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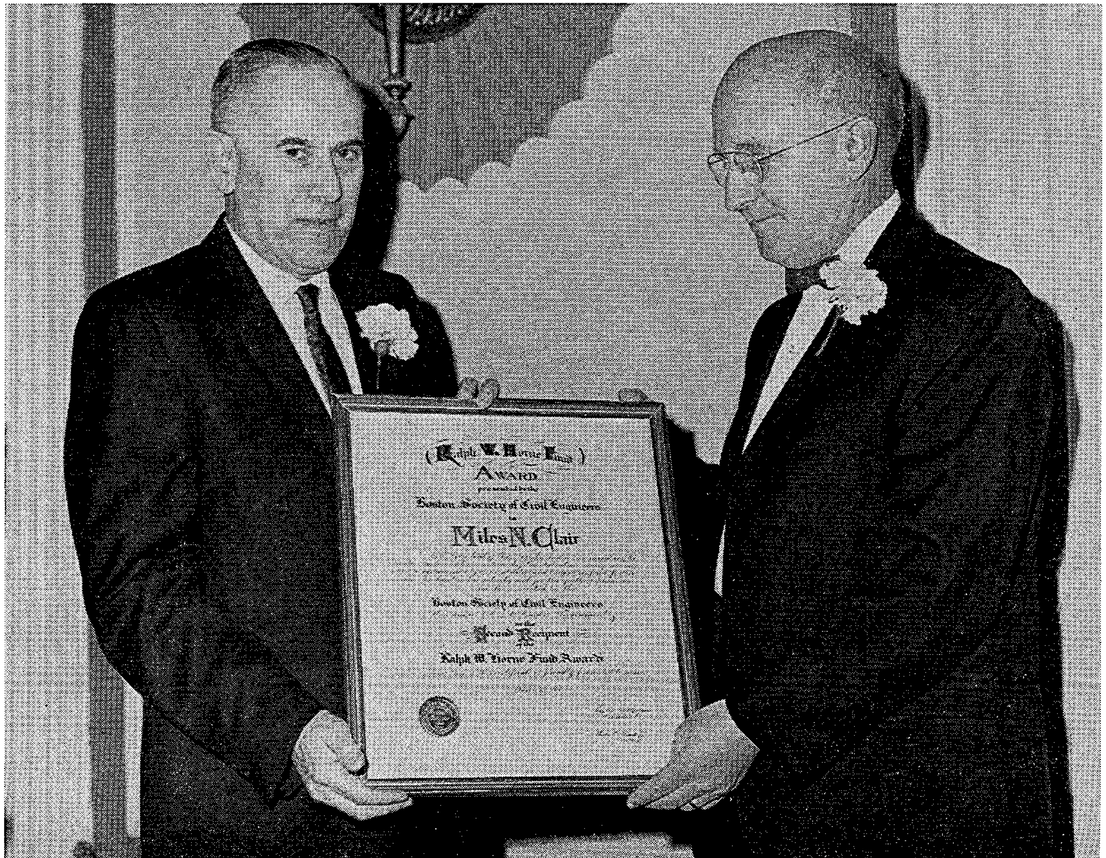

JOURNAL OF THE

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JET DIFFUSION AND CAVITATION

BY HUNTER ROUSE*

Presentation of this first John R. Freeman Memorial Lecture has meant a very great deal to the writer, for Dr. Freeman was a man whom he knew and deeply respected. An able experimenter in his own right, Freeman spoke with the voice of a prophet in behalf of experimental hydraulics, and he inspired and supported the study and work of many a young follower. All in all, his influence upon American hydraulics has probably been more far-reaching than that of any predecessor or successor [1].

Freeman himself would probably agree, if he could see things today, that not everything he began had turned out as he originally intended. Of the several dozen Freeman Scholars whom his engineering-society endowments sent to Europe and around the States, some are no longer alive, some were poor choices, and some went into other professions, but many have done their sponsor credit, and a few have even exceeded his expectations. (The writer can speak quite freely, for he was not a Freeman Scholar, but rather one of three M.I.T. students who held in succession an Institute fellowship which Freeman had arranged to be diverted temporarily to hydraulics.) The National Hydraulic Laboratory, founded within the Bureau of Standards at Freeman's instigation, has unfortunately just about reached its last days. For some reason there was little connection between these two undertakings: as far as the writer knows, only two of the forty-two Freeman Scholars have ever been on the Bureau's laboratory staff. Most interesting, indeed, is the fact that the man who made such a

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name as the laboratory acquired was not even an experimental hydraulician, but a mathematical physicist.

So far as Freeman's own investigations are concerned, his finest work—on the resistance of smooth pipes—was conducted years before that of Stanton and Nikuradse and was of fully comparable quality, yet was not published until so many years thereafter [2] that it played essentially no role in the advancement of the subject. For his work on fire streams, however, he received much acclaim, including the highest award of the American Society of Civil Engineers [3]. It is somewhat ironical that the principal factor involved in the poor performance of fire streams—the turbulence of the issuing water—was scarcely suggested in Freeman's papers. Probably the world of fire fighters and their equipment manufacturers is still not ripe for talk of turbulence, for though a paper of the last decade was written to clarify its role in jet diffusion [4], nozzles and monitors are still made much as they were at Freeman's time.

The tremendous impetus that he gave to American hydraulics may not have carried in the direction that he intended, but there is little doubt that he would have approved the healthy condition in which the subject presently finds itself. The writer had his mentor especially in mind, in fact, when he selected the topic of this paper, for it not only deals with jets and turbulence, but it carries their consideration through territory not then foreseen, and it finally brings the subject matter to useful application, just as Freeman believed that research as a whole should do.

A submerged jet differs from a free jet, on the one hand in the lack of gravitational influence, and on the other hand in the interaction between the jet and the surrounding fluid. In fact, from the latter point of view, the submerged jet can be regarded as the limit of the free jet as the density of the fluid stream approaches that of the fluid into which it is ejected. Except at low Reynolds numbers, the intense shear between the two regions results in the formation of turbulence throughout the shear zone, the effect of which is to entrain the surrounding fluid, diffuse the jet, and gradually dissipate its energy. The first mean-flow analysis of the phenomenon was published by Tollmien [5] just a year before the first Freeman Scholar reached Europe, and less than a decade ago Townsend [6] dealt at length with the jet in his general analysis of turbulent shear flow. A good digest of the many intervening papers is given by Hinze [7]. Essentially all of these studies, it should be remarked, were contributed by physicists and aeronautical engineers. While the behavior of a submerged jet surely does not vary with the observer, the point of view and the ultimate use of the observations definitely do. Not only is the writer a hydraulician, but work in certain aspects of jet diffusion has been taking place in his laboratories for the past two decades. He must hence be pardoned for emphasizing his own point of view and observations in the subsequent discussion.

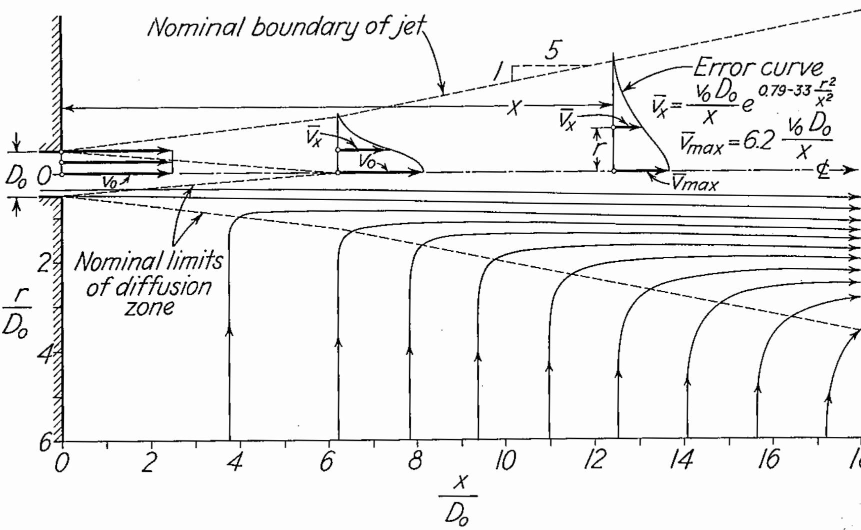

FIG. 1.—MEAN-VELOCITY CHARACTERISTICS OF JET DIFFUSION [8]

If it is assumed that the zone of jet diffusion is one of hydrostatic pressure distribution, then it follows for the boundary geometry shown in Fig. 1 [8] that the same momentum flux must occur past all successive sections, since there is no external force at hand to change it. If it is further assumed that the Reynolds number is so high that at all sections beyond the initial region of establishment a state of similarity exists for both the mean flow and the turbulence, it will follow that the jet must expand linearly and that the velocity along any line will vary inversely with distance from the nozzle [9]. Assumption of a velocity distribution like the error curve shown in the figure will then permit evaluation of the changing rates of volume flux and energy flux past successive sections—except for a single numerical factor, which must be determined empirically. Certain gross aspects of the turbulence can be approximated from the phenomenological relationships of Prandtl; indeed, the reverse process—assumption of the turbulence characteristics and evaluation of the corresponding velocity distribution—was the approach originally used by Tollmien. However, whereas the mean flow is relatively insensitive to the assumed distribu-

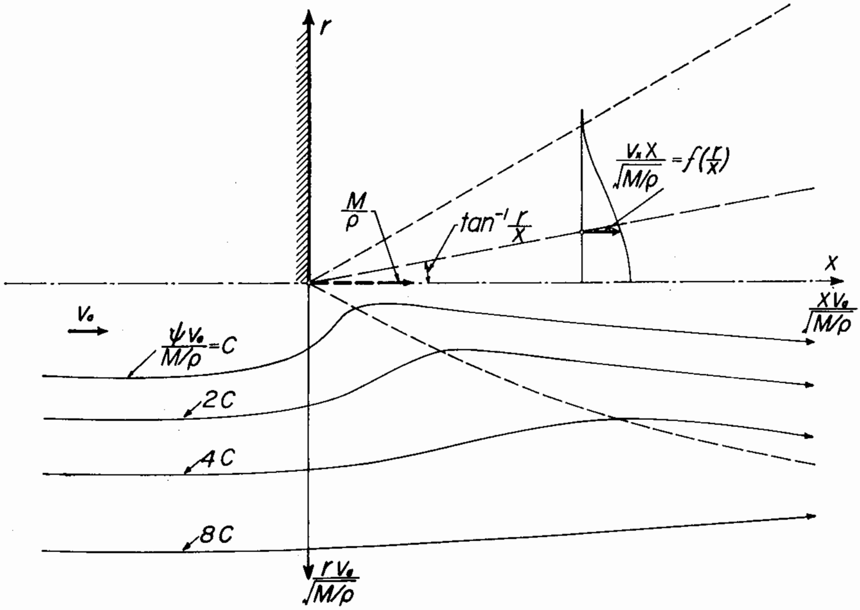

FIG. 2.—POINT SOURCE OF MOMENTUM FLUX: ABOVE, IN A NORMAL WALL; BELOW IN AN AMBIENT FLOW

tion of the turbulence, the reverse is not true, and, as will be illustrated later, information as to the actual turbulence characteristics has continued to depend upon direct measurement.

Consideration of the foregoing discussion will lead to the conclusion that at considerable distances from the efflux section it is neither the efflux velocity nor the outlet diameter which is significant, but the momentum flux $M = \rho v_0^2 \pi D_0^2/4$. In other words the quantity M/ρ (with the dimension $[L^4/T^2]$) becomes the sole independent parameter characterizing the jet as a whole, and the efflux section is reduced to a point source of momentum in the axial direction. As with any source flow, there is no reference length, and all cases are perforce dynamically similar and without linear scale. As indicated schematically in the upper half of Fig. 2, every section is similar to every other one in such a rendition, for mean-flow (the actual values of which can be obtained from those of Fig. 1) and turbulence characteristics alike. This intuitive dimensional method of approach can readily be extended to flow from a nozzle in an ambient stream. The normal wall containing the

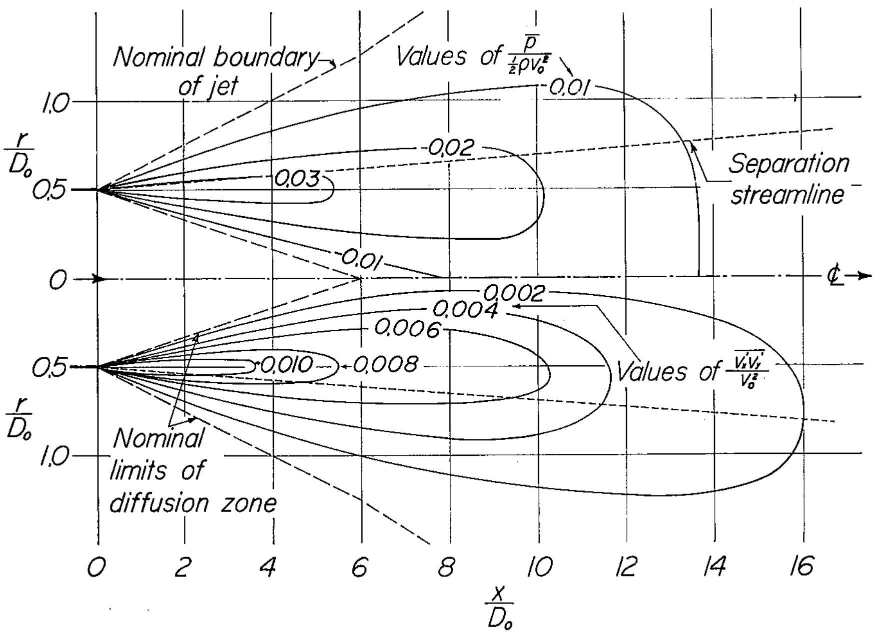

FIG. 3.—DISTRIBUTION OF PRESSURE (ABOVE) AND SHEAR (BELOW) IN A DIFFUSING JET (NOTE SCALE DISTORTION)

outlet must then be eliminated, of course, which necessarily changes the direction of the approaching fluid even for the case of entrainment without ambient flow. If it is now realized that the nondimensional coordinates must incorporate both the kinematic momentum flux M/ρ and the velocity of the ambient flow, they will be found to take the form $rv_a/\sqrt{M/\rho}$ and $xv_a/\sqrt{M/\rho}$. Further thought will show that large values of the radial coordinate will correspond to small rates of momentum flux (or large ambient velocities), and vice versa. The nondimensional pattern of streamlines must hence have a form much like that shown schematically in the lower half of Fig. 2, the streamlines becoming more and more nearly parallel to the axis with increasing relative radius; that is, a single picture will represent all possible conditions ranging from those of relatively high jet strength and weak ambient flow in an enlargement of the pattern (i.e., close to the axis) to those of relatively strong ambient flow and low jet strength in a reduction of the pattern (i.e., far from the axis). To the best knowledge of the writer, this manner of composite presentation of the entire range of conditions from pure jet flow to pure ambient flow is new; he leaves determination of the exact geometry of the pattern to an interested thesis student.

So far as the writer is presently concerned, it is the details of the zone of flow establishment rather than of the zone of established flow which are of special interest. This zone has the distinguishing feature of a central, essentially irrotational core into which the vorticity generated in the surrouding shear zone gradually diffuses. If the distinction between efflux from a cylindrical outlet in a normal boundary and efflux from a nozzle (the difference being restricted largely to the surrounding low-velocity flow) is ignored, the distribution of the mean velocity shown in Fig. 1 can be regarded as typical of either. Although the usual simplified analysis rests upon the assumption of hydrostatic pressure distribution, in actuality there are appreciable departures therefrom: to a minor degree because of the necessary radial change in the velocity of the entrained fluid, and to a greater degree because of the variation in the radial component of turbulence intensity. The two factors can be evaluated through the momentum equation for the radial direction [10]. As-yet-unpublished measurements made at Iowa by Thomas Carmody with a static-pressure probe yield the contours of piezometric head shown in the upper half of Fig. 3 [11]. The head is seen to be lowest in the zones of most-pronounced turbulence, as will immediately be delineated. Far more important than the distribution of pressure or normal stress accomplishing the diffusion of the jet fluid, however, is the distribution of shear or tangential stress. At sufficiently high Reynolds numbers, the viscous contribution to the shear can be neglected in comparison with the so-called Reynolds stresses of the turbulence. Contours of the Reynolds stress $-\rho\overline{v'_x v'_y}$ acting on coaxial cylindrical surfaces, from measurements at Iowa by Sedat Sami, are shown in the lower half of Fig. 3 [11]. In general the maximum intensities of shear would be found to occur at points of maximum mean-velocity gradient and turbulence intensity. Sami's hot-wire data for the several components of the intensity have been combined to yield the contours of $\sqrt{\overline{v'^2}} = \sqrt{\overline{v'_x{}^2} + \overline{v'_y{}^2} + \overline{v'_z{}^2}}$ shown at the top of Fig. 4. The geometrically similar contour plot shown at the bottom of the figure represents Sami's results for the fluctuating pressure obtained by means of a cylindrical piezoelectric crystal flush-mounted at the usual piezometer location on a ⅛-inch-diameter static-pressure

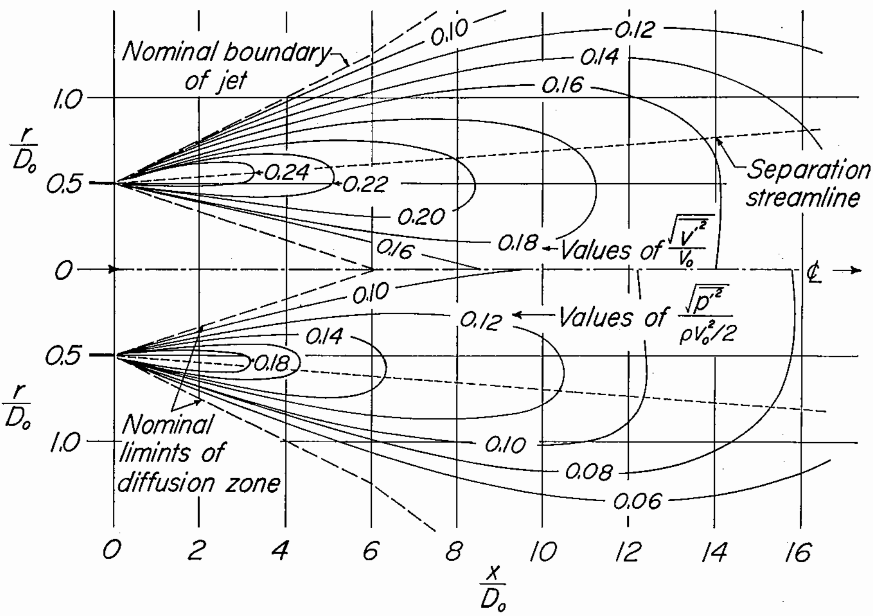

FIG. 4.—DISTRIBUTION OF VELOCITY FLUCTUATION (ABOVE) AND PRESSURE FLUCTUATION (BELOW)

probe. Careful comparison of the two contour families would show that at essentially all points of pronounced turbulence $\sqrt{\overline{p'^2}} = 3.6\,\rho\overline{v'^2}/2$.

Pressure-fluctuation measurements of this sort are relatively new, and the instrumentation involved has not yet been perfected to the extent of matching the hot-wire anemometer in either size or refinement. However, the nearly perfect correlation between the root-mean-square pressure fluctuation and the mean-square velocity fluctuation would lead one to hope that certain pressure characteristics which are not yet measurable may be represented by their velocity counterparts. Principal among these are their linear scales, i.e., the dimensions of the zones over which individual fluctuations extend. Since it is the eddy structure of the turbulence that is responsible for the two types of fluctuation, it is only reasonable that similar spectral distributions should be characteristic of both. In Fig. 5 are shown contour lines of two scales determined from temporal records of the longitudinal velocity fluctuation [11]: above, the size of the average eddies, determined from the correlation (i.e., degree of agreement) between values

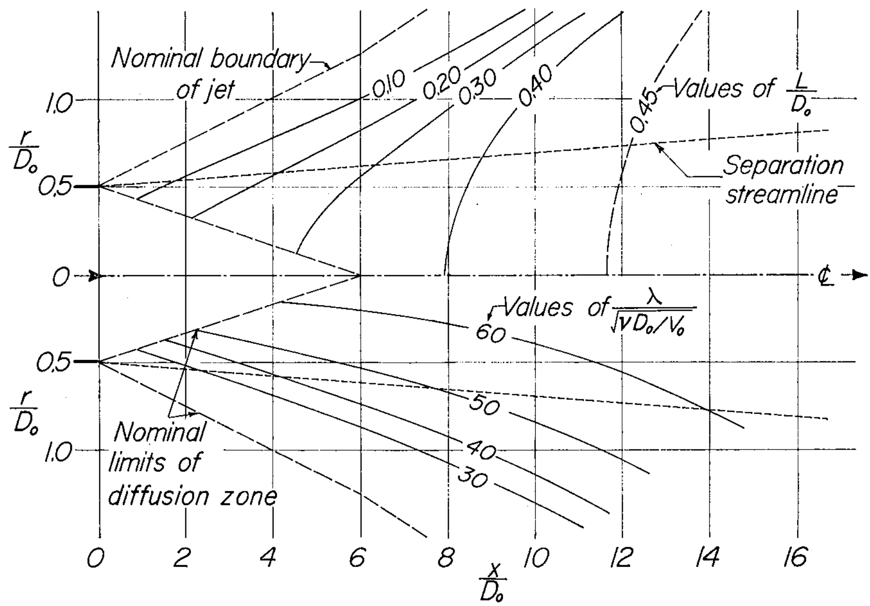

FIG. 5.—DISTRIBUTION OF MEAN EDDY SIZE (ABOVE) AND DISSIPATION LENGTH (BELOW)

at successive instants a variable time interval apart; and below, the average size of the smaller eddies, known as the dissipation length, determined from the time rate of change of the velocity.

If Freeman could have read the foregoing comments and examined the corresponding illustrations, he might well have lauded the Yankee ingenuity so obviously required by the painstaking and novel experiments that they represent, but he would most certainly have asked what engineering use they could possibly have. As it happens, two primary engineering applications of the information involve phenomena already known at Freeman's time, if not long before. One is the generation of noise. Each pressure fluctuation is a source of sound, whether it occurs in a diffusing jet or along the wall of a conduit, and considerable theoretical study has been given to the phenomenon [12]. However, the fluid compressibility is involved to a considerable degree, and the problem of noise generation is basically one of aerodynamics rather than hydraulics, though it must be granted that plumbing systems are among the most disturbing noise sources in modern civilization.

As a matter of fact, it is cavitation that is usually the cause of noise in plumbing systems, just as it is cavitation that is the other phenomenon involved in applying the foregoing information about pressure fluctuation in submerged jets. The study of jet cavitation as such seems to have originated with the writer some fifteen years ago in connection with the proposed jet propulsion of ships—i.e., the placing of enlarged propellers in pressure passages within the hulls to prevent blade cavitation. It was readily demonstrated that the low-pressure cores of the eddies generated in the shear zone around the emerging jet could well produce cavitation considerably in advance of the ducted propeller. Although the propeller would thus be protected from cavitation damage, the elimination of cavitation as a source of noise and bubbles in the wake would not necessarily be realized. The original studies conducted on the phenomenon at Iowa yielded a magnitude of about 0.6 for the cavitation index $\sigma = (h_0 - h_v)/(V_0^2/2g)$ under conditions of incipiency, as well as the first measurements that appear to have been made of the distribution of pressure fluctuations in the zone of turbulence generation, and the two photographs of vapor bubbles shown in Fig. 6 [13].

Because it was still impossible to predict other than empirically either the magnitude of the cavitation index for incipiency or the position at which collapse of the vapor bubbles could be expected to occur, i.e., the location of the noise source, the two phases of the investigation were continued independently. On the one hand, detailed measurements of the mean-flow and turbulence characteristics of submerged jets without cavitation (for convenience, air into air) were undertaken. These required not only the development of improved instrumentation but the perfection of measurement techniques to the state that the resulting values satisfied the equations of motion in detail as well as in gross. Only now being readied for publication elsewhere [11], the information thus obtained was used by S. T. Hsu of the Iowa Institute staff in preparing Figs. 3, 4, and 5 for the present paper. On the other hand, detailed sonic studies were made by Appel [14] of a cavitating jet at various values of the cavitation index, not only to define the location of the collapse zone but also to clarify a number of apparent anomalies in the earlier study.

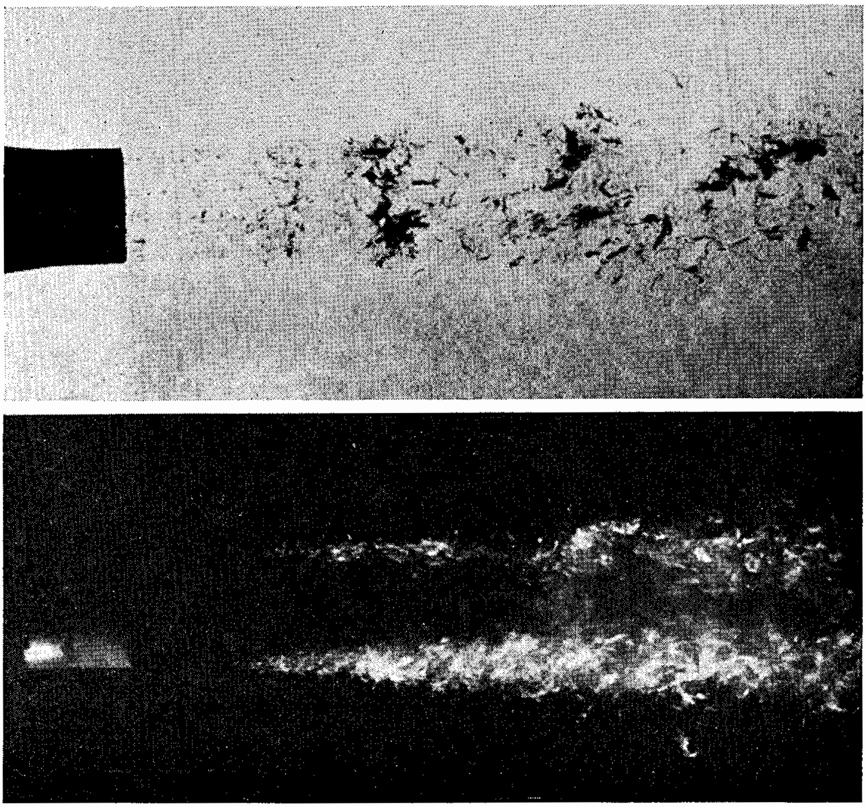

FIG. 6.—HIGH-SPEED PHOTOGRAPHS OF EDDY CAVITATION IN A DIFFUSING JET: ABOVE, SINGLE EXPOSURE, REAR ILLUMINATION; BELOW, 25 EXPOSURES, VERTICAL SHEET ILLUMINATION [13]

As is evident from Fig. 6, the intensity of cavitation-bubble concentration is distributed in much the same manner as the intensity of turbulence, shown by the contours of velocity and pressure fluctuation in Fig. 4. The smaller the cavitation index, moreover, the greater do experiments show the longitudinal and radial range of the visible bubbles to be, as one would expect from the general eddy-cavitation concept. Somewhat surprising, in this regard, was Appel's observation that the maximum source of noise was invariably at the end of the zone of flow establishment, approximately five nozzle diameters downstream from the efflux section. Although bubble collapse must evidently occur beyond any section at which bubbles are still visible, for some reason not yet clear the greatest rate of collapse thus coincides with the zone in which the turbulence reaches the jet axis. Appel's sonic measure-

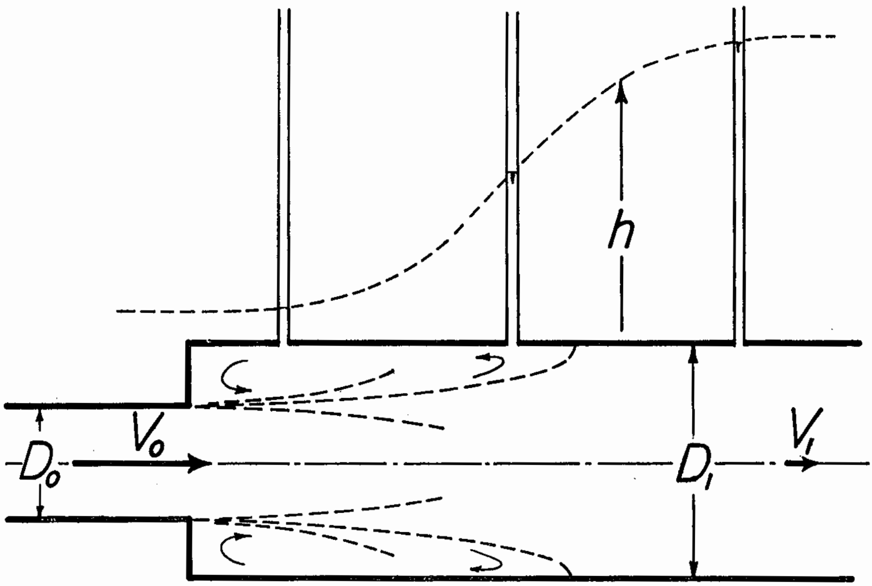

FIG. 7.—JET DIFFUSION AND PRESSURE VARIATION AT A CONDUIT EXPANSION

ments further revealed that at any point the variation in audible noise with the cavitation index involved its frequency but not its amplitude. In other words, each individual bubble collapse produces a pressure fluctuation of essentially the same magnitude, and the relative level of noise is a measure of the relative number of such collapses occurring per unit time. Moreover, the frequency of collapse was found to be quite randomly distributed. Definition of the point of incipiency is thus rather arbitrary. The writer's previouse hope [13] of being able to predict at least the point of incipiency from the turbulence data presented herein is now seen to meet yet another obstacle: whereas the distribution of eddy scale shown in Fig. 5 permits the relative magnitude of average eddy size to be predicted at any point (and hence the relative number of cavitation bubbles present at any instant), not only is this a statistical average, but an unknown coefficient of proportionality is also involved. As a result, although two conditions can now be com-

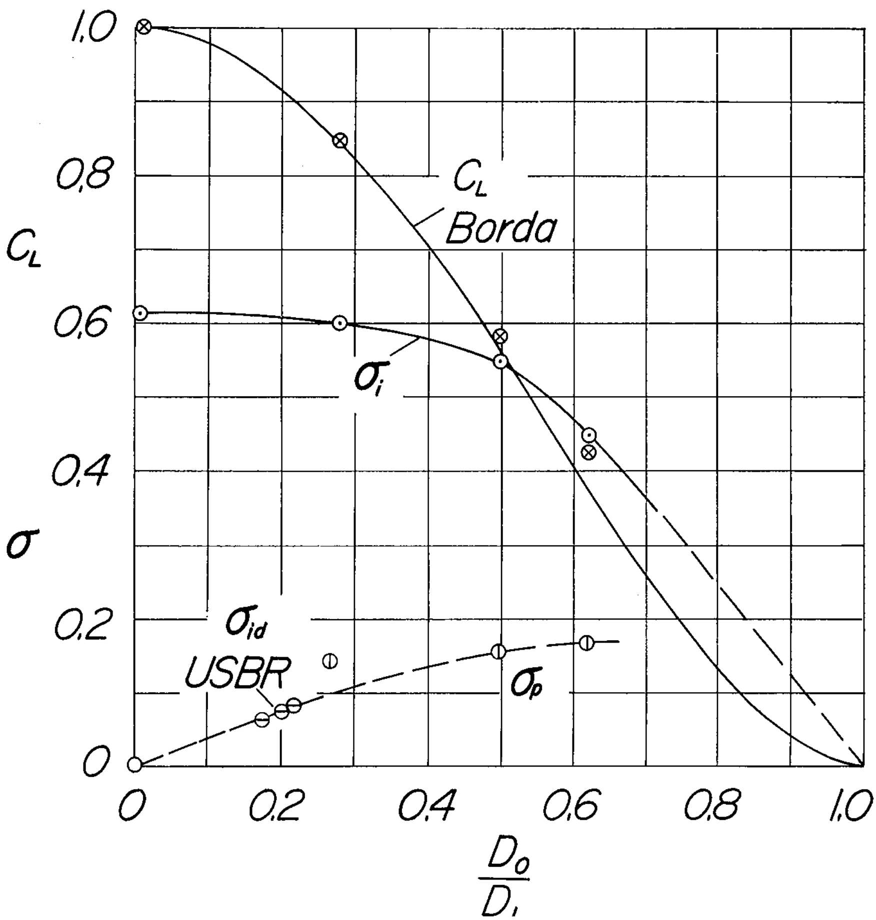

FIG. 8.—COEFFICIENTS OF LOSS, INCIPIENT CAVITATION, AND CAVITATION DAMAGE VERSUS EXPANSION RATIO [15]

pared quantitatively, one or the other still requires experimental evaluation.

Whereas the direct application of such knowledge about eddy diffusion and cavitation is apparently restricted to the phenomenon of jet propulsion, small changes in boundary geometry can extend its range of interest greatly. Consider, for example, the current use of conduit expansions as a means of energy dissipation in hydroelectric installations. Since these are simply diffusion zones of limited radial extent, all of the foregoing flow characteristics should again be encountered, plus a few more. Principal among the latter (see Fig. 7) are the region of reverse flow enclosing the main stream and the longitudinal rise in pressure as the jet expands. The rate of energy loss at such an expansion can be expressed in terms of the Borda coefficient

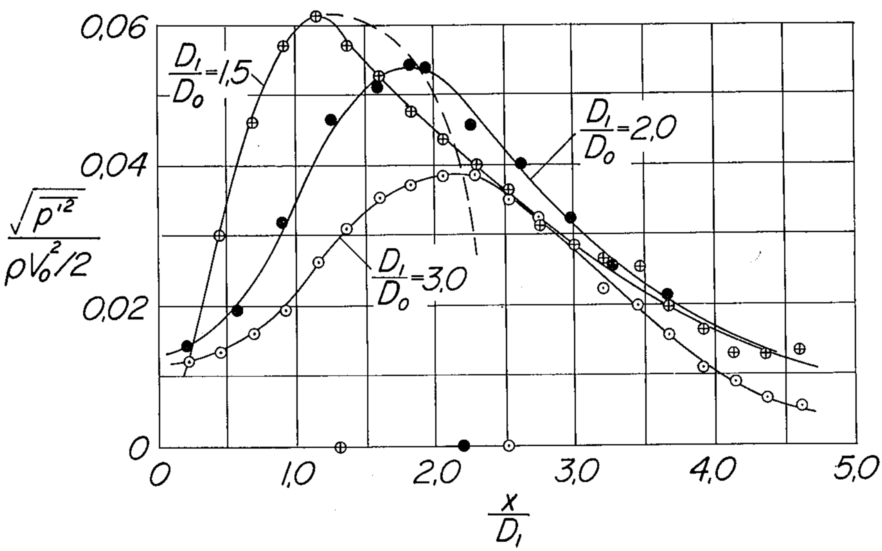

FIG 9.—LONGITUDINAL DISTRIBUTIONS OF WALL-PRESSURE FLUCTATION [17]

$C_L = (V_1 - V_0)^2/V_0^2$ as a function of expansion ratio, as plotted in the uppermost curve of Fig. 8 [15]. Conditions of incipient cavitation, determined experimentally, are seen from the middle curve to agree with those for the unconfined jet at the limiting expansion ratio. While even incipient cavitation is to be avoided in most design, it should be noted that actual danger to the structure from cavitation damage will occur only when the bubble collapse takes place in the vicinity of the wall. Such conditions as determined sonically at Iowa (σ_p) are compared near the bottom of the figure with re-evaluated measurements of actual cavitation damage (σ_{id}) conducted at the Bureau of Reclamation [16].

Since cavitation of this nature results from the eddy structure of the turbulence, the pressure fluctuations which the eddies produce cannot be made to disappear, as can the cavitation itself, simply by raising the ambient pressure. Hence, not only must an energy dissipator of the expansion type be designed to perform without cavitation, but the pressure fluctuations must also be considered in the structural design. Such considerations involve not only the magnitude of the pressure departure from the mean but also the size of the region over which the

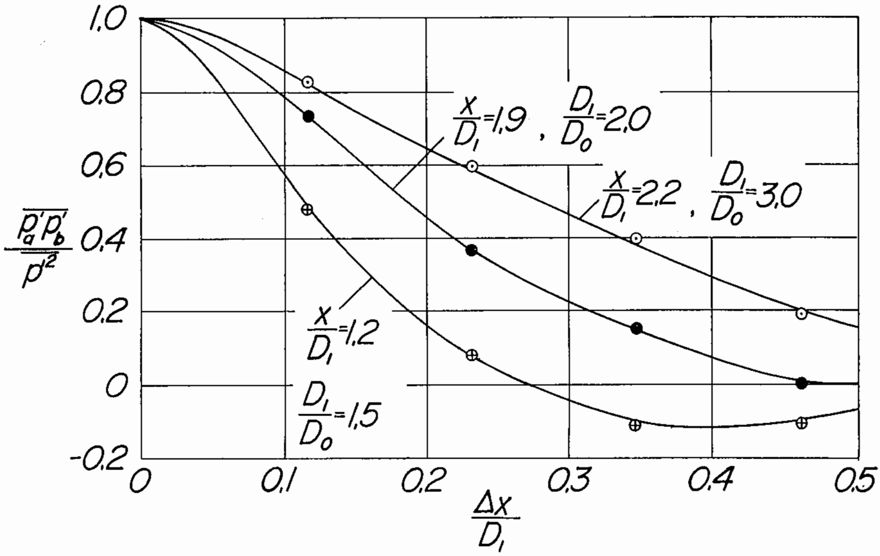

FIG. 10.—LONGITUDINAL WALL-PRESSURE CORRELATIONS [17]

average fluctuation extends and its degree of vibration-producing regularity. In a much more recent study [17], an initial exploration of these characteristics was made for the three expansions involved in the previous experiments (which, in turn, had been made with the same equipment used in the original investigations of the submerged jet itself).

Figure 9 reproduces the distribution of root-mean-square pressure fluctuation along the wall of each expansion. Since the fluctuation of pressure, like that of velocity, approximates the Gaussian probability function, magnitudes greater than the root-mean-square can be expected some 16 percent of the time. The root-mean-square values themselves are seen to increase between the initial section and one slightly upstream from that at which the separation streamline reaches the wall (marked by one of the three points along the abscissa scale), and thereafter to decrease. As must be concluded from the broken curve connecting the maximum values, an optimum condition is reached at about the smallest expansion ratio tested; it is, of course, only

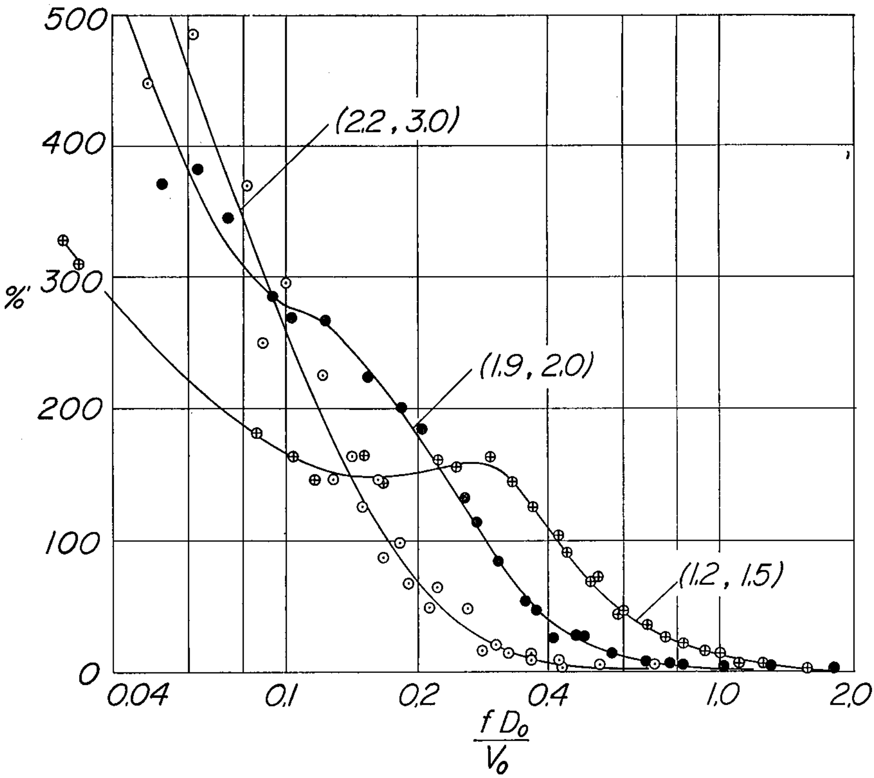

FIG. 11.—FREQUENCY SPECTRA OF WALL-PRESSURE FLUCTUATION [17]

logical that this curve should approach zero as the expansion ratio becomes either negligible or very large.

In order to obtain an indication of the average size of the zone over which individual fluctuations extend, measurements were made of the correlation (degree of agreement) between pressures at points a and b varying distances $\Delta x/2$ either side of the point of maximum root-mean-square fluctuation. With a negligible separation, the two values should be almost identical, and the correlation coefficient $\overline{p_a'p_b'}/\overline{p'^2}$ (see Fig. 10) should then be very close to unity; if, on the contrary, the separation is greater than the distance over which the individual fluctuation is likely to extend, the correlation should be essentially zero (the tendency of such a curve to dip below the axis is a sign of some degree of regularity of the fluctuations). As is seen from Fig. 10, the size of the fluid elements involved in the individual fluctuations is generally a small fraction of the conduit diameter.

Finally, the spectral analyses of the fluctuations, again at the sections of maximum root-mean-square fluctuation, are shown in Fig. 11. The ordinate scale is such as to yield an area of 100 percent under each curve when plotted arithmetically, and the abscissa scale includes the frequency of the fluctuations passed by the filter. It is actually the form of the curves rather than their elevation which is significant, for local humps (such as that in the curve for the 1.5 expansion ratio) indicate a tendency for fluctuations of a particular frequency to predominate and possibly excite elastic vibrations in the structure. It is evident from the figure that this tendency rapidly diminishes with increasing expansion ratio.

The points that the writer has emphasized are but a few of the conclusions that can be reached from the experimental data that have been presented. Likewise, the boundary forms used for illustration are probably but a few of the many which will eventually be investigated in a comparable manner. Phenomena of turbulence and pressure fluctuation are in the air, hydraulically speaking, both here and abroad, particularly, it should be noted, in the Soviet Union. Dr. Freeman would certainly have relished both the situation and the challenge that it brings.

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CONSTRUCTION OF EMBANKMENTS ACROSS PEATY SOILS

By L. Casagrande,* *Member*

Introduction

In response to an invitation by the U.S. Waterways Experiment Station, the author submitted in 1964 a study on mass displacement of soft organic soils by a combination of surcharging and blasting. It included the following topics:

1. Summary of American Practice.
2. Summary of German Developments including brief description of major applications.
3. Discussion of possible further developments of this method.
4. Literature as needed in connection with the above points.

The author considered himself qualified to undertake such a study because he made vital contributions to the procedures of displacement of peat by blasting which were developed in Germany before the second world war.

Although displacement of soft organic deposits (peat, marsh, swamp, muck, muskeg) for the purpose of building highway embankments on firm ground originated in the United States in the early Thirties [6, 26, 27],[1] it has become a neglected art on this continent. On the other hand, further developments of this method have continued in Germany since the author first introduced it into German highway construction practice. Therefore, as part of this study, the author visited Germany in 1964, specifically for the purpose of personally acquainting himself with the most recent developments in this field.

The following is a revised version of the above mentioned study.

Properties of Peaty Soils

The literature contains a number of detailed descriptions and classifications for peaty deposits (bog, marsh, muck, muckeg). An excellent

* Cons. Engr., Prof. of the Practice of Foundation Engineering, Harvard University, Cambridge, Massachusetts.

[1] The numbers in brackets refer to the bibliography at the end of this article.

summary on this topic, including an up-to-date bibliography, has been prepared by Pihlainen [65]. Since the pertinent engineering properties of these materials are quite similar, for the sake of simplicity these highly organic materials will be referred to in the following as peaty soils.

Description of Peaty Soils

With some exceptions peaty deposits are brown to dark brown in color and very soft and spongy in the undisturbed state. Except for the surface crust, the natural water content of such deposits is very high. Although the fibers or leaves may be clearly visible in the undisturbed state, they frequently disintegrate, without much effort, into an amorphous mass during remolding. In spite of peaty soils being usually very soft in the undisturbed condition, they possess distinct brittleness. Upon remolding, they may lose all of their strength and turn into a more or less homogeneous, soupy state.

In the undisturbed condition peat possesses a very high void ratio and a relatively high permeability. Upon load application, or when disturbed, both the void ratio and the coefficient of permeability decrease to very much lower values.

At locations where the grandwater table is somewhat below the surface of a peaty deposit, a drying crust (meadow mat) may exist which has appreciably greater strength, much less moisture, and may be much less sensitive to disturbance.

Water Content

Depending on the origin and on the thickness of a peaty deposit, the water content ranges between wide limits. Whereas the water content of the crust may be as low as 100% (by dry weight), the peat beneath the surface crust may possess water contents ranging between several hundred and several thousand per cent. Since peat, as a rule, has only a negligible submerged weight, the decrease in water content with depth is barely perceptible.

In Table I are recorded the ranges of water contents for typical peat deposits on this continent and in several European countries. In spite of the wide range of water contents, there are great similarities between a typical Canadian peat [2] and an average English peat [40], or between a peat deposit in Northern Ireland [34] and one in Florida [23].

TABLE I

Reference	Location	Water Content %
Adams [1]	Canada	375 - 430
Anderson and Hempstock [2]	Canada (Alberta)	600 - 1350
Brawner [8]	Canada	200* - 300*
		800 - 1400
Casagrande [14]	USA (Mass.)	250 - 800
Colley [23]	USA (Florida)	480 - 900
Dücker [29]	Germany (Schleswig Holstein)	400 - 1250
Duncan, et al [30]	Northern Ireland	400 - 800
Erlenbach [33]	Germany	600 - 900
Feustel and Byers [34]	USA	3200
Flaate and Rygg [35]	Norway	>2000
Garras [38]	Germany	600 - 800
Hanrahan [41]	Ireland	700 - 1400
Hardy and Thomson [42]	Canada (Northwest)	470 - 760
Helenelund [43]	Finland	1000 ->2000
Johnson [46]	USA	200* ->2000
Lake [51]	Scotland	1850
Legget [52]	Canada	up to 1400
MacFarlane [55]	Northern Ireland	700 - 1200
Margason and Fraser [56]	Northern Ireland	300 - 1500
Micklebrough [58]	Canada	480
Moos, von and Schneller [60]	Switzerland	100* - 2100
Morton [62]	USA (New Hampshire)	300 - 650
Ripley and Leonoff [74]	Canada	220* - 1460
Root [77]	USA (California)	550 - 1000
Tresidder and Fraser [86]	Scotland (Shetland)	400 - 1600
Usinger and Garras [88]	Germany	400 - 1200
Ward [90]	England (Wales)	660 - 1300

* Near surface or along edge of deposit.

Plasticity

The plasticity of peat ranges from low plasticity for thoroughly weathered deposits to non-plastic for highly fibrous deposits. As shown on the plasticity chart in Fig. 1, all points are located below the A-line. The two heavy lines and the encircled area represent an average of numerous test results for thoroughly weathered peat in Northern Germany [29] and slightly fibrous peat deposits in the United States [12, 14], with liquid limits ranging between approximately 300 and 1000. With a smaller degree of weathering, i.e., with increase in fibrous matter, the points on the plasticity chart move down and to the left, as suggested by the two dashed lines in Fig. 1.

Although only few investigations have been concerned with the plasticity of peat, it is believed that such tests would provide the engineer with a simple method for estimating the character and the engineering properties of peat deposits.

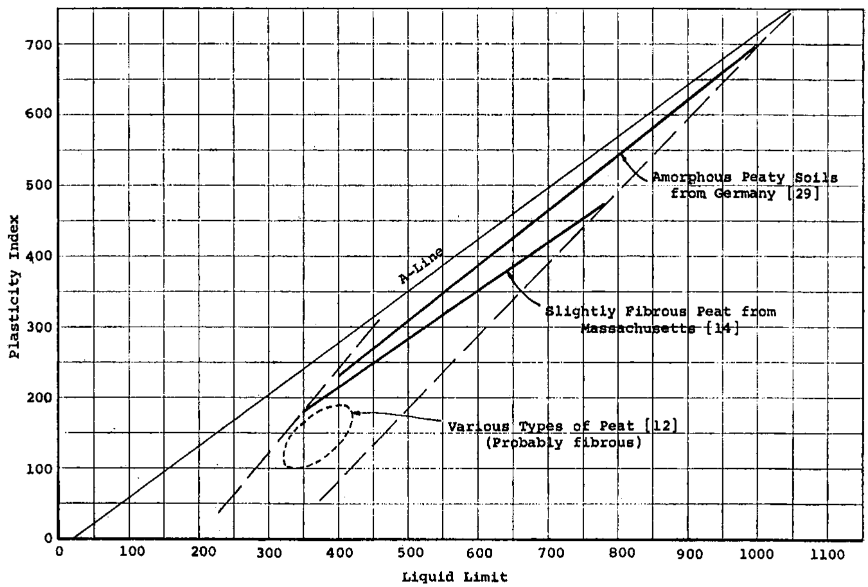


FIG. 1.—PLASTICITY CHART

Shear Strength

The results of investigations of shear strength of peaty soils are summarized in Table II and in Figs. 2 and 3. The majority of these investigations were made by means of in situ vane tests. Where unconfined compression tests were also made, the two sets of strength data agreed reasonably well.

The relationship of shear strength to depth for investigations from widely scattered sites are plotted in Fig. 2. It can be seen that the results for peat deposits in the Boston area [14] and in the vicinity of Vancouver, B.C. [49], are in reasonable agreement. They show a slight effect of surface drying which extends to a depth of about 10 or

TABLE II

Reference	Location of Peat	Shear Strength psf	Natural Water Content %
Anderson and Hempstock [2]	Canada (Alberta)	100 - 250	700-1400
Casagrande [14]	USA (Littleton, Mass.)	100 - 370	230- 750
	USA (Dedham, Mass.)	70 580	400- 800
	USA (Neponset, Mass.)	270	400- 550
	USA (Neponset, Mass.)	70 - 190	250- 380
	USA (Neponset, Mass.)	1000*	110
Dücker [29]	Germany (Schleswig Holstein)	20 -1000*	400- 800
Fraser [36]	Northern Ireland	280 - 560	680-1450
Hardy and Thomson [42]	Canada (Northwest)	100 - 600	470- 760
Lea and Brawner [49]	Canada (Alberta)	110 - 300	
Margason and Fraser [56]	Northern Ireland	340	790
Moos, von and Schneller [60]	Switzerland	100 - 300	220-1460
Ripley and Leonoff [74]	Canada	200 - 430	100-2100
Smith [82]	England	70 - 360	
Tresidder and Fraser [86]	Scotland (Shetland)	70 -1870	400-1600
Ward [90]	England (Wales)	125	800-1000

* Shear strength of drying crust.

15 ft, and below that one is dealing presumably with normally consolidated material which has a shear strength ranging between 400 and 500 psf at a depth of 20 ft. In contrast to these two investigations are the results obtained in Scotland [86] which are distributed over a very wide range as indicated in Fig. 2. Most of the results of these

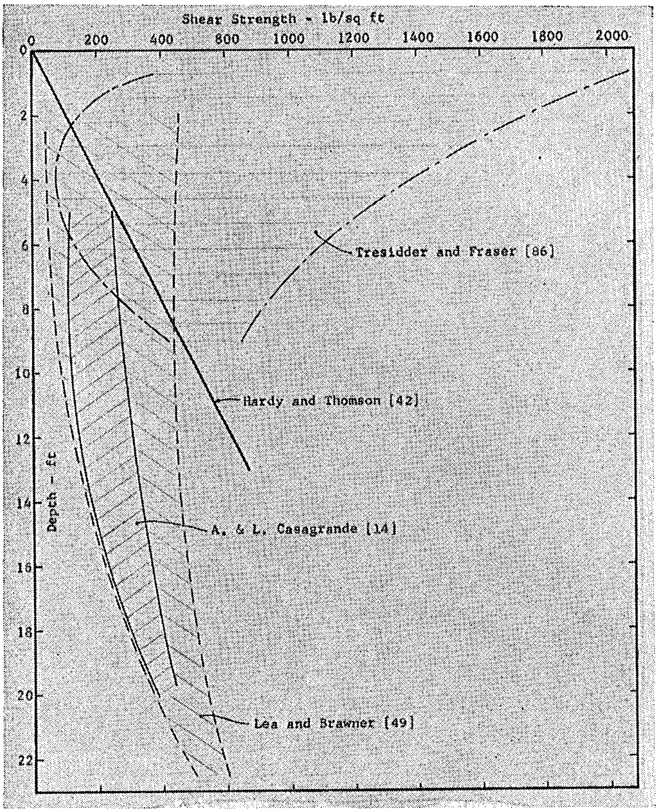


FIG. 2.—SHEAR STRENGTH FROM IN SITU VANE TESTS VS. DEPTH

vane tests, however, are concentrated in the middle of this area, and they indicate substantial preconsolidation by drying. Even at a depth of 9ft, the maximum depth at which tests were made in this investigation, the material is probably overconsolidated. Finally, the test results on muskeg reported by Hardy and Thomson [42] fall into a very narrow range which can be well averaged by the straight line shown in

Fig. 2 that starts with zero strength at the ground surface. Usually such a line would indicate a normally consolidated soil. However, it is very unlikely that a normally consolidated peaty soil would have a shear strength of 600 psf at a depth of only 12 ft. Therefore, some other factor must be responsible for the rapid increase of shear strength with depth at that site.

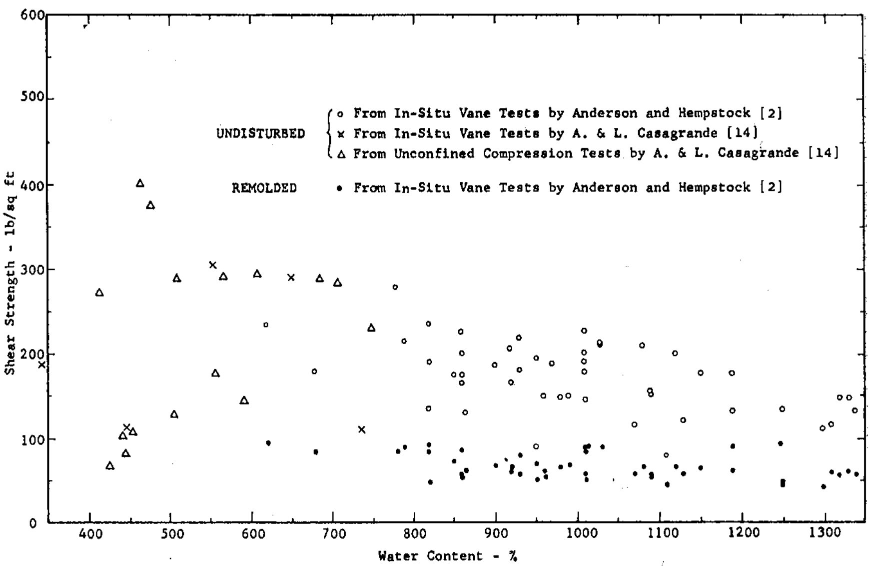


FIG. 3.—SHEAR STRENGTH OF PEATY SOILS

The relationship of shear strength and water content is reflected by the data in Table II and by Fig. 3. In Fig. 3 the results of two sets of investigations are plotted for which individual test results are available [2, 14]. In spite of the wide scattering of the test results, this plot shows the expected increase in shear strength with decreasing water content. It is of interest that the range for the strength of the remolded material is apparently independent of the water content, and that the loss in strength upon remolding is very much greater for the lower range of water contents, for which the strength drops to about one-third of the undisturbed strength. For the highest water content, remolding causes about a 50% loss in strength.

Void Ratio and Permeability

Both the void ratio and the coefficient of permeability of peaty soils range between wide limits. Whereas surface layers and shallow peat deposits which are subjected to air drying may possess void ratios between 2 and 5, for deep deposits they increase to between 5 and 28, with the majority lying between 5 and 15.

From the available information on permeability tests on undisturbed specimens of peat made prior to the start of consolidation

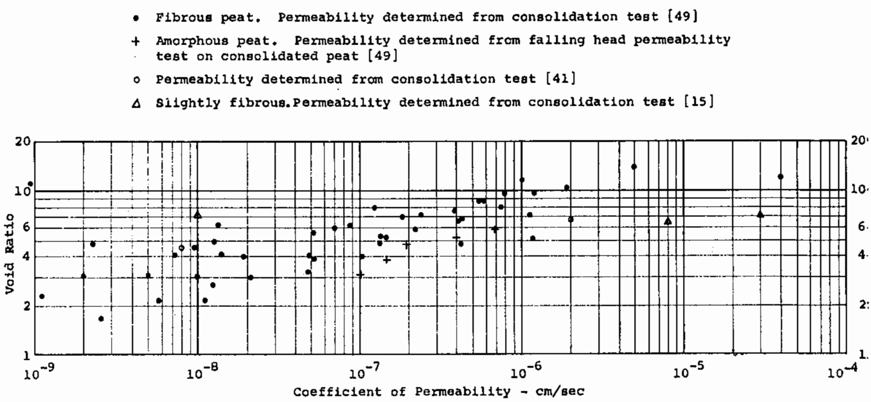


FIG. 4.—VOID RATIO VS. COEFFICIENT OF PERMEABILITY

tests one can conclude that the coefficient of permeability ranges between 10^{-2} and 10^{-4} cm/sec. Upon application of even small loads the permeability decreases rapidly. If the load is increased to 0.6 kg/sq cm the permeability gradually decreases to the order of 10^{-6} cm/sec after two days, and to the order of 10^{-8} cm/sec after a period of seven months [41].

Extensive investigations by Lea and Brawner [49] resulted on a log-log plot, Fig. 4, in a straight line relationship between void ratio and permeability. For comparison, in this figure are also shown the test results obtained by other investigators [15, 41], which fall into the same range.

A summary of the determinations of void ratios and coefficients of permeability for various peat deposits is contained in Table III.

TABLE III

Reference	Location of Peat	Effective Stress kg/sq cm	Void Ratio	Coefficient of Permeability cm/sec
Adams [1]	Canada	No load	3.4	
Casagrande [15]	USA (Mass.)	No load	7.1	3×10^{-5}
		0.02 for 12 hrs	6.6	8×10^{-6}
		0.16 for 12 hrs	3.1	1×10^{-8}
Colley [23]	USA (Florida)	No load	4.6-10.3	
Cook [24]	Canada (B.C.)	No load	2.8-13.1	
Hanrahan [41]	Ireland	No load	12 (12-25)	4×10^{-5}
		0.6 for 2 days	6.8	2×10^{-6}
		0.6 for 7 mths	4.5	8×10^{-9}
Lea and Brawner [49]	Canada (B.C.)	No load	8 -28	$10^{-2} - 10^{-4}$
Micklebrough [58]	Canada	No load	3.2- 9.7	
Root [77]	USA (Calif.)	No load	6 -17	
Thompson and Palmer [83]	USA (E. Coast)	No load	5.1- 7.1	
Ward [90]	England (Wales)		>16	

Primary Consolidation

Because of the relatively high permeability in the undisturbed state, primary consolidation develops rapidly [13, 41, 83, 90]. This is confirmed by numerous settlement observations on highway construction projects on this continent and in European countries which showed that primary consolidation of peat deposits develops practically simultaneously with load application [8, 46, 49, 61, 73].

Secondary Consolidation

After completion of the primary consolidation, the secondary compression of peaty soils proceeds usually along a straight line when plotting the settlements vs. log of time. However, some peats develop secondary compression which increases at a greater rate than such a straight-line relationship, so that predictions of settlements based on short-term laboratory tests may result in values which are too small.

Over a period of many years the secondary compression of a peat deposit often exceeds the primary consolidation. Furthermore, because of the high permeability of many peaty soils, one finds that most of the primary consolidation develops already during construction of the embankment. For these reasons the secondary compression of peaty soils is much more important than the time-lag due to primary consolidation [13, 77, 83].

Typical examples of the magnitude of the settlement due to secondary consolidation from observations are tabulated below [77, 83]:

Thickness of Peaty Soil ft	Height of Fill ft	Rate of Settlement per cycle on logarithmic time-scale ft
13	4.5	1.0
17	5.5	1.5
42	15	4.0
20	?	4.0

Difficulties and costly repairs result from such large settlements of highways due to secondary compression of peaty deposits [10, 28, 49, 61].

The literature contains numerous examples which show that secondary consolidation of peaty soils extend over hundreds of years. A number of medieval buildings in Europe which were built on such soils are still settling. Settlement observations on a 22 ft thick peat deposit in England resulting from draining this peat (i.e., by changing the submerged weight of peat into full weight condition) are reported by Chatwin [22] as follows:

Time Elapsed Years	Settlements ft
12	4.8
22	7.7
27	8.2
44	10.2
84	10.7

In an effort to control settlements of fills on peaty deposits, sand drains have been used extensively. In the judgment of the author, sand drains are of no value for the purpose of accelerating or controlling the large secondary compression of highly organic deposits. In fact, sand drains installed by the conventional method may cause more harm than good because their installation seriously disturbs the structure of the deposit and may result in much larger settlements.

Displacement of Peaty Soils by Blasting

In American highway construction the following methods for displacing peaty soils with explosives have been used:

(1) Toe shooting.
(2) Underfill blasting.
(3) Ditching.
(4) Relief method.

To these four basic procedures of peat blasting should be added a method which was developed in the State of New Hampshire. This procedure is a combination of the above methods and will, in this report, be referred to as:

(5) New Hampshire method.

Finally, in Germany a modified method of underfill blasting was developed which proved highly successful. Hence in this report reference will be made to:

(6) German method.

The basic principles involved in methods (1) to (4) are well described in the literature [5, 18, 21, 31, 44, 55, 81, 95]. Therefore, in this paper are discussed chiefly lesser known details of the various displacement methods. In addition, new concepts are presented that may contribute to greater efficiency and economy in the construction of highways over peat deposits.

FIG. 5.—JETTING OF SMALL CHARGES TO BREAK UP SURFACE MAT OR CRUST

Treatment of Surface Mat or Crust

Most highway engineers consider it good practice to break up the vegetative surface mat or drying crust before placement of any fill material. The purpose is threefold:

(1) To lessen the danger of cracks developing in the vegetative mat which may lead to sudden sliding during filling operations.
(2) To facilitate a uniform settlement of the fill.
(3) To prevent the crust from being trapped beneath the fill.

The dense surface mat is usually blasted with light charges which are either pushed or jetted to a depth of several feet at 1.5 to 3 ft spacing, as shown in Fig. 5. As a rule this operation extends over the full width and length of the proposed filling operation.

The breaking up of the surface mat to the full width of the embankment has the following important disadvantages:

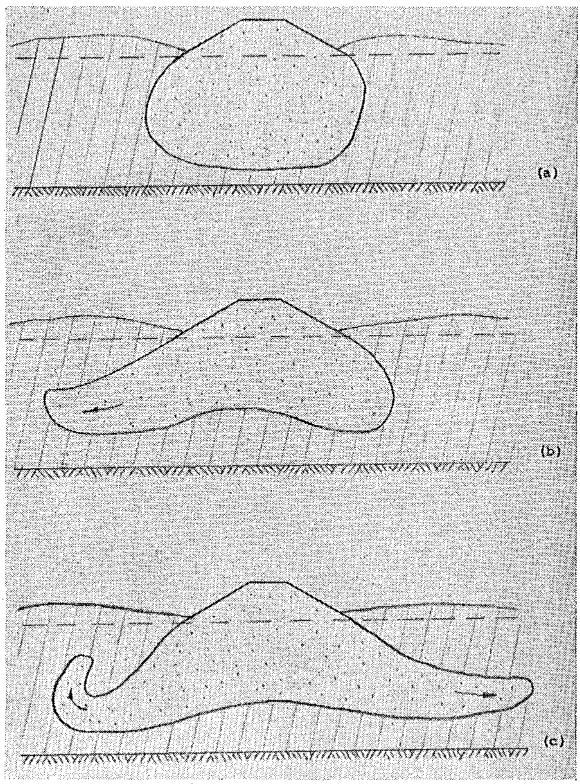


FIG. 6.—TYPICAL FILL SETTLEMENTS ON PEAT

(1) Placement of fill on the surface of peat or other highly organic soils may result in a portion of the fill being lost by excessive "spreading" [10, 28, 43]. Instead of the fill settling in a bulky manner, as shown by the cross-section in Fig. 6a, it may settle in a highly irregular manner and with excessive horizontal spreading, as shown in the cross-section in Fig. 6b. In extreme cases cross-sections of the type shown in Fig. 6c [43] are known to develop.

(2) Blasting of the surface mat disturbs the soft peat for some distance beyond the blasting operations and also to a certain depth beneath the mat. On a number of projects it was observed that the water content of the soft peaty soils had decreased noticeably in the vicinity of the applied fill. For

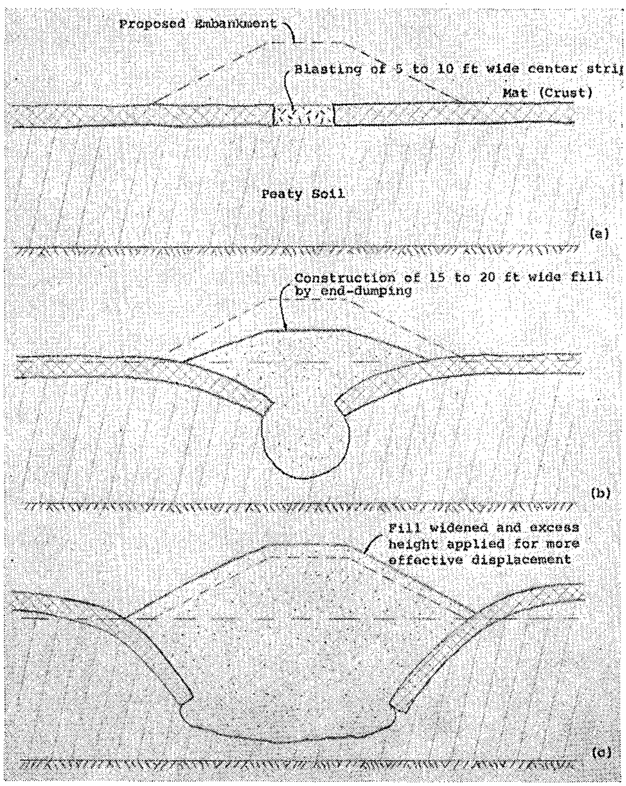


FIG. 7.—PROPOSED PROCEDURE FOR FILL CONSTRUCTION

this reason Morton [62] believes that blasting of the crust must not extend too far in front of the fill operation since the exposed unstable material "tends to stiffen." In his opinion this is due to rapid evaporation. However, this phenomenon is also known to occur at locations where the peat is completely submerged. Toe blasting operations for an embankment in Northern Ireland [36] resulted in a marked decrease in the water content to a distance of up to 200 ft. This was accompanied by a noticeable increase in shear strength within the affected areas. Similar observations were made on a second project in Northern Ireland [56].

It is the author's opinion that a procedure of breaking up the crust over a width of between 5 and 10 ft along the centerline, as shown in Fig. 7a, rather than over the full width of the embankment, and end-dumping a narrow fill at a fast rate and well ahead of placing the fill over the full embankment width, ensures that a narrow core of fill penetrates deep into the peaty soil. This is illustrated in Fig. 7b. Spreading of this core is inhibited by the restraining effect of the undamaged portions of the mat. Once this narrow core of fill is completed over the full length, it can gradually be widened without danger of sudden slides or excessive spreading. Placement of additional fill will result in the mat being deflected downward. Such rotational deflection gradually leads to an increase in width of the submerged portion of the fill, as illustrated in Fig. 7c. Completion of the desired width of displacement can then be achieved with explosives, as discussed under a subsequent heading.

The method of constructing first a narrow fill along the centerline over the full length of the peat deposit has been applied by the Michigan State Highway Department for peat in excess of 20 ft in depth [10]. It has also been used successfully on several German projects [16, 88].

Toe Shooting

The standard procedure of toe shooting consists of disturbing the peat by blasting ahead of the fill for a distance of between 25 and 50 ft per operation. The author considers it preferable to shoot only one row of charges at a time. These charges are arranged close to the toe of the fill, as illustrated in Fig. 8. Individual charges should be small enough so as not to throw out peat, including adjacent fill. The desirable quantity of explosives per hole should be determined by experimentation and will for average conditions not exceed $\frac{1}{2}$ in lbs, where h is the effective depth of the charge as shown in Fig. 8.

The standard procedure of toe shooting is slow and will not always guarantee full displacement of the peat. Experience has shown that large pockets of peat are frequently trapped beneath the fill and cannot be displaced at a later stage without excessive efforts and costs.

Cognizant of these difficulties, German engineers have in recent years developed a new method of toe shooting. It makes use of the desirability of increasing the confining load above individual charges, thus enabling the use of larger charges which in turn permit great efficiency and therefore also greater economy. This procedure consists of the following steps:

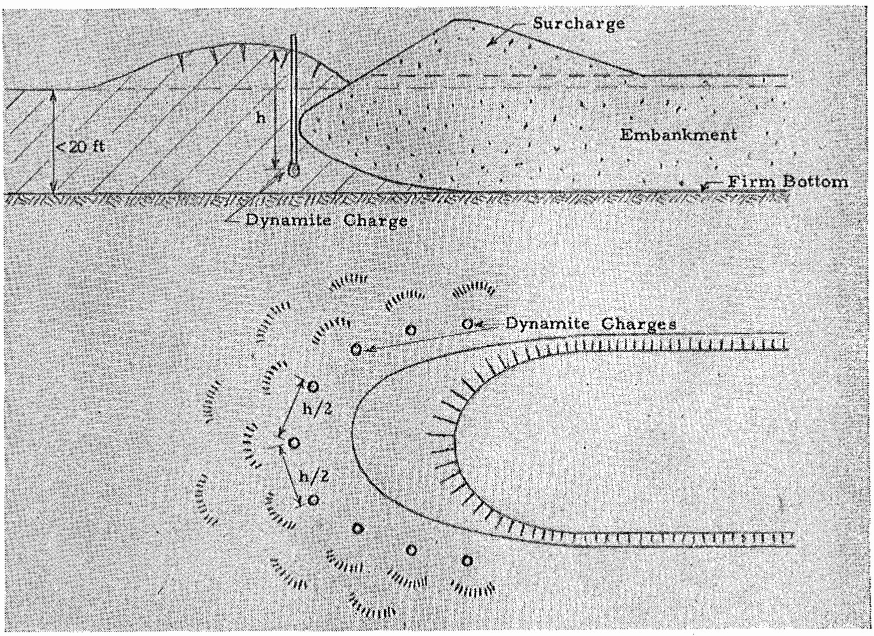


FIG. 8.—DISPLACEMENT OF PEAT BY SURCHARGING AND TOE SHOOTING

(1) As illustrated in Fig. 9a, ahead of the completed embankment already resting on firm bottom, enough sand is spread to permit the crew to operate on the surface of the peat. Depending on the consistency of the peat, the thickness of this sand platform ranges between 10 in. and 2 ft.

(2) From the surface of this sand platform, 10 in. diameter holes are jetted to firm bottom, using a jetting device described below. Depending on the depth of the peat stratum and on the available degree of confinement, the jetted holes are spaced between 6 and 15 ft, as illustrated in Fig. 9b.

(3) Immediately upon completion of one hole, the charges are lowered to the bottom of the hole. It was found that if the surface of

the slurry which develops during the jetting operation is kept above groundwater level, such jet holes will remain open long enough for the charges to be lowered without collapse of the hole. Depending on the depth of the hole, between one and five 20 lb units of explosives are used per hole. These units are delivered by the explosives manufacturer already assembled in cylindrical waterproof cardboard containers.

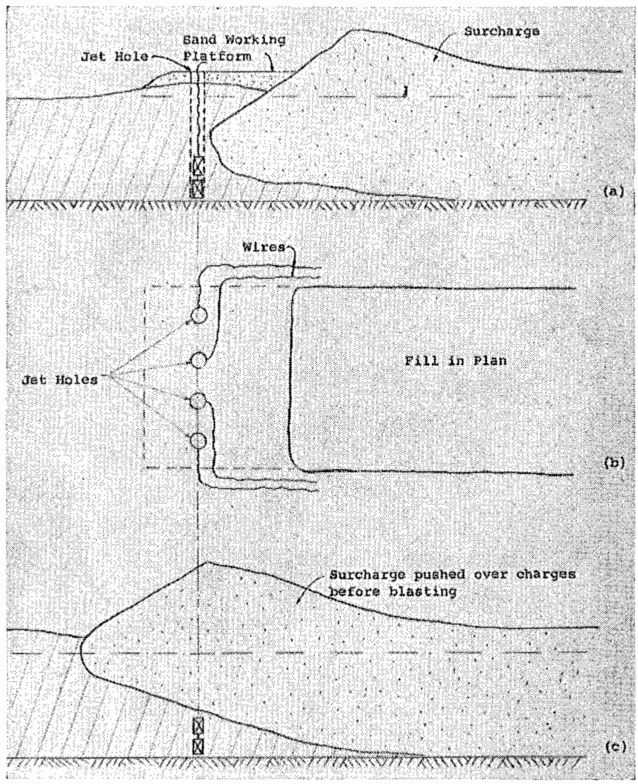


FIG. 9.—GERMAN PROCEDURE OF TOE SHOOTING

(4) As illustrated in Fig. 9b, on the ground surface wires are carried to both sides of the proposed fill so that they will not be damaged during subsequent filling operations. For depths of peat greater than 30 ft it was found expedient to protect the electrical circuits by stringing the wires through plastic tubing.

(5) Fill material is then pushed over the area of the jet holes

until the full height of fill including the desired surcharge is reached. This step is illustrated in Fig. 9c.

(6) Then blasting is carried out. The process is repeated in the same order.

This method does not require the fill to be extended in a "V" shape but makes use of a blunt head of fill, as illustrated in Fig. 9b.

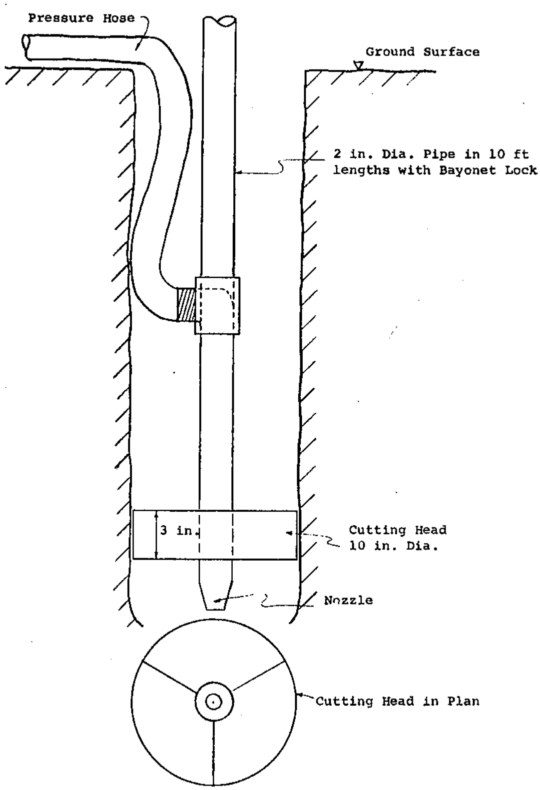


FIG. 10.—SKETCH OF GERMAN JETTING DEVICE FOR TOE SHOOTING

Fig. 10 shows a sketch of the jetting device used for the German method of toe shooting. It consists of a cutting head, a pressure hose connected to the jetting pipe approximately 2.5 ft above the cutting head, and 10 ft lengths of pipe which can be extended quickly by means of a simple locking device. The 1 in. nozzle extends approximately 3 in. below the lower edge of the cutting head. This jetting device is handled by a two-man crew using a water pressure of up to several hundred psi. As soon as the pump is started the jetted hole develops by the up and down movement of the jetting device. Penetration to depths of over 70 ft is usually achieved within several minutes. The author watched jet holes of 30 ft depth being completed within approximately 1 minute.

Currently this method is being employed very successfully along the Autobahn near Emmerich, close to the Dutch border. In April, 1964 the author inspected this work in progress, and also other peat blasting projects which have been completed in recent years. Without exception this method seems to have worked very successfully, both regarding the effectiveness of displacing peat and costs.

Underfill Blasting

The American procedure of underfill blasting is carried out as follows. The vegetative mat is thoroughly broken up by blasting. Then the fill is placed and explosives are pushed through the fill well into the peat by means of 1.5 to 5 in. diameter pipes driven or drilled through the fill. For great depths of peat, explosives experts [5] recommend settling the peat by blasting in stages, handling 10 to 15 ft of peat at a time, and in sections of embankment between 100 and 200 ft in length.

The literature contains descriptions of numerous applications of this method for depths of peat ranging up to over 50 ft and using charges of up to 100 lb per hole [25, 45, 64]. One of the largest projects of this kind was an 1800 ft long peat crossing of over 50 ft in depth [45].

As will be described under a separate heading, a modified method of underfill blasting was developed in Germany in the Thirties. Large pre-assembled charges are jetted through the sand fill into position and blasting is performed in one operation over the full length of the peat crossing, settling the fill to full depth. In this manner the cost per unit of displaced peat decreases with increase in depth of peat.

Ditching

This method has been used in the past for peat deposits not exceeding 15 feet in thickness. A ditch is blasted along the centerline of the proposed embankment to as nearly the full depth of the peat as practical and in sections not exceeding 50 ft. Such ditches have been blasted to widths up to 50 ft in one operation. Each section of the ditch is filled by end-dumping immediately following the blast.

To the author's knowledge this method has not been used in recent years. The reasons for this are probably twofold:

(1) Soft peat of relatively shallow depth can usually be displaced by surcharging, toe shooting, or by a combination of both.

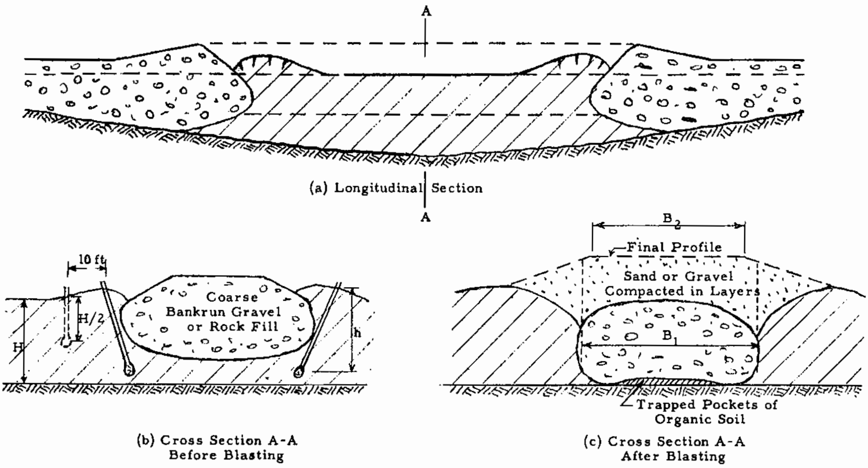


FIG. 11.—UNDERFILL BLASTING—NEW HAMPSHIRE PROCEDURE

(2) Development of large equipment has gradually made excavation of peat to firm bottom competitive.

Relief Method

This method involves the blasting of ditches on both sides of the fill so as to enable the peat to escape from underneath the fill. Normally this method is combined with underfill blasting [5, 31, 44].

New Hampshire Method

For depths of peat of between 10 and 40 ft the New Hampshire Highway Department has developed an economical method of settling fill to firm bottom. Where the peat possesses a surface crust it is first disrupted by blasting in short sections. Then the fill is placed by end-dumping. The two end sections of the embankments, where the thickness of soft material is less than 10 ft, are usually settled to hard bottom by surcharging only. The main body of the fill is then placed over the full length as indicated by the dashed lines in Fig. 11a. The soft material which remains beneath the fill is displaced by means of the following procedure of underfill shooting. Along both sides of the fill dynamite charges are placed at 10 ft centers felow the toe of the fill as indicated in cross-section A-A in Fig. 11b. This is achieved by means of 1.5 in. pipe that is pushed down with a special handle in such a manner that the bottom of the pipe is located beneath the toe of the fill. The quantity of dynamite in lbs per charge is about twice the depth H in feet.

If considerable quantities of soft materials have piled up on both sides of the fill (Fig. 11b), or if the meadow mat or crust has not been blasted effectively, thus causing too much interference with lateral displacement of the liquefied material, a second row of small charges is placed about 10 ft from the main row, as indicated in Fig. 11b, and fired a fraction of a second after the main rows. In some older procedures which employ two or more rows on each side of the centerline, the outer rows were exploded first. However, experience in New Hampshire has proved that a greater efficiency is achieved when the main row is exploded with resistance on both sides.

The dashed lines in the cross-section in Fig. 11c indicate the final profile of the fill which is built upon the settled fill. In the experience of the New Hampshire State Highway Department, if relatively thin lenses of the compressible material are caught beneath the fill, they consolidate quickly and do not cause objectionable settlements after completion of the pavement.

The New Hampshire Highway Department found it advantageous to use for the main body of the fill as coarse material as possible, either a very bony bankrun gravel or even rockfill. Coarse granular fill has the following advantages:

(1) The fill settles with much less spreading than experienced by a sand fill;
(2) Coarse fill has the ability to effectively arch across peat pockets that may be caught beneath the fill.

The additional fill which is placed upon the settled fill, shown by the dashed area in Fig. 11c, may consist of any suitable granular material, and is placed and compacted in layers.

German Method

In connection with the construction of the German Autobahnen, from 1934 to 1940, numerous crossings of soft organic deposits had to be made, some of which were over a thousand feet long and with a maximum depth of about 60 ft. After some experimentation with

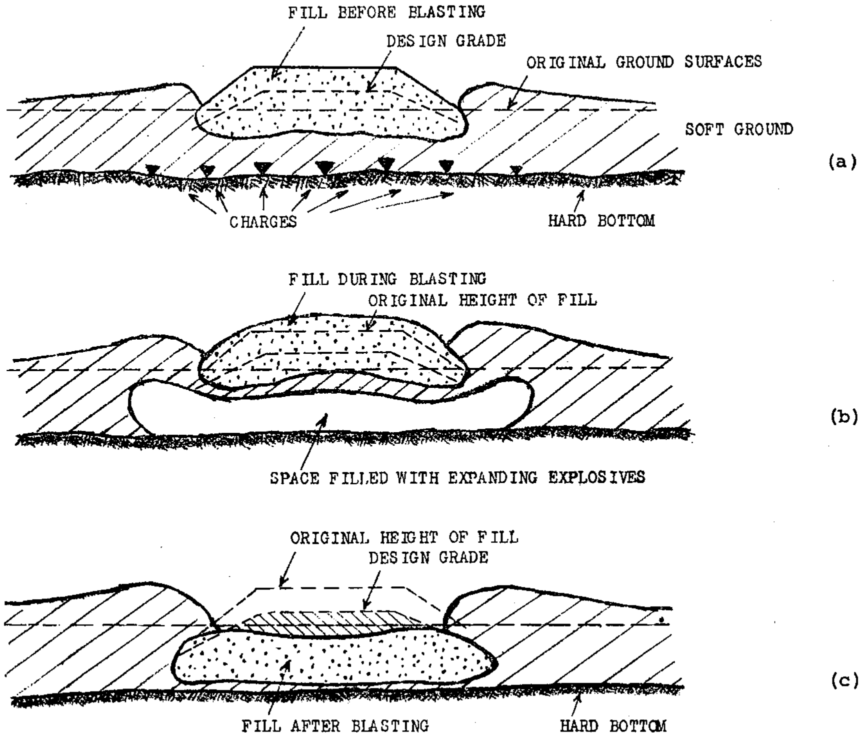


FIG. 12.—GERMAN PROCEDURE OF UNDERFILL BLASTING

the American procedure of underfill blasting [19], the following method evolved. Depending on the consistency of the surface crust, either a narrow strip is disrupted along the axis of the proposed embankment followed up without delay with construction of a narrow fill which is gradually widened, as illustrated in Fig. 7; or the entire fill is placed for the full length of the crossing, on top of the undisturbed mat, as shown in Fig. 12 [16, 17, 18, 20]. The height to which the fill is placed is so controlled that after blasting only minor additional filling is required. The thickness of peat beneath the fill can readily be determined by jetting a pipe through the fill, and observing the color of the returning wash water and the resistance of the jet pipe.

Fig. 13.—Torpedo Charge Ready To Be Jetted Through Fill

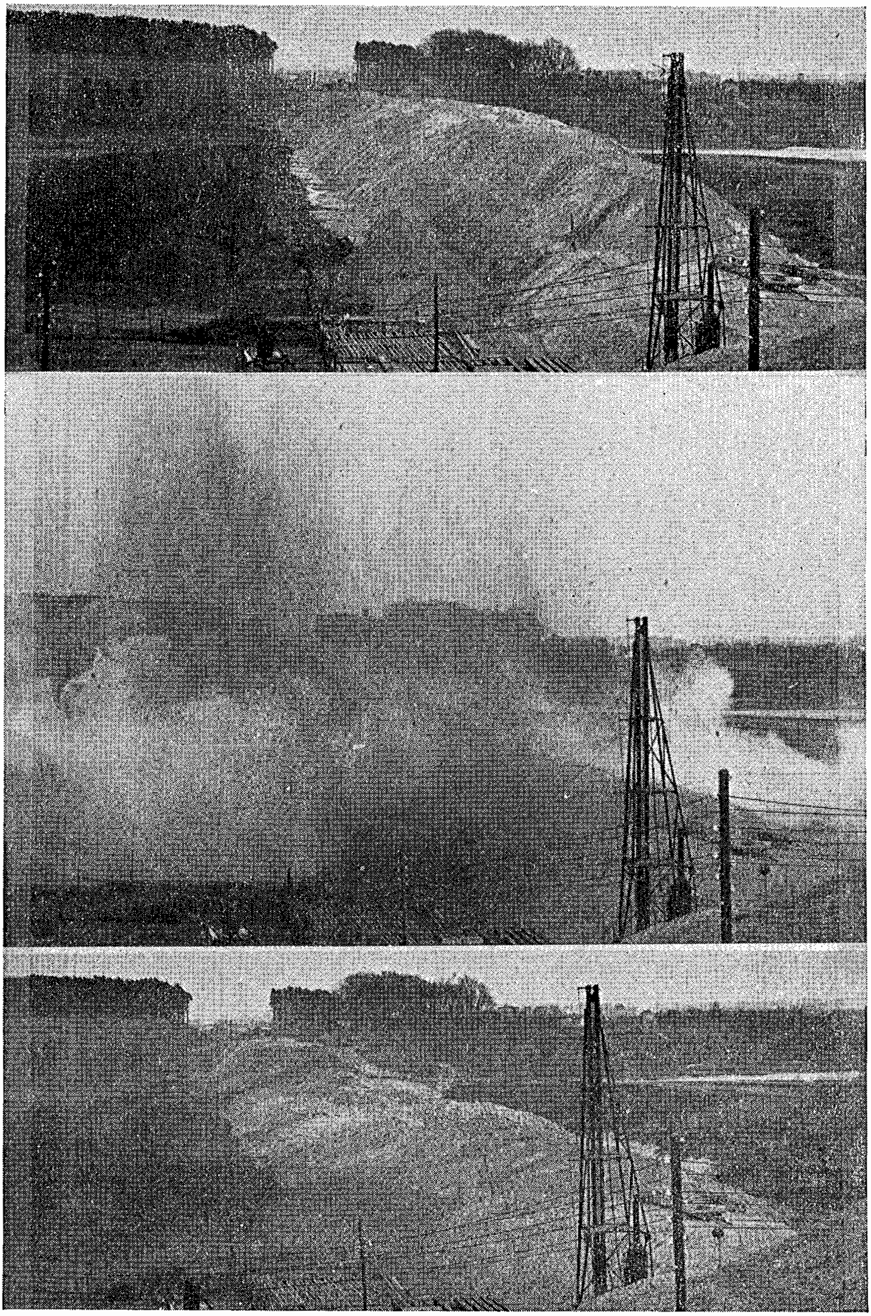
Then, charges in much greater units than is customary in the United States are jetted into position through the fill down to firm bottom, and are fired simultaneously over the full length and width of the fill, as illustrated by Figs. 12a and 12b. Such a simultaneous explosion of charges below the fill is highly effective in destroying the strength of the soft material. Furthermore, the explosion lifts the whole fill bodily for several feet, Fig. 12b, and then the mass drops down upon the underlying disrupted peat which is thereby effectively displaced as indicated in Fig. 12c.

Several methods were developed for the purpose of jetting preassembled charges through the fill and to the bottom of the underlying peat. In the method shown in Fig. 13, a torpedo shaped metal

FIG. 14.—ECCENTRIC JETTING DEVICE IN OPERATION

case containing the explosive is connected to a jetting pipe in such a manner that the pipe can be salvaged after the charge is in place. Another device that proved very satisfactory is shown in Fig. 14. It consists of four pet nozzles which form a sleeve to which the jet pipe is attached. This device is jetted to a depth at which the charge is to be placed. Then, with the jet remaining in operation to keep the hole open, the case containing the explosive is lowered by means of an electric cable and finally the jetting device is withdrawn. Both devices worked well only if the fill did not contain large amounts of coarse gravel; otherwise this coarse gravel would gradually collect at the bottom of the jet hole, finally blocking further progress.

In the German procedure three methods of fill construction were used.



(a) Before

(b) During

(c) After

FIG. 15.—UNDERFILL BLASTING AT GOLLINGWIESE

(1) Embankment constructed to the full width and length and blasted in one operation (Fig. 12).

(2) As illustrated in Fig. 7b, first a narrow fill is constructed along the centerline to the full length. After blasting of this fill it is then widened on both sides and settled to firm bottom by blasting. Depending on the dimensions of the embankment and the depth of peat, this widening and additional blasting may be done in one operation or in several stages.

TABLE IV

Method of Displacement	Peat Displaced cu yd	Amount of Explosives lb	lb/cu yd Displ. Peat
Filling	10,400		
1st Blasting	11,700	8,100	0.7
Surcharging	1,300		
2nd Blasting	6,500	4,200	0.6
Surcharging	1,300		
3rd Blasting	7,800	6,100	0.8
Surcharging	500		
4th Blasting	2,100	5,100	2.4
Surcharging	400		
5th Blasting	3,500	11,400	3.3

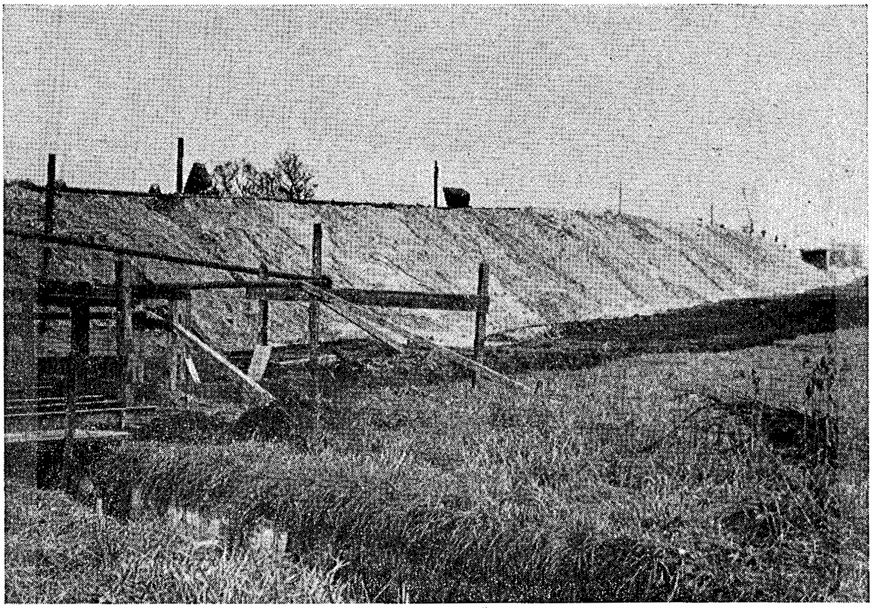
(3) First a narrow fill is constructed along one side of the embankment to the full length, and then settled to firm bottom by blasting. The fill is then gradually widened and blasted again in either one or several additional stages.

Typical examples of the German procedure are summarized below:

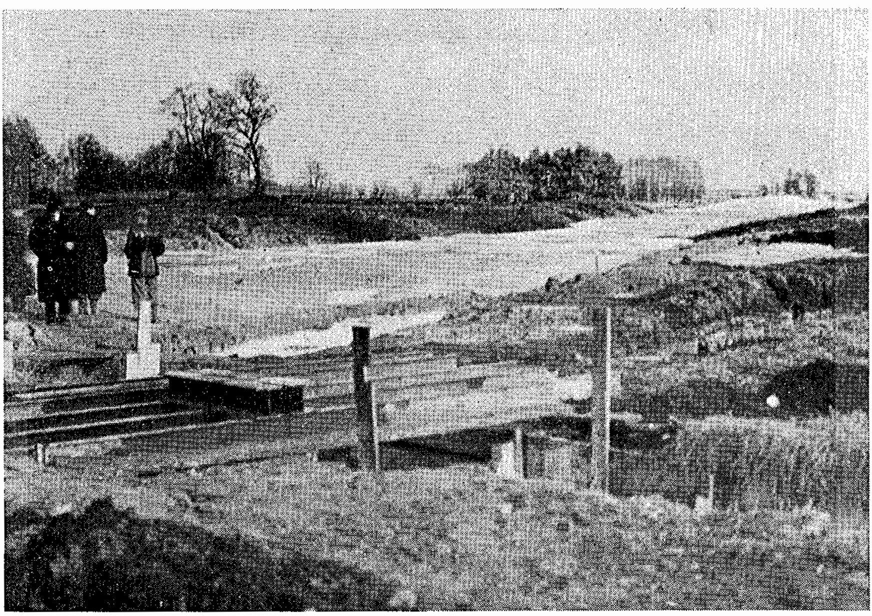
Gollingwiese [88]

The length of peat crossing was approximately 300 ft and the greatest depth about 40 ft, with the top 10 ft consisting of a rather tough vegetative mat. The crest width of the embankment was 75 ft.

The peat was excavated from both ends to a depth of 20 ft. Then the sand fill was constructed to the full width and length. In order to displace as much as possible of the peat without blasting, an excess surcharge of 15 ft was used. Since the firm bottom was sloping toward one side of the fill, several stages of blasting were required



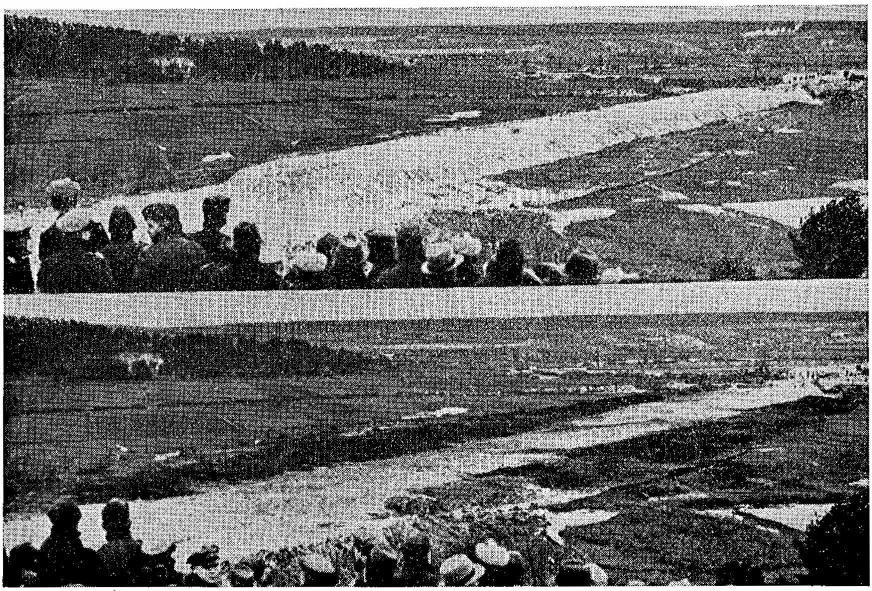
(a)
Before
Blasting



(b)
Seconds
Later

FIG. 16.—UNDERFILL BLASTING NEAR SAARMUND—FIRST BLAST

before the desired width of peat was displaced. The total amount of peat displaced was 45,000 cu yd, and a total of 35,000 lb of explosives were used. The condition before, during, and after blasting is illustrated by the photographs in Figs. 15a, b, and c. Fig. 15b shows large amounts of sand being thrown out which is indicative of excessive concentrations of explosives having been used.



(a) Before

(b) After

FIG. 17.—UNDERFILL BLASTING NEAR SAARMUND—SECOND BLAST

In Table IV are compared the quantities of peat displaced during filling operations and achieved by the various stages of blasting. As revealed by the last column in this tabulation, the amount of explosives used per cu yd of displaced peat increased rapidly with each subsequent blast.

Saarmund [*16*]

This peat crossing is about 700 ft long with an average depth of 25 to 30 ft and a maximum depth of 35 ft. The water content of the peaty soil ranged between approximately 400 and 1100%.

The initial sand fill had a crest width of 30 ft and a height of approximately 25 ft above the original peat surface. This fill was settled by blasting to its full length in one operation using four rows of charges which were jetted into position. After having been widened along both sides to the full width of 75 ft, this fill was settled by additional blasting. The largest individual charges were 250 lb and the smallest ones 65 lb. The total amount of peat displaced was approximately 200,000 cu yd using approximately 70,000 lb of explosives. The conditions prior and after the first blast are shown in the photographs in Figs. 16a and b. The effect of the second blast on this project is illustrated by Fig. 17. From personal observations during these blasting operations the author concludes that also in this instance excessive concentrations of explosives were used.

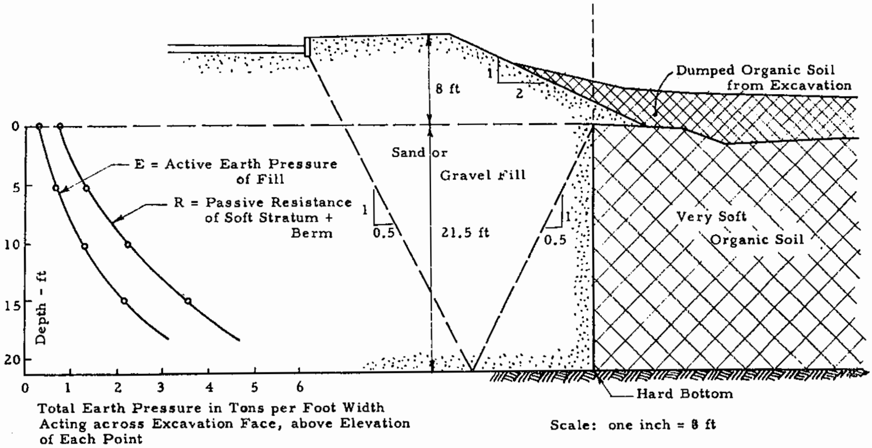
Konigsberg [*33*]

This several hundred foot long peat crossing with a depth of up to 60 ft was blasted in stages starting with a central dike of approximately 30 ft in width. After settling this fill by blasting, it was widened and blasted in two additional stages. The total peat displaced was approximately 130,000 cu yd and the total quantity of explosives was about 70,000 lb.

Required Width of Peat Displacement

For many years highway engineers were of the opinion that because of the small strength of peat the slope of the fill extending below the original peat surface should be approximately the same as above ground. Measured by this standard and depending on the method of peat blasting used, the bottom width of the fill after displacement of the peat may either turn out to be excessive or inadequate.

In recent years highway engineers on this continent and in Europe made the observation that in cases where according to previous requirements the base width was deficient, very few or no difficulties developed on the finished slopes of such embankments. This is not surprising considering the fact that during filling and blasting operations the peat will consolidate and increase in strength for some distance from the slopes, as has been discussed under a previous heading. There is no doubt that because of increase in resistance to lateral movement the body of fill need not be as large as required for stability above ground. German engineers are now satisfied with an average slope of 1 on 0.75 from the crest of the fill to the toe of the fill at firm bottom. This requirement is based purely on empirical knowledge. Obviously if an attempt were made to analyze such a problem one would have to take into consideration a number of variables including consolidation and strength characteristics of undisturbed and partially disturbed peaty soil, depth of the peaty deposit, dimensions of the proposed fill and the method of blasting. In contrast to such a difficult undertaking, analysis of the required base width of a fill, which is constructed by excavation of the peat, is a much simpler problem. If the results of such an analysis are applied to

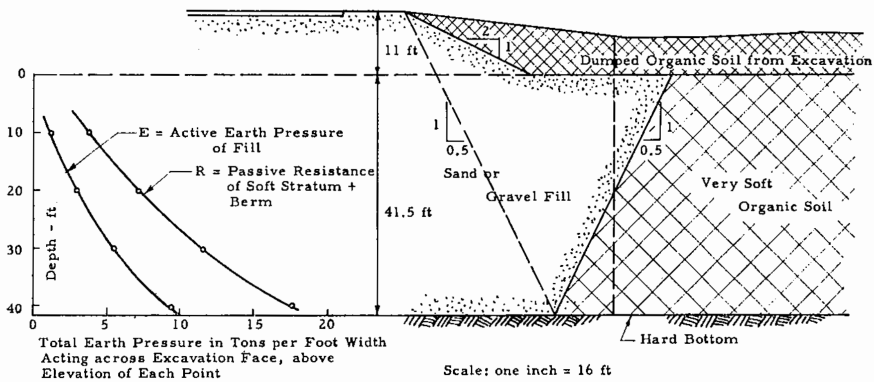


DEDHAM SITE—ROUTE 1
FIG. 18.—ANALYSIS OF ACTIVE EARTH PRESSURE OF GRANULAR FILL AND OF PASSIVE RESISTANCE OF SOFT STRATUM

the necessary width of fill, as developed by blasting, such a procedure would then be on the conservative side.

There are a number of rules-of-thumb in existence in the United States for the required width of excavation of peat in highway construction. The State of Massachusetts, for instance, has been using until recent years several methods for arriving at the desired width of excavation. For the example shown in Fig. 18, the design width was determined by a line sloping 1 on 0.5 from the edge of the pavement to hard bottom, and from that point another inclined line with the same slope in opposite direction to its intersection with the ground surface. Through this intersection a vertical line is drawn which establishes the width of excavation.

When using the above rule-of-thumb it was found that the shoulders and sometimes also the outer portion of the paved areas were subject to settlements and cracking. Although these effects were not very serious, the Department of Public Works found it expedient to support a research project [14] in order to provide assurance that highways built in this manner will not be subject to severe sliding. An analysis of the active and passive earth pressures for the example in Fig. 18 showed that sliding could not develop for these conditions. On the other hand, all portions of a sand and gravel fill, located outside the angle of repose drawn from the toe at hard bottom, are obviously



LITTLETON SITE—ROUTE 2

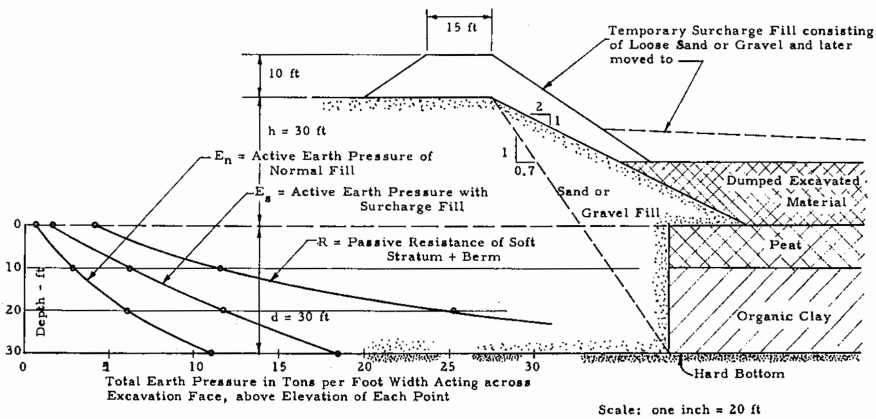
FIG. 19.—ANALYSIS OF ACTIVE EARTH PRESSURE OF GRANULAR FILL AND OF PASSIVE RESISTANCE OF SOFT STRATA

subject to movement due to gradual lateral consolidation of the adjacent peat.

In Fig. 19 is shown another example where the design width was determined by means of a rule-of-thumb which is slightly different from that shown in Fig. 18. In spite of the observed movements in the outer portion of this embankment, it was again found by comparison of active and passive earth pressure that there was no danger of major sliding. Finally, in Fig. 20 a third method is shown for determining the desired width of excavation. Also, in this instance the analysis showed that major instability could not occur. However, none of the embankments built with the use of the above rules-of-thumb are safe against cracking and subsidence within the outer portions of the fill on account of lateral deformation and consolidation of the peat stratum. The magnitude of this lateral yielding depends chiefly on the character of the peaty deposit. In many instances it may be sufficient to know the natural water content of the material in order to judge whether excessive yielding may be anticipated.

There are two ways one can reduce to a tolerable magnitude the lateral deformations of the fill:

(1) By increasing the width of excavation.
(2) By applying a temporary surcharge along the outer portions of the embankment, Fig. 20, which by increasing the active



NEPONSET SITE—SOUTHEAST EXPRESSWAY
FIG. 20.—ANALYSIS OF ACTIVE EARTH PRESSURE OF GRANULAR FILL AND OF PASSIVE RESISTANCE OF SOFT STRATA

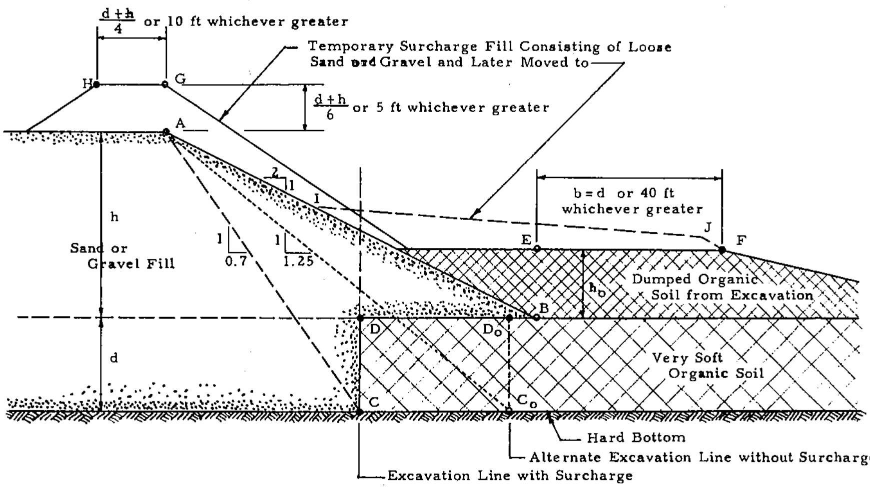
earth pressure will cause "prestressing" of the peaty soil. If such a surcharge is left in place for a period of several weeks and is then moved to the side of the embankment to form a permanent load on the outside of the vertical boundary between the natural fill and the peat, it would in addition result in an increase of the passive resistance of the peat.

This suggested procedure of surcharging should in most cases prove to be more economical than widening of the excavation. The surcharge fill need not extend over the full width of the embankment but only along the edges in the manner illustrated in Fig. 20.

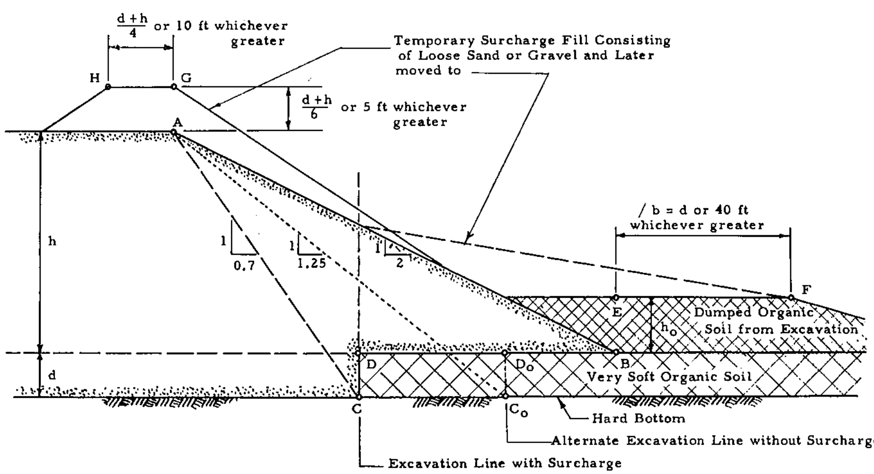
The analysis of the examples shown in Figs. 18 to 20 has led to the following two alternative rules for the required width of peat excavation [14]:

Rule I—Without Temporary Surcharge

From the edge of the crest of the embankment, point A in Fig. 21, draw a line sloping 1 on 1.25 to its intersection C_o with the hard bottom. The vertical line C_oD_o fixes the width of the excavation of the soft



PROCEDURE—H/D = 2
FIG. 21.—FOR H/D = 2 REQUIRED MINIMUM H_0 = 0.37H



PROCEDURE—H/D = 5
FIG. 22.—FOR H/D = 5 REQUIRED MINIMUM H_0 = 0.25H

stratum. The slope of the embankment should be approximately 1 on 2.

The above width of excavation assures full protection against objectionable settlements of the crest of the embankment.

In the example illustrated in Fig. 21, the toe of the embankment, point B, lies only slightly outside the fill. Therefore, there is no danger of substantial sloughing of the lower portion of the slope. However, when h/d is large, as, e.g., in Fig. 22, the portion of the toe of the embankment which is resting on the peat surface would not stand at the slope of 1 on 2. One may either allow the lower slope of the embankment to assume a considerably flatter equilibrium slope, or the toe can be protected against sloughing by building up a berm outside the embankment slope consisting of dumped excavation of the peat. The filling of this protective berm should not lag too far behind the elevation to which the embankment has been built up. The height of the protective berm may be determined by the method which is described under Rule II and illustrated in Fig. 25.

Rule II—With Temporary Surcharge

From the edge of the crest of the embankment, point A in Fig. 21, draw a line sloping 1 on 0.7 to its intersection with the hard bottom, point C. The vertical line CD establishes the width of the excavation of the peaty stratum, unless the theoretical toe of the embankment, point B in Fig. 21, is located closer to the center line of the fill, in which case the width of the excavation shall be fixed by the vertical through point B, as is the case in Figs. 23 and 24.

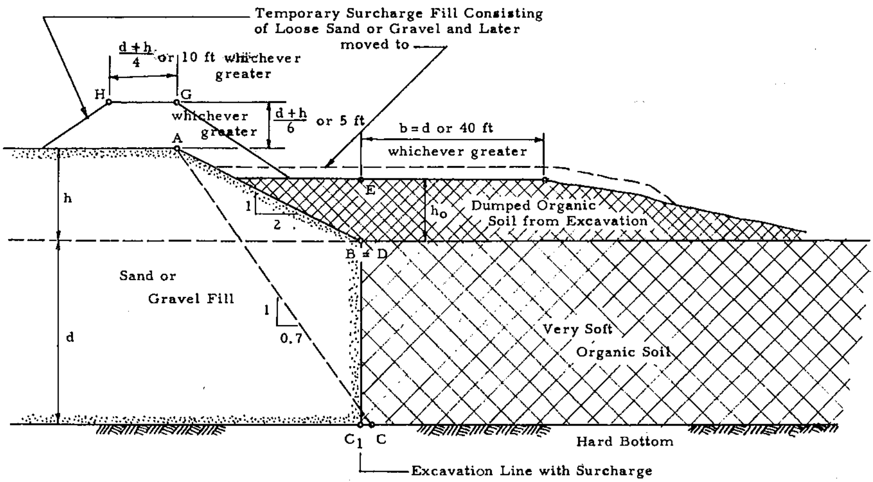
The design slope of the embankment should be 1 on 2. Simultaneously with the construction of the embankment above ground surface, an adjacent protective berm consisting of the excavated peat, should be constructed which should not lag too far behind the embankment in elevation. The minimum height, h_o, of this berm should be obtained from Fig. 25 as a function of the ratio h/d. The minimum width, b, of the berm should be equal to the depth, d, of the peaty stratum, or 40 ft, whichever is greater.

After completion of the design cross-section of the embankment the temporary surcharge is applied in the form of a dumped dike of a cross-section which is determined as follows:

(1) The outer edge of the crest of the dike, point G in Fig. 21, is located directly above the outer edge of the crest of the embankment.

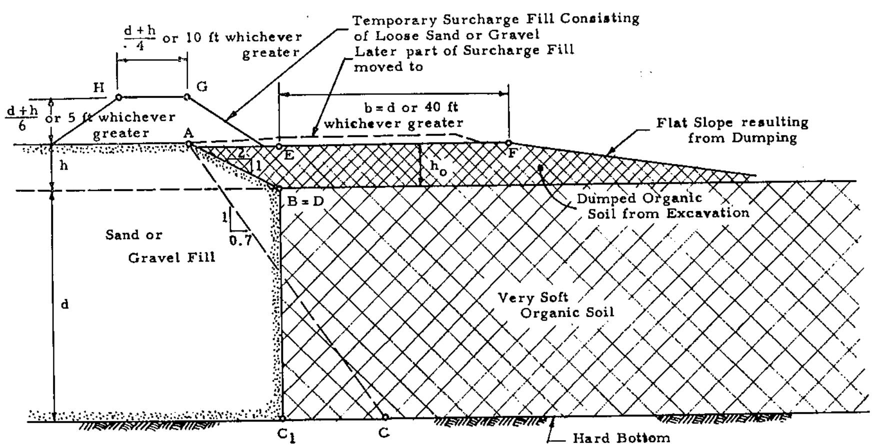
(2) The height AG should be equal to (d + h)/6, or 5 ft, whichever is greater.

(3) The crest width of the surcharge dike should be equal to (d + h)/4, or 10 ft, whichever is greater.



PROCEDURE—H/D = 0.5

FIG. 23.—FOR H/D = 0.5 REQUIRED MINIMUM H_0 = 0.67H



PROCEDURE—H/D = 0.2

FIG. 24.—FOR H/D = 0.2 REQUIRED MINIMUM H_0 = 0.9H

In general, the surcharge fill should be left in place for a period of several weeks, unless settlement observations indicate that the settlements are continuing at an excessive rate. After sufficient consolidation has been achieved, the surcharge fill should be spread out over the protective berm, as indicated by the dashed line IJ in Fig. 21.

The application of the above rules to various other ratios of h/d

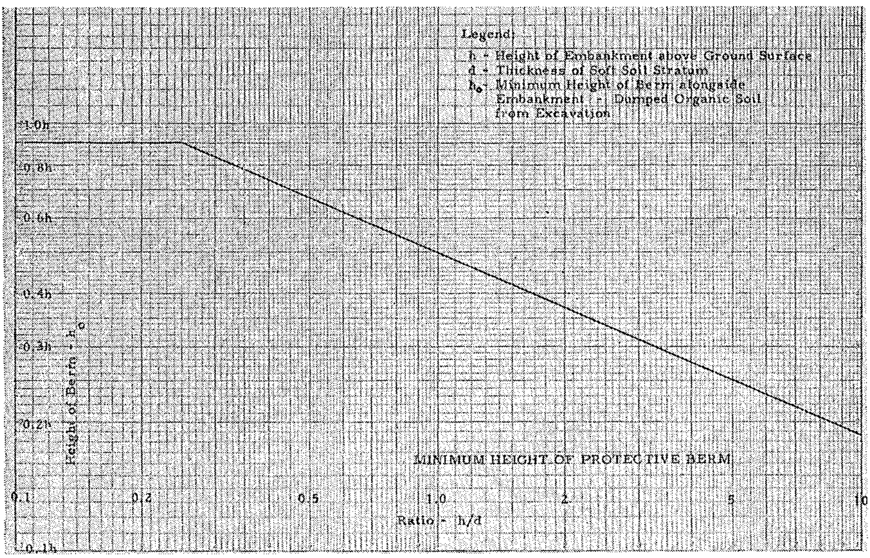


FIG. 25.—MINIMUM HEIGHT OF PROTECTIVE BERM

than those used in Figs. 20 and 21, are shown in Figs. 22, 23, and 24. It will be noted that in Figs. 23 and 24, i.e., for low embankments as compared to the depth of excavation, the width of excavation is governed by the vertical through the theoretical toe of the embankment, i.e., point B, and not point C.

When the thickness of the peaty stratum is small as compared to the height of the embankment, as, e.g., in Fig. 22, the percentage increase in the volume of fill by using Rule I as compared to Rule II, is so small that it will be more economical to use Rule I without the surcharge.

It is believed that the proposed Rules I and II are on the conservative side, and that on the basis of careful observations of embankments built in this manner, supplemented by the necessary tests of the soft materials, it may eventually be possible to modify these rules and permit smaller widths of excavation.

Explosives and Timing

For peat blasting operations the use of 40 to 60% gelatin dynamite has been standard practice. Small delays were commonly used both on this continent and in European countries for this type of work.

TABLE V

Thickness H of Peat Deposit	Distance D Between Rows of Charges	Spacing B Between Charges within a Row		
		Below Crest	Below Slopes	Outside Toes
<30 ft	0.75H	H	0.75H	0.5H
30 to 60 ft	0.75H	0.5H + 15 ft	0.5H + 7.5 ft	0.5H
>60 ft	45 ft	45 ft	38 ft	30 ft

TABLE VI

Thickness H of Peat Deposit—feet	Amount of Explosives W in. lb/charge Along Center Line of Fill	
	For Width of Crest <30 feet	For Width of Crest >30 feet
10	50	60
20	80	100
30	120	150
40	160	200
50	200	250
60	250	300

However, the author believes that the only benefit derived from delayed firing is a lessening of the shock set up by blasting large quantities of dynamite. Personal observations of blasting by the underfill method has convinced the author that the simultaneous firing of neighboring rows of charges without delay is more effective than the customary small delays between the blasting of outer and inner rows of charges.

On projects of peat blasting it is considered good practice to seek expert advice from explosives manufacturers. Such advice usually not only includes transport and storage, safety measures, installation of charges and the blasting operation itself, but also decisions regarding type of explosive, the spacing of individual charges, delays, etc. For large projects, test sections are strongly indicated. Excessive charges used in settling highway fills are not only wasteful but may result in decreased effectiveness of the blasting method.

Evaluation of the most important underfill blasting projects has led to tentative recommendations contained in Tables V and VI. Eventually improved versions of such recommendations will also have to take into account differences in the properties of the materials to be displaced.

New Types of Blasting Agents

Among the new types of explosives which have been developed in recent years, the most interesting one is a combination of ammonium nitrate and fuel oil. Although this blasting agent has so far not been used for peat blasting operations, it is believed that it would have important advantages for the underfill blasting procedure, for the following reasons:

(1) Slower reaction time, combined with development of appreciably greater volume of gas.
(2) Much greater safety in handling, in transport, and in storage.
(3) The cost is on an average 5 cents per pound as compared with 30 to 35 cents per pound for 40% gelatin dynamite.

Ammonium nitrate is very sensitive to moisture, but since preassembled charges can be sealed in water-tight containers, this should not be considered a serious problem. A second reservation could arise from the fact that for best results it requires adequate charge confinement. In applications of underfill blasting, this requirement may be sufficiently fulfilled by the combination of confining the blasting agent in containers, and also by the weight of an appreciable height of fill overburden.

Conclusions and Recommendations

The purpose of this section is to present the author's conclusions and recommendations for highway construction, for temporary road construction, for the construction of levees, and for future research.

1. *Construction of Highways Across Peaty Deposits*

The most efficient and most economical procedure for displacing soft, organic soil by sand, gravel or rockfill for the purpose of constructing a highway embankment, consists of the following steps:

(a) *Blast center strip*—A 5 to 10 ft wide center strip of the mat or crust of the peaty deposit is destroyed by blasting, as shown in Fig. 7a.

(b) *Dump initial fill*—A fill is built by end-dumping over the blasted strip with a crest width of 15 to 20 ft, and to such height that after the initial subsidence the crest will be about 3 ft above the adjacent ground surface, as shown in Fig. 7b.

(c) *Construct and settle by blasting the main fill*—The procedure will differ in detail depending on whether or not the explosive charges can be jetted through the fill material. The more desirable procedure is to dump the main fill first and then place the charges by jetting through the fill.

(1) *For sand or small gravel fill,* the initial fill (see Fig. 7c) should be widened and increased to the full design width and to a sufficient excess height so that the total volume will be about equal to the estimated requirement of fill volume after the blasting operation. For great thickness of the soft deposit or a narrow embankment it may not be possible to place the full estimated volume of fill within the desired width. Then the preassembled charges should be jetted through the fill down to the bottom of the soft stratum. Tables V and VI may serve as a basis for selecting the size and the layout of the charges. In general, the entire fill length should be blasted in one operation, as illustrated in Fig. 12. For large projects it is desirable to blast a test section at least 100 ft in length, principally for the purpose of checking the amount and distribution of explosives.

(2) *For coarse gravel and rockfill,* through which the charges cannot be jetted, it is necessary to install the charges before dumping of the main fill. The charges should be jetted on both sides and close to the initial fill, in a manner similar to that illustrated in Fig. 9a. The electric wires should be carried through plastic tubing well beyond the toe lines of the final fill. The main fill is then built and blasted as described in the preceding paragraph.

(d) *Explore shape of fill below ground surface and if necessary widen fill by supplementary blasting*—The exploration can be carried out quickly by means of jet probings either through the fill, or by slanting holes from both sides. For guidance in establishing a desired cross-section, Rules I and II as discussed in the section on REQUIRED WIDTH OF DISPLACEMENT, may be used. These rules are believed to be conservative. It is possible that a contact width of the fill along the firm bottom after blasting need not be greater than the crest width of the embankment.

2. *Construction of Temporary Roads Across Peaty Deposits*

The present empirical knowledge of construction of fills across peaty deposits is sufficient to suggest a procedure for the construction of a roadfill which would probably satisfy the following specifications: (1) Minimum volume of suitable fill material; (2) minimum time for construction; (3) safety against sliding; (4) settlements which can be readily repaired by constant maintenance operations. The author believes that these requirements can be fulfilled by using the following construction procedure:

(a) Blast a 5 to 10 ft wide center strip of the mat or crust of the peaty deposit, as shown in Fig. 7a.

(b) By end-dumping, build a fill with the desired crest width, but not more than 20 ft, and to a height such that after the initial subsidence the crest will be about 6 ft above the adjacent ground surface. If the desired crest width is more than 20 ft, the additional fill should be placed on both sides, and at the same time any deficiency in height which will develop during this operation, should be compensated by additional fill.

(c) Remove the top 3 ft of the fill by bulldozing it to both sides, creating shoulders with flat slopes which extend over the adjacent surface of the mat or crust of the organic deposit. The purpose of these flat shoulders is threefold:

(1) To increase the factor of safety of the main fill against sliding.

(2) To reduce the settlements of the main fill by creating counterweight berms.

(3) To form road shoulders which because of their flat slopes will provide a measure of safety against vehicles dropping off the narrow main fill, and which in addition can be used as emergency strips for disabled vehicles or light traffic. However, heavy vehicular traffic will have to be confined by appropriate markers to the crest of the main fill.

A road fill constructed in this manner will continue to settle and will require grading operations and occasional addition of fill.

A roadway so constructed would lend itself to further widening, or to the construction of a fill which is settled by blasting to firm bottom as described under a preceding heading.

For the major volume of the fill one can also use a variety of materials other than granular soils, such as stiff clay, clay shale, and badly weathered or disintegrated rocks. Such fill would finally have to be covered with a minimum thickness of a good quality granular fill, or with landing mats if granular soils are not available.

Certain procedures which have been used for settling fills through soft deposits for highway construction should not be used for temporary roads for which speed of construction and minimum volume of fill material are basic requirements. Particularly objectionable because of the danger of slides and because of waste of fill material are: (1) the breaking up of the surface mat or crust over the full base width of the embankment, and (2) the "relief method."

3. *Construction of Low Levees Across Peaty Deposits*

The author believes that for the construction of low levees across highly organic soil deposits, a procedure similar to that described above for the construction of temporary roads would be suitable and economical.

4. *Suggestions for Further Investigations*

Among the many topics which deserve investigation, the following are of particular interest:

(a) *The necessary width of contact of the fill with the firm bottom,* to achieve (1) positive protection against sliding, and (2) settlements of tolerable magnitude. It is believed that for many conditions it would be sufficient if this contact width is made equal to the crest width of the embankment. Ideally, this question would require systematic, full-scale experimentation. However, it is likely that considerable progress could be made by investigating the in situ stress-deformation characteristics of the soft deposits combined with theoretical studies of the stress conditions at the interface of fill and displaced mass.

(b) *The use of ammonium nitrate—fuel oil* as blasting agent for settling fills through peaty deposits would deserve investigation because of several potential advantages as discussed under a preceding heading. However, the only satisfactory approach to this question would be by full-scale experimentation in close cooperation with the manufacturers of this type of blasting agent.

(c) *Installation of charges before dumping of main fill* has important advantages as compared to the placement of charges by jetting through the fill. Experience in Germany with this procedure has been very encouraging. It is much faster, simpler and cheaper, and can be used for all types of fill materials including rock fill. The only objection which has been raised against this procedure is that the electric wires may be torn by the mass displacements. On the projects on which it was used, there has been no evidence that any significant number of the charges did not fire in spite of the fact that no special efforts were made in stringing out the wires to include extra slack, or to provide such slack by spiralled wires. For a systematic investigation of the installation of the effectiveness of charges installed ahead of the filling, it is recommended that an actual peat crossing project be utilized. Dummy charges should be installed ahead of the filling, in every respect as the real charges, but with the wires connected within the dummy. After the filling operation, the integrity of the wires should be checked. Various types of wires and methods for providing slack should be investigated in this manner.

(d) *The most desirable combination of details for construction of temporary roads on peaty deposits* deserves full-scale experimental research. The procedure recommended above would form a basis for the start of such an investigation. Certainly the type of organic deposit, the thickness and strength of the mat or crust, and the character of the fill material all are important variables. A systematic investigation of these variables by full-scale experimentation would yield invaluable guiding rules for such construction operations.

Acknowledgments

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STRUCTURAL METALS—UNDERSTANDING THROUGH MATERIALS SCIENCE

By Russel C. Jones*

The micro and macro levels of materials behavior are being increasingly tied together by understanding growing out of materials science. In this discussion the elasticity and strengthening mechanisms of structural metals are shown to be areas where strong bridges between micro and macro level behavior can be made. The structural steels, including the newer high strength steels, are discussed in terms of this materials science understanding. Possibilities for the control of the elastic moduli of structural metals are explored.

Introduction

In recent years the materials science approach has led to a more basic understanding of the principles underlying the behavior of materials of interest to civil engineers. With this information supplementing the more traditional materials testing approach, the micro and macro levels of materials behavior have been related to various degrees in all structural materials. The understanding of the relation between mechanical properties and the detailed structure and composition of materials has led to innovations in design and the development of improved and new materials. Control of structure and composition at the micro level can result in the improvement of properties at the macro level.

Civil Engineering materials can be grouped into three classes, based on similarities at the micro level—ceramics, organics and metals. Ceramic materials, which have complex atomic level structures and are often amorphous (non-crystalline), include the glasses, ceramics, and Portland cement concrete. The organic materials, such as wood, plastics, rubbers and asphalts, are characterized by long chain molecules at the micro level. Metals, such as steels and aluminums, are characterized by the regular crystalline arrangement of their atoms, which are held together by a particular kind of bonding.

The following discussion will be limited to structural metals, using them as examples of how properties at the macro level can be related to micro level composition and structure.

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Elasticity

The elasticity of metals is an example of a relatively easy bridge between the macro and micro scales. Consider the case of two atoms, attracted to each other by electron sharing but repelled from each

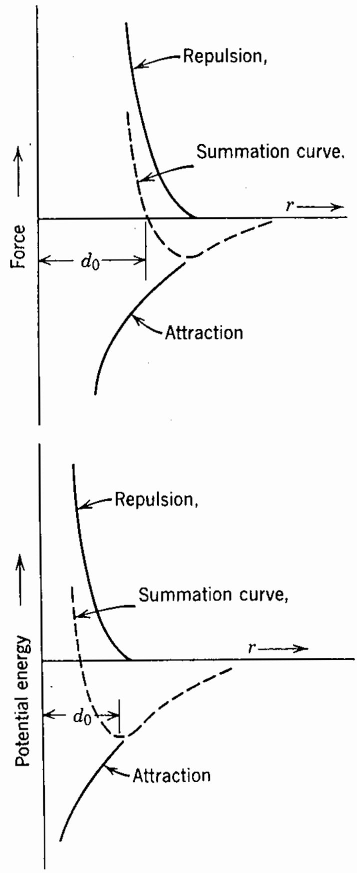


Fig. 1.—Qualitative Curves Showing (a) Force and (b) Potential Energy as Functions of Interatomic Spacing, r

other as they approach closely enough to begin to overlap in space. Fig. 1a shows schematically how the attractive and repulsive forces combine to produce a stable situation where the atoms have zero force between them at an equilibrium interatomic spacing. The equilibrium spacing, d_0, is shown in Fig. 1b to correspond with the minimum of potential energy. since the equilibrium position is one of minimum energy, this spacing between the atoms will be stable. If a disturbing force were to cause a small displacement of the atoms from the equilibrium spacing, they would return to the stable energy minimum upon removal of the force.

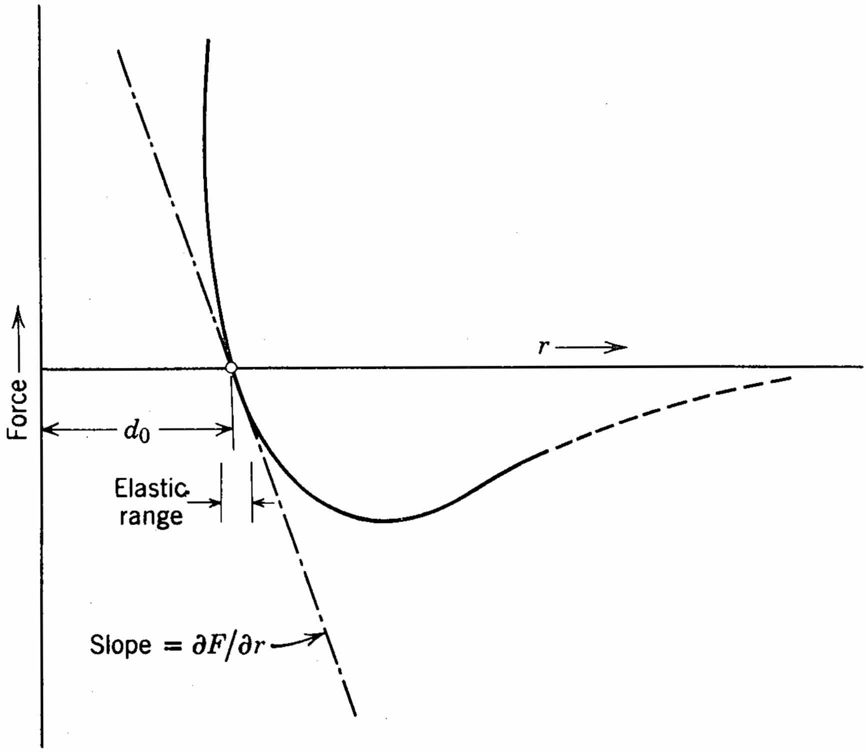


FIG. 2.—SUMMATION CURVE FOR FORCE VS. INTERATOMIC SPACING IS NEARLY LINEAR FOR SMALL DISPLACEMENTS FROM THE EQUILIBRIUM SPACING, GIVING RISE TO MACRO LEVEL LINEAR ELASTICITY

The same kinds of curves, drawn in Fig. 1 for two atoms, apply to large aggregations of atoms. Thus, in crystalline groupings of atoms the individual atoms are regularly spaced at an equilibrium distance from each other. The equilibrium interatomic spacing in this more general case is determined by the minimum of potential energy of the entire system consisting of many atoms.

In many materials of engineering importance the force-interatomic spacing curve is nearly linear in the vicinity of the equilibrium spacing. This direct proportionality of force and deformation for small displacements gives rise to the modulus of elasticity on the macro scale. Fig. 2 shows how the force summation curve can be approximated by a

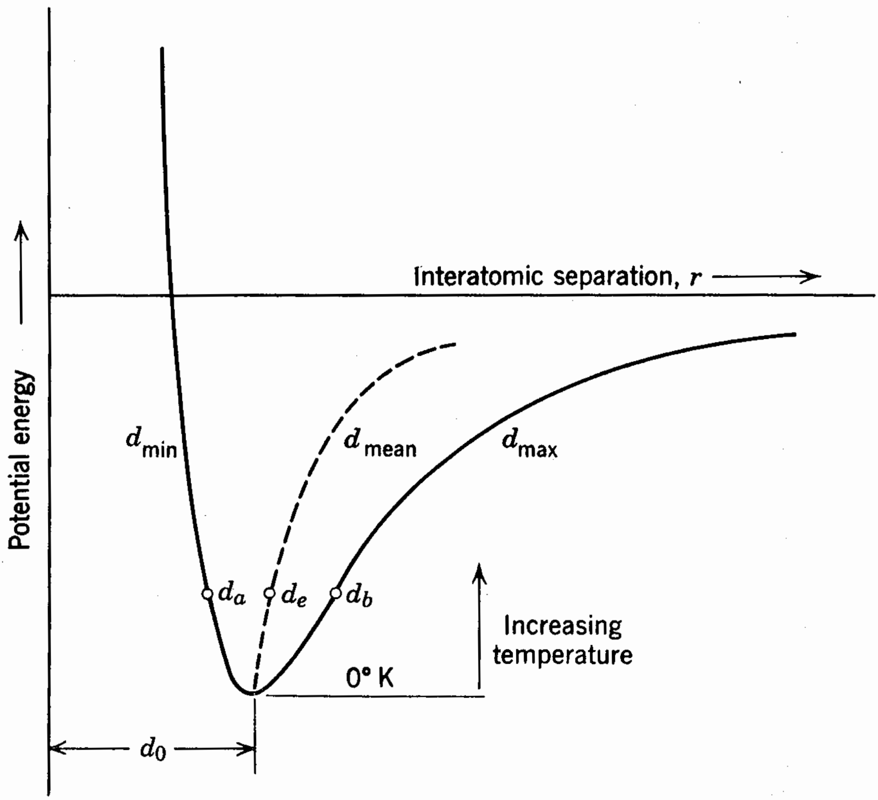


FIG. 3.—CHANGE IN INTERATOMIC SPACING WITH INCREASING TEMPERATURE; ASYMMETRY OF POTENTIAL ENERGY CURVE GIVES RISE TO MACRO LEVEL THERMAL EXPANSION (AFTER WULFF)

straight line at the atomic level. The integration of this force-displacement relationship over the large number of atoms present in a crystalline matrix results in the elastic modulus observed in conventional materials tests.

Having related atomic level potential energy relationships to the mechanical response of materials, it is easy to extend these concepts to the thermal response. At temperatures above 0° K, thermal energy causes atoms to vibrate about their equilibrium positions. As shown in Fig. 3, the atoms oscillate between the minimum and maximum interatomic spacings possible at any given energy level. The mean position represents the average interatomic spacing at a given temperature. Because the potential energy curve is asymmetrical, the mean position increases as the temperature increases. This increase in average interatomic spacing at the atomic level corresponds with the thermal expansion observed at the macro level. If the energy is increased by thermal

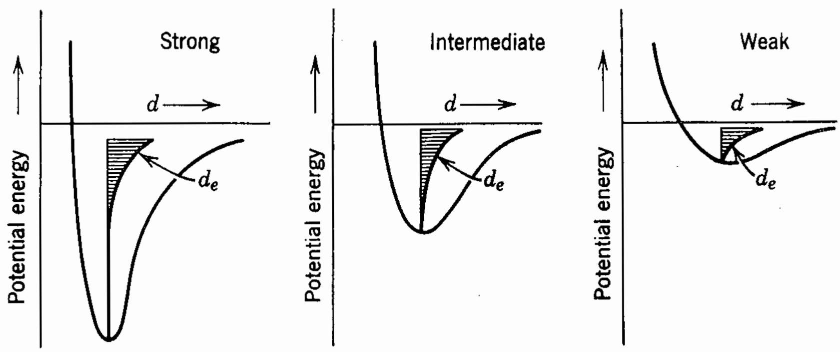


Fig. 4.—Relation of Elastic Modulus, Coefficient of Thermal Expansion and Melting Points to Each Other as a Function of Atomic Bonding Strength (After Wulff)

addition until the depth of the potential energy well is reduced to zero, the material melts.

The elastic modulus, coefficient of thermal expansion and melting point can thus be seen to be related to each other, since each is based upon potential energy-interatomic spacing relationships at the micro level. Modulus of elasticity is inversely proportional to the radius of curvature at the bottom of the potential energy well. The melting point is directly proportional to the depth of the well. The coefficient of thermal expansion is proportional to the degree of asymmetry of the energy curve. The way in which these three quantities vary with the relative strength of atomic bonding is shown in Fig. 4. A strongly bonded material has high modulus of elasticity and high melting point, with a low coefficient of thermal expansion. A weakly bonded material has relatively low values of modulus and melting point, with a high thermal expansion coefficient.

Strength and Fracture

The strength of materials, observed on the macro scale, can also be related to micro level structure and composition. Early calculations for the strength of a crystalline material considered the relative shear of two planes of atoms, as shown in Fig. 5. Taking atomic bonding

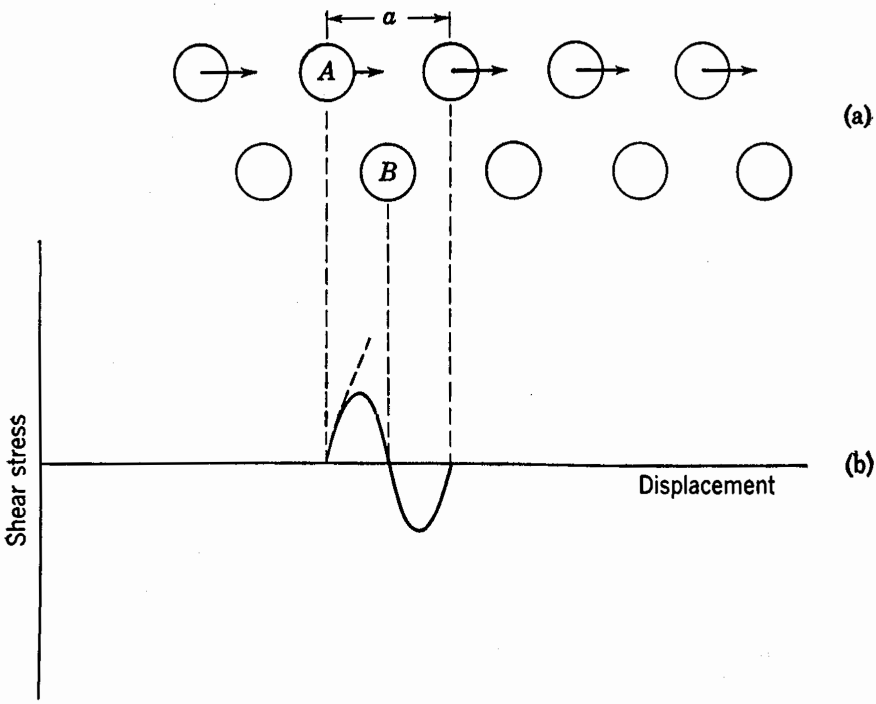


Fig. 5.—Relative Shear Between Planes of Atoms: (a) Schematic Representation of Two Planes of Atoms, (b) Shear Stress as a Function of Displacement of the Atoms From Their Equilibrium Positions

forces into consideration, theoretical strength was calculated as that stress required to permanently displace a whole plane of atoms from their equilibrium positions. Using this block-slip model, the elastic limit, B, is found to be approximately one-sixth the shear modulus, G, or $G/B \approx 6$. This theoretical value can be compared with actual observed G/B ratios in Fig. 6. An obvious discrepancy exists, with real materials having much lower strengths than those predicted by the block-slip atomic bonding model.

The dislocation model of slip in a crystal, introduced in the 1930's, resolves the discrepancy between calculated and observed strengths. In this model slip occurs in a stepwise manner caused by the presence of atomic scale imperfections, called dislocations, in the crystalline matrix. An edge dislocation can be visualized as an extra half-plane of atoms in a crystal. Dislocations are grown into crystals as accidents of growth, and multiply as the crystalline material is stressed. As shown schematically in Fig. 7, the dislocation moves under an applied stress by successively breaking and making new bonds with the extra half plane

	Shear Modulus G (dynes/cm^2)	Elastic Limit B (dynes/cm^2)	G/B
Sn, single crystal	1.9×10^{11}	1.3×10^{7}	15,000
Ag, single crystal	2.8×10^{11}	6×10^{6}	45,000
Al, single crystal	2.5×10^{11}	4×10^{6}	60,000
Al, pure, polycrystal	2.5×10^{11}	2.6×10^{8}	900
Al, commercial drawn	$\sim 2.5 \times 10^{11}$	9.9×10^{8}	250
Duralumin	$\sim 2.5 \times 10^{11}$	3.6×10^{9}	70
Fe, soft, polycrystal	7.7×10^{11}	1.5×10^{9}	500
Heat-treated carbon steel	$\sim 8 \times 10^{11}$	6.5×10^{9}	120
Nickel-chrome steel	$\sim 8 \times 10^{11}$	1.2×10^{10}	65

FIG. 6.—COMPARISON OF SHEAR MODULUS AND ELASTIC LIMIT (AFTER MOTT)

in a stepwise manner. An analogy can be drawn between this process and the moving of a floor rug: instead of pulling on one edge of a rug to move it a distance horizontally, one can effect a similar translation by making a hump in the edge of the rug opposite the desired direction of movement then running the hump across the rug. The stepwise movement takes less force, but finally achieves the same result. This is true in the crystal also, as show in Fig. 7b. The permanent deformation resulting from the passage of a dislocation through the crystal is similar to that which would result from the block slip of two portions of the crystal, but the required force for the former mechanism is considerably less.

The detailed strain field around an edge dislocation is shown schematically in Fig. 8. A region of tension exists on one side of the dislocation, with a region of compression on the other side where the extra atomic half-plane distorts the lattice. Foreign atoms of sizes different from the host matrix atoms tend to migrate to dislocation sites, replacing or fitting between the host atoms to partially relieve these strain fields. This results in a decrease in strain energy, and a dislocation with such an atmosphere of foreign atoms tends to be locked against movement. The stress required to move a locked dislocation would be higher than that required to move a dislocation in a pure matrix, since the energy to move the dislocation out of its lower energy

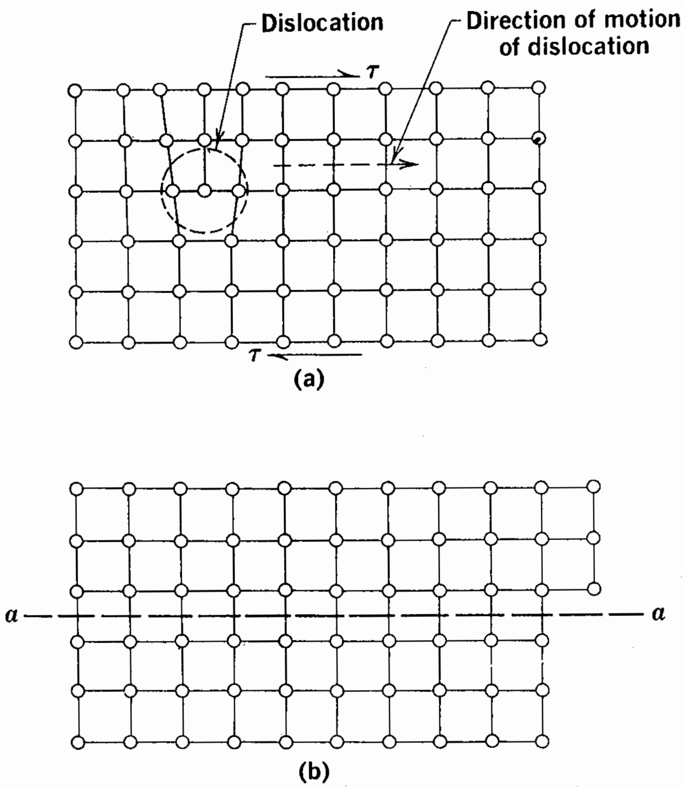


FIG. 7.—MOVEMENT OF AN EDGE DISLOCATION THROUGH A CRYSTAL, CAUSING SLIP

atmosphere and back into the pure matrix would have to be supplied. Once the dislocation has been broken from its atmosphere it can be kept in motion through the matrix by a somewhat lower applied stress. The upper and lower yield stress phenomenon in carbon steels is a macro level reflection of the dislocation unlocking phenomenon.

Once moving in a small single crystal, a dislocation travels to the edge of the crystal and pops out as plastic deformation. This is illustrated in Fig. 9 for the movement of three types of dislocations. A pure screw dislocation in a shear-type imperfection often found in crystals. A general dislocation consists of components of the pure edge dislocation described above and the pure screw dislocation. As shown in Fig. 9, the movement of any of these types of dislocations results in permanent deformation at the edge of the crystal.

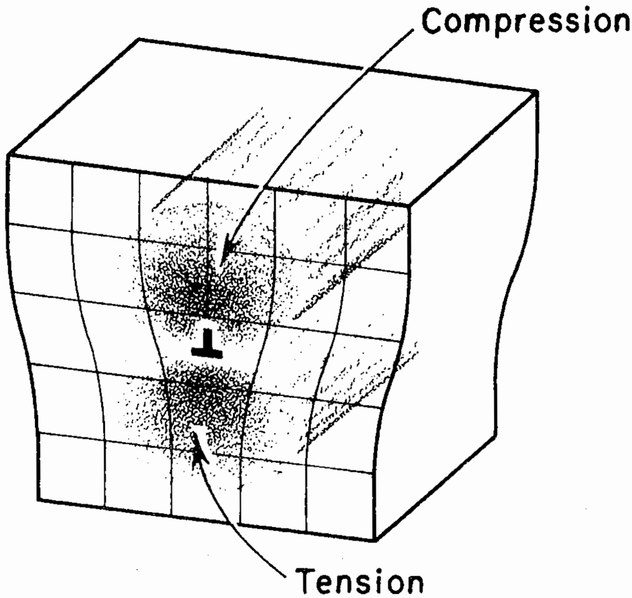


FIG. 8.—COMPRESSIVE AND TENSILE STRAIN FIELDS AROUND AN EDGE DISLOCATION

In polycrystalline metals, however, dislocations are not usually able to get a crystal surface and instead tend to pile up grain boundaries within the polycrystal. Fig. 10 shows how edge dislocations, generated by a multiplication source, pile up at an obstacle such as a grain boundary. Gross yielding of the material occurs when dislocations pop through the grain boundary, affecting adjacent grains.

Dislocation interactions can also inhibit the movement of dislocations through the matrix. The intersection of two dislocations can result in a jog in the dislocation line, as shown in Fig. 11. In certain situations such a jog anchors the dislocation, inhibiting further motion. The interaction of a dislocation with second-phase particles lying in the slip plane can also inhibit movement of the dislocation under a given stress. Fig. 12 illustrates how a dislocation line must be forced past such second-phase particles, requiring a higher stress than that required for motion through a pure matrix. These dislocation interaction mechanisms, fully operative in the plastic range of loading where large scale movements of dislocations occur, lead to work hardening of

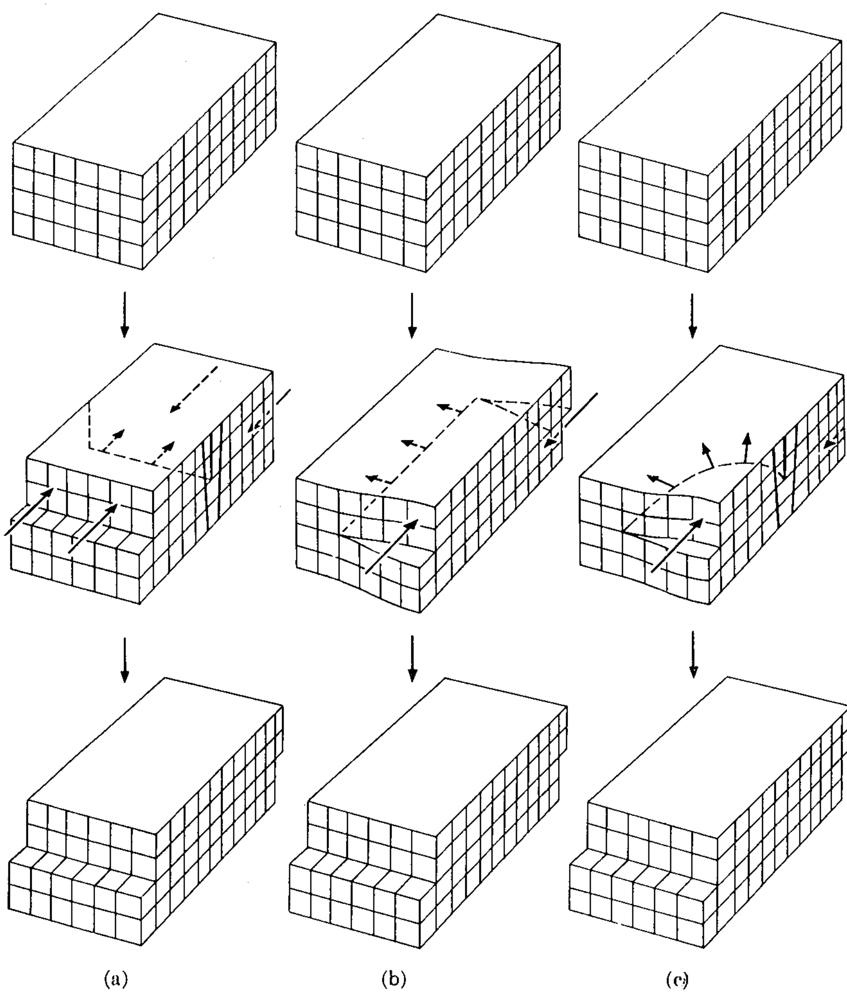


FIG. 9.—PLASTIC DEFORMATION CAUSED BY THE MOVEMENT OF (A) PURE EDGE, (B) PURE SCREW, AND (C) MIXED DISLOCATIONS THROUGH A CRYSTAL

the material. Thus in general, an increased stress is required to cause continued deformation in the plastic range.

Dislocation interactions can also lead to crack formation, thus contributing to the fracture of materials. Fig. 13 shows how dislocations can combine to open a crack at the intersection of slip bands. The extra half-planes of these edge dislocations lead to a large tensile buildup which results in localized crack formation.

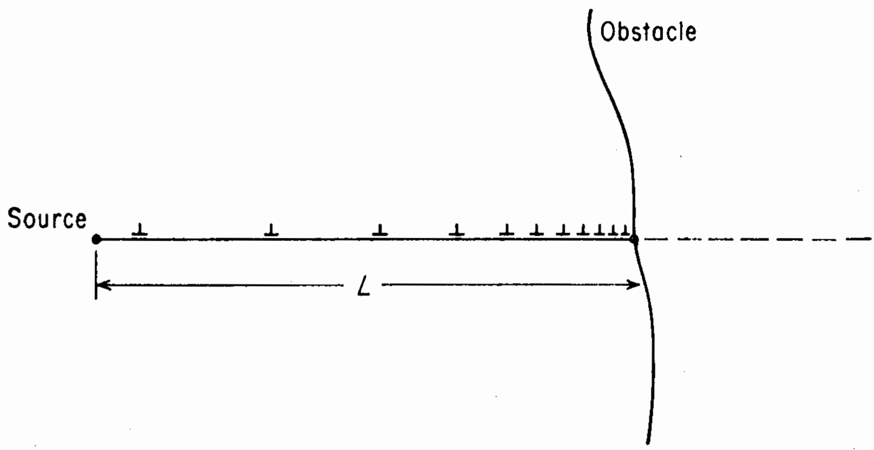


FIG. 10.—PILE-UP OF DISLOCATIONS AT AN OBSTACLE

DISLOCATION OBSERVATIONS

Although dislocations are an atomic scale phenomenon, evidence of their existence can be observed using microscopy techniques. Surface traces of dislocations can be optically observed after being marked by etch pitting, and bulk material observations of dislocations can be made using thin foil electron transmission microscopy.

Dislocation lines cannot end inside a crystal, so they either form closed loops or run out at a surface of the crystal. As an edge dislocation line cuts a surface, it is still surrounded by the localized tensile and compressive stresses which are caused by the presence of the extra half-plane of atoms. Fig. 8 is a schematic view of the atomic arrangement in a surface layer of atoms as an edge dislocation cuts through it. There is a finite strain energy stored in the portion of the crystal immediately surrounding the dislocation, due to the atomic misfit. When the entire surface is attacked by a strong etching solution, preferential etching occurs at the sites of dislocation due to the higher energy states of the atoms there. An example of such etch pits, these in lithium fluoride, is shown in Fig. 14a. The arrangements of etch pits in a line through the center of this micrograph is a typical configuration called

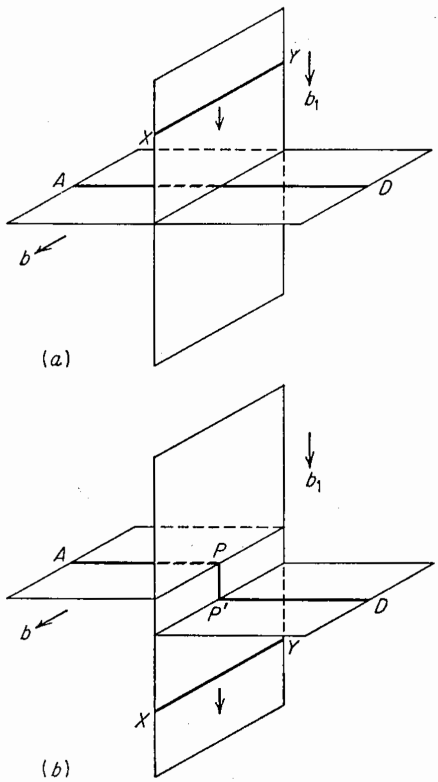


FIG. 11.—INTERSECTION OF TWO DISLOCATIONS, RESULTING IN THE FORMATION OF A DISLOCATION JOG (AFTER READ)

a low-angle grain boundary. The corresponding schematic representation of a low-angle grain boundary in Fig. 14b shows how a small change in crystal orientation occurs across such an arrangement of edge dislocations. The usual grain boundary observed at low magnification after metallographic etching is a large-angle orientation change which can be pictured as a complex arrangement of many more dislocations.

In order to observe dislocation arrangements in the bulk of the material, a thin section specimen is carefully cut from it. The section is cut in such a way that little mechanical damage is incurred by the specimen, then is thinned to 10^{-5} to 10^{-6}ch thickness by chemical

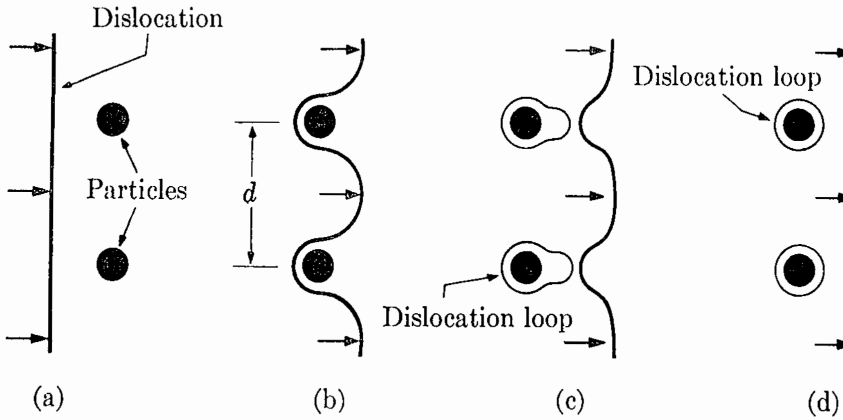


FIG. 12.—INTERACTION OF A DISLOCATION WITH SECOND-PHASE PARTICLES IN ITS SLIP PLANE

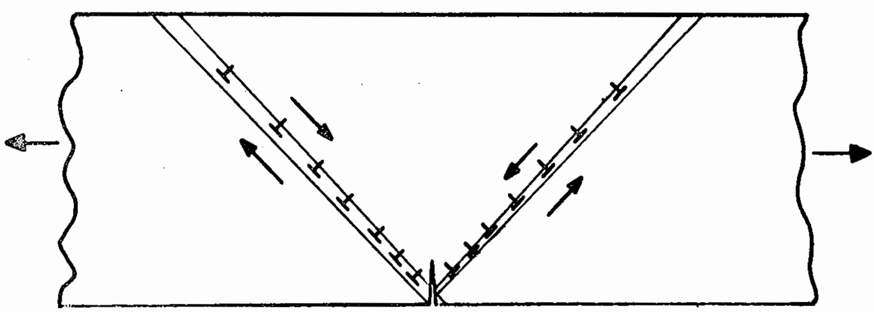
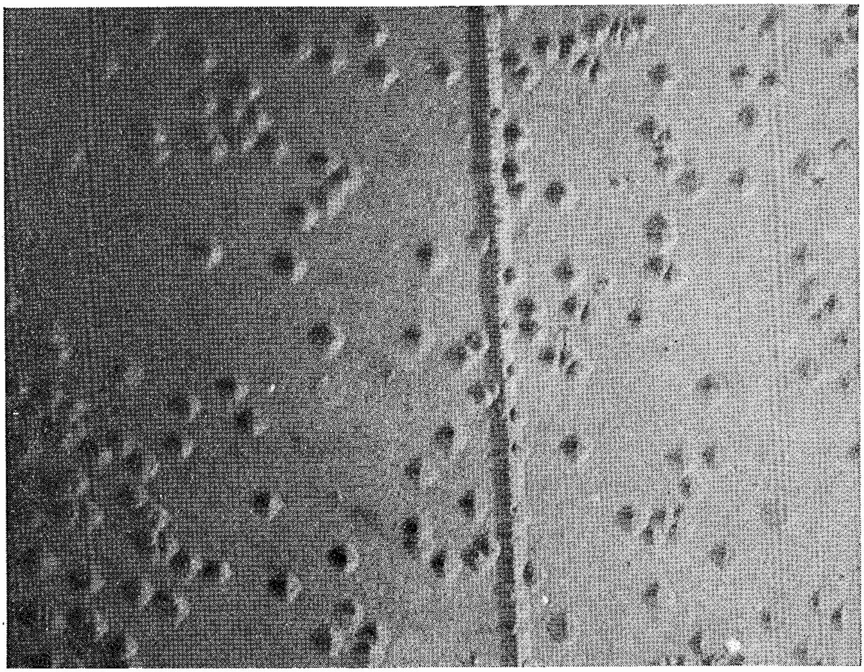


FIG. 13.—FORMATION OF A CRACK DUE TO THE COMBINATION OF EDGE DISLOCATION SLIP BANDS

polishing. In this form electrons accelerated through a high potential can be transmitted through the thin foil in a conventional electron microscope. Electrons hitting the areas of crystalline imperfection surrounding dislocations are diffracted out of the electron beam, so dislocations show up as dark lines on the fluorescent screen struck by the



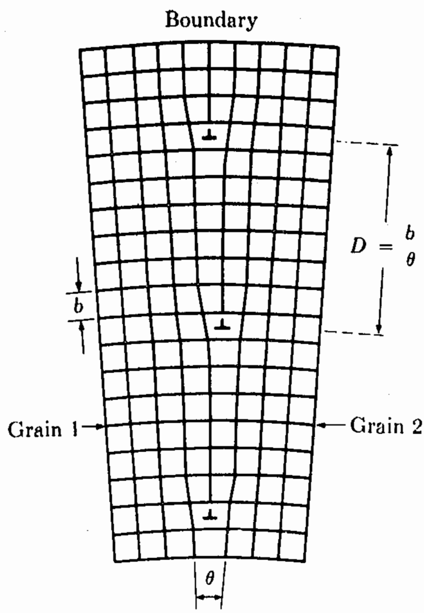


FIG. 14.—(A) ETCHED SURFACE OF LITHIUM FLUORIDE SHOWING ETCH PIT PATTERN CORRESPONDING TO LOW ANGLE GRAIN BOUNDARY. (B) SCHEMATIC REPRESENTATION OF LOW ANGLE GRAIN BOUNDARY OF EDGE DISLOCATIONS

undisturbed transmitted electrons. Fig. 15 shows dislocation lines in a thin foil of aluminum which has been prepared from a cyclicly loaded specimen.

The observation of dislocations by such techniques as etch pitting and thin foil transmission electron microscopy allows checks to be made on atomic scale models employed to explain the mechanical behavior of crystalline materials.

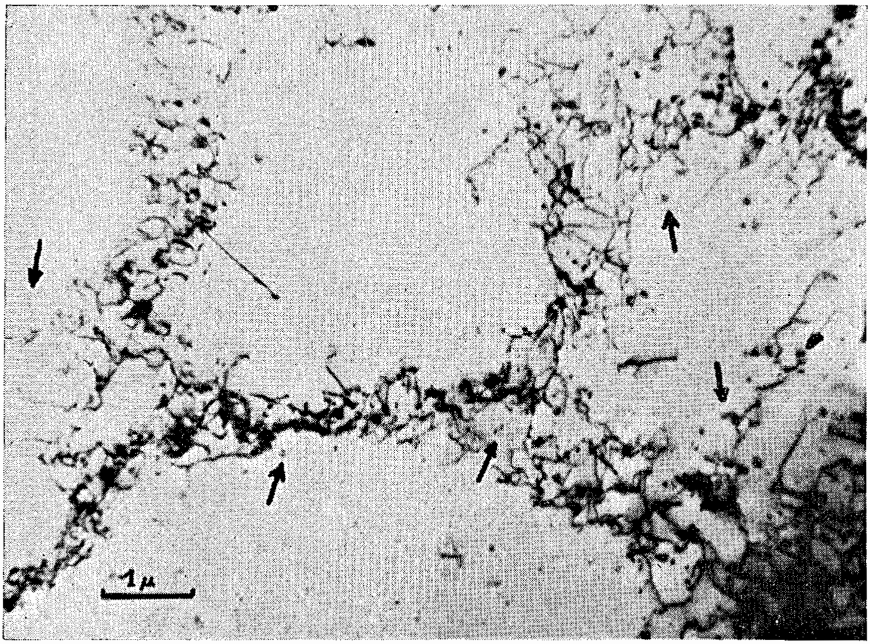


FIG. 15.—DISLOCATION STRUCTURE IN THE BULK OF AN ALUMINUM SPECIMEN, REVEALED BY THIN FILM ELECTRON MICROSCOPY TECHNIQUES. (FROM R. N. WILSON AND P. J. E. FORSYTH, ROYAL AIRCRAFT ESTABLISHMENT TECHNICAL NOTE MET, 311, JUNE, 1959)

STRESS-STRAIN CURVE

With the above introduction to dislocation theory, it is now possible to relate the micro and macro levels through the several stages of a full-range stress-strain diagram which represents the behavior of a metal subjected to a one-time loading to failure. In the stress-strain curve shown in Fig. 16, typical for a mild steel in tension, the micro-macro tie can be made at four stages: the elastic region, the yield point, the work hardening range, and fracture.

The macro level in the elastic portion of the curve is characterized by a linear relation between stress and strain and by fully recoverable deformation. This elastic behavior is based upon small changes in inter-atomic spacing with force at the micro level, as shown in Fig. 2. As long as the atoms return to their original equilibrium positions when

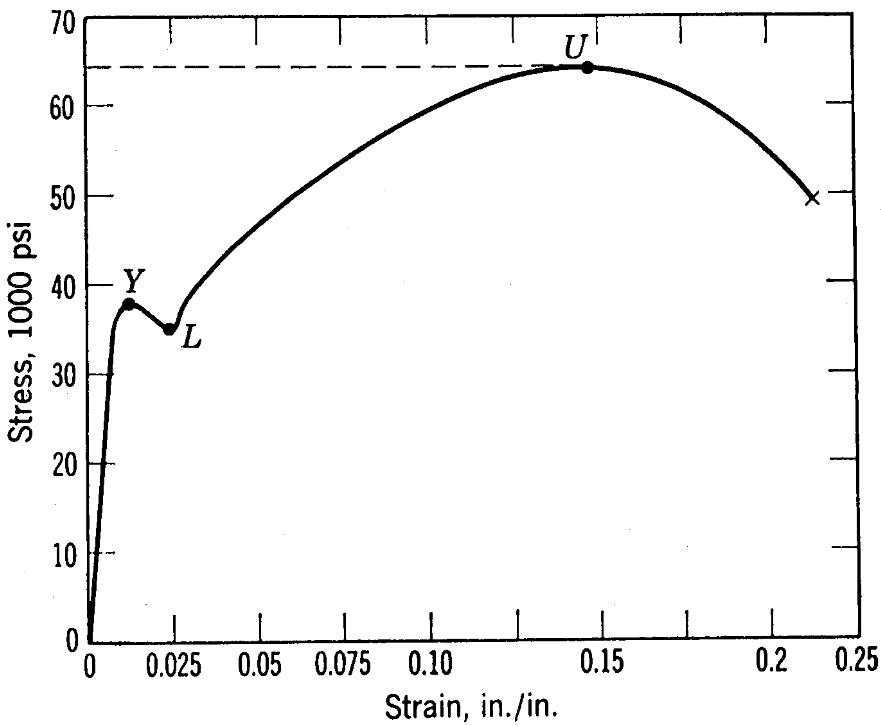


FIG. 16.—FULL RANGE STRESS-STRAIN DIAGRAM FOR A MILD STEEL LOADED TO FAILURE IN TENSION

the applied stress is removed, reversible elasticity is observed at the macro level.

When the stress is increased beyond the elastic limit of the material, plastic flow or yield occurs. At the macro yield point, dislocations at the micro level have begun to move across slip planes and pop through the grain boundaries. The summation of such atomic level rearrangements results in the plastic strain associated with post-yield behavior. In the particular case of low carbon steel, the yield phenomenon consists of an upper and a lower yield point. These correspond with the initial breaking away of dislocations from their surrounding locking atmospheres of solute atoms and the subsequent easier continued movement of the dislocations across the slip planes.

As the dislocations continue to be moved after the yield point, dislocation multiplication and intersections begin to make it more difficult for continued movement. This increased stress required for continued plastic straining appears as work hardening at the macro level. The work hardening phenomenon continues right up to fracture, and results in increased hardness and strength of the material. On a true stress-strain curve the stress would continue to rise until fracture occurred, but on the engineering curve shown in Fig. 16 nominal stress decreases as the material decreases in area due to localized necking.

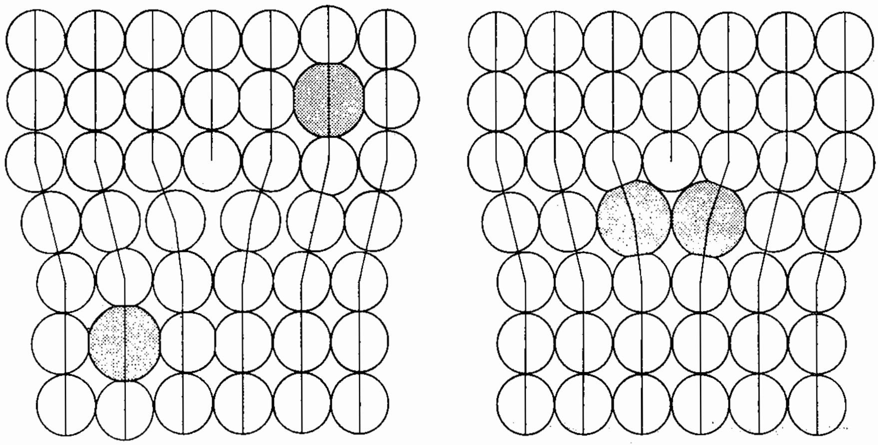
Microcracks formed at the head of dislocation pileups and at other stress concentrators begin to grow at high stresses. Such cracks eventually join together and grow to a critical size for rapid propagation to macro level fracture.

Strengthening Mechanisms

Several strengthening mechanisms are employed to raise the strength levels of structural metals. These mechanisms—solid solution alloying, precipitation hardening, grain size control, and work hardening—can each be understood in terms of dislocation mechanics.

Solid solution alloying begins with the random distribution of relatively small numbers of alloying element atoms throughout the metal matrix. Since it represents a lower strain energy state than does the random solution, these solute atoms regroup to form atmospheres around the dislocations present. This formation of an atmosphere is shown schematically in Fig. 17 where the two large solute atoms shown are only representative of the large number that may constitute the atmosphere. Once anchored by an atmosphere of solute atoms a dislocation requires a higher stress to move it, so the presence of the alloying element atoms has the effect of strengthening the host metal matrix.

If the alloying element atoms are formed into small second phase particles throughout the metal matrix instead of being in solution, a different kind of strengthening occurs. Second phase particles can be dispersed through the host matrix by a process known as precipitation hardening. The metallic alloy is first raised to a temperature high enough for the alloying element to be distributed randomly, then the material is quenched to a low temperature to lock in this random arrangement. The metal is then raised to a moderate temperature just high enough to give diffusional mobility to the alloying atoms. These



(a) (b)

FIG. 17.—FORMATION OF A LOCKING ATMOSPHERE OF SOLUTE ATOMS AROUND AN EDGE DISLOCATION

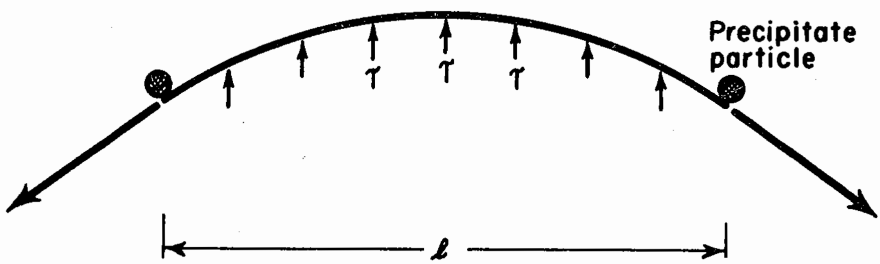
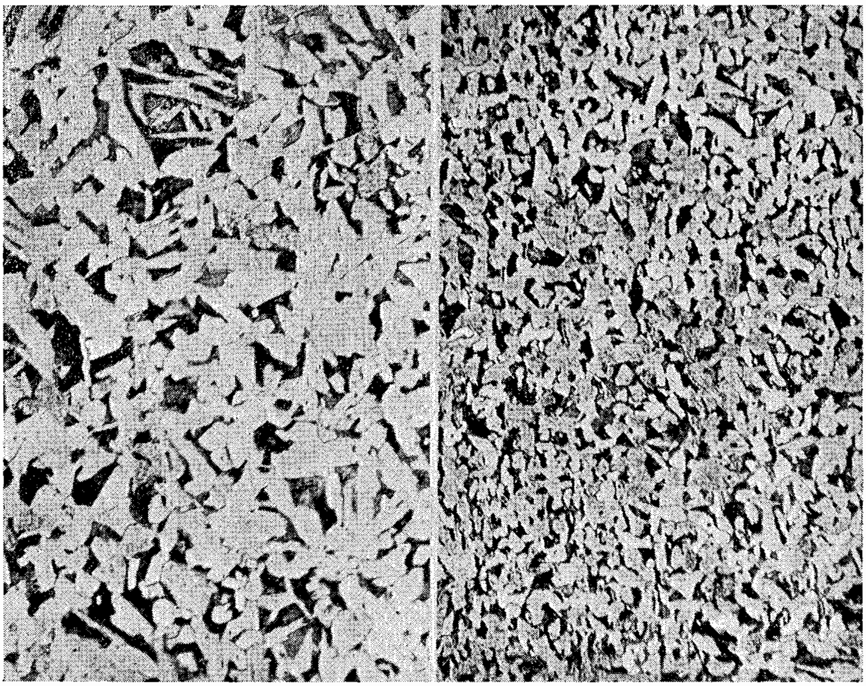


FIG. 18.—BLOCKING OF DISLOCATION MOVEMENT BY CLOSELY SPACED PRECIPITATE PARTICLES

atoms, since they are less soluble in the host matrix at relatively low temperatures, precipitate out as a second phase. If the heat treatment is properly programmed, many small precipitates are formed at close spacings. As shown in Fig. 18, such precipitate particles serve to restrain the movement of dislocations which are being driven by an applied stress. The stress required to force a dislocation past precipitate particles at a spacing l is proportional to $1/l$, so the more finely spaced the precipitate particles the more the metal is strengthened. It is important to note that a precipitation hardened alloy subjected to a high temperature, such as occurs locally in welding, can loose its strength as small



a.) b.)

FIG. 19.—MICROSTRUCTURE OF STRUCTURAL STEELS AT 100 ×: (A) ASTM-A36 STEEL, (B) A SIMILAR STEEL WITH SMALL AVERAGE GRAIN SIZE

precipitate particles merge into fewer large particles at greater spacings.

Grain boundaries, the discontinuities between crystalline regions of different orientations in polycrystalline metals, are also effective in inhibiting dislocation movements. Fig. 19a shows a typical micrograph of an A36 steel, etched and shown at 100× magnification to reveal the grain structure. Polycrystalline metals with a small average grain size exhibit higher strength than those with larger grain size, with strength being proportional to $d^{-1/2}$, where "d" is the average grain diameter.

Thus, the steel shown in Fig. 19b with a smaller average grain size than that in Fig. 19a, would show higher strength than the virgin A36 steel.

If a metal is strained well past the yield point and into the plastic range, dislocation interactions result in work hardening of the material. If it were unloaded from a point on the work hardening portion of the stress-strain curve, the metal would show a yield strength approximately equal to the stress from which it had been unloaded, if it were then loaded a second time. Thus, work hardening can be employed to strengthen metals, using dislocation interactions to inhibit plastic deformation due to dislocation movement.

These several strengthening mechanisms have in common the inhibiting of dislocation movements under an applied stress. Each of these mechanisms, and often combinations of them, are used to strengthen various commercially available structural metals.

High Strength Steels

The strength levels of structural steels have been increasingly raised in the past couple of decades as the strengthening mechanisms described above have become more understood and more used. The stress-strain curves sketched in Fig. 20 show the wide range of structural steels commercially available today.

The lowest curve shown in Fig. 20 represents ASTM A36, the carbon steel which is used in most standard structural applications today. In this steel carbon is the main alloying element, and its presence and subsequent dislocation locking develop the observed strength levels. No special heat treatment is applied, but microstructure and grain size are roughly controlled in the hot rolling process where the material is formed into structural shapes.

The second curve represents the high strength, low alloy steels. These somewhat higher strength steels, with yield strengths in the 50,000 psi range, were added to the AISC building code in 1961 to give an option to the standard carbon steels. In the past few years these steels have been heavily used in buildings and other constructed facilities. The high strength, low alloy steels derive their strength levels from the addition of other alloying elements to the usual carbon. These alloying elements produce a solid solution strengthening of the iron matrix. The third curve in Fig. 20 represents a similar class of steels, the columbium-vanadium bearing steels. These steels employ small percentages of columbium and/or vanadium as alloying elements.

As in the case of the structural carbon steels, neither the high strength, low alloy steels nor the columbium-vanadium bearing steels are given a special heat treatment in their production.

In order to raise strength levels higher, structural steel producers

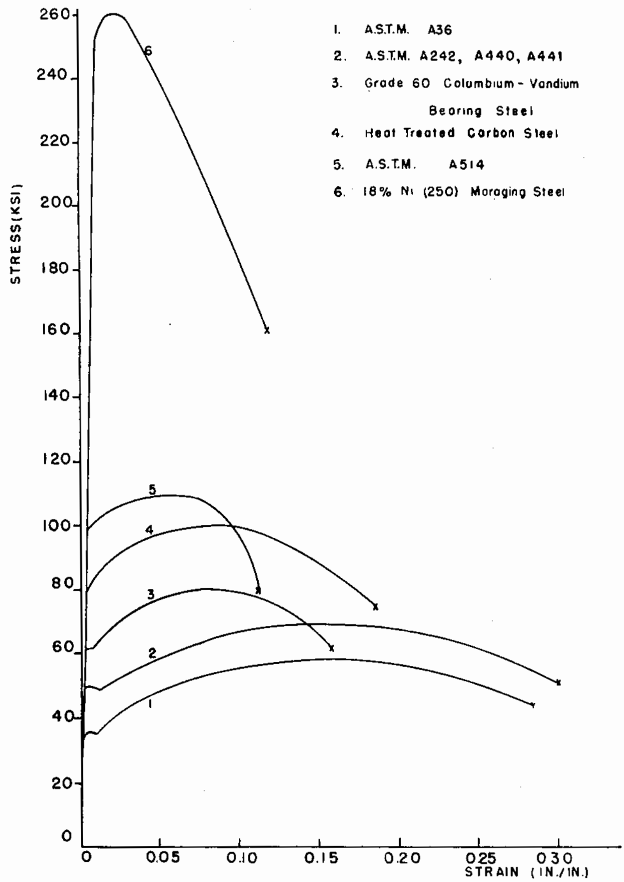


FIG. 20.—TYPICAL STRESS-STRAIN CURVES FOR SEVERAL COMMERCIALLY AVAILABLE STRUCTURAL STEELS

have devised heat treatments which will produce precipitation hardening of the basic iron matrix. The fourth curve in Fig. 20 represents a heat-treated carbon steel, produced by *quenching* the steel from a high temperature then *tempering* it at a moderate raised temperature to allow precipitate particles to form. The fifth curve shows a heat-treated constructional alloy steel, produced by quenching and tempering a steel to which both carbon and other alloying elements have been added. The heat-treated constructional alloy steels, available in the 100,000 psi strength range since the early 1950's, have recently been designated ASTM A514. Both the heat-treated carbon steels and

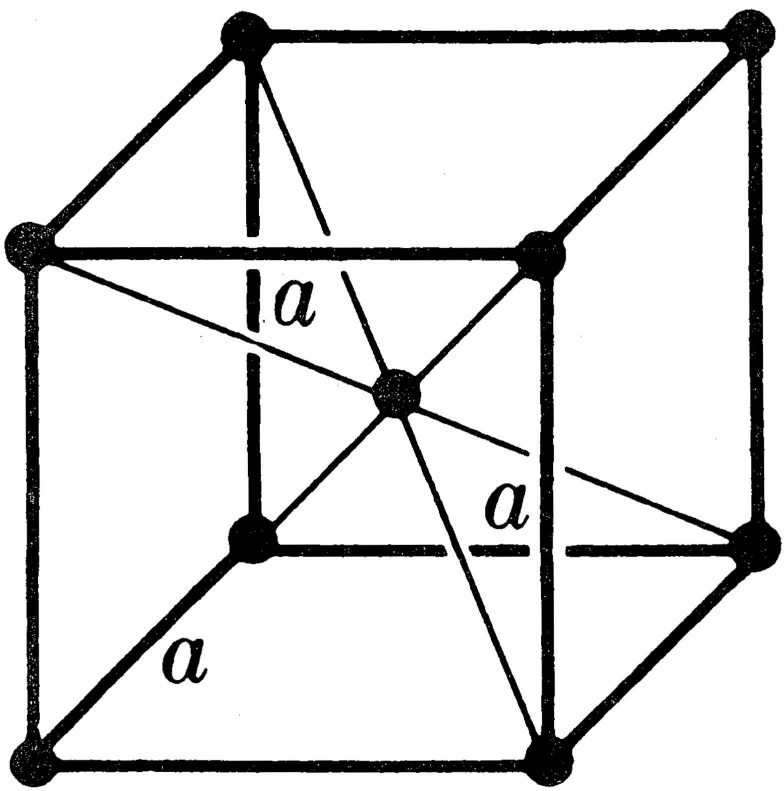


FIG. 21.—BODY-CENTERED CUBIC ARRANGEMENT OF ATOMS IN A CRYSTAL

heat-treated constructional alloy steels rely on carbon-based precipitation for their precipitation hardening strengthening.

The highest stress-strain curve shown in Fig. 20 represents the maraging steels. These steels are unique among structural steels in that they do not employ carbon as an alloying element. Instead of carbon, relatively large percentages of nickel are employed to produce strength levels in the 200,000 to 300,000 psi range. Though a special quenching and tempering heat treatment, the maraging steels are precipitation hardened by a finely dispersed nicked-based precipitate. The close spacing of these precipitate particles results in very effective blocking of dislocations, thus allowing the development of high strength levels.

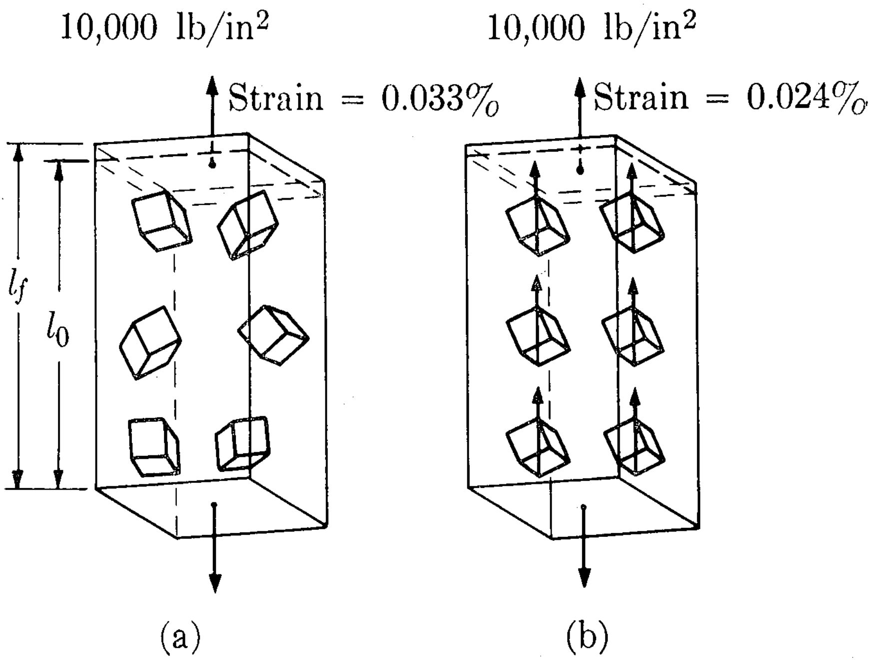


FIG. 22.—EFFECT OF PREFERRED ORIENTATION IN A POLYCRYSTALLINE IRON BAR ON ITS ELASTIC BEHAVIOR: (A) RANDOM ORIENTATION, (B) PREFERRED ORIENTATION WITH BCC CUBE DIAGONALS ALONG THE AXIS OF THE BAR (AFTER GUY)

STIFFNESS CONTROL

Although the strength levels of the steels shown in Fig. 20 range over almost an order of magnitude, the modulus of elasticity of each of the steels shown is essentially the same. This fact often creates a problem in the use of higher strength steels in structural applications in that deflection and stability considerations become limiting factors. The usefulness of the higher strength steels would be enhanced if their moduli of elasticity could be increased as strength levels are improved.

The elastic modulus is determined by the amount of strain observed at various imposed stresses. This macro level strain arises from elastic stretching of atomic bonds and from small recoverable movements of dislocations at the micro level. Stiffness can be enhanced by

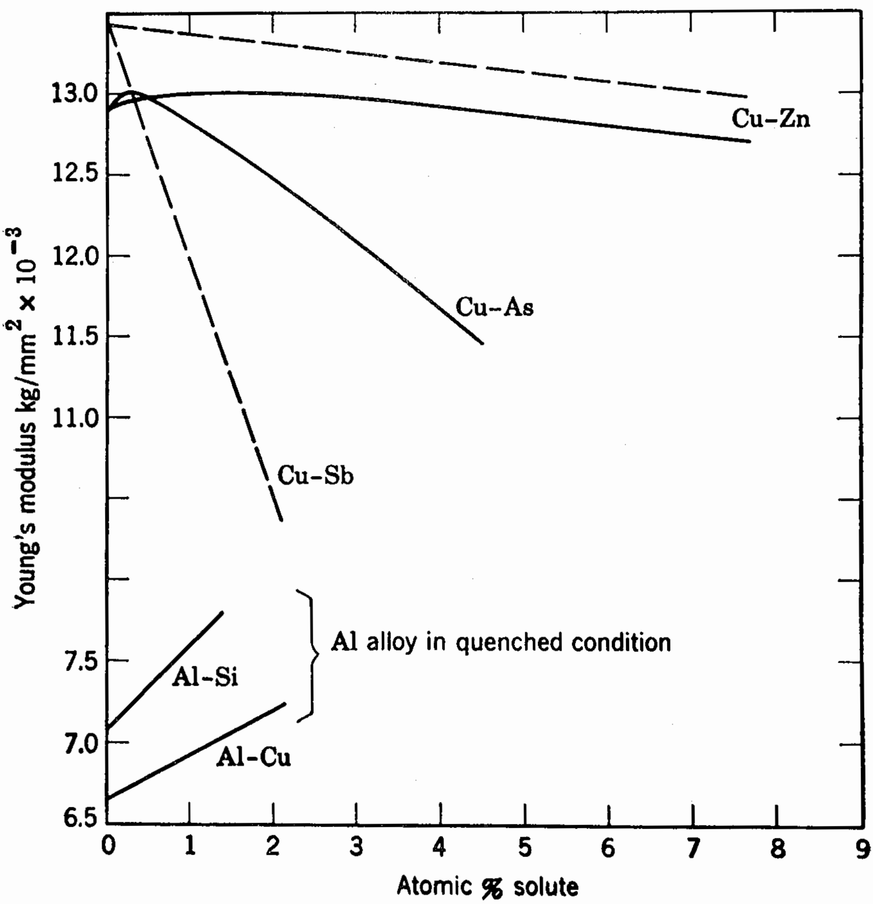


FIG. 23.—EFFECT OF SOLUTE ATOMS ON ELASTIC MODULUS (AFTER MCLEAN)

improving the effectiveness of the atomic bonds and by more effectively blocking even the small movements of dislocations.

Atoms in a metal have a characteristic crystalline arrangement. Iron atoms in steel at normal temperatures have the body-centered-cubic arrangement shown in Fig. 21. Aggregates of atoms in this form show different amounts of deformation under equal forces applied in various directions. For example, if a crystal of the bcc arrangements shown in Fig. 21 were loaded alternatively with the same stress on the cube faces and across the diagonal of the cube, different amounts of strain would result. Bonding across the cube diagonal would be seen to be more effective than that across cube edges, so strain would be

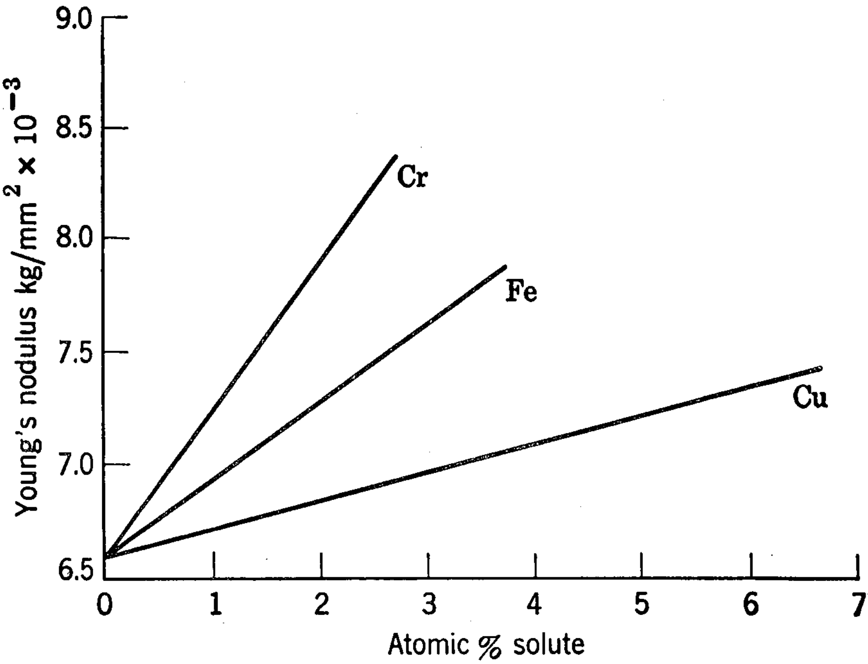


FIG. 24.—EFFECT OF SECOND PHASE PARTICLES ON THE ELASTIC MODULUS OF ALUMINUM (AFTER DUDZINSKI)

lower across the diagonal. If a large single crystal of iron were oriented with the bcc diagonal in the direction of loading, a minimum value of elastic modulus would be observed. In a polycrystalline metal it is often possible to produce preferred orientation of the crystalline structure by rolling operations. The effect of preferred orientation in an iron bar is shown schematically in Fig. 22. Here the elastic behavior of a polycrystalline alloy is enhanced if the normal random orientation of the crystal axes is changed to a preferred orientation along the bcc cube diagonal.

The elastic modulus can also be affected by the addition of solute atoms to the basic metallic matrix. Fig. 23 shows how the moduli of some aluminum and copper systems change as a function of the addition of certain solute atoms which form a solid solution. The effect of the addition of solute atoms which form hard second phase particles, due to a heat treatment processing, is shown in Fig. 24.

Evidently the modulus of elasticity of certain metals can be changed by atomic level manipulations. To date the ability to change modulus in desired directions has not been perfected for commercial quality metals such as the high strength steels. If an answer to this stiffness problem is found, it will be based largely on the increased understanding of the macro-micro relationships made possible by an ever developing materials science.

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WATER SUPPLY PROBLEMS IN EAST PAKISTAN

By Robert C. Marini*

Description of Province

Located on the eastern side of the Indo-Pakistani sub-continent, East Pakistan is one of the two provinces of Pakistan. As shown on Fig. 1, East Pakistan is bordered by the Bay of Bengal on the south, India on the west, north, and northeast, and Burma on the southeast. East Pakistan is separated from West Pakistan by India, and the two provinces are about 1,000 miles apart.

The province is almost entirely flat in topography with the exception of rolling hills along the eastern and southeastern margins and covers a surface area of approximately 54,000 sq miles. The province is a deltaic plain which was formed by the Ganges, Brahmaputra, and Meghna Rivers and their tributaries. A typical aerial view of the countryside is shown in Fig. 2.

To better visualize East Pakistan, the reader might imagine that an area the size of the State of Illinois was to be transferred to the Mississippi River delta in Louisiana.

The average annual rainfall in East Pakistan is 75 inches. The maximum annual rainfall is as high as 200 inches in the Sylhet District, which is located in the northeast sector of East Pakistan. This vast tract of low-lying alluvial plain is subjected to inundations during the monsoon season.

The climate of the province is tropical with the area being subject to the cyclic rainfall pattern of the monsoons. Dry summer weather starts in March with a steady rise in temperature until the end of May. Storms, accompanied by rain, are frequent during May. The monsoon rains usually start in June and continue through September. The humidity is generally high throughout the year running from 75 to 82 per cent in the winter months and 84 to 90 per cent during the remainder of the year. As compared to the climatic conditions in the United States, the most significant characteristic of the climate of East Pakistan is that extremes of hot and cold weather are not experienced.

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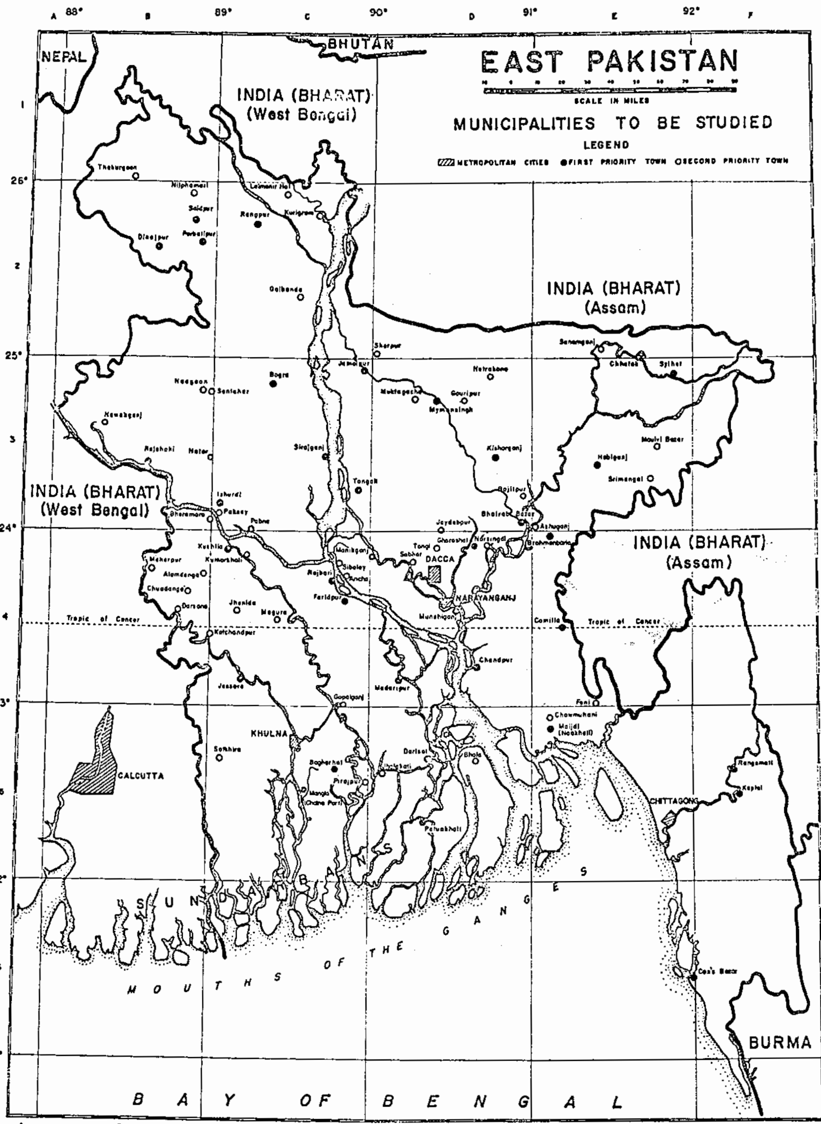

Fig. 1.—Map of East Pakistan

Area and Population

East Pakistan is one of the most densely inhabited areas of the world. It has an area of about 54,000 sq. miles and a population of approximately 57,000,000 people, the average density being about 1,050 people per sq. mile, as compared to 60 people per sq mile in the U.S.A. The province makes up about 1/7 of the area of Pakistan, but it contains approximately 57 per cent of its total population.

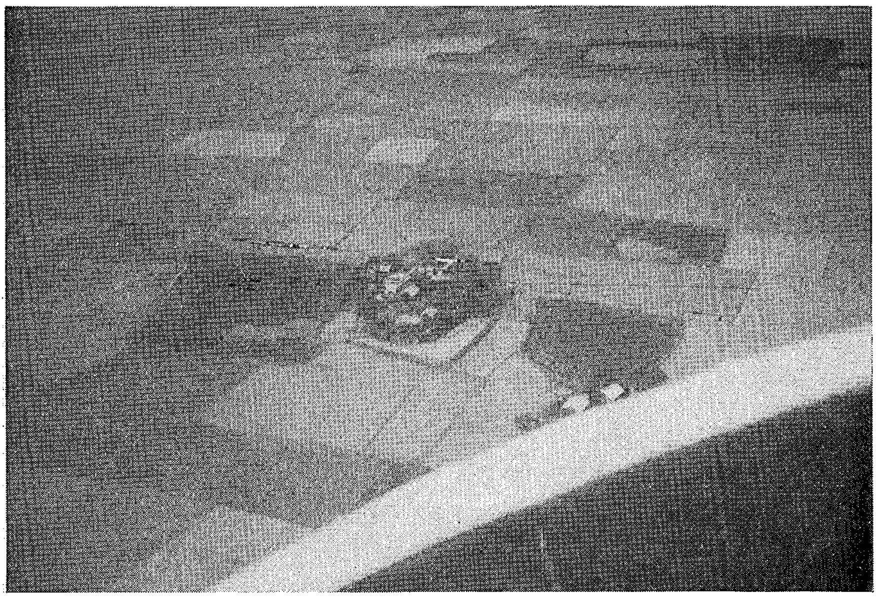

Fig. 2.—Aerial Photograph of East Pakistan

At the present time, approximately 2.9 million of the people in East Pakistan live in urban areas. It has been predicted by responsible planners, both Pakistani and American, that the population in 1985 in the urban areas may increase tenfold to approximately 27 million people because of the current migration of rural habitants to industrial urban areas. It is further predicted that if the current trends continue in East Pakistan, the population in the urban areas will increase between 25 to 30 times by the year 2015.

Currently, there are no cities within the province that have a population of one million or more. It is anticipated by the planners that between 4 to 8 cities with populations in excess of 1 million people will exist in East Pakistan by 1985 and possibly 10 to 16 of these large metropolitan areas will exist by 2015. The work that must be done to provide these future metropolitan areas with adequate and potable water supply simply staggers the imagination of most sanitary engineers.

Approximately 70 per cent of the East Pakistanis currently reside in unfinished huts (Katcha), constructed of bamboo and mud without interior plumbing, heating, or modern facilities. The average annual income of the Pakistani is about $65.00.

Another problem in the country is that currently about 75 per cent of the populace cannot read or write any language. This factor seriously impedes many of the progressive works which are being implemented to change the water consumption and personal hygiene habits of a majority of the populace.

History of Water Works

In 1947, the Indian sub-continent was partitioned by the British to form the two independent nations of Pakistan and India. Prior to independence, East Pakistan was known as the East Bengal Province of the Indian sub-continent. With the exception of the principal cities of Dacca and Chittagong, no significant development had taken place in the East Bengal Province prior to 1947. East Pakistan, since 1947, has made great strides in previously undeveloped fields, and now possesses a system of roads, hydroelectric power, schools, permanent buildings, and other major developments in very necessary fields. The history of water supply development in East Pakistan is a significant indication of the progress being made in other allied fields.

Prior to partition, water works within the province consisted mainly of very small water treatment plants processing surface water from rivers for the consumption of a privileged few. These were all built under the British rule in the major cities or government headquarters of Dacca, Comilla, Narayanganj, Mymensingh, and Paksey.

Some ground-water sources were developed in other municipalities, such as in Chittagong and Khulna. These facilities were usually of insufficient capacity and suitable only to serve a limited population. In the past, the majority of the population relied upon river water and ground-water "tanks" (open ponds) for bathing, drinking, and other miscellaneous uses. At the present time, as indicated on Fig. 3, these

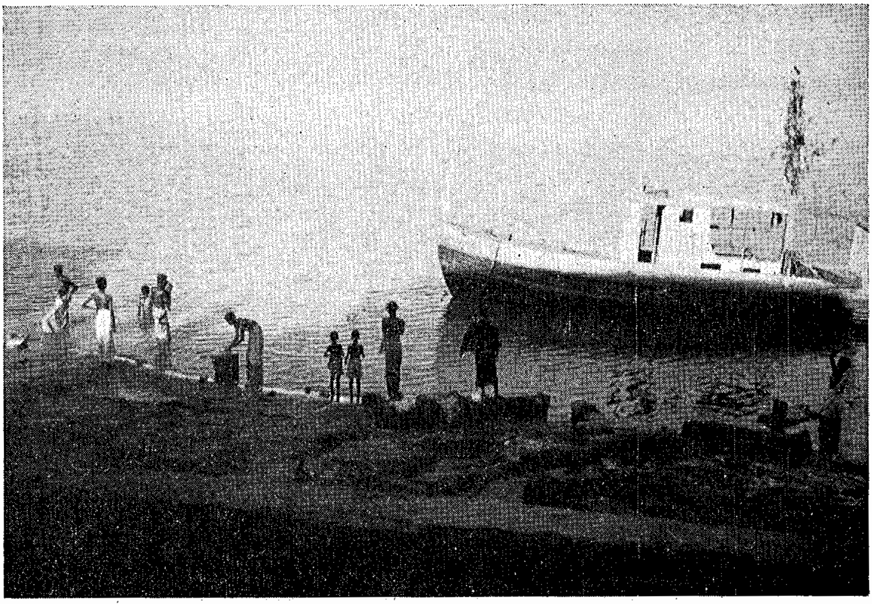

Fig. 3.—Typical River Scene

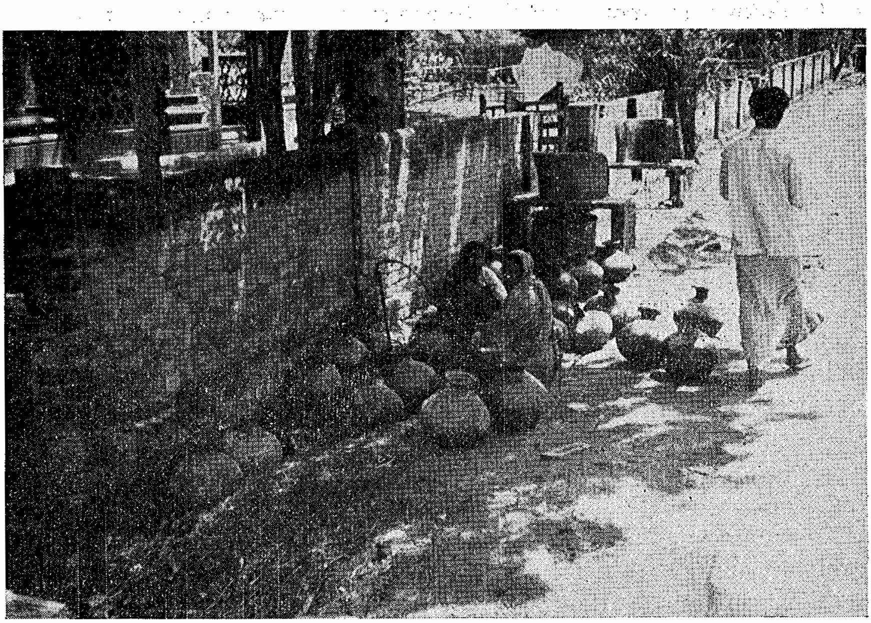

Fig. 4.—Women Obtaining Water for Household Use

same unsafe sources of water are being used by the people throughout the province for most of their water needs. As a result, East Pakistan is one of the remaining areas in the world where cholera and other water-borne diseases are still endemic.

Currently, where small distribution systems have been constructed, the majority of the population obtains its drinking and cooking water from street taps, shown on Fig. 4, which are usually under pressure from 4 to 8 hours a day. It has been estimated that from 200 to 400 people are currently served by each street tap.

Present Water Works in East Pakistan

Urban Water Supply Systems

The City of Dacca is now divided into two distinct sections; the old city, which was built before independence and the new city which has been constructed since partition. Dacca, which is also the capital of East Pakistan, has a water works system in the old city that was initially constructed in 1872 and after several modifications can deliver 4 mgd to the population. This main source of water, which still serves the old portion of Dacca, consists of a water treatment plant which treats water taken from the Buriganga River. Fig. 5 is a picture of the influent, baffle mix, and flocculation portion of this plant. This water is distributed to the system for a period of approximately 4 to 6 hours a day. The remainder of the city is served by 14 separate ground-water supply systems. All of the existing systems within the City of Dacca can only supply 14 mgd (million gallons per day). Based on a present municipal population of approximately 900,000 people and an average per capita consumption of 30 gallons per day, it is estimated that the average daily supply to the city should be about 27 mgd.

The City of Chittagong, which is the main seaport of the province, has a water supply system which was initially constructed in 1916 and expanded in 1952. The combined capacity of this water supply system is approximately 1.3 mgd. Based on the existing population and per capita consumption of 30 gallons per day, it is estimated that the average daily demand of the city should be around 17 mgd.

It is evident that both of the water systems of Dacca and Chittagong are grossly inadequate to take care of the present needs of the people.

In other metropolitan areas such as Comilla, Mymensingh, Ishurdi, and Paksey studies were made on the existing distribution systems to

FIG. 5.—MIXING AND FLOCCULATION BASIN AT OLD DACCA TREATMENT PLANT

FIG. 6.—WATER MAIN LAID IN DRAINAGE DITCH

determine the amount of water that people were actually getting under the present method of operation. It was estimated that the consumer was being supplied with from 0.6 to 10.0 gallons per capita per day. Bacteriological analyses made on the water being supplied to the consumer in various areas indicated that although a potable water was being put into the distribution system, contamination was entering the distribution system during periods of no pressure. As shown on Fig. 6, the water mains are normally laid in the drainage ditches which transport the communities' waste water.

In many of the areas within these metropolitan centers, numerous hand-pumped tube wells have been installed to supplement the existing water systems. The spasmodic operation of the present distribution systems, the amount of potable water which is available for human consumption, and the habits of some of the people are just a few of the important reasons which create the health hazard that exists within East Pakistan today.

Rural Water Supply

Prior to partition, the rural water supply which served the bulk of the populace consisted of shallow wells or untreated surface water which was available from either the rivers or from ground-water "tanks." To improve the unsatisfactory conditions that existed throughout the rural areas, the Government of East Pakistan acting through the Directorate of Public Health Engineering initiated a program in 1960 to install 1-in. to 1 1/2-in. diameter tube wells equipped with hand pumps. The Government of East Pakistan initially planned at the beginning of the second 5-year plan (1960) to install 58,000 new tube wells and replace 25,000 defunct tube wells in the rural area within the 5-year plan period. At the end of the second 5-year plan period (1965), approximately 83,000 tube wells were constructed and 17,000 old tube wells were replaced. The enactment of this single program has brought untold benefits to the bulk of the populace, providing people who live in this agrarian country with potable water for their cooking and drinking needs. The installation of so many tube wells has been one of the contributing factors to the recent decrease in the incidence of waterborne diseases, especially cholera.

Availability of Water and Water Quality

As stated hereinbefore, the deltaic plain known as East Pakistan is crossed by the Ganges, the Brahmaputra, and Meghna Rivers which drain the entire Himalayan Mountain ranges. Furthermore, because of the flatness of the land, the high ground-water table and the presence of suitable water-bearing substrata, the province is richly endowed with great ground-water resources in most areas. As a result of these factors, adequate water resources are available for development throughout East Pakistan. The surface water sources within the province are highly turbid during certain periods of the year and must be treated and disinfected to provide a good quality potable water. Because of the very flat slopes of the rivers and the tidal fluctuations in the Bay of Bengal, care must be exercised by the engineer in selecting the proper intake location so that high saline water will not be taken into a treatment plant. Recently the intake of a proposed treatment plant had to be relocated 7 miles upstream from the initially proposed location because it was found that during low river flows and high tide conditions the seawater was backed upstream from the Bay of Bengal many more miles than were originally conceived. Fortunately, this discovery was caught during the planning stage rather than the construction stage and indicates the necessity for good preliminary planning.

Although the ground water within the province is usually potable without disinfection, the quality in many areas is seriously impaired because of high iron, hardness and total dissolved solids.

Analysis of ground-water supplies throughout the province is currently being conducted and has indicated that the ground water in the Comilla area has an iron content that varies from 0.9 mg/1 up to as high as 7.0 mg/1. Analyses of the ground water in the Rajshahi District indicate that hardness expressed as calcium carbonate is in excess of 400 mg/1. Although the ground water in the Khulna area does not possess excessively high iron or hardness, it has a tendency to be high in total dissolved solids. The dissolved solids in the ground water in the Khulna area have run as high as 1,300 mg/1.

The limited analyses done thus far on the ground-water quality within the province clearly indicates that treatment of this water may be required in the very near future when it is demanded and can be paid for by the consumers.

It is anticipated that at least initially the majority of the water supplies within East Pakistan will be of ground-water origin with only the larger metropolitan areas requiring the construction of the more costly surface water supply facilities.

Future Plans for Providing Water to East Pakistan

Urban Areas

Final plans and specifications for the construction of modern water works facilities for the principal cities of Dacca and Chittagong were completed by Ralph M. Parsons, Inc., Los Angeles, California. These plans consisted of detail layouts for conventional water treatment, transmission, and distribution facilities for the municipalities. Because the bids received in 1965 for these projects were far in excess of the funds available for construction, the scope of these projects is currently being revised. Recently, Camp, Dresser & McKee were retained by the Government of East Pakistan as general advisors in public health engineering to study and review the planning done on these cities for their respective water supply and sewerage authorities (WASA).

Current thoughts are to utilize ground-water resources more extensively, where practicable, in order to reduce the cost of this project.

It is anticipated that if the revised project costs for furnishing potable water are within the budget authorized by the World Bank and Government of Pakistan, construction will start on these projects in the near future.

A water study report has been completed for the City of Khulna, which is currently the third largest city (220,000 people) within East Pakistan, by James M. Montgomery, Inc., of Pasadena, California, and Zafar & Associates, a Pakistani concern. This report has recommended that ground-water supply be utilized for an interim period until it becomes necessary to construct a modern water treatment plant to handle the water supply needs of the municipality. No final planning has been initiated thus far for implementing the recommendation of the report.

Camp, Dresser & McKee's contract with the Government of East Pakistan specifies that the firm shall "provide general advisory services in public health engineering for the province of East Pakistan, with the ultimate goal of developing within East Pakistan the capacity to plan, design, install, manage, operate, and maintain water and sewerage systems, and to develop manufacturing capability in the private sector for the greater part of the materials and equipment required for their installation and operation."

In cooperation with the Directorate of Public Health Engineering, we are currently studying the water requirements for about 80 communities within East Pakistan. Comprehensive preliminary planning has been completed for Comilla, Mymensingh, and Ishurdi and is almost completed for Narayanganj and Rajishahi.

Ground-water resources are going to be developed for use in most of the municipalities as the main source of water supply. Initially, no treatment of the supply will be incorporated into the design, but space will be allotted in the stations for the addition of chlorination equipment.

The water works improvements which have been recommended in the completed reports fall into three basic categories, which are as follows:

1. Correction of the deficiencies within the existing water systems by improving pumping operations, eliminating severe leaks in the distribution system, repairing improperly operating street taps, and roof tanks, etc.
2. Immediate construction of a modified distribution system which will be under pressure 24 hours a day to serve a population of about 100 families who live in "Katcha" homes. The modified system will consist of small distribution piping and a "Fordilla-Type" outlet in front of each house. The "Fordilla-Type" outlet is equipped with a dash pot arrangement which automatically closes after discharging a predetermined volume of water. This valve must be manually activated for each discharge. If this system proves to be acceptable, it will be utilized extensively throughout the province.
3. Construction of a new distribution system which will consist of multiple-feed points from wells and elevated tanks which are strategically located within the core of the municipality. It is anticipated that by the use of control valves, piping, and roof tanks for each individual house connection that the "fluctuations in draft experienced in the U.S.A. can be substantially reduced in East Pakistan. It is further planned that no additional capacity will be built into the distribution systems to handle fire flows. Since the permanent buildings are of masonry construction and the area is literally strewn with ground-water "tanks" and rivers, this design factor, in the writer's opinion, is not a severe one. Small fluctuations in draft and elimination of additional fire flow capacity will substantially reduce the sizing of the distribution piping and, therefore, the cost.

Because of the scope of work which is involved in developing water supply throughout the province, many other allied studies are proceeding while pertinent water supply studies are being made. It is necessary, for example, to determine the applicable design criteria to be used by sanitary engineers, the commodity requirements in order to supply the equipment and materials from local sources to implement these plans, and many other necessary factors to get the province in the position so that it will be able to supply potable water in modern systems to the consumers in the metropolitan areas. It has been estimated that in order to provide potable water of acceptable quality for the future urban areas in the province, the cost would be well over a half billion dollars. It is evident that it will require a considerable amount of engineering, planning, and construction in order to implement the works being planned for East Pakistan.

Rural Areas

Because of the success of the present hand tube well program, a goal for providing a hand tube well for every 200 people within the rural areas has been set by the DPHE. The hand tube well program has been made an important part of the third 5-year program (1965-1970), and work has been scheduled so that the above goal can be met.

Conclusion

In conclusion, it has been the intent of the writer to present only a generalized idea of the present water supply problem which must be met and resolved within East Pakistan. Future water work facilities which must be planned, constructed, operated, and maintained within the province are numerous and far exceed the number of suitable works that have been constructed thus far. Although a start has been made to solve this very important problem in East Pakistan, the challenge that awaits the sanitary engineers in the province is tremendous.

AN ADDRESS

By Hon. John F. Collins*

(Presented at the Annual Meeting of the Society, March 23, 1966)

For most of the post-war period—for over twenty years—our nation's cities have been in danger of being overwhelmed by an urban society, a society which litters our countryside, clogs our highways and blights our cities.

Today there are 90 million motor vehicles on our almost 3 million miles of paved highway.

Daily we each dispose of 4 pounds of trash, a total of 10 million pounds in our metropolitan area, 540 million pounds throughout the nation.

Our poor number over 30 million, most are children (15 million) or elderly (5 million).

Of all families headed by an elder American (65 or over) 44% have an income of $3,000 dollars or less.

Thirty-seven per cent of all Negro families have an income of $3,000 dollars or less.

Almost 70% of the household heads in poor families have no more than a grade school education.

Today of this nation's 193 million people, virtually 7 out of 10 live in urban areas. Our farm population which was 15% of our total population in 1960, today accounts for less than 10% of our population.

Each year 3 million infants are born. All must be educated.

Each year sees the formation of more and more families, who must be provided with employment, housing, and transportation facilities.

Each year an ever increasing number of us reach the age of 65, and our elderly, if they are to enjoy their retirement years, will require improved medical and health services, leisure time, and recreational activities.

Each year every additional 1000 metropolitan residents require an additional:

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* Mayor, City of Boston, Massachusetts.

4.8 elementary school rooms
3.6 high school rooms
1.0 hospital beds
100,000 gallons of water
1.8 policemen
1.5 firemen
Each 1000 residents urbanize 100 acres.

Spreading urbanization has made acute the needs for adequate transportation, schools, open space, recreational areas, sewer and water facilities. The congregation of a large number of people in compact but overlapping communities has generated new needs, has given rise to an unending chain of political and socio-economic problems.

The strength and vitality of our older central cities has slowly eroded. Our older cities have so far been unable to compete with the attractiveness of suburban green and the age of the motor car.

With rising incomes and aspirations many have left the cities to search for their own plot of grass. Business and retail activity soon follow their former customers.

Into our old central cities come the new immigrant, the displaced farmer, the Negro, the Puerto Rican, almost penniless, inexpert and often friendless, to live in slums or to generate new slums through simple overcrowding; to find that they are unable to secure employment. It is here that poverty, limited skills, and ignorance keeps them. That our old central cities are centers of anti-social behavior of crime, delinquency and mental illness should come as no surprise.

These are the problems of our municipal governments, induced by population growth, mobility and prosperity.

And yet, the Great Society which we all seek must, as Walter Lippman has pointed up, be built if it is to be built at all in the great cities, which are now the center of American life.

We were not prepared for affluence, for the metropolis. The results of unpreparedness are obvious; bulging suburbs and blighted cities, racial tensions, delinquency, polluted air, water shortages, inadequate mass transit facilities, clogged highways, depleted municipal revenues and mounting tax rates.

We were unprepared because we were slow to leave the farm, we were reluctant to admit that the desires and needs of a highly mobile urbanized society differ greatly from those of a society comprised of relatively separate, self-contained individual communities.

But thanks to the Housing Act of 1949 and later additions which substantially altered inter-governmental relationships, we, the creators and creatures of our urban society, now have a clearer understanding of what must be done, agree on how it is to be done and are prepared to act.

The renewal process taught us that we cannot concern ourselves solely with projects and neighborhoods but that our concern must be for the community at large that the revitalization of a community depends not only upon the adequacy of residential housing but also upon the adequacy of transportation, the vigor of the business community, the adequacy of its parks and playgrounds and its educational programs. It taught us that massive surgery is both expensive and painful, nor does it allow for selectivity. To retain the worthwhile while discarding the valueless.

We now realize that housing is not the only social need of residents, that they need and expect—they deserve—improved educational programs, better recreational activities, more open space, improved health and welfare services and more and better economic opportunities. We now know that to wait for blight and decay to appear before taking corrective action is to fight yesterday's battles.

On the horizon is the City Demonstration Act, a program which laboratories, the cauldron into which we pour the energies, talents, and resources of our municipalities, of our technologists, and our universities to dispel the myths and discover the realities of our urban ecology.

Slowly a nation of individualists, accustomed to the idea that each person must fend for himself as an independent unit, moves into an age of interdependence; slowly recognizing and organizing the institutions which such an age requires.

Reluctant state legislatures, long viewed by political scientists and other public figures as 17th-century anachronisms unable and unwilling to conduct the business of 20th-century government, to adequately cope with the problems of urbanism, have begun to challenge the old order.

Our own legislature has, in recent years, established a commonwealth service corps, revised its housing and code enforcement laws, revitalized and reorganized our entire educational system and approved a sweeping racial imbalance law. It has reorganized our mass transportation system. It has just passed a major revenue program and we, the cities, are the principal beneficiaries. We will now be better able to service the rising needs and aspirations of all our citizens.

We have discovered that the age of the motor car and mobility make it impossible to easily discern and separate purely local concerns from those concerns which are regional or national.

Health and welfare problems are not evenly distributed among communities, nor are the problems of race and aging. They are not because the Negro, the poor and the elderly are city dwellers; city dwellers because they lack mobility for various reasons, some self-imposed, some economic, some social and some political. The cities have the poor in health, education, and income.

Today we recognize that to ask our older cities to continue to provide for the education, health and welfare of the disadvantaged, be they young or old, is to admit that the disadvantaged will always be with us and at the same address.

We recognize that the problems of the poor and the neglected are the concern of all, that each of us has a right to safe and sanitary housing, that all are entitled to the opportunity for social and economic improvement.

We now realize that a child, who has been deprived of educational opportunities, of parental guidance, care and example, will not suddenly emerge as a mature young adult ready to take his place in society.

We now understand that what Little John did not learn, Big John does not know.

We have the Office of Economic Opportunity and its many educational, retraining and cultural programs. We are beginning to heed Jane Addams who long ago warned us against doing good to people. "One does good, if at all, with people, not to them." We are asking the poor to help us, to mobilize themselves, to assist us in developing neighborhood centers for health, education and recreation.

And so it is that we slowly admit that the organizational, financial, and legal prescriptions necessary to sustain and promote the growth of an agrarian society may be neither sufficient nor germane to the problems of an urban society. We reluctantly part with habit.

Today we have the new cabinet post, the Department of Housing and Development, to foster the growth and development of our urban areas, to deal with the problems of our cities.

Tomorrow we may have a new cabinet post, the Department of Transportation, to foster the development of a coordinated transportation system, to permit travelers and goods to move quickly, conveniently and efficiently.

And so we see developing a new federalism. A creative federalism which began because of the growing awareness that the urban problems of physical and social decay are national problems and that to resolve them there must be a reallocation of all our nation's resources. A federalism not based on separate functions for each level of government but of shared functions for all levels. This new federalism has given to our cities an unprecedented opportunity to deal more effectively with our urban environment.

I am convinced that we will attain our visionary Great Society and that our great cities will serve as its foundation, for our nation will no longer permit the plundering of its cities, for we now realize that a society which allows the plundering of its cities is in no position to protect the assets of the country.

I would be more secure in my belief that the Great Society will be ours, more secure that the passage of time will prove me to be more sage than charlatan; if I did not have to ask who is to build it. Who is to build this Great Society?

As a nation we have the technical competence, the mechanical, the electrical and the civil engineering skills to plan, design and construct the highways, the schools, the water and sewer facilities, and the housing necessary to satisfy the physical demands of our citizens.

The emerging dynamic or creative federalism, organizational changes and new methods of finance present us with an opportunity to subject our urban environment to the "scientific method," the tool which has contributed so much to our technological advance but has as yet directly contributed so little to social progress. We can now ask the questions.

Are our existing organizations and institutions mere anachronisms or do they bear some resemblance to reality?

Are our core cities, like the dinosaur, destined for extinction?

Is it true that 60 years ago, our central centers were the "melting pots" of America, the great synthesizer only because people lacked mobility and could not escape?

What are the real needs, social, economic, and political of an urban society?

We may ask, but in many communities who will answer?

The crisis of the Great Society may well be the lack of human resources.

Here in Boston age and retirement continue to deplete our roster of administrative, professoinal, and technical personnel.

The Great Society will not Topsy-like "just growed." It will not come about because of some grand vision or legislative fiat, it must be built. If all we have is grand visions and legislative victories, we may win battles but we will surely lose the war. For laws are not self-executing, nor are public facilities self-constructed.

Whether families will live in pleasant, attractive neighborhoods or slums depends upon public decisions, made by public officials, as to housing, and the location of schools, clinics, parks and playgrounds.

Whether business activity flourishes or dies is partly the result of the decisions of public officials as to traffic flow, transportation facilities, parking, and zoning.

Whether health and welfare programs are utilized or unused depends on the decisions of public officials as to an evaluation of the services desired, the location of clinics, and the availability of professional personnel. The management of local governments in an urban environment is a complex activity. It requires able, skilled and mature men and women.

Yet our local governments face major manpower shortages while our youngsters rush to the Peace Corps to accept the challenge of underdeveloped nations or to Vista to accept the challenge of Appalachia. Do they not realize that to find the same challenge, the same opportunity for significant service, they need look no further than their own community.

Any growing enterprise must compete for manpower and all must face the possibiliy of manpower shortages. The competitiveness of our local governments is severely inhibited by its public image. Any unit of government has a reputation and presents an image to the public based upon its performance. For too long politics and public administration have been depicted as "a strife of interest masquerading as a contest of principles" or the "conduct of public business for private advantage." For too many years local governments have been forced to labor under *the shame of our cities*. The public only reluctantly remembers that Lincoln Steffans was commenting on the realities not of 1966 but of 1904.

For too many years cities have operated under rigid, negative, and overprotective civil service systems, designed more to reward persistence and stamina than talent.

For too many years cities have been depicted as bankrupt enterprises and able and ambitious people do not go to work for failing organizations.

Cities themselves have not been active recruiters of talent, nor in providing programs of continuing education or developing in-service training programs.

Universities have only recently turned their eyes from the national and international horizons, and to focus attention on metropolitan and local issues.

A Great Society demands excellent people and excellence in all its people.

The stewards of this society, the architects, builders, artisans and craftsmen, must be equal to the task, for the social, economic, political, and ethical values of our constantly changing urban environment must be understood if our communities are to be suitable places in which to live and work.

Our success in building a Great Society will be judged not only in terms of our ability to deal effectively with physical blight, in achieving a land-use plan and transportation system more in tune with 20th-century criteria than those of the 18th century, but also in terms of our ability to deal effectively with the social and moral issues of our times.

Our ability—you and I, creators and creatures of our urban environment—to deal effectively with the social and economic issues of today, to restore in the less fortunate a sense of purpose, a sense of belonging, to secure for all the jobless, the aged, the deprived—the good life: to liberate the poor from the chains of poverty, to liberate the aging from the bonds of boredom, illness and idleness, will in a large measure determine whether we will realize our dream of the Great Society.

If we fail we will be the greatest of cowards for we will have much to be ashamed of.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

APRIL 14, 1966:—A joint meeting of the Boston Society of Civil Engineers, with the Structural Section was held this evening in the Adams Room, United Community Services Building, 14 Somerset Street, Boston, Mass., and was called to order by President John M. Biggs, at 7:00 P.M.

President Biggs stated that the minutes of the previous meeting held March 23, 1966, 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 Biggs announced the death of the following members:—

George A. Sampson, Past President, elected a member April 20, 1910, who died March 3, 1966.

John J. Devine, elected a member November 15, 1939, who died March 9, 1966.

Richard W. Johnson, elected a member March 17, 1937, who died March 22, 1966.

Frank A. Cundari, elected a member March 21, 1945, who died in September, 1965.

The Secretary announced the names of applicants for membership in the Society.

President Biggs stated that this was a joint meeting with the Structural Section and turned the meeting over to Robert L. Fuller, chairman of that Section to conduct any necessary business at this time.

Chairman Fuller introduced the guest speaker of the evening, Edward H. Barker, Sverdrup & Parcel & Assoc., Inc., Washington, D.C., who gave a most interesting illustrated talk on "Technique of Construction of the Chesapeake Bay Bridge-Tunnel Crossing."

A question period followed the talk.

27 members and guests attended the dinner preceding the meeting and 50 members and guests attended the meeting.

Meeting adjourned at 8:30 P.M.

Respectfully submitted,
CHARLES O. BAIRD, JR., *Secretary*

MAY 11, 1966:—A joint meeting of the Boston Society of Civil Engineers, with the Structural Section of BSCE, and the Massachusetts Section of the American Society of Civil Engineers was held this evening in the New Architects Building, 238 Newbury Street, Boston, Mass.

No business was conducted at this meeting. Chairman Robert L. Fuller of the Structural Section called meeting to order at 7:30 P.M., and introduced the guest speaker of the evening, Mr. Omar W. Blodgett, Design Consultant, The Lincoln Electric Company, Cleveland, Ohio, who gave an interesting illustrated talk on "What the Structural Engineer Should Know about Welding."

A discussion period followed the meeting.

105 members and guests attended the dinner preceding the meeting. 110 members and guests attended the meeting.

The meeting adjourned at 8:35 P.M.

Respectfully submitted,
CHARLES O. BAIRD, JR., *Secretary*

MAY 25, 1966:—A joint meeting of the Boston Society of Civil Engineers with the Construction Section was held this evening in the Society Rooms, 47 Winter Street, Boston, Mass., and was called to order by President John M. Biggs, at 7:00 P.M.

President Biggs stated that the minutes of the previous meeting held May 11, 1966 would be published in a forthcoming issue of the Journal and that the reading of those minutes would be waived unless there was objection.

The Secretary announced the names of applicants for membership and that the following had been elected to membership this date:—

Grade of member—Robert M. Pope,

James L. Rodgers, Rubin M. Zallen

President Biggs stated that this was a joint meeting with the Construction Section and turned the meeting over to Robert J. Van Epps, chairman of that section to conduct any necessary business at this time.

Chairman Van Epps introduced the guest speaker of the evening, Richard Halloran of the Perini Corporation, who gave a most interesting talk on "A Descriptive System for Estimating for Heavy and Other Construction." Outline of talk was passed out to those attending the meeting.

A question period followed the talk. Meeting adjourned at 9:45 P.M.

Respectfully submitted,
CHARLES O. BAIRD, JR., *Secretary*

HYDRAULICS SECTION

MAY 4, 1966:—A meeting of the Hydraulics Section of the Boston Society of Civil Engineers was held in the Adams Room of the United Community Services Building, 14 Somerset Street, Boston, Massachusetts. The meeting was called to order by the chairman of the section, Mr. Nicholas Lally, who introduced the new officers of the Hydraulics Section. The chairman also introduced Professor John M. Biggs, President of the Boston Society of Civil Engineers. Professor Biggs greeted the Hydraulics Section briefly.

Mr. Lally then introduced the speaker of the evening, Dr. Ronald T. McLaughlin, Associate Professor of Civil Engineering at the Massachusetts Institute of Technology. Dr. McLaughlin spoke on the "Sensitivity Analysis in the Design of Pipe Networks." The speaker discussed a computerized approach in analyzing the interrelated rates of change of various parameters affecting pipe network design.

The meeting had an attendance of 27 and was adjourned at 9:00 P.M.

Respectfully submitted,
ATHANASIOS A. VULGAROPULOS, *Clerk*

SANITARY SECTION

JUNE 1, 1966:—The annual outing of the Sanitary Section was held today as a joint meeting with the Boston Society of Civil Engineers and included a tour of the Carling Brewing Company, Route 9, Natick, Massachusetts.

Members assembled at the plant at

5:00 P.M., and after sampling the Carling products, were formed into several groups and conducted through the plant. The barley, grain and malt preparation, and the mashing, wort, hopping and boiling, fermentation, aging, carbonation, filtration, pasteurization, and packaging processes were visited. The tours were about 45 minutes in length and were led by capable plant guards.

At 7:00 P.M. the members reassembled at the Beacon Terrace (opposite Shopper's World, Route 9, Framingham) for a dinner meeting.

Chairman Robert L. Meserve introduced Vice-President Harry Kinsel of the Society, who extended greetings from the Society.

Chairman Meserve then introduced Mr. Don Davidson, Quality Control Manager of the Natick plant of Carling Brewery, who gave a most interesting and detailed talk on the beer and ale-making processes.

Major points brought out by the speaker included the following: Well water from Natick is used for the making of beer and ale. Lake Cochituate water is used for cooling purposes only and is returned to the lake. The brewing process uses malt, corn, corn syrup, hops and other ingredients, including carbon dioxide. Temperature control (both hot and cold) plays an important part in the process. The beer is filtered through diatomaceous filters and is pasteurized after packaging. Strict control over the sanitary aspects of the plant is accomplished by competition among all Carling breweries supervised by an independent agency and also by the Federal Drug Administration and other health agencies. By-products of the process include spent grain (used for cattle feed), yeast and hops. The beer and ale brewers do not rely on "old recipes handed down from generation to generation," but also on improved process methods and advanced technology to stay ahead in the field.

A very lively question and answer period followed Mr. Davidson's talk. Fifty-one members and guests attended the meeting. The meeting was adjourned at 9:40 P.M.

CHARLES A. PARTHUM, *Clerk*

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AND
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BOSTON SOCIETY OF CIVIL ENGINEERS

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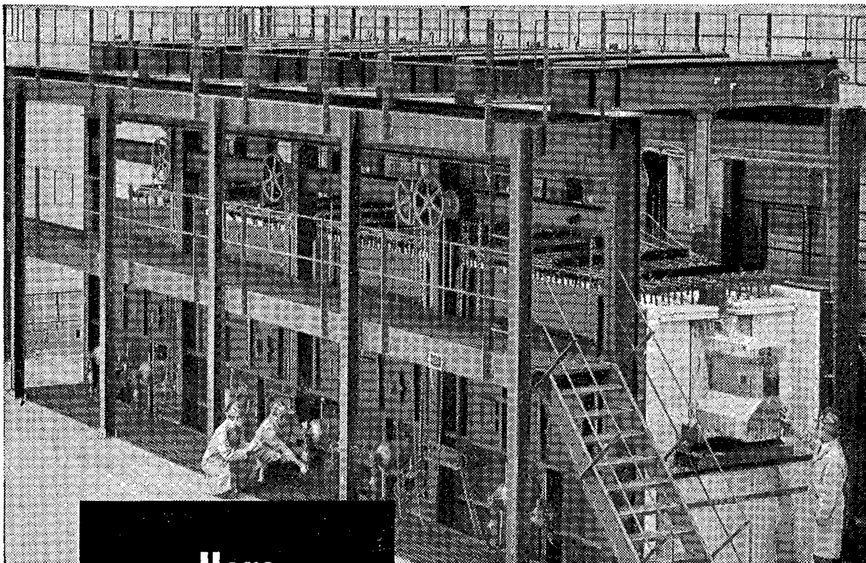

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Robert L. Fuller

Harry L. Kinsel
Harl P. Aldrich, Jr.
Charles O. Baird, Jr.

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Nicholas Lally
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Saul Namyet

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STRUCTURAL SECTION AWARD

Max D. Sorota, *Chairman*

Saul Namyet

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HYDRAULICS SECTION AWARD

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Ronald T. McLaughlin

TRANSPORTATION SECTION AWARD

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