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SOME PRACTICAL IMPLICATIONS OF ELEMENTARY SAFETY ANALYSIS

By
C. A. CORNELL[1] and E. H. VANMARCKE[2]

(Presented before Annual Meeting of the American Society of Civil Engineers October, 1967. Professor Cornell spoke on the subject of structural reliability before the Boston Society of Civil Engineers on April 20, 1967.)

A fundamental benefit of the theory of structural reliability is that it puts the understanding of what affects structural safety on a firm basis. It permits one to isolate factors that are widely known to affect safety and to determine new, less obvious, factors. In both cases the factors can be studied quantitatively to determine the situations under which they are significant and, just as important, the circumstances under which they are relatively unimportant. In this paper a number of such factors are identified and known results, determined from previous theoretical studies, are deduced and interpreted in an elementary manner. The intent is to keep the discussion non-theoretical and qualitative in nature, relying upon the reader's intuitive understanding of simple probability notions[3] to justify the conclusions.

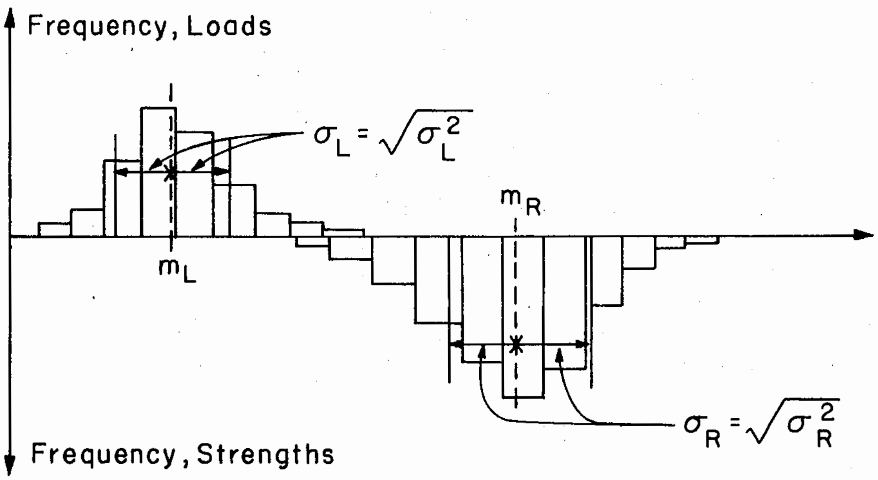
The succeeding sections will cover some lessons to be learned from focusing attention on, in turn, the individual components of a structure, a structure subjected to an entire history of loading, complex structure under a known load, and finally a similar structure under an uncertain load.

1. Associate Professor, Massachusetts Institute of Technology
2. Research Assistant, Massachusetts Institute of Technology
3. Meyer, P. L. "Introductory Probability and Statistical Applications," Addison-Wesley, 1965

LOAD AND STRENGTH VARIATION INFLUENCE

It is widely recognized that the safety of a member is determined not only by the relationship between the typical or central values of material strengths and applied loads, but also by the degree of variability or dispersion demonstrated by both strengths and loads. Owing to the nature of the material and to the conditions under which it is manufactured, relative dispersion in concrete strengths is known, for example, to be larger generally than in steel strengths. This variability influences a member's reliability, i.e., the likelihood that it will perform properly in service. Similarly, the wide variation in earthquake accelerations, storm wind velocities, and numerous other natural loadings makes it unwise to design for their average annual maximum values; their variability too is significant.

But how do these factors influence safety and to what extent? Sketches of histograms of numerous strength and load observations can be converted to similar units, and juxtaposed as shown in Figure 1. Measures of the central or typical values are the average or *mean* load, m_L, and mean strength or resistance, m_R. A measure of the variation in the load is σ^2_L, the *variance*, or moment of inertial of the histogram about its mean:



HISTOGRAMS OF LOAD AND STRENGTH OBSERVATIONS

Figure 1

$$\sigma_L^2 = \frac{1}{n} \sum_{i=1}^{n} (x_i - m_L)^2 \qquad 1$$

in which the x_i are the observed values of the load. The square root of this variance, σ_L, called the *standard deviation,* has the same units as the load and its length is plotted on the sketches. Analogous measures of the central value and dispersion of the resistance are also shown, m_R and σ_R.

Clearly a particular member will perform properly only if its particular strength, R, exceeds the particular load, L, to which it is subjected. From the histograms shown one can estimate the proportion of combinations of one strength and one load which will perform satisfactorily (i.e., for which $R > L$), and this proportion or probability is called the *reliability* of the member. Through reliability theory it can be shown (see Appendix II) that this number, P_s, is at least

$$P_s \geq 1 - \frac{K_R \sigma_R^2 + K_L \sigma_L^2}{(m_R - m_L)^2} \qquad 2$$

in which the two constants K_R and K_L depend upon the shapes of the corresponding histograms. This bound may be well below the actual value (which the theory can also determine[4-5]), but the lower bound's simple form facilitates direct interpretation.

This simple relationship can be used to gain an appreciation for the effect of means and dispersions on reliability. It is somewhat more straight forward to discuss one minus the reliability, $P_f = 1 - P_s$, which is called the *probability of failure,* failure to perform satisfactorily with respect to safety or serviceability, depending on the problem. From Eq. 2

4. Freudenthal, A. M., Garrelts, J. M., Shinozuka, M. "The Analysis of Structural Safety", *Journal of the Structural Division,* Proceedings of ASCE, Vol. 92, No. ST1, February, 1966
5. Turkstra, C. J., "A Formulation of Structural Design Decisions," thesis presented to the University of Waterloo, at Waterloo, Canada, 1962, in partial fulfillment of the requirements for the degree of Ph. D. in C. E.

$$P_f \leq \frac{K_R \sigma_R^2 + K_L \sigma_L^2}{(m_R - m_L)^2} \qquad 3$$

Eq. 3 reveals that the probability of failure (as reflected in this upper bound) depends not only on the difference in central values $m_R - m_L$, but also on the dispersions of both resistance and load, and in a symmetrical, additive way.

Effect of Resistance Variation

Letting $V_R = \sigma_R/m_R$ and $V_L = \sigma_L/m_L$, Eq. 3 also can be rewritten as

$$P_f \leq \frac{K_L m_L^2 V_L^2 + K_R m_R^2 V_R^2}{(m_R - m_L)^2} \qquad 4$$

In this form it is apparent that, for given central values, m_R and m_L the (upper bound on the) probability of failure increases linearly with the square of the ratio $\sigma_R/m_R = V_R$. This non-dimensional ratio, called the *coefficient of variation,* can be used to compare the variability of different phenomena, such as two materials. Mill tests on the yield stress of mild steel, for example, have shown[6-7] a coefficient of about 7 to 8%, while the same coefficient for concrete compressive stress may vary from 10 to 25% depending upon the quality control exercised.[8]

In addition to inherent material variability, dispersion due to fabrication and other factors must in general be included in the definition of the coefficient of variation of the resistance of a member in place, as will be discussed.

6. Julian, O. G., "Synopsis of the First Progress Report of the Committee on Factors of Safety," *Journal of the Structural Division,* Proceedings of ASCE, Vol. 83, 1957
7. Freudenthal, A. M., "Safety and Probability of Structural Failure," *Journal of the Structural Division,* Proceedings of ASCE, Vol. 121, 1956
8. ACI Standard 214-65, "Recommended Practice for Evaluation of Compression Test Results of Field Concrete," Jan. 1965

Effect of Load Dispersion

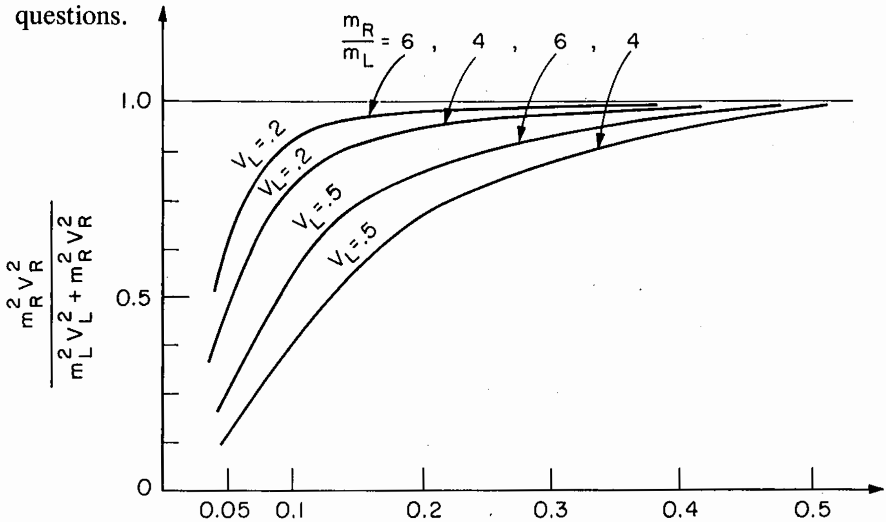
At the same time Eq. 4 shows that the coefficient of variation of the load affects reliability in a parallel way. For given central values, a structure is less safe with respect to wind loads[9] where V_L = 30 to 50% than with respect to dead loads[10-11] where V_L is less than 10%. Most present building codes reflect the influence of uncertainty primarily through load factors of different magnitudes. Although seldom in a quantified manner, nominal working loads and specified strengths may also reflect σ_L and σ_R as well as m_L and m_R. The most recent codes[10] make these relationships more explicit, choosing design loads, for example, proportional to mean (maximum) loads plus a fixed number of standard deviations.

Effect of Resistance Versus Load

It is commonly stated that the uncertainty in the loading on structures is so great that the variability in the strengths of most common construction materials is negligible in comparison. Eq. 4 permits a ready quantitative evaluation of this statement. Notice that the coefficients of variation appear squared emphasizing the difference between V_L and V_R. Representative values of the former might range from 0.1 to a typical value of 0.5 or above, while the latter might range from 0.05 through a typical value of 0.1 or 0.15 to 0.3 or above. The ratio of V^2_L/V^2_R for typical values is of the order of 20, which seems to verify the common statement. Notice, however, that in the expression for (a bound on) the probability of failure (Eq. 4), there appear the products $K_R m^2_R V^2_R$ and $K_L m^2_L V^2_L$. Assuming the shape factors K_R and K_L are about equal, it is the relative values of $m^2_L V^2_L$ and $m^2_R V^2_R$ which should be compared. For a typical value[4] of the "central" safety factor $m_R/m_L = 5$, we find $m^2_L V^2_L/m^2_R V^2_R$ (or σ^2_L/σ^2_R, Fig. 1) about equal to unity for typical values of V_L and V_R. The relative contribution of resistance variation to the upper bound on the probability of failure is shown in Fig. 2 for several cases. The implication is that under typical situations the presently available data fail to justify the common adage that resistance variation has a relatively negligible influence upon the safety of a structure. The larger the value of the central safety

9. Thom, H. C. S., "On Extreme Winds in the United States," ASCE Trans., Vol. 126, 1960
10. C. E. B. Code, "Recommendations for an International Code of Practice for Reinforced Concrete," published by the American Concrete Institute and the Cement and Concrete Association
11. Rosenblueth, E., "Safety in Structural Design," Ch. 19, of a handbook on reinforced concrete design, edited by B. Bresler. To be published.

factor the more important resistance variation becomes. These conclusions, however, are subject to many qualifications, ranging from the lack of sufficient empirical data (both in number and kind), through the assumption that the coefficients of variation are approximately independent of means, to the inadequacy of the theoretical bound in expressing true reliability. Nonetheless the way is clear to begin adequate studies of such questions.



PROPORTION OF RESISTANCE CONTRIBUTION TO TOTAL P_f BOUND VERSUS V_R

Figure 2.

Effect of Dead Versus Live Load

It is possible to go on to investigate the factors which make up the loads and resistances in order to understand better the influences upon structural safety. The total load, for example, is often considered to be the sum of two or more independent loads, say dead load, D, plus live or service load, S. Larger dead load/live load ratios are often stated to be advantageous to safe structures. This conclusion presumably arises from engineers' reflection upon the relative uncertainty in dead and live loads. This uncertainty may be expressed in the variances. If the loads are additive, their means and variances are known to add to be the mean and variance of the total load, L,

$$m_L = m_D + m_S \qquad 5$$

$$\sigma_L^2 = \sigma_D^2 + \sigma_S^2 \qquad 6$$

For given values of the central safety margin, $m_R - m_L$, of the resistance characteristics, m_R and σ_R, and of the shape factors, K_R and K_L, Eq. 4 reveals that the reliability depends upon V^2_L alone. This coefficient can be expanded as follows:

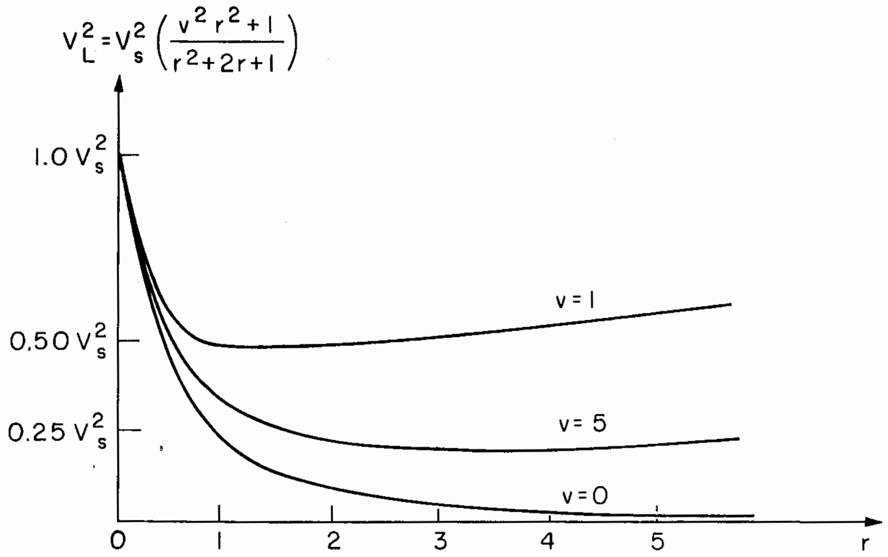
$$V_L^2 = \frac{\sigma_L^2}{m_L^2} = \frac{\sigma_D^2 + \sigma_S^2}{(m_D + m_S)^2} = \frac{V_D^2 m_D^2 + V_S^2 m_S^2}{(m_D + m_S)^2}$$

$$= \frac{V_S^2 m_S^2}{m_S^2} \quad \frac{\left(\frac{V_D^2}{V_S^2}\,\frac{m_D^2}{m_S^2} + 1\right)}{\left(\frac{m_D}{m_S} + 1\right)^2} \qquad 7$$

Replacing the ratio of central values of dead load to live load m_D/m_S by r, and the ratio of the corresponding coefficients of variation V_D/V_S by v,

$$V_L^2 = V_S^2 \left(\frac{v^2 r^2 + 1}{r^2 + 2r + 1} \right) \qquad 8$$

In virtually all cases $0 < v < 1$. The above function is plotted for several values of v in Figure 3. Notice that larger dead-load-to-live-load mean ratios, r, are advantageous to safe structures only up to the point where V^2_L reaches its minimum, that is when $r \leqslant 1/v^2$. Beyond this point, V^2_L is a slowly *increasing* function of r. In the range of values of v and r of usual interest however, the intuitive conclusions are analytically justified.



DEPENDENCE OF THE SAFETY UPON THE DEAD LOAD/LIVE LOAD RATIO

Figure 3

Effect of Resistance Factors

Resistance variation is also not solely a function of one factor. The dispersion in the ultimate moment capacity of nominally similar cross-sections of reinforced concrete beams, for example, depends on the variability of steel yield stress, of concrete compressive stress, and of fabrication. The last factor includes the rolling of the steel bar, the construction of the forms, and the placing of the steel and concrete. A simple approximation (see Appendix III) implies that the variance of the ultimate moment is the sum of the variances of the independent variables multiplied by the squares of corresponding "sensitivity factors." These latter factors indicate the effect of the variable upon the beam's capacity; they are simply the partial derivatives of the ultimate moment with respect to the variables (evaluated at the variables' mean values). Formally

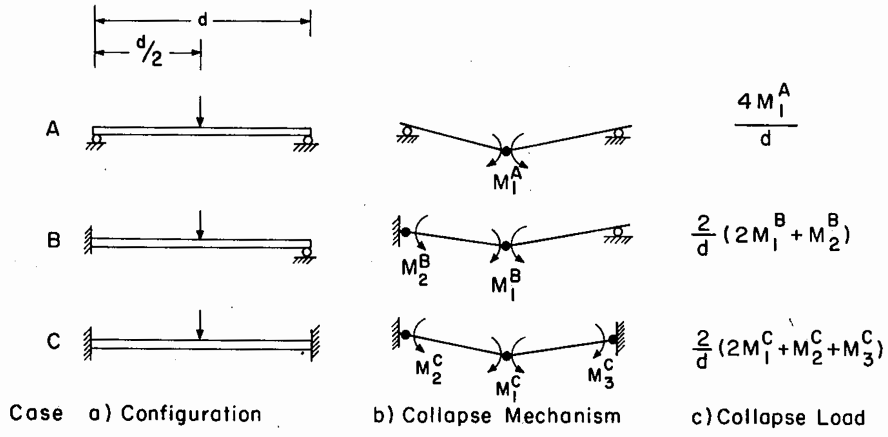
$$\sigma_R^2 \cong \sum \left(\frac{\partial R}{\partial X_i}\right)^2 \sigma_{X_i}^2 \qquad 9$$

in which the X_i's are the variables upon which the cross-sectional resistance depends (steel yield stress, depth of steel, width of beam, etc., in the case of reinforced concrete beam), the $\sigma^2_{X_i}$ are the variances of these variables, and the $\partial R/\partial X_i$ are the partial derivatives of the expression relating R to the X_i's (e.g., the equation for the ultimate moment capacity appearing in the ACI code[12]). Studies involving such approximations and typical values of the mean and variance of the variables can be immediately revealing. Even though concrete compressive strength variation is relatively large, for example, it has a negligible influence on the dispersion of the moment capacity of an under-reinforced cross-section.[13] This is true because the sensitivity factor is very low for this variable, sufficiently so to offset the high variance. Steel yield force and depth of steel contribute the dominant proportions of resistance variation in this case. Variation in latter arises, of course, owing to construction practice and workmanship variability. The shear capacity variance, on the contrary, may depend strongly upon concrete strength variability, depending upon the amount of transverse reinforcement. Such observations, crude as they may be, can have important implications on where an input of effort and expense (such as more strict inspection of certain operations) could most effectively improve structural safety.

Effect of Statical Indeterminacy

Yet another influence upon the safety of a member is thought to be its degree of statical indeterminacy. Reference to "multiple load paths" providing redundancy or to "back up" resistance is often heard in such discussions. Reliability theory can shed light too upon this influence. Consider the three prismatic beams, pictured in Figure 4. By proper choice of the beam sizes the nominal or mean capacities m_R^A, m_R^B and m_R^C can be made equal. Choose the mean moment capacity of a cross-section of Beam B equal to 2/3 that of Beam A, and choose the mean moment capacity of Beam C equal to 1/2 that of a cross-section of Beam A. Assuming perfect elasto-plastic behavior, the mean capacities are then equal (Figure 4):

12. ACI 318-63, "Building Code Requirements for Reinforced Concrete," published by the American Concrete Institute, June, 1963
13. Plum, N. M., "Quality Control of Concrete, Its Rational Basis and Economic Aspects," *Inst. Civ. Eng.*, Proc., Part I, 1953



DETERMINATE VERSUS INDETERMINATE SYSTEMS

Figure 4.

Case A: $$m_R^A = (4/d)m^A \qquad 10$$

Case B: $$m_R^B = (2/d)(2m^B + m^B) = (6/d)m^B = (4/d)m^A \qquad 11$$

Case C: $$m_R^C = (2/d)(2m^C + m^C + m^C) = (8/d)m^C = (4/d)m^A \qquad 12$$

In which m^A, m^B and m^C are the mean values of the moment capacities M_i^A, M_i^B and M_i^C of the cross-section of beams A, B and C, respectively, and d is the distance between the beam supports.

The variances of the beams' capacities may differ considerably, however. Assuming that the coefficients of variation, V^A, V^B, and V^C, of all beams' cross-section capacities are equal, the standard deviations of the cross-sections are

$$\sigma^A = m^A V^A \qquad 13$$

$$\sigma^B = m^B V^B = (2/3)m^A V^A = (2/3)\sigma^A \qquad 14$$

$$\sigma^C = m^C V^C = (1/2)m^A V^A = (1/2)\sigma^A \qquad 15$$

In which σ^A, σ^B and σ^C are the standard deviations of $M_1{}^A$, $M_i{}^B$ and $M_i{}^C$, respectively. The variances of the resistances of the beams follow from elementary probability theory as

$$\sigma^2_{R^A} = \left(\frac{4}{d}\right)^2 \left(\sigma^A\right)^2 \qquad 16$$

$$\sigma^2_{R^B} = \left(\frac{2}{d}\right)^2 \left\{ 4\left(\sigma^B\right)^2 + \left(\sigma^B\right)^2 \right\} \qquad 17$$

$$\sigma^2_{R^C} = \left(\frac{2}{d}\right)^2 \left\{ 4\left(\sigma^C\right)^2 + \left(\sigma^C\right)^2 + \left(\sigma^C\right)^2 \right\} \qquad 18$$

These results depend upon the assumption that the cross-sectional moment capacities are probabilistically independent, a notion which will be returned to shortly. In terms of σ^A or simply σ,

$$\sigma^2_{R^A} = \frac{16\,\sigma^2}{d^2} \qquad 19$$

$$\sigma^2_{R^B} = \frac{20}{d^2}\left(\sigma^B\right)^2 = \frac{40}{3d^2}\,\sigma^2 = \frac{13.3\,\sigma^2}{d^2} \qquad 20$$

$$\sigma^2_{R^C} = \frac{24}{d^2}\left(\sigma^C\right)^2 = \frac{12}{d^2}\,\sigma^2 \qquad 21$$

Under these conditions, although mean member resistances are equal, the variability in the resistance decreases, while reliability increases, as the degree of indeterminacy grows. Reliability theory here supports and quantifies the engineer's intuition about ductile, indeterminate structures. (Brittle structures do not share this advantage.)[14]

14. Shinozuka, M., "On Fatigue Failure of a Multiple-Load-Path Redundant Structures," Vol. 2, Proc. of the First Intern. Conf. on Fracture, Sendar, Japan, 1965

A critical assumption in the previous argument was that of probabilistic independence among the cross-section capacities within a beam. Such independence implies the assumption of a lack of correlation or coherence among these capacities compared to the capacities of all beams. In fact, owing to their common background (same batch of steel, same rolling and cooling experience, etc.), some degree of probabilistic dependence undoubtedly exists among the cross-section capacities within a beam. If the capacity of one cross-section in a beam is above the average among all such beams, it is very likely that other cross-sections of the same beam are also above average. In other words, if the capacity of one cross-section in a beam were tested and found to be a particular value, say 10% more (or less) than the population average, another cross-section in the same beam will most likely have a capacity very near that same value, rather than continuing to be about equally likely to be either higher or lower than the population average.

Such correlation is subject to experimental estimation. Its effect on the previous results can be large.[15-16] If as a limit, the correlation is perfect, it implies that all cross-sectional capacities (although still subject to variation from beam to beam) are equal within a beam. In Case B, this perfect dependence would imply $M_1{}^B = M_2{}^B$. In this case

$$\sigma^2_R{}^B = \left(\frac{6}{d}\right)^2 \left(\sigma^B\right)^2 = \frac{36}{d^2}\left(\frac{4}{9}\sigma^2\right) = \frac{16}{d^2}\sigma^2 \qquad 22$$

While in Case C

$$\sigma^2_R{}^C = \left(\frac{8}{d}\right)^2 \left(\sigma^C\right)^2 = \frac{64}{d^2}\left(\frac{1}{4}\sigma^2\right) = \frac{16}{d^2}\sigma^2 \qquad 23$$

In short, if this dependence or correlation is very nearly perfect, which intuition and initial evidence[15] suggests is the case, the variability of an indeterminate beam is no less than that of a determinate one. Consequently it is no more safe. It is *unconservative* to ignore this de-

15. Cornell, C. A., "Bounds on the Reliability of Structural Systems," *Journal of the Structural Division,* Proceedings of ASCE, Vol. 93, No. ST1, February, 1967
16. Tichy M. and Vorlicek M., *Safety of Reinforced Concrete Framed Structures,* Proceedings of the International Symposium on Flexural Mechanics of Reinforced Concrete, Miami, Fla., November, 1964, ASCE, 1965

pendence in a safety study of redundant structures. Reliability theory thus may reveal that there exist unsuspected influences on structural safety. Probabilistic independence versus dependence is one such influence that appears throughout more thorough reliability studies.[15-16-17]

LENGTH-OF-LIFE INFLUENCE ON STRUCTURAL SAFETY

A structure which must serve for a longer time should be designed more conservatively. This common axiom too is easily investigated through some elementary ideas of structural reliability analysis. Given the reliability, P_S, of a member or structure subjected to a single load, the reliability of the same structure subjected to a sequence of such loads can be estimated as follows.

Elementary probability theory states that the probability of two independent events occurring is the product of their probabilities. So the probability of getting two successive heads on two flips of a well-balanced coin is (½) (½) or ¼. If the events are not independent, but dependent or related in some way, the second factor must be replaced by the *conditional probability* of the second event given that the first has occurred. So the probability that the result of the throw of the die is both odd *and* less than four is (½) (⅔)= ⅓. The first factor is the probability of an odd number, 1, 3, or 5. The second factor is the conditional probability of a number less than four *given* that the outcome was odd, 1 or 3. (The answer is *not* (½) (½), the probability of an odd number times the probability of a number less than four.) So, to determine the probability that a structure survives two load applications, P_{S_2}, we must multiply the probability of a survival on the first load application, P_{S_2}, times the conditional probability of survival on the second given survival on the first, $P[s_2 \mid s_1]$. (Read $P[A \mid B]$ as the probability of event A given that event B occurred.)

$$P_{s_2} = P_s P[s_2 \mid s_1] \tag{24}$$

17. Ang, A. H. S., and Amin, M., "Studies of Probabilistic Safety Analysis of Structural Systems", University of Illinois Civil Engg. Studies, Struct. Res. Series No. 320, 1967

(If the successive loads are taken to be the annual maximum loads, the number of loads can be equated to years of service.)

The value of this conditional probability may range from a number as low as P_s to one as high as unity.[5–15] If the successive loads are independent, the conditional probability *may* be about P_s. Then

$$P_{s_2} \cong P_s P_s = (1 - P_f)(1 - P_f) \cong 1 - 2P_f \tag{25}$$

For n loads the result becomes

$$P_{s_n} \cong 1 - n\,P_f \tag{26}$$

In this case the reliability of the structure (now defined as the likelihood that it will perform satisfactorily throughout its lifetime) decays almost linearly with the number of anticipated loads. This conclusion is a quantitative verification of the statement opening this section. It now might be re-stated: "if a structure must serve satisfactorily for n years, it should be designed with a probability of unsatisfactory performance under one annual maximum load, i.e., with a P_f, equal to about one-half that of a similar structure which need serve only for n/2 years."

The situation may not, however, dictate that the conditional reliability, $P[s_2 \mid s_1]$ is equal to P_s. For example, if there is virtually no variation in the load, but resistance variation is large, then knowledge that the structure performed adequately under the first load almost guarantees that a low resistance does not exist and hence that in following years the performance will be equally satisfactory. The first load has in essence performed a proof test. This case is illustrated in Figure 5.

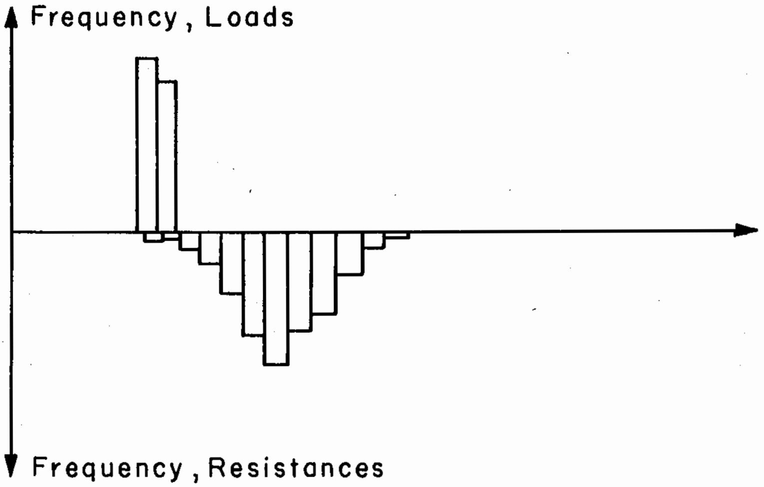
In this case

$$P\left[s_2 \mid s_1\right] \cong 1 \text{ and}$$

$$P_{s_2} \cong P_{s_1} = 1 - P_f \tag{27}$$

and more generally

$$P_{s_n} \cong 1 - P_f \qquad 28$$



SMALL LOAD VARIABILITY SITUATION $V_L \ll V_R$

Figure 5.

Such a situation occurs in the static design of a dam. The maximum load each year may be always very nearly the capacity of the dam, while the resistance involves significant uncertainty in material properties, abutments strengths, etc. Also, if the significant uncertainty in a structure's resistance lies not in material variability, but in possible design or workmanship errors, blunders, or omissions, the first major load (even if it is only an average one) will be likely to cause the failure *if* it is going to occur. Many construction failures support this observation. In these situations the reliability is not as length-of-life sensitive. (Time-dependent or deteriorating strength is not under consideration here, although this, too, can be treated.[4-15-18])

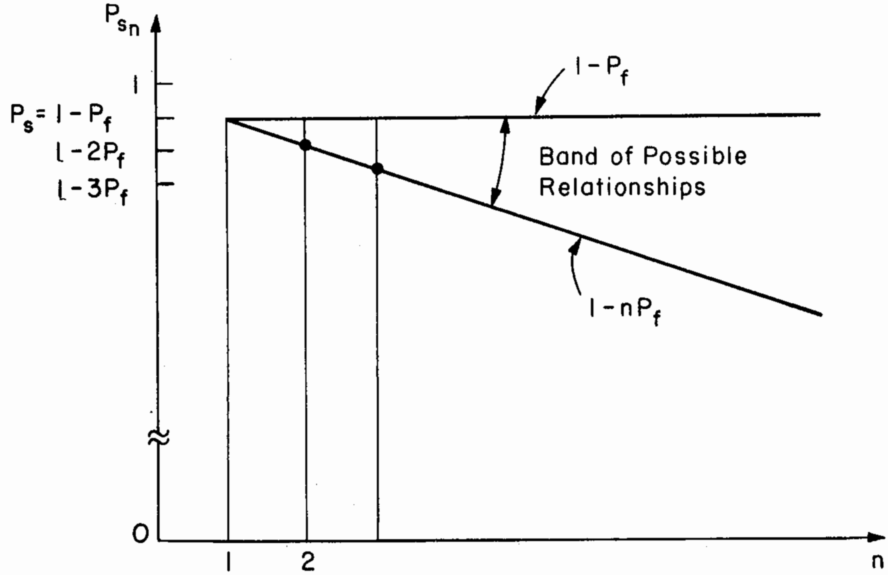
18. Leve, H. L., "Reliability Framework for Structural Design," *Journal of the Structural Division,* Proceedings of ASCE, Vol 89, No. ST1, August, 1963

Quite another situation may lead to the conclusion that $P[s_2 | s_1] \doteq 1$. Here, the relatively unfamiliar notion of probabilistic dependence is again the critical factor. If the successive loads are highly correlated, as say dead loads are, then knowledge that in the first year the maximum load didn't exceed the capacity implies that future loads (which are not necessarily well known, but, owing to dependence, will be of very nearly the same magnitude as the first) will also fail to exceed the capacity. The conclusion is, then, the same as Eq. 28.

In general, for n loads the likelihood of satisfactory performance lies between

$$1 - nP_f \leq P_{s_n} \leq 1 - P_f \qquad 29$$

as illustrated in Figure 6. The lower bound[4-15] $1 - nP_f$ may estimate well the common case where load variability, σ_L, exceeds significantly resistance variability, and when successive load values may be treated as independent.

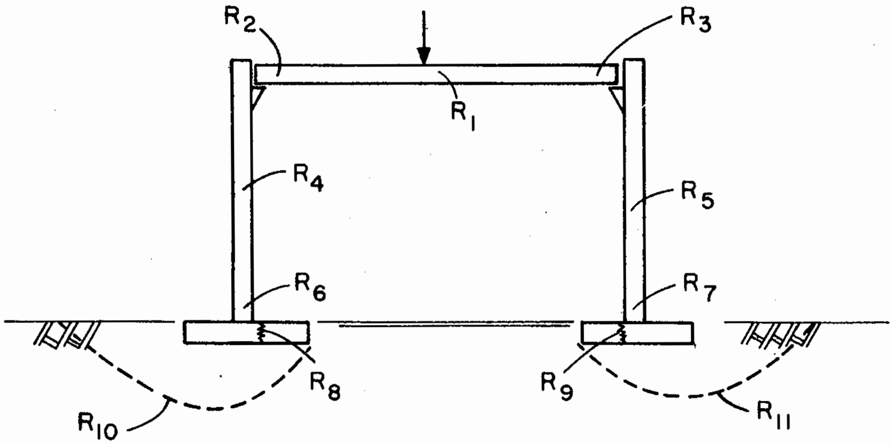


INFLUENCE OF TIME ON STRUCTURAL SAFETY

Figure 6.

STRUCTURAL SYSTEM RELIABILITY UNDER GIVEN LOAD

When structures become more complicated than single members, their safety may be jeopardized by failure of any of several potential modes of failure. A determinate truss will fail if any bar fails. A building may collapse owing to one of several potential weaknesses in its frame, or due to a shear failure of a footing, or because of a foundation stability problem (Figure 7). The analogy is sometimes made to a chain of many links in order to argue that the structural system is no stronger than its weakest mode. Hence the number of modes and the safety of *each* member or mode must influence the total system's reliability.



R_1 : beam in bending
R_2, R_3 : beam in shear
R_4, R_5 : column instability
R_6, R_7 : column local buckling
R_8, R_9 : footing in shear
R_{10}, R_{11} : foundation instability

SCHEMATIC COLLECTION OF POTENTIAL MODES OF COLLAPSE

Figure 7.

These effects are best studied in isolation of loading variation, the influence of which will be discussed in the next section. Assume then that the load is given exactly, or that it demonstrates relatively little dispersion about its known central value, or that for artificial reasons, such as a legally specified maximum load, interest lies only in the safety of the structure under a given load.

By an argument parallel to that in the previous section it is clear that Ps, the likelihood of survival of two modes of the system, is the product of $P_{1}s$, the reliability of the first mode or member, times $P[_2s \mid {}_1s]$ the conditional reliability of the second mode *given* survival of the first.

$$P_s = P_{{}_1s} \, P\left[{}_2s \mid {}_1s\right] \qquad 30$$

Since the load is fixed, if the capacities of the first and second modes are probabilistically independent, survival of the first contains no information about the safety of the second. Thus $P[_2s \mid {}_1s] = P_2s$, the reliability of the second mode. Then

$$P_s = P_{{}_1s} P_{{}_2s} = (1 - P_{{}_2f})(1 - P_{{}_2f}) \cong 1 - (P_{{}_1f} + P_{{}_2f}) \qquad 31$$

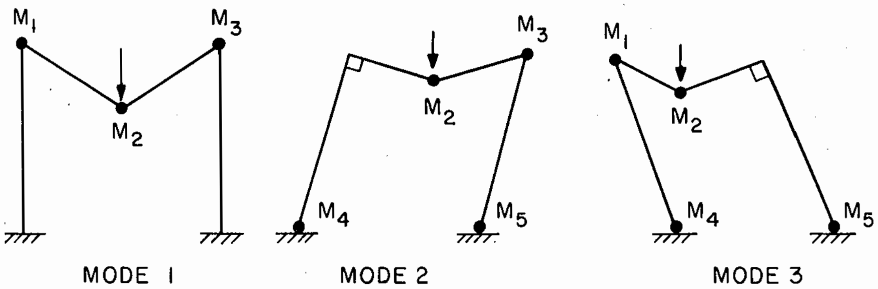
in which $P_{1}f$ and $P_{2}f$ are, respectively, the probabilities that the capacities of modes 1 and 2 are insufficient. Under these circumstances the system probability of failure, P_f, is approximately equal to the sum of the m modes' probabilities of failure.[4-15]

$$P_f \cong \sum_{i=1}^{m} P_{{}_if} \qquad 32$$

The conclusion is that each mode "contributes" to the system failure likelihood in an almost additive way. Under these circumstances, more complex systems can be made as safe as simple systems only if one increases the individual modes' reliabilities in order to maintain the *sum* of their failure probabilities at the desired value.

In fact, the assumption of independence of the capacities of the modes may not be realistic in many cases. It is true that in the "structure" in Figure 7 the foundation instability modes, R_{10} and R_{11}, which depend on soil properties, may be unrelated to the strength of the beam in bending, R_1. But, the strengths of the columns and the beam may very well be correlated with one another because of common production and fabrication histories.

More directly, in some structures modal resistances may be probabilistically dependent because they are functionally related to the same cross-section capacities.[15-16] Consider, for example, the three simple collapse modes of the portal frame in Figure 8. The first and second modes have resistances which could be treated as the indeterminate beams in Figure 4 were. But now both modal resistances depend in part upon the same cross-section capacities, M_2 and M_3. Given that the capacity of one mode is higher than average, the capacity of the other is very likely to be similarly located with respect to the population of all such modes, simply because M_2 and M_3 are probably greater than average values. Additionally, as discussed in an earlier section and above, the various cross-section capacities are undoubtedly correlated. Owing to their occurrence in the same member or in neighboring members, they are likely to have undergone similar previous histories.



COLLAPSE MODES SHARING COMMON HINGES

Figure 8.

If, as a limiting case, this dependence is perfect, the fact that the mode most likely to fail survived will imply that all others survived also. Then all the conditional survival probabilities of the remaining modes are unity. For two modes,

$$P_s = P_{1^s} \, P\left[{}_2s \,\middle|\, {}_1s\right] = P_{1^s} \tag{33}$$

in which P_{1^s} is the smaller of the P_is. Or, in general,[15]

$$P_s = \min_i \, (P_{i^s}) = 1 - \max_i \, (P_{i^f}) \tag{34}$$

or

$$P_f = \max_i \, (P_{i^f}) \tag{35}$$

In reality the dependence among some modal resistances will be strong and among others it will be weak. In some cases one might obtain a good estimate of the system reliability by looking first at appropriate groupings or sub-systems of highly dependent modes. In Figure 7 one might group together the modal resistances one through seven, namely those associated with the steel frame. The probability of failure of such a subsystem would be very nearly the maximum of all the modal probabilities within the subsystem, one of the column instability modes, say, in Figure 7. There may be, however, little correlation between resistances in different subsystems. Hence, the probability of failure of the system would be approximately equal to the sum of the subsystem probabilities of failure. In Figure 7, the sub-systems in addition to the steel frame might be the concrete footings, R_8 and R_9, and the soil instability modes, R_{10} and R_{11}. Then the system reliability under a given load would be approximately

$$P_s \cong 1 - P_{4^f} - P_{8^f} - P_{10^f} \tag{36}$$

In general, about any system or subsystem it can be stated that,

$$1 - \sum_{i=1}^{m} P_{f_i} \leq P_s \leq 1 - \max\left(P_{f_i}\right) \qquad 37$$

Again probabilistic dependence plays a key role in determining the safety of structures. In this case, however, a conservative lower bound is found by ignoring it.

STRUCTURAL RELIABILITY UNDER AN UNCERTAIN LOAD

When the load is not known to be a given value but displays marked variation about its central value, the results of the previous section are altered. In fact, they are simplified. For in this case, even if the capacities of the various modes are independent, probabilistic dependence is set up among the survival *events* by fact that each mode is subjected to the same load, or same loading environment. This *environmental* dependence[15-18-19] causes the conditional probabilities such as $P[{}_2s \mid {}_1s]$ to be very close to unity, when Eq. 35 again holds:

$$P_f \cong \max\left(P_{f_i}\right) \qquad 35$$

This dependence is best understood by considering the limiting case when the resistances are all fixed in value, i.e., lacking any variability. Then, if it is known the weakest mode survived, it is known that the load was less than *all* the resistances, implying all resistances survived. More generally, this condition holds in approximation when the variances of the resistances are small compared to those of the load. This environmental dependence operates, to a greater or lesser extent, any time that the load displays some variation. The degree of its effect is not well understood except in the limiting case mentioned above where the variation in the resistances (or at least in those resistances with the larger P_{f_i}) is negligible compared to that in the load.

Nonetheless, the multitude of factors causing dependence among the modal survival events (namely dependence among cross-section re-

19. Moses, F. and Kinser, D. E., "Analysis of Structural Reliability", private communications

sistances, dependence among modal resistances, and environmental dependence) suggests that in reality the safety of complex, multiple-mode-of-failure systems is closer to $1 - \max(P_i f)$, the reliability of its most unreliable mode, than to $1 - \Sigma P_i f$, a cumulative composite of the many modes of failure. The immediate design implication is that it does not create a significantly safer structure if one designs an already non-critical mode to be even more conservative. The design effort should go into the most unreliable mode, but only until it is safer than the next least reliable mode of failure, when any additional expense should go into the latter. Readers familiar with critical path scheduling will note the analogy[20] of this procedure with the one used in improving a critical path schedule.

Combining this conclusion with the result judged to be "most common" in the section on sequences of loads, it is concluded that the reliability of a typical structural system over a period of n years is

$$P_{s_n} \cong 1 - n \max\left(P_{f_i}\right) \qquad 38$$

in which the $P_i f$ are the probabilities of failure of the various modes with respect to a single (annual maximum) load.

It is important to realize that these conclusions extend beyond the study of total collapse. A code (such as ACI 63[12]) might choose to define as failure of the structural system the yielding or crushing of any cross-section of a frame. In this case, each potential yield region becomes a possible mode of failure. Assuming consistent levels of reliability are being sought from building to building, the fact that the code does not require higher safety factors as the number of such cross-sections increases can be justified only if high dependence among survival events exists (i.e., only if P_S equals $1 - \max(P_i f)$ not $1 - \Sigma P_i f$.) Unsatisfactory performance with respect to excessive cracking or excessive lateral sway may be defined as a "serviceability failure" and studied in the same way.[21] In these situations, the determination of the probability of no failure may not be sufficient, because some such failures are

20. Byers, William G., private communications.

21. Robertson, L. E., and Chen, P. W., "Glass Design and Building Code Implications of Recent Wind Load Research for Tall Buildings," Building Research Institute Fall Conference, Nov. 15-17, 1966

tolerable. It becomes important to estimate the number of such failures, for example the number of windows which might crack under high wind loads. If dependence is in fact high, the failure of one mode (window) might imply that several or many more have failed too. (Physically, the same extreme gust, if it occurs, is likely to break several of the weaker windows not just the weakest. These failure events, having been caused by the same gust, are not independent.)

SUMMARY AND CONCLUSIONS

While the effort is not exhaustive (multiple kinds of loads, for example, are not considered), a number of the commonly stated factors affecting structural safety have been reviewed through elementary reliability analysis. These include dispersion in strength and load, the relative influence of resistance and load uncertainty, the dead load/live load ratio, statical indeterminacy, the length of the structure's lifetime, and the complexity of the system. In most cases quantitative verifications have been demonstrated, along with indications as to the degree of significance and conditions for validity of these notions. In still other cases some less familiar factors, most notably probabilistic dependence, have been found critical. The relative variability of load and resistance variation has been seen to be a significant factor in determining the degree to which the length of lifetime and the complexity of the structure affect its safety. The likelihood of a structure's failure may depend most strongly on its most unreliable potential mode of failure rather than on the total of the various modes' unreliabilities.

APPENDIX I — NOTATION

The following symbols are used in this paper:

b = positive constant

D = dead load

d = distance between beam supports

$K_{\bar{R}}$, $K_{\bar{L}}$ = constants depending on the shape of the probability distribution of R and L, respectively

L = load

M_1^A, M_i^B, M_i^C = moment capacities of the cross-sections of beams of A, B, and C respectively

M_i = moment capacity of cross-section i

m^A, m^B, m^C = mean value of M_1^A, M_i^B and M_i^C, respectively

$m_{\bar{D}}$, m_L, m_R, m_S = mean values of D, L, R and S, respectively

m_{R^A}, m_{R^B}, m_{R^C} = nominal or mean capacity of beams of A, B and C respectively

m_{X_i} = mean value of X_i

n = number of loads, or time units (years, say)

P_f = probability of failure

P_{if} = probability of failure of the i^{th} mode, or member

P_{is} = reliability of the i^{th} mode, or member

P_S = reliability (probability of survival)

P_{S_i} = probability that the structure survives i load applications

$P[s_2 \mid s_1]$ = conditional reliability at the second load application, given survival on the first

$P[{}_2s \mid {}_1s]$ = conditional reliability of mode 2 given survival of mode 1

$Q = R - L$ = safety margin

R = capacity or resistance

r = ratio m_D/m_S

R^A, R^B, R^C = capacity of beams A, B and C respectively

S = live load or service load

V^A, V^B, V^C = coefficient of variation of $M_1{}^A$, $M_i{}^B$ and $M_i{}^C$, respectively

V_D, V_L, V_R, V_X = coefficient of variation of D, L, R and S, respectively

x_i = observed value of the load

X_1, X_2, . . . = Variables on which the cross-sectional resistance depends

$\sigma = \sigma^A$

σ^A, σ^B, σ^C = standard deviation of MA, MB and MC, respectively

σ_D, σ_L, σ_R, σ_S = standard deviation of D, L, R and S, respectively

$\sigma_R{}^A$, $\sigma_R{}^B$, $\sigma_R{}^C$ = standard devaiation of the capacity of beams A, B, and C respectively.

APPENDIX II

It will be shown in this section that, for positive load L and resistance R[22]

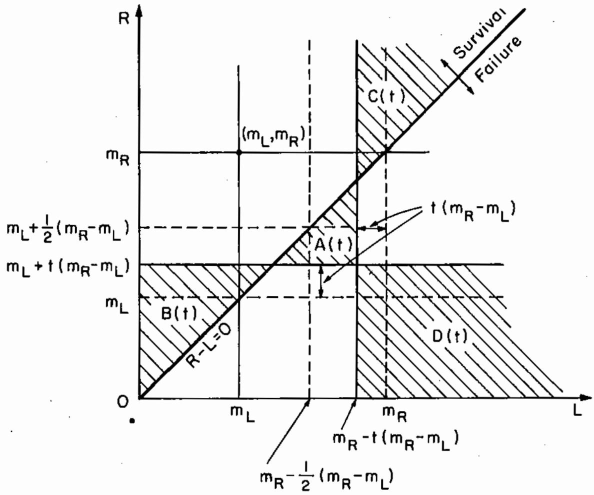
$$P_f \le \frac{K_R \sigma_R^2 + K_L \sigma_L^2}{(m_R - m_L)^2} \qquad 3$$

in which K_R and K_L are factors, which depend on the shape of the probability density function of R and L, resp.

Proof:

(1) From the representation of the two-dimensional sample space (Figure 9), for any $t \le ½$

22. Vanmarcke, E. H., "Reliability in the Design of Structures," thesis presented to the faculty of the University of Delaware in partial fulfillment of the degree of Master of Science in Civil Engineering



SAMPLE SPACE OF LOAD AND RESISTANCE

Figure 9.

$$P_f = P\left[R - L \leq 0\right] = P\left[L \geq m_R - t(m_R - m_L)\right] + P\left[R \leq m_L + t(m_R - m_L)\right] - \varepsilon(t) \qquad 39$$

in which

$$\varepsilon(t) = P\left[B(t)\right] + P\left[C(t)\right] + P\left[D(t)\right] - P\left[A(t)\right] \qquad 40$$

It follows that

$$P_f \leq P\left[L \geq m_R - t(m_R - m_L)\right] + P\left[R \leq m_L + t(m_R - m_L)\right] \qquad 41$$

if $\varepsilon(t) \geq 0$

Note that for $t = ½, \varepsilon(t)$ is strictly positive, since the only negative term in (40) vanishes.

Both terms on the right hand side of (41) can be approximated by conservative inequalities of the Chebychev type.

(2) For any random variable X with mean m_X and standard deviation σ_X, and for any constant $a \leq m_X$

$$P\left[X \leq a\right] = P\left[m_X - X \geq \frac{(m_X - a)\,\sigma_X}{\sigma_X}\right]$$

$$= c_X\, P\left[|m_X - X| \geq \frac{(m_X - a)\,\sigma_X}{\sigma_X}\right] \qquad 42$$

in which $c_X \leq 1$. If the probability density function is symmetrical about m_X, then $c_X = ½$.

Also, for any such random variable X, having a finite variance and for every $b > 0$, the Chebychev inequality holds.[23-24-5]

$$P\left[|m_X - X| \geq b\,\sigma_X\right] \leq b^{-2} \qquad 43$$

A less conservative approximation, known as the Gauss inequality, applies if the distribution of X is known to be unimodal with mode $\mathcal{M}_X$. Then[23-24]

$$P\left[|m_X - X| \geq b\,\sigma_X\right] \leq \frac{4}{9}\,\frac{(1 + \lambda_X^{\,2})}{\left(1 - \frac{|\lambda_X|}{b}\right)^2}\, b^{-2} \qquad 44$$

provided $b > |\lambda_X|$, where $\lambda_X = \dfrac{m_X - \mathcal{M}_X}{\sigma_X}$ 45

In particular, when mean and mode coincide (e.g., when X is symmetrically distributed), then $\lambda_X = 0$, and

$$P\left[|m_X - X| \geq b\,\sigma_X\right] \leq \frac{4}{9}\, b^{-2} \qquad 46$$

All these bounds can be put in the form

$$P\left[|m_X - X| \geq b\,\sigma_X\right] \leq k_X\, b^{-2} \qquad 47$$

23. Mood, A. M., "Introduction to the Theory of Statistics," McGraw Hill, 1950
24. Wadsworth and Bryan, "Introduction to Probability and Random Variables," McGraw Hill, 1960

in which $k_X = 1$ corresponds to the Chebychev inequality, and

$$k_X = \frac{4}{9} \frac{(1 + \lambda_X^2)}{(1 - \frac{|\lambda_X|}{b})^2} \qquad 48$$

corresponds to the Gauss inequality.

Finally, introducing (47) into (42), with $b = \frac{m_X - a}{\sigma_X}$ yields

$$P\left[X \leq a\right] \leq \frac{c_X k_X \sigma_X^2}{(m_X - a)^2} \qquad 49$$

In an analogous way it can be shown that, for any random variable X with mean m_X and finite variance σ_X^2, and for any $a' > m_X$

$$P\left[X \geq a'\right] \leq \frac{c_X k_X \sigma_X^2}{(a' - m_X)^2} \qquad 50$$

(3) From (49) and (50), in which a is substituted by $m_L + t(m_R - m_L)$, a′ by $m_R - t(m_R - m_L)$ and X by R and L, resp.,

$$P\left[L \geq m_R - t(m_R - m_L)\right] + P\left[R \leq m_L + t(m_R - m_L)\right]$$

$$\leq \frac{1}{(1-t)^2} \frac{c_L k_L \sigma_L^2 + c_R k_R \sigma_R^2}{(m_R - m_L)^2} = \frac{K_L \sigma_L^2 + K_R \sigma_R^2}{(m_R - m_L)^2} \qquad 51$$

in which

$$K_L = \frac{c_L k_L}{(1-t)^2} \;; \quad K_R = \frac{c_R k_R}{(1-t)^2} \qquad 52$$

Introducing (51) into (41) completes the formal proof of the existence of bound (2) for some c_R, c_L, k_R, $k_L \leqslant 1$ and some $t \leqslant ½$.

(4) In this section some simplified approximations for K_L and K_R are suggested for reliability practice. The value of $\varepsilon(t)$ gradually decreases as t decreases (see figure 9) and bound (3) becomes less conservative. Strong evidence suggests that (3) still holds true, when $t = 0$ in (52).

(i) When the variation of the load L is negligible compared to the strength variation ($\sigma_L = 0$, $L = m_L$), then, from (49)

$$P_f = P\left[R \leq m_L\right] \leq \frac{c_R k_R \sigma_R^2}{(m_R - m_L)^2} \tag{53}$$

(ii) When the strength is a deterministic quantity ($\sigma_R = 0$, $R = m_R$), then, from (51)

$$P_F = P\left[L \geq m_R\right] \leq \frac{c_L k_L \sigma_L^2}{(m_R - m_L)^2} \tag{54}$$

Thus, when either σ_L or σ_R approaches zero, (3) is seen to hold for values of $K_L = c_L k_L$ and $K_R = c_R k_R$, i.e., $t = 0$.

(iii) When R and L are independent random variables (as is commonly assumed in safety analysis), then the variance of the safety margin $M = R - L$, equals the sum of the variances of R and L, $\sigma_M^2 = \sigma_R^2 + \sigma_L^2$. From (49), in which X is substituted by M and a by 0,

$$P_F = P\left[M \leq 0\right] \leq \frac{c_M k_M (\sigma_R^2 + \sigma_L^2)}{(m_R - m_L)^2} \tag{55}$$

Disregarding the effect of the factors c_R, c_L and c_M, it can be argued that, whenever $k_M \leqslant k_R = k_L$, t may again assume the value zero in (52). This follows from the fact that the values K_R and K_L may be chosen such that bound (55) equals bound (3). The central limit theorem indicates that, when L and R have unimodal density functions with skewness characteristics λ_R and λ_L, the density function of their difference tends to remain unimodal, but with a lower skewness characteristic λ_M. It is precisely these skewness characteristics which determine the factors k_R, k_L and k_M when the density functions are unimodal. In case nothing can be said about the distributations of R and L, then $k_R = k_M = 1$ again leads to a choice $t = 0$.

Thus, substituting c_L and c_R by unity, their upper bound, and putting $t = 0$ in (52), the values of K_L and K_R, to be used in (3) become $K_L = k_L$ and $K_R = k_R$. Note that, when the distribution of R is unimodal with mode $\mathcal{M}_R$

$$K_R = k_R = \frac{4}{9} \frac{(1 + \lambda_R^2)}{\left(1 - \frac{|\lambda_R| \sigma_R}{m_R - m_L}\right)^2} \simeq \frac{4}{9}(1 + \lambda_R^2) \qquad 56$$

because in practice

$$|\lambda_R| = \frac{|m_R - \mathcal{M}_R|}{\sigma_R} << \frac{m_R - m_L}{\sigma_R} \qquad 57$$

Similarly, when the distribution of L is unimodal with mode $\mathcal{M}_L$

$$K_L = k_L \simeq \frac{4}{9}(1 + \lambda_L^2) \qquad 58$$

because in practice

$$|\lambda_L| = \frac{|m_L - \mathcal{M}_L|}{\sigma_L} << \frac{m_R - m_L}{\sigma_L} \qquad 59$$

The Chebychev inequality guarantees that the values K_R and K_L do not exceed unity.

In summary it is suggested that Equation 3 be used with $K_R = 1$ if nothing is known about the distribution of R, and with $K_R = \frac{4}{9}(1 + \lambda_R^2)$ of R is known to have a unimodal distribution. Analogous conclusions hold for K_L.

APPENDIX III

The cross-sectional resistance R is a function of the random variables X_i

$$R = R(X_1, X_2, X_3, \ldots,)$$

A multidimensional Taylor expansion about the mean value m_{X_i} is suggested by observing the fact that the X_i are likely to lie close to m_{X_i} if their variability is not substantial.

$$R\ (X_1, X_2, \ldots) = R(m_{X_1}, ,m_{X_2}, \ldots)$$

$$+ \sum_i (X_i - m_{X_i}) \left.\frac{\partial R}{\partial X_i}\right|_{m_{X_i}} + \ldots \qquad 60$$

Taking the expectation of both sides

$$E(R) \cong R(m_{X_1}, m_{X_2}, \ldots)$$

since $E(X_i - m_{X_i}) = 0$. 61

Also, since $\mathrm{Var}\left[R(m_{X_1}, m_{X_2}, \ldots)\right] = 0$,

$$\mathrm{Var}\,[R] = \mathrm{Var}\left[\sum_i (X_i - m_{X_i}) \left.\frac{\partial R}{\partial X_i}\right|_{m_{X_i}}\right]$$

$$= \sum_i \left(\left.\frac{\partial R}{\partial X_i}\right|_{m_{X_i}}\right)^2 \mathrm{Var}[X_i] \qquad 62$$

$$+ \sum_i \sum_{j \neq i} \left(\left.\frac{\partial R}{\partial X_j}\right|_{m_{X_j}}\right)\left(\left.\frac{\partial R}{\partial X_j}\right|_{m_{X_j}}\right) \mathrm{cov}(X_i, X_j)$$

If the X_i are uncorrelated, then cov $(X_i, X_j) = 0$ $(i \neq j)$, and the approximation is simply

$$\mathrm{Var}[R] = \sum_i \left(\frac{\partial R}{\partial X_i} \bigg|_{m_{X_i}} \right)^2 \mathrm{Var}[X_i] \qquad 63$$

This result merely states that the variability of the cross-sectional resistance is the net result of contributions of variability of all related parameters. The contribution of each component depends on its variability and on its relative importance in determining the overall resistance.

THE WARRAGAMBA PIPELINES STILLING BASIN

By

THOMAS F. CHEYER*

(Presented at the meeting of the Hydraulics Section, Boston Society of Civil Engineers, May 3, 1967.)

INTRODUCTION

Approximately 2.8 million people are now served by the Metropolitan Water, Sewerage and Drainage Board of Sydney, Australia, at an average per capita rate of 98 gallons[1] per day, resulting in an annual average consumption of 275 mgd (million gallons per day). The present maximum daily consumption during the year may reach 500 mgd. Projected annual average consumption for the area served by the Board is estimated to be 455 mgd in the year 1985 and 745 mgd in 2010.

The Warragamba Catchment supplies the major portion of the present water needs of the Sydney area. This catchment, with its 3,480 square miles of drainage area, is tapped for water supply by the newly constructed Warragamba Dam, creating Lake Burragorang with its 452,505 million gallons capacity. The estimated safe draft of this supply is 263 mgd, or about 74 per cent of the total developed supply capability of the Sydney system. Thus, the Warragamba Catchment is of prime importance in meeting the water supply needs of the metropolitan Sydney area for many years to come.

Camp, Dresser & McKee was retained by the Board in August, 1964, to make an engineering investigation of the Board's water supplies with the primary emphasis on preparing preliminary plans for the treatment of the Warragamba supply. The investigation was completed in August, 1965, and Camp, Dresser & McKee is at present preparing final plans for a water treatment facility with a capacity of 700 mgd. This paper will report on one of the many studies conducted in conjunction with the design of this treatment plant, called the Prospect Water Treatment Works.

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1. The term "gallon" as used in this article refers to the Imperial gallon, equivalent to a volume of 0.16 cubic feet.

THE WARRAGAMBA PIPELINES

Warragamba Dam is situated approximately 40 miles west of Sydney. Water is transported by pipeline to Prospect Reservoir, a storage and balancing reservoir located approximately 8 miles west of the city. In addition, water from the Metropolitan Catchment is also brought to Prospect Reservoir, but by open canal. From Prospect, the supply is distributed to the service reservoirs feeding the various reticulation systems. A schematic representation of the supply is depicted in Fig. 1. The proposed water treatment plant will be constructed adjacent to Prospect Reservoir, thus being located at the junction of the Warragamba and Metropolitan supplies.

The original pipeline between Warragamba and Prospect (See Fig. 1) was approximately 90,250 ft in length and consisted of 13,650 ft of two parallel cement-lined 106-in diameter conduits, emanating from Warragamba Dam and tying into a single 84-in diameter cement-lined conduit, which completed the connection to Prospect Reservoir. The maximum static head on the pipeline is 191 ft, giving a maximum flow capability of 190 mgd, or less than the net safe draft of the catchment. This pipeline was built as an interim transmission main.

Additional transmission capacity from the Warragamba Catchment is required in order to keep up with a rapidly increasing demand (consumption is doubling approximately every twenty to twenty-five years). In fact, there is difficulty in meeting peak system demands even now. Accordingly, the final phase of the pipeline is now proceeding with the addition of a 120-in pipeline, which will increase the maximum capacity to 550 mgd. Design of this main has been completed by the Sydney Water Board and construction is underway. It is anticipated that the new pipeline and outlet works will be completed by 1969.

The 120-in mild steel, cement-lined pipeline ties into one of the existing 106-in diameter mains near the source (See Fig. 1) and parallels the existing 84-in main for all but the last 2,000 ft. At this point the two pipelines are interconnected and two 120-in mains traverse the southern perimeter of Prospect Reservoir, a distance of approximately 2,000 ft to the new outlet works. From the outlet works water can enter either Prospect Reservoir or the proposed treatment plant.

A plot of the pipeline characteristics is shown in Fig.2. From the figure it can be seen that for Warragamba Reservoir full, the maximum discharge is about 550 mgd, while for the reservoir at elevation R.L. 300 (85 ft available head) the maximum discharge is about 380 mgd.

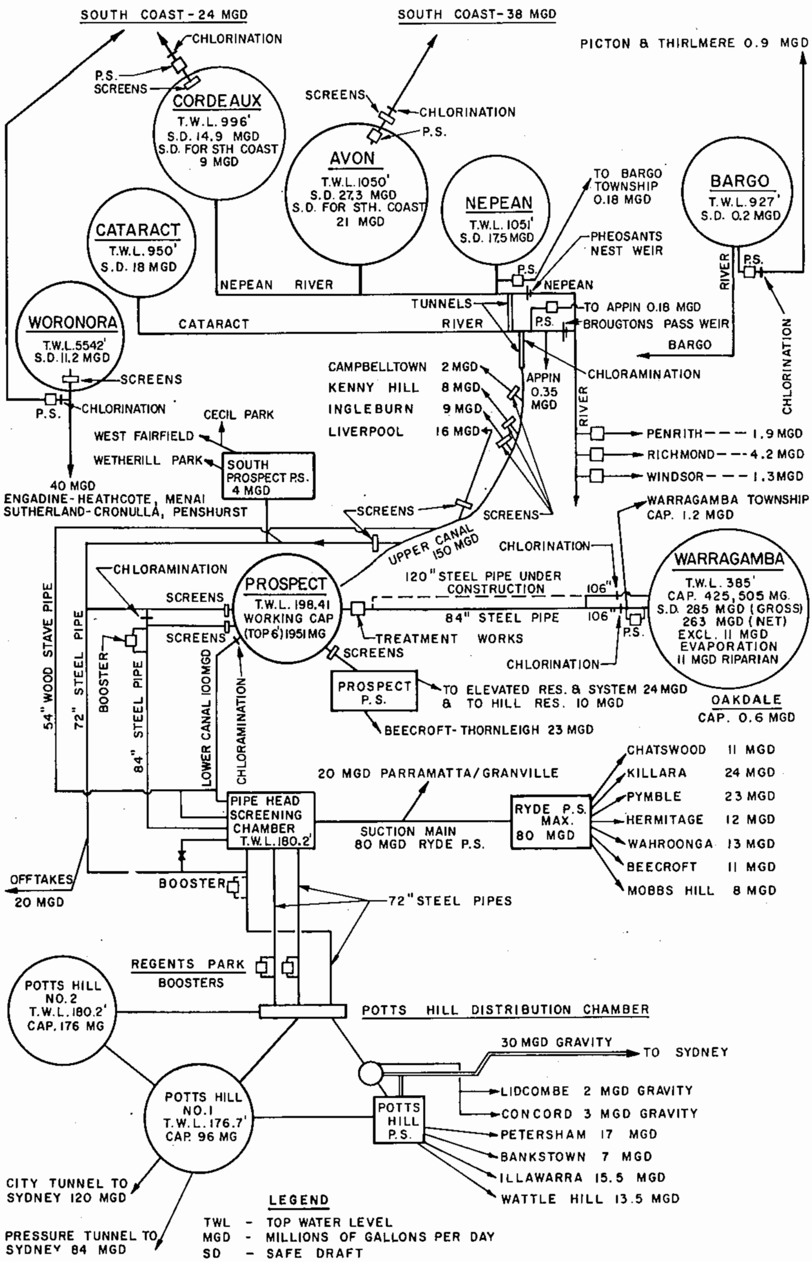


Fig. 1 — Schematic of Sydney Water Supply System

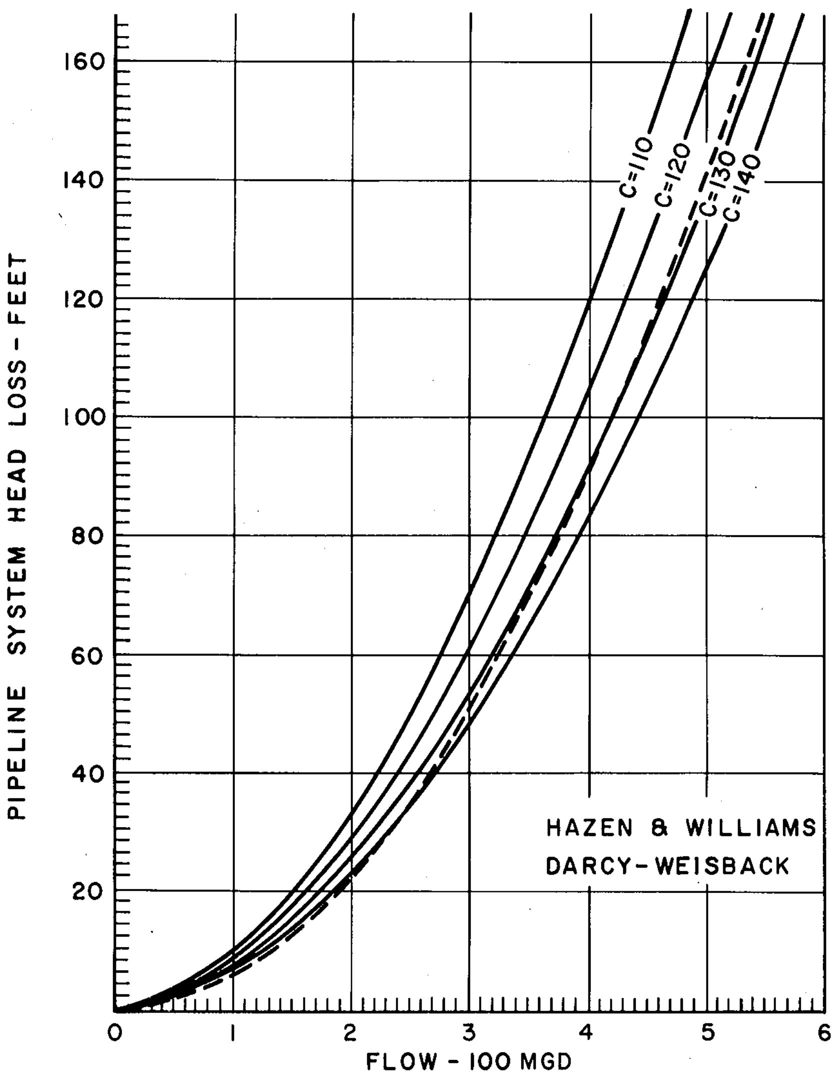


Fig. 2 — Head Loss - Discharge Rlationships for the New Warragamba Pipelines

THE NEED FOR FLOW CONTROL AND ENERGY DISSIPATION DEVICES

As indicated previously, the pipeline outlet works can discharge either to Prospect Reservoir or to the treatment plant. The treatment plant will be constructed in two stages. The first stage will provide only for clarification, while the second stage will both clarify and filter the water. During the first phase, Prospect Reservoir will be essentially "on line" when needed, while during the second stage this reservoir will be used only as emergency storage, and a covered filtered water reservoir will be used in its stead to meet system fluctuations in demand. For both stages of treatment it is necessary to control flow of the two sources of supply, with this control being more critical when complete treatment is provided. This follows from the fact that only a certain mismatch between supply and demand can be tolerated and that limited storage is available during the second phase to balance this mismatch.

For a given elevation of the water surface at Warragamba Dam, the energy dissipation per pound of water flowing from Warragamba to the outlet works at Prospect is a constant. Thus, referring to Fig. 2, if the reservoir is full, each pound has to dissipate 168 foot-pounds of energy. For flows less than the pipeline maximum, in this case less than 550 mgd when the reservoir is full, only part of this dissipation is by the pipeline, and the remainder has to be dissipated by some device.

The *rate* of energy dissipation by this control device is, however, variable. For no flow, there is, of course, no power dissipation. For the maximum flow at a given reservoir elevation, there is again no power dissipation by this device, all the dissipation occurring by wall friction and local losses in the transmission main. Thus, the maximum rate of energy dissipation by the flow control device does not occur at the maximum flow rate but at some flow intermediate between the maximum and zero flow rate. A plot of the power dissipation by the flow controller for the maximum operating level of Warragamba Reservoir is depicted in Fig. 3. From the figure it is seen that 7,300 horsepower (HP) will be dissipated by the flow control device at 320 mgd, or at 58 per cent of the maximum flow rate.

METHODS OF FLOW CONTROL AND ENERGY DISSIPATION

It is convenient to classify the methods of energy dissipation by the place where the energy is dissipated. Thus, the excess energy can be destroyed either within the pipeline itself, or upon exit from the pipeline. Within the

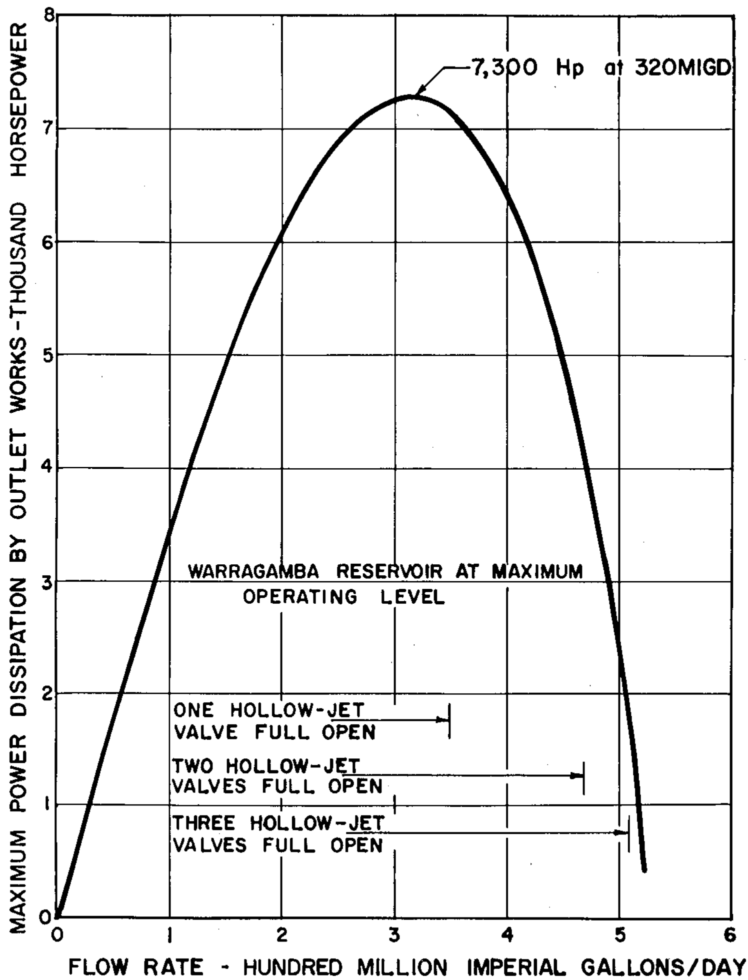


Fig. 3 — Power Dissipation - Discharge Relationship for Warragamba Pipelines Outlet Works at Maximum Reservoir Operating Level

pipeline, throttling valves may be used, or sections of the parallel pipeline system may be closed so that part of the system acts as a single pipe[2], or a combination of these two methods may be used. Dissipation of the excess energy outside the pipeline system is normally accomplished in a stilling basin or stilling pool when the required energy dissipation is large.

Dissipation of the energy entirely within the pipeline has the advantage that a stilling basin with its appurtenances is not required. Also, the flow exits from the pipeline in a relatively quiet manner, so that wave action in the channel leading to the treatment works is not a problem. This mode of operation does, however, increase the possibility of cavitation in the pipeline system, particularly at existing sectioning valve locations, because of consequent reduction in pressure due to energy dissipation and because of the valves' locations and elevations. If the rates of flow are to be controlled by partial throttling of the sectioning valves, then specially constructed valves and controls are required to withstand the hydrodynamic forces for a given flow rate and the additional forces induced by changing from one flow rate to another. If control of the flow rate is to be effected by sectioning of the pipeline (complete closure of certain sectioning valves), then special valves may not be required if the operation is to be intermittent, i.e., the flow would have to be stopped by downstream control valves while the sectioning valves were adjusted in still water. Operation of the system in this fashion (using parallel pipes for some sections, and a single pipe for other sections) would give a stepped, rather than continuously variable, control of the flow rate. Both methods of control by dissipation of the energy within the pipeline have the disadvantage that control of the system is scattered throughout the length of the pipeline. This type of operation would require that, for each different flow pattern, a number of pipeline protective devices (pressure or velocity sensors) would have to be adjusted. Incorrect setting or malfunction of any of these devices could endanger the pipeline.

Control of the flow rate by dissipation of the energy outside the pipeline system has the decided advantage of decreasing the possibility of cavitation within the pipeline to a minimum by maintaining the pressure within the pipeline as high as possible for all flow rates. Further, it allows continuous control of the flow rate, centralized at the point of dissipation so that performance can be readily observed. In addition, it decreases the number

2. Considerable preliminary investigation on flow control by pipeline sectioning was conducted by the Board, but was not recommended as a method of control.

of specially constructed valves required. However, this type of dissipator requires a stilling basin that must give minimal wave action at the exit to the basin.

Outlet flow control has been chosen for the Warragamba pipeline as it is more advantageous to the overall design.

OUTLET REGULATING WORKS

For flow rates less than the maximum and with excess energy dissipation outside the pipeline system, the excess head is converted to a high exit velocity from the particular flow-regulating device utilized. This velocity must be dissipated in a controlled manner in order to avoid spray, waves in the channel leading to the treatment works, cavitation damage to the pipeline exit basin, or cavitation or vibration damage to the regulating device. The flow from the pipeline can be discharged either above or below the waterline of the receiving basin. Discharge below the waterline normally requires a deeper stilling basin, but it is easier to control waves and spray. Further, it has the advantage of allowing the maximum possible effective head to be produced over the length of the pipeline for a given tailwater elevation in the stilling basin so that the largest possible flow can be obtained.

The control of the flow rate at the pipeline exit may be either continuously variable (by valves), or it may be discontinuous or stepped, by using a number of nozzles or orifices.

The usual practice for control of the flow when discharging above the surface of the stilling pool is to use valves that break up the flow rather than leave it as a solid jet. Nozzles and orifices are generally not used because they produce solid jets. Breaking up the jet makes use of air resistance to dissipate some of the energy, thus reducing wave problems.

One type of free discharge valve is the cone dispersion valve, also known as the sleeve regulator or Howell-Bunger valve. This valve has the advantage of being relatively inexpensive for its size and of being quite efficient (the coefficient of discharge is 0.85 at 100 per cent stroke). The normal location of this valve is well above the stilling pool so that only spray falls into the pool. This type of operation produces considerable mist or fog, particularly on windy days, which would be objectionable in the Project Reservoir area. Elevating the valve sufficiently above the stilling pool so that the jet is broken up would also reduce the maximum flow rate by about 35 mgd at low reservoir levels. If this type of valve is located immediately above the water surface, the stilling basin design would have to incorporate the effects of the unbroken jet of water. Design criteria for such basins have not been developed and extensive model tests would be required to develop the appropriate basin.

Hollow-jet valves, developed by the U.S. Bureau of Reclamation, have become increasingly popular, since their performance is virtually cavitation-free. The use of hollow-jet valves had been considered among other alternatives by the Sydney Water Board prior to the engagement of Camp, Dresser & McKee. The Bureau of Reclamation has developed extensive design criteria for stilling basins to be used in conjunction with these valves, which operate close to the water surface so that the maximum head is available. The basin design criteria and valve design criteria were developed from extensive model tests, and subsequently confirmed by results from prototypes.

Submerged discharge of the pipeline can be controlled either by use of a number of nozzles (or orifices), or by control valves.

Nozzles have the advantage of being cheaper and of having no moving parts, so that wear is minimized. However, this type of control is not continuously variable, and allows only a limited number of distinct flow rate selections, depending upon the number and size of nozzles installed. Moreover, the design of this type of stilling basin for the dissipation of large velocity heads must again be determined by extensive development model tests. On the other hand, if the requirement is the dissipation of small velocity heads, this can easily be accomplished through the use of impact stilling basins developed by the Bureau of Reclamation.

Control of a submerged discharge by valves at the pipeline exit requires that these valves be designed to be cavitation-free for the desired flow range. This is often difficult to obtain in any valve that first decreases the flow area and then increases it (as in butterfly valves, needle valves, and the like) and cannot be guaranteed without an adequate testing program. It is a better policy to use a valve that does not have this flow pattern, such as a cone dispersion valve which, however, must be aerated in order to permit flow expansion at the base of the moving cone and prevent cavitation at that point. The air required is of the order of the volume of water being passed, and this is sometimes difficult to supply satisfactorily.

It is felt that the difficulties enumerated above with respect to flow regulation at the pipeline exit will be avoided if hollow-jet valves are utilized in combination with submerged discharge nozzles. The hollow-jet valves will be utilized for the flow range from minimum flow to near maximum flow, while the nozzles will be utilized for maximum flow rates. Thus, continuously variable, free discharge control will be used for most of the flow range, with a final step to maximum discharge by the nozzles. The hollow-jet valves will insure cavitation-free performance through the critical portion of the operating range. Stilling basin design and performance for the hollow-jet valves have been investigated quite thoroughly by the Bureau of Reclamation, and no difficulties in operation of this valve and associated stilling basin are envisioned. Further, extensive developmental model tests are not necessary for this type of exit control and hence the design of the entire project will not be impeded.

THE HOLLOW-JET VALVE

Many different types of valves have been developed during the last half century in the search for a control valve that would operate satisfactorily at any opening position and under high heads. The hollow-jet valve seems to adequately meet these criteria. Development was begun in 1940 on this type of valve at the Denver Office of the Bureau of Reclamation by Byron H. Staats and G. J. Hornsby [1, 2][3]. The valve was specifically developed for Anderson Ranch Dam, but the first large prototype (96-in diameter) was installed at Friant Dam in California. The Bureau of Reclamation has since used the hollow-jet valve at many of its installations.

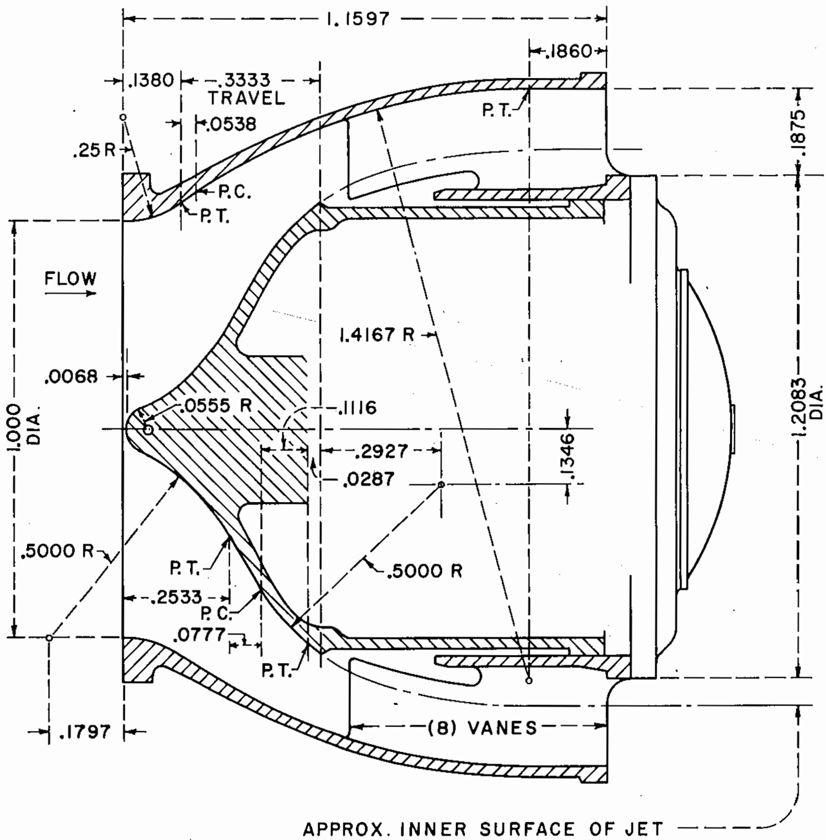
The proportions of the valve in terms of the valve diameter are shown in Fig. 4. The valve became known as the hollow-jet valve because the jet, as it leaves the valve, issues forth in the shape of a doughnut. This doughnut-shaped cross section is segmented into parts by the splitter vanes. From the figure it is also apparent that the hollow-jet valve is in reality a form of needle valve.

The hollow-jet valve consists of an inner needle and a valve casing with its upstream end having the same diameter as the supply conduit. The needle is centrally positioned within the casing by the equally spaced splitters. Free access of air to the interior wall of the hollow-jet is permitted through both the cavities in the hollow splitters and through the slits in the walls of the jet itself caused by the splitters.

As water flows through the valve, it leaves the curvilinear portion of the needle at the knife-edge end of the needle seat ring and clears the downstream cylindrical portion of the needle. This is possible owing to the aeration of the cylindrical portion of the needle by the splitter vanes.

Because of the way the valve functions, it was not intended to be used for discharging under water. However, it can be used in a closed conduit provided adequate provision is made for the admission of air at the appropriate pressure at the outlet end of the valve.

3. The numbers in brackets refer to the references at the end of this article.



NOTE : ALL DIMENSIONS IN TERMS OF DIAMETER UNLESS OTHERWISE NOTED

Fig. 4 — U.S. Bureau of Reclamation Hollow-Jet Valve: Proportions of Valve and Waterway

The coefficient of discharge of this valve, based on the area of the inlet, is 0.72 for the valve fully open [3]. The discharge coefficient is essentially linear with valve opening between the fully closed and half-open positions, being, of course, zero when the valve is closed and 0.41 when the valve is 50 per cent open. Between half and fully open, the variation in coefficient is curvilinear.

As has been mentioned, the hollow-jet valve is reputed to be cavitation-free in its performance. In order to document this assertion, data from reference 2 have been utilized. Fig. 5 shows the pressure distribution in dimensionless form plotted as a projection on the component parts of the valve for flow through the valve when it is fully open. It is seen that the pressure is everywhere positive for the casing, while the pressure along the needle is only slightly negative a short distance downstream from where the jet springs clear of the needle. Fig. 6 shows the pressure distribution for the valve 10 per cent open. In this position, both the needle and the outer casing experience slightly negative pressures of the order of 2 per cent of the total valve head (pressure plus velocity head one diameter upstream of the valve). The two figures were prepared from data for a 24-in diameter valve discharging under a head of 330 ft, and are intended to indicate only the qualitative form of the pressure variation. Data for a 6-in and a 96-in valve discharging under different heads produce similar pressure variations at corresponding valve openings. For none of these valves at various valve openings did the negative pressure exceed 2.5 per cent of the total valve head. Thus, for heads up to at least 500 ft, cavitation will not occur, provided the valve does not have local irregularities due to casting or machining. It was concluded, therefore, that this type of valve does adequately meet the conditions existing in this instance.

Three 48-in diameter hollow-jet valves have been selected as flow controllers for the Warragamba pipelines. Fig. 7 indicates the discharge capabilities of the pipelines with these valves. Three valves were chosen because of reliability considerations. Thus, with one valve out of service, the maximum flow capability is reduced by only 8 per cent. The valve size of 48-in was selected because it reduces the size of stilling basin required and, in addition, permits the flow rate to be reduced to about 115 mgd at 20 per cent opening, which is an appropriate minimum flow rate and valve stroke.

THE HOLLOW-JET VALVE STILLING BASIN

In the first stilling basins constructed for use with the hollow-jet valve, the valve was aligned to discharge horizontally. A trajectory-curved floor was placed near the valve to assist in spreading the jet uniformly in the

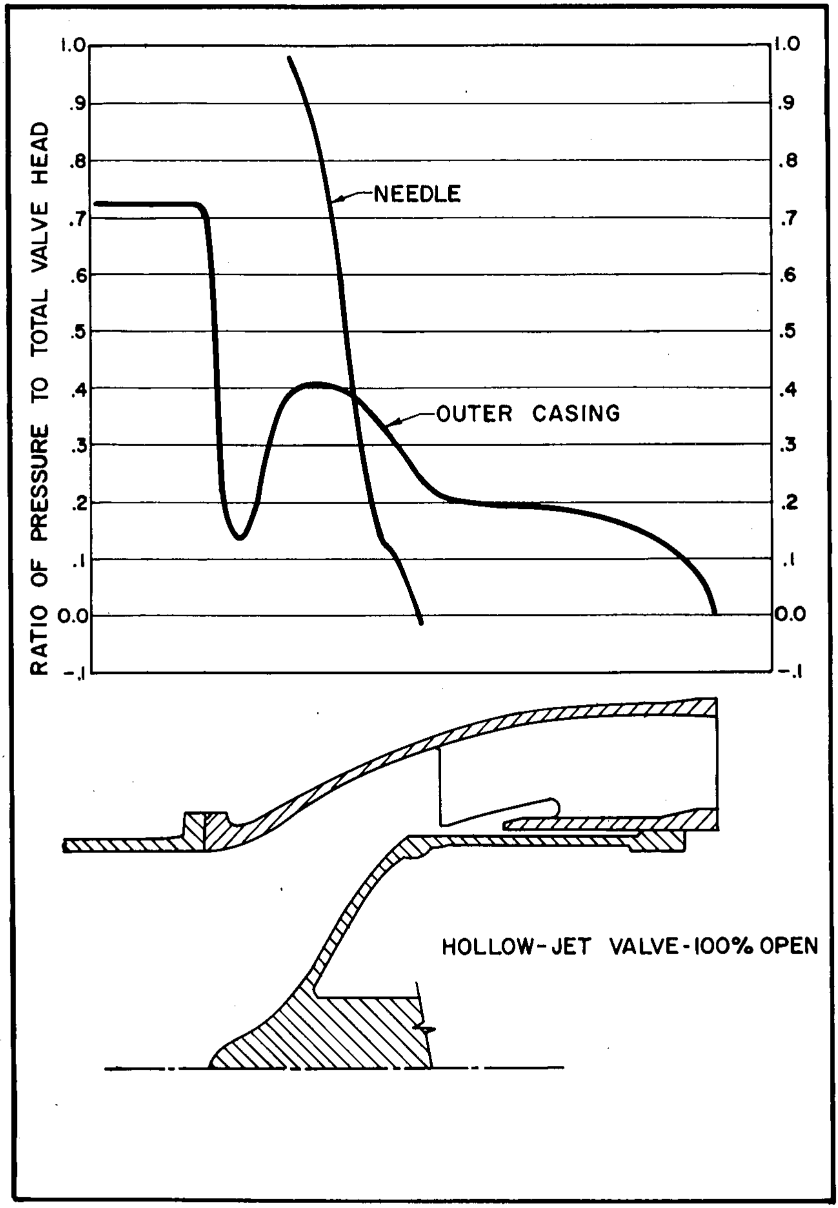


Fig. 5 — Pressure Distribution Within the Hollow-Jet Valve: 100% Open

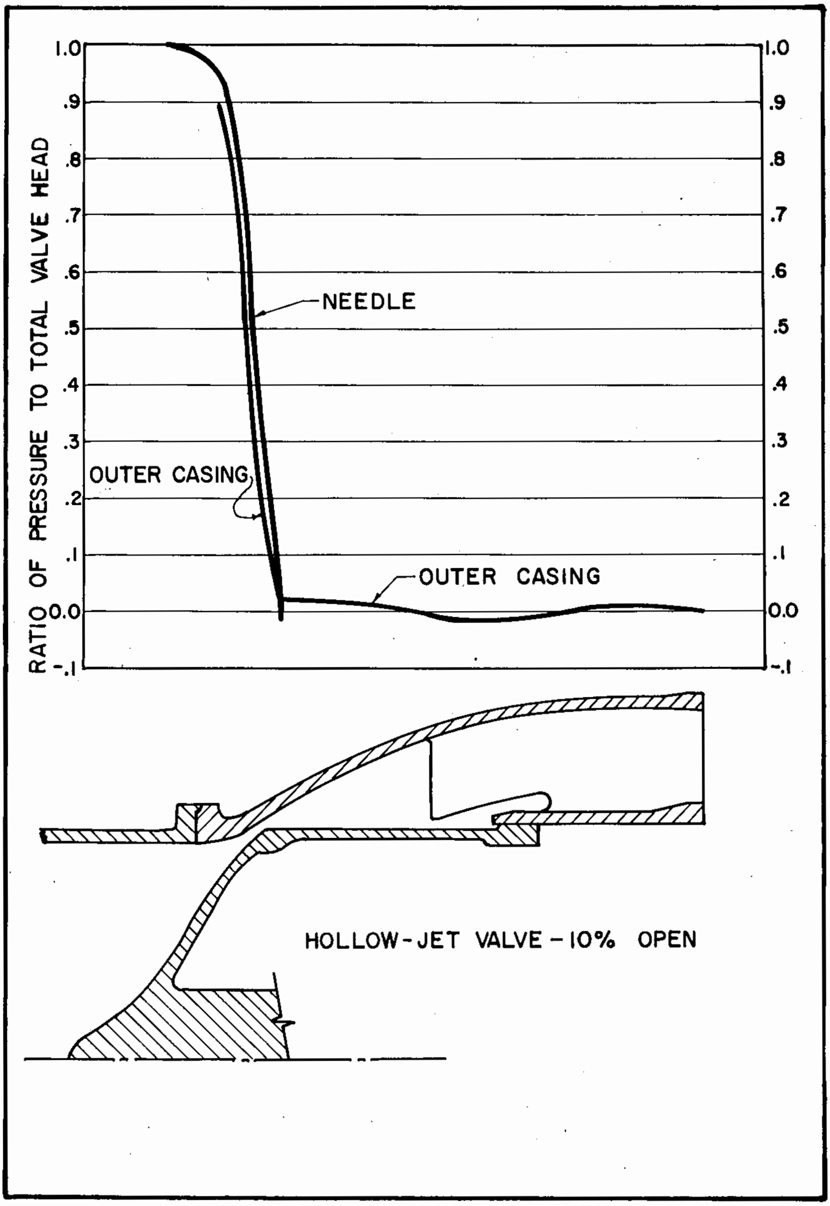


Fig. 6 — Pressure Distribution Within the Hollow-Jet Valve: 10% Open

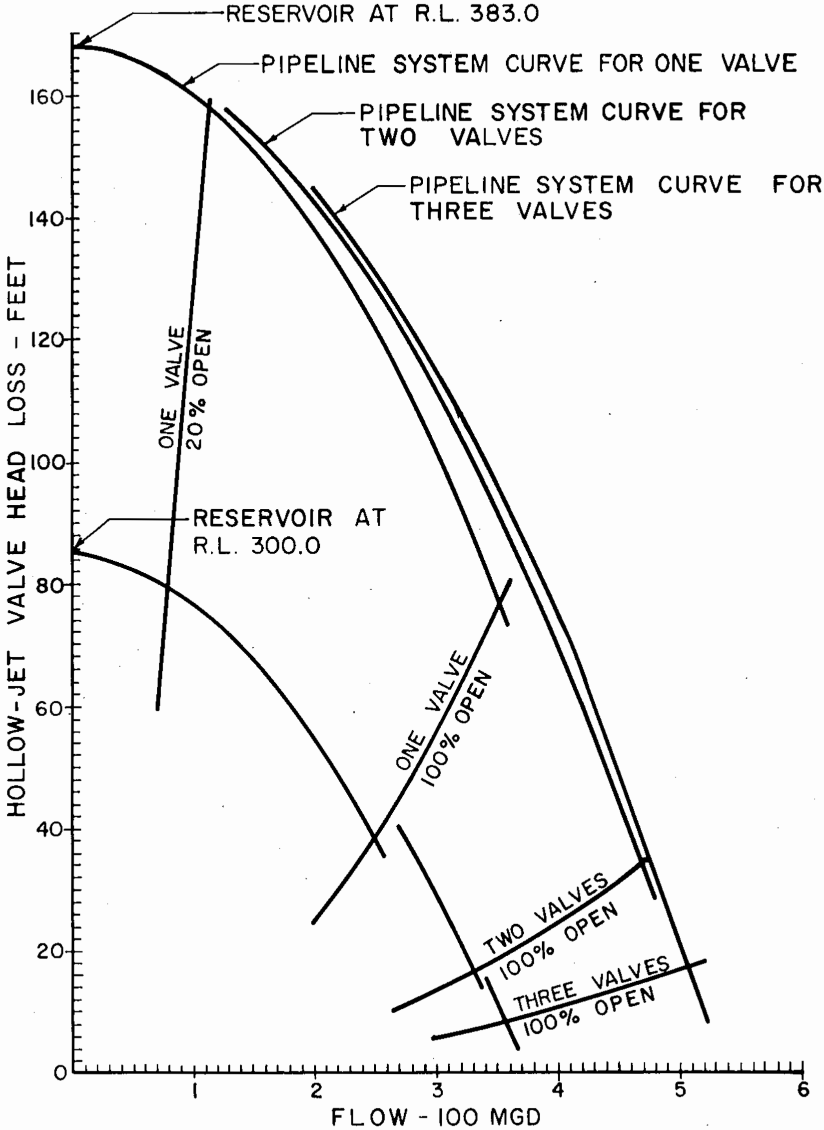


Fig. 7 — Head Loss - Discharge Relationships for the New Warragamba Pipelines Regulating Works

horizontal plane. The jet then entered a hydraulic jump stilling pool. This type of design resulted in an extremely long structure, and containing the jump within the basin was sometimes difficult.

For the U.S. Bureau of Reclamation's Boysen Dam, a different type of basin was developed, which eventually led to the Bureau's standard Basin VIII. The valve was aligned downward rather than horizontally. It was found that the optimum angle of entry of the jet into the tailwater pool was 24 degrees from the horizontal. For lesser angles the jet skipped along the tailwater surface, while for steeper angles the jet penetrated the pool but about-faced and rose vertically to the surface to form a large boil. The floor beneath the jet where the jet first entered the basin could not be horizontal, but had to essentially follow the jet to project it from turbulent eddies. A floor slope of 30 degrees from the horizontal was found to be satisfactory. Converging sidewalls on this 30-degree floor materially aided the performance of the basin. These converging walls effectively controlled surface boiling, created additional energy loss due to the expansion of the jet at their ends and forced the jet to penetrate the tailwater pool for a longer length. Fig. 8, reproduced from reference 3, indicates the form of the basin as it has developed over the years.

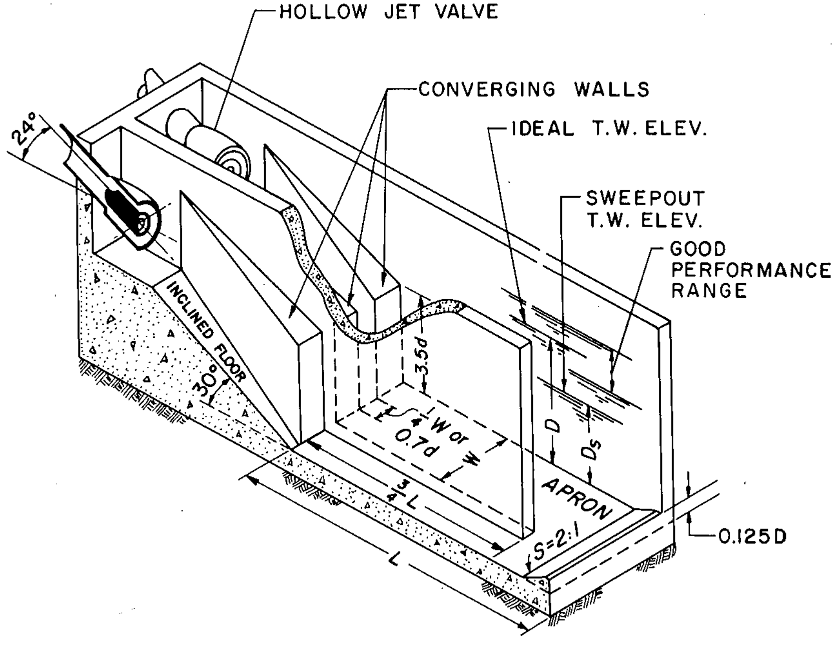


Fig. 8 — Generalized Form of the Hollow-Jet Stilling Basin

It became apparent to the Bureau of Reclamation after model testing a number of basins of the Boysen type that the design could be generalized, and hence it developed a testing program to do so. From Fig. 8, it is seen that the size of the basin is effectively specified by the length, L; the width, W; the ideal depth, D, or the sweep-out depth, D_S. The ideal depth was judged by visual appearance and quality of the stilling action and by smoothness of the tailwater surface. The sweep-out depth was based on the minimum tailwater elevation necessary to maintain the stilling action within the basin and prevent the jet from sweeping out the basin. The generalization tests verified that the basin size parameters were a function of the valve diameter, d, and the valve discharge head and opening, and consequently related to the power dissipation by the basin. The Bureau of Reclamation monograph [3] presents the normalized basin size parameters, D/d, D_S/d, L/d, and W/d as graphical functions of the normalized head, H/d, and the valve opening percentage. All of the normalized basin size parameters increase with increasing values of H/d, and for a constant value of H/d, decrease with decreasing valve opening.

It is interesting to see if the basin size parameters can be represented as a function of a single parameter rather than by both H/d and the valve opening percentage. If the head on the valve is replaced by its equivalent expression, $8Q^2/\pi^2gC^2d^4$, where C is the valve discharge coefficient at the appropriate valve opening, then H/d at a particular valve opening can be expressed as a function of Q^2/d^5. If the basin size parameters from the Bureau's monograph are now plotted versus Q^2/d^5, it is found that this is a fairly adequate parameter. (For the sweep-out parameter, D_S/d, valve openings of 100 and 75 per cent give almost perfect agreement, while that for the 50 per cent opening is greater at most by 16 per cent.) All of the normalized basin size parameters increase with increasing values of Q^2/d^5. Thus, for a given valve configuration, determination of the critical flows governing basin size can generally be found by examining values of Q^2/d^5 for each valve rather than comparing performance for the valves at different openings.

As indicated previously, three 48-in hollow-jet valves had been chosen as the primary control valves for the Warragamba pipelines outlet works. From Fig. 7 it is seen that the maximum discharge for a single valve is 350 mgd; for two valves operating simultaneously, 470 mgd; and for three valves, 510 mgd, or the discharge per valve is 350, 235, and 170 mgd respectively. Thus, it should be expected that a single valve operating fully open (this gives the largest value of Q^2/d^5) will determine the basin size and that 350 mgd through a single valve will have more of a tendency to sweep out than any other flow. Further, reference to Fig. 3 indicates that a single valve discharging 350 mgd will have to dissipate 7,150 HP, while two valves operating simultaneously will dissipate 2,050 HP apiece, and three valves 583 HP apiece. Fig. 3 also shows that the maximum power dissipation by the outlet works occurs very close to 350 mgd so that maximum power dissipation for the outlet works as a whole is approximately the same as that through a single valve.

A single hollow-jet valve 48-in in diameter discharging 350 mgd (650 cfs) requires a head of approximately 80 ft. Therefore, H/d is equal to 20. From the Bureau of Reclamation's monograph, the requisite basin size is: length of 57 ft; width of 10 ft; an ideal depth of 16 ft; and a predicted sweep-out depth of 13 ft.

The discharge elevation of the outlet works is governed by the requirements of the treatment plant and the desirability of using Prospect Reservoir for storage when needed, and hence was set 17 ft above the high water elevation of the reservoir. The outlet works discharge can be routed either to Prospect Reservoir or to the clarifiers of the treatment works. When flow is directed to the clarifiers, the tailwater will vary by 1.5 ft in the stilling basin for flows between zero and maximum flow. A control weir is required in the Prospect Reservoir discharge channel to ensure that the stilling basin tailwater variations will not be such as to, on the one hand, submerge the hollow-jet valves and cause them to cavitate, and, on the other hand, cause sweep-out to occur. It is desirable to limit the width of the control weir and associated channels and hence allow the tailwater variations in the stilling basins to be the largest possible.

As a consequence of the above conditions, the floor of the hollow-jet stilling basin was set at R.L. 198, the top elevation of Prospect Reservoir, and the hollow-jet valves were set so that their center line was 17 ft above the floor level. The resulting ideal depth of the stilling basin for 350 mgd is 1 ft below the valve and the sweep-out depth is predicted at 4 ft below the valve. This allows the control weir in the Prospect Reservoir channel to have a head variation of about 5 ft which produces a reasonable width of weir and channel. Because of the tightness of the design, it is desirable to verify the predicted basin performance by model tests and, in addition, to obtain further design information.

THE IMPACT STILLING BASIN

The impact stilling basin was developed by the U.S. Bureau of Reclamation primarily for use with irrigation projects [3]. As with the hollow-jet stilling basin, a series of generalization tests led to its development as a standard basin (Basin VI). The basin size is a function of the maximum discharge, while performance is essentially independent of tailwater variations.

The impact basin is an open rectangular box, with a supply pipe at one end of the box and a control sill at the opposite end of the box (see Fig. 9B). Intermediate between these two points is a hanging baffle. The jet from the supply pipe is supposed to impinge upon this baffle, spread out, and flow under the baffle. Thus, the designation of the basin is the impact stilling basin. The control sill at the end of the basin forces the jet to spread out within the basin in order to meet the requirement of discharge over the entire sill.

For the Warragamba pipelines outlet works, four impact basins were proposed. Each impact basin would be fed by a 72-in diameter supply pipe and would have a 16.5 ft inside width. The maximum discharge capacity of each basin would be 339 cfs, with the total possible discharge through the four basins being 730 mgd. This would allow for increased capacity through the outlet works in the event that the pipeline capacity is increased in the future.

THE COMPOSITE STILLING BASIN

As discussed above, the Warragamba pipelines stilling basin will consist of three hollow-jet basins and four impact stilling basins. The three hollow-jet basins will be be identical in size, so that any one of them will be able to discharge the critical flow of 350 mgd, thus providing for an outlet works of high reliability, and also capable of handling increased flows in the event of any future pipeline expansion. Fig 9 shows the original layout of the proposed basin, with the hollow-jet basins in the middle and the impact basins adjacent. The entire complex of basins connects to the exit channel by a smooth curvilinear transition.

The Warragamba pipelines terminate at a 120-in manifold. From the manifold, supply pipes bring the flow to each of the seven separate basins. All of the supply lines are equipped with guard valves to permit servicing of the operating valves. The supply pipes to the hollow-jet valves are designed to keep the pipe velocity to a reasonable value. The center-line elevation of the manifold is depressed below the center-line elevation of the outlet in order to keep the pressure in the manifold positive at maximum flows.

The preliminary design was done by Camp, Dresser & McKee. The Board is responsible for the final design and preparation of construction drawings for the composite basin, and has performed rather extensive model tests to verify the hydraulics of the basin and determine other

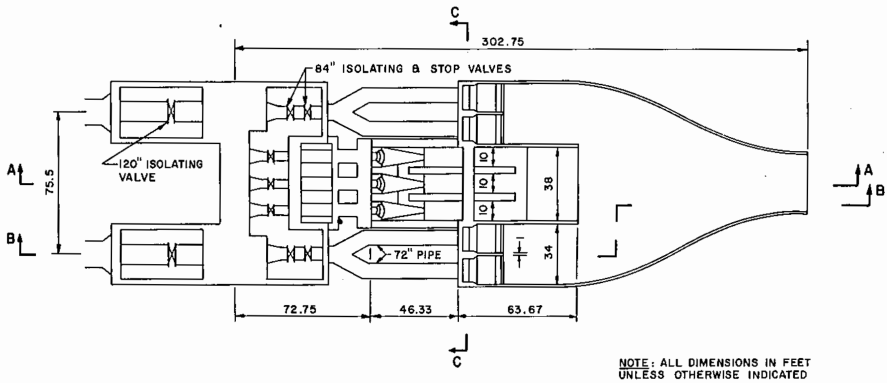


Fig. 9A — The Warragamba Pipelines Stilling Basin - Plan

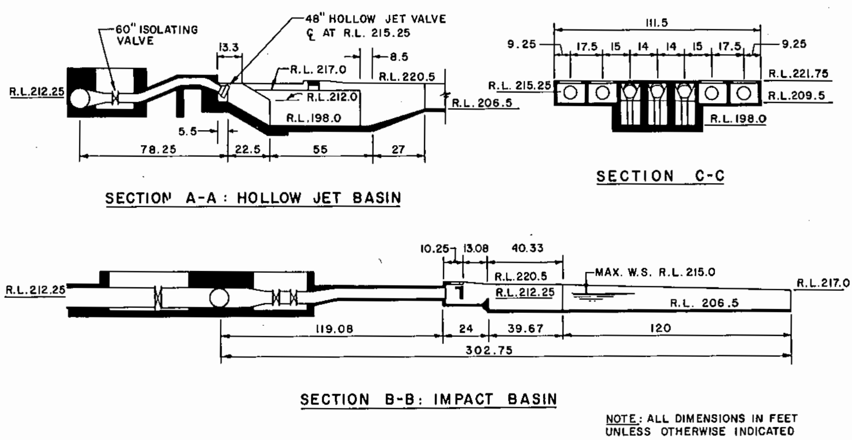


Fig. 9B — The Warragamba Pipelines Stilling Basin - Sections pertinent design information. Some of the findings of the model tests follow. A complete description of the model tests and the results obtained will be contained in a paper soon to be published by the Board's Engineers in charge of the work.

MODEL OF THE COMPOSITE BASIN AND HIGHLIGHTS OF TEST RESULTS

The Sydney Water Board built a 1:20 scale model of the composite stilling basin at its Manly Hydraulics Laboratory. Tailwater level control was by a tilting weir in the discharge channel.

The overall hydraulic performance of the model basin for the outlet works discharging to the treatment plant was very good. As expected, the critical discharge was through a single hollow-jet valve fully open, producing the maximum amount of turbulence in the basin (see Fig. 10). Wave heights at the entrance to the discharge channel were estimated at 5-in prototype for this condition, and negligible for two or three hollow-jet valves discharging simultaneously. Discharge through the four impact basins (hollow-jet valves closed) gave excellent stilling action, being somewhat better at the higher tailwaters.

The sweep-out depth as determined by Sydney's model tests was in excess of that predicted by reference 3. The Bureau of Reclamation's graph gives a depth of 13.1 ft. The model test of the composite basin had a sweep-out depth of 15.3 ft, or 16 per cent greater. Fig. 11 shows in a rather dramatic fashion the effect of sweep-out. Because of the greater elevation of the sweep-out depth, the control weir in the Prospect discharge channel has to be redesigned from its preliminary dimensions.

The model tests showed good stilling action in the impact basins. At flow rates less than the maximum rates given by the U.S. Bureau of Reclamation, the impact action does not occur. Instead, the jet from the supply pipe dove under the impact baffle, but the stilling action was not impaired. As the size of these basins was selected conservatively, the impact action is not anticipated, and the baffles will not be installed.[4]

Tests were also conducted to determine the magnitude of the loads on the walls of the hollow-jet stilling basins. Reference 3 had indicated that in the model tests for Boysen Dam, the maximum pressure on the stilling basin floor beneath the impinging jet was less than one-fourth of the total valve head. Communication with the U.S. Bureau of Reclamation by the Sydney

4. Since the presentation of this paper, additional model testing by the Board has resulted in further design modifications. These consist primarily of reducing the dimensions of the stilling basin areas in front of the 72-in diameter nozzles. As noted, the Board will publish a description of the final design of the composite basin.

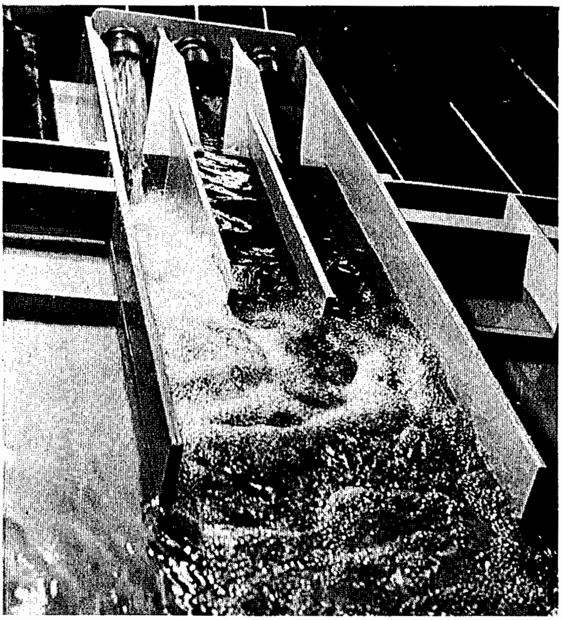


Fig. 10 — Model With One Hollow-Jet Valve Discharging 365 MGD at a Depth of 16 ft.

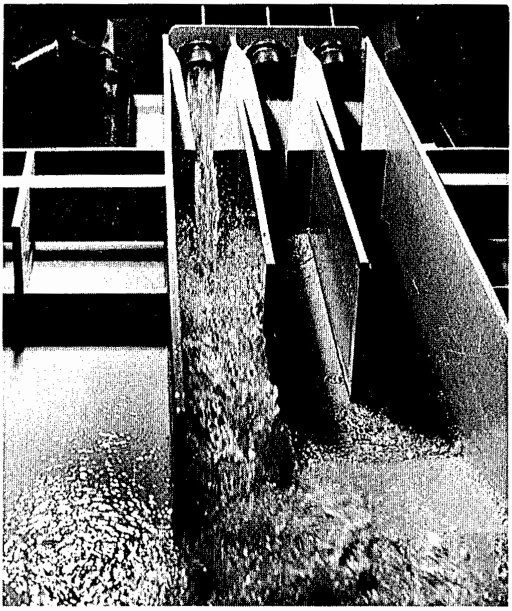


Fig. 11 — Model with One Hollow-Jet Valve Discharging 365 MGD at Sweep-Out Depth

Water Board indicated that the Bureau had recently found that pressure differentials across the dividing walls in multiple hollow-jet basins reached up to 70 per cent of the valve velocity head, and that this dividing wall vibrated at frequencies of 3.5 to 6.9 cps (prototype). Sydney has found that the maximum load on the walls of the basin can be represented by the following: 0.4 of the maximum hydrostatic pressure at the water surface with a linear increase to 1.6 times the maximum hydrostatic pressure at the base of the wall. This represents both the static and dynamic loads. For the purposes of design, the dynamic portion of the load was multiplied by a factor of three. Pressure fluctuation frequencies were also measured in the Board's model, corresponding to fluctuations of 1.2 to 4.5 cps in the prototype.

CONCLUSIONS

Generalized stilling basin designs such as those developed by the U.S. Bureau of Reclamation enable a designer to choose among different types of basins and know essentially how they will function. This can accelerate the design process immensely by eliminating developmental tests. However, all important and large designs should still be tested, both for verification and for further information that is invaluable to the complete design picture.

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THE MASSACHUSETTS WATER POLLUTION CONTROL PROGRAM

By

JOHN R. ELWOOD* Member

The Massachusetts Division of Water Pollution Control has been in operation for a little over one year and therefore, it seems an appropriate time to describe the structure of this organization and also summarize its principal activities to date.

The Division was established on September 6, 1966 by an act of the legislature called the Massachusetts Clean Waters Act (Chapter 685, Acts of 1966). This law states that "there shall be in the Department of Natural Resources subject to the control of the Water Resources Commission a division of water pollution control". The Commission now has two divisions under its jurisdiction as shown in Figure 1.

In effect, the Act created a new "regulatory agency" for water pollution control and has transferred this function from the Department of Public Health to the Department of Natural Resources.

LEGISLATION

There were actually four legislative acts enacted in 1966 which established the Water Pollution Control Program and these are summarized below:

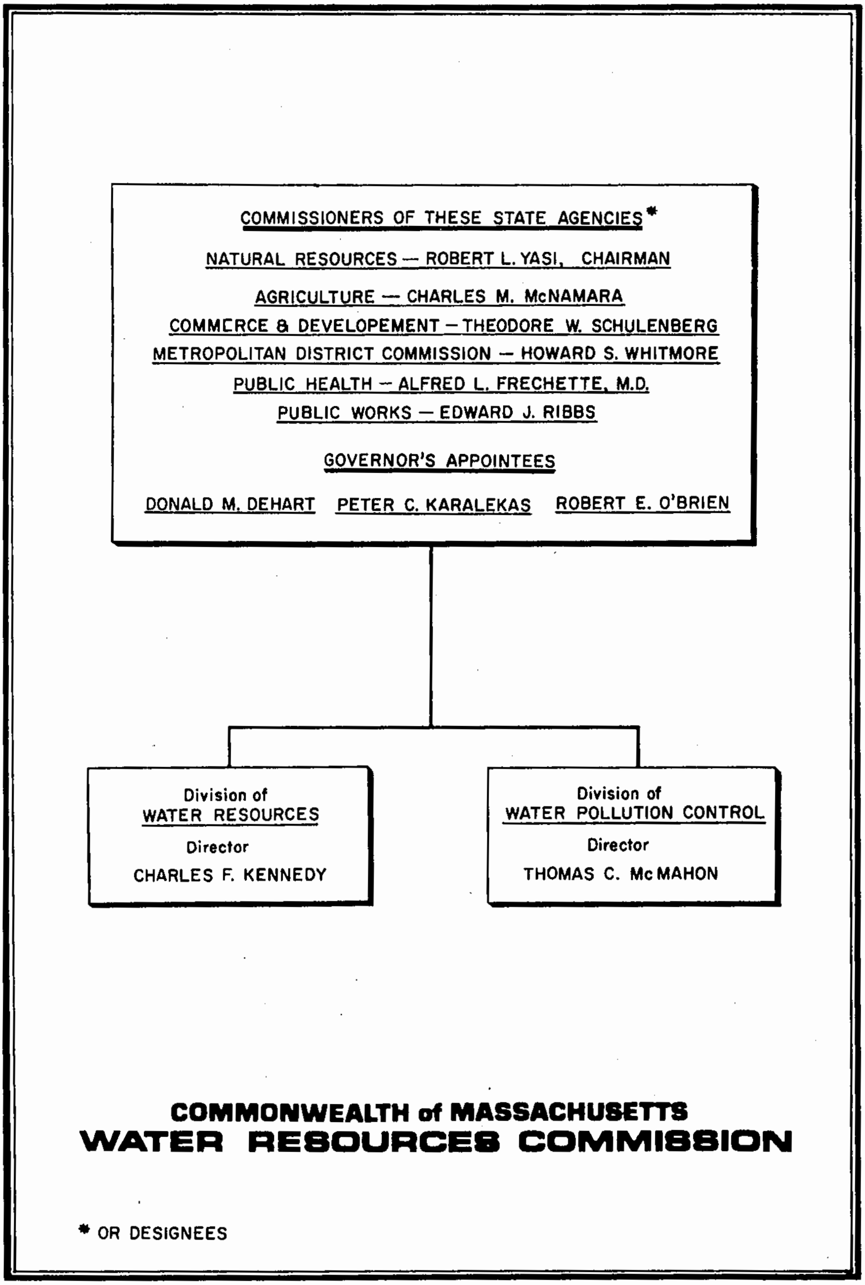
Chapter 685 Created the Division of Water Pollution Control, established a program for pollution abatement, including enforcement procedures, and gave the Division authority to establish pollution abatement districts.

Chapter 687 Provided for $150 million bond issue for construction grants, research and demonstration projects and construction of low flow augmentation reservoirs.

Chapter 700 Provided for local property tax exemption for new and existing industrial waste treatment facilities.

Chapter 701 Provided for deduction and exemption, under the business and corporation excise tax, of the expenditures for construction of industrial waste treatment facilities.

* Supervising Sanitary Engineer, Massachusetts Division of Water Pollution Control


COMMISSIONERS OF THESE STATE AGENCIES*
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WATER RESOURCES COMMISSION
* OR DESIGNEES

Figure 1.

There were certain inconsistencies in Chapters 685 and 687 which prevented the awarding of construction grants from the funds authorized under Chapter 687. The principal problem was that Chapter 685 called for the establishment of pollution abatement "districts" while Chapter 687 provided for aid to "cities and towns." For this and other reasons it was necessary to amend these acts before a workable construction grants program could be placed in operation.

In addition to the corrections, the amendment act also included further refinements such as:

1. Providing for pre-financing of federal grants with state funds.
2. Providing for a planning advance of up to seven percent of the project cost.

The amendment act (Chapter 873, Acts of 1967) was enacted on January 5, 1968.

WATER QUALITY STANDARDS

The principal activity of the Division during its early stages was the development of a set of water quality standards. This project required the immediate attention of the Division in order to satisfy the requirements of the Federal Water Quality Act of 1965. Further, the development of acceptable water quality standards is required in Section 26 of Chapter 685 and, under the Federal Clean Water Restoration Act of 1966, qualifies Massachusetts communities for the maximum federal construction grants.

The water quality standards are a group of criteria which have been selected to establish levels of water quality. The criteria used were: dissolved oxygen, solids, color, turbidity, coliform bacteria, ammonia, phosphate and phenol concentrations. Parameters for these criteria were then established based on the intended use of the water (Figure 2).

In addition to the water quality criteria, the Water Quality Standards contain several notes pertaining to minimum waste treatment requirements. The principal items covered are summarized below:

Inland Waters

1. All wastes shall receive secondary treatment or its industrial waste treatment equivalent, and including disinfection, except where a higher degree of treatment is required.
2. The amount of disinfection required shall be equivalent to a free and combined residual of at least 1.0 mg/1 after 15 minutes contact time.

FIGURE 2

SUMMARY OF WATER QUALITY STANDARDS FOR WHICH PARAMETERS HAVE BEEN ESTABLISHED

ITEM	INLAND WATERS				COASTAL WATERS		
	CLASS A	CLASS B	CLASS C	CLASS D	CLASS SA	CLASS SB	CLASS SC
DISSOLVED OXYGEN							
Minimum, mg/l	5.0	5.0	3.0 (2)	2.0	6.5	5.0	3.0 (2)
Percent Saturation (1)	75	75	—	—	—	—	—
COLIFORM BACTERIA							
Average value per 100 ml	50	1000	NONE (3) (3)	NONE (3)	70 •	700	NONE
pH (Min — Max)	(4)	6.5-8.0	6.0-8 5	6.0-9.0	6.8-8.5	6.8-8.5	6.5-8.5
TEMPERATURE, Deg. F							
Cold water fishery	(4)	68	68	—	—	—	—
Warm water fishery	(4)	83	83	90	—	—	—
Maximum increase	(4)	4	4	—	—	—	—
TOTAL PHOSPHATE, mg/l as P	—	0.05	0.5	—	0.07	0.07	0.07
AMMONIA, mg/l as N	—	0.5	1.0	—	0.2	0.2	1.0
PHENOLS, mg/l	—	0.001	0.002	—	—	—	—

(1) During 16 hrs of a 24 hr period.
(2) Minimum of 5.0 mg/l during 16 hrs of a 24 hr period.
(3) None in such concentrations that would impair uses assigned this class.
(4) As naturally occurs.

NOTE: The remaining criteria (solids, color and turbidity, taste and odor, chemical constituents and radioactivity) have not been assigned limiting values. Allowable concentrations depend on most sensitive water use. The complete Water Quality Standards have been published and are available from the Division of Water Pollution Control, 100 Cambridge Street, Boston, Massachusetts.

Coastal Waters

1. Appropriate treatment is defined as the degree of treatment with disinfection required for the receiving waters to meet their assigned classification.
2. Disinfection requirements are the same as for inland waters.

The Water Quality Standards were adopted by the Water Resources Commission on March 3, 1967, after a public hearing and were approved on August 8, 1967 by the U. S. Department of Interior. Massachusetts was the first New England state and fourth state in the nation to receive federal approval of its standards.

CLASSIFICATION

The various water uses have been grouped into a set of classifications which are Classes A, B, C and D for inland waters and Classes SA, SB and SC for coastal and marine waters. The following is a brief summary of how the various classes relate to water use:

Inland Waters

CLASS A: Waters designated for use as public water supplies.

CLASS B: Suitable for bathing and recreational purposes including water contact sports. Acceptable for public water supply with appropriate treatment.

CLASS C: Suitable for recreational boating and good habitat for fish and wildlife.

CLASS D: Suitable for industrial use.

Note: Class D will be assigned only where a higher water use class cannot be attained after all appropriate waste treatment methods are utilized.

Marine Waters

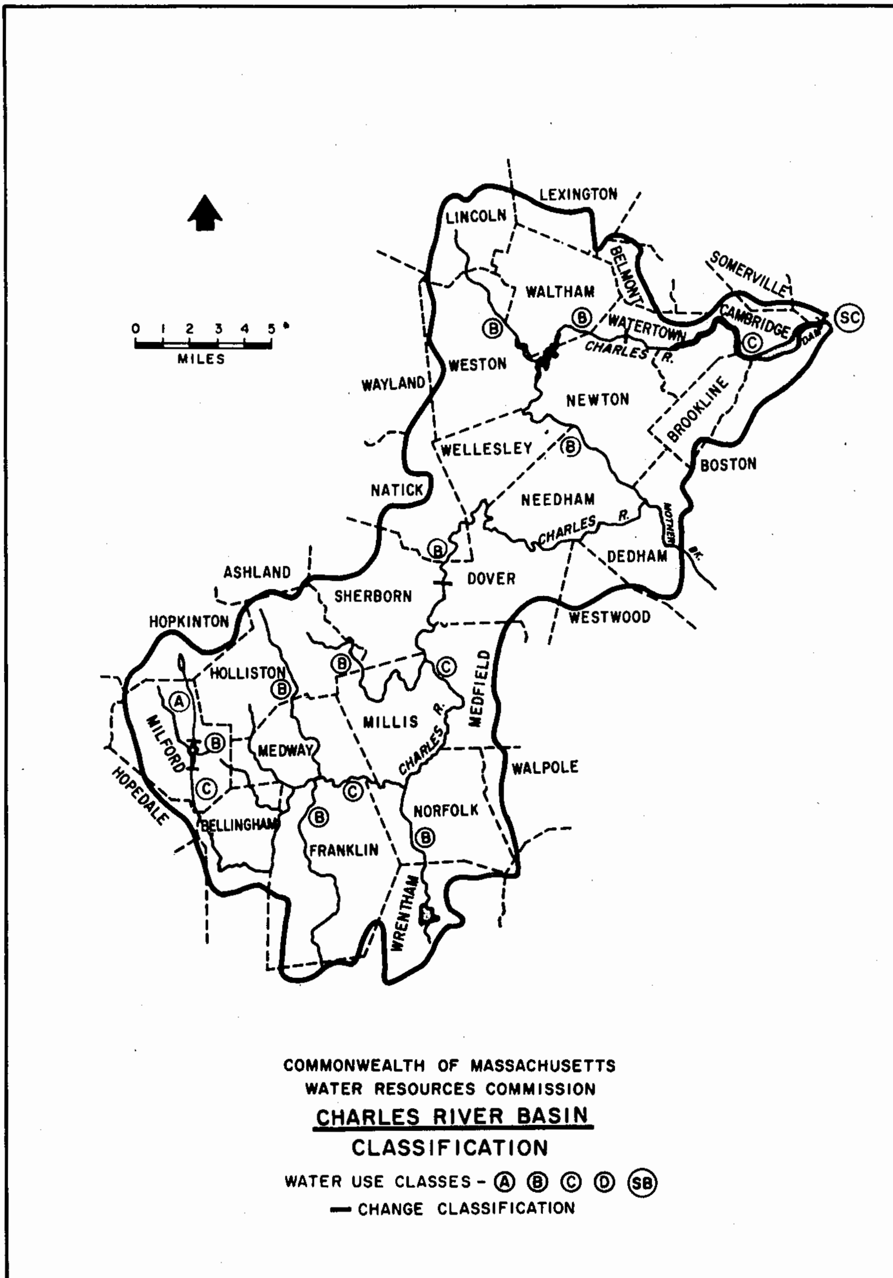
CLASS SA: Suitable for bathing, water contact sports and shellfish.

CLASS SB: Suitable for bathing, water contact sports, and industrial use. Suitable for shellfish with depuration.

CLASS SC: Suitable for fish and wildlife habitat and industrial use.

Following a series of public hearings held in the various river basins, all waters of the Commonwealth were classified and a series of maps were prepared which show the classification of all major streams, lakes, estuaries and coastal waters (Figure 3).* Minor streams and ponds not shown on

* Both the maps and written descriptions of the classifications are available from the Division of Water Pollution Control.



FORM WPC28 1M-2-67-944469

Figure 3.

these maps are considered to have a B classification. The classifications shown are proposed classifications and in most instances do not indicate the present quality of a body of water.

IMPLEMENTATION PLAN

The third principal activity of the Division was the development of a Plan of Implementation. This plan is essentially a schedule for water pollution abatement. Under the plan all known sources of pollution, both municipal and industrial, were listed. Dates were then determined for the accomplishment of each of the several steps required to construct the necessary pollution abatement facilities. Dates have normally been determined for the following six steps:

A. Submission of a preliminary engineering report.
B. Appropriation of funds.
C. Acquisition of site.
D. Submission of final construction plans.
E. Start of construction.
F. Completion of construction.

As of this date implementation schedules have been developed for 108 municipalities and 227 industries. As additional sources of pollution are investigated and evaluated, their names are added to the list, schedules are determined, and the polluters notified.

CONSTRUCTION GRANTS

As mentioned previously, the Division did not award any state construction grants during 1967. Federal grants were available, however, and a total of $2,632,367 in federal grants was allocated. Procedures and requirements for a state grant program were developed and many applications for both state and federal grants were processed. Preliminary engineering reports and final construction plans for a number of projects were reviewed and approved, and many meetings were held with consulting engineers and municipal officials regarding proposed pollution abatement projects.

For the remainder of the Fiscal Year 1968 (ending June 30, 1968), there will be $14.7 million in state funds and $5.0 million in federal funds available for construction grants in Massachusetts.

The Massachusetts Clean Waters Act provided for the reimbursement of a portion of the cost (usually 30%) of all pollution abatement projects constructed since 1956. These projects were reviewed and the amount of payment was determined for each project. A total of $591,134 will be paid to Massachusetts communities during Fiscal Year 1968 under the reimbursement program.

STREAM SURVEYS AND SPECIAL INVESTIGATIONS

Stream surveys were made on the Charles, Mystic and Aberjona Rivers in 1967 and a special study was made of Lake Cochituate in Natick. Other field activities consisted of complaint investigations and the investigation of industrial waste sources.

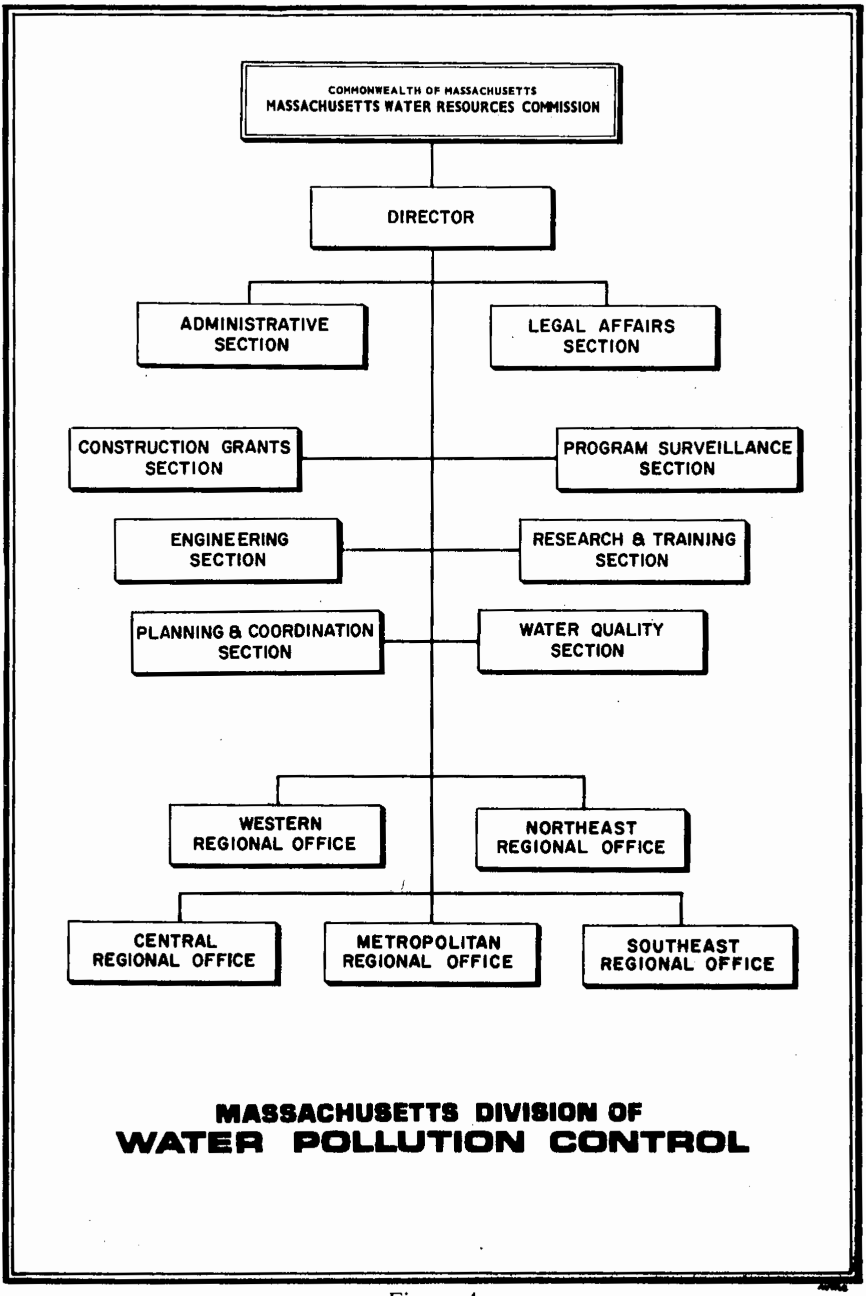
Conservation officers of the Department of Natural Resources filled a gap in the Division's field forces by performing the bulk of the sampling on the stream surveys and by making the initial investigations of complaints. These officers showed a strong interest and good capability for these activities and were a tremendous asset to the Division during its early stages of development. The Division of Law Enforcement plans to assign at least six officers to full time activity on water pollution control.

ORGANIZATION

The Division has been authorized a staff of 55 people which includes approximately 30 engineers and other technical personnel. The main office will be organized into six principal operating sections and there will be six field offices as shown in Figure 4. The staff will be multi-disciplinary in that it will consist of sanitary engineers as well as biologists, chemists and civil engineers. A diversity in backgrounds already evident in the group will add to its broad-based capability for program development. A majority of the engineers engaged to date have had experience with consulting engineer firms and/or public health departments.

CONCLUSION

The Division has the legal strength through its enforcement powers and the financial means through construction grants to develop an effective pollution control program. In the final analysis, though, its success will depend on the caliber of its personnel and on the degree of cooperation they receive from consultants, municipal officials, industrial leaders, sewage treatment plant operators and other agencies. We can all have a part in making the Division of Water Pollution Control an effective instrument for the enhancement and preservation of water quality.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MASSACHUSETTS WATER RESOURCES COMMISSION
DIRECTOR
ADMINISTRATIVE SECTION
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WATER POLLUTION CONTROL

Figure 4.

SEWERAGE PROGRESS IN BOGOTA, COLOMBIA

by

DAVID R. HORSEFIELD* Member

(Paper presented at the Meeting of the Sanitary Section, Boston Society of Civil Engineers; October 4, 1967)

DESCRIPTION OF THE CITY

Colombia, located in the northwest corner of South America, is closer to the United States than any other South American country. As shown on Fig. 1, Bogota, the capital, is in the mountainous interior of the country, only three hours by jet from Miami. It is situated on a high plateau called the Sabana in the eastern range of the Andes at an altitude of about 8,500 ft above sea level and about 4° north of the equator. The Sabana itself is the site of a prehistoric lake surrounded by mountains which rise steeply another 3,000 ft immediately east of the city.

The Rio Bogota is the principal watercourse and only outlet of the Sabana, and the water supply for Bogota is obtained from this river, some 20 miles upstream. The river meanders sluggishly through the flat plains of the Sabana until it reaches control gates at Alicachin, which is located about 10 miles downstream from Bogota. Several miles below the gates, which control river levels for power generation purposes, lies the Tequendama Falls where the river pours over a 480 ft cliff into a lush tropical region. All waste water from Bogota passes untreated over these falls. The climate of Bogota is temperate the year around, with two distinct rainy periods, principally during the months of April and November. There is considerable variation in the occurrence and amounts of rainfall from place to place throughout the Bogota area.

During the rainy seasons, heavy thundershowers occur often at midday. Such storms usually occur over the city with surface winds from the southwest, the direction of the falls. These air currents are forced upwards on the steep ridges behind the city and the prevailing northeast winds then carry the thunderheads back over the city. The average annual rainfall on the plateau is about 36 inches, with over 80 inches on some of the higher elevations in the mountains and about 15 inches in areas several miles southwest of the city.

*Project Engineer, Camp, Dresser & McKee.

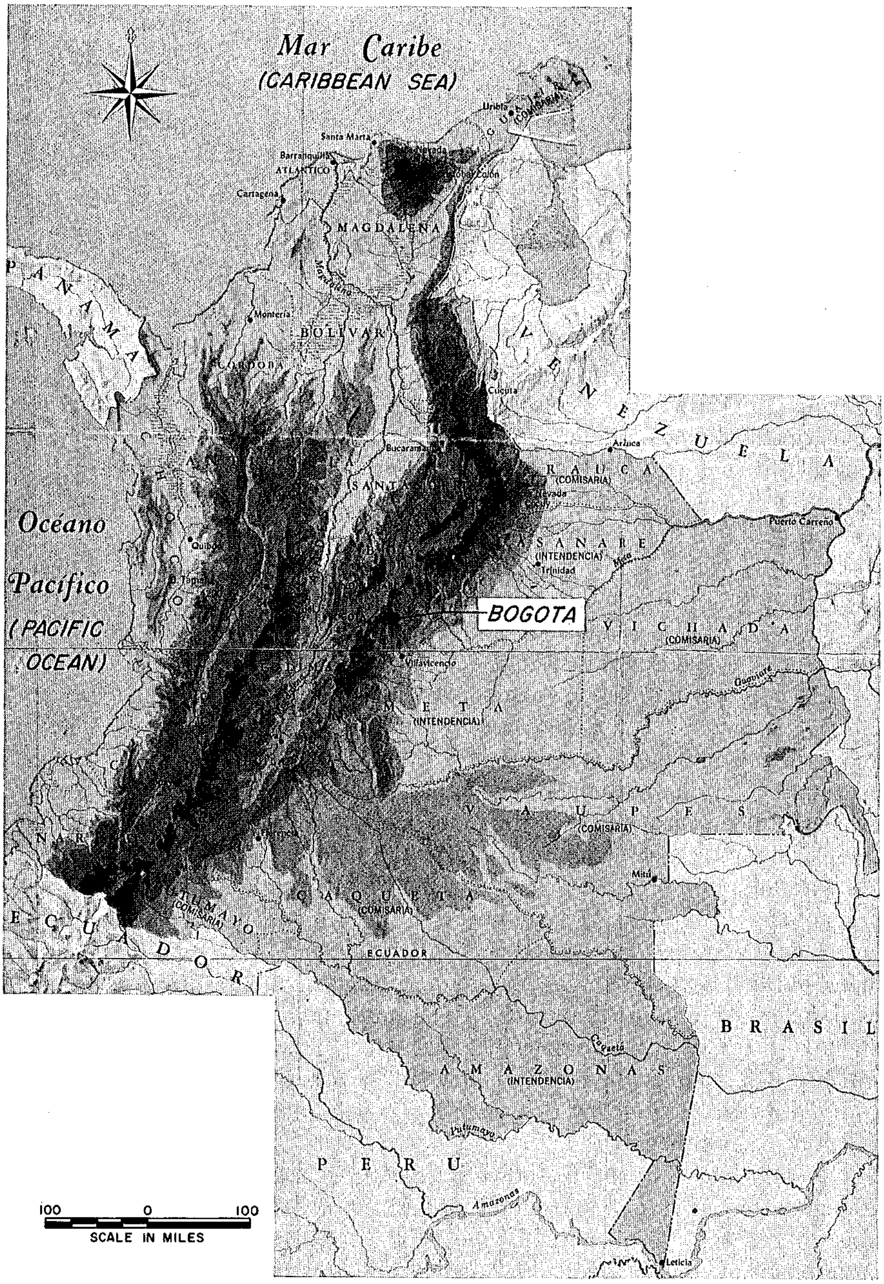

Fig. 1 — Map of Colombia

Portions of the city built on the lower slopes of the mountain range bordering the city on the east have generally hard rocky soil. The lower and more level sections of the city, which are in the area of the ancient lake, generally have soils consisting of organic silt and clay near the surface to clays with some layers of loose gravel and clayey sand at lower levels. The water table is located at depths of between 6.6 and 26 ft.

NEED FOR THE MASTER PLAN

Bogota, like most Latin American cities, is in the midst of a population explosion with ever-increasing sewage flows and worsening stream conditions. These factors, coupled with severe storm-water drainage problems and overflows of mixed sewage and storm water, result in serious hazards to public health throughout the Bogota Sanitary District. The Master Sewerage Plan for Bogota, completed in 1962, provides for a feasible program of staged construction with the goal of eliminating these hazards. Construction of the First-Stage project began early this year, and the long-range program, consisting of five stages, may take about 15 years to complete.

There are two objectives in the Master Plan for the disposal of sewage: (1) Protection of the public health, and (2) elimination of odor nuisances. The most important of these of course is the protection of the public from infection by disease organisms present in sewage.

In 1960, the Empresa de Acueducto y Alcantarillado de Bogota (Bogota District Water and Sewer Department) engaged Camp, Dresser & McKee and C.I.S. (Compania de Ingenieria Sanitaria), a Colombian sanitary engineering firm, to make engineering studies, and to develop a master plan for the disposal of sewage and storm water from the Bogota Sanitary District, comprising an area of approximately 46,000 acres. In 1964, after completion and adoption of the Master Plan, the First-Stage Feasibility Study was prepared by Camp, Dresser & McKee for the Empresa in accordance with the requirements of the United States Agency for International Development (AID). This study was prepared in Bogota with Empresa engineers working under our direction. The feasibility study recommended a First-Stage Construction program, consisting of canals and intercepting sewers which, when completed, will form a part of the long-range Master Plan. The author lived in Bogota for two years during the preparation of this study and construction plans.

The First-Stage program, shown by means of dotted lines on Fig. 2, is now under construction, and the Second-Stage program is under consideration. In general, the objective of the First and Second-Stage Construction programs is to construct collectors, canals and appurtenant facilities serving

Fig. 2 — Plan of Bogota Master Sewerage Program

the greatest problem areas which are above flood water levels in the Rio Bogota. Detailed planning of future stages of construction depends on whether the Rio Bogota channel is to be improved.

To prevent infections, closed conduits are proposed for all sewage, whether or not it is mixed with storm water. To eliminate the odor nuisance, it is recommended that future dry weather flows of sewage be conveyed in closed conduits to chlorination stations and thence to stabilization lagoons before it is discharged to open stream channels at locations well removed from densely populated areas.

PRESENT SANITARY CONDITIONS

There are well over 100 miles of open drainage ditches within the Sanitary District which carry mixed sewage and storm water. High water levels in the Rio Bogota and heavy storms over the city now cause the lower reaches of the main channels of the Rio Salitre and the Rio Fucha to overflow their contents of combined sewage into adjacent low-lying areas. Since most of the area is flat, these areas are quite extensive. Children play and animals roam in these areas, as shown on Figs. 3 and 4, and as a result, disease may be readily contracted and transmitted. Many new settlements have recently developed around the city as a result of heavy immigration from rural areas. Such settlements often are without water or sewer service.

In general, improvement in food sanitation, personal hygiene, sanitary conveniences, and sewage disposal has not kept pace with the improvement in quality of the public water supply. In 1961, the reported rate of cases of diarrhea and enteritis in children under 2 years of age was almost 12,500 per 100,000 children. It is reasonable to assume that the rate would be much higher if the number of unreported cases was known. Other water-borne diseases, such as amoebic dysentery, shistosomiasis and infectious hepatitis are also commonly experienced by members of the general population and foreigners staying in Bogota.

EXISTING SEWERAGE AND DRAINAGE SYSTEMS

It is estimated that about 1 million inhabitants are served by the existing sewerage system through over 100,000 household, commercial, industrial, institutional and public connections. There are approximately 27,600 acres of built-up area in the Sanitary District of which about 24,700 acres are presently sewered. Of these 24,700 acres, approximately 14,800 acres are presently served by combined sewers, and the remainder by a separate system of sanitary sewers. As in the United States, areas served by the existing separate systems consist for the most part of more recent housing

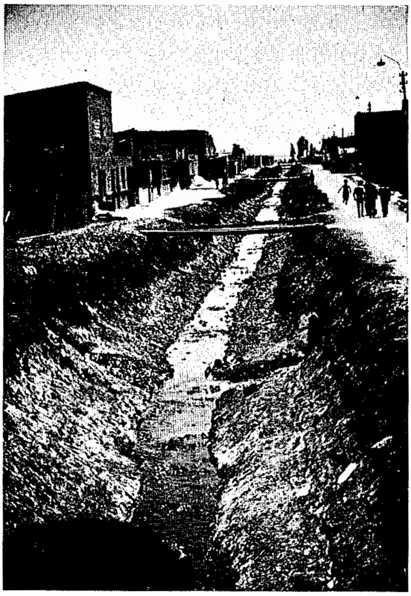

Fig. 3 — Unsanitary condition in Residential Area

Fig. 4 — Typical Conditions along Rio Salitre

developments. All new sewer systems are constructed on the separate plan. Separation of existing combined sewer areas is not recommended because its estimated cost is almost double the cost of collecting and treating combined flows.

DESIGN CRITERIA

Sanitary Sewers

Figure 5 shows past and projected population and water consumption trends for Bogota. Based upon evidence available at the time of the preparation of the Master Plan, it was expected that a population of approximately 3,200,000 might be attained by the year 1985. It may be seen from Fig. 5 that actual population increases from 1960 to the present time have exceeded those which were projected because of the heavy immigration mentioned above and a high birth rate. It thus appears that the projected population of 3,200,000 may be exceeded before 1985 if the present rate of growth continues. Such a population increase means that the consumption of water and the discharge of sewage will increase substantially, thus increasing the urgency for water and sewer facilities. There is very little use of the public water supply for lawn sprinkling, air conditioning and industrial water, or fire protection.

It is anticipated that by the design year of 2010 the entire population will be served with potable water in such a manner that water consumption will more closely reflect the water requirement of the area than it does at present. At the present time, many homes, particularly in poorer areas, are not directly served with water, and inhabitants must utilize the nearest public faucet. Approximately 10 per cent of the inhabitants of Bogota are supplied with water from such unmetered public fountains.

The average per capita sewage flow for the adequately served residential population is estimated to be about 62 gpcd. It is estimated that Bogota now has a total population of about 1,700,000 with an annual average sewage flow of about 88.2 mgd, or about 52 gpcd. It is estimated that between the years 1985 and 2010 the total water consumption and sewage flow for the Bogota Sanitary District will reach about 315 mgd, or about 97 gpcd.

Bogota has been painstakingly zoned for various land uses by the Bogota District Planning Department. Population densities for tributary areas were estimated, based upon these land uses. The average population density in residential areas by the year 2010 is estimated to be about 100 persons/acre. This results in an average per acre sewage flow by the year 2010 of approximately 9,800 g/acre/day.

Sewage flows have been based upon estimates of probable fixture unit

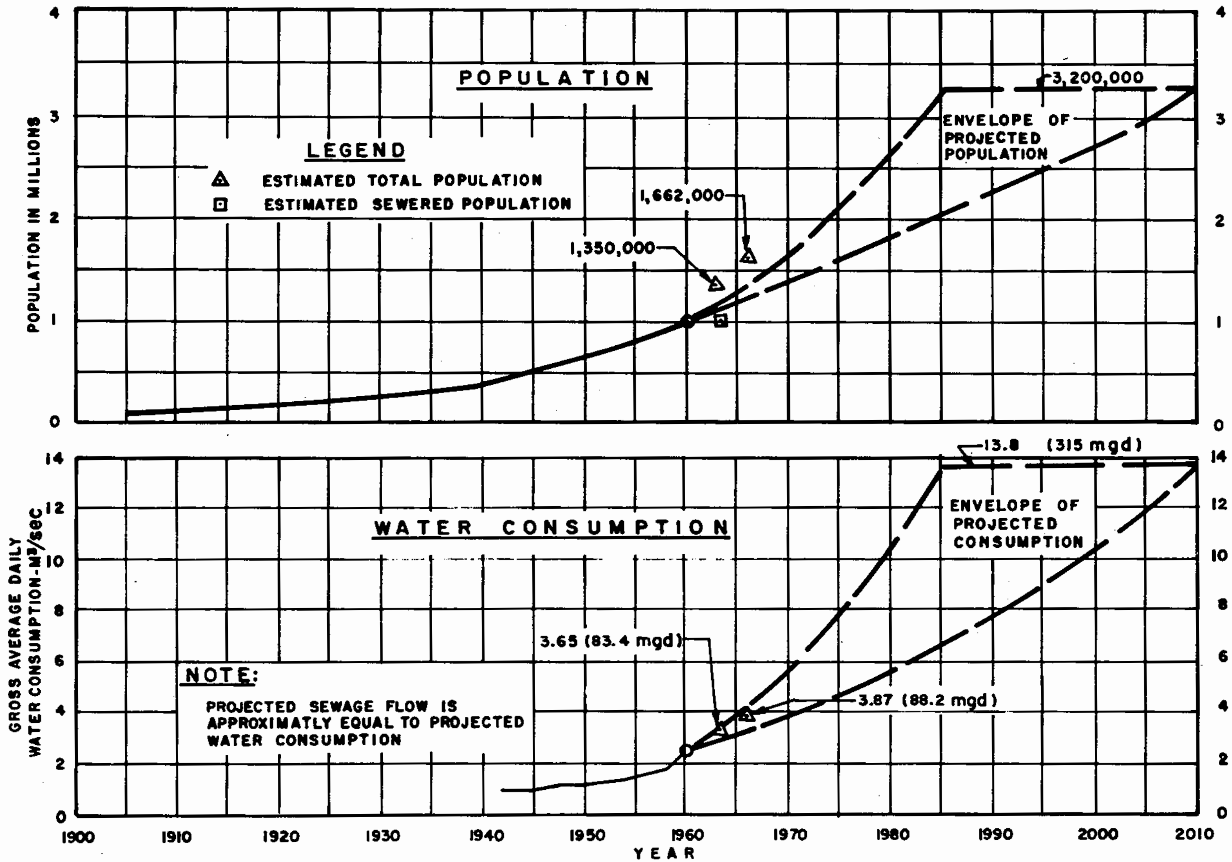

Fig. 5 — Population & Water Consumption Trends for Bogota Sanitary District.

densities for small areas, water consumption data and from gagings taken in various parts of the existing sewerage system. The flows are based upon 90 percent of saturation population for areas of 1,235 acres or more for design purposes.

Land use zones were arranged into six groups: A,B,C,D, E and F. The land use zones having the least population density are grouped under group A, while those having the highest population density are shown under group D. Groups A, B and C comprise most of the area to be occupied by residential water users. Zoned areas in Groups E and F are considered individually.

The design average flow for tributary areas of 1,235 acres and greater has been estimated at 7,400 g/acre/day for group A; 10,600 g/acre/day for groups B and C, and 16,600 g/acre/day for group D. Most of the tributary areas entering into the design of the Master Plan works are greater than 1,235 acres.

Peak flows of sanitary sewage for design purposes have been estimated, based upon the curves shown in Fig. 6. The curves on this figure were developed from work done in many communities in the United States and from curves found in the revised ASCE Manual of Engineering Practice No. 37, currently being published. Results of gagings taken throughout the Bogota sewerage system are shown.

The estimated rates of ground water infiltration are added to the estimated dry weather flow of sanitary sewage for the design of all sanitary sewers and interceptors. Our estimates are based upon extensive construction and maintenance experience of the Empresa throughout the Bogota area, upon observations of many exploratory test pits and upon soil analyses conducted by the Empresa. The zones of lowest infiltration lie in the higher elevations of the city near the base of the mountains, and the zones of highest expected infiltration lie in the lowlands toward the west and nearer the Rio Bogota. The estimated maximum infiltration rates in existing sewers vary between 1,850 and 3,700 g/acre/day. All sewers and interceptors proposed in the Master Plan will have preformed rubber joints to reduce rates of infiltration.

Sanitary sewers and dry weather interceptors for Bogota are designed to carry simultaneously estimated peak sanitary sewage flow, ground-water infiltration and industrial waste water flow for the design period. The design capacities of the sewers and interceptors flowing full are obtained by dividing the estimated peak flow by the following factors:

0.6 for pipes 8 to 21 inches in diameter

0.7 for conduits 24 inches to 47 inches (1.20m) in diameter

0.9 for conduits 49 inches (1.25m) and larger in equivalent diameter

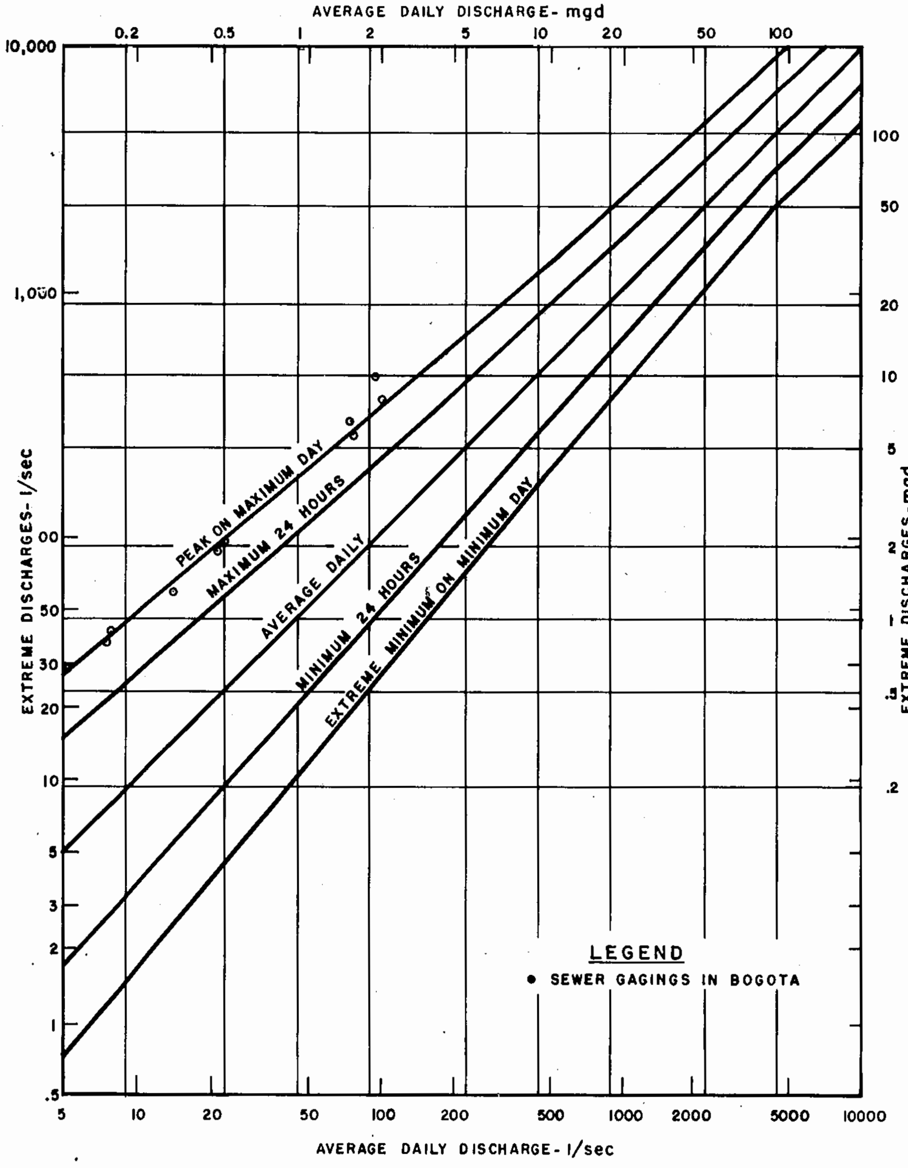

Fig. 6 — Relation of Extreme Discharges to the Average Daily Discharge of Sanitary Sewage

(It should be noted that pipe is manufactured locally according to the English system of units up to 36 inches in diameter, and according to the metric system from 1.00 m.

For pipes from 8 to 15 inches in diameter, a Manning's "n" value of 0.014 is used with slopes sufficient to produce scouring velocities of not less than 2 fps at flowing-full capacity. For pipes and conduits 18 inches in diameter and greater, an "n" value of 0.013 is used.

In addition to the above criteria, conduits with diameters greater than 47 in. (1.20m) are investigated to determine critical depth, particularly in areas of steep topography.

Conduits for Mixed Sewage and Storm Water

To protect the people from water-borne diseases, dry weather flows of sewage from existing combined sewer areas are to be received at points of overflow by interceptors designed in accordance with the criteria for the design of sanitary sewers. The intercepted sewage will ultimately be conveyed to chlorination stations and treatment works designed for the dry weather flow only. Open ditches and streams flowing through the combined sewer area, which receive overflows of mixed sewage and storm water, will be paved, and provisions will be made for enclosing such streams in the future. These paved streams or canals will carry flows to future chlorination stations prior to discharging to the Rio Bogota. These stations will be located on the Rio Fucha, Rio Salitre and Rio San Francisco.

The design of conduits carrying mixed sewage and storm water is simplified by estimating the rates of storm water runoff only, as discussed below. The size of rainstorm used for the determination of the design runoff for a particular conduit depends upon the damage which may result from an overflow or flood. The size of the rainstorms used for design are designated in terms of the frequency of their occurrence in years, as shown in Table 1.

Precipitation has been recorded in Bogota since about 1806. Continuous records have been made since 1922, from which we determined the amount of precipitation for five-minute intervals and developed intensity-duration curves.

On December 4, 1966, a severe rainstorm occurred in the mountains east of the city. Comparison of the rainfall intensity and duration for this storm with curves based on previous records indicated a recurrence frequency of about 5 years. Storm runoff velocities from this storm of about 15 fps were observed in downtown Bogota streets, resulting in severe damage to property, as indicated in Fig. 7.

TABLE 1
DESIGN STORM FREQUENCIES OF CONDUITS FOR VARIOUS TYPES OF AREAS

TYPE OF AREA	DESIGN RAINSTORM RECURRENCE FREQUENCY IN YEARS			
	TYPE A CANAL		TYPE B CANAL	
CONDUITS FOR UNCHLORINATED STORM WATER OR CHLORINATED MIXED SEWAGE AND STORM WATER	Pipe or Closed Conduit	Usable Channel	Paved Portion	Usable Channel
Located in Low Value Residential Areas	3	10	3	10
Located in Multiple Family & High Value Residential Areas; Medium Value, Residential, Commercial and Industrial Areas	5	20	5	20
Located in Principal Administrative Areas; High Value Commercial and Industrial Areas	10	40	5	40
Serving Tributary Area of over 1000 Hectares	—	20	5	20
Acequias - Storm Water Interceptors	—	40	10	40
Spillways from Acequias or Open Channels across Slopes	—	100	10	100
CONDUITS FOR UNCHLORINATED MIXED SEWAGE AND STORM WATER				
Located in Low Value Residential Areas	5	10	3	10
Located in Multiple Family & High Value Residential Areas; Medium Value, Residential, Commercial and Industrial Areas	10	20	5	20
Located in Principal Administrative Areas; High Value Commercial and Industrial Areas	20	40	5	40
Serving Tributary Areas of over 1000 Hectares	—	100	10	100

NOTE: All open channels for mixed sewage and storm water upstream from proposed chlorination stations are designed to be covered in the future.

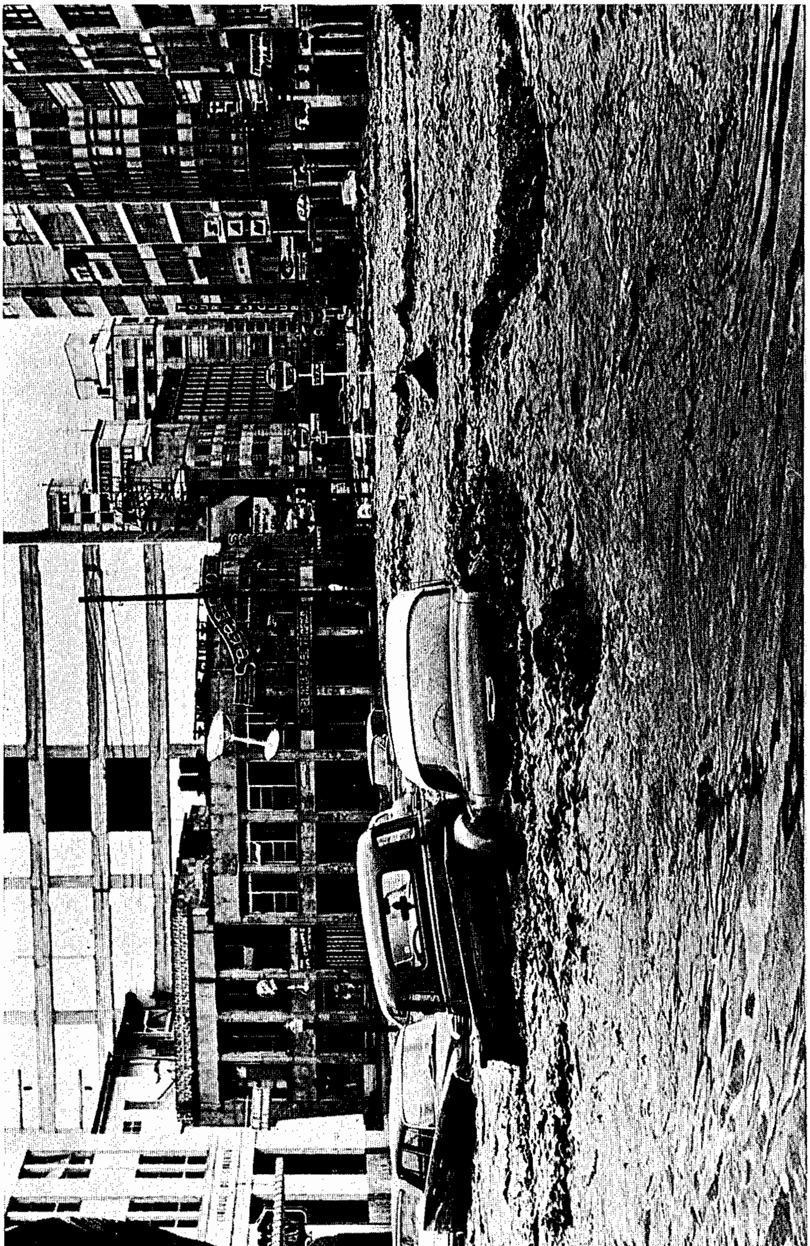

Fig. 7 — Downtown Street Scene during Storm of December 4, 1966

Rainfall intensities over large drainage areas are modified for design purposes by the use of reduction factors inasmuch as the average intensity of rainfall over a large area is considerably less than the intensity shown by the rain gages. These factors are supported by studies made by the U.S. Weather Bureau and others. Storm water runoff in the Bogota Sanitary District was estimated by the "Rational Method". Horner coefficients of runoff were adapted for use in Bogota by assuming that the intense rainfall starts after the ground has been wetted for ten minutes and by applying the "zone principle" to compute the average coefficients throughout the drainage area under investigation. Percentages of relative imperviousness were estimated for different land uses. The time of concentration of flow, which is the time of flow from the most remote point of a tributary area to the loading point on a conduit, was determined for estimated time for surface runoff and flow time in mountain streams, swales, ditches, gutters and conduits.

During the preparation of the Master Plan, many different canal sections were considered. A trapezoidal section (Type B) with paved and sodded portions was chosen as being most suitable and requiring very little structural design. A rectangular section (Type A) is considered for use in locations where lateral clearance is a problem. The recommended arrangement of the trapezoidal canal and interceptors is shown on Fig. 8. All pipes and closed conduits for mixed sewage and storm water are designed to flow full.

The various materials and Manning's roughness coefficients "n" which are used are listed below:

Conduit Size and Shape	*Material*	*Manning's "n"*
12 - 15 inch pipe	Vitrified clay	0.014
18 - 30 inch pipe	Vitrified clay	0.013
Pipes larger than 30 in. (0.76 m)	Reinforced concrete	0.013
All sizes and shapes	Brick	0.016
Type A or B canals	Reinforced concrete	0.013
Type A or B canals	Brick or soil cement	0.016
Type A or B canals	Sodded slopes	0.035

The effective "n" value for a full open canal with two different lining materials, such as concrete and sod, is computed from the following formula:

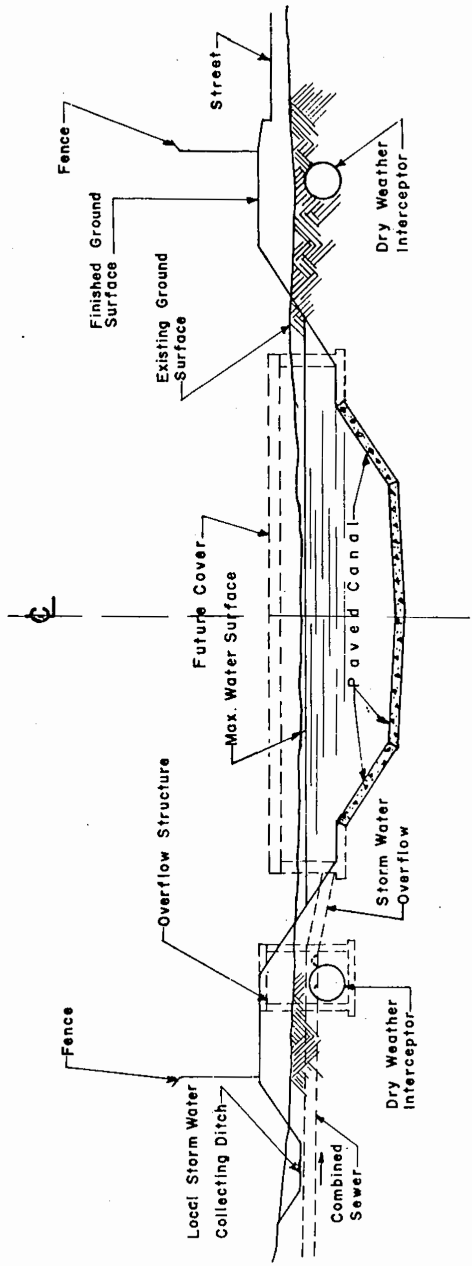

Fig. 8 — Recommended Arrangement of Canal and Interceptors

$$n = \left(\frac{P_1 n_1^2 + P_2 n_2^2}{P_1 + P_2} \right)^{1/2}$$

Where n_1 is the "n" value for the concrete portion, n_2 is the "n" value for the sodded portion, P_1 is the wetted perimeter of the concrete portion, and P_2 is the wetted perimeter of the sodded portion.

The slope of the energy line for all canals or other conduits for mixed sewage and storm water is computed to provide mean velocities flowing full of not less than 3.3 fps. Maximum design velocities are governed by the relation of critical depth to the depth of flow under consideration. Special junction and transition structures are designed where flows pass through critical depth, where a conduit discharges to one of larger size or where larger flows enter a conduit. The Froude Number at the design flow must be less than 0.9 or more than 1.1 in order to avoid the unstable flow and high friction factors which develop at or near the critical depth. In addition, losses are computed in cases where high velocities will occur on canal curves.

FIRST-STAGE CONSTRUCTION PROGRAM

The First-Stage Construction program consists of the construction of canals, intercepting sewers and collectors in various locations in the city. It includes the construction of about 12 miles of canals having bottom widths of from about 6.6 ft to over 115 ft, and the construction of about 22 miles of intercepting sewers and collectors ranging in size from 10 in. to over 10 ft. in equivalent diameter.

The First Stage has been arranged for construction into 22 separate contracts with the first contract awarded in December 1966. At present, 21 contracts have been awarded, all of them to Colombian contractors, and plans are being prepared on the last one. The total value of the 21 construction contracts plus the estimated cost of Contract 22 is about 13.7 million dollars, as shown on Table 2, based on an exchange rate of 9 pesos to the dollar. The various contracts in the First Stage are shown by numbers and dotted lines on Fig. 2.

Progress of the construction work is excellent with Contracts 12, 13 and 14 approaching completion. In general, the quality of work being performed is quite high and is improving as both the Empresa and the contractors become familiar with stricter requirements of inspection and construction than hitherto prevailed.

TABLE 2 FIRST STAGE CONSTRUCTION COSTS

Contract No.	*Name*	*Estimated Construction Cost X 10^6*
1	Comuneros I	6.11 pesos
2	Comuneros II	6.99
3	Comuneros III	7.76
4	Albina	3.27
5	Salitre IA	8.73
6	Rio Seco	6.66
7	Boyaca	6.01
8	Rio Nuevo	10.38
9	Cordoba A	6.56
10	El Cedro	4.15
11	Cordoba B	6.56
12	San Vicente IA	0.88
13	San Vicente IB	2.94
14	San Vicente IC	1.08
15	San Vicente II	1.53
16	Contador	8.62
17	Del Norte	7.97
18	San Francisco A	8.73
19	San Francisco B	8.73
20	San Francisco C	8.73
21	Salitre II	24.57
22	Salitre IB[1]	1.74
	Estimated Total Construction Cost[1]	148.70 pesos = 16.5 million dollars U.S.[2]
	Total Value of Contracts	122.98 pesos = 13.7 million dollars U.S.[2]

NOTES: [1]Contract 22 not yet awarded.
[2]Based on an exchange rate of 9 pesos to the dollar.

CONSTRUCTION METHODS

Circular conduits are constructed of vitrified clay pipe, precast reinforced concrete pipe and brick. Because of the ample supply of unskilled but easily trained labor and low cost (about 1.00 dollar per day for a common laborer), there is a tendency in Bogota to utilize less precast units and labor-saving equipment than in the United States. Brick is readily available, and as a result, brick conduits are being built as part of the Master Plan work, as shown on Fig. 9. Economic comparisons made prior to the awarding of construction contracts indicated that circular brick sections, precast reinforced concrete pipe sections, and cast-in-place reinforced concrete box culvert sections of equivalent hydraulic and structural capacity were about equal in cost. High quality reinforced concrete pipe is produced locally in quantities sufficient to support construction of the Master Plan. Installation of such pipe is shown on Fig. 10.

During the preliminary design phase of the First Stage facilities, it was considered feasible to use precast reinforced concrete canal slabs with temperature steel for the 1 1/2 to 1 (horizontal to vertical) side slopes and cast-in-place concrete for the base. However, tests run prior to construction, as well as subsequent construction experience in Bogota, indicated that casting the concrete side slopes in place on that slope is entirely feasible and results in a quite satisfactory job. Bedding for the slabs on the side slope is simplified by using vertical gravel-filled troughs spaced at about 6.6 ft intervals in the clayey subbase for drainage. The remainder of the slope is covered with a ½ in. to ¾ in. layer of mortar to prevent drying of the clay and provide a good working base. This construction is shown on Fig. 11.

Rectangular conduits are constructed of cast-in-place and precast reinforced concrete. At present, precast reinforced concrete slabs are being used for the side walls of box culverts on two contracts (contracts 8 and 21, as shown on Fig. 12). After these wall slabs are set in place, they are cast into the base slab. Spaces between slabs permit lapping of reinforcing steel to provide structural continuity.

FUTURE CONSTRUCTION PROGRAMS

The total estimated construction cost of the entire Master Plan program, consisting of a grand total of about 75 miles of open and covered canals and 87 miles of intercepting sewers, together with pumping and treatment facilities, is about 85,000,000 dollars U.S.

It is expected that facilities for the treatment of dry weather sewage flows will be constructed as part of the final stage. These treatment facilities were proposed to be stabilization lagoons because of the availability of land and shortage of electric power. Pilot plant studies are now under way to determine the loadings which may be utilized in Bogota. Because of increases in available electric power and the cost of land since 1964, further study appears warranted to determine whether conventional treatment facilities may now be more advantageous than lagoons.

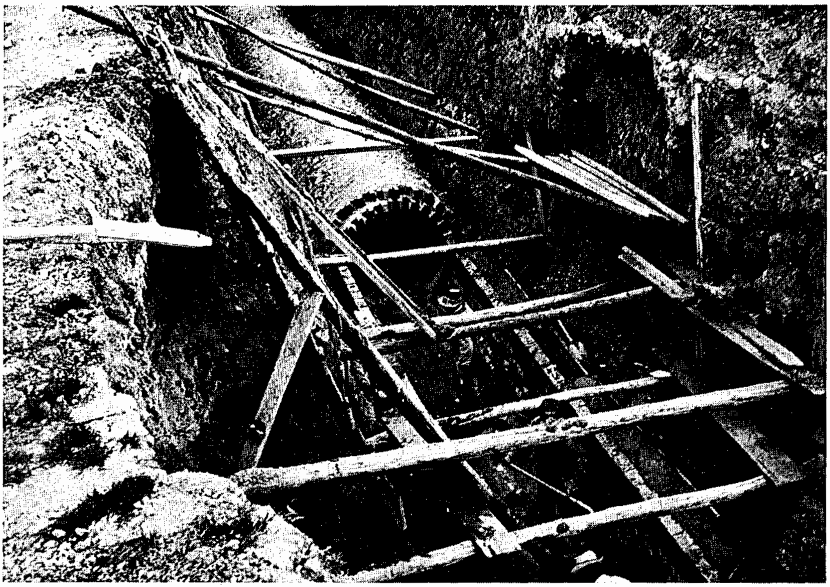

Fig. 9 — Construction of Brick Sewer

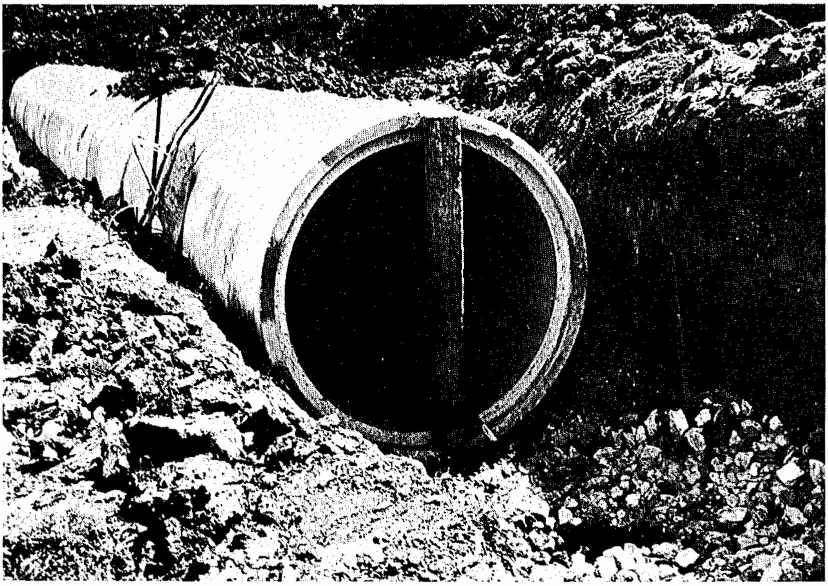

Fig. 10 — Installation of Reinforced Concrete Pipe

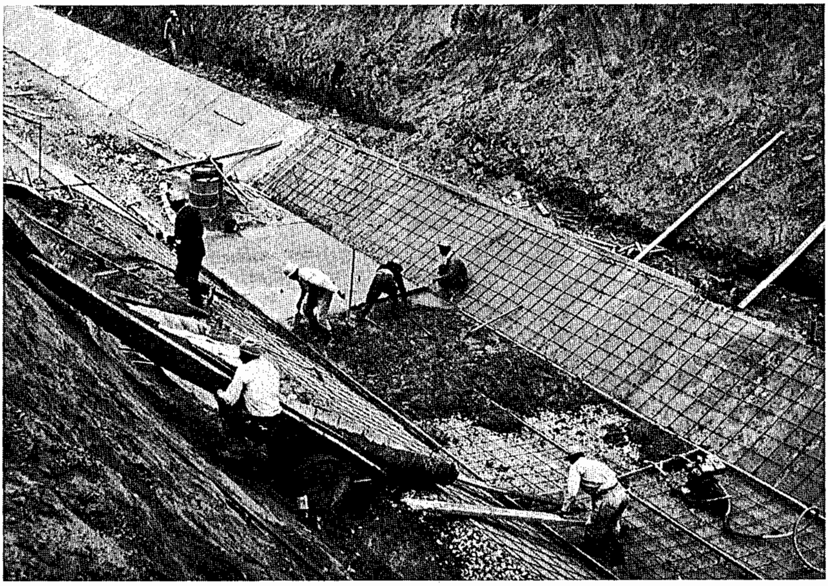

Fig. 11 — Paving Canals of Trapezoidal Cross-Section

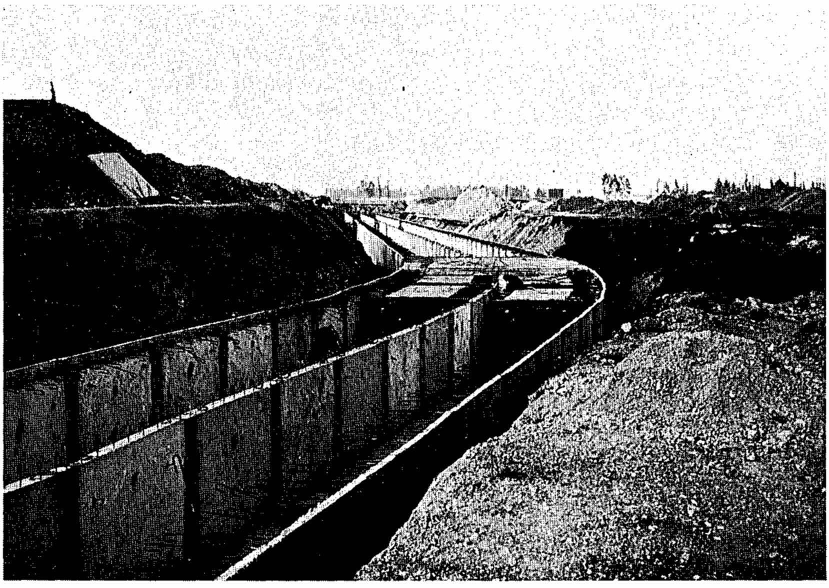

Fig. 12 — Double Box Culvert Canal Section under Construction

CONCLUSION

A Master Sewerage Plan has been developed for Bogota, Colombia which provides a feasible and orderly program for the elimination of serious flooding and water pollution conditions which now exist in the city. At the present time, detailed planning and construction activities to implement the Plan are progressing at a rapid pace. The program effectively utilizes Colombian engineering talent to solve a most pressing public health need.

Based upon work accomplished to date, several conclusions may be drawn:

1. Local (Colombian) engineering talent is of a high order.
2. The generally good quality of construction by local contractors is being improved by their growing familiarity with and use of modern techniques.
3. Low wage scales, coupled with comparatively high material prices, result in the utilization of considerably more manual labor and less mechanized equipment than is the case in the United States.
4. Public awareness of the need for proper sewage disposal and drainage in Bogota is now being followed by energetic efforts by local public agencies to achieve it.
5. United States consultants working in foreign countries must be flexible in their approach to solutions. In particular, it is essential that we recognize that approaches to solving sewerage and drainage problems in the United States may require modification to be fully applicable overseas.
6. Through its adoption of modern engineering practices and construction techniques adapted to local needs, material and labor, Bogota is now progressing toward pollution abatement on a large scale for the benefit of all of its citizens.

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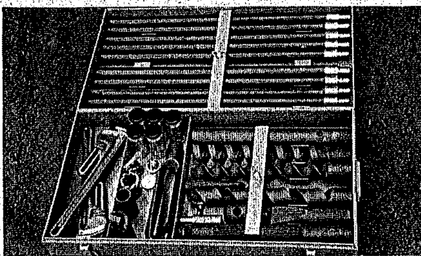

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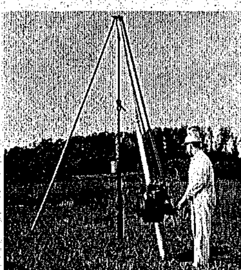

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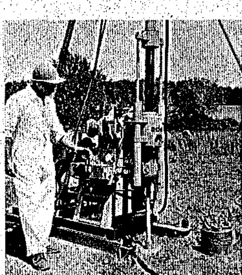

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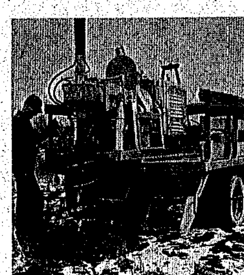

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