VOGTLE 9510

8.7.1 Aquifer Tests for
Construction Dewatering

8.7.a

May 3,
C. R.
SHEET

Vogtle Nuclear Plant Excavation
Proposal for dewatering test well program

Test wells (2) - 80 feet deep, 10" Ø hole, 4" dia. casing, 10 feet of well screen, gravel pack.

Obs. Pts. (4) - two 80 feet deep; and two 65 feet deep, 4" dia. with 2" casing, 10 feet of well screen.

1. Drilling, setting casing and gravel pack of test wells;
   est. 160 feet - cost per linear foot

2. Drilling, setting casing and gravel pack of obs. pts.
   est. 290 feet - cost per linear foot

3. Cleaning and development of test wells
   est. 40 hrs. (20 each) - cost per hour

4. Test Pumping of wells
   est. 144 hrs. (72 each) - cost per hour

5. Move in, set up, and clean up
   linear sum cost

TOTAL COST ESTIMATE

Pay Item	Unit of Measure	Cost per Unit	Estimated Total Units	Total Cost
1	linear foot	$25.00	160	$4000.00
2	linear foot	$ 6.00	290	1740.00
3	hour	$30.00	40	1200.00
4	hour	$30.00	144	4320.00
5	lump sum	$3,000.00	1	3000.00
TOTAL COST				$14,260.00

FORM BC-S 116
20M ISPCO

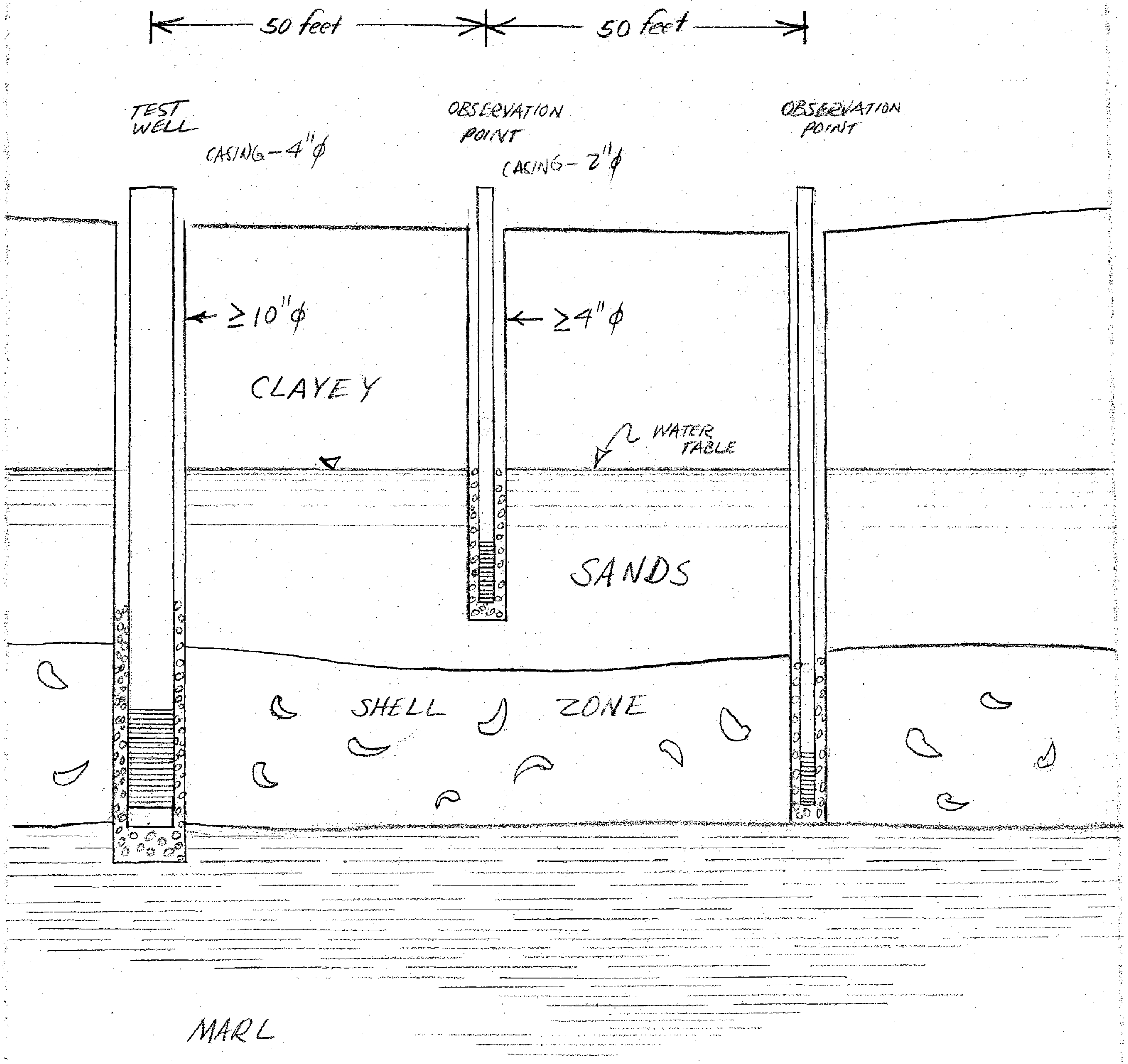
CALCULATION SHEET

BECHTEL CORPORATION
4620 SEVILLE AVE.
VERNON, CALIFORNIA

SIGNATURE Clifford Farrell DATE May 3, 1972 CHECKED ______ DATE ______

PROJECT Vogtle Nuclear Plant Excavation JOB NO. 9510-001

SUBJECT Test Wells for Dewatering SHEET 2 OF 2 SHEETS



SKETCH — NOT TO SCALE

FORM BC-S 116
20M ISPCO

BECHTEL

CALCULATION SHEET

BECHTEL CORPORATION
4620 SEVILLE AVE.
VERNON, CALIFORNIA

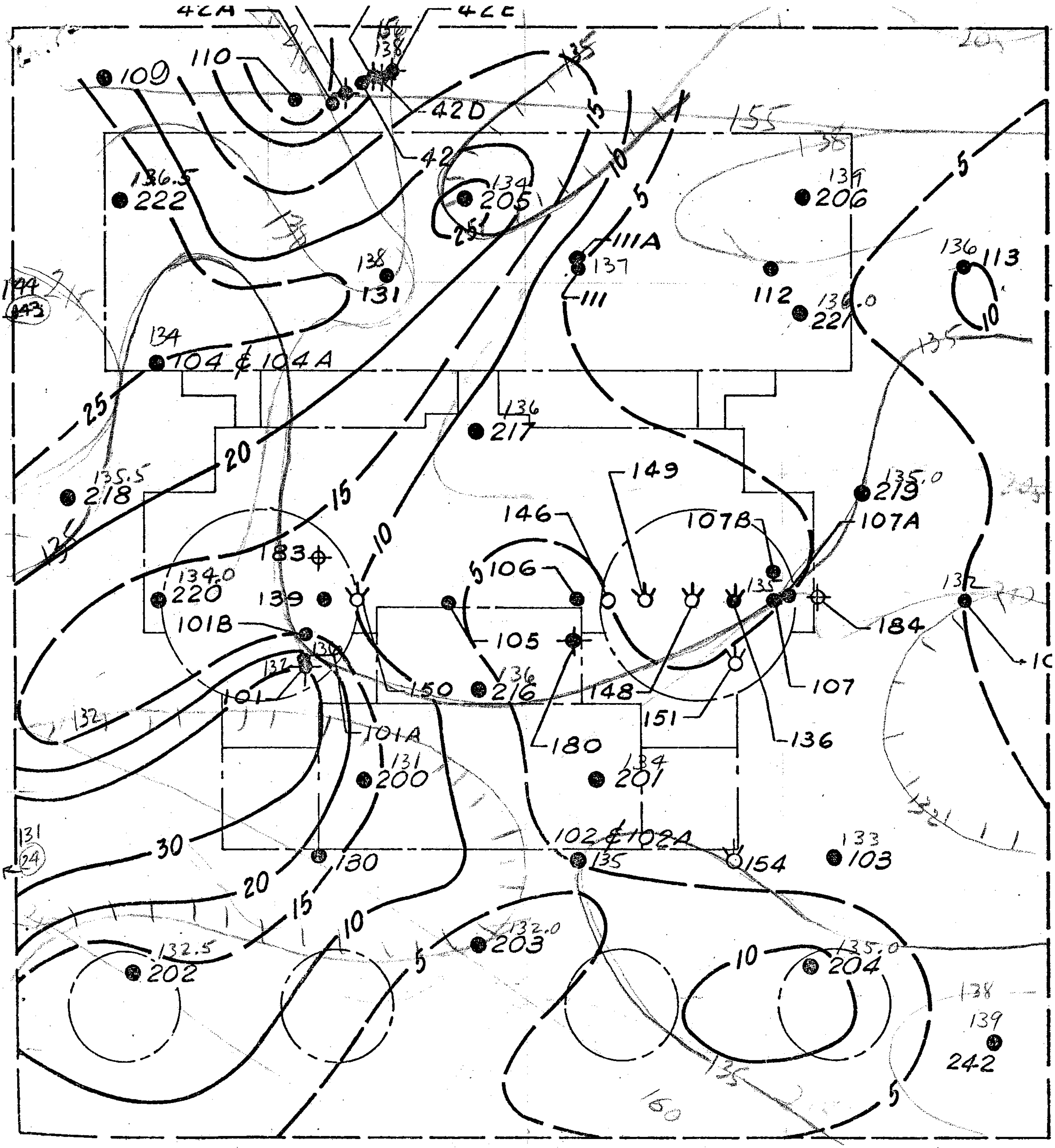
SIGNATURE ____ DATE ____ CHECKED ____ DATE ____

PROJECT ____ JOB NO. ____

SUBJECT ____ SHEET ____ OF ____ SHEETS

DH #	E	N
(GEOL)		
114	623,526	1,143,503
116	623,928	1,143,503
113	624,026	1,143,256
OBSERV. PTS (SHALLOW)		
42D	623,571	1,143,403
245	623,917	1,143,501
140	622,702	1,142,846
143	622,738	1,143,283
OBSERV. PTS. (DEEP)		
101	623,517	1,142,945
42A	623,535	1,143,380
24	623,093	1,142,850

TEST WELLS FOR DEWATERING



DETAIL A

SCALE 1" = 100'

135 - T.O.C.

156 - W.L.

8.7.

Bechtel Corporation

Inter-office Memorandum

To	Files	Date	May 10, 1972
Subject	Investigation for Dewatering of Plant Excavation, Vogtle Nuclear Plant Job No. 9510-001	From	C. R. Farrell
		Of	Geology
Copies to	W. Holland A. Luft C. McClure W. Ferris	At	E & I Division

On Friday, April 28, R. Bush, consulant to the project, attended a meeting in our offices to discuss dewatering problems we might expect in the excavation for the plant site, and at the water intake structure near the river. I briefly attended the meeting to provide clarification of our intrepretation of ground water conditions at the site.

Site Excavation Dewatering

Mr. Bush is concerned that wells might not be an effective means for dewatering the area. He is basing this concern on the information collected to date; pump-in tests within the shallow sands and the description of materials in the shell zone overlying the marl (bearing unit). Although the experience of drilling and knowledge of the materials suggests that the shell zone is relatively high in permeability, it is not certain that it would act as an effective underdrain for dewatering the overlying sands. Should the proposed plan for well points in the shell zone not adequately drain the sands, serious delay in construction scheduling, as much as 2 or 3 months, could occur. I agreed with Bush that our knowledge of the permeabilities was not firm enough to preclude this possibility. It was decided that a testing program be conducted.

Test wells selected at two sites, representing the most favorable conditions and the least favorable conditions, as evidenced from our exploration of the site for Units 1 and 2, will provide data to evaluate a well system. After selecting the sites, and preparing a tentative construction plan, I contacted Layne-Atlantic of Savannah, Georgia, concering their availability to do the work. After verifying their willingness, I contacted R. Bush by telephone, Thursday, May 3, to review the details of test well construction.

There was apparently some misunderstanding as to Bush's primary objective for the test wells; I had thought it was to determine the permeability of the shell zone. Although this will be desireable, Bush is first concerned about the maximum yield of wells. Construction wise, this does not make a large difference (primarily it will call for 15 to 20 feet of perforations opposite the upper sands also, in order to intercept all inflows of water available to the well.

With these additional factors in mind, the test wells and observation points to be constructed will consist of the following:

<u>Test Wells</u> (2)

Depth: 80 feet (± 5 feet)
Diameter of bore: 12-inch
Casing diameter: 6-inch
Well screen: length; 15 feet
diameter; 4-inch
slot opening; 1/8-inch

<u>Observation Points</u>

Quantity: 3 points for each well
Depth: 80 feet (± 5 feet)
Diameter of bore: 4-inch
Casing diameter: 2-inch
Screen: length; 15 feet
diameter; 2-inch
slot opening; 1/8-inch

After placing the screen and casing in the bore, the annular space in the wells and the observation points will be filled with clean, fine-gravel up to height of 15 feet above the screened intervals. During placement of the gravel, clean water will be pumped through the casing to clean the hole of drilling fluid. The observation points will then be "pumped" by air injection to confirm hydraulic continuity with the aquifer zone.

The wells will be developed by pumping, possibly preceded by air injection. It is anticipated that 8 to 12 hours of development will be sufficient before commencing a testing of the well. The pumping tests will be conducted at a constant discharge rate for a continuous period of 72 hours (3 days).

I have asked Terry Scafidi of Layne-Atlantic to submit an estimate of cost for the work as a lump sum to be added to the present contract. He will submit an estimate by the end of this week. They would be able to conduct the work following completion of the test well construction and testing.

Intake Structure

Invert elevation of the intake structure adjacent to the Savannah River will be at elevation 54 feet, or approximately 10 feet below the base of the marl. Piezometric levels measured at various depths below the marl in the vicinity of the plant site indicate the level below the marl is at elevation 110 feet.

However, where the confining marl is breached, as in the river channel, the upward flow reduces the point hydraulic head, and it is believed that piezometric levels adjacent to the river will not be as high as 110 feet. This will be significant both for dewatering at the intake structure and in considering possible uplift pressures. It is therefore, recommended that an observation point be placed at the intake structure, to a depth corresponding to elevation 45. The point should be isolated by grouting the annular space above elevation 65. This could possibly be done by a Law Engineering drilling rig presently at the site conducting soils exploration for Units 3 and 4. Following completion of that work, a piezometer could be easily constructed by them, as they are familiar with the site and have placed similar ones in the vicinity. It is my understanding that data for dewatering conditions are not needed for the PSAR, so that construction of the piezometers can be planned on the availability of a drilling rig. If it is not convenient for Law Engineering to do it, we can arrange for placement of the point by Layne-Atlantic.

CR Farrell

C. R. Farrell

P. 7.

R. Y. Bush
Consulting Engineer
543 N. Stanford Avenue
Fullerton, California 92631
Telephone (714) 879-7812

DEWATERING STUDY

ALVIN W. VOGTLE NUCLEAR PLANT
GEORGIA POWER COMPANY

Purpose:

The purpose of this report is to present the results of our study of the dewatering problem anticipated in connection with the construction of the subject project.

Description of Study

This investigation consisted of a review of preliminary construction drawings; studies of geological information which included borings logs, a draft of a ground water report by Mr. C. Farrell of Bechtel, and various maps of geological conditions at the site; studies of rainfall intensity as related to possible flood damage in the excavation area; analyses of pump test data obtained by your personnel; and the preliminary design of a combination dewatering and storm water pumping system. As of the date of this report, the writer has not had an opportunity to personally visit the project site.

Groundwater Conditions

The report draft on groundwater conditions by Mr. Farrell provided valuable information. Significant items contained in this report are:

How deep?

1. "The impervious marl, or bearing unit, acts as an aquiclude (impervious barrier) to groundwater."
2. The only source of recharge to the unconfined groundwater above the marl is rainfall, and
3. A highly pervious shell zone of limited thickness (10'+-) exists directly above the marl.

January 12, 1973

The report describes the outflow from Mathes Pond as an estimated 300 gpm which is considered to be the amount corresponding to a final equilibrium condition during dewatering. It is pointed out that initial pumping for dewatering would be considerably greater than this amount. An excellent check on the 300 gpm was obtained by a planimeter measurement of the tributary area to the site, which appears to be about 367 acres. For this area, a rate of 300 gpm would correspond to 50 inches per year with 30% infiltration, both reasonable values.

?

Data obtained from two pump tests were analyzed. Descriptions of the test wells follow:

1. Well #1-total depth 94'; white sand with shells 72'-80'; marl below 80'; coordinates N1, 142,660 and E623,570.
2. Well #2-total depth 87'; white sand with shells 52'-61'; shell, hard, limestone 61'-85'; marl below 85'; coordinates N1, 143,225 and E623,075. In addition to the pumped wells, 8 observation wells, 4 per test well, were installed to permit the measurement of water levels during pumping.

Well #1 was pumped for approximately four days at rates of generally in the range of 30 to 38 gpm. Well #2 was pumped for about 27 hours at rates of 10 to 15 gpm. Pumping on well #2 was discontinued due to the lack of response of the water levels in the observation wells. Additional "pump in" tests were performed on well #2 observation wells. Due to the relatively small rate of pumping from well #2 and the correspondingly small amount of lowering of water, a quantitative evaluation of permeability was not possible in this case. This test did indicate that transmissibility at this location is very small.

Data obtained from test well #1 was analysed on the basis of nonequilibrium methods, using data obtained during both drawdown and rebound periods. Attached plots indicate fair agreement between the various observation wells with the exception of 1-c.

The erratic behavior was due to interruptions in the rate of pumping and to a lesser extent due to variations in barometric pressure during a storm period.

Based on our analysis, the transmissibility of the unconfined aquifer is estimated to be in the range of 0.7 to 1.8 ft^2/min. (7,560 to 19,440 gal/day ft.). Corresponding permeability values, based on an aquifer thickness of 10' would be 0.07 to 0.18 ft./min. Average permeabilities in the area are probably less than this due to the fact that well #1 was probably located in a relatively high permeability area. Considering the variable nature of the shell zone, a wide range of local permeability should be anticipated.

Permeable material is considered 300-1000 gals/day/ft^2.

Dewatering and Pumping

Although the apparent permeability of the shell zone is relatively high, because of its limited thickness, the transmissibility of the aquifer is quite low. Due to this condition, which results in low individual well capacity, the application of predraining methods employing deepwells or wellpoints is not considered practical or economically feasible.

The volume of water to be removed during the initial dewatering period until the final "equilibrium condition" is reached, is estimated at about 140,000,000 gal. An average rate of 1,000 gpm would therefore require about 100 days which should coincide reasonably well your anticipated excavation rate. An initial dewatering plant having a minimum capacity of approximately 1,500 gpm is recommended. The rate of pumping would gradually decrease with time until finally the sustained condition, estimated at 300 gpm, is reached.

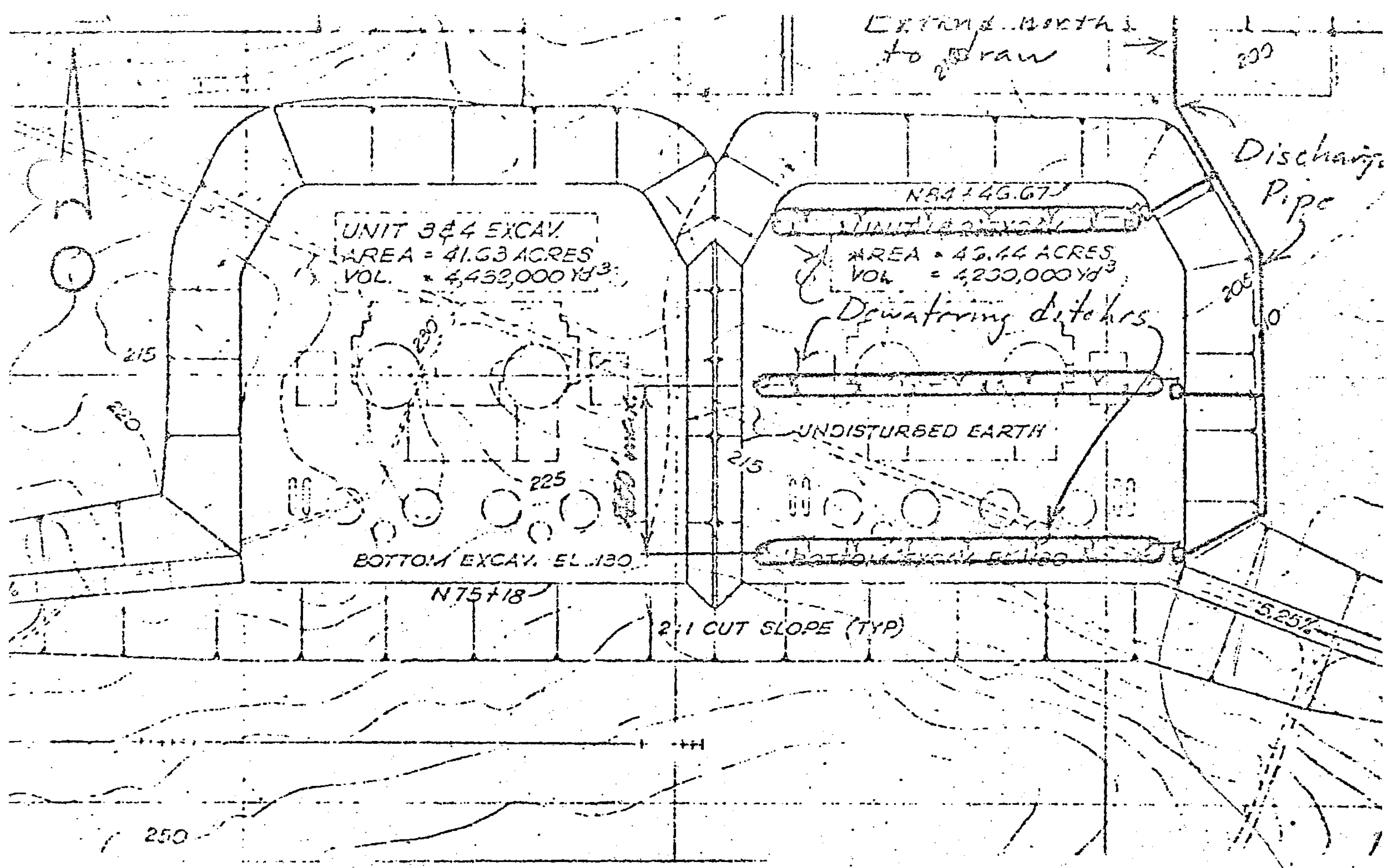
A system of ditches and sumps is recommended to perform this dewatering. The basic scheme is illustrated on figures 3,4,5,6. It should be emphasized that the sketches are of necessity quite rough and should be considered as schematic only. It is recognized that various construction considerations unknown to the writer could necessitate the extensive revision of the layouts as proposed.

The basic dewatering scheme proposed consists of the following:

1. Preliminary excavation is made to an elevation slightly above the initial water table.
2. Ditches are excavated across the excavation area to allow the wet materials to drain by gravity flow through the ditches to sumps from which the water is pumped. It should be noted that the spacing of the ditches is indicated as a 400' maximum. This is to insure that dewatering of the materials between ditches occurs in a reasonably short period of time.
3. Excavation continues to the surface of the marl, the bearing material, at which time the rate of pumping should have diminished to a relatively small rate, approaching the sustained rate.
4. At this time ditches are excavated in the marl to provide drainage during periods of high intensity rainfall. This item is discussed in greater detail subsequently.
5. Prior to backfill, a perimeter porous drain pipe is installed to allow dewatering during the backfill period. This drain leads to vertical pump wells from which the water can be pumped during the backfilling operation. This pumping on the perimeter drain would continue until backfill has reached a sufficiently high elevation, and the weight of the concrete placed is sufficiently heavy so that no further control of hydrostatic uplift is required.

Stormwater Pumping

The major pumping requirement will be to remove stormwater from the excavation during periods of high intensity rainfall which must be anticipated in this area. The combined effects of this high intensity rainfall with the extremely large area of the excavation results in extremely high rates of pumping required during storms to keep the excavation free of water. Figure 7 illustrates a simple plot of rainfall intensity versus gal/min for the area



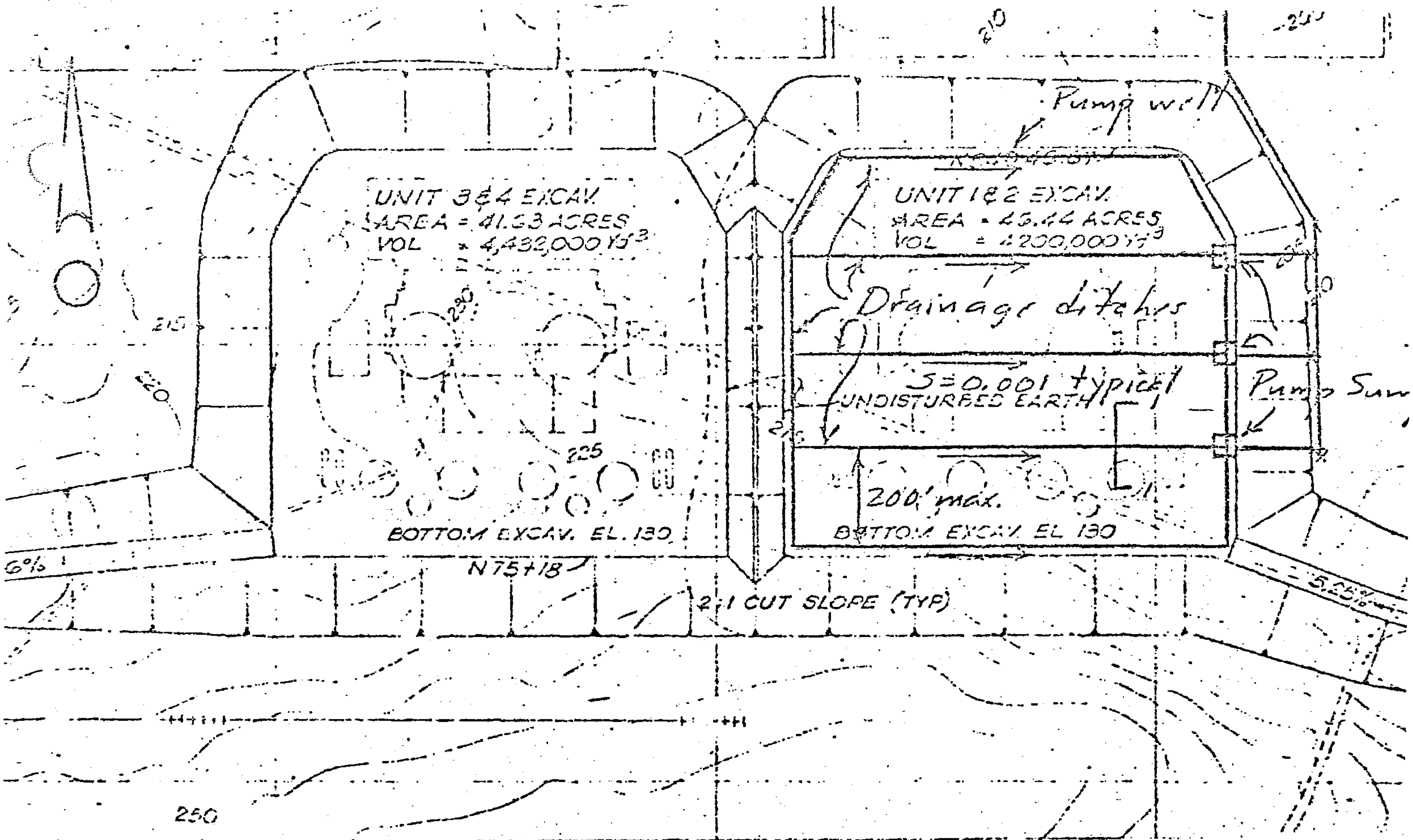
Stage I

1. Install discharge pipe. Size of line to be based on maximum design rate of pumping to handle storm water.

2. Excavate with scraper equipment down to about 3'± above water table.

3. Excavate drainage-dewatering ditches with Gradall or backhoe. Maximum spacing between ditches to be 400'. Provide initial pumping capacity of 1500 gpm minimum. Final pumping rate for de-watering estimated at 300 gpm.

Figure 3

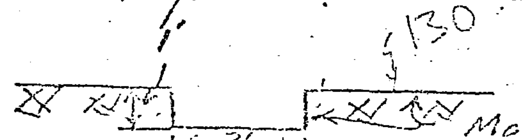


Stage II

1. After completion of excavation to subgrade, cut storm water drainage ditches in marl as indicated. Install storm water pumps.

2. Prior to starting backfill, install porous concrete drain pipe around perimeter in a properly graded filter envelope to allow dewatering during period of backfilling up to water table. Install pump well in backfill.

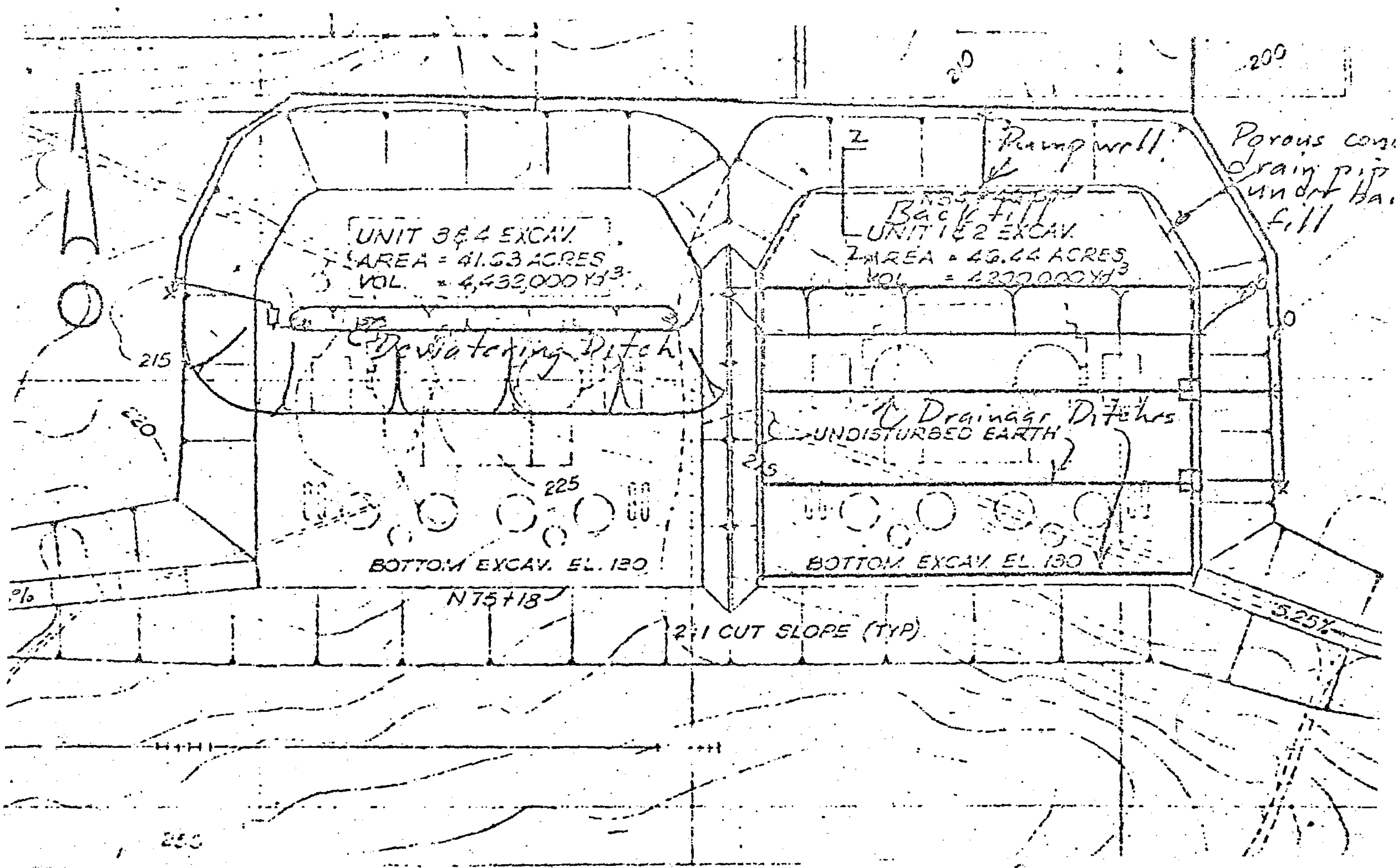
3. Provide appropriate ditches in backfilled areas to allow drainage of storm water as required.



Section 1-1 Typ

N.T.S.

Figure 4

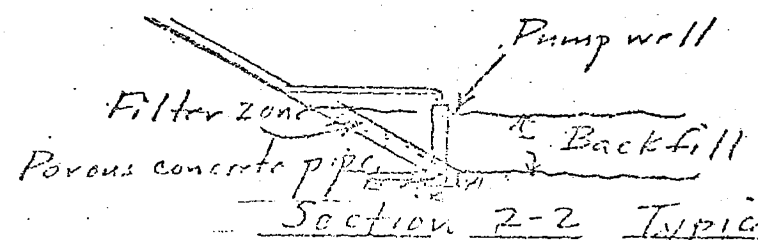


Stage III (Phase III Excavation, Drwng SK-C-5...

1. Dewatering in Units 1 & 2 Turbine Bldg area by perimeter drain during backfill.

2. Install additional discharge piping to Units 3 & 4 area. Dewatering by ditching similar to Stage I.

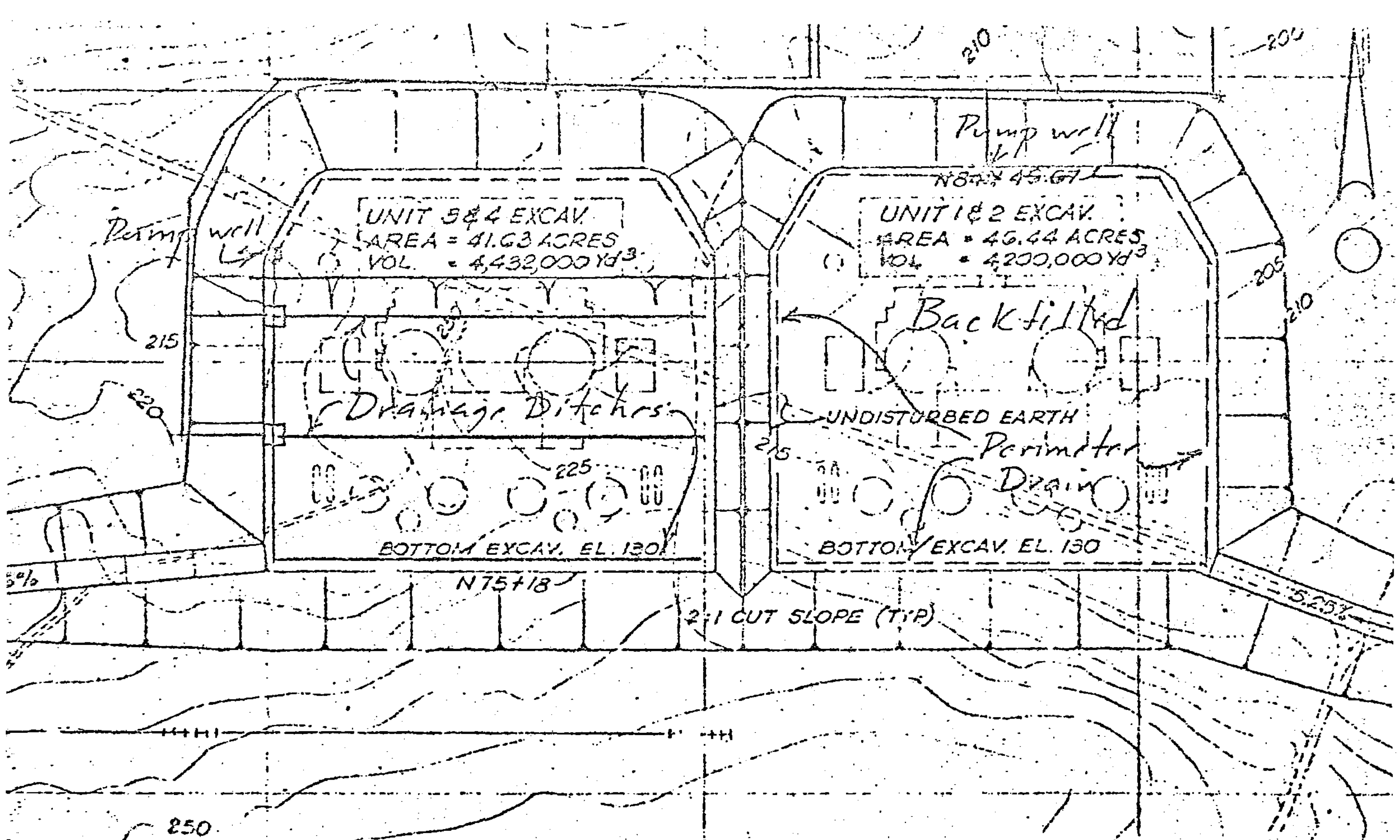
3. Provide storm water pumping equipment in areas as required.



Section 2-2 Typical

N.T.S.

Figure 5



Stage IV (Phase IV Excavation, Drwng SK-C-51)

1. Continue pumping on perimeter drain in Unit 1&2 area as required to (a) permit backfilling in "the dry," (b) control hydrostatic uplift under partially completed structures, and (c) control seepage from east for completion of construction of Units 3 and 4

2. Install additional discharge piping to south portion of Units 3 and 4 area. Dewater by ditching similar to Stage I.

3. Excavate ditches in marl for handling storm water similar to Stage II.

4. Install perimeter drain around north, west, and south sides of Unit 3 and 4 area. Complete construction.

Figure 6

PROJECT Alvin W. Vogtle Nuclear Plant PROJECT NO. 230
BY RHB DATE 1/11/73 CHECKED BY ______ DATE ______ SHEET NO. ___ OF ___

Stormwater Pumping Study

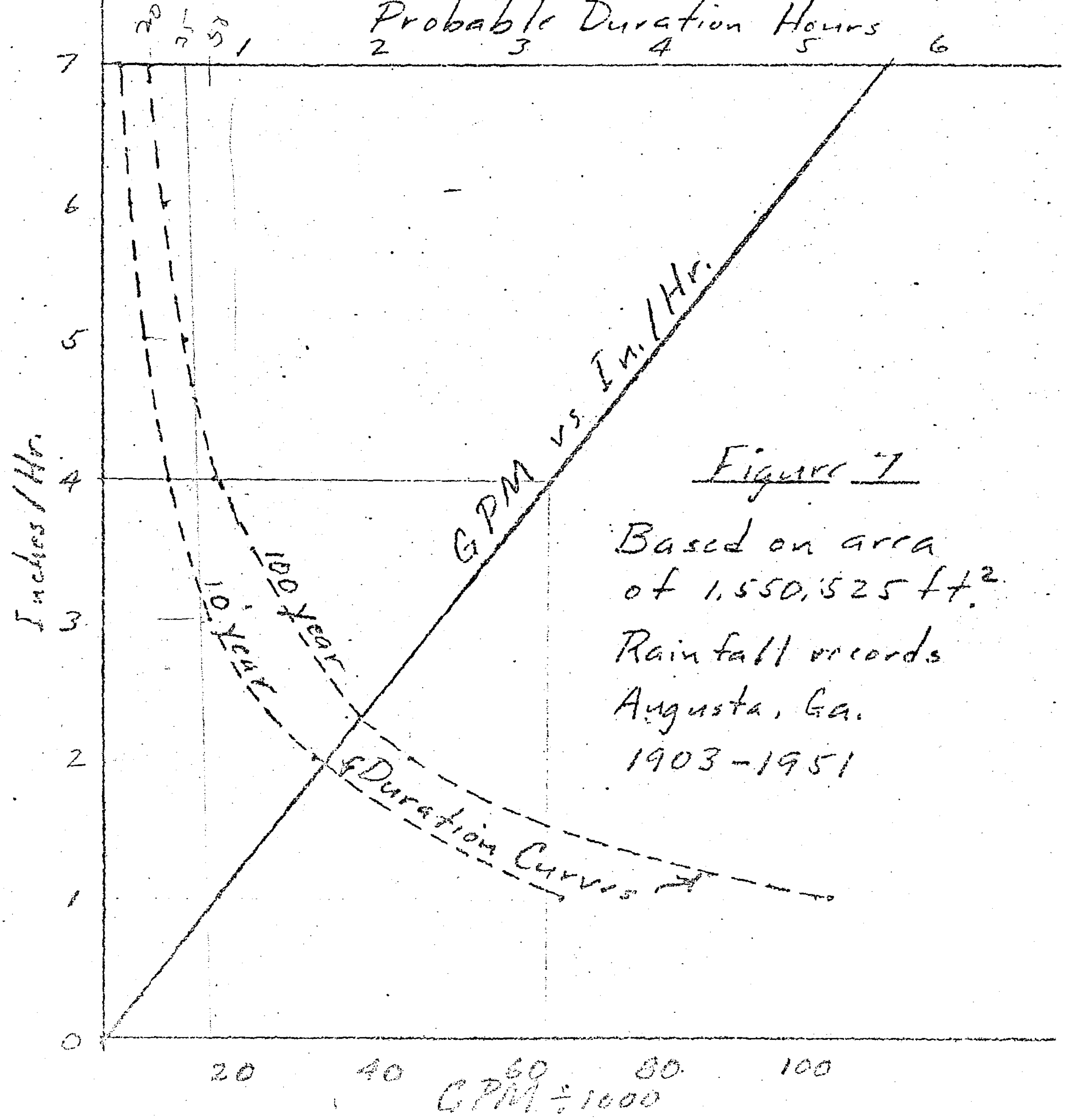
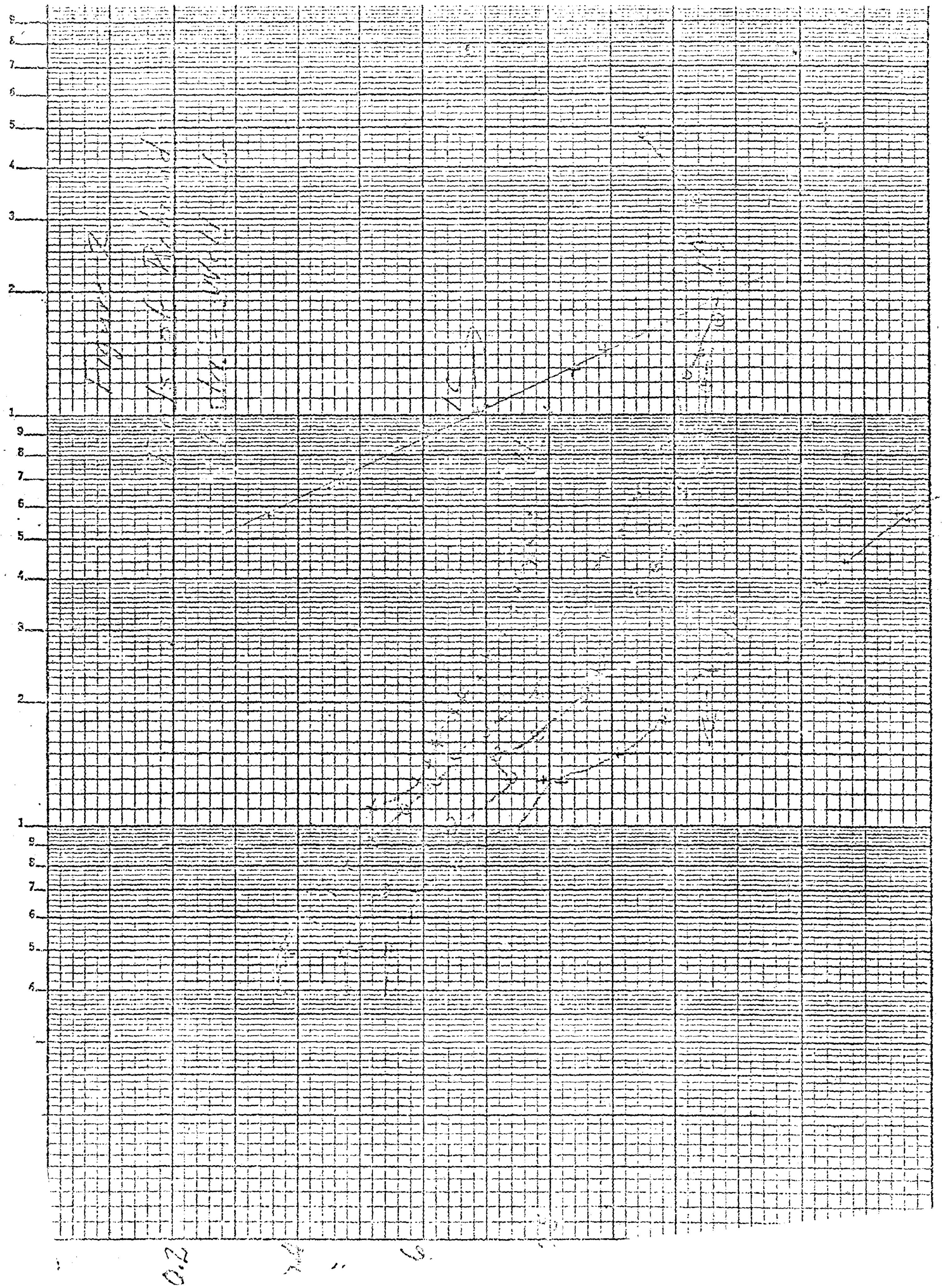
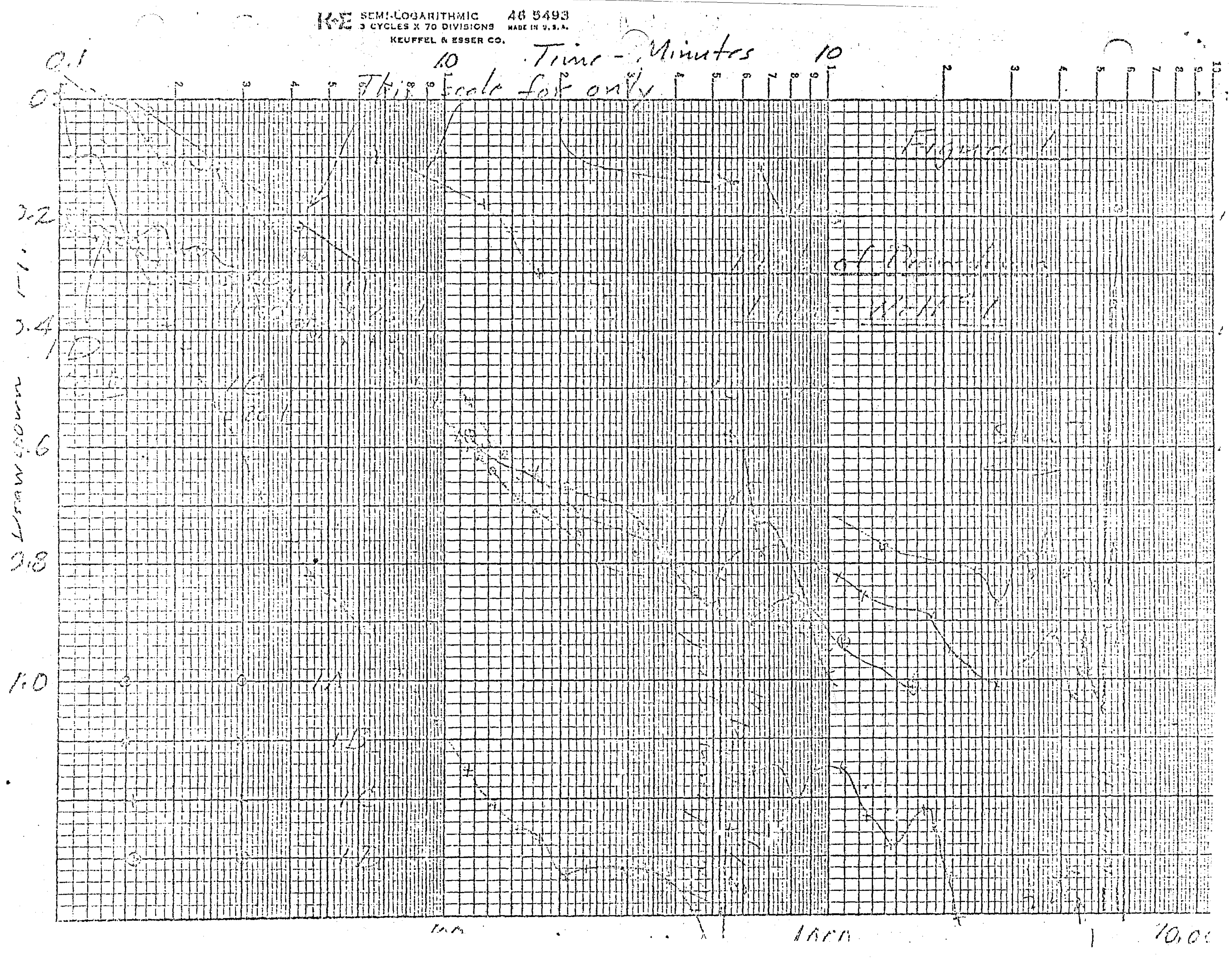


Figure 7

Based on area of 1,550,525 ft.^{2}

Rainfall records Augusta, Ga. 1903-1951




K+E SEMI-LOGARITHMIC 46 5493
3 CYCLES X 70 DIVISIONS MADE IN U.S.A.
KEUFFEL & ESSER CO.
Time - Minutes
This scale for only
0.1
10
10
Figure
0
0.2
0.4
0.6
0.8
1.0
Drawdown

represented by Units 1 and 2. It is assumed that the top of the excavation slope is provided with proper drainage ditches and that therefor only rainfall falling on the actual excavation area would be pumped from it. Consideration should be given to the use of appropriate stabilizing materials to the slope to minimize erosion.

During normal conditions, only a portion of the pumping equipment would be required to operate for dewatering; that is, handling the groundwater entering the excavation. <u>The design of the discharge piping for the combined system would obviously be based on the pumping rate during the storm period</u>. The actual size of the system must be based on a careful consideration of the financial consequences of a heavy rainstorm due to damage caused to concrete and other operations, weighed against the probability of extreme storms occurring say of the 50 to 100 year variety. We will not attempt to evaluate this complex problem since we are not sufficiently acquainted with the various cost and construction considerations involved on this project. It would appear that a pumping plant to provide reasonable protection against storm damage <u>should have a capacity in the range of from 5,000 to 10,000 gpm.</u>

5000+1000

<u>Conclusions and Recommendations</u>

1. Although the permeability of the shell zone immediately above the marl appears to be quite high, due to its limited thickness, the transmissibility in this area is quite low, in the range of 0.7 to 1.8 $ft.^2$ / min.
2. Due to the limited thickness of the pervious zone directly above the marl, along with other considerations such as the difficulty and high expense of drilling, the application of predraining methods employing wellpoints or deepwells is considered impractical and economically not feasible.
3. A method of ditches and sumps should be used to perform the dewatering of the excavation.
4. The size of the pumping plant provided should be based on a consideration of handling stormwater since this pumping rate will greatly exceed the anticipated rate of dewatering.

~~5. A perimeter drain should be installed to allow dewatering and~~

5. A perimeter drain should be installed to allow dewatering and hydrostatic uplift control during backfill operations.

R. Y. Bush

R. Y. Bush

January 12, 1973