Diffuser Design and Study for the
Seabrook Wastewater Facility Outfall
Seabrook, New Hampshire

Prepared by
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Tech 797
Spring, 1995

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Sea Grant
New Hampshire/Maine

Special Acknowledgments

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Dr. Tom Ballestero
Jon Pavlos
Phil Coyne
Pete Morrow
John Scott
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Introduction

This report contains recommendations for the design of the outfall diffuser for the Seabrook wastewater treatment plant, Seabrook, New Hampshire, located on Write Island in the Hampton -Seabrook salt marsh. The current conditions of the plant fail to meet state and federal regulations, therefore an upgrade to secondary treatment is under construction, with a design flow of 1.8 million gallons per day (mgd). The treated wastewater disposal is to the Atlantic Ocean through an ocean outfall pipe that terminates with a diffuser.

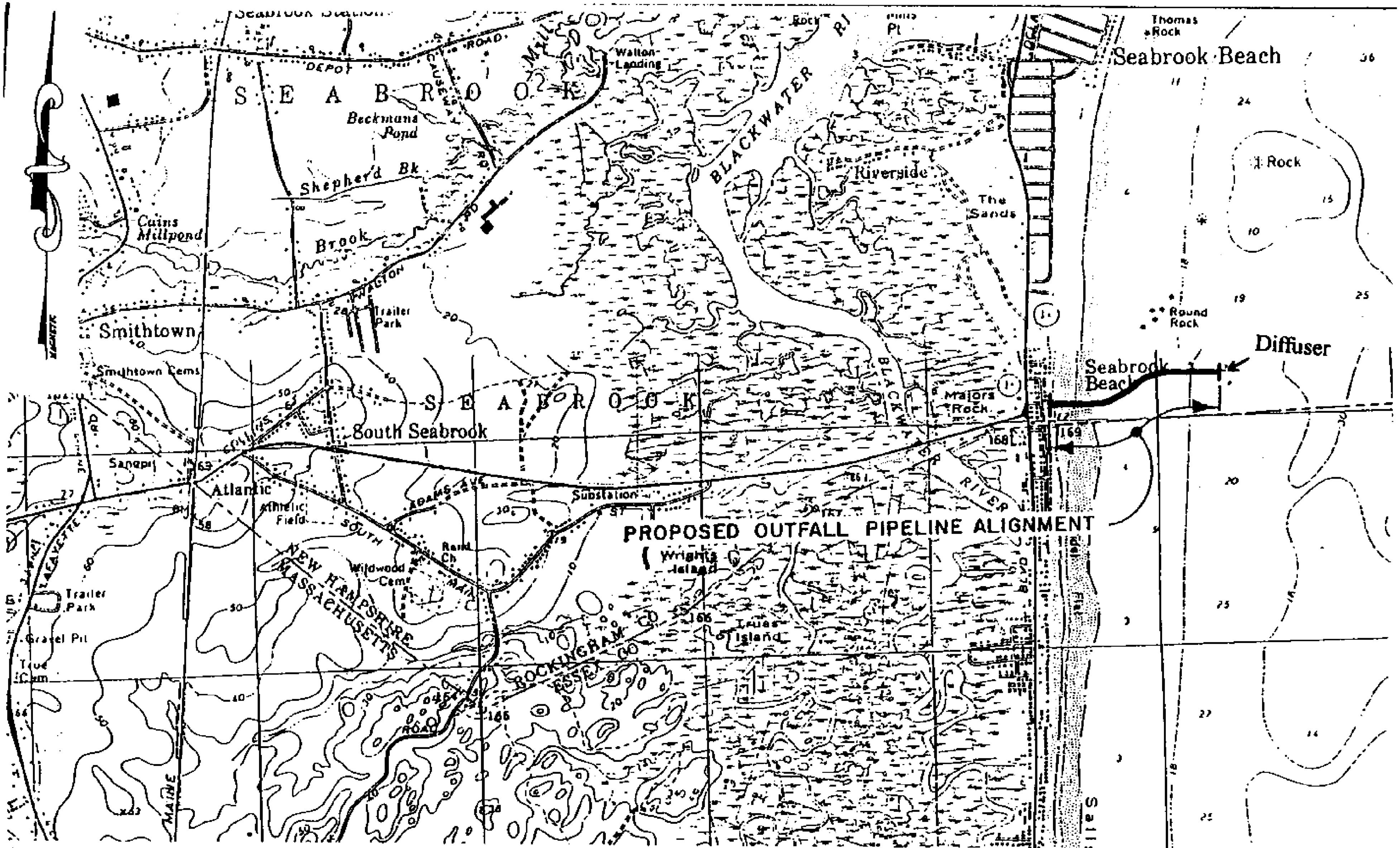
A diffuser system is one that disperses the effluent water from the treatment plant into the ambient water, in this case the saline water of the Atlantic Ocean. This dispersion allows little environmental disturbance at the diffuser. Our goal was to design an efficient, strong, and easily maintainable diffuser for the community.

The first step was to research the subject and locate the important variables. Such things as the flow from the treatment plant, diffuser components, and site description were found. From this information, the plume model MERGE was used to define parameters of a design concept for the diffuser. Once completed, an anchoring system was developed to aid in failure protection from the forces of waves and erosional events. The final stage of this study was to investigate the plume dilution, which was performed using a physical model.

The proposed diffuser location is approximately 2100 feet offshore, in the Gulf of Maine (fig.1), at the end of a 2 foot diameter, high density polyethylene (HDPE) pipe. The mean low water depth at this point is 30 feet and the diffuser is to be oriented parallel to the shore line to provide maximum dilution and dispersion. The outfall pipe, which has already been installed, is located in an armored trench. By having the pipe buried, it is protected from failure due to wave forces and erosional events.

An extensive literature search was conducted and numerous articles were identified, describing the outfall being built in Boston, Massachusetts. Other investigated papers and books included general design guidelines, procedures, and methods of modeling. Reports from Stearns and Wheler, and GZA GeoEnvironmental, Inc., who actively investigated the Seabrook site, were used for the background information and in situ characteristics.

To aid in the physical understanding of this project, a trip was scheduled and extra reading was completed. The purpose of the trip was to see what a pre-installed diffuser looked like. The trip was to the Deer Island wastewater treatment facility in Boston. While at the site, The tunnel for the opening of the outfall was viewed and a diffuser head was investigated to see the actual design involved.



GEOTECHNICAL ENGINEERING RECOMMENDATIONS PROPOSED SEWER OUTFALL PIPELINE JONES & BEACH/STEARNS & WHELER JOINT VENTURE SEABROOK, NEW HAMPSHIRE	DES'D BY : M.J.I.
	CHK'D BY : M.D.B.
	APP'D BY : W.E.J.
	DRAWN BY: D.K.T.
LOCUS PLAN	SCALE : 1"=2000'
	DATE : JUNE 1994

PROJECT No.: 21145

FIGURE No.: 1

GRAPHIC SCALE 2000' 0' 2000' 4000'

GZA GeoEnvironmental, Inc.
Engineers and Scientists
380 HARVEY ROAD
MANCHESTER, NEW HAMPSHIRE 03103
(603) 623-3600

Figure 1. Seabrook wastewater treatment facility outfall route

Initial Design

Analytical prediction of outfall plume dilution

In 1973, the plume model MERGE was developed and published by the Environmental Protection Agency (EPA). MERGE was used to model the dilution of the effluent plume as it aspirates into the ambient water of the receiving body. EPA ran MERGE for a variety of standard ambient water characteristics, resulting in a series of charts to evaluate diffuser dilution. The output obtained from the model yields a value of dilution, for a specified diffuser design, and a corresponding trapping level. The trapping level (fig 2, USEPA, 1985) is the level where the diffused plume ceases to rise and then drifts with the ambient fluid velocity. This had great consideration in the design because it is undesirable to have a plume rise to the water-air interface.

Assumptions incorporated in MERGE include :

- steady state flow,
- the plume maintains a circular cross section everywhere until the trapping level and,
- the density difference between the effluent and receiving water is very small.

These conditions are not always met by the prototype but have been accepted in design. The basic feature of MERGE is that it includes forced entrainment : which is the dilution of the effluent plume due to the shear that occurs between an effluent jet from the diffuser port and the ambient flow. Even in the absence of current, which is not found off the coast of Seabrook, the shear created by the plume jet results in the entrainment of ambient water into the plume, causing plume dilution.

There are important limitations of MERGE that must be considered (USEPA, 1985) :

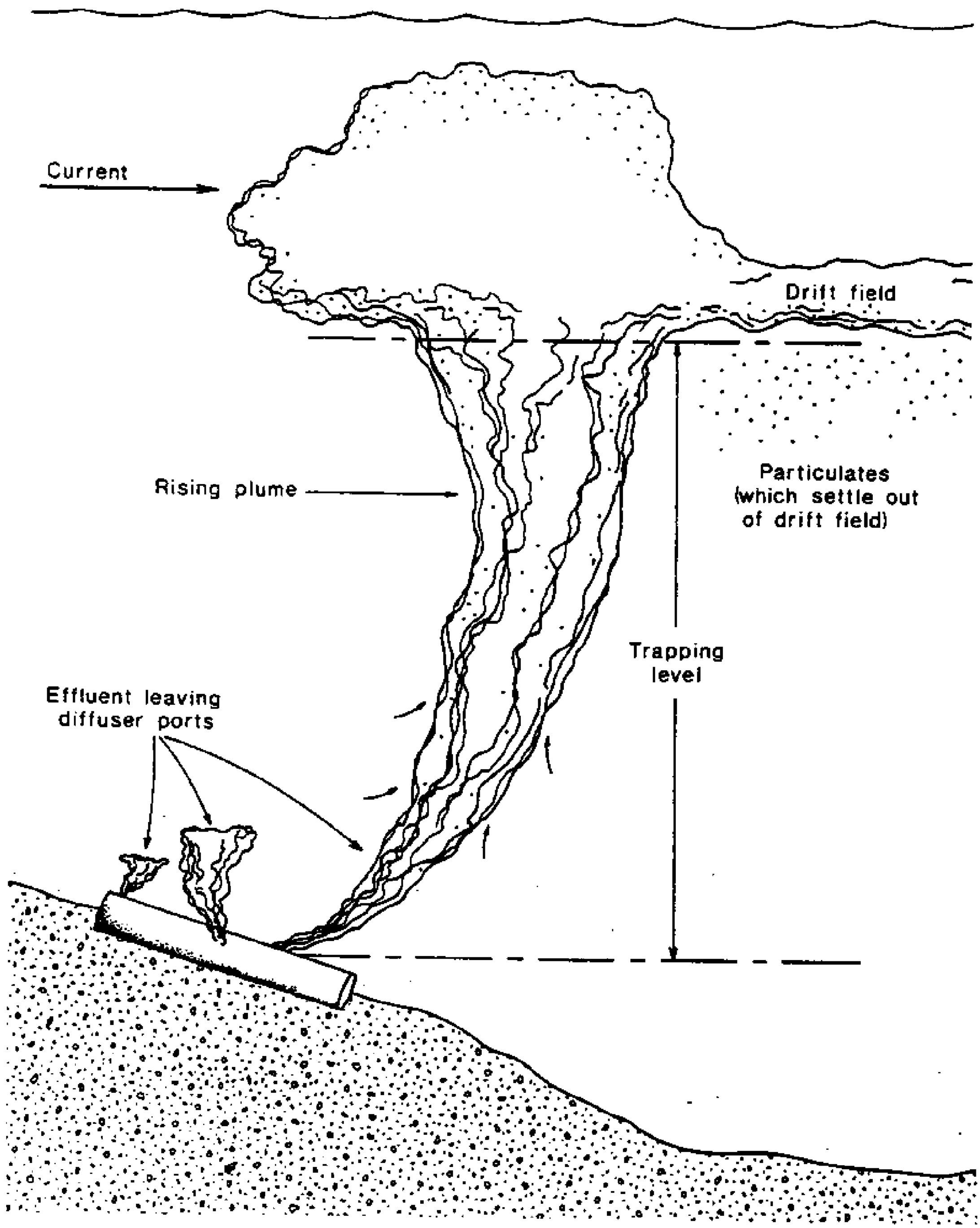


Figure 2. Plume trapping level

• Diffuser jets are parallel to the ambient current: The model does not predict plume dilution for cases of current flowing parallel to the diffuser pipe. This is a severe limitation especially in some ocean applications. Ambient flow parallel to the pipe result in the lowest initial dilution's for the effluent.

• Boundary conditions: The model does not properly account for interfacial boundary conditions. Dilution near the water surface or bottom may be overestimated since entrainment is being modeled where water is unavailable for such entrainment.

• Horizontal homogeneity: The model assumes a homogeneous horizontal ambient current, however, bottom topography, internal waves, and/or other factors may cause considerable spatial flow variations. This is in addition to temporal variations which are excluded by virtue of the assumed steady-state.

• Uniform discharge: It is assumed that an infinitely long diffuser exists for which there is no port-to-port variation in effluent characteristics.

MERGE parameters

Four similarity parameters must be calculated before entering the MERGE dilution tables. These are the densimetric Froude number (F_r), stratification parameter (SP), current to effluent velocity ratio (k), and the port spacing parameter (PS). The Froude number (eq-1) is a dimensionless parameter dependent on the effluent velocity from an individual port and relates the density stratification between the effluent and the ambient sea water.

$$F_r = v/\sqrt{g' d_o}$$ eq-1

where :

v = port velocity effluent
g' = reduced gravity (eq-2)
d_o = port diameter

and :

$$g' = ((\rho_a - \rho_e)/\rho_e)^* g$$ eq-2

where :

ρ_a = ambient density
ρ_e = effluent density
g = gravitational acceleration

The stratification parameter (eq-3) is also related to the density stratification but involves the effective diameter of the port and the density of ambient fluid as a function of the depth.

$$SP = \rho_e/(d_o{}^* d\rho_a/dz)$$ eq-3

where :

z = depth of port below the water surface

The current to effluent ratio is a comparison between the two velocities (eq-4) and the port spacing (eq-5) is the ratio of the distance between the ports to the port diameter.

$$k = \mu_a / v$$ eq-4

where :

k = current to effluent ratio
μ_a = ambient velocity
v = effluent velocity

$$PS = S_1/d_o$$ eq-5

where :

S_1= distance between ports

To begin the diffuser design using MERGE, the effluent flow (using the average daily discharge) per port was calculated, different values for the number of ports were chosen and the port diameters were estimated. The port diameters used were 1, 2, 3, and 4 inches, and the number of ports were 10, 15, 20, 25, and 30. Port sizes and numbers outside of these ranges resulted in excessive head losses at the diffuser or nonuniform port flows. Velocities were computed for the different combinations of the port diameters and number of ports, then all of the dimensionless parameters were calculated for the respective MERGE variables. The current to effluent ratio for both the strongest and weakest in situ currents were applied. For the change in ambient density to depth ratio used in the stratification parameter, a value of 1.0 was used, indicating that for depths from 30 - 2500 feet, there is very little change in density (Grace, 1978).

Diffuser port spacing and port diameter

With the various combinations of parameters, the MERGE dilution tables (USEPA, 1985) were employed to estimate dilution and trapping level values. From the port spacing and number of ports, the length of the diffuser (L) could be obtained from equation 6. This was useful to eliminate larger models from consideration due to the limited space of the actual design site. The known geology of the site was only valid for 75 ft to either side along the pipe route. The length, along with the dilution value and plume rise were the final factors in deciding the final outfall design.

$$L = PS * n \qquad \text{eq-6}$$

where :

n = number of ports

After trying several combinations, the most practical design, (considering the plume dilution, trapping level, and port size), for the two foot diameter pipe consisted of 20 ports of 2 inch diameter each, which produced a dilution of 530 and trapping levels of 15 and 26 feet for the fast and slow currents, respectively. The port spacing of 50 inches, on center, was used, to yield a port spacing parameter of 25 and a total diffuser length of 83.3 feet, which was rounded to 85 feet for simplicity of construction. Figure 3 depicts this design. (The stratification parameter of infinity was used because the density stratification was considered to be 0).

TABLE 1.

Summary of MERGE Design Parameters

Design Parameters	Values
Froude number	17.170
Stratification parameter	infinity
Port spacing parameter	25
k strong	0.211
k weak	0.055

These values are dimensionless and lead to the following design

Prototype

Design Variables	Values
Number of ports	20
Port diameter	2 inches
Distance between ports	50 inches
Length of diffuser	85 feet (>83.3 ft)
Velocity of effluent	6.4 feet/second

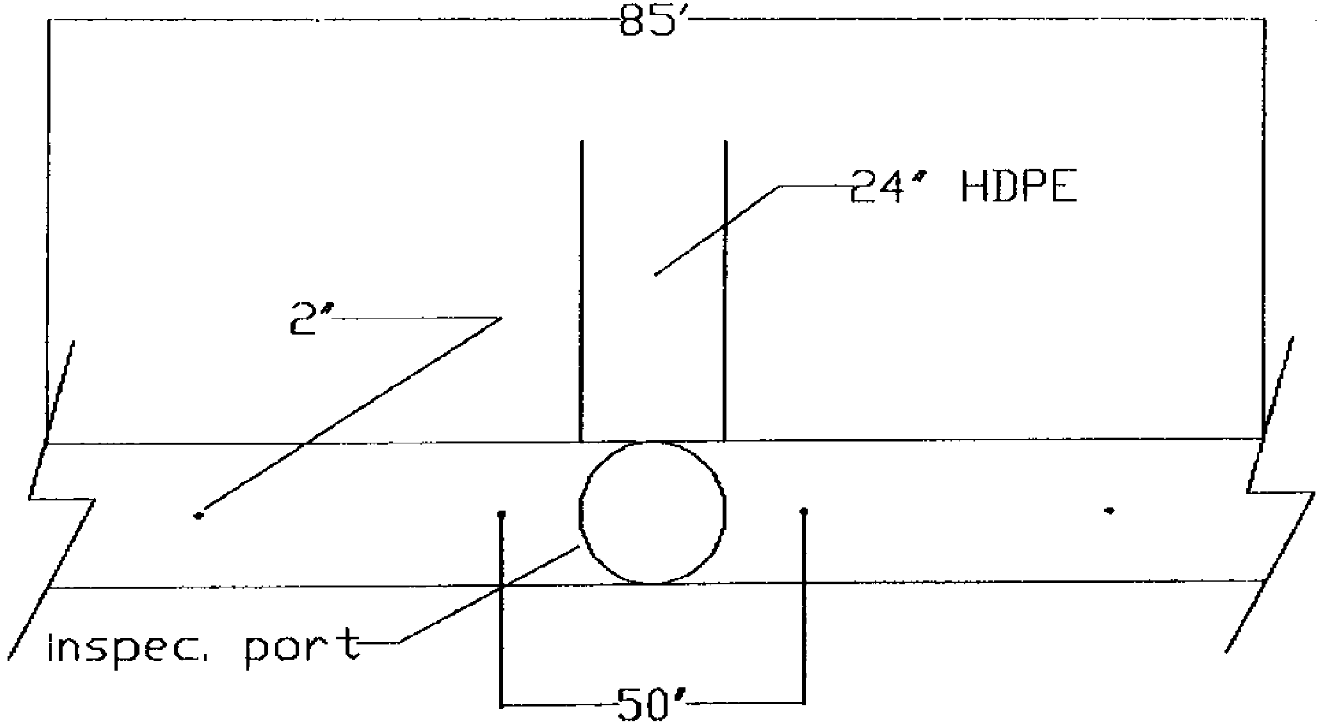


Figure 3. Initial diffuser design concept

Design Considerations

The considered designs were based on research of existing diffusers that could meet the needs of the Seabrook plant in durability, ease of maintenance, as well as cost efficiency. The two diffusers had to meet the following requirements:

- Able to withstand erosion or deposition
- Survival through severe storm
- Maintenance and inspection
- Biofouling

Durability

GZA GeoEnvironmental , Inc., (GZA, 1994) performed a soil profile of the outfall route (fig 4), and determined that the sediment is mostly a stratified sand and glacial silt with some marine clay. Normandeau Associates, Inc. (NAI, 1975) performed a study in the area of the pipeline/diffuser route and determined that a plus or minus 6 inch depositional or erosional event could occur. GZA also performed an acoustic survey and concluded that the sediments were unstratified and there was a subbottom at a depth of 5 feet. GZA defined the subbottom to be that of a hard layer of soil that would not be susceptible to a significant erosional event(GZA, 1994). Stearns and Wheler arrived at the conclusion that the diffuser should not only be designed for the plus or minus 6 inch event, but also the possibility of a 5 foot erosional event.

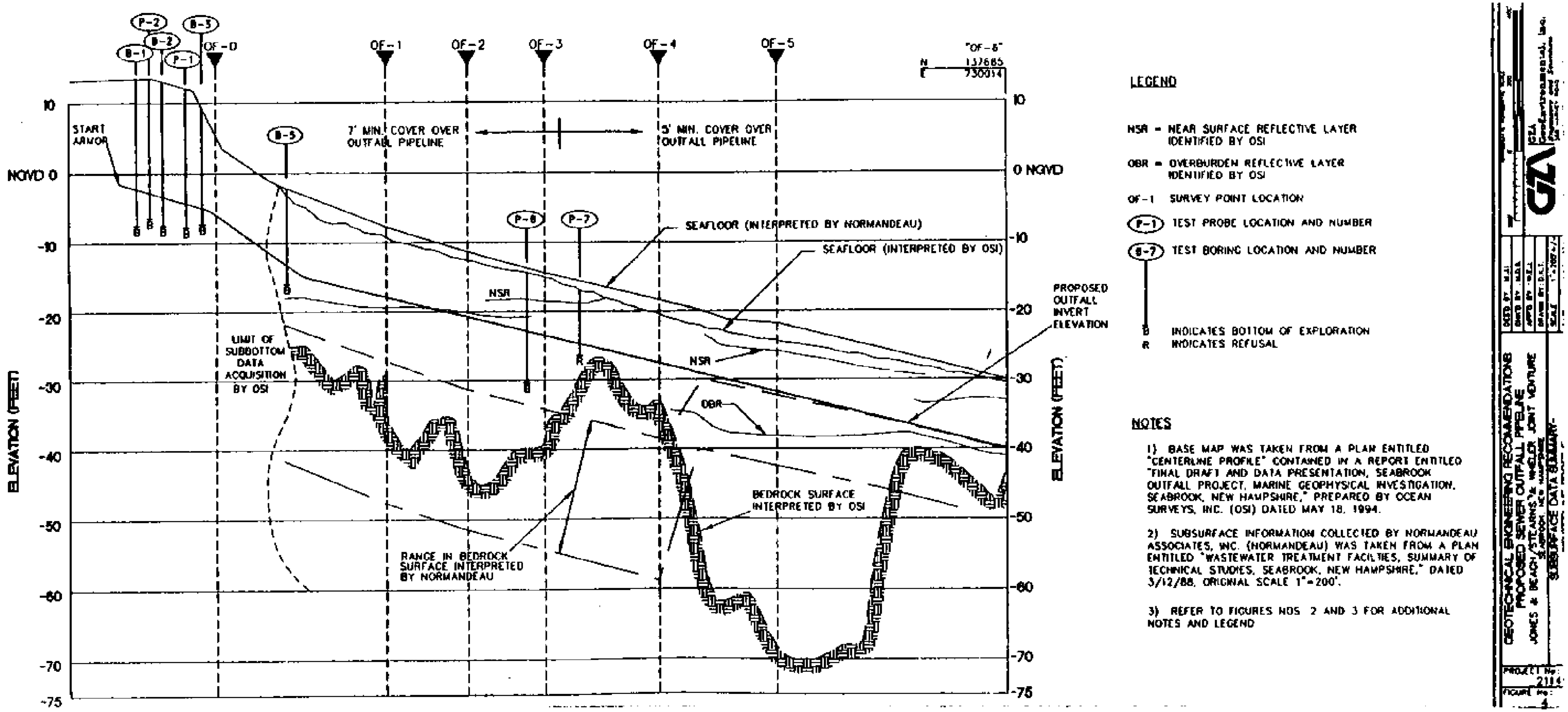


Figure 4. Seafloor profile at outfall location

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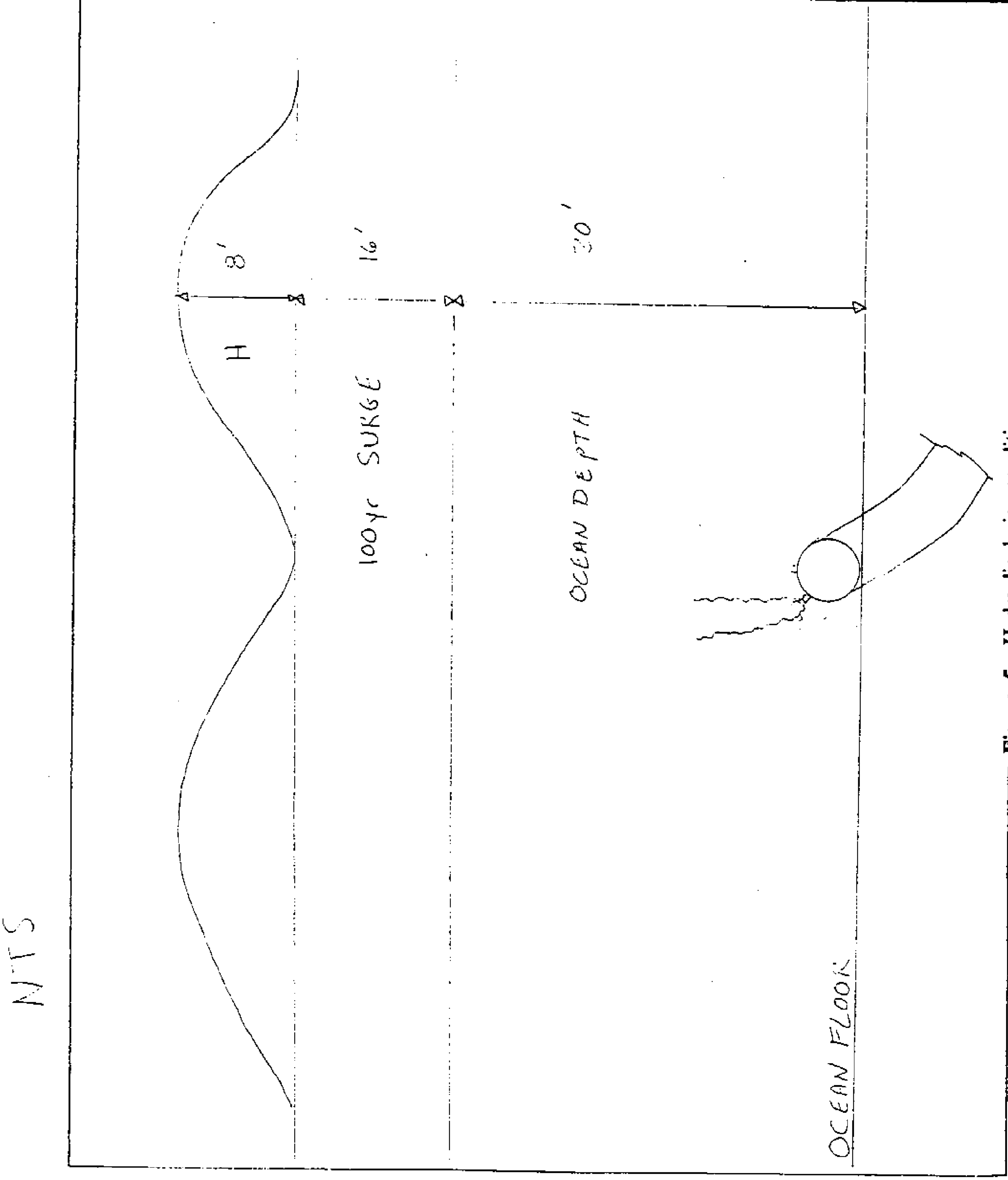

Figure 5. Hydraulic design conditions

Survival through the 100 year storm

The diffusers in this study were designed to withstand the 100 year storm conditions. To obtain such storm characteristics, a study was conducted by NUMERIC Environmental Modeling. (NUMERIC, 1994) Using a program called the Regional Coastal Process Wave Propagation Model (RCPWAVE) the effects of the 100 year storm at the proposed site were calculated. RCPWAVE parameters used for the design of the diffuser include: the wave period, the surge water level, and the wave height. NUMERIC found the wave periods to be between 10 and 14 seconds. RCPWAVE calculated a surge water depth of 16 feet, and a significant wave height of 8 feet.

The surge height is the local rise in water level due to the shoreward transport of water from wind stress due to the storm. On top of the surge water level is the storm wave height of 8 feet (fig 5). The significant wave is that of which one third of waves, on the average, are larger (Shore Protection Manual, Vol. 1, 1984).

The storm surge is important because shallow water wave heights are limited by water depth. Waves will break when the wave height is roughly 0.78 times the water depth. The energy of a wave is proportional to the square of the wave height. The added depth of a storm surge allows more energetic waves to reach the diffuser site before breaking and thus more energy would be transferred to the diffuser (Dean & Dalrymple, 1984). At the Seabrook site, Shoals three miles eastward prevented the occurrence of the significant wave heights larger than 8 ft.

With this data, vertical as well as horizontal bottom velocities and accelerations during the 100 year storm were computed using Airy Wave theory. (Shore Protection Manual, Vol. 2, 1984)

$$Uh = \frac{H}{2}\sigma\frac{\cos k(h+z)}{\sinh kh}\cos(kx-\sigma t) \qquad \text{horizontal velocity} \qquad \text{(eq-7)}$$

$$Uv = \frac{H}{2}\sigma\frac{\sinh k(h+z)}{\sinh kh}\sin(kx-\sigma t)$$ vertical velocity (eq-8)

$$Ah = \frac{H}{2}\sigma\frac{\cosh k(h+z)}{\sinh kh}\cos(kx-\sigma t)$$ horizontal acceleration (eq-9)

$$Av = \frac{H}{2}\sigma\frac{\sinh k(h+z)}{\sinh kh}\cos(kx-\sigma t)$$ vertical acceleration (eq-10)

where :

H = wave height (ft)
k = wave number = $2\pi/L$ (L = wave length) (1/ft)
$\sigma = 2\pi/T$ T = wave period (sec)
x = horizontal distance to reference point (ft)
h = water depth + surge (ft)

The resulting design storm velocities and accelerations at the diffuser are shown in Table 2.

TABLE 2.

Airy wave theory design wave parameters

Wave length L= 304 ft.	Wave height H= 8 ft, h= 46 ft	Period T= 10 sec
	Velocities (ft/s)	**Accelerations (ft^2/s)**
Maximum horizontal	3.863	2.426
Maximum vertical	3.109	1.952
Wave length L= 433 ft.	Wave height H= 8 ft, h= 46 ft	Period T= 14 sec
	Velocities (ft/s)	**Accelerations (ft^2/s)**
Maximum horizontal	3.301	1.481
Maximum vertical	2.164	0.971

Since the velocities and accelerations are greater using the 10 sec period, these calculations were used in the force analysis for the diffuser.

The analysis of the wave - induced forces on a diffuser was divided into two parts. The first part is the force caused by hydrodynamic drag. This force is created by the steady current of water on the diffuser by passing waves. The second part of the wave force is the inertial force, which is caused by the acceleration of a passing wave on the diffuser. The total force was calculated using Morison's equation: (Shore Protection Manual Vol. 2, 1984)

$$F = F_d + F_i \qquad \text{(eq-11)}$$

where :

F = total wave/ current force (lbs)
F_d = drag force (lbs)
F_i = inertial force (lbs)

and

$$F = \frac{1}{2} C_d \rho A U|U| + C_i \rho v \frac{du}{dt} \qquad \text{(eq-12)}$$

where :

C_d = drag coefficient
ρ = density of water
A = projected area/unit elevation of the pipe (ft^2)
U = relative velocity of fluid with respect to the pipe (ft/s)
C_i = inertia coefficient
V = volume of pipe/ length (ft^3)

Equation 12 was also used to compute the minimum stable armor stone size. This then can be translated into the minimum weight of an armor stone that would not be moved by this storm. The determination was that any smaller than a 1.5 ft^3 rock (assuming a Sg = 2.6) could possibly be moved by this storm.

The first diffuser design consideration concept was to have the diffuser on the ocean floor without any kind of support. This proved to be infeasible due to the chance of an erosional event where the diffuser would have no support and could possibly fail. Therefore, the following two diffuser designs were evaluated: a buried diffuser with riser effluent ports and an anchored diffuser at the ocean floor.

Diffuser Design 1

The first diffuser design was similar to the system being constructed for the Deer Island plant, except at much smaller scale (fig 6). The main outfall pipe lies on the hard layer 5 feet beneath the ocean floor. The two inch diameter risers were designed to each have a cone shaped protective shield which is meant to deflect floating debris or objects (for example lobster traps, anchors, etc.) that could possibly break them off. These cones, along with an inspection pipe, are the only visible structure under the current elevation of the ocean floor. Table 3 depicts the forces exerted on this design before and after the design erosion event. It can be seen that when the risers are buried, little force is exerted on them. When the diffuser is fully exposed after the five foot erosional event, a different situation arises, and the force on each riser greatly increases, giving them the possibility of failure.

Diffuser Design 2

The second diffuser design is one that is exposed above the existing ocean floor (fig 7). The outfall pipe comes out at a gradual bend to a tee at the center of the diffuser. The bottom of the diffuser pipe is roughly 1 foot above the ocean floor. This allows up to three feet for the deposition discussed earlier. The force on this diffuser greatly increased during the five foot erosional event (table 4) and to counter this, 6 precast 7'x3'x3' concrete anchors were designed to be buried to the hard layer below. Even though the force increases after the erosional event the concrete anchors are heavy enough to resist this increase in force, even if the diffuser was filled with air. The weight of the anchors are also such that even considering a friction factor of 1, the diffuser does not have to be back filled with armor. This design has two inspection ports, one at each end, yielding greater inspection ease than the first design. To contain the diffuser on the anchors, a stainless steel strap is recommended that will bolt into the anchor on each side.

Diffuser Material

The material chosen for these designs was that of a high density polyethylene (HDPE). This is consistent with the outfall pipe material and will ease installation. This material is non corrosive in the ocean, environmentally safe, strong, and does not invite biofouling. Its only negative aspect is that it is light weight, which makes the construction easier, but results with a higher buoyancy. It was calculated that for the worst case, with the diffuser filled with air, the buoyant force was 131 lbs/ft. This force is taken into account in the diffuser designs

FORCE CALCULATIONS

F= Fd +Fi

Table 3

Design 1

Diffuser pipe covered with exposed risers

			Horiz	Vert
base dia =	1 ft		**Horiz**	**Vert**
height =	2 ft	Drag Force(lbs/riser)=	15	3
projected area =	1 ft^2	Inertial Force(lbs/riser)=	33	0
volume =	2.09 ft^3	**Total(lbs/riser)=**	**48**	**3**
Cd =	1			
Ci =	3.29	force on diffuser(lbs)r=	960	60

After erosional event

base dia =	20 in		**Horiz**	**Vert**
height =	3 ft	Drag Force(lbs/riser)=	37	21
projected area =	2.5 ft^2	Inertial Force(lbs/riser)=	139	111
volume =	8.72 ft^3	**Total(lbs/riser)=**	**176**	**132**
Cd =	1	force on pipe(lbs)=	2469	0
Ci =	3.29	force on diffuser(lbs)r=	5989	2640

Table 4

FORCE CALCULATIONS

F= Fd +Fi

Design 2

Diffuser pipe with concrete anchoring system

base dia =	2 ft		**Horiz**	**Vert**
length =	83.3 ft	Drag Force(lbs)=	3020	1606
projected area =	203 ft^2	Inertial Force(lbs)=	5888	4738
volume =	370 ft^3	**diffuser Total=**	**8908**	**6344**
Cd =	1			
Ci =	3.29			

After erosional event

base dia =	2 ft		**Horiz**	**Vert**
length =	83.3 ft	Drag Force(lbs)=	4359	516
projected area =	293 ft^2	Inertial Force(lbs)=	10186	8195
volume =	640 ft^3	**diffuser Total=**	**14545**	**8711**
Cd =	1			
Ci =	3.29			

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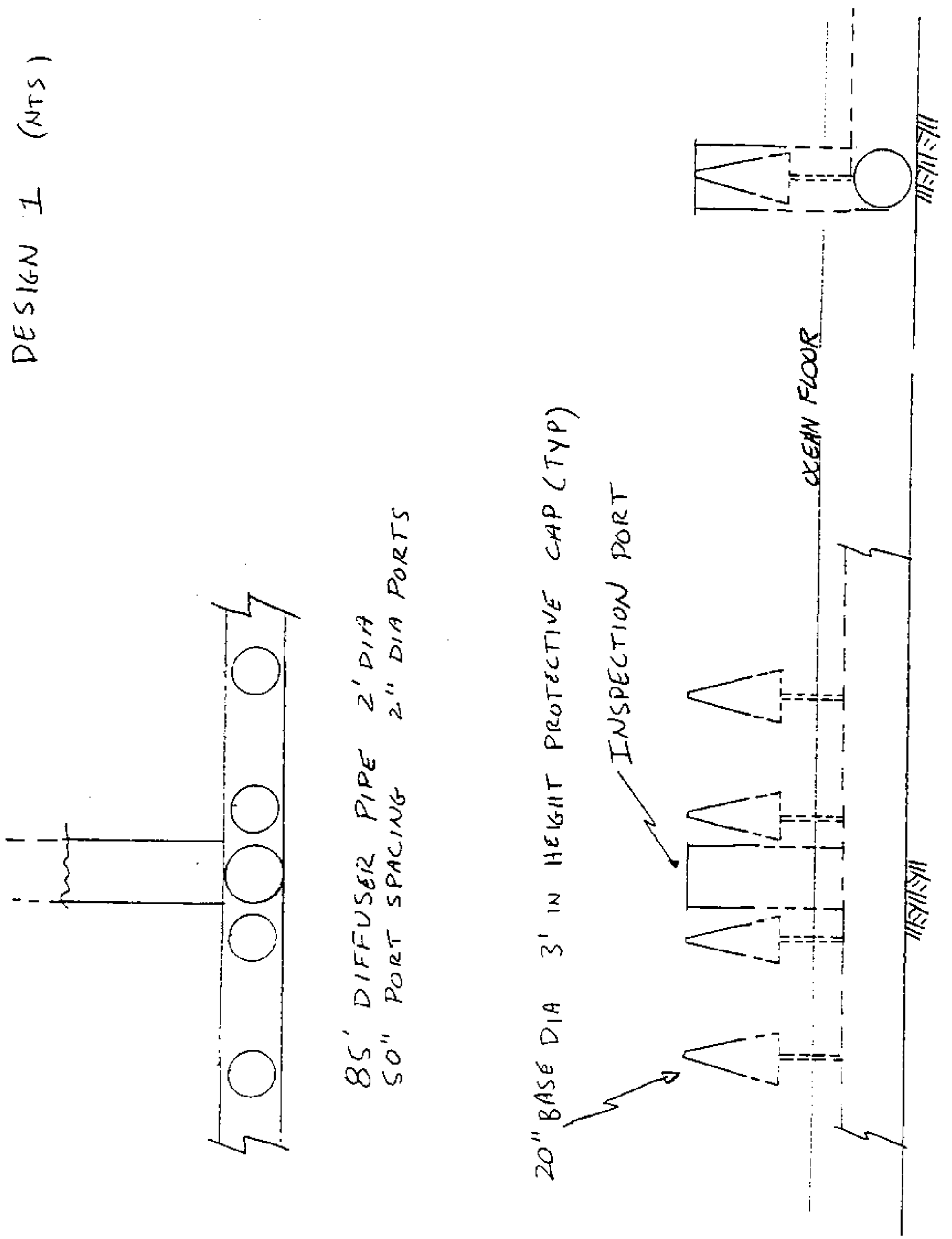

Figure 6. Diffuser system 1

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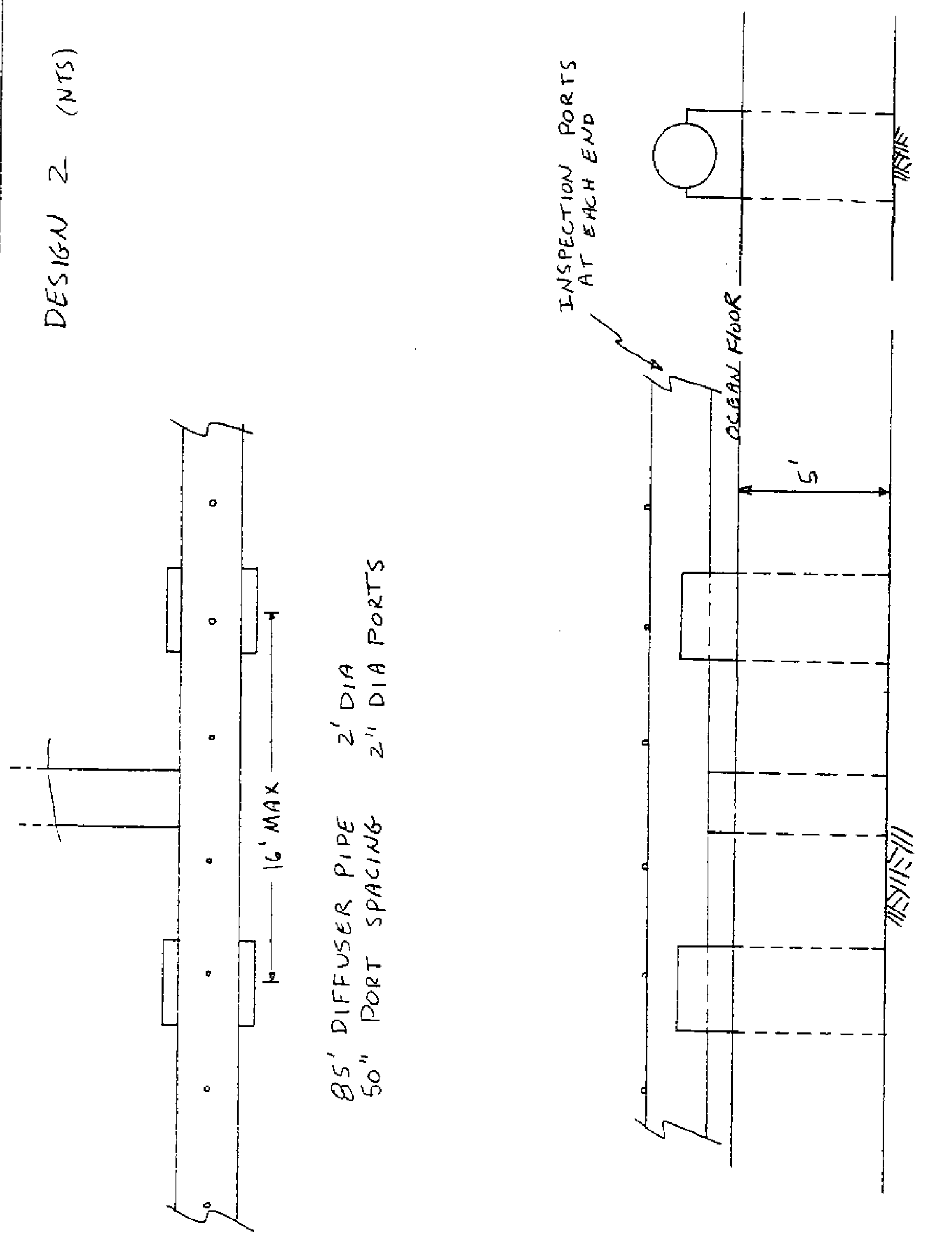


Figure 7. Diffuser system 2

Physical Modeling

Physical modeling was performed to test for the dilution's predicted by MERGE. While modeling, several factors were observed. Among these were:

- merging of plumes,
- current affects on the plume, and
- plume dilution after aspiration.

Model considerations

To physically model the diffuser, geometric similitude (eq-13) was considered for the size of the model and Froude similarity (eq-14) was used to calculate the model discharge.

$$L_m/L_p = 1/10 \qquad \underline{\text{eq-13}}$$

where :

L_m = model length
L_p = prototype length

$$Q_m/Q_p = (L_m/L_p)^{5/2} \qquad \underline{\text{eq-14}}$$

where :

Q_m = model discharge
Q_p = prototype discharge

By assuming a geometric scale, the dimensions of the model were calculated, then applied to determine whether the size was feasible for the tank. Major considerations that went into the scale were, the number of ports that could be used within the width of the tank to help reduce side effects, and the size of the ports to have a noticeable flow. The scale chosen was 1:10, which lead to the model dimensions of a 2.4 inch diameter pipe, and ports of 0.2 inches. Due to availability, a 2 inch PVC pipe and 0.206 inch ports were used. These standard materials yielded an actual port diameter scale ratio of 1 : 9.7. Schematics of the model can be seen in figures 8 - 10.

The discharge for the model was calculated by equation 14 using the prototype design flow per port and the geometric scale. The model discharge from the prototype flow of 1.8 MGD was 0.79 gallons per minute (gpm).

For successful hydraulic modeling, there are four requirements that should be observed and these include (Sharp, 1981):

- geometric similarity; no exaggeration of scale in all dimensions
- equal densimetric Froude numbers in model and prototype
- Reynolds number greater than approximately 2500; and
- equal Froude numbers, $v/(gd_o)^{1/2}$, in model and prototype.

The point of having equal densimetric Froude numbers, is to ensure that the density difference between the ambient and effluent remains the same as the prototype. By preserving these model relationships, the model acts hydraulically as the prototype.

Model set-up

In order to run the model, two variable speed peristaltic pumps were used in parallel to acquire the desired flow of 0.79 gpm. Calibration of the pumps was conducted over a period of three days. The two pumps were set at different speeds and the time was measured for them to displace 2.0 gallons from a bucket of water. The desired time of 2.53 minutes, (2 minutes and 32 seconds), was reached quickly but many more trials were used to obtain an acceptable average. The best combination was to set the pump settings at 8 and $8^1/_2$ for the older and newer pumps, respectively. Number 18 Masterflex tubing

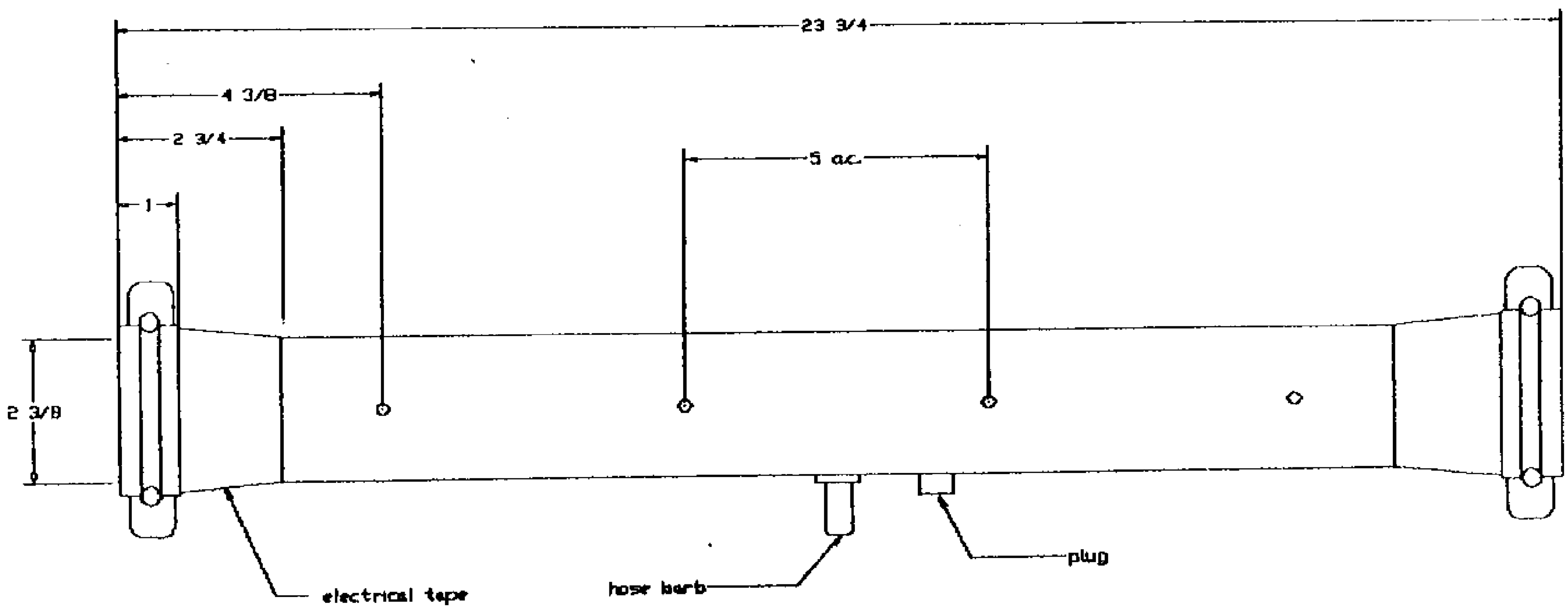


Figure 8. Physical model - top view

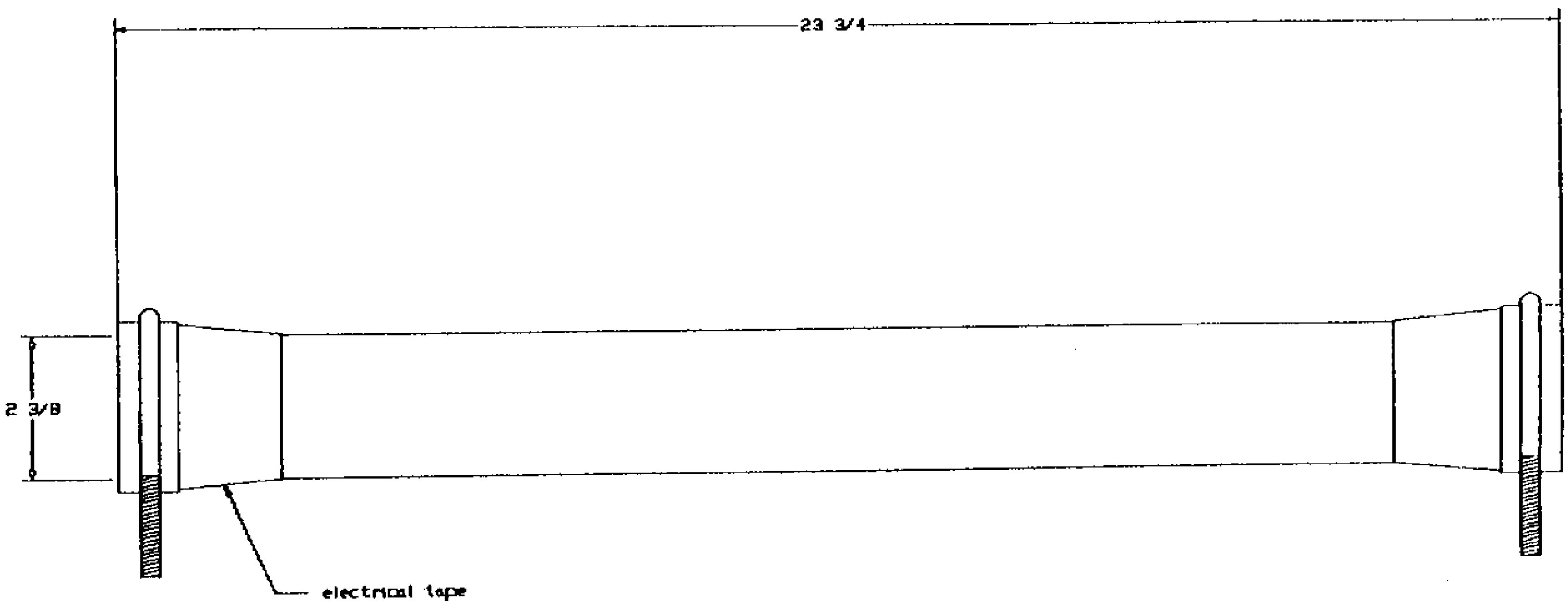


Figure 9. Physical model - front view

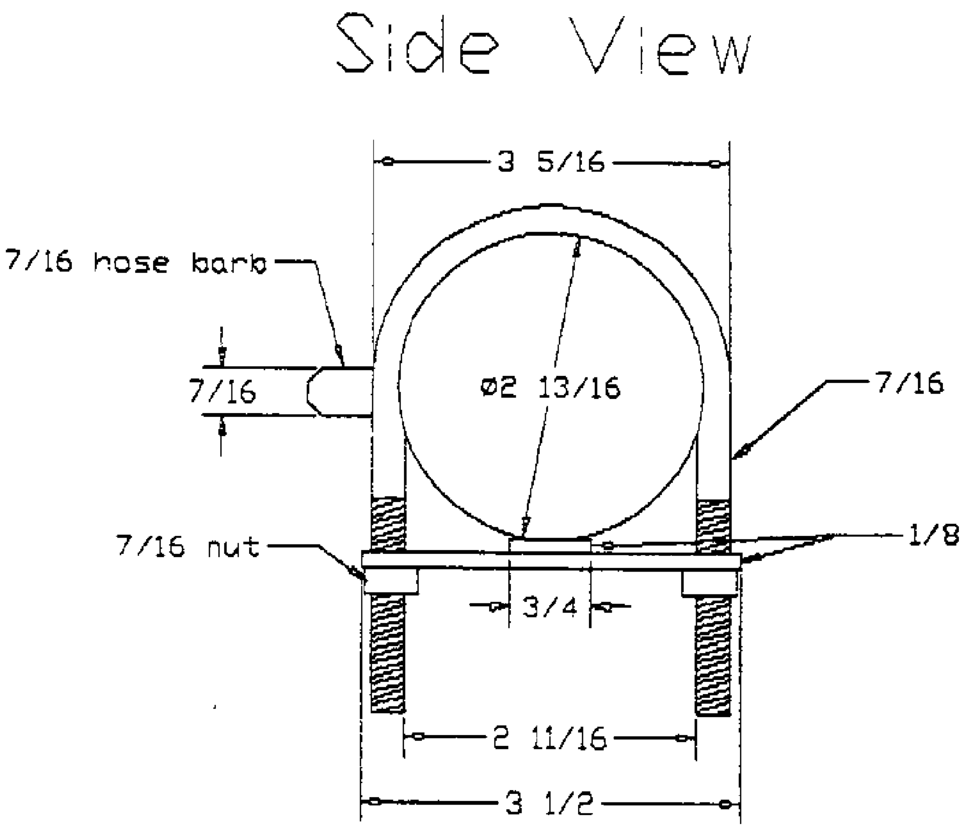


Figure 10. Physical model - side view

was used and connected from each pump by a tee, and then a single feed line continued into the tank and the diffuser. The hose was connected to the model by a $^{5}/_{8}$ inch hose barb, located at the center on the upstream side.

The tank measured roughly 40 feet long, 2 feet wide, and 3 feet deep representing a 30 foot in situ depth and a 20 foot in situ width. The tank consisted of PVC piping at the head to create uniform flow and it was long enough to view the effects of the plume downstream. The diffuser was placed at approximately one third of the total length from the head box and the three downstream sampling locations were situated at arbitrary distances of 34, 83, and 129 inches. The simulated ambient flows were 0.8, 0.6, 0.4, and 0.2 knots in the prototype and were calculated by the Froude relationship of equation 15.

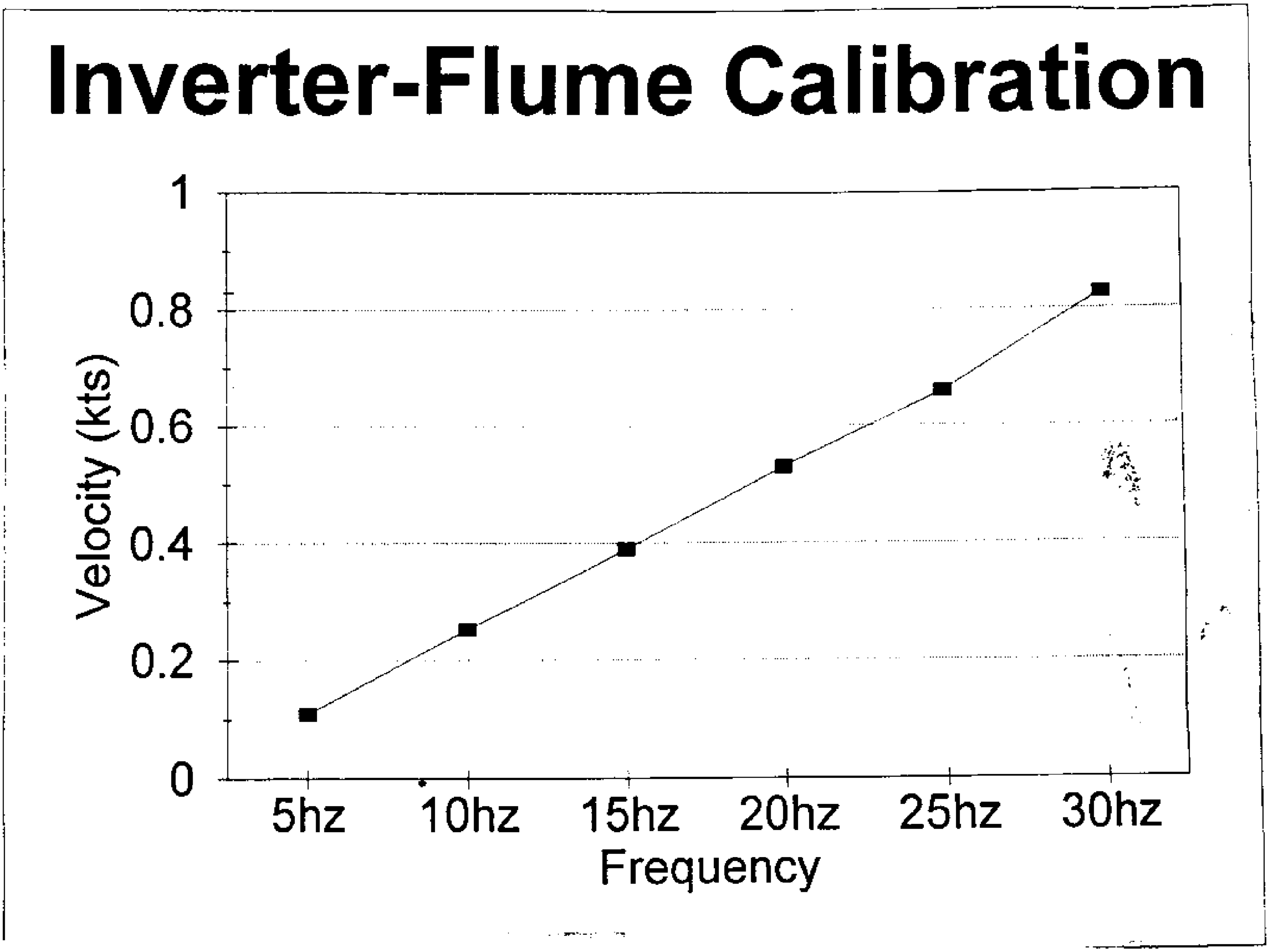
$$V_m/V_p = (L_m/L_p)^{1/2} \qquad \text{eq-15}$$

where :

V_m = model velocity
V_p = prototype velocity

The motor speeds of the tank were measured in hertz, so a calibration chart (figure 11) was used to obtain the desired water velocities of 0.253, 0.190, 0.126, and 0.063 knots for the model. These calibrated velocity values were measured 2 inches above the physical model by a laser system.

Samples from the tank were taken at 4 stations, one upstream and three downstream of the diffuser. A station consisted of a #18 Masterflex tube connected to a 4 foot glass rod and extended over the tank side to waist level. At the intake end, a bent wire of 90 degrees was inserted into the rod and the tubing was fastened to this, and pointed upstream. After running the physical model for a test run, the sampling stations were installed at the observed plume centerline, both vertically and horizontally. The


Inverter-Flume Calibration
Velocity (kts)
1
0.8
0.6
0.4
0.2
0
5hz
10hz
15hz
20hz
25hz
30hz
Frequency

samples were extracted from the tank, through the tubing, by negative pressure (a siphon effect), in order for the plume to remain undisturbed by movement. A stopper was attached to the effluent end of the sampling tube to prevent sampling when not taking samples. The port located upstream of the model was used as a measure for the background concentration, and the three downstream were used to measure the diluted plume concentrations.

For visual effects and the measurement of plume concentrations, Rhodamine WT dye was used. A 55 gallon drum was filled with water, dye was added until a noticeable pink color occurred, then samples of this effluent water were taken. All of the samples were placed in clean, opaque bottles and then immediately stored in boxes. This was done to prevent degradation of the Rhodamine by sunlight.

When samples were taken, the stations were unplugged and run to purge the stagnant fluid inside the tubing. Once tube purging was completed, bottles were filled nearly simultaneously, capped, and boxed. To obtain average values, four samples for each tube were taken for each run (each run had a different current velocity), and then the diffuser was shut down while the tank continued to flow in order to equilibrate the dye within the ambient fluid (approximately 3 minutes). Upon completion of testing, the 3 downstream stations were vertically stacked 4 inches apart, 34 inches away from the diffuser, to investigate the vertical concentration gradient of the plume. These results are shown in figure 12.

Sample measuring

A fluorometer (Turner Designs, Model 10 Series), was used to measure the

Rhodamine WT concentrations in the samples. The fluorometer works by estimating the fluorescence of the sample in an enclosed cuvette holder. From a meter reading the concentration is computed from a series of equations. The fluorometer must first be calibrated using a blank sample (distilled water) and standards (these were previously made in concentrations of 0.1 ppm and 0.01 ppm). The calculated concentrations are shown in the appendix.

The actual concentrations downstream are found by taking the observed concentrations and subtracting the upstream (background) concentrations. By use of equation (eq-16, USEPA, 1985) and the dilution predicted by MERGE (530), the desired concentration is determined. The desired concentration is where the dilution is equal to 530. Figures 12-15 were developed from the information in table 5 and show where the desired concentration is located in the physical model. The maximum length from the diffuser to reach the desired concentration appeared in test 1, at 40 inches. This value, multiplied by the scale factor, would represent 33.3 feet in situ (table 6). As the currents decreased in speed, the desired dilution occurred closer to the diffuser. The explanation for these results is that when a faster current is present, the diffuser plume does not aspirate as readily and thus moves further downstream before achieving the desired dilution. When this happens, the plume velocity takes longer to equilibrate with the ambient flow, which is when dispersion is more effective.

$$C_d = C_a + (C_e - C_a)/S_a \qquad \text{eq-16}$$

where :

C_d = concentration at the completion of initial dilution
C_a = background concentration
C_e = effluent concentration
S_a = initial dilution

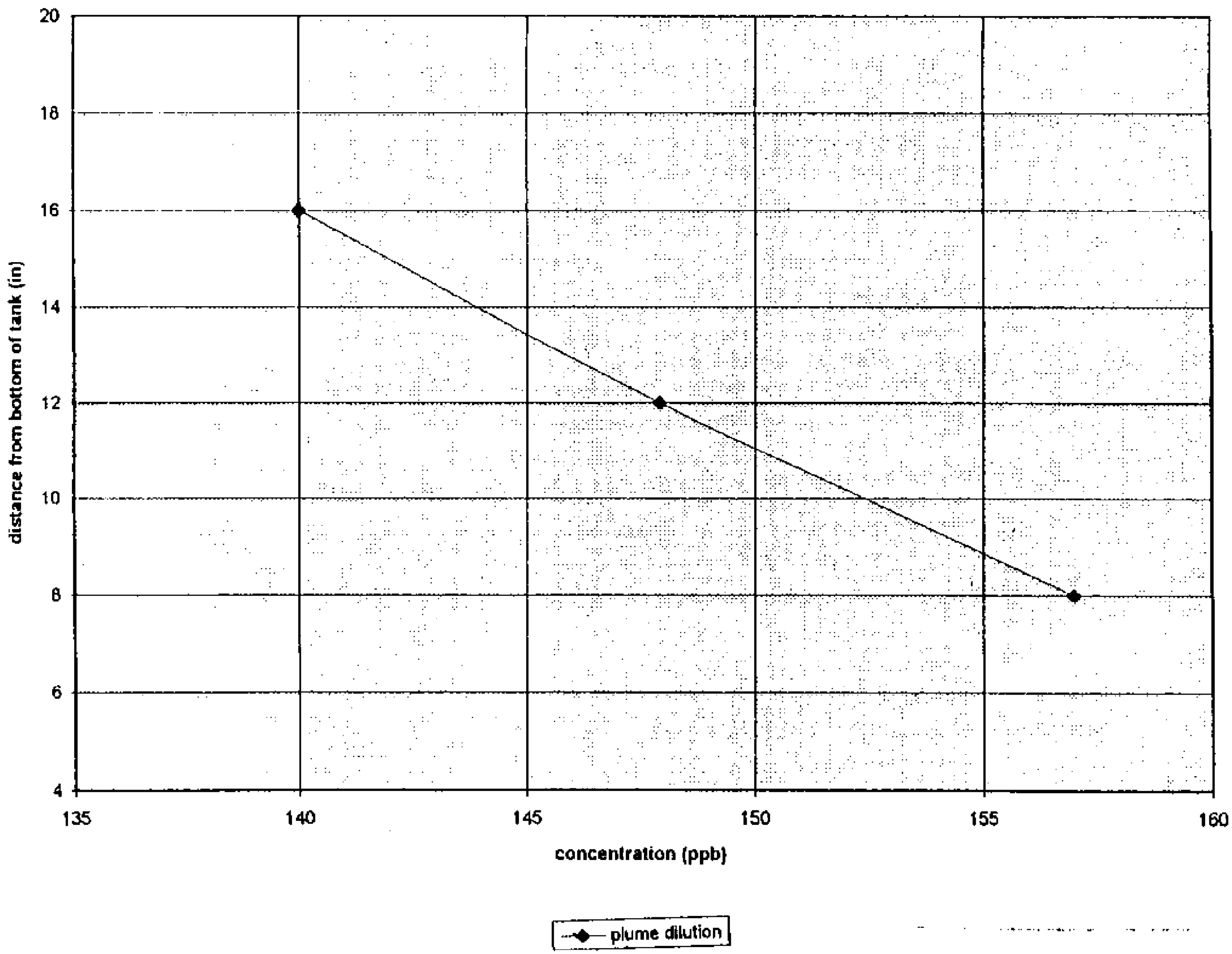


Figure 12. Vertical dilution in physical model

TABLE 5

PLUME CONCENTRATIONS OF PHYSICAL MODEL

Test	Port	Measured conc. (ppb)	Actual conc. (ppb) of model
1	1	35.45	
	2	80.79	45.34
	3	54.77	19.31
	4	51.29	15.84
2	1	70.68	
	2	106.69	36.02
	3	87.47	16.79
	4	87.14	16.46
3	1	101.40	
	2	150.84	49.45
	3	127.64	26.24
	4	125.00	23.60
4	1	127.37	
	2	180.38	53.01
	3	163.64	36.26
	4	148.47	21.10

TABLE 6.

Test	Desired concentration (ppb)	Distance from physical model (in)	Distance from prototype (ft.)
1	36.45	40.0	33.3
2	71.56	26.0	21.7
3	102.23	21.0	17.5
4	128.15	17.5	14.6

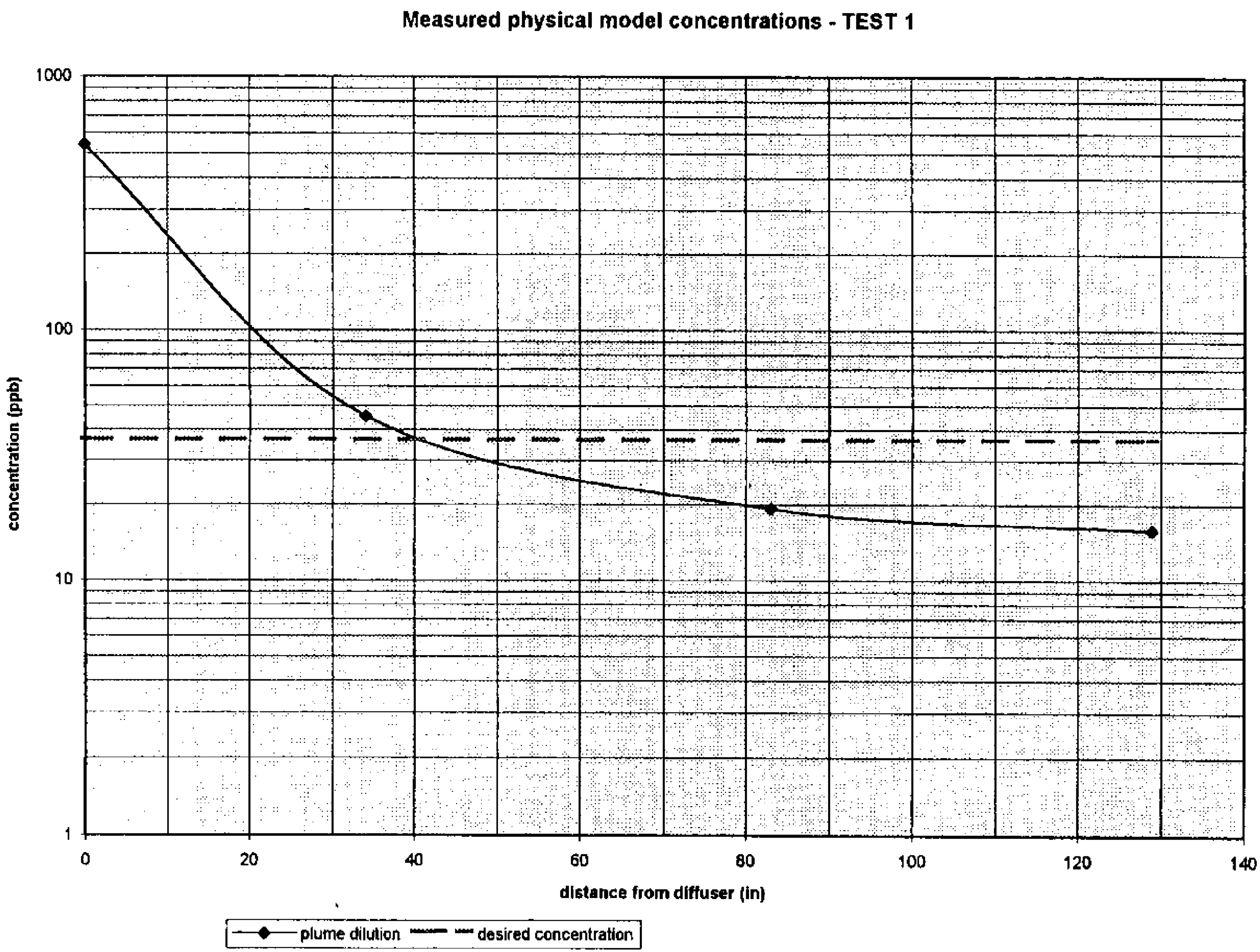


Figure 13. Measured physical model concentrations

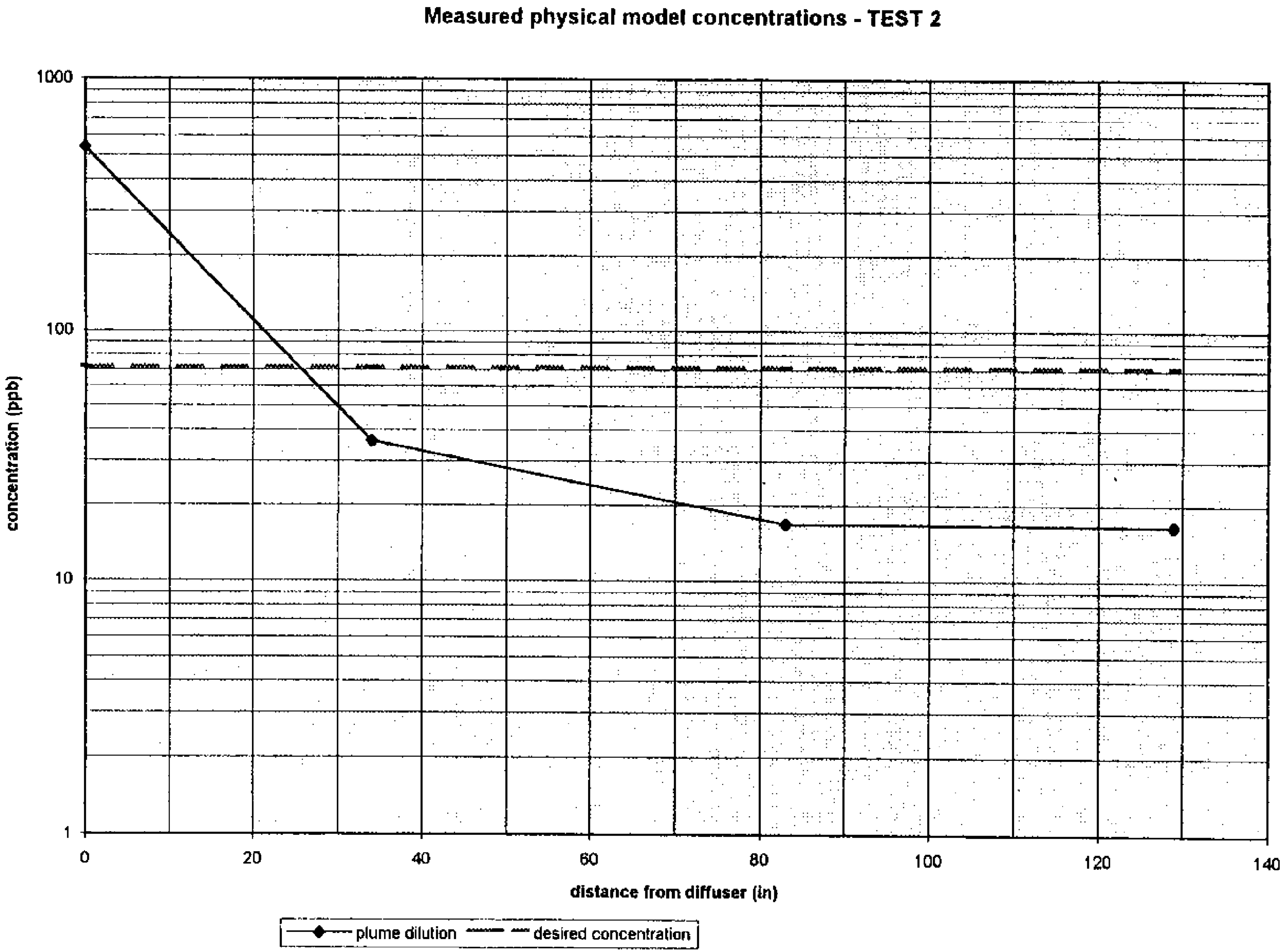


Figure 14. Measured physical model concentrations

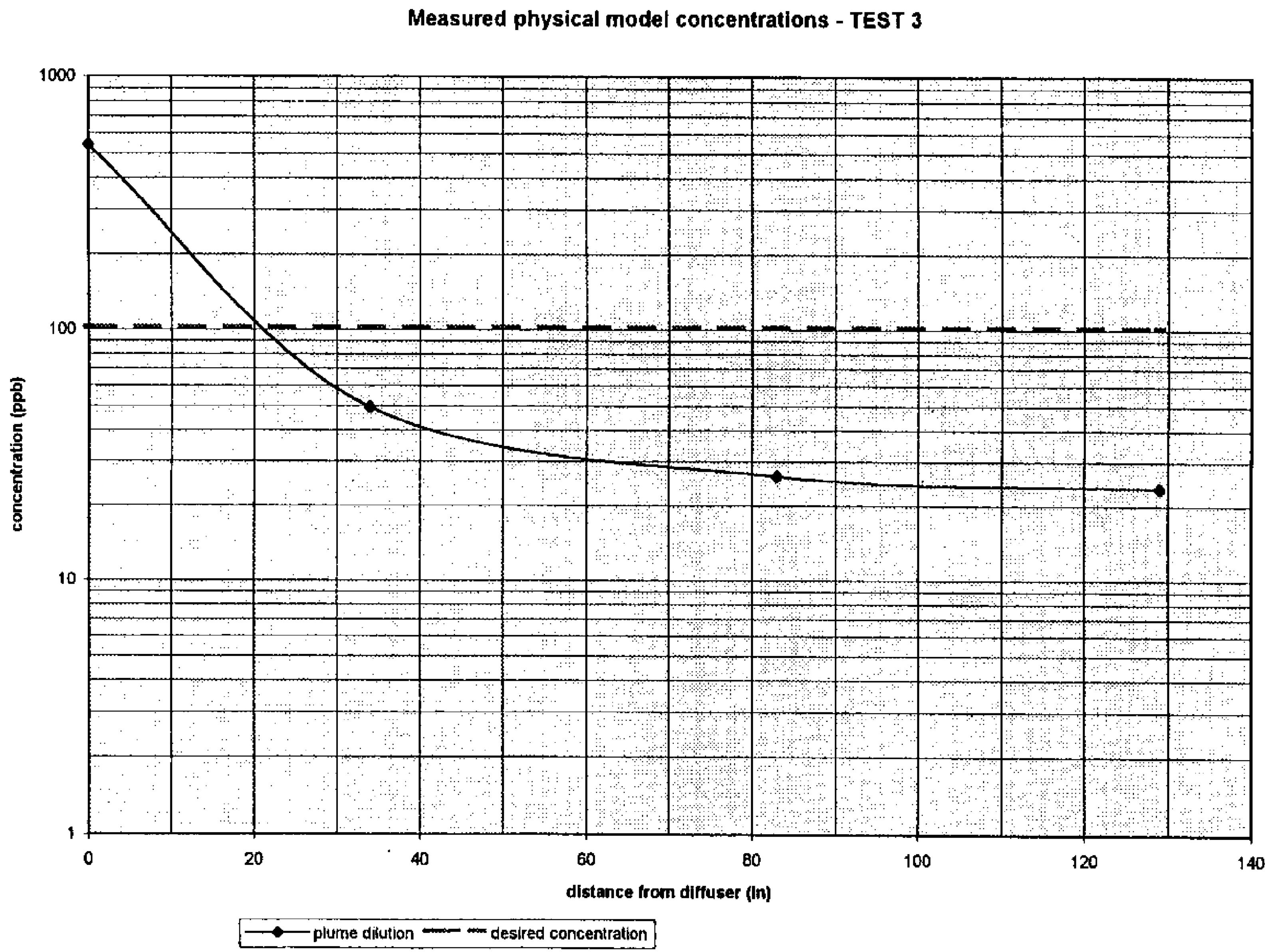


Figure 15. Measured physical model concentrations

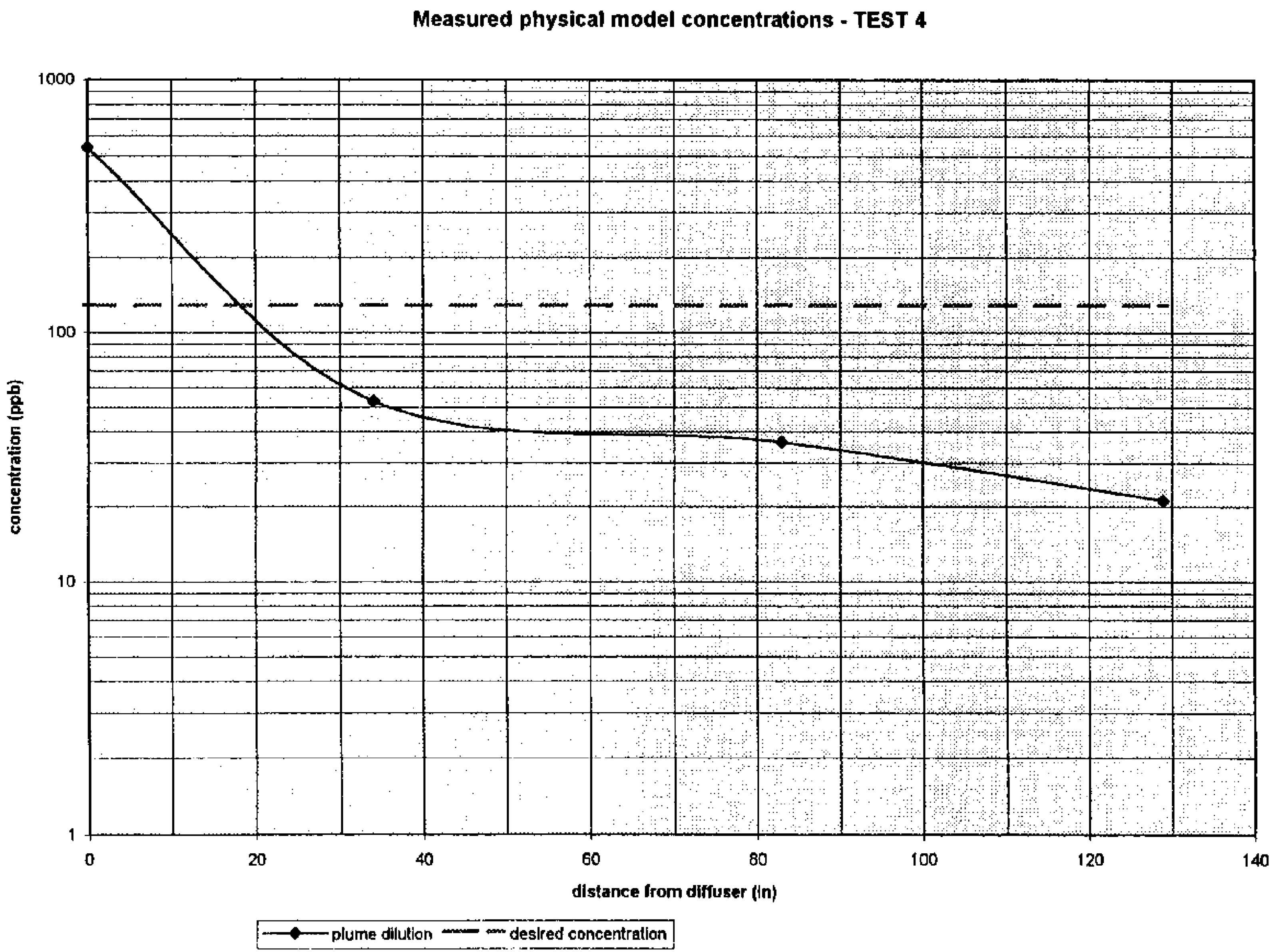


Figure 16. Measured physical model concentrations

Recommendations

The proposed design for the Seabrook wastewater treatment plant is that of design 2. This design meets all of the requirements that were considered in the analysis with an added degree of safety:

- Able to withstand erosion or deposition
- Survival of severe storm
- Maintenance and inspection
- Biofouling

This design was determined to withstand the above conditions in a manner that meets standard engineering practice. It proved to be efficient, strong, (able to withstand the 100 year wave induced horizontal and vertical force of 23,256 lbs with its own 34,398 lbs of submerged weight.) and can be easily maintained. The anchoring system proved to be very stable in the 100 year storm conditions, as well as the possible 5' foot erosional event. It contains two inspection ports and is designed above the ocean floor, which will make any possible maintenance easier.

Design 1 which works on a plant the size of Boston's, would work on this scale as far as dilution is concerned, and the force on this design was determined to be much less, but the riser design is to susceptible to damage during the design storm with the calculated size stone that could possibly become mobile, which could lead to failure. The diffuser for the most part is buried under the ocean floor, thus making inspections harder, as well as any possible maintenance.

Appendix

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Project/Problem Set: Merge
Detail/Problem: Flow calcs parameters
Class: Prof:
Sheet: of
Calc. by: RJF
Date:
Chck. by:
Date:

Flows: Avg (MGD)
1.8

$$Q = 1.8 \frac{MG}{D}\left(\frac{1\,day}{24\,hr}\right)\left(\frac{1\,hr}{3600\,sec}\right)\left(\frac{ft^3}{7.481\,g}\right)\left(\frac{10^6\,g}{MG}\right) = 2.78\ ft^3/s$$

(20.8 gal/s)

for 24" pipe (to diffuser)

$$V_o = \frac{Q}{A} = \frac{2.78\ ft^3/s}{\pi(1\,ft)^2} = 0.885\ ft/s$$

using 20, 2" ports

$$V_{port} = \frac{Q}{A_{port}(\#\,ports)} = \frac{2.78\ ft^3/s}{\left(\frac{1}{12}\right)^2 \pi\ 20_{ports}} = 6.37\ ft/s \quad (\text{at each port})$$

MERGE:

Froude number (Fr)

$$Fr = V/\sqrt{g' d_o}$$

$$= 6.36\ ft/s \Big/ \sqrt{0.830\tfrac{ft}{s^2} \cdot \tfrac{1}{6}ft}$$

$$Fr = 17.10$$

$v = 6.37$ ft/s
$d_o = 1/6$ ft

$$g' = \left(\frac{\rho_a - \rho_c}{\rho_c}\right) g = \left(\frac{1.99 - 1.94}{1.94}\right) 32.2 =$$

$$g' = 0.830\ ft/s^2$$

Stratification Parameter (SP)

$$SP = \frac{(\rho_a - \rho_a)}{d_o \frac{d\rho_a}{dz}}$$

$$= \frac{(1.99 - 1.94)}{\frac{1}{6}ft \cdot 1}$$

$$SP = 0.3 \quad \text{(however use infinite)}$$

$\frac{d\rho_a}{dz} = 1$ no change in density with depth

$d_o = \frac{1}{6}$ ft
$\rho_a = 1.99$ $\rho_c = 1.94$

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Durham, New Hampshire 03824

Project/Problem Set: Merge
Detail/Problem: parameters
Class: ______ Prof: ______
Sheet: ____ of ____
Calc. by: RJF
Date: ______
Chck. by: ______
Date: ______

Port Spacing (PS)

$$PS = \frac{\text{distance between ports}}{\text{effective diameter}} = \frac{X}{d_o}$$

$$= \frac{50 \text{ in}}{2 \text{ in}} \Rightarrow PS = 25$$

Current to effluent ratio k

$$k = \frac{U_a}{V}$$

U_a = current velocity (strong = 8 knots = 1.35 fps)
V = effluent velocity = 6.37 ft/s

$$= \frac{1.35 \text{ ft/s}}{6.37 \text{ ft/s}} \Rightarrow k = 0.212$$

The Department of Civil Engineering The University of New Hampshire Kingsbury Hall Durham, New Hampshire 03824	Project/~~Problem Set~~: DIFFUSER DESIGN Detail/~~Problem~~: AIRY THEORY CALCS Class: Prof:	Sheet: of Calc. by: JRW Date: Chck. by: Date:

$$U_h = \frac{H}{2}\sigma \frac{\cosh k(h+z)}{\sinh (kh)}$$

Pg 2-7
SHORE PROTECTION
EKART 1952

T = 14

$L = \frac{32.2\,(196)}{6.28}\sqrt{\tan 0.188}$

$L = 433$ ft

$k = \frac{2\pi}{T} = 0.0145$

$\sigma = \frac{2\pi}{T} = \frac{6.28}{14}$

cosh = 0.0145 (46+8) = 1.32
k (h+z)

sinh = (0.02065)(46) = 0.7176

for u_v

sinh = 0.8655 k(h+z)

T = 10

$L = \frac{gT^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{T^2}\frac{d}{g}\right)}$

$L = 304$ ft

$k = \frac{2\pi}{L} = \frac{6.28}{304} = 0.02065$

$\sigma = \frac{2\pi}{T} = \frac{6.28}{10}$

cosh 0.02065(46+8) = 1.69

sinh (0.02065)(46) = 1.099

$$u_v = \frac{H}{2}\sigma \frac{\sinh k(h+z)}{\sinh (kh)} \quad (1)$$

sinh = 1.36 k(h+z)
sinh = 1.099 (kh)

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Project/~~Problem Set~~: DIFFUSER DESIGN
Detail/Problem: Sizing Rubble
Class: Prof:
Sheet: of
Calc. by: BW
Date:
Chck. by:
Date:

Minimum size Rubble   MORISON

uh max = 3.863 ft/s   Ah max = 2.426 ft^2/s
uv max = 3.109 ft/s   Av max = 1.952 ft^2/s

Assume Rubble wt = 165 lbs/ft^3

Boyancy = −64
101 lbs/ft^2

$Fd = \frac{1}{2}(1)(1.994)(A)(u^2)$ C_i = 3.29

$F_i = 3.29(1.994)(V)(A_i)$ Cd = 1

For

2.5³ ft Rubble

A = 6.25
V = 15.63
$wt = 15.63(1. \frac{lb}{ft^3}) = 1579$ lbs

$Fd = \frac{1}{2}(1.994)(6.25)(3.863)^2 + \frac{1}{2}(1.994)(6.25)(3.109)^2 = 153$ lbs

$F_i = 3.29(1.994)(15.63)(2.426) + 3.29(1.994)(15.63)(1.952) = 448$ lbs

F = 601 lbs < 2578 lbs

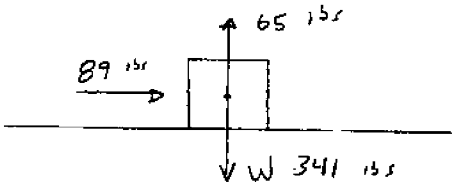
1.5³ ft Rubble

A = 2.25
V = 3.375
wt = 341 lbs

$Fd = \frac{1}{2}(1.994)(2.25)(3.863)^2 + \frac{1}{2}(1.994)(2.25)(3.109)^2 = 55.5$ lbs

$F_i = 3.29(1.994)(3.375)(2.426) + 3.29(1.994)(3.375)(1.552) = 98$ lbs

F = 154 lbs < 341



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Durham, New Hampshire 03824

Project/~~Problem Set~~: DIFFUSER DESIGN
Detail/~~Problem~~: Sizing Anchors
Class: Prof:

Sheet: of
Calc. by: BRW
Date:
Chck. by:
Date:

MORISON $F = F_d + F_i$

Sizing Concrete Anchors for Design 2
6 Anchors

Concrete = 155 lbs/ft³
Boyancy −64
91 lbs/ft³

$u_h = 3.863$ ft/s $A_h = 2.426$ ft²/s
$u_v = 3.109$ ft/s $A_v = 1.952$ ft²/s

$F_d = \frac{1}{2}(1)(1.994)(A)(u^2)$

$F_i = 3.29(1.994)(V)(A_1)$

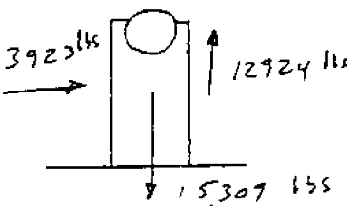
Size
7' High x 1.5' Wide x 3' Deep
Designed For worst Case – Front View with max erosional event

Horiz A = 229 ft² cd = 1 vert A = 2(83.3) = 166.6 ft²
V = 450 ft³ Ci = 3.29
Pipe DIA = 2'
length = 83.3'

$F_d = \frac{1}{2}(1)(1.994)(229)(3.863)^2 + \frac{1}{2}(1)(1.994)(166.6)(3.109)$

$F_d = 3923$ lbs

$F_i = 3.29(1.994)(450)(2.426) + 3.29(1.994)(450)(1.952)$



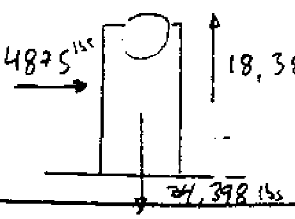
$F_i = 12{,}924$ lbs
16,847 lbs > 15,309 lbs
Force wt of Anchors

NO GOOD

Size 7' High x 3' Wide x 3' Deep
Designed for worst case – Front View max erosional event

Horiz A = 293 ft² cd = 1 Vert A = 166.6 ft²
V = 640 ft³ Ci = 3.29
PIPE DIA = 2'
length = 83.3'

$F_d = \frac{1}{2}(1)(1.994)(293)(3.863)^2 + \frac{1}{2}(1)(1.994)(166.6)(3.109)$
$F_d = 4875$ lbs
$F_i = 3.29(1.994)(640)(2.426) + 3.29(1.994)(640)(1.952)$
$F_i = 18{,}381$ lbs



23,256 lbs < 34,398 lbs
Force wt of Anchors

GOOD

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Project/~~Problem Set~~: DIFFUSER PROJECT
Detail/~~Problem~~: BUOYANCY CALCS
Class: ______ Prof: ______

Sheet: ______ of ______
Calc. by: GW
Date: ______
Chck. by: ______
Date: ______

SOURCE CHEVERON CHEMICAL PLEXCO TECH INFO 10/91

NORMAL CONDITIONS

24" HDPE PIPE ALONE	+ 4.80 lbs/ft	
WASTEWATER	+ 4.91 lbs/ft	
TOTAL	+ 9.71 lbs/ft	BOYANCY IN PIPE + WW EFFLUENT

TOTAL BUOYANCY = 806 lbs

WORST CASE - DIFFUSER FILLED WITH AIR

BORE VOLUME = 2.0472 ft^3/ft

BUOYANCY DUE TO TRAPPED AIR = 2.0472 (64)

= 131 lb/ft

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Project/~~Problem~~ Set: DIFFUSER DESIGN
Detail/~~Problem~~: SPECS ON Polyethylene
Class: ______ Prof: ______
Sheet: ____ of ____
Calc. by: BW
Date: ______
Chck. by: ______
Date: ______

Source - CHEVRON CHEMICAL
PLEXCO Techical INFO
"Recomended Support Spacing" 10/91

MAX Spacing 24" SDR 11 => 16'

ASSUMPTION
wt of pipe
+
H_2O

Polyethylene

- Good strength
- Light weight
- Is Already in Place
- Good Hydraulics!
- Does Not support Biological Growth
- Non Contaminating to environment.

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Project/Problem Set: Modeling
Detail/Problem: Froude / Dilution
Class: ______ Prof: ______

Sheet: ___ of ___
Calc. by: RJF
Date: ______
Chck. by: ______
Date: ______

Froude Modeling: (no distortion)

estimated scale = 1:10

prototype ⇒ $Q_{port} = Q_p = \frac{1250\text{ gpm}}{20\text{ ports}} = 62.5$ gpm   for 4 ports $Q_p = 250$ gpm

model ⇒ $Q_m = Q_p \left(\frac{L_m}{L_p}\right)^{5/2} = 250\text{gpm}\left(\frac{1}{10}\right)^{5/2}$

L_m = length of model
L_p = length of prototype

$Q_m = 0.791$ gpm

geometric scale (no exageration)

port diameter (prototype) = 2 in

@ 1/10 scale ⇒ port diameter (model) = 0.2 in

Reynolds #: for 1 port

$$V_m = V_p\left(\frac{1}{16}\right)^{1/2} = 6.37\,ft/s\left(\frac{1}{16}\right)^{1/2} = 2.01\,ft/s$$

$$R_e = \frac{VD}{\nu} \quad @\,80°F \quad \nu = 0.930\times10^{-5}\,ft^2/s \quad \text{(Roberson/Crowe)}$$

$$R_e = \frac{2.01\,ft/s\,\left(0.206\,in\left(\frac{1\,ft}{12\,in}\right)\right)}{0.930\times10^{-5}\,ft^2/s} = 3717 \quad > 2500$$

Fluorometer calcs:

example → Test 1 port 1
(fluorometer ⇒ X=1)

to get conc:

$$\frac{\text{measure scale}}{\text{sensitivity scale}} \times 100 = \text{conc (ppb)}$$

i.e. $\text{con} = \frac{6.2}{31.6} \times 100 = 19.6\ ppb$

avg concentrations taken for each port.

$$C_d = C_a + \frac{(C_e - C_a)}{S_a} \qquad S_a = 530$$

for Test 1 $C_a = 35.45$ ppb $C_e = 541$ ppb

$C_d = 36.40$

for Test 2 $C_a = 70.68$ $C_e = 541$ $C_d = 71.57$

for Test 3 $C_a = 101.40$ $C_e = 541$ $C_d = 102.23$

for Test 4 $C_a = 127.37$ $C_e = 541$ $C_d = 128.15$

	TEST 1							
PORT 1								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	19.62	30.50	40.00	47.00	137.12			
2	19.30	30.50	39.00	46.00	134.80			
3	21.50	35.00	45.00	52.00	153.50	CONC (ppb)		
					425.42	35.45		
PORT 2								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	69.62	77.53	64.87	99.68	311.70			
2	67.50	77.00	65.50	96.00	306.00			ACTUAL
3	78.00	87.50	75.50	110.76	351.76	CONC (ppb)		CONC.
					969.46	80.79		45.34
PORT 3								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	38.00	39.00	60.00	72.78	209.78			
2	38.00	39.00	58.00	73.50	208.50			
3	44.00	44.50	69.00	81.40	238.90	CONC (ppb)		
					657.18	54.77		19.31
PORT 3								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	33.00	38.00	64.00	63.00	198.00			
2	33.00	37.00	62.00	62.50	194.50			
3	37.00	44.00	71.00	71.00	223.00	CONC (ppb)		
					615.50	51.29		15.84

Cd = 36.45

	TEST 2							
PORT 1								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	62.00	69.62	77.53	85.44	294.59			
2	51.00	59.00	65.50	71.50	247.00			
3	66.00	72.78	80.70	87.03	306.51	CONC (ppb)		
					848.10	70.68		
PORT 2								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	101.27	121.84	88.61	132.91	444.63			
2	85.00	97.00	73.00	107.59	362.59			ACTUAL
3	110.76	129.75	91.77	140.82	473.10	CONC (ppb)		CONC.
					1280.32	106.69		36.02
PORT 3								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	72.78	82.28	94.94	113.92	363.92			
2	62.00	69.00	80.00	91.77	302.77			
3	79.11	87.03	98.10	118.67	382.91	CONC (ppb)		
					1049.60	87.47		16.79
PORT 4								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	79.11	91.77	98.10	94.94	363.92			
2	66.00	76.00	84.00	75.95	301.95			
3	82.28	94.94	104.43	98.10	379.75	CONC (ppb)		
					1045.62	87.14		16.46

46

C = 71.56

	TEST 3							
PORT 1								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	94.94	106.01	110.76	117.09	428.80			
2	79.11	85.44	91.77	94.94	351.26			
3	98.10	107.59	110.76	120.25	436.70	CONC (ppb)		
					1216.76	101.40		
PORT 2								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	148.73	177.22	164.56	148.73	639.24			
2	120.25	145.57	132.91	123.42	522.15			ACTUAL
3	148.73	180.38	167.72	151.90	648.73	CONC (ppb)		CONC.
					1810.12	150.84		49.45
PORT 3								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	101.27	145.57	142.41	158.23	547.48			
2	82.28	117.09	110.76	126.58	436.71			
3	101.27	145.57	140.82	159.81	547.47	CONC (ppb)		
					1531.66	127.64		26.24
PORT 4								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	117.09	129.75	132.91	151.90	531.65			
2	94.94	104.43	104.43	126.58	430.38			
3	117.09	131.33	134.49	155.06	537.97	CONC (ppb)		
					1500.00	125.00		23.60

$C_d = 102.23$ ppb

	TEST 4							
PORT 1								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	126.58	139.24	142.41	137.66	545.89			
2	102.85	112.34	113.92	107.59	436.70			
3	126.58	137.66	143.99	137.66	545.89	CONC (ppb)		
					1528.48	127.37		
PORT 2								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	174.05	205.70	202.53	185.13	767.41			
2	142.41	169.30	161.39	150.32	623.42			ACTUAL
3	175.63	210.44	204.11	183.54	773.72	CONC (ppb)		CONC.
					2164.55	180.38		53.01
PORT 3								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	170.89	180.38	170.89	180.38	702.54			
2	136.08	145.57	136.08	145.57	563.30			
3	167.72	181.96	167.72	180.38	697.78	CONC (ppb)		
					1963.62	163.64		36.26
PORT 4								
TEST PD	CONC-1	CONC-2	CONC-3	CONC-4	SUM			
1	156.65	158.23	161.39	158.23	634.50			
2	125.00	129.75	129.75	123.42	507.92			
3	158.23	161.39	161.39	158.23	639.24	CONC (ppb)		
					1781.66	148.47		21.10

$C_d = 128.15$

Bibliography and Usage

Boston's Sewage Outfall, Civil Engineering, April 1991, David Palmer

- Hydraulic considerations from plant to diffuser
- Choosing location of diffuser
- Design of diffuser

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