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Risk-based floodplain management: A case study from Greece

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ABSTRACT
The use of engineering risk analysis and multi-objective decision-making under risk are considered as tools for floodplain management and extreme flood protection. Distinction is made between (a) the *catchment or large scale planning* and (b) the *local* or *small-scale design* of protection measures. After defining the risk of flooding at different scales and specifying the multiple criteria that must be considered when choosing between alternative measures of floodplain protection and management, the methodology used is illustrated in a case study from Greece (Giofyros Basin, Crete Island), where a devastating flash flood occurred on 13th January 1994. Possible remedial structural and non-structural solutions are analysed in order protect the inhabited area and important public buildings from future extreme floods.

Keywords: Flood control; detention basins; risk analysis; multi-objective decision-making; floodplain management; numerical simulation.

1 Introduction

Extreme floods are essentially natural hazards that occur infrequently. In most cases excessive precipitation is the main cause of catastrophic floods. However, anthropogenic factors, such as human occupation of floodways, extensive urbanisation and structural measures to mitigate floods (e.g. flood levees and walls, cutting of river meanders and river training) have modified the natural characteristics of extreme floods [6,13]. Recent catastrophic floods both in Europe and the USA (Rhine River, 1995; Elbe River, 2002; Mississippi River, 1993) have shown that human activity and traditional river engineering works may result in an increase in the frequency of extreme floods and the water stage with serious negative economic consequences such as loss of or damage to property as well as danger to or loss of human life.

In the Mediterranean area, flooding conditions are unique, given the influence of a semi-arid climate, geological characteristics and the socio-economic environment. The main characteristics of floods in the Mediterranean basin may be described as follows:

1. The presence of heavy rainfall in autumn and winter may produce flash floods in catchments and streams, which remain dry throughout much of the year. These flash floods are of short duration (from a few minutes to a few hours) and have high flood peaks (many hundreds of m^3/s).
2. During flash floods, soil erosion and sediment transport are important and may lead to the failure of flood-defensive engineering structures (reservoirs, spillways, gates).
3. In karst areas, which make up more than half of the Mediterranean drainage basin, flash floods are more acute and much more violent [2,9–11]. Excessive flooding occurs in these areas after the karstic cavities are filled by a huge amount of rainfall water.
4. Heavy concentrations of population in urban and residential areas around the centres of historic cities have, in many cases, resulted in the occupation of the beds and floodways of ephemeral streams. This phenomenon has been recorded mainly near the coastal areas, where tourist activity has dramatically increased in recent years. As the existing infrastructure in sewer systems is inadequate and its completion is very expensive, great volumes of storm water cannot be evacuated after heavy rains. As a consequence, the lower areas of cities become flooded and serious damage to public and private property occurs [9].

In view of the limited economic means of local authorities, the implementation of traditional engineering measures to prevent floods, such as the building of dams and drainage tunnels, is very expensive. In populated areas, extension of the existing storm sewer system is not easy, due to the high cost of replacing the existing sewers and the impact of engineering works on urban activities such as trade, tourism and traffic. A risk-based design of alternative measures may be appropriate to reduce costs and to improve the reliability of the design [8].

Floodplain management and flood control involve *alternative measures* (*structural*: levees, dikes, retention basins, channel modifications, or *non-structural*: flood warning, land uses), *different natural conditions* (type of climate, socio-economic

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environment) and *various_preferences* (economic, environmental, aesthetics, etc.). For the management of risks related to floods, various hydrological, economic and environmental uncertainties should be assessed and quantified. The US Army Corps Engineers' flood reduction structures use traditional principles of risk-based design, in that they only consider hydrological risk, which maximizes the net economic benefits from the project under various uncertainties [3].

In this paper, the general principles of engineering risk analysis are used to develop a multi-objective risk-based approach to floodplain management. The various steps taken in a comprehensive application of engineering risk analysis to flood control are: (1) identification of hazards, (2) risk quantification, (3) consequences of risk, and (4) risk management. This multi-objective risk-based floodplain management approach is illustrated in the case of the Giofyros Basin near the city of Heraklion, on the island of Crete. Hydro-meteorological data for this area, as well as data from the 1994 flood, are available.

2 Impact of the 1994 flood

2.1 Geological and hydrological characteristics

As shown in Figure 1, the hydrologic basin of the Giofyros (189 km^2) lies in the northern part of the island of Crete. The Giofyros stream outfalls through the western suburbs of the city of Heraklion to the Aegean Sea. In terms of its catchment area, the Giofyros is one of the biggest streams of this Mediterranean island, although it has a constant flow only during the humid part of the year (i.e. autumn and winter). The main geomorphological characteristics of the catchment are:

Total area:	189 km^2
Max hydraulic route:	31 km
Max altitude:	1000 m
Mean altitude:	353 m
Mean slope:	0.22

The soil is mainly alluvial and contains a relatively high percentage of clay, and some areas of rock. The area is constantly cultivated and covered mainly by vineyards and olive trees, with some forests. The climate is typically Mediterranean, with hot, dry summers and mild winters. Rainfall is quite considerable during the winter period (from October to March) and the mean, maximum and minimum annual precipitations are:

Mean annual precipitation:	827 mm
Max. annual precipitation:	1217 mm
Min. annual precipitation:	469 mm

Data from the Aghia Varvara hydrometeorological station from 1954–1994 show that the monthly precipitation in December and January has exceeded 550 mm/month at least once during the last 40 years. Many rainfall gauge stations, and some

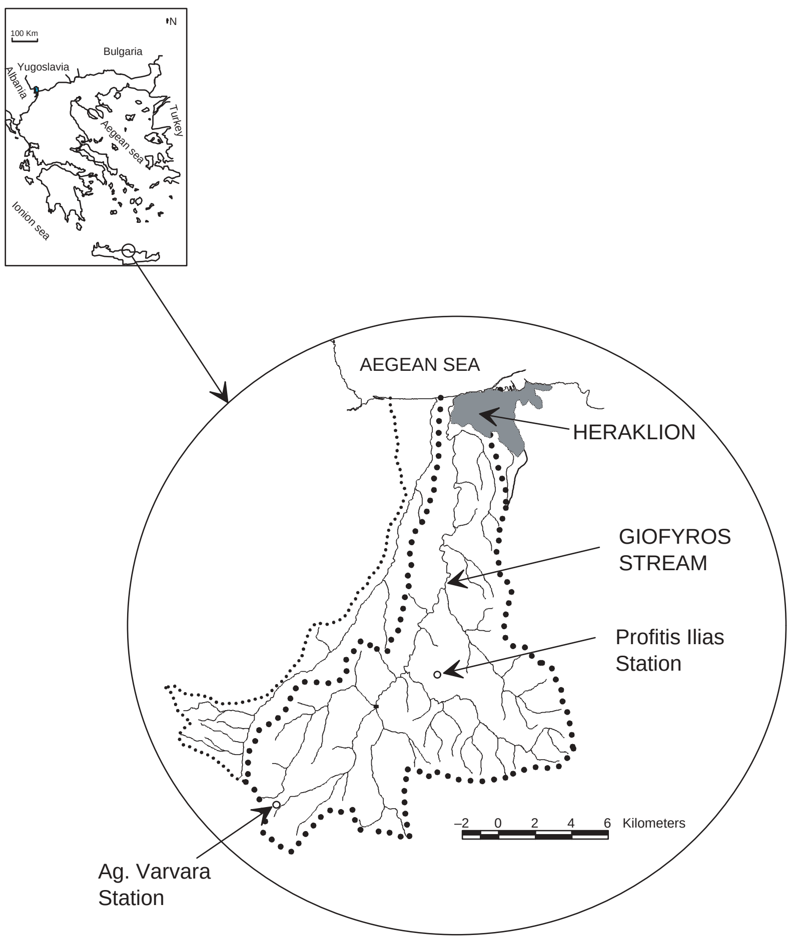

Figure 1 Location of the Giofyros catchment and hydrometeorological stations.

full meteorological stations are located in the watershed and in the neighbouring basin. The data from these stations are not easily and fully exploitable due to errors or missing periods. The best and most reliable rainfall data and rainfall intensity data are available for a period of less than a decade from the official meteorological station installed at Heraklion airport, which is out of the catchment area on the coast. The analysis of these data showed a good representation of conditions for the catchment area.

2.2 The 1994 flash flood

On 13th January 1994, a devastating flash flood occurred in the Giofyros basin. The extreme flood resulted in a series of events, which may be summarised as follows:

(a) *Heavy rainfall*. The total rainfall recorded on the day of the flood was about 185 mm, which is equal to about half of the mean annual precipitation in the region of Heraklion. A maximum rainfall intensity of 37 mm/h was recorded at the hydro-meteorological station of Aghia Varvara (Figure 2). In 6 h, which is about the retention time for the Giofyros basin, a total rainfall of 143 mm was recorded.

(b) *Soil conditions*. Rainfall of a light intensity had persisted several days before the critical storm of 13th January 1994. The soil was almost completely saturated and runoff was high during the critical storm.

(c) *Other*. Deforestation and the removal of several hectares of vineyards during the months preceding the storm probably also influenced the increased intensity of the flood.

Many houses located downstream, near the coast, were flooded and material damage was evaluated at several hundreds of thousands of Euros. The most important effect of the flood was the damage caused to the city's wastewater treatment plant, which was still under construction at the time. Many of the plant's reservoirs, made of concrete, were rendered unserviceable or completely destroyed by the force of the incoming water.

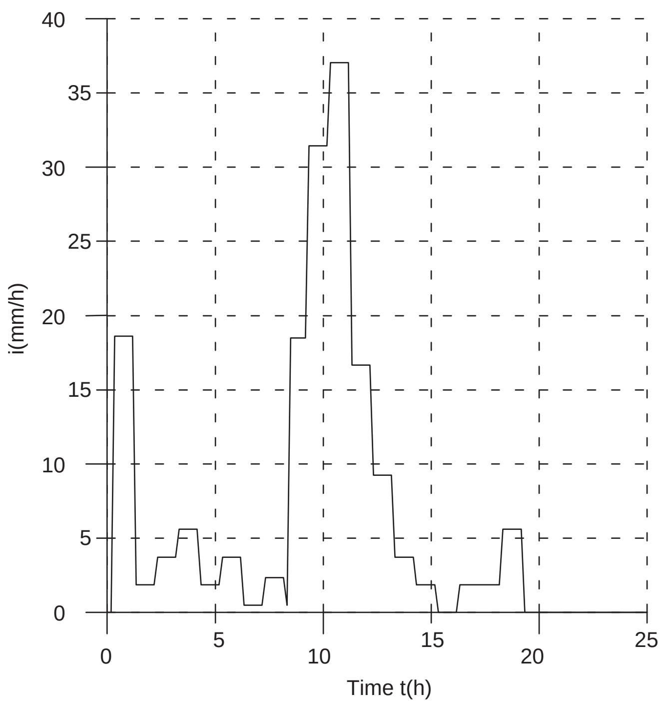

Figure 2 Relationship between rainfall intensity i (mm/h) and duration t (h) between 13–14 January 1994 (Ag. Varvara Station).

3 Risk-based floodplain management

By definition, floodplain management is an integrated consideration of all structural (engineering) and non-structural (administrative) measures to minimize losses due to flooding on the catchment scale. Selection among alternative measures to prevent floods may be made on different scales. It is useful to distinguish between

(1) *catchment scale planning*: a large scale or regional scale "optimal" selection between various alternative measures,

(2) *local scale design*: a small-scale area (sub-basin) design of hydraulic structures.

On both scales, the risk of flooding is traditionally related to hydrological uncertainties (hydrological risk). If the engineering risk is defined as the probability of failure [7,8] then on the *catchment scale* (regional scale) we have:

$$\text{risk of flooding} = P(Q > Q_T) \tag{1}$$

where P(.) is the probability, Q is the actual flood in the catchment area, and Q_T is the T-year flood.

On the *local scale* of a hydraulic structure, the risk may be defined as the probability of overtopping. As shown in the case of a simple flood levee (Figure 3), failure occurs when $h_0+z > H$.

For this case the flooding risk can be expressed as:

$$\text{risk of failure} = P(h_0 + z > H) \tag{2}$$

where h_0 is the mean water level, z is the surrevelation for a given flood, and H is the height of the levee.

Traditional risk-based design (US Corps of Engineers) incorporates uncertainty analysis under risk into an optimisation framework [3]. The objective is to select the *optimal risk-based design* that maximizes the *net economic benefits*.

If $C_D(x)$ is the expected annual damage cost due to flooding failure, $C_I(x)$ the annual installation cost and x a vector of decision variables relating to structural sizes, then the optimal risk-based design may be expressed as

$$\min_x\{C_I(x) + C_D(x)\} \tag{3}$$

under some design specifications $g(x) = 0$.

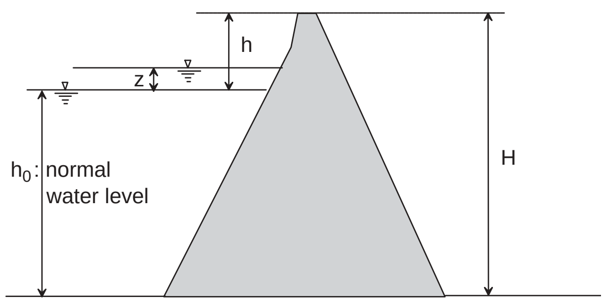

Figure 3 The flood levee problem.

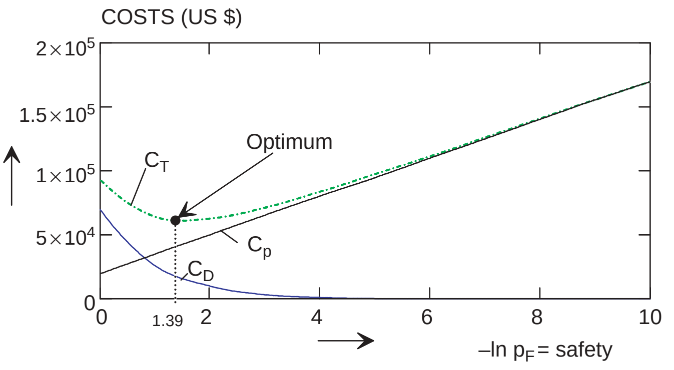

Figure 4 Total cost-reliability relationship for a flood levee.

The design level x is the structural size H for the flood levee, which may be related to the hydrological risk P_F or the hydrological reliability $(\ln p_F)$ [10]. The result of minimizing the expression (3) for a special case [10] is shown in Figure 4.

This approach has only *one objective*: the *total cost* or the *total net benefit* of the project, in other words maximized (benefit) or minimized (cost) as a function of the flooding risk. The procedure is suitable mainly for small-scale design (e.g. sizing a flood levee or a hydraulic structure), where a trade-off between costs (or benefits) and risk (reliability) may be obtained through the optimisation procedure.

On the *catchment* or *regional scale planning process*, a multi-objective approach to flood control alternatives is recommended [1]. The main *objectives* or *criteria* to be taken into consideration are

1. *Economic objectives*: Costs and benefits such as project cost, operation and maintenance costs, external costs, reduction of flood damage benefits, land enhancement, indirect benefits;

2. *Environmental objectives*: These may be positive or negative environmental impacts, such as increase or decrease in the number of species, flora and fauna modifications, losses of wetlands, landscape modification; and

3. *Social objectives*: Risk of extreme flooding, duration of construction, employment increase, impacts on transportation.

After the definition of the objectives, the steps to be undertaken for the multi-objective planning of flood control alternatives are the following [8]:

1. Define a set of *alternative actions*, which include structural and non-structural measures of flood protection;
2. Evaluate the *outcome* or *impact matrix*, i.e. assign rates to each specific objective, corresponding to each particular action; and
3. *Rank the alternative actions*, using an appropriate multi-objective analysis technique.

Different techniques are available for multi-criteria decision-making [5,12,14] and recently, distance-based techniques have been most developed, such as the following:

– ELECTRE I to III
– Compromise programming
– Goal programming
– Sequential multi-objective optimisation
– Game theory.

In selecting the most appropriate method, important criteria are the kind of objectives (quantitative or qualitative), the number of decision-makers (one or a group) and whether objectives are involved *a priori*, a posteriori or interactively. ELECTRE I to III techniques are more suitable for qualitatively expressed criteria [1]. Game and team theories [4] are mainly interactive techniques. Uncertainties and risk may be quantified by using probabilities or fuzzy sets, and can be handled better by compromise programming techniques.

Multi-criteria decision-making analysis is actually under investigation for the catchment scale planning of flood defensive measures in the Giofyros stream.

4 Application to the Giofyros basin

4.1 Large scale planning

A distinction should be made between: (a) the downstream plain area of the Giofyros stream and (b) the upstream catchment area. The downstream plain area represents about 20% of the total area of the basin and has a mean hydraulic length of 11 km. This is about 1/3 of the mean hydraulic length of the basin (30 km). Apart from some minor hydraulic works in the plain area, no other structural measures (such as reservoirs, regulation structures, etc.) have yet been implemented in the entire basin.

The hydraulic risk of flooding was first evaluated for the entire catchment area. Because no gauged data are available for flow rates, the discharge-frequency relationship was estimated by analysing the maximum rainfall-frequency data. Then, a rainfall-runoff model, such as the HEC-1, was used to estimate runoff.

The relationship between the *maximum rainfall height* (mm) and the *return period* T(yr) is shown in Figure 5 for a 2-h rainfall duration. In order to evaluate different uncertainties that influence the extrapolation results over 50- and 100-year return periods, three different methods were applied [9] including: (a) fitting a Gumbel distribution, (b) fitting the data, and (c) fitting the A and B coefficients. These coefficients appear in the following relationship:

$$h(t, T) = A(T)t^{1-B(T)} \tag{4}$$

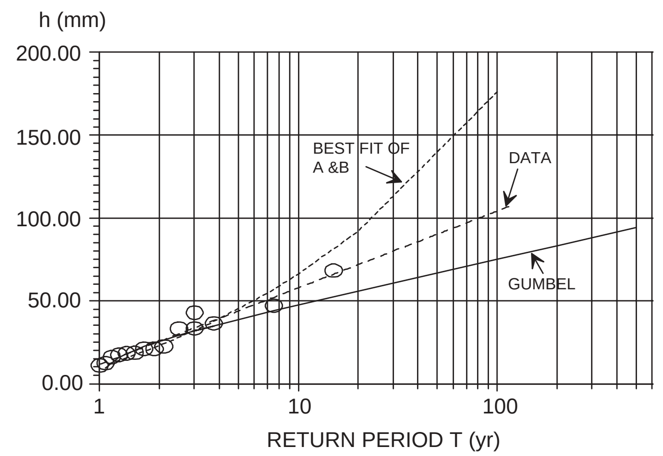

Figure 5 Maximum rainfall height (mm) versus the return period T(yr) for 2-hour rainfall duration.

Table 1 Estimated maximum rainfall height h_{max}(mm) and peak flood discharge Q_{max}(m^3/s) for return period T = 30, 50 and 100 years.

T	h (max) (mm)	Q max (m^3/s)
30	125	450
50	152	580
100	193	900

where t is the rainfall duration (min), h the rainfall height (mm) and T the return period (yr).

The maximum rainfall height and the peak flood discharge corresponding to T = 30, 50, and 100 years are summarized in Table 1.

Alternative measures for floodplain protection are combinations of three different approaches:

1. Regulation of the downstream cross-section of the Giofyros stream in order to increase the hydraulic capacity. Due to some constraints (existing bridges) the maximum hydraulic capacity can reach the 20-year flood ($Q \cong 300\,m^3/s$). Environmentally sound regulation may avoid any concrete scaling and stream training: regulation should be based on the enlargement of the cross-section, use of natural materials for fixing the bed and earthen flood levees, and should be well-integrated into the landscape.
2. Design and construction of a multi-purpose reservoir to retain a substantial volume of the critical flood. Two reservoirs of different capacities were proposed:
   2.1 A $28 \times 10^6\,m^3$ reservoir to be realized by an earthen dam of about 70 m in height
   2.2 A smaller reservoir with a total capacity of $15 \times 10^6\,m^3$
   It should be noted that the net annual water balance for the catchment is estimated at $20 \times 10^6\,m^3$, although the maximum volume of a 50-year flood is about $5 \times 10^6\,m^3$.
3. Use of a storm water detention basin network distributed over the catchment. The principal function of the system should be to reduce the peaks of the flood hydrographs. At the same time, significant volumes of water may be retained locally for agricultural purposes.

   Design of the detention basin system (i.e. size and site of flood detention reservoirs) should be adequate to sustain floods of T = 30-, 50-, or 100-year return period.

By combining the above three structural solutions, the following alternatives are currently under investigation:

1. Regulation of the downstream level of the stream (R) and construction of a large capacity reservoir (LR)
2. (R) + Construction of a small capacity reservoir (SR)
3. (R) + Detention Basin network of T = 30-yr floods (DB30)
4. (R) + Detention Basin network of T = 50-yr floods (DB50)
5. (R) + Detention Basin network of T = 100-yr floods (DB100)

The main objectives for ranking the above 5 alternatives are: (a) costs and benefits, (b) risk of failure, (c) environmental impact, and (d) social effects.

4.2 Local scale flood protection

Local authorities expressed their desire for an urgent undertaking of the necessary flood protection measures for the city's wastewater treatment plant. The issue was to determine the size of the flood levees around the sewage treatment facility in order to protect important civil and mechanical equipment from future extreme floods. Emphasis was placed on safety rather than cost, because of the importance of the plant and the relatively small volume of the levees.

For the design of the flood levees on the local scale, a two-dimensional mathematical model was used to propagate the flood hydrograph. Different hydrographs representing the historical flood (13 January 1994) and the T = 30-, 50-, 100-year return periods were simulated. The mathematical model consists of the following mass continuity and Saint–Venant equations:

$$\frac{\partial h}{\partial t} + \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = 0 \tag{5}$$

$$\frac{\partial q_x}{\partial t} + \frac{\partial}{\partial x}\left(\frac{q_x^2}{h}\right) + \frac{\partial}{\partial y}\left(\frac{q_x q_y}{h}\right) = -gh\left\{\frac{\partial h}{\partial x} - (I_{fx} - I_{0x})\right\} \tag{6}$$

$$\frac{\partial q_y}{\partial t} + \frac{\partial}{\partial x}\left(\frac{q_x q_y}{h}\right) + \frac{\partial}{\partial y}\left(\frac{q_y^2}{h}\right) = -gh\left\{\frac{\partial h}{\partial y} - (I_{fy} - I_{0y})\right\} \tag{7}$$

where h is the flood stage in m, q_x, q_y the flow rates per unit width in m^3/s/m, I_{0x}, I_{0y} the bed slopes, and I_{fx}, I_{fy} the friction slopes.

The Manning formula was used to compute I_{fx} and I_{fy} as functions of q_x, q_y and h. Numerical integration of the above equations was performed over a two-dimensional grid using finite differences. A 100 m grid size was selected. The model was validated by comparing the numerical results with data available from the historical flood of 13th January 1994. On that day, the maximum water levels at different locations inside the wastewater treatment plant were recorded.

Results of the numerical simulation indicating the contour lines of the water stage during the 1994 flood are shown in Figure 6. For the same flood, water stage hydrographs computed at characteristic locations are shown in Figure 7. After defining the size of the flood levees around the wastewater plant, results of the simulation of the T = 100-year flood are shown in Figure 8. It can be seen that the space where the wastewater treatment plant is located is well protected from this extreme flood. Further protection of the local area will be provided after implementation of the flood detention basin network in the upstream catchment area as described in Section 4.1.

5 Conclusions

Special attention should be paid to the floodplain management measures in areas with semi-arid climates. In these areas, flash floods in ephemeral streams can be violent and unpredictable. Risk-based design methodologies for protection measures may result in trade-offs between risk and costs, as well as having environmental and social impact.

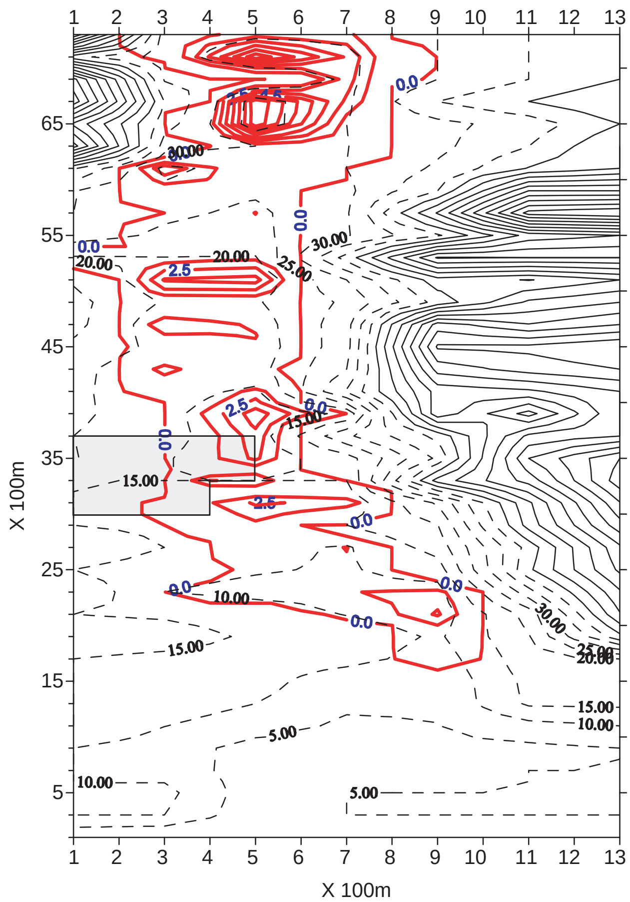

Figure 6 Contour lines of water stage for the 1994 flood (no flood levees around the wastewater treatment plant).

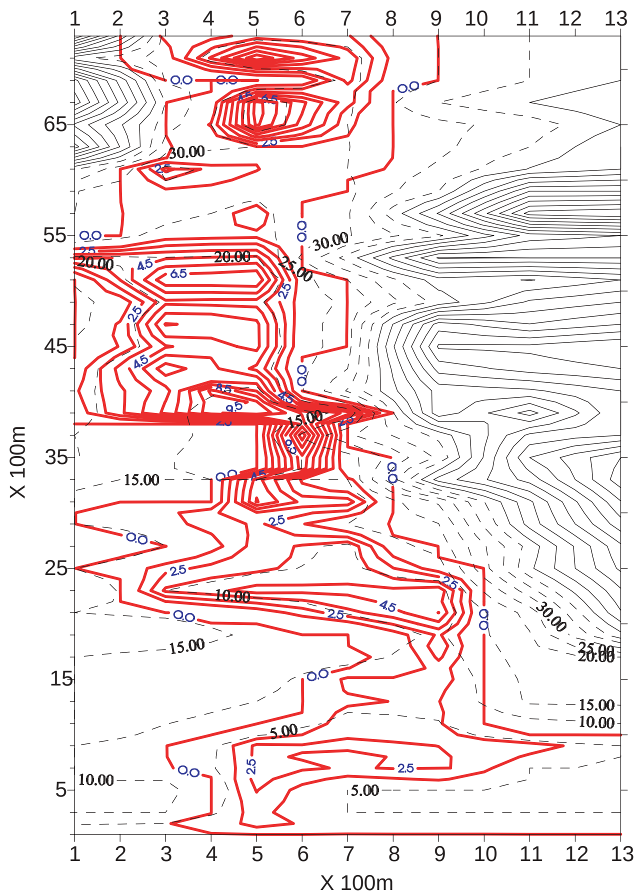

Figure 8 Contour lines of water stage for the T = 100-year flood, after construction of the flood levees around the wastewater treatment plant.

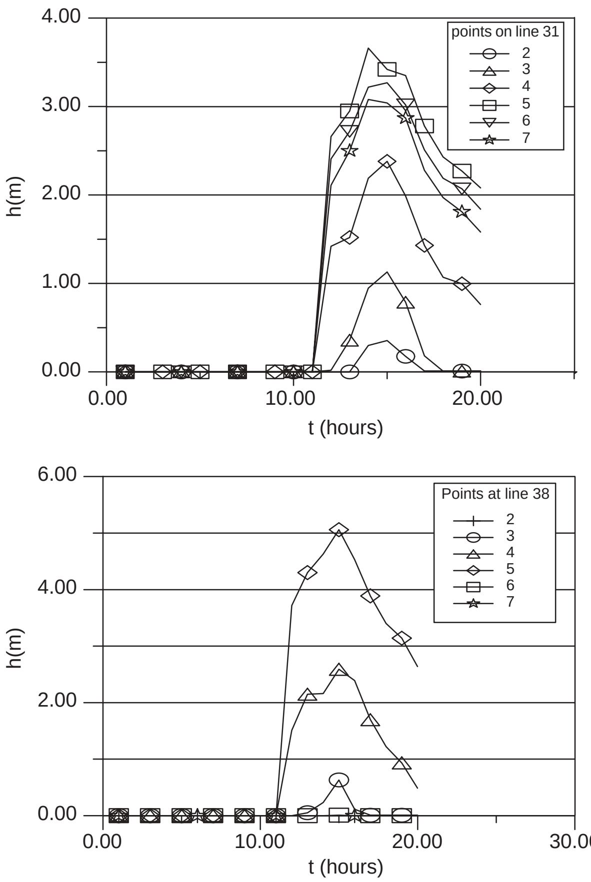

Figure 7 Water stage hydrographs h(t) at characteristic locations.

Distinction is made between catchment scale planning and local scale protection from floods. On the former scale, a multi-criteria decision making approach to areas under risk may help in selecting between different alternatives. In areas without too many constraints (e.g. high population or intensive agriculture) a storm water detention basin system distributed over the entire catchment area seems to be the most appropriate. On a local scale, reliability of the protection measures may be based on more traditional techniques involving hydrological and hydraulic modelling of two-dimensional steady flows.

The above methodology was applied to the Giofyros basin, on the island of Crete, Greece.

Notations

A, B	=	fitting coefficients
C_D	=	annual damage cost
C_I	=	annual investment cost
h	=	flood stage
H	=	height of the levee
h_0	=	mean water level
h_{max}	=	maximum rainfall height
i	=	rainfall intensity
I_{0x}, I_{0y}	=	bed slopes
I_{fx}, I_{fy}	=	friction slopes
P(.)	=	probability
P_F	=	hydrological risk

Q	=	flow rate
Q_T	=	T-year flood
q_x, q_y	=	flow rates per unit width
T	=	return period
t	=	time
z	=	surrevelation

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