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The Boston Society of Civil Engineers, now a section of the American Society of Civil Engineers, announces an annual prize of $2000 for an exceptionally useful paper in the field of hydraulic engineering. To qualify for the prize, the paper submitted must be comprehensive. It must trace the historic development of a thesis, explain its theoretical basis and give detailed practical examples of, and cite pitfalls avoided by, its application. The paper must be original in its overall presentation, though it can cover the state-of-the art in a superior manner. It should be well illustrated and edited.

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The prize is available through the generosity of the late John R. Freeman, one of the great civil engineers of the early part of the century, who left a substantial sum of money to the Boston Society of Civil Engineers for purposes such as this. Papers will be judged by the Section's John R. Freeman Fund Committee. They should be submitted by registered mail to the Freeman Fund Committee, c/o Boston Society of Civil Engineers Section American Society of Civil Engineers, 230 Boylston Street, Boston, Massachusetts 02116. The first prize(s) will be awarded in December 1975 and papers, to be eligible, must be received by the Committee before July of 1975. Interested applicants are invited to correspond with the Committee before preparing final applications.

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HEAT DISPOSAL IN THE WATER ENVIRONMENT

by
Donald R. F. Harleman,[1] M. ASCE
(The ninth John R. Freeman Lecture, presented before the Boston Society of Civil Engineers on April 3, 1974.)

Preface

The author was honored to give the 9th annual John R. Freeman Memorial Lecture before a joint meeting of the Boston Society of Civil Engineers and the Massachusetts Section of the American Society of Civil Engineering in Boston on April 3, 1974. The privilege of a personal remark is requested in view of the unexpected death, only two days later, of the author's friend, teacher and colleague, Professor Arthur T. Ippen.

Professor Ippen attended the lecture and had himself delivered the 6th John R. Freeman Memorial Lecture in 1971. It is entirely fitting that this paper be prefaced by a brief tribute to a man who epitomized the ambition of John R. Freeman to improve the profession of hydraulic engineering. Both men had immeasurable influence on the profession by their encouragement and support of young engineers. In the author's case this happy association as a student and colleague of Arthur T. Ippen covered a period of almost thirty years.

A tribute to one's teacher also requires an acknowledgment of those who represent the future of our profession. Learning is a continuing process, and a teacher should learn as much from his students as they in turn learn from him. This paper is a reflection not only of past influences but of the many things the author has learned from his students. In particular, the contributions of Keith D. Stolzenbach, Patrick J. Ryan and Gerhard H. Jirka are gratefully acknowledged.

Abstract:

The need for continuing development of techniques for predicting temperature distributions due to waste heat discharges into lakes, rivers, estuaries and the oceans is discussed. Emphasis is on the interactive role of basic laboratory experiments, analytical and numerical techniques and field

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observations. Diffusion of buoyant jets is discussed, including heated surface jets and multiple jets issuing from a submerged multiport diffuser. In the near-field analysis of surface jets the important problems are related to the lateral spreading caused by buoyancy. Comparison of theoretical predictions with laboratory and field observations are given. The mechanics of multiport diffusers for heated discharges in shallow receiving waters are discussed in contrast to sewage diffusers. The important problem is the degree to which stratification can be maintained in order to minimize local reintrainment and reduction of dilution capacity. Criteria for stable and unstable flow regimes are provided. A mathematical model for temperature distribution, with or without waste heat addition, in unsteady flows under time-varying meteorological conditions is given.

Introduction

Heat is by far the largest waste product associated with the generation of electricity from a thermal energy source. Modern fossil fuel stations have an efficiency of about 40 percent and the present generation of commercial nuclear stations about 32 percent. Thus in a nuclear power station, for every kilowatt of electrical power produced, the equivalent of two kilowatts is rejected to a heat sink (the surrounding air or water) through the steam condenser system.

Accounting for differences in plant and stack losses between fossil and nuclear units, the following heat rejection rates to condenser cooling water may be used to quantify the heat disposal problem for a unit producing 1000 megawatts $\left(MW_e\right)$ of electrical power:

Fossil:	4.2 x 10^9 BTU/hr (4.4 x 10^{12} joule/hr)
Nuclear:	6.6 x 10^9 BTU/hr (7.0 x 10^{12} joule/hr)

Thus a nuclear unit rejects approximately 1½ times as much waste heat as a corresponding fossil unit. A typical condenser water flow rate for a 1000 MW unit is about 1500 cfs (42.5 m^3/s) and the temperature increase across the condenser is 20°F (11°C) for a nuclear unit.

If a continual supply of new water is available at the condenser water intake from an adjacent body of water, the process is called *once-through* cooling. If the cooling water is recirculated and the heat transferred directly to the atmosphere by an auxiliary mechanism, such as evaporative cooling, the process is called *closed-cycle* cooling.

Under the 1972 amendments to the Federal Water Pollution Control Act, the EPA is directed to establish guidelines for effluent limitations including waste heat, identifying the best *practicable* control technology available. These requirements are to be met by 1977. In addition, EPA must identify the best *available* technology to be met by 1983. Recent EPA guidelines imply that only closed-cycle, evaporative cooling processes meet the above conditions. However, with respect to thermal discharges, the 1972 Amendments state that if it can be demonstrated that an EPA limitation is more stringent than that necessary to protect the propagation of fish and wildlife, then EPA may permit less stringent control on a case by case basis. The additional economic costs of closed-cycle cooling are usually significant with respect to once-through cooling. Therefore it is important to continue the development of techniques for predicting temperature distributions and the environmental costs of adding heat to natural bodies of water, particularly for power plant sites in the coastal zone and along the Great Lakes.

The objective of this paper is to illustrate some recent developments in the area of temperature prediction in the water environment. These are drawn from recent experience in the Ralph M. Parsons Laboratory at M.I.T. The paper is not intended to be a complete state-of-the-art review, the objective is to emphasize the interactive role of basic laboratory experiments, analytical and numerical techniques and field observations.

The problem of temperature prediction for waste heat disposal is distinguished by the simultaneous occurrence of two factors which add a degree of complexity beyond that for other predictive models for effluent dispersion and water quality in lakes, rivers or coastal waters or air quality in the atmosphere. These two factors are a) the buoyancy of the discharge, and b) the considerable volume and momentum of the effluent. The buoyancy of the discharge which is associated with the temperature change as the cooling water passes through the power plant condenser requires the simultaneous determination of fluid motion (velocity distribution) and heat distribution within the receiving water body. The volume and momentum of the discharge will affect the ambient flow field. Hence, it is not possible to consider the heated discharge as a passive tracer introduced into the ambient flow. This is particularly the case in a zone close to the discharge area (near-field zone) where advection and free turbulence created by the shearing action of the discharge with respect to the ambient water causes jet diffusion of the heated water. Outside this immediate near-field exists a considerably larger far-field zone in which the heat is distributed by buoyancy driven currents and through diffusion and advection by ambient currents.

Predictive models for hydrothermal analysis can be broadly classified into two groups: complete models and zone models. In the *complete models* the governing equations are solved in their more general form over the whole region of interest. These models promise a significant advance through the use of high speed computers with large memories. Yet several problems must be recognized: 1) The state of the art requires many simplifying assumptions regarding turbulent fluid flow and heat fluxes. 2) The flow and temperature field induced by a thermal discharge exhibits distinctly different hydrodynamic zones. Consequently, the simplifying assumptions utilized in the formulation of the complete model are not uniformly valid throughout the region of interest. This may cause considerable error and thus restrict the utility of such computer models. 3) Boundary conditions at the edge of the solution domain, notably open fluid boundaries, are difficult to specify.

In the *zone models* the whole region of interest is divided into several zones with distinct hydrodynamic properties (such as near-field and far-field). For each zone it is then possible to simplify the governing equations by dropping unimportant terms (through a formal scaling process). This gives a considerable advantage in the mathematical treatment and improved accuracy in the solution. Despite this advantage, problems remain since some of the assumptions which yield simplified governing equations may not be appropriate in the actual application. Furthermore, there may be a lack of criteria on how to establish a correct division of the whole region into zones.

In this paper, models for the diffusion of buoyant jets are discussed as one class of zone models which are of particular importance in the prediction of the near-field behavior of heated discharges. The simplifying assumptions pertinent to jet diffusion are discussed and the restrictiveness of these assumptions in the development of actual buoyant discharge models is analyzed. This is done for two types of models: buoyant surface discharges and buoyant submerged discharges. In the final section, a far-field model for temperature distribution in unsteady flows under time-varying meteorological conditions is discussed.

Diffusion of Buoyant Jets

The dominant transport processes in jets are the convection by the mean velocities along the trajectory of the jet and the lateral turbulent diffusion normal to the jet trajectory through the irregular eddy motion within the jet. The convective mechanism is due to the initial discharge momentum and/or the vertical acceleration in the case of submerged buoyant jets.

In general, the governing equations of the boundary layer type are formulated in a local coordinate system following the trajectory of the jet. Exact similarity solutions to these equations can be found if semi-empirical mixing length assumptions are made (Schlichting [15]). For engineering purposes, however, it is more practical to specify similarity profiles *a priori*. By integrating across the jet, the governing partial differential equations are then easily reduced to ordinary ones with the axial distance as the independent variable. This integral technique (method of moments) has been found useful and sufficiently accurate in many applications. Examples include buoyant jets in deep (unconfined) receiving water, either non-stratified or stratified. A further advantage of the integral technique is the possibility of considering a flow of the receiving water by defining a gross force acting on the jet. As in all problems of turbulent flow, empirical coefficients appear in the analysis and have to be determined from experiments.

Buoyant Surface Jets — Near-Field Analysis

Three-dimensional predictive models of buoyant surface jets which take account of the underlying transport phenomena have been proposed by Stolzenbach and Harleman (17, 18), Prych (12) and Stefan and Vaidyaraman (16). The theoretical premises on which these models are built have been examined in detail by Jirka and Harleman (6).

Figure 1 defines the problem under consideration: Discharge parallel to the free surface of the receiving water which is deep and quiescent. In order to illustrate the various assumptions, only the steady state horizontal momentum equations in the x, y directions are considered in the following:

$$\frac{\partial u^2}{\partial x} + \frac{\partial uv}{\partial y} + \frac{\partial uw}{\partial z} = \frac{g}{\rho_a}\int_z^{-\infty} \frac{\partial \Delta\rho}{\partial x}\,dz - \frac{1}{\rho_a}\frac{\partial p_d}{\partial x} - \frac{\partial u'^2}{\partial x} - \frac{\partial u'v'}{\partial y} - \frac{\partial u'w'}{\partial z} \qquad (1)$$

$$\frac{\partial uv}{\partial x} + \frac{\partial v^2}{\partial y} = \frac{\partial vw}{\partial z} = \frac{g}{\rho_a}\int_z^{-\infty} \frac{\partial \Delta\rho}{\partial y}\,dz - \frac{1}{\rho_a}\frac{\partial p_d}{\partial y} - \frac{\partial u'v'}{\partial x} - \frac{\partial v'^2}{\partial y} - \frac{\partial v'w'}{\partial z} \qquad (2)$$

where x,y,z = cartesian coordinates; u,v,w = time-averaged velocities in x,y,z; u′,v′,w′ = turbulent velocity fluctuations; $\Delta\rho$ = local density difference with respect to ambient density ρ_a; p_d = dynamic pressure. The terms involving $\Delta\rho$ arise from the buoyancy of the flow. An additional effect due to buoyancy is the damping of vertical turbulence, thereby reducing vertical spreading and entrainment. The importance of the buoyant effects is given by the local densimetric Froude number

$$F_L = u_c\left(\frac{\Delta\rho_c}{\rho_a}\,gh\right)^{-1/2} \qquad (3)$$

where u_c, $\Delta\rho_c$ are values of u and $\Delta\rho$ at the jet axis on the water surface and h is a measure of the verticle thickness of the jet.

In the limiting case of the isothermal surface jet, $F_L \rightarrow \infty$, and the buoyancy terms are negligible. In addition, boundary layer arguments lead to the neglect of the lateral momentum equation (2) and the second and third terms to the right of the equal sign in equation (1). Thus, the classical iso-

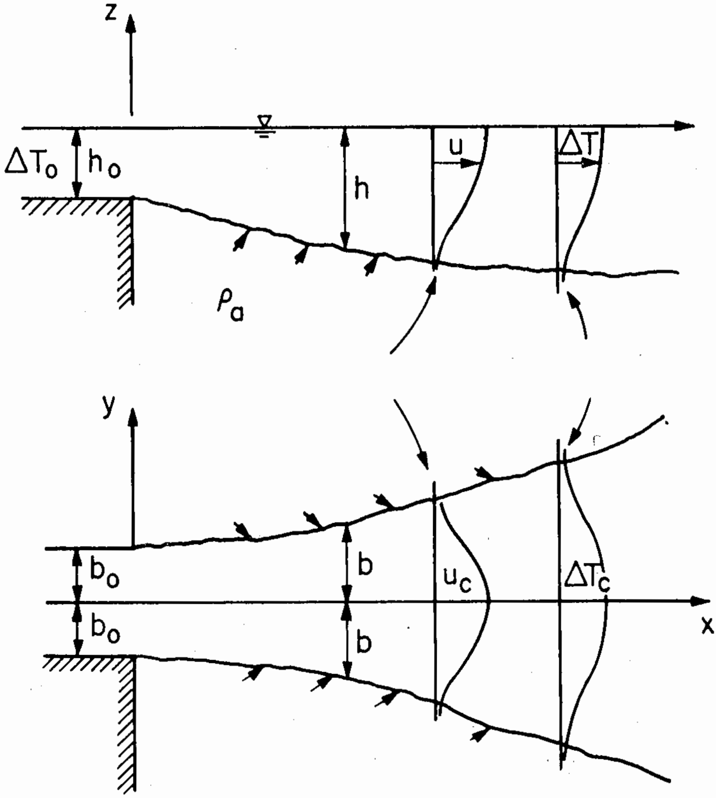

Fig. 1. - Buoyant Surface Jet - Near Field Zone

thermal jet equation expresses the balance between convective transport of axial velocity u and lateral and vertical diffusion.

In the other limiting case of a strongly buoyant surface jet, $F_L \rightarrow 1$, and the buoyancy terms are of the same order of magnitude as the convective terms. As the buoyant pressure terms act in both horizontal directions, both the x and y momentum equations must be retained. In contrast to the isothermal case, only vertical diffusion terms are significant for $F_L \rightarrow 1$. Thus, the lateral velocity and temperature distributions will not have jet-like sheared profiles.

If profile assumptions on the lateral and vertical distribution of u, v and T (or $\Delta\rho$) are made, the partial differential equations can be reduced to ordinary differential equations (jet integral analysis). However, as the equations should describe the transition in surface buoyant jets from large to small values of F_L, the following difficulties arise: a) Only the vertical jet profiles are truly of shear type. The lateral profiles change from shear type to a more uniform distribution (shear acting only at the edges). b) The distribution of the lateral velocity, v, is not readily specified in terms of centerline quantities. Some hypothetical assumptions have to be made in this respect.

Despite these theoretical restrictions it is useful to retain the assumptions of jet-like lateral profiles in order to describe the deviation due to buoyancy from the more non-buoyant behavior, without attempting to describe the limiting case of strongly buoyant behavior. The utility and range of applicability of such an approach have to be demonstrated by comparison with experiments. If this procedure is followed, the distributions for velocity and density are given by

$$u/u_c = f(\eta, \zeta);$$

$$\Delta T/\Delta T_c = \Delta\rho/\Delta\rho_c = g(\eta, \zeta);$$

where $\eta = y/b$ and $\zeta = z/h$ and f and g define bell-shaped jet distributions. After integration in the lateral direction the governing equations acquire the following general form

$$\frac{dQ}{dx} = c_1\alpha_o u_c h + c_2\alpha_v u_c b \tag{4}$$

$$\frac{dM}{dx} = \frac{dP}{dx} \tag{5}$$

$$\frac{dH}{dx} = -c_3 k \Delta T_c b \tag{6}$$

$$\frac{db}{dx} = \epsilon + \left(\frac{db}{dx}\right)_B \tag{7}$$

where c_1, c_2 and c_3 are profile-dependent coefficients, k is a coefficient for surface heat loss, ΔT_c is the excess centerline temperature of the surface and ϵ is the rate of lateral spreading for a non-buoyant jet.

The above equations are amenable to numerical solution using appropriate initial conditions at the discharge point. The integral quantities are defined as follows:

Volume flux $$Q = \int_A u \, d\eta \, d\zeta \tag{8}$$

Momentum flux $$M = \int_A \rho_a u^2 \, d\eta \, d\zeta \tag{9}$$

Pressure force $$P = \int_A \left[\int_{-\infty}^{\zeta} g\Delta\rho \, d\zeta \right] d\eta \, d\zeta \tag{10}$$

Temperature flux $$H = \int_A \Delta T \, u \, d\eta \, d\zeta \tag{11}$$

where A is the cross-sectional area of the jet.

The continuity equation (4), uses the entrainment concept proposed by Morton, Taylor and Turner (10) which relates the normal velocities at the jet boundary to the centerline velocity by means of a proportionality coefficient. α_o is the constant coefficient for lateral entrainment, and α_v is the variable coefficient for vertical entrainment which is a function of the local buoyant damping of turbulent entrainment, characterized by F_L, so that $\alpha_v = \alpha_o f(F_L)$ as indicated by the data of Ellison and Turner (2). Equation (5) expresses the balance between longitudinal momentum flux and buoyant pressure force. The heat conservation equation (6) allows for excess heat decay to the atmosphere. The jet spreading equation (7) represents the assumption that the buoyancy of the jet causes spreading of the jet width $\frac{db}{dx}_B$, in addition to the usual non-buoyant turbulent spreading, ϵ. Closure of the equations requires specification of $(db/dx)_B$ through the use of the lateral momentum equation. Different hypotheses are possible:

i) Stolzenbach and Harleman (17, 18) assume the local lateral velocity, v, to be proportional to the local lateral density gradient, $\partial\Delta\rho/\partial y$, and the local longitudinal velocity, u; the proportionality constant being equal to $(db/dx)_B$. This specification allows integration of the lateral momentum equation.

ii) Prych (12) and Stefan and Vaidyaraman (16) solve the lateral momentum equation under the simplifying assumption that $\left(\frac{db}{dx}\right)_B = f(F_L)$.

Buoyant Surface Jets — Experimental Results

Figure 2 shows a comparison of surface jet theoretical predictions and laboratory experimental results for a buoyant jet having an initial densimetric Froude number of 1.8. Good agreement is obtained for the longitudinal decrease of the excess surface temperature $\Delta T_c/\Delta T_o$ along the jet centerline and for the jet thickness. Both theories (refs. 17 and 12) tend to over-predict the lateral width of the jet when the ratio $\frac{b}{h} \sim 100$. The half-width ($b_{1/2}$) shown in Figure 2 is the distance from the centerline to the point at which the surface temperature excess is equal to ½ the centerline temperature rise. The half-depth ($h_{1/2}$) is defined in a similar manner. The over-prediction of the lateral width is probably due to two facts: (1) The retention of lateral jet-like profiles at small F_L;(2) the neglect of lateral shear $\overline{v'w'}$ in the lateral momentum equation which is done in all of the analyses described above.

Figures 3, 4 and 5 show examples of field data for a heated surface discharge at Pilgrim Nuclear Power Station collected by Pagenkopf, et al (11). Comparison between the field data and three-dimensional, steady state mathematical models is complicated by the fact that the densimetric Froude number of the discharge varies over a range from 2 to 11 during the six-hour change from high to low tide. Nevertheless, data obtained during a period of an hour on either side of the time of high tide represents reasonably steady state conditions. Figure 3 shows the centerline temperature decrease, for a jet near high tide having an initial densimetric Froude number of 2.2, as a function of longitudinal distance in comparison with the Stolzenbach-Harleman (17, 18) prediction. A plot of the observed vertical temperature distribution along the centerline of the plume is shown in Figure 4. This indicates that for the low Froude number jet there is very little interaction between the development of the jet plume and the sloping ocean bottom offshore of the discharge channel.

Frequently, thermal discharge criteria formulated by regulatory agencies are specified in terms of a permissible water surface area within a certain isotherm of excess temperature. The analytical models described above can be used to predict such areas as shown in Figure 5. The Stolzenbach-Harleman plume area prediction is shown in comparison with areas determined from successive field measurements taken during the period near high tide. The tendency to over-predict the isotherm areas having small values of $\Delta T_c/\Delta T_o$ is again evident.

Additional analytical and experimental studies on both the near and far-field behavior of buoyant surface jets are underway in the R. M. Parsons Laboratory. These include: The interaction of the jet plume with the bottom boundary of the receiving water, either an abrupt step or a gradual slope; the interaction of the jet plume with an along-shore current in the

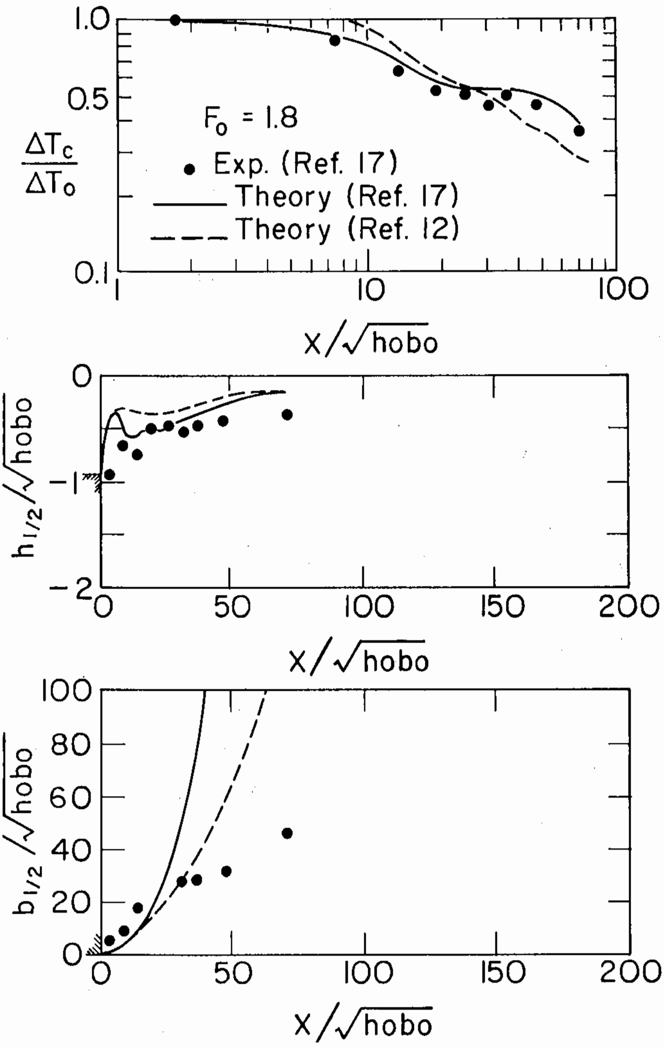

Fig. 2. - Comparison Between Laboratory Experiments and Theoretical Predictions: Buoyant Surface Jet, $F_o = U_o \left(\frac{\Delta\rho_o}{\rho_a} g h_o \right)^{-1/2} = 1.8$; $h_o/b_o = 0.87$ and $k/U_o = 6.2 \times 10^{-5}$

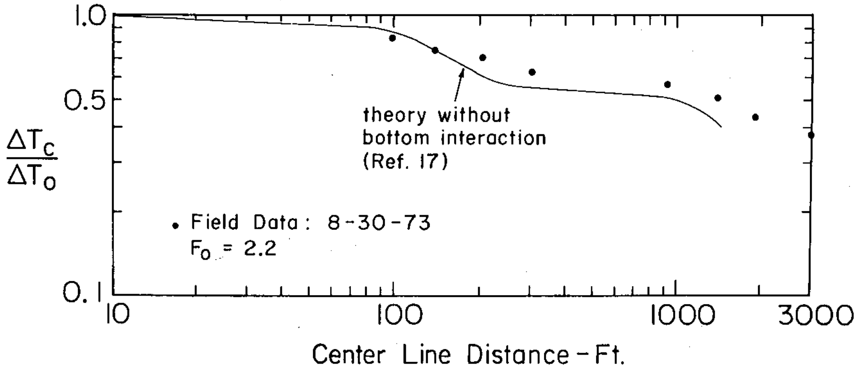

Fig. 3. - Comparison Between Field Data and Theory for Surface Excess Temperature Along Plume Centerline During High Tide at Pilgrim Nuclear Power Station: $F_o = U_o\left(\frac{\Delta\rho_o}{\rho_a} gh_o\right)^{-1/2} = 2.2$; $h_o/b_o = 0.47$ and $k/U_o = 1 \times 10^{-5}$

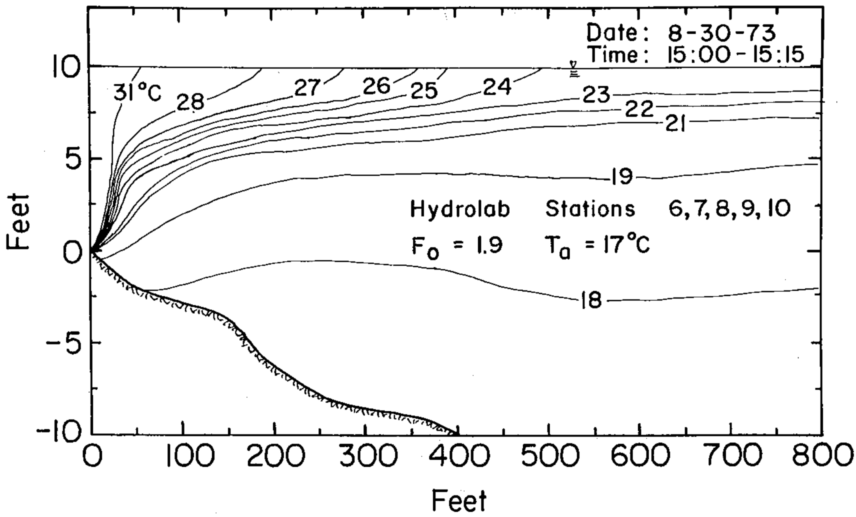

Fig. 4. - Observed Vertical Temperature Distribution Along Plume Centerline During High Tide at Pilgrim Nuclear Power Station

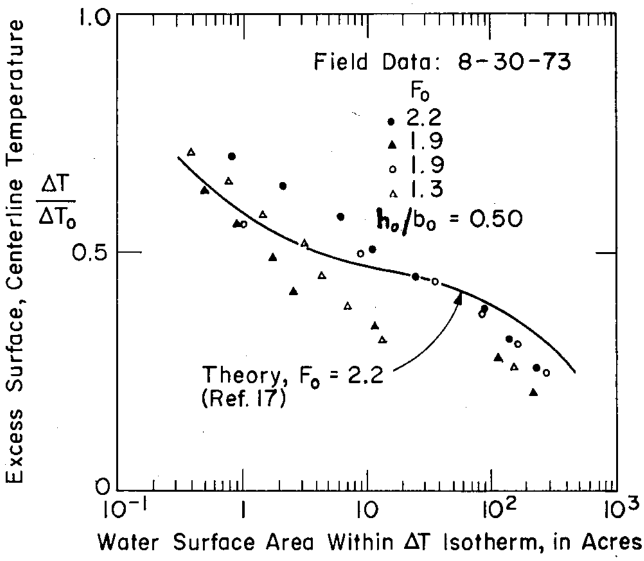

Fig. 5. - Comparison Between Field Data and Theory for Water Surface Areas Within ΔT Isotherms During High Tide at Pilgrim Nuclear Power Station

receiving water and the extension of the temperature prediction to the far-field where ambient currents, diffusion and surface heat loss dominate the process. Some preliminary results have been published by Harleman, Adams and Koester (4) in connection with studies for the Atlantic Generating Station, a floating nuclear power station off the New Jersey coast.

Multiport Diffusers in Shallow Water

Basic analytical and experimental studies on the mechanics of submerged multiport diffusers has been conducted by Jirka and Harleman (6, 7). A multiport diffuser consists of a pipeline, laid on the bottom of the receiving water, with many nozzles of diameter, D, attached at a regular spacing, ℓ. The individual round jets emanating from the nozzles interfere after a short distance and form a two-dimensional jet zone. It has been shown that the jet parameters in this two-dimensional zone are equal to those of an "equivalent slot diffuser" with slot width $B = \pi D^2/4\ell$ and equal discharge velocity, U_0. Using the concept of the "equivalent slot diffuser" reduces the number of dimensionless parameters characterizing a multiport diffuser and thus provides a means to compare different designs.

Over the years a considerable body of knowledge has been built up on the design of offshore multiport diffusers for sewage. Only recently have multiport diffusers been considered for thermal discharges in coastal waters and in the Great Lakes. The design problems for the two types of diffusers are quite different. Sewage diffusers generally require a dilution at the water surface of the order of 100. This has generally limited their application to water depths of 100 feet or more where the large dilution is due to the long trajectory of the buoyant jet rising toward the surface. The vertical thickness of the mixed zone at the surface is a relatively small fraction of the total depth as shown in Figure 6 (a) and (c) for vertical and non-vertical discharges in deep water. In contrast, thermal diffusers generally require a dilution at the water surface of the order of 10 and, particularly in the East coast and Great Lakes, are located in water depths much less than 100 feet. Another important difference is the relative buoyancy of the two types of discharges. In the case of a sewage diffuser, the density difference between the effluent and the receiving water is about an order of magnitude higher than for a heated discharge. These two factors, the greater depth and buoyancy, usually insure that the mixed layer at the surface is stably stratified and the near-field dilution is little affected by diffuser orientation or currents in the receiving water.

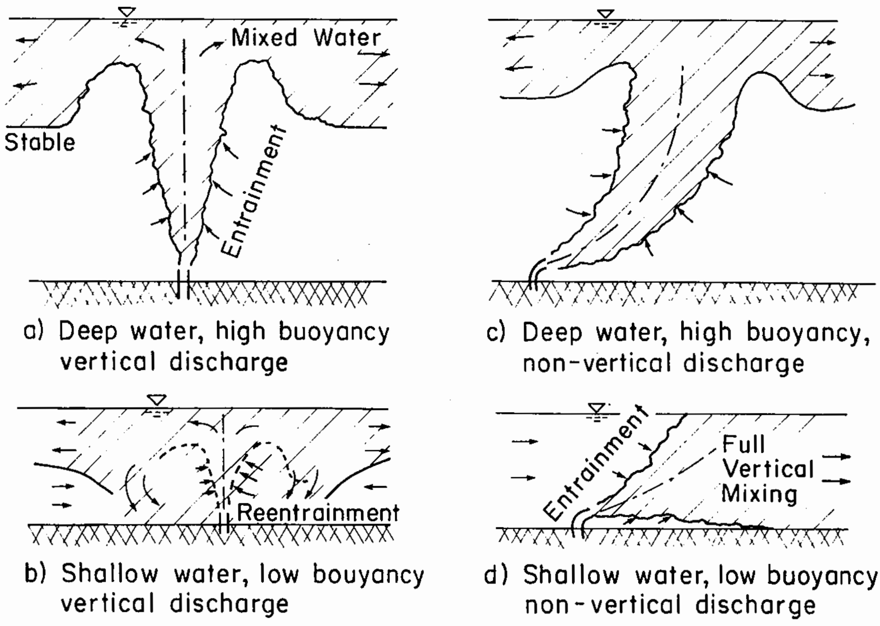

Fig. 6. - Submerged Buoyant Jets: Effects of Relative Buoyancy, Submergence and Angle of Discharge

Stability Analysis and Prediction of Dilution

The analytical and experimental studies of Jirka and Harleman (6) have shown that, because of low buoyancy, thermal diffusers in shallow water almost always exhibit an unstable stratification in the near field, thus leading to reentrainment or full vertical mixing as shown in Figures 6 (b) and (d). Prediction of the dilution of such diffusers is based on the two-dimensional stratified flow regions shown in Figure 7. These regions are (1) a

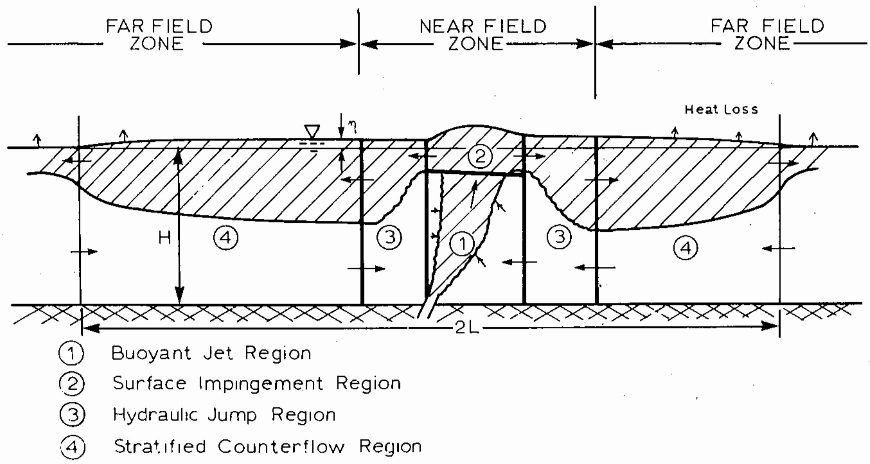

Fig. 7. - Zones of Stratified Flow in Two-Dimensional Analysis of Submerged, Buoyant Slot Jet

buoyant jet region, (2) a surface impingement region, (3) an internal hydraulic jump, and (4) a stratified counterflow region. These regions, each with different hydrodynamic properties allowing simplifying approximations, can be analyzed separately. Successive matching of the individual solutions yields a description of the total flow field. The objectives of the analysis are to determine the limiting condition of a stable flow field, that is, the criterion line between stable and unstable regimes and the dilutions which occur in the two regimes.

Stability is primarily dependent on regions (1), (2) and (3). Inspectional analysis of the problem gives the following governing parameters:

Slot densimetric Froude number: $$F_s = U_o \left(\frac{\Delta\rho_o}{\rho_a} gB \right)^{-1/2} \qquad (12)$$

Relative water depth: H/B

Angle of discharge: θ_o

where $\Delta\rho_o$ = initial density difference and H = water depth.

(i) Buoyant Jet: A hydrostatic pressure distribution is assumed in this region. This is tantamount to the assumption that the pressure disturbance due to the rise in surface elevation as a result of impingement is limited to the impingement region. This assumption is essentially verified by experiments. A buoyant jet analysis, utilizing the entrainment concept, is performed to give predictions of jet dilutions and trajectories.

(ii) Surface Impingement: The surface impingement region provides the transition between the jet flow, with a strong vertical component, and the horizontal spreading motion. The process is complex and is most conveniently analyzed by a control volume approach, using a continuity equation, a horizontal momentum equation and two energy equations (account being taken of the energy loss in the flow transformation). Results of the analysis give the thickness of spreading layer, h_1, and thus the elevation to which effective jet entrainment occurs. Furthermore, the dynamic characteristics of the spreading layer, represented by a densimetric Froude number

$$F_1 = u_1 \left(\frac{\Delta\rho}{\rho_a} gh_1\right)^{-1/2} \tag{13}$$

where u_1 = layer velocity, $\Delta\rho$ = relatively density difference between upper and lower layer, can be calculated. The upper layer thickness is about ⅙ of the total water depth and is only weakly dependent on F_S and H/B. The Froude number F_1 is strongly dependent on F_S and H/B and generally supercritical.

(iii) Internal Hydraulic Jump: Experimental observations indicate that following the surface impingement the thickness of the surface layer suddenly increases by the formation of an internal hydraulic jump.

The governing equations for internal hydraulic jumps have been derived by Yih and Guha (21). A simplified solution for small density differences has been obtained by Jirka and Harleman (6). The equations indicate that for certain upstream conditions no solution is possible, that is, a stable subcritical downstream condition does not exist.

The dividing line between stable and unstable conditions (in which reentrainment occurs) is shown in Figure 8 for a vertical discharge. The parameters defining stability are the relative submergence H/B and the equivalent slot densimetric Froude number F_S given by equation (12).

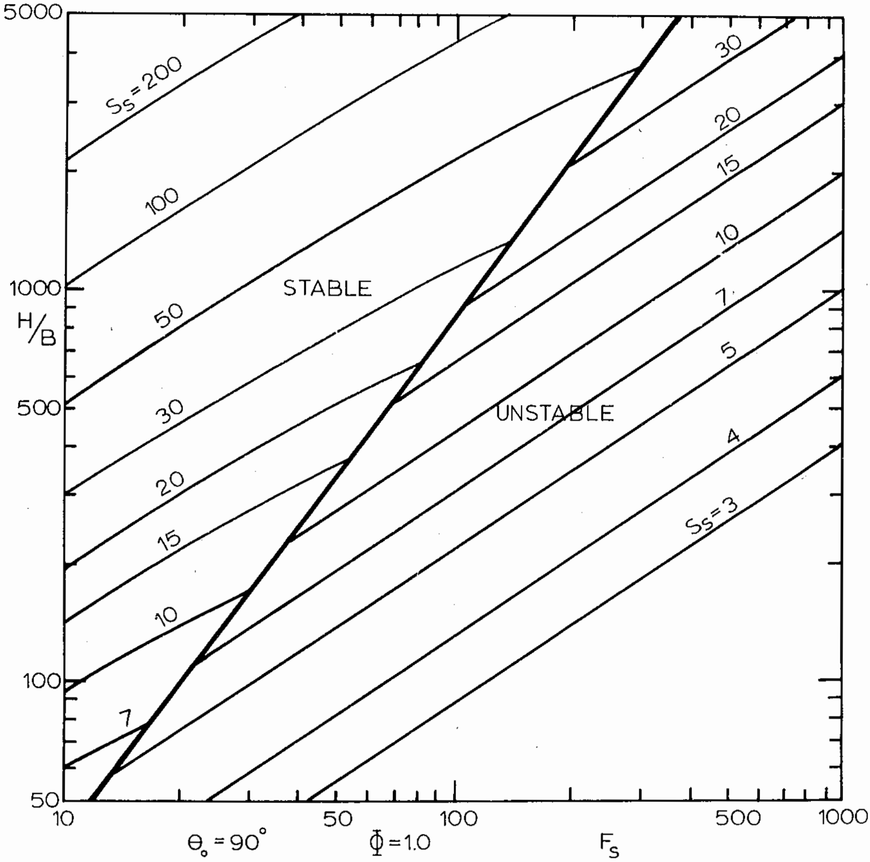

Fig. 8. - Surface Dilution S_S as a Function of Relative Submergence H/B and Slot Densimetric Froude Number F_S: Dividing Line Between Stable and Unstable Regimes

The variable of major importance in the design of submerged multiport diffusers is the near-field surface dilution S_S. For the case of a stable near-field S_S can be directly obtained from the buoyant jet analysis, if account is taken of the thickness of the impingement layer.

Whenever the near-field is unstable and reentrainment into the jet zone occurs, a simple buoyant jet analysis is not valid. In this case the near-field dilution is directly dependent on the stratified counterflow system in the far-field (region 4). This counter-flow presents a balance between buoyancy forces and frictional forces (dependent on geometry, boundary and interfacial friction). Values for S_S, evaluated through analysis of these different far-field conditions by Jirka and Harleman (6) are shown in Figure 8. Upon crossing the dividing line from the stable to the unstable zone, the surface dilution decreases due to reentrainment. For typical conditions encountered in condenser water discharges, thermal diffusers are almost always in the unstable zone. For a specified dilution S_S, it is generally desirable to design as close to the dividing line as possible in order to promote vertical stratification. This requires reducing the diffuser nozzle velocity U_O; however, practical considerations of internal diffuser hydraulics dictate that U_O should not be less than approximately 2 m/sec.

Three-dimensional aspects of multiport diffusers, including the interrelation between the diffuser length, the water depth and the total waste heat load were also considered by Jirka and Harleman (6). Other important design considerations are the orientation of the diffuser nozzles and the diffuser axis with respect to currents in the receiving water. In the case of reversing ambient currents (such as unsteady wind-driven or tidal currents), it is desirable to design the diffuser to promote stratification and to be effective in either current direction. This objective can be met by the use of alternating direction nozzles (no net horizontal momentum). The studies described above are useful in making preliminary design estimates and for screening alternative discharge schemes for further investigation in a hydrothermal scale model. A number of such model studies for specific power plant sites have been carried out in the R. M. Parsons Laboratory (3, 8, 20).

Analytical and experimental analysis of the four flow regions shown in Figure 7 for a single round buoyant jet discharging vertically upward has been conducted by Lee (9). Ungate (19) has experimentally investigated the effect of Reynolds number, in the laminar-turbulent transition range, on the entrainment of buoyant jets covering a wide range of densimetric Froude numbers. This study is helpful in the choice of length scales for physical models.

Far-field Temperature Distribution in Unsteady Flow

The mathematical model developed by Harleman, Brocard and Najarian (5) is a one-dimensional, transient numerical model in which water temperature is a function of longitudinal distance and time. The model is applicable to rivers, shallow reservoirs and estuaries in which temperature variations in the longitudinal direction are more important than the vertical.

In post-operational studies of once-through cooling processes, it is generally necessary to demonstrate compliance with established thermal discharge criteria by determining the incremental temperature rise due to condenser water discharge. This implies that ambient temperatures are known and can be subtracted from observed temperatures to determine the incremental rise. Because of buoyancy and unsteady flow reversals (due to tidal motion in estuaries and hydroelectric regulation in rivers and reservoirs) thermal effects of power plants may extend both upstream and downstream of the site. Therefore, it is difficult to determine natural or ambient water temperature by direct measurement. Temperature prediction models are also required to provide input to water quality models where the rate constants governing biochemical reactions are temperature dependent.

The flow field is unsteady and non-uniform and is determined by simultaneous solution of the continuity and momentum equations. The governing equations for one-dimensional flow in a variable area channel are: the continuity equation

$$\frac{\partial A}{\partial t} = \frac{\partial Q}{\partial x} = q \tag{14}$$

and the longitudinal momentum equation

$$\frac{\partial}{\partial t}(AU) + \frac{\partial}{\partial x}(QU) = -gA\frac{\partial h}{\partial x} - g\frac{Q|Q|}{AC^2R_h} \tag{15}$$

where, x = distance along longitudinal axis; t = time; h = elevation of water surface with respect to horizontal datum; Q = cross-sectional discharge; q = lateral inflow per unit length of channel; U = average cross-sectional velocity in the channel, = Q/A; g = acceleration of gravity; A = cross-sectional area of channel; C = Chezy coefficient; and R_h = hydraulic radius of channel.

In equation (15) a term which represents the effect of a longitudinal density gradient and which is significant only within the salinity intrusion region of estuaries has been ignored. Boundary conditions must be specified (either water surface elevation h or discharge Q) at the upstream and downstream sections of the river, reservoir or estuary being modeled. The solution of equations (14) and (15) for Q = f(x,t) and A = f(x,t) can be obtained numerically by schemes as described by Dailey and Harleman (1). The solution requires the specification of initial conditions for h and Q and advances in time in accordance with the values of the time varying boundary conditions.

The quantity ρcT represents the amount of heat per unit volume of water and the one-dimensional conservation of heat equation is

$$\frac{\partial}{\partial t}(A\rho cT) + \frac{\partial}{\partial x}(Q\rho cT) = \frac{\partial}{\partial x}\left[AE_L\frac{\partial}{\partial x}(\rho cT)\right] + \phi_n b + S \tag{16}$$

where, ρc, the product of density and specific heat of water has the dimensions of heat content, BTU (joules) per unit volume per degree; T = water temperature; E_L = longitudinal dispersion coefficient; ϕ_n = net heat influx per unit water surface area, BTU/ft².sec (joules/m².sec); b = width of water surface and S = source term accounting for waste heat discharges and lateral heat input from tributary inflows. For a power station with once-through cooling S is the heat rejection rate in BTU/sec (joules/sec) divided by the effective longitudinal length over which the heat injection occurs.

The steady state form of eq. (16) has been used to determine the longitudinal temperature distribution in rivers under natural and artificial heat inputs. Since ρc can be assumed constant and the dispersive term can be neglected in rivers, the steady form for a reach with no tributary ($\partial Q/\partial x = o$) can be written as

$$\frac{dT}{dx} = \frac{\Phi_n b}{\rho c Q} \tag{17}$$

This first order differential equation can be solved by specifying a temperature boundary condition at the upstream end such that $T = T_o$ at $x = o$. Additional simplifications to facilitate an analytical solution include the assumption that the surface width b is constant and linearization of the net heat flux into the water surface.

$$\Phi_n = -K \quad T - T_E \tag{18}$$

where K is the surface heat transfer coefficient in BTU/ft².day.°F (joules/m².day.°C) and T_E is the equilibrium temperature for which $\Phi_n = o$. The solution of equation (17) is

$$T = T_E + \left(T_o - T_E\right) \exp\left(-\frac{bKx}{\rho c Q}\right) \tag{19}$$

provided both K and T_E are independent of x.

For unsteady flow problems with time varying meterological conditions, the solution of equation (16) must be carried out simultaneously with the continuity and longitudinal momentum equations (14 and 15) by numerical techniques. In the unsteady case the linearization of the surface heat flux ϕ_n by means of equation (18) is of little value since both K and T_E are functions of time. Since meterological data are necessary to determine T_E, the same data can be used to compute ϕ_n in equation (16) directly. Ryan and Harleman (13) and Ryan, et. al. (14) have made a careful study of surface heat transfer mechanisms and have proposed formulae by which ϕ_n can be calculated as a function of time from standard meterological observations. The net influx of heat per unit surface area is the summation of five terms: the net incident solar (short wave) radiation; the net incident atmospheric (long wave) radiation; the long wave, back radiation from the water surface; the evaporative and conductive heat fluxes.

The transient model has been used to predict ambient temperatures in Conowingo Reservoir (5) on the Susquehanna River. Unsteady flows, including flow reversals, occur due to the transient operation of a pumped-storage plant and the hydroelectric power stations at the upstream and downstream boundaries of the reservoir. The objective of the study was to determine the temperature rise (above ambient) due to the discharge of waste heat from Peachbottom No. 2 Atomic Power Station which is located on the reservoir. The mathematical model computes natural reservoir temperatures under given meterological conditions and boundary conditions. The temperature rise due to Peachbottom is determined by subtracting the calculated ambient temperatures from temperatures measured in the reservoir during operation of the nuclear power plant.

The reservoir length is about 12 miles (19.2 km) and values of $\Delta x = 1500$ ft. (460 m) and $\Delta t = 12$ min. were used in the unsteady flow calculations. For the temperature calculations the same time step was used; however, Δx was reduced to 500 ft. (165 m).

Velocities and temperatures were measured in Conowingo Reservoir during 1972 prior to the operation of Peachbottom No. 2. A comparison of calculated and observed mean velocities at two different cross sections for a three day period in September 1972 is shown in Figure 9. Flow reversals due to the hydroelectric operation are clearly indicated. A comparison of the calculated and observed longitudinal temperature profile at the beginning of September 3, 1972 is shown in Figure 10. Another verification of the ambient temperature prediction model was made during a 10 day period in April 1972. Figure 11 shows the predicted and observed temporal variations in temperature at a section located about 5 miles below the upstream boundary.

Conclusions

A number of analytical and experimental techniques for predicting water temperature distributions due to waste heat discharges have been discussed. Predictive techniques are needed in the preparation of environmental impact statements for pre-operational site studies in order to evaluate the economic and environmental costs of alternative cooling water systems. These techniques are also useful in post-operational studies, inasmuch as field observations can be carried out only under a limited number of ambient conditions. Mathematical models can be used in interpreting field data and for providing additional information for receiving water conditions other than those measured.

The use of mathematical and/or physical models for the planning and design of field monitoring programs has received relatively little attention. It is suggested that a considerable amount of time and expense could be saved by making use of temperature prediction models in planning field surveys.

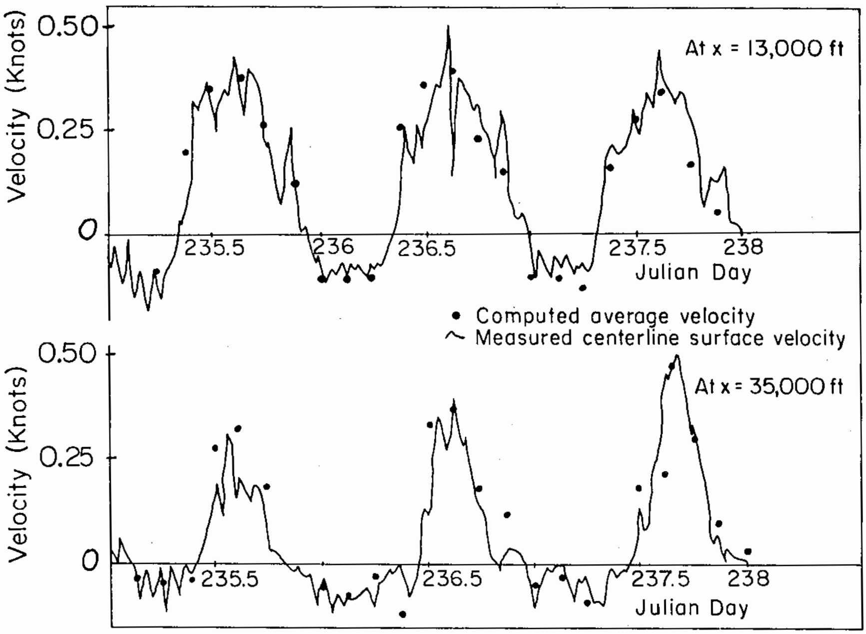

Fig. 9. - Comparison Between Measured and Predicted Velocities in Conowingo Reservoir from September 1-3, 1972: Upper Curve at x = 13,000 ft., Lower Curve at x = 35,000 ft.

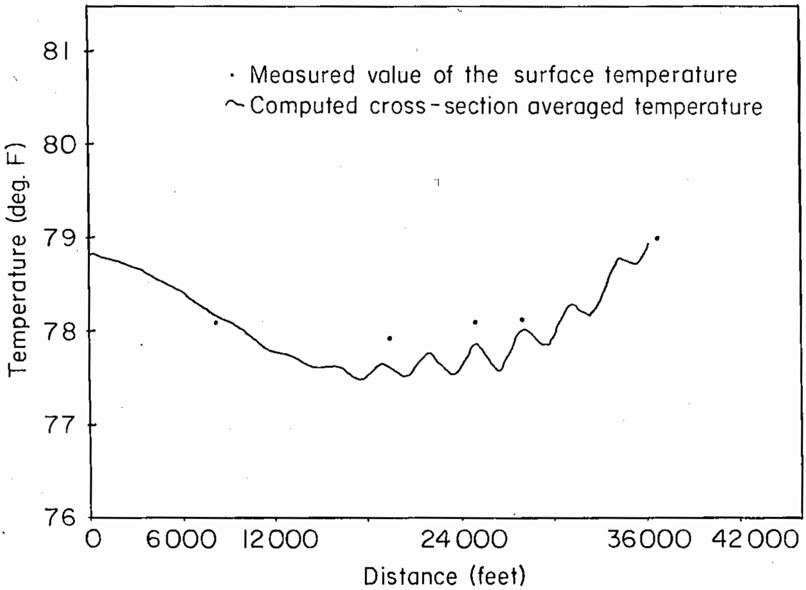

Fig. 10. - Comparison Between Measured and Predicted Longitudinal Temperature Profile in Conowingo Reservoir at 0 hrs. on September 3, 1972

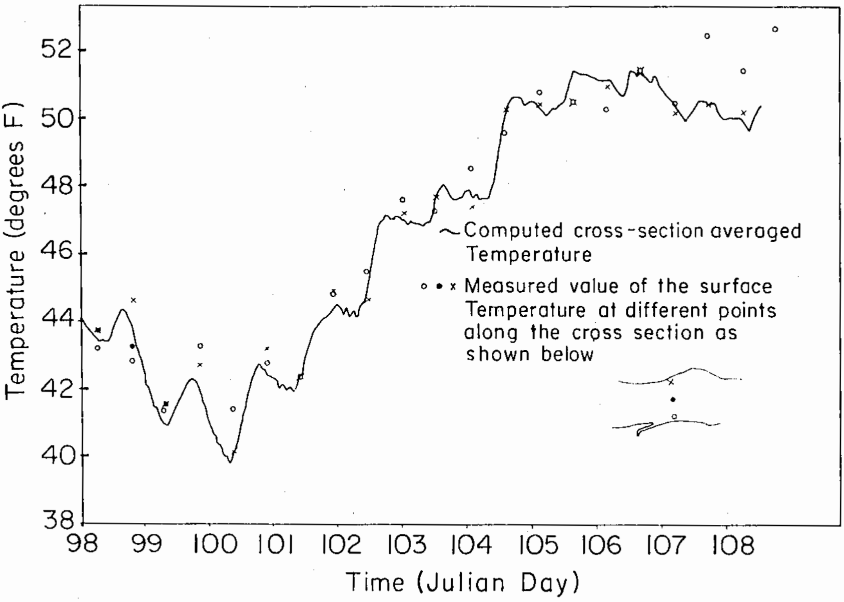

Fig. 11. - Comparison Between Measured and Predicted Temperatures in Conowingo Reservoir from April 8-18, 1972 at x = 25,000 ft.

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Appendix II. - Notation

The following symbols are used in this paper:

A	=	cross sectional area;
b	=	lateral width of jet;
$b_{1/2}$	=	value of b on water surface where $\Delta T = \Delta T_c/2$;
B	=	slot width;
c	=	specific heat of water;
C	=	Chezy coefficient;
D	=	nozzle diameter;
E_L	=	longitudinal dispersion coefficient;
F_1	=	densimetric Froude number of spreading layer;
F_L	=	local densimetric Froude number;
F_S	=	slot jet densimetric Froude number;
g	=	acceleration of gravity;
h	=	vertical thickness of jet;
$h_{1/2}$	=	value of h where $\Delta T = \Delta T_c/2$;
h_1	=	vertical thickness of spreading layer
H	=	temperature flux; or total water depth;
k	=	surface heat loss coefficient;
K	=	surface heat transfer coefficient;
ℓ	=	nozzle spacing
M	=	momentum flux
p_d	=	dynamic pressure
P	=	pressure force

q	=	lateral inflow per unit length of channel;
Q	=	volume flux
R_h	=	hydraulic radius;
S	=	source term for waste heat discharge;
S_s	=	near-field surface dilution;
t	=	time;
T	=	water temperature
T_E	=	equilibrium temperature;
T_o	=	water temperature at $x=0$;
ΔT	=	local temperature difference with respect to ambient;
ΔT_c	=	value of ΔT at jet axis on water surface;
u,v,w	=	velocity components in x,y,z directions;
u', v', w'	=	turbulent velocity fluctuations in x,y,z directions;
u_i	=	horizontal velocity of spreading layer;
u_c	=	value of u at jet axis on water surface;
U	=	average cross-sectional velocity;
U_o	=	nozzle discharge velocity;
x,y,z	=	rectangular Cartesian coordinates;
α_o	=	lateral entrainment coefficient;
α_v	=	vertical entrainment coefficient;
ϵ	=	rate of lateral spreading for a non-buoyant jet;
ζ	=	z/h;
η	=	y/b;
ϕ_o	=	discharge angle of slot;
ρ_a	=	ambient density;
$\Delta\rho$	=	local density difference with respect to ambient;
$\Delta\rho_c$	=	value of $\Delta\rho$ at jet axis on water surface;
$\Delta\rho_o$	=	initial density difference with respect to ambient; and
φ_n	=	net heat influx per unit water surface area

THE USE OF COMPACTED FILL FOUNDATION SCHEMES FOR MULTI-STORY BUILDINGS IN THE NEW ENGLAND AREA

by
Rebecca Grant*

(Presented before the Geotechnical Group of the BSCES/ASCE on May 1, 1974.)

Introduction

The purpose of this paper is to review the use of foundation schemes for multi-story buildings in the New England area which incorporate shallow footings or mats on compacted fill. This subject is of importance to foundation engineers since some regulatory agencies express hesitation and even flat refusal to consider this type of foundation design for multi-story structures. Because experience has shown foundations on compacted fills to be satisfactory and economical in many instances, the reluctance to adopt such a design is surprising. Accordingly, it was decided to poll local engineers to learn their opinions and their experiences with such foundations. This was accomplished by sending a questionnaire to geotechnical and structural engineers in this region who are involved in foundation design. The returned questionnaires indicated that the use of compacted fill foundation schemes for heavy foundations including multi-story structures was generally accepted as a good engineering design. The conclusions of this questionnaire are presented in this article for the benefit of our engineering colleagues both in private practice and governmental agencies.

Background

First the term "compacted fill" will be clarified and situations in which such a foundation scheme is advantageous will be briefly described.

Compacted fill is used to create a raise in grade and/or to replace unsuitable materials and the first step in preparing a site is to remove all unsuitable material (compressible soils, loose fill, trash, etc.). The next step is to place the fill.

For compacted fills supporting foundations, a granular fill material is generally used. Often it will consist of a relatively well-graded naturally occurring sand and gravel with less than 15 percent passing the U.S. No.

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200 sieve. A typical specification for fill material in accordance with the Boston Building Code is:

Sieve Size	Percent Finer
6-inch	100
No. 10	30-95
No. 40	10-70
No. 200	0-8

This granular fill is placed in lifts and compacted to a specified degree. Approval of each lift is based on performance specifications, such as: percent compaction as related to a maximum dry unit weight, or satisfactory behavior when proof-rolled. Sometimes, a specific compactive effort such as the number of passes of certain kinds of equipment may be required. A major criterion for compacted fill is that the work be supervised by qualified engineering personnel.

Several situations commonly occur in which a foundation scheme utilizing shallow footings on compacted fill is advantageous. For sites where the depth to a suitable bearing stratum is less than ten feet, a compacted fill foundation scheme is often the most economical approach. The cost of excavating the unsuitable upper materials and replacing them with compacted fill so that shallow footings and a slab-on-grade can be used is frequently lower than the cost of either extending footings to underlying firm stratum together with a structural floor slab, or using deep foundations carried to a lower stratum. Sites with shallow peat deposits and loose miscellaneous fills are frequently treated in this manner.

Another instance in which a compacted fill foundation design is beneficial occurs where a suitable bearing stratum is underlain by a deep compressible soil. By placing the footings on compacted fill above the firm stratum, the concentrated stresses from the footing loads have a greater depth in which to dissipate. Thus the underlying compressible layers experience a lesser stress increase and consequently less settlement will occur. A good example of this occurs in Boston, where the stiff yellow clay crust (bearing stratum) overlies the softer Boston blue clay (deep compressible soil). Footings extending through the miscellaneous fill and organic soils and bearing directly on the yellow clay generally cause larger stress increases in the blue clay than do footings bearing on compacted fill which has replaced the unsuitable materials above the yellow clay.

Compacted fills can also be used to reduce differential settlement when the density of the bearing stratum is somewhat erratic. The compacted fill layer acts as a raft, spreading the stresses from the footings over a large area and arching over soft areas. This reduces the chance of one footing settling much more than an adjacent footing.

Questionnaire Results

The results of the opinion poll regarding the use of compacted fill foundation schemes for multi-story buildings are presented below. Of the nineteen returns, six were from structural engineers, and thirteen from soils engineers.

The five questions together with a synopsis of the answers are as follows:

Question No. 1. "Should the height of the building be the controlling factor governing the use of a foundation scheme involving footings on compacted fill?"

Seventeen engineers replied no, one replied yes, and one no answer. Thus, the overwhelming consensus is that the number of stories of a building is not the critical factor in determining the applicability of a compacted fill foundation scheme.

Question No. 2. "On how many occasions have you or your firm designed and inspected a foundation scheme involving footings or mat on compacted fill for buildings over two stories?"

Six engineers replied that they had designed no buildings over two stories bearing on compacted fill. Eleven had designed well over 100 such structures. Two engineers did not reply. Details on some of the structures which were designed were included and these will be described in a subsequent section. All of the structures which were described are reported to have performed satisfactorily. Thus it would appear that it is a well accepted practice.

Question No. 3. "Would you recommend the use of the compacted fill foundation scheme for buildings over two stories?"

Eighteen of the engineers recommended the use of a compacted fill foundation scheme, and one did not. Again, this indicated general acceptance of such a foundation design for multi-story structures.

Question No. 4. "Have you encountered any problems peculiar to foundations on compacted fill?"

Thirteen replied that they had had no problems, and five replied yes. However, it is important to note that of the five engineers who reported having problems, only one had ever designed a compacted fill foundation scheme for a multi-story structure. Thus, the occurrence of problems cannot be correlated with the use of compacted fill for multi-story buildings.

The second part of this question asked for a description of any problems which had been encountered. In the responses, three categories of difficulties were mentioned. The first type of problem concerned the negative attitudes of many members of the construction industry towards compacted fill and the need to convince them of the advantages of designs involving its use. The second area in which problems arose was during the excavation and compaction stages of the construction. These difficulties included dewatering the excavation, especially when fine-grained soils occurred at the bottom, and obtaining proper compaction of the fill. The third category was post-construction difficulty; only one engineer cited problems with settlement due to inadequate compaction of the fill.

From the problems which were described it may be concluded that the field liaison person is extremely important in jobs involving compacted fills, both to allay any doubts on behalf of the contractor and owner, and to ensure that satisfactory results are achieved. Technical problems with compacted fill foundations generally surface during the construction stages and are of a procedural type. An experienced soils technician or engineer is required to handle these.

Question No. 5. "What aspects of foundations on compacted fills require the most careful consideration?"

The overwhelming majority of engineers stressed the importance of good field inspection to verify that all poor quality material was excavated and that the fill was compacted properly. The presence of a qualified soils engineer or technician is felt to be absolutely imperative to insure that a high quality compacted fill is attained. The second most important factor is the type of material being used for fill. Other factors mentioned as requiring careful consideration included:

a. The nature of the soils underlying the fill.
b. The bearing capacity of the fill and underlying soils.
c. The drainage of the excavation.
d. The stress changes in the underlying soil due to the fill.
e. The post construction monitoring.

The above responses indicate that foundation designs for multi-story buildings utilizing shallow footings on compacted fill are widely accepted by engineers practicing foundation design in this area. As in the construction of any type of foundation, qualified inspection is required to insure a satisfactory and safe product.

Examples of Multi-Story Buildings on Compacted Fill

As mentioned earlier in this paper, over one hundred buildings in the Boston area designed with compacted fill foundations were reported in the questionnaires. A partial listing of these structures, their statistics, and their performance is presented in Table I. Many three to eight story buildings, and even some structures as high as twenty-two stories, have been constructed on compacted fill. The design footing pressures vary from two to five tons per square foot according to the nature of the fill material and the underlying soil strata. The fill thicknesses range from zero to twenty-six feet; however, the fill is generally less than ten feet thick. In all instances satisfactory behavior of the completed structure was reported.

Several interesting cases are included in Table I. In one case where the structure is bearing partially on natural ground and partially on compacted fill, no problems with differential settlements were encountered even with a sensitive precast structure. In another case a fill thickness ranging from 0 to 26 feet was reported with no problems with differential settlement.

Unfortunately, several of the structures listed in Table I which were designed with compacted fill foundation schemes were not constructed due to codes prohibiting the use of such a foundation scheme. In some cases more costly and less satisfactory foundation alternatives were adopted or the project was abandoned.

Summary

In summary, this paper has attempted to present representative information on the use of compacted fill foundations for multi-story structures. It is hoped that a cooperative effort between foundation engineers and public agencies will result in the modification of regulatory codes to reflect the state of the art in foundation design.

TABLE I

Name and Location of Building	Height	Design F'ting Press. (TSF)	Thickness of Compacted Fill (feet)	Nature of Fill	Settlements or Conclusions Regarding Performance of Building
B.H.A. Apartment Building, Roxbury	8 stories	3-5	10	granular, less than 15% passing U.S. #200 sieve	satisfactory
Lexington Street Apartments, Woburn	3 stories	2	10	silty sand, some gravel	satisfactory
Turn Key Housing Quincy	8 stories	4	6-8	fine to coarse sand	not built
128 Office Building, Newton	5 stories	2	0-12	granular, less than 15% passing U.S. #200 sieve	satisfactory
Back Bay Manor, Boston	22 stories	4	0-10	granular, less than 5% passing U.S. #200 sieve	satisfactory
Queen Anne Nursing Home, Gloucester	3 stories		7	silty sand-gravel	problem with silty fine sand subgraded during construction, satisfactory after construction
New Greater Boston Academy, Stoneham	3 stories		0-several	clean sand and gravel	satisfactory
Brockton High School	3 stories		2.5.4		satisfactory
Broadlawn Apartments, West Roxbury	5 stories	2	0-several		satisfactory
Mill Street Apartments Weymouth	5 stories	3	1-5	granular, less than 5% passing U.S. #200 sieve	satisfactory
Franklin Street Housing, Holyoke	7 stories	2	0-10	granular, less than 8% passing U.S. #200 sieve	excellent performance for 7 years

TABLE I

Name and Location of Building	Height	Design Fting. Pres. (TSF)	Thickness of Compacted Fill (feet)	Nature of Fill	Settlements or Conclusions Regarding Performance of Building
Hudson Street East Project Worcester	17 stories	4	2-3	granular	not constructed
Housing for Elderly, Dover, New Hampshire	7 stories	3	3-6	granular	satisfactory
Waterview Villa Nursing Home	45 feet	2	10-12		not constructed
Criss Cadillac Company, Providence, R.I.	24 feet	2	0-26		no visible settlement no cracking due to differential settlement
Hek Headquarters, Lexington, Mass.	3-4 stories	3	0-12		no complaints with sensitive precast structure
Xerox 307 Webster, N.Y.	8 stories	3	4-9		under construction
Overlook Hospital Garage	6 stories	2	0-18		partly on rock no problems with differential settlement
Portions of MFB Insurance Building, Johnson, RI	3 stories	4	up to 14		no settlement
—————————	40 feet	3	8		under construction
Greater Lowell Voc. Technical High School	3 stories	2	3		satisfactory
Melrose High School	3 stories	2.5	3		satisfactory
Pittsfield, Mass.	5 stories	5	10-12		satisfactory
Binghampton, N.Y.	2 and 3 stories	5	less than 10	rock fill	satisfactory

TABLE I

Name and Location of Building	Height	Design Fting. Pres. (TSF)	Thickness of Compacted Fill (feet)	Nature of Fill	Settlements or Conclusions Regarding Performance of Building
Blanchard Road Cambridge	4 stories	1.5-2.5	2-4	granular	satisfactory
Mystic Towers	8-14 stories	2-4	8	granular	maximum settlement after ½ yr. of construction is 1 inch; maximum angular distortion is 1/1500
over 40 structures	1-9 stories	2.5-3	1-20		satisfactory
Xerox 200 Webster, N.Y.	1 story with 500 ton column loads	3	4-15	clayey till fill material placed in winter	maximum settlement was 1.1 inch

ENGINEERING THE BEAR SWAMP PROJECT

by
Robert W. Kwiatkowski[1] and David R. Campbell[2]

(Presented at the meeting of the Hydraulics Group of the Boston Society of Civil Engineers Section, ASCE, 1974)

Introduction

The intent of this presentation is to describe some of the principal civil engineering features of the Bear Swamp Pumped Storage Project. The goal, here, is to convey in general terms an understanding of the overall project by relating a brief history of the project background together with the most salient of the hydrological aspects, and a broad description of the various project structures.

Although some may argue that a hydroelectric project is primarily an electrical feat while others insist that it is mechanical, all civil engineers realize that the prime-moving discipline required for a successful hydroelectric development must be civil engineering.

Thus, in order to avoid confusing verbosity, all mechanical and electrical details will be excluded from this discussion. Furthermore, with a project of this magnitude, it is simply not feasible to attempt an exploration of individual features in great detail without losing perspective of the total project.

General

As shown on Fig. 1, the Bear Swamp Pumped Storage Project is located on the Deerfield River in the northwestern corner of Massachusetts, straddling the towns of Florida and Rowe. It is situated about 18 miles west of Greenfield and about 8 miles east of North Adams, very close to the eastern portal of the historic Hoosac Tunnel. The nearest main road to the Project is the well-known Mohawk Trail.

The project is owned by the New England Power Company, a subsidiary and the principal wholesale company of the New England Electric System. The New England Electric System also includes the New England Power Service Company, the Massachusetts Electric Company, the Narragansett Electric Company, and the Granite State Electric Company.

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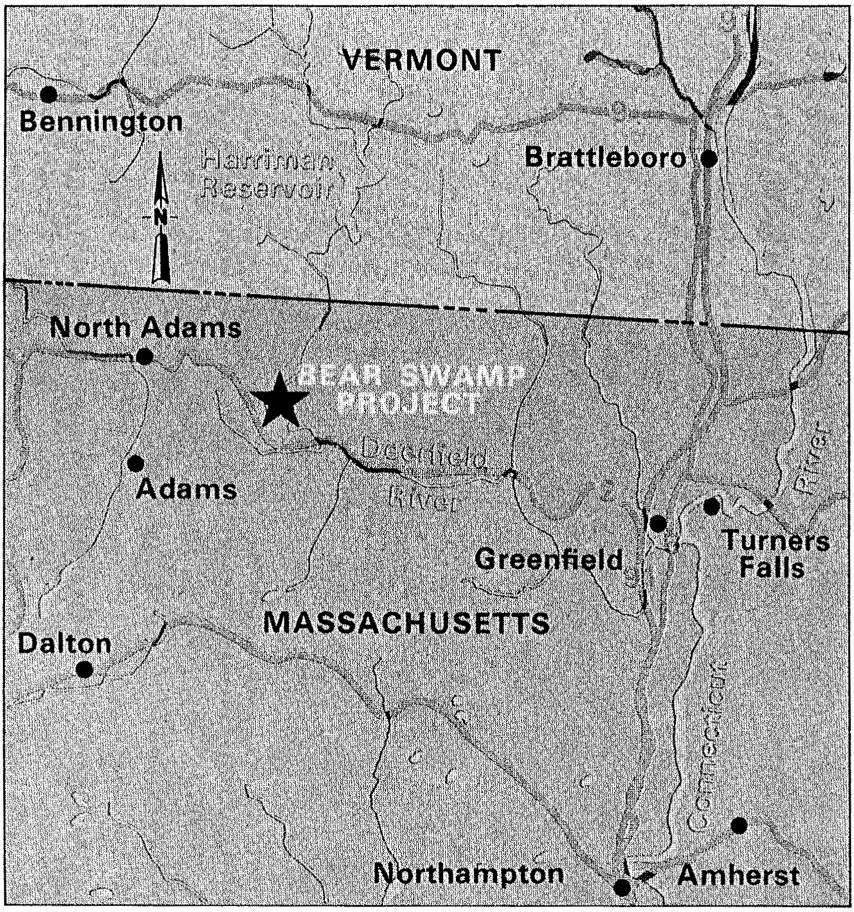

Fig. 1 - General Location Plan

The total project cost was approximately 125 million dollars, which includes, in addition to the 600 megawatt pumped storage plant through the high side of the main transformers, two small conventional hydro stations, namely, the 10 megawatt Fife Brook Station and the 15 megawatt Deerfield No. 5 Redevelopment. The total cost also represents all direct and indirect costs, exclusive of transmission facilities.

The principal features of the Project, as shown on Fig. 2, include an upper storage reservoir enclosed by rock fill dikes and a lower reservoir created by constructing a rock fill dam and spillway across the Deerfield River. A 10 megawatt station was built into the downstream side of this

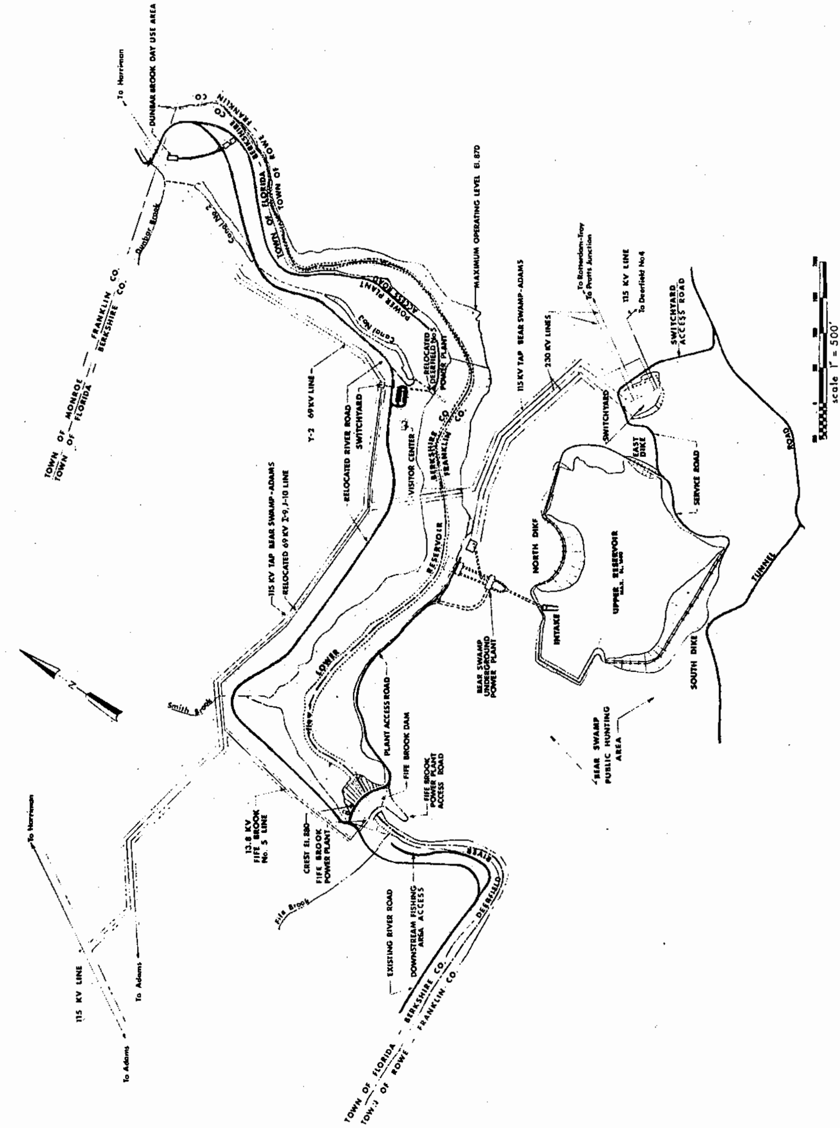

Fig. 2 - Key Plan

dam to regulate river flow, thus supplying water to four existing downstream hydro stations. The two reservoirs are linked by an underground waterway leading from the upper reservoir to an underground powerhouse and thence to the lower, or so-called, Fife Brook Reservoir. This waterway consists of a vertical concrete-lined shaft connecting to a concrete-lined tunnel bifurcating into two smaller tunnels which contain steel penstocks at their lower ends and terminate at two vertical reversible pump-turbines, each rated at 410,000 horsepower and two motor-generators, each rated at 300 megawatts with directly connected starting motors. The flow is controlled by two 11-ft. diameter spherical valves, at the inlet section of the units. From the powerhouse, two tailrace tunnels lead the discharge to the lower reservoir. There are two additional tunnels, one for access for personnel and equipment and the other to bring the generator main leads from the powerhouse to two main transformers located outside the tunnel portal, adjacent to the lower reservoir. Power is transmitted from this area by two 230 kV lines to a switchyard near the upper reservoir.

The lower reservoir now inundates the remnants of an old 3-unit 15 megawatt station which was located on the river edge. This station was demolished and a new single unit 15 megawatt station built about a half-mile upstream, using the same dam and much of the same waterway system which supplied the old plant. Most of these features will be described in greater detail during the course of this presentation.

A visitor's center and recreational plan were also a part of the total project.

Deerfield River

The Deerfield River is an extremely flashy stream flowing in a narrow valley with steep slopes rising as much as a thousand feet above the valley floor. It originates in southern Vermont and flows southerly and then easterly emptying into the Connecticut River just south of Greenfield, Massachusetts. The Deerfield has a drainage area of close to 600 square miles and in its total length of approximately 75 miles, the river drops over 2,000 feet. There are now nine conventional hydroelectric plants and two storage reservoirs along its length, developing 1,250 feet of head with a total installed capacity of 130,000 kilowatts; all, except the Bear Swamp complex, having been built between 1911 and 1927.

For the student of hydraulic engineering, a week spent along the Deerfield River offers the unique experience of a broad range of examples of hydraulic techniques. Almost every conceivable type of hydroelectric structure is represented in the series of plants along this comparatively small stream. There are surge tanks, wood stave penstocks, canals, tunnels, timber crib dams, semi-hydraulic fill dams, concrete dams, a morning glory spillway and other minor features too numerous to mention.

With the addition of the Rowe Yankee Atomic Plant and the Bear Swamp Pumped Storage Station, two of the most modern practical generation techniques, the Deerfield has become one of the hardest working rivers in the world.

The average flow of the river, as measured at the Charlemont U.S.G.S. gage, some 11 miles downstream of the Fife Brook Dam, is about 870 c.f.s. with historical extremes ranging from a minimum of 5 c.f.s. to a maximum of 56,000 c.f.s. which occurred in September 1938. Minimum flow in the future, however, will never approach this extreme since New England Power has agreed to pass 50 c.f.s. continuously through the Fife Brook Dam and to increase this minimum to 100 c.f.s. during the fishing season.

Plant Operation

With pumped storage having become so popularized in the last several years, there is no need to dwell at length on that phase. Briefly then, a pumped storage plant is simply a way of storing electrical energy for use at a more favorable time. By using incremental energy produced by large, base-load fossil or nuclear fueled thermal plants during the late night or weekend hours when the demand for electricity is minimal, a supply of water is pumped from a low-level reservoir to a high-level reservoir. For Bear Swamp this is shown on Fig. 3. When the demand for energy is high, thus having the most value, the same stored water is used to generate power as it makes the return trip from the high level to the low level reservoir. Naturally, there are several variations of this concept currently in operation.

At Bear Swamp, the ratio of pumping energy to generation is about 1.3 to 1, that is to say, for every 3 kilowatts of power generated, about 4 kilowatts of pumping power are required to raise the necessary water. Assuming, then, that pumping energy is available at 10 mils per kilowatt-hour and peaking energy is worth 25 mils per kilowatt-hour, for every 40 mils it costs to pump water to the upper reservoir, 75 mils worth of generation could be realized.

The Bear Swamp Station will be capable of generation at full load, or 600 megawatts, for about five hours and will require about 6.5 hours of pumping time to replenish the upper reservoir; theoretically using the same water over and over again, except for a small amount of make-up for evaporation and seepage losses. This is one of the principal advantages of pumped storage that is, a large continuous supply of water is not needed.

The Fife Brook power station, on the other hand, will be used to regulate and pass downstream the normal flow of the Deerfield River as it is received from the upstream Harriman and Sherman power stations. The Redeveloped Deerfield No. 5 Station will use the existing dam and much of the waterway system of the old station to produce power at a slightly lower head but at greater efficiency than the plant is replaces.

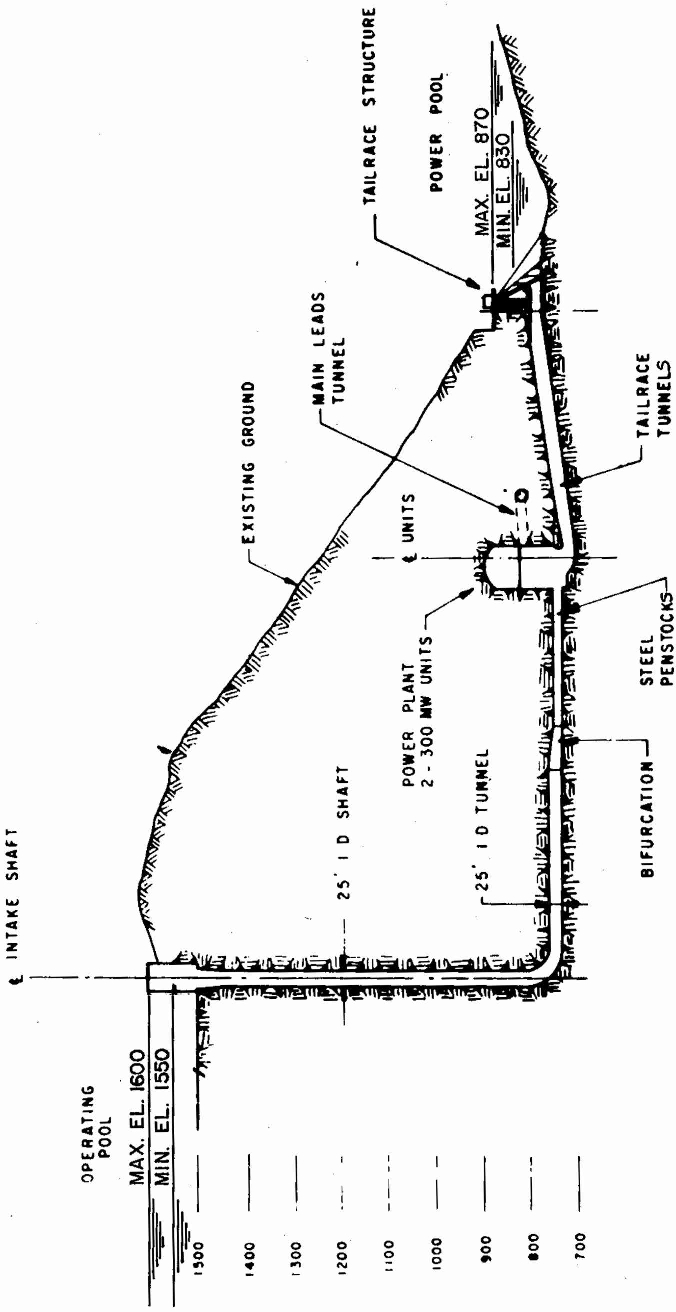

Fig. 3 - Profile Thru Waterways & Powerhouse

Historical Development of the Project

The concept of pumped storage is not new. It was first used in Europe back in 1892 and had its first application in the United States in 1928 at the Rocky River Plant of Connecticut Light and Power on the Housatonic River.

Actually, the New England Power Company first studied the use of pumped storage at Bear Swamp back in 1927, when the upper reservoir was first surveyed as part of a study of a 50 to 75 megawatt pumped storage scheme. All of these early schemes were based on the use of separate turbines and motor-driven pumps.

Probably the biggest reason for the resurgence of interest in pumped storage projects was the development of a single machine operating with high efficiency in both the pumping and generating cycles and capable of operating as a generator when rotating in one direction and as a motor when rotating in the opposite direction. These reversible units were first announced by Allis-Chalmers in 1950, and shortly thereafter, reversible units were installed at Flatiron, Colorado and Hiwassee, North Carolina.

More recently, there have been other incentives to the development of pumped storage, such as the development of much larger thermal units and the rapid growth and inter-connection of utility systems which justified the installation of larger capacity plants and consequent lower costs per kilowatt. Therefore, in 1960, the Power Company again began studying the Bear Swamp site, and in March 1965, plans had proceeded to the point where Chas. T. Main, Inc., was authorized to perform preliminary engineering services. Also, in 1965, aerial survey maps were prepared, seismic surveys were initiated and preliminary exploratory drilling was started.

In 1967 preliminary meetings were held with the Massachusetts Department of Natural Resources, the Department of Public Works, the Massachusetts Division of Fisheries and Game, and the Army Corps of Engineers.

Application for a License from the Federal Power Commission was submitted on February 15, 1968 and, at the same time, application was made for amendment of the License for the Deerfield River projects which included the redevelopment of the existing Deerfield No. 5 station. After a pre-hearing conference on April 28, 1969, hearings in May, another pre-hearing conference on July 10th and final hearings from September 9 to 11, 1969, the Licenses were granted on April 28, 1970.

Construction began in July 1970 with the establishment of a field office and clearing and grubbing activities in the two reservoir areas. Underground excavation started on June 8, 1971 and was completed by June of 1972. By July of 1974 the lower reservoir was completely filled, and initial pumping into the upper reservoir was accomplished. On September 30, 1974, the first unit went into commercial operation.

Permits

In addition to the F.P.C. license, many additional permits were required. Numerous local permits and variances were required from the towns of Rowe and Florida. An agreement with the Army Corps of Engineers was required for flood flow coordination. On the state level, a permit under the Hatch Act and a Water Quality Certification were required from the Department of Natural Resources and licenses to construct and authorizations for reservoir dikes were required from the Department of Public Works. Additional authorizations, permits and certifications required numerous meetings and hearings with all of the following groups over a three year period.

Massachusetts Department of Natural Resources
Massachusetts Department of Public Works
Massachusetts Department of Public Utilities
Massachusetts Department of Labor and Industries
Massachusetts Department of Public Health
Massachusetts Department of Fisheries and Game
Vermont Agency of Environmental Conservation
Army Corps of Engineers
United States Department of the Interior
Federal Water Quality Administration
Bureau of Outdoor Recreation
Geological Survey
Fish and Wildlife Service
Bureau of Sport Fisheries and Wildlife
Franklin County Commission
Franklin County Planner
Berkshire County Commission
Berkshire County Regional Planning Commission
Pioneer Valley and Berkshire Air Pollution Control Districts
Whitingham Planning Commission
Town of Florida — Assessors and Selectmen
Town of Rowe — Assessors, Selectmen, Zoning Board and Board of Appeals

Also, as a condition of the Federal Power Commission License, the Peabody Museum at Harvard University was commissioned to make an archaeological survey of the project area. This investigation resulted in archaeological digs which uncovered evidence of an early Archaic Indian culture dating back to 400 B.C. The major excavation area, just downstream of the Fife Brook Dam, is being preserved and the artifacts recovered are on display at the project Visitors' Center.

A short line railroad, the Hoosac Tunnel and Wilmington, affectionately called the "Hoot & Toot & Whistle", ran along the south side of the river while the so-called River Road ran along the other. Both of these features were within the area to be flooded by the lower reservoir. A three and one-half mile section of River Road was relocated at a higher elevation, and after prolonged negotiations, the railroad was abandoned and the right-of-way purchased by the Power Company.

Geology

As part of the preliminary engineering, geologic surveys, including surface reconnaissance, seismic surveys and exploratory rock and soil drillings were initiated in 1965. The results of this program confirmed the favorable geologic conditions in the upper reservoir area; however, rather unfavorable conditions were revealed in the area of the proposed lower reservoir dam and Fife Brook powerhouse due to deep overburden. Further studies in 1966 resulted in the recommendation of an alternate dam site about a mile downstream, just above the mouth of Fife Brook. By moving the dam downstream, an additional 60 feet of head were gained and the capacity of the plant, which had been fixed at 540 megawatts, was correspondingly increased to 600 megawatts.

Here in New England, one of the important factors to consider in interpreting the underlying formations at a dam site is the impact of the Glacial Age upon the area and the manner in which it altered the surface and subsurface conditions. It is known that for many thousands of years the Deerfield River had been eroding its bed, scouring out a winding gorge in the rock floor of the valley. When the continental glacier engulfed the area, stream erosion was halted and the uplands were partially worn down. As the ice finally melted, a thin layer of glacial debris was left on the uplands and large quantities of glacial drift were deposited in the valleys. The Deerfield River then resumed its course as the ice melted, following its old valley, winding over the surface of glacial drift, but since its gorge was buried under glacial materials, its new course had only a general relation to its old channel. The new course crossed and recrossed the buried gorge, sometimes cutting down to bedrock on one side or the other of its old valley.

At the Fife Brook damsite on the west side of the river, bedrock outcrops just above river level. The east side, however, contains the preglacial gorge and bedrock is overlain with glacial till some 170 feet deep.

During the summer of 1968, an extensive subsurface exploratory drilling program was undertaken at the Fife Brook dam site. Similar exploration as well as a complete seismic survey of the perimeter of the reservoir was completed in the upper pool area during the summer of 1969. A 700-foot deep vertical hole was also drilled in the vicinity of the proposed vertical shaft.

The drilling and testing at the Fife Brook dam site revealed that the buried gorge contained highly compacted glacial till which was relatively impervious and entirely capable of supporting the embankment planned for the main dam. The Fife Brook powerhouse and spillway were located on the bedrock outcrop area on the west side of the river.

In the upper reservoir area, the rock was found to be sound and tight with more than adequate strength to support the upper reservoir and dike structures.

The underground powerhouse and the waterway system are located in an area of dense, massive rock classified as a fine grained, light green, garnet bearing chlorite micaschist. An exploratory tunnel was driven into the east end of the proposed powerhouse cavern and along the full length of the centerline of the cavern prior to construction. Enlargements were made at each end and in situ stress measurements were made by the overcoring method to verify the proposed orientation of the powerhouse axis. Results indicated that the principal stress within the rock was parallel to the longitudinal axis of the cavern, confirming the original orientation of the cavern. Rock bolts were tested in the enlarged area of the adit to determine the best type of rock anchorage and the time that installed load would be retained. Valuable information was also gained with respect to drilling patterns and explosive loadings.

A few faults were discovered during the exploratory program and these were carefully mapped and procedures were adopted in the detailed design and layout of the structures to allow for these planes of weakness.

The summer of 1970 was spent investigating potential borrow areas in the river valley and in the vicinity of the upper reservoir for core and filter materials to be used in the embankments. Sufficient quantities of impervious glacial till were located and a quarry site was established in the perimeter of the upper reservoir. Subsequent construction activities showed that the rock for use in the shells of the embankment was extremely friable and had poor abrasion resistance. This precluded its use after crushing to provide a free-draining filter material and it was necessary to go further afield than had been expected to get a suitable substitute. It also necessitated some modification of the zones of material designed for the dam cross-section, increased the cost of the crushing and separating activities and necessitated opening an auxiliary quarry in the lower reservoir area.

Model Studies

The upper reservoir and intake were hydraulically modelled (See Fig. 4) at a scale of 1 to 50 at Worcester Polytechnic Institute's Alden Research Laboratories to study the effects of scouring along the upper reservoir north dike and the elimination of air entraining vortices from the vertical shaft intake during the generating mode. The complete reservoir was simulated

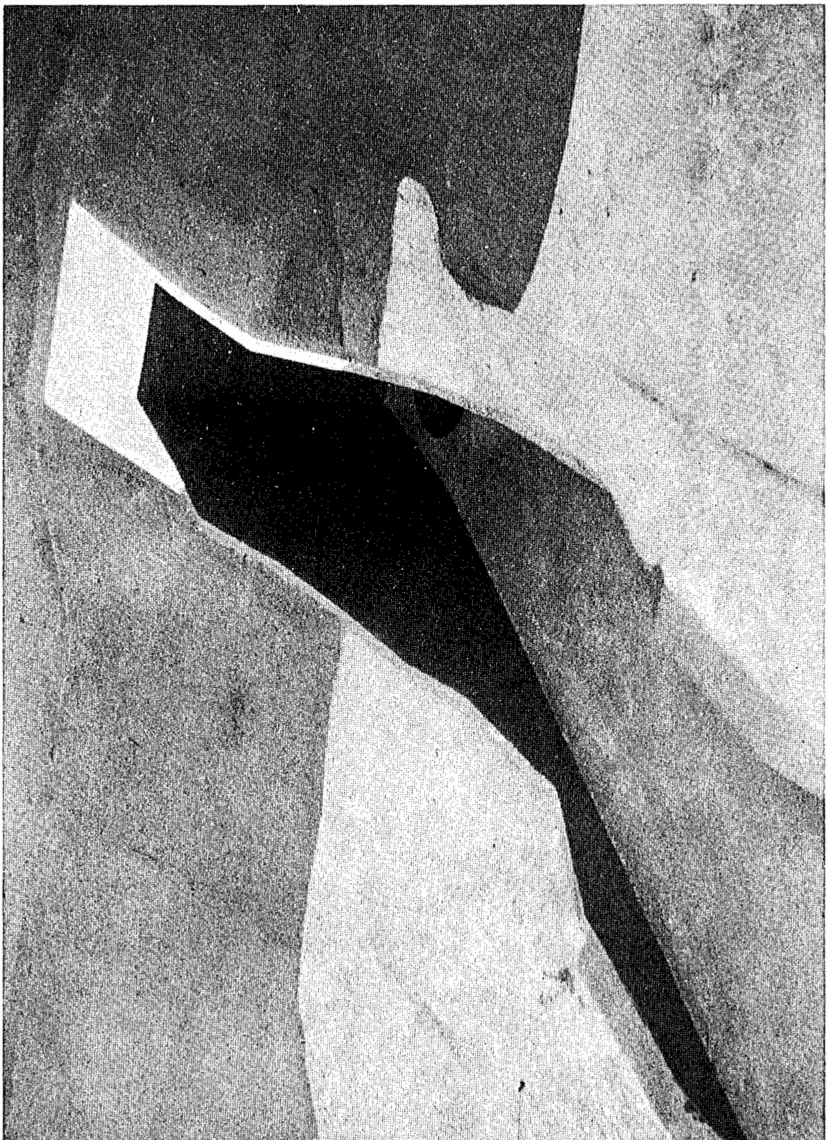

Fig. 4 - Upper Reservoir - Intake Model.

so that approach velocities to the intake could be correctly represented, that timing for each operation could be properly scaled and operating characteristics during the pumping and generating phases could be observed.

The intake location was originally chosen by topographic study. This location was changed when model testing indicated high velocities along the face of the north dike and was again changed to take advantage of a deep pocket in the rock encountered during overburden excavation, thus effecting a reduction in excavation costs.

Test results showed that the box-like rock faces around the vertical shaft should be vertical and as close to the shaft opening as possible. These rock walls were supplemented with vertical concrete walls extending above the rock topography and sloping toward the reservoir. The vertical shaft is 25 feet in diameter, and as a result of the model tests, the upper 75 feet were changed from a constant 25-foot diameter to a conical section expanding to a 40-foot diameter. With all these conditions met, the vortex action was essentially eliminated and a relatively simple intake design obtained. No trash rack structure is provided at the intake since the entire reservoir was grubbed and cleared 50 feet outside the upper water level contour in order to eliminate the debris which might cause problems to the unit.

The laboratory was also commissioned to model the Fife Brook spillway including a portion of the lower reservoir and part of the river downstream of the spillway channel. Tests were performed to determine the best bucket configuration to dissipate energy and minimize downstream scouring and bank erosion. The model was operated and modified, resulting in a final channel which sloped below the existing river bed, with side walls aligned so as to create a hydraulic jump in a bucket configuration at the lower end.

Tests were run to assure that the spillway could safely and efficiently pass any anticipated runoff from the 250 square miles of drainage area above the dam. The spillway capacity was determined by the unit hydrograph method using a precipitation of 20.2 inches in 48 hours over the total drainage area resulting in a runoff of 15.4 inches. The resulting flow for each incremental drainage area above the upstream developments was routed through the various reservoirs and intervening valleys assuming all upstream reservoirs and the Fife Brook reservoir full at the beginning of the storm. These studies indicated a maximum possible discharge of 73,900 c.f.s. which resulted in an ultimate elevation of 874.25 feet above mean sea level, leaving 5.75 feet of freeboard on the dam. This flow and the elevation were checked on the model and discharge coefficients determined for the spillway ogee and gates. Scouring at flood flow conditions was minimal and riprap installed at two downstream locations will minimize bank erosion.

Lower Reservoir

The lower reservoir, formed by a 130-foot high dam across the Deerfield River just upstream of its confluence with Fife Brook, is located on a solitary reach of the Deerfield River in an area covered by beech, birch, maple, oak and hemlock. Topographically, the river has carved a steep walled valley ideally suited for this pumped-storage reservoir.

This 2.4-mile long, narrow pool, occupying a maximum surface area of a little more than 150 acres, will be subject to a fluctuation of 40 feet during the pumping and generating cycles. Seven hours of pumping could deplete the 4,600 acre-feet of power storage capacity. Because of this extreme water level fluctuation, the entire reservoir has been fenced for the safety of the public. It should also be mentioned, in passing, that the valley topography within the flowage of the pool is such that flooding is not expected to have any significant effect upon terrestrial forms of wildlife.

As shown on Fig. 5, the Fife Brook or lower reservoir dam is simply an earthfill embankment with an upstream rockfill shell. This cross section represents the final stage of the dam's evolution from the original concept of a rolled earth embankment proposed for the project in 1967.

The foundation for this dam varies from rock at the right abutment to thick overburden at the left abutment. The rock is part of the Rowe formation, a granular schist with quartz seams, as described earlier. The overburden consists of glacial till, sufficiently impervious to preclude the necessity of a cutoff to rock. Because of local seeps in the vicinity of the abutment, however, limited upstream and downstream seepage blankets were incorporated into the design of the dam.

The dam section, itself, consists of a compacted glacial till protected on the downstream side by a thin zone of dumped rock fill and on the upstream side by a thicker zone of compacted rock fill. In order to insure free drainage of the compacted rock fill zone, the transition and filter zone material was extended laterally through the rock fill at the elevation of the minimum reservoir operation level. This feature was specified after observation of the results of initial rock placement. Rehandling, dumping, spreading, and final compaction tended to break down some of the rock in place; thus the additional drainage layer was adopted for added security.

Since the contractor's cofferdam was left in place, the area between the cofferdam and the upstream toe was used for disposal of waste material which could not be used in other construction.

All material for construction of the dam was obtained from a combination of required excavation and borrow areas in the immediate vicinity of the dam and lower reservoir.

Since the bedrock in this area was more broken and open-jointed than elsewhere, the grouting of the right abutment at Fife Brook was accomplished by three rows of grout holes. Because of the deep and relatively impervious overburden at the left abutment, the grout curtain did not extend across the entire length of dam. At any rate, the average take per foot of hole was just under 0.3 cubic feet per foot.

Early investigation of the site for the lower reservoir revealed that cor-

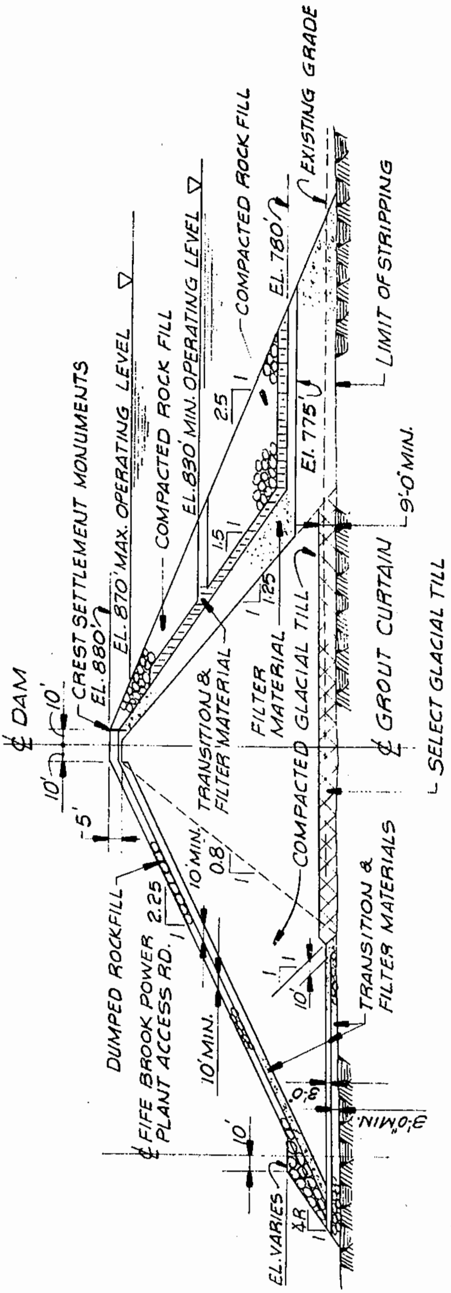

Fig. 5 - Lower Reservoir - Typical Fife Brook Dam Section.

rective measures would be required along certain portions of the reservoir side slopes to insure stability during drawdown. Visual examination combined with an intensive topographic map study identified the areas requiring slope angle correction. In general, action taken consisted in either excavation or filling or a combination of both in order to flatten the slopes. Analysis of the existing slopes indicated that correction for stability would be required where the natural slope was steeper than 1 vertical on 2.5 horizontal within the range of reservoir drawdown. Thus, upon identification of all critical areas within the reservoir, a separate economic solution was implemented for each critical section contingent upon length of haul for disposal or placement, availability of material, timing relative to construction activities, height of slope, and finally, esthetics.

This lower dam also incorporates a concrete, two bay, gated spillway as shown on Fig. 6. Control of flows in excess of plant operation and minimum downstream releases will be accomplished by conventional taintor gates, 40 feet high by 36 feet wide, operated by individual gate hoists. As discussed under the section on model studies, the spillway can safely discharge about 74,000 c.f.s. The original conceptual design of this spillway called for three gates, but further study revealed the economic advisability of larger gates. With only two gates capable of releasing the same 74,000 c.f.s. flow, it was possible to maintain rock foundations for the concrete structures in this confined space with minimum excavation.

Upper Reservoir

The Upper Reservoir is situated within a natural depression, known as Bear Swamp, above the Deerfield River valley. It lies in a formerly swampy basin at a point where the wooded hill breaks sharply to the river below. The reservoir, contained by three dikes in addition to the natural terrain, will occupy an area of some 120 acres with a normal power storage capacity of about 4,600 acre-feet out of the gross storage capacity of 8,900 acre-feet. Of the 8,900 acre-feet, an additional 500 acre-feet will be available for emergency use. The surrounding cover is second-growth mixed hardwoods and softwoods growing on long since abandoned agricultural land. The upper pool, similar to the lower, will experience extreme, rapid, daily fluctuations of water level. The total water level fluctuation for this reservoir will be about 50 feet. Since it will not support public fishing, it has been completely fenced for public safety. The west end of the reservoir served as a quarry area for obtaining rock as well as filter zone material for construction of the dikes. In addition, natural sand filter material was obtained from an esker formation in the lower river valley.

The overburden thickness in the foundation areas is variable, but most of the dike foundations sit directly upon bedrock. In the case where the entire foundation is not in contact with rock the glacial till core is trenched into

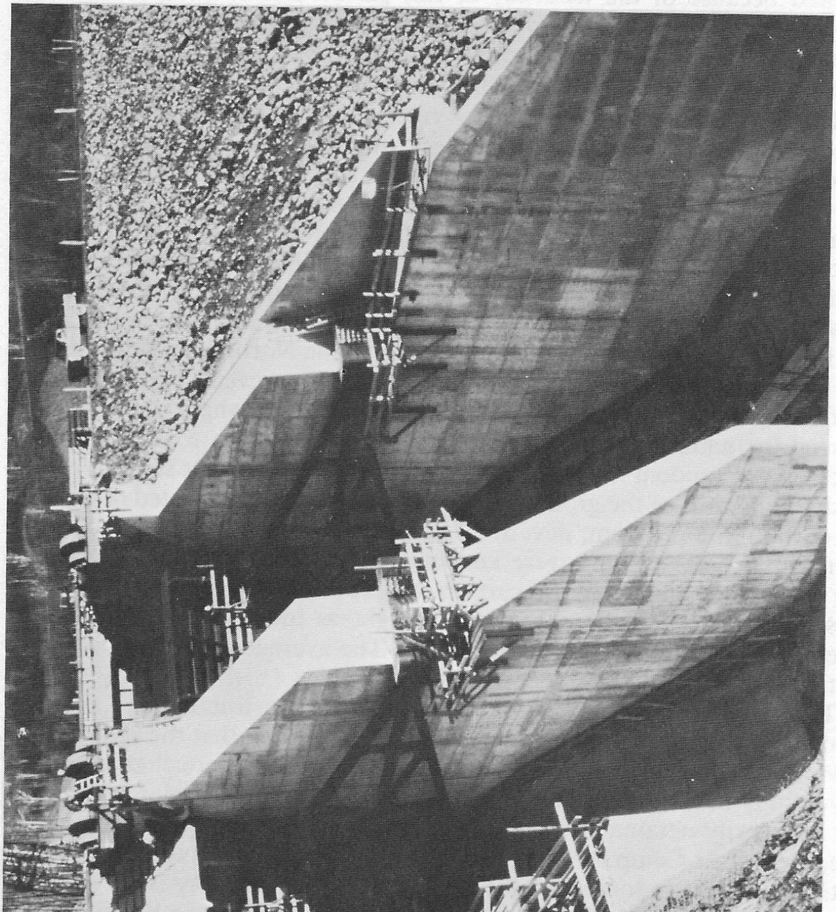

Fig. 6 - Fife Brook Spillway

the existing overburden, effecting a cutoff to rock. This cut-off was required because of the tendency of this glacial soil to be lenticular and relatively pervious in some layers. The underlying rock, although of three different geologic formations, could be singularly classified as schist with almost identical engineering properties.

The original concept for the upper reservoir dams envisioned a rock fill section with a bituminous concrete deck over the upstream face. As field explorations progressed, it became apparent that the quarry site would be able to produce sufficient rock adequate for rock fill shells for both upstream and downstream sections of the dikes. When nearby borrow areas for core material were discovered, it became feasible to adopt a more conventional design cross section with rock shells and a wide central core. These sections are shown on Fig. 7 and 8. Because of the narrow, downward sloping valley at the maximum section of the North dike, it was necessary to curve the dike sharply into the reservoir to prevent the downstream toe from running away downslope. At the South dike where the rock shell rests on the natural glacial till foundation rather than on rock, an 80-foot wide berm of random fill was constructed to insure sufficient stability.

Although most of the grouting in the upper reservoir consisted of a single line curtain, the program, nevertheless, was considered to be thorough and extensive. Wherever there were known small faults or large joints, special care was taken to increase the curtain width by additional grout holes. The schists were found to be exceptionally tight as indicated by small water losses during drilling and low grout takes. The average grout take was only a little more than 0.1 cubic feet per foot of hole.

The original project design did not require the construction of a spillway in connection with the upper reservoir since it was felt that the redundancy provided by the automatic controls would have been more than ample to prevent any inadvertent overpumping. However, since an emergency spillway was easily obtainable simply by the shaping of rock east of the North dike, the Power Company, prodded by the F.P.C., decided to add this precautionary measure to the project. The crest of this spillway is set two feet above the operating level of the upper reservoir. Acting as a broad crested weir, it is long enough to discharge the maximum pumping capability without overtopping any of the dikes. Any inadvertent pumping will simply return to the Deerfield River into the lower reservoir.

Reservoir Level Control

A water level monitoring system, designed to perform several functions, has been installed at Bear Swamp. In addition to having the capability to prevent overpumping, which is apparently the most common and important concern of designers, it will also prevent over-generation, as well as acquire, develop, transmit and display data for project operation.

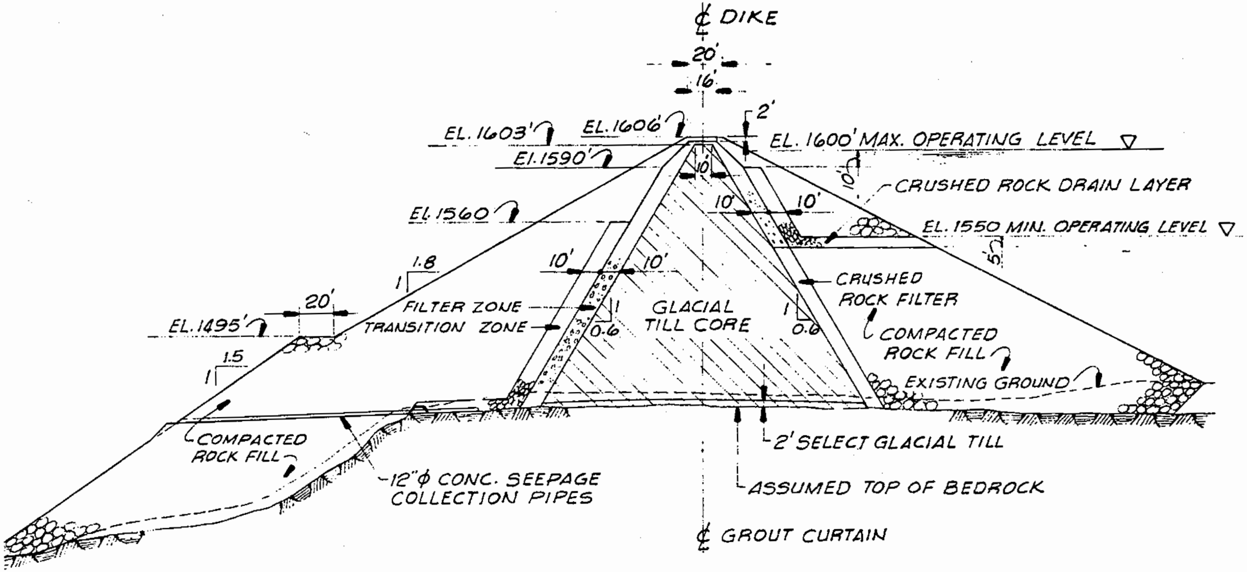

Fig. 7 Upper Reservoir Typical North Dike

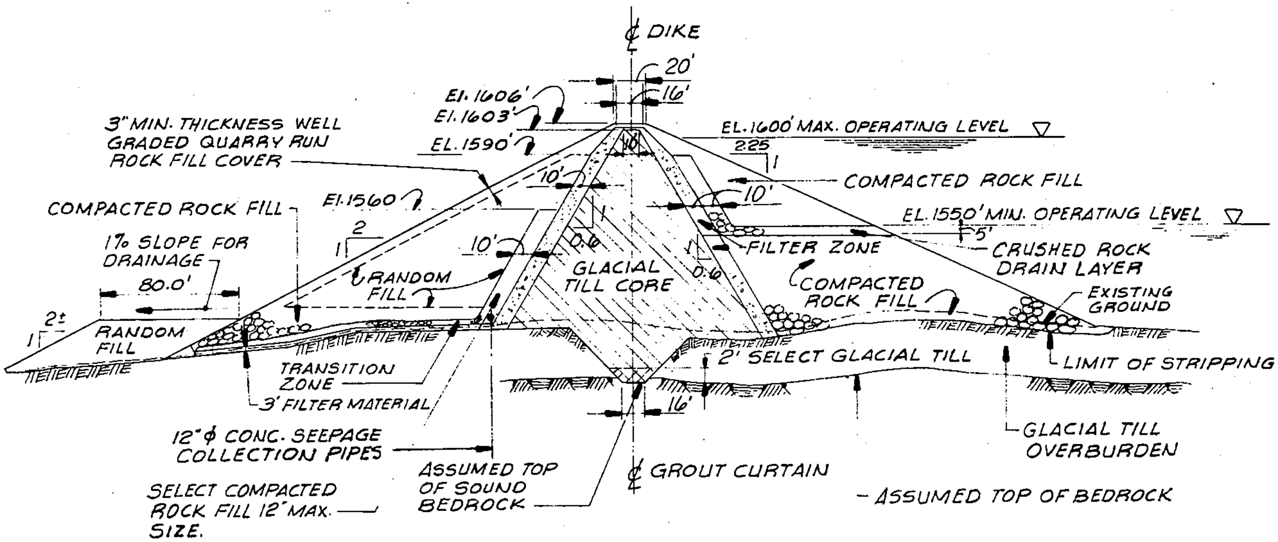

Fig. 8 Upper Reservoir Typical South Dike

The upper reservoir level variation is measured by three differential pressure transmitters located in a well at the North Dike. Signals from this well are hard wired to the Bear Swamp switchyard, from where they are transmitted by microwave to an emergency generator building at the powerhouse transformer yard. As a precaution, the microwave link also has an alternate route which may be followed. From the emergency generator building, the signals are carried by cable through the main leads tunnel to a control panel in the powerhouse. Safeguards are provided against the effects of extreme temperatures and adverse weather.

Besides the normal recording of reservoir levels, this system will actuate an alarm when the reservoir drops below a predetermined level or rises above a level 2.5 feet below the crest of the emergency spillway. Moreover, should the reservoir level continue to increase to within 1.5 feet below the emergency spillway crest, automatic pump shutdown will be initiated if this has not already begun. In the case where the reservoir level is dropping, the system will automatically shut down the generators should the level continue dropping to the next lower predetermined elevation.

An additional measure of redundancy is provided in the above system by means of a radio link from the pressure transmitters to the powerhouse access tunnel portal, from which point cable is run through the access tunnel to the control panel in the powerhouse. This alternate also has the same capability of alarms and controls.

Another completely independent system is provided to further reduce the possibility of overfilling the upper reservoir. This consists of a conductance type sensing probe located in a well at the East Dike. Information is transmitted to the emergency generator building via space radio, and from the generator building to the main control board in the plant via cable in the main leads tunnel. This back-up system will initiate pump shutdown should the reservoir level reach an elevation one foot below the emergency spillway crest. This condition could only occur if all other systems had failed.

The lower reservoir level is sensed in a well located at the Bear Swamp Plant tailrace. This well houses a float actuated transmitter whose signal is received at the power house control board via cable in conduit. Equipment related to this system indicates and records the Lower Reservoir levels, trips the generators on high reservoir level, trips the pumps on low reservoir level and, in conjunction with the Upper Reservoir controls, delivers a gross head signal to the governor for optimizing wicket gate position during pumping.

Tunnels

One of the most advantageous features of this project is its simplicity. Although there are many individual features, intersecting each engineering discipline, the overall simplicity facilitates rapid comprehension. One contributing factor is the tunnel system. Requiring no surge tanks or chambers, all design as well as construction proved to be very direct, as shown on Fig. 9.

The earliest project construction was related to the design of the rock cavern for the underground powerhouse and consisted of driving a 6 ft. by 8 ft. exploratory adit into the hillside to the proposed chamber. This adit, which was plugged prior to filling the Lower Reservoir, is about 500 ft. long from portal to powerhouse chamber. The adit also ran some 220 ft. along the longitudinal axis of the powerhouse. Construction of this adit permitted the performance of rock mechanics testing, as mentioned earlier, first hand examination of the rock to be encountered during excavation of the main chamber, rock bolt testing and additional core boring for geological examination. Final powerhouse design also utilized this adit for piping.

The hydraulic power supply tunnel originates at the intake in the Upper Reservoir as a 25-ft. diameter concrete-lined vertical shaft. This shaft, which was lined by using a slip form procedure with a travel of about one inch every three minutes, reaches down for a distance of 740 ft. where it bends to form a nearly horizontal tunnel, joining a bifurcation 410 ft. away. Both the vertical and horizontal portions of the tunnel are concrete lined with a minimum thickness of 12 inches. A grade of 7.5 percent was adopted for this tunnel on the basis of utilizing rubber-tired equipment for construction. Economic studies also dictated the advisability of a vertical shaft rather than an inclined shaft. This represented a saving in the order of $1,000,000.

From the bifurcation, which was lined with reinforced concrete, two 17.5-ft. diameter penstock tunnels continue on to the spherical valves in the powerhouse. Each of these penstocks is 175 ft. long and steel lined. The steel requirements were dictated by consideration of notch toughness, weldability, high tensile strength, and economics. After determining local availability, ASTM A-516 Grade 70 steel with a yield of 38,000 psi was specified. Although the allowable stress for penstock design is normally 60 percent of the minimum yield stress or ⅓ of the ultimate tensile stress, the liner was designed for 50 percent of the minimum yield stress since final synoptic model curves were not yet available at the time the liner had to be designed and material ordered. All joints were subjected to complete radiographic examination and the steel was also drained.

Grouting of the penstock tunnels was required by the specifications. In order to fill any voids between the concrete and the rock and possible cracks in the crown of the rock arch, contact grouting was accomplished by holes in the roof approximately on the tunnel centerline at 10-ft. centers and drilled one foot into rock. For contact between the steel liner and concrete, tapped holes were provided in the liner at the horizontal centerline on 5-ft. intervals. These holes did not penetrate the concrete lining. Finally, a curtain or fan grout pattern of radial holes grouted under pressure formed a barrier just upstream of the steel liners.

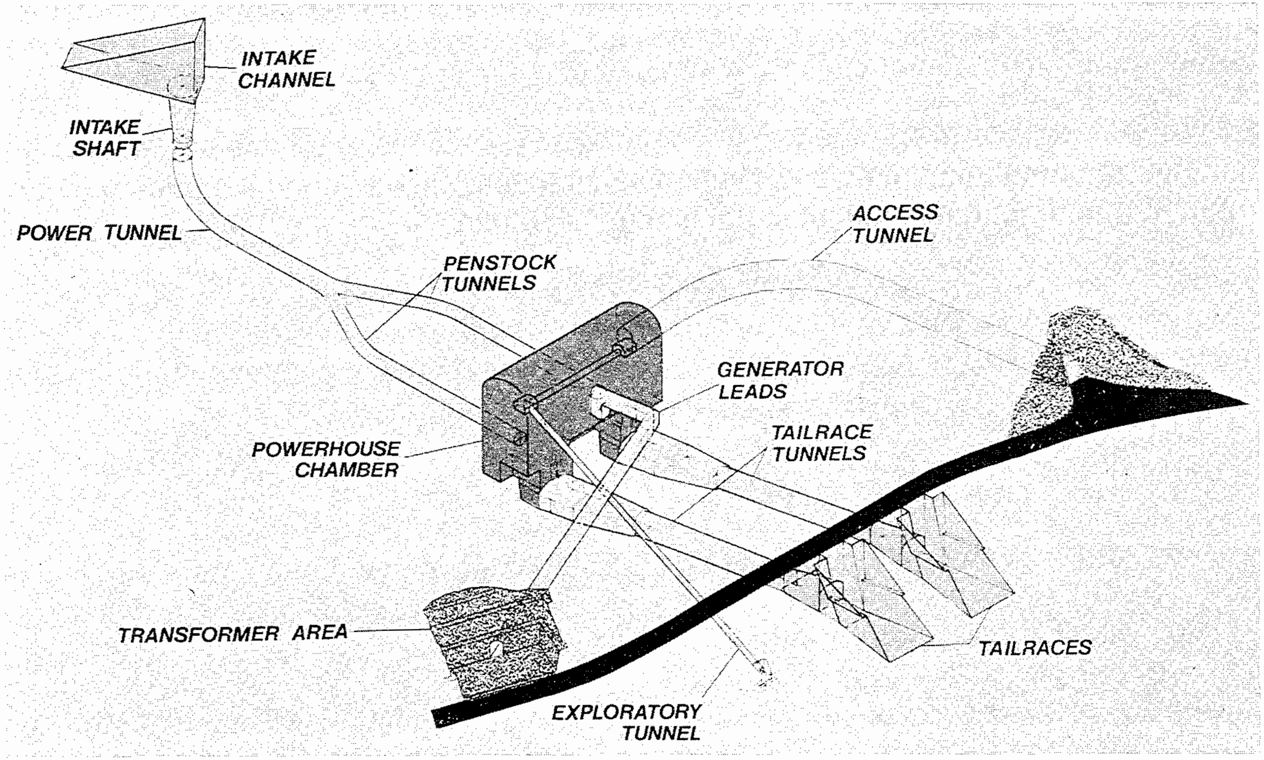

Fig. 9 - Schematic Diagram Underground Pumped Storage Project

Each of the two tailrace tunnels is 500 ft. long with a concrete lined transition from double-barrelled to single barrel shape at each end. The remaining 300-ft. length is horseshoe shaped 22 ft. wide by 29 ft. high. Although the invert of the horseshoe shaped portion was not paved, the side walls and roof were shotcreted in addition to being bolted to guard against future rock falls. Prior to filling, the inverts were carefully inspected following thorough clean-up. All loose or potentially loose rock was removed and all seams which offered any possibility of future spalling were filled with dental concrete anchored into the rock. Grouting for these tunnels was confined to a curtain established at the end of the concrete lining for the draft tube transition.

The tailrace tunnels can be closed to the lower pool by means of slide gates (two per tunnel) at the outlet structure. Each gate is operated by its own fixed hoist. This outlet structure is also fitted with trash racks.

Access into the underground powerhouse is by a 25-ft. wide by a 30-ft. high horseshoe shaped tunnel 690 ft. long. This tunnel, as shown on Fig. 10, is unlined except for the invert and penetrates the powerhouse chamber at its west end at the erection floor level. The size of the tunnel was predicated upon accommodating the largest piece of equipment to pass through it either during construction or for future maintenance. This turned out to be the turbine runner. The arch area has also been utilized for duct space required for powerhouse ventilation and lighting.

During construction of this tunnel, the excavation contractor proposed and was allowed to excavate an adit branching off from the point of curvature of the access tunnel and rising to penetrate the powerhouse cavern near the roof of the powerhouse arch. This enabled the contractor to proceed with the main cavern excavation quite rapidly. The designers quickly took advantage of this adit by using it for permanent access to the area above the powerhouse drip ceiling as well as locating the CO_2 system adjacent to the main access tunnel.

The final location of the main transformers was the subject of a considerable amount of study by the Power Company. Alternatives considered included: Transformers located within the underground powerhouse cavern; placing the transformers on the surface directly above the cavern with the main leads running vertically in a shaft to the transformers; locating the transformers in a switchyard on the bank of the Deerfield River; and, finally, the scheme adopted, constructing a separate transformer yard just upstream of the tailrace on the bank of the lower reservoir. The scheme which was adopted as the most economical was also compatible with environmental considerations.

As shown on Fig. 11, the main leads tunnel carries not only the bus enclosures for the main leads on either side of each wall, but also turbine vent piping, cable trays for electrical controls, ventilation ducts and light-

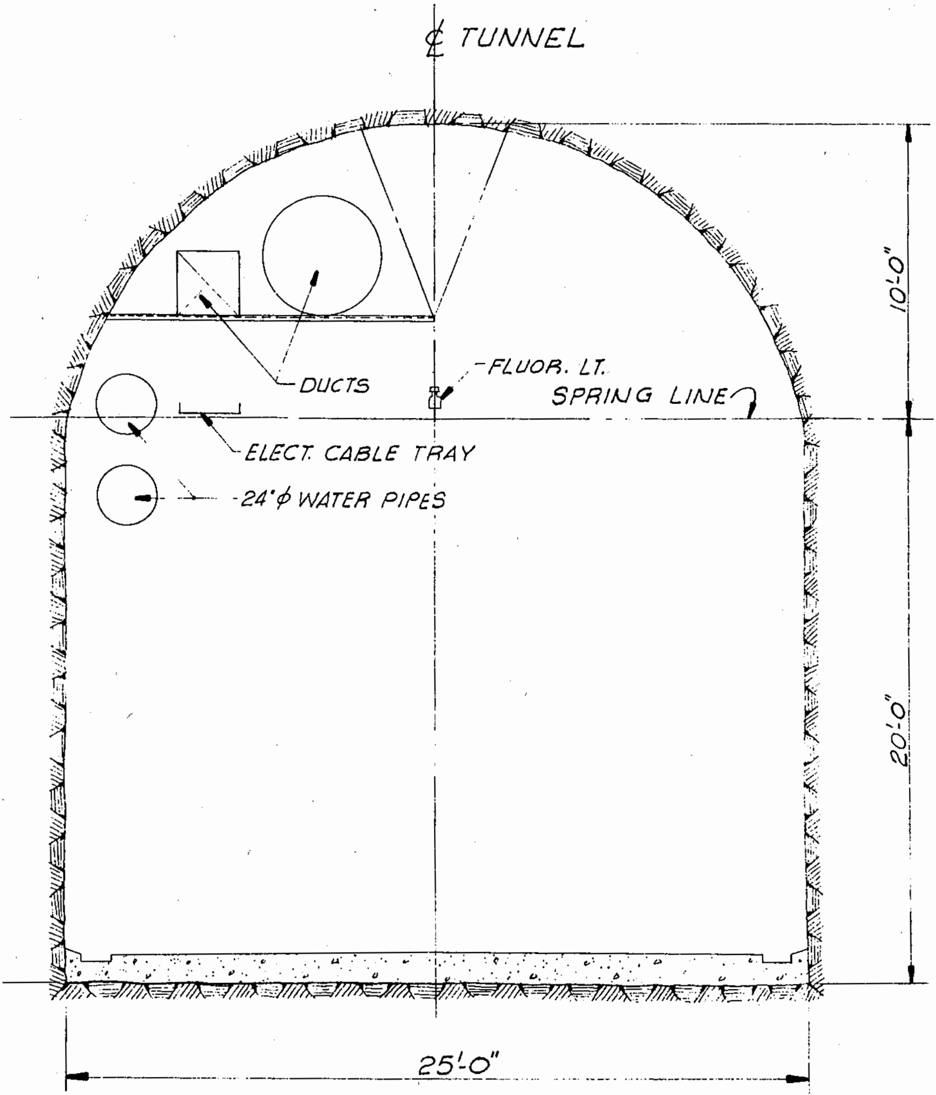

Fig. 10 - Access Tunnel

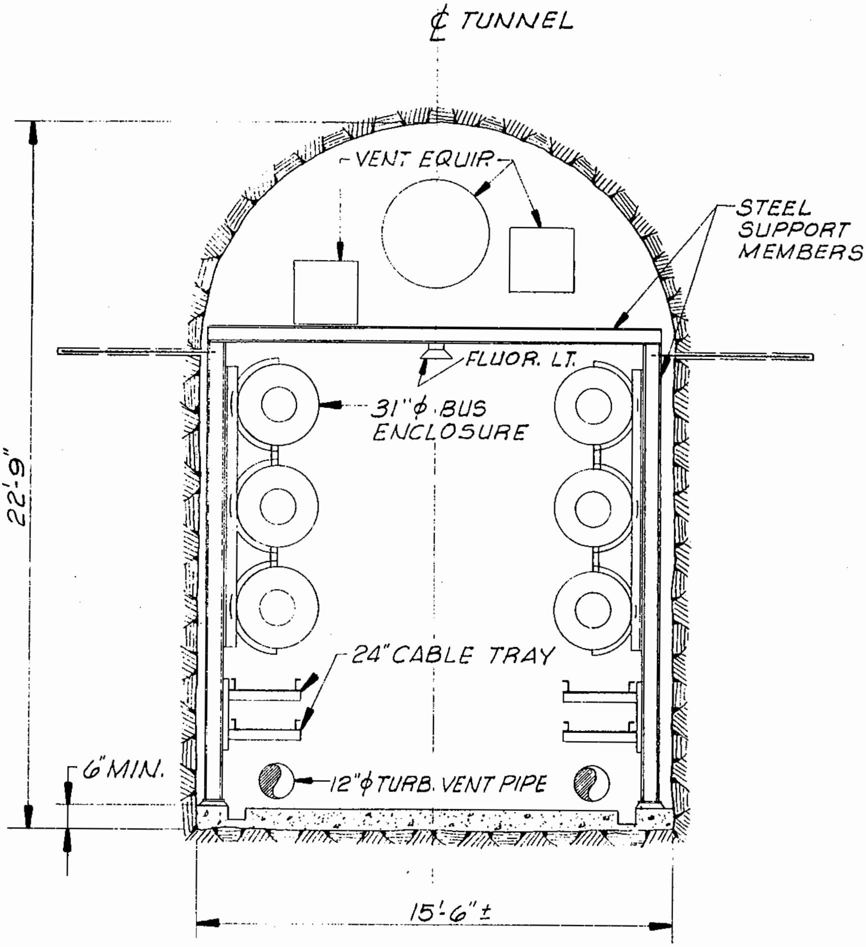

Fig. 11 - Main Leads Tunnel

ing. Clearances established were the minimum required for future maintenance.

Before leaving this discussion of tunnels, some mention should be made of hydraulic transient considerations for the project. Although it was almost intuitively obvious that a surge tank was not required by virtue of the favorable ratio of waterway length to head as well as the low tailrace tunnel water velocity, preliminary computer runs were made to determine potential superpressures and overspeeds. Since data for the Bear Swamp pump turbines were not yet available, the computations were based upon model test curves for another machine which had approximately the same specific speed. Final computations were made upon receipt of the actual model characteristics from the manufacturer. Without going into detail, it will suffice to say that a variable speed governor was adopted in order to minimize waterhammer pressures during operation. Thus, the wicket gates initially close at a faster speed followed by a slower rate to final closure. Actual tests up to this point in time reveal that the theoretical superpressures nearly coincide with the actual.

Bear Swamp Powerhouse

The Bear Swamp powerhouse is only the third complete underground pumped storage plant in the United States. It is located in an excavated chamber (Fig. 12) about 600 feet into the hillside of the left bank of the Deerfield River and occupies a space of about 80 ft. in width by 225 ft. in length, 150 ft. high. The chamber houses two pump-turbine units and generator-motors, along with the usual auxiliary equipment, governors, valves, air compressors, unwatering pumps, drainage sumps, battery rooms, shops and the like.

The original, conceptual design visualized an above ground, semi-outdoor powerhouse for the pumped-storage plant. However, as studies progressed for the overall project, it became obvious that an underground powerhouse would be more desirable, not only esthetically, but also economically. Comparative, detailed studies showed that, based upon early 1970 prices, a saving of at least $2,500,000 would be possible by adopting the underground design. This saving also accounted for the additional geological exploration and rock testing which would not have been required for a surface type powerhouse. A substantial amount of this cost reduction was made possible by the elimination of mass concrete which would have been necessary with a surface powerhouse.

Once it was decided to proceed with final design on the basis of an underground plant, it soon became apparent that access from the west end of the plant was most desirable from the standpoint of economy combined with construction maneuverability during the various stages of equipment delivery and erection.

In order to allow early erection of crane rails, three alternatives were considered. The first possibility, consisting of cantilevering concrete supports from the side walls of the cavern, was discarded for fear of damage which might have resulted from quarry excavation techniques below the arch. The second alternative, excavating a haunch in the rock itself, would have proved much too expensive because of the increased span required for the crane itself combined with rock sculpturing which would have been necessary. The third alternative, that which was finally adopted, consisted of a structural steel framing. The design, in this case, was controlled by

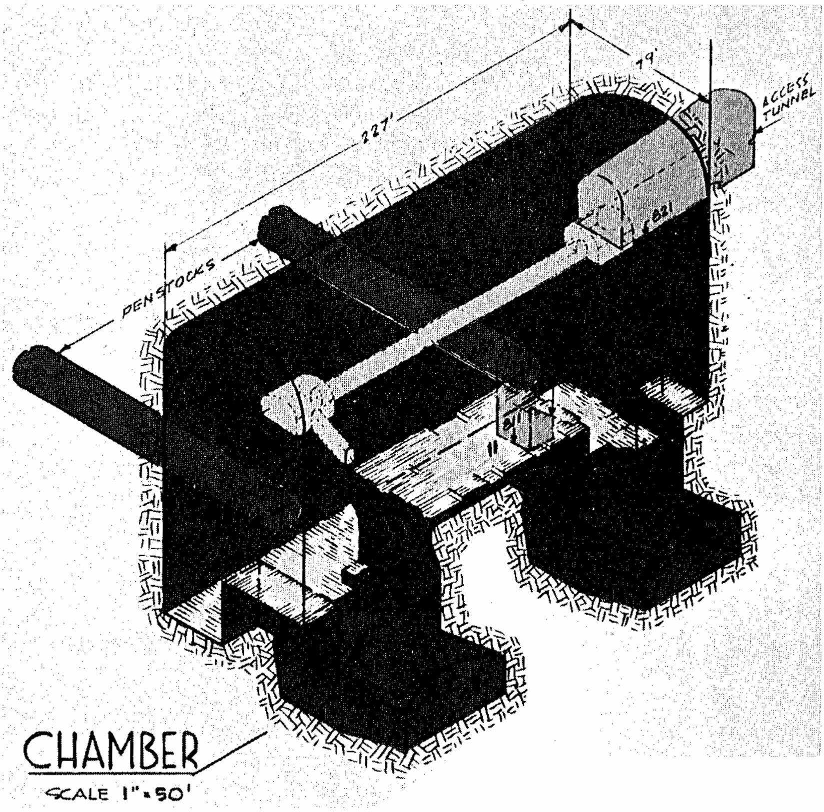

Fig. 12 - Excavation Diagram Underground Power Plant Chamber

construction conditions. The steel had to be adequate for crane loadings without the benefit of sub-structure mass concrete in order to allow multiple construction operations in a confined space and was therefore bolted to the cavern rock face. Thus, early erection of both a temporary crane and the permanent 606-ton bridge crane was possible. The permanent crane, incidentally, was also used as the space support for installation of the arch supported "galbestos" drip roof. Fig. 13 and 14 show the cross section and longitudinal section through the powerhouse.

After settling upon a steel structure within the powerhouse, a technique for constructing the floors was borrowed from bridge builders. That is, corrugated bridge decking forms were used for placing the concrete floors. This allowed clear space between floors for early use by the various trades during construction.

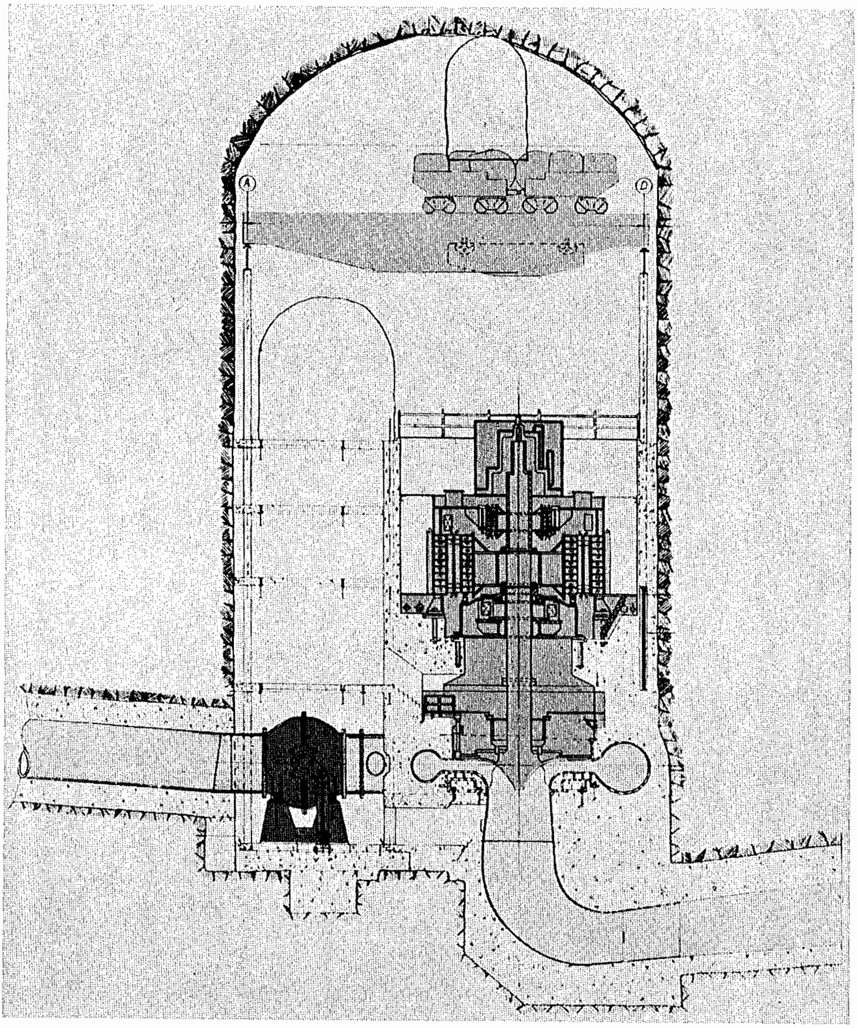

Fig. 13 - Cross Section Bear Swamp Underground Power Plant

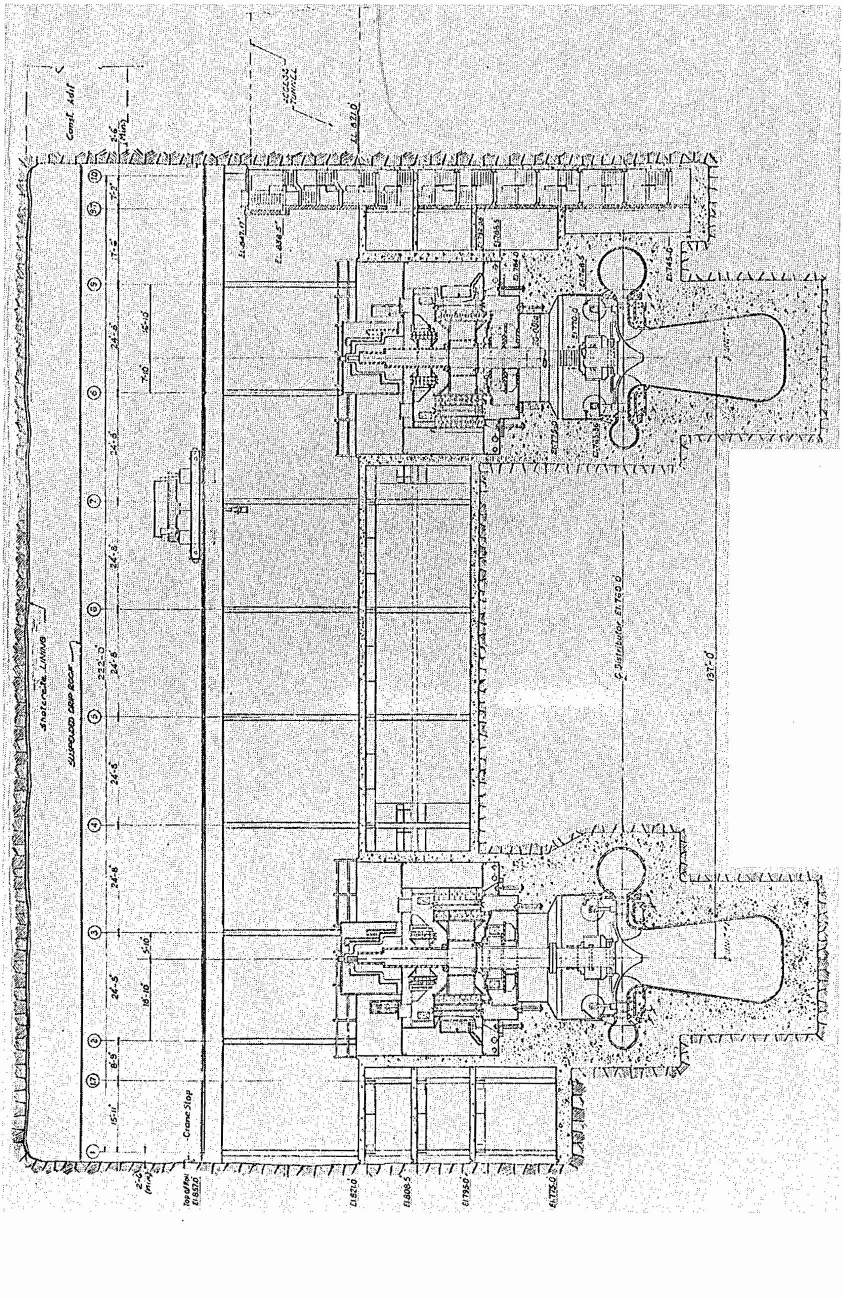

Fig. 14 - Longitudinal Section Bear Swamp Powerhouse

The top floor was designed to serve as an unloading area, providing for assembly of generator rotors and spherical valves and space for set-down of turbine and generator parts during disassembly of a unit for future repairs.

The plant layout, itself, was a combined effort involving various engineering disciplines and the key personnel who would be charged with maintenance and operation of the plant after completion. It should also be mentioned here that the plant has been designed for remote, fully automatic operation.

Fife Brook Powerhouse

As indicated earlier, the dam forming the lower reservoir will incorporate a gated spillway at the right abutment with the intake for the 10 megawatt powerhouse between the spillway and the dam. This station, known as the Fife Brook Powerhouse, is located at the downstream toe of the dam and consists of a single Francis unit powerhouse, penstock and intake structure, designed for power generation using only the run-of-the-river flow of the Deerfield.

This above ground powerhouse, 80 ft. long by 40 ft. wide and 90 ft. high, is a reinforced concrete structure with a roof of precast double-tee slab sections. (Fig. 15) It houses a 13,500 horsepower vertical Francis turbine directly connected to a 12,500-KVA generator, a 60-ton overhead bridge crane and the customary mechanical and electrical auxiliary equipment.

The intake is simply a single bay, concrete gravity structure which also acts as an integral part of the spillway west abutment. It contains a single 8-ft. by 12-ft. wheeled gate and fixed hoist, trash racks and provision for stop logs. The face of the structure also contains a sluice gate for maintenance of a 30-inch, low flow release by-pass line which runs essentially parallel to the penstock and discharges into the tailrace. This by-pass permits the continuous release of 50 c.f.s. (or 100 c.f.s. during the fishing season) into the Deerfield whenever the turbine is not in operation.

A steel penstock, fabricated from ordinary A442-Grade 60 steel, 10 ft. in diameter and about 200 ft. long, is completely encased in concrete and connected to the inlet section of the turbine spiral case inside the powerhouse. Unlike the Bear Swamp powerhouse, no control valve was required since an intake gate has been provided.

Deerfield No. 5 Re-development

Prior to construction of this project, New England Power owned and operated a small 15-megawatt plant known as "No. 5 Station" which was located about a mile upstream of the planned lower dam. Since the lower reservoir would inundate this station, it was demolished and rebuilt about

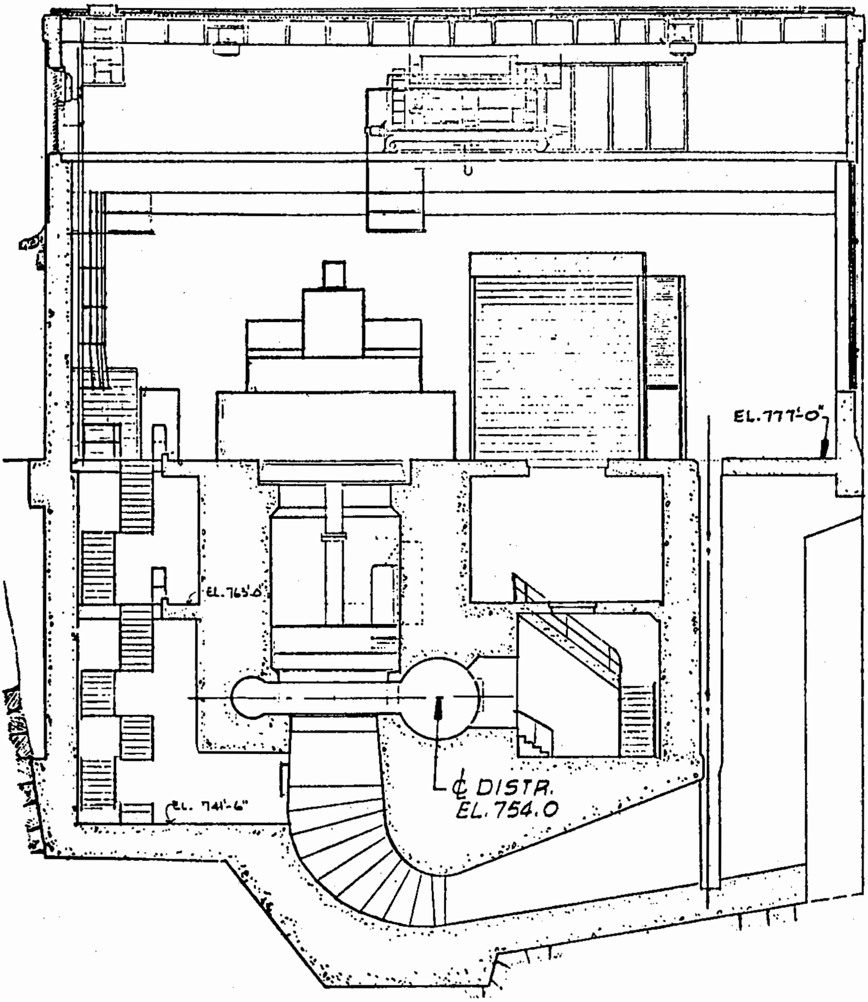

Fig. 15 - Cross Section Fife Brook - Power Station

2,000 feet upstream of its old location. Instead of the three 5-megawatt units which were built in 1915, the No. 5 re-development was constructed as a single 15-megawatt unit.

The new No. 5 plant utilizes most of the waterway features which were appurtenant to the old plant. These features consist of a rock-fill timber crib diversion dam a considerable distance upstream on the Deerfield River, a concrete intake and spillway section, water conveyance facilities comprised of concrete conduits, canals, and a tunnel, totaling about 13,000 feet at the present time.

A portion of the pre-project existing conditions is an earth canal known as Canal No. 3. This canal now feeds the new powerhouse via a new intake structure, constructed on the south bank of the canal. This structure is provided with a single 8-ft. by 12-ft. wheel-type gate, trash racks and stop log guides.

The intake structure is connected to the powerhouse by an ordinary steel (A442 grade 60) penstock, 10 feet in diameter, supported on ring girders in a rock trench. This trench was subsequently backfilled and landscaped to blend in with the surrounding area. Prior to backfilling, the penstock was painted with an initial coat of zinc rich paint followed by two coats of coal tar epoxy.

The powerhouse, which is 79 ft. long by 44 ft. wide and 114 ft. high, consists of a concrete substructure and a steel rigid-frame superstructure with a metal deck roof. The exterior walls above the operating floor are constructed of concrete block with brick facing.

The powerhouse contains a single vertical Francis turbine rated at 20,800 horsepower directly connected to a generator rated at 19,500 KVA at 0.9 P.F. The station is also equipped with a 60 ton overhead traveling crane and the usual auxiliary equipment, such as: air compressors, governor, unwatering pumps, drainage sumps, battery room, control panels, switchgear and the like. While the main powerhouse pump-turbines were manufactured by Hitachi in Japan, this Francis turbine as well as the one at Fife Brook, was manufactured by Leffel.

Diversion

Although the spillway is capable of passing almost 74,000 c.f.s., it is quite obvious that such a flow could not be considered economical for temporary construction facilities. Due consideration was given to the historical fact that in more than 35 years of record, a daily average inflow of 3,000 c.f.s. has been experienced only 0.2 percent of the time. Thus the diversion structure for the lower dam was designed for 3,000 c.f.s. and a freeboard of 6 ft. Actually, flows of up to 5,300 c.f.s. were experienced during construction. Fortunately, however, this occurred at a time when the main embankment had attained sufficient height above the cofferdam to prevent overtopping, and the only flood damage incurred was from high tailwater below the dam.

The structure itself is simply a double-barrelled horseshoe shaped reinforced concrete conduit, 9 ft. wide by 9 ft. high. Since the dam was built directly over it, the conduit was founded on rock and provided with seepage collars. Although the early design studies considered other possibilities for diversion, this scheme was adopted as the simplest and most economical. In the area where the dam would not be over the diversion conduit, two temporary corrugated metal pipes were connected to the end of the concrete section. These pipes were subsequently removed prior to completion of project construction. Closure was accomplished by installation of two slide gates at the conduit intake portal. Once the gates were in place, permanent closure was made within the conduit by 25-ft. long concrete plugs located beneath the central portion of the dam.

Quality Assurance

The contract between the owner and contractor required the contractor to act as the agent, constructor and construction manager at the site. Thus, the contractor was responsible for supervision, scheduling, coordination and execution of construction and installation of equipment. The contractor was also charged with the first line of quality control, and the specifications required control in both field construction and equipment manufacturing.

By the use of independent testing and inspection agencies, augmented by periodic checks by the design and engineering staff, work on major equipment was reviewed and tested through the progressive stages of fabrication, assembly and shop testing.

Field construction was also monitored, but by a minimal engineering group and restricted to foundations and dam construction. Prior to placement of embankments on either bedrock or earth foundations, the prepared foundations were inspected and approved. Pertinent geologic features encountered in the foundations were mapped.

Test embankments were prepared under controlled conditions and compactive effort required to produce stable, water-tight embankments was determined.

Systematic, in-place record tests were made of foundations as well as embankments. Where inspection by visual, index or detailed laboratory tests indicated unsatisfactory construction, the materials in question were either reworked or removed and replaced as necessary to insure proper construction.

In an effort to obtain concrete of high quality and durability, the specifications provided for rigid control of the quality of all materials entering the mixes, proportioning the materials in the mix, and placing and curing operations. To this end, the owner engaged an independent testing and inspection company to cover the inspection, mix design, batching plant operation and testing of concrete.

Visitors' Complex

As an adjunct to the project, New England Power developed a visitors' center complex to provide interpretive, sight-seeing and picnic facilities for the use of the general public.

Located just off the relocated River Road above the lower reservoir, the complex features an underground display containing models of both reservoirs, a cutaway model of the underground powerhouse, and visual and audio devices explaining the concept of pumped storage. Space has also been allotted for displays by the Massachusetts Department of Natural Resources. In addition, there are included comfort facilities and an observation deck directly accessible from the main display area.

The total recreation plan also provides picnic areas and facilities, hiking trails and parking space for fishing access.

Conclusion

Although problems were encountered during design and construction of the project, none of the difficulties was of such magnitude as to hamper steady progress. All physical measurements made during as well as subsequent to construction, as part of the monitoring program, have been within expected design limits. At the present time, all generating units are in commercial operation.

Perhaps the best conclusion should be a direct quote from the Board of Consultants' final report: ". . . it is the Board's judgement that the project has been well designed and constructed and that its operation can be expected to be fully satisfactory."

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