JOHN E. SIRINE AND ASSOCIATES, LTD.
SURVEYORS AND ENGINEERS
4317 BONNEY ROAD
VIRGINIA BEACH, VIRGINIA 23452
(804) 486-4910

JOHN E. SIRINE, C.L.S.
VIRGINIA AND NORTH CAROLINA
KENTON B. PATRICK, P.E.
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PAUL E. GARRETT, C.L.S.
VIRGINIA AND NORTH CAROLINA

ROBERT R. PETERS, P.E.
VIRGINIA
CHARLES E. NEWBAKER, III, C.L.S.
VIRGINIA
DENNIS J. GERWITZ, P.L.S.
NEW YORK

February 14, 1984

Cape Hatteras Water Association, Inc.
Box 578
Buxton, North Carolina 27920

Re: Evaluation of Potential Ground-Water Development
on Hatteras Island

Gentlemen:

We are pleased to enclose our report on the potential ground water which may be developed on Hatteras Island.

We appreciate this opportunity to be of service to you.

Respectfully submitted,

JOHN E. SIRINE AND ASSOCIATES, LTD.

Robert R. Peters, P. E.
Executive Vice President

RRP:b

EVALUATION OF POTENTIAL GROUND WATER DEVELOPMENT ON HATTERAS ISLAND

FOR

CAPE HATTERAS WATER ASSOCIATION, INC.
BOX 578
BUXTON, NORTH CAROLINA 27920

By
Robert R. Peters, P. E.

February 14, 1984

John E. Sirine and Associates, Ltd.
Surveyors-Engineers-Planners
4317 Bonney Road
Virginia Beach, Virginia 23452
(804) 486-4910

Introduction

The Cape Hatteras Water Association, Inc. is currently investigating water supply improvements for future expansion of its existing water system. John E. Sirine and Associates, Ltd. has conducted an on-site investigation to ascertain the present condition and system. In addition, we have consulted with the North Carolina Department of Natural Resources and Community Development to collect all available data on the water supply for the Hatteras Island area. In addition, we have consulted with the United States Geological Survey to obtain and confirm the same information.

The purpose of this report is to assist the Cape Hatteras Water Association by providing information in regard to the availability of ground water supply additions on Hatteras Island and the impact of such development and to ascertain the long-range potential yield of the aquifer system. This is a preliminary evaluation and is based on personal observation together with what information is available. No detailed analyses or tests have been conducted.

Ground Water Availability

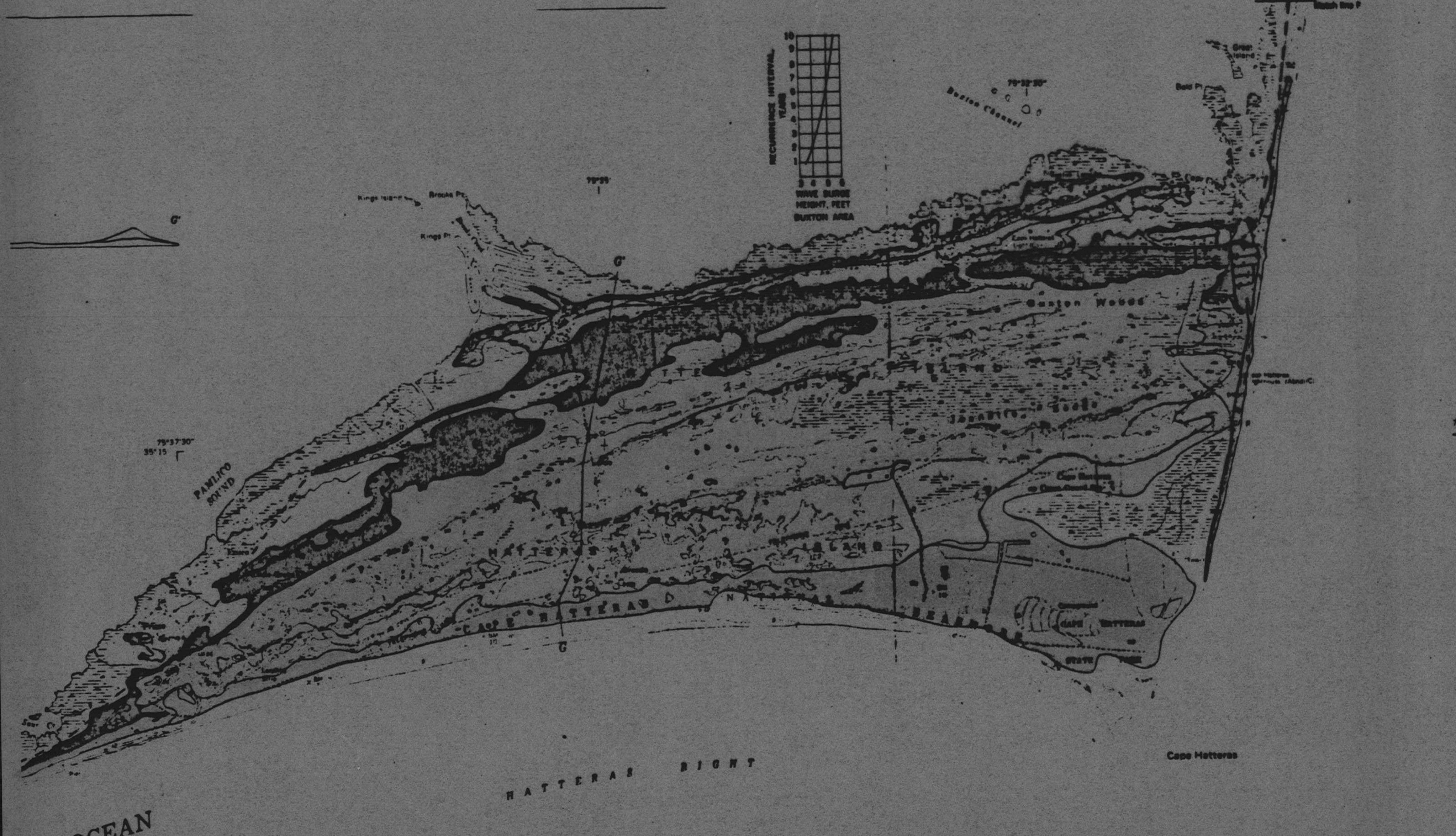
The Cape Hatteras Water Association, Inc. system is located on Hatteras Island, which is one of a chain of barrier islands known locally as the Outer Banks that extend from the Virginia State Line southward for approximately 175 miles. Hatteras Island is about 600 feet wide near the village of Hatteras and nearly three miles wide at Cape Hatteras. The area for potential ground water development extends from approximately Billy Mitchell Field, near Frisco, to Buxton, a distance of approximately five miles. On the ocean side, it is bordered by an almost continuous dune ridge, the crest of which is as much as 20 feet above sea level. The center section consists of numerous fresh water swamps that are below the crest of the dunes. The yearly average rainfall is approximately 50 to 55 inches. (Refer to Figure One.)

Assuming an average rainfall of 50 inches, this would mean a total rainfall on the five square mile area of 4.356 billion gallons per year. The very sandy area allows considerable recharge, up to approximately 80 percent of this amount; however, for safety purposes, we are only assuming a recharge of one-half of this total precipitation, which means we have 2.178 billion gallons of water that are going into the underground aquifer system per year. (Refer to Figure Two.)

Present usage during the summer period is approximately one million gallons per day, which could calculate out to 365 million gallons per year. By increasing this withdrawal by one hundred percent, we would be withdrawing 730 million gallons per year. This, then, means that one and one-half billion gallons of water per year would still be flowing into the ocean from Cape Hatteras Island.

These figures are conservative and are not taking into account evaporation and transpiration. Practically all of the water from rainfall is recharged to the ground; therefore we feel that the fifty percent figure is a safe number to be used as water available in the aquifer system.

The water table aquifer system is basically a large fresh water



RECURRENCE INTERVAL, YEARS
WAVE SURGE HEIGHT, FEET
BUXTON AREA
Buxton Channel
Buxton Woods
Kings Pt.
Brooks Pt.
Bald Pt.
PAMLICO SOUND
CAPE HATTERAS
HATTERAS BIGHT
OCEAN
Cape Hatteras
G
G'
79°37'30"
35°15'
79°35'
79°32'30"

FIGURE ONE

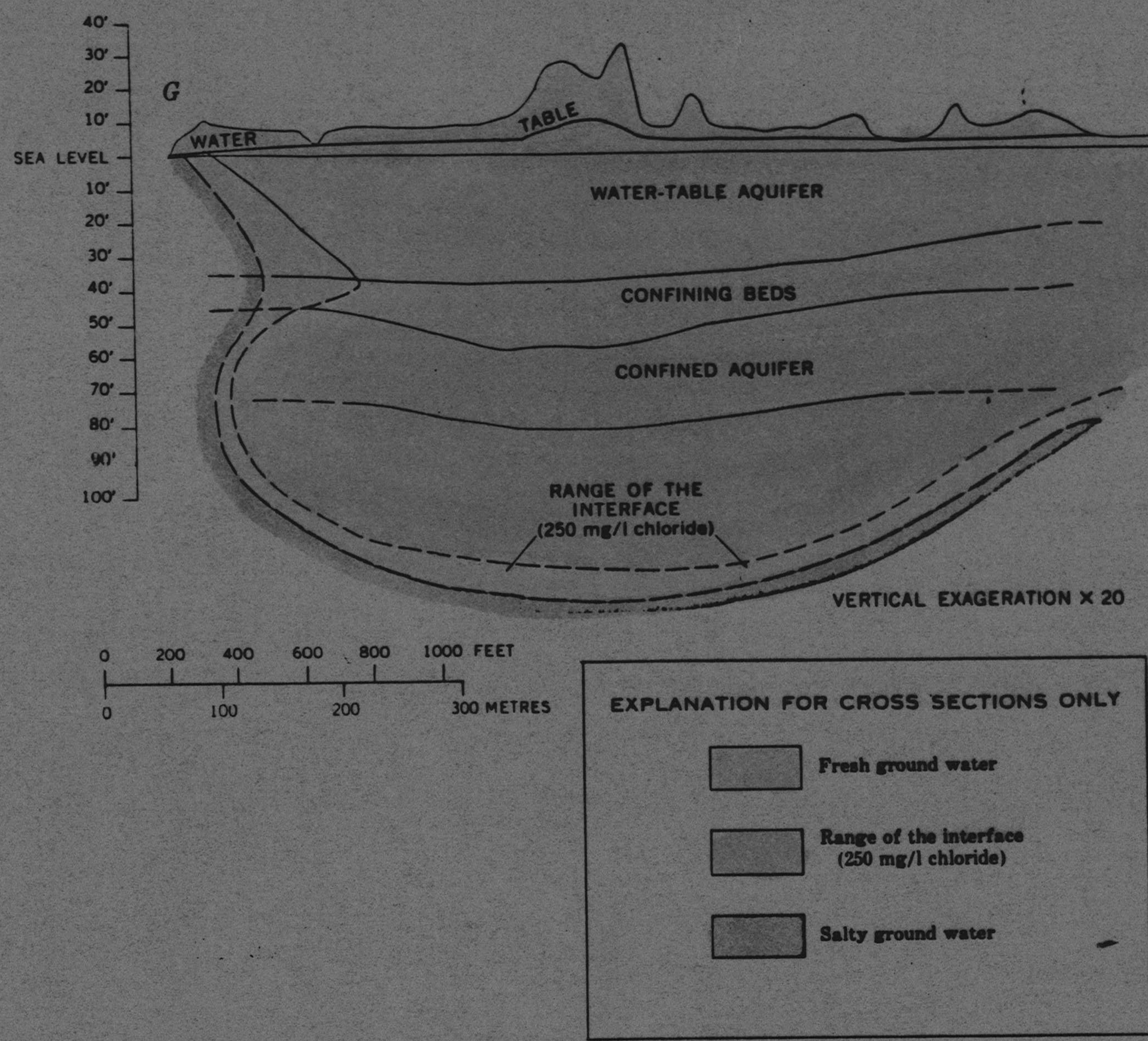

40'
30'
20'
10'
SEA LEVEL
10'
20'
30'
40'
50'
60'
70'
80'
90'
100'
G
WATER
TABLE
WATER-TABLE AQUIFER
CONFINING BEDS
CONFINED AQUIFER
RANGE OF THE INTERFACE (250 mg/l chloride)
VERTICAL EXAGERATION X 20
0
200
400
600
800
1000 FEET
0
100
200
300 METRES
EXPLANATION FOR CROSS SECTIONS ONLY
Fresh ground water
Range of the interface (250 mg/l chloride)
Salty ground water

FIGURE TWO

bubble that is floating on an underlying salt water system. As long as proper above sea level water levels are maintained, this system will remain intact. However, the only method for extraction of water in this island-type aquifer condition is by limited pumping of individual wells over distances, thereby skimming the fresh water off the top of the salt water system.

Underlying this water table aquifer system is a relatively impervious formation that prevents sizable downward movement and also limits upward coning, or intrusion, into the water table aquifer system.

The other area of importance is the very swampy area lying approximately down the center of Cape Hatteras Island. The major danger for a loss of this water supply would be a salt water intrusion problem, which could occur either laterally or vertically by improper construction of wells or improper spacing of same.

The factor limiting this investigation has been the lack of data available to establish water quality and quantity trends. It is extremely important that water level data be maintained on pumping wells together with quality analyses of individual wells. In addition, observation wells should be placed at strategic points to record water levels and water quality.

In order to maintain equilibrium in the aquifer system over a significant period of time, the amount of recharge to the system must equal the amount of recharge. Any reduction in the recharge versus the amount of withdrawals will cause the salt water interface to move inland from the ocean and eventually destroy the ground water aquifer system.

This, however, would occur over a relatively long period of time; therefore there is the necessity for monitoring wells at strategic locations to provide knowledge at an early date when ample time is available for corrections to be made.

The other potential source of salt water intrusion would be upward, and this would occur by overpumping of wells or spacing them too closely together. These data can be obtained by the conducting of pumping tests with observation wells to ascertain the maximum capacities or safe yield from each individual well field system.

The swampy area, or shallow lake system that occurs in the center of the Island, is also a good indicator for water level declines and at several points should have gauging stations installed within them.

Conclusions

(1) It is our conclusion from visual observations that the present water resources of one million gallons per day can be maintained.

(2) With the data available at the present time, it can also be assumed that this supply can be expanded one hundred percent to a capacity of two million gallons per day.

Recommendations

(1) The present system should be monitored as to capacities, water levels and qualities of individual wells.

(2) All future wells should be monitored as to capacity, water level and quality.

(3) A series of observation wells should be installed at critical points to ascertain the possibility of water level changes or salt water intrusion. These wells would also be monitored and water level and periodic analyses of the quality conducted.

(4) In order to develop the new well field, a pumping well should be installed with several observation wells at specified distances and a pump test conducted on same to establish hydraulic data which will enable safe yield pumping from individual wells and well spacing.

(5) Gauging stations should be installed in several of the lakes and this data recorded.

REFERENCES

Winner, Jr., M. D., 1975, Ground Water Resources of Cape Hatteras Seashore, North Carolina.

PUMPING TEST AND EVALUATION

OF

POTENTIAL GROUND WATER

DEVELOPMENT ON HATTERAS ISLAND

FOR

CAPE HATTERAS WATER ASSOCIATION, INC.
BOX 578
BUXTON, NORTH CAROLINA 27920

By
Robert R. Peters, P. E.

October 15, 1984

JOHN E. SIRINE AND ASSOCIATES, LTD.
SURVEYORS-ENGINEERS-PLANNERS
4317 BONNEY ROAD
VIRGINIA BEACH, VIRGINIA 23452
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VIRGINIA
DENNIS J. GERWITZ, P.L.S.
NEW YORK

October 15, 1984

Cape Hatteras Water Association, Inc.
Box 578
Buxton, North Carolina 27920

Re: Pumping Test and Evaluation of Existing Well Field

Gentlemen:

We are pleased to enclose our report on the pumping test and evaluation of your existing well field.

We appreciate this opportunity to be of service to you.

Respectfully submitted,

JOHN E. SIRINE AND ASSOCIATES, LTD.

Robert R. Peters, P. E.
Executive Vice President

RRP:b

Enclosures

PUMPING TEST AND EVALUATION

OF

POTENTIAL GROUND WATER

DEVELOPMENT ON HATTERAS ISLAND

FOR

CAPE HATTERAS WATER ASSOCIATION, INC.
BOX 578
BUXTON, NORTH CAROLINA 27920

By
Robert R. Peters, P. E.

October 15, 1984

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INTRODUCTION

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The purpose of this report is to assist the Cape Hatteras Water Association by providing information in regard to the availability of ground water supply additions on Hatteras Island and the impact of such development and to ascertain the long-range potential yield of the aquifer system. The preliminary evaluation has been done and was based on personal observations together with what information is available. A pumping test has been conducted and is included in this report.

GROUND WATER AVAILABILITY

The Cape Hatteras Water Association, Inc. system is located on Hatteras Island, which is one of a chain of barrier islands known locally as the Outer Banks that extend from the Virginia State Line southward for approximately 175 miles. Hatteras Island is about 600 feet wide near the village of Hatteras and nearly three miles wide at Cape Hatteras. The area for potential ground water development extends from approximately Billy Mitchell Field, near Frisco, to Buxton, a distance of approximately five miles. On the ocean side, it is bordered by an almost continuous dune ridge, the crest of which is as much as 20 feet above sea level. The center section consists of numerous fresh water swamps that are below the crest of the dunes. The yearly average rainfall is approximately 50 to 55 inches.

Assuming an average rainfall of 50 inches, this would mean a total rainfall on the five square mile area of 4.356 billion gallons per year. The very sandy area allows considerable recharge, up to approximately 80 percent of this amount; however, for safety purposes, we are only assuming a recharge of one-half of this total precipitation, which means we have 2.178 billion gallons of water that are going into the underground aquifer system per year.

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These figures are conservative and are not taking into account evaporation and transpiration. Practically all of the water from rainfall is recharged to the ground; therefore we feel that the fifty percent figure is a safe number to be used as water available in the aquifer system.

The water table aquifer system is basically a large fresh water bubble that is floating on an underlying salt water system. As long as proper above sea level water levels are maintained, this system will remain intact. However, the only method for extraction of water in this island-type aquifer condition is by limited pumping of individual wells over distances, thereby skimming the fresh water off the top of the salt water system.

Underlying this water table aquifer system is a relatively impervious formation that prevents sizable downward movement and also limits upward coning, or intrusion, into the water table aquifer system.

The other area of importance is the very swampy area lying approximately down the center of Cape Hatteras Island. The major danger for a loss of this water supply would be a salt water intrusion problem, which could occur either laterally or vertically by improper construction of wells or improper spacing of same.

The factor limiting this investigation has been the lack of data available to establish water quality and quantity trends. It is extremely important that water level data be maintained on pumping wells together with quality analyses of individual wells. In addition, observation wells should be placed at strategic points to record water levels and water quality.

In order to maintain equilibrium in the aquifer system over a significant period of time, the amount of recharge to the system must equal the amount of discharge. Any reduction in the recharge versus the amount of withdrawals will cause the salt water interface to move inland from the ocean and eventually destroy the ground water aquifer system. This, however, would occur over a relatively long period of time; therefore there is the necessity for monitoring wells at strategic locations to provide knowledge at an early date when ample time is available for corrections to be made.

The other potential source of salt water intrusion would be upward, and this would occur by overpumping of wells or spacing them too closely together.

These data can be obtained by the conducting of pumping tests with observation wells to ascertain the maximum capacities or safe yield from each individual well field system.

The swampy area, or shallow lake system that occurs in the center of the Island, is also a good indicator for water level declines and at several points should have gauging stations installed within them.

OBSERVATIONS

Well data as per Exhibit 'A" has been recorded and where possible included the amperage loading, the depth, the screen setting, the pumping level below the top of the casing, the well construction data, static levels and design capacities. This information also has notations of conditions found at time of observation. In addition, on page one of Exhibit 'A', we also noted the two different types of flow control devices that were being used. Of particular notation it appeared that the flow control devices were restricting capacities of the wells. We recommended that these flow control devices be removed and flow control be maintained with a gate valve.

In computing the capacities of the wells, we found that the below ground head and the above ground head, together with friction loss in the pipeline, would not allow the wells to pump in excess of 20 to 25 gpm each. From numerical data on the submersible well pumps that you are using at the present time, which is a Berkley Model 4BM7, we have plotted a capacity and head curve which points out this information. (Exhibit 'B').

We conducted a pumping test on Well No. 21. The reason for selection of this well was there were three observation wells in the immediate area. One observation well was located six feet from the pumping well and at a depth of 88 feet. Another observation well was located a distance of seven feet from the pumping well and had a total depth of 8'-8". A third observation well was located at a distance of 18 feet from the pumping well and was at a depth of 124 feet.

The pumping test was conducted for a period of four hours at which time the pumping level dropped to 11.65 feet. We recorded the water levels and plotted the recovery test with the water level recovering to 5.65 feet in 350 minutes. We have plotted a distance drawdown for this unit, and it appears we have no interference beyond 200 feet of the well. With this information we feel the capacity of the wells in the well field can be increased to 25 gpm each.

At the present time you have a total of 35 wells in existence. Utilizing the present well field, you can add eight additional wells by installing Wells 1, 1A, 2, 2A, 3A, 4A, 5A and 6A. This would provide a total of 43 wells in the existing well field. Allowing for capacity to be increased to 25 gpm will give you a total of 1,075 gpm from the existing field.

You are desirous of producing a total of 1,400 gpm or 2 million gallons per day for your system, which would require an additional 13 wells be installed and we would recommend two spares. Well No. 19 is out of service at the present time due to what has been reported as a field pumping condition. This may possibly be rectified by cleaning out this well, air surging and swabbing. However, there is no guarantee that this work would be successful, and it would be on a trial basis to see if it could be rectified. If this could not be rectified, however, a replacement well could be installed in the area utilizing present electrical and pipeline availability. (Exhibits 'C', 'D', 'E', 'F').

If the additional well field is increased to 1,400 gpm, this capacity carries an increased friction head in the existing pipeline to a point whereby you could not get the flow into the plant with the present well pumps. A rather inexpensive method of correcting this would be to install a booster pump at a point in the line to overcome this friction loss. This would require approximately 15 horsepower to operate. The cost to install this unit would be much less than the replacement cost of the well pumps in the field. However, a detailed analysis of flow in your pipeline and its existing internal condition may require some paralleling of the pipeline to reduce these heads.

CONCLUSIONS

(1) The results of the testing done allows us to conclude that the present field can be expanded to a capacity of 25 gpm for each well.

(2) Eight additional wells can be installed in the existing well field.

(3) A booster pump can be installed to overcome friction loss.

RECOMMENDATIONS

Periodic water quality analyses should be made on selected wells to establish background data on water quality. The selected wells should be at each end of the well field and several in between. This should be done on a frequent (approximately monthly) level at the beginning and then expanded over to six months to a year if no changes are being detected. This will allow you to detect any possible problems prior to their affecting the entire field.

If sites are available, two remote wells out from the well field should be installed for water quality analyses. The flow control valves should be removed and replaced with gate valves and meters. A one-inch meter installed in the line, however, will generate approximately five pounds of pressure loss. The meters, however, allow you to maintain a balanced flow from the individual wells and give you a record as to the amount of capacity being taken from each unit.

Water levels should be taken on the wells at periodic intervals to update the information included with this report.

We would recommend a simple gauging station in the lake to ascertain any change in this level, as this is a direct source of the water that you are pumping from the ground.

A maintenance program should be incorporated to periodically check conditions of individual wells.

REFERENCES

Peters, Robert R., P.E., 1984, Evaluation of Potential Ground-Water Development on Hatteras Island.

EXHIBITS

EXHIBIT 'A'
4-26-84

CAPE HATTERAS WATER ASSOC.

WELL NO.	AMPS	DEPTH	SCREEN SETTING	*PUMPING LEVEL	WELL CONSTRUCTED	STATIC WATER LEVEL	CAPACITY	NOTES
				*Measuring point top of casing.				
3	6.75							
4	7.5							
5	7.0							
6	6.5							
7	7.5							
7A	6.75			6.2'				Closed G. valve to stop back press.
8	Off							
8A	7.0	48'	41' - 46'	7.5'				E. J. Baker 5480-3 Muesco #116-5 - Orange, CA Flow control vol. set at 50 psig. 1-1/2" - Stock No. 70393-06 Code MF - Clay Mfg., Newport Beach, CA.
9	7.0							
9A	7.0	48'	41' - 46'		12-14-78	4'	20 gpm	
10	7.6							
10A	7.0	48'	41' - 46'	8.8'	4-1-46	4' - 1"		

EXHIBIT 'A'

CAPE HATTERAS WATER ASSOC.

WELL NO.	AMPS	DEPTH	SCREEN SETTING	*PUMPING LEVEL	WELL CONSTRUCTED	STATIC WATER LEVEL	CAPACITY	NOTES
				*Measuring point top of casing.				
11	7.75							
11A	7.25	48'	41' - 46'	13.3'	2-7-78	3' - 10-1/2"	20 gpm	
12	7.25							
12A	7.0	48'	41' - 46'	7.75'	2-1-78	3.75'		
13								Off - Motor disconnected - Motor O.K. - pipe not connected to main (Saddle Blow-out).
13A	6.5	45'	40' - 45'	7.0'	1-25-78	3'-2"	20 gpm	
14	7.5			14.4'				
14A	7.0	44'	37' - 42'	9.35'	1-26-78	3'-5"	20 gpm	
15	6.75							
15A	6.5	48'	41' - 46'	12.8'	1-15-78	2'-8"	20 gpm	
16	7.5			12.3'				
16A	7.25			7.7'				Turned pump off - diaphragm leaking.
17	7.5							Turned off when we arrived.
17A	6.75	44'	37' - 42'	12.95'	2-16-78	2'	20 gpm	Capac. disconnected (needs screw). Leaking diaphragm and top of casing slightly higher than others.

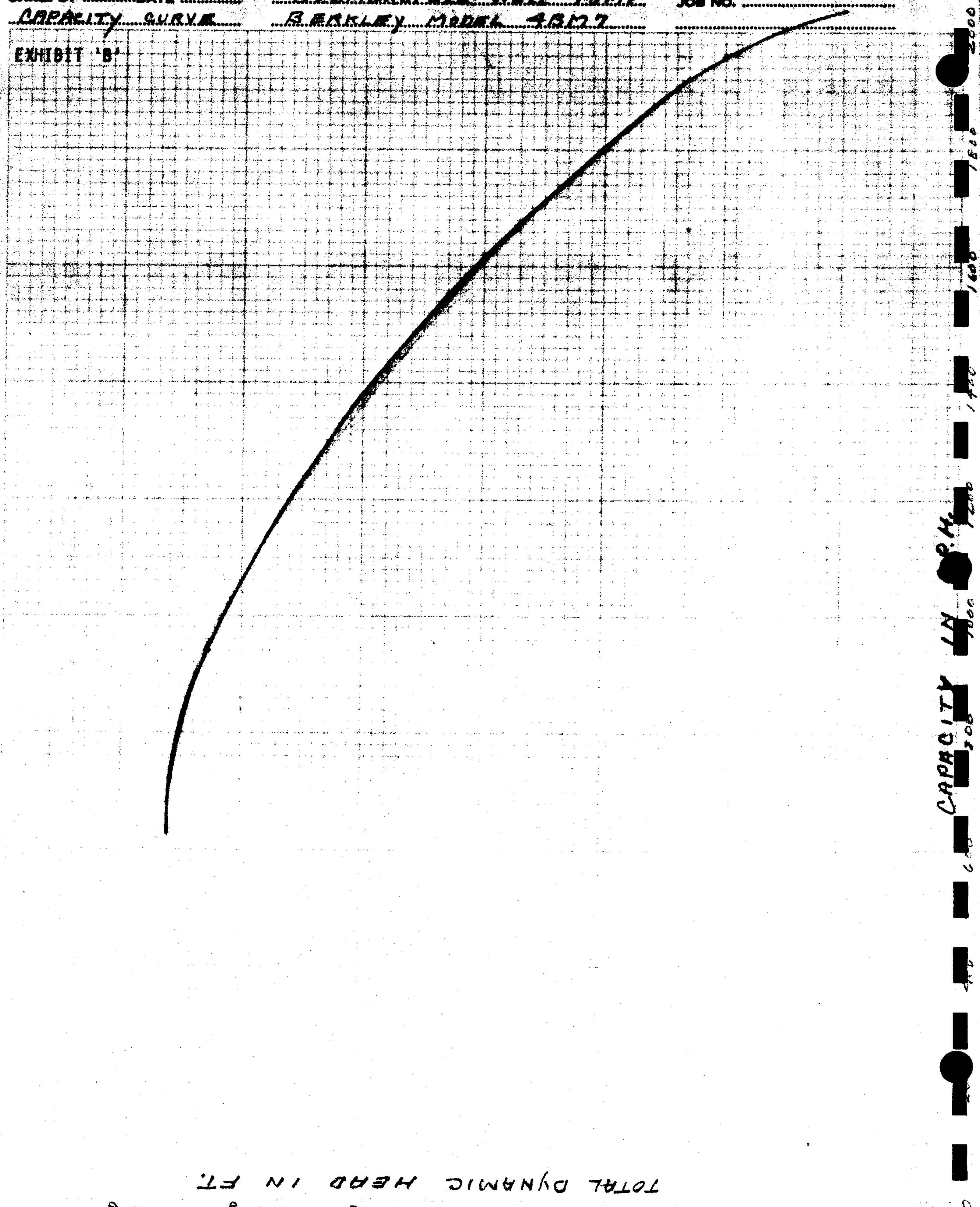
EXHIBIT A

CAPE HATTERAS WATER ASSOC.

WELL NO.	AMPS	DEPTH	SCREEN SETTING	*PUMPING LEVEL *Measuring point top of casing.	WELL CONSTRUCTED	STATIC WATER LEVEL	CAPACITY	NOTES
18	7.25			10'		4'	20 gpm	
18A	6.75	44'	37' - 42'	8.35'				Out of service - discharge line disconnected.
19						4'		Pump off.
19A								
20	7.5					4'	20 gpm	Pumped this well for a few minutes.
20A	7.0	46'	39' - 43'	4.3'	12-14-77			
21								Out of service - 4 obs. wells. Pipe found in field. 25.2' pump setting, approx.
21A		47'	41' - 45'		12-7-77	3.6'	25 gpm	This system down due to 21 being off.
22	7.0			12.1'				

BY R.R.P. DATE 8-17-84 SUBJECT HATTERAS WATER ASSO SHEET NO. 1 OF 1
CHKD. BY .... DATE .... SUBMERSIBLE WELL PUMP JOB NO. ....
CAPACITY CURVE BERKLEY MODEL 4BM7

EXHIBIT 'B'

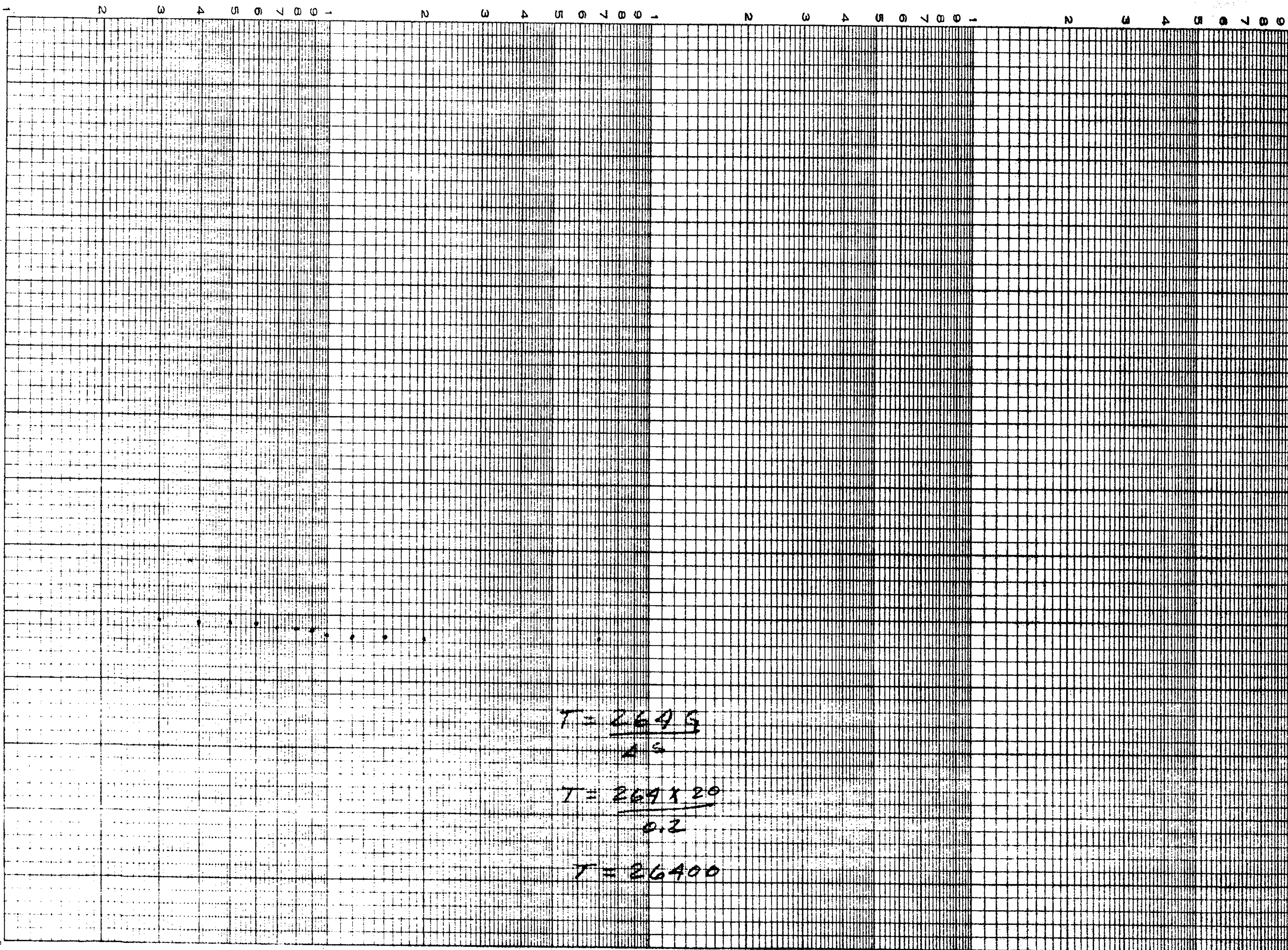


NO. 340-L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

EXHIBIT 'C'

PUMP TEST - WELL NO. 21

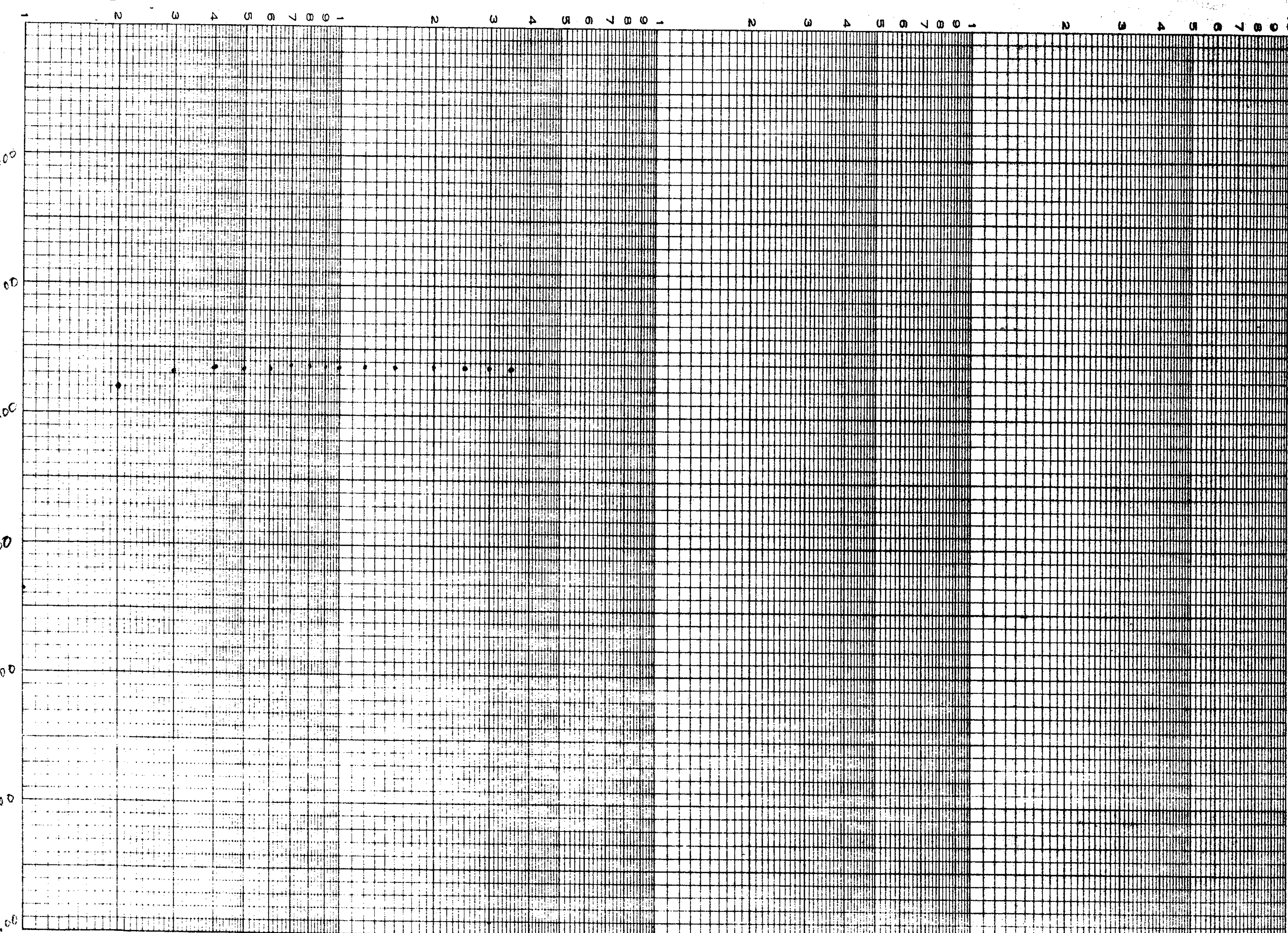


NO. 340-L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

EXHIBIT 'D'

RECOVERY TEST - WELL NO. 21



NO. 340-L410 DIETZGEN GRAPH PAPER
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4 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
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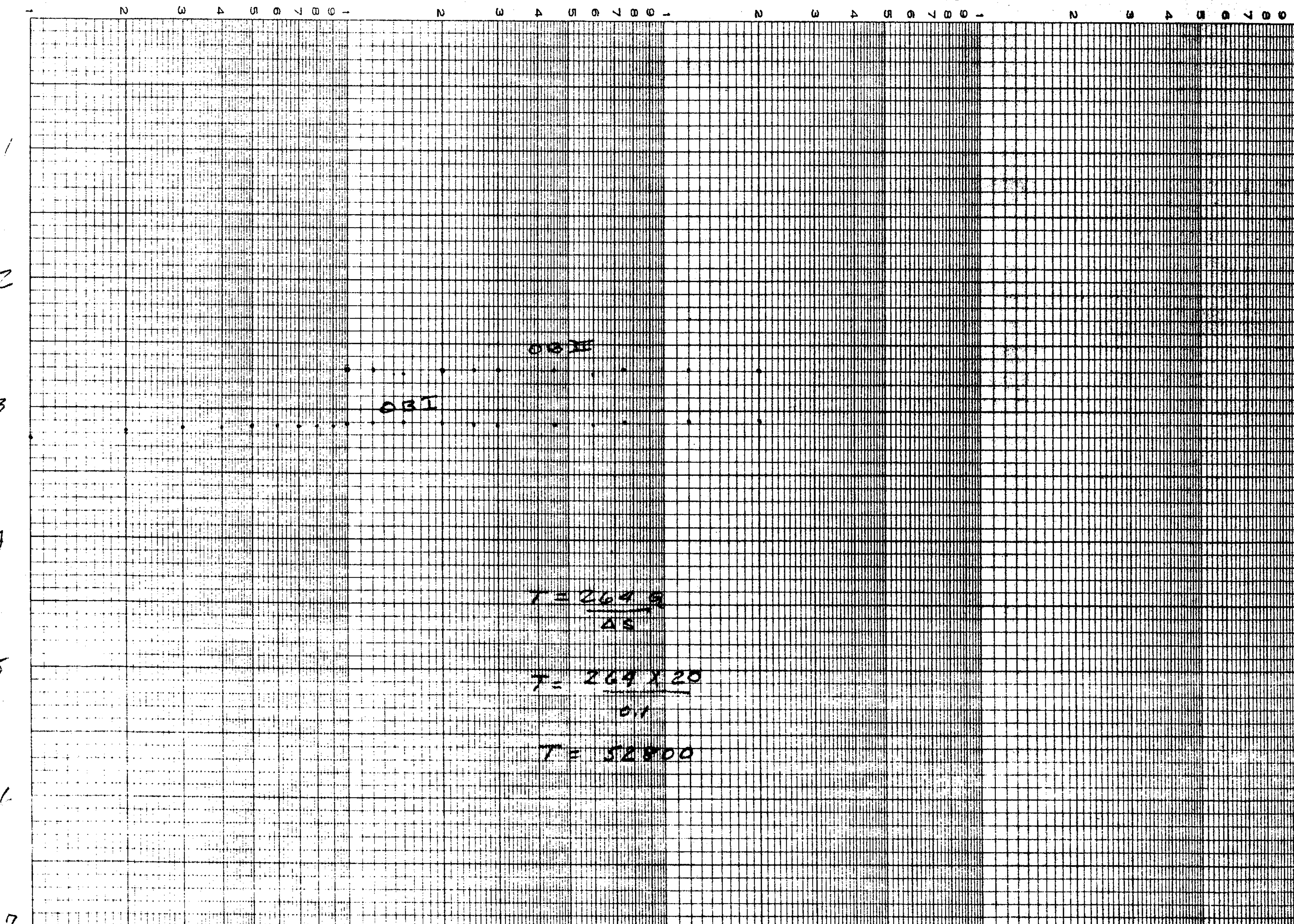
EXHIBIT 'E'

RECOVERY TEST - OBSERVATION WELLS I AND II

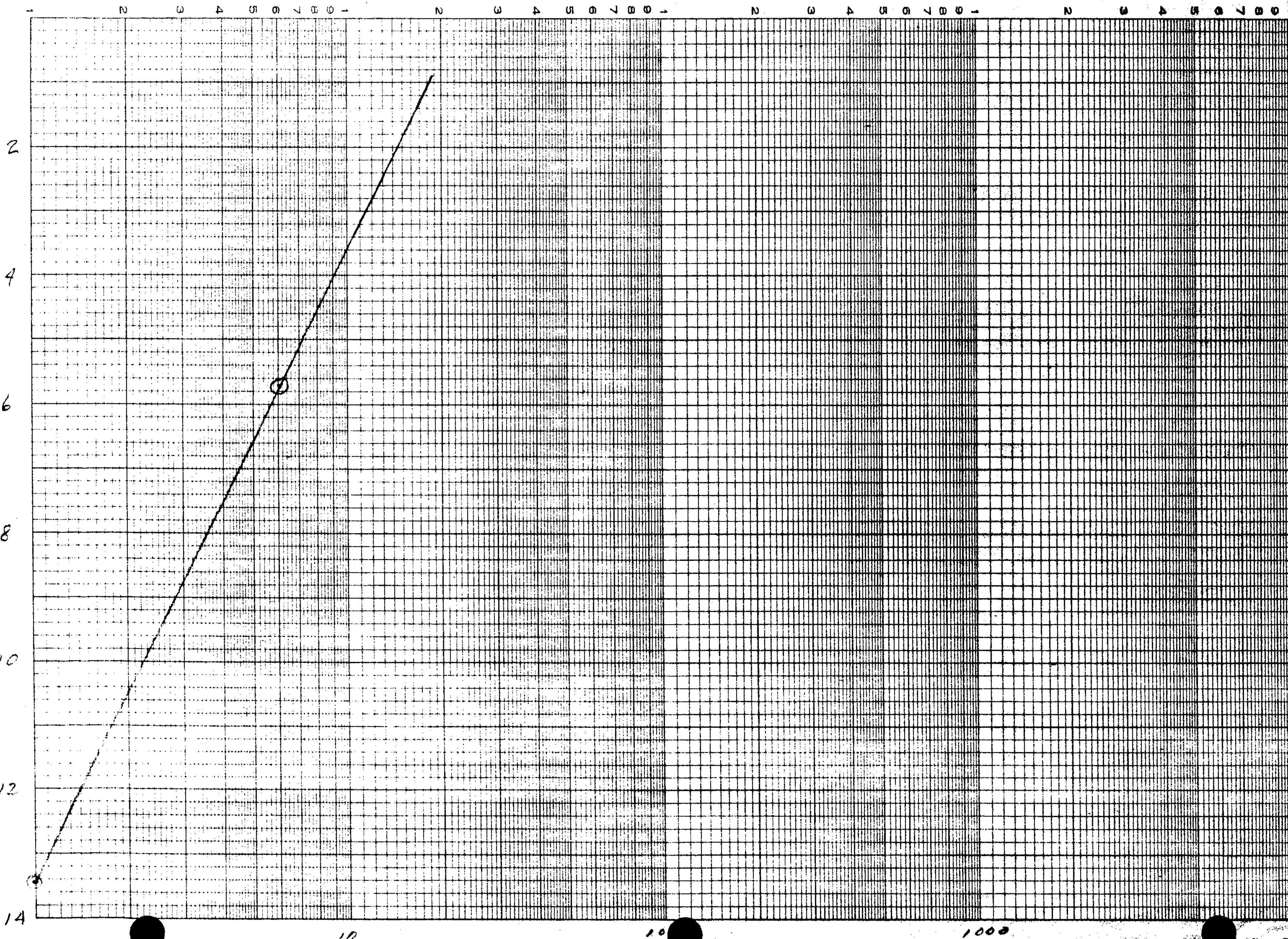


NO. 340-L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 10 DIVISIONS PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.

EXHIBIT 'F'

WELL 21 - DISTANCE DRAWDOWN



Westinghouse Environmental
and Geotechnical Services, Inc.

3100 Spring Forest Road (27604)
P.O. Box 58069
Raleigh, North Carolina 27658-8069
(919) 872-2660
Fax (919) 790-9827

February 27, 1991

Diehl and Phillips
219 East Chatham Street
Cary, North Carolina

Attention: Mr. Bill Diehl

Reference: Report of Subsurface Investigation
Cape Hatteras Water Association
Treatment Plant Expansion
Buxton, North Carolina
Westinghouse Project No. 1051-91-132B

Gentlemen:

Westinghouse Environmental and Geotechnical Services, Inc. has completed the authorized subsurface investigation to evaluate foundation support considerations for proposed additions to the existing water treatment plant in Buxton, North Carolina. Subsurface conditions at the site were investigated by drilling three test borings at approximately the locations shown on the attached Figure 1. These locations were established in the field by taping distances from existing on-site structures and the indicated locations that should therefore be relatively accurate. The surface elevations shown on the attached generalized subsurface profile were interpolated from point shots presented on the attached Figure 1. The test borings were advanced to termination depths of 30 to 35.5 feet beneath existing site grade utilizing standard penetration test procedures at selected intervals to evaluate the consistency and density of the subsurface soils. This report presents the findings of the investigation and our recommendations for site grading and foundation support for the various structures which will be constructed as part of this project.

PROJECT SITE DESCRIPTION

It is our understanding that the existing water treatment facilities in Buxton, North Carolina will likely be expanded in the near future with an extension of the existing treatment plant building, construction of a new sludge basin, and construction of a new finished water tower. Based on the existing plan drawings, it appears that the existing structure is supported on timber piling foundations bearing at a depth of not more than 25 feet beneath existing site grade. We understand that the 200,000 gallon sludge basins which exist on the property and exert a foundation bearing pressure on the order of 1,000 pounds per square foot are supported on shallow foundations. Based on a visual reconnaissance of the existing facilities, no significant settlement cracking could be observed in either the water treatment plant structure nor in the sludge basin structure.

We understand that the proposed sludge basin addition will likely be a 60 feet diameter tank with a standing water depth of 15 feet, which will also result in an approximately 1,000 pound per square foot bearing stress in the vicinity of test boring W-2. The existing water treatment plant building will be expanded in the vicinity of test boring W-3 and although structural loads are not yet known, it is anticipated that wall loadings will not be particularly heavy. In addition to the expansion to the water treatment plant building in the area of W-3, an interior portion of the structure not specifically designed for equipment loadings initially will be used to support new filters supported on four legs each imparting a load of approximately 21 kips to the slab. We further understand that consideration is being given to possibly raising the existing treatment plant building in the front portions to provide additional space.

The exact location which will be considered for a new finished water tower has not yet been identified, however, the tower may have a height of as much as 100 feet and a diameter of 30 feet in the vicinity of test boring W-1 or between V-1 and W-2. Based on a 100-foot height, the contact stress beneath this finished water tower would be on the order of 6,000 psf.

SUBSURFACE CONDITIONS

A generalized subsurface profile prepared from the test boring data is attached to this report as Figure 2 to graphically illustrate subsurface conditions encountered at this site. More detailed descriptions of the conditions encountered at the individual test boring locations are then presented on the attached test boring records.

The test borings performed for this project encountered similar conditions at all three test locations with the exception that a zone of organic silt or sand encountered in all three borings was thicker and more organic in test boring W-2 than in the other borings. Typically, the near surface soil consisted of approximately three feet of slightly silty fill exhibiting penetration resistances in the range of 7 to 9 blows per foot underlain by a thin veneer of organically stained silty sand. The organically stained sands had a relatively low organic content at test locations W-1 and W-3, but had a high organic content and were classified as a organic clayey silt at test location W-2. As such, the material encountered from a depth of 4 to 6 feet at test location B-2 had the characteristic appearance of a low grade peat with a high sand content.

With increasing depth, the test borings penetrated loose to firm slightly silty sand which became increasingly denser with depth. These sands exhibited penetration resistances in the range of 9 to 22 blows per foot to a depth of 5 feet and increased in resistance to typically 28 to 70 blows per foot below a depth of 5 to 6 feet to the termination depth of the borings. The only exception to this was a zone of 16 blow per foot silty sand encountered from 21 to 26 feet below grade at test location W-1.

The ground water level was measured after 24 hours and all three test borings had water levels ranging from 2.75 feet to 3.5 feet beneath existing site grade. We note that the soils on this site consist predominantly of relatively clean sands and that it is anticipated that the water level will vary with significant tidal changes in the area.

RECOMMENDATIONS

The following recommendations are made based on our review of the attached test boring data, our understanding of the proposed construction, and past experience with similar projects and subsurface conditions. Should site grading or structural plans change significantly from those now under consideration, we would appreciate being provided with that information so that these recommendations may be confirmed, extended, or modified as necessary. Additionally, should subsurface conditions adverse to those indicated by this report be encountered during construction, those differences should be reported to us for review and comment.

SUPPORT OF PROPOSED WATER TOWER

The proposed finished water tower will likely be a narrow, tall ground storage tank with a diameter of 30 feet and a height of 100 feet constructed somewhere in the area between W-1 and W-2. Based on an empirical settlement analysis using the FHA method, we estimate that a 6,000 psf contact stress on a 30 foot diameter tank will result in a total center settlement on the order of 3.2 inches. Due to the relatively great height of the structure and narrow base, it is our opinion that this magnitude of settlement may be unacceptable. If approximately 3 inches of settlement cannot be tolerated, we recommend that the water tower be supported on piling foundations extended to bear at a depth of approximately 25 feet beneath existing grade. These piling would have to be installed by predrilling or jetting to a depth on the order of 17 feet and then driving until achieving an indicated set indicative of the design load.

The finished water structure could be supported on timber piling designed for a working load of 20 to 30 tons or on concrete piling designed for loads on the order of 40 to 100 tons. Under the current North Carolina building code, any pile having a capacity in excess of 40 tons will have to be load tested in order to confirm the design. If it is desired to utilize a 40 ton pile and eliminate the pile load test, the pile section may consist of either a ten inch square or 12 inch square pile. If it is desired to achieve a higher capacity on the order of 100 tons, we recommend driving a 12 inch square concrete pile. We anticipate a 20 to 30 ton timber piles will take up at a depth of no more than 20 to 25 feet and that 40 ton concrete piles will be achieved at 25 to 30 feet beneath existing grade after preaugering. 100 ton piles will likely also be developed at depths on the order of no more than 30 to 35 feet beneath existing site grade. If a piling foundation is used, post construction settlements will likely be negligible.

SUPPORT OF SLUDGE BASIN

The proposed sludge basin to be installed in the vicinity of test boring W-2 will likely have a contact stress of only 1000 pounds per square foot. Assuming that a two foot zone of low grade peat which was encountered at test location W-2 is removed and replaced, we anticipate that a ground supported tank utilizing a concrete slab on grade and ringwall foundation will experience total center settlements on the order of 1.2 inches and perimeter settlements on the order of 1/2 inch at this location. Accordingly, provided the organic zone is repaired beneath the foundation for this tank, we anticipate that this structure can be easily supported on shallow foundations.

The organic soils which exist in the vicinity of test boring W-2 may be discontinuous and may not exist over the entire base of the foundation in that area. As such, we recommend that preparation of the subgrade for this tank be accomplished by excavating approximately three feet to the top of the ground water table over the area of the tank and then performing a detailed hand auger inspection of the underlying 3 feet of soil to identify whether or not the organic materials are present. If the organic soils are present, they should be removed and wasted off-site or in non-structural areas. A clean sand fill, such as the material excavated to achieve the three foot excavation levels should then be placed back into the excavation and those soils should be compacted from above the water table with a vibratory roller. Any additional fill required to achieve the desired subgrade level should be placed and compacted to not less than 95% of the standard Proctor maximum dry density. Following completion of these repairs, the subgrade should provide adequate support for a ringwall foundation sized for a bearing pressure of up to 3,000 pounds per square foot and the resulting subgrade should limit total settlements to not more than approximately 1" in the center of the tank.

TREATMENT BUILDING EXPANSION

Subsurface conditions in the area of the proposed treatment plant appear to be excellent for support of foundations for the structural addition. Based on the results of test boring W-3, it appears that significant loadings in that area could in fact be tolerated by shallow spread footing foundations if differential settlement between the new structure and existing structure are not considered to be a problem. Figure 3 attached with this report provides an indication of the anticipated settlement versus column load for an assumed bearing pressure of 3,000 pounds per square foot. If settlements on these orders of magnitude can be tolerated between the new and existing structure, it is our opinion that the new structure could be supported on shallow spread footing foundations.

If shallow spread footings are used, it should be recognized that all footing excavations will have to be carefully inspected to identify existing trench areas will require repair. The addition area should be proofrolled following clearing to identify trench and soft soil areas, and any area which ruts or pumps excessively should be repaired by excavating to firm bearing and then backfilling with properly compacted fill. All new fill should consist of clean or silty sands compacted to not less than 95 percent of ASTM D-698. Care should be exercised during compaction to prevent excessive vibration of the existing structure. Following proper completion of grading, the subgrade soils should provide adequate support for a concrete slab-on-grade assuming a 150 pci subgrade modulus.

If it is desired to limit settlements in the new addition area to negligible values in order to prevent significant differential movement between the two structures, we recommend that the new addition be supported on short timber pilings sized for a design working load in the range of 20 to 30 tons. The piles should be installed by preaugering or jetting to a depth of 15 feet and then driving to a final set indicative of the desired capacity. Based on the results of the borings, we anticipate that the piles will not penetrate more than 1 to 3 feet below the preauger or jetting depth for the piling.

We understand that a non-structurally supported slab on the interior of the structure will be converted to equipment support use. Based on the results of the test borings, the subgrade conditions beneath the non-structurally supported slab are likely excellent for support of the new machine loadings providing past construction did not result in densification of the underlying sands and a slight settlement of those sands away from the slab on grade. We also are assuming that a zone of relatively organic material present at test location W-2 does not exist beneath the structural slab area.

In order to verify the preceding assumptions, it is recommended that a limited number of core borings be taken through the existing non-structurally supported slab prior to final design of the machine foundation mounts. If it is determined that a void exists beneath the slab or that zones of organic material exist at depth, it will be necessary to improve the soils to a depth of approximately 3 to 4 feet by compaction grouting. This would be accomplished by boring small diameter holes through the slab at the leg locations and then pressure grouting the sands within the desired zone of improvement prior to regrouting of the core hole through the slab on grade. Whether or not this approach will be required will be dependent upon whether or not a void has developed beneath the non structurally supported slab and whether or not organics exist at depth. Following core borings through the slab and an evaluation of these two items, we will be happy to evaluate the structural adequacy of the existing slab for the subgrade if you will provide us with the anticipated new slab loadings.

SUMMARY

In summary, subsurface conditions at the water treatment plant site appear to be excellent for the support of the new construction except where a zone of organic material was encountered at a depth of 4 to 6 feet at test location W-2.

These zones of slightly organic material are often discontinuous over a site of this nature and may not exist uniformly in the area of test boring W-2. However, we do recommend that consideration be given to pushing off the upper three feet of soil beneath the foundation in this area and then performing a detailed subgrade inspection in order to evaluate whether or not additional undercutting may be required. The materials pushed out from the excavation may be used as structural backfill which can be placed below the water table and then compacted from above the water table if necessary. The sludge basin may then be supported as a ringwall foundation with a concrete slab on grade. The proposed building addition may be supported on shallow spread footings if anticipated settlements can be tolerated or may be supported on light to moderate capacity timber pilings if it is desired to limit settlements of the new addition to negligible values. The proposed finished water tower will have relatively high contact stresses and will result in settlements on the order of three inches if shallow foundation support is used. Accordingly, you may wish to consider supporting that structure on short piling.

We appreciate the opportunity have provided you with our services during this phase of the project. Please contact us if you should have questions regarding this report or if we may be of further assistance.

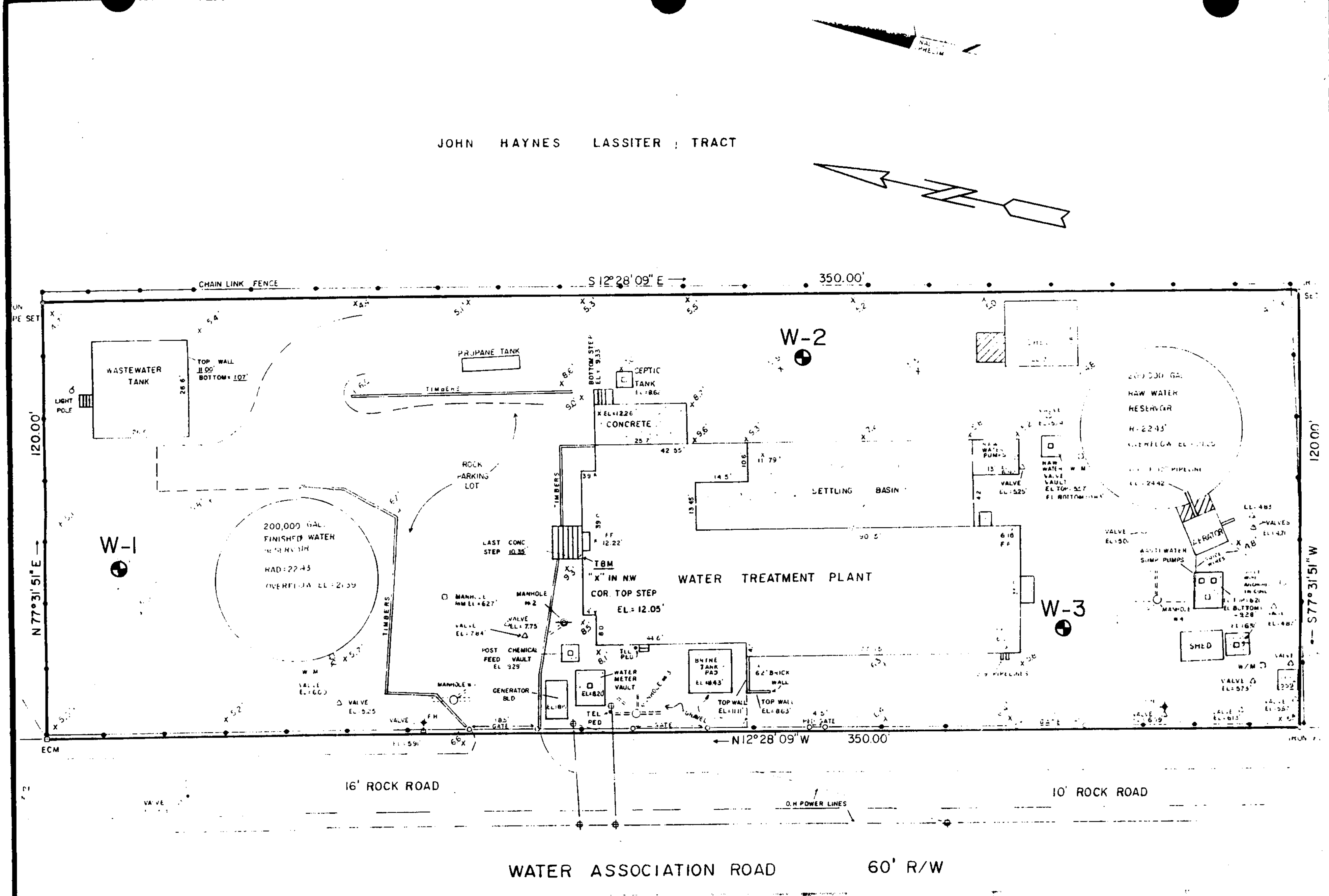
Sincerely,

WESTINGHOUSE ENVIRONMENTAL AND
GEOTECHNICAL SERVICES, INC.

Edward B. Hearn, P.E.
NC Registration No. 9520

John R. Browning, P.E.
Branch Manager

EBH/JRB/als/R91-132B



PROJECT

WATER TREATMENT FACILITY
BUXTON, NORTH CAROLINA

Westinghouse Environmental and Geotechnical Services, Inc.

SCALE: N.T.S.

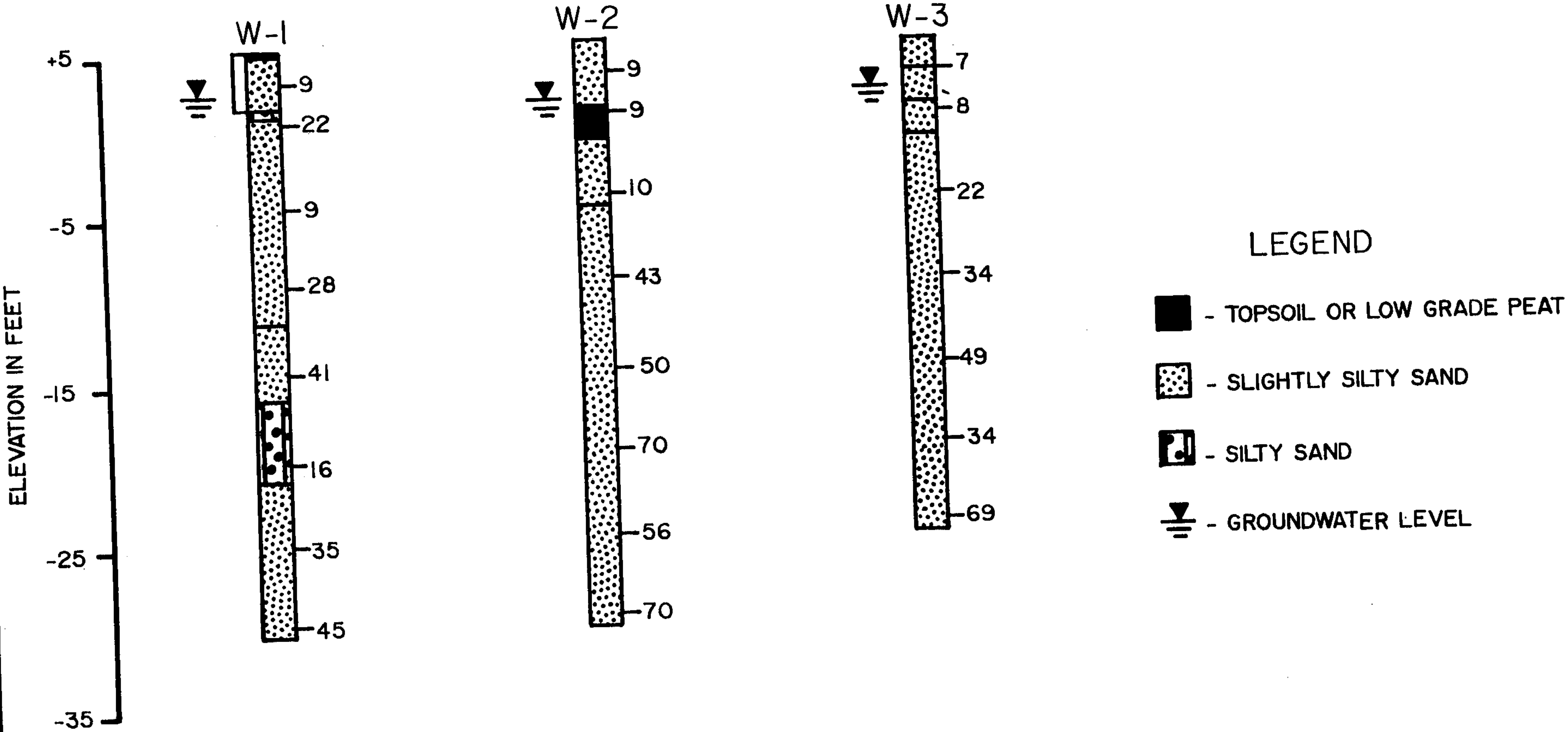
JOB NO: 1051-91-132

FIG NO: 1

154D

GENERALIZED SUBSURFACE PROFILE

PROPOSED BUXTON WATER TREATMENT PLANT EXPANSION



PROJECT WATER TREATMENT PLANT EXPANSION BUXTON, NORTH CAROLINA	Westinghouse Environmental and Geotechnical Services, Inc.	SCALE: AS SHOWN JOB NO: 1051-91-132 FIG NO: 2

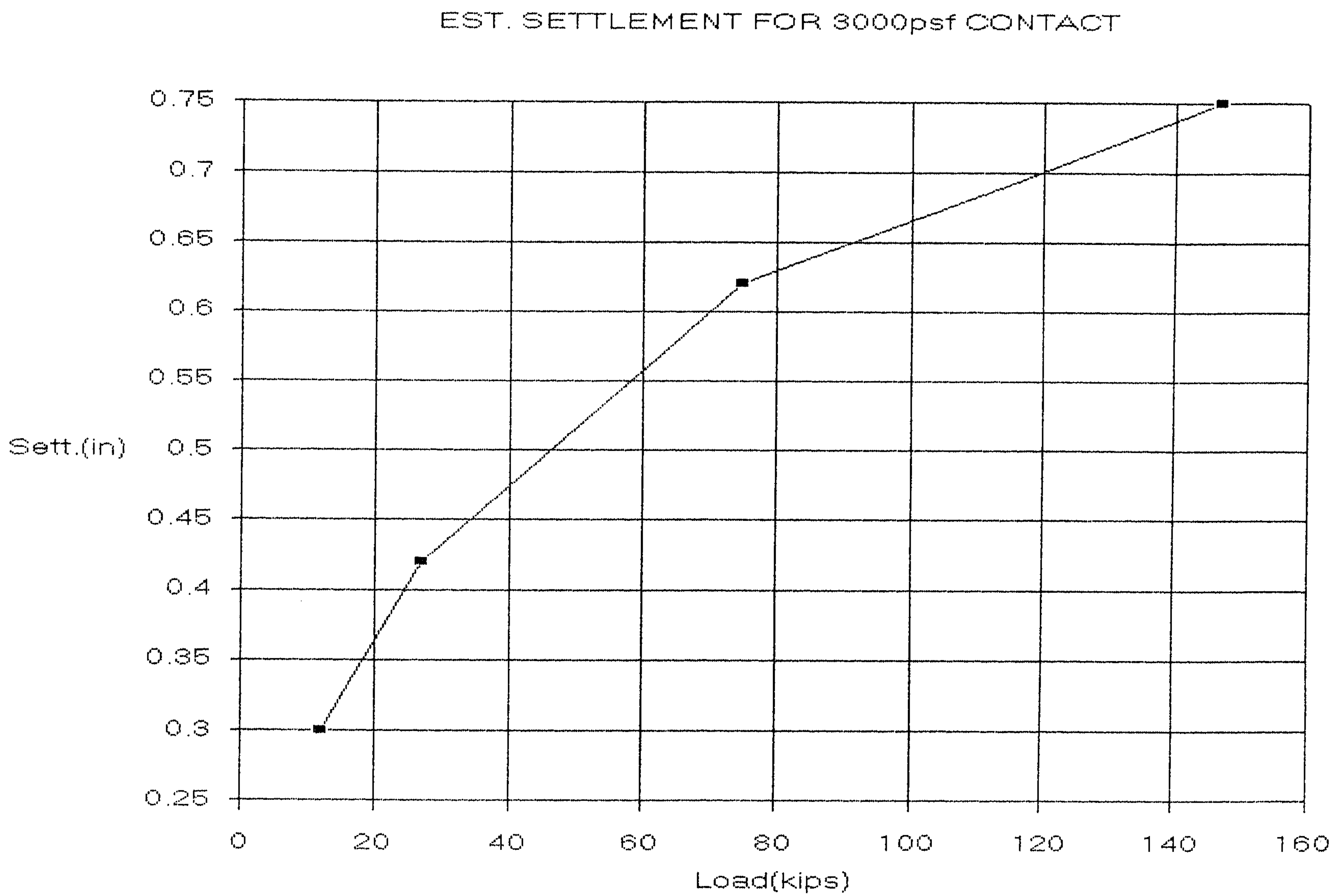
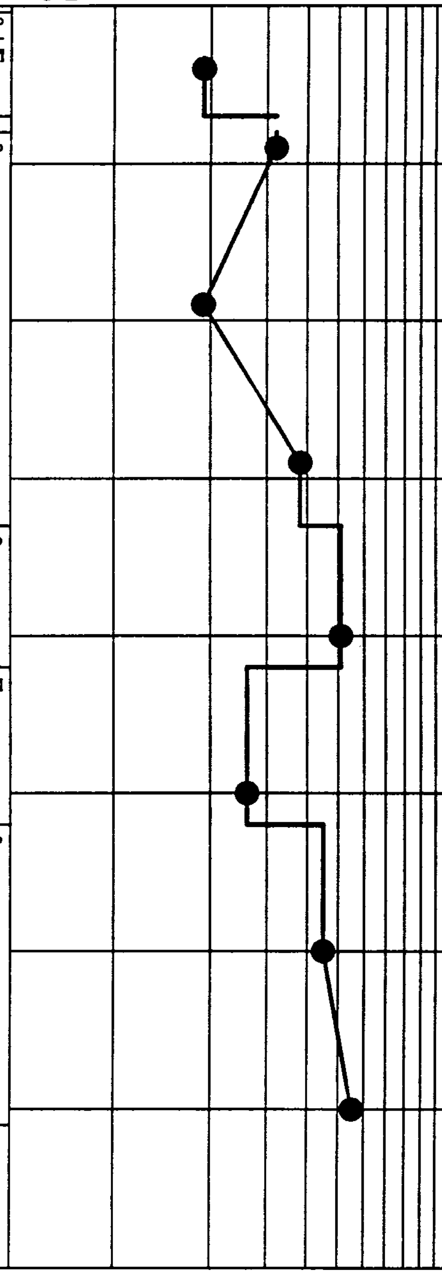

EST. SETTLEMENT FOR 3000psf CONTACT
Sett.(in)
0.75
0.7
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0
20
40
60
80
100
120
140
160
Load(kips)

FIGURE 3

TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.) +5.2 0	PENETRATION (BLOWS/FT.) 10 20 40 60 100	BLOWS PER SIX IN.
0.0	Roots and Grass	SP			
0.2	Loose Tan Shelly Fine to Medium SAND (Fill)	SM			3-5-4
3.5					2.75
4.0	Loose Dark Gray Organic Silty Fine SAND with Trace Organic (Wet) OM	SP			4-10-12
	Loose to Firm Light Gray Fine to Medium SAND Trace Shells				2-4-5
					9-12-16
16.5	Dense Light Gray Slightly Silty Fine to Medium SAND (Trace Shell)	SP			12-20-21
21.0	Loose Gray Silty Fine and Fine to Medium Shelly SAND	SM			7-10-6
26.0	Dense Light Gray Slightly Silty Fine SAND	SP			11-15-20
					15-21-24
35.5	Boring Terminated at 35.5'				



REFER TO ATTACHED SHEET FOR EXPLANATIONS AND SYMBOLS

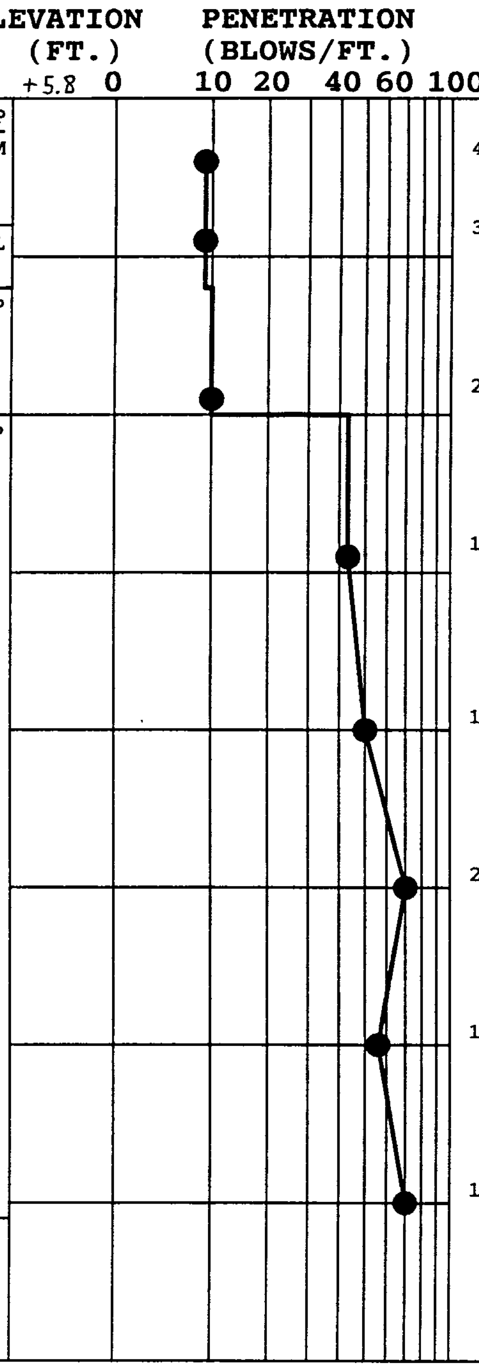
JOB NUMBER 1051-91-132
BORING NUMBER W-1
DATE 1-31-91

PAGE 1 OF 1

S&ME

TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION		ELEVATION (FT.) +5.8	BLOWS PER SIX IN.	
0.0	Loose Tan Slightly Silty Fine to Medium SAND with Trace Shells (Possible Fill)	SP / SM		4-4-5	
4.0	Stiff Black Organic Fine Sandy Clayey SILT (Lo Grade Peat)	OL		3-3-6	3.5
6.0	Firm Tan Fine to Medium SAND	SP		2-4-6	
10.0	Dense and Very Dense Tan and Gray Fine to Medium SAND with Trace Shell	SP		14-19-24	
				13-20-30	
				22-30-40	
				18-26-30	
				15-30-40	
35.5	Boring Terminated at 35.5'				



REFER TO ATTACHED SHEET FOR EXPLANATIONS AND SYMBOLS

JOB NUMBER 1051-91-132
BORING NUMBER W-2
DATE 1-31-91

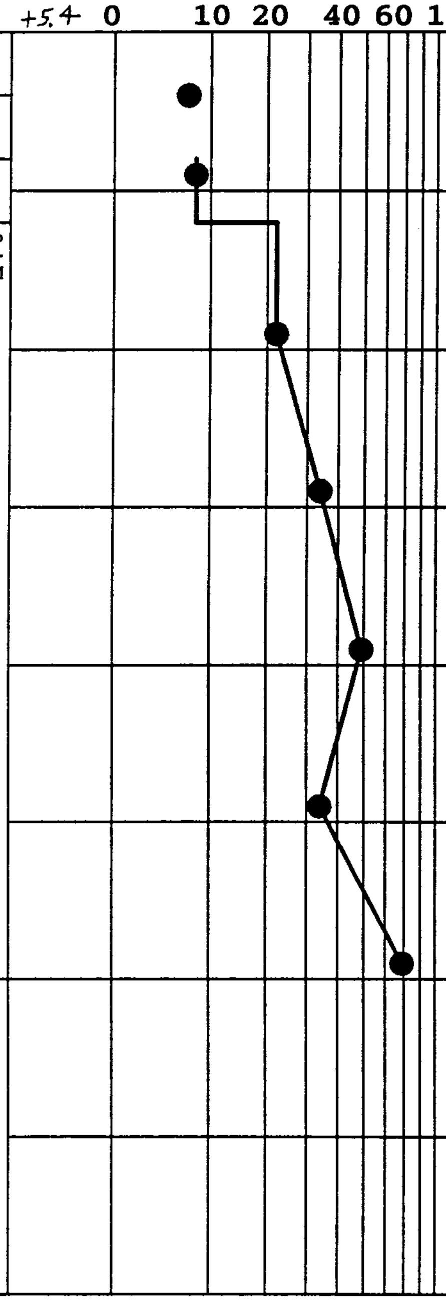
PAGE 1 OF 1

S&ME

TEST BORING RECORD

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.) +5.4	PENETRATION (BLOWS/FT.) 0 10 20 40 60 100	BLOWS PER SIX IN.
0.0	Loose Tan Slightly Silty Fine to Medium SAND with Trace Shells (Fill) SP/SM			5-5-2
2.0				
4.0	Loose Brown Organically Stained Slightly Silty Fine to Medium SAND SP/SM			4-3-5
6.0	Loose Brown Slightly Silty Fine to Medium SAND SP/SM	SP/SM		
	Firm to Dense Gray Slightly Silty Fine and Fine to Medium SAND			7-9-13
				15-17-17
				17-21-28
				17-17-17
				21-29-40
30.0	Boring Terminated at 30.0'			

3.0 (water level)



REFER TO ATTACHED SHEET FOR EXPLANATIONS AND SYMBOLS

JOB NUMBER 1051-91-132
BORING NUMBER W-3
DATE 2-1-91

PAGE 1 OF 1

S&ME