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CAVITATION PHENOMENA IN HYDRAULIC SYSTEMS

BY JAMES W. DAILY,* *Member*

(Presented as one of the John R. Freeman Lectures on Fundamental Hydraulic Processes in Water Resources Engineering, Fall, 1963).

INTRODUCTION

Cavitation is a phenomenon involving a rapid and essentially uncontrolled change of phase from liquid to vapor. It occurs in a liquid system whenever the absolute pressure in the liquid drops by hydrodynamic means to or below some critical value. In all the common situations cavitation first appears when the pressure is low enough to unbalance the equilibrium of minute volumes or nuclei of undissolved gas or free vapor which are trapped on entrained foreign matter or in the containing walls of the liquid. The result is a transient unsteady phenomenon characterized by a growth of holes or cavities. The bubble growth will be "explosive" if it is primarily the result of vaporization into the cavity. This we call the cavitation and it is of prime importance. The slow growth by diffusion of gas into the cavity is sometimes called *gaseous cavitation* when acoustically induced.

Cavitation, at least in its inception stages, is local boiling at essentially constant temperature. However, cavitation is hydrodynamic (fluid motion causing the unbalancing of nuclei equilibrium) while boiling is thermodynamic (heat addition causing the unbalance).

Cavitation has several effects. First, it modifies the non-cavitating hydrodynamic flow characteristics. This modification changes the performance of hydraulic devices. Since these devices are designed for non-cavitating conditions, the result is usually poorer performance. Second, cavitation causes material damage to the nearby solid bounda-

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ries. Third, it is the source of some auxiliary problems of differing importance in different situations. Noise, unwanted release of gases and mechanical breakdown of some types of fluids are examples.

When properly controlled, cavitation can be used beneficially. Its damaging ability makes it an effective "scrubber" for certain types of surface cleaning operations. Its noise producing characteristic has been applied to a high energy echo-ranging device for ocean bottom surveying. Its ability to mechanically break down certain chemicals and compounds has important possibilities. Currently there is much research effort going into developing hydraulic machinery and equipment designed to operate with cavitation and allow an important extension of operating pressure and velocity ranges.

Cavitation Mechanics

A. *Inception*

The critical pressure at which cavitation occurs is usually taken to be the vapor pressure. However, the problem is not so simple as experiments show. While cavitation occurs near the vapor pressure, there are deviations from it with both water and other liquids. Vapor pressure is defined as the equilibrium pressure, at some specified temperature, of the liquid's vapor which is in contact with an existing free surface. On the other hand, cavitation inception is related to the equilibrium of liquid-free pockets and involves other properties of the liquid, its bounding walls and any entrained contaminants. We can illustrate this by discussing two points.

Let us examine first the stability of a gas volume or *nucleus,* in a normal liquid. For convenience we will consider a sphere, the shape a very small free gas bubble will assume. We will assume irrotational motion inside and outside the bubble as the sphere changes size. The velocity potential for the liquid motion (having zero velocity at infinity) which accompanies bubble size change is

$$\phi = \frac{R^2\, dR/dt}{r} \tag{1}$$

where

$R = R(t) =$ bubble radius at time t

$r =$ radial distance from the bubble center.

The radial velocity at any r is

$$u = -\frac{\partial \phi}{\partial r} = \frac{R^2\, dR/dt}{r^2} \tag{2}$$

which reduces to $u = U = dR/dt$ for the bubble wall radial velocity. Actually, because of evaporation or condensation at the wall, this latter is an approximation for a cavitating bubble. With constant liquid density, the equation of motion can be written

$$-\frac{\partial\phi}{\partial t} + \frac{u^2}{2} + \Omega + \frac{p(r)}{\rho} = F(t) = \frac{p_\infty}{\rho} \tag{3}$$

where

$p(r)$ = local pressure at radius r
p_∞ = pressure at infinity in the liquid
Ω = gravity potential.

Neglecting gravity and reducing to $r = R$ gives for the bubble wall

$$-R\frac{dU}{dt} - \frac{3}{2}U^2 = \frac{p_\infty - p(R)}{\rho} \tag{4}$$

where

$p(R)$ = pressure at the bubble wall.

For a *gaseous bubble,* let

$p_v = p_v(T)$ = vapor pressure
$p_g = \dfrac{NT}{R^3}$ = gas partial pressure (assuming a perfect gas)
$\sigma = \sigma(T)$ = surface tension.

Then, referring to Fig. 1, which is a section through a spherical bubble, we see

$$p(R) = p_v + p_g - \frac{2\sigma}{R} \tag{5}$$

so that the equation of motion of the bubble wall becomes

$$\rho\left[R\frac{dU}{dt} + \frac{3}{2}U^2\right] = -\left[p_\infty - p_v + \frac{2\sigma}{R} - \frac{NT}{R^3}\right] = f_1(R) \tag{6}$$

With the bubble in equilibrium $U = \dfrac{dR}{dt} = 0$ and the equilibrium radius is given by

$$f_1(R) = -\left[p_\infty - p_v + \frac{2\sigma}{R} - \frac{NT}{R^3}\right] = 0 \tag{7}$$

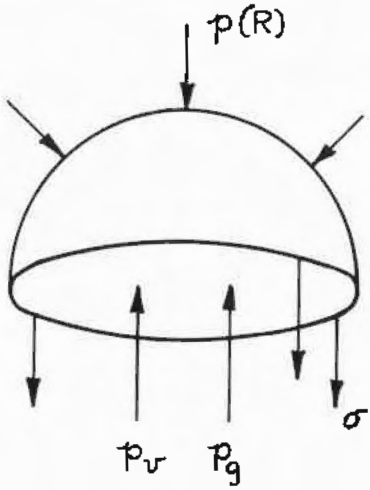


FIG. I SECTION THROUGH SPHERICAL BUBBLE

The quantity f_1 is the force encouraging growth. Ignoring the dynamic effects we have

$$\text{for stability } \frac{\partial f_1}{\partial R} < 0 \qquad (8)$$

$$\text{for unstability } \frac{\partial f_1}{\partial R} > 0$$

For constant temperature and constant gas content N, the critical radius is

$$R = R^* = \left(\frac{3NT}{2\sigma}\right)^{1/2}$$

Combining with $f_1 = 0$ we get

$$R^* = \frac{-4\sigma}{3(p_\infty - p_v)_{crit}}$$

or

$$(p_\infty - p_v)_{crit} = -\frac{4\sigma}{3R^*} \qquad (9)$$

In Fig. 2 is shown $(p_\infty - p_v)$ versus nucleus diameter $(D = 2R)$ for a constant temperature (and surface tension) and various constant values of gas content. The minimum of each curve corresponds to the critical value R^*. The diagram shows clearly that once $(p_\infty - p_v)$ reaches its critical or minimum value, equilibrium is unstable and a bubble will continue to grow without further pressure reduction. It also shows that the ambient pressure p_∞ must fall below the vapor pressure to cause instability of a gas bubble. The magnitude of this drop below p_v depends on the amount of gas in the nucleus, and hence on the original size of the nucleus. For smaller and smaller amounts of gas the critical value of $(p_\infty - p_v)$ becomes increasingly negative giving high tensions in the liquid.

For a *vapor bubble*, p_g is zero and the equilibrium is given by

$$f_2(R) = -\left[p_\infty - p_v + \frac{2\sigma}{R}\right] = 0 \tag{10}$$

or

$$R_{equil.} = \frac{-2\sigma}{p_\infty - p_v} \tag{11}$$

For constant temperature

$$\frac{\partial f_2}{\partial R} = \frac{2\sigma}{R^2} > 0 \text{ always} \tag{12}$$

so that there is no stable equilibrium.
Then

$$\begin{aligned} &\text{for } R < \left[\frac{2\sigma}{p_v - p_\infty}\right] \text{ the bubble collapses} \\ &\text{for } R > \left[\frac{2\sigma}{p_v - p_\infty}\right] \text{ the bubble expands indefinitely.} \end{aligned} \tag{13}$$

Thus, the static equilibrium radius is the critical radius for a vapor-filled bubble and

$$R^*_{vap} = \frac{3}{2} R^*_{gas} \tag{14}$$

Now let us examine the origin of nuclei. First we note that simple free spherical bubbles will dissolve under surface tension forces after some time. Nuclei can only be stabilized on the solid boundaries of the system or on entrained foreign matter. All solid surfaces contain micro-

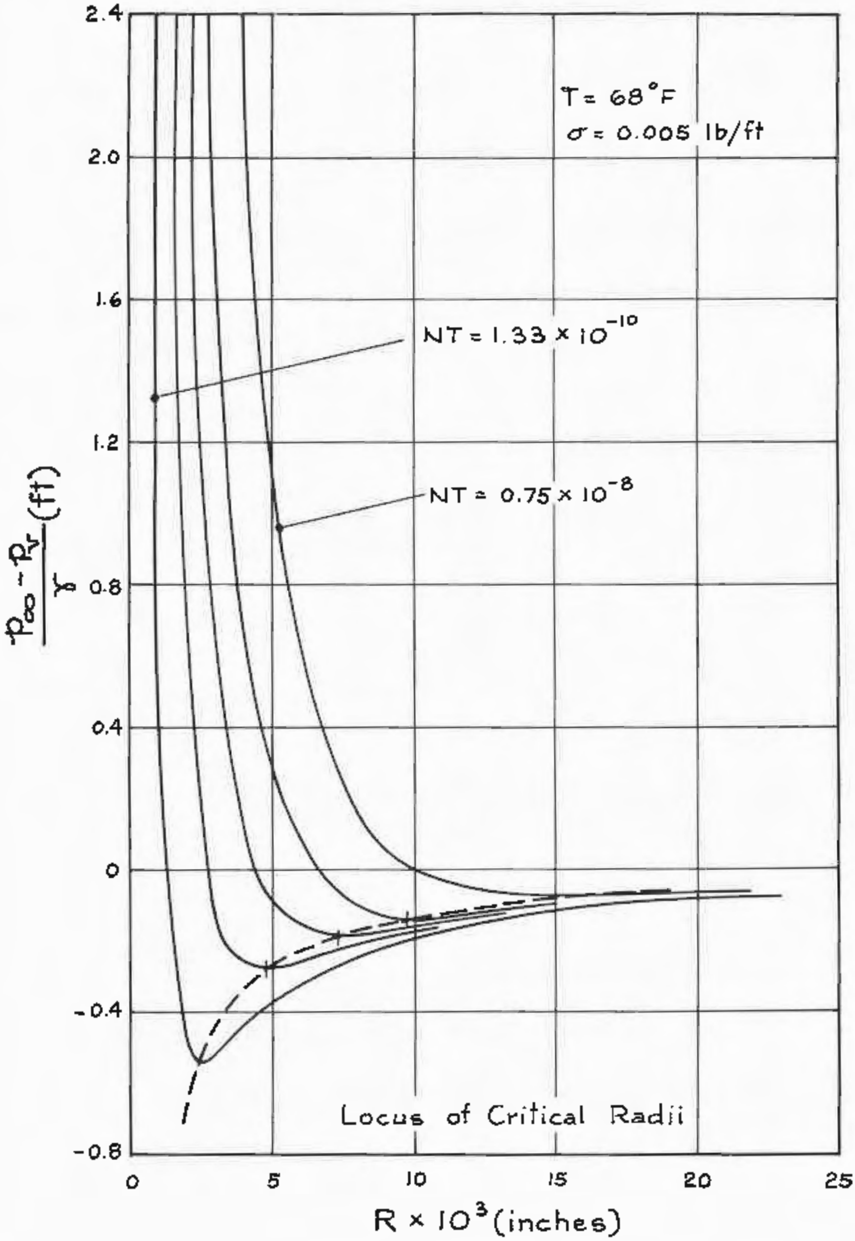


FIG. 2 PRESSURE VERSUS BUBBLE SIZE FOR GAS NUCLEUS

scopic crevices and cracks and in most cases these surfaces and cracks are unclean and hence hydrophobic and "non-wettable". It has been shown that gas can be trapped in such a crevice. This is possible because under certain circumstances the surface tension of the gas trapped in the crevice acts to decrease rather than increase the gas partial pressure. In Fig. 3 a hydrophobic (non-wettable) conical crack

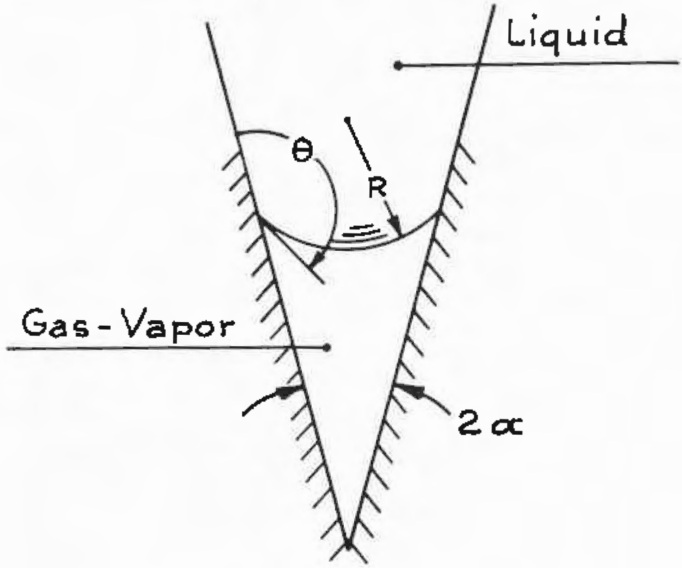


FIG. 3 INTERFACE EQUILIBRIUM IN HYDROPHOBIC CONICAL CRACK

has the liquid-gas interface with contact angle $\theta = (\pi/2 + \alpha)$. For this condition equilibrium is given by

$$p_g + p_v = p - \frac{2\sigma}{R} \tag{15}$$

Assuming p and p_v both constant, let p_g exceed the saturation value. Gas will dissolve causing the interface to advance into the crevice.

The radius R is simultaneously reduced causing $p_g = NT/R^3$ to drop until diffusion equilibrium occurs. If p_g is less than the saturation value, the reverse will occur, gas diffusing from the liquid and the volume growing.

If the receding angle of contact θ_R remains

$$\theta_R \geqslant (\pi/2 + \alpha) \tag{16}$$

equilibrium will be established. On the other hand, if the receding angle becomes

$$\theta < (\pi/2 + \alpha) \tag{17}$$

the curvature of the liquid gas interface changes sign and p_g continues to decrease. Then gas release will continue and the gas pocket will emerge beyond the crevice to be swept away by passing flow. Thus we have a mechanism for "seeding" a liquid stream with gas nuclei.

If only vapor exists in the hydrophobic crevice, equilibrium requires

$$p_v = p - \frac{2\sigma}{R} \tag{18}$$

Assume p exceeds the equilibrium value. The interface will advance into the crevice and the radius R will decrease. It is possible then for the vapor pocket to become stabilized and serve as a nucleating agent on reversal of the pressure p.

Of course, pure liquids *completely void* of dissolved gases will be encountered rarely (possibly some cryogenic liquids may be gas-free). It appears that in all practical cases cavitation originates from gas nuclei.

B. *Travelling Cavities*

After inception, cavitation occurs in different degrees and assumes different physical appearances depending on magnitude of the pressure reduction and on the configurations of the containing solid boundaries. The various manifestations can be grouped conveniently into two types or classifications called *travelling cavities* and *fixed cavities.*

Travelling cavities are individual transient bubbles which form from nuclei and move with the liquid as they expand, shrink and then collapse. These cavities may originate at low pressure points along a solid boundary or in the liquid interior either at the cores of moving vortices or in a highly turbulent shear zone. At the points of their collapse, high "pin-point" pressures occur. Fig. 4 shows travelling cavities originating at the minimum pressure zone on an immersed "ogive" surface. These cavities, which were "stopped" with microflash photography, grow rapidly to a maximum diameter and then begin to collapse as they sweep into a region of pressure higher than the vapor pressure. The life of a single bubble is of the order of milliseconds. To the naked eye a case of travelling cavitation will appear as a quasi-steady zone of "froth." Flash photography or high speed moving pictures are needed to show the individual cavities clearly.

The rate of growth and collapse of travelling cavities and the final pressure at collapse depends on various properties and factors. Compressibility, viscous effects and entrained air tend to retard collapse and reduce the maximum pressure. Surface tension acts to aid collapse. As we have seen, gas content is important in the inception process. However, beyond inception the vaporization rate is so much faster than gas diffusion from the liquid, that gas content becomes a negligible factor in growth. Only in the final stages of collapse are the gas effects again important.

For spherical cavities the assumption of an empty bubble and incompressible liquid in Eq. (3) will predict bubble growth and collapse accurately down to about one-fourth maximum diameter. Beyond this the other effects must be introduced. Particularly, heat transfer at the bubble wall and the effect of temperature on surface tension is important in the last stages of collapse. Because of the complexity of the problem there is no agreement on the theoretical value for the maximum collapse pressures. Computations have given values ranging from several hundred to several thousand atmospheres. Experimental values range from 10 atmospheres at 10 cm distance from the collapse point up to 12,000 atmospheres at the collapse point on a boundary. The latter value is interpreted from stress waves appearing in a photoelastic material exposed to cavitation. It is only agreed that extremely high intensities are possible.

Mechanical damage is associated with these high pin-point pressures which at least initiate local deformation and material removal. Other factors, such as corrosive and electrochemical effects, may contribute in an important way but are not necessary to the destruction. The interaction between the high pressure forces and mechanical response of the solid material appears to be primary. Different materials

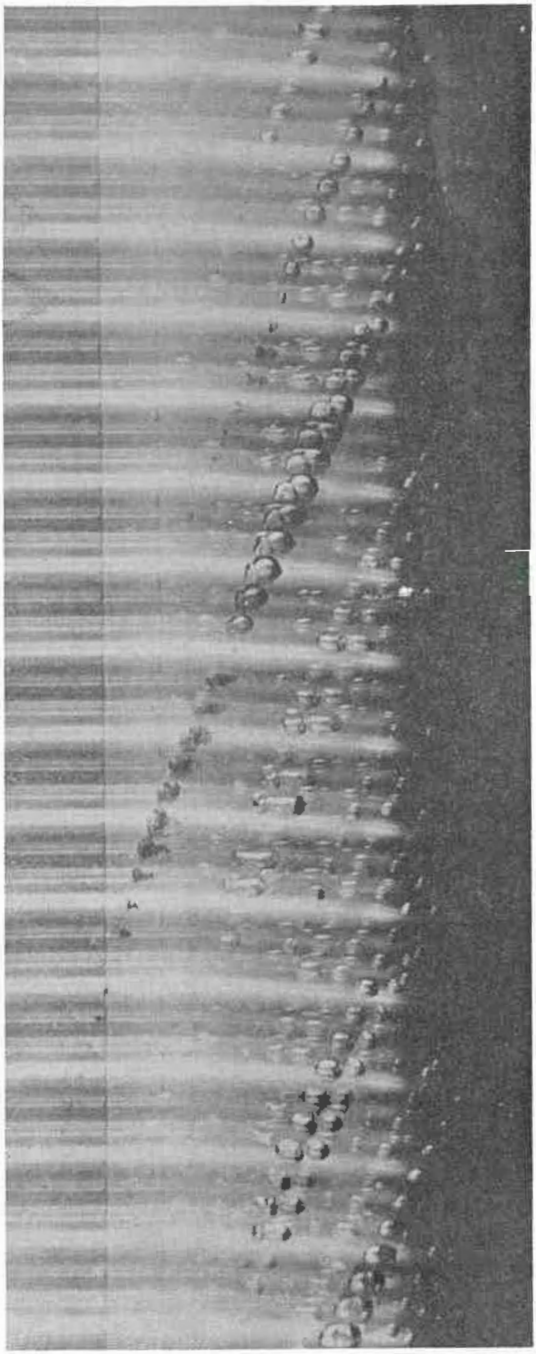


FIG. 4.—TRAVELLING CAVITIES ON THE OGIVE NOSE OF A CIRCULAR CYLINDER

deform and may not lose weight rapidly, while brittle materials may lose weight very soon after exposure cavitation.

Travelling cavities may oscillate as they change size. Also, complete collapse is often followed by a series of re-openings and reclosures. Both are evidences of a pulsating pressure condition in the cavitation process in addition to the intermittency of the appearance and demise of individual bubbles. It is not clear what role "rebounding" at collapse plays in the mechanism of damage, but such oscillations do contribute to the noise generated in a cavitating zone.

C. *Fixed Cavities*

At advanced stages of cavitation, a fixed cavity may occur in the form of a pocket attached to a rigid boundary around which the main liquid flow passes. In hydraulic equipment, the fixed or attached cavities are very important. As the pressure and velocity conditions for inception are exceeded, cavitation advances to a stage where it causes the main flow to separate from the guiding surface leaving a cavitated zone or space between the solid surface and the boundary of separation. The main flow follows a trajectory determined by the pressure field and usually returns to the solid surface at some point downstream. It is at this point that damage usually occurs. While there is a zone of condensation at the terminus of the cavitated volume, sudden collapse is not readily apparent. A question is raised about the damage process and the applicability of the ideas from travelling cavity considerations. Also these flows are definitely unsteady giving rise to unsteady pressure and force variations.

The conditions for inception and character of subsequent stages of cavitation depend on the shape of the solid boundary or boundaries of the liquid space. In hydrodynamic machines and structures, cavitation is due to pressure reductions occurring as the result of a relative flow past an immersed body, vane, hydrofoil or other guiding surface. In all cases as the liquid is deflected around the solid boundary, a curvilinear flow field is established having pressure gradients normal to the streamlines opposing the centripetal accelerations. In addition, convective accelerations along streamlines will cause negative pressure gradients in the flow direction and decelerations will cause positive gradients. Thus, on the solid boundaries the pressure intensity tends to fall if the boundary curves away from the flow or if the flow accelerates along the boundary. The pressure tends to rise if the boundary curves into the flow or the flow decelerates along the boundary. There results a pressure distribution along any immersed surface consisting of a minimum pressure point with higher pressures preceding and following the minimum.

Since boundary layer separation tends to occur in regions of high adverse pressure gradients or excessively sharp boundary curvature, a distinction can be made between two general classes of bodies or guiding surfaces. They are:

1. Streamlined surfaces for which non-cavitating flow boundary layer separation is avoided.
2. Projecting surfaces which present sharp or possibly zero curvature projections to the relative flow so that non-cavitating boundary layer separation coincides with the minimum pressure point.

For the streamlined surfaces, so long as boundary layer separation does not occur, the liquid will remain in contact with the boundary until sufficient tension is developed by pressure reduction to cause "rupture" at weak points. Initially such ruptures will occur as discrete and nearly spherical bubbles and the large bulk of the liquid will pass the minimum pressure point without break in continuity. For the projecting type surfaces, cavitation generally will occur first at the cores of vortices established in the zone of high shear along the separation boundary, and well away from the solid surface. Due to the highly vortical motion of the liquid, as at the rupture points, the resulting cavities develop irregular shapes.

In general, as the pressure is reduced or relative velocity increased in a given situation, bubbles appear more diffusely and with greater frequency and persist longer. There results for streamlined type boundaries a gradual blanketing by coalescing bubbles. The mingling cavities effectively break the continuity of the liquid stream and complete breakaway from the boundary may occur causing a fixed or attached cavity.

The maximum length of a fixed cavity depends on the pressure field and termination may be on the solid boundary or may extend well beyond the body before the liquid stream closes together again. Cavities which are long compared to the body are often called supercavities. Supercavitation has become important in connection with high speed propellers and hydrofoils. By purposely designing to operate in the supercavitation range, the speed of operation has been extended. Currently, attention is being given to development of supercavitating hydraulic turbines and pumps. Figs. 5, 6 and 7 show fixed cavity examples.

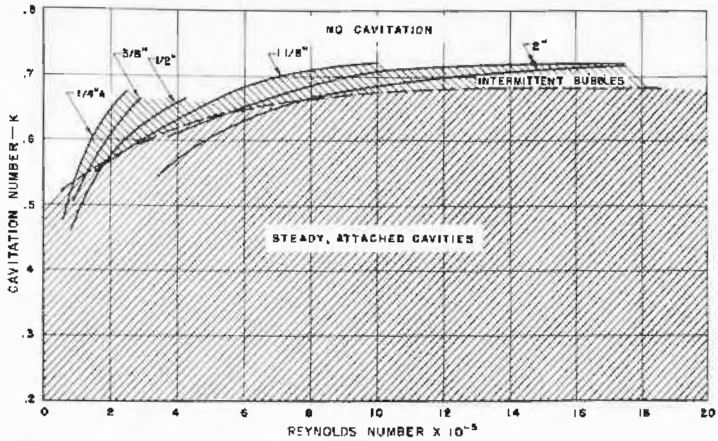
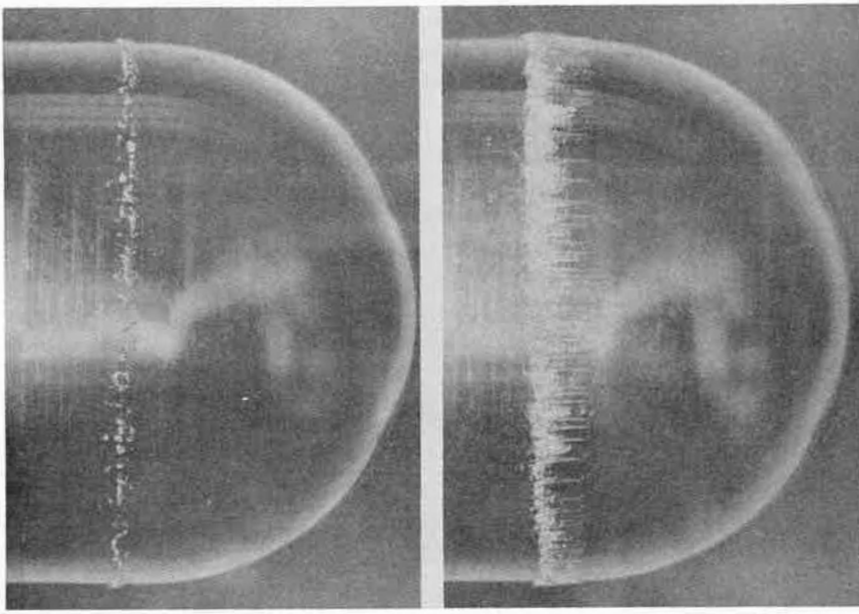


Fig. 5a - Regions of intermittent bubble and clear, attached cavity cavitation

For low cavitation numbers cavities formed by the same body have been observed either as

1. a smooth transparent walled void apparently enclosing a vapor or gas-filled volume,
2. a cavitated region with a turbulent fluctuation interface surface. This surface is seemingly composed of an intermingling of transient cavities, and its roughness reduces its transparency, sometimes making it opaque.

The smooth transparent cavities are practically steady state in that the cavity wall is nearly stationary with fluctuations and oscillations only at the trailing end. The most useful examples of supercavitation are such smooth transparent cavities. The forward portions of these cavities are essentially free-streamline flows and certain examples have been analyzed as such. The rough and less transparent cavity walls are steady only in the sense that there is an average envelope of fixed dimensions. The reasons for these differences have not been completely established, but there is evidence that the smooth transparent free stream type is associated with the presence of large amounts of non-



$K = 0.67$ $K = 0.64$

FIG. 5b—INTERMITTENT BUBBLE (LEFT) AND ATTACHED CAVITY (RIGHT) CAVITATION ON 2-IN. DIAMETER HEMISPHERE

$V = 50$ fps, $R_e = 8.2 \times 10^5$

Figs. 5a, 5b taken from Kermeen (1)

condensable gas. It is known, for example, that a clear cavity can be maintained behind an immersed body by venting the wake with air. It is also clear that the mechanics of cavity maintenance and downstream closure will be different in the two cases. In general, vaporization and gas release through the cavity walls must be in equilibrium with the vapor condensation and gas solution at cavity closure. The smooth transparent walls are indications that very little vaporization is occurring and suggests that the smooth walled cavity can be maintained only if it can accumulate a large mass of non-condensable gas. For small amounts of gas the cavity must be maintained by boiling through the turbulent interface.

The difference in steadiness between the two cases affects resulting vibration and noise in hydraulic equipment. In fact, the favorable effect of bleeding air into cavitating hydraulic turbines suggests that a transition from the unsteady to the more steady type may

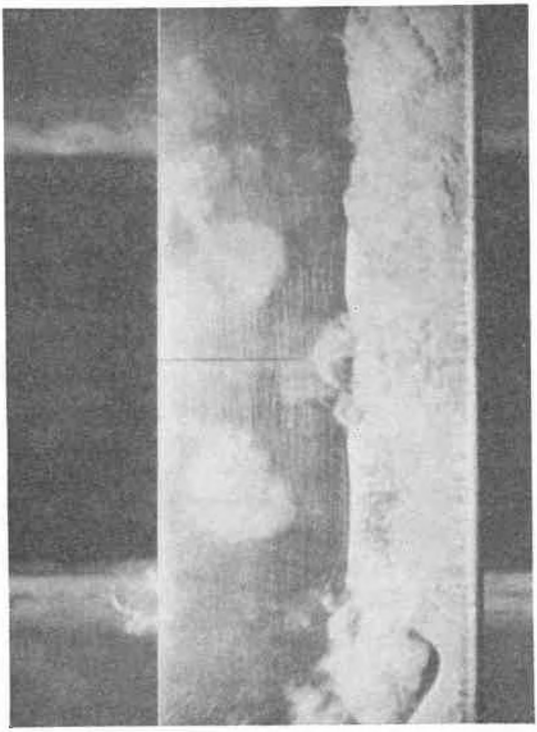


FIG. 6.—FIXED CAVITY ON HYDROFOIL SURFACE

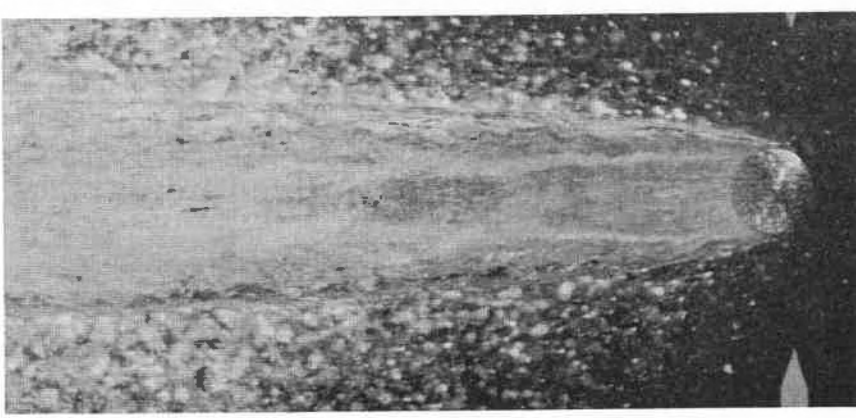


FIG. 7.—SUPERCAVITY ATTACHED TO CIRCULAR CYLINDER

take place. Then too, it is suspected that, if the air mass is important in distinguishing the two types, the damage potential of the more turbulent type is greater than that of the smooth walled cavity.

In some cases fixed cavities display a cycling process. Following initial formation, the cavity grows by a process of liquid entrainment out of the pocket region and then refills by a re-entrant liquid jet from the high pressure zone caused by the main flow closing back on the solid boundary as shown in the schematic diagram in Fig. 8. When the re-entrant jet reaches the upper end of the cavity, the process is disrupted and the bulk of the cavitated zone breaks off and is swept downstream. The process is then repeated. The cycle time is short (of the order of 0.02 seconds at 50 feet per second) and decreases with main flow velocity increase. The series of motion picture frames in Fig. 8 show the cavity growth, filling and breakoff. The re-entrant flow is seen clearly in fourth and fifth frame. The last frame shows the breakup of the surface which occurs at complete filling. As indicated in the schematic below the photographs, the liquid in the re-entrant flow is largely fluid that was adjacent to the cavity free surface, a point of significance in the observed damage pattern. The surface of the main cavity has been observed to be covered with a multitude of small bubbles which grow like travelling cavities. These bubbles are swept into the downstream stagnation pressure zone (S) in Figure 8 and disappear. It is thought that a number collapse at the boundary and are the cause of the observed damage. With cycling, the stagnation point (S) sweeps the length of the cavitated zone but with greater concentration near the end of the fully developed cavity. This is the region of greatest damage.

The cycling just described does not occur below some critical velocity. Probably insufficient energy and momentum prevent complete cavity filling and lead to stability. With the increasing use of higher velocities in structures and machines the possibility of cycling becomes a more important consideration.

Basic Data for Design Applications

A. *Cavitation Number and Scale Effects*

In comparing cavitation conditions, a useful *cavitation number* (called sigma) is defined as

$$\sigma = \frac{p_\infty - p_b}{\rho\, U_\infty^2/2} \qquad (19)$$

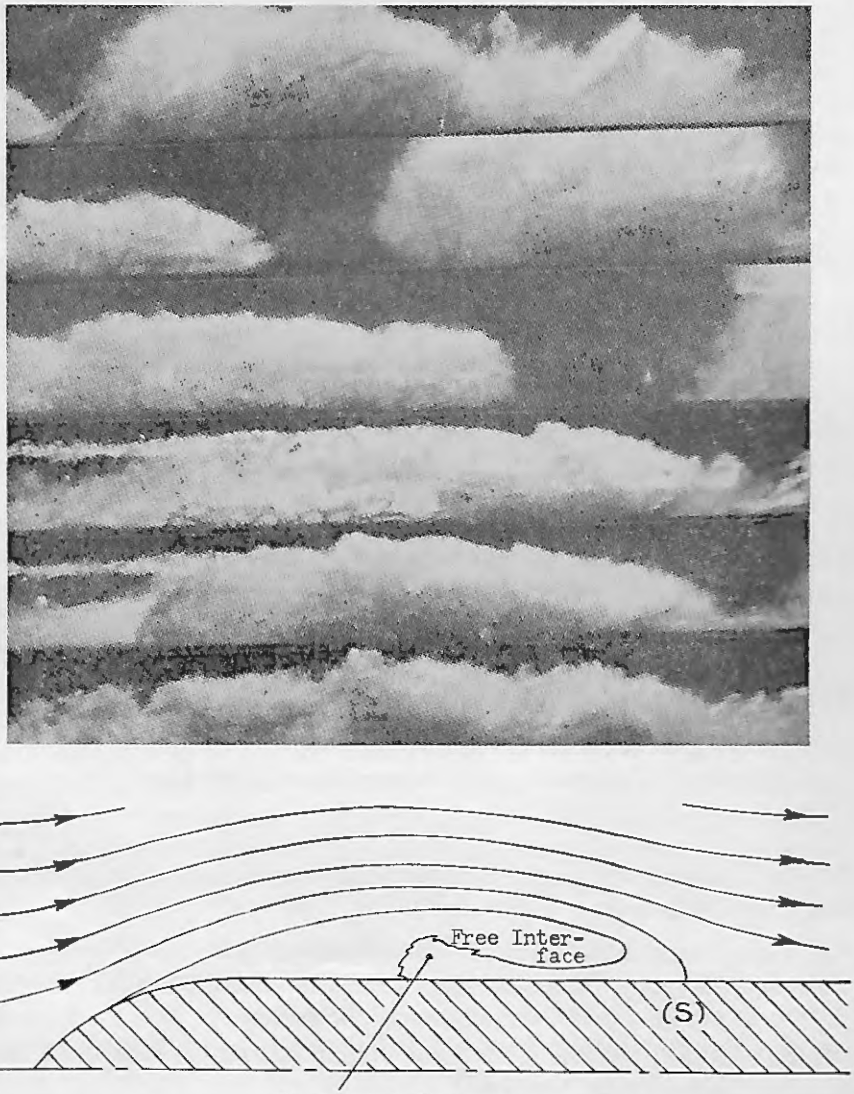


Fig. 8 Formation and Re-entrant Flow in a Cycling Fixed Cavity [After R.T. Knapp (2)]

where

p_b = local pressure in the cavitated region
p_∞ = free stream pressure
U_∞ = free stream velocity.

If $p_b = p_{min}$, the minimum pressure on a submerged boundary, sigma equals the minimum pressure coefficient, or

$$C_{p_{min}} = \frac{p_\infty - p_{min}}{\rho U_\infty^2/2} \tag{20}$$

At cavitation inception $\sigma = \sigma_i$ defined as

$$\sigma_i = \frac{p_i - p_b}{\rho U_\infty^2/2} \tag{21}$$

where

p_i = free stream pressure at which inception occurs some place in the flow system.

For convenience it is customary to use the vapor pressure instead of p_b. Thus we write

$$\sigma_i = \frac{p_i - p_v}{\rho U_\infty^2/2} \text{ and } \sigma = \frac{p_\infty - p_v}{\rho U_\infty^2/2} \tag{22}$$

where

p_v = vapor pressure.
If cavitation inception occurs when $p_{min} = p_v$ we have

$$\sigma_i = C_{p_{min}} \tag{23}$$

Departures from the equality of Eq. (23) are known as *scale effects.* They are related to

(a) the nucleation process already discussed,
(b) Reynolds number effects on the distribution of average pressure over the immersed boundary,
(c) turbulence and separation eddy effects on the local minimum pressure in the fluid,
(d) roughness effects.

These effects combined may lead to inception pressures higher or lower than predictions using vapor pressure. Roughness is a special problem which we will discuss in the next section. Turbulence causes local pressures to fluctuate about the mean. Turbulence in a boundary layer has been observed (4) to cause a local minimum of 0.40 to 0.57 feet of water below the existing mean pressure. This would make the measured σ_i [by Eq. (22)] higher than $C_{p_{min}}$ [by Eq. (20)]. Separation and eddy effects produce similar results, but the magnitudes are not well established. Nucleation from gas nuclei, which we have discussed, tends to make the measured σ_i less than $C_{p_{min}}$ so long as we adhere to the definition that cavitation is the explosive growth of the bubbles. On the other hand, with large quantities of entrained undissolved gas, growth by diffusion may lead to cavitation-like conditions and to fixed cavities at pressures equal, or even above, the vapor pressure. In experiments, therefore, every effort is made to operate at pressures corresponding to undersaturated air solutions. If the air is completely dissolved, there appears to be no problem. Consequently, pressurized samples of water show great tensions before cavitation starts because all gas (and hence most nuclei) is driven into solution.

For advanced stages of cavitation

$$\sigma < \sigma_i \tag{24}$$

The pressure p_b remains approximately constant at the vapor pressure once cavitation occurs and the pressure distribution over the immersed surface is distorted from the non-cavitating value. Hence, lowering p_∞ or increasing U_∞ causes σ to fall below σ_i.

B. *Roughness Effects*

Cavitation-free operation of hydraulic devices involves both the proper design and control of the surface finish. Roughness, whether randomly distributed or composed of isolated projections, can result in early cavitation leading to either performance changes or damage or both. There have been many well-publicized examples of concrete structures, hydraulic turbines and centrifugal pumps which have suffered costly damages due to cavitation which roughness initiated.

In general, the extra potential for cavitation inception and its various consequences is measured by the relative height of the surface projections and the boundary layer thickness. Holl (3) has investigated isolated roughnesses protruding into a turbulent boundary layer. A sample of his results is shown in Fig. 9 in the plot of incipient cavitation number σ_i versus the ratio h/δ of roughness height

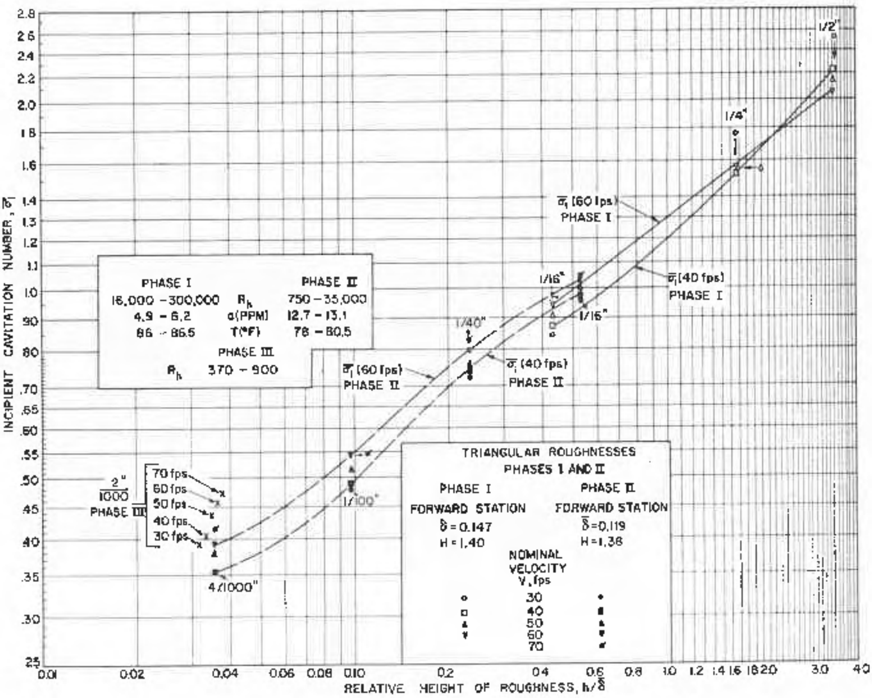


Fig. 9 Incipient Cavitation Numbers for Triangular Roughnesses [After Holl (3)]

to boundary layer thickness. In these the "inception" pressure of cavitation was measured as the pressure at which cavitation disappeared. This procedure gave more consistent results. In addition, average values of several determinations of σ_i and δ for each flow condition are used. The roughness elements were two-dimensional cylindrical strips attached to a plane surface normal to the flow direction. Using these results, Holl gives rules for computing the percent increase of the inception cavitation parameter of a parent body with a projection which we will describe.

To predict the effect, four factors must be known:

(a) roughness height,
(b) boundary layer thickness in the vicinity of the roughness,

(c) pressure distribution on the parent body in the vicinity of the roughness,

(d) the incipient cavitation number of the roughness considered alone (that is when the roughness is on a flat plate).

The minimum pressure coefficient for a parent body with roughness can be written

$$C_{p_{minR}} = \frac{p_\infty - p_{minR}}{\rho U_\infty^2/2} \tag{25}$$

$$= \frac{p_\infty - p}{\rho U_\infty^2/2} + \frac{p - p_{minR}}{\rho U^2/2}\left(\frac{U}{U_\infty}\right) \tag{26}$$

where

p_∞ = free stream pressure
p_{minR} = min. press. caused by roughness irregularity
U_∞ = free stream velocity
p = pressure in the vicinity of the roughness
U = velocity outside the boundary layer in the vicinity of the roughness.

The pressure coefficient for the smooth body at the location of the roughness is

$$C_{ps} \approx \frac{p_\infty - p}{\rho U_\infty^2/2} \tag{27}$$

Then, for small roughness,

$$\left(\frac{U}{U_\infty}\right)^2 \approx 1 + C_{ps} \tag{28}$$

Finally, the minimum pressure coefficient of the roughness (as would be obtained when the roughness is on a flat plate) is

$$C_{p_{min}} = \frac{p - p_{minR}}{\rho U^2/2} \tag{29}$$

We have then

$$C_{p_{minR}} = C_{ps} + [1 + C_{ps}]\, C_{p_{min}} \tag{30}$$

Assuming that $p_{minR} = p_v$ at the inception of cavitation

$$\sigma_{i\,R} = C_{ps} + [1 + C_{ps}]\, \sigma_i \tag{31}$$

where

σ_{iR} = the incipient cavitation number of the body with roughness

σ_i = the incipient cavitation number of the roughness element on a plane.

The roughness is most detrimental when located at the minimum pressure point on the body. Then $C_{pS} = C_{p_{minS}}$ and

$$\sigma_{iR} = C_{p_{minS}} + (1 + C_{p_{minS}})\, \sigma_i \tag{32}$$

The thickness of the boundary layer can be estimated using the power law relation

$$\delta = \frac{0.37x}{\mathbb{R}_x^{1/5}} \tag{33}$$

so long as the boundary layer is fully turbulent. The values of σ_i are obtained from test results.

Computations for a triangular projection on a hydrofoil's minimum pressure point are summarized below. The right hand column gives the percent increase and is computed using the approximation that for the smooth body $\delta_{iS} = C_{p_{minS}}$. Then

$$\%\ \text{increase} = \frac{100\,(\sigma_{iR} - C_{p_{minS}})}{C_{p_{minS}}} \tag{34}$$

Calculations for $C_{p_{minS}} = 0.5$, $\delta = 0.048$, in.

h, in.	h/δ	σ_i	σ_{iR}	Percent increase in σ_{iS}
0.001	0.021	0.34	1.01	102
0.002	0.042	0.45	1.18	135
0.004	0.083	0.67	1.50	201
0.008	0.167	0.90	1.85	270

An extrapolation formula to other roughness elements is suggested as

$$\sigma_i = \left(\frac{u_h}{U}\right)^2 \sigma_{i_\infty} \tag{35}$$

where

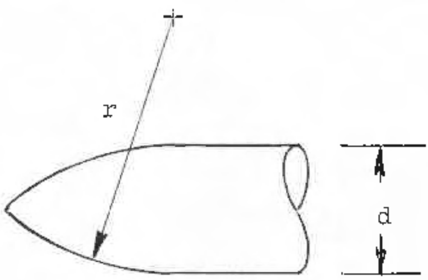
σ_{i_∞} = incipient cavitation number of a roughness element in a uniform stream (i.e. $h/\delta = \infty$)

u_h = velocity in the boundary layer at the height of the roughness projection.

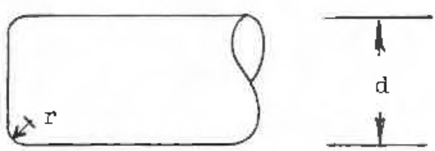
For sharp-edged roughness elements it is suggested that σ_i for sharp-edged disks might be used to approximate $\sigma_{i\infty}$.

C. *Cavitation on Bodies of Revolution*

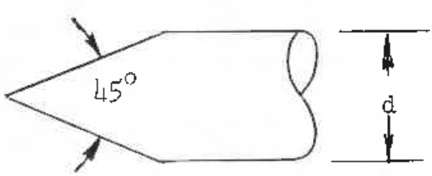
Among the basic design elements are simple bodies of revolution. In the accompanying Table I are values of cavitation numbers, measured in a water tunnel, for families of ogival and conical tips, or noses, attached to circular cylindrical bodies. The ogive and the cones are defined as shown in the sketches below.



r = 2d   2-Caliber Ogive



r = d/4   1/4-Caliber Ogive



45° Cone

The cavitation numbers are compared with measured minimum pressure coefficients at inception and at advanced stages of cavitation. The $C_{p_{min}}$ values at inception are the non-cavitating values. The $C_{p_{min}}$ values after inception are, of course, those measured under cavitating conditions. All the data are for Reynolds numbers at which non-cavitating flow separation was assured.

Some points are of special interest. First, the observed value of σ_i tends to be greater than the measured minimum pressure coefficient. This is attributed to microscale cavitation occurring in locally separated regions in which the local pressure reached the cavitating condition before the pressure at the surface of the body dropped that low. Second, for advanced stages of cavitation (at which a statistically steady attached cavity existed) σ is equal to the measured $C_{p_{min}}$. This indicates that the mean pressure in the cavitated region is the vapor pressure, to the accuracy of the measurements. Third, for those noses with sharp corners and severe separation $C_{p_{min}}$ rises slightly after inception before it decreases (ultimately linearly with σ). Apparently a small amount of cavitation changes the intensity of the separation and, hence, the pressure in the separation zone.

TABLE I

Cavitation Numbers Compared with Minimum Pressure Coefficients

[Adopted from Rouse and McNown (5)]

	At Inception		Beyond Inception	
	σ_i	$C_{p_{min}}$	σ	$C_{p_{min}}$
Ogives				
2-caliber	$\geqslant 0.40$	0.32	0.20	0.20
1-caliber	$\geqslant 0.55$	0.48	0.24	0.24
½-caliber	$\geqslant 0.80$	0.73	0.20	0.20
¼-caliber	$\geqslant 1.30$	1.08	0.20	0.20
⅛-caliber $\mathbb{R} = 2.4 \times 10^5$	1.30	0.82	0.30	0.29
$\mathbb{R} = 3.2 \times 10^5$	$\geqslant 1.30$	1.16	0.30	0.29
0-caliber (square-ended)	$\geqslant 1.80$	0.62	1.50	0.64
			0.30	0.29
Conical Heads				
45°	$\geqslant 1.40$	1.10	1.25	1.17
			0.30	0.30
90°	$\geqslant 1.60$	0.69	1.20	0.76
			0.30	0.30
135°	1.70	0.63	1.30	0.68
			0.30	0.30
180° (0-caliber, square-	$\geqslant 1.80$	0.62	1.50	0.64
ended)			0.30	0.29
135° *concave*	$\geqslant 1.80$	0.62	1.30	0.65
			0.30	0.30

An interesting comparison with the 0-caliber nose is the result of experiments with disks normal to the flow. Kermeen et al. (6) made tests over a wide Reynolds number range, as summarized in Table II, using disks from 1/16″ to 1-1/2″ diameter. Note that the 0-caliber nose (and the 135° concave nose) falls in the disk range.

TABLE II
INCEPTION CAVITATION NUMBERS FOR DISKS
[Adopted from Kermeen et al. (6)]

Reynolds Number $\times 10^{-5}$	σ_i
0.25	1.0
0.50	1.3
1.0	1.5
2.0	1.8
3.0	1.9
4.0	2.0
6.0	2.1

Beyond inception the flow resistance changes. This is typified by drag on bodies. As the fixed cavity stage develops, the drag coefficient at first increases over the non-cavitating value. As a supercavity is formed the coefficient then decreases. In the supercavity range the drag coefficient for bodies of revolution is approximately

$$C_D(\sigma) = C_D(0) + \sigma \tag{36}$$

where

$$C_D = \frac{\text{Drag}}{(\rho\, U_\infty^2/2)\, A} \tag{37}$$

with

$$A = \text{projected area of the body}$$
$$C_D(0) = \text{coefficient at } \sigma = 0.$$

Typical values of the zero sigma C_D are given in Table III as obtained by extrapolating measurements at finite sigmas back to zero. The data are from various sources.

D. *Cavitation on Struts*

As a final example of design data, Table IV gives some values from various sources of the zero sigma drag coefficients for struts. Such data is of interest, not only in connection with struts as such,

TABLE III
DRAG COEFFICIENTS AT ZERO CAVITATION NUMBER
BODIES OF REVOLUTION

Nose Form	$C_D(0)$
Disk	0.8
2-caliber Ogive	0.15
Hemisphere	0.26
Hollow Cylinder	1.0
Cones	
15°	0.12
30°	0.36
45°	0.52
90°	0.81

but relates to flow over the leading edges of all kinds of vanes and hydrofoils. For such two-dimensional bodies the drag coefficient in the supercavitating range near zero sigma is approximately

$$C_D(\sigma) = C_D(0)\ (1 + \sigma) \qquad (38)$$

Compared with Table III, this shows higher σ for the two-dimensional cases. This is in agreement with the higher $C_{p_{min}}$ of non-cavitating flows for these cases.

TABLE IV
DRAG COEFFICIENTS AT ZERO CAVITATION NUMBER
TWO-DIMENSIONAL STRUTS

Strut Form	$C_D(0)$
Ellipse (2:1)	0.35
Circle	0.50
Wedges	
15°	0.28
30°	0.50
45°	0.63
90°	0.88

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HYDRO ELECTRIC POWER IN THE GREAT NORTHERN PAPER COMPANY

By Dr. R. S. Kleinschmidt,* *Member*

(Presented at a meeting of the Hydraulics Section, B.S.C.E., on October 28, 1964.)

Shortly after the Bangor and Aroostook Railroad was pushed into Northern Aroostook County toward the end of the 19th century, an engineer and lumberman by the name of Charles Mullen was cruising the territory around Grand Pitch on the West Branch of the Penobscot River. He realized the potential for developing water power at this site, where the river with an average flow of about 3000 cfs dropped 110 ft., and the extensive forests of spruce and fir in all directions made it seem a natural site for a paper mill. Mr.

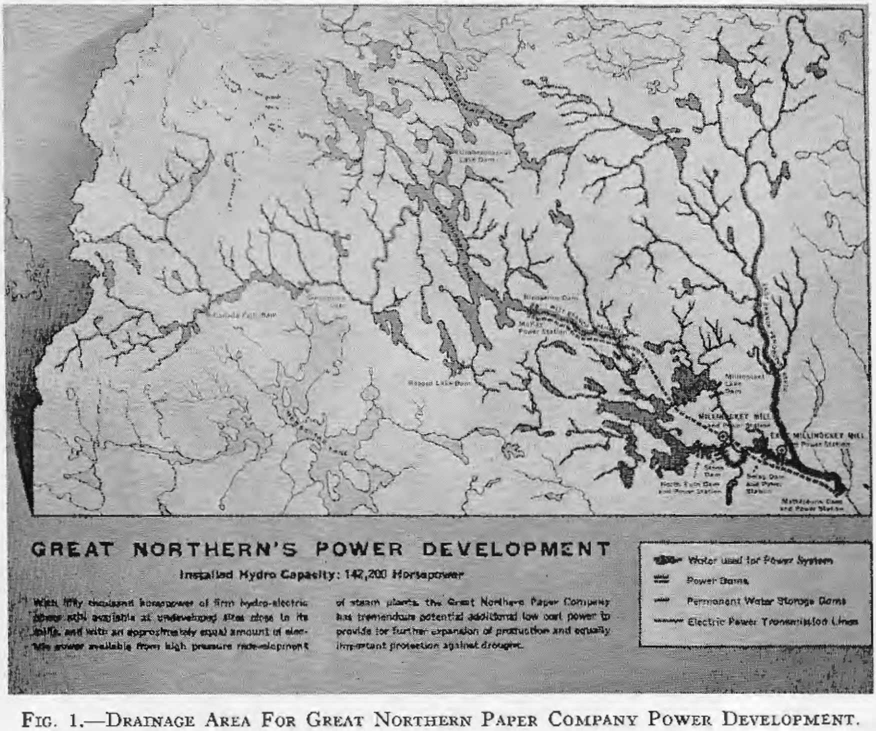

Fig. 1.—Drainage Area For Great Northern Paper Company Power Development.

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Mullen discussed his ideas with Garrett Schenck, who was already a successful paper mill builder and operator. Mr. Schenck was undoubtedly enthusiastic about the plan and in 1897 formed the Northern Development Company to build a mill. A young engineer by the name of Hardy S. Ferguson was engaged to prepare plans and specifications for the new mill, and in 1899, construction was started. At this time the name of the enterprise was changed to Great Northern Paper Company. Mr. Ferguson was a man of exceptional ability. He explored all the possibilities of the area most thoroughly and a large part of the success of the venture was due to his skill and judgment in taking best advantage of the natural features of the site. The eight paper machine mill with a capacity of 240 tons per day was completed during the Fall of 1900, and on November 1st, Mr. Schenck officially started up the mill by closing the switch that put the Wood Room machinery into operation. The first roll of newsprint was produced eight days later. For many years the Millinocket Mill was the largest newsprint mill in the world. Technical descriptions of the power development at Millinocket and the other power sites will be given later in the paper.

To make better utilization of the annual river flow, upstream storage was required. In 1903, an old log driving dam at the outlet of North Twin Lake was replaced by a new concrete structure giving approximately 15 bcf (billion cubic feet) of storage and what was formerly four separate lakes now flowed into one large lake.

Business was good and demand for newsprint caused the Company to expand rapidly. In 1907, the Mill at East Millinocket was built approximately 10 miles below Millinocket. For this purpose two power sites on the river were developed. At the paper mill itself at East Millinocket, a dam was built across the river at Burntland Rips, with a developed head of about 25 ft. In addition, a much larger dam was built about a mile upstream at Dolby, where a 50 ft. fall could be developed. At East Millinocket, the entire power development was used to mechanically drive pulp wood grinders. A pulp mill was also included at Dolby and in addition two hydro electric generators were installed.

The East Millinocket Mill went into production in August 1907 with three machines producing a total of 120 tons of newsprint per day. In 1913 a fourth machine was added at East Millinocket.

In 1914 the historic ninth machine was installed at Millinocket. This was the first paper machine designed to run at a speed of 1000 ft.

per minute. From time to time during this period, small less efficient water wheels were replaced with larger and more efficient units.

By 1916 it was evident that to meet the demands for power more storage would be required to better regulate the river flow. At Ripogenus Gorge, about 35 miles upriver from Millinocket, there was a perfect dam site where the relatively high land of the upper basin dropped rapidly to the lower elevations around Millinocket. In 1916, Ripogenus Dam was built at this point. At the time it was the largest storage dam ever built by a private corporation for its own use. Ripogenus Dam provided 30 bcf more of storage on the West Branch and achieved almost complete control of the river. From 1916 to 1930, increase in power demand was met by expanding and improving the installations at the existing stations. Four new water wheels were installed at Millinocket in 1916 and four others were replaced in 1922. Other replacements were made from time to time at East Millinocket and Dolby, but, by 1930, full development of these three power sites had been completed. To meet the ever increasing demand for power, a station was installed at the existing North Twin Dam utilizing 28 ft. of head with a full pond. At Dolby, a new fixed blade propeller type turbine was installed to improve efficiency at this plant and the Grinder Room at Dolby was discontinued. Generators were installed on the grinder turbines and Dolby thus became exclusively a hydro electric station. Larger four foot grinders with helper motors were installed on the water wheels at East Millinocket to make up for the loss of grinders at Dolby.

Toward the end of the '30's still more power was needed. A 40 ft. drop in the river at Mattaceunk Rips was next developed and put in service in 1939. This is the most southerly station located near Mattawamkeag, Maine, approximately twelve miles below Millinocket. The station was planned for four units but only two were installed initially. The third unit was added in 1940 and the fourth in 1941. Recently the name of Mattaceunk Hydro Electric Station was changed to the Weldon Station in honor of Roy V. Weldon, a former Vice-President of the Company who was most instrumental in getting it built.

During World War II all modernization or expansion was, of course, curtailed; however, following the war, paper machines were modernized or replaced and production again increased. By 1950 it was evident that more power would again be needed. From the crest of Ripogenus Dam to the crest of North Twin Dam, the West Branch drops approximately 450 ft., most of this being at or just below Ripogenus Dam. In the early 1950's approximately 185 ft. of this head was developed with the construction of McKay Station (see Fig. 2).

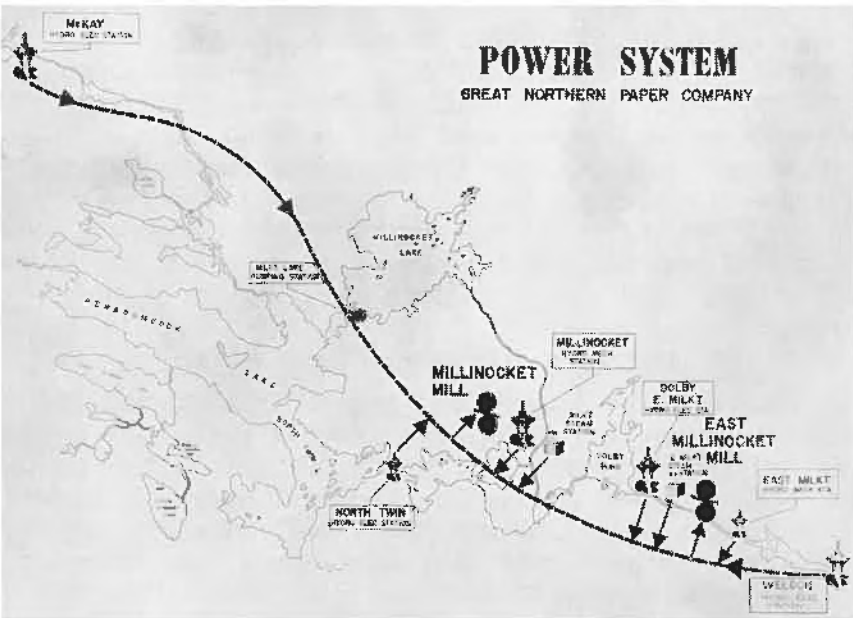

FIG. 2.—SCHEMATIC DIAGRAM OF GREAT NORTHERN PAPER COMPANY POWER SYSTEM.

During the '50's plans were made to make a major increase in production capacity. Two new machines were installed at East Millinocket, Nos. 5 & 6. These were two of the largest and fastest paper machines ever built, with a width of over 20 ft. and paper-making speeds up to 2500 ft. per minute. With installation of these machines, the necessity of a major increase in the size of the Steam Plant brought about the decision to go to high pressure steam boilers with reducing steam turbines to help carry the power load. In this way, cheap by-product steam power would be available to supplement the hydro power and, in addition, condensing steam power could be used to back up the hydro system during periods of drought.

Two steam stations were built, one at East Millinocket and one at Millinocket, each having two steam turbines rated at about 20,000 horsepower each. One turbine at each mill is strictly a reducing turbine or back pressure turbine and the other is a condensing turbine with extraction.

To complete our brief history of the power system, during the late 1950's supervisory control equipment was installed at Weldon Station and at North Twin with the control point being the Control Room of the Millinocket Steam Station. Also, protective equipment was installed at McKay Station and operation converted to completely automatic control. The Generator Room at Millinocket was put on remote control from the Boiler House as far as changing load on the two units or adjusting power factor is concerned. Thus, aside from the two Steam Stations, Dolby is the only manned power station at the present time. But enough of history, let us look further at the various power stations and consider some of their more interesting features.

Ripogenus Dam and McKay Station

Ripogenus Dam, almost 700 ft. long with a maximum height of about 73 ft. above the river bed, controls a lake with a usable storage of 30 bcf. It is a gravity concrete structure with stop log flash boards, two log sluicing gates and four deep gates, which were used originally for the release of water. Since McKay Power Station was built, the deep gates are seldom used. Most of the water now passes through the 16 ft. diameter tunnel 4,100 ft. to the power station at the foot of Ripogenus Gorge. McKay Power Station was built for three 16,590 HP Francis Turbines, but to date only two of these have been installed; however, the penstock and draft tube are in place for the third unit, which will be installed when power demand justifies. To the best of the author's knowledge, operation of this station is unique in that loading is controlled by adjusting system frequency. The station is unattended, except for twice-daily inspections by the gate tender at Ripogenus Dam and checking two or three times a week by a traveling crew which takes care of routine maintenance and lubrication. Supervisory devices are installed to protect the units in the event of failure and, in the event of trouble, a radio signal is transmitted to the load dispatcher at Millinocket. The two governors are connected by a load leveling device and they are both set to a very small droop, approximately 0.25%. Thus, by small changes in system frequency, this station can be loaded or unloaded at will.

North Twin Station

North Twin Dam is a gravity concrete structure about 350 ft. long with 620 ft. of earth wings with concrete core walls (see Fig. 3).

Fig. 3.—North Twin Dam and Station.

There are 14 deep gates which were used to release water before the power house was built and are still used in handling flood flows. There are two log sluices and a fishway. The power house contains three propeller type units, two of which are fixed blade and the third is a Kaplan. The fixed blade units are rated 3,050 HP and the Kaplan has a rating of 3,250 HP. With this combination of units high efficiency can be maintained at any desired flow rate. The Twin Lakes behind North Twin Dam contain approximately 15 bcf of usable storage. Since this is one of the seasonal storages, the head on the Station varies considerably over the year. In fact, there have been times when the water in North Twin was so low that no power could be generated.

The units at North Twin are controlled by a supervisory control system from the Millinocket Boiler House Control Room. Loads can be changed by adjusting the speed setting, reactive power can be controlled by adjusting the field and readings can be taken on gate setting, load, reactive load, and voltage on all units. Since this station is but five miles from Millinocket, remote starting and synchronizing were not provided. If one unit is not needed, however, it can be placed at zero gate reducing the flow to nothing but leakage. Only a small amount of power is drawn from the system to keep it turning and instantly available.

Millinocket Mill

The power development at the Millinocket Mill is the largest and also the most complex installation (see Fig. 4). Stone Dam, 1,086 ft. long with a maximum height of 24 ft., diverts water through a gate

Fig. 4.—Millinocket Mill.

section and canal into Ferguson Pond, a shallow artificial body of water contained by several earth dykes. From Ferguson Pond the water flows through a short canal to the penstock head gates. Six penstocks, 10 ft. in diameter, conduct the flow approximately 1,000 ft. in distance and 110 ft. down to the Grinder Room where each feeds a 5,250 HP double discharge horizontal Francis turbine, directly connected to five four-foot Great Northern grinders. Since the turbines do not have sufficient power to drive these grinders, each line also contains a large motor. Four of these are 4,000 HP and the other two are 6,000 HP. The seventh penstock is 11 ft. in diameter and feeds two 5,500 HP double discharge Francis turbines, each connected to a 5,000 KVA generator. The tailraces discharge into Millinocket Stream approximately 1½ miles above its confluence with the West Branch. Quakish Lake is relatively small having an area of 1,400 acres, but the small pondage available here is useful in handling load fluctuations.

Dolby

Dolby Dam has a total length of 1,390 ft. Of this, 821 ft. are of gravity concrete and contain a spillway section, gate section and the power house. The remaining length is concrete core wall earth construction. The power house contains eight units. The last of these to be installed is a vertical propeller type unit rated 3,900 HP at 49 ft. head. Units 1-7 are set in concrete flumes forming part of dam. Each of these has twin runners with separate draft tubes for each. One draft tube is formed in the concrete work of the dam and the other is made of plate steel. Units 1, 2 and 3 are 1,800 HP; unit 4 is 2,400 HP. These first four units are replacements to the original equipment installed when the power house was built and are more or less conventional wicket gate type Francis turbines. Units 5, 6 and 7 are of particular interest. They are ancient Holyoke runners put in in 1907 when the power station was built. They have drum gates and mechanical Holyoke governors, and are believed to be 2,100 HP each. These veterans are normally not operated except when another unit is down for repairs or during the Spring flood when excess water is available. While their efficiency is not up to a modern unit when they are operated at 100 gate, they still turn out a good block of power.

East Millinocket

East Millinocket is our lowest head development with a dam 423 ft. long and 21 ft. above riverbed. There are six units set in open flume settings, each unit consisting of two double wheels on a horizontal shaft, discharging into two plate steel draft tubes. Each of the six units is rated 1,640 HP at 240 rpm and is direct-connected to two four-foot Great Northern grinders along with a helper motor. There is no hydro electric development at East Millinocket.

WELDON STATION

The dam at Weldon is a concrete gravity structure 951 ft. long with a height of about 40 ft. above the riverbed (see Fig. 5). The spillway section is 658 ft. long. There is a roller gate 90 ft. wide

FIG. 5.—WELDON DAM AND STATION.

with a discharge capability of about 22,000 second-feet and the power house with four propeller units in it. Two of these are fixed blade and the other two are Kaplans. All are rated 6,000 HP at 40 ft. head. This station has the greatest discharge capability of any of the power stations. It is located below the confluence of the East and West Branches and, therefore, gets the run-off from a considerably larger area than any of the others. Also, the East Branch is nowhere near as well controlled as the West Branch with the result that flood flows are considerably heavier here than at the stations above on the West Branch.

Weldon Station is on complete remote supervisory control. All operations of the station are controlled from the Mill No. 1 Boiler House. Units can be started and synchronized and stopped, loads can be changed and field strength can be adjusted. Also, the roller gate can be opened and closed as necessary. In addition, reading of load, gate setting, reactive power, bus voltage, head water, tail water readings, and the percent opening of the roll gate are all obtained remotely at Millinocket. An intrusion alarm is also provided in the event unauthorized entry to the station is attempted. Routine checking and maintenance of the station is performed by the traveling crew which visits it several times a week.

Operation

In operating the power system, an attempt is made to achieve the maximum economic benefit to the company. Toward this end, a load dispatcher is on duty 24 hours a day to supervise the overall operation. Beyond his routine duties of keeping records of flows, loads, ponds' elevations and so on, he has the responsibility of seeing that water is used as efficiently as possible. Toward this end, he distributes the load in such a way as to balance the flows in the river to keep all ponds below North Twin as near full as possible without wasting water. He is also responsible for seeing that stations operate at efficient gate settings, in so far as this is possible.

At present we have three sources of power available, hydro power at no fuel cost, reducing steam power at very low cost and condensing steam power at approximately four times the cost of condensing power. In determining how the load is to be split between these three sources, two things must be considered: first, reliability of operation, that is being sure that enough power will be available to handle the load under any eventuality; and second, best overall economy. The first condition is taken care of by operating our 57 bcf of storage according to a rule curve. When storage is above the rule curve, river flows are maintained at as high a level as can be economically used at all stations, usually around 3400 cfs. When storage drops below the rule curve, however, the average flow must be cut back to 2,400 cfs to assure not running out of water more often than once every 100 years. This flow of 2400 cfs is about the minimum that will maintain full production.

During the early Spring, snow surveys are conducted and estimates made of the expected spring runoff. The results of these snow surveys will often alter our operation from that which would result from going strictly according to the rule curve. For example, early last Spring our ponds were considerably above normal; however, due to the almost complete lack of snow during the winter, we knew a dry season was ahead. Therefore, we cut back on our flows in order to carry more water over into the following year. On the other hand, at other times the ponds may be very low, but, with an exceptionally heavy snow cover, it might be desirable to go considerably below the rule curve during the winter in order to be able to hold all the spring runoff in the reservoirs.

Since the entire output of the Power System goes to the paper mills, we do not have a utility type load curve. On the average, our load is very steady, depending only on the level of production; however, the moment to moment variations can be quite severe, as can be seen by the fact that in the East Millinocket New Grinder Room, there are six 7,000 HP synchronous motors which are started directly across the line. Each of these represents quite a block of power on a system with a total capability of only some 200,000 HP.

To further complicate the problem, two of the steam turbines are often operated on back pressure governing in order to get the maximum amount of low cost reducing power. Since the output of these turbines then depends on the steam demand, big changes in this steam demand will cause sizeable changes in the generating capability. Swings of 4,000 to 5,000 kilowatts are not unusual from this source. In spite of these problems, we try to maintain frequency stability within plus or minus 2/10 of a cycle.

To sum up, operation is a problem of juggling hydro, reducing and condensing steam power to achieve maximum economy consistent with the desired reliability of operation.

Hydraulic Engineering

There are many problems of unusual interest to a hydraulic engineer at Great Northern Paper Company. As well as the routine operating problems of how best to utilize available flow and the constant problem of trying to out-guess the weather, there are many possibilities for improving efficiency of operation and maintenence which require engineering investigations of considerable depth to solve. As an example, we are contemplating a computer program for guiding the load dispatcher. At present, this is only in the brainstorming stage, but we envision a program into which would be fed all available weather information, both past records and future forecasts, as well as pertinent stream flow data, load predictions, etc. Out of this program would come the best mode of operation as to river flows and so on for the next interval of time.

Another question which arises is the economic advantage of maintenance to increase efficiency. In order to get better answers to this question, we are developing apparatus and planning to run complete efficiency tests on all our water wheels. This will involve measuring the discharge as well as power output at various gate settings on each of our 31 major water wheels. As a result of these tests, we will be able to tell where best to spend our maintenance dollars to achieve the greatest return.

As production increases and mill power demand goes up, the question arises where best to get the required additional power. We still have a number of good hydro electric sites on the West Branch which could be developed. We could get a considerable increase in power by replacing some old inefficient water wheels with more modern units, or steam or atomic power may turn out to be the best solution. As a hydraulic engineer, I am prejudiced, but I certainly hope that hydro power turns out to be the answer. Designing and constructing a major hydro electric development of, say, 40,000 kilowatts would be a very challenging and satisfying job.

We have in the head waters of the West Branch above Ripogenus Dam a number of small ponds varying from about 4 bcf of storage down to less than ½ bcf. Three of these have concrete dams which were built during the 1930's, while most of the others are timber crib gate sections with earth wing walls (see Fig. 6). Most of these were originally built as driving dams to move wood down the river to the mills. As the older dams fall into disrepair, the question arises whether it is worthwhile to rebuild or maintain them. Our experience with concrete, particularly in fairly light sections, has not been good in this

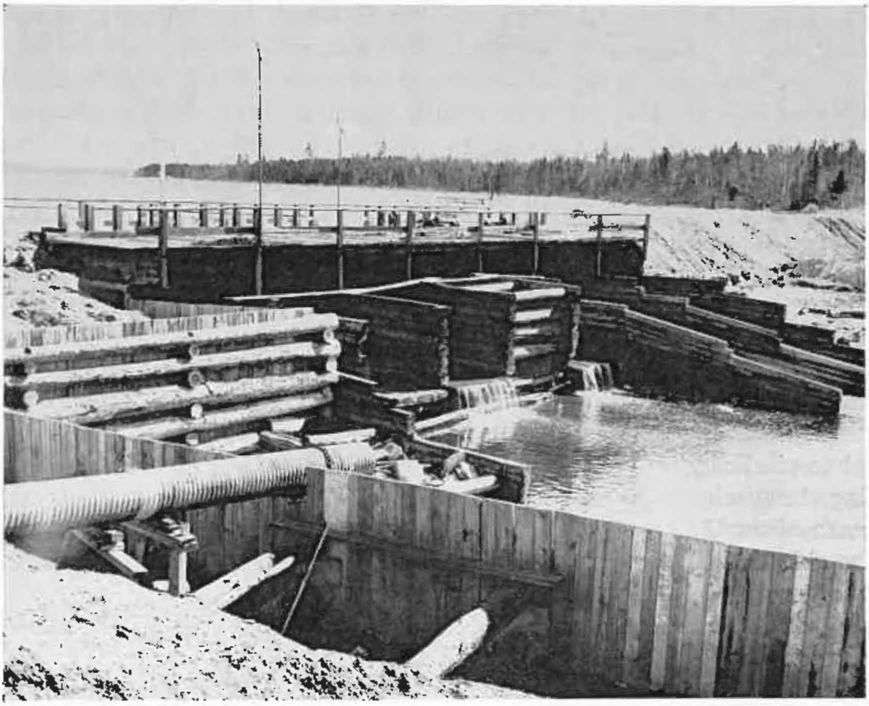

FIG. 6.—BIG BOG DAM.

severe climate. On the other hand, untreated timber has a life of about 12 to 15 years where it is not kept constantly submerged. Thus, we have here another interesting challenging problem.

Over 150,000 cords of pulpwood are driven down the river from the timberlands to the company's mills at Millinocket and East Millinocket. Passing of this wood by the dams in log sluices involves the use of considerable quantities of water. Here is another opportunity for hydraulic engineering to achieve economies in operation.

Even outside the limits of the power system and the normal civil engineering phases of hydraulics, there are many interesting fluid mechanics problems in the paper industry. Thus, there appears to be no end to challenge and opportunity. In fact, the only limit appears to be one's initiative and imagination.

MAJOR ADDITIONS AND REINFORCEMENTS REQUIRED TO MEET FUTURE DEMANDS OF PROVIDENCE WATER WORKS

BY PHILIP J. HOLTON, JR.*

(Presented at a meeting of the Hydraulics Section, B.S.C.E., on May 5, 1965.)

The Water Works of the City of Providence, although municipally owned and controlled, operates in the capacity of a metropolitan district. In addition to Providence, it serves the City of Cranston along with the Towns of North Providence and Johnston from a distribution system owned and maintained by Providence. The City of Warwick, the East Smithfield Water District, the Town of Smithfield and the Kent County Water Authority purchase water from the City but control and operate their own distribution system. In 1962, enabling legislation was passed that permits the City of East Providence to receive Providence water and this community should be connected to our system by 1967. When this tie-in takes place, the water district will be serving, in whole or in part, four cities and five towns representing over 45 percent of the population of the State. Under the 1915 Water Act, the Towns of Scituate, Foster and Glocester are entitled to receive water from our system, but at the present time there is no public water supply in any of these communities. The area of all cities and towns that may obtain water from Providence totals 390.80 square miles or around 36 percent of the land area of Rhode Island.

Providence obtains its water from a surface supply located on the north branch of the Pawtuxet River. Water is impounded in the main Scituate Reservoir and five smaller reservoirs from a drainage basin that contains 92.8 square miles. This is about 9 percent of the land area of the State of Rhode Island and about five times the area of the City of Providence. The City owns in fee 23.93 square miles or about 25 percent of the entire catchment area. The total capacity of all reservoirs at their respective spillway elevations is 41,268 million gallons. The main Scituate Reservoir has a capacity of 37,011 million

* Chief Engineer, Providence Water Works, Providence, Rhode Island.

gallons and is capable of impounding 400 million gallons for every square mile of drainage area.

The annual rainfall on the watershed based on the 49-year average is 48.25 inches with a maximum of 66.28 inches for the 1958 fiscal year and a minimum of 33.43 inches in 1957. The yearly runoff, or water actually collected in the reservoir, based on the same period, is 24.86 inches with a maximum of 35.92 inches in 1956 and a minimum of only 12.02 inches for the year 1930. Every inch of runoff is equivalent to 1,612.75 million gallons. The average daily yield over the last 49 years is 109.77 million gallons. However, we cannot plan that this quantity of water will always be available. We are limited to what is known as the "Estimated Safe Yield". This is the maximum dependable draft which can be made continually upon a source of water supply during a period of extended drought when the greatest deficiency in runoff is likely to occur. The "Estimated Safe Yield" of the Scituate supply is 84.02 million gallons daily but under State statute, we are compelled to release 12 million gallons daily to mills below Gainer Dam leaving 72 million gallons daily for water supply purposes.

According to the department's long range studies, our average daily consumption plus plant usage, compensatory water to the mills and the requirements of East Providence will reach the limits of the "Estimated Safe Yield" by the year 1983. This means that Providence must have available a supplementary source of water ready for operation as of that date. Unfortunately, Rhode Island cannot lay claim to an abundance of drainage basins. The entire State contains only eight principal catchment areas with three holding favorable potential for the development of surface water supply reservoirs.

The most desirable project consists of Big and Wood Rivers. The first is located on the south branch of the Pawtuxet and the other, which would be a diversionary reservoir, is on the Pawcatuck River basin. The watershed for this combination would have a drainage area of 65.66 square miles, a storage capacity of 33,265 million gallons, equivalent to 507 million gallons of storage per square mile, and an "Estimated Safe Yield" of 61.3 million gallons daily. This could be expanded at some future date to include the Flat River also located on the south branch of the Pawtuxet. The addition of Flat River would increase the drainage area to 94.46 square miles, storage capacity would be 35,140 million gallons, equal to 372 million gallons per square mile, and the "Estimated Safe Yield" woud be 83 million gallons daily. Flat River, like Wood River, would be a diversionary reservoir.

In 1962, a referendum was submitted to the voters of Rhode Island covering the acquisition of land for the eventual development of Big and Wood River Reservoirs. It was rejected by the voters. In 1964, a similar referendum, with some modifications in the Act, was presented for the second time and was approved by a small majority. It carried an appropriation of $5,000,000 for land acquisition only. If the State of Rhode Island intends to assume a leading role in the coordination and development of our surface water resources, they should proceed within the next few years with the actual development of both reservoir sites. Past experience proves that it usually requires twelve to fifteen years to complete major projects of this nature. Such a program would enable the City of Providence and other communities to purchase water from the State on a wholesale basis. If the State fails to recognize its responsibility, the City of Providence will have no choice but to proceed on its own and obtain legislative authority to develop Big and Wood River Reservoirs.

Since the start of the Scituate supply in 1926, water has been conveyed by gravity from the intakes at the dam through twin 60-inch aqueducts as far as the junction chamber where they converge into a single 94-inch aqueduct to the Water Purification Works. Up to the present time, no pumping whatsoever has been required between the Reservoir and the Plant. In discussing the capacity of this aqueduct, we cannot consider the average daily requirements. It must be capable of delivering sufficient quantities to meet the maximum day's demand. Our records show that the maximum day is about 194 percent of the average, so for general design purposes we use 200 percent. In other words, where we show the reservoir capable of producing an average of 72 million gallons daily for water supply purposes, we must be able to deliver through the aqueduct twice this volume to meet the maximum daily demand, or 144 million gallons.

It is anticipated that the City of East Providence will connect into our system around the latter part of 1967 and at that time our estimated maximum daily demand will reach 105 million gallons. In order to convey this volume of water to the Plant, Scituate Reservoir cannot drop below elevation 272.2 or about 12 feet lower than the spillway elevation of 284.0. If drought conditions should occur in 1967 similar to what took place from April, 1910 to October, 1911, the actual level in Scituate Reservoir would drop to elevation 266.6, or 5.6 feet below the level required for gravity flow without pumping, based on an average daily water supply requirement of 52.5 million gallons plus 12 million gallons to the mills.

Within the next few years, the department will be compelled to construct a pumping station at the base of Gainer Dam capable of delivering about 150 million gallons daily from the reservoir to the Plant. An additional source of power, in addition to that supplied by the local power company, will be required as the output from our Hydro Station would be negligible under these low water conditions. It is our intention to finance this capital improvement from our Water Depreciation and Extension Fund thereby eliminating the necessity of a bond issue.

The Water Purification Works is all electric and is operated from a Power Control Center and a Central Control Board located on the first floor of the Central Operations and Control Building. Two sources of power are available, one from the Hydro Electric Station and the other from the local power company. As the Hydro Station will be inoperative during periods of low runoff, we will plan to obtain power from the supplementary system at the proposed new pumping station.

The Water Purification Works must have sufficient filter capacity to treat the draft of the maximum day from Scituate Reservoir which will be 144 million gallons. At present, we have 14 rapid sand filters, each capable of filtering 7.5 million gallons daily or a total of 105 million gallons daily. We have space to add four additional units of 7.5 million gallons each which will bring the plant output to 135 million gallons per day. In order to reach the required capacity of 144 million gallons daily with 18 filters, we will be required to increase the rate of filtration from 3 gallons per square foot per minute to 3.2 gallons. This will not affect the quality of the effluent whatsoever but will reduce the length of filter runs.

When East Providence comes into the system, the maximum day will equal our existing plant capacity in the year 1967. Plans are now being prepared by the department's Engineering Division to increase the number of filters to eighteen. The cost of this new construction together with other plant improvements is estimated to be $2,500,000. This project is covered by a bond issue approved by the voters of Providence at the last general election. It is now pending ratification of the Rhode Island General Assembly.

Just as the aqueduct must have the capacity to deliver the maximum draft of 144 million gallons daily from the reservoir to the plant and the plant have sufficient filter capacity to treat that volume of water, we must have a transmission system that is capable of delivering this quantity of water, the maximum daily demand, from the filter plant into the system. At present, the tunnel and aqueduct that transports the water from the plant to the siphon chamber in the City of Cranston is able to deliver only 100 million gallons daily. On June 27, 1963, we etablished a new record for the maximum day of 87.215 million gallons and if East Providence was being supplied, it would total 95 million gallons. Our maximum demand will reach the limits of this single tunnel and aqueduct within the next few years. The present tunnel and aqueduct was designed by the department's engineers in 1916 and at that time, our average daily consumption was only 17.75 million gallons and the maximum day 22.92 million gallons. They displayed excellent judgment in designing this facility for a capacity of 100 million gallons.

Our studies show that by the year 2001 the maximum daily demand will reach 173 million gallons. Scituate Reservoir can supply 144 million gallons daily and the remainder will have to come from the Big and Wood Rivers development. It is anticipated that the first section will be a surface aqueduct 6′-6″ in diameter running from the plant in Scituate to the Riverpoint section of West Warwick, a distance of 22,750 feet. This will be capable of conveying an additional 44 million gallons daily from the present plant. At this point, it will eventually converge with the reinforcement from Big and Wood Rivers, will be 8′-6″ in diameter, and will run for an approximate distance of 27,300 ft. to the Budlong Road section of Cranston. Instead of limiting the capacity to 73 million gallons daily, it will be designed for 100 million gallons daily, thereby providing a total of 200 million gallons daily from both the existing and proposed construction. The cost of the new aqueduct is estimated to be $10,500,000 and the bond issue was approved by the voters at the last general election. Ratification is expected at this session of the General Assembly along with the approval of a land condemnation act.

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THE ANALYSIS AND DESIGN OF ANTENNA TOWER FOUNDATIONS

BY H. M. HORN

("The Analysis and Design of Antenna Tower Foundations" appeared in the July, 1964 Issue of the *Journal*.)

DISCUSSION BY A. SRIDHARAN*

The author has to be complimented for his very interesting and lucidly presented paper on the Design of Antenna Tower Foundations. Use of different design methods for such foundations have been reviewed with particular reference to their limitations. The author has rightly pointed out the uncertainties involved in the evaluation of soil properties such as E, G and ν.

With regard to the calculation of natural frequency, further clarification on the following points are sought. Among the various parameters affecting the natural frequency of a foundation soil system, the depth of embedment of the foundation forms one of the factors. Due to various considerations, foundations are laid at a particular depth below ground level. In Example 2 of the paper the depth of foundation is reported to be 10 ft. and theories used to calculate the natural frequency are those developed for foundations resting on the ground level. Thus the effect of depth on the natural frequency has not been taken into account.

The writer's laboratory experiments on a 12 in. $\times$ 12 in. foundation footing show that the resonant frequency reduces as the foundation depth increases. The experiments were conducted on dry sand contained in a masonry tank ($4\frac{1}{2}$ ft. $\times$ $4\frac{1}{2}$ ft. $\times$ $4\frac{1}{2}$ ft.) using a Degebo type vibrator. The vibrator was designed to produce a single harmonic dynamic force in the vertical direction. The experiments were conducted keeping the footing at the surface, at 6 in., 10 in. and 16 in. depth. Fig. 1 shows the relationship between the resonant frequency and the depth of embedment for two static intensities and for two different unbalanced masses.

Even for rocking and torsional mode (similar to that considered

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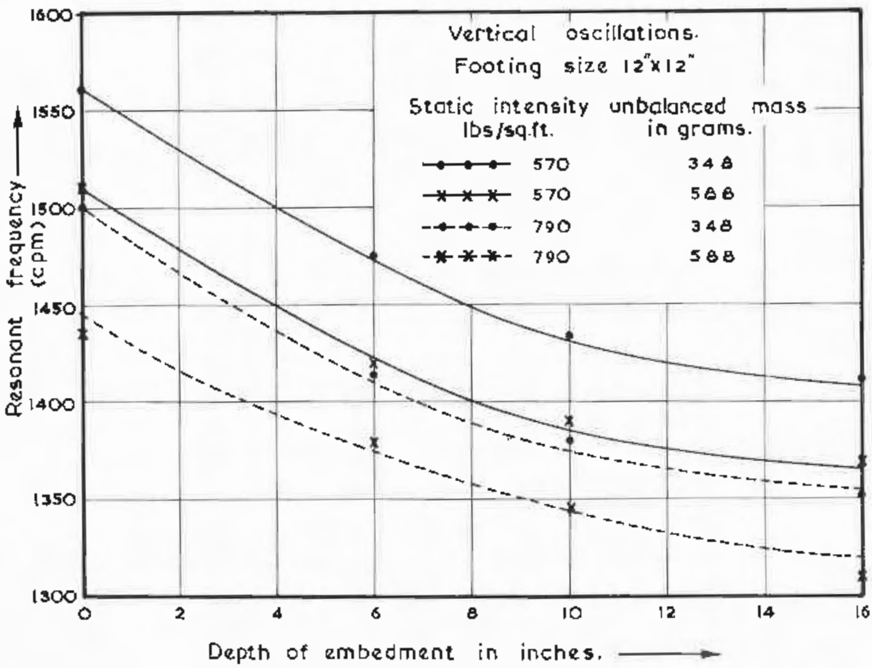

FIG. 1.—RELATIONSHIP BETWEEN RESONANT FREQUENCY AND DEPTH OF EMBEDMENT.

in the paper) the writer is of the opinion that the natural frequency will become reduced as the depth of embedment increases. Crockett and Hammond (1) are also of the opinion that increase in depth of foundation should reduce the natural frequency of the system.

Secondly, the analysis made use of for calculating the natural frequency is for a rigid base. Since the relative stiffness factor affects the natural frequency considerably, the analysis carried out purely on rigid base approximation needs further examination. This, when viewed from the findings of Sung (2) that both the parabolic loading and uniform loading conditions give a lesser natural frequency when compared to that of a rigid base condition, makes one wonder whether the design approaches the limiting condition.

An alternative design has also been reported in the paper using steel for the super structure in order that a higher natural frequency will be obtained for the system. The writer has calculated the value of "b" for the same reinforced concrete super structure changing only the diameter of the foundation mat. The value of "b" has been reduced to 1.015 from 1.73 for an increase in the diameter from 52 ft. to 60 ft. Increasing the size of the mat foundation may thus prove more effective in increasing the natural frequency.

The writer is of the opinion that $\frac{\bar{f}_r\ (\nu = 0.25)}{\bar{f}_r\ (\nu = 0)}$ is related to $\sqrt{\frac{1 + 0.25}{1 + 0}}$ rather than $\sqrt{\frac{1 - 0}{1 - 0.25}}$ as reported in the paper. This also needs clarification from the author of the paper.

REFERENCES

1. CROCKETT, J. H. A., AND R. E. R. HAMMOND (1949), "The Dynamic Principles of Machine Foundations and Ground," Proc. IME (Lond.), Vol. 160, pp. 512-531.
2. SUNG, T. Y. (1963), "Vibrations in Semi-infinite Solids Due to Periodic Surface Loading," Symp. on Dynamic Testing of Soils, ASTM, Sp. Tech. Pub. No. 156, pp. 35-63.

CLOSURE TO DISCUSSION BY A. SRIDHARAN OF "DESIGN OF ANTENNA TOWER FOUNDATIONS"

By Harry M. Horn,* *Member*

Laboratory model tests performed by Mr. Sridharan indicate that as the depth of the foundation increases, the natural frequency of the foundation-soil system decreases. Data obtained from tests performed on a 12 in. by 12 in. footing resting on dry sand and vibrating in a vertical mode are presented by the writer which clearly show that natural frequency is a function of foundation embedment.

The ratio of depth to the diameter of the mat in Example 2 is 0.192, which corresponds to a depth of 2.3 inches in Mr. Sridharan's tests, for which he found that the natural frequency was about 2 per cent less than that of an equivalent surface footing. The author believes that such a small difference would not merit consideration in an actual design problem. In addition, the soil conditions in Example 2 are quite different from those in the model tests. The mat in that example rests on the surface of a deposit of very stiff clay which is overlain by very loose sand. The author believes that in such a situation, the influence of foundation embedment would be considerably less than if all of the soil involved were sand having a uniform relative density. Relationships such as those presented by Mr. Sridharan should be considered when designing a deeply embedded mat or footing. Embedment is particularly significant when the top of the foundation is below the ground surface, in which case the overlying soil acts as part of the vibrating foundation.

Mr. Sridharan questions the reliability of a design based on relationships derived on the assumption that a rigid base-elastic foundation contact exists between the footing or mat and the underlying soil. The author realizes that such an assumption leads to computed natural frequencies which are greater than those based on theory in which either a parabolic or a linear distribution of contact stress is assumed for the static case. However, when foundations rest on very stiff cohesive soils, such as in Example 2, the author believes

* Assistant Professor of Civil Engineering, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts.

that the rigid base assumption is reasonable. He agrees that this point is open to question, and feels that it can only be resolved by studying performances of large-scale models and prototype structures.

Mr. Sridharan is correct in his contention that the natural frequency can be increased by increasing the diameter of the foundation. The author believes, however, that this is often an inefficient means of achieving such an increase. For example, if the diameter of the mat in Example 2 were increased from 52 ft. to 60 ft., as the writer suggests, the value of "b" would be 1.015 and the frequency factor, $\bar{a}_o \approx 1.37$, for $\nu = 0$. The computed natural frequency, $\bar{f}_r$, would be:

$$\bar{f}_r = \frac{1.37}{1.10} \times \frac{26}{30} \times 5.04 = 5.44 \text{ cps} < 5.82 \text{ cps}$$

In order to raise the computed natural frequency of the system to 5.82 cps, a mat with a diameter of approximately 70 ft. would be needed. Such a mat would have a volume that was approximately 80 per cent greater than that needed for one of equal thickness and 52 ft. in diameter.

The relationship:

$$\frac{\bar{f}_r\ (\nu_1)}{\bar{f}_r\ (\nu_2)} = \sqrt{\frac{1 - \nu_2}{1 - \nu_1}}$$

used by the author to account for the influence of Poisson's ratio on natural frequency is based on theory for the natural frequency of an oscillator having a rigid base, vibrating in a rocking mode while resting on an elastic but weightless half space (Richart, 1960). If this oscillator had a circular base, the natural frequency of the system would be:

$$\bar{f}_r = \frac{1}{2\pi}\sqrt{\frac{d^3\ G}{3\ (1 - \nu)\ I_o}}$$

Therefore,

$$\bar{f}_r = F\ [\sqrt{1/(1 - \nu)}]$$

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

MAY 17, 1965:—A Joint Meeting of the Boston Society of Civil Engineers with the Transportation Section was held this evening in the Society Rooms, 47 Winter Street, Boston, Mass., and was called to order by President Leslie J. Hooper, at 7:10 P.M.

President Hooper stated that the Minutes of the previous meeting March 18, 1965, would be published in a forthcoming issue of the Journal, and that the reading of those Minutes would be waived unless there was objection.

President Hooper announced the deaths of the following members:—

Edward H. Cameron, elected a member May 19, 1920, who died November 28, 1964.

Charles A. McManus, elected a member June 16, 1943, who died February 20, 1965.

Allen E. Rucker, elected a member December 15, 1926, who died December 2, 1964.

The Secretary announced the names of applicants for membership in the Society and that the following had been elected to membership May 17, 1965:

Grade of Junior.—Thomas J. Quinn, Jr., George G. Hamparian.

President Hooper stated this was a Joint Meeting with the Transportation Section and turned the meeting over to Benedict J. Quirk, Chairman of that Section to conduct any necessary business at this time.

Chairman Quirk introduced the speaker of the evening, Frank P. Davidson, President, Technical Studies, Inc., New York, who gave a most interesting illustrated talk on "Channel Tunnel: The View from Dover Castle."

A question period followed the talk.

35 members and guests attended the meeting.

Meeting adjourned at 8:10 P.M.

CHARLES O. BAIRD, JR. *Secretary*

STRUCTURAL SECTION

APRIL 14, 1965:—A regular meeting of the Structural Section was held this date in the Society Rooms. The meeting was called to order by Robert L. Fuller, Acting Chairman, at 7:10 P.M.

The Acting Chairman introduced the speaker of the evening, Prof. Russell C. Jones of M.I.T., who spoke on "Structural Metals-Understanding through Materials Science."

The speaker began by defining Materials Science as an attempt to interpret and predict macroscopic behavior in terms of microscopic behavior. He showed how the mechanical properties of metals can be related to their composition and their microscopic structure.

Several examples were given of possible ways of improving metal properties through a rational interpretation of the operative microscopic mechanisms.

After an extensive question and answer period, the meeting was adjourned at 8:45 P.M.

Attendance was 19.

CHARLES C. LADD, *Clerk*

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BOSTON SOCIETY OF CIVIL ENGINEERS

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