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# AQUIFER CHARACTERISTICS OF THE INTAKE WELLS ON DAMODAR RIVER NEAR RANIGANJ, WEST BENGAL, INDIA

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**Abstract:** Since the British period the Damodar River has been the main source of water supply in Raniganj, Asansol and Durgapur area of West Bengal State in India. Afterwards since nineties, the drinking water demand was increasing rapidly with the population rise. There has been a considerable scarcity in water supply especially in summer months. Recently, to augment the water supply situation, considering future demand and population rise, a hydro-geological study has been carried out during dry period at different locations on the river bed. The observation were undertaken in and around the existing pumping area on Damodar river located in the southern part of Raniganj. This research is aimed at assessing the yield test at exploratory bored wells to find out the minimum quantity of water available to design radial collector wells at three different locations with respect to their drawdown, transmissibility and storativity. Secondly, a field method for the mechanical analysis of bore well log material of the study area was attained to determine the material size and the required artificial gravel packing. On the basis of the field study and investigations, data collection, collasion and analysis, this paper has been prepared highlighting the characteristics of the aquifer, the location of the intake wells and yield of the bed.

**Keywords:** Damodar river, aquifer, bore well, yield test, drawdown, transmissibility, storativity, sieve analysis.

## 1. INTRODUCTION

River Damodar is a perennial river receiving water from Chota Nagpur Plateau of Bihar State in India. The river inundates vast area of West Bengal State and the industrial belt of Durgapur Asansol area, receives water from its source. In the study area, the Gondwana Sandstone of lower to middle Palaeozoic age occurs with intercalations of shale and coal seams and forms the basement. Over the basement, coarse sands and gravels with minor fraction of medium to fine sand are overlain for a thickness of 10.5-12.0 m which are generated through weathering of sandstone occur in the upstream of Damodar river. Due to existence of a perennial channel in the river bed, many local establishments in the area draw water from the river. The existence of a permanent aquifer in Damalia area near Asansol is proven, and it is feasible to draw sub-surface water from river during the lean months of the year.

The availability of quantum of 10 million gallon per day (MGD) of water at a time needs to be confirmed by actual testing during lean period. Thickness of the porous overburden has been confirmed through test drilling undertaken in the river valley within 300 m upstream and downstream of the proposed intake wells site. The basement rock i.e. Gondwana Sandstone does not form prolific aquifers. Furthermore, because of artificial disturbance in the subsurface because of excavation for coal mining, construction of bore wells are also not feasible in many areas. Consequently, since the British period, the Damodar river has been the main resource of water supply in Raniganj and adjoining areas. Formerly due to the absence of pollution in the upper catchments, as also because of abundance in river discharge, the surface water of Damodar river was directly pumped for water supply and it was being supplied after filtration and disinfection. Afterwards since eighties, to tackle the water scarcity problem, tube wells were also constructed tapping the top coarse sand with gravel aquifers deposited on the Gondwana basement. However, there has been a considerable scarcity in water supply especially in the summer months.

A collector well lowers the water table and thereby induces infiltration of surface water through the bed of the river to the well [1-2]. In this manner greater supplies of water can be obtained than would be available from groundwater alone. The sensitivity features of drawdown to aquifer parameters under constant rate pumping conditions in one dimensional flow systems were examined to different characteristics of transmissivity and storativity by [3].

Recently to mitigate the increasing drinking water demand with population rise as also keeping in view of future demand and population rise, water availability depending on the aquifer supply through infiltration galleries in tiers and two collector wells has been envisaged.

It is, therefore, thought prudent to check for two numbers 5 MGD each capacity collector wells adjacent to each other at about 150 m apart so that the drawdown curve for one do not disturb the flow of the other. It is also obvious that initially one or two 5 MGD capacity collector well/s may be constructed depending upon the availability of water. Individual wells will be connected to the shore by two separate approach bridges each about 144 m long. The approach bridge may be provided with central approach walkway and 0.45 m diameter delivery pipe.

As such ground water occurs under unconfined condition in the unconsolidated coarse sand with gravel aquifer and it gets recharged directly from the surface flow in Damodar. The width of the river section is 600 m which is abutted on both the sides by the Gondwana rocks evident from the rock exposures. The top sand with gravel layer is highly porous and because of perennial flow in the river throughout the year, the depth to water level even in the dry portions in the area remain within maximum 0.5-0.7 m below ground level during peak summer. The pumping tests were conducted to find out the aquifer parameters, also envisage very low drawdown and negligible radius of influence. However, the problems of clogging of strainer with clays especially during rainy months and monitoring of proper gravel packing, sand filling etc may be taken properly.

## 2. EXPERIMENTATION

Two categories of experiments were carried out: (a) yield test at exploratory bore wells (as shown in the Fig. 1) on Damodar river near Raniganj, West Bengal and (b) mechanical analysis of bore well log material of the study area by using a nest of sieves (IS standard) and mechanical vibrating shaker combination at the Fluvial Hydraulics Laboratory of the School of Water Resources Engineering, Jadavpur University, Kolkata, West Bengal, India.

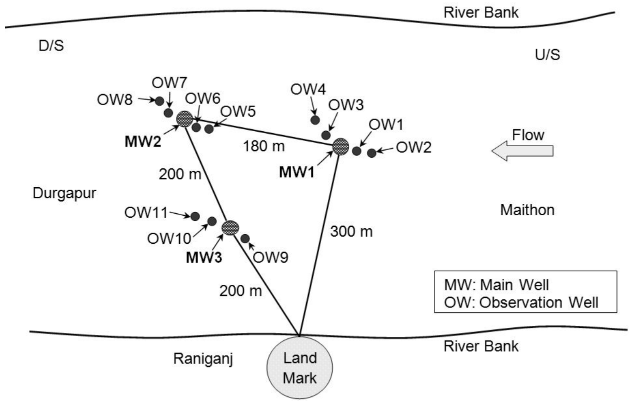


**Fig. 1** Layout of the main and observation wells

## 3. METHODOLOGY

On the very first day, beside the main bore well (MW1) situated 300 m from the land mark near Raniganj there were four observation wells (OW) were constructed 0.5 m (OW1), 2.1 m (OW2), 5.1 m (OW3) and 10.2 m (OW4) away from the main well 1 (Fig. 1). Only one pump was used at the study area. Simultaneously data collection during the yield test of the three pumping wells was performed in consecutive three days. For the other main bore well (MW2) 400 m from the land mark, there were four observation wells situated 0.5 m (OW5), 3.45 m (OW6), 2.25 m (OW7) and 8.5 m (OW8) away from the main well 2 (Fig. 1). For the third main bore well (MW3) situated 200 m from the land mark, there were three observation wells situated 0.64 m (OW9), 2.13 m (OW10), 7.1 m (OW11) away from the main well 3 (Fig. 1).

All the distances of the observation wells were measured with respect to the respective main wells. For all the main wells the transmissivity or transmissibility coefficient (the volume of water that can be transmitted horizontally through a unit thickness of an aquifer under a hydraulic gradient of 1) was taken much lesser than the actual value considering sufficient factor of safety in each case. Similarly, for all the main wells, the storativity or storage coefficient (the volume of water released from a saturated confined aquifer per unit cross-sectional area of the aquifer column, per unit decrease in hydraulic head) was taken as at least 0.33 or 33% of the actual value taking sufficient factor of safety [4]. Constant well discharge ($Q$) was measured by a V-notched weir placed 50 m away from the main well. Then the well discharge was calculated using the Eq. 1.

$$Q = \frac{8}{15} C_d \tan\frac{\theta}{2} \sqrt{2g} H^{5/2} \quad (1)$$

where, coefficient of discharge $C_d$ was taken equal to 0.5; the vertex angle ($\theta$) was equal to $90^o$ and $H$ is the mean depth of water over the V-notched weir crest.

Transmissivity ($T$) and storativity ($S$) were determined from Copper-Jacob method of solution [5-6] by using Eq. 2 and Eq. 3, respectively.

$$T = \frac{2.3Q}{4\pi\Delta s} \quad (2)$$

$$S = \frac{2.25Tt_0}{r^2} \quad (3)$$

where $r$ is the distance of the observation well from the discharging well, $\Delta s$ is the drawdown difference log cycle of $r$, and $t_o$ is the time for zero drawdown taken from the time-drawdown curve. Calibration of the V-notched weir was done at site with the help of volumetric tank and the stop watch. Entrance velocity $V_e$ was considered equal to 3 mm/sec. Average value of the discharge was considered during the constant pumping. The diameters of the main wells are 20 cm each having a depth of 11.55 m whereas the diameters of the observation wells are 4 cm each having a depth of 6 m. Total 15 observation wells were constructed but four of them were found faulty.

The high flow level (HFL), low water level (LWL) and average bed level of the river were presumed +77.0 m, +71.0 m, and +70.0 m, respectively. Operating platform level of pumps inside the intake well was +78.5 m. River level of the left side of the river bank was found +77.0 m.

It is proposed to install four numbers (three working and one stand-by) vertical turbine type raw water pumps of 360 $m^3$/hr capacity each with estimated total of head of 20 m. The column heights of the pumps were 12 m approximately. The total power requirement for each intake well shall be assessed.

## 4. INPUT AND OUTPUT DATA

In day one there was a constant discharge during the 135 min constant pumping. Arithmetic mean water level in the V-notched weir ($H$) was measured as 18.5 cm. Well discharge ($Q$) was 62.56 $m^3$/hr. In second day there was a constant discharge during the 105 min constant pumping. Arithmetic mean water level in the V-notched weir ($H$) was measured as 17.95 cm. Well discharge ($Q$) was 58 $m^3$/hr. In day three, there was also a constant discharge (= 59.43 $m^3$/hr) during the 174 min constant pumping. Arithmetic mean water level in the V-notched weir ($H$) was measured as 18.12 cm. Fig. 2 and Fig. 3 illustrate the time-drawdown curves i.e. the variation of river water level (RWL) and V-notched weir reading during the three day analysis, respectively.

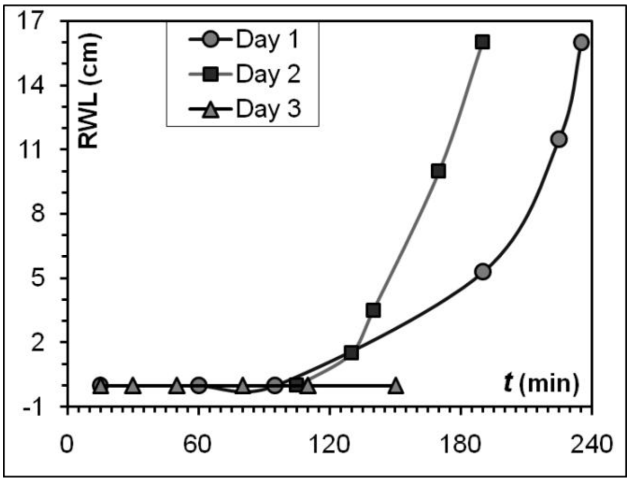


**Fig. 2** Water level in the river

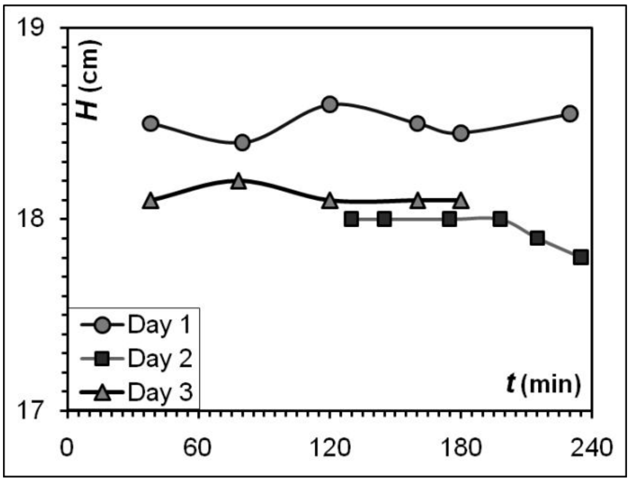


**Fig. 3** V-notched weir data

In day one, the pump was started (7:20 am IST) 30 min after taking the initial data and then the drawdown readings were taken in MW1 and four observation wells (OW1-OW4) as shown in Fig. 4. Table 1 illustrates the initial reading in day one. For all the wells magnitudes of $t_0$ and $\Delta s$ have been determined from Fig. 4. Then the values of $T$ and $S$ were calculated using Eq. 2 and Eq. 3 as shown in Table 2. The pump was stopped after 135 min (9:35 am IST) and then an interval of 15 min was given for recuperation. After stopping the pump, within 1 min all wells reached steady state having 2 cm constant drawdown for the main well and 1 cm constant drawdown for the observation wells 3 and 4.

**Table 1** Initial data (before pumping) for MW1

| | **Well used** | ***r*** (m) | ***t***(min) | ***h*** (cm) |
|---|---|---|---|---|
| Day1 | MW1 | — | 0 | 135.0 |
| | OW1 | 0.5 | 0 | 19.8 |
| | OW2 | 2.1 | 0 | 23.0 |
| | OW3 | 5.1 | 0 | 105.0 |
| | OW4 | 10.2 | 0 | 147.0 |
| Day2 | MW1 | — | 180 | 145.1 |
| | OW1 | 1.0 | 180 | 53.1 |

**Note:** $h$ is the depth of water from datum (considering RWL)

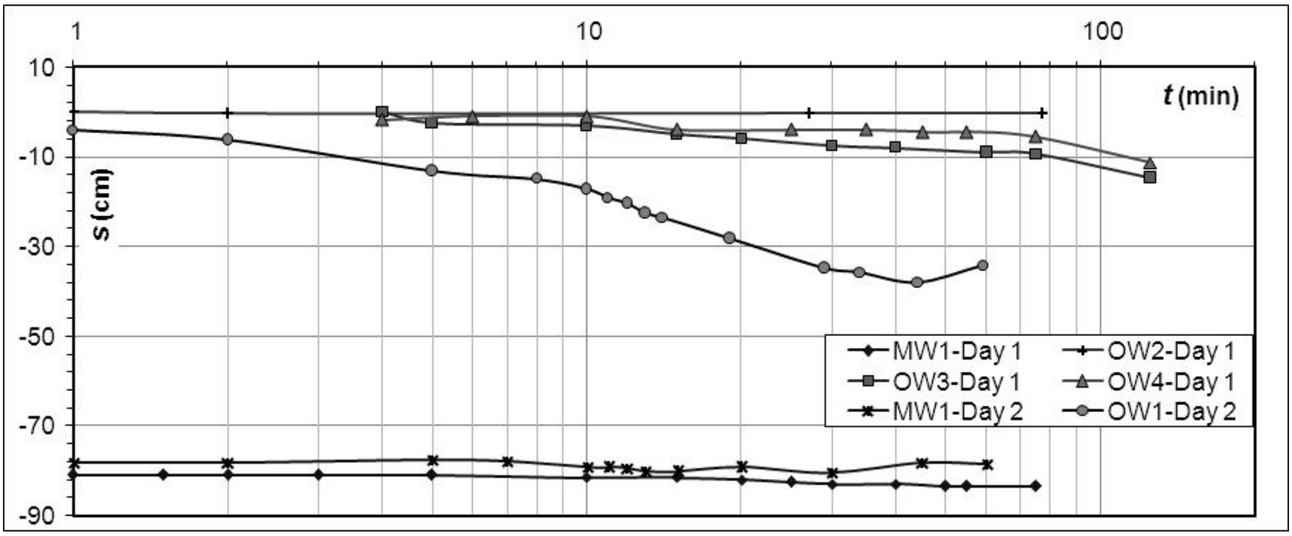


**Fig. 4** Drawdown in main well 1 and four observation wells on Days 1 and 2

**Table 2** Measured and calculated data (after pumping) for MW1

| | $t_0$ (min) | $s$ (cm) | $\Delta s$ (cm) | $r$ (m) | $Q$ ($m^3$/hr) | $T$ ($m^2$/day) | $S$ (-) |
|---|---|---|---|---|---|---|---|
| MW1 | | 83.50 | | 0.0 | 62.56 | | |
| OW3 | 7.5 | 14.71 | 10.8 | 5.1 | 62.56 | 2545.78 | 1.1470 |
| OW4 | 9.0 | 11.31 | 9.5 | 10.2 | 62.56 | 2894.16 | 0.3911 |
| $\left(\sum OW_i\right)/n$ for $i = 3, 4$ | | | | | | 2719.97 | 0.7690 |
| OW1 | 70.0 | | 142.0 | 1.0 | 62.56 | 193.62 | 21.1800 |

In day two, the pump was started (8:57 am IST) 12 min after taking the initial data and then the drawdown readings were taken in MW2 and four observation wells (OW5 to OW8) as depicted in Fig. 5. Table 3 illustrates the initial reading in day one. For all the wells magnitudes of $t_o$ and $\Delta s$ have been determined from Fig. 5. Then the values of $T$ and $S$ were calculated using Eqs. 2-3. The pump was stopped after 105 min (10:42 am IST). Summary of observation data of drawdown, transmissivity and storativity of MW2 and its respective observation wells are listed in Table 4.

**Table 3** Initial data (before pumping) in Day 2

| **Well used** | $r$ (m) | $t$ (min) | $h$ (cm) |
|---|---|---|---|
| MW2 | — | 0 | 125.26 |
| OW5 | 0.50 | 0 | 71.26 |
| OW6 | 3.45 | 0 | 38.26 |
| OW7 | 2.25 | 0 | 64.26 |
| OW8 | 8.50 | 0 | 45.26 |

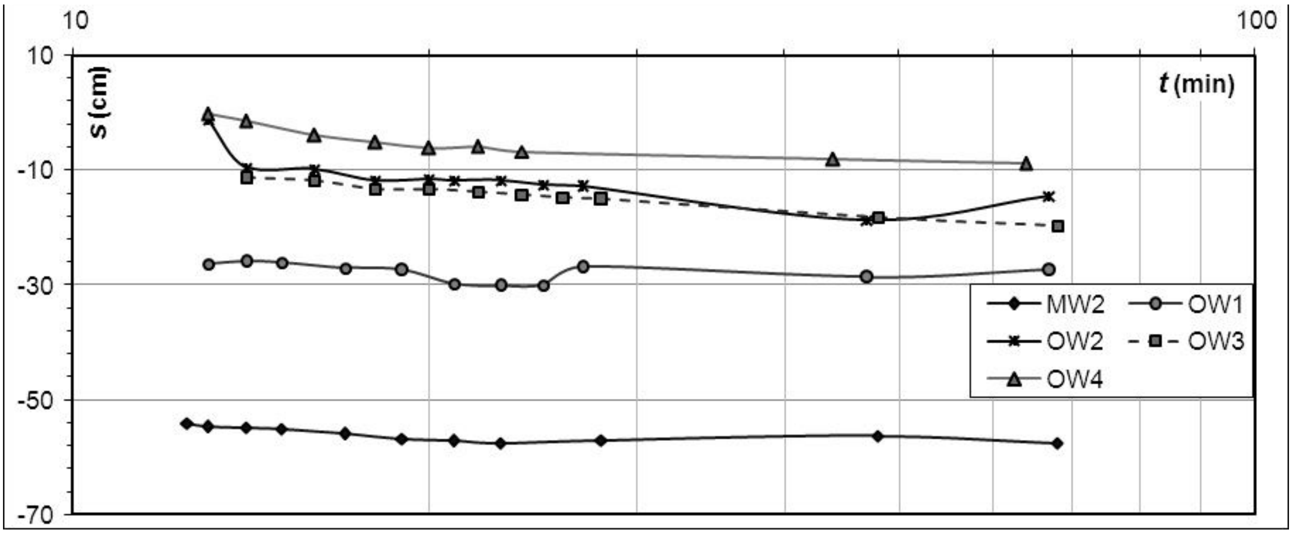


**Fig. 5** Drawdown in MW2 and four observation wells on day 2

**Table 4** Measured and calculated data (after pumping) in day two

| | $t_0$ (min) | $s$ (cm) | $\Delta s$ (cm) | $r$ (m) | $Q$ ($m^3$/hr) | $T$ ($m^2$/day) | $S$ (-) |
|---|---|---|---|---|---|---|---|
| MW2 | | 57.57 | | | | | |
| OW5 | 0.1 | 30.07 | 11.50 | 0.50 | 58 | 2216.56 | 1.380 |
| OW6 | 6.0 | 18.58 | 20.00 | 3.45 | 58 | 1274.52 | 1.000 |
| OW7 | 1.5 | 19.53 | 11.50 | 2.25 | 58 | 2216.56 | 1.020 |
| OW8 | 2.5 | 8.78 | 6.25 | 8.50 | 58 | 4078.47 | 0.220 |
| $\left(\sum OW_i\right)/n$ for i = 5, ..., 8 | | | | | | 2446.52 | 0.905 |

In day three, the pump was started (7:31 am IST) 1 min after taking the initial data and then the drawdown readings were taken in MW3 and four observation wells (OW9-OW11) as displayed in Fig. 6. Table 5 shows the initial reading in day one. For all the wells magnitudes of $t_o$ and $\Delta s$ have been determined from Fig. 6. Then the values of $T$ and $S$ were calculated using Eq. 2 and Eq. 3. The pump was stopped after 174 min (10:25 am IST). Summary of observation data of drawdown, transmissivity and storativity of MW3 and its respective observation wells are listed in Table 6. From Fig. 6 it may be noted that as the pump was stopped for 30 s (from 10:25 am 10:25:30 am) for checking the recovery of the drawdown. So in 30 s MW3 recovered of 55.2 cm which was 97% of the depth of water from the bed (RWL) at that time.

**Table 5** Initial data (before pumping) in day 3

| **Well used** | $r$ (m) | $t$ (min) | $h$ (cm) |
|---|---|---|---|
| MW3 | — | 0 | 110.5 |
| OW9 | 0.64 | 0 | 51.9 |
| OW10 | 2.13 | 0 | 48.5 |
| OW11 | 7.10 | 0 | 59.5 |

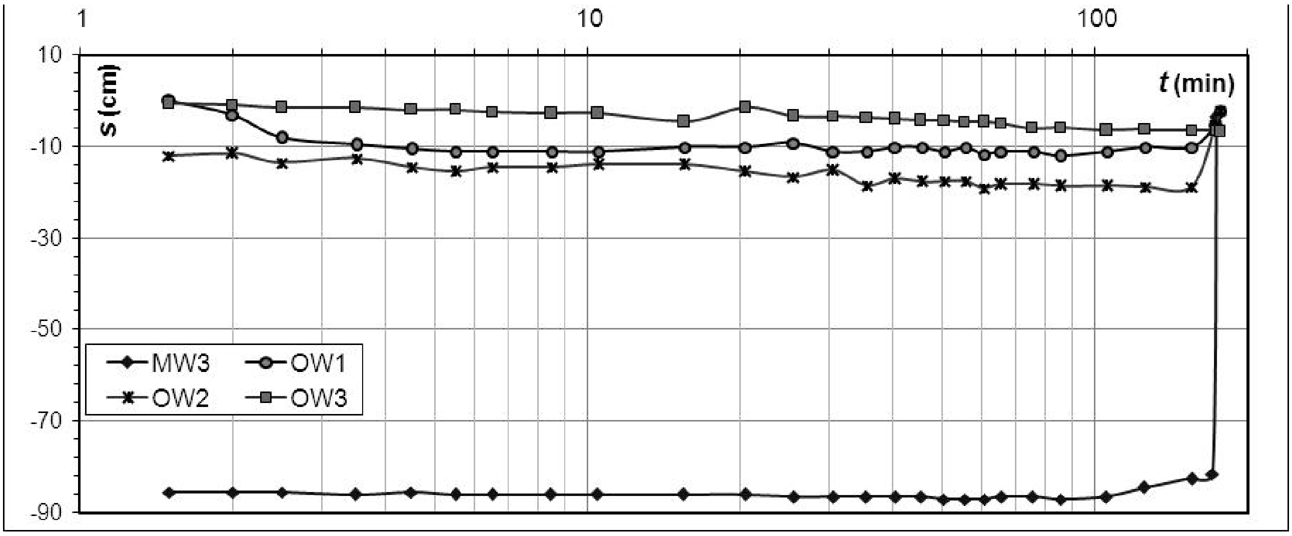


**Fig. 6** Drawdown in main well 3 and four observation wells on Day 3

**Table 6** Measured and calculated data (after pumping) in day three

| | $t_0$ (min) | $s$ (cm) | $\Delta s$ (cm) | $r$ (m) | $Q$ ($m^3$/hr) | $T$ ($m^2$/day) | $S$ (-) |
|---|---|---|---|---|---|---|---|
| MW3 | | 87.00 | | 0.00 | 59.43 | | |
| OW9 | 0.380 | 12.20 | 5.0 | 0.64 | 59.43 | 5223.78 | 7.57 |
| OW10 | 0.005 | 18.80 | 4.4 | 2.13 | 59.43 | 5936.11 | 0.01 |
| OW11 | 1.600 | 6.70 | 3.5 | 7.10 | 59.43 | 7462.58 | 0.37 |
| $\left(\sum OW_i\right)/n$ for $i = 9, ..., 11$ | | | | | | 6207.49 | 2.65 |

## 5. DESIGN OF RADIAL COLLECTOR WELL AT THREE DIFFERENT LOCATIONS WITH RESPECT TO THEIR TRANSMISSIVITY

**Main well 1:** The collector well was located at a distance of 300 m from the land mark and to be designed such a way to fulfill a demand of 5 MGD or 22700 $m^3$/day. From Table 2, average transmissivity ($T$) and storativity ($S$) of the location were found 2719.97 $m^2$/day and 0.76905, respectively. Now, assuming, $T$ = 2500 $m^2$/day and $S$ = 0.33 (considering sufficient factor of safety), from the Theis non-equilibrium equation [4,7-8] i.e. Eqs. 4 and 5, the dummy variable $u$ was calculated using Eq. 4 where $r$ is the radial distance from the pumped well and $t$ is the time since beginning of pumping. Form Theis type curve, the well function $W(u)$ was determined equal to 1.2 for $u = 0.22$. The value of $s$ was computed 0.86752 using Eq. 5.

$$u = \frac{r^2 S}{4Tt} \tag{4}$$

$$s = \left(\frac{Q}{4\pi T}\right) W(u) \tag{5}$$

The cumulative effect of both image pumping well (Fig. 7) (which simulates the barrier and recharge boundaries [4,8]) is 1.74 m and the total drawdown allowable for the real collector well is only 3.2–1.74 = 1.46 m (adopting maximum drawdown as 3.2 m). From Theis equation, $W(u)$ is equal to 2.0264. Form Theis type curve, $u$ = 0.0833, $r$ = 615.3343 m, for $W(u)$ = 2.0264.

**Fig. 7** Image pumping well

The design of the well screen consists of the length of the screen, its location, percentage of open area, size and shape of this slots and selections of the screen material. The optimum length of the screen was chosen in relation to the aquifer thickness, available drawdown and stratification of the aquifer. In homogeneous artesian aquifer about 70 to 80% of the aquifer thickness was screened. Length of the screen or radial strainer or lateral ($L_s$) was equal to 770 m

$$\boldsymbol{Q = \left(pdnL_s p\right)V_e} \quad (6)$$

where, $p$ is the effective percentage of opening, equal to 12%; $V_e$ is the entrance velocity, equal to 3 mm/sec; $d$ is the lateral diameter, equal to 300 mm; $n$ is the number of laterals spaced equally around the circumference of the caisson; and $Q$ is the discharge, equal to 22700 $m^3$/day.

Using Eq. 6, the value of $n$ was estimated as 1.212. Hence, the total length of strainer is 933.4 m (=770×1.212). Therefore, 36 strainers of length 30 m each giving a total length of strainer of 1080 m may be provided in the design. Thereby the design will be safe.

**Main well 2 :** The collector well was located at a distance of 400 m from the land mark and to be designed such a way to fulfill a demand of 22700 $m^3$/day. Average values of $T$ and $S$ of the location were equal to 2446.518 $m^2$/day and 0.905, respectively. Assuming, $T$ = 2000 $m^2$/day and $S$ = 0.33 (considering sufficient factor of safety). From Eq. 4 the value of $u$ was calculated 0.275. Form Theis type curve, well function $W(u)$ = 0.98 for $u$ = 0.275. Therefore, s = 0.8856 m.

The cumulative effect of both image well is 1.77 m and the total draw-down allowable for the real collector well is only 3.2–1.77 = 1.43 m (adopting maximum drawdown as 3.2 m). From Theis equation, $W(u)$ = 1.5812.

Form Theis type curve, $u$ = 0.1428, $r$ = 720.6 m, for $W(u)$ = 1.5812 . The length of the screen ($L_s$) is equal to 900.757 m for $p$ = 0.12, $V_e$ = 3 mm/sec, $d$ = 300 mm and $Q$ = 22700 $m^3$/day. Therefore using Eq. 6 the value of $n$ was calculated 1.031. Hence, the total length of strainer is (900.757×1.031) 928.68 m. Therefore, 36 strainers of length 30 m each giving a total length of strainer of 1080 m may be provided in the design. So the design is safe.

**Main well 3 :** The collector well is located at a distance of 200 m from the land mark and to be designed such a way to fulfill a demand of 22700 $m^3$/day. Average values of $T$ and $S$ of the location were chosen 6207.49 $m^2$/day and 2.65, respectively. Assuming $T$ = 3600 $m^2$/day and $S$ = 0.33 (considering sufficient factor of safety), $u$ = 0.1528. Form Theis type curve, $W(u)$ = 1.65 for $u$ = 0.1528. Therefore $s$ = 0.8284 m.

The cumulative effect of both images well is 1.656 m and the total drawdown allowable for the real collector well is only 3.2–1.656 =1.544

m (adopting maximum drawdown as 3.2m). From Theis equation, $W(u) = 3.07$.

Form Theis type curve, $u = 0.0313$, $r = 452.2665$ m, for $W(u) = 3.07$. The length of the screen ($L_s$) 565.334 m where $p = 0.12$, $V_e = 3$ mm/sec, $d = 300$ mm and $Q$ = discharge = 22700 $m^3$/day. Therefore, the magnitude of $n = 1.6445$. Hence, the total length of strainer is (565.2334×1.6445) 930 m. Therefore 36 strainers of length 30 m each giving a total length of strainer of 1080 m may be given in the design. So the design is safe.

## 6. MECHANICAL ANALYSIS OF DATA

A mechanical analysis was carried out with the bore well log material and from that the material size was determined and also the artificial gravel packing size was determined. The effective size ($d_{10}$) is the size corresponding to 10% of the material being finer and 90% coarser and an index of fineness of the soil. The uniformity coefficient ($C_u$) is the average slope of the grade curve between 10% and 60% sizes and is given by $C_u = d_{60}/d_{10}$ and gives an idea of the grading or particle size distribution in the material. Lower values ($C_u < 2$) indicate more uniform material or poor grading and higher values indicate well graded material. Another term to indicate the effective distribution of grain size is the range sizes ($C_r$) [Eq. 7] defined on the basis of the mean slope of the grain size curve.

$$C_r = 2\log_{10}\left(d_{100} / d_0\right) \qquad (7)$$

From the grading curve (Fig. 8), $d_{10} = 0.75$ mm, $d_{30} = 1.50$ mm and $d_{60} = 2.50$ mm. Therefore the uniformity coefficient ($C_u$) = 3.33 > 2, which indicates aquifer materials are well graded and $C_r = 5.76$.

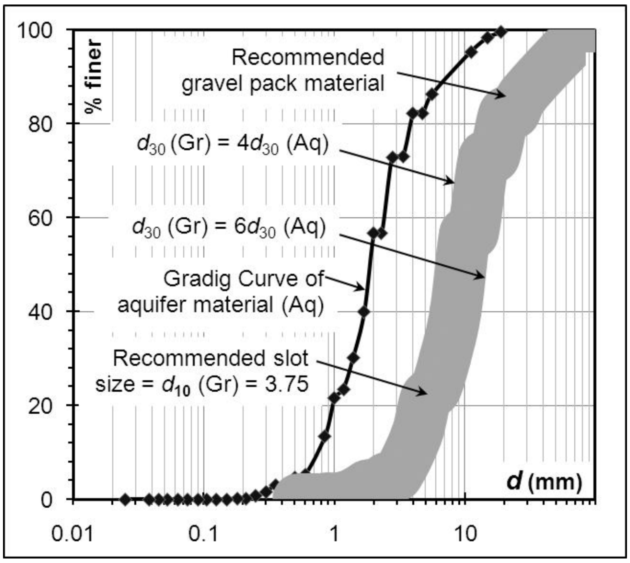


**Fig. 8** Grading curve of the aquifer material (Aq) and design of gravel (Gr) pack

The volume of water, expressed as a percentage of the total volume of the saturated aquifer, that can be drained by gravity is called the specific yield and the volume of water retained by molecular and surface tension forces, against the force of gravity, expressed as a percentage of the total volume of the saturated aquifer, is called specific retention and corresponds to field capacity. Porosity is the summation of specific yield and specific retention. From the sieve analysis, specific yield, specific retention and porosity of the aquifer material are obtained 24.0%, 19.5% and 43.5%, respectively.

From the mechanical analysis 0.53% coarse gravel, 7.23% medium gravel, 9.74% fine gravel, 25.51% very fine gravel, 35.08% very coarse sand, 16.98% coarse sand, 3.39% medium sand, 1.26% fine sand, 0.23% very fine sand, 0.019% coarse silt, 0.00016% medium silt and 0.03084% other type of soils were present in the aquifer of the study area.

**Gravel pack design:** For coarse sand and non-uniform aquifer (like this case), $d_{30}$ of the gravel pack material [$d_{30}$ (Gr)] is equal to 4 to 6 times of $d_{30}$ of aquifer material [$d_{30}$ (Aq)] i.e. 6 to 9 mm as depicted in Fig. 7. With these points for $d_{30}$ for the gravel pack material smooth curves were drawn such that $C_u$ for the gravel pack material is 2.5. The shaded area in Fig. 7 shows the recommended gravel pack material. Clean pea gravel round shaped of size 5 to 10 mm may be used. The slot size was kept at $d_{10}$ of the gravel pack material which is 4.0 mm. The thickness of the artificial gravel pack material may be 15-20 cm.

## 7. DESIGN RECOMMENDED

The optimum length of the well screen is chosen with respect to drawdown, aquifer thickness and stratification of the aquifer. After all type analyses (mechanical, theoretical, numerical etc), it may be recommended that for the radial collector well, 70% of total aquifer thickness may be screened.

According to the design 36 laterals of length 30 m each are to be placed to fulfill the requirement of 5 MGD from each well and a total of 10 MGD from two well. 36 laterals are to be placed in three layers each have 12 laterals spaced uniformly at 30° around the circumference of the caisson. The aquifer thickness was taken as 11.55 m. The layer may be placed at 4.572 m, 7.62 m and 10.668 m. The diameter of the caisson and the screen length may be taken as 1.5 m and 8.085 m, respectively.

The sizes of the slots depend upon the gradation and the size of the formation material. In case of artificial gravel packing, $C_u$ is less than 3 and the effective grain size is less than 0.25 mm. The design procedure for selecting the gravel material is to determine which is equal to 4 to 6 times of the $d_{30}$ of the aquifer material obtained from the mechanical analysis. The slot size of the strainer is kept at $d_{10}$ of the gravel pack material to avoid segregation of the fine particles near the strainer opening ranging between 1.5 to 4 mm and the length 5 to 12 mm. Minimum of 12% opening was recommended.

After the length of the screen and slot size screen diameter was selected in such a way that the entrance velocity near the well screen will not exceed 2.5 to 6.0 mm/sec assuming 22700 $m^3$/day capacity for the radial collector well. For entrance velocity $V_e$ = 3 mm/sec the screen diameter may be proposed as 300 mm. This is the dimension as per the details provided earlier. It is dependent of the mineral content of the water, presence of bacterial slimes, DO level, pH of the water, $CO_2$ content, chloride content, hardness and iron content etc. Stainless steel or brass can be used as the screen material and V type continuous slot will be preferred here.

## 8. CONCLUSIONS

From the above calculations it is observed that the main wells 1 and 3 are most suitable place for construction of the intake wells to fulfill the demand of 5 MGD. In case of each main well, 36 strainers of length 30 m each giving a total strainer length of 1080 m are provided in the design. So the design is safe. It is depicted from Fig.4 - Fig.6 that the drawdown in all observation wells is decreasing proportionately with respect to their corresponding main wells. From Fig. 6, it may also be observed that as the pump was stopped from 10:25 am to 10:25:30 am for checking the recovery of the drawdown. So in 30 seconds the main well 3 recovered of 55.2 cm which was 97% of the depth of water from the bed at that time. So, it can be said that there will be no deficit of water supply to fulfill the required demand. In the aquifer of the study area the amount of clay and silt are negligible with respect to gravel and sand. Here, 43.01% gravel and 56.94% sand are available. Specific retention was lesser than the specific yield. According to the mechanical analysis, artificial gravel packing is not required because aquifer materials are well graded. But an artificial gravel packing is necessary for sustainable safe design.

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