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Reliability Assessment of Water Structures Subject to Data Scarcity Using the SCS-CN Model

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Abstract: When no discharge measurements are available, design relies on using frequency analysis of rainfall data, and applying a rainfall-runoff transformation to estimate a hydrograph. Rainfall-runoff transformation could be undertaken using, for example, the Soil Conservation Services (SCS) Curve Number (CN) unit hydrograph method. Calibration of the CN and the time of concentration (TC), from nearby gauged watersheds, is limited and subject to high uncertainties due to scarcity of data. Therefore, the inherent uncertainty/variability in the SCS parameters may have considerable ramifications on the safety of design.

In this research, a reliability approach is used to evaluate the impact of incorporating the uncertainty of the CN and the TC. The stochastic sensitivity of the probabilistic outcome to the basic uncertainty in the input parameters is calculated using First Order Reliability Method (FORM). The results from FORM are compared with the conventional SCS results taking solely the uncertainty in the rainfall event. Moreover, the relative importance of the uncertainty of the SCS parameters is estimated. It is found that the conventional approach used by many practitioners may grossly underestimate the risk of failure for water structures, due to neglecting the probabilistic nature of the SCS parameters and especially the Curve Number. The most predominant factors under which the SCS-CN method is highly uncertain are when the rainfall average value is low; its coefficient of variation is not significant – due to small sample sizes for example –. A case study is presented for Egypt using the rainfall data and values for CN driven from satellite information to determine the regions of acceptance of the SCS-CN method.

Keywords: Uncertainty; FORM; SCS; Curve Number; Time of concentration; Arid region

1 Introduction

Hydrology of arid regions is gaining increasing interest in recent scientific literature with the accumulation of satellite based data in desert regions and because of the devastating floods that recently occurred in regions like Jeddah, Saudi Arabia (El-Hames and Al-Wagdany 2012) and Muscat, Oman (Fritz et al. 2010). Although there is no agreement among hydrologic experts on the distinct classification of arid regions based on their annual rainfall, areas where the annual total rainfall is less than 70 mm/year and evaporation exceeds the yearly rainfall may be classified as hyper-arid (Soliman 2010). It is worth mentioning that two-thirds of the Middle East region can be classified in this category. Furthermore, areas where annual total rainfall is between 70 and 200 mm/year with sparse vegetation are called arid regions (Soliman 2010).

One of the most widely used methods to estimate the direct surface runoff, in arid zone hydrology, is the Soil Conservation Service (SCS) Curve Number (CN) method (Food Agriculture Organization 1981). The

SCS-CN method gained its popularity from its simplicity, stability and its ease of understanding and application. The SCS-CN method was originally developed for small agricultural watersheds and has since been extended and applied to rural, forest, and urban watersheds (Mishra et al. 2012) and even applied to arid and hyper-arid regions. The SCS-CN method dates back to the early works of Mockus (1949) and was first published in details in 1956 in Section 4 of the National Engineering Handbook of the Soil Conservation Service (now called the Natural Resources Conservation Service). The publication has since been revised from 1969 till 2009, introducing the concept of initial abstraction (US Dept. of Agriculture 1972) and the Antecedent Moisture Content lately termed the Antecedent Runoff Conditions (US Dept. of Agriculture 1985). Despite several limitations of the method and even questionable credibility at times, for example, Singh (1992) and Pilgrim and Cordery (1993) for concise summaries, it has been in continuous use for the simple reason that it works fairly well at the field level (Mishra and Singh 2003). Although the method is relatively old, its relevant research is still in the heart of recent publications to the extent that the Journal of Hydrologic Engineering of the ASCE has devoted a whole issue to the method to “present the state-of-the-art developments/advancements in the SCS-CN methodology and its potential and practical applications in hydrology” (Mishra et al. 2012).

2 The SCS CN Method

The general runoff equation introduced by the USDA-Soil Conservation Service (US Dept. of Agriculture 1972; US Dept. of Agriculture 1985; US Dept. of Agriculture 2004) is

[1] $$Q = \frac{(P-I_a)^2}{(P-I_a-S)} \quad for\ P \geq I_a \quad \text{and} \quad Q = 0 \quad for\ P < I_a$$

where Q = direct runoff (mm), P = total rainfall depth (mm) (taken in this research as the 24-hr rainfall depth), S = potential maximum retention (mm), and I_a is called the Initial Abstraction (mm) which consists of interception and surface depression storage and assumed to be a fraction of S (i.e. $I_a = \lambda S$).

The USDA-SCS developed a further relation to calculate the value of S through a dimensionless coefficient called the curve number (CN) which varies from CN=0 (theoretically) to CN = 100.

[2] $$S = 25.4\left(\frac{1000}{CN} - 10\right)$$

The fraction (λ) has been traditionally assumed by USDA-SCS to be of 0.20. Although recent studies indicate this value to be unusually high, it’s still the value used extensively by practitioners in flood and drainage field, mainly because the tabulated CN values are based on λ equaling 0.2 (Woodward et al. 2003).

To transform the calculated runoff (mm) to a peak discharge (m^3/s), the concept of unit hydrograph (UH) (Sherman 1932) is used convoluting increments of rainfall depths on the UH (Singh 1992). To achieve the segmentation of rainfall and obtain a UH for ungauged sites, the SCS has also developed four (4) temporal rainfall distribution each to be applied to a specific US region. Awadallah and Younan (2012) have shown that the SCS type II rainfall distribution is the one conforming to many rainfall patterns in the Middle East. Furthermore, to obtain a UH for ungauged sites, the SCS has also developed a synthetic UH in which the only parameter is the lag time (T_{lag}) defined as 0.6 times the time of concentration (Tc). Thus, to summarize the input required for SCS-CN runoff method + SCS-UH method (shortly termed hereafter the SCS method), the practitioner inputs the *design* 24-hr P (the 100-year 24-hr precipitation for example), a value for CN (from lookup tables), an initial abstraction ratio λ (usually assumed to be 0.2 to conform to the tabulated CN values), a catchment area A (which is only a multiplicative factor) and the storm distribution type (taken as type II in arid regions as previously mentioned).

By adopting such input configuration, the practitioner is implicitly assuming that the level of risk of failure for the resulting peak discharge is the one associated with the *design* rainfall (i.e. 0.01 for the 100-year rainfall example). This implicitly assumes also that the sole source of randomness is the rainfall variability. This research addresses the following question: if the other input variables of the SCS method, such as CN or T_{lag}, were also random variables, what is the expected probability of failure (P_f)? In fact, it was always assumed that the rainfall variability is predominant to the extent that it masks the variability of CN and T_{lag}. This is true when the *design* rainfall is high as it is the case in humid regions or even in semi-arid regions. However, with design rainfalls of 20 or 30 mm/day, which we might encounter in arid and hyper-arid regions, the P_f value – expected to be very close to 0.01 – might exceed 0.05, as will be presented later in this paper when applying to a case study of Egypt.

In practice, the CN value for design purposes is selected for ungauged watersheds from tabulated values in published handbooks such as the SCS National Engineering Handbook Section 4: Hydrology (NEH-4) (US Dept. of Agriculture 1985) or Technical Release 55 (TR-55) (US Dept. of Agriculture 1986) based on hydrologic soil group (HSG), land use, and surface treatment. In fact, there is no single CN value for the same above mentioned conditions. The USDA-SCS introduced the concept of Antecedent Moisture Condition (AMC) where CN may vary from a low value for dry conditions in the five days prior to the simulated event (CN-I) to a high value for wet conditions (CN-III) and the average value of CN (termed CN-II) is the one found in the lookup tables. The CN-III, CN-II and CN-I were later found to represent approximately 90%, 50%, and 10% exceedance probabilities of runoff depth for a given rainfall (Hjelmfelt 1991), and the concept of Antecedent Runoff Criteria (ARC) (US Dept. of Agriculture 2004) was introduced by the Natural Resources Conservation Service (NRCS) as a replacement for AMC with published CN values representing the average condition, or ARC II. The AMC criteria based on five-day antecedent rainfall is no longer recommended by the NRCS (US Dept. of Agriculture 2004)

The current research is focussed on reducing the SCS-CN method inherent uncertainty by introducing in CN determination the role of additional factors not included in the original method. Such factors take account of storm duration and rainfall intensity (Jain et al. 2006), steep slopes of catchments (Ebrahimian et al. 2012), presence of rock fractures … Even with the recent availability of online spatial soil data, such as the NRCS Web Soil Survey (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm), Soil Data Mart (http://soildatamart.nrcs.usda.gov/), FAO Digital Soil Map of the World and FAO landuse (http://www.fao.org/geonetwork/srv/en/#), the reliability of assigning CN values for ungauged watersheds remains problematic (D'Asaro and Grillone 2012). This might be the reason behind considering the CN as a random variable with 100-CN following a two-parameter Gamma distribution (McCuen 2002).

3 Uncertainty and reliability of peak discharge estimates

In dealing with uncertainty in the estimation of the peak discharge (Q_p, m^3/s), it is often desired to know how sensitive the computed Q_p is to the changes in the actual values of λ, P, CN and T_{lag} which cannot be estimated with certainty. Sensitivity analysis, thus, presents a viable method to evaluate, without repetitious computations, the proper weight and consideration that must be taken when each of the influencing factors is evaluated. For the runoff (Q) estimation (equation 1), closed form mathematical solutions are available. However, as the peak discharge calculations involve convolutions, the sensitivity analysis approach might not be feasible. Chen (1982) gave examples of the sensitivity of runoff estimates to variations in CN using differentiation and Singh and Yu (1990) presented similar results based on Taylor series expansion using first order analysis. While sensitivity analysis provides information about how much the output changes with small changes in particular input parameters, uncertainty analysis provides the statistical properties and an estimate of the statistical distribution of the output from the statistics or distribution of the input. With knowledge of the output Q_p statistical distribution, the probability of failure (P_f) can be assessed. Uncertainty analysis contains various methods, among which the well-known Monte Carlo Simulation. Other methods including approximate sampling methods and alternative point estimation procedures attempt to achieve results similar to those of Monte Carlo with fewer model runs. Among the methods of approximate sampling methods is the First Order reliability method (FORM).

First-Order Reliability Method (FORM) was developed in the structural engineering field to evaluate the probability of failure of structural components and systems, and was applied in the fields of groundwater flow and transport (Hamed and Bedient 1997), assessment of public health risk (Hamed and Bedient 1997) and soil contamination (Hamed and El-Beshry 2006) among other applications in the environmental and water fields. FORM is based on formulating the problem in terms of a limit state function, g(X), where X is a vector of n basic random variables $X_1, X_2, \ldots, X_n$. In our study, X is the matrix of input parameters /

variables of the SCS method such as CN, P, T_{lag}. The limit state function is formulated in such a way that g(X) = 0 defines the failure domain. We assume that the function is in the failure zone when the calculated peak discharge using the FORM procedure exceeds the peak discharge calculated with constant CN, T_{lag} and the 100-year rainfall value; i.e. assuming that the sole variability is due to rainfall random variation. The boundary of the failure domain, g(X) = 0, represents the limit state surface. The goal of the reliability analysis is to evaluate the probability of failure (P_f), which is the probability that the random vector takes on values in the failure domain. P_f may be determined by the following integral:

[3] $$P_f = \int_{g(x)\leq 0} f(X)dx$$

where f(x) is the joint probability density function of the random variables.

The name of First Order Reliability Method (FORM) comes from the fact that the performance function g(X) is approximated by the first order Taylor expansion (linearization). The probability integrations in Eq. 3 are visualized with a two-dimensional case in Fig. 1. The figure shows the joint pdf f(x) and its contours, which are projections of the surface of f(x) on X_1 – X_2 planes. All the points on the contours have the same values of f(x) or the same probability density. The integration boundary g(X) = 0 is also plotted on X_1 - X_2 plane.

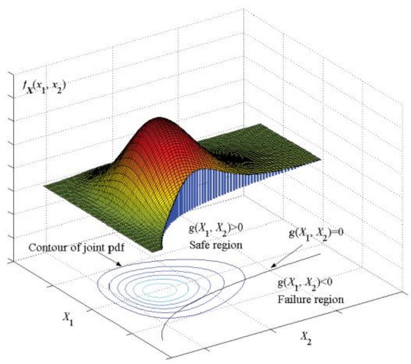


Figure 1: Probability Integration (after DU 2005)

The probability integration in Eq. 3 is the volume underneath the surface (hyper-surface) of the joint pdf f(x) in the failure region *g*(X) < 0 or the safe region *g*(X) > 0. Imagine that the surface of the integrand f(x) were cut by with the curve *g*(X) = 0, the surface would be divided into two parts. If the part on the side of *g*(X) < 0 were removed, the part left would be on the side of *g*(X) > 0 as shown in Fig. 1. The probability of failure will be the volume underneath f(x) on the side of failure region *g*(X) < 0, the removed part. The reader is referred to Du (2005) for complete details regarding mathematical formulation and solutions of the method. A premade MATLAB code called FERUM (*Finite Element Reliability Using Matlab*) was used in this study to perform FORM calculations. FERUM code, initiated in 1999 at the University of California at Berkeley (UCB), consists of an open-source MATLAB toolbox, featuring various stochastic methods, aiming to provide researchers with a tool which is very accessible, and which they can develop for research purposes. The latest available version of this code is FERUM 4.1, available at *the Institut Français de Mécanique Avancée (IFMA)* website (Institut Français de Mécanique Avancée 2010). FORM requires full knowledge of the statistical distributions of input variables. This knowledge can be summarized in Table 1 below.

Table 1: Summary of random variables used in FORM analysis

	Maximum 24-hr Precipitation	Lag time	Curve Number
Probability Distribution	Gumbel	Log-Normal	(100-CN) follows Gamma
Range of the Mean Value	P_{mean} : 10 → 100 (mm)	230 min	51→ 85
Coefficient of Variation/ Standard Deviation	P_{COV}: 0.05 → 3.0 (COV)	10 → 60 min (STD)	3 → 7 units (STD)

Several notes are worth mentioning regarding the above table:

- Maximum 24-hr rainfall is assumed to follow Gumbel distribution. In fact, this distribution is the only one explicitly mentioned in Middle East highway design codes of practice, such as the Egyptian road code (Ministry of Housing Utilities and Urban Development 2008), Saudi Arabian highway code issued by the Ministry of Communication (1997), Qatar highway design manual (Ministry of Municipal Affairs and Agriculture 1997), …etc. The ranges of the mean of max. 24-hr rainfall (P_{mean}) and the coefficient of variation of the max. 24-hr rainfall (P_{COV}) shown in Table 1 are much wider than the range encountered in arid regions rainfall data; however, the full range will be investigated to compare the case of humid regions to that of arid regions.
- 100-CN, as previously mentioned, is found to fit gamma distribution (McCuen 2002). Range of values for CN mentioned in Table 1 is chosen from NEH lookup tables (US Dept. of Agriculture 1985) assigned to arid/semi arid areas.
- Standard deviation range assigned to CN variable is based on confidence intervals proposed by McCuen (2002) narrower than the ARC range of USDA (2004).
- Since the time of concentration and hence its corresponding lag time, is not the main focus of this study, a constant mean lag time of 230 min was arbitrary chosen. However, this lag time was considered also as a random variable following a log-normal distribution with various tested coefficients of variation for lag time.
- Initial abstraction λ was assumed = 0.2 and the rainfall distribution was assumed as type II SCS 24-hr. The 100 year return period was used to complete the definition of the design storm. These values are chosen based on the most common practice in designing major drainage structures in arid areas.

4 Results and discussions

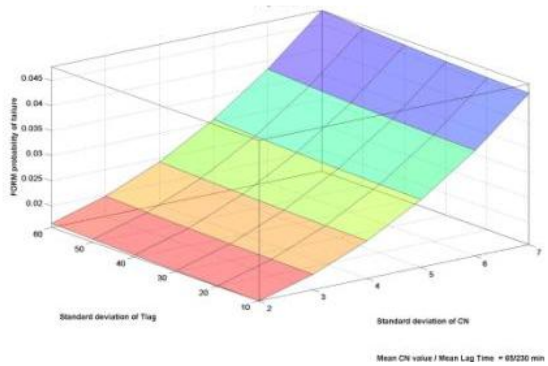
The results show in general consistent patterns for the variation of the probability of failure (P_f) due to variation of precipitation, Curve number and lag time. A sample of the results is shown in Figure 2 (a to f). Each graph shows the standard deviations of the SCS-CN input parameters (T_{lag} and CN) on the X-Y axes, while the Z-axis shows P_f at every combination of values on X & Y axes. All curves are monotonically increasing surfaces bounded at their lower limits by 0.01 which is the value corresponding to the 100-yr return period (the target design case). All P_f should in fact exceed 0.01 which is the expected probability of failure if only the rainfall is the sole source of variability. The results are summarized in the following 2 subsections: the effect of precipitation variation whether it is coming from P_{mean} or P_{COV} and the effect of CN variability. Finally, the P_f values exceeding 0.04 (4 times the target risk of 0.01) are highlighted in the summary tables. A case of Egypt is presented at the last subsection.

4.1 Effect of Precipitation variation over P_f

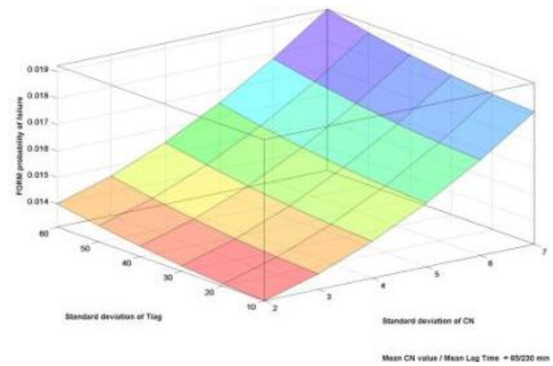
To describe the impact of rainfall variation on the maximum attained P_f, the effect is due to the mixed effect of P_{mean} and P_{COV}. In general, as shown on Figure 2 (a to f), the variability of curve number is the predominant factor compared to that of T_{lag} leading to high P_f, in the cases where P_{COV} is small (<2) and P_{mean} is less than 30 mm (Fig. 2 a, b, d and e). When P_{mean} increases, T_{lag} becomes predominant (Fig. 2f). The results (not shown in Fig. 2) illustrate also that low P_{COV} – even if the value of the P_{mean} is high – has considerable impact on the P_f associated with the generated runoff. Furthermore, it can be seen that for low P_{COV} (< 0.25), P_f values exceed 0.045 while for relatively high P_{COV} (between 1.00 and 3.00), P_f values don't exceed 0.021. This pattern is consistent for all CN mean values.

4.2 Effect of Curve Number (CN) variation over P_f

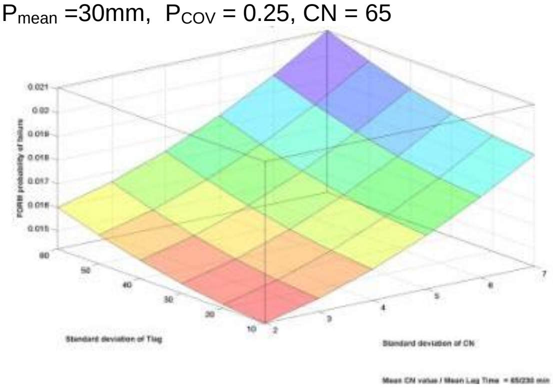
The results of the analysis show that P_f increases with mean value of CN. The higher the mean CN, the higher is the value of P_f under the same incremental variation expected of the mean CN. Several values of CNs are plotted against resulting P_f under the same P_{mean} and T_{lag} conditions. By changing P_{mean} and T_{lag} and redrawing new curves, a complete family of CN behaviors can be reproduced. Figure 3 (a to d) shows the results obtained.



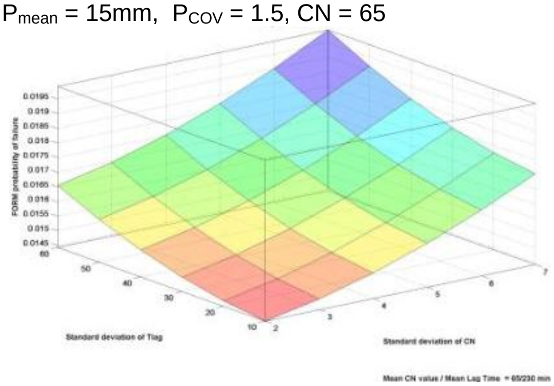
a) P_{mean} =30mm, P_{COV} = 0.25, CN = 65



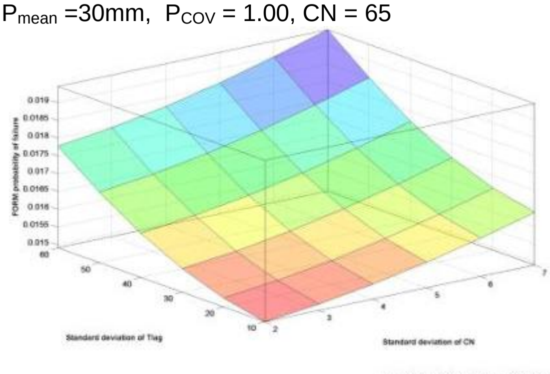
d) P_{mean} = 15mm, P_{COV} = 1.5, CN = 65



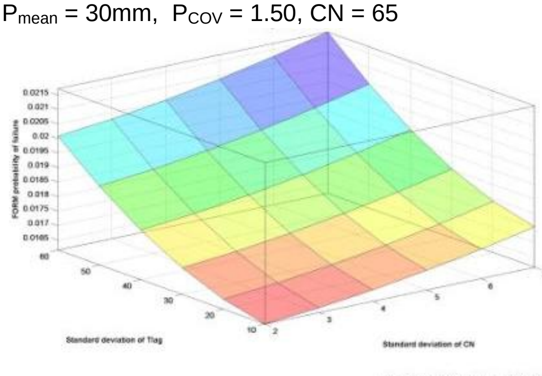
b) P_{mean} =30mm, P_{COV} = 1.00, CN = 65



e) P_{mean} = 30mm, P_{COV} = 1.50, CN = 65



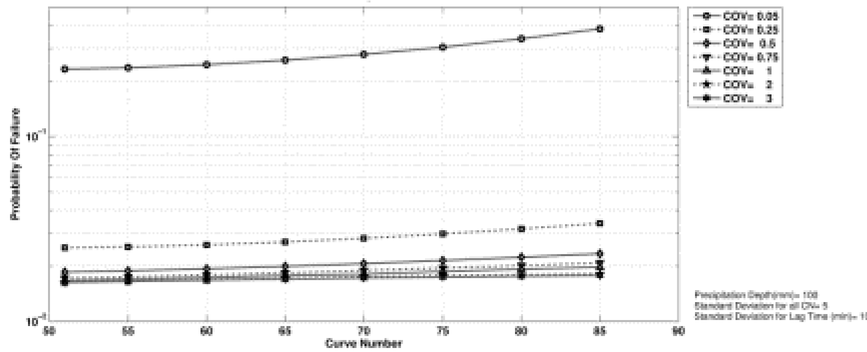
c) P_{mean} =30mm, P_{COV} = 3.00, CN = 65



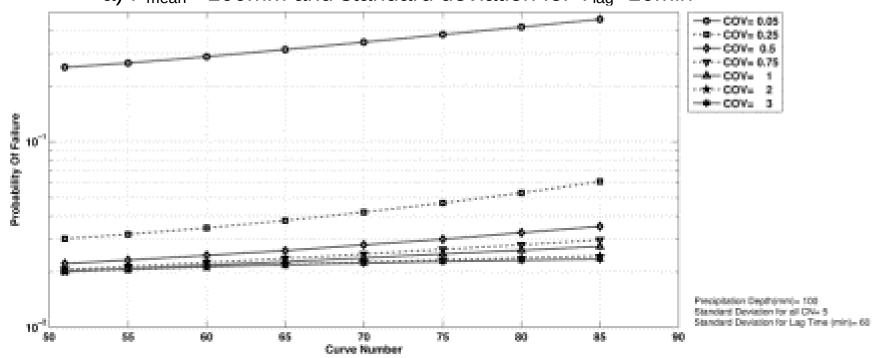
f) P_{mean} = 70mm, P_{COV} = 1.50, CN = 65

Figure 2: Results curves showing P_f for various P_{mean} and P_{COV} and CN = 65

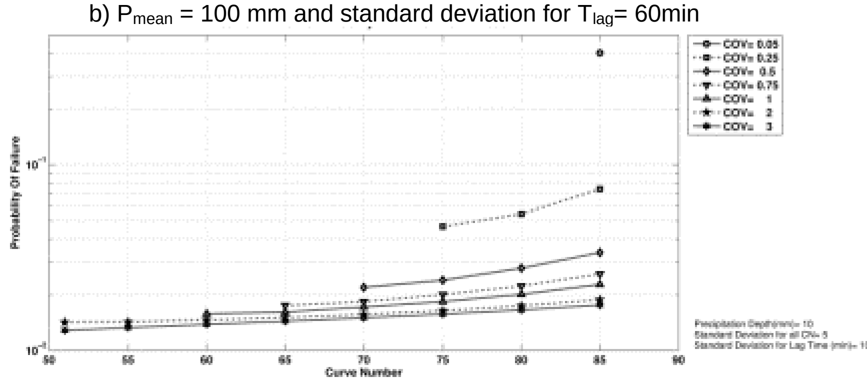
In order to reach concluding remarks, the results from all graphs are grouped and filtered based on P_f values obtained. For a conventional design aiming at a risk of 0.01, if P_f reached 0.04 (4 times 0.01), this is considered as a risky design. If P_f values are between 0.02 and 0.04, this is considered as critical (yet not recommended). If P_f values are less than 0.02, the design is considered acceptable. In the below Tables 2 and 3, risky cases are marked with “R” (with an orange hatch) and critical cases are marked with “C”. Tables 2 and 3 are samples of the complete analysis undertaken, Table 2 being for CN = 85 and Table 3 for CN = 55. From the tables, it is shown that for CN=85, all P_{mean} values are risky for P_{COV} less than 0.5. For CN=85, the upper limit of risky P_{COV} is 1.25 when P_{mean} is 3 mm. For CN=55, the pattern is reversed, for low P_{mean}, all P_{COV} are risky and the horizontal extent of the risky cases is reduced till P_{mean} of 20 mm only. It is to be noted that the selection of 0.04, for defining a *risky* case, is subjective and based on intuition, as most codes of practice usually set criteria of 50- and 100-year to important structures like bridges and culverts; while they set lower (25- and 10-year) for less important works like road positive drainage and cutoff ditches.



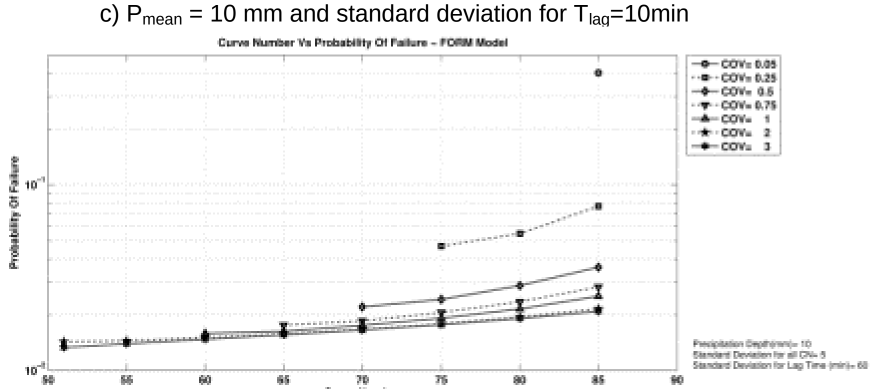
a) P_{mean}= 100mm and standard deviation for T_{lag}=10min



b) P_{mean} = 100 mm and standard deviation for T_{lag}= 60min



c) P_{mean} = 10 mm and standard deviation for T_{lag}=10min



d) P_{mean} = 10 mm and standard deviation for T_{lag}=60min

Figure 3: Probability of failure corresponding to each Curve number value under various Means of Max 24-hr (P_{mean}) and standard deviations for T_{lag}

Table 2: Minimum acceptable P_{mean} based on maximum P_f = 0.04, Curve number = 85

CN=85		P_{mean} (mm)											
		3	4	5	7	10	15	20	25	30	35	50	>60
P_{COV}	0.4	R	R	R	R	R	R	R	R	R	R	R	R
	0.5	R	R	R	R	R	R	R	R	R	R	R	C
	0.55	R	R	R	R	R	R	R	C	C	C	C	
	0.6	R	R	R	R	R	R	C					
	0.65	R	R	R	R	R	C						
	0.75	R	R	R	R	C							
	0.9	R	R	R	C								
	1	R	R	C									
	1.15	R	C										
	1.2	R											
	1.25	R											
	1.3	C											

Table 3: Minimum acceptable P_{mean} based on maximum P_f = 0.04, Curve number = 55

CN=55		P_{mean} (mm)											
		3	4	5	7	10	15	20	25	30	35	50	>60
P_{COV}	0.3	R	R	R	R	R	R	R	C	C	C	C	C
	0.35	R	R	R	R	R	R	C					
	0.6	R	R	R	R	R	C						
	1.05	R	R	R	R	C							
	1.6	R	R	R	C								
	2.35	R	R	C									
	3	R	C										

4.3 Case Study

To identify the applicability of the SCS-CN method in Egypt as an example, the above tables 2 and 3 and similar tables for other values of CN, are illustrated on a map of Egypt. This is undertaken by gathering the following information: (1) P_{mean} (Fig. 4) and P_{COV} (Fig. 5) calculated using rainfall data from 1960 till 1990, taken from the Egyptian road code of practice (Ministry of Housing Utilities and Urban Development 2008); (2) estimated CN values (Fig. 6) based on FAO Global Soil Map of the World (Food Agriculture Organization 2007) and FAO local landuse of Egypt (Food Agriculture Organization 2009). Using "if" statements on ArcGIS software and the result tables similar to Tables 2 and 3, a map (Fig. 7) for P_f actual values is reproduced. From Fig. 7, it is clear that most of P_f values in Egypt exceed 0.02, posing risky or critical conditions on the applicability of the SCS-CN method. Consequently, while the designer is aiming at a design risk of 0.01 (100-year return period), in fact the actual probability of failure exceeds 0.02 (50-year return period).

5 Conclusions and Recommendations

A reliability approach, based on First Order Reliability Method (FORM), is used in this research to evaluate the impact of incorporating the uncertainty of CN and T_{lag} and not only the randomness of the rainfall. The relative importance of the uncertainty of these parameters is assessed. It is found that the conventional approach used by many practitioners may grossly underestimate the risk of failure for water works, due to neglecting the probabilistic nature of the SCS parameters. The most predominant factors under which the SCS-CN method is highly uncertain are when the rainfall average value is low; its coefficient of variation is not significant – due to small sample sizes for example –. A case study is presented for Egypt using actual rainfall data and CN driven from satellite data. Based on the results of the present study, it is recommended that:

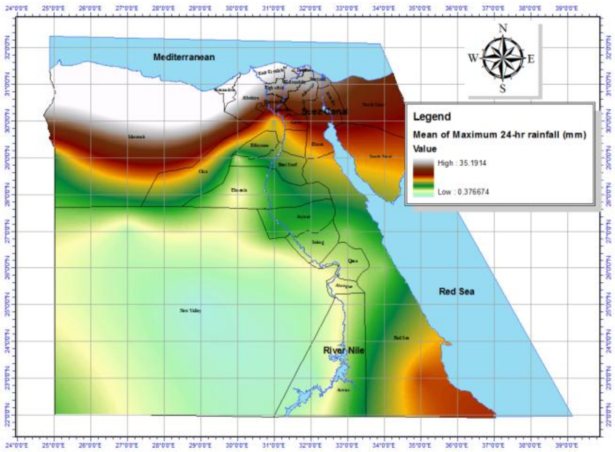


Figure 4: Map of P_{mean} for Egypt

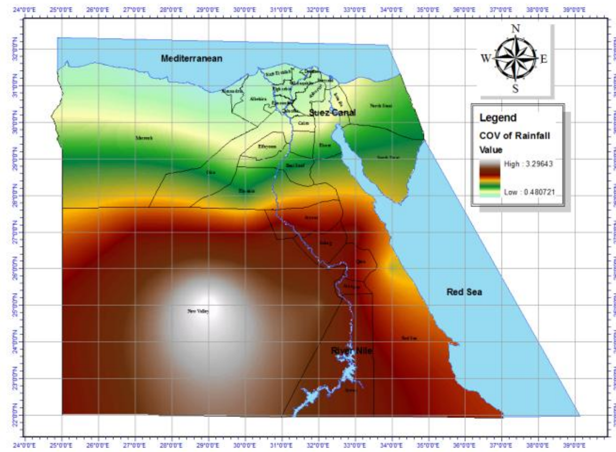


Figure 5: Map of P_{COV} for Egypt

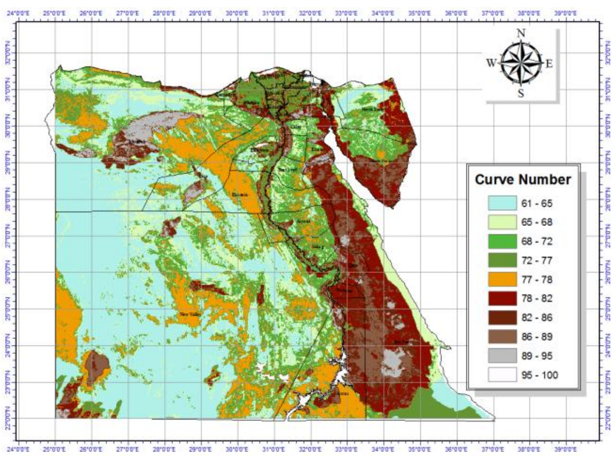


Figure 6: Map of Curve Number (CN) as deduced from FAO Digital Soil Map of the World and Landuse of Egypt

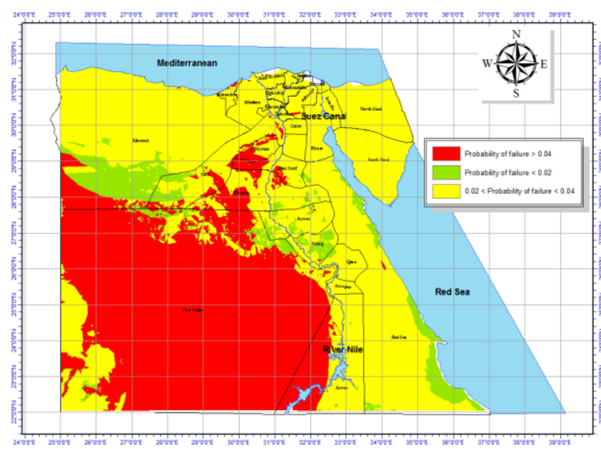


Figure 7: Resulting Probability of Failure if the CN values are considered random and if the Design Probability of Failure was 0.01 (design for the 100-year return period)

- In regions where low coefficient of variation of rainfall is located (P_{COV} < 0.50), SCS-global methodology should be avoided.
- For regions, where the combination of P_{mean}, P_{COV} and CN would lead due to actual probability of failure between 0.02 and 0.04, it is recommended to increase the used CN to account for the increased level of risk.

Future studies should be conducted to check the effect of change of λ (I_a/S ratio) on the resulting probability of failure. Studies should be conducted with different target design return period other than the 100-year design target.

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