Technical Note

Tropical, Seasonal River Basin Development through a Series of Vented Dams

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Abstract: Tropical rivers are predominantly seasonal in nature, and managing water resources during the deficit period is becoming more difficult because of the rapidly increasing demand for water. The present investigation focuses on harvesting Netravathi River water in the southern Indian peninsula through a series of vented dams with an estimated storage capacity of 102 Mm^3 for use during the deficit period. A brief hydraulic design of a vented dam at a specific location is presented. The spacing and capacity of these reservoirs were worked out on the basis of the dam height and the river characteristics. The proposed vented dams are seasonal dams, and the closure of the vents will be decided on the flow available (i.e., 95% dependable flow), the storage capacity, and the minimum water release required for the downstream ecosystem. The appropriate time to start storing water in the vented dams was estimated to be in the month of November, and the entire process of storing water in the vented dams may last for about 41 days. An operational protocol for the storing process is presented. The investigations of aquifer parameters were performed by using electrical resistivity, pumping, and soil tests. The results indicated that the aquifer is shallow, unconfined in nature, and had a depth ranging from 18 to 30 m and hydraulic conductivity ranging from 62.6 to 406 m/day. A multiple regression model developed to assess the groundwater recharge in the adjoining well fields indicated that water table fluctuations may be 30% of reservoir level fluctuations. Because the river is also tidal in nature, a saltwater exclusion dam is proposed at the lower reaches of the river to prevent the entry of saltwater along the river during the summer period. **DOI: 10.1061/(ASCE)HE.1943-5584.0000316.**

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Introduction

India has 2.45% of the world's land resources, 4% of the world's fresh water, and 16% of the world's population. Most parts of India receive significant rainfall during the southwest monsoon season from June to September. India receives an average annual precipitation of about 4×10^6 Mm^3. Only about 0.69×10^6 Mm^3 of the available surface water can be utilized. With the addition of replenishable groundwater resources of about 0.432×10^6 Mm^3, the total usable water in the country is about 1.122×10^6 Mm^3 (Central Water Commission 2006). However, India will be water stressed because of the rapidly rising population and increasing agricultural, industrial, and other requirements (Central Water Commission 2006; Rakeshkumar et al. 2005). For the projected population in the year 2050, the per capita availability of water in the country will be 1,140 m^3/year (Gupta and Deshpande 2004). The Dakshina Kannada district is one of the fast growing coastal districts of the Karnataka state. This study proposes river basin development through a series of vented dams across the tropical Netravathi River in India. The Netravathi River drains an area of about 0.3657 Mha carrying an average annual discharge of about 11,502 Mm^3 (Central Water Commission 2006). The study area description and the river flow statistics are provided elsewhere (Shetkar and Mahesha 2011). The Netravathi River is joined by the Kumardhara River at Uppinangady (i.e., the confluence point) and flows as the Netravathi River until it joins the Arabian Sea. The district receives an average annual rainfall of about 3,930 mm and still experiences an acute shortage of fresh water in the nonmonsoon (i.e., deficit) periods.

A vented dam is a structure with several openings called vents through which the flow during the surplus period is allowed without obstruction. These dams do not impede environmental flow but increase groundwater recharge. The storage of water is achieved by the closing of the gates between the piers during low flow periods. The height of vented dams is below the bank level so that water storage is restricted to within the river banks, and adjoining lands are not inundated. Fig. 1 shows a vented dam across the Netravathi River.

The recharge of aquifers by rivers stores water that would have otherwise flowed quickly out of the basin. The conjunctive use of surface and groundwater needs to be promoted wherever practical (Rosenberg and Lund 2006). Groundwater provides dry weather flows, which, if managed properly, can provide a stable flow and water for the preservation of high biodiverse habitats (Griffith et al. 2006). Seepage from surface water bodies into aquifers augments and preserves the groundwater (Korkmaz and Halil 2006). Recharged water in the form of groundwater seepage enters back into the river as base flow. This is particularly important for regions

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J. Hydrol. Eng., 2011, 16(3): 292-302

Fig. 1. Photo of the vented dam

in which rainfall and runoff are unevenly distributed over the year (Konrad 2006). The construction of various structures for the purposes of recharge have been successful in many parts of India. After the construction of such structures, the groundwater table variation trend has changed from declining to rising (Karve 2003; Somratne et al. 2005). Studies have shown that up to 50% of the depth of water infiltrated in a recharge zone contributes to the groundwater below the root zone (Singh et al. 2003).

The dynamic behavior of the groundwater is primarily attributable to the recharge and withdrawal of water. A variety of approaches on the basis of stream flow data, basin characteristics, climate topography, land use data, and remote sensing techniques has been adopted to model the recharge depending on the circumstances (Kumar 2000). Uncertainties in each approach underscore the need for the application of multiple techniques to increase the reliability of recharge estimates (Scanlon et al. 2002). Groundwater model calibration or inversion is used to predict the recharge rates from the information on the hydraulic heads, from hydraulic conductivity, and from other parameters (Sanford 2002). Because recharge and hydraulic conductivity are often highly correlated, model inversion by using hydraulic head data is limited to estimating the ratio of the recharge to hydraulic conductivity. Healy and Cook (2002) reviewed the methods that use groundwater level data for recharge estimation. The advantage of an estimation from groundwater table data is its simplicity and insensitivity to the mechanism by which water moves in an unsaturated zone (Sanford 2002). The water table fluctuation method is best applied to shallow water tables that display a sharp rise and decline and is usually used for the estimation of long term mean annual groundwater recharge (Lee et al. 2006). Rushton and Ward (1979) reviewed the methods of estimating groundwater recharge in temperate climates. The multiple tracer approaches offer the best potential for a reliable recharge estimate in studies that require point information (Jacobus and Sinners 2002). A linear regression model was used to quantify recharge at the regional level in Minnesota (Lorenz and Delin 2007), which compared well with the estimates made by using the other method. The Monte Carlo technique was used to simulate the groundwater recharge of the Manawatu region, New Zealand (Bekesi and McConchi 1999), which compared well with the actual scenario. For an initial, quick estimate of recharge, geographic information system (GIS) toolkits were successfully used at several locations in the United States (Lin et al. 2008), which may be used as input for detailed modeling.

This study proposes river basin development by surface water storage, groundwater recharge, and the prevention of saltwater intrusion. The present investigations primarily focus on the estimation of possible locations for vented dams across the river, their storage capacity, a brief hydraulic design, and a schedule of vented dam gate operation. In addition, the estimation of aquifer parameters by using pumping tests, soil tests, and geoelectrical tests are discussed. The establishment of the correlation between the water level in the wells and other parameters including reservoir level are also discussed. The results from the investigation may be useful for the design of strategic plans to develop tropical, coastal, and seasonal river basins.

Methodology

Google Earth (2004) images were used to record latitude and longitude and to measure river width, bank height, and bed elevation at 100 m intervals. A reconnaissance survey to check the field conditions and suitability of the vented dam locations was performed. In this paper, a brief hydraulic design of the primary body of the vented dam structure for one of the vented dams is presented. For the design of small hydraulic structures, a flood frequency of 20–25 years is normally adopted. From probability studies, the peak flow for a return period of 25 years is 6,987 m^3/s observed by Gumbel's method, and the corresponding maximum flood level is 11.7 m (Shetkar and Mahesha 2011). A brief hydraulic design of the vented dam is provided considering stability and seepage factors.

For the estimation of vented dam capacities, the average bank heights on either side were determined after measuring the bank heights at 100 m intervals along the river. The height of the dam was fixed considering the bank height and afflux. For the calculation of storage, the sections were considered at 100 m intervals for which the reduced levels were obtained from the *Google Earth* (2004) satellite images and verified with ground checks. The depth of water at each subsequent section from the vented dam was computed until it became zero or negative. Knowing the width of the river and the depth of water, calculations for the flow area were performed. For the calculation of the reservoir storage, a program was written in *FoxPro* (2009), and the storage capacities for various heights of the dam were computed. Similar analyses were performed for all proposed vented dams across the rivers Kumardhara and Netravathi.

The reservoir operation policy for each vented dam was updated on the basis of the 95% dependable inflow at the site, storage capacity, and the water release required for environmental aspects. Hydrogeological investigations were performed at 19 locations to characterize the aquifer and to assess the extent of saltwater intrusion. The details on well locations, saltwater intrusion, and the vulnerability assessment were performed for the study area (Shetkar and Mahesha 2011). An electrical resistivity test was conducted at nine locations near the observation wells in the study area by using the Schlumberger configuration (Nath et al. 2000). The electrode half spacing (i.e., $AB/2$) range was 10–80 m. The soil samples were collected from these locations for the determination of porosity, specific gravity, and grain size. *AQTESOLVE* was used to determine the hydraulic conductivity from the observations of the pumping test by using the Neuman method (1974). The aquifer properties were also determined from the measurable soil parameters as per Barr (2001). The soil samples were collected from the locations near the wells.

For this study, the water table variation in the adjoining wells attributable to the storage in the vented dam was studied. The effect

J. Hydrol. Eng., 2011, 16(3): 292-302

of water storage in the reservoir was attempted through multiple linear regressions by using the *Essential Regression and Experimental Design for Chemists and Engineers* software, which is an add-on package for Microsoft Excel. The input parameters for the model included the hydraulic conductivity of the aquifer, the observed water table, the distance from the river or reservoir, the reduced level of the ground surface near the observation well, and the depth of the well and the reservoir water level.

Fig. 2. Existing vented dam at Thumbe across the Netravathi River

The water table fluctuations were monitored at 19 open wells on the banks of the Netravathi River on a weekly basis during 2006–2007 to form 304 observations. From this, 153 were used for the model calibration. The multiple variable regression equation used for the analysis is

$$y = b_0 + b_1x_1 + b_2x_2 + b_3x_3 + b_4x_4 + b_5x_5 \quad (1)$$

where y = water table elevation in the well (m); x_1 = distance of the observation well from the river or reservoir (km.); x_2 = water level in the reservoir (m); x_3 = hydraulic conductivity (m/day); x_4 = reduced level of ground surface near the observation well (m); and x_5 = depth of the observation well (m).

Results and Discussion

Present Status of Natravathi River Basin Development

At present, the domestic as well as the industrial water requirements for the region are met by Netravathi River water, which is being supplied by the city of Mangalore by a vented dam near Thumbe (Fig. 2). The capacity of this reservoir is estimated to be

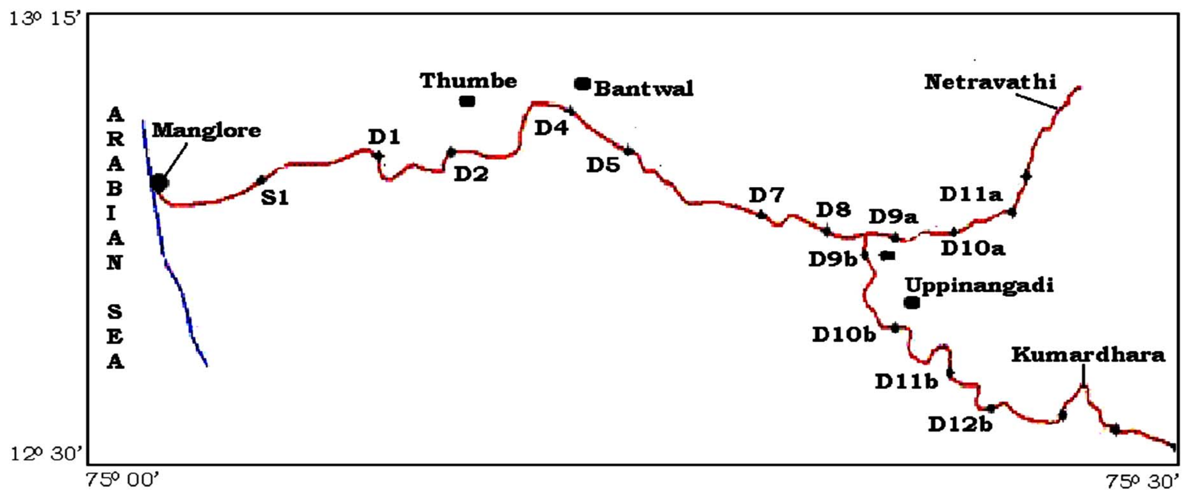

Fig. 3. Locations of the proposed vented dams

Table 1. Details of the Proposed Locations for Construction of Vented Dams

Serial number	Dam identification	Latitude	Longitude	Distance from previous dam (km)	Width of river (m)	Remark
01	S_1	12°51'06.76" N	74°53'38.62" E	7.70	400.0	
02	D_1	12°51'55.28" N	74°57'33.73" E	7.90	330.0	
03	D_2	12°52'20.98" N	75°00'17.85" E	6.60	330.0	Thumbe[a]
04	D_4	12°53'51.80" N	75°03'59.02" E	9.24	320.0	
05	D_5	12°52'43.11" N	75°06'02.59" E	4.32	320.0	Sarapadi[a]
06	D_7	12°50'55.76" N	75°10'43.09" E	9.64	240.0	
07	D_8	12°50'24.42" N	75°13'12.22" E	5.00	240.0	
08	D_{9a}	12°50'24.64" N	75°15'19.92" E	3.91	180.0	
09	D_{9b}	12°49'40.26" N	75°14'33.32" E	3.70	160.0	Uppinangadi[a]
10	D_{10a}	12°50'33.48" N	75°17'26.70" E	4.20	160.0	
11	D_{10b}	12°47'31.78" N	75°15'28.88" E	5.18	180.0	
12	D_{11a}	12°51'25.63" N	75°19'33.09" E	4.28	140.0	
13	D_{11b}	12°46'25.65" N	75°17'37.64" E	6.62	160.0	
14	D_{12b}	12°45'6.93" N	75°18'56.29" E	4.50	160.0	

[a]Existing vented dams.

J. Hydrol. Eng., 2011, 16(3): 292-302

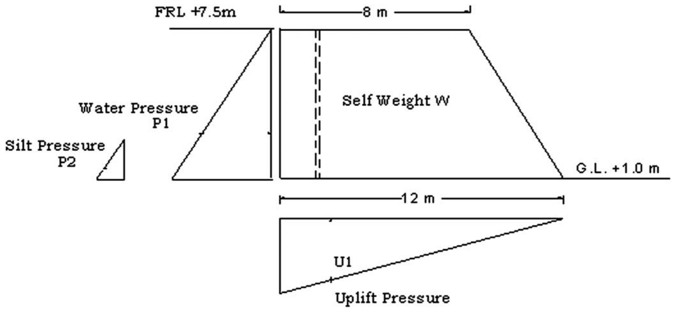

Fig. 4. Forces acting on the pier

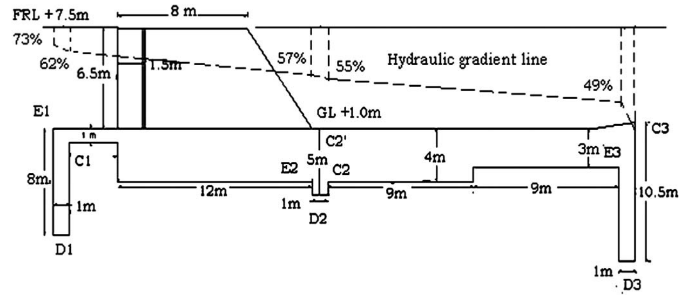

Fig. 5. Cross section of the designed vented dam

Table 2. Storage Capacities of Dams S_1 and D_1 to D_8 (Netravathi River)

	Reduced level of top of dam (m)	Storage capacity (10^3 m^3)	Reduced level of top of dam (m)	Storage capacity (10^3 m^3)	Reduced level of top of dam (m).	Storage capacity (10^3 m^3)
Serial number	S_1 7.70 km from sea		D_4 9.24 km from D_2		D_7 9.64 km from D_5	
01	2.00	18.37	6.00	0.60	14.00	0.22
02	2.50	20.86	6.50	0.80	14.50	0.31
03	3.00	23.24	7.00	1.03	15.00	0.40
04	3.50	25.73	7.50	2.29	15.50	0.52
05	4.00	27.74	8.00	2.89	16.00	0.56
06			8.50	3.91	16.50	0.72
07	D_1 7.90 km from S_1		9.00	4.52	17.00	0.77
08			9.50	5.64	17.50	1.29
09	2.00	1.79	10.00	6.20	18.00	1.41
10	2.50	3.49	10.50	7.32	18.50	1.95
11	3.00	6.86	11.00	8.07	19.00	2.01
12	3.50	8.06	11.50	9.18	19.50	2.77
13	4.00	8.97	12.00	9.89	20.00	2.97
14	4.50	10.17			20.50	3.45
15	5.00	11.07	D_5 4.32 km from D_4		21.00	3.63
16	5.50	12.27			21.50	4.13
17	6.00	12.33	13.00	2.92	22.00	4.35
18			13.50	3.32	22.50	4.92
19	D_2 6.60 km from D_1		14.00	3.71	23.00	5.07
20			14.50	4.13	23.50	6.02
21	2.00	0.22	15.00	4.53	24.00	7.07
22	2.50	0.69	15.50	4.95	D_8 5.0 km from D_7	
23	3.00	1.01	16.00	5.29	25.00	1.47
24	3.50	2.95	16.50	5.76	25.50	1.65
25	4.00	3.48	17.00	6.10	26.00	1.9
26	4.50	5.15	17.50	6.54	26.50	2.07
27	5.00	6.07	18.00	6.76	27.00	2.24
28	5.50	7.98	18.50	7.55	27.50	2.4
29	6.00	9.24	19.00	7.75	28.00	2.59
30	6.50	10.89			28.50	2.75
31	7.00	12.29			29.00	2.94
32	7.50	13.95			29.50	3.04
33	8.00	15.50			30.00	3.13
34	8.50	17.15			30.50	3.38
35	9.00	18.64			31.00	3.51
36	9.50	20.29			31.50	3.76
37	10.00	21.74			32.00	3.87
38	10.50	23.30			32.50	4.12
39					33.00	4.19
40					33.50	4.44
41					34.00	4.45

J. Hydrol. Eng., 2011, 16(3): 292-302

Table 3. Storage Capacities of Dams D_{9a}, D_{10a}, D_{11a} (Netravathi River), D_{9b}, D_{10b}, D_{11b}, D_{12b} (Kumardhara River)

	Reduced level of top of dam (m)	Storage capacity (10^3 m^3)	Reduced level of top of dam (m)	Storage capacity (10^3 m^3)	Reduced level of top of dam (m)	Storage capacity (10^3 m^3)
Serial number	D_{9a} 1.4 km from confluence point		D_{9b} 1.1 km from confluence point		D_{11b} 6.8 km from D_{10b}	
01	33.00	1.05	36.00	4.55	64.50	1.44
02	34.00	1.40	37.00	5.17	65.00	1.47
03	35.00	1.97	38.00	5.49	65.50	1.63
04	36.00	3.43	38.50	6.09	66.00	1.68
05	37.00	3.91	39.00	6.65	66.50	1.84
06	38.00	4.52	39.50	7.33		
07	39.00	5.09	40.00	7.87		
08	D_{10a} 4.2 km from D_{9a}		40.50	8.67	D_{12b} 4.4 km from D_{11b}	
09	38.00	0.36	41.00	9.27	66.00	0.31
10	38.50	1.10	42.00	9.99	66.50	0.41
11	39.00	2.90	D_{10b} 5.2 km from D_{9b}		67.00	0.50
12	39.50	3.30	46.00	0.46	68.00	0.59
13	40.00	3.44	47.00	1.04	69.00	0.74
14	40.50	3.81	48.00	1.48	70.00	0.87
15	D_{11a} 4.28 km from D_{10a}		49.00	1.90		
16	40.00	0.59	50.00	2.26		
17	40.50	0.67	51.00	2.71		
18	41.00	0.75	52.00	3.04		
19	42.00	0.85	53.00	3.20		
20	43.00	1.17	54.00	3.40		
21	44.00	1.44				
22	45.00	1.68				
23	46.00	1.79				
24	47.00	2.07				

about 4.0 Mm^3 with the crest level of the vented dam at +4.0 m above mean sea level. Recently, because the capacity of the Thumbe reservoir has been reduced by silting and significant leakage through the structure and along the pipelines, the water supply to the city has been drastically reduced. The increasing demand for water in the near future will be so high that it may be impossible for the existing system to fulfill the demand. A status report (Mahesha 2008) indicated a total storage of about 14 Mm^3 from the existing vented dams across the Netravathi River and its tributaries.

Proposed Vented Dams

The present storage is negligible compared to the average annual flow in the river (i.e., 11,502 Mm^3), and the water demand is exponentially increasing because of urbanization, the development of satellite towns, and the expansion of irrigation. Climate change effects are found to be adversely affecting the water resources of the region as well (Shetkar and Mahesha 2011). Hence, a need for river water harvesting by a series of small storage structures called vented dams is needed without causing the inundation of fertile land and environmental degradation. About 14 locations were tentatively proposed for the construction of vented dams across the river considering the bed width, slope, geography, backwater length, bank height, and the capacity of the proposed vented dams. The locations of the proposed vented dams are shown in Fig. 3 and the details obtained from *Google Earth* (2004) are given in Table 1.

Hydraulic Design of Vented Dam

For the site D_2, the maximum storage level is 7.50 m and the average bed level at the site is +1.0 m. Providing 101 vents of 2.0 m width and 100 piers of 1.25 m width, the total waterway = 327 m. The calculation for the required waterway was performed with Lacey's wetted perimeter formula (Garg 1997) and is 397 m with a looseness factor of 0.823. The corresponding silt factor and the normal scour depths are 1.1 and 8.80 m, respectively (Indian Standard Code 1989). The bottom of the upstream and downstream cutoffs may be kept at −1.55 and 5.90 m, respectively. The velocity of approach estimated from the discharge, cross-sectional area, and the scour depth is 2.2 m/s. The corresponding velocity head is 0.23 m and the elevation of the total energy line above the bed level is about 11.93 m. The afflux attributable to the obstruction is estimated to be about 2.12 m. The vent way provided was checked against the safe passage of the design discharge.

The foundation level is fixed at 2.5 m below the bed level (i.e., at −1.5 m), which is adequate for structures designed as nongravity structures on pervious foundations. The length of the piers provided varies linearly from 8.0 m at the top to 12.0 m at the bottom with the upstream face vertical and the downstream face sloping.

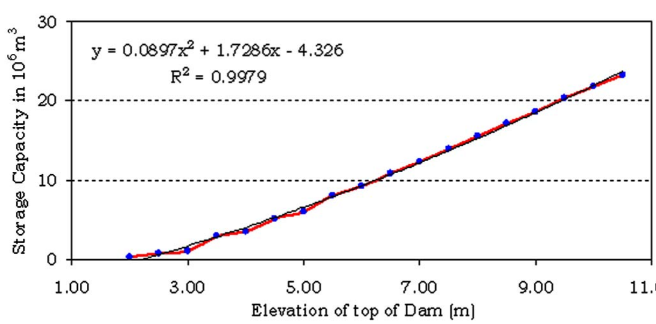

Fig. 6. Stage-storage capacity curve for various heights of dam D_2

Table 4. Regression Coefficients for the Calculation of Storage of Vented Dams

Serial number	Dam identification	Regression coefficients $ax^2 + bx + c = 0$ a	b	c	R^2	Reduced level of top of dam (m)	Depth of water (m)	Storage capacity (10^3 m^3)
1	S_1	−0.2429	6.1791	6.9577	0.9996	2.0	2.0	18.37
2	D_1	−0.5501	7.0350	−10.087	0.9880	6.0	6.0	12.33
3	D_2	0.0897	1.7286	−4.3260	0.9979	9.0	7.5	18.64
4	D_4	0.0647	0.4755	−4.9507	0.9954	11.0	5.0	08.07
5	D_5	0.0046	0.6601	−6.4279	0.9974	19.0	6.0	07.75
6	D_7	0.0520	−1.3292	8.5856	0.9911	24.0	10.0	07.07
7	D_8	−0.0029	0.5025	−9.2738	0.9974	34.0	9.0	04.45
9	D_{9a}	−0.0079	1.2907	−33.199	0.9753	39.0	6.0	05.09
10	D_{10a}	−0.7429	59.615	−1192.5	0.9541	40.0	2.0	03.44
11	D_{11a}	0.0034	−0.0797	−1.7108	0.9901	45.0	5.0	01.68
12	D_{9b}	0.0702	−4.4946	75.256	0.9844	41.0	4.0	09.27
13	D_{10b}	−0.0238	2.7472	−75.594	0.9983	53.0	7.0	03.20
14	D_{11b}	0.0429	−5.4123	172.23	0.9713	66.0	2.0	01.68
15	D_{12b}	−0.0041	0.6953	−27.605	0.9927	70.0	5.0	00.87
							Total	101.91

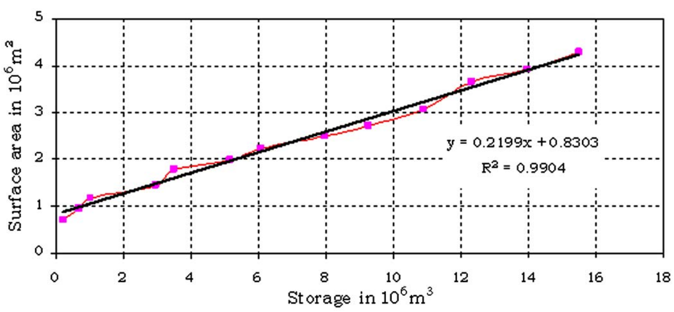

Fig. 7. Surface area per unit storage in the reservoir D_2

The stability analysis performed for the section shown in Fig. 4 indicates a factor of safety of 2.98 (i.e., > 2) against overturning and 1.55 (i.e., > 1.5) against sliding. The eccentricity of 0.56 m is within the limits (i.e., 2 m). The stresses at the foundation are less than 158.17 kN/m^2, which is within the permissible limit of 200 kN/m^2 for the site. Assuming a groove of 10 cm on either side and providing a clear span of 2.0 m, the thickness of the steel shutters required for resisting the bending moment is about 50 mm with a 0.5 m width and a 2.20 m length. Proper rubber beading of good quality to make the joints water tight to arrest the leakage of freshwater into the downstream side is required.

The calculation for the length of downstream apron was performed by using Bligh's formula and is about 21 m. The total length of the apron is about 36 m. Of this, 4.0 m is on the upstream side, 12 m is along the body wall, and 20.0 m is on the downstream side. On the downstream side of the apron, the first 10.0 m is 4 m thick, and for the remaining length, it is 3 m thick as shown in Fig. 5. The length of the apron beyond the downstream side of gate is 32 m, which is adequate according to Bligh's empirical formula. The design aspects of the stilling basin, the floor length, the thickness, and the exit gradient are incorporated as outlined in Garg (1997). The designed cross section of the structure is shown in Fig. 5.

Saltwater Exclusion and Freshwater Storage Dam

The dam S_1 is a saltwater exclusion and freshwater storage vented dam proposed for the lower-most reach (i.e., at a distance of about 7,700 m from the sea) near Kannur, Mangalore (Fig. 3). This dam is supposed to be a low-height dam because the surrounding area is a coastal plain with a low elevation. This dam may act as a saltwater exclusion dam by preventing the entry of backwater into the upper reaches of the river. The height of the dam proposed may be kept at about a +2.0 m level considering the high tide level in the region. The downstream side cutoff wall should be extended to the hard rock level or to the low permeable soil layer (i.e., like clay) if it is below the designed maximum cutoff level on the downstream side to arrest saltwater intrusion (Mayya et al. 2003). This dam serves the dual purposes of preventing saltwater intrusion from the downstream side and of storing fresh water on the upstream

Table 5. Flow Analysis for the Operation of Gates (Kumardhara River)

Serial number	Month	Date	95% Dependable flow m^3/s	95% Dependable flow 10^3 m^3/day	% Flow available	Flow (10^3 m^3)	Flow after 10% release	Cumulative flow (10^3 m^3)	Name of dam to be closed
1	November	21	67.690	5.848	0.292	1.708	1.537	1.537	D11a, D10a
2		22	59.670	5.155	0.292	1.505	1.355	2.892	D10a
3		23	58.760	5.077	0.292	1.480	1.332	4.224	D10a
4		24	54.376	4.698	0.292	1.369	1.233	5.456	D10a
5		25	51.482	4.448	0.307	1.367	1.230	6.686	D9a
6		26	49.952	4.316	0.316	1.362	1.226	7.912	D9a
7		27	46.760	4.040	0.316	1.275	1.148	9.060	D9a
8		28	52.028	4.495	0.316	1.419	1.277	10.337	D9a

J. Hydrol. Eng., 2011, 16(3): 292-302

side. The adjoining well fields along the river bank are protected from saltwater intrusion during January–May for a distance of about 15,000 m upstream. Considering bank storage up to a distance of 600 m on either side of the river with an average aquifer thickness of 20 m and a porosity of 0.3, the volume of the prevention of contamination may be approximated to about 108 Mm^3.

Storage Calculations

The accuracy of the *Google Earth* (2004) image dimensions were verified by field checks. The length parameter was found to be accurate up to 99.89% and the elevation was found to be accurate up to 99%. The corrections for the elevations were applied before the analysis. The storage calculations for the vented dams across the Netravathi and Kumardhara Rivers are given in Tables 2 and 3. These were plotted and the regression coefficients were obtained by fitting the second-order polynomial for the proposed vented dams, D_2, as shown in Fig. 6. The coefficients and the R^2 values obtained are given in Table 4, which indicate encouraging results. The storage versus surface area for the vented dam, D_2, is shown in Fig. 7. The total river water storage estimated for the proposed vented dams is about 102 Mm^3 (Table 4).

Table 6. Flow Analysis for the Operation of Gates (Netravathi River)

Serial number	Month	Date	95% Dependable flow m^3/s	95% Dependable flow $10^3\ m^3/day$	% flow available	Flow ($10^3\ m^3$)	Flow after 10% release	Cumulative flow ($10^3\ m^3$)	Name of dam to be closed
1	November	18	68.278	5.899	0.506	2.985	2.687	2.687	D12b, D11b
2		19	63.888	5.520	0.506	2.793	2.514	5.200	D10b
3		20	62.640	5.412	0.506	2.739	2.465	7.665	D10b, D9b
4		21	67.690	5.848	0.506	2.959	2.663	10.328	D9b
5		22	59.670	5.155	0.506	2.609	2.348	12.676	D9b
6		23	58.760	5.077	0.506	2.566	2.310	14.986	D9b
7		24	54.376	4.698	0.530	2.491	2.242	17.228	D9b, D8
8		25	51.482	4.448	0.555	2.467	2.221	19.449	D8
9	December	26	49.952	4.316	0.560	2.415	2.174	21.622	D7
10		27	46.760	4.040	0.560	2.261	2.035	23.657	D7
11		28	52.028	4.495	0.560	2.516	2.264	25.921	D7
12		29	46.190	3.991	0.891	3.557	3.201	29.122	D7, D5
13		30	44.544	3.849	0.891	3.430	3.087	32.209	D5
14		1	46.544	4.021	0.917	3.686	3.318	35.527	D5
15		2	43.824	3.786	0.917	3.471	3.124	38.651	D4
16		3	42.800	3.698	0.956	3.535	3.181	41.832	D4
17		4	42.260	3.651	0.956	3.490	3.141	44.973	D4, D2
18		5	40.668	3.514	0.974	3.421	3.079	48.052	D2
19		6	40.264	3.479	0.974	3.387	3.049	51.101	D2
20		7	38.926	3.363	0.974	3.275	2.947	54.048	D2
21		8	35.912	3.103	1.000	3.103	2.793	56.841	D2
22		9	35.714	3.086	1.000	3.086	2.777	59.618	D2
23		10	35.248	3.045	1.000	3.045	2.741	62.359	D2, D1
24		11	34.300	2.964	1.000	2.964	2.667	65.026	D1
25		12	32.860	2.839	1.000	2.839	2.555	67.581	D1
26		13	31.988	2.764	1.000	2.764	2.487	70.068	D1
27		14	31.608	2.731	1.017	2.778	2.500	72.568	D1
28		15	31.680	2.737	1.017	2.784	2.506	75.074	D1, S1
29		16	31.092	2.686	1.017	2.732	2.459	77.533	S1
30		17	30.140	2.604	1.017	2.649	2.384	79.917	S1
31		18	17.076	1.475	1.017	1.501	1.351	81.267	S1
32		19	16.788	1.450	1.035	1.501	1.351	82.618	S1
33		20	13.662	1.180	1.035	1.222	1.100	83.718	S1
34		21	13.692	1.183	1.035	1.225	1.102	84.820	S1
35		22	13.128	1.134	1.035	1.174	1.057	85.877	S1
36		23	12.408	1.072	1.035	1.110	0.999	86.876	S1
37		24	13.740	1.187	1.035	1.229	1.106	87.981	S1
38		25	13.662	1.180	1.035	1.222	1.100	89.081	S1
39		26	11.340	0.980	1.035	1.014	0.913	89.994	S1
40		27	10.932	0.945	1.035	0.978	0.880	90.874	S1
41		28	12.816	1.107	1.035	1.146	1.032	91.905	S1

J. Hydrol. Eng., 2011, 16(3): 292-302

Reservoir Operation Guidelines

The demand and flow analysis results indicated that the 95% dependable flow and demand variation over the year is such that the storage of 25.735 Mm^3 is required to fulfill the domestic water requirements for the region. The water deficit period starts on January 19 and continues to June 5 according to the hydrological analysis for the region (Shetkar and Mahesha 2011). The reservoirs need to be filled before the flow in river is drastically reduced (i.e., before the end of November). As there exists only one gauging station, calculations for the flow data are worked for the other reservoirs by using area weighted averages (Tables 5 and 6). Accordingly, the flow data are generated for all the proposed dam locations. For most of the rivers, the exact environmental water requirements are unknown (Smakhtin et al. 2007), and a minimum allowance of 20% of the existing flow may be sufficient (Rakeshkumar et al. 2005). For the efficient operation of the vented dams during the filling period and to restrict any further degradation of the ecosystem, 10% of the existing flow was let off downstream for the present analysis as required by the National Water Development Agency (NWDA) guidelines (NWDA 1993). Knowing the flow and keeping a provision for the minimum tailwater flow (i.e., the environmental requirements), calculations for the filling duration for each reservoirs were computed (Table 7). The gates of the dams were closed starting with the uppermost dam. The total time required to fill of all the reservoirs was 41 days commencing November 18. The entire process of the operation of the gates was simulated by considering the inflow, storage, and release over time.

The reservoir operations are scheduled for normal flow conditions and any deviations attributable to flash floods need to be handled as special cases. The stage-storage relationship (Fig. 8) and the depth of flow-inflow relationship (Fig. 9) developed are the two operating curves that will assist with operating the reservoirs. From these relationships, the projected water storage may be estimated. In the present investigation, the water demand, the deficit and surplus periods, and the gate operation dates were devised on the basis of these relationships.

Aquifer Characterization

The results of the vertical electrical sounding (VES) tests are given in Table 8. The tests were conducted near the well locations and are

Table 7. Schedule for the Operation of Gates

Serial number	Dam name	Date to be closed	Number of days required to fill
1	D12b	18th November	01 Day
2	D11b	18th November	01 Days
3	D10b	19th November	02 Days
4	D9b	20th November	05 Days
5	D11a	21st November	01 Days
6	D10a	21st November	04 Days
7	D9a	25th November	04 Days
8	D8	24th November	02 Days
9	D7	26th November	04 Days
10	D5	29th December	03 Days
11	D4	02nd December	03 Days
12	D2	04th December	07 Days
13	D1	10th December	06 Days
14	S1	15th December	14 Days

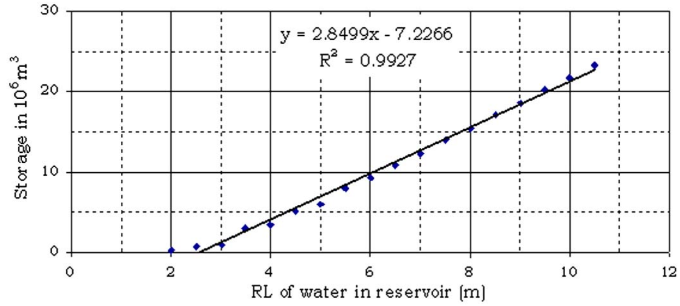

Fig. 8. Stage-storage capacity curve for vented dam at D_2

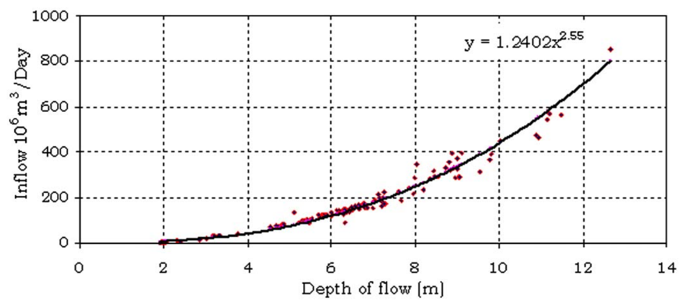

Fig. 9. Depth-inflow curve for dam D_2

numbered accordingly. The transmissivity and hydraulic conductivity estimations were made according to Nath et al. (2000). The test results indicated that the aquifer thickness in the study area ranges from 18 to 30 m. At three locations (i.e., near the wells W_4, W_{19}, and the river location F_{01}), the saltwater intrusion effect in the form of reduced resistance was observed. It was predicted that the depth to the saltwater could be 40, 16, and 24 m, respectively. The probable fracture zones were also detected at depths ranging from 25 to 55 m. In most of the places, the litholog consists of soil, laterite, clay, and gneiss. The details of the parameters evaluated, including the hydraulic conductivity, are given in Tables 8 and 9.

The hydraulic conductivity estimated from the soil analysis ranged from 150 to 248 m/day. The pumping test was also conducted to evaluate the hydraulic conductivity (285 m/day), transmissivity (1,440 m^2/day), and specific yield (0.1). The details of the pumping tests were reported elsewhere (Shetkar 2009). The results indicated that the hydraulic conductivity and the transmissivity agree well with the modified Theis method for unconfined aquifers and recovery analyses.

From the preceding analyses, it is evident that the area is a shallow, unconfined aquifer with good groundwater potential. The hydraulic conductivity values were generally high indicating the pervious nature of the aquifer. Even though this will ensure adequate groundwater storage in the well fields adjoining the reservoir, the effect of saltwater intrusion from the river during the summer season may also have greater impact on the fresh water aquifers.

Multiple Regression Analysis

The results of the multiple variable regression between the water level in the wells and the other parameters are given in Table 10. The analysis was performed at a 95% confidence interval. The summary of the statistical analysis gives a very good value for the correlation coefficient R^2, which is standardized, is equal to 0.992, and

J. Hydrol. Eng., 2011, 16(3): 292-302

Table 8. Observations and Results from the Electrical Resistivity Tests

Schlumberger configuration (AB/2)	Resistance offered by the aquifer for various depths (Ohm)								
	ER_1/W_{02}	ER_2/W_{04}	ER_3/W_{06}	ER_4/W_{12}	ER_5/W_{14}	ER_6/W_{16}	ER_7/W_{17}	ER_8/F_{01}	ER_9/W_{19}
10 m	205.90	366.30	107.16	342.70	62.30	148.20	78.20	28.80	37.20
20 m	713.40	99.10	435.00	163.60	46.90	426.30	341.10	41.70	5.20
30 m	671.60	152.30	1194.20	43.50	49.70	771.28	205.20	6.20	18.60
40 m	1168.10	106.60	653.20	533.80	62.80	1149.20	263.70	332.80	87.90
50 m	911.40	39.20	313.60	245.00	156.80	686.00	1205.40	225.40	9.80
60 m	1043.40	70.50	352.50	310.20	56.40	1128.00	98.70	—	1283.10
70 m	—	538.50	523.30	215.40	353.97	1569.70	1385.10	—	246.20
80 m	—	—	1024.60	2692.10	—	—	—	—	—
Overburden (thickness)	12 m	17 m	16 m	12 m	16 m	12 m	12 m	16 m	18 m
Depth to salinity zone	—	40 m	—	—	—	—	—	24 m	16 m
Depth of probable fractures	25, 40 m	32, 40 m	40, 48, 56 m	25, 40, 55 m	32, 50 m	40, 50 m	50, 60 m	40 m	25, 40, 55 m
Litholog	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
	—	Sand, silt	—	—	—	—	—	Sand, silt	Sand, silt
	Laterite	Laterite	Laterite	Laterite	Laterite	Laterite	Laterite	Laterite	Laterite
	Clays	Clays	Clays	Clays	Clays	Clays	Clays	Clays	Clays
	Gneiss	Gneiss	Gneiss	Gneiss	Gneiss	Gneiss	Gneiss	Gneiss	Gneiss

Table 9. Analysis and Results from the Electrical Resistivity Tests

	ER_1/W_{02}	ER_2/W_{04}	ER_3/W_{06}	ER_4/W_{12}	ER_5/W_{14}	ER_6/W_{16}	ER_7/W_{17}	ER_8/F_{01}	ER_9/W_{19}
Aquifer resistance for 10 m thickness (ρ_0) Ohm	205.90	366.30	107.16	342.70	62.30	148.20	78.20	28.80	37.20
Transverse resistance, Ohm	2059.00	3663.00	1071.60	3427.00	623.00	1482.00	782.00	288.00	372.00
Predicted transmissivity m^2/day	2649.29	4060.01	1780.87	3852.45	1386.33	2141.82	1526.17	1091.70	1165.57
Hydraulic conductivity (m/day)	**264.93**	**406.00**	**178.09**	**385.24**	**138.63**	**214.18**	**152.62**	**109.17**	**116.56**
Conductiviy of water	180.00	83.00	145.00	99.00	245.00	243.00	67.00	326.00	135.00
Pore resistance (ρ_0) Ohm	55.56	120.48	68.97	101.01	40.82	41.15	149.25	30.67	74.07
Formation factor = Aquifer resistance/pore resistance	3.71	3.04	1.55	3.39	1.53	3.60	0.52	0.94	0.50
Hydraulic conductivity (m/day)	**359.77**	**298.01**	**160.14**	**330.70**	**157.59**	**350.04**	**64.62**	**103.10**	**62.60**

Table 10. Results from the Multiple Regression Analysis

Sl mumber	Regression coefficients		P value	Standard error	95% Confidence range		t-statistics
					Lower	Upper	
1	b0	1.357	0.000858	0.399	0.569	2.144	3.403
2	b1	0.506	0.03364	0.236	0.03973	0.973	2.144
3	b2	0.311	0.00166	0.09709	0.119	0.503	3.204
4	b3	−0.000331	0.629	0.000682	−0.00168	0.00102	−0.485
5	b4	0.920	1.5284E − 146	0.00791	0.904	0.935	116.20
6	b5	−0.814	1.95263E − 60	0.02925	−0.871	−0.756	−27.82

is assumed to be 0.991 for prediction. The standardized coefficients are better measures of sensitivity than common statistical parameters. The standard error is minimum for all the regression coefficients. The developed model was validated for the remaining data sets and the results indicated that the R^2 value for the best fit line is 0.9905, and therefore, the performance is good (Fig. 10). The results of the sensitivity analysis (i.e., the regression coefficients in Table 10) indicated that the reduced level of the ground surface near the observation well was the most sensitive parameter, and the water table elevation varied directly with the variation in the ground surface level. The second-most sensitive parameter was the depth of the well. The water table elevation varied inversely with the depth of the well. The other sensitivity parameters were the distance from the reservoir and the water level in the reservoir. The least sensitive parameter was the hydraulic conductivity because the area of consideration had more or less the same range of hydraulic conductivity.

The model was used to predict the water levels in the observation wells when the reservoir water level varied between zero and 7.0 m (i.e., the maximum level). The predicted water levels in the wells are shown for these two cases in Fig. 11. The figure gives useful information about the effect of induced recharge attributable to the vented dam on the surrounding well fields. It was also estimated that the surface storage by the vented dams may induce groundwater recharge to the extent of about 108 Mm^3. The bank storage during the surplus period may be released to the river during the deficit period and in long run, may help in maintaining the flow of the river even during the summer period.

J. Hydrol. Eng., 2011, 16(3): 292-302

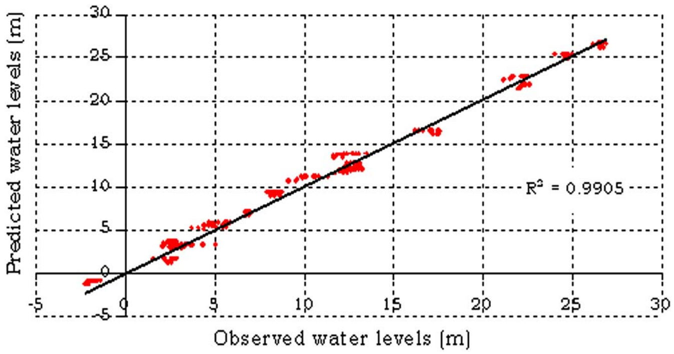

Fig. 10. Model verification—predicted water level versus observed water level

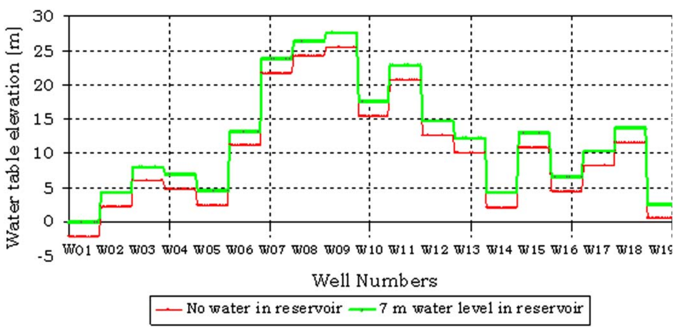

Fig. 11. Model prediction for minimum and maximum reservoir water levels

Summary and Conclusions

The proposed river basin development by a series of vented dams, 14 in number, across the Netravathi River was estimated to provide about 102 Mm^3 of surface water storage within the river banks, which is about 1% of the average annual yield of the river. The storage calculations with the depth and area relationships were established precisely at each vented dam location. The timing and quantity of surface storage were determined on the basis of a 95% dependable flow in the river, the height of the dam, and the water spread area. The storage capacity and the minimum flow to be released on the basis of river flow were used to establish an operation policy that included the timing of the closure of the gates of the vented dams. It was estimated that the closure of the gates of the vented dams at the uppermost location must begin by November 18 and should proceed downstream. Filling of all 14 reservoirs may require about 41 days. This schedule may be useful information for the stakeholders of the river basin for the wise use of water. The opening of the gates, however, depends on the onset of the monsoon in the region, which is usually during the first week of June. The electrical resistivity survey conducted in the basin indicated that the aquifer is shallow, unconfined in nature, with depth ranging from 18 to 30 m. The tests of aquifer parameters indicated that the aquifer has good groundwater potential with hydraulic conductivity ranging from 62.6 to 406 m/day. The groundwater simulation predicted that the water table fluctuation in the well fields was about 30% of the reservoir level fluctuations. It was estimated that the surface storage by the vented dams may induce groundwater recharge into the adjoining well fields which, over time, may assist in transforming the seasonal river into a perennial river.

The tidal nature of the river is affecting the adjoining well fields up to a distance of about 22,000 m along the river course from the sea during April–May. As a preventive measure, the vented dam at the lowermost reach (i.e., about 7,000 m from the sea) will also act as a saltwater exclusion dam, restricting the entry of saltwater upstream. The results from the investigation may be useful for tropical, seasonal river basin development programs established to meet the challenges of the increasing demand for fresh water.

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