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Feasibility Study of Rainwater Harvesting in Semi-Arid Region

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ABSTRACT

Rain water harvesting in hot arid and semi-arid regions should be based on the study of overland flow on porous surface, effect of surface soil treatment on quantity and quality of runoff produced, extent of land treatment to get required volume of water to be conserved. In view of the short duration rainfall and high infiltration and evaporation rates in the arid regions, the time steps used for computations should be much less than the duration of rainfall. Runoff produced from short duration rainfall may be lost during transmission to ephemeral stream. Therefore the runoff should be collected before it is lost during transmission. The runoff should be stored in underground storage tanks so that the evaporation losses are minimum.

From the study of overland flow on porous surface it is found that the catchment should be treated such that the hydraulic conductivity of the treated surface soil should be less than one third of the rainfall intensity. If the in-situ soil has conductivity less than one third of rainfall intensity, overland flow occurs which can be harvested.

Drinking water can be supplied at a rate of about Rs. 42 per 1000 litres in an area receiving an annual rainfall of 300 mm and having potential evaporation loss of 1800 mm. For an area where the annual rainfall is 250 mm and potential evaporation loss is 3000 mm, water can be harvested at a rate Rs. 50 per cubic meter (i.e. paisa 5 per litre) to a population of 1000 with water requirement of 125 litres per capita per day. Detailed hydrological investigations are necessary before implementation of the water harvesting project.

1.0 INTRODUCTION

Water is the source of life, hence it should be conserved and utilised properly. A large population in the remote rural areas of arid regions having no access to community water supply or other drinking water resources like deep wells, step wells, ponds, lakes, hand pumps etc. have been thriving on rain harvested water to quench their thirst by devising their own mechanism of storing rainwater in their respective homes as per their capacity to mitigate their annual domestic water need. Rainwater harvesting is defined as artificial methods for collecting and storing precipitation until it can be used for watering livestock, small-scale subsistence farming, and domestic use. The water harvesting system includes catchment area, usually prepared to improve runoff efficiency, and a storage facility for the harvested water, unless the water is to be immediately concentrated in the soil profile of a smaller area for growing short hardy plants. A water distribution scheme is also required for those systems devoted to irrigation (Brooks et al, 1998) .

The arid zone of India occupies an area of 3.2 lakh sq. km. of hot desert mostly in Rajasthan, Gujarat, Haryana and Karnataka. Rainwater harvesting and water conservation are the most important initial activities to support other activities like afforestation for preventing desertification, grassland management, livestock farming, dryland agriculture and horticulture, and exploitation of wind and solar energy. With appropriate watershed treatment, it is possible to harvest runoff water before it is lost through the process of infiltration in ephemeral stream. Once the harvested water is judiciously applied either by sprinkle or drip irrigation, the land resource could be prevented from desertification through appropriate forestry and range management. For rainwater harvesting, an understanding of the hydrological processes in arid and semi-arid regions is required in respect of surface runoff, recharge and evaporation. In the process of storage of harvested rainwater, various kinds of losses occur such as transmission losses, evaporation losses and infiltration losses. To minimise these losses, the storage tank should be near the harvesting catchment and under ground.

2.0 REVIEW

The geometric configuration of water harvesting system depends on the topography, the type of catchment treatment and the intended use. The apron type of harvesting system is used primarily for livestock, wildlife, and domestic water supplies. It is designed for minimum maintenance and the catchment is fenced. The catchment area is treated to obtain a high runoff efficiency, unless an existing impermeable surface is in place. Gravel-covered asphalt-impregnated fibreglass is a common treatment. A storage tank with evaporation control is required (Brooks et al. 1998). The apron-type system is the simplest to design. As a first approximation for the size of apron required, the following equation has been suggested by Brooks et al. (1998):

$$A = b\ U/P$$

where A=catchment area (m^2); b=1.13, a constant; U=annual water requirement (litre); and P=average annual precipitation (mm).

Roaded catchments are well suited for growing high-value horticultural crops such as fruit trees, nut trees, and grapes and for providing water for livestock. These catchments are best adapted to very gently sloping ground. A roaded catchment consists of parallel rows of drainage 100 m or less long and spaced 15-18 m apart. Trees or horticultural species are planted in the drainage. The areas between drainage are shaped much like high-crowned roads to serve as catchments. Side slopes of the catchment roads and longitudinal slopes of the drainage should not be more than about 2% to prevent erosion. The catchments are cleared of vegetation, smoothed, and treated to reduce infiltration. Sodium chloride has been found to be effective on expanding-clay soils. If high-value horticultural crops are grown, water storage is necessary to provide supplemental irrigation water which is accomplished easily by diverting excess water from the drainage into a storage facility.

Water harvesting for agriculture requires a more complex system. The size of catchment area of a water harvesting system in relation to that of the agricultural area must be balanced against crop demands and water storage capability. However, the system can be readily adapted to existing topography, provided there is a level area to farm and care is taken with catchment construction to provide gradual slopes or, in steep terrain, short slopes broken by diversions. Since relatively large quantities of water is usually required, application of catchments are necessary, but treatments can be expensive. NaCl, one of the least expensive treatments, is effective in locations where the soil has a sufficient quantity (about 10% or more) of expanding clays.

2.1 Catchment Area

The catchment area of a water-harvesting system should be sufficiently impermeable to water to produce runoff. Some examples of different catchment surfaces are:

1. Natural surfaces, such as rock outcrops.
2. Surfaces prepared with minimal cost and effort, such as those cleared of vegetation or rocks and smoothed, or both smoothed and compacted.
3. Surfaces treated chemically with sodium salts, silicones, latex, or oils.
4. Surfaces covered with asphalt, concrete, butyl rubber, metal foil, plastic, tar paper, or sheet metal.

The particular surface treatment selected depends largely upon the cost and availability of materials and labour. In general, the greater the runoff efficiency and life of treatment, the greater the cost. At one end of the scale, simple smoothing and compaction of nonporous soils is effective but requires annual maintenance. On the other end of the scale is asphalt-impregnated fibreglass covered with gravel, which can last 20 yrs or more.

Desirable characteristics of catchment treatments include:

1. Runoff from the surface must be nontoxic to humans, plants and animals.
2. The surface should be smooth and impermeable to water.
3. The surface material should have high resistance to weathering and should not deteriorate because of chemical or physical treatments.
4. The surface material need not have great mechanical strength, but it should be able to resist damage by hail or intense rainfall, wind, occasional animal traffic, moderate water flow, plant growth, insects, birds, and burrowing animals.
5. The surface material should be inexpensive and require minimum site preparation.
6. Maintenance should be simple.

Obviously, no single treatment would have all these characteristics. Some trade-off is necessary, but lowest cost over the long term is often the overriding objective. Estimates of the costs of water harvesting using various catchment treatments in the United States are given in Table 1.

Keeping in view the various problems in western Rajasthan, particularly in border districts (Barmer, Bikaner and Jaisalmer where soils have high infiltration rate and acute problem of drinking water exist), **Central Arid Zone Research Institute, Jodhpur,** carried out experiment for a period of four years involving the techniques of water harvesting from small catchments to increase the water yield for dug out Nadis and Tankas in the desert region.

Table 1 : Water costs for various water-harvesting treatments

Treatment	Runoff %	Estimated life of treatment (yr)	Initial Treatment cost (US$/ha)	Annual amortized cost (US$/ha)	Water cost in a 500 mm rainfall zone (US$/$10^3$ m^3)
Rock outcropping	20-40	20-30	<120	<240	58-119
Land clearing	20-30	5-10	120-230	<120	79-119
Soil smoothing	25-35	5-10	600-840	120-240	66-188
Sodium dispersant	40-70	3-5	840-1440	120-240	34-119
Silicone water repellents	50-80	3-5	840-1440	120-240	34-119
Paraffin wax	60-90	5-8	1440-2160	240-280	58-188
Concrete	60-80	20	3600-4800	600-1200	132-394
Gravel-covered membranes	70-80	10-20	24,000-60,000	2040-5280	499-1725
Asphalt fibreglass	85-95	5-10	6000-8400	480-1200	119-335
Artificial rubber	90-100	10-15	12,000-24,000	1680-5760	346-1321
Sheet metal	90-100	20	24,000-36,000	2040-3120	399-679

The treatment materials were so selected that these materials are easily or locally available. These were: (1) Bentonite (20%) mixed with soil, (2) Soil cement (8%) mixture, (3) Mud plaster (Tank silt + wheat husk), (5) Lime concrete, (6) Jantha emulsion premix (a type of asphalt), (7) Mechanical stabilisation, (8) Sodium carbonate spray (Dhobi soda) 1 kg/10 sq. m., (9) Mud plaster (tank silt + husk 3% + Jantha emulsion 2%). The technique consists in the preparation of artificial catchments and compacting these with different materials mentioned above to the thickness of 1.25 cm. The results of treatments studies are given in Table 2.

Selection based only on cost can be a mistake, however. For example, simple, smoothed catchments produce water at low cost, but they do not provide runoff from small storms that characterize rainfall periods in many arid zones. A large, expensive structure might have to be built to store water for use during the period when no runoff occurs. The cost of a simple, smoothed catchment plus the large storage required could be greater than the cost of a more expensive catchment that provides runoff from small storms.

Table 2 : Treatment type, run-off efficiency, and life of treated catchment

S. No.	Year	1972	1973	1974	1975	Cost of treatment material per sq. m. (Rs.)
	Rainfall in mm	316.90	502.05	130.60	497.32	
	No. of rainy days	11	19	5	21	
	Treatment	Runoff %	Runoff %	Runoff %	Runoff %	
1.	Control (No treatment)	57.42	22.15	29.81	6.62	0.00
2.	Bentonite 20% mixed with soil, 1.25 cm thick	87.53	62.71	51.30	12.80	1.25
3.	Cement 8% mixed with soil, 1.25 cm thick	41.14	28.52	22.74	7.17	0.90
4.	Mud plaster (local) 1.25 cm thick	66.62	52.00	38.23	9.18	0.45

5.	Lime concrete 5 cm thick	74.48	65.21	47.99	36.07	0.45
6.	Jantha emulsion premixed 1.25 cm thick, 8% solution of Jantha emulsion and K. oil @ 4.1	94.06	82.26	66.20	29.20	3.10
7.	Mechanical stabilization	65.22	48.28	28.15	7.78	0.30
8.	Sodium Carbonate spray @ 1kg / 10 sq. m over 1.25 cm thick tank silt compacted	91.75	75.70	63.46	34.40	0.60
9.	Mud plaster (RRL) mixture of mud, bhusa and Jantha emulsion (95:3:2)	78.76	67.62	48.82	20.27	1.20

A catchment surface has no standard shape. Flexibility is encouraged to utilize the natural topography to minimize the construction costs. The slope of the surface should be only as steep as necessary to cause runoff; ideally, the slopes should be less than 5%. Slopes that are too steep can erode and produce high amounts of sediment in the runoff water. The catchment surface must be cleared of vegetation, rocks, and other debris that might reduce the durability of a treated surface or retain water on the surface.

From the results given in Table 2, it has been inferred that in first year, the percentage of runoff was the highest i.e. 94.06 percent of rainfall from Jantha emulsion treatment followed the next highest yield i.e. 91.75 percent of rainfall by sodium carbonate spray. In subsequent years, the runoff generated from Jantha emulsion treatment and sodium carbonate spray was 82.26 %, 66.20 %, 29.20 % and 75.70 %, 63.46 %, 34.40 % respectively. While comparing these results with respect to control (no treatment), it can be seen that the Jantha emulsion and sodium carbonate spray generated 63.81 % and 59.78 % more runoff in the first year. Due to inherent problems with other treatments, the runoff generated by them in first year was much less than these two treatments except Bentonite mixed with sand. Bentonite treatment, initially though it creates water proofing in the surface, but subsequently it fails in the long run due to the development of cracks at the surface on drying. In general, the efficiency of all the treatments was reduced year by year because of deterioration of materials. But still sodium carbonate spray generated the maximum i.e. nearly 5 times than the control even after four years of application of treatment. In general, its efficiency varies from 34.40 to 91.75 percent of rainfall occurred within the years. Looking to the cost of treatment per sq.m., percentage of runoff generated and life span of sodium carbonate spray is much economical than any other tried treatment.

In arid and semi-arid zone of Australia, where the average annual rainfall is less than 250 mm, surface runoff is widely used to stock water and for diluting the saline ground water.

Hydrological study on arid region of Western New South Wales shows that although soils vary considerably from one part of the region to other, and the hilly and flat areas are physically very different, these factors don't appear to greatly influence the production of runoff. "Soils and vegetation adopt to their respective regional climate in a way which result in surface runoff production under the regime of low precipitation as well as under the high rainfall regime (Pilgrim et al., 1979; Cordery et al., 1983)".

From the hydrological study conducted by Cordery et al. (1983), it has been found that there is a large amount of surface runoff in both hilly and flat areas of Semi Arid Fowlers Gap Catchment. It had been proposed to store the runoff in deep excavated tanks to minimize the effects of the 3.5 m annual evaporation. The stored water is supplied upto two to three years after a runoff event. An important finding of their study is that rainfall well above 50 mm is required for runoff to be assured in humid zone, but a depth of 20 mm in 24 hours or 5 mm in one hour produces runoff from arid area. The initial losses are much lower in arid zone than the losses in humid zone.

For rainwater harvesting, the runoff should be collected near its source. Otherwise they will be lost as transmission loss in the influent stream. In arid lands, potential evaporation , affected mainly by solar radiation, is much higher than annual rainfall. In Sudan in the vicinity of Khartoum, daily evaporation reaches 7.5 mm, and near Wadi Haifa 7.9 mm. In some arid regions of India, the annual evaporation is up to 3000 mm (Worthington, 1976, pp-11). From the atlas of Rajasthan published by CGWB (1994), the annual evaporation in arid zone of

Rajasthan varies from 1400 to 2000 mm per year (Fig.1). The normal annual rainfall in Rajasthan varies from 250 mm in north west to 1100 mm in south east. The large potential evaporation of arid region dictates that the most useful surface storage are excavated tanks that are deep with minimum surface area.

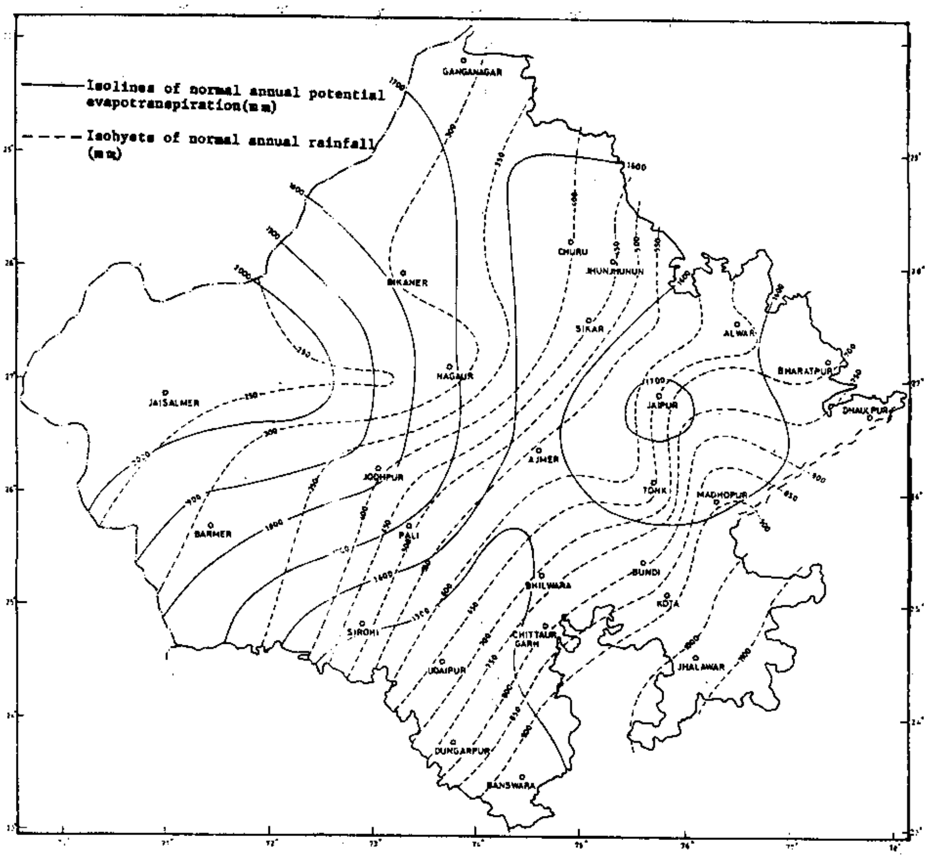

Fig. 1: Isohyets of annual rainfall and isolines of annual potential evapotranspiration.

3.0 POSSIBILITY OF RAINWATER HARVESTING IN RAJASTHAN

The semi-arid zone of Rajasthan, India, receives rainfall during July to October and the average annual rainfall is of the order of 225 mm. The possibility of harvesting rainwater in the desert area of Rajasthan needs investigation.

Rainwater harvesting in Rajasthan is an age old practice. Evidence of practice of rainwater harvesting is conspicuous in Bikaner district near village Darbari (Fig.2)

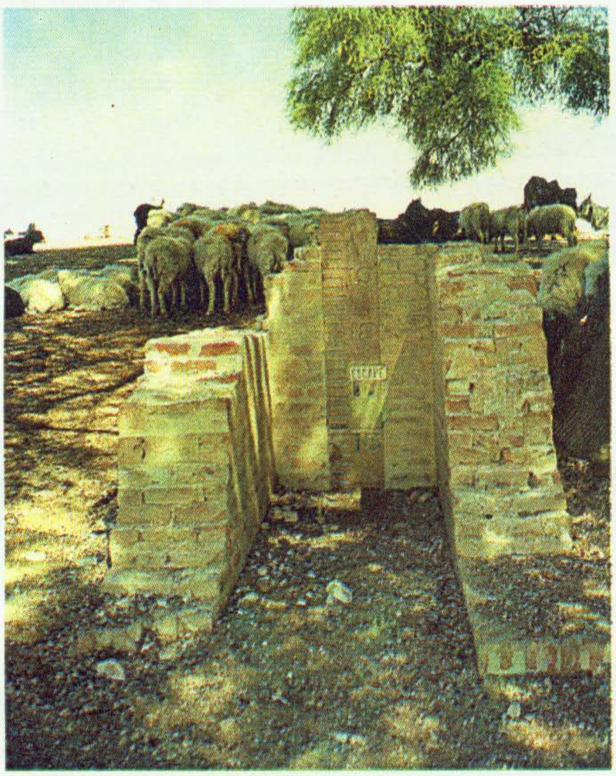

Fig.2 : Exit constructed in the bank of the pond to allow excess water to escape

In Fig.2, one can see exit constructed for the escape of the excess water stored in a pond. Rain water is directed to the pond (Fig.3) from catchment (Fig.4). The harvested water stored in pond, is available up to March and beyond. Domestic animals use this water for drinking and bathing. Trees and shrubs around the pond also use this water. The quantity of water stored could be increased by treating the catchment. Small concrete structures are constructed by the farmers to collect overland flow during monsoon period (Fig.5). The size of the structure can be increased to store more runoff and this may eventually lead to construction of a pond of large size.

Fig.3 : Water diverted from a catchment to a pond which gets filled during monsoon

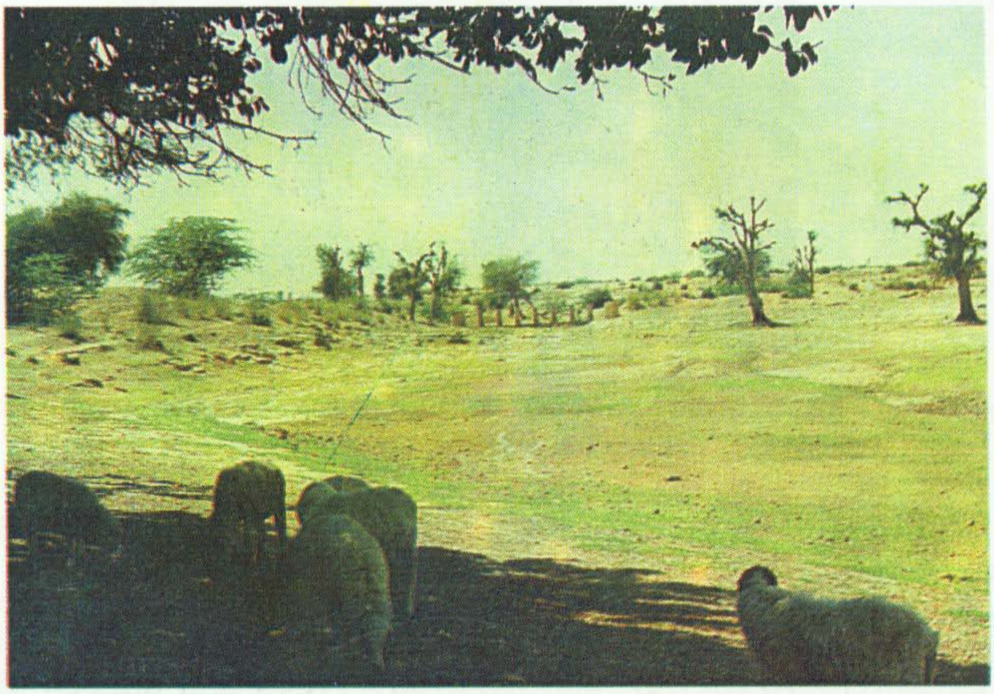

Fig. 4: A view of the catchment near village Darbari; surface runoff from this catchment is diverted to the pond.

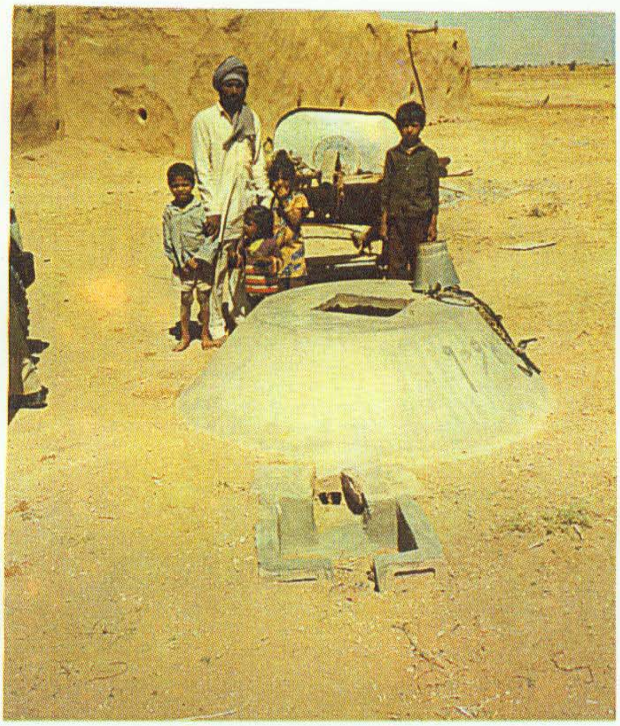

Fig. 5 : Small concrete structure constructed to store water from overland flow during monsoon

In some parts of the lift canal command of IGNP Stage II, there exists a hydrologic barrier layer whose conductivity is of the order of 0.02 m/day to 0.002 m/day. At some places, the barrier layer is also exposed at ground surface. The barrier layer is lying at an average depth of 5.5 m below ground surface. The thickness of the barrier layer is more than 20 m. Rainwater can be collected from the region where the barrier layer is exposed and can be stored in tanks constructed in the barrier layer. If necessary, the surface of the catchment can be treated so as to reduce the infiltration rate and the transmission losses. The area of the surface to be treated can be ascertained from analysis of overland flow on porous surface. From the analysis of time scale of rainfall and time scale of runoff, the necessity of treating the catchment for control of infiltration can be ascertained. The chemical treatment to be given to the surface to reduce infiltration is to be studied keeping in mind its durability under high temperature in arid region and its influence on water quality. The size of the tank should be determined so that evaporation losses are minimum.

4.0 DATA REQUIREMENT

The following data are required for designing the rainwater harvesting system:

1. Quantitative estimates of the surface water resources;
2. The runoff production process and their spatial variability;
3. Minimum rainfall for which runoff may be expected;
4. Land system identification on the basis of relief, lithology, soils and vegetation; and
5. Climatological Data:

(a) mean maximum daily temperature (in a month),
(b) mean minimum daily temperature,
(c) relative humidity, daily
(d) observed mean monthly pan evaporation,
(e) continuous record of rainfall, daily
(f) mean number of wet days,
(g) upper quartile of monthly rainfall,
(h) lower quartile of monthly rainfall.

The data should be collected from natural basin with defined stream channel, from small runoff plots within the natural basin, and from large plots on that part where there is no defined channels or basin boundaries.

Existing water storage can be conveniently used as stream gauging sites because of the following reasons:

i) Lack of suitable natural control

ii) Cost of construction and difficulties with artificial controls

iii) Impossibility of calibration of stations with conventional gauging

iv) Problem associated with high sediment loads.

Water level records used in conjunction with storage volume calibrations permit computation of volumes and rates of runoff.

The following types of storage can be created for estimation of the runoff in the catchment.

(1) **Storage created by constructing earth banks :** For basin area upto 11 km^2, storage is created by constructing earth embankment across the stream.

(2) **Excavated tank storage :** They are excavated at the down slope ends to act as collecting pits and are fitted with water level recorder. They are constructed to collect water from large plots on flat land. Large plots have been delineated with earth banks.

(3) **Cylindrical pits :** They are used for small plots of around 25 m square size. The plots are diamond shaped delineated by galvanized steel cut off. Runoff is collected in a cylindrical pit fitted with a recorder. The aim of the plots is to obtain data on spatial variation of volumes, rates and timing of runoff and loss, and to compare these with the values for the whole basin.

5.0 RAINFALL - RUNOFF MODELLING FOR THE TREATED CATCHMENT

An analytical study using simple overland flow modelling has been carried out considering infiltration to find the relation between a given rainfall and runoff volume produced from small treated watershed. For this purpose, one-dimensional shallow water flow equations have been solved along with the two-dimensional Richards equation for calculating the infiltration at the ground surface. The governing equations for surface and subsurface flow have been given below.

5.1 Governing Equations

Mathematical modelling of rainwater harvesting involves the solution of the governing equations for both the surface flow and the subsurface flow with infiltration at the ground surface acting as the connecting link. In the present study , the surface flow is represented by one-dimensional flow equations in the x-direction while the subsurface flow is represented by the two-dimensional Richards equation in the x and z directions.

5.1.1 Surface Flow Equations

Surface flow is assumed to occur in a prismatic channel of rectangular section. The one-dimensional shallow water flow equations in conservation form for such a case are given by:

<u>Continuity equation</u>

$$\frac{\partial h}{\partial t}+\frac{\partial q}{\partial x} = (R-I) \tag{1}$$

<u>Momentum Equation</u>

$$\frac{\partial q}{\partial t}+\frac{\partial}{\partial x}\left[\frac{q^2}{h}+\frac{gh^2}{2}\right] = gh(S_o-S_f) \tag{2}$$

Equations (1) and (2) can also be written in vector forms as:

$$\frac{\partial U}{\partial t}+\frac{\partial F}{\partial x} = S \tag{3}$$

in which, U, F and S are vectors and are given as

$$U = \begin{Bmatrix} h \\ q \end{Bmatrix}, \quad F = \begin{Bmatrix} q \\ \frac{q^2}{h}+\frac{gh^2}{2} \end{Bmatrix} \quad \text{and} \quad S = \begin{Bmatrix} (R-I) \\ gh(S_o-S_f) \end{Bmatrix} \tag{4}$$

where, h=depth of flow (m); q=discharge per unit width (m^2/s); R=volumetric rate of rainfall per unit surface area (m/s); I=volumetric rate of infiltration per unit area (m/s); S_o=bottom slope in the direction of flow; S_f=friction slope; g=acceleration due to gravity; x=distance along the flow direction (m) and t=time (s). The derivation of the above equations have been reported by Chaudhry (1993). The assumptions in deriving the above equations are as follows:

1. The pressure distribution is hydrostatic. This is true if the vertical acceleration is small, i.e. if the streamlines do not have sharp curvatures.

2. The slope, θ, of the channel bottom is small so that $\sin\theta \approx \tan\theta$ and $\cos\theta \approx 1$.

3. The velocity distribution along the depth is uniform.

4. The channel is straight and prismatic.

5. The transient-state friction losses may be computed using formulae for the steady-state friction losses.

The friction slope can be computed by using the Darcy-Weisbach formula and taking into account the effect of rainfall on frictional resistance. The Darcy-Weisbach equation for computing the friction slope is given by

$$S_f = f_d \frac{q^2}{8gh^3} \tag{5}$$

where, f_d = frictional resistance coefficient. Evaluation of f_d depends on the instantaneous state of flow (Akan and Yen, 1981) and is given by the following formulae.

(i) Laminar flow (Re < 900):

$$f_d = \frac{C_L}{Re} \tag{6}$$

(ii) Transitional flow (900 < Re < 2000):

$$f_d = \frac{0.223}{Re^{0.25}} \tag{7}$$

(iii) Fully turbulent flow (Re > 2000):

$$f_d = \left(2 \log \frac{2h}{k} + 1.74\right)^{-2} \tag{8}$$

in which,

$$C_L = 24 + 0.21743 \ R^{0.407} \tag{9}$$

In Equations (6)-(9), Re = Reynolds number = q/ν (ν = Kinematic viscosity of liquid), k = a length measure of surface roughness and R=rainfall.

5.1.2 Subsurface flow equation

The subsurface flow is considered as two-dimensional motion of a single-phase incompressible fluid. The two-dimensional, transient, unsaturated flow equation in an isotropic porous medium is derived by applying the principle of conservation of mass and the basic Darcy's law for unsaturated flow and making the following assumptions:

(i) Compressibility of the medium and the water are negligible, and
(ii) The air phase is stagnant and is at atmospheric pressure.

The two-dimensional continuity equation without sources and sinks term within the flow domain can be written as

$$\frac{\partial \theta}{\partial t} + \frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial Z} = 0 \tag{10}$$

in which, θ=volumetric moisture content; V_x and V_z = Darcy flow velocities in the x and z directions, respectively, and, x and z are distances along the two coordinate directions. z is taken positive down wards. It is assumed that the Darcy's law is applicable for evaluating the velocity components. The Darcy's law for unsaturated flow in the x and z directions for an isotropic soil is

$$V_x = -K(\psi) \frac{\partial \psi}{\partial x}, \qquad V_z = -K(\psi) \left(\frac{\partial \psi}{\partial z} - 1\right) \tag{11}$$

in which, ψ =pressure head (m); and $K(\psi)$= unsaturated hydraulic conductivity (m/s), which depends on the pressure head, ψ. Substitution of Equation (11) in Equation (10) yields the Richards equation (Freeze and Cherry, 1979):

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x}\left[K(\psi) \frac{\partial \psi}{\partial x}\right] + \frac{\partial}{\partial z}\left[K(\psi)\left(\frac{\partial \psi}{\partial z} - 1\right)\right] \tag{12}$$

Equation (12) is said to be in "mixed form" since it includes both the dependent variables θ and ψ. Most of the earlier studies on unsaturated ground water flow have employed the Richards equation in either pressure head form or moisture content form. The numerical models based on the mixed form of Richards equation can guarantee mass balance while having no limitations when applied to field problems. The main difficulty in applying the Richards equation to actual field situations is the estimation of the parameters of the soil characteristic curve. Characteristic relationships between the pressure head, ψ and the hydraulic conductivity, K, (ψ-K relationship) and between the moisture content, θ and the pressure head, ψ, (ψ-θ relationship) are needed while solving Eqs. (10) and (11) in the unsaturated zone. In general, ψ-K and ψ-θ relationships are not unique and soils exhibit different behaviour during wetting and drying phases. This hysteresis in soil characteristic has not been considered for the cases studied in the present work. However, the hysteresis can be included by employing different ψ-K and ψ-θ relationships for wetting and drying processes.

5.2 Numerical Solution

5.2.1 Surface Flow

Surface flow equations constitute a set of nonlinear hyperbolic partial differential equations. A recently developed high resolution Essentially Non-Oscillating (ENO) scheme for solving the shallow water flow equations (Nujic, 1995) has been employed in the present study to solve the surface flow equations. The scheme, proposed by Nujic (1995), has been suitably modified by Singh (1996), to account for the non-zero source term in the continuity equation. This scheme, unlike many other classical second-order accurate schemes such as the MacCormack method, is non-oscillatory even when sharp gradients in the flow variables are present. The main advantages of this scheme are its simplicity, ease of implementation and ease of extension to two-dimensional case. It is also very attractive for the present application because it is possible to account for the variable bottom topography in a convenient and accurate way. It is an explicit, two-step predictor-corrector scheme which results in second-order accuracy in both space and time. A detail description of the method as applied in the present study is outlined in Singh (1996). Only the predictor and corrector parts have been reproduced here.

Predictor Part

The finite-difference analog of the Eq. (3) is written here for a finite-difference grid where the subscript i refers to the grid point in the x-direction and the superscripts n and * refer to the values at the known time level and the predictor level, respectively.

The finite-difference form of Eq. (3) for the explicit determination of U^*_i is written as

$$U_i^* = U_i^n - \frac{\Delta t}{\Delta x}\left[F^n_{i+\frac{1}{2}} - F^n_{i-\frac{1}{2}}\right] + \Delta t\ S_i^n \tag{13}$$

where, $F^n_{i+1/2}$ represents the numerical flux through the cell face between nodes i and i+1. Δx is the grid spacing and Δt is the computational time step. The numerical flux is given as

$$F_{i+\frac{1}{2}} = \frac{1}{2}[F_R + F_L - \alpha(U_R - U_L)] \tag{14}$$

in where, α = a positive coefficient, $F_R = f(U_R)$ = the flux computed using the information from the right side of the cell face and $F_L = f(U_L)$ = the flux computed using the information from the left side of the cell face. U_R and U_L are obtained using the following equation.

$$U_L = U_i + \frac{1}{2}\delta U_i \quad and \quad U_R = U_{i+1} - \frac{1}{2}\delta U_{i+1} \tag{15}$$

There are several ways to determine δU_i and δU_{i+1} using different slope limiter procedures (Alcrudo et al., 1992). The "minmod" limiter has been followed in this study, according to which

$$\delta U_i = minmod\,(U_{i+1} - U_i,\ \ U_i - U_{i-1}) \tag{16}$$

$$\delta U_{i+1} = minmod\,(U_{i+1} - U_i,\ \ U_{i+2} - U_{i+1}) \tag{17}$$

where the minmod function is defined as

$$minmod = \begin{cases} a \ \ if \ \ |a| < |b| \ and \ ab > 0 \\ b \ \ if \ \ |b| < |a| \ and \ ab > 0 \\ 0 \ \ if \ \ ab \le 0 \end{cases} \tag{18}$$

The positive coefficient α is determined using the maximum value (for all the nodes) of the largest eigen value of the Jacobian of the system of equations. This is approximately given as

$$\alpha \ge \max\left|\left(V_i + \sqrt{g h_i}\right)\right| \qquad where\ , \qquad i = 1\ to \ \ N \tag{19}$$

in which, N is the total number of grid points, and V_i is the resultant velocity. All terms on the right hand side of the Eq. (13) are evaluated at the known time level n and therefore, U^*_i can be computed explicitly. The vector equation gives the predicted values of h and q at the unknown time level at any node i.

Corrector Part

The vector U at the unknown time level n+1 and at node i i.e. U_i^{n+1} is computed using the predicted values and the values at the time level n.

$$U_i^{n+1} = 0.5\left[U_i^n + U_i^* - \frac{\Delta t}{\Delta x}\left(F^*_{i+\frac{1}{2}} - F^*_{i-\frac{1}{2}}\right) + \Delta t\ S_i^*\right] \tag{20}$$

where

$$F^*_{i+\frac{1}{2}} = \frac{1}{2}\left[F_R^* + F_L^* - \alpha(U_R^* - U_L^*)\right] \tag{21}$$

In Eq. (20), only the source term in the momentum equation (the second component of the vector equation) is evaluated using the predicted values of h and q. However, the source term in the continuity equation i.e. (R-I) is evaluated using the values at the known time level instead of the predicted values. Strictly speaking, this procedure decouples the subsurface flow computations and the surface flow computations during the computational time step Δt. However, the response of the subsurface flow to the variation in surface flow depth is much slower than the response of surface flow to changes in the rate of infiltration (Akan and Yen, 1981). Therefore, the above decoupling does not affect the results significantly. In fact, numerical experimentation showed that determination of the infiltration rate, I during the corrector step by using the predicted flow depth did not alter the results. On the other hand, the decoupling procedure resulted in significant savings of the computational time since the subsurface flow is computed only once during a time step.

Initial and Boundary Conditions

The values of flow depth, discharge and infiltration rate are specified at all the nodes at time t = 0 as the initial conditions. The initial infiltration rate is equal to the rainfall rate. Although the initial flow depth and the discharge are equal to zero (overland flow on an initially dry surface), a very thin water film of depth h_{ini} and corresponding uniform flow discharge, q_{ini} are assumed to exist at time t = 0. This assumption is made to overcome the numerical singularity in a simple way. However, it should be noted that the outflow hydrograph may be sensitive to the h_{ini} value and therefore, it should be chosen as small as possible.

The explicit finite-difference scheme described earlier can be used to determine h and q at the unknown time level at the nodes i = 2 to N-1. The values of the variables at the upstream and the downstream ends of the domain are determined using the appropriate boundary conditions.

The discharge at the upstream end is equal to zero. However, the discharge at the upstream end is specified as q_{ini} to be consistent with the initial conditions. The flow depth at the upstream end can then be determined using the negative characteristic equation (Chaudhry 1993). In the present study, a simple extrapolation procedure has been adopted to determine the upstream flow depth from the depth at the interior nodes. Numerical experimentation in the initial stages of the model development showed that the extrapolation procedure gave satisfactory results. Similar extrapolation procedure is adopted to determine h and q at the last node N.

Stability Condition

The high-resolution Lax-Friedrichs scheme adopted in the present study is an explicit scheme. Therefore, the computational time step, Δt is chosen using the CFL condition.

$$C_n = \frac{\Delta t}{\Delta x}\left[\frac{q}{h} + \sqrt{g\,h}\right] \leq 1 \tag{22}$$

in which C_n = Courant number. Δt is chosen dynamically in the numerical model such that Eq. (22) is satisfied at all the nodes i = 1,2.....N.

5.2.2 Subsurface Flow

In order to determine the infiltration rate I in the continuity equation for the surface flow, two-dimensional Richards equation for subsurface flow is to be solved along with an appropriate boundary condition at the ground surface. In the present study, a recently developed strongly implicit finite-difference scheme (Hong et al., 1994) for the mixed based formulation of the Richards equation has been used to simulate the unsaturated subsurface flow conditions. This scheme ensures mass balance in its solution regardless of time step size and nodal spacings, and has no limitations when applied to field problems. It is also easy to incorporate different types of boundary conditions in this scheme.

Numerical solution of the Richards equation is described in the following section. The subsurface flow domain is divided into a number of rectangular blocks (Fig.6). The moisture content, θ and the pressure head, ψ are specified at the centre of the block (the node), while the velocities are specified at the interblock faces. The subscript i refers to the block number in the x-direction and the subscript j refers to the block number in the z-direction. The superscripts n and n+1 refer to the known and the unknown time levels, respectively. The finite-difference form of the Eq. (10) is

$$\frac{\theta_{i,j}^{n+1} - \theta_{i,j}^{n}}{\Delta t} + \frac{\bar{V}_{III} - \bar{V}_{I}}{\Delta x} + \frac{\bar{V}_{IV} - \bar{V}_{II}}{\Delta z} = 0 \tag{23}$$

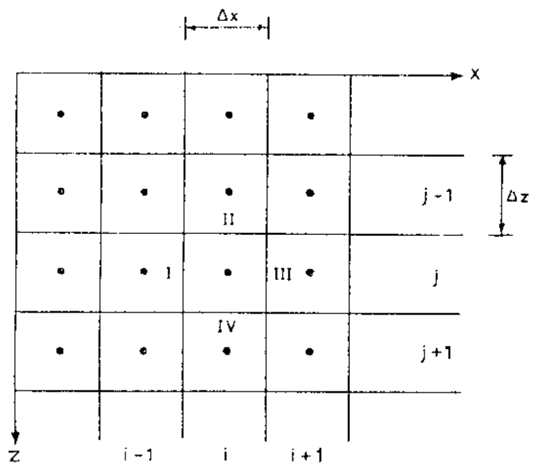

Fig.6 : Finite difference grid for subsurface flow

where, the bar is used to denote the time averaged value of the velocity. Δx and Δz are the nodal spacings in the x and z directions, respectively. The time-averaged velocities are determined by

$$\bar{V} = wV^{n+1} + (1-w)\,V^{n} \tag{24}$$

in which, w=time weighting factor, w = 1.0 for fully implicit scheme and it is equal to 0.5 for the Crank-Nicolson scheme. The velocity at any interblock face is determined using the pressure heads at the neighbouring cell centres. For example ;

$$V_{IV} = -\frac{K_{IV}\{(\psi_{i,j+1} - \psi_{i,j}) - \Delta z\}}{\Delta z} \tag{25}$$

and

$$V_{I} = -\frac{K_{I}\,(\psi_{i,j} - \psi_{i-1,j})}{\Delta x} \tag{26}$$

in which, K_{IV} and K_{I} are the unsaturated hydraulic conductivities evaluated at the interblock faces IV and I, respectively. Substitution of Eqs. (24), (25) and (26) in Eq. (23) yields

$$
\begin{aligned}
Res_{i,j}^{n+1} = {} & \frac{w\Delta t}{\Delta x^2}\left[-K_{III}^{n+1}\left(\psi_{i+1,j}^{n+1} - \psi_{i,j}^{n+1}\right) + K_{I}^{n+1}\left(\psi_{i,j}^{n+1} - \psi_{i-1,j}^{n+1}\right)\right] \\
& + \frac{w\Delta t}{\Delta z^2}\left[-K_{IV}^{n+1}\left(\psi_{i,j+1}^{n+1} - \psi_{i,j}^{n+1} - \Delta z\right) + K_{II}^{n+1}\left(\psi_{i,j}^{n+1} - \psi_{i,j-1}^{n+1} - \Delta z\right)\right] \\
& + \theta_{i,j}^{n+1} - \left[\theta_{i,j}^{n} - (1-w)\frac{\Delta t}{\Delta x}\left(V_{III}^{n} - V_{I}^{n}\right) - (1-w)\frac{\Delta t}{\Delta z}\left(V_{IV}^{n} - V_{II}^{n}\right)\right] = 0
\end{aligned} \tag{27}
$$

The unsaturated hydraulic conductivity at an interblock face is estimated using the pressure heads at the neighbouring cell centres. Haverkamp and Vauclin (1979) state that the geometric mean is the best choice for estimating the interblock conductivities. However, Hong et al. (1994) reported that the iterative solution of Eq. (27) fails to converge if the above procedure is adopted for estimating the K. This is especially true for infiltration into initially very dry soils. The geometric mean is strongly weighted towards the lower value and therefore, water can not drain easily if the soil is initially dry. This results in a non-physical build up of pressure. In this study, the interblock hydraulic conductivity is estimated by the weighted arithmetic mean. For example,

$$
K_{IV} = \gamma K(\psi_{i,j}) + (1-\gamma) K(\psi_{i,j+1}) \tag{28}
$$

in which, γ is the weight coefficient. Hong et al. (1994) suggested a value of 0.5 for γ.

Equation (27) is written for all the blocks in the flow domain and this results in a set of simultaneous algebraic equations in the unknowns $\psi(i,j)^{n+1}$. These simultaneous equations are highly non-linear since θ^{n+1} and K^{n+1} are non-linear functions of ψ^{n+1}. In the present study, they have been solved by using the Newton-Raphson technique.

$$
Res_{i,j}^{n+1,r} + \frac{\partial Res_{i,j}^{n+1,r}}{\partial \psi_m}\delta\psi_m = 0 \tag{29}
$$

in which, r is the previous iteration level and $\delta\psi = (\psi^{n+1,r+1} - \psi^{n+1,r})$. Subscript m indicates the summation of the second term over all the blocks. Substituting of Eq. (27) in Eq. (29) yields a linear equation in $\delta\psi$ having the following form.

$$
\begin{aligned}
& W_{i,j}^{n+1,r}\delta\psi_{i-1,j} + E_{i,j}^{n+1,r}\delta\psi_{i+1,j} + T_{i,j}^{n+1,r}\delta\psi_{i,j-1} \\
& + B_{i,j}^{n+1,r}\delta\psi_{i,j+1} + P_{i,j}^{n+1,r}\delta\psi_{i,j} + Res_{i,j}^{n+1,r} = 0
\end{aligned} \tag{30}
$$

in which, W, E, T, B, and P are the elements of the Jacobian of the system of equations, Eq. (27). Equations for evaluating these are presented in Appendix-I. Equation (30), when written for all the blocks in the domain, constitutes a matrix equation

$$A^{n+1,r}\,\delta\psi = -Res^{\,n+1,r} \tag{31}$$

in which, the coefficient matrix A is banded. Equation (31) has been solved in the present study using an efficient NAG subroutine (D03EBF) especially designed for such systems.

For convergence in iteration of Eq. (29), it is required that

$$\left| Res_{i,j}^{n+1,r} \right| < \epsilon \tag{32}$$

in which, ϵ = water content convergence tolerance. Equation (32) practically represents the principle of mass conservation because usually a very small value of ϵ is imposed.

Boundary Conditions

(i) *Flux-Type Boundary Conditions*: In the adopted scheme, the grid is arranged in such a manner that the boundaries of the flow domain coincide with an interblock. Therefore, flux or velocity-type boundary condition can be incorporated in a natural way in Eq. (23).

(ii) *Pressure Head-Type Boundary condition*: Referring to Fig.7, let ψ_b be the imposed pressure head at the ground surface of the flow domain. This pressure head ψ_b is used along with the values of ψ_1 and ψ_2 to determine the flux at the ground surface as given below.

$$V_z\big|_{z=0} = -K(\psi_b)\left[\frac{\left(-\dfrac{8\,\psi_b}{3} + 3\,\psi_1 - \dfrac{\psi_2}{3}\right)}{\Delta z} - 1\right] \tag{33}$$

Second-order forward finite-difference analog is used to determine the above Eq. (33). Equation 27 and equations for the coefficients W, E, T, B and P are appropriately charged to include the boundary conditions before the matrix $A^{n+1,r}$ in Eq.(31) is formed.

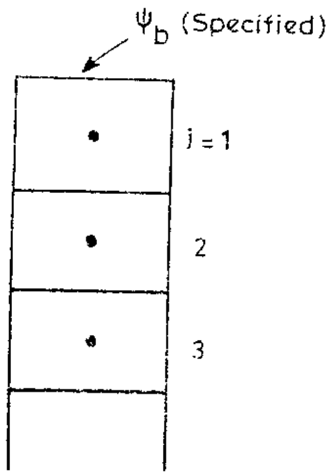

Fig.7 : Sketch for boundary conditions for subsurface flow.

5.2.3 Surface and Subsurface Flow Interaction

Surface and subsurface flow components are interrelated by a common pressure head and the infiltration at the ground surface. The top boundary condition for the subsurface flow is determined by the surface flow depth. In turn, the infiltration term in the surface flow equation is controlled by the subsurface flow conditions. The following procedure is adopted for simulating the interaction between the surface and the subsurface flow components.

1. Subsurface flow solution at time level n is used to determine the infiltration rate at the ground surface.

2. Surface flow equations are now solved using the infiltration rate from step 1 to determine q and h at the unknown time level n+1.

3. The surface flow depth at the time level n+1 is used as the top boundary condition and the subsurface flow equations are solved. This gives the θ and ψ distribution in the subsurface at time level n+1.

4. Steps 1-3 are repeated up to the required time level.

As mentioned earlier, it is a decoupled approach which reduces the CPU time by half without significantly affecting the accuracy of the results.

Boundary Conditions

For subsurface flow resulting from rainfall infiltration, the top boundary condition changes with time. During the initial stages of the rainfall, there is no ponding and the infiltration rate is equal to the rainfall rate. The top boundary condition for such a situation is the specification of the flux equal to the rainfall rate. As time progresses, the upper layers of the subsoil get saturated and then infiltration rate starts decreasing. Before starting the solution of the Richards equation for any time step, the flux at the top boundary is estimated by taking $\psi_b = 0$. If this flux is greater than the rainfall rate, then the flux type of boundary condition is applied and the flux will be equal to the rainfall rate. Otherwise, a head type of boundary condition ($\psi_b(x) = h(x)$ = water flow depth at that point) is applied. The time to ponding comes out as a part of the solution.

A no flux boundary condition is imposed at the right and left boundaries of the domain. The ψ values at the bottom boundary are obtained using a simple extrapolation from the interior points. This approximation does not introduce errors because the bottom boundary is taken fairly deep and the moisture front does not reach there for the computational times considered.

5.3 Results and Discussion

The present model has been used to simulate the overland flow over a porous surface of semi-arid region. For this purpose, a hypothetical catchment has been considered. In this catchment, length = 100 m; width = 1 m; longitudinal slope of the catchment, S_0=0.0018; surface roughness, S_{rf}= 0.0013; saturated hydraulic conductivity, K_s=1.368 cm/hr; saturated moisture content, θ_s=0.18; and residual moisture content, θ_r=0.005. The intensity of rainfall was $3.8X10^{-6}$ m/s and duration of rainfall was 600 sec. In the numerical simulation using present model, the distance step Δx = 2.0 m, the Courant number, C_n= 0.6 and the specified initial depth, h_{ini}=0.00001 m. were taken.

Fig.8 shows the effect of treatment of catchment on runoff for different percentages of length treated. In this figure, the ratio of the volume of water collected at downstream side to the volume of total rainfall occurred on the catchment has been shown on y-axis and the factor (i.e. ratio of hydraulic conductivity and intensity of rainfall) has been shown on x-axis. There are four curves in the figure for different percentage of length of hypothetical catchment treated (i.e. 25 % length means 25 % length of catchment from downstream side has been treated). It can be seen from figure that as the factor, K/R, (K=hydraulic conductivity of the trated surface soil) increases, the ratio, V_c/V_r (V_c=total volume of water collected at downstream end, V_r = total volume of rainfall), decreases i.e. by decreasing hydraulic conductivity of the surface soil, the volume of water collected at the downstream side increases. Now keeping in view the requirement of volume of water collection, the hydraulic conductivity can be modified by treating the surface soil of the catchment.

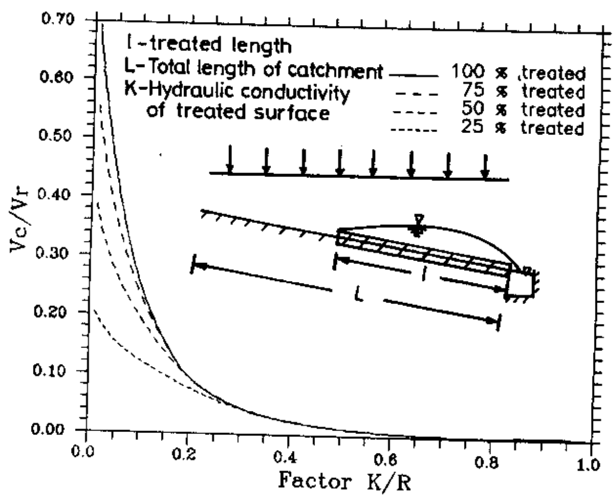

Fig. 8 : Effect of treatment of catchment on runoff for different percentages of length treated.

Input Data Required for the Application of the Model

i Length of the catchment;
ii. Slope of the catchment;
iii. Surface roughness of the catchment;
iv. Saturated hydraulic conductivity;
v. Saturated moisture content;
vi. Residual moisture content;
vii. Soil characteristic relationship;
viii. Intensity of rainfall;
ix. Duration of rainfall.

6.0 ECONOMICAL DESIGN OF A STORAGE TANK

Let the cost of excavation of unit volume of soil at depth z be $c_0 + c_1 z$, where c_0 and c_1 are the excavation cost parameters. Hence the total cost of excavation upto depth D is given by:

$$C_e = A\,(c_0 D + c_1 D^2/2)$$

where, A = Surface area of the tank (pond)

D = depth of the pond.

A and D are unknown.

Since the tank has to store the drinking water requirement and the water lost by evaporation (AE_p), therefore

$$AD = V + E_p A$$

or,

$$A = \frac{V}{D - E_p}$$

where,

E_p = Yearly potential evaporation loss from unit surface area

V = Volume of drinking water to be supplied

Let,

C_w = Unit price of water

n = Life of the treatment of the catchment

S = Surface area of the treated catchment (unknown)

C_s = Cost of surface treatment per unit area

The catchment area to be treated to supply the drinking water volume V, allowing the evaporation AE_p from the tank surface, is given by :

$$S = \frac{V + AE_p}{f\,R}$$

where, f is runoff efficiency of the treated catchment and R is the average annual rainfall.

Therefore cost of surface treatment is equal to SC_s.

Cost of concrete structure for square shaped and closed tank is given by :

$$C_c = (2A + 4\sqrt{A}D)\,c_c$$

Cost of concrete structure for square shaped and open tank is given by :

$$C_c = (A + 4\sqrt{AD})\, c_c$$

Let, i be the interest rate and n be the life of the treatment. Let, the cost factor be defined as

$$C_a = \frac{i(1+i)^n}{(1+i)^n - 1}$$

Hence, annual cost of water is given by

$$C_w = (C_s + C_e + C_c)\, C_a$$

Therefore unit cost of water is equal to C_w/V.

This cost is function of depth D of the tank. Considering the variation of cost with respect to depth, the depth for which the cost of water is minimum can be obtained. A typical variation of cost with D is given in Fig.9.

The dimension of the storage tank to supply water for 1000 people at the rate of 125 litres per capita per day has been worked out. The following data have been used for the computation :

Catchment treatment : Lime concrete (100%)
Life of the treatment : 5 years
Average runoff efficiency : 50 percent
Cost of treatment : Rs. 12 per m^2 of catchment
Cost of excavation : C0 = Rs. 20 per cubic meter and C1 = Rs. 5 per cubic meter
Cost of concreting the walls = Rs. 300 per square meter.

The results have been obtained using the following two sets of rainfall and evapotranspiration data

	Set-I	Set-II
Average annual rainfall :	250 mm	300 mm
Yearly potential evaporation from open tank :	3000 mm	1800 mm
Yearly evaporation loss from closed tank :	300 mm	180 mm

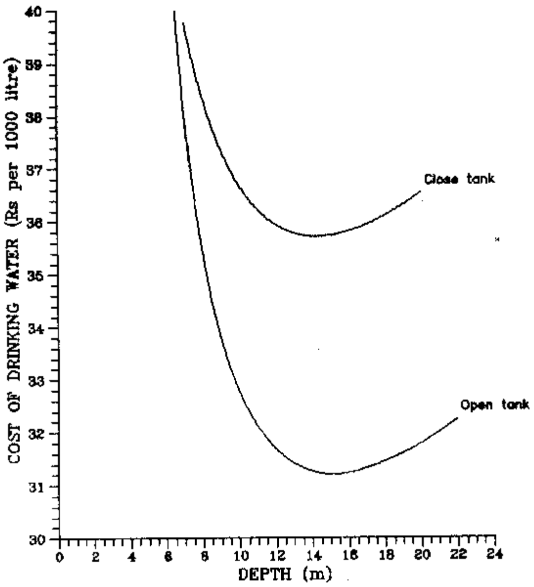

Fig.9 : Variation of cost of harvested water with depth of tank.(Rainfall = 250 mm; potential evaporation = 3000 mm)

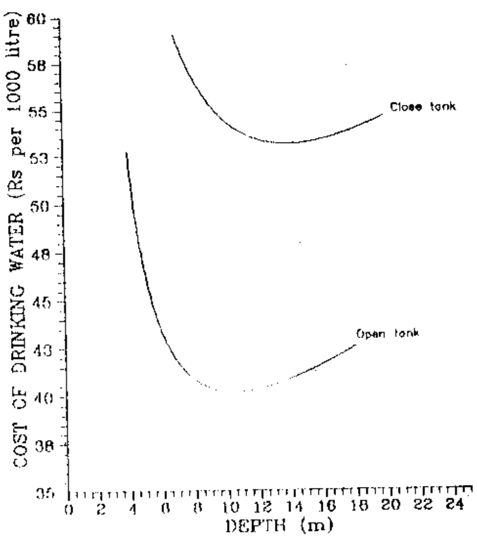

Fig.10 : Variation of cost of harvested water with depth of tank.(Rainfall = 300 mm; potential evaporation = 1800 mm)

The variation of cost of unit volume of water with depth of the storage tank follows a parabolic equation (as seen from the Fig.9 and Fig.10). For an area which receives an annual rainfall of 250 mm and experiences 3000 mm annual potential evaporation, the minimum cost of harvested water occurs for an open tank of size at 62x62x15.1 m. For a closed tank, the size of the tank is 58x58x16.7 m. The minimum cost of water at these depths for open and closed tanks is Rs.50.61 and Rs. 57.86 per 1000 litres respectively.

For another area which receives normal annual rainfall of 300 mm and experiences 1800 mm annual potential evaporation, the minimum cost of harvested water for open and closed tanks is Rs.42.20 and Rs. 53.11 per 1000 litres respectively. The optimal size of the open tank is 74x74x10 m and of the closed tank is 58x58x15.4 m.

The cost of the harvested water will depend upon the annual rainfall, the life and efficiency of treatment, evaporation losses and cost of concreting the storage structure. FORTRAN program for calculation of cost of water with test data is given in Appendix-III.

7.0 CONCLUSIONS

From the study of overland flow on porous surface, it is found that the catchment should be treated such that the hydraulic conductivity of the treated surface soil should be less than one third of the rainfall intensity. If the in-situ soil has conductivity less than one third of rainfall intensity, overland flow occurs which can be harvested.

Drinking water can be supplied at a rate of about Rs. 42 per 1000 litres in an area receiving an annual rainfall of 300 mm and having potential evaporation loss of 1800 mm. For an area where the annual rainfall is 250 mm and potential evaporation loss is 3000 mm, water can be harvested at a rate Rs. 50 per cubic meter (i.e. paisa 5 per litre) to a population of 1000 with water requirement of 125 litres per capita per day. Detailed hydrological investigations are necessary before implementation of the water harvesting project.

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ELEMENTS OF THE BANDED MATRIX

$$W_{i,j}^{n+1,r} = \frac{w\Delta t}{\Delta x^2}\left[-K_I^{n+1,r} + \frac{D_{i-1,j}^{n+1,r}}{2}\left(\psi_{i,j}^{n+1,r} - \psi_{i-1,j}^{n+1,r}\right)\right]$$

$$E_{i,j}^{n+1,r} = \frac{w\Delta t}{\Delta x^2}\left[-K_{III}^{n+1,r} - \frac{D_{i+1,j}^{n+1,r}}{2}\left(\psi_{i+1,j}^{n+1,r} - \psi_{i,j}^{n+1,r}\right)\right]$$

$$T_{i,j}^{n+1,r} = \frac{w\Delta t}{\Delta z^2}\left[-K_{II}^{n+1,r} + \frac{D_{i,j}^{n+1,r}}{2}\left(\psi_{i,j}^{n+1,r} - \psi_{i,j-1}^{n+1,r} - \Delta z\right)\right]$$

$$B_{i,j}^{n+1,r} = \frac{w\Delta t}{\Delta z^2}\left[-K_{IV}^{n+1,r} - \frac{D_{i,j}^{n+1,r}}{2}\left(\psi_{i,j+1}^{n+1,r} - \psi_{i,j}^{n+1,r} - \Delta z\right)\right]$$

$$\begin{aligned} P_{i,j}^{n+1,r} = C_{i,j}^{n+1,r} &+ \frac{w\Delta t}{\Delta x^2}\left[K_{III}^{n+1,r} - \frac{D_{i,j}^{n+1,r}}{2}\left(\psi_{i+1,j}^{n+1,r} - \psi_{i,j}^{n+1,r}\right)\right. \\ &\qquad \left. + K_I^{n+1,r} + \frac{D_{i,j}^{n+1,r}}{2}\left(\psi_{i,j}^{n+1,r} - \psi_{i-1,j}^{n+1,r}\right)\right] \\ &+ \frac{w\Delta t}{\Delta z^2}\left[-K_{IV}^{n+1,r} - \frac{D_{i,j}^{n+1,r}}{2}\left(\psi_{i,j+1}^{n+1,r} - \psi_{i,j}^{n+1,r} - \Delta z\right)\right. \\ &\qquad \left. + K_{II}^{n+1,r} + \frac{D_{i,j}^{n+1,r}}{2}\left(\psi_{i,j}^{n+1,r} - \psi_{i,j-1}^{n+1,r} - \Delta z\right)\right] \end{aligned}$$

$$\begin{aligned} Res_{i,j}^{n+1,r} = \theta_{i,j}^{n+1,r} - \theta_{i,j}^{n} &+ \frac{w\Delta t}{\Delta x}\left(V_{III}^{n+1,r} - V_I^{n+1,r}\right) + \frac{(1-w)\Delta t}{\Delta x}\left(V_{III}^{n,r} - V_I^{n,r}\right) \\ &+ \frac{w\Delta t}{\Delta z}\left(V_{IV}^{n+1,r} - V_{II}^{n+1,r}\right) + \frac{(1-w)\Delta t}{\Delta z}\left(V_{IV}^{n,r} - V_{II}^{n,r}\right) \end{aligned}$$

Appendix -II

Notations

C_n : Courant number;
c : wave speed;
ds_r : lengths of the four walls which contour the cell (i,j);
f_o : ostiakov empirical coefficient (m/s);
f_d : frictional resistance coefficient;
g : acceleration due to gravity (m/s^2);
h : low depth (m);
h_{ini} : depth of flow for the initial condition
I : volumetric rate of infiltration per unit area (m/s);
iflag, jflag : represents wave front location in x and y direction respectively
$K(\psi)$: unsaturated hydraulic conductivity (m/s);
k : length measure of surface roughness;
n : Manning roughness coefficient
n_x & n_y : unit vectors in x and y directions respectively.
N : total number of grid points in x direction;
Q : total discharge (cumec)
q : discharge per unit width (m^2/s);
q_{ini} : discharge specified as the initial condition;
R : volumetric rate of rainfall per unit surface area (m/s);
Re : Reynolds number = q/ν;
S_o : bottom slope in x direction;
S_f : friction slope in x direction;
t : time (sec);
U, F and S : vector;
V : resultant velocity (m/s);
V_x : Darcy flow velocity in the x direction;
V_z : Darcy flow velocity in the z direction;
x & z : distances along the two coordinate directions;
α : a positive coefficient;
Δt : time stepping;
ψ : pressure head (m);
ψ_b : imposed pressure head at the ground surface;
ψ_1 : pressure head at first grid under the ground;
ψ_2 : pressure head at second grid under the ground;
θ : volumetric moisture content;
ν : Kinematic viscosity of liquid

Superscripts

n,*,** : refers to the values of the variables at known time level, the predicted and the corrected values;

Subscripts

i : refer to the grid point in x-direction;
j : refer to the grid point in y-direction;
r : denotes the walls

```
C     FORTRAN PROGRAM FOR ECONOMIC ANALYSIS OF RAINWATER
C     HARVESTING

      OPEN(1, STATUS= 'OLD',FILE= 'HARVEST.DAT')
      OPEN( 2, STATUS='UNKNOWN',FILE= 'HARVEST.OUT')
C
C
C     INPUT DATA
C
C     EP=YEARLY POTENTIAL EVAPORATION LOSS
C     R=ANNUAL RAINFALL
C       V=VOLUME OF WATER TO BE SUPPLIED FOR DRINKING
C
C     S=SURFACE AREA TO BE TREATED
C       CS=COST OF SURFACE TREATMENT PER UNIT AREA
C     F=RUNOFF EFFICIENCY
C       D=DEPTH OF THE TANK
C     DELD=INCREMENTAL DEPTH
C     A=AREA OF THE TANK
C       EN=LIFE OF CATCHMENT
C     EI=ANNUAL INTEREST RATE
C     DRINKR=DRINKING WATER REQUIREMENT PER CAPITA PER
DAY
C     PEOPLE=POPULATION
      READ(1,*)EP,R,F
      READ(1,*)EN
      READ(1,*)PEOPLE,DRINKR
      READ(1,*)CS,C0,C1,CC,EI
      CA = EI*(1.+EI)**EN/((1.+EI)**EN - 1.)
      READ(1,*)DELD
      V=PEOPLE*DRINKR*365./1000.
C     AREA OF THE TANK IS NOT KNOWN
C     DEPTH OF THE TANK IS NOT KNOWN
C     ASSUME DEPTH OF THE TANK
C     DEPTH OF THE TANK SHOULD BE MORE THAN EP
      WRITE(2,2)
2     FORMAT(2X,'DEPTH',2X,'TANK_AREA',2X,'CATCH_AREA',
     1  2X,'COST_OPEN',2X,'DEPTH',1X,'TANK_AREA',1X,'CATCH_AREA',
     2  1X,'COST_CLOSE, '/)
      D1=EP+DELD
      D2=EP/10. + DELD
1     CONTINUE
c     COST OF PROJECT VS. DEPTH FOR OPEN POND
      A1=V/(D1-EP)
      S1=(V+A1*EP)/(F*R)
      COST1 = (S1*CS+A1*D1*(C0+0.5*C1*D1))*CA/V
c     COST OF PROJECT VS. DEPTH FOR CLOSED TANK
```

```
      A2=V/(D2-EP/10.)
      S2=(V+A2*EP/10.)/(F*R)
      COST2 = (S2*CS+A2*D2*(C0+0.5*C1*D2)+(2.*A2
     1 +4.*SQRT(A2)*D2)*CC)*CA/V
      WRITE(2,3) D1,A1,S1,COST1,D2,A2,S2,COST2
3     FORMAT(2X,F5.2,2X,F9.1,2X,F9.0,2X,F7.1,3X,F5.2,2X,
     1 F9.2,2X,F9.0,2X,F8.2)
      D1=D1+DELD
      D2=D2+DELD
      IF(D1.LT.20.OR.D2.LT.20) GO TO 1
      STOP
      END
```

DATA FILE - I

```
3.0 0.25 0.5
5.
1000. 125.
12. 20. 5. 300. .1
0.1
```

DATA FILE - II

```
1.8 0.3 0.5
5.
1000. 125.
12. 20. 5. 300. .1
0.1
```

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