Design of Subsurface Geodrain for Automated Industrial Unit – Case Study

D. L. Shah
The Maharaja Sayajirao University of Baroda, Vadodara, Gujarat, India

C. R. Shah
Consulting Engineer, Vadodara, Gujarat, India

H. M. Patel
The Maharaja Sayajirao University of Baroda, Vadodara, Gujarat, India

DESIGN OF SUBSURFACE GEODRAIN FOR AUTOMATED INDUSTRIAL UNIT – CASE STUDY

D L Shah
Applied Mechanics Dept.
Faculty of Tech. & Engg.
M S University of Baroda
Vadodara 390 001 (Gujarat)
INDIA

C R Shah
Consulting Engg.
2, Krishna Society
Old Padra Road
Vadodara 390 007 (Gujarat)
INDIA

H M Patel
Civil Engineering Dept
Faculty of Tech. & Engg.
M S University of Baroda
Vadodara 390 001 (Gujarat)
INDIA

ABSTRACT

This paper describes the pre-construction modeling for design and post-construction evaluation of subsurface drainage systems for an industrial plant. Rajshree Polyfil Ltd has a polyester filament manufacturing plant spread over 50 hectare area in Bharuch district of Gujarat State, India. The plant is fully automatic and robotics operated. The cable duct for control system was laid below formation level. The seepage water was observed in the cable trench and nearby vicinity. This seriously affects the functioning of computer controlled production system. Preliminary investigation revealed that the ground water level was around 1.0m depth below formation level, which was more than 15m depth during the construction of unit. Detailed subsurface investigations and field permeability tests are carried out. Subsurface drainage system was designed and its performance was estimated prior to construction of drain with the help of computer modeling using software MODFLOW. The model area was divided in three to five layers having different permeability values obtained from field test. After construction of subsurface geodrain, discharge was measured and water level was also measured at few piezometers installed near the drain. It is found that the performance of the drain is well in accordance with the design.

INTRODUCTION

Rajshree Polyfil Limited has a polyester filament manufacturing plant spread over 50 hectare area in Bharuch District of Gujarat State, India. The plant has fully automatic manufacturing unit with computer control system. The cable ducts for communication were laid below formation level and were found to collect seepage water from the surrounding low ground water level. This seriously affects the computer controlled production system. It was desired that suitable under drainage system should be designed to keep seepage water away from the critical buildings like Spinning block, Utility block, and D G block. The plant site is located in the alluvium deposits, which consist of clay and silt with lime kankar. There are occasional very thin layers of gravelly sand. The depth of water table was determined by drilling 14 boreholes. The high water table in the plant area was found due to perched water in gravel pockets, recharge due to rainfall and irrigation in command area. During subsurface investigation, the ground water table was found at variable depth. At spinning block, water table was found varying from 0.8m to 2.45m below ground level. At utility block, it was varying from 1.25m to 2.15m below ground level.

SUBSURFACE INVESTIGATIONS

Fourteen boreholes of 200mm diameter are drilled in the area considered for the treatment. The boreholes are drilled up to about 5.0m to 6.0m depth below ground level. Out of fourteen boreholes, six boreholes are drilled around spinning block area, four boreholes around utility block area, and four boreholes around DG block. Location map of boreholes ia shown in Fig. 1. During subsurface investigation, the ground water table was found at variable depth of 1.0m to 2.0m below ground level. Piezometers are installed to monitor the ground water level before and after construction of geodrain. Piezometers are constructed using 110mm diameter PVC pipe having perforations except in top 1.0m length. Clean gravel of size 3.0mm to 10.0mm is poured around the pipe in boreholes. Gravel packing is made up to the perforation level only. above perforation level, the boreholes are backfilled using plastic clay. The piezometers are developed immediately after pipe assembly is lowered and gravels are packed. The development is carried out by pumping out water with the help of hand operated pump. Water was pumped out up to bottom of borehole and is allowed to be filled up again. The cycle is repeated till clear water is pumped out. After the aquifer had been developed, sounding is taken and water level recorded with reference to the top of the pipe.

In-Situ Permeability Test

The in-situ permeability tests are carried out using bailer method. Total ten in-situ tests are carried out at different locations. Initial water level in borehole is recorded with water level indicator. Water from the borehole is then pumped out till the draw down of 2.0m to 3.0m is obtained. Water quantity bailed out is measured. Recovery after bailing stopped is observed with time elapsed. The test is repeated for four to five cycles to obtain representative value of coefficient of permeability.

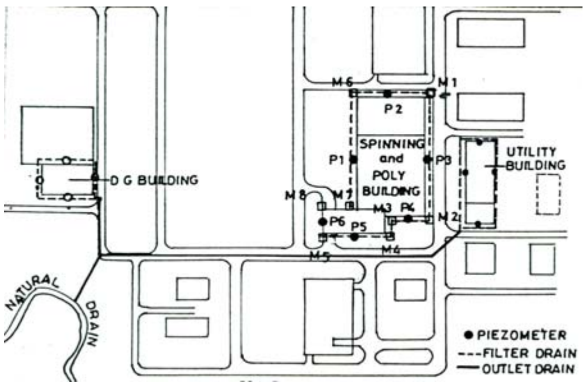


Fig. 1 Layout of piezometers installation, filter ad outlet drain

MODEL STUDIES

The data obtained from field permeability tests are used for computer modeling and design of under drainage system. Typical soil profile at spinning block used for computer model is shown in Fig. 2.

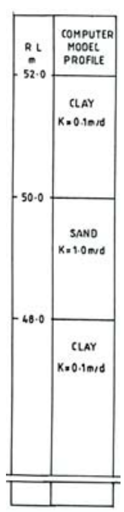


Fig. 2 Soil profile

The computer studies are carried out using MODFLOW ground water flow program developed by U S Geological Survey. Three different models were carried out for spinning block, utility block, and DG block with subsurface drains along the periphery of the buildings. One additional model is also carried out for spinning block, with drains only on three sides. Soil strata in each building was divided in to 3 to 5 layers based upon soil profile and permeability coefficient value as determined from soil investigation data. Constant head boundary was considered at 50m away from drains. Control level was fixed at 0.5m below lowest duct level for each building. Simulations were carried out with different drain elevation till computed maximum water level below building was obtained, at least 0.5m below control level. For simulation, DG block was considered as one system, while spinning and utility blocks were combined as one system. The distance between spinning block and utility block is only 30m. The simultaneous working will cause interference of draw down between two systems and will provide lower water levels. This aspect was desired to be studied to check whether lower water levels between two blocks can be helpful in design of road between two blocks. The road has settled due to higher water level and remedial measures were being considered to improve the road. Both the spinning and utility block drainage systems were modeled as a single system to find drain elevation and discharges with highest water level. The summary of input and output of computer studies is given in Table – 1.

Table 1. Summary of Input & Output of Computer Studies

Block	Input		Output		
	Layer RL (m)	K (m/day)	Control RL (m)	Max RL (m)	Discharge (m^3/day)
spinning	52-50	0.1	51.4	50.77	38.1
	50-49	1.0			
	49-47	0.1			
utility	47.5-47	1.0	50.85	50.33	28.8
D G	47-42	0.1	49.6	49.11	30.9
	52-50	0.1			
	50-49	1.0			
	49-42	0.1			

Computer runs were made with different drain elevations till computed maximum water level below building was 0.5m below control RL furnished by owner. Control RL for different blocks, maximum computed RL, and computed discharge for all drains using MODFLOW software is shown in Table 1. The models were run using software MODFLOW and the results of final simulation are shown in Fig. 3 to 5.

Water level contours and water surface profile at section xx for spinning block with drains on four sides are shown in Fig. 3 (a, & b). It is seen that maximum RL 50.77m is lower than central RL 51.40m by 0.43m and hence it is accepted.

Water level profile for utility block with drains on four sides as obtained from the computer output is shown in Fig. 4. It is seen that maximum RL 50.33m is lower than control RL 50.85m by 0.52m, hence it is accepted.

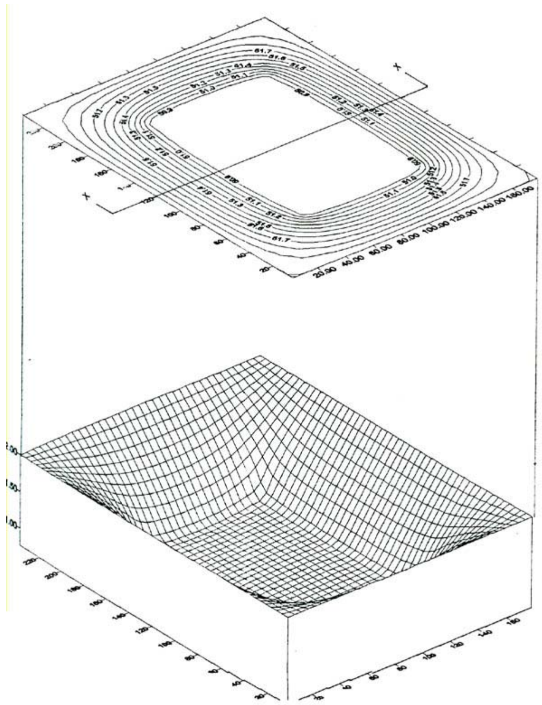


Fig. 3(a). Water level contours with drains on four sides for spinning block – MODFLOW output

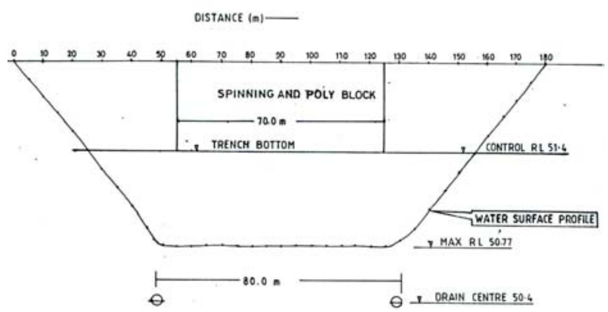


Fig. 3(b). Water surface profile at section xx for spinning block – MODFLOW output.

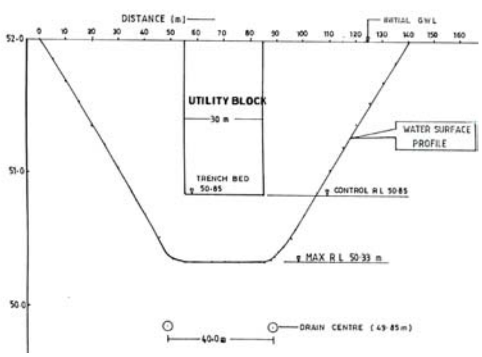


Fig. 4. Water surface profile for utility block – MODFLOW output

Water level profile for DG block with drains on four sides is shown in Fig. 5. It is seen that maximum RL 49.11 is lower than control RL 49.60m by 0.49m, hence it is accepted.

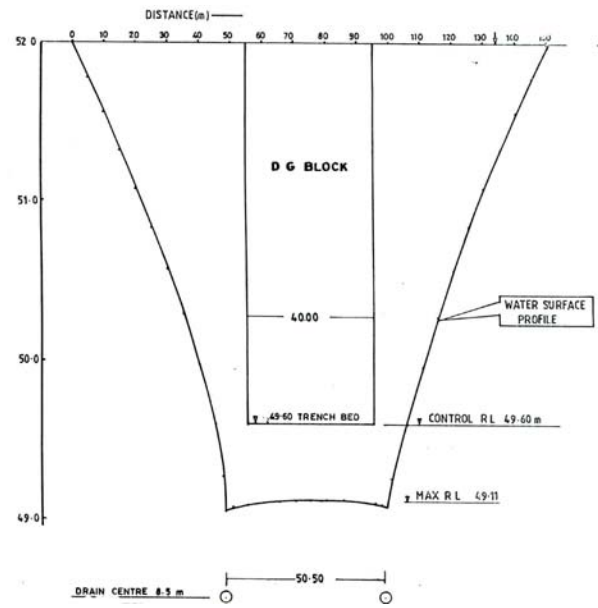
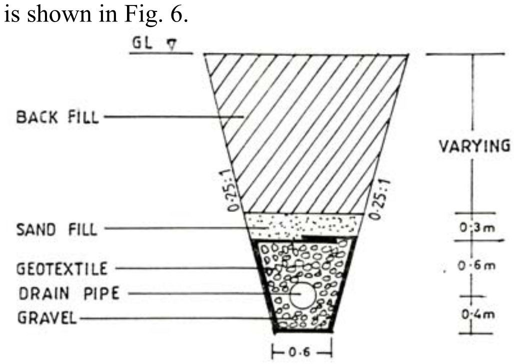


Fig. 5 Water surface profile for DG block – MODFLOW output.

The highest water level between two building reached at 50.4m i.e. 1.6m below road top and 1.3m below sub-grade which was considered to be safe and no special drainage was needed for road design.

DESIGN AND INSTALLATION OF DRAINS

It was proposed to provide filter drain at about 5.0m away from the wall of the building all round. Manholes are provided on filter drain at each location of change of alignment for maintenance purpose. The filter drain was designed with a gradient of 1 in 1000, so that free gravity flow of water takes place from all sides. The final grade was decided after checking in the model, that maximum water level remains at least 0.5m below control level. The typical section of filter drain is shown in Fig. 6.



.Fig. 6. Typical section of filter drain.

To permit high rate of flow and to prevent entry of silt, a geotextile material was proposed to cover the fill around the drain. The filter drain comprises of (1) filter pipe, (2) gravel packing, and (3) geotextile. The basic concept of design is to provide design discharge corresponding to clay and maximum water level at GL, i.e. at RL 52.0m. This discharge will be lower than unsteady state discharge, which will be maximum when SWL is maximum, reduce as draw down increases and will be constant at steady state condition. To dispose of high discharge rate of short duration, during unsteady state by providing gravel fill and geotextile with a high discharging capacity. Geotextile permits high rate of flow and prevents silt entry.

A typical section of filter drain is shown in Fig. 6. It comprises of six parts:

1. Excavated trench with bottom width of 0.6m, side slope of 0.25:1 and bed RL 0.4m below design centre of drain.
2. Geotextile surrounding gravel fills.
3. Gravel fill surrounding filter pipe from trench bed to 0.6m above pipe centre.
4. Filter pipes with diameter and centre as designed.
5. Sand fills above geotextile.
6. Back fill above sand fill.

The minimum base width of 0.6m is provided for filter drain considering practical construction facility. Slope of 0.25:1 is provided as minimum safe to keep gravel fill quantity minimum.

Geotextile is designed with reference to permeability and gradation of soil strata required draining. From the soil investigation, it inferred that at each building, generally top strata is clayey followed by sandy and again clay. However, there is wide variation in soil stratification both in length wise and depth wise. From the in situ permeability results, the design coefficient of permeability for clayey strata is considered as 0.1m/day, and for sandy strata 1m/day. The maximum discharge of 0.14m3/day per meter length of filter drain is obtained ay DG block. Mirafi-180N geotextile with normal permeability of 0.2cm/sec is considered. Flow per meter length through geotextile is: $Q = K_n \times A \times i$.

$$Q = \frac{0.2}{100} \times 0.4 \times \frac{3.4}{50} \times 3600 \times 24 = 4.7 m^3 / day$$

This is much more than requirement of 0.14m^3/day.

Gravel of 20mm to 40mm size is proposed around stone ware pipe. The coefficient of permeability of gravel is taken as 20cm/sec and slope = 0.001. Flow through gravel is $Q = K \ \times A \times i$.

$$Q = 0.2 \times 0.4 \times \frac{3.4}{50} \times 3600 \times 24 = 470 m^3 / day$$

This is much more than required.

The discharge capacity of 150mm and 200mm diameter for half-full pipe is calculated using Manning's formula:

$$Q = \frac{A}{n} \times R^{2/3} \times S^{1/2}$$

For 150mm diameter pipe, half-full pipe discharge capacity is worked out as 38.84m3/day, considering n = 0.018. stone ware pipe of required size was provided for each length of sub surface drain depending upon discharge to be carried out.

A sand layer of 0.3m is provided over geotextile to provide additional drainage thickness to accelerate drainage. The under-drainage system is designed for steady state flow. There will be some lag between rise of water level in monsoon and establishing of steady state water level. This time lag will be reduced by provision of sand layer.

CONCLUSION

MODFLOW software was used successfully to study the special problem of water logging. Different alternatives can be tested for site conditions. The complex three dimensional flow analysis using MODFLOW has provided safe solution for highly sophisticated industrial unit affected by seepage water. The construction of under drainage system was under taken, completed and put to operation in May, 1997. The subsurface drainage system for spinning, utility and DG block has worked satisfactorily. Water level in piezometers and manholes has indicated that water level has been below control level.