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A coastal groundwater management model with Indian case study

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Abstract

The complexity of the hydrogeological setup in coastal areas calls for the adoption of scientific groundwater management techniques. Excessive withdrawal of groundwater in coastal zones will lead to depression of the water table, with associated hazards such as putting the well out of use, rendering abstraction uneconomic with increased lift. A sustained regional groundwater drawdown below sea level runs the risk of saline water intrusion, even for confined coastal aquifers. Uncontrolled groundwater development may lead to reversal of the freshwater gradient, thereby resulting in saline water ingress into coastal aquifers. There are, however, several established methodologies to control and minimise the problems associated with groundwater extraction followed by saline water intrusion. This study developed a convenient and easily implementable analytical model for coastal groundwater management aimed at the control of saltwater intrusion. The technique includes withdrawal of coastal freshwater by means of qanat-well structures associated with artificial recharge through rainwater harvesting aided by percolation ponds and recharge wells. The proposed methodology is suitable specifically for not highly urbanised coastal areas with significant annual precipitation, good hydraulic conductivity of the aquifer and a low depth of fresh groundwater. As a case study, the model is applied to a coastal zone of the Purba Medinipur district of West Bengal, India. Adequate quantifications of the efficiency of the methodology are incorporated and relevant conclusions are drawn.

Keywords

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The complexity of the hydrogeological setup in coastal areas calls for the adoption of scientific groundwater management techniques. Excessive withdrawal of groundwater in coastal zones will lead to depression of the water table, with associated hazards such as putting the well out of use, rendering abstraction uneconomic with increased lift. A sustained regional groundwater drawdown below sea level runs the risk of saline water intrusion, even for confined coastal aquifers. Uncontrolled groundwater development may lead to reversal of the freshwater gradient, thereby resulting in saline water ingress into coastal aquifers. There are, however, several established methodologies to control and minimise the problems associated with groundwater extraction followed by saline water intrusion. This study developed a convenient and easily implementable analytical model for coastal groundwater management aimed at the control of saltwater intrusion. The technique includes withdrawal of coastal freshwater by means of qanat-well structures associated with artificial recharge through rainwater harvesting aided by percolation ponds and recharge wells. The proposed methodology is suitable specifically for not highly urbanised coastal areas with significant annual precipitation, good hydraulic conductivity of the aquifer and a low depth of fresh groundwater. As a case study, the model is applied to a coastal zone of the Purba Medinipur district of West Bengal, India. Adequate quantifications of the efficiency of the methodology are incorporated and relevant conclusions are drawn.

Notation

A_l	total area of vacant land in the community
A_{pond}	total pond area
A_{road}	total road area in community
A_{roof}	total roof cover area for all buildings in community
A_t	total area of community
d	depth of bottom surface of qanat legs
F	factor of safety
F_D	design factor of safety
H	aquifer thickness
H_f	height of fresh water above seawater interface
H_p	depth of pond to be excavated
k_h	hydraulic conductivity of the aquifer along horizontal direction
k_v	hydraulic conductivity of the aquifer along vertical direction
L	length of horizontal well
L_q	length of qanat leg
N_w	number of recharge chambers
P	population of the community
p	effective rainfall
Q	steady-state discharge
Q_q	steady-state discharge from the qanat
$Q_{q(max)}$	maximum discharge from qanat-well structure
R	design annual rainfall (in mm)
R_m	design monsoon rainfall (in mm)
r	infiltration recharge
r_q	inner radius of qanat legs
r_w	well radius
s	drawdown above the well centre
t	hourly pumping operation per day
V_0	annual excess volume of water

V_e	volume of water available annually from other sources
V_w	volume of recharge chambers in the community
W	average water consumption
Z_∞	value of z at infinite time
z	height of unconing
α	fraction of roof-collected rainwater
β	$(k_h/k_v)^{0.5}$
δ	off-centred eccentricity of the well centre in the vertical aquifer plane
η	recharge coefficient
η_1	runoff coefficient relevant to the area
ξ	non-dimensional function
ρ	radius of influence of the equivalent vertical well
ρ_f	freshwater density
ρ_s	saline water density

1. Introduction

Groundwater is the second largest reserve of freshwater on earth. About two billion people – approximately one-third of the world's population – depend on groundwater supplies, withdrawing about 20% of global water (600–700 km^3) annually, much of it from shallow aquifers (Patra, 2006). Localities near coastlines with a large population can experience saline water intrusion due to over-exploitation of groundwater, causing significant threat to freshwater resources. Sustainability strategies adopted to retard or halt the rate of saline water intrusion are necessary to protect these resources from further damage.

The objective of any control method of saltwater intrusion should be to prevent further encroachment and, if possible, reduce the area already intruded. Of the available methods, popular techniques are (Karanth, 1987; Raghunath, 1990; Todd, 1995)

- the creation of hydraulic barriers
- canal irrigation
- desalination and reverse osmosis
- rainwater harvesting
- artificial recharge methods.

Of the significant contributions on available models for coastal groundwater management, Rastogi *et al.* (2004) developed a diffused interface model to prevent ingress of seawater in multi-layer coastal aquifers to study a two-dimensional (2D) steady-state seawater intrusion problem involving hydrodynamic dispersion in a synthetic multi-layered confined coastal aquifer. Rao and Sreenivasulu (2004) studied the planning of groundwater development in coastal aquifers where seawater intrusion was controlled through a series of barrier extraction wells as a non-linear, non-convex combinatorial model and was solved using a coupled simulation–optimisation approach. Nichols *et al.* (2007) developed an improved technique for partitioning artificial recharge from simulated total recharge for inclusion in a groundwater model, thereby contributing to improvement in the calibration of a site-wide groundwater model. Mahesha (1996) carried out an analytical study on the transient effect of a battery of injection wells on seawater intrusion into coastal confined aquifers, analysed using a quasi-three-dimensional (3D) areal finite-element model derived based on the sharp interface approach. Mahesha (2009a) developed a conceptual model for the safe withdrawal of freshwater from coastal aquifers using a Galerkin finite-element model considering the sharp interface approach and studied the interface profile considering the effect of barrier and no-barrier conditions. Mahesha (2009b) carried out parametric studies to analyse the effect of freshwater recharge on seawater intrusion through a finite width strip parallel to the coast using a vertically integrated one-dimensional finite-element model. These studies included the effect of location, width, intensity and the period of recharge on seawater–freshwater interface motion. Abd-Elhamid and Javidi (2011) developed a cost-effective abstraction, desalination and recharge methodology to control seawater intrusion in coastal aquifers using a coupled transient density-dependent finite-element model to simulate fluid flow and solute transport.

Among the available mathematical models on freshwater abstraction using horizontal wells, Hantush and Papadapulos (1962) developed simplified classical drawdown formulas, which are probably the most widely used for predicting the yield of radial collector wells. Huisman (1972) used flow net analyses to develop reasonable corrections on the drawdown of vertical wells and horizontal galleries resulting from partial penetration. Zhan and Cao (2000) developed closed-form analytical solutions and semi-analytical solutions for capture times to horizontal under no-flow and constant-head boundaries. Zhan *et al.* (2001) developed transient 3D solutions on the intricacies of horizontal-well pumping tests in anisotropic confined aquifers. Birch *et al.* (2007) developed a steady-state 2D model along a horizontal well for the Dupuit–Forchheimer condition, employing the Cauchy boundary condition.

There are also significant contributions on rainwater harvesting. Rastogi and Pandey (1998) carried out finite-element modelling of artificial recharge basins of different shapes and the effect on the underlying aquifer system. Chen *et al.* (2008) conducted an analytical study to estimate the characteristics of groundwater recharge from precipitation and groundwater loss due to evapotranspiration by lysimeter measurement and using a multi-layer soil moisture model. Helmreich and Horn (2009) discussed the recent advancements and advantages of rainwater harvesting. Dourte *et al.* (2012) developed rainfall intensity–duration–frequency relationships for Andhra Pradesh, India, from the viewpoint of changing rainfall patterns and implications for runoff and groundwater recharge.

With the objective that field engineers will be able to capture and implement an analytical technique that is appropriate and easily implementable is proposed as a new approach for groundwater withdrawal and control of saline water intrusion.

It is evident that any withdrawal of groundwater from a coastal aquifer results in advancement of the saltwater–freshwater interface from the shoreline towards the point of withdrawal (Todd, 1995) unless the withdrawal is compensated by an equivalent artificial recharge. Rainwater harvesting is one of the most popular recharge techniques followed worldwide (United Nations, 2006). In the proposed methodology for reduction of saline water intrusion into the coastal aquifer and subsequent safe withdrawal of groundwater, the adoption of a qanat-well structure associated with artificial recharge by rainwater harvesting through recharge ponds and recharge wells is studied here. The salient features of the methodology are described by considering a design example adopted by the authors in the Contai Polytechnic campus in the district of Purba Medinipur in the state of West Bengal, India.

2. The proposed technique

The model developed for coastal groundwater management is convenient and easily implementable at a specific site, provided the values of in situ parameters are properly assessed and used. To meet the water requirements in the specific community under consideration, use of a qanat-well structure to extract fresh groundwater is proposed in the model since shallow wells are unsuitable for low yields and the adoption of deep tube wells initiates the problem of upconing (Bhattacharya *et al.*, 2004). While withdrawal of fresh groundwater in coastal regions is well established, as evidenced from the available literature, there is no specific methodology for the design of such qanat-well structure. In this paper, an attempt has been made to establish a design methodology by taking into account the already available discharge correlations for a horizontal well, the method of superimposition and the upconing phenomena, which is quite obvious for coastal aquifers. To mitigate the advancement of the freshwater–saltwater interface towards the point of withdrawal induced by such freshwater extraction by qanat-well structures, the rainwater harvesting method associated with recharge ponds and recharge wells is proposed. To design these recharge structures, correlations available from the literature are used. The proposed model is thus applicable for groundwater management in a specific locality in a coastal region and its successful implementation depends upon adequately chosen field parameters.

The proposed analytical model is convenient and easily implementable for coastal groundwater management aimed at the control of saltwater intrusion. As stated earlier, the model consists of the withdrawal of freshwater by means of qanat-well structures coupled with rainwater harvesting. However, the methodology is only suitable for coastal regions that have significant annual precipitation, good hydraulic conductivity of the aquifer, a low depth of fresh groundwater and are not overly urbanised.

2.1 Adoption of a qanat-well structure

A qanat-well structure is a group of horizontal wells (termed ‘legs’) connected together at their ends with a vertical riser (Figure 1), constructed in arid regions for groundwater withdrawal (Maity, 2011). Due to saltwater intrusion, deep tube wells are not recommended because of upconing problems. However, the adoption of a shallow well is also inappropriate because of

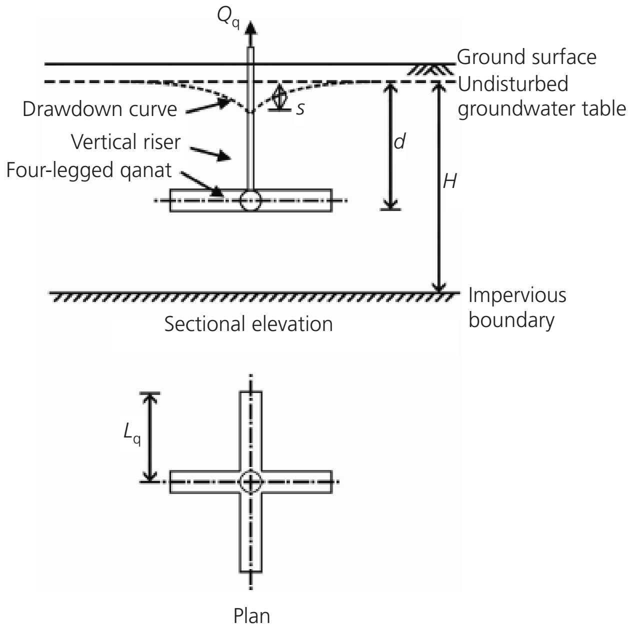

Figure 1. A typical four-legged qanat

significantly lower discharge. It is well established (Ball and Herbert, 1992; Raghu Babu *et al.*, 2004; Sawyer and Lieuallen-Dulam, 1998) that adoption of qanats in such conditions not only yields higher discharge but also significantly reduces the upconing problem. Several advantages of qanats over conventional vertical wells, specifically for shallow highly permeable coastal aquifers, have been highlighted in the past (Ball and Herbert, 1992; Haitjema *et al.*, 2010; Sawyer and Lieuallen-Dulam, 1998). Horizontal wells are more efficient than conventional vertical wells for environmental remediation of groundwater for a number of reasons, such as the following.

- Greater reservoir contact with the well screen increases the productivity of the well.
- The geometry of the groundwater zone is conducive to greater access with a horizontal well than a series of vertical wells.
- Access to groundwater zones with vertical wells is often hindered by obstacles such as buildings, paved surfaces or other topographical obstructions.

In many coastal regions in India, horizontal qanats are widely used for freshwater abstraction (Raghu Babu *et al.*, 2004). Various mathematical models for flow near horizontal wells have been presented in the literature (Birch *et al.*, 2007; Hantush and Papadapulos, 1962; Huisman, 1972; Zhan and Cao, 2000; Zhan *et al.*, 2001). While a numerical simulation method like the finite-difference or finite-element scheme might be very illuminating from a research point of view, it is unlikely that field engineers will be able to capture the intricacies of such an approach. Keeping this in mind, an analytical model has been developed by the authors to predict the maximum steady-state yield of a qanat by combining the analysis of Beljin and Losonsky (1992) and Joshi (1986) for horizontal wells and the effect of upconing by the classical analysis of Dagan and Bear (1968), which is obvious in a coastal environment.

Beljin and Losonsky (1992) provided a generalised solution, based on the work of Joshi (1986), for estimating steady-state discharge for withdrawal of groundwater from a vertical aquifer by means of a horizontal water well. The solution provided was given by

$$Q = \frac{2\pi k_\mathrm{h} H s}{\ln\left[\left(\dfrac{[1+(1+64\rho^4/L^4)^{1/2}]^{1/2} + [-1+(1+64\rho^4/L^4)^{1/2}]^{1/2}}{2^{1/2}}\right)\left(\dfrac{(\beta H^2/2)+2\beta\delta^2}{H r_\mathrm{w}}\right)^{\beta H/L}\right]} \quad (1)$$

where Q is the steady-state discharge, s is drawdown above the well centre, L is the length of the horizontal well, r_w is the well radius, k_h is the hydraulic conductivity of the aquifer along the horizontal direction, H is the aquifer thickness, $\beta = (k_\mathrm{h}/k_\mathrm{v})^{0.5}$, k_v is the hydraulic conductivity of the aquifer along the vertical direction, δ is the off-centred eccentricity of the well centre in the vertical aquifer plane and ρ is the radius of influence of the equivalent vertical well in the same aquifer for the same drawdown, which can be reasonably estimated using the available correlations (e.g. Sichardt's formulae).

Equation 1 has been modified, applying the method of superimposition, to reasonably estimate the steady-state discharge by means of a four-legged qanat as

$$Q_\mathrm{q} = \frac{2\pi k_\mathrm{h} H s}{\ln\left[\left(\dfrac{[1+(1+4\rho^4/L_\mathrm{q}^4)^{1/2}]^{1/2} + [-1+(1+4\rho^4/L_\mathrm{q}^4)^{1/2}]^{1/2}}{2^{1/2}}\right)\left(\dfrac{(\beta H^2/2)+2\beta\delta^2}{H r_\mathrm{q}}\right)^{2\beta H/L_\mathrm{q}}\right]} \quad (2)$$

where Q_q is steady-state discharge from the qanat, L_q is length of a qanat leg (see Figure 1) and r_q is inner radius of qanat legs.

The maximum discharge from the qanat under full flow conditions can be obtained by putting $s = d - 2r_\mathrm{q}$ (neglecting the wall thickness of the qanat legs) in Equation 2. Thus,

$$Q_{\mathrm{q(max)}} = \frac{2\pi k_\mathrm{h} H (d-2r_\mathrm{q})}{\ln\left[\left(\dfrac{[1+(1+4\rho^4/L_\mathrm{q}^4)^{1/2}]^{1/2} + [-1+(1+4\rho^4/L_\mathrm{q}^4)^{1/2}]^{1/2}}{2^{1/2}}\right)\left(\dfrac{(\beta H^2/2)+2\beta\delta^2}{H r_\mathrm{q}}\right)^{2\beta H/L_\mathrm{q}}\right]} \quad (3)$$

where $Q_{\mathrm{q(max)}}$ is the maximum possible discharge from the qanat and d is the depth of the bottom surface of the qanat legs below the undisturbed water table.

Using Sichardt's formulae $\rho = 3000(d - 2r_\mathrm{q})(k_\mathrm{h})^{1/2}$ in Equation 3, where all terms should be in SI units, Equation 3 can then be written as

$$Q_{\mathrm{q(max)}} = \frac{2\pi k_\mathrm{h} H_\mathrm{f} (d-2r_\mathrm{q})}{\ln\left[\xi\left(\dfrac{(BH^2/2)+2\beta\delta^2}{H_\mathrm{f}\mathrm{r_q}}\right)^{2\beta H_\mathrm{f}/L_\mathrm{q}}\right]} \quad (4)$$

where

$$\xi = \frac{(1+\{1+4[3000(d-r_\mathrm{q})(k_\mathrm{h})^{1/2}]^4/L_\mathrm{q}\}^{1/2})^{1/2} + (-1+\{1+4[3000(d-r_\mathrm{q})(k_\mathrm{h})^{1/2}]^4/L_\mathrm{q}\}^{1/2})^{1/2}}{2^{1/2}}$$

Although upconing in the case of qanat-well structures is rare, specifically when the saltwater–freshwater interface is situated at a significant depth below the bottom of the structures, the upconing problem may be catastrophic for a shallow depth of interface in the area near the sea. Therefore, the present analysis is extended to consider upconing as well (Figure 2). After the recommendation of Dagan and Bear (1968)

$$\mathrm{Z}_\infty = \frac{Q_\mathrm{q}}{2\pi[(\rho_\mathrm{s}/\rho_\mathrm{f})-1]k_\mathrm{h}(H_\mathrm{f}-d)} \quad (5)$$

where Z_∞ is the value of z at infinite time. The value of Q attains the maximum value when $Z_\infty = H_\mathrm{f} - d$, from which the following equation can be derived

$$Q_{\mathrm{q(max)}} = 2\pi k_\mathrm{h}\left(\frac{\rho_\mathrm{s}}{\rho_\mathrm{f}} - 1\right)(H_\mathrm{f} - d) \quad (6)$$

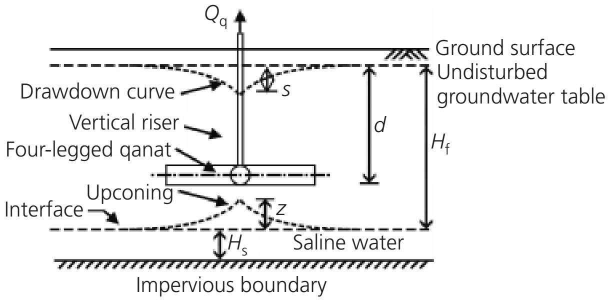

Figure 2. Withdrawal of groundwater by qanat in salinity-affected aquifer

The optimum values of $Q_{q(max)}$ shall be the least of the two values obtained from Equations 4 and 6. The design values of the depth of qanat-well structures are calculated unrealistically high and therefore not feasible, and the number of such qanat-well structures may be reasonably increased as required.

2.2 Groundwater recharge by rainwater harvesting

If freshwater in a coastal area is withdrawn regularly, the saltwater–freshwater interface is progressively advanced horizontally as well as vertically unless the withdrawal is subsequently compensated by a suitable groundwater recharge technique (natural or artificial). The method proposed here includes rainwater harvesting by means of recharge ponds and recharge wells. A hybrid method considering ponds and recharge wells combines the best of both methods, since providing only ponds would take up a huge amount of unnecessary space whereas providing only wells would necessitate pressure injection into the aquifer.

2.2.1 Recharge area

Usually, for a particular community under consideration, neglecting the area of recharge well

$$A_t = A_{roof} + A_{road} + A_{pond} + A_l \tag{7}$$

where A_t is the total area of the community, A_{roof} is the total roof cover area for all buildings in the community, A_{road} is the total road area in the community, A_{pond} is the total pond area of the community and A_l is total area of vacant land in the community.

2.2.2 Factor of safety for rainfall recharge

For a particular community in a coastal area, the net volume of freshwater withdrawal in a certain period of time should not exceed the volume of recharge available for that period. With this concept, the corresponding factor of safety for the particular community was formulated for volumetric constancy, considering also possible recharge available from other sources such as nearby streams/canals, irrigation recycle, etc. (MoWR, 2009) as

$$F = \frac{\text{Volume of water annually available for recharge}}{\text{Volume of water annually extracted}}$$

$$= \frac{[(A_t - A_{roof})\eta + \alpha A_{roof}]R + V_0}{365WP} \tag{8}$$

where W is average water consumption in the community (litres/capita per day), P is population of the community, R is the design annual rainfall (mm), η is the recharge coefficient, α is the fraction of rainwater collected on roofs that is directed towards the recharge well and V_0 is the volume of water available annually from other sources such as, nearby stream/canal, irrigation recycle, etc.

This technique is most effective when the value of the factor of safety F is slightly higher than unity. An excessively high value of F may be necessitated for adequate drainage in the area under consideration to avoid undesirable circumstances such as waterlogging and flooding. Conversely, when F is less than one, the situation can be compensated for by either reducing the withdrawal of groundwater or increasing the catchment area for rainfall recharge.

In the case that F is excessively high for a given locality, the total excess volume of water may be calculated using

$$V_e = [(A_t - A_{roof})\eta + \alpha A_{roof}]R + V_0 - F_D(365WP) \tag{9}$$

where V_e is the annual excess volume of water and F_D is the design factor of safety (to be chosen arbitrarily, slightly greater than unity). An adequate drainage facility should be constructed to mitigate this excess volume of water.

2.2.3 Percolation pond

The design precipitation chosen depends on the design return period of the precipitation. The longer the design return period, the greater the precipitation. The water collected from the roof area in the community is partly allowed to percolate through the recharge chamber/recharge well and the remaining portion is stored for future use (e.g. firefighting, domestic use). Therefore, the total area of the recharge pond for the community may be reasonably estimated considering the net volume of water to be stored in the pond during the monsoon period. Therefore

$$A_{pond} = \frac{[(A_t - A_{roof} - A_{road})\eta_1 + A_{road}]R_m}{1000H_p - (1 - \eta_1)R_m} \tag{10}$$

where A_{pond} is the area of the pond, R_m is the design monsoon rainfall (in mm), η_1 is the runoff coefficient relevant to the area and H_p is the depth of the pond to be excavated (in mm).

2.2.4 Recharge chamber with recharge well

Sarkar (2007) designed and successfully implemented a pilot project using roof top harvesting for the purposes of artificial groundwater recharge and firefighting by means of a recharge structure comprising recharge wells and chambers in a certain locality within the city of Kolkata, which is situated about 100 km from the study area. The subsoil, hydrogeological conditions and rainfall pattern of both places are similar. Therefore, it is expected that the recharge wells are going to work properly in the study area.

The main objective was to collect rainwater to store in the surface or subsurface for future use, with the aim of minimising surface runoff through drains and drainage channels to the rivers and sea. The components of the artificial recharge structure were first flushing provisions, a diversion line, filtration unit, recharge chamber and gravity head recharge well.

The first flushing provision is an arrangement made to drain off the first two or three spells of rain to flow to stormwater drains and thereafter directed to the recharge chambers to remove silt and other foreign materials. The filtration unit takes care of desilting and filtering the harvested rainwater before it is directed to the recharge chamber. This unit has two components – a silting chamber for the settling down of heavier foreign materials and the filtration chamber to arrest lighter particles and silts. In this chamber, filtered rainwater is received for recharge. The outlet of the filter unit is the inlet for the recharge chamber. In this chamber, larger filter materials (pebbles of 20–40 mm) are provided. This filter bed, together with the filtration unit, forms an inverted filtering arrangement, slowing down the rate of flow and thereby helping the settlement of suspended materials within the rainwater. Thereafter, the clean rainwater flows to the gravity head recharge wells towards the groundwater.

Following the recommendations of Sarkar (2007), the dimensions and number of recharge chambers fitted with 100 mm diameter recharge wells for the community under consideration may be reasonably estimated using

$$\frac{V_{\mathrm{w}}N_{\mathrm{w}}}{V_{\mathrm{wS}}N_{\mathrm{wS}}}=\frac{\alpha RA_{\mathrm{roof}}}{\alpha_{\mathrm{S}}R_{\mathrm{S}}A_{\mathrm{roofS}}} \tag{11}$$

where V_w is the volume of the recharge chamber in the community and N_w is the number of recharge chambers fitted with recharge wells of 100 mm diameter adopted in the community. The suffix S denotes the corresponding parameter relevant to Sarkar (2007).

3. Typical case study and quantification

The proposed methodology was used for an intensive case study with adequate quantification in a typical locality of the east coast of India, namely, Contai Polytechnic campus. Although the area is quite small ($\approx$ 90 000 m^2), the model could also be applied to significantly larger coastal localities provided the relevant field parameters are suitably chosen.

3.1 Site characterisation

Contai Polytechnic, a technical institute under the directorate of Technical Education and Training, Government of West Bengal, India, is situated 7 km from Contai town, the headquarters of the district of Purba Medinipur, West Bengal, India. Its geographical location is 21°46′40″N latitude and 87°44′50″E longitude, with an elevation of about 6 m above mean sea level. The geographical location and plan of the community under consideration are shown in Figures 3 and 4, respectively.

A subsurface survey report reveals that the area consists of very thick (up to 300 m) unconsolidated sandy sediments of recent alluvium. From the available hydrogeological data, the coastal area consists of an unconfined aquifer, with the average freshwater table 2–3 m below the ground surface and the average saline water and freshwater interface varying in the range 20–50 m (Goswami, 1968; Sarkar, 2005). From electro-logging test results at the specific site, the average value of H_f for the present analysis was chosen to be 40 m. During 2001–2010, annual precipitation in the area varied in the range 1296–2259 mm (IMD, 2010; OPCCF, 2008). The average monthly precipitation in the study area is shown in Figure 5. Since there is no nearby stream/canal or irrigation land in the locality, the water available from other sources is negligible and thus V_0 is taken as zero. To carry out the study using the model described in Section 2, the input parameters chosen are listed in Table 1.

3.2 Design of qanat-well structure

The aquifer in the study area is unconfined, having an average hydraulic conductivity of $k_h = 3{\cdot}512 \times 10^{-4}$ m/s and $k_v = 3{\cdot}614 \times 10^{-4}$ m/s (from laboratory tests). From the available data (United Nations, 2006; WHO, 2010), the value of W (water demand) was chosen as 140 litres/capita per day. The average population in the campus is 250. Considering a 20% increase, the value of P is taken as 300. On the basis of daily water demand in the campus with hourly pumping operation per day (t) chosen as 2, 3, 4 and 5, the various geometrical parameters of the qanat-well structure (e.g. L_q, r_q and d) were calculated using Equation 4. The value of $Q_{q(\max)}$ was calculated as

$$Q_{\mathrm{q(max)}}=\frac{WP}{\mathrm{t}\times 3600\times 10^{3}} \tag{12}$$

This value of d obtained using Equation 4 is substituted in Equation 6 to check for upconing. If the discharge calculated from Equation 6 exceeds the design discharge as estimated previously, upconing does not take place. Otherwise, the depth d may be calculated considering upconing (i.e. using Equation 6). It was observed that upconing does not occur at the study area.

The values of the required depth of the qanat-well structure for

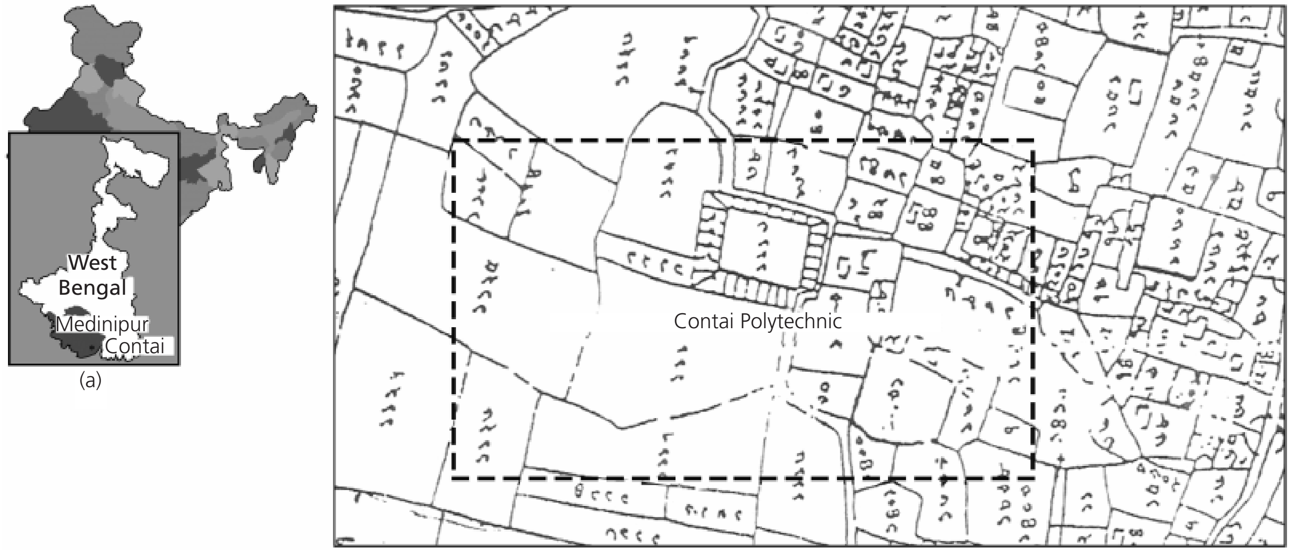

Figure 3. Geographical location of (a) Contai city (b) Contai Polytechnic (not to scale)

different values of the length and radius of the qanat leg are plotted in Figure 6: parameter d decreases following a curvilinear pattern with the length of leg L_q. The variation is quite sharp in the range of $1\ m \leqslant L_q \leqslant 3\ m$ and assumes a linear pattern for $L_q > 3$ m. The curves in Figure 6 should be helpful for design engineers to adopt suitable values of the qanat parameters (r_q, L_q, d and t) considering other design aspects such as maximum depth of water table, feasibility of construction, etc.

3.3 Design of recharge structures

The design of the recharge structures was carried out using the methodology described in Section 2.2.

3.3.1 Rainfall recharge and factor of safety

Successful application of the proposed model depends largely on the estimation of rainfall infiltration, which in turn depends not only on the porosity and arrangement of soil particles and the degree of compaction, but also on other factors such as vegetative cover, climatic conditions, existing soil moisture, extent of porous media below ground level, etc. (Linsley *et al.*, 1982). Adequate estimation of the recharge volume can be done primarily by means of field tests and secondarily from the available literature for the area under consideration and engineering judgement. Infiltration tests – either by infiltrometers or through recharge wells and pits – are mostly recommended for field-based investigations (Water Aid in Nepal, 2011).

As noted earlier, total annual rainfall in the district of Purba Medinipur for the period 2001–2010 varied from 1296 to 2259 mm (IMD, 2010; OPCCF, 2008). For design of a recharge pond equipped with recharge wells for the given community, the factor of safety F for recharge may be estimated from Equation 8 using $A_t = 90\,169\ m^2$ and $A_{roof} = 12\,000\ m^2$.

The Ministry of Water Resources of the Government of India has carried out extensive survey and data acquisition on studies undertaken in various water balance projects in different parts of the country and summarised reasonable values of recharge coefficients based on soil type and other environmental conditions (MoWR, 2009). These recommendations relevant to the study area were taken for the values of the recharge coefficient η. Chaturvedi (1973) proposed a rainfall–infiltration relationship based on field study relevant to the specific hydrogeological conditions at Ganga-Yamuna Doab of India. Kumar and Seethapathi (2002) made reasonable improvements on this model based on field investigations at Upper Ganga Canal, Uttar Pradesh, India. Since both these places are near the present study area, these precipitation–recharge relations were used for the present case study. Wu and Zhang (1994) developed a statistical model to establish correlations between annual rainfall, effective precipitation and groundwater recharge. The concept of effective precipitation (P) was introduced to account for the effect of soil surface evaporation between rainfalls in an effective rainfall event. Effective precipitation was defined statistically, considering the length of the rainfall interval and water surface evaporation in terms of the evaporation ratio and evaporation time. Although this model is based on field conditions applicable to China, it was used for the current case study due to its versatility. In this paper, values of the recharge coefficient η were estimated from the available literature (Chaturvedi, 1973; Kumar and Seethapathi, 2002; MoWR, 2009; Wu and Zhang, 1994), as shown in Table 2.

Wu and Zhang (1994) performed regression analysis correlating

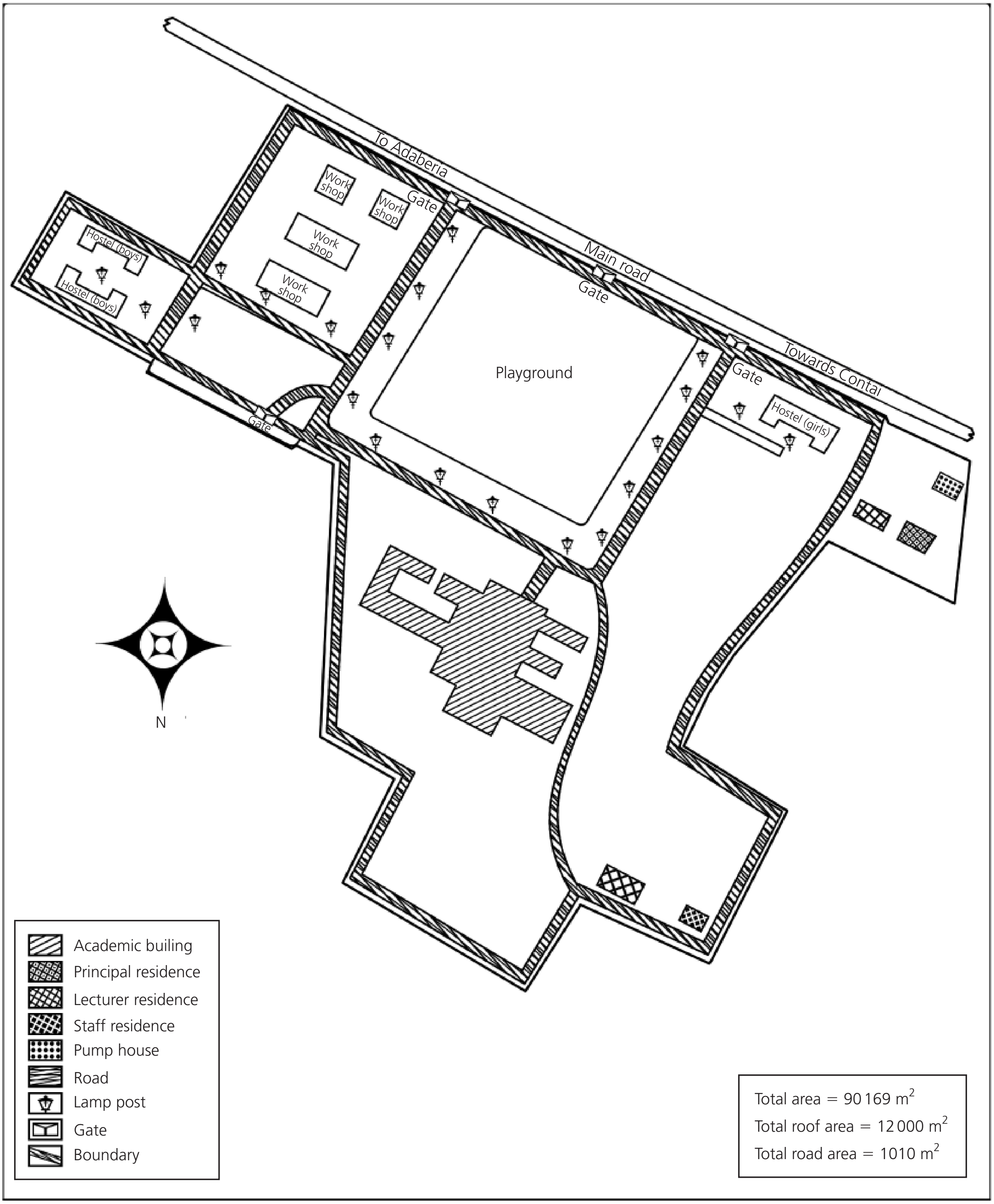

Figure 4. Plan of Contai Polytechnic showing salient features (not to scale)

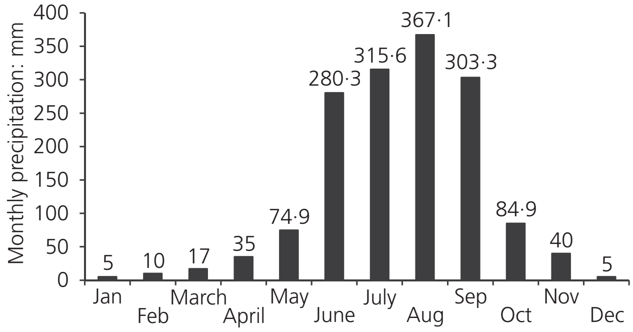

Figure 5. Average monthly precipitation in the study area

annual rainfall, effective precipitation and groundwater recharge, and obtained a functional relationship between annual precipitation p and the total amount of recharge r produced

13. $$r = 0{\cdot}87(p - 5{\cdot}25)$$

where both r and p are in millimetres.

Chaturvedi (1973) derived an empirical relationship for recharge as a function of annual precipitation in the Ganga-Yamuna river basin in India as

14. $$r = 6{\cdot}807(p - 355{\cdot}6)^{0{\cdot}5}$$

where r and p are in inches (1 inch = 25·4 mm).

The formula of Chaturvedi (1973) was later modified by Kumar and Seethapathi (2002) to yield

15. $$r = 3{\cdot}175(p - 388{\cdot}1)^{0{\cdot}76}$$

Following the recommendation of the Ministry of Water Resources, Government of India (MoWR, 2009)

16. $$r = \eta p$$

MoWR (2009) summarised reasonable values of recharge coefficients based on soil type and other environmental conditions. For alluvial areas in the vicinity of the east coast of India, the value of η may be reasonably estimated as 0·16.

Variation of the factor of safety for rainfall recharge with annual precipitation was studied and is shown in Figure 7: the factor of safety increases linearly with an increase in annual precipitation, which is in good agreement with Equation 8. Both the magnitudes and slopes of the curve relevant to the work of Wu and Zhang (1994) are significantly higher ($6 < F < 11$) than the range ($1{\cdot}25 < F < 3$) for the equations of Chaturvedi (1973), Kumar and Seethapathi (2002) and MoWR (2009). For Indian conditions, the results obtained using the correlation of MoWR (2009) are most reliable.

Input parameter	Value
Maximum design discharge, $Q_{q(max)}$	Calculated using Equations 4 and 6
Freshwater head above the saltwater interface, H_f	40 m
Horizontal hydraulic conductivity, k_h	$3{\cdot}512 \times 10^{-4}$ m/s (laboratory test)
Vertical hydraulic conductivity, k_v	$3{\cdot}614 \times 10^{-4}$ m/s (laboratory test)
Conductivity contrast, $\beta = (k_h/k_v)^{0{\cdot}5}$	0·9857
Well eccentricity, δ	0
Population of Contai Polytechnic campus, P	300
Total area of the college campus, A_t	90 169 m^2
Total roof area including student hostels, A_{roof}	12 000 m^2
Total road area A_{road}	1010 m^2
Total area of pond, A_{pond}	1204 m^2
Area of vacant land, A_l	75 955 m^2
Design annual precipitation, R	Various values
Design monsoon precipitation, R_m	Various values
Per capita water consumption, W	140 litres/capita per day
Volume of water available from other sources, V_0	0
Recharge coefficient, η	Chosen from available literature
Runoff coefficient, η_1	Chosen from available literature

Table 1. Values of input parameters used for analysis

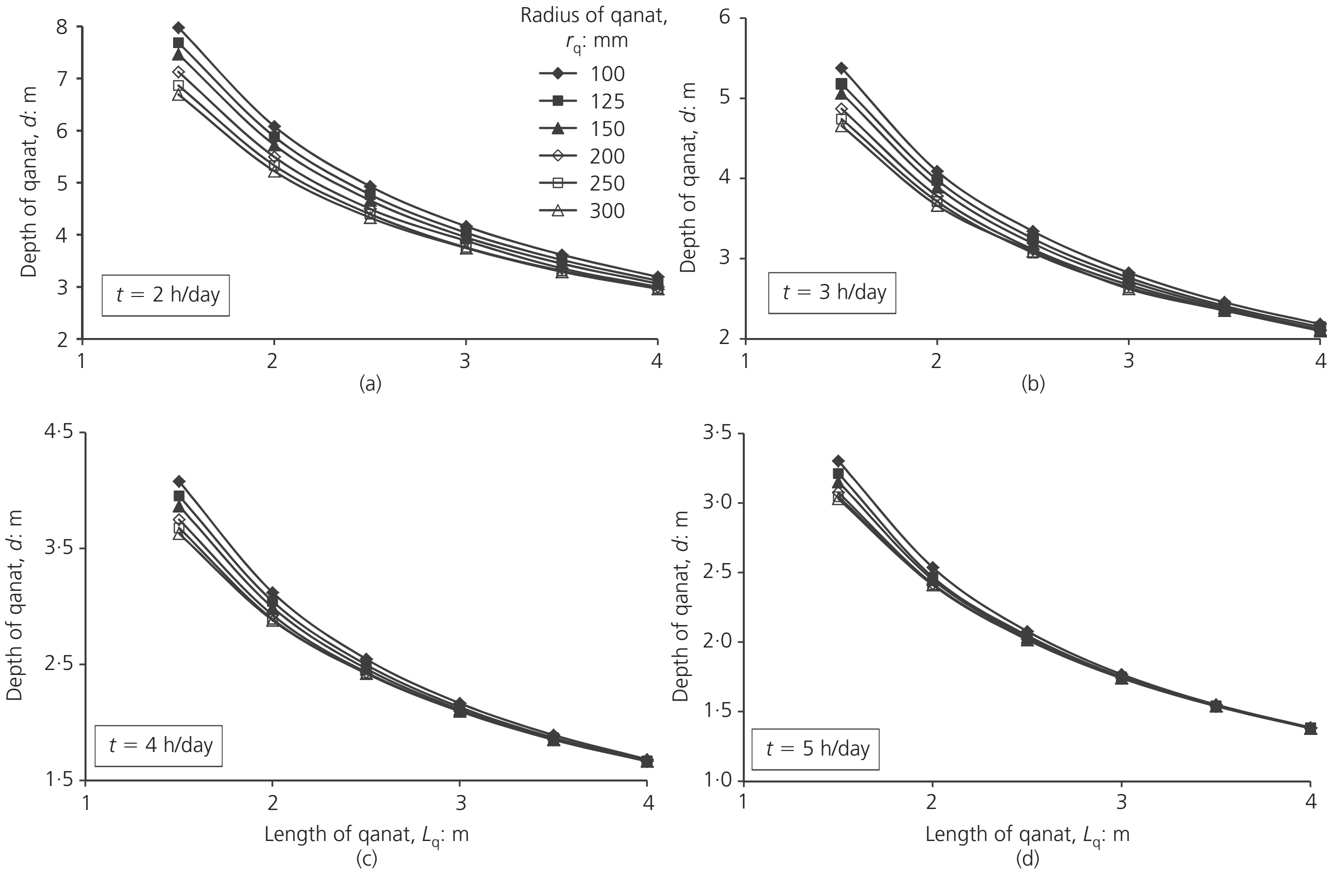

Figure 6. Variation of depth of qanat at base versus length of qanat leg for pumping duration (a) $t = 2$ h/day, (b) $t = 3$ h/day, (c) $t = 4$ h/day and (d) $t = 5$ h/day

Mean rainfall: mm	Recharge coefficient η			
	Wu and Zhang (1994)	Chaturvedi (1973)	Kumar and Seethapathi (2002)	MoWR (2009)
1296	0·866	0·161	0·187	0·16
1525	0·867	0·153	0·189	0·16
1750	0·868	0·145	0·188	0·16
1975	0·869	0·139	0·188	0·16
2259	0·870	0·131	0·186	0·16

Table 2. Values of recharge coefficient calculated according to different formulae

3.3.2 Percolation pond

Runoff should be assessed accurately for the design of recharge structures and may be calculated from

$$\text{Runoff} = \text{Catchment area} \times \text{Rainfall} \times \text{Runoff coefficient}$$

The runoff coefficient plays an important role in assessing runoff availability and depends on the catchment characteristics. The runoff coefficient adopted considered the previous analysis carried out by Patra (2006) for West Bengal, India (see Table 3). The relevant value for the study area was entered into Equation 10 to calculate the total area of percolation pond required. The pond area A_{pond} is plotted against pond depth H_p for different values of average monsoon precipitation R_m in Figure 8. The

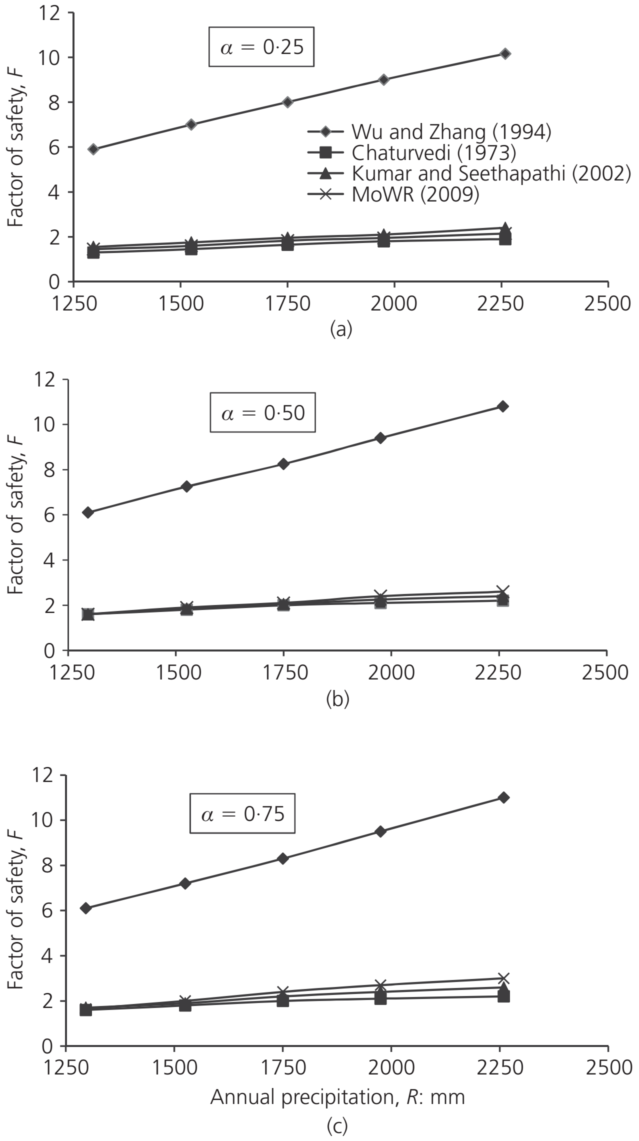

Figure 7. Variation of factor of safety F with annual precipitation R for (a) $\alpha = 0{\cdot}25$, (b) $\alpha = 0{\cdot}50$ and (c) $\alpha = 0{\cdot}75$

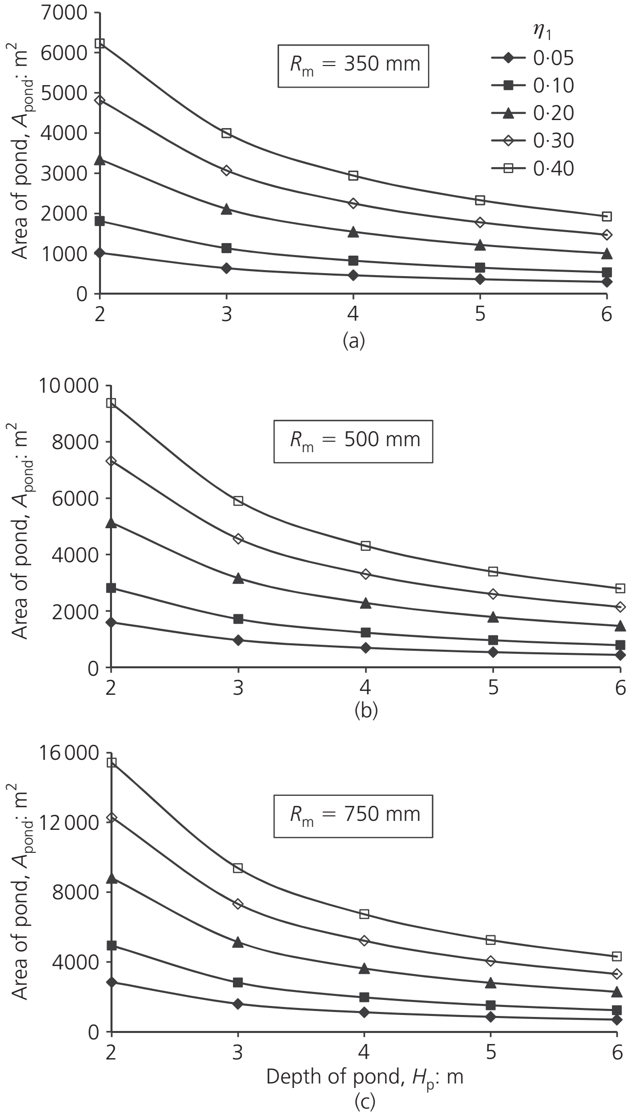

Figure 8. Plots of recharge pond area versus pond depth for monsoon precipitation of (a) 350 mm, (b) 500 mm and (c) 750 mm

Type of catchment	η_1
Roof top catchments	
Tiles	0·8–0·9
Corrugated metal sheets	0·7–0·9
Ground surface coverings	
Concrete	0·6–0·8
Brick pavement	0·5–0·6
Untreated ground catchments	
Soil on slopes less than 10%	0·0–0·3
Rocky natural catchments	0·2–0·5
Green area	0·05–0·1

Table 3. Values of runoff coefficient η_1 (after Patra, 2006)

required area of recharge pond decreases in a hyperbolic manner with the depth of pond to be excavated, which is well in agreement with Equation 10.

3.3.3 Recharge chamber with recharge well

The relevant calculations for a recharge chamber with a recharge well were carried out using Equation 11. Figure 9 shows the required number of recharge chambers N_w plotted against the volume of recharge chambers V_w for different values of annual precipitation R ($1750\ \text{mm} < R < 2259\ \text{mm}$) and roof rainwater collection factor α ($0{\cdot}25 < \alpha < 0{\cdot}75$). The figure shows that N_w decreases fairly exponentially with V_w. The rate of decrease is pronounced for $V_w < 5$, beyond which a stabilising tendency is noted.

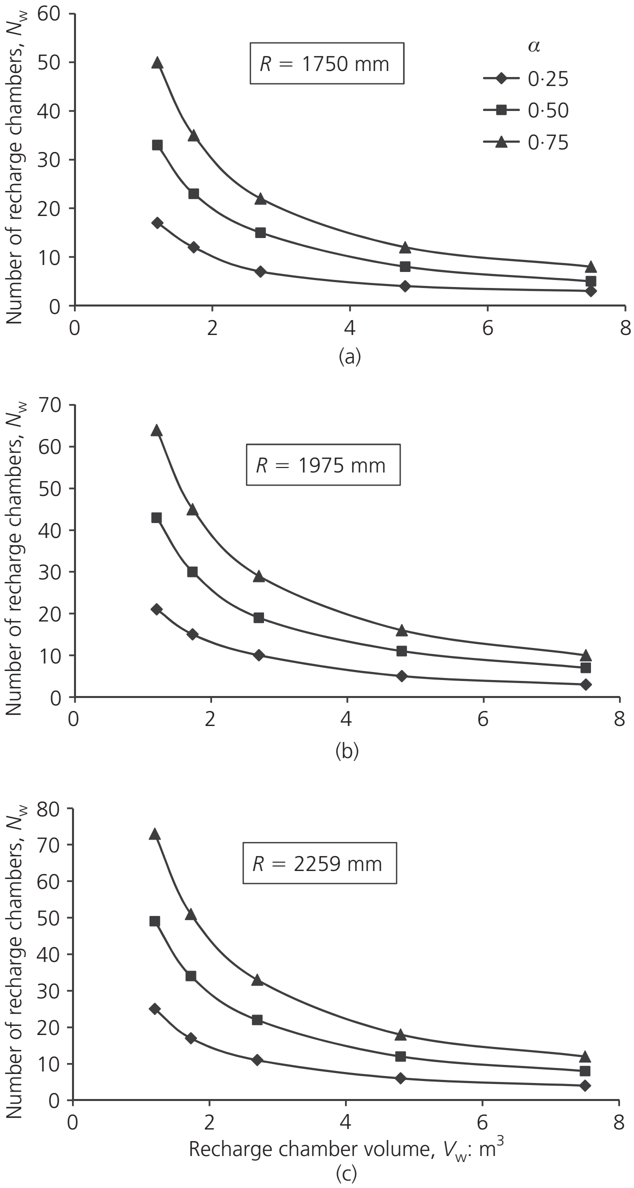

Figure 9. Variation of recharge chamber volume with number of chambers for (a) $R = 1750$ mm, (b) $R = 1975$ mm and (c) $R = 2259$ mm

3.3.4 Appropriate engineering design

3.3.4.1 QANAT-WELL STRUCTURE

Using the data from Figure 6, the suggested parameters are $t = 3$ h, $r_q = 125$ mm, $L_q = 2{\cdot}5$ m and $d = 3{\cdot}25$ m. (For future safety provision, two qanat-well structures are recommended for alternative use.) Thus, the design depth of the qanat below ground level is d + maximum depth of groundwater table = 3·25 m + 3 m = 6·25 m.

3.3.4.2 FACTOR OF SAFETY

Using the data from Figure 7, the safety factors for minimum and maximum rainfall were calculated and are presented in Table 4

Rainfall	Factor of safety F	Method
Minimum	5·88	Wu and Zhang (1994)
	1·33	Chaturvedi (1973)
	1·49	Kumar and Seethapathi (2002)
	1·27	MoWR (2009)
Maximum	11·23	Wu and Zhang (1994)
	2·76	Chaturvedi (1973)
	3·47	Kumar and Seethapathi (2002)
	3·17	MoWR (2009)

Table 4. Calculated safety factors for maximum and minimum rainfall

($\alpha = 0{\cdot}5$). For Indian conditions, the recommended value as per MoWR (2009) is most suitable. The value of the factor of safety under minimum rainfall in the last 10 years should not fall below 1·0, therefore the values are satisfactory in terms of reasonable compensation of withdrawal of groundwater. Furthermore, for maximum rainfall in the last 10 years, the factor of safety exceeds 3·0, which should introduce sufficient push back of the saline water interface. For excessively high values of factor of safety F, adequate drainage should be facilitated at the site towards a nearby stream channel.

3.3.4.3 AREA AND DEPTH OF PERCOLATION POND

From Figure 5, the design rainfall data was chosen as $R_m = 367{\cdot}1$ mm. The depth of the pond was chosen as $H_p = 3$ m. Therefore using data from Figure 8, the area of pond required may be interpolated as $A_{pond} = 1202{\cdot}436$ m^2. Hence, for Contai Polytechnic campus, four ponds, each of area 301 m^2 and depth 3 m are recommended.

3.3.4.4 RECHARGE CHAMBER WITH RECHARGE WELL

Adopting a total roof area for the site of 12 000 m^2, $\alpha = 0{\cdot}5$ and $R = 2259$ mm (maximum rainfall in the last 10 years), keeping the recharge chamber dimensions as 2 m length, 2 m width and 1·2 m height, the number of recharge chambers with recharge wells was obtained from Figure 9 as $N_w = 12$. The dimensions of the recharge wells adopted here are as per the recommendations of Sarkar (2007) and are presented in Table 5. Application of the proposed scheme is illustrated in Figure 10(a) and details of the recharge well are shown in Figure 10(b). The methodology of

Diameter of well	100 mm
Strainer diameter in aquifer	100 mm
Length of strainer	12 m
Diameter of inlet strainer in recharge chamber	150 mm
Length of 150 mm strainer	800 mm

Table 5. Dimensions of recharge wells

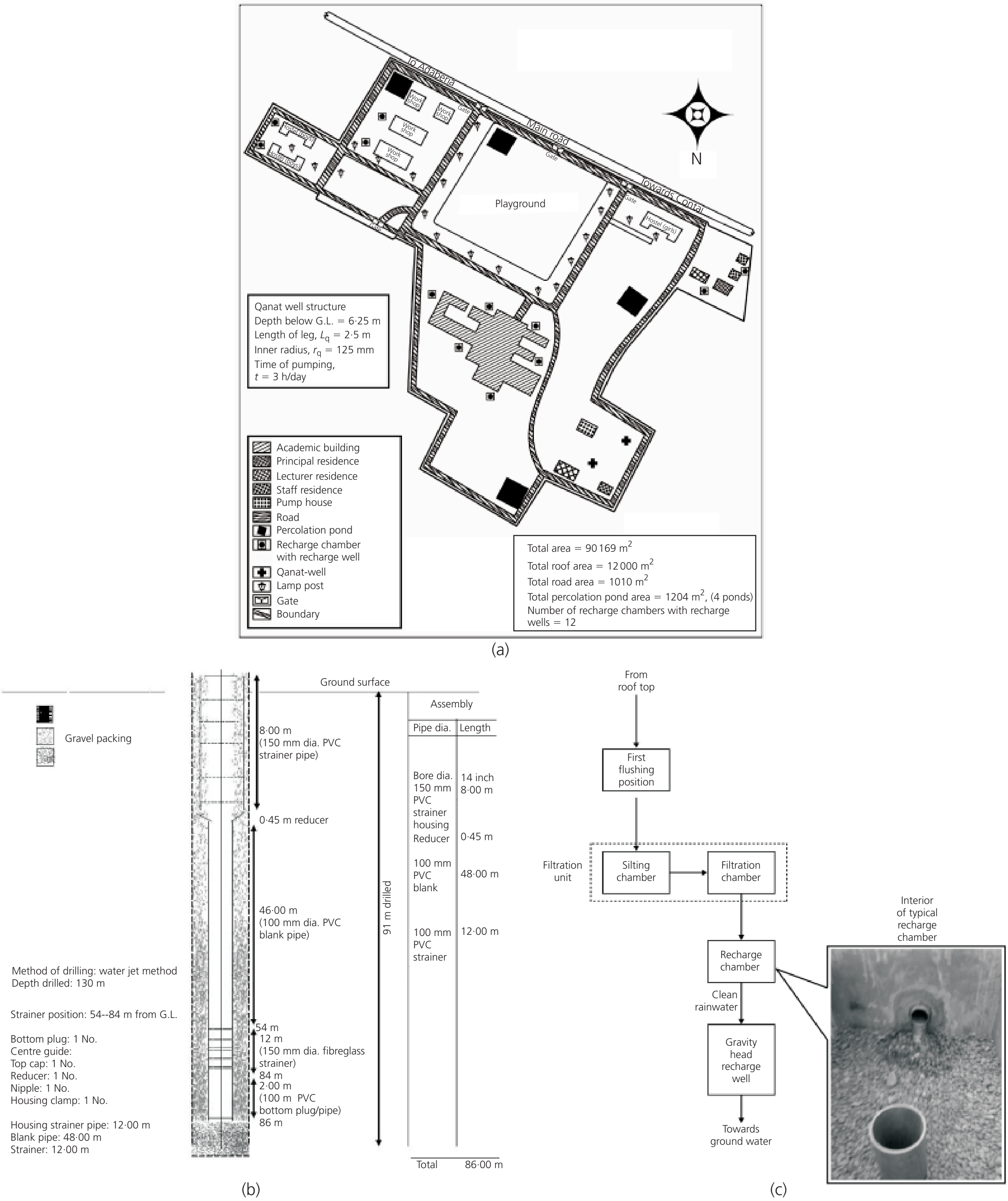

Figure 10. Application of proposed technique. (a) Contai Polytechnic campus area showing proposed locations of qanat-well structures, percolation ponds and recharge chambers with recharge wells (not to scale). (b) Cross-section of recharge well adopted. (c) Methodology of recharge through recharge wells and recharge chambers, after Sarkar (2007)

recharge through recharge wells and recharge chambers, as per Sarkar (2007), is shown in Figure 10(c).

4. Model limitations

The proposed simplified analytical model can be reasonably adopted for groundwater management relevant to coastal aquifers, but it does have a few inherent limitations.

- The parameters for qanat-well structures, recharge wells, etc. used in the design are highly site-specific. Successful application of the entire method is only possible when the design parameters are adequately chosen for a particular site.
- Field conditions are mostly heterogeneous, but saline water intrusion modelling was carried out with the simplified idealisation of a homogeneous aquifer. By adopting certain engineering judgement, an approximate analytical solution can be arrived at using the proposed model for heterogeneous aquifers as well.
- A sharp interface between saline water and freshwater was assumed in the mathematical analysis for the qanat-well structure considering upconing. However, in practice a brackish water zone exists.

5. Conclusions

An innovative technique for groundwater management in coastal aquifers from the viewpoint of the safe withdrawal of freshwater and simultaneous control of saltwater intrusion has been suggested. This new method consists of withdrawal by qanat-well structures with reasonable compensation by rainwater harvesting by means of recharge ponds and recharge wells. The salient features of the methodology are described by considering a design example adopted at Contai Polytechnic campus of the district of Purba Medinipur in the state of West Bengal, India.

It was observed that the depth of the qanat d decreases following a curvilinear pattern with the length of the qanat leg L_q. The variation is quite sharp in the range 1 m $\leqslant L_q \leqslant 3$ m and assumes a linear pattern for $L_q > 3$ m. The factor of safety for rainwater recharge in the selected location increases linearly with increases in annual precipitation. The curves relevant to the formulae of Chaturvedi (1973) and Kumar and Seethapathi (2002) almost coincide. Both the magnitudes and slopes of the curves following Wu and Zhang (1994) are significantly higher. The proposed technique is most effective when the factor of safety is slightly higher than unity. The required area of recharge pond decreases in a hyperbolic manner with the depth of pond to be excavated. The number of recharge chambers N_w decreases fairly exponentially with the volume of the recharge chambers V_w. The rate of decrease is pronounced for $V_w < 5$, beyond which a stabilising tendency is noted.

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