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## SOIL INSTRUMENTATION FOR THE I-95 MIT-MDPW TEST EMBANKMENT

By

L. Anthony Wolfskill<sup>1</sup> and Cetin Soydemir<sup>2</sup>

### Abstract

This paper describes an instrumentation scheme adequate to document the stress-deformation history of a heavily loaded, soft foundation. The instrumentation includes hydraulic piezometers, vibrating wire piezometers, settlement plates and anchors, inclinometers, and vibrating wire total stress cells. The instrumentation is described, installation procedures outlined, and instrument performance evaluated.

### I. Introduction

#### *A. History of Research Project*

In June 1965, the Soil Mechanics Division of the Department of Civil Engineering, M.I.T., began an in-depth research study for the Massachusetts Department of Public Works on stability and deformation of heavy embankments on thick deposits of soft soils. The primary objective for this research was to obtain critically needed fundamental information on the reliability of techniques of predicting stability and deformation of such embankments. The strategy for accomplishing this objective was to instrument an actual embankment, predict its behavior, document the measured behavior, compare reality with prediction, and then evaluate methods of prediction. This paper is limited to the description and evaluation of the instrumentation.

The first two and one-half years of the research work involved preconstruction activities. First, a field test site was chosen, being Station 246 of an embankment for Interstate 95 to be built across a soft, compressible tidal meadow in Saugus, Massachusetts. The construction plan for the embankment

<sup>1</sup>Director of Field Engineering Research, Dept. of Civ. Engrg., MIT, Cambridge, Mass.

<sup>2</sup>Associate Prof., Dept. of Civ. Engrg., Orta Dogu Teknik Univ., Ankara, Turkey.

envisioned these operations: excavation of thin surface peat, placement of a granular backfill below water, compaction of granular embankment to about Elev. +18 ft, placement of a temporary overload to about Elev. +40 ft, removal of the overload after a suitable period of time, and then construction of the pavement.

Following selection of the test site, undisturbed samples of high quality were taken from a deep boring at the test site. Laboratory tests were performed to support preliminary predictions of foundation performance. Concurrently, the researchers developed specifications for field instrumentation to be placed at the test site. The instrumentation design provided for the following:

1. Observation wells and piezometers to measure the pore water pressures under and adjacent to the embankment;
2. Settlement rods and platforms to measure vertical deformations;
3. Inclinometers to measure lateral and vertical deformations;
4. Total stress cells to measure vertical embankment loading.

All of these instruments were placed after the peat had been removed and granular backfill placed to Elev. +9 ft. To establish initial conditions in the foundation soils, a few piezometers and settlement rods were installed at nearby Station 245 before any construction activities.

Early construction work began in December 1967 at the test section. Readings were taken at Station 245 instrumentation before and during the installation of the Station 246 instrumentation (February to July 1968). All Station 246 instrumentation was in operation by early July 1968. Readings of the Station 245 instruments were discontinued after a few weeks overlap with the main instruments. Readings were taken daily on most instruments during the embankment construction period. The embankment was completed to Elev. +36 ft in November 1968. Filling then stopped and was resumed in April 1969 to Elev. +40 ft by May 1969.

### *B. Objectives of this Paper*

This paper has two primary objectives:

1. Description of a field instrumentation scheme adequate to document the stress-deformation history of a heavily loaded, soft foundation;
2. Evaluation of the performance of the various instruments used.

There are several unusual features of the instrumentation that need evaluation, such as the instrumentation tunnel, the total stress cells buried in the embankment, the piezometers off center line and beyond the toe of the embankment, and the use of electric vibrating wire instruments. Actual field data are portrayed to illustrate the performance of the instruments. A separate publication will use the field data to evaluate methods of design and analysis.

## II. Field Instrumentation Scheme

### A. Soil Conditions

The site for the I-95 embankment crosses a tidal meadow consisting of a thick deposit of soft grey clay known locally as the Boston blue clay. It is a silty clay of medium plasticity with numerous sand and silt lenses. The plan location of the project is shown in Fig. 1; the soil profile in Fig. 2. At the M.I.T. test section the peat is about 6 ft thick, the sand 10 ft thick, and the clay 130 ft thick. The clay is overconsolidated at the top due to sea level lowering and becomes normally consolidated at about Elev. -80 ft. The undrained strength near the top is about 1.0 tsf, ranging to a low of about 0.65 tsf near the middle. The average compression index for the clay profile is about 0.2. Under the heavy embankment, stability of the foundation was expected to be a serious problem, and settlements on the order of several feet were predicted.

### B. Instrumentation Plan

Figure 3 shows the instrumentation as installed at the Preconstruction Test Section (Station 245), and Fig. 4 at the M.I.T. test section (Station 246). Detailed information on the installation of individual instruments were documented in a recent report (MIT 1969). Table 1 summarizes all the instrumentation.

Several features of the M.I.T. test section stand relatively unique:

#### 1. Instrumentation Tunnel

Rather than carrying the instrument leads to the surface of the embankment concurrently with construction progress, the instrumentation located under the embankment terminates in a 5-ft diameter, 110-ft long flexible steel plate tunnel in order to minimize interference with construction activities and damage to instruments. The tunnel was placed initially at a slope (i.e., Elev. +8 ft at the center line and Elev. +6 ft at the toe) in order to offset expected differential settlement. The leads of piezometers and stress cells were brought into the tunnel through flexible protective hosing. The tunnel sits along a line where all the settlement platforms, settlement rods, inclinometers, observation wells and the bench mark were installed. All instruments can move independent of the tunnel. The tunnel is illustrated in Fig. 5. In Fig 5a the left end of the tunnel rests at about the center line of the embankment. Figures 5b and 5c show the details of connecting the piezometers to the tunnel.

#### 2. Total Stress Cells

Three clusters of stress cells, each cluster consisting of three cells, were installed at Station 246 +33 at the center line, and at 30 ft and 60 ft offset from the center line. The decision for using stress cells was made after the filling had

Table I  
FIELD INSTRUMENTATION FOR I-95  
MIT TEST EMBANKMENT

TYPE OF INSTRUMENT	NUMBER OF INSTRUMENTS	
	STATION 245	STATION 246
Double tube hydraulic piezometer	6	33
Vibrating Wire electric piezometer	0	6
Observation well	0	5
Settlement rod	6	12
Settlement platform	0	5
Inclinometer	0	6
Benchmark		1
Total stress cell	0	9
Total drilling, ft		
6"	0	957
4"	0	1001
3"	677	3089
Total cost		
Instrumentation	\$4000	\$85,000
Tunnel	0	\$17,000

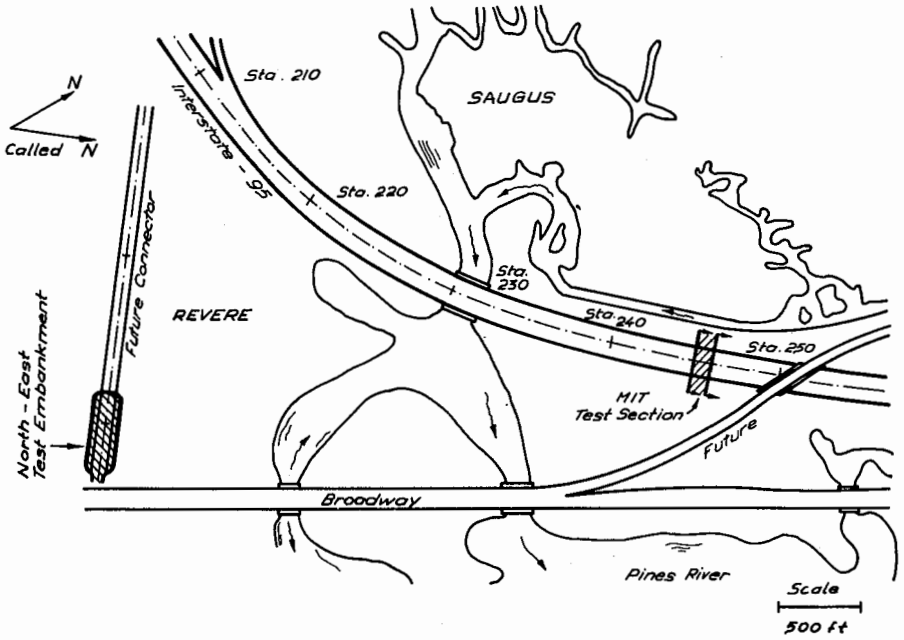


Figure 1. Location of the Test Embankments

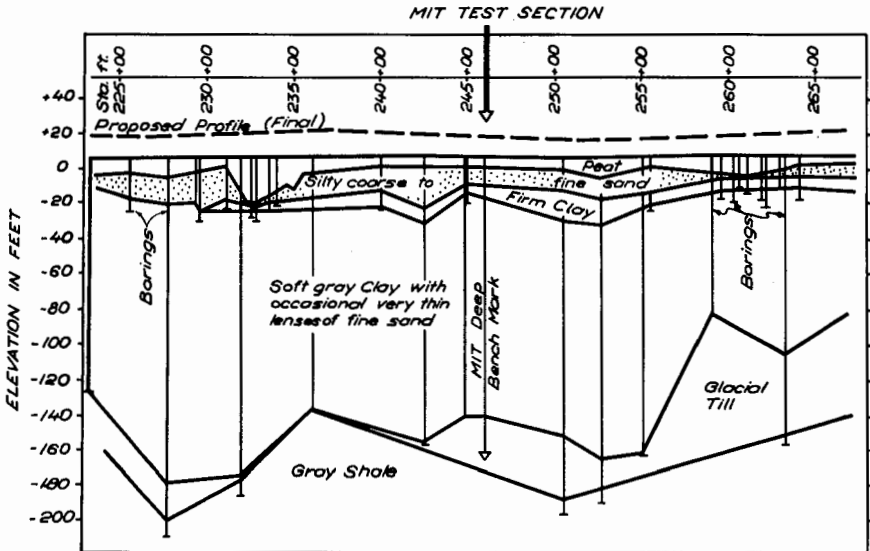


Figure 2. Soil Profile Along I-95

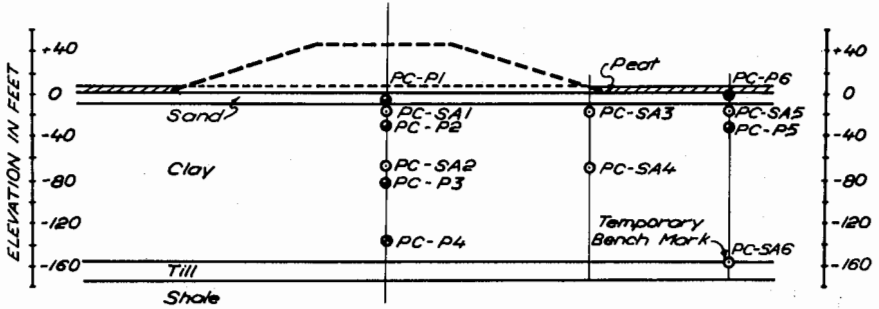


Figure 3. Preconstruction Instrumentation, Station 245

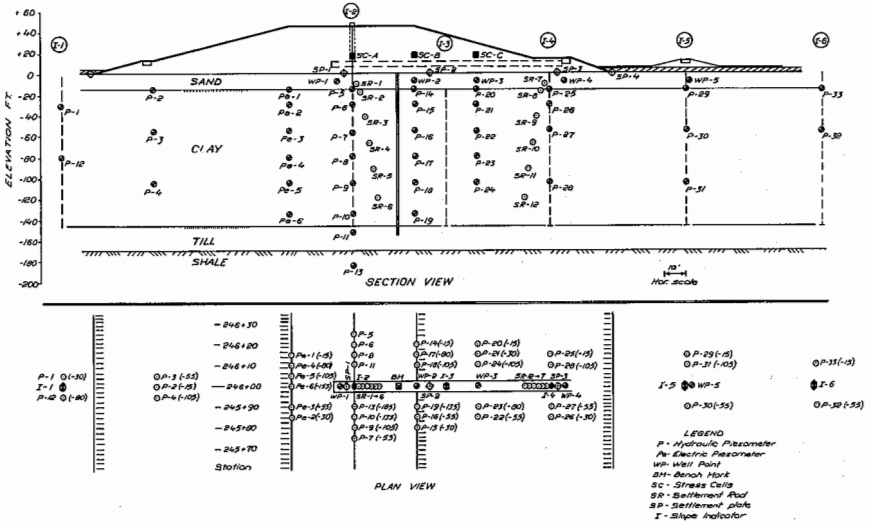
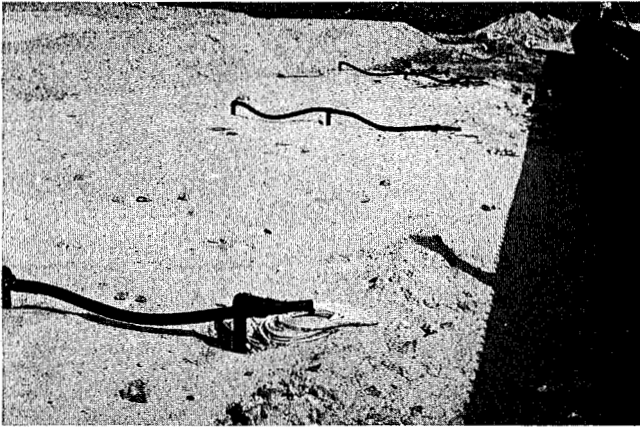


Figure 4. MIT-MDPW Test Section Instrumentation, Station 246

A



B



C

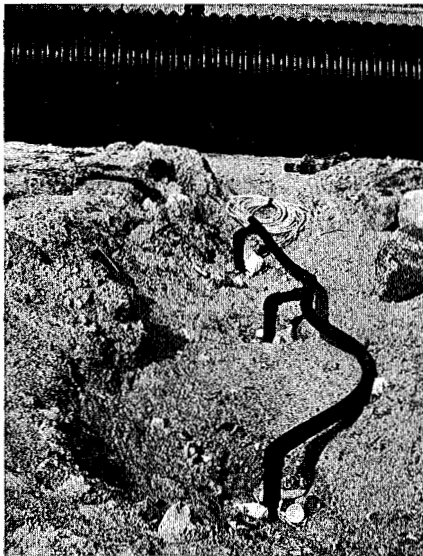


Figure 5. Instrumentation Tunnel

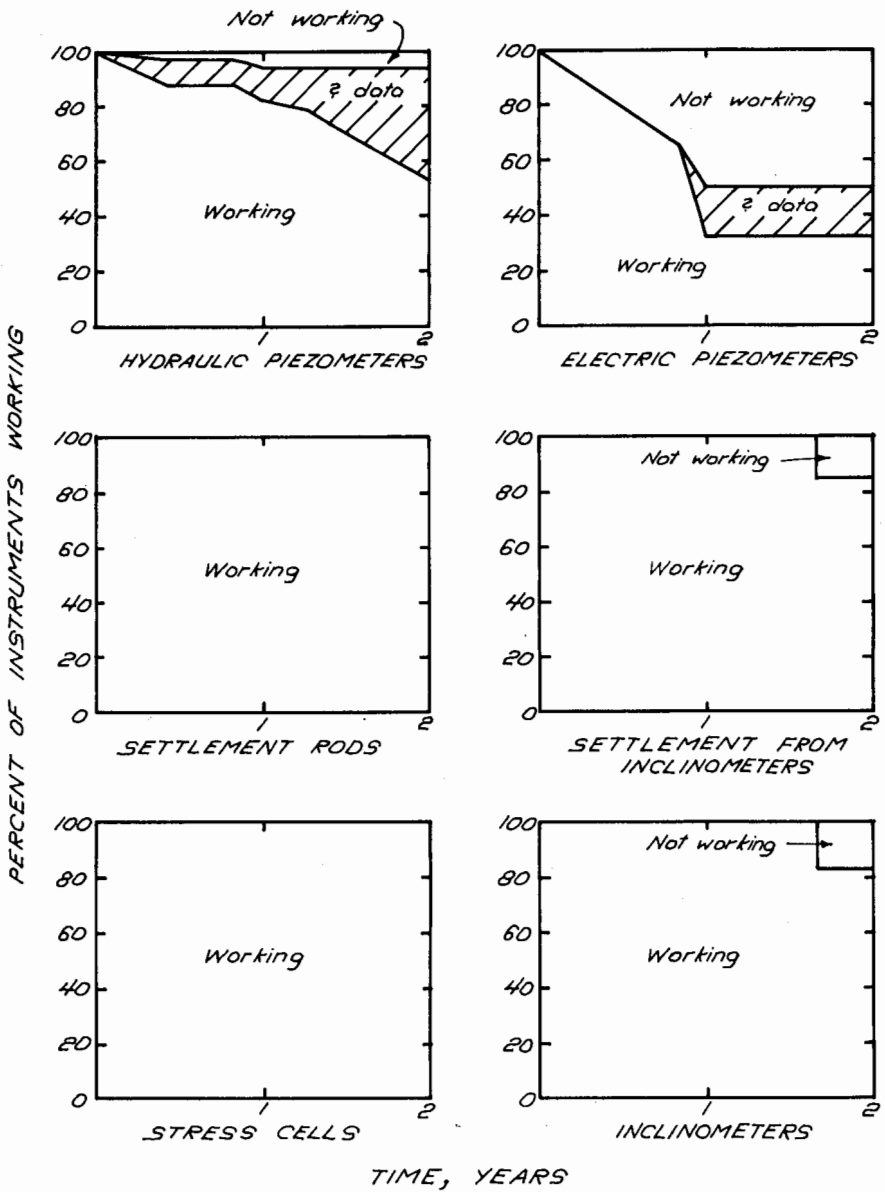


Figure 6. Two Year Performance Record of Instruments

already started, thus they were installed within the embankment, approximately at Elev. +17 ft. The stress cells measure total vertical stresses.

### 3. Off-center Line Piezometers

Besides a thoroughly instrumented center line profile, the test section consists of lines of piezometers at 30, 60, 95, 160, and 225 ft offset from the center line. The intention has been to observe the pore pressure response in the full lateral extent, since this information is essential both for stability and settlement analyses.

The data collected from off-center piezometers contributed significantly to the decision not to build the side berms which were included as options in the original design.

### C. Longevity of Instruments

Figure 6 indicates the percentage of instruments working during the first two years of operation. Three categories of evaluation are shown:

1. Working
2. Working, but data questioned
3. Not working.

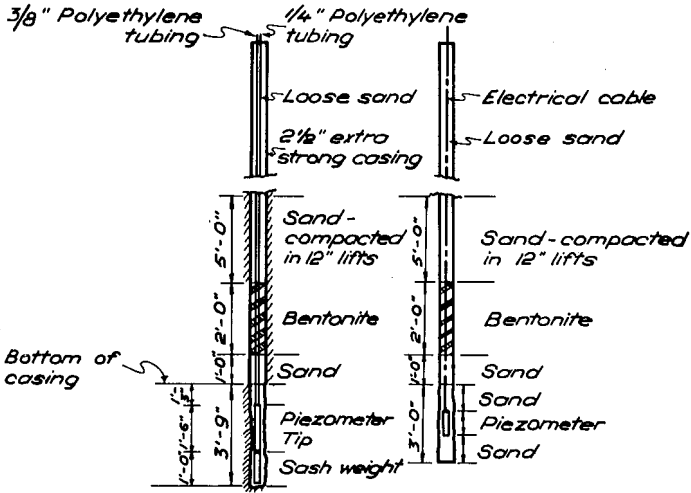
The stress cells and settlement rods performed without any instrument failure. Piezometers, both hydraulic and electric vibrating wire types, lost about 50 percent by the end of the two-year period. Although this loss is high, it is not unusual for similar installations where large settlements occurred.

## Details of Instruments

### A. Piezometers

**Station 245.** Figure 3 shows the locations of the five hydraulic piezometers, designated PC-P1 through PC-P5, installed at the Preconstruction Section. The piezometers were Geonor A/S, M-206 field piezometers. The piezometer consists of a hollow metallic stem with a conical point at the bottom; three sintered bronze filters are mounted on the stem and are separated by chamfered brass rings. The top of the hollow stem was attached to a drilling rod (E-rod) connector and sections of E-rod were attached directly to the sensor. Two 3/16-inch plastic tubes were used as riser pipes. The installation procedure is described in Appendix B.

**Station 246.** Figure 4 shows the location of the thirty-three hydraulic and six electrical piezometers installed at the MIT-MDPW Test Section. The piezometers under the embankment terminate at the instrument tunnel, whereas the ones beyond the toe are grouped in four manholes.



HYDRAULIC VIBRATING WIRE TYPE  
(0 kg/cm<sup>2</sup> to 10 kg/cm<sup>2</sup>)

Figure 7. Details of Hydraulic and Vibrating Wire Piezometers

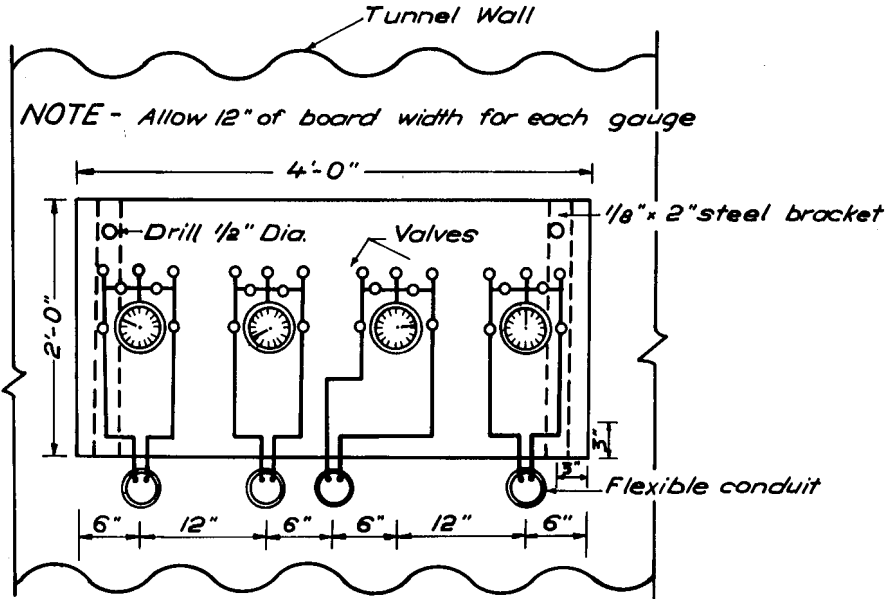


Figure 8. Gage Boards in Instrument Tunnel

The hydraulic piezometers were manufactured by Geomeasurements, Inc., and consist of an 18-in. long porous plastic ("Vycron") sensor with two plastic riser pipes, a 3/8-in. reading lead and a 1/4 in. flushing lead. A sash weight was tied to the tip to aid installation. Details of the installed peizometer are shown in Fig. 7. All leads were brought into the instrument tunnel or a manhole by attaching an elbow to the top of the steel casing and running flexible hose from the elbow to the tunnel or manhole. The leads are connected to a 4½-in. diameter weatherproof combination gauge with a range from -30 in. mercury to +60 psi. A typical arrangement of gauges is shown in Fig. 8. The installation procedure is described in Appendix B.

The electrical piezometers were Geonor A/S vibrating wire piezometers with a range of 0-5 kg/cm<sup>2</sup> or 0-10 kg/cm<sup>2</sup> depending on depth of installation. The piezometer output is the frequency of vibration of a wire stretched from a fixed support to a sensing diaphragm. The frequency is read directly by a digital frequency meter. The piezometers were calibrated in the laboratory for hydrostatic pressure and temperature. A final calibration was made during installation by recording the frequency every five feet as the instrument was lowered into the cased hole full of clear water. The installation procedure is described in Appendix B.

**Observed Data.** The piezometer data have been summarized by plotting water level elevation or total head. This is simply the elevation to which water would rise in an open standpipe whose bottom is at the same elevation and location as the corresponding piezometer. Data from shallow observation wells are also summarized and, therefore, excess pore pressures (in ft of water) may be computed as the difference of the piezometer and well readings.

Figure 9 summarizes the piezometer data at the Preconstruction Section, Station 245. Almost no data could be collected from mid-February to mid-April 1968 due to freezing weather conditions. Later experience showed an antifreeze mixture (4 parts by volume methanol, 2 parts glycerol, 3 parts water) to be effective under sustained cold spells with temperatures of -10°F to -20°F. During the second half of July 1969, the Preconstruction Section piezometers were monitored concurrently with the MIT-MDPW Test Section peizometers at Station 246. After a consistent behavior was observed between the two sections, further readings of the Preconstruction piezometers were terminated and they were buried under the rising embankment.

Figures 10 through 14 summarize the piezometer data at the MIT-MDPW Test section through June 1969. All the piezometers were in operation in mid-July 1968, with the embankment at Elev. +10 ft. Continuous construction brought the embankment fill to Elev. +36 ft on 22 November 1968. The piezometers were monitored daily (except weekends) during construction. Construction stopped until mid-April 1969; the piezometers were read twice a week during this period. Construction resumed in mid-April and the embankment was brought to its final elevation, Elev. +40 ft, by mid-May 1969. During this last stage the piezometers were monitored daily.

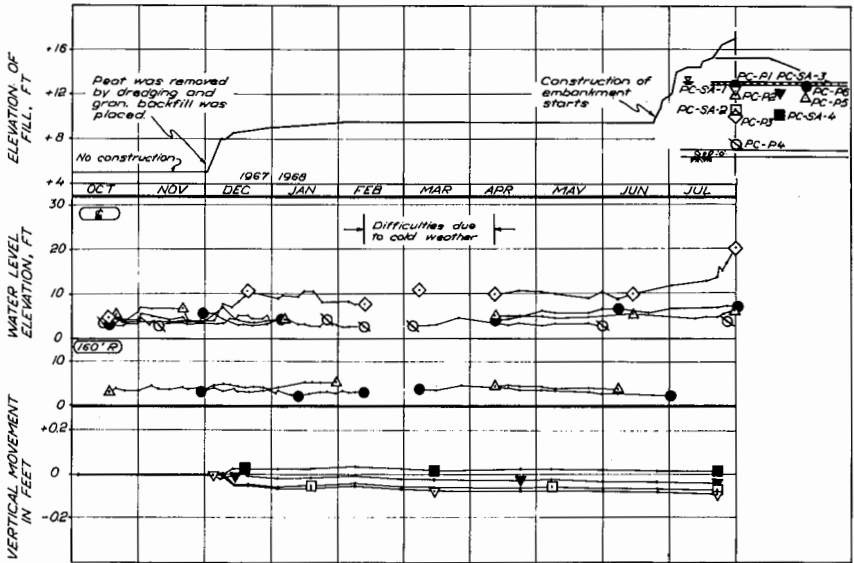


Figure 9. Preconstruction Instrumentation Data

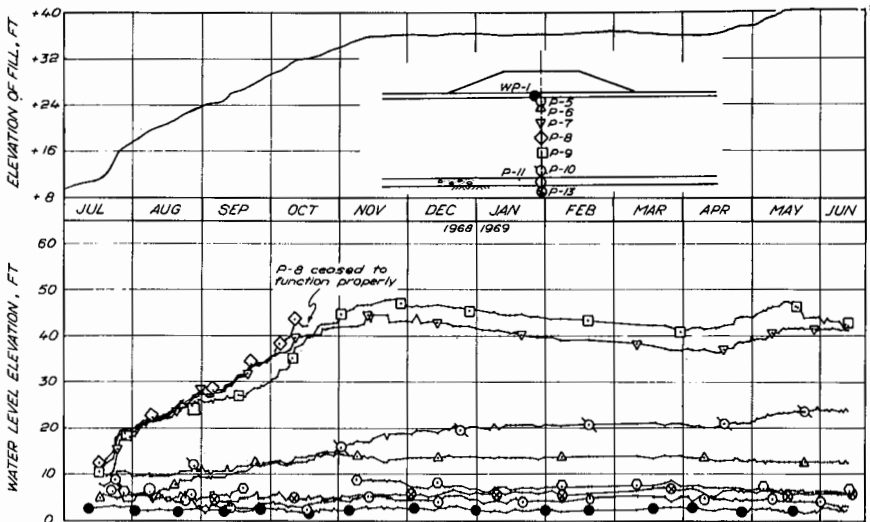


Figure 10. Pore Pressures Along the Center Line

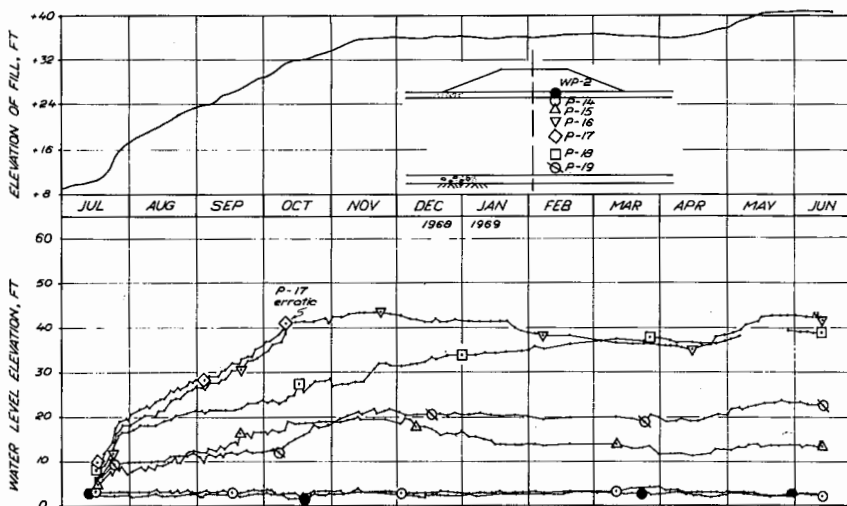


Figure 11. Pore Pressures 30 Feet Right of Center Line, Hydraulic Piezometers

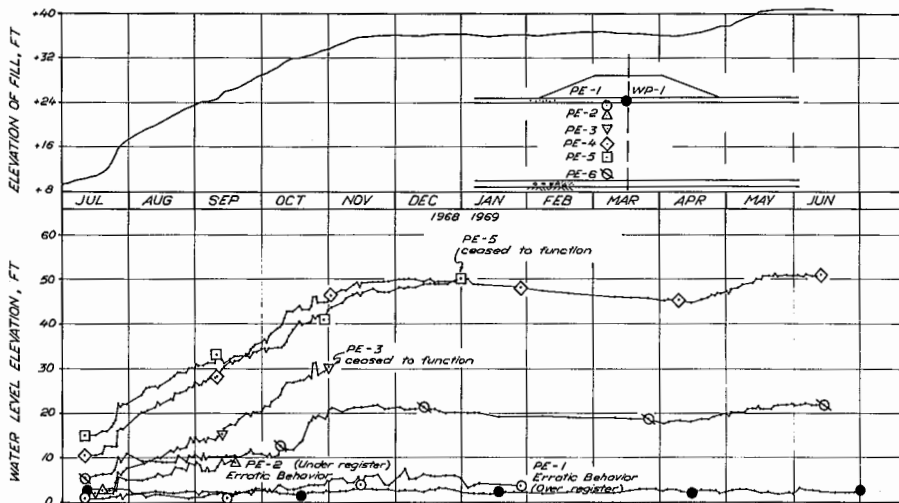


Figure 12. Pore Pressures 30 Feet Left of Center Line, Electric Vibrating Wire Piezometers

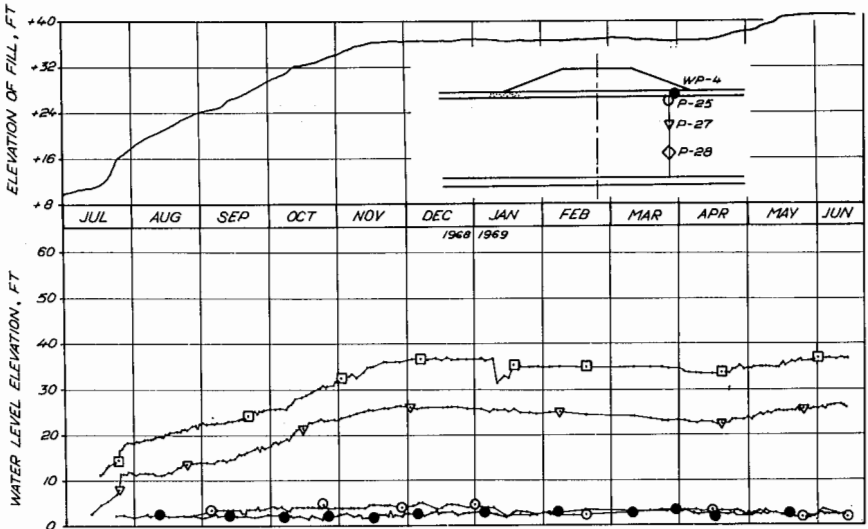


Figure 13. Pore Pressures 96 Feet Right of Center Line

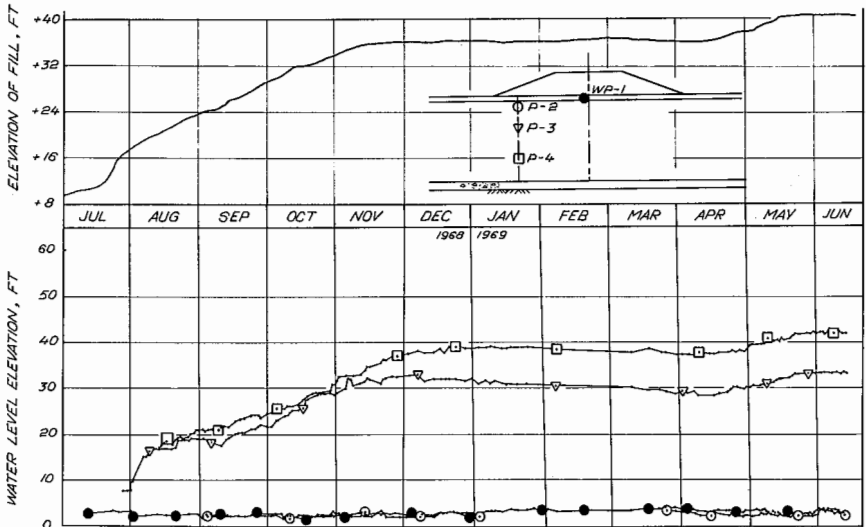


Figure 14. Pore Pressures 95 Feet Left of Center Line



**Evaluation of Piezometers.** Two-year performance records for double tube hydraulic piezometers and vibrating wire electrical piezometers are summarized in Fig. 6. The detailed status of piezometers at the end of this two-year period, July 1970, is summarized in Table 2. These data clearly show that a high percentage of all piezometers fail within two years of installation. Particularly alarming is the width of the crosshatched band of questionable data.

The hydraulic piezometers have performed for a longer period than the vibrating wire piezometers. However, during the embankment construction period, the electrical piezometer data have been compatible with data from symmetrical hydraulic piezometers.

One main cause of piezometer failure has been the differential settlement of the casing and the piezometer sensor. Due to large casing settlement the plastic tubing just above the sensor is cut or pinched off. This was observed visually when one of the piezometers monitored by the consulting engineers was pulled out of the ground for inspection. The sudden loss in pressure experienced by several piezometers supports this explanation. Several additional causes of failure may be cited for electrical piezometers: shift in zero frequency, leakage through O-rings at the sensor, and air at the sensor.

Piezometers P-6, P-8, and P-10, located at the center line of the embankment, were replaced in December 1970 to ensure that reliable pore pressure data are obtained during consolidation and subsequent removal of overload.

### *B. Settlement Rods, Anchors and Platforms*

**Station 245.** Figure 3 shows the location of five settlement anchors and one temporary bench mark, designated PC-SA1 through PC-SA6, which were installed at the Preconstruction Section. These instruments measured the early settlement and heave prior to the installation of the Station 246 instrumentation.

The settlement anchors were manufactured by Borros A/B. The anchor consists of two parts: the point and three anchor rods. The point is a 1¼-in. hollow steel cylinder with three circular milled grooves. A short section of 1 in. steel pipe is connected to the point and additional sections added as necessary. The anchor rods are approximately 3/8 in. in diameter and are welded together at their upper ends and connected to a round plate. A short section of ¼-in. steel pipe is then connected to the plate and additional sections added as necessary. The installation procedure is described in Appendix A.

**Station 246.** Twelve settlement rods, five settlement platforms, and one permanent bench mark were installed at Station 246 (see Fig. 4). All of the settlement rods and three of the settlement platforms terminate at the instrument tunnel. Level surveys are made from the permanent bench mark which is in the tunnel.

The settlement rod (see Fig. 15) is a 1-in. standard pipe with a hollow disk welded 3.0 ft from the bottom. The disk is  $1\frac{1}{2}$  in. in diameter and  $\frac{1}{2}$  in. thick. The settlement platform (see Fig. 16) is a 4.0-ft square,  $\frac{1}{2}$  in. thick steel plate. A  $1\frac{1}{2}$ -in. steel pipe is welded to the steel plate and extended through the fill as necessary. The installation procedures are described in Appendix B.

**Observed Data.** Settlement-heave data at the Preconstruction Section are shown in Fig. 9. Figure 17 summarizes the settlement data at the MIT-MDPW Test Section through June 1969. Data for SP-4 and SP-5 are now shown due to their lack of precision. These two platforms are outside the tunnel and had to be surveyed using a distant bench mark.

**Evaluation of Settlement Rods, Anchors, and Platforms.** All of the settlement rods, anchors, and platforms performed satisfactorily during the period of embankment construction.

### C. *Inclinometers*

**Station 246.** Figure 4 shows the locations of the six inclinometers installed at the MIT-MDPW Test Section. These are dual purpose instruments since they give data on vertical movement as well as horizontal movement within the foundation.

The inclinometers were manufactured by Slope Indicator Co., Seattle, Washington. The inclinometer, shown in Fig. 18, consists of a 3-in. inside diameter aluminum casing having four continuous longitudinal grooves. The casing is assembled from five-foot sections connected by telescoping couplings which allow 6 in. of vertical movement at each coupling. The casing is installed in a 6-in. open borehole which extends through the clay and 5 ft into glacial till. The longitudinal grooves are aligned parallel and perpendicular to the axis of the embankment and the annular space backfilled with sand or peastone.

Lateral movements were determined by a special electronic torpedo which is lowered into the casing and rides in the longitudinal grooves. The sensing element of the torpedo is essentially a pendulum-actuated Wheatstone bridge circuit. A reading is made at three points within each five-foot section of casing. The slope of the casing at each point is calculated, and integrating from the fixed bottom yields a plot of lateral movement versus depth.

Vertical movements were determined by a special settlement torpedo which is lowered down the casing with a steel tape. Spring loaded arms catch the bottom of each five foot section and the depth to the bottom of each section is recorded. The elevation of the top of the inclinometer casing is then used to convert the depth readings to elevations.

**Observed Data.** Periodic observations of the inclinometers were made. The data for horizontal displacements in the plane of the embankment are summarized in Fig. 19 as a function of embankment height and time. The horizontal displacements along the axis of the embankment (north-south direction) were negligibly small, i.e., less than 1 inch.

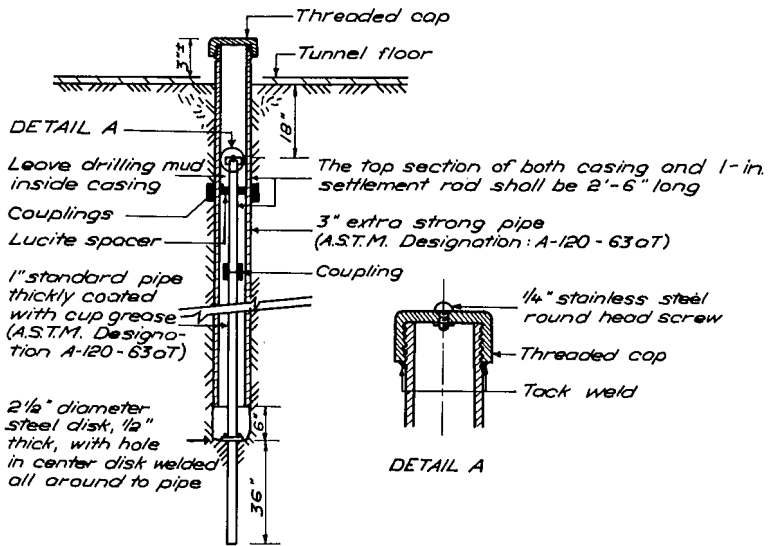


Figure 15. Details of the Settlement Rod

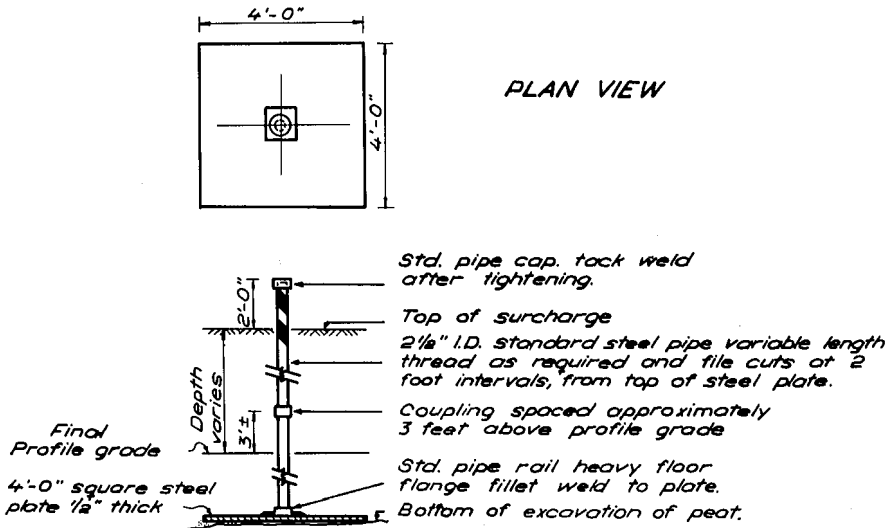


Figure 16. Details of the Settlement Platform

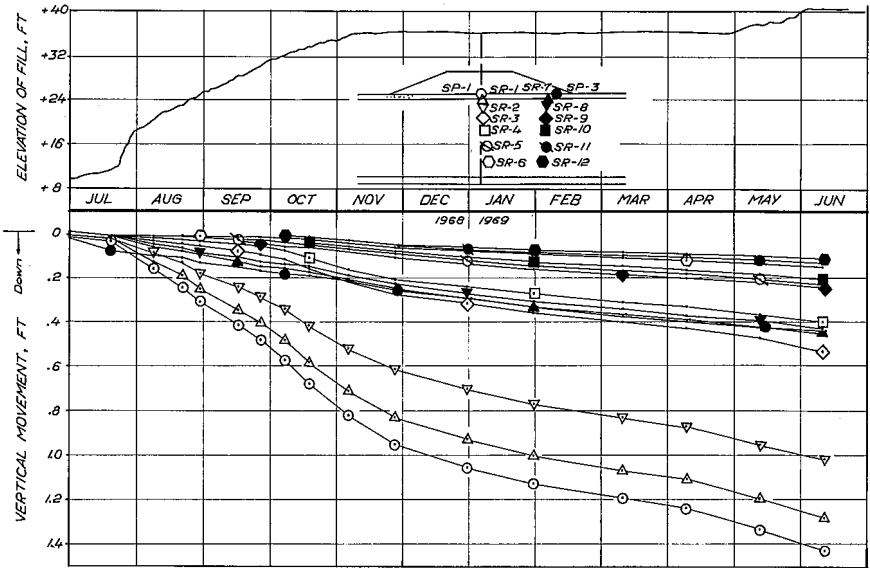


Figure 17. Settlement Data at the MIT Test Section

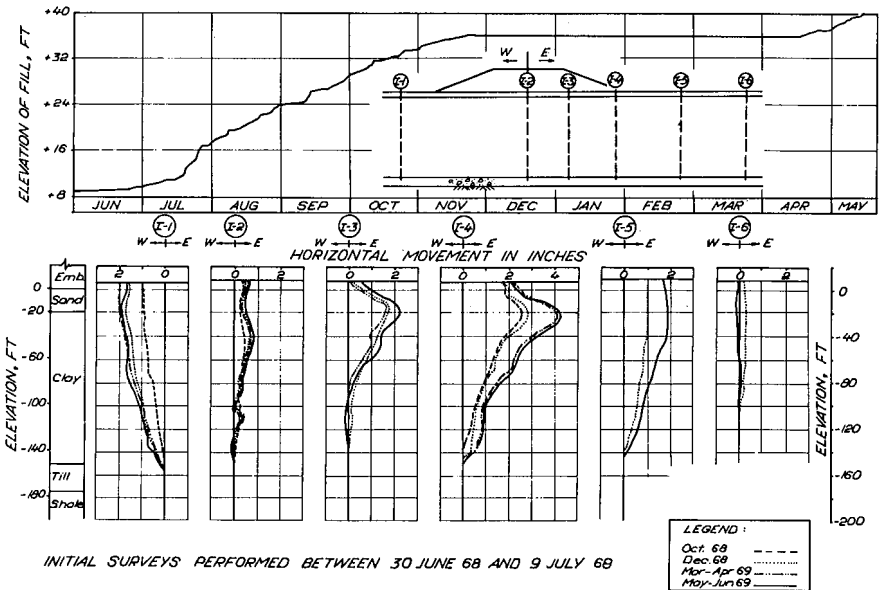


Figure 19. Horizontal Movement at the MIT-MDPW Test Section

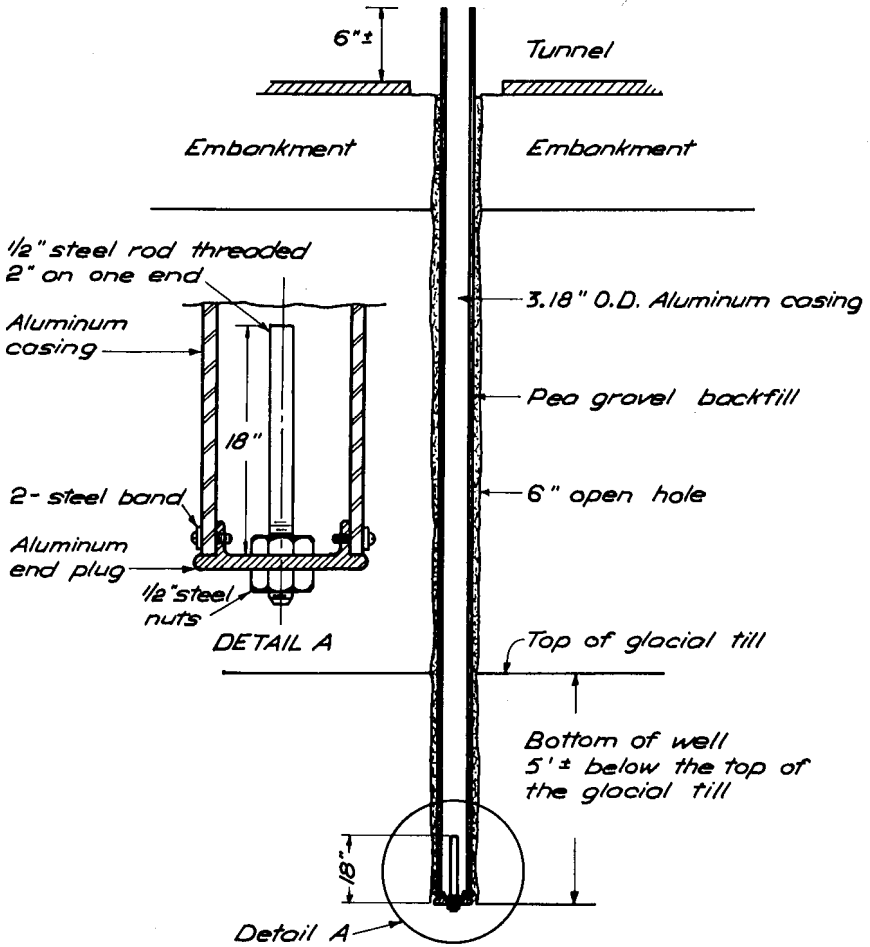


Figure 18. Details of the Inclinometer

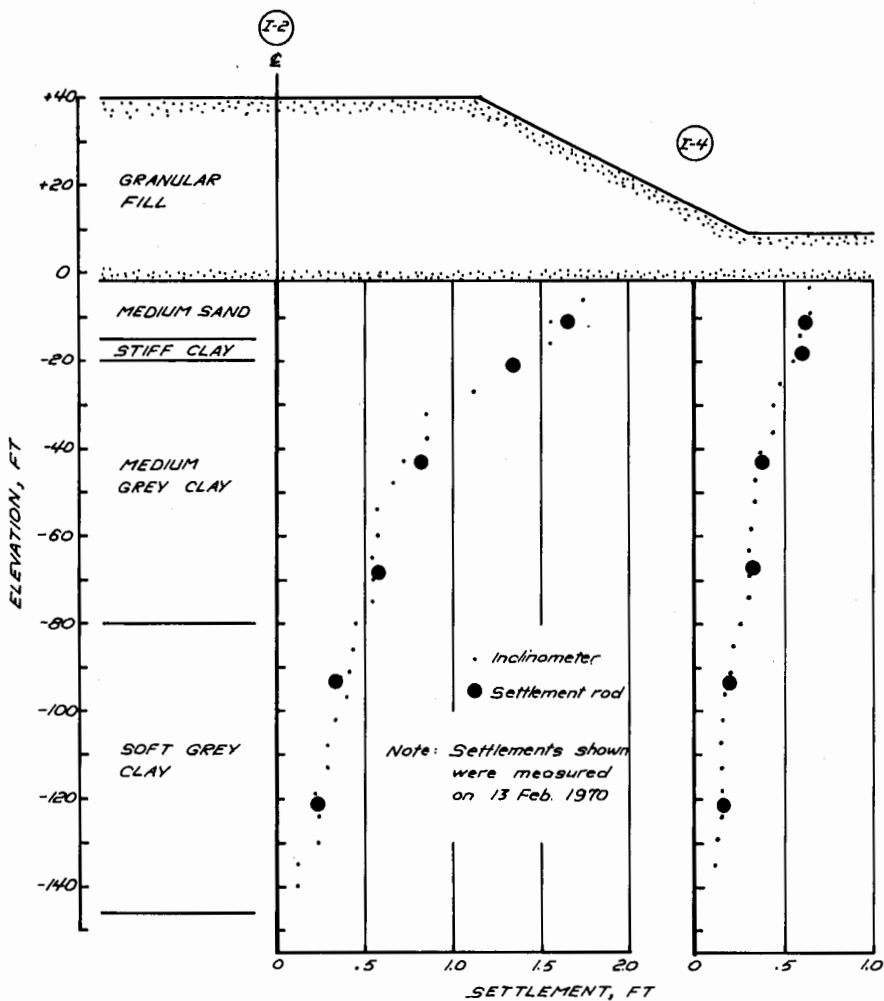


Figure 20. Comparison of Settlement Data From Inclinometers and Settlement Rods

Settlement data obtained in February 1970 from inclinometers I-2 and I-4 are compared with the settlement profile from the settlement rods in Fig. 20. The agreement is excellent. Figures 21 and 22 compare the settlement rod data to the corresponding settlement data obtained from vertical inclinometer surveys. Again the agreement is good.

**Evaluation of Inclinometers.** There have been no technical problems with the inclinometers. The data on lateral movement appear reasonable, and the vertical movement determined from coupling surveys is in excellent agreement with the settlement rod measurements. One inclinometer, I-6, was lost when vandals filled it with stones.

The lateral movement data appear reasonable but the following factors are thought to affect the precision of the measurements:

1. It is most likely that a 150-ft long inclinometer will not be installed exactly vertical. For the six inclinometers installed, the eccentricity from vertical varied from 4 to 38 in. Due to this initial imperfection in orientation, an inclinometer may not be able to reflect the imposed soil displacements with high precision.
2. During the installation a number of couplings may collapse and impose a condition where certain segments of the inclinometer are relatively less flexible for bending and cannot respond to vertical displacements.
3. In the survey of an inclinometer of 150 ft in length, approximately ninety readings are made. The position of the inclinometer is computed by integration of recorded slopes from the bottom (i.e., fixed point) to the top. Thus a very small bias in measuring system, which is hard to detect by the field technician, may accumulate to significant errors.

It is usual procedure to verify the horizontal location of the top of the casing by an independent optical survey. However, the location of the inclinometers inside the tunnel precluded this independent check. All inclinometer data reported here are based on the assumption that the bottom of the casing was fixed.

#### *D. Total Stress Cells*

**Station 246.** A principal feature of the instrumentation at the MIT-MDPW Test Section is the total stress cells. The objective is to measure the distribution of vertical total stress imposed by the embankment on the foundation soil. The common assumption in practice is that the vertical stresses at the embankment foundation interface are distributed in direct proportion to the height of the embankment. The total stress measurements enable one to check the validity of this assumption and evaluate recent theoretical work done in this area.

**Mechanical Features of Total Stress Cells.** The total stress cells were manufactured by Geonor A/S of Norway and commercially available under the brand name of P-100 Earth Pressure Cells. Figure 23 shows a plan and cross

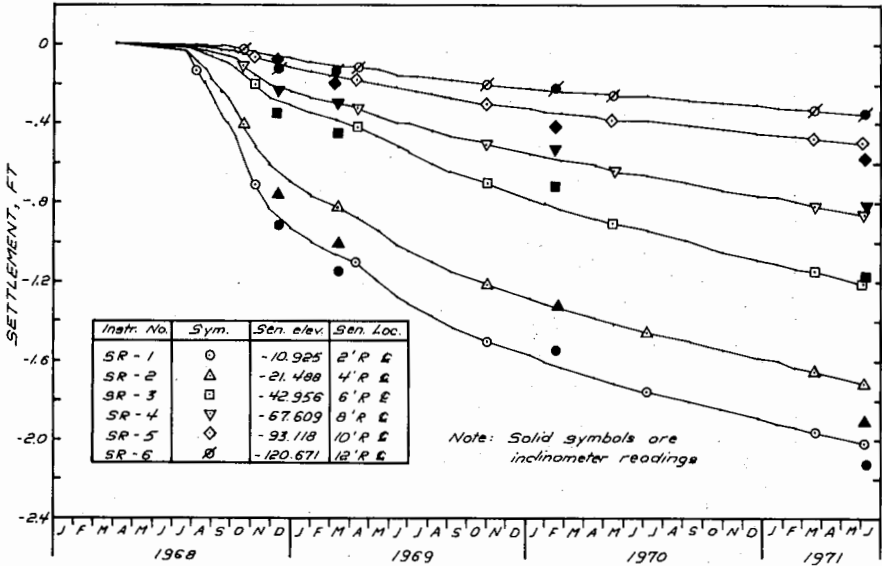


Figure 21. Comparison of Center Line Settlement SR-1 to SR-6 vs. I-2

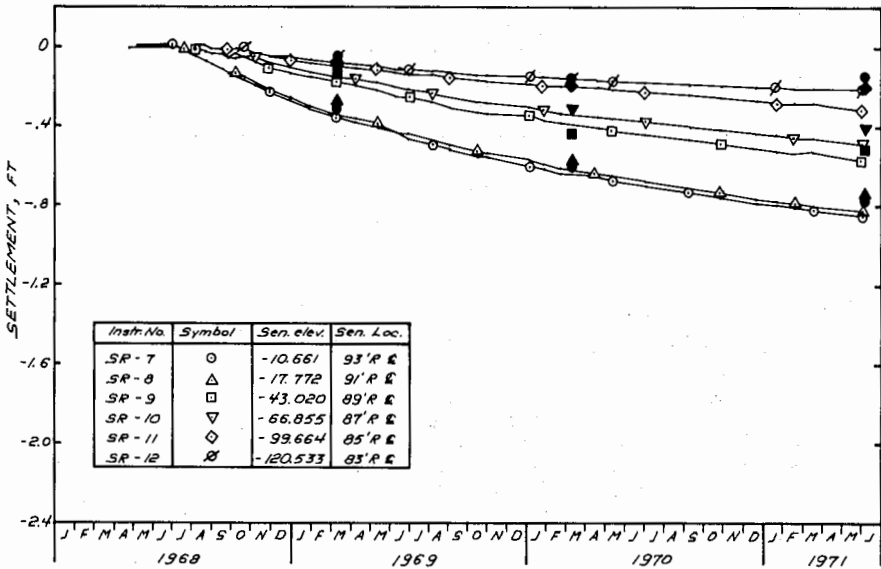


Figure 22. Comparison of Settlement 95 Fleet Right SR-7 to SR-12 vs. I-4

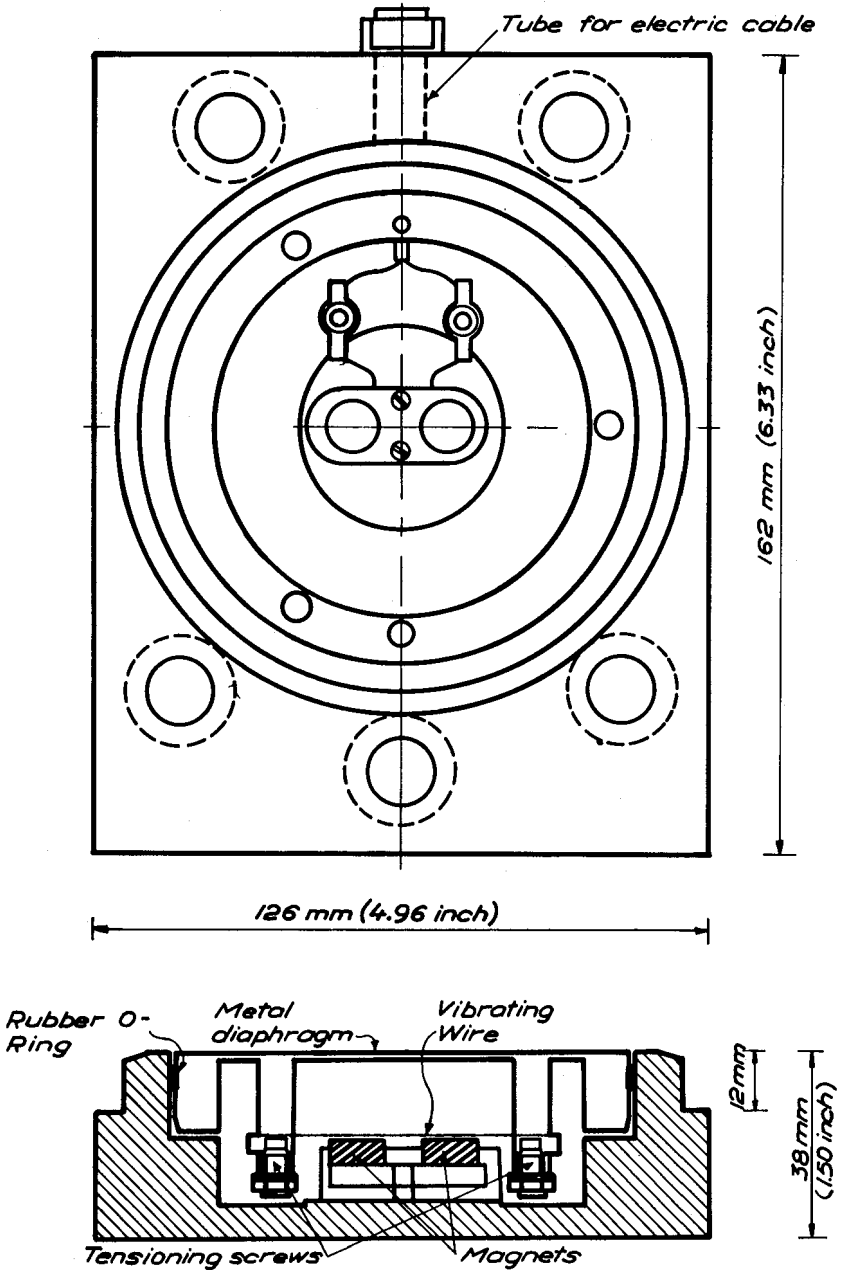


Figure 23. Details of the Total Stress Cell

section of the cell. The particular geometry of the cell is due to the fact that they were designed to be mounted on sheet piles at another ICEP project. Difficulties encountered in driving the sheet piles on which the cells were mounted led to abandonment of that attempt to measure total stress. The remaining stress cells were used for the I-95 project.

The stress acting on the metal diaphragm deforms it and causes tension in the wire stretched between the two diaphragm posts. The natural vibrating frequency of the wire depends on the tension and thus the stress action on the diaphragm. One electromagnet activates the wire and the other picks up the vibrating frequency which is amplified and measured directly with a digital frequency counter.

**Laboratory Calibration of Total Stress Cells.** Since the cells were originally designed to measure lateral earth pressures in soft and medium-soft clays, Geonor recommended using the hydrostatic calibration obtained by means of a water pressure tank. The I-95 embankment is coarse to medium sand with scattered gravels and boulders. Therefore, it was felt necessary to observe the behavior of the cells in a granular medium. The ultimate goal of this study was to develop the best approach in determining the calibration curves for granular soil.

The laboratory tests were carried out in a 35.12 in. diameter steel bin, 27 in. high. The dry uniform medium-fine sand (commercially called No. 1/2 sand) was placed by the "raining technique" at a rate of 5 pounds per minute with a height of fall of 10 in. In this way a uniform density of  $102 \pm 5$  lb/cu ft was achieved. The bin was filled 9 in. and then the cell placed horizontally with the sensing diaphragm up. Another 9 in. of sand was placed above the cell. The pressure was applied through a cylindrical pressure bag and a 5 in. plywood shim was placed between the pressure bag and the steel top cap which is bolted to the bin during loading.

Figure 24 shows that the response of the cell to applied pressure through the medium sand followed a pattern close to the water pressure calibration. A tendency of underregistration for the dense state and overregistration for the loose state was observed. This is consistent with the tests of Plantema (1953). A hysteresis effect was observed with higher registering in unloading than in loading. This is in agreement with the tests of Leussink and Prange, and Plantema (1953). The hysteresis effect should be considered in a case of loading and then unloading. Such is the case at I-95.

Figure 24 also shows that within the range of loading at the I-95 embankment (i.e., up to a level of 25 to 30 psi) the calibration for the loose sand backfill is almost linear with a slope somewhat different than that of the water pressure calibration.

**Cluster Scheme for Field Observation.** The measurement of total stresses in a soil mass is a most difficult task. Depending on the characteristics of the

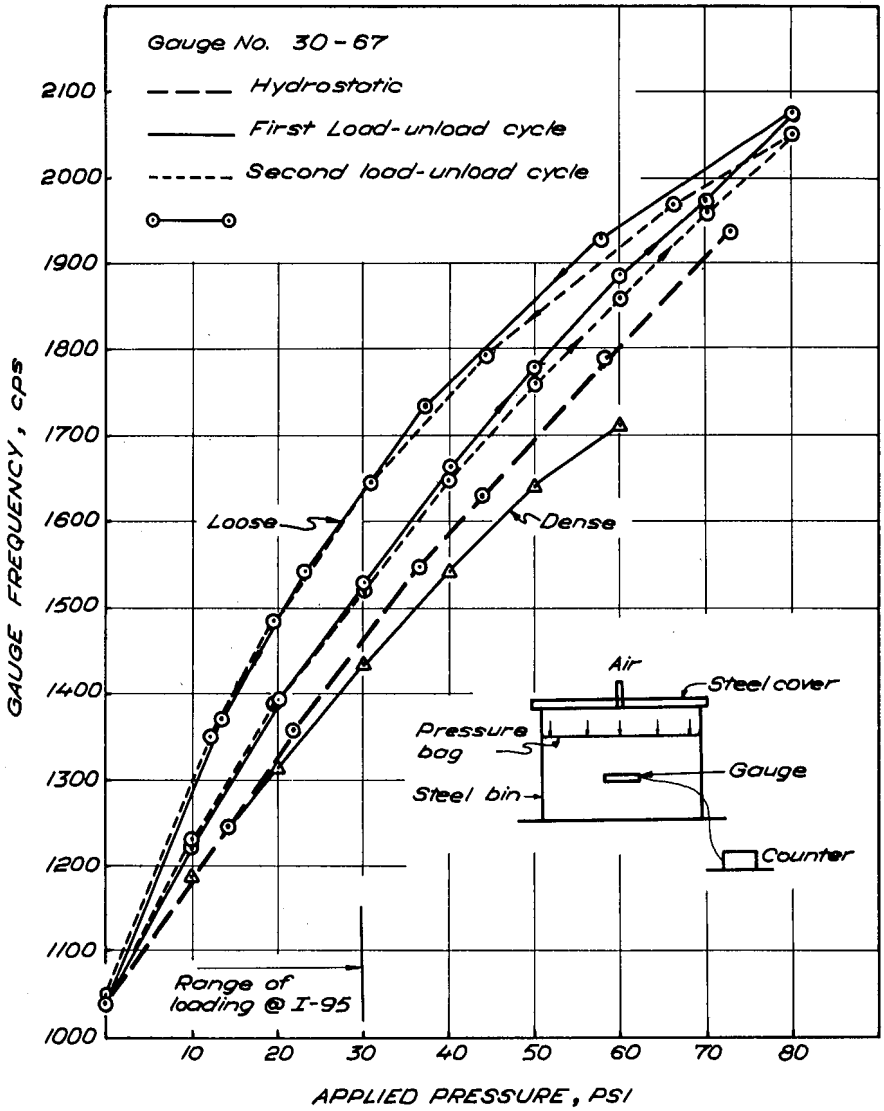


Figure 24. Laboratory Calibration of the Total Stress Cells

measuring device and the way it is placed in the soil, the device will sense more or less stress than would have existed had the device not been present. Taylor (1945) studied the effect of geometry and compressibility of a stress cell in reference to over- and underregistering. Guidelines for diameter to thickness and diameter to compressibility ratios were proposed to minimize these effects. Hadala (1967) examined the influence of placement methods on the response of the Waterways Experiment Station type total stress cells placed in sand and clay. Also, based on his observations on data scatter, he concluded that using a single stress cell to measure the magnitude of stress in a soil mass with any reasonable degree of confidence is a fruitless effort. Hadala recommends that an average of at least three cell measurements should be used if 20 percent accuracy is required nine times out of ten. Taylor's (1945) observations on three Carlson cell clusters placed in Arkabutla Dam (U.S. Corps of Engineers) are in agreement with Hadala's suggestions.

In light of this previous experience with total stress cells installed in a soil mass the scheme shown in Fig. 25 was adopted. This scheme calls for three stress cells bolted on a 2.0-ft diameter 1 in. thick circular steel plate. The cells are placed at  $3\frac{1}{4}$ -in.,  $5\frac{1}{4}$ -in. and  $7\frac{1}{4}$ -in. distances away from the center of the plate to observe the overall stress distribution imposed on the plate.

**Field Installation and Field Calibration.** The stress cells were installed in three clusters: at the center line, 30 feet right of the center line, and 60 feet right of the center line. Figure 26 shows the location of the three clusters. At the time the cells were installed, the average fill elevation was +20.0 ft and three circular funnel type holes approximately 3 feet in depth were excavated (see Fig. 26). To prevent potential arching the average diameter of the hole was about eight times the diameter of the cell cluster. The cell cluster was placed on firm soil, leveled and monitored for zero load reading. The backfilling was made by "raining" the material through a No. 4 sieve. Slight compaction by tamping was used. After 2 ft of backfill were placed, filling continued with regular embankment fill. After each foot of backfill the cells were monitored and field density tests were performed. Above Elev. +20 ft the filling was carried on in the regular way with rubber-tired compaction, and field densities were taken at an average of one per foot of fill. Assuming that up to Elev. +25 ft the vertical total stress acting on the cells would be practically equal to the average fill density times the height of fill, the observations in this range were treated as field calibrations. Figure 27 shows a typical field calibration established in this manner. The hydrostatic calibration was used as a reference line in extrapolating the field calibration for higher stress levels. The nearly linear behavior of the field calibration for the particular range of stress level was deduced from the laboratory calibration studies (see Fig. 24).

**Typical Data of One Cluster.** Figure 28 shows elevation of fill versus total vertical stress measured by the three cells of Cluster A. The maximum difference

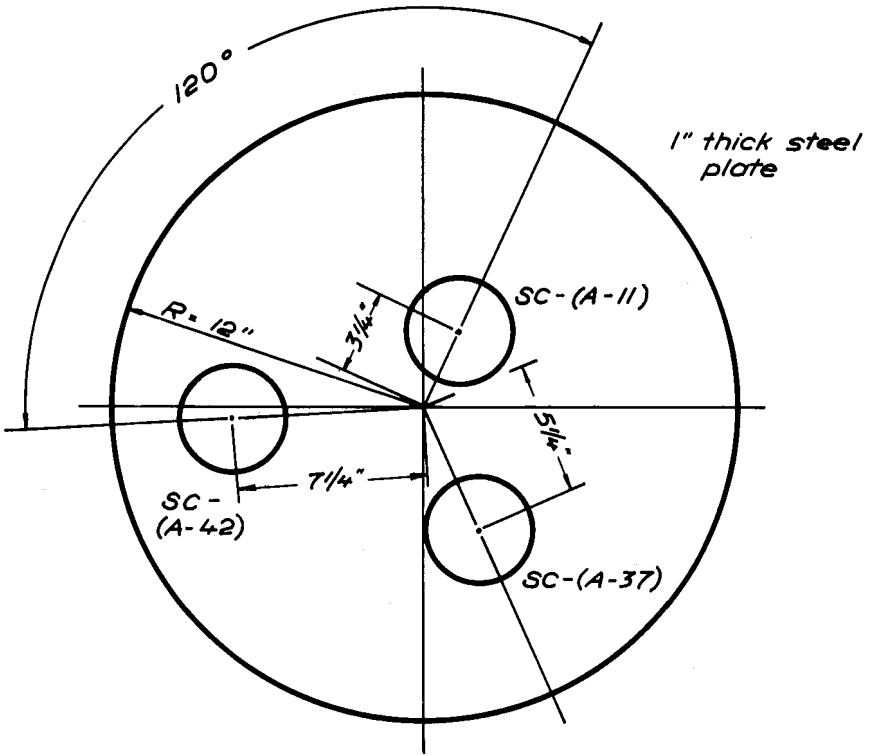


Figure 25. Details of Total Stress Cell Cluster

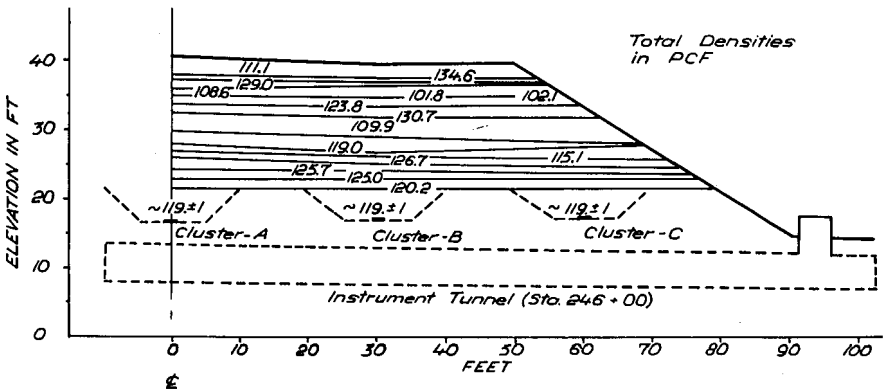


Figure 26. Location of Total Stress Cell Clusters

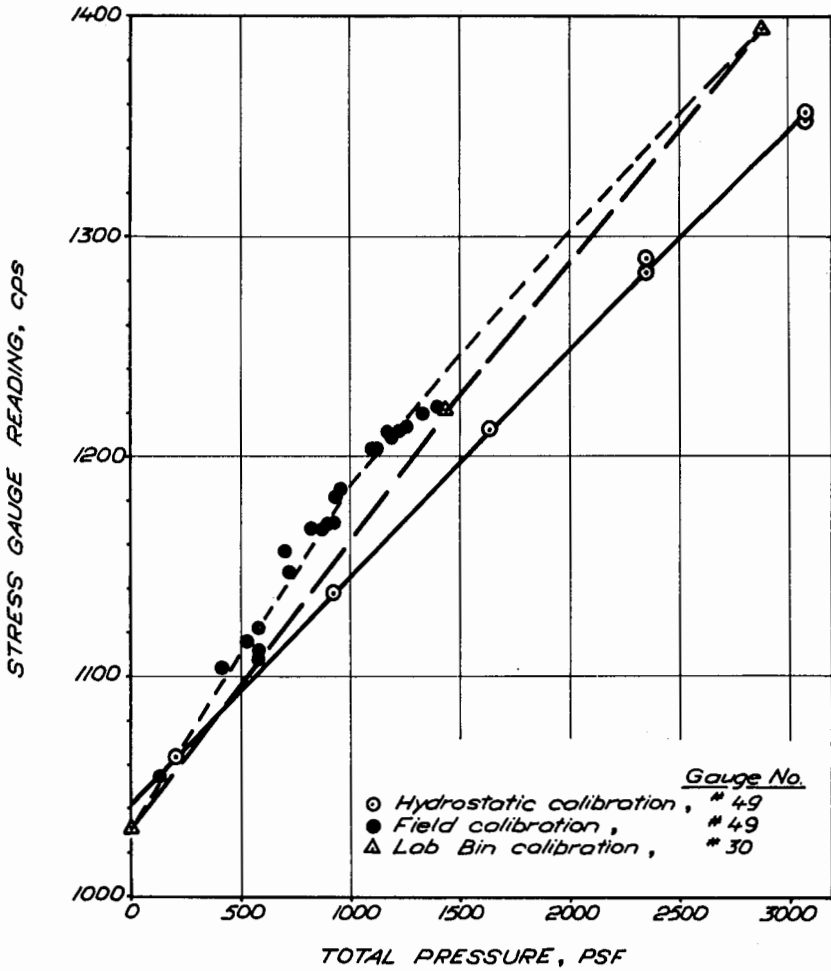


Figure 27. Field Calibration of Total Stress Cells

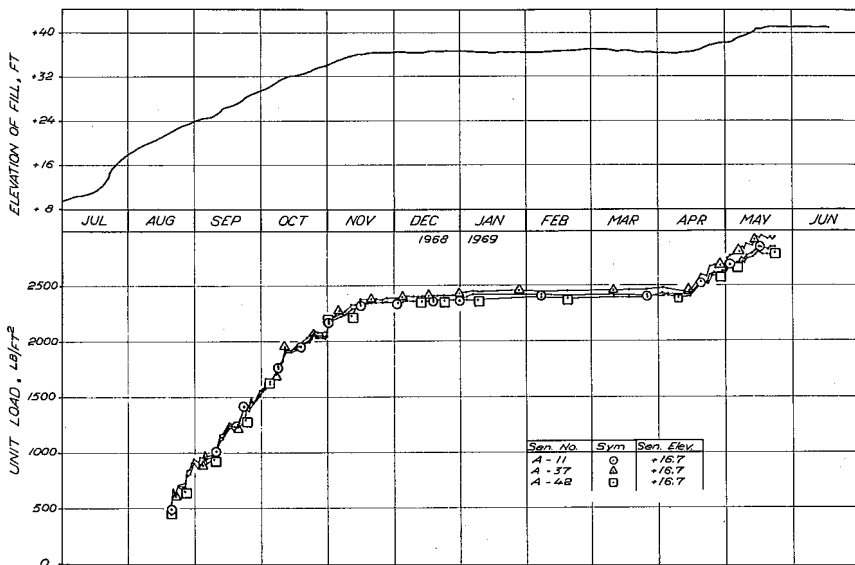


Figure 28. Total Stress Cell Measurements for Cluster "A"

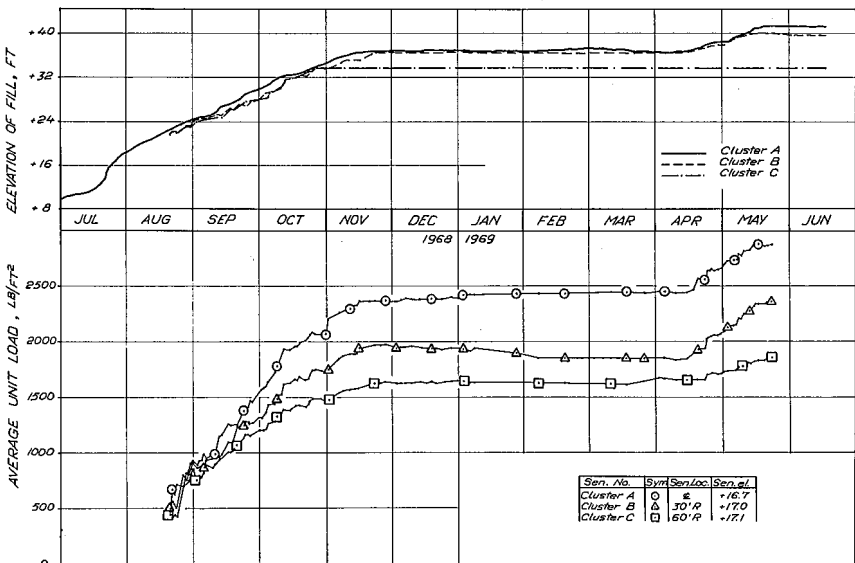


Figure 29. Total Stress Measurements for Clusters "A", "B", and "C"

in individual cell readings is about  $150 \text{ lb/ft}^2$  over a total magnitude of nearly  $3000 \text{ lb/ft}^2$ . Similar observations were made from Clusters B and C.

**Data of Average of Clusters A, B, and C.** Figure 29 shows elevation of fill versus average measured total vertical stress for Clusters A, B, and C. Note that between November 20, 1968 and April 20, 1969, during which time no filling was made, Cluster A and Cluster C have picked up an increment of approximately  $100 \text{ lb/ft}^2$  and  $50 \text{ lb/ft}^2$ , respectively, and Cluster B experienced a release of approximately  $100 \text{ lb/ft}^2$ .

Figure 30 shows for several fill heights the geometry of the embankment, the total vertical stress distribution equivalent to average unit weight times the embankment height, and the measured total vertical stresses. For the maximum fill configuration, Elev. +40 ft, the theoretical prediction given by Perloff et al (1967) for homogeneous, isotropic, linear elastic embankment and foundation has been shown for comparison. The agreement is very good.

#### IV. Conclusions

1. The instrumentation system has sufficiently documented the performance of a heavily loaded, compressible foundation during the loading period. Replacement of some piezometers was made to adequately document the post-construction period.

2. The deformation and total stress instruments have performed very satisfactorily and only minor changes to the designs and installation procedures would be made for subsequent jobs.

3. Major changes in the design and installation procedures for piezometers, both electrical and hydraulic, will be made for future installations. There are many cost and time incentives for developing reliable, multiple sensor piezometers to fit in one borehole, as well as to solve longevity problems due to large deformations, shifting calibrations (electric) and trapped air (hydraulic). Piezometer sensors will not be placed below rigid casings where large settlements are expected. Double tubing for hydraulic piezometers will be used at all times.

4. Major improvements are also planned for data acquisition and data handling problems. The procedures used for this project were slow, used a lot of skilled manpower and produced job records that were somewhat unwieldy for rapid and diverse use.

#### V. Acknowledgments

The research described in this paper was supported by the Department of Public Works, Commonwealth of Massachusetts and the Bureau of Public Roads, U.S. Department of Transportation. The authors appreciate the support they

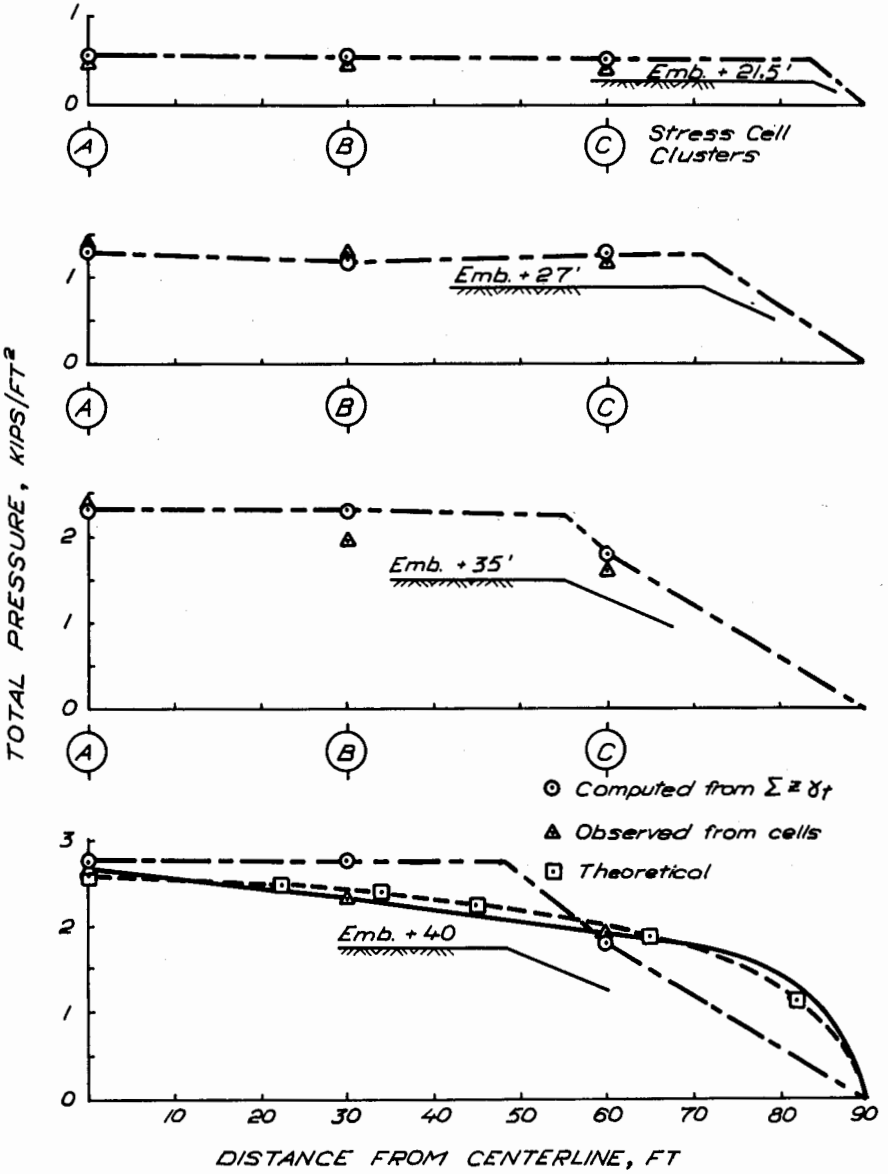


Figure 30. Comparison of Theoretical and Measured Total Stress Values

have received from the sponsors, especially Mr. Charles Whitcomb, Deputy Chief Engineer for Highway Design, Mr. Robert T. Tierney, Deputy Chief Engineer for Highway Maintenance, Mr. John J. Lyons, Research and Materials Engineer, and Mr. Paul McHugh, Assistant Research and Materials Engineer, all of the Massachusetts Department of Public Works.

The Perini Corporation constructed the embankment and installed the main test section instrumentation through a subcontract with Geomeasurements, Inc.

The M.I.T. principal researcher was T. William Lambe, Edmund K. Turner, Professor of Civil Engineering. Walter Beckett, Joseph Guertin, Kjell Karlsrud, Robert Kirby, and Robert McPhail of the field measurements staff and several Northeastern University Coop students have done most of the work. Robert Kirby made important additions and editing to the text. The contributions of all are appreciated.

The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Massachusetts Department of Public Works or the Bureau of Public Roads.

## APPENDIX A — REFERENCES

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## APPENDIX B – INSTALLATION PROCEDURES USED AT STATIONS 245 and 246

### I. Hydraulic Piezometers – Preconstruction Section

1. Advance 2½-in. diameter casing to within 50 ft of piezometer sensor elevation by any method.
2. Wash to bottom of casing.
3. Attach the piezometer to a sufficient length of E-rod, being careful not to crimp the plastic tubing which goes inside the E-rod.
4. Press or drive the piezometer to the desired elevation, adding E-rod extensions as required. Leave the E-rod in place with approximately 5 ft extending up from the ground surface. The natural clay provides the piezometer seal.

### II. Hydraulic Piezometers – MIT-MDPW Test Section

1. Advance 2½-in. diameter casing through fill by any method; seal casing by pushing into the top of the clay layer.
2. Mix thick mud; sealing properties stressed not weight.
3. Wash ahead of casing with mud to 10 ft above center line of sensor.
4. Push casing to 2.0 ft above center line of instrument; wash out to bottom of casing with clean water.
5. Take split spoon sample.
6. Wash 1.75 ft below the center line of the sensor until the returning water is reasonably clear (do not recirculate water). Stop pumping, raise the wash rods 5 ft and wait for 10 minutes. lower the rods to the bottom of the hole and pump for 5 minutes. If the wash water remains clear the rods can be removed.
7. Lower the piezometer and sash weight into the hole. Place the sash weight on the bottom and check to see if the bottom is firm. If the bottom feels soft remove the piezometer and rewash the hole.
8. Keeping the piezometer leads tight, slowly add sand until it fills the collection zone and is up in the casing. Tamp thoroughly using a tamping hammer.

9. Siphon 4 in. of water out of the casing. Add about one cup of bentonite balls, one at a time, until the water is  $1\frac{1}{2}$  in. below the top of casing. Wait 3 to 5 minutes and then add sand (or  $\frac{1}{4}$  in. pea gravel) until the water is  $\frac{1}{2}$  in. below the top of casing. Wait 3 to 5 minutes and then tamp the seal, gradually at first (i.e., 6-in. drop), then increasing the drop to 12 and then 18 in. Continue tamping until the seal does not compress more than  $\frac{1}{2}$  in. for 10 blows. Repeat the above process until the seal is 24 in. thick.
10. Place 5 ft of sand in 1-ft lifts, tamping each lift thoroughly; then, keeping the leads tight, backfill the rest of the hole with loose sand.

### III. Electrical Piezometers — MIT-MDPW Test Section

1. After hole has been advanced to final depth and cleaned by the same methods as specified for hydraulic piezometers, lower the electrical piezometer to the bottom of the hole with the porous brass tip removed. Measure the exact depth of the sensor.
2. Monitor the piezometer for a minimum of 30 minutes and record readings every 5 minutes. When it is established that the instrument is in equilibrium, pull the instrument up taking readings at 5-ft intervals for a field calibration.
3. Without touching the instrument, remove from the water and record the frequency reading. This is called a zero reading.
4. Turn the instrument upside down in a bucket of warm water and put on the previously deaired porous brass tip under water. Cover the tip and the filter with two prophylactics and secure with an elastic. Securely fasten a thin string to the tip of the prophylactics.
5. Lower the instrument several feet into the water-filled hole and tear the membranes with the string.
6. Proceed as for hydraulic piezometers.

### IV. Settlement Anchors — Preconstruction Section

1. Advance a  $2\frac{1}{2}$ -in. diameter open hole either through overlying granular soils or to within 50 ft of the proposed anchor elevation, whichever is closer.
2. Attach a 1.0-ft section of 1-in. pipe to the anchor point. This section of pipe should have one left hand and one right hand thread. The left hand threaded end is attached to the point using a special coupling. Count the number of turns necessary to tighten the coupling.
3. Attach lengths of  $\frac{1}{4}$ -in. rod and then the 1-in. pipe by sliding it over the  $\frac{1}{4}$ -in. rod.

4. Lower the anchor to the bottom of the open hole adding lengths of  $\frac{1}{4}$ -in. rod and 1-in. pipe as necessary. Drive the anchor to the desired depth below the bottom of the hole.
5. Hold the 1-in. pipe securely, tap the  $\frac{1}{4}$ -in. rod down 5 in. This will force the three prongs at the tip of the anchor out into the surrounding soil.
6. Unscrew the 1-in. pipe by rotating clockwise at least the number of turns noted when it was attached to the anchor.
7. Bump the 1-in. pipe back 3.0 ft.

#### V. Settlement Rods —MIT-MDPW Test Section

1. Advance casing through fill by any method; seal at the top of the clay layer.
2. Mix thick drilling mud; sealing properties stressed.
3. Using mud wash ahead of casing to 3.0 ft above proposed sensor elevation.
4. Push casing to 6 in. above sensor elevation.\*
5. Wash to sensor elevation with drilling mud. Do not clean out hole with clear water.
6. Lower the settlement rod into the hole until it rests on the bottom and then push it 3 ft into the clay.

#### VI. Settlement Platforms — MIT-MDPW Test Section

1. Excavate to Elev. +5 ft through embankment fill using a grade-all.
2. Place platform at bottom of excavation and carefully level it.
3. Backfill around settlement platform with sand.

#### VII. Inclinometers

1. Advance 6-in. diameter casing through the fill by any method; seal the casing by pushing into top of clay layer.
2. Mix thick mud with sealing properties stressed, not weight.
3. Advance 6-in. open hole to bottom of clay and then drill 5 ft into the underlying till. Keep hole full of drilling mud at all times to prevent cave-in.
4. Lower aluminum casing into hole. Successive prefabricated 11-ft sections (made up of two 5-ft sections joined together by a 2-ft coupling with 6 in. left between each section) are lowered using special clamps. Each 11-ft

\*For future installations, push casing to no closer than twice the estimated differential settlement between sensor location and top of ground.

section is joined together by a 2-ft coupling with 6 in. left between each section. The couplings are coated with beeswax and the casing kept full of clear water to overcome bouyance.

5. Align grooves parallel and perpendicular to embankment.
6. Backfill annular space between aluminum casing and borehole with pea gravel. Continuously vibrate the aluminum casing during backfilling. Rodding the backfill with a ½-in. steel rod proved most effective for controlling arching of the backfill.
7. Wash casing out until you can circulate clear water.

# SETTLEMENT OF RUBBISH FILL YELLOW FREIGHT SYSTEM FACILITY – BOSTON

by William S. Zoino,\* Member

(Presented before the Boston Society of Civil Engineers Geotechnical Section – Forum Meeting on Problems of Construction on Rubbish Fill and Peat January 19, 1971).

## Introduction

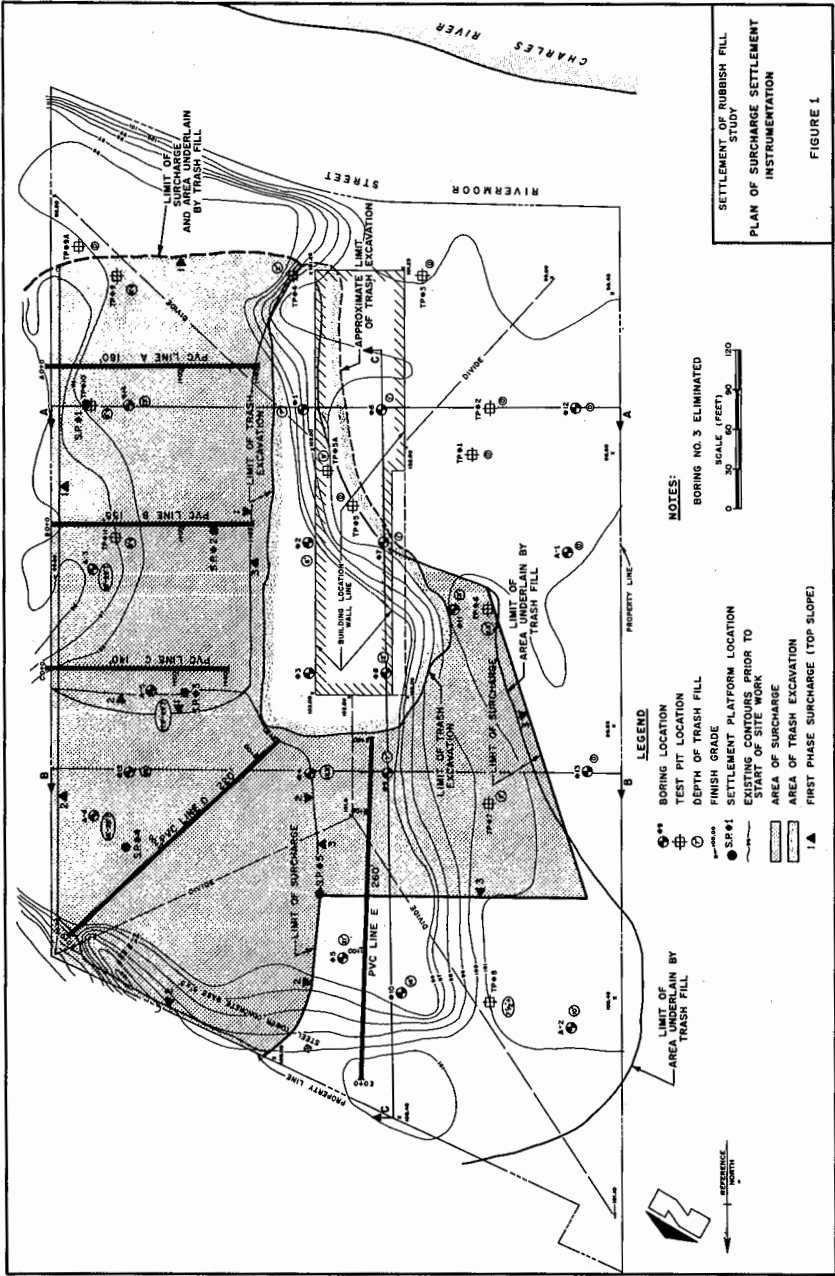
This paper presents the results of settlement measurements obtained at the site of a trucking facility presently under construction in West Roxbury, Massachusetts. The terminal building is located on a suitable portion of the site which allowed use of normal shallow foundations. However, the proposed paved yard area is underlain by a maximum of 25 feet of rubbish fill which had been placed on the site in the mid 1950's. The cost of excavation below water and replacement of the rubbish with granular fill in this area would have been prohibitive. It was therefore decided to leave the rubbish in place and to surcharge the site with an excess of fill above the required raise-in-grade to minimize post construction settlement beneath pavements.

A program of field instrumentation was undertaken to monitor the settlement performance and to identify areas that might undergo severe post construction differential settlement. Provision was made in the construction specifications to provide additional surcharge in these areas using observed settlements as a guide. However, only minor revisions to the planned surcharge program resulted. A description of the site and the pertinent results of the settlement measurements are presented.

## 1.00 Site Description

The site is located on Rivermoor Street in West Roxbury, Massachusetts, as shown on Figure 1. It is bounded by Rivermoor Street on the south, by property of the Rexall Drug Company and United Liquor Company on the west and east respectively, and by a Boston Edison Company transmission line on the north. In 1969, the site was characterized by a low wet area over most of the eastern portion, and by relatively flat, slightly higher flanking terrain on the western portion. The absence of vegetation was indicative of a filled situation and possibly a cut situation in the western portion.

\*Goldberg-Zoino Associates, Inc., Cambridge, Mass.



According to local residents, the site previously had been used as a sand and gravel pit and later abandoned. At a later date the site was used as a dump by the City of Boston. Both observations are consistent with subsurface data obtained from borings and test pits.

## 2.00 Test Borings and Test Pits

A total of 14 test borings and 13 test pits were made at the site in late April and early May 1969. These explorations are located as shown on Figure 1.

The borings ranged in depth from 25 feet to a maximum of 60 feet. Samples were taken using the conventional 1-3/8-inch I.D. split-spoon sampler, and the standard penetration resistance of the sample in blows per foot was obtained. Test pits were excavated with a ¼-cubic yard backhoe in order to examine the materials in place.

## 3.00 Subsurface Conditions

Subsurface profiles showing representative conditions are given on Figure 2. These conditions may be further generalized as follows:

### 3.1 Conditions in South and West Portions of Site

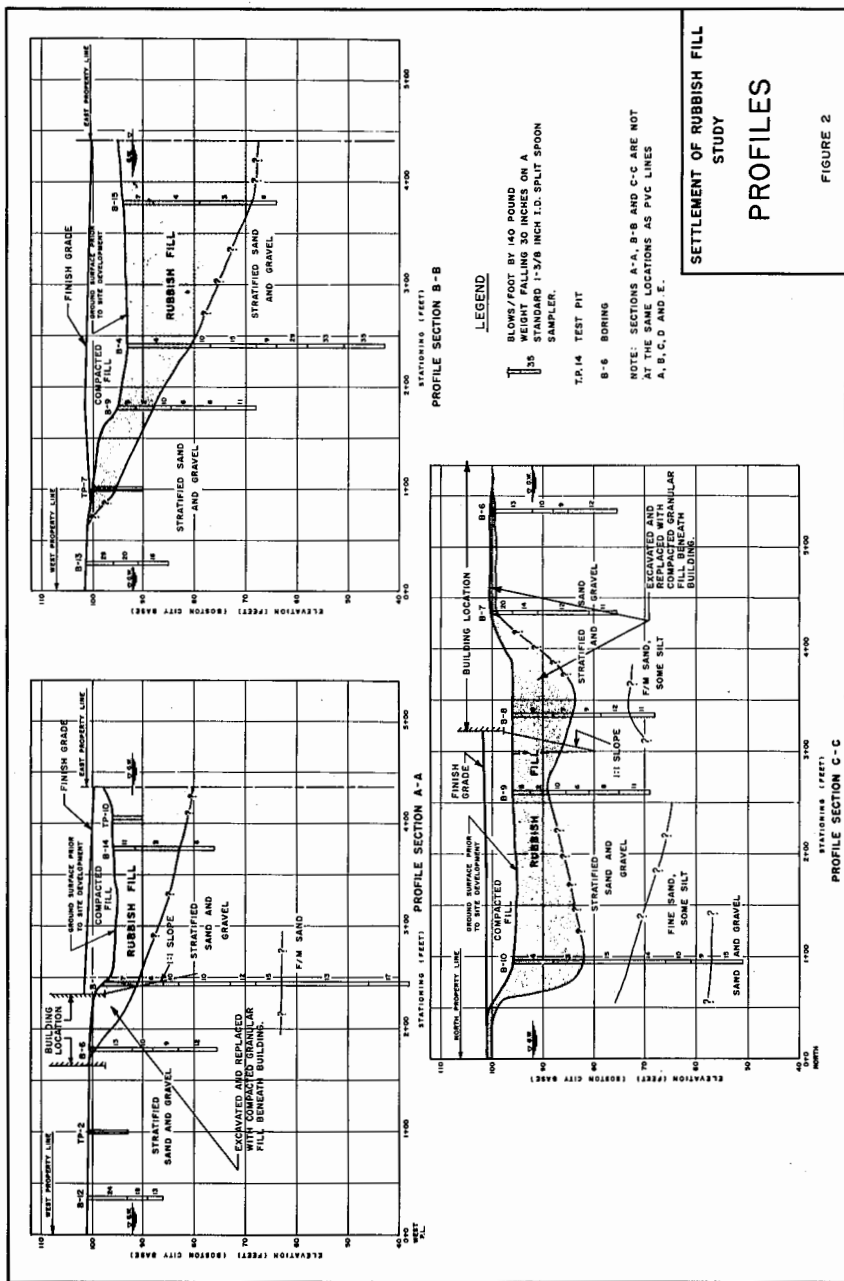
Prior to recent construction the subsurface conditions in this portion of the site were characterized by loose to medium dense granular deposits. These are natural soils containing fine to coarse sands and fine gravel in the upper 10 to 25 feet, and containing fine to medium sands with some silt and little or no gravel below depths of 19 to 25 feet and extending to depths of at least 60 feet below present ground surface.

### 3.2 Conditions in East and North Portions of Site

The subsurface conditions in the eastern and northern portions of the site are characterized by the presence of rubbish fill within the approximate limits indicated on Figure 1. The maximum depth of fill was encountered at Boring No. 15 where it was found to be in excess of 25 feet. Information from boring logs and penetration resistance indicated that the rubbish fill was very loose. Test pits excavated into this material verified the loose and compressible nature of the deposit. The fill is underlain by loose to medium dense natural granular deposits consisting of fine to coarse sand and fine gravel.

## 4.00 Construction at Site

After investigation of the site in early 1969, excavation and filling of the site was started in September 1969. The limits of the various items of work are shown on Figure 1. The work consisted of:



1. Within the building limits: Removal of rubbish fill and replacement with compacted granular fill.
2. Areas to be paved: Surcharge of rubbish fill with approximately six feet of temporary earth surcharge for periods not less than 30 days.
3. Other areas: Direct filling over zones underlain by rubbish.
4. Construction of the building on normal shallow foundations, starting in September 1970. Construction was completed in August 1971.

## 5.00 Settlement Measurements

### 5.10 Settlement Devices

Measurements of settlement during filling and surcharging were undertaken along five horizontal lines (PVC Lines A to E) and at five settlement platforms located as shown on Figure 1. The settlement measurements were made using a hose settlement device capable of obtaining a continuous profile of settlement along the buried PVC lines. Detailed descriptions of the apparatus and the measurement technique are appended hereto.

### 5.20 Results of Measurements

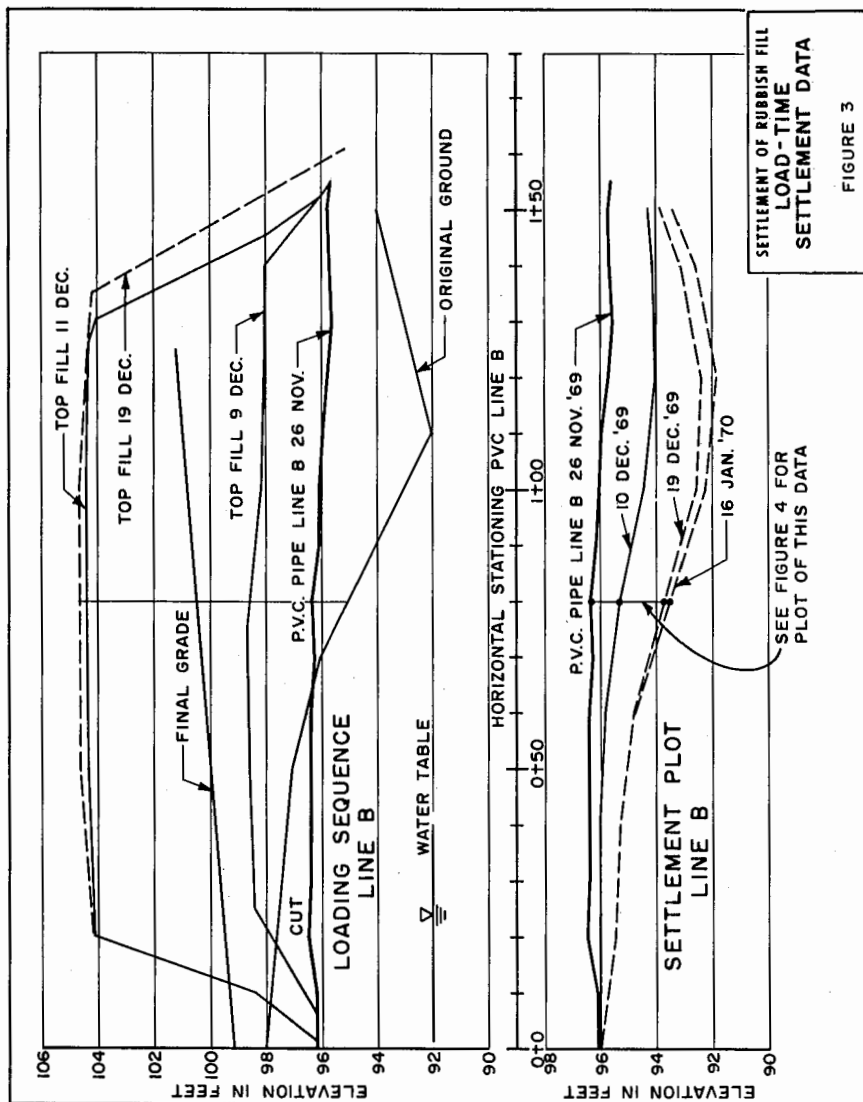
Observations of settlements were made at several points along the five buried PVC pipes at periodic intervals during filling and surcharging. Typical data obtained along PVC Line B are shown on the bottom portion of Figure 3 and on Figure 4. It may be noted that maximum differential settlements of the order of four feet were experienced along this line. The filling and loading sequence along this line is shown in the upper portion of Figure 3.

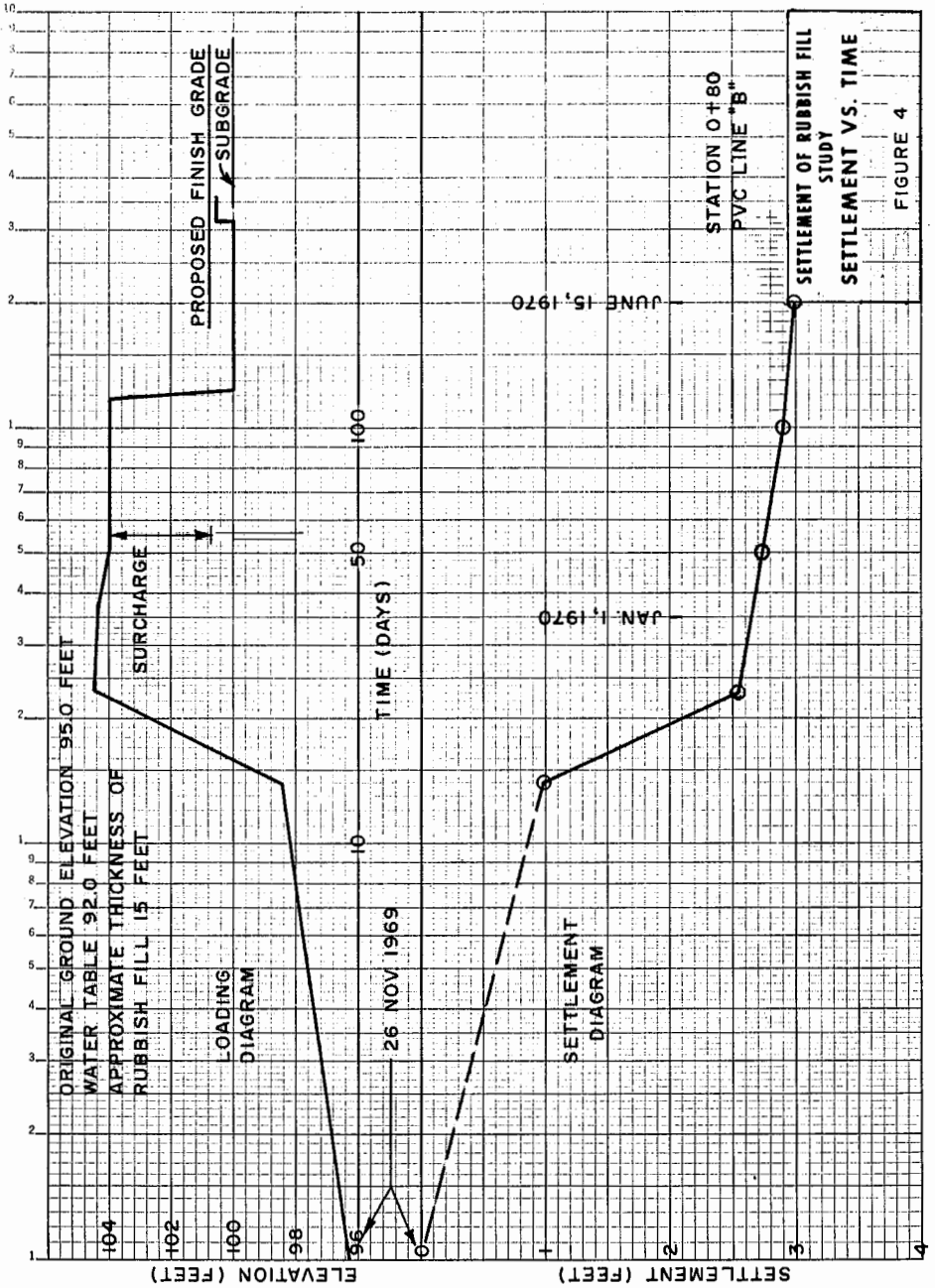
The unit compression defined by the total settlement divided by initial thickness of rubbish fill was calculated for each of the observation points. These data were plotted against applied surface load as shown in Figure 5 for a typical point. Although the scatter was wide, most of the points fell within the maximum and minimum lines shown on Figure 5.

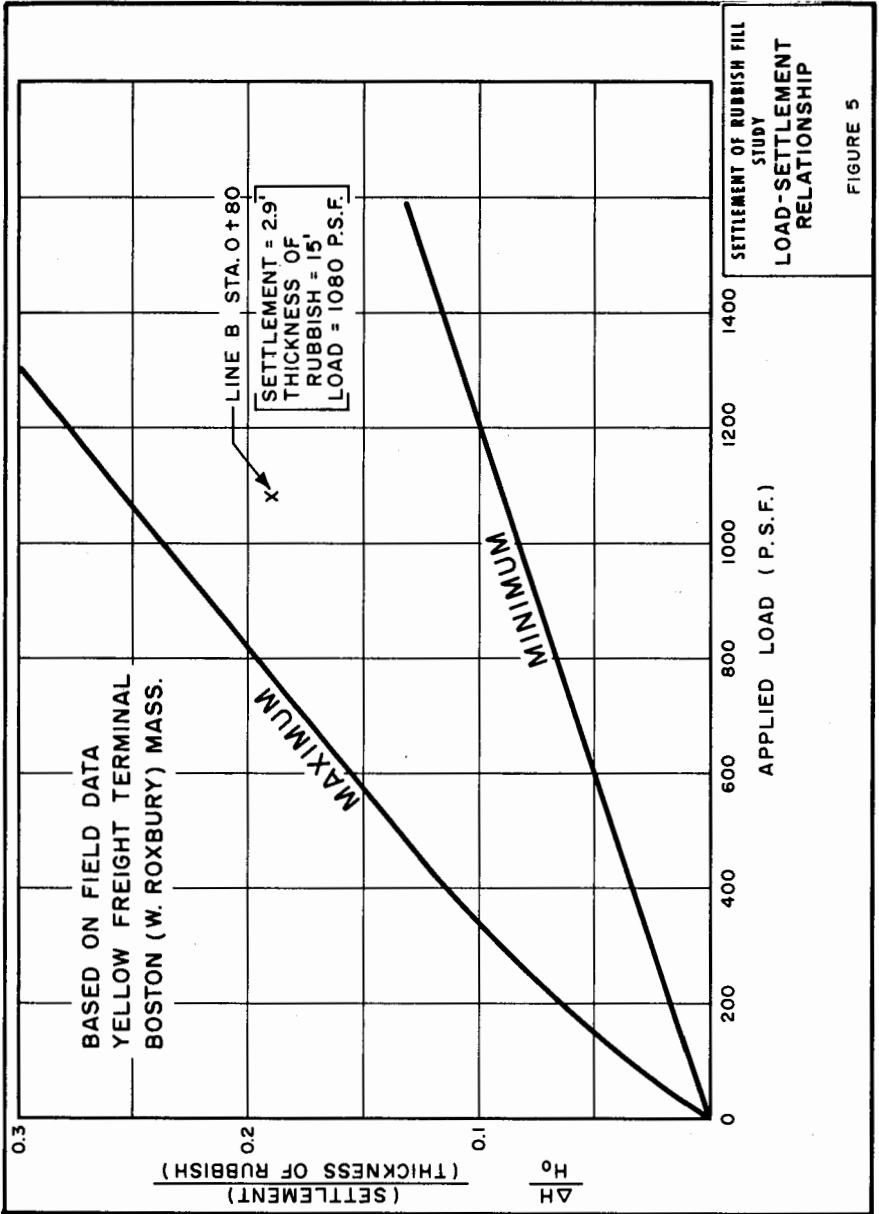
### 5.30 Settlement Parameters

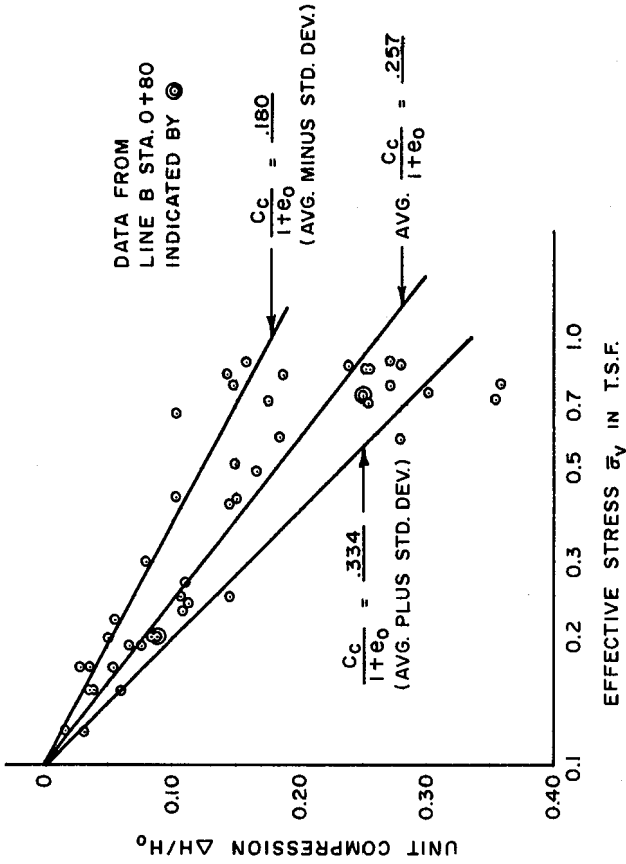
An attempt was made to analyze the data in terms of the conventional parameters that are used with one-dimensional consolidation theory. These parameters are  $C_c/(1+e_0)$ ;  $c_v$  and  $C_\infty$ . The "e log p" plot for all data points is given in Figure 6. In this plot the assumption is made that the unit compression versus pressure relationship is linear on a semi-log plot.

The results shown on Figure 6 and the other parameters were compared with field results from other projects and from published data. This information is given on Table 1. It should be noted that the time-settlement mechanism of rubbish fill may not bear any theoretical resemblance to Terzaghi consolidation theory. For example, the coefficient of secondary compression in rubbish fill is









PLOT OF SETTLEMENT DATA  
(normalized to common  $\bar{\sigma}_{v0}$  of 0.1 T.S.F.)

SETTLEMENT OF RUBBISH FILL STUDY  
COMPRESSION VS. LOG PRESSURE  
FIGURE 6

Table 1. Summary of Compression Characteristics of Rubbish Fill

SITE	$C_c$ $1+e_0$	$C_v$ $Ft^2/Day$	$C_\alpha$
Yellow Freight Terminal West Roxbury, Massachusetts (Reference 1)	.257±.077	3.35±2.71	.024±.020
Columbia Point Housing Project Boston, Massachusetts (Reference 2)	—	—	.021±.003
Sanitary Landfill Site — Georgia (G.F. Sowers — Reference 5)	.3 (est)	—	.007 to .015
Large Diameter Cell Tests California (References 3 and 4)	.192	2.12	.020
Typical Peat Values (Reference 2)	.25 to .45	0.1 to 0.8	.015 to .035

partly related to chemical decomposition. Nevertheless for this project the selected parameters were a convenient way of presenting the results and for comparison of results with other projects. It may also be seen in Table 1 that the results were of the same order of magnitude as might be anticipated in highly organic soils such as peat.

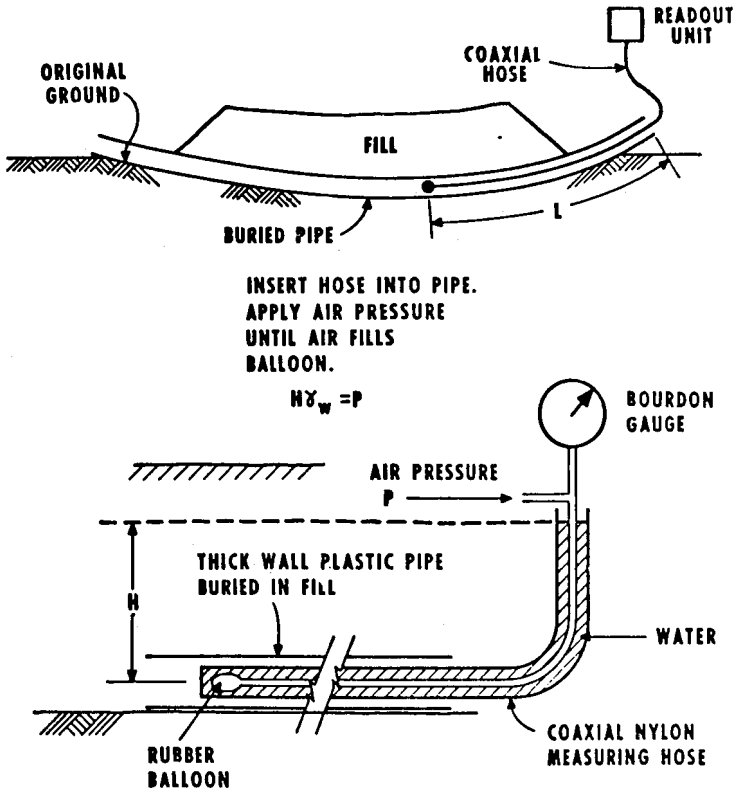
### **Acknowledgement**

The writer wishes to acknowledge the help of Mr. Peter J. Riordan and Mr. John P. Sullivan of Goldberg-Zoino & Associates, Inc. in the preparation of this discussion. The writer also wishes to acknowledge the advice and help of Mr. C.J. Dunncliff of Soil and Rock Instrumentation, Inc. for the selection, installation and data acquisition of the field instrumentation.

### Principle of Operation of Full-Profile Settlement Gauge

The principle of operation is based on the method proposed by Broms and Bergdahl (Reference 6).

A 1½-inch I.D. thick wall PVC pipe is laid in a shallow trench on the original ground surface or at any elevation within the fill, and subsequently covered by fill material. To measure the elevation of *any* point of the pipe, a probe, connected to plastic tubing, is pulled along the pipe. By achieving an air/water pressure balance across a thin rubber balloon housed in the probe, the elevation of the probe can be determined, as shown below.



The major advantages of the gauge, as compared with conventional settlement platforms, are:

1. It does not interfere with earthmoving operations.
2. It provides a full profile of readings along the buried pipe (which is of particular significance if thickness or compressibility of comparable material varies in a horizontal direction).

3. The same instrument can be used on many successive jobs, the only item expended on each job is the buried pipe.
4. Accuracy of measurement is adequate for settlement measurements above rubbish fill and peat.

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# SEWAGE DISPOSAL IN FALMOUTH, MASSACHUSETTS

## I. "EXPERT" OPINION AND PUBLIC POLICY<sup>1</sup>

W. Redwood Wright, Dean F. Bumpus and Ralph Vaccaro<sup>2</sup>

### Abstract

The problems of providing useful scientific information on controversial public issues are considered in the light of an experience at the local level — the debate in the Town of Falmouth, Mass., over building a secondary sewage treatment plant with an outfall in Vineyard Sound. Woods Hole scientists made surveys of both land and sea disposal sites and concluded, with some reservations, that no environmental threat existed. The weight of scientific opinion strongly favored the outfall but it was roundly defeated. The town is still wrestling with the sewage problem.

Some questions for the scientific community:

How can we make convincing recommendations while preserving scientific detachment?

How can we disagree publicly without hopelessly confusing the lay public?

How can we help the layman separate fact from opinion, or thoughtful disagreement from the not-so-thoughtful?

How can we reconcile the need for more study with the need for action?

How was it that we knew so little about our waters to begin with?

Some conclusions: We must condition the public to expect uncertainty and not revealed truth in scientific utterances.

We must learn to commit ourselves effectively despite this uncertainty.

And we must try to anticipate public questions and begin to accumulate data.

### Introduction

Falmouth, Massachusetts, located at the southwest corner of Cape Cod, (Figure 1) is in many ways a typical New England waterfront resort community. Its government is based on the town meeting, which in Falmouth consists of 259 elected representatives of the townspeople. Its economy is based on the tourist dollar. Its population, which triples in the summertime, is concentrated along the seashore. And it is beset by growing pains, reflected in a tax rate rise of 50 percent in less than ten years, a new school under construction every couple of years, and increasing concern about zoning and conservation and about pollution of natural waters, both fresh and salt.

<sup>1</sup>This is the first of three papers relating to the main subject. It was delivered, in slightly different form, at the annual meeting of the American Society of Limnology and Oceanography, Kingston, R.I., August 1970.

<sup>2</sup>All, Woods Hole Oceanographic Institution, Woods Hole, Mass.

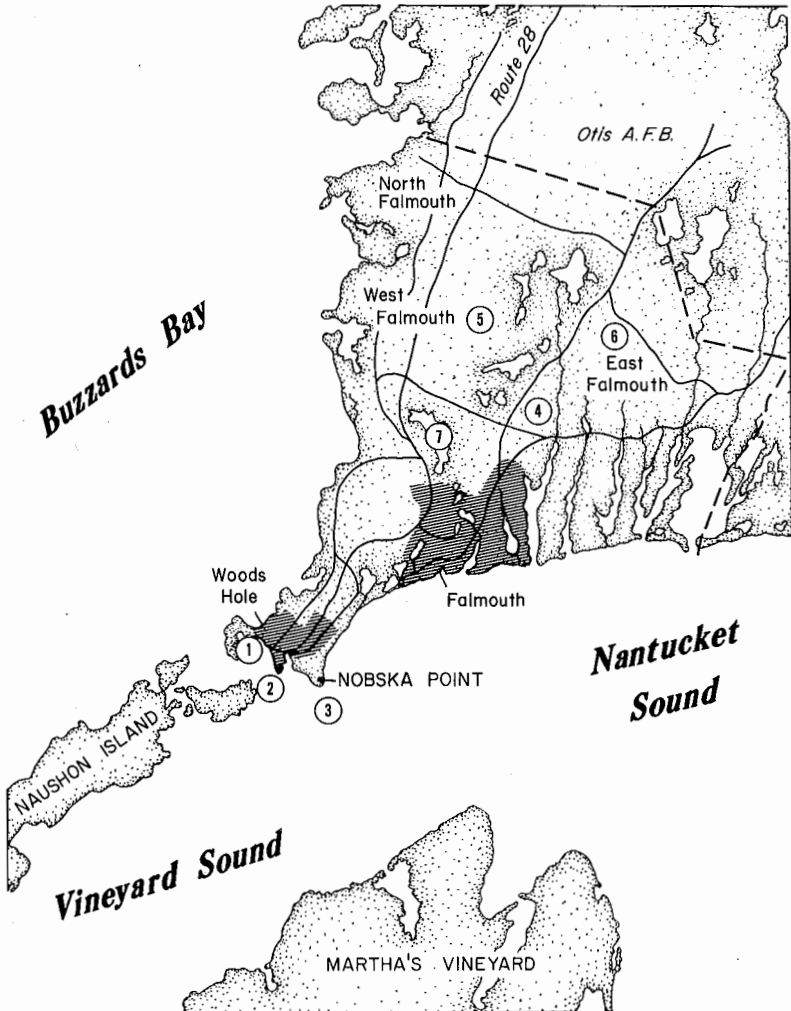


Figure 1. Falmouth, Mass., and vicinity. The hatched areas would be served by the proposed sewer. Circled numbers identify sites mentioned in the test, as follows: 1. Existing outfall in Great Harbor, Woods Hole. 2. Outfall site originally proposed by engineers. 3. Outfall site recommended by W.H.O.I. scientists. 4. Possible inland disposal site – Brick Kiln Road. 5. Possible inland disposal site – Blacksmithshop Road. 6. Possible inland disposal site – Hayway Road. 7. Town water supply – Long Pond.

Falmouth is unusual in that it is the home of a large marine science complex. The village of Woods Hole boasts the Marine Biological Laboratory, a laboratory of the National Marine Fisheries Service, and the Woods Hole Oceanographic Institution. Many of the people in the scientific community are year-round residents of Falmouth, many spend only the summer months on Cape Cod. In either case, they are as interested as any other Falmouth resident in keeping the town a pleasant place to live. For several decades, members of the scientific community have been active in the affairs of Falmouth, serving on town committees, in elective office or working through non-governmental organizations. Several are town meeting members. The scientists in Falmouth have worked with their neighbors in dealing with the problems of the town, but have done so as citizens and not specifically as scientists.

Recently a local problem arose which involved the professional capabilities of the Woods Hole scientists — namely, a proposal to build a sewage treatment plant with an ocean outfall. The question of sewers in Falmouth is by no means new — efforts to provide some sort of sewage system for the most densely populated parts of the town have been made, and defeated, since 1926, and shortly after World War II a sewer was installed to serve a portion of Woods Hole, including the scientific institutions. It is now the only sewer in Falmouth. This sewer deposits about a quarter million gallons a day of raw sewage — chopped and chlorinated but otherwise untreated — into Great Harbor in Woods Hole, within a few hundred yards of the sea water intake pipes of the laboratories. So far as can be determined there have been no ill effects.

### Narrative

In 1965 the town began its latest effort to develop a more comprehensive sewer system, in part because of concern about the threat to health and aesthetics in proliferating cesspools, and also because of prodding by the state about the inadequacies of the Woods Hole sewer. A sewer study committee was formed to advise the Board of Public Works, and the Boston engineering firm of Whitman & Howard Inc. was hired to make a preliminary report.

In December 1968, the report was submitted to the public.<sup>1</sup> It proposed a three-stage system of sanitary sewers which would eventually serve all of the built-up areas of town along Nantucket Sound from Woods Hole to Maravista. The sewage would be pumped to a plant in Woods Hole, near Nobska Point, on land owned by the Marine Biological Laboratory. There it would be subjected to primary treatment and secondary treatment by the activated sludge method, before being pumped overboard through an outfall pipe into Woods Hole passage between Juniper Point and Nonamesset Island. The tidal currents run as fast as six knots through the Hole, so rapid dispersion could be expected. As a less attractive alternative, land disposal after identical treatment was suggested at

either of two inland locations. This would have involved the acquisition of much greater land area for settling beds as well as greater expense, both in construction and annual operating costs.

The committee's reaction was prompt, unanimous and negative. The engineers were told that Falmouth's beaches and sparkling water were its lifeblood, and they should go back and consider the inland sites more carefully.

They did so, but reported back eventually with the same recommendations. In the meantime, noting that the scientific institutions had a special concern about the availability of clean water for experiments, Dr. Paul M. Fye, director of the Woods Hole Oceanographic Institution, wrote the town selectmen, and before long an informal committee was set up with representatives of all three laboratories, the engineers and town officials. It is safe to say that from the outset the burden of proof was upon the advocates of the sea outfall.

Very early in the discussion one suggestion was made which was readily adopted by the engineers:<sup>2</sup> to put the outfall off Nobska Point rather than in the Woods Hole passage, so as to take advantage of the much greater flow of water through Vineyard and Nantucket Sounds.

Initially two investigations were made. Three of us, the authors of this article, measured the currents in the vicinity of Nobska Point and estimated the degree of dilution to be expected and the extent to which the proposed outfall might affect the biological cycle of the Sounds.<sup>3</sup> The other study<sup>4</sup> was concerned with the implications of inland disposal and with the impact on the town's fresh water supply that might be expected if the treated effluent were pumped out to sea and not put back into the ground. The work was done principally during the summer and fall of 1969 and the reports were submitted to the town several months before the annual town meeting in March of 1970.

These reports are published in modified form as Sections II and III of this series. In effect they said either inland disposal or a sea outfall appeared to be a reasonable and safe way of disposing the effluent.

Needless to say, both reports were carefully hedged by the authors to avoid any possibility that we would be accused of telling the town what to do. In transmitting the reports to the town, Dr. Fye took the further step of disassociating the Oceanographic Institution from the controversy. He wrote:

"It should be understood that the opinions expressed herein are those of the authors and do not constitute an endorsement by the Woods Hole Oceanographic Institution for or against the disposal of sewage effluent in the marine environment."

The day the Bumpus, Wright and Vaccaro report was released, the Board of Public Works voted to ask town meeting to authorize construction of a sewer, with a sea outfall off Nobska Point at the location we had selected.

The first public comment was that of the local newspaper, which editorialized that it was time to proceed with the sewer now that the scientists have confirmed "what most of us thought all along — that pouring treated sewage into the tide race off Woods Hole will not impair the quality of sea water". (Falmouth [Mass] Enterprise, Dec. 9, 1969.) But dissenting opinions quickly appeared, at first mostly in the form of letters to the editor.

Actually there had been rumblings of disagreement within the scientific community as word of our conclusions got around. Most of the opposition was from biologists, who know well the pitfalls of predicting what will happen in a marine environment. Several members of the biology department at W.H.O.I. prepared what they call an "ancillary report", pointing out the uncertainties and potential hazards. Their report was circulated around the laboratory but never made public, although the points it raised were all well aired, and amplified, during the following months.

As time went by, the length and frequency of the letters to the editor increased. Some were very emotional but most of the letters were thoughtful and dealt with relevant questions.

The principal objections raised to the outfall were:

1. Too little is known about how the ocean responds to a perturbation of this sort to be confident that an outfall will function safely.

2. The proposed outfall might be suitable for the population it is meant to serve, but the Sound could not accommodate the contamination that would result if Falmouth were to grow rapidly or if other towns on the Cape or the Islands were to build similar installations.

3. The town's water table has dropped seriously in recent droughts and might not be able to withstand the continuous loss of 3.7 million gallons per day (the estimated average summer load for the completed sewer).

4. No one knows what unanticipated dangers may turn up in the effluent from a secondary treatment plant. (Later on this objection focused on the problem of viruses.)

5. Woods Hole occupies a position of leadership in the marine sciences in eyes of the world and we should not set an example by dumping our wastes in the sea.

There were very few letters in favor of the outfall. The authors of the reports took no part in the public discussion at first.

In February, a month before the 1970 annual town meeting, we became more active. The sewer study committee belatedly scheduled a public forum at which all of us spoke and answered questions. Dean Bumpus spoke to the Falmouth Rotary Club. Finally, as the town meeting deadline approached, we went in turn to each of the town's five precincts to talk with the town meeting members and other interested citizens. None of us openly advocated either disposal method (even in private we found it a difficult choice) but we did try to

respond to the principal objections which were raised, and we tried to keep the problem in perspective. All these sessions were thoroughly covered by the newspapers. By the time town meeting opened, in other words, there had been ample opportunity for everyone to speak up or ask questions.

The scientific institutions in Woods Hole went separate ways. In a statement issued on February 13, 1970, the trustees of the Marine Biological Laboratory expressed doubts about our conclusions regarding the outfall, suggested more thought be given to advanced treatment and inland disposal, and raised a number of other questions about the wisdom of locating the plant in Woods Hole. In contrast, after Dr. Fye's initial disclaimer, the Oceanographic Institution kept discreetly silent. Although some of the strongest arguments in favor of the outfall were presented at town meeting by high-ranking W.H.O.I. officials, they made it clear that they were speaking for themselves. The National Marine Fisheries Service, as a federal institution, remained completely aloof.

One entire night of the town meeting was devoted to the sewer question. The debate lasted four hours, including one more presentation of our reports. Strong support for the outfall was voiced by several scientists of high standing among both their professional colleagues and their fellow citizens in Falmouth. Strong opposition came from several sources, three of the most effective being the town conservation officer, a much-respected local physician, and the long-time chairman of the town planning board, who is also on the staff at W.H.O.I. He urged that no action be taken until a thorough study could be made of several methods of advanced treatment.

The vote came on the outfall proposal. Because of the expenditure of money involved (about \$8 million), a two-thirds majority was required for passage. The tally was 100 in favor, 86 against, so that the question failed by 24 votes. More than 70 town meeting members either abstained or had gone home to bed before the vote was taken, or had not come to town meeting in the first place.

The next night, when passions had cooled a bit all around, the town voted to ask the sewer committee to look further into the possibilities of land disposal and advanced treatment and to report back with some new proposals.

The indefatigable committee responded before the next special town meeting in October with a third inland site, near Hayway Road (Fig. 1). The possible effects of effluent disposal at this site were considered in a report<sup>5</sup> which is incorporated in Section III of this series of articles. The conclusions were familiar: the site would be acceptable if ground-water monitoring and space for further advanced treatment facilities were included. There was also a section on the potential effect on the water table of putting the treated effluent into the ocean through an outfall. This report was the last of the formal scientific contributions to the sewer discussion.

At a special town meeting on October 15, 1970, there was a choice between the outfall and the Hayway Road site. Residents of the eastern end of

the town provided formidable opposition to the inland alternative, and the Finance Committee recommended postponement of a choice pending further study. On the first ballot both possibilities failed to get the necessary two-thirds majority. However, the outfall had many more supporters than the inland site, and in a new ballot it was carried by a three-to-one vote.

Two weeks later the town learned from bonding counsel that the motion which provided for financing the construction of the sewer was technically improper. This meant another special town meeting to correct the motion, and it gave the outfall opponents their chance. An organization called the Association for the Preservation of Vineyard Sound was formed and began an anti-outfall advertising campaign. The pro-outfall forces also organized a campaign, but much later.

In December, in an effort to cool the rapidly mounting controversy, the Oceanographic Institution made public a memorandum by Bostwick H. Ketchum, Associate Director, which set forth the principal advantages and disadvantages of the various disposal systems under consideration. The basic conclusions were:

1. If cost is of no consideration, the preference would be tertiary treatment with inland disposal, secondary treatment with inland disposal, or secondary treatment with ocean disposal, in that order.
2. However, when properly done with adequate control, any one of the three would be satisfactory.
3. Conflicting preferences should not be used as an excuse to do nothing.

The statement was calm and well-balanced — so well so that both sides quoted it in support of their positions, and the controversy raged on.

The special town meeting was held on January 6, 1971. There was little new argument except for the appearance of state officials who outlined the consequences if the town failed to act. Town meeting members voted 146-55 for the correct motion to finance the outfall, well over the necessary two-thirds, but that was not the end of the matter.

A state statute provides that the action of a representative town meeting can be challenged and a referendum held so that all the voters can pass on the action of their representatives. The procedures are complicated and the requirements stiff, and only three such challenges had been attempted in the 34-year history of representative town meeting in Falmouth. Nevertheless the outfall opponents, who were clearly both well-organized and well-financed, quickly obtained the necessary petitions. The advertising campaigns were stepped up, and when the referendum was held, Feb. 12, about half the town's 8856 voters overturned town meeting and defeated the outfall by a two-to-one vote. It was agreed by both sides that many of the negative votes were from those who did not wish to pay for any sewer system, regardless of location.

A new sewer study committee has been appointed, and the state has initiated proceedings to force Falmouth to build a sewer system. So the story of sewage disposal in Falmouth is far from finished.

### Discussion

Reflecting on the past two years we might ask, first, whether the scientific community accomplished anything at all by injecting itself into the discussion.

The "experts" – if there were any – were those of us who had studied the situation, made observations, worked up the data and reached some conclusions. We were not certain that the sea outfall was better than land disposal, but we were firmly of the conviction that either method would be safe and effective, and that either was better than no action at all.

On that basis we failed, at least temporarily. Falmouth still has no sewer, except the antiquated system in Woods Hole, nor is there any solid ground for confidence that one will be voted in the near future.

However, on several other counts we feel that we certainly helped and that both the study and the time we spent explaining it were well worth while.

We provided a basis for rational discussion of the problem. And by and large the discussion *was* rational, at least until the advertising campaigns were initiated. It was clear that the townspeople were concerned about the outfall and wanted some information. They asked intelligent questions and they listened carefully to the answers.

We enabled the engineers to choose a more suitable outfall site than the one they first proposed, originally by recommending the move from Woods Hole passage to the Sound, and later by pinpointing the optimum location in the Nobska Point area. Similarly, the reports on the inland sites showed which ones presented the greatest threat of contaminating or depleting the fresh water supply.

We learned a great deal, about our own local environment and about techniques that could be used elsewhere. And, incidentally, some of our fellow citizens learned that oceanography can have some practical value.

Perhaps more important, we found out how little we really did know about the physical and biological processes in our own home waters. There were few historical observations of value; we had to obtain our own data on the current system off Nobska Point, and some of our biological calculations were based on data from the Gulf of Maine.

Intentionally or not, we stimulated a good deal of interest in the local marine environment among Woods Hole scientists of various disciplines who had previously been content to make their first observations at the 100-fathom curve.

There are some other pertinent observations.

Many Falmouth residents are concerned about the environment. This was evident from the questioning and the discussion in town meeting, and from the number of people who were pushing for the much more expensive solution of tertiary treatment. In Falmouth we might expect more concern than in some other communities, because our economy is tied so closely and so obviously to clean air and water. But we have no evidence even in Falmouth that people are willing to spend any large amount of money to preserve their environment. Despite all the talk the sewer was defeated.

In a situation involving a public decision about spending public funds, it is a lot easier to kill a proposal than to carry it. Doomsday statements — such as picturing Vineyard Sound as a future Lake Erie — are effective. And we must report that the scientists were no different than the laymen in our town when it came to employing, or being influenced by, such statements. Perhaps it can be argued that this sort of thing is necessary to arouse the public to the perils of pollution. We doubt it, but in any case it does not help the decision-making process at the local level.

Our town is suspicious of “outside” experts and with some reason, historically. The original report by the engineers — outsiders from Boston — was almost rejected outright. The scientific studies were made and presented by four people who had lived in Falmouth for a combined total of more than 60 years, and who had been active in a variety of town affairs: music, sports, scouting and education, as well as politics. We believe that any success we had in explaining our work and our conclusions to our neighbors resulted from their knowing us as human beings like themselves, capable of error of course, but genuinely interested in the well-being of the community and willing to work for it.

In the light of our Falmouth experience, let us now consider some of the general questions posed in our abstract about the role of scientists in the community. We should note that the questions are not new and that they have been asked with increasing frequency, publicly and privately, in recent years.

How can the experts disagree without hopelessly confusing the public? The only answer is that they cannot, and the voters simply have to make the best decision they can on the basis of the information they get, considering the “expert” opinion along with a lot of other factors like economics, politics, local animosities and even lateness of the hour. In this case there were no real “experts” — no one really knew the answers — but there was plenty of strong and conflicting opinion among highly-respected scientists, and public confusion was natural. However, we have seen nothing in the past two years to suggest that the experts, such as they are, are more capable of choosing a reasonable course of action than is the public, even the bewildered public. We retain our confidence in the democratic procedure.

There is also the question of reconciling the need for action with the need for more study. It is always tempting to delay action in order to obtain a few more facts, but anyone who has descended even briefly from the ivory tower

knows that it is almost always necessary to act on the basis of imperfect knowledge. Most scientific papers are written on that basis too. There are situations, of course, that cry out for further study, but in our case we felt that even another year or two of observations would not really resolve the issue — and construction costs are going up at 15 percent a year. It also seems that there is a point beyond which further technical input is useless: no one listens; emotions and politics have taken over and minds are made up. That point was reached in Falmouth shortly after the October town meeting.

We must simply recognize that if we are to become involved in public issues we will have to give up the luxury of further study and do the best we can with what we know. We *can* improve this situation, however, by trying to anticipate problems and by getting started now on the kind of background studies that will enable us a) to make predictions with a little more foundation, and b) to determine, after the fact, what the consequences have been. If our experience is at all typical, this sort of information is sadly lacking.

Finally there is the question of impartiality: do we lose our credentials as humble seekers after truth by taking sides in a controversy? Thimann, in a recent issue of *Science*,<sup>6</sup> says we do. "The politicization of science... is the road to its destruction." He feels we should stick to research and teaching. But Eipper<sup>7</sup>, in the same journal a few weeks earlier argued that we have two obligations: that of the scientist to present the information and interpret it as clearly and fairly as possible, and that of the citizen to speak out and vote on complex issues. We quote from Eipper's article:

"The role of the scientist, as such, should not extend beyond presentation and defense of his estimate of pollution hazards and an assessment of alternatives. He has an obligation to make available information from his profession that will help the voter make a more enlightened decision, but he must scrupulously avoid telling him how to vote. Because decisions on environmental management are so complex, they must represent the best possible reconciliation of many different interests; hence they must be public decisions. The scientist can contribute much to the basis for a public decision, but in making that decision he has only one vote. He is no more entitled — and no more qualified — than any other citizen to elect which of various alternative courses should be followed.

"On the other hand he is no less entitled or qualified to choose. The scientist should not, from fear that his professional identity will give him unfair advantage, shrink from exercising the political rights of a private citizen to express his personal views (so identified) on a controversial issue. Although quite properly concerned about his credibility as a scientist, he should not disregard his credibility as a human being and voter with genuine convictions."

We share Eipper's sentiments and we would add, further, that the scientific ideal of objectivity is another luxury, one which we were allowed in this case only because the issue was not clear-cut. We were not, ourselves, sure which method of disposal was better and we could therefore refrain from speaking out. But suppose our investigations had convinced us that the proposed outfall would blight our swimming beaches and ruin our laboratory aquaria? Would we have presented our evidence and conclusions and then sat back to let the people decide? Of course not. Is there any doubt that the entire scientific community, individuals and institutions, would have jumped into the political process in a vigorous attempt to stop such an outrage?

There has been growing concern about the divergence between science and technology on one hand, and the general public on the other, a growing public disenchantment with the methods and fruits of science and technology. We see in the environmental crisis an opportunity to help reverse the trend. It is one area where the layman can observe the scientist at work on a subject of which the layman has some understanding and with which they are both concerned. If we can establish contact here there is hope that we can bridge the gap in more esoteric areas.

So let us not complain if we are stripped of our former safeguards: the mask of impartiality and the opportunity to duck the issue by further study. Instead there is a better opportunity, to face up to our two-fold obligation as scientists and citizens. We believe that both science and society will benefit.

### Acknowledgment

The authors are grateful to George L. Moses, moderator of the Town of Falmouth, and John Pena, chairman of the Town Board of Public Works, who reviewed the accuracy of the narrative.

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# SEWAGE DISPOSAL IN FALMOUTH, MASSACHUSETTS

## II. PREDICTED EFFECT OF THE PROPOSED OUTFALL<sup>1</sup>

Dean F. Bumpus, Senior Scientist; W. Redwood Wright, Assistant Scientist and Ralph F. Vaccaro, Associate Scientist<sup>2</sup>

### Abstract

The proposed addition of nearly 4 million gallons per day of effluent, from a secondary sewage treatment plant in Falmouth, to Vineyard Sound off Nobska Point is evaluated by means of current measurements, dilution estimates, and potential stimulation of phytoplankton growth. In terms of current usage no impairment of the water quality is indicated.

### Introduction

The Whitman & Howard report<sup>1</sup> to the Town of Falmouth recommended a sea outfall (off Juniper Point, near the entrance to Woods Hole Passage) for its domestic sewage system. From inspection of the U.S. Coast and Geodetic Survey Charts and current data, it appeared likely that an outfall off Nobska Point would offer superior dilution and mixing and would minimize the chances of the effluent being carried into Buzzards Bay, the harbors and estuaries, or onto the beaches of the Town. No place east of Nobska Point along the Falmouth shore offered comparable tidal dilution and dispersion. On this basis, studies were concentrated on the area off Nobska Point (Fig. 1) to answer the following questions.

1. How far offshore should an outfall be located to minimize the return of effluent to the harbors and estuaries and beaches of Falmouth, and to avoid its entrainment in the current through Woods Hole Passage?

2. What dilution of the effluent would be expected as a result of the volumes of water moving past Nobska Point?

3. What effect would be expected upon the ecological cycle of Vineyard and Nantucket Sounds from the added phosphorous, nitrogen and carbon, the principal fertilizing elements in sewage?

Vineyard and Nantucket Sounds form a long shallow embayment lying south of Cape Cod and the Elizabeth Islands and north of Nantucket and Martha's Vineyard, see USC&GS charts 1209 and 1210. They open to the east into the Gulf of Maine, to the west into the Atlantic Ocean, and to the north in the western part through "holes" into Buzzards Bay. Semidiurnal tidal currents sweep forth and back through the sounds at speeds varying from nil (for only a

<sup>1</sup>This is the second of three papers relating to the main subject. It is identified as Contribution No. 2700 of the Woods Hole Oceanographic Institution.

<sup>2</sup>All, Woods Hole Oceanographic Institution, Woods Hole, Mass.

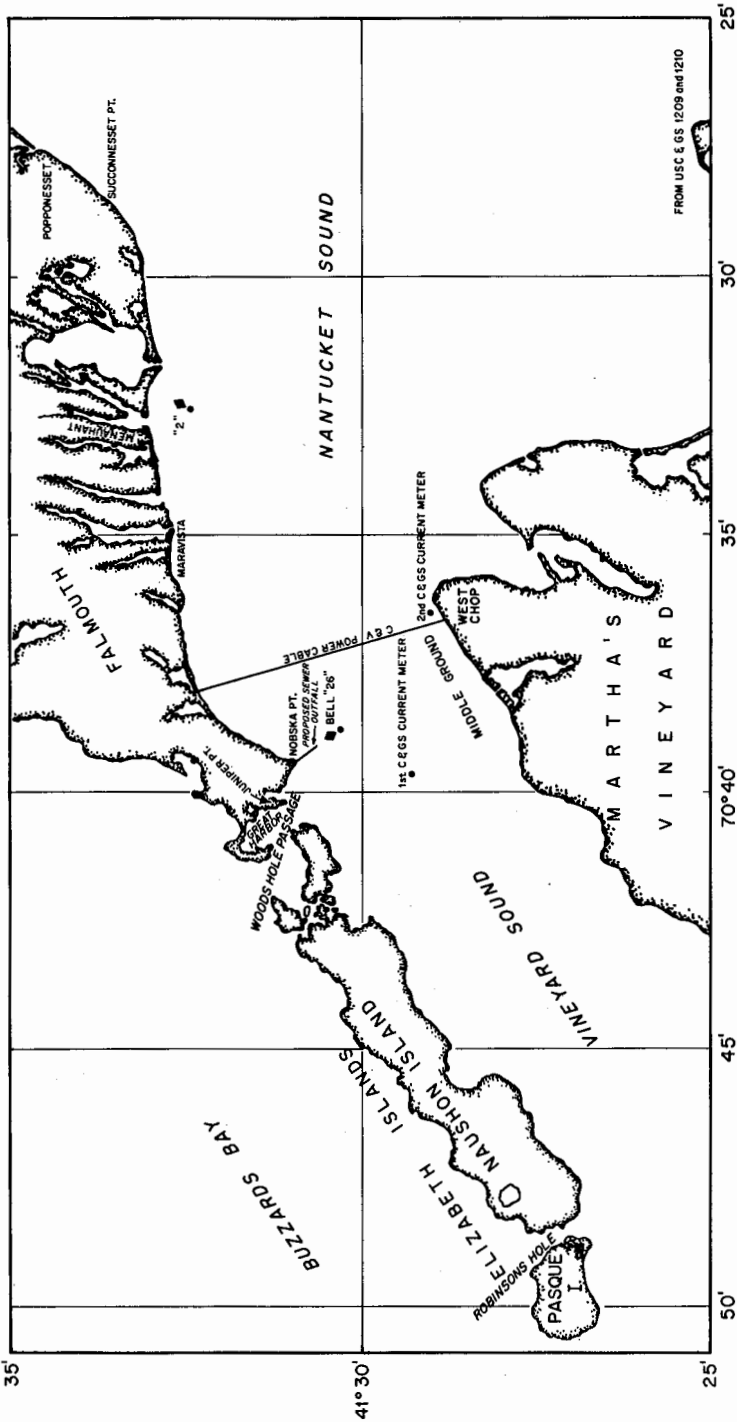


Figure 1. Chart of Vineyard and Nantucket Sounds showing places and objects mentioned in the text.

few minutes) to over three knots.<sup>2</sup> The tidal rise and fall averages about three feet at each end of the sounds but only 1.5 feet near Woods Hole, attributed by Redfield<sup>3</sup> to the interference of the tidal waves entering the sounds from the opposite ends. A further peculiar characteristic is the unequal duration of flow in opposite directions in the Woods Hole Passage. The eastward current duration exceeds the westward current duration by about two hours.

The current flowing easterly immediately off Nobska Point includes a branch from Woods Hole Passage which joins the flow from Vineyard Sound. When the current flows to the west, a portion passes through Woods Hole Passage (and Great Harbor) and the remainder travels down Vineyard Sound. We needed to ascertain the offshore extent of the separation between the Woods Hole flow and the flow to the westward south of the Elizabeth Islands. We wished also to determine the length of the tidal excursion in order to compare the projected volume of treated sewer effluent with the amount of sea water available for mixing during the tidal cycle. It was also desirable to estimate the exchange rate, i.e. how much new sea water is added to the system to replace that leaving the area within a given period of time.

### Data Available

The anticipated average summertime outflow from the proposed secondary treatment plant, including a possible extension to East Falmouth, is estimated by the engineers at 3.7 million gallons per day, which is about 14 thousand  $m^3$ /day or 7 thousand  $m^3$  per tidal cycle.

The proposed system is for domestic sewage only; no storm sewers or industrial wastes are included. Suspended solids and biochemical oxygen demand (BOD) would be 85% to 95% removed. The principal pollutants therefore are the nutrients, phosphorous and nitrogen. Their presence in large quantities could lead to overfertilization of the waters, with harmful effects on the resident plant and animal life. The anticipated concentration of these substances in the effluent are 30 mg/l of phosphorous as phosphate, equivalent to 10 thousand  $mg/m^3$  of  $PO_4\text{-P}$  and 40 thousand  $mg/m^3$  of nitrogen (letter from Whitman & Howard to Bumpus, April 10, 1969). The dissolved carbon content should be about 40 thousand  $gram/m^3$ . The background concentration of these nutrients in the coastal waters adjacent to Vineyard Sound ranges from 25  $mg/m^3$  of  $PO_4\text{-P}$  and 140  $mg/m^3$  of nitrogen in winter to 6.2  $mg/m^3$  of  $PO_4\text{-P}$  and 7  $mg/m^3$  of nitrogen in the summer.<sup>4</sup> The dissolved organic carbon varies from 1000  $mg/m^3$  in the summer to 2300  $mg/m^3$  in the winter. (Menzel, personal communication)

The area and mean-low-water volumes of the two sounds were calculated by dividing USC&GS charts 1209 and 1210 into small rectangles at 2½ minutes of latitude and longitude and then determining the average depth of each rectangle or fraction thereof between lines connecting Gay Head to Cuttyhunk

and Monomoy to Great Point, Nantucket. Using  $70^{\circ}40'W$  as the boundary separating the sounds, the results were:

	Area (km <sup>2</sup> )	Volume (km <sup>3</sup> )
Vineyard Sound	189.2	3.31
Nantucket Sound	<u>1538.4</u>	<u>13.68</u>
Totals	1727.6	16.99

On this basis the average depth at mean low water is 17.5 m (57.4 ft) in Vineyard Sound and 8.9 m (29.2 ft) in Nantucket Sound.

Mangelsdorf<sup>5</sup> measured the flow between Juniper Point and Nonamesset Island with a stationary salt bridge. He found a westward transport of 11 million m<sup>3</sup> per tidal cycle and an eastward transport of 27 million m<sup>3</sup> per tidal cycle, which means a total flow of 38 million m<sup>3</sup> per tidal cycle and a net flow to the eastward, into the Sound, of 16 million m<sup>3</sup> per cycle.

The tidal flow in the sound between Nobska Point and Middle Ground Shoal is about 4000 million m<sup>3</sup> per tidal cycle — 100 times that through the Woods Hole Passage — as estimated from the tidal current and bathymetric charts of the USC&GS. The estimate was refined somewhat with current meter records obtained by the USC&GS over a 15-day period in the summer of 1966. One instrument was at 15 feet in a water depth of 81 feet about 1.6 miles south of Nobska Point. The record showed an average peak velocity of 2.18 knots on the flood (east tide) and 2.36 knots on the ebb. As the velocity curve approximated a sine curve, the mean speed can be estimated at  $2/\pi$  times the peak, or 1.45 knots. Applying this speed to a cross section area of the sound, from Nobska to Middle Ground Shoal, gives a total flow per tidal cycle of 2800 million m<sup>3</sup>. The record also indicated a net westerly flow of 0.13 knots, equivalent to a net westerly transport of about 250 million m<sup>3</sup>/tidal cycle. A second meter, located between Middle Ground and West Chop, showed a net easterly velocity of 0.60 knots. Applying this to the cross section area between Middle Ground and West Chop gives a net easterly transport of 75 million m<sup>3</sup>/tidal cycle in that part of the sound. These results are tabulated in Table 1. It should be recognized that using observations made at one point in a tidal current to determine the transport through an entire section is not very reliable, except to give a gross idea of the magnitude of flow.

### Experiments Conducted

Initially, we set out drogued buoys (ballasted Chlorox bottles with pilot chutes attached with 12 feet of cord) at intervals along a line between Nobska Point and buoy "26" at various phases of the tide, and observed their departures from the points of release by fixing their position from time to time through

Table 1. Vineyard Sound Transport Estimates

Method	Location	Date	Mean Speed (kts)	Cross Section Area (m <sup>2</sup> )	Total Transport per tidal cycle (x 10 <sup>6</sup> m <sup>3</sup> )	Net Speed (kts)	Net Transport per tidal cycle (x 10 <sup>6</sup> m <sup>3</sup> )
US C & GS current meter	41°29'18"N 70°39'38"W	21 Aug-4 Sept 1966	1.45	86,500 <sup>1</sup>	2800	0.13 W	250 W
US C & GS current meter	41°29'00"N 70°36'30"W	26 Jul-9 Aug 1966		5,700 <sup>2</sup>	170 <sup>3</sup>	0.60 E	75 E
Cape & Vineyard <sup>4</sup> Electric Co. cable	Falmouth beach to West Chop	summer - 1969	2	92,000	4200	0.1 E	200 E
Geodyne current meter	41°30'43"N <sup>5</sup> 70°39'05"W	26 Nov-10 Dec 1969	1.15			0.06 E	

<sup>1</sup> Nobska Point to Middle Ground Shoal<sup>2</sup> Middle Ground Shoal to West Chop<sup>3</sup> Calculated by integrating velocity curves<sup>4</sup> See page 260<sup>5</sup> At proposed site for outfall

horizontal sextant angles on known landmarks. Drift bottles and occasionally sea-bed drifters were released with the drogue buoys to help determine residual drift and dispersion. These experiments were conducted on 2 and 9 July and 11 August 1969. We were favored with calm to light winds during all of the above experiments.

Corollary to our experiments with drifting equipment, the use of a spare electric power cable extending from Falmouth shore, near the intersection of Oyster Pond Road and Surf Drive, across the Sound to West Chop on Martha's Vineyard was permitted the Woods Hole Oceanographic Institution, courtesy of the Cape and Vineyard Electric Co. This cable was used to measure the variations in the electric field generated by the tidal currents flowing through that section of Vineyard Sound, after the electromagnetic method of von Arx<sup>6</sup>, Wertheim<sup>7</sup>, and Trites and MacGregor<sup>8</sup>. This information, when integrated, also informs us of the net flow. A summer fellow, Reinhard Flick, under the guidance of Dr. Thomas Sanford, conducted these observations.

An effort to determine the net flow past Nobska Point was conducted in the autumn through the good offices of Paul F. Smith of the Geodyne Corporation and the Woods Hole Oceanographic Institution Buoy Group in which a self-recording meter was moored for two weeks in November-December, 1969.

A brief dye experiment by two graduate students was conducted in August 1970 in order to determine the rate of diffusion at the surface.

## Results

### *1. Flow across a section off Nobska Point at various phases of the tide.*

Diagrams of the results of a dozen sets of drogue releases on a section between Nobska Point and buoy "26" are shown in Figure 2. The diagrams are related to the time of Slack, Flood Begins or Slack, Ebb Begins at Pollock Rip Lightship. Inspection of the diagrams reveals a general tendency for the east flowing current within  $\frac{1}{4}$  mile of Nobska to run nearly parallel with the shore line, whereas drogues released farther off-shore were diverted slightly off-shore by the shoal immediately east of Nobska Point. At certain stages of the west flowing current, drogues released within  $\frac{1}{4}$  mile of Nobska Point were carried into Woods Hole Passage, whereas these released farther off-shore headed westerly past the Elizabeth Islands. Thus to avoid direct flow toward the beaches or into Woods Hole Passage, the outfall should extend more than  $\frac{1}{4}$  mile off Nobska Point.

Normalization of the drogue speed data reveals that the average speed of the current increases with distance off-shore, approaching a plateau at 0.3 miles (Figure 3). In comparing the time when the tide turned at 0.1, 0.2, 0.3 and 0.4 miles off Nobska, it is noted that the west flowing tide turns to the east about  $\frac{1}{2}$



Figure 2. Drift of drogue buoys from the offing of Nobska Point at various times during the tidal cycle, related to tidal time at Pollock Rip Lightship.

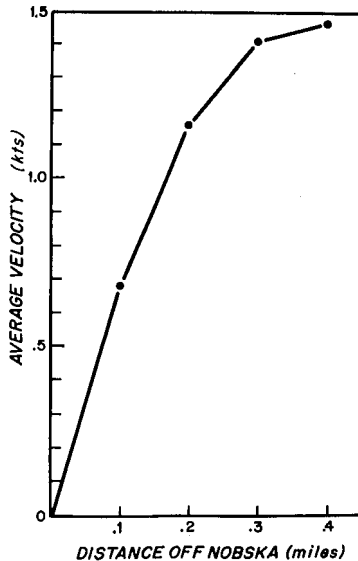


Figure 3. Average velocity of current off Nobska Point relative to distance offshore.

hour later inshore than offshore and that the east flowing tide turns west about an hour earlier inshore (Figure 4). It is further interesting to note that the volume of water flowing eastward increased with distance offshore (Table 2).

Table 2. Comparison of the Relative Volume of East and West Flow at Various Distances off Nobska

Distance off	Volume East	Volume West
0.1 mile	24%	76%
0.2	45%	55%
0.3	53%	47%
0.4	63%	37%

An echo sounder profile of the section from Nobska to Bell "26" (Fig. 5) showed a maximum depth of 28 m (91 ft) about 700 yards offshore. It thus appears that minimum return to shores, harbors and estuaries, coupled with maximum dilution and minimum length of outfall pipe, could be achieved at a distance of 0.375 miles south of Nobska Point (700 yards  $141^{\circ}$  T) from Nobska Lighthouse in 90 feet of water.

## 2. The length of a tidal excursion

Drogues were set out on three occasions at the time the tide was turning from one direction to the other, in order to ascertain how far a parcel of water

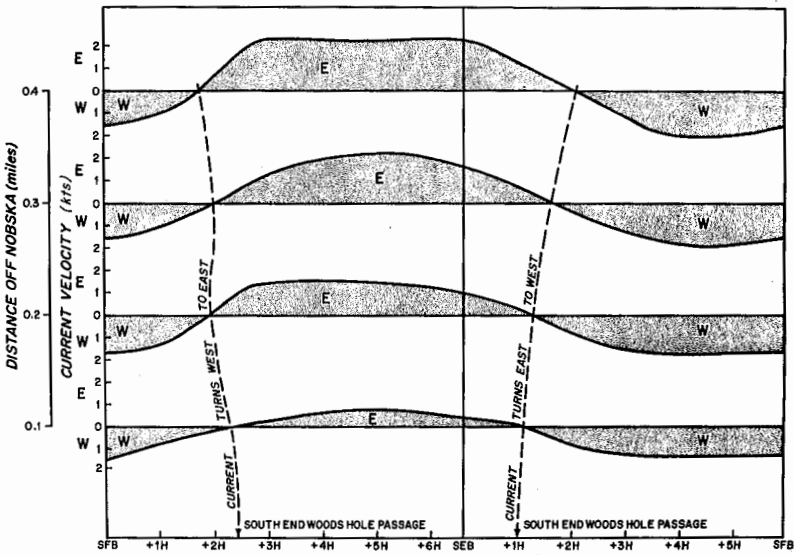


Figure 4. Diagram of the tidal flow at various distances off Nobska Point relative to tidal time at Pollock Rip Lightship.

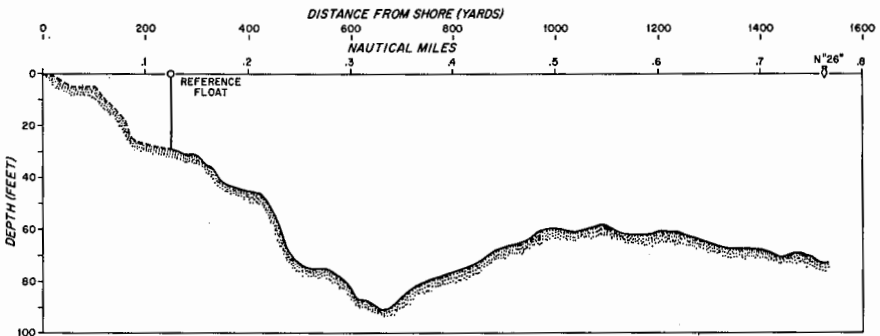


Figure 5. Bottom Profile, Nobska Point to Bell "26" azimuth  $164^{\circ}$ M. Position of reference float  $41^{\circ} 30' 51''$  N;  $70^{\circ} 39' 11''$  W. Depths seaward of reference float from echo-sounder run, 1355 - 1402, 15 July, '69. Depths inshore of reference float from C & GS chart #348.

may travel before it stops and begins to flow in the opposite direction. The experiment on July 2 (Figure 6) commenced shortly after 8 A.M. as the tide was turning east. The drogue released at a distance of 0.05 mile off Nobska Point went aground by the sign under Nobska warning of a Cable Crossing. The drogue launched 0.12 mile offshore tended toward the Falmouth shore and was eventually recovered off Maravista, undoubtedly after it had commenced to move westerly. The other three drogues launched at 0.20, 0.28 and 0.35 miles off Nobska drifted over 8 miles (14.8 km) in six hours to a point about one mile off Succonnet Point. This experiment was conducted at the time of spring tide, hence the distance travelled may have been greater than average.

Similarly, drogues were launched on 9 July at the time of slack water and were followed as they moved westerly. The innermost drogue, launched at a distance of 0.25 mile off Nobska, eventually grounded off Witches Glen on Naushon, over 4 miles from its origin. The remainder, launched at 0.34, 0.55 and 0.66 miles off Nobska, travelled in company almost to Robinson's Hole, a distance of over 7 miles (13 km) in six hours.

A third excursion experiment was conducted on 11 August in which drogues were released in the offing of Bell "2" off Menauhant, just before the tide turned west, and were tracked for nearly six hours, actually until they had ceased westerly progress. This excursion was 4.5 miles (8.3 km) west.

The interesting points to note from this set of excursion experiments are:

- a. Excursions from Nobska are on the order of 7-8 miles (13-14 km);
- b. Return excursion from off Menauhant toward Nobska was substantially reduced, suggesting the net flow is toward the east.
- c. With the exception of those drogues launched inshore of 0.25 mile off Nobska, all drogues drifted along together, being only slightly farther apart after 6 hours than they were initially.

### 3. *Drift Bottles*

Drift bottles were released with each drogue, and in some instances farther offshore out to buoy "26", in order to gain some insight into the extension of the drift beyond that defined by the drogue experiments. The drift bottles moved at a rate and direction comparable with the drogues. Of the 416 bottles released, 275 or 66% were recovered between West Dennis to the east and Cape Hatteras to the west. Those recovered were about equally distributed between east of the point of release along the north side of the Sound (32%), westward into Vineyard Sound to Rhode Island, New York and North Carolina (37%), and northward into Buzzards Bay (29%). Only 1% were recovered from Martha's Vineyard. When we break down the recoveries, relative to how far offshore they were launched and in what direction they eventually stranded (Table 3), we see that a high percentage of those released within  $\frac{1}{4}$  mile off Nobska Point entered Buzzards Bay where the currents are weak; whereas beyond  $\frac{1}{4}$  mile off Nobska the flow to Buzzards Bay was 25% or less.

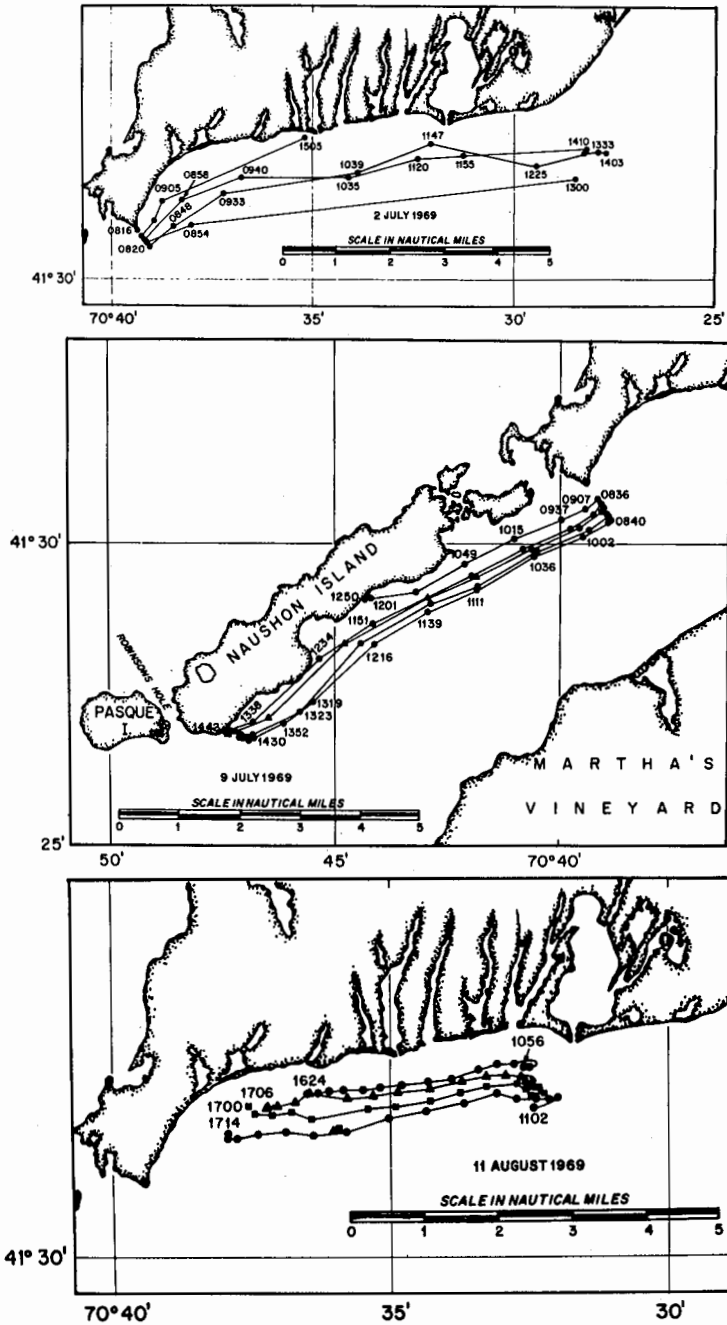


Figure 6. Drift of drogue buoys during the tidal excursion experiments.

Table 3. Percentage of Recovered Drift Bottles which Stranded Northerly, Easterly or Westerly Relative to Distance of Release off Nobska Point

Distance Offshore	N	E	W
0.00 – 0.25 mile	40	25	35
0.25 – 0.35	22	38	38
0.35 – 0.50	25	36	39
> 0.50	20	39	38

These returns show that, of those launched within  $\frac{1}{4}$  mile off Nobska, 50% were recovered just over 4 miles from their point of release, whereas of those launched more than  $\frac{1}{4}$  mile off Nobska, 50% were recovered within 6 miles of their origin. However, in view of our observations that bottles and drogues appear to drift together, we do not believe this means the bottles drifted directly to the beach where they stranded. Instead, they probably drifted back and forth for several tidal cycles. The north shore of the sound, east of Nobska, is a sandy shoreline well peopled in June, July and August; hence, there should be little or no delay in their recovery. Sixty-eight bottles were recovered here between 1 and 49 days after their release, of which 50% were recovered within 5 days. In contrast, the Elizabeth Islands present only patches of beach here and there, and as a whole are more sparsely peopled. Fifty-three bottles were found along the shore of the Elizabeth Islands between 0 and 65 days, of which 50% were recovered within 10 days.

#### 4. *Sea-Bed Drifters*

A sea-bed drifter is a "drift bottle" designed to drift along the bottom. It is a plastic disc with a 2-foot stem ballasted with a brass ferrule so it has slight negative buoyancy in sea water.<sup>9</sup> These were distributed at intervals along the section between Nobska Point and buoy "26". Of the 140 sea-bed drifters released, 45 were recovered; 40 drifted eastward, one went 2 miles to the west, one to Little Harbor, two to Wood Neck Beach in Sippewisset and one to Martha's Vineyard. Of those which went easterly, many stranded at Popponneset. The average distance of strand was 7.8 miles from Nobska in 19 days. There were no significant differences in drift related to the off-shore distance of release.

#### 5. *Net Flow as Determined by Geomagnetic Electrokinetograph*

The geomagnetic electrokinetograph measurements of Sanford and Flick during the summer of 1969 showed an easterly flow of 2200 million  $m^3$  on the flood and a westerly flow of 2000 million  $m^3$  on the ebb, for a net easterly flow of 200 million  $m^3$  per tidal cycle, through the section from Surf Drive, Falmouth, to West Chop, Martha's Vineyard, in reasonable agreement with the results of the current meter calculations. The mean speed across the section was 2.0 knots.

### 6. *Current Meter Record at Proposed Outfall Location*

A self-recording current meter<sup>10</sup> was moored at a depth of 30 feet in 90 feet of water at 41° 30' 43" N, 70° 39' 05" W (at the point of maximum depth between Nobska Point and Bell "26") for a 14-day interval, 26 November to 10 December 1969. The current ranged from 0.5 to 149.5 cm/sec (0 – 2.8 kt) with an average speed of 60 cm/sec (1.15 kt). The progressive vector diagram (Fig. 7) shows tidal excursions on the order of 13 km (7 miles) comparable to the evidence from the drogue experiments. The easterly current flowed toward 060° T at an average speed of 62 cm/sec for 6.28 hours, the westerly current flowed toward 234° T at an average speed of 45 cm/sec for 6.09 hours. The net flow was easterly at a rate of 1.45 km per tidal cycle, roughly 5% of the total flow, in good agreement with the 5% net easterly flow evidenced by the geomagnetic electrokinetograph measurements.

### 7. *Dye Experiments*

Two brief dye experiments were conducted in the summer of 1970 by two graduate students in the Woods Hole Oceanographic Institution–Massachusetts Institute of Technology Joint Program and were described in a typewritten report.<sup>11</sup> The experiments ran for about four hours each on both easterly and westerly tides. In each case Rhodamine B dye diluted to a specific gravity of 0.965 was released at the sea surface above the proposed outfall point. The spreading dye was photographed from the air and its concentration was determined in several samples taken by a small boat criss-crossing the plume. In addition, drogues were set out at various depths and tracked for the duration of the experiments.

Although too short to be conclusive, the dye experiments generally corroborated the results of the 1969 studies. For example, the drogues all stayed together and, on the easterly tide, both drogues and dye streak were diverted offshore by a shoal just east of the outfall point. The principal differences from the earlier experiments were (1) on the westward tide, with a southeast wind blowing, some of the dye came very close to the shore on Nonamesset Island and was carried into Woods Hole Passage, although most of it went down Vineyard Sound; and (2) the dye showed a good deal more horizontal dispersion, approximately 1 square nautical mile/hour (3.3 km<sup>2</sup>/hr), than was evident in the drogue experiments.

## **Dilution and Dispersion**

The degree of contamination to be expected from the sewage plant effluent cannot be estimated to better than an order of magnitude because of uncertainties about the degree of mixing with the waters of the sounds and the net transport of water through the sounds. It was concluded early in the

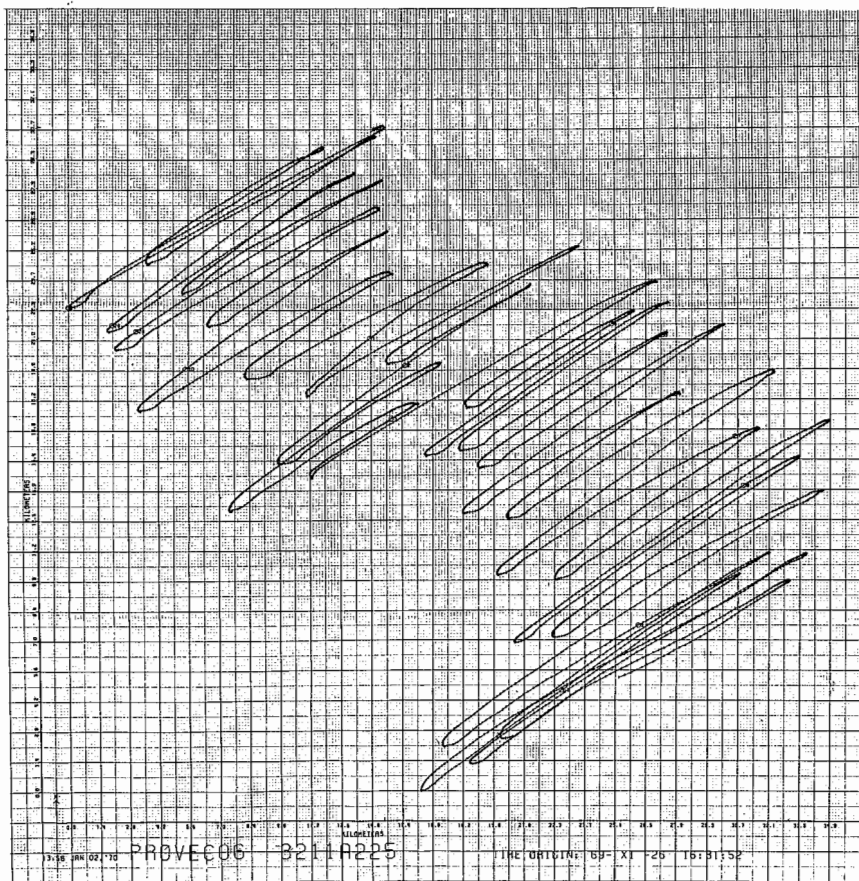


Figure 7. Progressive vector diagram of current meter data, 26 November to 10 December 1969 at  $41^{\circ} 30' 43''$ N;  $70^{\circ} 39' 05''$ W at 30' in 90' of water.

investigation that it would be fruitless to try to apply any refined theory to an area of strong reversing tidal currents. Instead, the emphasis has been on estimates based on the above measurements. We have tried to use conservative figures so that our results would be on the pessimistic side.

There are two fundamental and interrelated problems involved in predicting the concentration of pollutants from the sewer outfall: the rate of flushing and the mixing characteristics. The data in the previous sections permit calculation of the flushing rate by the methods of tidal exchange and net flow. Both methods assume complete mixing within the volume affected, and the first assumes, in addition, that the exchange of contaminated water for uncontaminated water at each change of the tide is a known quantity. Neither assumption is valid, of course, but reasonable approximations can presumably be made.

There is evidence of some exchange in the drift bottle results. More than 28% of all the returns were from outside the sounds, either to the north in Buzzards Bay or to the west. If we consider only those bottles launched more than  $\frac{1}{4}$  mile offshore, 25% of the recoveries were from outside the system and a third of those were from the west. Several tidal cycles are required, of course, to transport a float west from Nobska Point past Cuttyhunk, out of the sounds; the earliest return from beyond Cuttyhunk was 9 days.

Nearly complete vertical mixing, whether caused by wind stirring or by rapid flow over an undulating sea floor, can be assumed with some confidence; among the hundreds of bathythermograms in the W.H.O.I. files on the two sounds, only a handful of mid-summer observations show any thermal stratification. And, although the effluent will be warmer and less saline than the waters of the sounds, and hence, less dense, it will be introduced at the bottom of the sound, with downward diffusers on the outlet to encourage mixing, into water which is moving rapidly almost all the time. On the other hand, there is very little evidence of horizontal mixing except as provided by the dye experiments. Very little separation of drogues was observed on the three tidal excursion runs, except along the axis of flow at the beginning and end of the tidal cycle, when the inshore drogues changed direction earliest. The lack of vertical shear is shown by the surface drift bottles moving right along with the drogues at 12 feet depth. In one experiment comparing drogues at the surface and at 30 feet, the shallow drogues outdistanced the deep ones but only by about ten percent. Finally, the recovery of only 1 percent of the drift bottles on Martha's Vineyard argues that water flowing past Nobska keeps to the northern part of the sounds.

To allow for these mixing conditions it will be assumed in the following discussion that the mixing of effluent is restricted to one-tenth of the total volume of the sounds, and that one-tenth of the water entering the system on a change of tide is "new" water. We think these are conservative estimates; in combination they will predict concentrations 100 times greater than if mixing through the entire volume and 100% "new" water were assumed.

### 1. Tidal Exchange

The tidal exchange method is described by Ketchum<sup>12</sup> and applied in combination with tidal excursion data in Ketchum<sup>13</sup> for estuaries with river flow. He developed an expression for the quantity  $Q$  of river flow present in a given segment of an estuary after many tidal cycles, assuming complete mixing within the segment, as

$$Q = R/r$$

where  $R$  is the volume of river water introduced in a tidal cycle and  $r$  is the ratio between the intertidal volume (tidal prism) and the high tide volume of the segment. Because of the complications of large tidal excursions we will simply consider the entire sound as one segment.

$Q$  becomes the amount of effluent in the sound after a steady state is reached.  $R$  is the effluent introduced during a tidal cycle (7 thousand cubic meters) and  $r = MP/(P+V)$  where  $M$  is the fraction of new water in the entering tide (0.1),  $P$  is the volume of the tidal prism and  $V$  is the mean low water volume of the system (17 cubic kilometers). Assuming a mean tidal rise of 0.7 m,  $P$  is 1.2 cubic kilometers, so that

$$r = \frac{0.1 \times 1.2 \text{ km}^3}{1.2 \text{ km}^3 + 17 \text{ km}^3} = \frac{1}{151}$$

$Q$  then equals  $151 R = 1057 \times 10^3 \text{ m}^3$ , or approximately one million cubic meters of effluent. If this quantity is mixed with 10% of the total volume of the sound, the dilution will be

$$\frac{1.7 \times 10^9 \text{ m}^3}{1 \times 10^6 \text{ m}^3} = 1700 \text{ sea water to 1 effluent}$$

The reciprocal of  $r$ , 151 cycles, is equivalent to residence time so it can be said that the effect of any unusual event, even discharge of untreated sewage, would be dissipated in less than three months.

### 2. Net Flow

A better estimate is probably that based on net flow, which represents the amount of uncontaminated water entering the system on each tidal cycle. When a steady state is reached, the ratio of sea water to effluent in the contaminated volume of the sound will be the same as the ratio of net flow per tidal cycle to the effluent introduced in a tidal cycle.

The net flow according to the cable measurements is about 200 million  $\text{m}^3$  per cycle. If we apply the 10% assumption as before, we find that 7000  $\text{m}^3$  of effluent mixed with 20 million  $\text{m}^3$  of sea water has a dilution of about 2850 to 1.

### 3. *Immediate Mixing*

The foregoing applies to the general dispersion of effluent throughout the affected body of water, but the amount of contamination will, of course, be greater near the outfall.

We will assume that the sewage effluent is lighter than sea water and will be mixed by entrainment throughout the whole water column as it rises.

The absence of reliable information on mixing characteristics and the added complications of tidal fluctuations make it impossible to predict the effect accurately, but a minimum dilution figure can be established as follows:

Assume the effluent comes out of a diffuser 10 meters long at a depth of 28 meters at a rate of  $14 \times 10^3 \text{ m}^3$  per day. The current meter data, at the prospective outfall site, give evidence that the tidal flow past the outfall location amounts to 52 kilometers per day. Multiplying this rate of flow by the cross sectional area ( $10 \text{ m} \times 28 \text{ m} = 28 \text{ m}^2$ ) =  $14.6 \times 10^6 \text{ m}^3$  of sea water to mix with the  $14 \times 10^3 \text{ m}^3$  of effluent, producing an initial dilution of 1000  $\text{m}^3$  of sea water for each cubic meter of effluent.

### 4. *Horizontal Diffusion*

Let us take the 1000 to 1 figure as a minimum estimate for immediate mixing as no allowance has been made for spreading of the effluent plume as it mixes upward through the water column. More important is horizontal diffusion as the effluent will be further diluted with two to ten times as much sea water within a few hours of discharge.

For example, the equations developed by Brooks<sup>14</sup> indicate that the contamination will spread to about three times its original width and the original dilution will be doubled by the time the effluent has been carried 1 km from the outfall. This sort of spreading was indicated by the dye experiments off Nobska Point, although quantitative data are not available.

Bowles et al <sup>15</sup> observed a continuous dye release in the English Channel in an area where the tidal heights and currents are somewhat greater than in our sounds. They found a minimum dilution of 4000 to 1 at slack water near the source. They also found a transverse eddy coefficient (k) of  $10^4 \text{ cm}^2/\text{sec}$ , and observed that the dye concentrations decreased downstream in inverse proportion to the square root of the distance. Their factor of proportionality, if applied to the Falmouth sewage effluent, would give a minimum dilution of 6000 to 1 a kilometer from the outfall. At one-quarter km from the source the concentration would be 3000 to 1; at 4 km it would be 12000 to 1.

Both the eddy coefficient of around  $10^4 \text{ cm}^2/\text{sec}$  and the inverse relationship with the square root of the distance are supported by other investigators. Ketchum and Ford<sup>16</sup> obtained k values from  $2 \times 10^3$  to  $7 \times 10^3 \text{ cm}^2/\text{sec}$  for pollutant discharges in the wake of a barge. Whipple<sup>17</sup> argues that the eddy coefficient in coastal waters should be of the order of  $UD$  where  $U$  is

the tidal speed and  $D$  is the depth, which would give  $k$  of  $10^5$   $\text{cm}^2/\text{sec}$  for a depth of 15 m and a speed of 1.15 knots. He also found that the plume width doubled in less than a kilometer and the dilution was quadrupled in 4 km. Bowden<sup>18</sup> tabulated  $k$  values ranging from  $10^4$  to  $10^6$   $\text{cm}^2/\text{sec}$  in English coastal waters.

The eddy coefficient can be used with the following expression developed by Reid<sup>19</sup>:

For the minimum dilution  $D_{\min}$  at a distance  $x$  downstream from a continuous point source  $S$  at a depth  $H$  in a uniform current of speed  $U$ :

$$D_{\min} = \frac{2H\sqrt{2\pi kUx}}{S}$$

Thomas<sup>20</sup> uses the same expression without the factor 2 under the radical. For  $U = 60$   $\text{cm}/\text{sec}$ ,  $H = 15$  m and  $k = 10^4$   $\text{cm}^2/\text{sec}$ , Reid's expression gives a minimum dilution of more than 10000 to 1 at 1 km from the outfall.

To summarize the calculations above: initial mixing would be at least 1000:1; assuming the net flow through 10% of the cross section, the dilution would be 2850:1; or assuming the steady state tidal prism method the dilution would be 1700:1. Horizontal diffusion and mixing should increase the dilution several fold. For the calculation of the effect of nutrients on plant production in the next section we have chosen the conservative steady state dilution of 1700:1.

### Effects of Nutrients on Plant Production

The effects of a sewer outfall on the ecology of the sounds will depend ultimately on the magnitude of change in those factors which regulate the biochemical cycle in those waters. Usually, the effects of both organic substances which are decomposed by bacterial activity and inorganic chemicals which stimulate the growth of marine plants must be considered. Here, however, the addition of plant nutrients would appear to be the more relevant consideration providing the outmoded Woods Hole outfall becomes inoperative. Calculations by Whitman & Howard predict an overall decrease in organic loading of the neighboring waters once effective secondary treatment is established.

All treated sewage effluents characteristically contain significant concentrations of phosphorus and nitrogen which can ultimately influence standing crops at all levels of the food chain. When excessive, such fertilization leads to an over-abundance of organic production having undesirable effects on water quality. On the other hand, a shortage of these same elements frequently limits the productivity of coastal waters.

The engineers have estimated that each cubic meter of effluent from the proposed plant will contain 10 grams of phosphorus, 40 grams of nitrogen and 40 grams of dissolved organic carbon. With a minimum dilution of 1700 to 1, the final concentrations to be added to those naturally present should not exceed 6.0 mg of phosphorus, 25 mg of nitrogen or 25 mg of carbon per cubic meter. We have studied the effect of this enrichment increase by evaluating its effect on the marine food chain at the primary production level, assuming complete utilization. The predicted changes in production are then compared to the natural biochemical variations known to occur under existing conditions.

Typically, the amounts of phosphorus and nitrogen in a representative phytoplankton cell vary according to cell size and the available concentration of nutrient elements outside the cell. Table 4 compares the elementary composition (in terms of phosphorus, nitrogen and carbon) for a small specie (*Cyclotella nana*) and a very large specie (*Thalassiosira fluviatilis*). From this information, the projected increase in cell numbers corresponding to a known increase in the concentration of a given nutrient element can be calculated as:

$$N = \frac{\Delta C}{C_1}$$

where N is the increase in cell numbers,  $\Delta C$  the external change in nutrient concentration and  $C_1$  the elementary content of an average cell. Thus the anticipated increase in phytoplankton numbers corresponding to a phosphorus and nitrogen increase of 6 and 25 mg per cubic meter respectively would be expected to range between 2 and 12 billion cells per cubic meter. How this increase compares to the normal population is included in the data shown in Table 5.

Table 4. Carbon, Nitrogen and Phosphorus Contents of Small and Large Marine Phytoplankters in Growing Culture<sup>1</sup>

	Mean Cell Volume ( $10^{12}$ cm <sup>3</sup> )	Elementary Composition (mg x $10^{-8}$ per cell)		
		Phosphorus	Nitrogen	Carbon
<i>Cyclotella nana</i>	34	0.04-0.06	0.40-0.60	1.00-4.00
<i>Thalassiosira fluviatilis</i>	4000	0.20-0.40	2.00-4.00	10.0-20.00

<sup>1</sup>Personal communication, Dr. Louis Hobson of the Woods Hole Oceanographic Institution.

To evaluate other parameters useful to this analysis we have used certain previously established relations descriptive to the physiology and growth of marine phytoplankton. August studies in the Gulf of Maine<sup>4</sup> have shown that

Table 5. Maximum Variations<sup>1</sup> in Some Biochemical Parameters in Vineyard Sound and Predicted Changes Following Outfall Installation

	Pre-outfall		Potential Outfall Effluent Effect	Post-outfall Totals	
	Max	Min		Max	Min
Dissolved constituents mg/m <sup>3</sup>					
Phosphorus	25	6	6	31	12
Nitrogen	140	7	25	165	32
Organic carbon	2300	1000	25	2325	1025
Particulate constituents mg/m <sup>3</sup>					
Organic carbon	13690	200	(207) <sup>2</sup>	13897	407
Nitrogen	1650	20	25	1675	45
Phosphorus	(183) <sup>3</sup>	(4) <sup>4</sup>	6	189	10
Chlorophyll A	125	5	(3.0) <sup>2</sup>	128	8
Phytoplankton cell counts cells/m <sup>3</sup>	25 x 10 <sup>9</sup>	1.0 x 10 <sup>9</sup>			
Phytoplankton estimates cells/m <sup>3</sup>					
large cells			2 x 10 <sup>9</sup>	27 x 10 <sup>9</sup>	3 x 10 <sup>9</sup>
small cells			12 x 10 <sup>9</sup>	37 x 10 <sup>9</sup>	13 x 10 <sup>9</sup>
Dissolved oxygen as oxygen mg/m <sup>3</sup>	10000	7200	-654 <sup>5</sup>	9346	6546
ml/l	7.0	5.0	-0.46 <sup>5</sup>	6.54	4.54

<sup>1</sup> Analysis compiled by Department of Marine Biology, W.H.O.I.

<sup>2</sup> Assuming all nitrogen and phosphorus from effluent is converted to plant cells at a C : N : P ratio of 8.3 : 1.0 : 0.12

<sup>3</sup> Estimated from N : P ratio of 9 : 1, see text for April conditions in Gulf of Maine

<sup>4</sup> Estimated from N : P ratio of 5 : 1, see text for August conditions in Gulf of Maine

<sup>5</sup> Assumes complete oxidation of effluent organic carbon (25 mg/m<sup>3</sup>) plus the expected increase in particulate carbon (207 mg/m<sup>3</sup>) and allows 2.82 milligrams of oxygen for each milligram of carbon oxidized.

when the supply of plant nutrients in the surface water is at the annual minimum, the ratio (by weight) in the phytoplankton of nitrogen:phosphorus approximates 5:1. In April, however, with an excess of plant nutrients available, the same relation increases to about 9:1. Comparable observations from Vineyard Sound when the standing crop of phytoplankton was at the annual maximum showed that the ratio of carbon:nitrogen corresponds to 8.3:1. In combination with other direct analysis on the dissolved and suspended matter in Vineyard Sound water, the above ratios have been used to develop the information shown in Table 5 where the additive effect attributable to a functional outfall is superimposed on the seasonal conditions which now prevail in these waters.

It can be seen in this table that the amount of phosphorus added by the proposed sewer represents a doubling of the natural summer concentration and about a 25 percent increase in the winter background level. For nitrogen the corresponding increase would be about 5 times the summer value and 18 percent over the winter value; the dissolved organic carbon changes would be negligible during both seasons.

The table indicates that these additions will have an effect on plant production in the sounds which will not exceed the normal seasonal variation. Any measurable increase in fertility would appear restricted to the summer months when the levels of standing crop are already depressed due to a lack of sufficient plant nutrients. Similarly, no untoward oxygen depletion is foreseen, the predicted maximum annual variation being from 4.54 to 6.54 ml/l (6500 and 9300 mg/m<sup>3</sup>) respectively for the summer and winter months. During the more productive seasons of the year these calculated changes can be expected to become progressively less significant in terms of the normal background situation until such time as the annual maximum in standing crop (spring bloom) begins to recede.

It should be emphasized that the above analysis presupposes an effluent entirely composed of domestic sewage which has undergone effective secondary treatment. Excessive amounts of organic carbon or toxic substances, such as heavy metals and nondegradable hydrocarbons which can be concentrated in marine organisms could cause deleterious effects and a significant departure from these predictions.

## Conclusions

If the town should elect to dispose of its sewage effluent in the marine environment, a location off Nobska Point would be preferable to a proposed location off Juniper Point. The data indicate that the sewer outfall should be extended to a location 0.375 miles from Nobska Point where minimum return of the effluent to the beaches, harbors and estuaries of Falmouth would be

achieved, where the tidal flow reaches maximum speeds, and where the depth of water is greatest to provide for maximum immediate dilution.

A steady state condition would be reached in 75 days when the sewage effluent would be diluted by a volume ranging from at least 1700-2800 parts of sea water for each part of effluent. Slightly higher concentrations could be expected in the immediate vicinity of the outfall pipe.

The concentrations of nutrients added to the sounds and ecological effects would be less than is experienced in the normal annual cycle.

There would be no impairment of the quality of the water.

### **Acknowledgement**

We acknowledge with thanks advice and criticism from A.C. Redfield and B.H. Ketchum. This research was supported in part by N.I.H. Grant EC 00178.

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# SEWAGE DISPOSAL IN FALMOUTH, MASSACHUSETTS

## III. PREDICTED EFFECTS OF INLAND DISPOSAL AND SEA OUTFALL ON GROUNDWATER<sup>1</sup>

Robert H. Meade<sup>2</sup> and Ralph F. Vaccaro<sup>3</sup>

### Abstract

Both inland and offshore disposal of 3.7 mgd of secondary-treated sewage effluent would affect the groundwater in the Town of Falmouth. Inland disposal would probably increase the concentrations of nitrogen and other dissolved constituents in groundwater downstream of disposal sites, and it would alter slightly the existing patterns of groundwater flow. The seriousness of these effects would vary according to the location of the disposal site relative to present and future sources of water. Disposal through a sea outfall would ultimately lower the levels of some fresh water ponds by several feet.

### Introduction

The Town of Falmouth, in the southwest corner of Cape Cod, has been considering alternative methods of disposing of 3.7 million gallons per day (mgd) of secondary-treated municipal sewage effluent. The two alternative methods that were proposed to the Town by its consulting engineers in 1968-1970 would provide ultimate disposal (1) through an outfall pipe into the ocean, and (2) through sand filter beds into the ground. As a complement to the studies of the effects of the proposed sea outfall that were reported in the preceding paper of this series,<sup>1</sup> we studied the groundwater conditions in Falmouth. The results of this study and recommendations were submitted to the Town of Falmouth in 1970 in two informal reports,<sup>2-3</sup> which are combined (and somewhat revised and amplified) in this presentation.

The proposed inland disposal sites are labeled I, II, and III on Figure 1. Details of the sea outfall and its location are summarized in the preceding paper. Our predictions of the effects of inland disposal concerned both the influence on water quality and the resulting patterns of recharge and groundwater flow with respect to present and future sources of water supply. Our predictions of the effects of the sea outfall concern its influence on the water supply and groundwater levels. As emphasized in the first paper in this series,<sup>4</sup> we confined our studies largely to assessing the environmental effects of the courses of action

<sup>1</sup>This is the third of three papers relating to the main subject. It is identified as Contribution No. 2701 of the Woods Hole Oceanographic Institution.

<sup>2</sup>Woods Hole, Massachusetts

<sup>3</sup>Woods Hole Oceanographic Institution, Woods Hole, Massachusetts

proposed by the Town's consulting engineers, and we did not attempt to recommend fundamental changes in the design of the sewage-treatment and sewage-disposal works.

## Groundwater in Falmouth

### *Geologic Setting*

The geology of Cape Cod and the Falmouth area has been described in several reports.<sup>5-6-7</sup> For purposes of selecting inland sewage-disposal sites, the geologic setting of Falmouth can be divided conveniently into three areas: moraine, outwash, and mixed moraine-outwash-marsh. The following descriptions are taken mainly from Mather, Goldthwait, and Thiesmeyer.<sup>6</sup>

The Buzzards Bay moraine forms a band 1 to 1½ miles wide that passes through western Falmouth from Woods Hole to the Bourne town line (Figure 1). At the surface it consists of heterogeneous and discontinuous associations of silty clay, hardpan, glacial till, clean sand and gravel, and large boulders up to 20 feet in diameter. The limited amount of information available from a few borings and deep pits suggests that, below the first few tens of feet of heterogeneous surficial material, the moraine consists mainly of sand and gravel. However, the spatial arrangements of the different types of material in the moraine are unpredictable on an *a priori* basis, and any site on the moraine should be subjected to an intensive survey by closely spaced borings before it is selected for sewage disposal.

East of the Buzzards Bay moraine lies the outwash plain that covers most of the area of Falmouth. Outwash material consists mainly of loose and permeable stratified sand and gravel. Gravel, containing boulders that are usually less than one foot in diameter, is present along the western edge of the outwash within a mile or so of the contact with the moraine. Farther east, the outwash material is essentially all sand. With the exception of Falmouth Heights and a few small localized areas of Teaticket, the subsoil in the outwash area can be considered sandy, permeable, and of good quality for rapid percolation. While any potential sites on the outwash should be surveyed by boreholes before being selected, disposal sites can be proposed in the outwash with a fair degree of confidence that their subsoil characteristics will prove to be suitable.

The parts of West and North Falmouth that are west of the Buzzards Bay moraine are underlain by isolated areas of morainal material that are separated by areas of outwash and coastal marsh. Subsoil characteristics of possible disposal sites west of the Buzzards Bay moraine, therefore, are as unpredictable and require the same intensity of investigation as those beneath sites on the moraine itself.

Information on the spatial distribution and composition of the deeper subsoils that lie below Falmouth is limited. Many wells and a few test borings

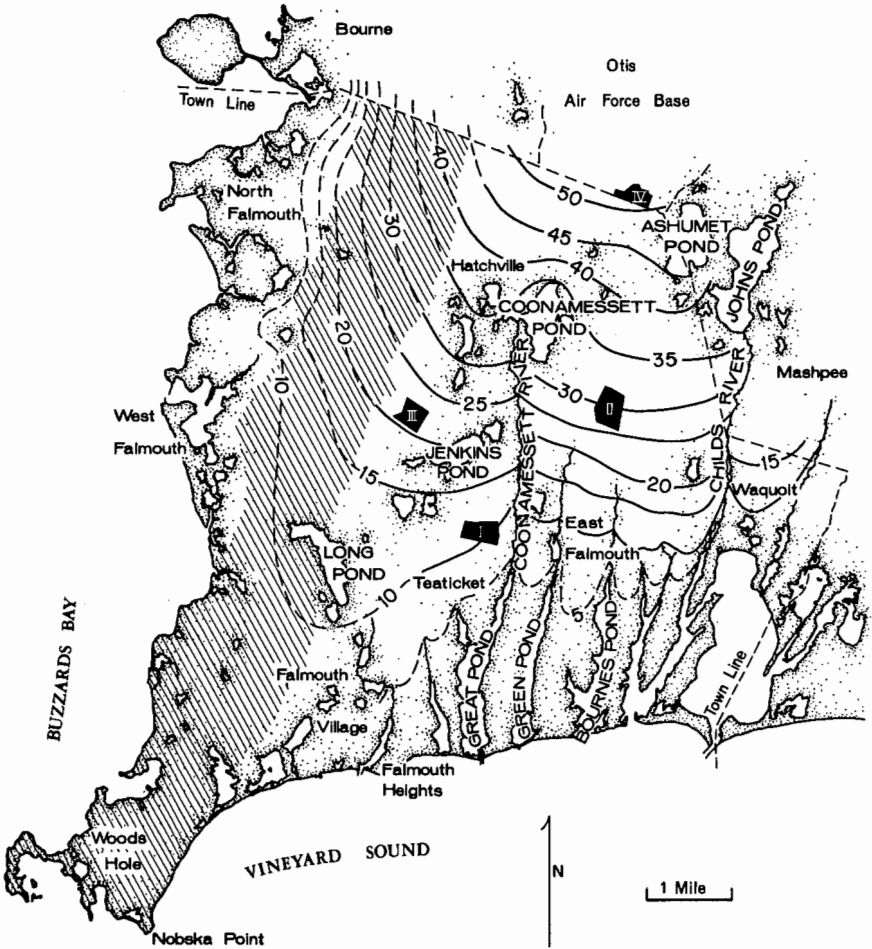


Figure 1. Map of Town of Falmouth, Cape Cod, Massachusetts, showing locations of constituent villages and pertinent geologic and hydrologic features. Outcrop area of Buzzards Bay moraine (from Mather *et al*<sup>6</sup>) shown by cross hatching. Contours show elevation of water table, in feet above mean sea level, in late spring in a year of normal rainfall (June 1970); contours dashed where uncertain—particularly uncertain is location of 10-ft contour between Long Pond and Woods Hole. Inland sewage disposal sites indicated in solid black: proposed sites at Brick Kiln Road (I), Hayway Road (II), and Blacksmith Shop Road (III); site of existing disposal plant at Otis Air Force Base (IV).

have been driven or drilled, especially in the outwash area, to depths a few feet below the water table. Seismic surveys have shown that the surface of bedrock under the town ranges from about 100 feet below sea level in the northern part to about 300 feet in the southeastern part.<sup>8</sup> The characteristics of the 150 to 300 feet of material that lie between the water table and bedrock, however, are essentially unknown.

### *Groundwater Levels*

Elevations of the water table (the boundary between unsaturated and saturated subsoil) are shown in Figure 1. The elevations shown in the outwash area are fairly accurate: those greater than about 25 feet are based mainly on measured levels in wells and boreholes where land-surface elevations were determined by leveling; those less than about 25 feet were determined mainly from elevations of ponds and streams as shown on topographic maps (scale 1 to 24,000) of the U.S. Geological Survey. Elevations of the water table in the moraine and in the area to the west of the moraine, where many pond levels reflect locally perched water tables, are known with considerably less certainty. Elevations shown in the figure are typical of late spring in a year of average rainfall; elevations will be lower in autumn and during years of low rainfall.

Fluctuations in groundwater levels, as measured in three observation wells in Falmouth and neighboring towns, are shown in Figure 2. Levels in the spring season are normally about 2 feet higher than those in autumn in the low-lying areas near the coast (Falmouth 5 well), and they are 3 to 4 feet higher than autumn levels in the more upland areas (Mashpee 13 and Bourne 198 wells). Groundwater levels declined generally during the drought years 1962-1966. During the last two years of the drought they lay about 2 feet lower than their average annual levels during more normal years. The levels of most of the ponds, which are indicators of the level of the water table, also fluctuate to the same degree.

### *Water Balance*

All fresh water in Falmouth comes from rain that falls directly on inner Cape Cod. Falmouth's average rainfall of 46 inches per year (51-year average at Hatchville, 1919-1969) amounts to an average of 2.2 million gallons per day (mgd) per square mile. Using the detailed computations made in a geologically and hydrologically similar area of Long Island,<sup>9</sup> we can say that about half the rain that falls on the ground is returned to the atmosphere through direct evaporation or through transpiration by plants. The other half, or about 1 mgd per square mile, percolates into the ground and recharges the groundwater.

As the area of Falmouth exclusive of ponds is about 44 square miles, the groundwater recharge within the town averages, in round figures, about 40 mgd. This estimate relies on the assumption that most of the land area of Falmouth is

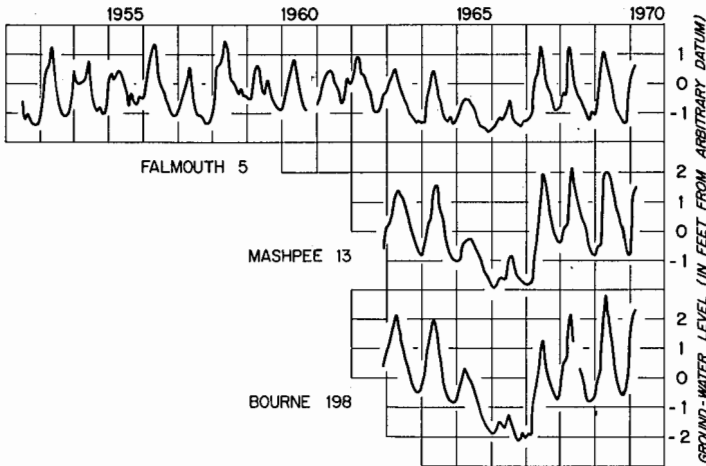


Figure 2. Fluctuations of groundwater levels in Falmouth and vicinity, 1952-1970. Based on monthly measurements in observation wells (which are not pumped); data from unpublished records, U.S. Geological Survey. Falmouth 5 well located where water table elevation averages between 5 and 10 feet, between East Falmouth and Waquoit. Mashpee 13 well located where water table elevation averages about 40 feet, in outwash area 2.5 miles east of northeastern corner of Falmouth. Bourne 198 well located where water table elevation averages about 25 feet, in outwash area west of Buzzards Bay moraine, 2.3 miles north of Falmouth-Bourne line.

suitable for recharge. While this assumption is not strictly true in some of the more heavily populated and paved areas or in the low-lying marshy areas along the coast, the lack of suitable recharge in these areas is at least partly compensated by groundwater that flows southward into Falmouth from Mashpee and Otis Air Force Base. In an attempt to make our estimates more conservative, we have chosen to exclude the several million gallons a day of groundwater which flow into Falmouth from the north.

The groundwater moves seaward through the subsoil at velocities of a few inches per day. Some of the groundwater returns to the surface in streams such as the Coonamessett River which derive essentially all their water from groundwater seepage. Most of the groundwater seeps eventually into coastal ponds and marshes or directly into Buzzards Bay and Vineyard Sound. East of a line from Hatchville through Long Pond, groundwater flows to the south toward Vineyard Sound (Figure 1: flow directions are perpendicular to water-table contours). West of the same line, groundwater flows westward toward Buzzards Bay. Over the long term the amount of fresh water that seeps or discharges into streams and coastal waters must average the same as the recharge, or about 40 mgd.

### *Water Supply*

The Town of Falmouth takes all its municipal water from one ground-water-fed reservoir, Long Pond, just north of Falmouth Village. An average of 2.0 mgd were pumped from Long Pond during 1970. Pumping during the summer months of July and August averaged 3.7 mgd, and nearly reached 5 mgd on the day of heaviest demand. On an average day in winter, about 1.5 mgd are pumped. Long Pond supplies water to all or part of each village in Falmouth. Households in the more remote or sparsely settled areas of town take their domestic water from individual wells. With the exception of about 0.3 mgd (year-around average) that is discharged through the existing sewer outfall pipe in Woods Hole Harbor, all wastewater in Falmouth is returned to the ground via cesspools and septic tanks.

For future sources of water, the Town has investigated well sites in the outwash area in northeastern Falmouth, and has found suitable sites in areas north and east of Hatchville that are now used as wildlife-management tracts.<sup>10</sup> Wells will probably be drilled in these areas if and when the population of Falmouth increases to the point where Long Pond can no longer supply its water needs.

### **Effects of Sewering on Water Levels**

Groundwater levels in parts of Falmouth will be lowered when sewers are installed. The proposed sewerage plan calls for collection pipes to be installed in three villages: Falmouth Village, Teaticket, and Woods Hole. The present groundwater levels in these areas are all less than 10 feet, and mostly less than 5 feet, above sea level. When wastewaters that are now returned to the ground in these areas via cesspools (presently on the order of 1 mgd) are transported to other parts of town or to Vineyard Sound for disposal, the resulting decline in groundwater level should be on the order of 1 to 2 feet.

### **Effects of Sewage Disposal on Groundwater Quality: Background**

To predict the effects of disposing of 3.7 mgd of secondary-treated effluent into the groundwater in Falmouth, we followed two approaches. First, we drew on experiences reported from other areas, particularly those from geologically similar regions. These experiences indicated two potential problems: contamination by nitrate, and a general increase in dissolved salts. Second, we studied the quality of groundwater in the effluent area of the existing sewage-treatment plant at Otis Air Force Base (labeled "IV" in Figure 1). Neither of these approaches was entirely satisfactory because they did not provide a basis for making numerically specific predictions of the effects on groundwater of effluent from a disposal plant of the type and capacity proposed by the consulting engineers in Falmouth. However, they did alert us to some of the potential hazards.

### *Contamination by Nitrate*

Experience in other areas of the country shows that the main contamination problem near inland sewage-disposal sites is the accumulation of nitrate in groundwater. Reduced nitrogen compounds in sewage effluents are typically oxidized to nitrite and finally nitrate when they become exposed to oxygenated soils and subsoils. Unlike some other contaminants and constituents of treated sewage effluents (especially phosphate, probably also DDT, and possibly viruses and some heavy metals) which are effectively adsorbed by soil particles and removed from solution, nitrate often persists in solution without being removed by the soil.

Studies on the mobility of nitrate in groundwater have been made in Texas and on Long Island, in sandy and gravelly materials similar to the subsoils of Falmouth. In the Texas experiment,<sup>11</sup> water that contained 24 ppm nitrate was pumped through a recharge well into a sandstone aquifer at a rate of 0.5 mgd. Water in an observation well 66 feet away from the recharge well eventually contained 23 ppm nitrate, indicating that the nitrate passed essentially undiminished in concentration through the sandstone. In the Long Island experiments,<sup>12</sup> groundwater quality was monitored downstream of household cesspools. Where cesspools were well above the water table and effluents percolated down to the water table through an unsaturated and oxygenated subsoil zone, nitrate formed in substantial concentrations. These nitrate concentrations persisted in the groundwater with essentially no decrease for horizontal distances as great as 45 feet (the greatest distance tested at the site) downstream of one cesspool, and they diminished somewhat after travelling 31 feet (greatest test distance at the site) downstream of another cesspool. That is, at one site the nitrate remained essentially constant and at another site it diminished somewhat as the groundwater moved away from the source of contamination.

Concentrations of nitrate in groundwaters have been reported in other areas where sewage effluents are disposed into sandy and gravelly subsoils. The town of Clovis (near Fresno), California, has a primary sewage treatment plant that has operated intermittently since the early 1940s (continuously since 1955) and has processed an average of about 1 mgd of sewage in recent years. Effluent is disposed into ponds and allowed to percolate down to the water table. Nitrate concentrations as great as 50 ppm are found in groundwater below the treatment plant, and they decrease to 25 ppm half a mile downstream of the plant.<sup>13</sup> Whether the decrease in concentration is due mainly to dilution by uncontaminated groundwater or to some chemical or bacterial process in the sandy subsoil is unknown. In Nassau County on Long Island, the composition of waters from public wells has been monitored annually for about 15 years, and the nitrate concentration of many of these waters has been increasing steadily. Of 333 wells whose waters were analyzed, 80 showed a progressive increase in nitrate concentration, 20 already (1969) exceed the safety limit recommended by the

U.S. Public Health Service (45 ppm), and 53 more will exceed this limit within the next 50 years if the present trends continue.<sup>14</sup> That is, more than one-fifth of the well waters that were analyzed either already contain excessive nitrate or will contain excessive nitrate by the year 2020. The organic nitrogen and ammonia of domestic sewage is the main source of this nitrate — about 800,000 people in Nassau County dispose of their sewage through cesspools.

As reported in the preceding paper,<sup>1</sup> Falmouth's consulting engineers predict that the total reduced nitrogen content of the effluent from the proposed activated-sludge treatment plant will be 40 ppm. If all this nitrogen is oxidized to the nitrate form, the nitrate concentration will be about 180 ppm. Although their prediction may be an overestimate — data from other sources<sup>15</sup> suggest that the total inorganic and available organic nitrogen in effluents after secondary treatment averages about 20 ppm — we used the figure of 40 ppm to maintain comparability with the study of the effects of the effluent on Vineyard Sound. In this event, we believe that the concentrations of nitrate probably would increase in groundwaters adjacent to and downstream of the proposed inland sewage disposal sites in Falmouth. To what degree this increase would affect Falmouth's groundwater is presently unknown. Nevertheless, any sewage disposal design for an inland site should anticipate an eventual need for further treatment to regulate the intrusion of nitrate into the groundwater.

#### *General Increase in Dissolved Salts*

When water is passed through a municipal distribution system and is used by people and their various household appliances, it accumulates dissolved matter. The used water that reaches a municipal sewage-treatment plant can be expected to contain 200 to 500 ppm more dissolved salts than the unused water that enters the municipal water supply system.<sup>15-16</sup> These are mainly common salts of substances such as sodium and bicarbonate which are not deleterious in themselves but which increase the general salinity of the wastewater. This salinity is not reduced substantially by secondary sewage treatment.

Considering that the total salt content of uncontaminated groundwater and the present water supply in Falmouth averages only 50 to 60 ppm, the addition of several hundred ppm would represent a substantial change in the quality of the water downstream of any sewage disposal site. Furthermore, because the U.S. Public Health Service recommends that the total dissolved salts in drinking water be kept below 500 ppm, this puts a limit on the number of times that such wastewaters, even if they can be made otherwise suitable for drinking purposes, can be disposed into the ground and later pumped out for further use as drinking water.

#### *Effects of Disposal at Otis Air Force Base*

The sewage treatment plant at Otis Air Force Base has been in operation since 1942. It uses trickling filters in its secondary treatment phase, and it

disposes effluent into the ground through sand filter beds. Although the Otis plant was designed to process an average of about 3 mgd, its load has averaged substantially less than 1 mgd for the last 10 years or more. Because this load is so much less than the design capacity, treated effluent is actually recycled (that is, processed more than once before being placed on the filter beds) in order that the plant may be operated more efficiently.

Samples of groundwater collected and analyzed during the spring months of 1970 show no chemical characteristics that can be clearly related to the effluent from the Otis treatment plant. The locations from which the samples were collected are shown in Figure 3 and the detailed results of the analyses are given in Table 1. Concentrations of nitrate in this group of samples are uniformly low and usually less than 5 ppm. Total dissolved salts range generally between 30 and 70 ppm. Whether the higher concentration of dissolved solids in some samples of this group is due to the effluent from the treatment plant or to strictly local contributions is not clear. But even if they are influenced by the effluent from the treatment plant, the accumulation is relatively minor and well within tolerable limits.

A few analyses suggest the existence of a zone of nitrate-bearing groundwater beneath the dry valley that eventually leads to Green Pond. Nitrate concentrations in samples from this valley — numbers 4 and 10 in Figure 3 as well as samples analyzed earlier from test well 3<sup>10</sup> and Otis Air Force Base well J (U.S. Geological Survey, unpublished data) — are anomalously high. Lower concentrations of nitrate are found in samples from uplands that adjoin the valley — those numbered 3, 8 and 9 in Figure 3. (Water from boring number 6, which is located within the valley, is also low in nitrate, perhaps because of influent seepage from Ashumet Pond.) Whether the Otis disposal plant is the source of the high concentrations of nitrate is not certain, but the valley seems to have served as a zone of concentration for nitrates, whatever their origin may have been.

These results have only limited application in making predictions of the effects of the proposed inland sewage disposal in Falmouth. The trickling-filter method of secondary treatment at the Otis plant is different from the activated-sludge method proposed for the Falmouth plant. And for the last decade or two, the Otis plant has processed one fourth or less of the amount of wastewater that is proposed for Falmouth. We can say that the Otis plant is not exceeding the capacity of the groundwater system to assimilate the chemical constituents reported in Table 1. However, we cannot say what the safe assimilation capacity of the system might be, and we cannot predict on the basis of the Otis example whether the system can safely assimilate the 3.7 mgd proposed for the Falmouth plant.

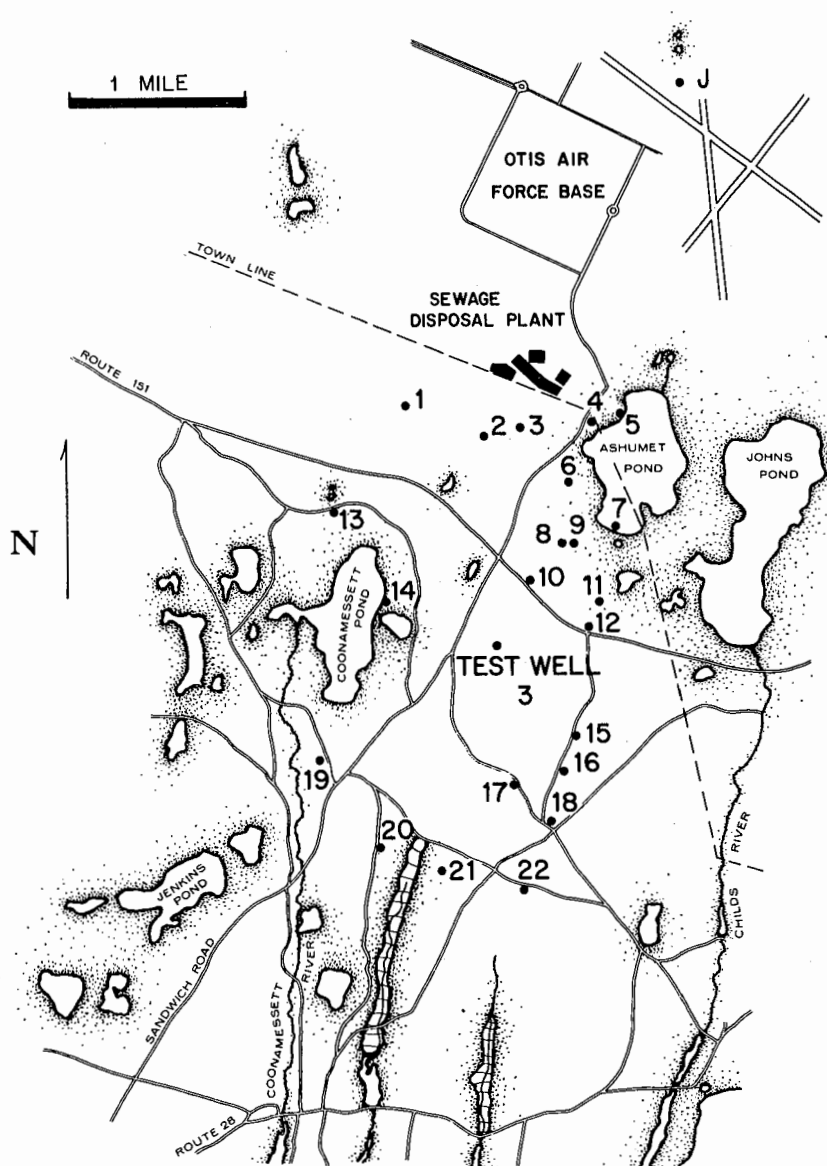


Figure 3. Location of wells sampled (spring 1970) in northeastern Falmouth in effluent area of sewage-treatment plant of Otis Air Force Base (IV in Figure 1). Numbers refer to chemical analyses of waters reported in Table 1.

Table 1. Selected dissolved constituents and total dissolved solids in water in northeastern Falmouth, spring 1970. Concentrations in parts per million. Chloride determined by titration; total dissolved solids by evaporation, analyst, F.T. Manheim. Other constituents determined by Nathaniel Corwin; analytical techniques described in preceding paper.<sup>1</sup>

Well	Number on Sampled	Date Sampled (1970)	Silica (SiO <sub>2</sub> )	Chloride (Cl)	Nitrate (NO <sub>3</sub> )	Nitrite (NO <sub>2</sub> )	Ammonia (NH <sub>4</sub> )	Phosphate (PO <sub>4</sub> )	Total dissolved solids
Coonamessett Airport	1	14 Jun	1.3	5.5	0.5	0.004	0.1	0.01	37
1970 test boring A-1	2	1 Jun	0.8	18.5	0.2	0.01	0.04	0.02	50
1970 test boring A-2	3	2 Jun	0.8	10.0	0.6	0.004	0.07	0.02	43
McMahon, Sandwich Rd.	4	15 Apr	13.8	12.2	6.4	0.004	0.03	0.01	78
	4	16 Jun	1.3	9.4	9.7	0.003	0.9	0.02	107
Cogut, off Sandwich Rd.	5	16 Jun	2.7	13.1	0.01	0.004	0.07	0.007	62
1970 test boring A-3	6	2 Jun	0.4	9.3	0.01	0.003	0.02	0.01	32
Ashmet Pond	7	14 Apr	0.2	10.6	1.5	0.01	0.3	0.002	53
Clubhouse, Club Valley Dr.	8	15 Apr	5.6	10.3	1.8	0.08	0.2	0.002	25
Hellawell, Club Valley Dr.	9	14 Apr	6.2	5.7	0.2	0.002	0.002	0.000	34
Harney Golf Course (irrigation)	10	14 Apr	7.5	13.1	4.2	0.004	0.003	0.02	59
Ashmet Holly Reservation	11	14 Apr	5.3	—	1.6	0.02	0.1	0.000	66
Mass. Highway Dept., Rt. 151	12	15 Apr	4.7	11.5	1.1	0.004	0.03	0.004	76
Hatch, Boxberry Hill Rd.	13	15 Apr	4.9	9.2	0.3	0.003	0.02	0.7	64
Clauson's Inn & Golf Course	14	15 Apr	8.4	8.6	0.5	0.001	0.02	0.03	47
Lydon, Currier Rd.	15	14 Jun	0.6	8.2	0.04	0.003	0.04	0.009	37
Rogers, Currier Rd.	16	14 Apr	4.9	7.9	0.02	0.001	0.009	0.007	36
Rose, Hayway Rd.	17	14 Apr	6.5	10.2	0.2	0.001	0.007	0.000	40
Rhodes, Hayway Rd.	18	14 Apr	4.7	11.1	0.05	0.004	0.006	0.004	52
Dimmock, Hatchville Rd.	19	16 Jun	0.5	5.7	1.4	0.003	0.03	0.007	36
Lopes, Old Meeting House Rd.	20	16 Jun	1.9	11.7	1.7	0.04	0.1	0.01	57
Falmouth Country Club	21	16 Jun	1.1	7.6	0.5	0.009	0.06	0.005	45
Texeira, Waquoit—Meeting House Rd.	22	16 Jun	4.0	7.6	0.1	0.002	0.07	0.01	40

Fig. 3

## Methods of Estimating Effects of Inland Disposal on Groundwater Flow

To estimate the effects of the disposal of 3.7 mgd of effluent on the patterns and directions of groundwater flow near the proposed disposal sites, we had few quantitative data to draw upon. Our estimates are necessarily qualitative, therefore, and they are presented with the understanding that they need further testing by more comprehensive investigations.

The few quantitative data that are available on the characteristics of groundwater flow come from a pumping test that was made in 1967<sup>10</sup> in the outwash deposits in northeastern Falmouth (Figure 4). To use these data to predict the effects of sewage disposal, we treated the disposal sites as large-diameter wells, and we assumed that the buildup of the water table due to 3.7 mgd of applied sewage effluent would equal the drawdown due to 3.7 mgd of pumped discharge. We applied the Thiem equilibrium formula (equation 2, page 91 of reference 17) to the drawdown data to calculate the transmissibility of the outwash aquifer. Using this calculated transmissibility and the same formula, and assuming a radius of influence of one mile, we estimate that the buildup of the water table directly beneath the disposal sites would average about one foot; at a radial distance of about half a mile from the center of a disposal site, the buildup would be about 0.3 foot.

The one-mile radius of influence that we assumed is considerably larger than the 800-foot distance at which the pumping test showed no measurable effect on water level. In the face of insufficient data, however, we chose to make a conservative estimate. If our estimate proves wrong, we have simply exaggerated the size of the contaminated areas upgradient from the disposal sites and at right angles to the water-table gradient. The areas directly downgradient from the disposal sites would be subject to contamination in any event.

In estimating the volume and velocity of groundwater flow, we assumed that the water-bearing materials have a permeability of 1,000 gallons per square foot per day at a standard reference water table gradient of one foot per foot. This is a reasonable assumption for medium-to-coarse sands that contain some gravel. The present gradients in the vicinity of the proposed disposal sites range from 5 to 15 feet per mile. The corresponding groundwater flows per square foot of aquifer cross section would range from 1 to 3 gallons per day. The corresponding velocities of groundwater flow, assuming a porosity of 0.3 in the sand and gravel, would range from about 0.5 to 1.5 feet per day.

## Predicted Effects of Inland Sewage Disposal

Three locations (I, II, III on Figure 1) were proposed for consideration as inland sewage-disposal sites. The suggested plant design at an inland site would provide secondary treatment of sewage by an activated-sludge system, and it

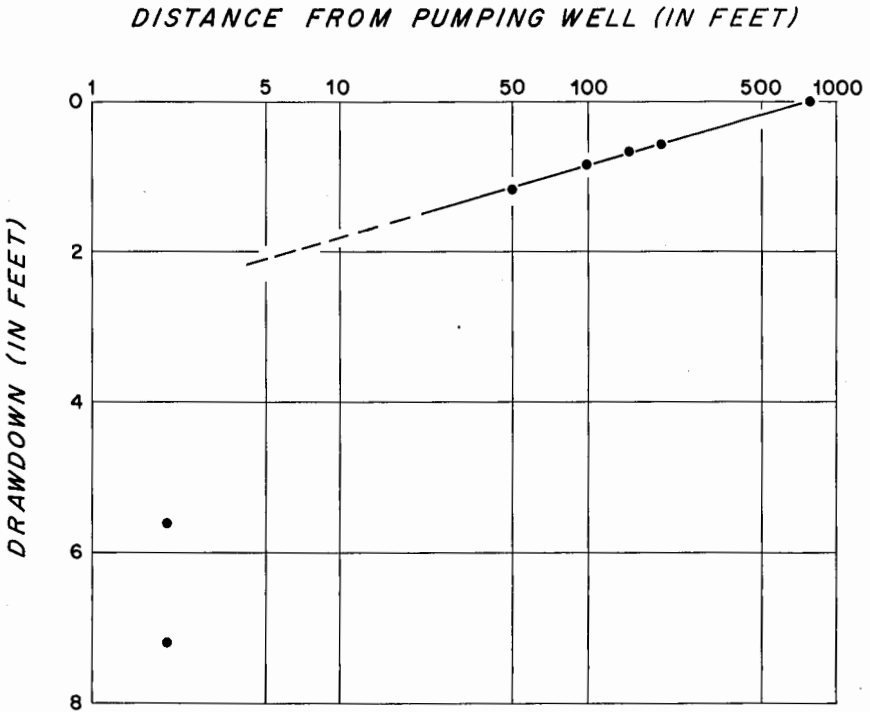


Figure 4. Drawdown in pumping 8-inch well (test well 3 shown in Figure 3) which reached a stable configuration after about 5 days of pumping at 0.47 mgd. Large drawdowns at distances of 2 feet are assumed to be due to turbulence or other effects near the pumping well.

would allow for the eventual disposal of 3.7 mgd of treated effluent onto some 40 acres of filter beds. In making our predictions, we considered each site individually.

#### *Effects at Brick Kiln Road Site*

If sewage effluent is disposed into the ground at the Brick Kiln Road site (I on Figure 1), most of it will seep southeastward and southward into the Coonamessett River or directly into Great Pond. Although we anticipate that the level of pumping of Long Pond eventually might be doubled to a year-around average on the order of 4 mgd, effluent from the Brick Kiln Road disposal site will not flow toward Long Pond under average conditions — nor is it likely to flow significantly near Long Pond even during droughts. However, the treated sewage effluent would form a slight mound on the water table directly below the site that would shunt part of the groundwater flowing from the north around the site to the westward and eastward. This shunting effect could enhance the recharge flow of uncontaminated water toward Long Pond.

The present groundwater flow beneath the Brick Kiln Road site is probably about 1 mgd\*, or considerably less than the 3.7 mgd that is the proposed effluent flow from the treatment plant. The greatest dilution of the effluent that can be expected near the plant, therefore, is about one part uncontaminated groundwater to four parts effluent. If we assume that a little more than half the nitrogen in the effluent reaches the water table in the oxidized state, the resulting nitrate concentration will be 100 ppm. Assuming further that the effluent will contain 350 ppm total dissolved salts, the one-to-four dilution with uncontaminated groundwater (salt concentration 50 ppm) will give the groundwater downstream from the disposal plant a salt concentration of about 300 ppm and a nitrate concentration of about 80 ppm. The portion of the effluent that reaches the Coonamessett River would be diluted further, probably in proportions of two or three parts river water to one part contaminated groundwater (the flow of the river in June 1950 was about 7 mgd<sup>18</sup>). Even when diluted by the Coonamessett River, however, the nitrate concentration of the inflowing water will be substantially greater than the concentration in Great Pond, which is usually less than 1 ppm.<sup>19</sup> The result of the added nitrate may well be an unpleasant excess of algae in Great Pond if other essential plant nutrients are also available in the water. The non-nitrogenous salts, however, probably would have little apparent effect on the algae population of the brackish waters of Great Pond.

The velocity of the groundwater flow would probably be such that the excess nitrogen and other contaminants would not reach the Coonamessett River for several years after the treatment plant began operation. It probably would not seep directly into Great Pond itself for several more years. However, should remedial action become necessary, an additional period of time of equal or perhaps even greater length would be required to flush the excess contaminants from the groundwater between the disposal site and Great Pond.

#### *Effects at Hayway Road Site.*

In the event that 3.7 mgd of treated sewage effluent is disposed into the ground at the Hayway Road Site (II on Figure 1), the groundwater would continue its general southward flow. Most of the effluent would eventually emerge in the streams and bogs that drain into Green Pond and Bourne Pond. The Coonamessett River should act as a hydrologic barrier to prevent the effluent from either contaminating or recharging the groundwaters that feed the Town reservoir at Long Pond.

Effluent from a disposal plant at the Hayway Road site probably would not contaminate the potential future sources of groundwater that lie beneath

\*In making this estimate we assumed a saturated aquifer cross section about 3000 feet long and 150 feet deep beneath the site and normal to the groundwater gradient; the flow at the existing gradient of about 10 feet per mile is about 2 gallons per day per square foot.

wildlife-management areas in the northeastern part of Falmouth. We arrived at this conclusion by postulating a municipal well at the location of test well 3 (shown in Figure 3) and constructing profiles of the water table between the well and the disposal site for different assumed conditions. Even if the well were being pumped at the excessively high rate of 3.7 mgd while the disposal plant added the same amount to the groundwater, the water table between the well and the disposal site (which now slopes southward at 10 feet per mile) probably would continue to slope slightly southward, and no sewage effluent would reach the pumping well. Pumping the well at a more practical rate for sustained yield of 1 to 2 mgd and locating it somewhat farther north would provide extra margins of safety.

The effects of the effluent from a plant on Hayway Road would be like those from the site on Brick Kiln Road. Nitrate would reach Green Pond, Bournes Pond, and perhaps Great Pond in concentrations greater than those now existing in these estuaries. The result of the added nitrate might be an unpleasant excess of algae. Furthermore, all households that still rely on private wells for their domestic supply in the area southward of the disposal site might have to be supplied by the Town water system.

One further matter we considered was the possible effect of the added nitrate and other dissolved salts on the cranberry bogs that lie downstream of the Hayway Road site in the stream valleys that are tributary to Green and Bournes Pond. Judging from data and opinions supplied by Dr. Chester E. Cross of the University of Massachusetts Cranberry Experiment Station, the interaction between cranberries and sewage plant effluent would probably be slight. Because the nitrogen requirements of cranberries are rather low, when compared to other crops, the cranberry plants would not remove a significant proportion of the nitrate in the effluent waters. Conversely, the effluent would have little if any effect on the cranberries.

#### *Effects at Blacksmith Shop Road Site*

At present, groundwater beneath the Blacksmith Shop Road site (III on Figure 1) flows toward the Town reservoir at Long Pond. This flow pattern would continue at an accelerated rate if sewage effluent is disposed at the site. Jenkins Pond, other nearby ponds, and private wells in the vicinity might also receive effluents from the plant.

The dilution of the effluent by groundwater might be slightly greater than at Brick Kiln Road, but it would not be significantly greater than one to four when the plant is discharging its proposed average effluent of 3.7 mgd at the Blacksmith Shop Road site. Groundwater immediately downstream of the disposal site might be expected to contain dissolved salts in concentrations of about 350 ppm, of which 80 ppm would be nitrate. How much these concentrations would be diminished by progressive dilution or by other processes in the subsoil before the effluent reaches Long Pond is uncertain.

The velocity of groundwater flow would be such that a decade or two would be required for the effluent from the plant at the Blacksmith Road site to reach Long Pond. Even if no further contaminants were percolated into the ground at the end of that period, another period of several decades would pass before contaminants ceased to enter Long Pond.

### **Predicted Effect of Proposed Sea Outfall on Groundwater**

The sea outfall proposed by the Town's consulting engineers would eventually dispose of 3.7 mgd of fresh water directly into Vineyard Sound. This amount is equivalent to about 10 percent of the natural recharge of the groundwater, and its disposal through the outfall would have an effect equivalent to that of reducing the natural recharge by 10 percent. Our predictions of the effects of the proposed outfall centered on two separate effects: the effect on the eventual water supply, and the effect on groundwater levels.

#### *Effect of Sea Outfall on Water Supply*

The sea outfall will not have a limiting effect on the water supply of Falmouth for some years to come. However, when the year-around population of the town begins to approach 100,000 (about 6 times the present population), the effect of the outfall might become critical. This assessment is based on the following approximate estimate of the population that can be served by the available groundwater in Falmouth.

1. Groundwater in Falmouth is recharged by rainfall at an average rate of about 40 mgd. All this water, however, is not available for water supply. As a rough rule of thumb, about half of it probably should be left in the ground to retard salt-water intrusion and keep the groundwater system dynamically operative. The other half, or 20 mgd, can be counted on for long-term sustained yield.
2. Pumping from Long Pond now averages 2 mgd on a year-around basis. This amount serves a year-around population of 10,000 people. (Actually it serves several thousand more year-around people. The total year-around population of Falmouth in 1970 was about 15,500 people. Of this total, several thousand people take their water from household wells. The remainder, supplied from Long Pond, amounts to somewhat more than 10,000 people. However, we used the figure of 10,000 to keep the arithmetic simple.)
3. The 20 mgd that can be counted on as sustained yield would then serve, at the present rate of water use, a year-around population of 100,000.

*Effect of Sea Outfall on Pond Levels*

A more immediately serious effect of the outfall on groundwater would be the reduction of the water levels of the inland ponds which are important scenic and recreational resources of the town. Because the levels of all the large freshwater ponds and most of the small ones are directly controlled by the level of the groundwater, a reduction of groundwater level would decrease the depths and surface areas of the deeper ponds and would empty some of the shallower ponds.

Rough numerical estimates of the declines in pond levels that would be produced can be based on Darcy's Law of groundwater motion. Darcy's Law states that the volume of flow is a direct function of the combination of three factors: the permeability of the aquifer materials, the cross-sectional area of saturated material in the aquifer, and the slope of the water table. If the volume of flow (recharge in this case) is decreased by 10 percent, the combined effect of the other factors would decrease accordingly. Because the permeability would remain constant for a given aquifer material, the other two factors would have to adjust to the decrease in recharge. And because adjustments in the elevation and slope of the water table in Falmouth are accompanied by proportionately much smaller adjustment in the saturated thickness, essentially all the response to the decreased recharge would appear as a decrease in slope. That is, a 10-percent reduction in recharge would cause about a 10-percent reduction in the slope of the water table. Groundwater levels that now average 20 feet above sea level would drop to a new average elevation of 18 feet, and levels that now average 40 feet above sea level would drop to a new average elevation of 36 feet.

There is some difficulty, however, in predicting the specific amount of water-level decline to be expected in any particular pond. The analysis given above assumes that the effects of the decreased recharge would be distributed uniformly over the whole area of the town, and it predicts that the elevations of the water table above sea level would decrease uniformly by 10 percent. In practice, however, the effects would show more strongly in areas where groundwater is being pumped and in areas where the wastewater that was previously returned to the ground via cesspools is being collected and taken elsewhere by sewers. Under these circumstances, the Falmouth Village-Long Pond area would probably be the first to show the effects. When groundwater sources in the Hatchville area are developed for future supplies, as they probably will be before the sewer system reaches its full design capacity of 3.7 mgd, water levels in the northeastern part of town would begin to be affected more strongly. In some other areas of town, however, even the long-term effects may be fairly small. In the light of this, we make the following estimate of the decline in the average groundwater levels that would be caused by disposing of 3.7 mgd of effluent through a sea outfall: average water levels in ponds in the low-lying areas in and near Falmouth Village and Teaticket could be expected to drop 1 to

2 feet; levels of ponds in the upland Hatchville area (Coonamessett and Ashumet Ponds, for example) could be expected to average 2 to 4 feet lower than their present average levels.

### Conclusions and Recommendations

If 3.7 mgd of sewage effluent is disposed through a sea outfall, pond levels in Falmouth would decline 1-2 feet in lowland areas and 3-4 feet in upland areas.

If secondary treated sewage effluent is disposed into the ground at the site on Brick Kiln Road, it would eventually enter the Coonamessett River and Great Pond estuary, and it would help conserve the waters that recharge the Town reservoir without contaminating them. Effluent disposed at the site on Hayway Road would eventually enter the streams that are tributary to the estuaries of Green Pond, Bournes Pond, and perhaps Great Pond. Effluent from the Hayway Road site would assist in conserving the groundwaters in northeastern Falmouth that are potential future sources of municipal water supply, but it would not recharge or conserve the groundwater that feeds the present Town reservoir. Effluent disposed at the Blacksmith Shop Road site would directly recharge and probably contaminate the Town reservoir. Of the three proposed inland sites, the one on Blacksmith Shop Road combines the greatest efficiency in water conservation with the greatest potential hazard to the quality of the Town's water supply.

Nitrate can be expected to persist in the effluents of inland disposal plants for extended periods. However, there is no certain way of estimating the concentrations of nitrate that would eventually reach the Town reservoir or the estuaries, and therefore no certain way of predicting whether or not their effects would be deleterious.

Taking this uncertainty into account, we recommend that consideration be given to the eventual need for an expanded installation to remove nitrate and perhaps other constituents from the effluent even though the immediate installation of such tertiary-treatment equipment be delayed. Specifically, we recommend the following course of action if the Town decides to locate a secondary sewage-treatment plant at any inland site.

1. Acquire the necessary acreage and design the disposal plant so that tertiary-treatment equipment for removal of nitrogen and other constituents can be installed in the future with minimum effort and expense if it becomes necessary.
2. Install a network of inexpensive observation wells to intercept the effluent groundwater at appropriate locations near the disposal site, and monitor the content of nitrate and other constituents frequently enough to give early warning of contamination so as to permit prompt remedial action.

3. If concentrations of nitrate and other constituents in the groundwater approach unacceptable levels, effective tertiary removal facilities should be installed at the plant. If concentrations do not increase significantly after many years of monitoring, such facilities should not be necessary.

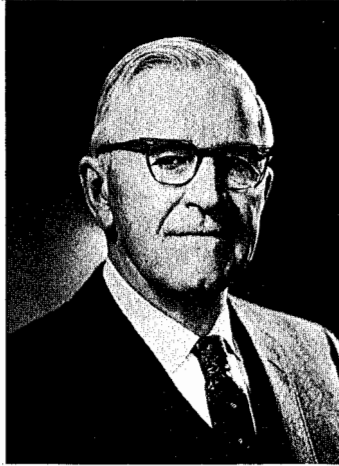
### Acknowledgments

We thank the following for assistance, helpful discussions, and reviews of our manuscripts: R.H. Backus, D.F. Bumpus, N. Corwin, K.O. Emery, H.W. Jannasch, B.H. Ketchum, and W.R. Wright of Woods Hole Oceanographic Institution; R.H. Brown, J.E. Cotton, G.T. Malmberg, F.T. Manheim, R.N. Oldale, S.S. Papadopulos, D.A. Rickert, and J.R. Williams of the U.S. Geological Survey; R.E. Hickman of Whitman & Howard, Consulting Engineers, and A.N. Strahler of Orleans, Mass.

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**Dr. Thomas Ringgold Camp**  
**1895-1971**



My acquaintanceship with Tom Camp, referred to for years by younger engineers and students as Professor Camp, was through the Boston Society of Civil Engineers. We joined at about the same time, Tom becoming active in the Sanitary Section while the writer directed his efforts toward the Designers Section. This friendship grew by repeated meetings on committees and boards as well as Society papers in which Tom took an active part. By 1944, when he decided to enter full-time consulting practice, I was employed as principal engineer with Samuel M. Ellsworth, a consulting sanitary engineer. Upon Mr. Ellsworth's death, Tom immediately acquired the Ellsworth business and its principal engineer. I am relating these events to show young engineers the importance of becoming acquainted with contemporary engineers through Society affiliations. One never knows when such a friendship will unexpectedly lead to an opportunity, such as mine, of becoming a partner with a man like Tom Camp.

Tom Camp was a native of Texas, having been born in San Antonio, and he received his B.S. Degree from Texas A&M College in 1916. His college yearbook referred to how creditably he handled the duties of Captain of "one of the best companies on the campus". The editor also appraised Tom remarkably well at that early date, for not only did he comment on his "distinguished" grades, but also his "unusually high sense of honor, and a man who will always be square regardless of the circumstances". With this background it was only natural that he devoted two years to his country, serving with the heavy artillery in France, 1917-1919.

Following World War I, seemingly by chance, he drifted toward sanitary engineering with several firms in Texas where he designed pumping stations, filtration works, and sewerage systems. Evidently he became aware of his

interest in the sanitary field and the need for further educational training, because he next became a graduate student at the Massachusetts Institute of Technology, receiving his Master's Degree in 1925. The partnership of Spoon, Lewis & Camp, which was formed in Greensboro, North Carolina, in 1925, specialized in the fields of waterworks and sewerage.

In 1929 he was called back to MIT as an instructor, moving rapidly to become head of the Department of Sanitary Engineering. He was a prodigious worker and, while handling his teaching schedule, proceeded to become self-taught in fields such as sanitary chemistry, bacteriology, advanced mathematics, and hydraulics, as well as treatment processes for both water and wastewater. Being of an inquiring mind and fruitful imagination, he and many of his graduate students embarked upon new fields of research, many of which had not been touched before. His personal study and research during this period became the basis for the production of many papers, resulting in his attaining a national reputation at a relatively young age. He was well-qualified through his constant searching for the truth by research, as well as by his constant efforts to expand his personal knowledge so that he could provide students with the highest calibre of training. Many of his graduate and undergraduate students assumed prominent positions in the fields of teaching or engineering practice.

He was frequently requested by this Society to present papers, as well as to serve as an officer and on a variety of committees. Of approximately one hundred papers and discussions which appeared in journals and periodicals, some of the earliest presentations appeared in our Journal. Copies of these papers were requested for many years by readers throughout the world and a substantial number were recognized by awards, as mentioned later.

The reputation which he established while teaching at MIT resulted in so many demands for his services as a consultant that, in 1944, he decided to devote full time to consulting practice. It was then that he acquired the business of Samuel M. Ellsworth, and together with the writer continued the business under Mr. Camp's name until the partnership of Camp, Dresser & McKee was formed in 1947. His practice was exclusively in sanitary engineering from the beginning. He had the courage and fortitude to base his reports and designs on the basis of the most up-to-date engineering principles including the results of some of his own research and development, rather than following only the long established basis of conventional design. From a modest beginning with two persons, Mr. Camp lived to see the staff of his firm grow to over 360 in Boston with clients in many locations throughout the world, and additional employees of about one hundred nationals in foreign offices.

His papers and articles are too numerous to enumerate but they appeared in practically all of the journals and periodicals relating to his field of activity. In addition, he authored a book entitled "Water and Its Impurities" as well as the Sections devoted to "Water Supplies, Water Distribution, and Water Treatment" in Davis' Handbook of Applied Hydraulics. In 1970, he was presented an

Honorary Doctor of Science Degree from Clarkson College. This award was based upon his 15 years of service as a teacher, as well as his professional activities for an additional period of 27 years. Many other awards were received by Dr. Camp including the following from the Boston Society of Civil Engineers: Hydraulics Section Award; Sanitary Section Award; Desmond FitzGerald Medal; Clemens Herschel Award; and he was selected to give a John R. Freeman Memorial Lecture, January 1969. He was a member of the John R. Freeman Fund Committee for a period of years.

Other honors included Honorary Membership in the Boston Society of Civil Engineers; American Society of Civil Engineers; New England Water Works Association; and Water Pollution Control Federation. His awards and medals from other Societies included the New England Award of the Engineering Societies of New England; the Dexter Brackett Memorial Medal on two occasions from the New England Water Works Association; and the Fuller Award of the American Water Works Association. The following awards were received from the American Society of Civil Engineers: the Society's Karl Emil Hilgard Hydraulic Prize; the J.C. Stevens Hydraulic Prize; the J. James R. Croes Medal; the Rudolph Hering Medal; and the Edmund Friedman Professional Recognition Award.

The BSCE honored Dr. Camp by establishing the Thomas R. Camp Fund in January 1971, the income from which is to be used annually for one or more lectures on "outstanding recent developments in the field of sanitary engineering." The Water Pollution Control Federation also established the Thomas R. Camp Medal in 1964, to be awarded annually to a Federation member who best exemplifies "the unique application of basic research or fundamental principles."

The Society activities of Dr. Camp resulted in his having served as a president of the Boston Society of Civil Engineers, the New England Water Works Association, the New England Water Pollution Control Association, and as one of the founders and the first chairman of the American Sanitary Engineering Intersociety Board.

Tom Camp was a loving and devoted husband and father. He saw his children obtain one or more college degrees and his oldest daughter receive her Doctorate. Following the death of his wife, Margaret Camp, in 1970, Dr. Camp, who had already retired in 1965, was happiest in the office continuing to carry on his research and development studies, on the basis of which he wrote several of his latest papers. It was during this period when he was alone that the partners and staff of his firm also became "his family" and drew even closer to him because his own children were all at a distance.

Thomas R. Camp, whose death occurred on November 15, 1971, was a man with an unusually keen mind together with an equally good memory, and through his self-training and teaching experience had assimilated an amazing amount of scientific knowledge. The writer, over a period of the last 45 years,

had occasion to know rather intimately many of the outstanding engineers in the Boston area. No one else had the ability to draw upon such a vast store of knowledge in such a wide variety of fields and apply it without hesitation, though it may have involved civil engineering, chemistry, mathematics, physics, and other branches of scientific and engineering knowledge. It was this breadth of his background which permitted him to conceive of and carry out the many activities of his professional career. He was not reluctant to share this knowledge with others.

The life of Dr. Thomas R. Camp, with his high ethical standards, his personal integrity, combined with willingness to work hard and bring his professional attainments to the summit which they achieved, will always remain an example and challenge to younger engineers.

(MEMOIR by Herman G. Dresser)

# PROCEEDINGS OF THE SOCIETY

## Minutes of Meetings

### Computer Section

October 27, 1971:— The Section meeting was called to order at 7:45, by Al Rimer, Section Chairman, at the Boston Playboy Club. About 55 people were present. Al urged all present who were not currently on the Section's mailing list to fill out and leave a card at the door. With no further business besides the evening's speaker, the Chairman turned the meeting over to the writer.

Robert Logcher introduced the speaker, Dr. James N. Jackson of Engineering Computer International, Inc., Cambridge, Massachusetts, and his topic, "Computerized Cost Estimating". Dr. Jackson spoke for 45 minutes, describing the problem of cost estimating and ways in which a computer

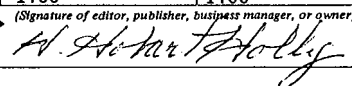
could improve this process. He described a system developed for and by Smith, Henschman and Grylls in Detroit, an architectural engineering firm. Input, hierarchical project description and development and a variety of cost report facilities were illustrated. A lively discussion period followed the end of the talk.

Al Rimer closed the meeting at 9:00 after announcing the next meeting on November 17 at the M.I.T. Faculty Club.

Respectfully submitted,

Robert D. Logcher  
Clerk, Computer Section

STATEMENT OF OWNERSHIP

U. S. POSTAL SERVICE <b>STATEMENT OF OWNERSHIP, MANAGEMENT AND CIRCULATION</b> <i>(Act of August 12, 1970: Section 3685, Title 39, United States Code)</i>		SEE INSTRUCTIONS ON PAGE 2 (REVERSE)
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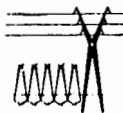
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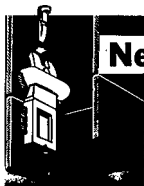
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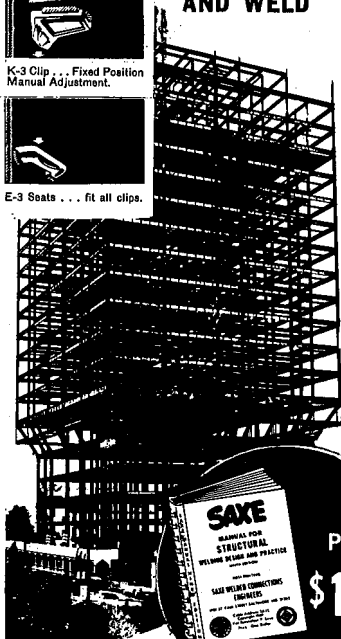
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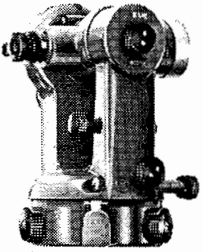


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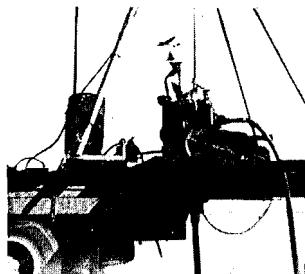
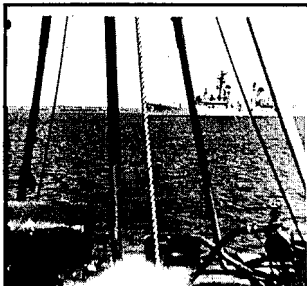
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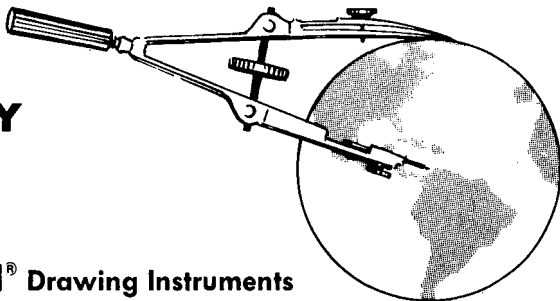
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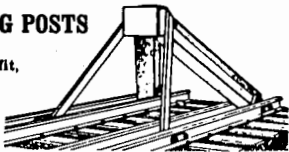
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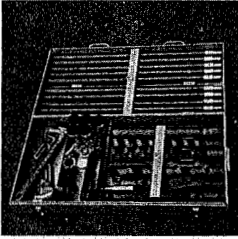
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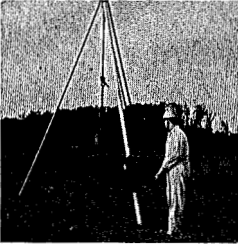
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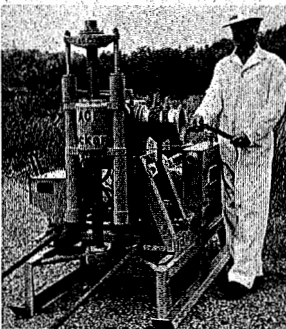
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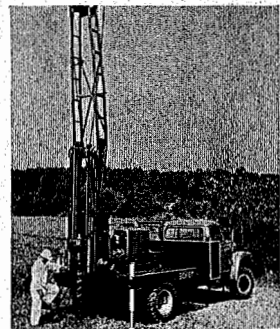
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