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Design of Groundwater Sustain Irrigation Network System in Panca Lautang Area, Sidrap Regency, South Sulawesi

B Hidayah[1], A Jaya[1], A I D Puspita[2], B R Maulana[1]

[1]Department of Geological Engineering, Hasanuddin University, Gowa, 92111, South Sulawesi, Indonesia

[2]Department of Civil Engineering, Hasanuddin University, Gowa, 92111, South Sulawesi, Indonesia

*coresponding author: bahrulhidayah@gmail.com

Abstract. Mining activities in open pit and underground mines will always be associated with rock breaking or stripping activities (both mechanical and blasting), so that this can affect the structure and strength of rocks. The strength of the rock is strongly influenced by the presence of initial cracks (pre-existing cracks) and rock anisotropy conditions associated with discontinuous plane conditions. Fracture mechanics is a science that illustrates how a fracture can occur and propagate during applied stress on material. The main parameter in fracture mechanics is called fracture toughness which shows the resistance of the material to propagate the crack. There are several mode in determining type I fracture toughness, one of which is type I fracture toughness Flattened Brazilian Disc (FBD) mode. Type I fracture toughness test is carried out using a compression machine in a laboratory and is conducted on concrete samples consisting of 3 (three) various samples, with a ratio of cement and sand composition of 1:1, 1:2, and 2:1. This test also uses different loading rate values, namely 2.50 mm/min, 2.70 mm/min, and 2.83 mm/min. The results of the type I fracture toughness value from each loading rate will be compared to determine the effect of the loading rate on the value of type I fracture toughness. The obtained fracture toughness value is also related to the physical and mechanical properties of the samples. Based on the results of tests, it can be seen that the loading rate affects the value of fracture toughness, the increase in fracture toughness value is followed by the higher loading rate. In addition, it can be seen that the fracture toughness value is directly proportional to the uniaxial compressive strength value and the indirect tensile strength value. The average correlation value obtained is $R^2 = 0.9884$ (indicating a strong relationship).

Introduction

Not all of the agricultural areas in the Panca Lautang area have irrigation networks, other than that the existing networks have not been able to meet all the needs of agricultural water [1]. Consequently, agricultural production does not reach maximum results, because its water needs are very dependent on rainwater, so that agricultural production is not optimal [2]. This limitation causes farmers to only be able to cultivate crops once a year and sometimes not at all, especially when there is a prolonged drought. Efforts to increase the availability of agricultural water by creating a groundwater irrigation network by taking water from boreholes, to support this, it is necessary to carry out the detailed design of a well-planned groundwater irrigation network. This research is intended to conduct a survey and make detailed planning in detail about the ideal groundwater irrigation network with an area of rice fields that can be irrigated according to the available water discharge in the borehole which will be used for irrigation needs. Activities are focused on areas that do not have adequate irrigation networks. The Panca Lautang area in the Sidrap Regency of South Sulawesi was the target of this research.

2. Geological Setting

Based on the geomorphological aspect, this area is divided into several types of morphology, such as plain to wavy landscapes with a height difference of <5 - 20 meters, the percentage of slope 0 - 3.6%, slopes ranging from 0 - 8°, flat - wavy relief. Based on van Zuidam's (1985) classification, the morphological units that form the dominant research area are plain and in the southern part of the location are wavy morphological units [3-4]. refers to the location of the type and the official lithostratigraphic naming, the lateral distribution of alluvial deposits (Qa) which is the constituent lithology, occupying almost the entire study area, or about 21.77 Ha.

Alluvial deposits at the study site are quarter-old (Qa) which consists of clay, silt, mud, sand, gravel deposits found around the shores of Lake Sidenreng and Lake Tempe. Local alluvial deposits contain remains of shellfish and coral limestone [5-7]. The material that makes up this sediment consists of weakly consolidated sediment deposits from land to transition (deltaic) and has not been compacted and undergoes a lithification process, consisting of sediment deposits in the size of gravel, sand, and mud deposited in the environment of lakes, swamps, deltas and beach (fining upward) [6].

The lithology of the constituent aquifers at the study site has medium-low permeability in coarse materials while low-permeability in clay-sized materials. Aquifer productivity is in the form of the aquifer with flow through inter-grain spaces and is a productive aquifer type with a wide lateral distribution [8].

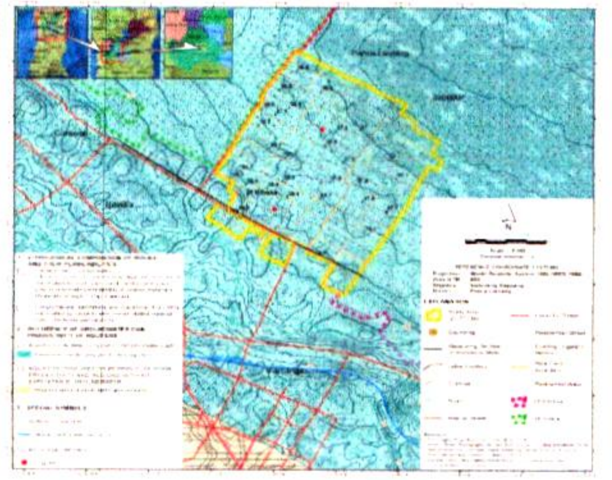

Figure 1. Research location on the Indonesian Hydrogeological Map scale 1: 250,000 [8]

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3. Methods

3.1. Literature Research

This stage is the pre-field activity stage, including: conducting a secondary data inventory of information on the research area, maps as well as on the methodology and instruments to be used.

3.2. Vertical Electric Sounding

Vertical electrical sounding (VES) is a geophysical method for the investigation of a geological medium. The method is based on the estimation of the electrical conductivity or resistivity of the medium. The working principle of this method is to actively inject an electric current into the soil to obtain a variation of the resistivity value of the rock [9-10], then the measurement data will be inverted using the Res2DInv software programs use the smoothness-constrained Gauss-Newton least-squares inversion technique to produce a 2D model of the subsurface from the apparent resistivity data [11], which will then be interpreted based on the resistivity type of rock [12].

3.3. Topographic Mapping

This stage is carried out using photogrammetry with and flat folders, to map the detailed topographic conditions of the service area, as well as to obtain the area, height difference, and digital surface models which will then be processed into topographic maps and cross-sections.

3.4. Geological and Hydrogeological Survey

Geological and hydrogeological mapping stages at the research location include:

1. Identify the types of soil and lithology (rock) in this area.
2. Physiographic observations, covering general morphological conditions, types of rivers, and the direction of surface water flow in this area.
3. Observation of geological structural conditions and other geological conditions
4. Identifying rock layers that could be the groundwater-bearing aquifer layer in this area
5. Identifying the potential and productivity of aquifer layers
6. Determine the recharge area (add) of groundwater.
7. Identify the surface runoff patterns around this area.
8. Measuring surface water conditions (rivers), conditions and shallow groundwater levels (community dug wells), and conditions and deep groundwater levels (boreholes) around the investigation area
9. Interpreting the physical conditions and quality of shallow and deep groundwater (boreholes) and surface water conditions in this area
10. Determine the Groundwater Basin Area (CAT) which is the location of the potential groundwater found in this area.
11. Measuring the potential for groundwater in the vicinity of the study area by conducting pumping tests of several wells in the area.

3.5. Hydrology, Climatology and Water Resources Analysis

Rainfall data, first the data must pass the test for data consistency using the multiple mass curve method and the RAP's method then a frequency analysis is made to see the distribution before use [14].

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3.6. *Irrigation Network Design*

Correlating surface and subsurface data as a reference in determining the point of drilling production wells, then designing the pump house as a place for the installation of control instruments and pump support, then designing the pipeline network based on the morphology and water requirements of the research area [12].

3.7. *Preparation of Technical Specifications and Design Cost Budgeting*

The preparation of technical specifications is based on needs analysis and technical guidance documents for piping irrigation network planning and piping irrigation planning standards [13-15], then prepare a budget draft based on unit price analysis.

4. Results

Based on the results of geoelectric measurements, at the sounding point, it is interpreted that sandstones containing freshwater ranging from a depth of 81 to a depth of 105 meters, with a layer thickness of 27 meters, a layered slope of 16°, an estimate of the optimum potential for groundwater discharge with a casing pipe with a diameter of 8" with a depth of 150 meters.Based on the Huisman equation, 1974, the maximum discharge (Q_{max}) of the well can be calculated as follows:

$$
\begin{aligned}
Q_{maks} &= 2\pi \ x \ r_W \ x \ D \ x \left(\frac{\sqrt{K}}{15}\right) \\
Q_{maks} &= 2\pi \ x \ 0{,}10116 \ x \ 29 \ x \left(\frac{\sqrt{0.00316}}{15}\right) \\
Q_{maks} &= 0.0219 \frac{m^3}{dt} \\
BQ_{maks} &= 84{,}06 \ x \ 0{,}0219 \\
BQ_{maks} &= 1{,}84 \ m \\
CQ_{maks}{}^2 &= 1250 \ x \ (0{,}0219)^2 \\
CQ_{maks}{}^2 &= 0{,}6 \ m \\
SW_{maks} &= 1{,}84 + 0.60 \\
SW_{maks} &= 2{,}44 \ m
\end{aligned}
$$

Table 1. VES measurement results

NO	MN (m)	MN/2 (m)	AB (m)	AB/2 (m)	K	ΔV (mVolt)	I (mA)	ρa (ohm-m)
1	1	0.5	2	1	2.357	2550.0	168	35.78
2	1	0.5	3	1.5	6.286	148.6	82	11.39
3	1	0.5	4	2	11.786	63.7	29	25.89
4	1	0.5	6	3	27.500	46.8	80	16.09
5	1	0.5	8	4	49.500	61.9	29	105.66
6	4	2	10	5	16.500	67.2	11	100.80
7	4	2	12	6	25.143	49.4	45	27.60
8	4	2	14	7	35.357	41.9	22	67.34
9	4	2	16	8	47.143	59.6	18	156.10
10	4	2	18	9	60.500	51.7	22	142.18
11	4	2	20	10	75.429	44.3	42	79.56
12	10	5	30	15	62.857	14.2	45	19.83
13	10	5	40	20	117.857	6.8	65	12.33
14	10	5	50	25	188.571	4.7	96	9.23
15	10	5	60	30	275.000	9.2	218	11.61
16	10	5	70	35	377.143	3.2	88	13.71
17	20	10	80	40	235.714	4.4	99	10.48
18	20	10	100	50	377.143	2.5	68	13.87
19	20	10	120	60	550.000	1.8	112	8.84
20	20	10	150	75	868.214	1.1	68	14.04
21	20	10	200	100	1555.714	0.9	52	26.93
22	30	15	250	125	1613.333	2.6	212	19.79
23	30	15	300	150	2333.571	1.7	307	12.92
24	30	15	350	175	3184.762	0.6	257	7.44
25	30	15	400	200	4166.905	0.3	92	13.59
26	60	30	450	225	2604.643	3.2	118	70.63
27	60	30	500	250	3226.667	4.2	120	112.93
28	60	30	600	300	4667.143	3.8	125	141.88
29	60	30	700	350	6369.524	3.6	128	179.14
30	60	30	800	400	8333.810	4.4	130	282.07

After that, scatter plotting was carried out using the Sichardt Graphical Method modified by Cashman (2001) [5], so that the discharge range was obtained from 0.004 m^3/s - 0.012 m^3/s, with an optimum discharge (Q_{opt}) of 0.012 m^3/s. And the maximum water level fluctuation (SW_{max}) is 2.44 m. So that it is known that the optimum well discharge in the study area can produce a well discharge of 4.0 - 12.0 m/s.

Furthermore, referring to the BPSDM water demand projection, 2018 the calculation of water needs for irrigation is based on factors of plant type, type of soil, method of providing water, method of soil processing, lots of rainfall, planting time, climate, maintenance of water channels and structures and so on. The amount of water for irrigation in rice fields can be formulated as follows [16-17]:

$NFR = E_{tc} + P + WLR - Re$

with;

NFR = Net Field Water Requirement, the need for clean water in the fields (mm/day)

E_{tc} = Plant evaporation (mm/day)

P = Percolation (mm/day)

Re = Effective rainfall (mm/day) in this case using TAKE so that Re is constant throughout the year.

WLR = second replacement of water layer

The results of calculating water requirements can be seen in table 2.

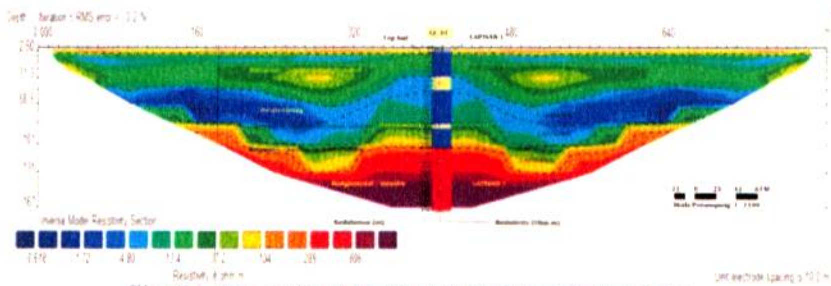

Figure 2. Cross-section of apparent resistivity inversion result at the sounding point

Table 2. Results of analysis of water needs in irrigation service areas

No	Bulan Periode		Satuan	Januari I	Januari II	Februari I	Februari II	Maret I	Maret II	April I	April II	Mei I	Mei II	Juni I	Juni II	Juli I	Juli II	Agustus I	Agustus II	September I	September II	Oktober I	Oktober II	November I	November II	Desember I	Desember II
1	Pola Tata Tanam																										
2	Koefisien Tanaman	K1		1.200	1.27	1.11	1.100	1.200	0.000		LP	LP	1.200	1.27	1.11	1.100	1.100	0.000		0.900	0.11	0.66	0.950	0.950	0.950	LP	LP
		K2		LP	1.200	1.27	1.11	1.100	1.100	0.000		LP	LP	1.200	1.27	1.11	1.100	1.100	0.000		0.900	0.11	0.66	0.950	0.950	0.950	LP
		K3		LP	LP	1.200	1.27	1.11	1.100	1.100	0.000		LP	LP	1.200	1.27	1.11	1.100	1.100	0.000		0.900	0.11	0.66	0.950	0.950	0.950
3	Rerata Koefisien Tanaman			0.000	0.400	1.267	1.100	1.110	0.867	0.650	0.000	0.000	0.000	1.000	1.267	1.100	1.110	0.867	0.650	0.250	0.605	0.887	0.673	0.828	0.617	0.650	0.000
4	Evapotranspirasi Potensial (Eto)		mm/hari	4.667	4.667	5.121	5.121	5.799	5.799	4.591	4.591	4.485	4.485	4.627	4.627	3.890	3.890	4.740	4.740	6.07	6.07	6.72	6.72	6.41	6.41	6.19	6.19
5	Kebutuhan Air Untuk Penyiapan Lahan (PL)		mm/hari	15.471	15.471	14.967	14.967	14.467	14.467	13.962	13.962	11.888	11.888	13.619	13.619	13.512	13.512	14.071	14.071	14.879	13.727	14.127	14.127	11.889	11.889	13.727	13.727
10	Penggantian Lapisan Air (WLR)		mm/hari		1.1	1.1	2.2	1.1	1.1					1.1	1.1	2.2	1.1	1.1									
11	Perkolasi		mm/hari	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
17	Kebutuhan Air Tanaman (Etc)		mm/hari	15.471	15.471	7.734	7.958	7.182	5.016	2.967	13.962	13.888	13.888	13.619	5.101	1.657	5.096	4.108	3.081	1.517	3.081	1.783	4.527	7.298	5.617	13.727	13.727
19	Debit Air Tanah hasil pemompaan selama 24 jam		mm/hari	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671	0.671
21	Kebutuhan Air Bersih Di Sawah (NFR)		lt/dt/ha	1.210	1.210	1.151	1.151	1.086	1.086	1.021	1.021	1.026	1.026	0.986	0.986	0.980	0.980	1.040	1.040	1.151	1.009	1.094	1.094	1.027	1.027	1.009	1.009

Flow velocity in the pipe

$$V_d = \frac{4Q_{ep}}{\pi D_d^{\,2}}$$

Where ;
Qep = Effective capacity of the pump = 0.012 m^3/s
Dd = Pipe Diameter = 0.15 m
then;

$$V_d = \frac{4(0.012)}{\pi(0.15)^2} = 0.679\ m/s$$

The total loss in the production pipeline can be calculated using the following equation:

$$h_{fd} = \lambda \frac{L_d}{D_d} \frac{V_d^{\,2}}{2g}$$

Where;
Dd = Pipe diameter

Ld = Pipe Length
λ = Friction Coefficient
Vd = Velocity of water in pipe
g = Gravitational Acceleration

then;
$\lambda = 0.020 + (0.0005/0.15)$
$= 0.023$

$$h_{f,d} = 0,023 \frac{105(0,679)^2}{0.15x2x9,8}$$

$$h_{f,d} = 0,023 \frac{48,4}{2.94}$$

$$h_{f,d} = 0,023 \frac{48,4}{2.94}$$

$$h_{f,d} = 0.37\, m$$

The total losses in the distribution pipe can be calculated using the following equation:

$$h_{fd} = \lambda \frac{L_d}{D_d} \frac{V_d^2}{2g}$$

Where:
Dd = Pipe Diameter
Ld = Pipe Lengths
λ = Coefficient of Friction
Vd = flow velocity in distribution pipe
g = gravitational acceleration
h_{mgv} = 0.2 12.155
h_{mgv} = 1.3 m
v = flow velocity (m/s)
g = Gravitational accelerations (m/s^2)
Then;
λ = 0.020 + (0.0005/0.15)
= 0.023

$$h_{fd} = 0,025 \frac{1108,4(0,679)^2}{0.1x2x9,8}$$

$$h_{fd} = 0,025 \frac{511}{1,96}$$

$$h_{fd} = 6.51\, m$$

Gate valve losses can be calculated by:
$h_{mgv} = f\,(v^2/2g)$
Where:
6" gate valve loss coefficient,
f = 0.2
Then;
The amount of loss
$h_{mgv} = 0.2\,(0.679^2/2 \times 9.8)$

The total pump head can be calculated by the following equation:
H = ha+hl+zb(Vd2/2g)
Where:
Vd = Average pipe flow velocity (m/s)

Ha = The difference in water level (m)
hl = Loss value in pipe (m)
g = Acceleration of gravity 9.81 (m/s^2)

The total pump head can be calculated by the following equation:
$H = ha + hl + zb(Vd2/2g)$
Where:
Vd = The average flow rate of the pipe (m/s)
ha = The difference in water level (m)
hl = Loss Value on the pipe (m)
g = Magnitude of acceleration due to gravity 9.81 (m/s^2)
Then:

$$\begin{aligned} H &= ha + hl + zb + (Vd2/2g) \\ &= (100-1.3) + 2(0.37+6.51+1.3) + (38.2-36)(0.6792/(2 \times 9.8)) \\ &= 98.7 + (8.45+1.3) + 2.2 + 0.023 \\ &= 98.7 + 8.18 + 2.2 + 0.023 \\ &= 109.103 \text{ m} \end{aligned}$$

Based on these data, the type of pump that will be used in the planning of this research's groundwater irrigation network is a submersible pump with a total head of 109 m.

The minimum amount of energy needed to distribute water per unit time:
$Pw = 0.163 \cdot \gamma.Q.H$
Where:
Pw = Water power (kW)
γ = density of fluid (kg/l)
Q = Capacity (m^3/min)
H = head of total pump (m)
So that:

$$\begin{aligned} Pw &= 0.163 \times 0.99983 \text{ kN/m}^3 \times 0.012 \text{ m}^3\text{/mnt} \\ &= 0.163 \text{ X } 9998.3 \text{ N/m}^3 \times 0.012 \text{ m}^3\text{/mnt} \times 109 \\ &= 2{,}131.7 \text{ watt} \end{aligned}$$

Then the amount of power required to move and rotate the pump shaft is the same as the water power plus the power loss in the pump. This power can be expressed as follows:
Where;
P = shaft power
Pw = Water Power
Np = pump work efficiency index

Then:
P = 2,132 Kw/efficiency index, EI > 0.70
P = 3.04 Kw

The results of the topographic mapping show that the slope of the land is almost flat because the height difference is only about 1.762 meters, the average data is less than 0.5 meters. The highest condition in this area is in the southern area which is relatively higher than the other areas, namely 38.40 meters, but the difference in height from the location of the well is only 0.178 meters (38.22 meters). The location of the lowest area with an altitude of 36.632 meters is in the part of the rice fields in the northeast. The slope of the rice fields in this area can be seen in Figure 4.

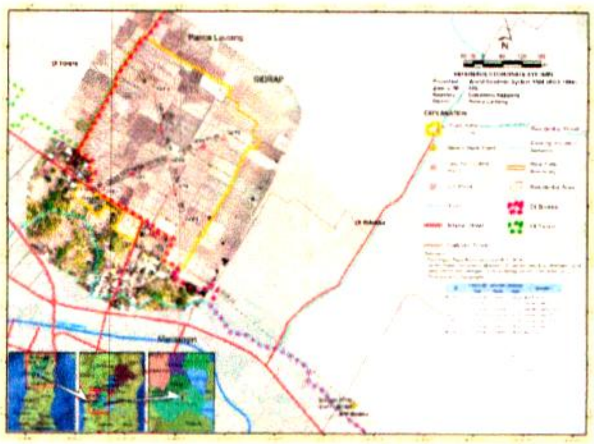

Figure 3. Photogrammetry and geodetic measuring points of the study site (ArcGIS Online Imagery overlay)

Aerial recording and shooting from the drones used to produce aerial photography conditions of this rice field area of which the survey area is 21.77 Ha. While the area of rice fields to be served is only about 19.51 hectares, the remaining area recorded in this study is the location of settlements (residents' houses), village roads, paddy fields and drainage channels. Aerial photos and irrigation service rice field areas to be reached by the Groundwater Irrigation Network (JIAT) can be obtained from the measurement results, referring to the benchmark point which is close to the location of the borehole and pump house in the southwestern part of the research location. In general, the placement of this dividing pipe distribution network to the service area will be distributed through a network of pipelines that are planted and buried in rice fields to a depth of 900 cm from the ground. The total length of the 6 inch pipeline network used is 1,127.35 meters.
Based on unit price analysis, the recapitulation of prices per work item is presented in table 3.

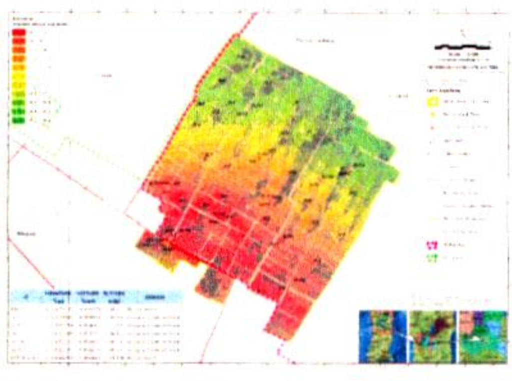

Figure 4. Topographic map of the research location (DTM data overlay)

Table 3. Results of the recapitulation of the budget plan based on the analysis of the unit price of work

NO.	TYPE OF WORK	TOTAL PRICE (Rp)	
I	PREPARED JOBS	Rp	16,686,750
II	CONSTRUCTION OF PUMP HOUSE	Rp	32,450,263
III	DIVIDER BOX WITH THOMPSON MEASUREMENT BUILDING	Rp	11,712,336
IV	2 WAYS BIG DIVIDER BOX	Rp	106,593,193
V	3 WAYS DEVIDER BOX	Rp	53,296,597
VI	PIPELINE NETWORK	Rp	272,973,706
VII	CONECTION PIPE	Rp	21,089,377
VIII	DEVELOPMENT OF DRILL WELLS	Rp	282,873,750
IX	PROCUREMENT OF MATERIALS	Rp	210,625,000
REAL COST		**Rp**	**1,008,300,973**
PPN 10 %		Rp	100,830,097
TOTAL AMOUNT		Rp	1,109,131,071
Rounding of Value		**Rp**	**1,109,100,000**

Conclusions

From this research, the following conclusions were drawn that the layer with a resistivity value of 4.5 - 117.8 m with a blue symbol is a layer of sandstone which is thought to be a layer of confined aquifers/deep aquifers (can store and pass water) Thickness of up to 25 meters (depth between 80 - 105 m) located in around the Panca Lautang area.

The calculation of potential discharge with scatter plotting using the Sichardt Graphical Method above obtained a discharge range from 0.004 m^3/s - 0.012 m^3/s, with an optimum discharge (Q_{opt}) of 0.012 m3/s and a theoretical optimum water level decrease (Sw_{opt}) is 1.17 m. And the maximum water level fluctuation (SW_{max}) is 2.5 m. Meanwhile, based on the calculation of the drill well discharge with a drilling depth of 105 meters, ignoring the groundwater pressure factor in the basin, with the construction of an 8-inch borehole, it is expected that this well can produce a discharge of 5,177 liters/sec.

Based on these data, the type of pump that will be used in the planning of the groundwater irrigation network in this study is a submersible pump with H> 109 m, namely (submersible pump). The choice of pipe type to be taken is PVC because the advantages of this pipe are flexible, can be bent and connected, rust-resistant and durable, various types are available according to pressure capacity, and light, easy to transport.

The total budget for the implementation of Production Well drilling activities followed by the construction of Groundwater Irrigation Network is IDR 1,478,400,000,- including 10% VAT.

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