation or calibration where observations are available; it remains much more difficult to predict the effective values for a more extreme storm or ungauged catchment. There are still problems of separating out the effects of runoff generation and routing in model parameterisations (and in fact this should be expected because of the real physical interactions in the catchment).

However, it is not impossible to make predictions, even with such simple models. Even in the precomputer era, the Rational Method evolved into the Graphical Estimation technique (see the work of Linsley, Kohler and Paulhus (1949) or Chow (1964) for full details). This was an attempt to summarise a wide range of analyses carried out for catchments in the USA into a set of graphs or nomograms that could be used to predict peak discharges under different rainfall and antecedent conditions (Figure 2.1). This approach has been used as a design tool for many years and has been put into mathematical form by, for example, Plate et al. (1988).


C. Rossmiller, (1982)


D. McPherson, (1969)

1.0  INTRODUCTION AND SUMMARY

Sewerage systems and canalized drainage works are generally owned, operated and maintained by local governments, and designed and constructed by local governments and private land developers. "Human life is seldom threatened by the flooding of these facilities. The principal detrimental effects of flooding are damage to the below-ground sections of buildings and hindrance of traffic. The consequences of flooding range from clearly assessable property destruction to annoying inconvenience. It follows that provision of complete protection from flooding can only rarely be justified. Instead, facilities are designed which will be overtaxed infrequently.”

The procedure used for design of storm sewers in the United States is almost exclusively the "rational method." This method has substantial liabilities and concerned engineers, such as the members of the ASCE Urban Hydrology Research Council, have long been seeking improved design procedures founded on field observations of the rainfall-runoff process. Very few catchment areas in the U.S. have been gauged. The U.S. Geological Survey is currently assessing information needs, and the purpose of this technical memorandum is to illustrate limitations of the rational method in an effort to help substantiate the urgent need for field data.

Flow in storm sewer systems is principally by gravity. Like natural drainage basins, smaller sewer branches unite with larger branches, and so on, until a main sewer is reached. The basic catchment area, about one to several acres in size, is that tributary to an inlet. "For most smaller areas in the upper reaches of an urban drainage system the time required to reach peak runoff after the beginning of a storm is a matter of minutes. Hence, high-intensity, short-duration rainfall is normally the main, if not sole, type of precipitation contributing to critical runoff rates. This type of rainfall is usually associated with thunderstorms."

Studies under way by the ASCE Urban Water Resources Research Program, to be reported in a subsequent technical memorandum, indicate that in some principal cities the median size of storm sewerage drainage system catchments is less than perhaps 200-acres in size; but about half the total sewer area of large cities is represented by catchments about 1,000-acres and smaller. To give an indication of relative flow rates accommodated by storm sewers, "roughly, capacities for storm sewers have been approximately 100 times the capacities provided in sanitary sewers."
Storm sewers have one-directional open-channel flow actuated by gravity, and seldom form closed-loop networks except in a limited sense where catchments are interconnected. A main drain not only transmits upper reach flow to a receiving watercourse, the usual sole function of a relief sewer, but also serves as a collector of surface runoff all along its route. Relief for overloaded storm sewers can often be achieved only by diverting flow from the upper reaches of the catchment in an 'express' outfall. A relief sewer is a very expensive alternative to the provision in the first place of main drains of adequate size to meet future flow increases. The cost of an error in design, as in water resources development generally, is thus, deferred, and the cost of rectification is almost universally greater than would have been the cost of adequate facilities at the outset.

The rational method applies to a very unique set of assumed conditions. Once a system is designed using the fixed features inherent in the method there is no logical way to analyze modifications, such as provisions for relief, reusing the method. Also, because there is no direct way to verify the method in the field, even the adequacy of the original design cannot be checked.

Three factors affect the magnitude of a design flow in using the rational method: C-value, inlet time and the frequency for the rainfall intensity-duration curve used. Computed flows are larger as the C-value is raised, as the inlet time is shortened and as a curve for a rarer rainfall frequency is used. Intelligent, though arbitrary, selection of values for these three variables has been found to give ostensibly "satisfactory" results in a number of cities. There are a number of factors which may contribute towards conservatively "safe" designs, such as the usual practice of designing sewers to accommodate at least the design flow rate at a flowing-full condition, whereas some degree of surcharge might be sustained without flooding. The probability of all design assumptions being satisfied simultaneously is less than the probability of occurrence of the rainfall rate used in the design, contributing in effect to a "safe" design. "Sewer design practice data from a number of municipal sewer designers show extensive differences in the methods and design factors used for storm sewer capacity design computations."

"There is an obligation to the public, insofar as it is practicable, to equate the cost of a given design to the probable protection and service which it will afford. Otherwise, an adequate criterion for the 'most economical design' does not exist." The tenuous linkage between the design rainfall frequency of the rational method and frequency of flooding have precluded evaluation of economic criteria.

In later sections of this memorandum an attempt will be made to indicate that the range of variability in the rarer peak flows of design significance might be much less than for natural catchments, suggesting that a high degree of precision must be achieved in gauging peak flows and associated rainfall if subtle differences occasioned by presently unidentified variables or characteristics are to be separated and quantified.

The essence of the immediately preceding arguments is that there is wide latitude in interpreting the concept and in the application of the rational method; there appears to be considerable regularity in the damping of rainfall input variability by the urban system;
and the apparent "success" of the method may well spring from a combination of a rather limited range of possible error in applying any method and conservatism arising from cumulative favorable factors, albeit inadvertent, inherent in the rational method. Under prevailing circumstances a debate on questions of under design or over design would be completely academic.

Attempts have been made to develop alternative improved methods for flow determinations based on hydrologic fundamentals, but these have, thus far, been usable only in the locality for which they were developed or their veracity has not been adequately confirmed by means of field gauging verification.

To state the situation very simply, development of improved design methods has been stymied for decades because of a lack of a suitable national field gauging rainfall runoff program. Mathematical models exist that could quite likely lead to vastly improved design methods were the field data available for their calibration and refinement. There is a real need to account for storm time and space variability in urban water developments, including storm drainage, but the impetus for research on metropolitan storms is inhibited so long as no tangible progress is made in the collection and analysis of rainfall-runoff data.

Essentially, an impasse exists. The foregoing criticisms of the rational method are not intended as a recommendation that its use be abandoned. Unless, and until, improved methods are developed, it is about as satisfactory as any other oversimplified, empirical approach.

More refined and extensive hydrologic information is required to develop adequate technical knowledge for reliable planning of water quality and quantity exchanges between the several urban water service functions for multi-purpose development; and for quantifying pollution loadings from combined and storm sewerage systems applicable nationwide. The rational method is of no use whatever for these purposes. However, if adequate information can be secured on rainfall-runoff-quality processes, the collective needs for storm sewer design, water exchange developments and evaluation of pollution loadings can be met simultaneously. It appears that a high degree of field gauging precision will be needed to achieve any one or all three of these objectives.

The value to the public of improved knowledge on stormwater pollution and methods of analyzing exchanges between service functions might substantially exceed the value from improved storm sewer design. As will be detailed in the final report for the USGS project, average annual expenditures over the next several years in the United States for urban storm drainage facilities is estimated at about $3.5 billion per year: $2.5 billion by local governments and $1 billion by developers. If only 5% of these projected costs could be saved by means of improved design criteria, the average annual savings would be almost $200 million per year.

2.0 BACKGROUND

The origins of the "rational method" of storm sewer design in the United States are generally traced3 to Kuichling.4* "More than 90% of the engineering offices throughout the United States that replied to a questionnaire on storm sewer design practice indicated
use of the ... method" and it "must be considered current practice."3 Practice has changed little since 1960. The rational method has been the predominant approach used for sizing storm sewers (separate and combined) over the last half of the 80-years since Kutchling's 1889 paper was published. Kuichling deplored the conduit sizing methods employed by his contemporaries, which were wholly empirical and founded on weak logic. If design peak flow rate was related to rainfall at all, the basis was a 1-hour or greater duration rain regardless of catchment area size. The central theme of Kuichling's paper was a call for recognition of the variability of rainfall so that storm sewers could be sized more realistically and reliably. 

The writer was long since impressed with the fact that during showers the volume of water discharged at the mouths of several large outlet sewers in the City of Rochester, New York, appeared to increase and diminish directly with the intensity of the rain at different stages, but that a certain length of time was required in each case after the termination of a brief and heavy downpour before the corresponding flood showed itself at the outfall; these floods, moreover, seemed to last about as long as the said showers themselves, and the conclusion was, therefore, reached that there must be some definite relation between these fluctuations of discharge and the intensity of the rain, also between the magnitude of the drainage area and the time required for the floods to appear and subside. The conclusion is accordingly irresistible that the rates of rainfall adopted in computing the dimensions of a main sewer must correspond to the time required for the concentration of the drainage waters from the whole tributary area when small, or from so much thereof as will produce an absolute maximum discharge when the area is very large. 

And also:

Kutchling was among the first to recognize the relationship between intensity and duration. 

... the maximum uniform intensity of the rainfall diminishes rapidly as its duration increases from a few minutes to one hour, and for rains of uniform intensity lasting more than one hour the rate of diminution is comparatively slow.


The following is from McPherson, (1974):

The procedure used in nearly all current storm sewer design is the 'rational method,' the numerous inadequacies of which have been discussed by McPherson [1969]. The method yields only an estimate peak flow. A complete hydrograph is needed for design of detention storage, for evaluation of pollutant burdens, for design of storm water pollution abatement facilities, for design of local protection works along streams, such as pumping stations for passing local drainage flows over levees and dikes, and as inputs (or design of stream and river development works. Also quantification of the effects of urbanization on the hydrologic regimen is dependent in many cases on the availability of sewer outlet hydrographs. Further-more, as urban water management problems
become increasingly acute, the need for multiple use of water becomes more evident. In exchanging one use for another, for example, by using storm water as a source of water supply, knowledge of the time histories of flows and water qualities is essential for reliable design of transfer facilities. In sum, there is essentially universal agreement that improved methods for design are sorely needed.

Model requirements for planning are less rigorous and require and permit less detail than those for design because investigation of a range of broad alternatives is at issue. What are sought for planning tools are general parameters or indicators for large-scale evaluation of various alternative schemes. Hence the degree of model detail required in metropolitan planning is much less than that for design. However, a certain amount of intensive detailed modeling is needed to establish parameters and indicators and to provide an underlying understanding of the governing hydrologic processes so that simplified expedients are not inadvertently misused. In the absence of suitable examples for sewered systems, applications of detailed models in planning are illustrated here by the development of simulation relationships for estimating stream discharge-and-stage frequency for use in connection with floodplain mapping activities [Hydro-comp International, Inc., 1971] and for identification of relationships between projected land use alternatives and degree of flooding and the cost of attendant flood protection measures [Reimer and Franzini, 1971].

And also:

Human life is seldom threatened by the flooding of urban drainage facilities. Because the principal local detrimental effects of flooding are damage to the below-ground sections of buildings and hindrance of traffic, the consequences of flooding range from clearly assessable property destruction to annoying inconvenience. It follows that provision of complete protection from flooding can only rarely be justified. Instead, facilities are designed that will be overtaxed infrequently. The major question in the analysis/design of drainage systems is the choice of storms to be used. Storm definitions used for deriving river basin extremes such as 'reservoir design floods' and 'spillway design floods' are irrelevant because urban sewer systems are expected to be overtaxed much more frequently than major river structures whose failures could be catastrophic. From this standpoint the mean frequencies of occurrence of flow peaks and volumes and quality constituent amounts are the issue, not the frequencies of the input rain-fall, and if it were possible to arrive at statistical series for discharge quality independently of rainfall, we could vastly simplify the storm characterization issue.

Furthermore, because there are inherent nonlinearities in most methods for processing inputs for linear models, and dynamic models are nonlinear by definition, the statistics of the rainfall input array may differ appreciably from the statistics of some or all of the arrays for runoff quality characteristics. That is, attempting to assign a mean frequency of probable occurrence to a 'design storm' is meaningless because of statistical nonhomogeneity of rainfall, runoff, and quality. Also such an approach neglects the effect of prior storms on the runoff from a given storm (Linsley, 1970).
And also:

Over and above he storm definition problem is the inherent difficulty with any runoff model in the necessarily subjective separation of abstractions (infiltration, depression storage, etc.) from total rainfall to resolve rainfall excess (amount and pattern), which is the input from which an equal volume of direct runoff is generated by models of one kind or another. This problem is greatly aggravated by the extreme scarcity of field data for calibrating and verifying models of all types.

F. Westphal, (2001):

The following is from Westphal, (2004):

The Rational Formula

The hydraulic sizing of drainage and conveyance structures in urban settings always requires estimation of peak flow rates. Historically, the venerable “Rational method” has been the tool of choice for most practicing engineers around the world. Although the method definitely has its place in hydrologic design, it is routinely misapplied and overextended. The roots of this methodology date as far back as 1851 (Mulvaney, 1851), and certainly as far back as 1889 (Kuichling, 1889). See discussion on chapter 1. The concept is attractive and easy to understand. If rainfall occurs over a basin at a constant intensity for a period of time that is sufficient to produce steady state runoff at the outlet or design point, then the peak outflow rate will be proportional to the product of rainfall intensity and basin area. In the United States, the method is commonly expressed by the equation known as the “Rational formula”:

\[ Q = C \cdot I \cdot A \] (4.1)

where \( Q \) _ peak runoff rate (cfs)\n
\( C \) _ dimensionless runoff coefficient used to adjust for abstractions from rainfall\n
\( I \) _ rainfall intensity for a duration that equals time of concentration of the basin (in / hr)\n
\( A \) _ basin area (ac)\n
In English units, it turns out that the dimensions of the product \( I \cdot A \) are ac in/hr, and 1.0 ac in/hr is very nearly equivalent to 1.0 cfs. In SI units, the equation must be made dimensionally homogeneous (e.g. if \( A \) is hectares and \( I \) is cm/hr, then the product \( C \cdot I \cdot A \) must be multiplied by 0.00278 to make the dimensions on \( Q \) equal to cms).

Since its inception, the Rational formula has been discussed extensively in the published literature and in theses. Most of its limitations and shortcomings are well documented, but these constraints are largely ignored by most practicing engineers. For credible engineering design, engineers must observe the constraints which limit the applicability of the Rational formula. An
exhaustive discussion of the rational method was done by Rossmiller (1982). The reader is urged to read Rossmiller for a cogent discussion of limitations that should be considered in its application.

And also:

There are a number of practical limitations to the application of the Rational formula. Records of mass rainfall curves for heavy storms invariably show that peak periods of constant rainfall intensity are usually of comparatively short duration (a few minutes or, at most, a few tens of minutes). This indicates that the assumption of constant rainfall intensity is more likely to be realized in basins that have a short time of concentration.

Isohyetal maps for storms on areas of less than 50 mi2 usually have elliptical isohyetal patterns. They show that the area of greatest rainfall depth is near the storm center and that depth decreases away from the storm center. Thus, for basin-centered storms, smaller basins are more likely to have a reasonably uniform spacial distribution of rainfall than are larger basins.

Like everything else in our physical world, rainfall-runoff relations in watersheds obey the Law of Conservation of Mass (and all other physical laws). In narrative form, this may be stated for any watershed (or stream reach) as: mass inflow rate minus mass outflow rate equals time rate of change of mass in storage. For instance, if we take a lined channel and introduce a hydrograph at the upstream end, we observe that the peak outflow rate at the downstream end is smaller than the peak inflow rate and it occurs later in time. The attenuation of the peak flow rate and the displacement in time is due to the channel storage in the intervening reach. It is a comparatively simple numerical task to demonstrate similar behavior on an idealized, impervious watershed that receives rainfall at a constant intensity. In the absence of storage, the peak outflow rate equals the steady-state rate of supply to the basin and it is coincident in time with the time of concentration of the basin. When storage is introduced into the computation, the peak outflow rate is attenuated and it occurs later in time than the time of concentration of the basin. Except in the case of zero storage in the system, the peak outflow rate from a basin must always be less than the peak rate of supply (e.g. the product of rainfall intensity and basin area). The storage in a basin (disregarding impoundments) will decrease as the area decreases and the percentage of the impervious area increases.

From the foregoing, it is clear that the application of the Rational formula should be limited to “small” watersheds. Although the term “small” is subjective, a reasonable upper limit under most circumstances would be about 200 acres (0.8 km2, 80 ha).

Time of concentration is defined as the length of time it takes for water to travel from the hydraulically most remote point in a basin to the outlet. Although this definition is attractive, its simplicity is deceiving. There is no practical way of measuring the time of concentration. From elementary considerations of free-surface flow (e.g. velocity of flow increases with increasing depth of flow), we know that for any given storm duration, greater rainfall depths will induce greater depths of flow in the drainage network, and travel times through the basin than will be less than those that will occur during smaller, more frequent rainfall events. In the
Rational formula, design rainfall intensity is a direct function of the time of concentration. Generally, the disparity between estimates of time of concentration by the various methods decreases as basin area decreases. In the case of runoff estimates for residential areas and other urbanized areas where the percentage of pervious surface is comparatively high compared to the area of impervious surfaces, the time of concentration should be based on travel time through the connected impervious part of the basin, and basin area should be taken as the area of the connected pervious surface.

The notion that the runoff coefficient, C, is a constant for any given watershed is flawed. As a percentage of storm rain, runoff increases with increasing rainfall depth and rainfall intensity. Published tables of suggested values of C are seldom explicit about the justifications for their guidance, but the general consensus in the literature seems to be that the Rational method be limited to events with return periods of 10-years or less (exceedance probabilities of 10% or more). For less frequent (more rare) events, there are published multipliers for C, but in no case should a value of C ever exceed 1.0.

Application of the Rational method requires that the duration of the design rainfall event be equal to the time of concentration of the basin. Many equations, nomographs and charts have been developed for estimation of the time of concentration of a basin. McCuen, et al. (1984) reviewed a number of equations that are in use for the determination of the time of concentration. No single approach has been demonstrated (or even claimed) to be superior to any of the others.

G. Young, (2012)

The following is from Young, (2012):

The Rational method has endured over 150 years since its original description by Irish engineer Thomas Mulvany in 1851 (Dooge 1957). Yen (1992) questioned whether the method endures because it is fundamentally sound or because the field of hydrology simply has not progressed. Indeed, the Rational Method is often considered a crude, simple rule-of-thumb approach to estimating design discharges on small streams.

Since its introduction to the United States in 1889 (Kuichling 1889), the Rational method has been used to size billions of U.S. dollars’ worth of drainage infrastructure. The method remains a popular hydrologic analysis and design tool, although its use is usually restricted to small, unregulated drainage areas. For larger unregulated drainage areas, engineers typically use regional flood-frequency relations; computer modeling is employed if flow in the watershed is significantly regulated. Even when these more sophisticated methods are used in design, the Rational Method often serves as a quick check or validation.

The Rational formula is commonly written as

\[ Q_T = kC_Ti_TA \]  

(1)
where $Q_T =$ design discharge (in cms) for the recurrence interval, $T$; $k$ is a unit conversion factor (0.278 for the SI units used in this paper); $CT$ is the Rational runoff coefficient; $iT =$ rainfall intensity (mm/h); and $A =$ watershed area (km$^2$) upstream of the point of interest. The rainfall intensity $iT$ is selected from intensity duration- frequency (IDF) relations for the recurrence interval of interest using a duration that is equal to a characteristic time (or averaging period) for the watershed. The watershed time of concentration $tc$ is commonly used to define the rainfall duration.

Previous studies have advanced the Rational method as a viable tool for hydrologic analysis. Schaake et al. (1967) published results from a major urban field study in the Baltimore, Maryland area, presenting the Rational formula as a frequency-based equation relating peak discharge for a given recurrence interval to the product $iT A$. Their analysis showed that the rainfall and runoff-frequency curves for individual stations are nearly parallel and could be related by a runoff coefficient that increases in accordance with recurrence interval.

This presentation of the Rational method as a frequency-based relationship differs from the common interpretation in which the runoff coefficient $CT$ is considered as the ratio of runoff volume to rainfall volume. Several published reports have supported the frequency-based application of the Rational method and have demonstrated that the Rational formula should not be used to predict the peak discharge for individual storms. French et al. (1974) conducted a large study of Australian watersheds, which showed that the Rational method performs poorly as a predictor of peak discharge for individual storms but works well as a frequency-based prediction tool for watersheds up to 250 km$^2$. Hotchkiss and Provaznik (1995) arrived at a similar conclusion using a set of gaged watersheds in Nebraska. The frequency-based version of the Rational method has been defended by Pilgrim and Cordery (1993) and Wong (2002), who responded to the question of Yen (1992) about the endurance of the Rational method.

And also:

There is a common misperception in practice that the Rational method should not be used for anything other than very small watersheds. McEnroe et al. (2007) demonstrated that the residuals of the Rational C equations in Table 9 are not a function of drainage area, which indicates that the form of the Rational method works equally well for the full range of watershed areas used in the research reported in this paper. French et al. (1974) and Hotchkiss and Provaznik (1995) showed that the Rational method can be used for much larger watersheds than is typically assumed. French et al. (1974) suggest that the form of the equation is valid for watershed areas up to 250 km$^2$ and perhaps larger watersheds as well. This study moves from the conceptual interpretation of the Rational method to the frequency-based method, which sees that the product $iT A$ is simply a predictor for $Q_T$. As such, $C_T$ can be backed out using $C_T = Q_T/iTA$ with $Q_T$ and $iT$ obtained from flood and rainfall frequency analysis data. If $C_T > 1.0$, then this could be caused by uncertainty in the flood frequency analysis or selection of the $iT$ value, or it could be due to a real cause that means that the predictor $iT A$ is lower than the real peak runoff rate for the event that causes $Q_T$. The strongest two reasons for this would
be spatial and temporal variability in rainfall, particularly in large watersheds. Storm movement through the watershed is another potential factor. Thus, using the frequency-based interpretation for the Rational method, there is no physical reason to expect $C_t$ to be less than 1.0.

The Rational method has been in widespread use for decades and has been used to design billions of dollars of drainage infrastructure. Many engineers assume that the Rational method is a rule-of-thumb technique or that it is inferior to regional flood frequency equations. This paper demonstrates that regional flood frequency analysis gives strong empirical support for the form of the Rational equation, as long as the Rational C values are properly determined for the area of interest. The Rational equations developed in this paper explain more of the variance in the flood quantiles than do equations of the form used by the USGS to predict peak flows for small rural watersheds in Kansas. The equations in this paper apply to small (less than 80 km2), rural, unregulated watersheds in the state of Kansas and are certainly not globally applicable.

H. Nash, (1958)

The following is from Nash, (1958):

The determination of the magnitudes and frequencies of discharges in sewers and in natural catchments drained by open streams by consideration of the amounts and frequencies of rainfall over the area has been the subject of many Papers published in the past half-century. It was perhaps inevitable that some confusion would arise, particularly since almost all such Papers deal exclusively with either urban or natural catchments. This has resulted in the growth of two different approaches to what is virtually the same problem. Failure to appreciate the identity of the problem (on both urban and natural catchments) has resulted in the public health engineer being deprived of the tools developed by the hydrologist, who at present is in a much more advanced position than his urban colleague.

It is the purpose of this Paper to review briefly the various methods developed over the years, to compare them with one another and isolate their common elements. As a result, what is believed to be a systematic method of investigation, suitable for either urban or natural catchments, emerges. This method is currently being applied by the Author to a series of natural catchments. In order, however, to distinguish more clearly between the method and its application in a particular case, as well as to avoid undue length it is proposed to keep the results of the application for a later Paper.

And also:

The relation between storm run-off and rainfall may be considered in three parts:-

(a) The relation between the volume of rainfall in a given storm, and the volume of storm run-off resulting.
(b) The manner in which the storm run-off is distributed in time. If \( i = i(t) \) represents the distribution in time of effective rainfall intensity (i.e. rainfall minus all losses) and \( Q = Q(t) \) represents the flow of storm run-off past the gauging station, then the transformation which the catchment performs on \( i(t) \) to produce \( Q(t) \) is the effect which must be found.

(c) The relation between rainfall frequency and discharge frequency.

If (a) and (b) are known, the hydrograph of storm run-off due to any given rainfall storm in any given circumstances can then be predicted. A means of predicting the frequencies of peak discharges from standard rainfall quantity duration-frequency curves would still, however, have to be found.

The first part of the problem is of much less significance on urban than on natural catchments because the percentage run-off on the former, at least in intense storms, is sufficiently near 100 (and comparatively independent of antecedent conditions) to make errors from incorrect assessment of this factor comparatively slight. It is perhaps on this account that investigators working on urban catchments have generally been content to use a fixed percentage runoff, while hydrologists working on natural catchments have had to adopt more elaborate methods, e.g. the work of Linsley and Ackermann in obtaining, by statistical analysis of the results of a large number of storms, correlations between the volume of run-off, and the volume of rainfall and indices representing the hydrological condition of the catchment at the time of occurrence of the storm.

The second part of the problem—the determination of the operation performed by the catchment on the input \( i(t) \) to produce the response \( Q(t) \)—is almost the same problem on either urban or natural catchments. The relation is, in fact, more easily determined on urban catchments, because of the almost complete absence of ground-water flow or base flow, which, on natural catchments, must be separated from the hydrograph of total discharge, before the storm run-off due to any particular storm can be isolated. Furthermore, on natural catchments, the determination of the distribution in time of rainfall losses during a storm is rarely accurately possible. On urban catchments where the losses are small if not entirely negligible this difficulty scarcely arises. Despite these disadvantages the present position is that almost all progress to date has been made by hydrologists working on natural catchments, while public health engineers are still compelled to use (with minor modifications) the method introduced to Great Britain by Lloyd-Davies in 1906.

The third part of the problem—the frequency relation—requires the prior solution of (a) and (b), and is thereafter purely analytical. Consequently its solution must be sought in the same way for both natural and urban catchments.

Parts (b) and (c) only will be dealt with in this Paper, assuming where necessary that the volumetric relation between rainfall and run-off can be achieved by multiplying the total rainfall by a run-off coefficient to obtain the effective rainfall. It will also be assumed that this coefficient can be predicted with sufficient accuracy from knowledge of local conditions. In fact, on any one
catchment, this coefficient is not as variable as might be expected, particularly during rather large storms which effectively saturate
the catchment during their earliest portions. This Paper will therefore be confined to discussing the following two problems:-

(a) Given i(t), the intensity of effective rainfall as a function of time, to find Q(r) the corresponding hydrograph of storm
run-off, particularly the peak discharge.

(b) Given a rainfall frequency formula expressing frequency of given quantities of rain in given storm-periods, to find the
frequency of a given peak discharge on a catchment for which a method is known of determining run-off from rainfall.

HISTORICAL DEVELOPMENT

The “Rational method”

The origin of this method is somewhat obscure. In Great Britain it is often referred to as the Lloyd-Davies method and hence by
implication ascribed to his Paper of 1906. It has been shown, however by Dooge that the principles of the method were explicit in
the work of Mulvaney in 1851. As currently understood the method may be stated as follows. For every catchment there is a
period, knowans the time of concentration Tc, which is the time required for a particle of water to flow from the farthest part of the
catchment to the gauging station. The discharge peak occurs when the whole catchment is contributing at the gauging station, i.e. a
period Tc, after start of rain, and is equal to the mean intensity of the effective rain during this period. This can be stated as:

\[ Q = C \cdot p \]  

(1)

where C is the coefficient of run-off, \( A \) denotes the area, and \( p \) the mean intensity of rainfall during the period \( T_c \). This formula is

known in the literature as the “rational formula”. Kutchling in 1889 suggested that C approached a constant value for a given
catchment as the magnitude of the storm increased. Kutchling’s purpose was not so much the prediction of the hydrograph of run-
off from the rainfall but rather the determination of the frequency of discharges from the frequency of rainfall. A set of curves may
be assumed, giving the frequency of any given quantity of rainfall in any given time or less as \( F=F(R, T) \) (i.e. frequency as a
function of quantity and period in which the quantity is to be expected). Such curves are the standard rainfall quantity-duration-
frequency curves. Given this information and assuming C and \( T_c \) to be known for a given catchment, then the frequency of any
given discharge can be obtained by reading from the rainfall frequency curves, the frequency of the quantity of rain required in \( T_c \)
to produce \( Q \).

In detail, to produce a peak discharge \( Q \) the effective rainfall must have a mean intensity equal to \( Q \) over the period of
concentration \( T_c \), i.e. a volume of effective rainfall \( T_c \cdot Q \) (corresponding to a volume of actual rainfall \( T_c \cdot Q/C \)) must occur in time \( T_c \).
The rainfall frequency chart or formula gives directly the frequency of this quantity in the given time \( T_c \). This frequency is the
frequency of \( Q \), because for every occurrence of \( Q \) there will also, according to the rational theory, be an occurrence of \( T_c \cdot Q/C \)
inches of rainfall in period \( T_c \), and conversely. It is important to appreciate that the reason why the frequency of \( Q \) is equal to the
frequency of a certain amount of rainfall in a certain time is that, according to the rational theory, the discharge $Q$ depends on the quantity of rainfall in a certain critical period. If on the other hand it was assumed that a quantity $R_1$ in a period $T_1$ or a quantity $R_2$ in a period $T_2$, or a quantity $R_3$ in a period $T_3$ could each produce $Q$, then it would be necessary to add the frequencies of the independent occurrences of these quantities ($R_1$ in $T_1$, $R_2$ in $T_2$, etc.) to obtain the frequency of $Q$. It is also worthy of note that if $C$ be assumed to vary with the conditions in the catchment at the time of occurrence of the storm, then the rational method of determining the frequency would not work, since the peak discharge $Q$ would no longer be related to a unique quantity of rainfall in a unique time, but could be produced by $R_1$ in $T_C$ or $R_2$ in $T_C$, depending upon the value of the coefficient of run-off occurring at the time of the storm.

While it is adequate to assume that the peak discharge is directly proportional to the total volume of rain falling in the time of concentration adequate, that is, in the sense that no further assumptions are required in the theory—it can be shown that this assumption is equivalent to a number of other assumptions which are known to be inaccurate. This will be dealt with later when a unified view is taken of the various methods proposed since the advent of the rational theory.

**XIX. Conclusion**

The conclusion is that the Rational Method is still relevant and useful and should be continued to be used.

**XX. Recommendations**

1. Visit: www.arr.org.au
   - Submit your comments to the ARR revision team.
   - Subscribe to eNews

2. Join Linkedin ARR group
   - Join discussion on Rational Method
   - Join discussion on Terminology for Design Rainfall
7.3 Hydrology and hydraulics
7.3.1 General
The following factors must be considered in the design and selection of the final drainage treatment:

(a) design discharges based on the ultimate development in the catchment;
(b) future maintenance requirements to ensure the drainage facility continues to meet its design performance;
(c) safety of persons, particularly children;
(d) erosion and siltation both within and on adjoining properties not increased as a result of the development;
(e) the existing treatments of other sections of the drainage system;
(f) the general amenity of the area and particular use of parkland;
(g) environmental issues, including vegetation protection orders, maintenance of natural channels and buffer vegetation, preservation and rehabilitation of flora and fauna habitats, riparian vegetation, archaeological values, heritage values, water quality and existing features such as wetlands;
(h) integration of total water cycle management.

7.3.2 Flow estimation methods
(1) For guidance to the design of urban drainage systems Council refers the designer to QUDM and Australian Rainfall and Run-off. Council will accept flow estimations using the rational method or from run-off or storage routing models (e.g. DRAINS, ILSAX, XP-RAFTS, WBNM, RORB).

(2) For complex drainage situations (particularly as part of a flood study for setting building development levels) or for sizing stormwater detention systems, a run-off storage routing model must be used to estimate flows and/or analyse the hydraulics of an urban drainage system.

7.3.3 Rational method assumptions
Where the rational method for flow estimation is suitable for flow estimation, the design is to be in accordance with QUDM and the following sections.

7.3.3.1 Fraction impervious
(1) Designers are to refer to QUDM section 4.5 for methodology in determining the run-off coefficients.

(2) The $C_{10}$ coefficients of discharge shown in Table 7.3.3.1.A are to be used for rational method calculations.
### Table 7.3.3.1.A—Coefficient of discharge $C_{10}$ for development

<table>
<thead>
<tr>
<th>Development category</th>
<th>$C_{10}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Central business areas (including in the Principal centre zone and Major centre zone)</td>
<td>0.90</td>
</tr>
<tr>
<td>Industrial uses and other commercial uses (including in the District centre zone and</td>
<td>0.88</td>
</tr>
<tr>
<td>Neighbourhood centre zone)</td>
<td></td>
</tr>
<tr>
<td>Significant paved areas (e.g. roads and car parks)</td>
<td>0.88</td>
</tr>
<tr>
<td>Medium density and high density residential land uses</td>
<td>0.88</td>
</tr>
<tr>
<td>Low–medium density residential land uses</td>
<td>0.87</td>
</tr>
<tr>
<td>Low density residential area (including roads)</td>
<td></td>
</tr>
<tr>
<td>Average lot $\geq 750m^2$</td>
<td>0.81</td>
</tr>
<tr>
<td>Average lot $\geq 600m^2 &lt; 750m^2$</td>
<td>0.82</td>
</tr>
<tr>
<td>Average lot $\geq 450m^2 &lt; 600m^2$</td>
<td>0.85</td>
</tr>
<tr>
<td>Average lot $\geq 300m^2 &lt; 450m^2$</td>
<td>0.86</td>
</tr>
<tr>
<td>Low density residential area (infill subdivision excluding roads)</td>
<td></td>
</tr>
<tr>
<td>Average lot $\geq 750m^2$</td>
<td>0.82</td>
</tr>
<tr>
<td>Average lot $\geq 600m^2 &lt; 750m^2$</td>
<td>0.81</td>
</tr>
<tr>
<td>Average lot $\geq 450m^2 &lt; 600m^2$</td>
<td>0.83</td>
</tr>
<tr>
<td>Average lot $\geq 300m^2 &lt; 450m^2$</td>
<td>0.85</td>
</tr>
<tr>
<td>Rural/environmental protection areas (2–5 dwellings per ha)</td>
<td>0.74</td>
</tr>
<tr>
<td>Open space areas (e.g. parks with predominately vegetated surfaces)</td>
<td>0.70</td>
</tr>
</tbody>
</table>

### 7.3.3.2 Time of concentration

Refer to [QUDM](#) section 4.6 for calculation of time of concentration (rational method).

### 7.3.3.3 Creek flow times

1. For open creek catchments ($< 100ha$), minor channel or creek flow times may be initially determined by assuming an average stream velocity of 1.5m/s.
2. For medium-sized open creek catchments (100–500ha), the stream velocity method ([QUDM](#) Table 4.6.6) or the modified Friend’s equation ([QUDM](#) Section 4.6.11) must be used.
3. For large open creek catchments (>500ha), the rational method should be used. However, detailed hydrological modelling of Brisbane’s major creeks indicates that the rational method provides a reasonable estimate of peak discharge if an average flow velocity of 0.9m/s is assumed.

### 7.3.4 Hydraulic calculations

Refer to [QUDM](#) section 7.16 for information regarding hydraulic calculations.

### 7.3.5 Pipe capacity assumptions

1. Pipe capacity for trunk stormwater systems is to be estimated using hydraulic grade line analysis of the drainage system for the relevant design storm or using a suitable computer model.
(2) Where estimating the capacity of existing small pipelines (1,050mm reinforced concrete pipe or less) for planning purposes for a development site <1,000m², the minor flow capacity can be estimated using pipe flowing full at grade assumptions. The adopted pipe velocity when using this method must not be greater than 3m/s, because various hydraulic losses in the drainage system at pits and bends will limit the allowable velocity.

(3) Where the pipe capacity is being estimated to determine the proportion of overland flow through a site as part of a flood study, the hydraulic grade line analysis must use a starting water level that is relevant to the major storm event (e.g. 2% or 1% AEP storm event).

7.3.6 Tailwater level assumptions

(1) Designers are referred to QUDM section 7.16 and QUDM section 8.0 for advice regarding the correct tailwater level requirements for drainage design of stormwater outlets.

(2) An allowance of 300mm for climate change must be assumed for the minor system design, where stormwater drainage discharges into tidal waterways or the Brisbane River.

(3) If tailwater is critical for managing major flows and setting flood immunity, a sensitivity check must be undertaken to examine impacts of higher sea level in accordance with best climate change predictions at the time.

(4) In areas situated beside Moreton Bay and lower parts of the Brisbane River near the river mouth, storm surge may occur at times of the most intense rainfall as a result of cyclones or significant low-pressure systems. In small catchments, this may result in concurrent flooding whereby the peak flow off the catchment will coincide with peak storm-tide levels. Drainage design should choose appropriate tailwater levels in the situation carefully if it influences flood immunity for development.

7.3.7 Hazard estimation

The hazard associated with stormwater flows is determined by the product of depth and velocity, and or maximum total depth of flow (refer to QUDM section 7.4). For pedestrian safety the following criteria will apply:

(a) The velocity by depth product in a roadway in the major storm is to be limited to 0.6m²/s in the kerb and channel.

(b) Where there is an obvious danger of pedestrians being swept away where the velocity by depth product is to be limited to 0.4m²/s.

(c) For areas involving small children (e.g. child care centres) the velocity by depth product should be limited to <0.2m²/s in all cases.

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