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BOSTON SOCIETY OF CIVIL ENGINEERS

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NOTES ON WATERFRONT PROPERTY LINES

By LORING P. JORDAN, JR., ESQ.*

(Presented at a meeting of the Surveying and Mapping Section, B.S.C.E., held on October 26, 1960.)

THE current plans for redevelopment of the City of Boston bring into focus a title problem which has existed in this Commonwealth for a good many years, i.e., that titles to tidelands filled after 1847 are not, generally speaking, freely marketable. Therefore, in any case in which the title attorney suspects that his locus may consist of filled tidelands, he must at a minimum ascertain the facts as to this matter and if he does determine that his locus was once tidal, he must attempt to ascertain whether it was filled before 1847. In view of the fact that a substantial portion of downtown Boston was once tidal, it may be reasonably anticipated that these difficult determinations must be made with respect to many redevelopment areas. However, it should not be thought that the City of Boston has a monopoly on the problem, for many areas in surrounding cities and towns also consist of filled tidelands.

In general the filling of such tidelands has been under authorizations granted by the General Court or by public bodies to whom the General Court has delegated this power. In 1941 the Supreme Judicial Court in *Commissioners of Public Works vs. Cities Service Oil Company,* 308 Mass. 349, suggested that such licenses are revocable. This suggestion is what has made necessary the more extensive examination of the titles to filled tidelands than that which the conveyancing bar generally feels is necessary for non-tideland titles. To put it another way, it is not common practice to examine titles to non-tidal properties back of 1890. On the other hand, it is essential that titles to filled tidelands be examined back to the first half of the nineteenth century, if not before, in order to determine whether the land has been filled under a revocable license, and to determine other circumstances which might

* Rackemann, Sawyer and Brewster, Counsellor at Law, Boston, Mass.

give rights to conflicting claims. There are at least three results of the *Cities Service Case*:

A. The examination of titles to filled tidelands, which titles are ultimately found to be good, is far more painstaking and hence more expensive than that of non-tidal titles.

B. Even if the title to filled tidelands is found not to depend upon a license which is revocable, it may well be that the evidence which leads to this conclusion is so obscure that the title cannot be said to be good and merchantable of record.

C. Many titles to filled tidelands are removed from commerce unless and until an act of the General Court is obtained which cures the defect, even though the land in question may have been filled and built upon for decades.

It is my understanding that the power to cure these defects is lodged only in the General Court and that no other state agencies (not even Redevelopment Authorities) have the power to cure them.

There follow notes prepared by the writer in connection with remarks which he made to the Boston Society of Civil Engineers, Surveying and Mapping Section, at its meeting on October 26, 1960. These notes will show the great dependence which the title attorney must place on the surveyor in the examination of titles to filled tideland. Whether the title is good or bad depends not only upon what is found in the Registry of Deeds, but also upon what existed on the ground many years ago and also upon the locations of the ancient harbor lines. It is obvious that only the surveyor can give the title lawyer this essential information.

I cannot conclude these prefatory statements without referring to the debt which I owe to my late good friend, Louis A. Chase, who, until his death, was a devoted member of the Surveying and Mapping Section of your Society and who introduced me to the law involved in this field. As a facet of his great professional knowledge, Louis A. Chase knew as much of the law of this difficult subject as did almost any title lawyer.

A. The Basis of Massachusetts Riparian Titles

1. *Title of the Crown*

"At the time of the settlement of Massachusetts and the other English colonies in America, the only source of title to the vacant and unsettled lands of this portion of the continent, claimed by the crown of England by right of discovery, was a grant from the king. It was not merely the only source of legal title to the soil, but the only source of authority for exercising limited powers of government, in and over the lands thus granted." *Comm. vs. City of Roxbury,* 9 Gray 451, 478 (1857)

2. *Indian Titles*

"The theory universally adopted, acted upon, and sanctioned by a long course of judicial decisions of the highest authority, was, that the Indians found upon this continent had no legal title to the soil, as that term was understood at the common law and among civilized nations, no fee in the land, but only a temporary right of occupancy, for which it was perhaps equitable to make them some allowance." *Comm. vs. City of Roxbury,* 9 Gray 451, 478 (1857)

3. *Grant to Massachusetts Bay Colony*

"All the lands were first granted by the crown to the Governor and Company of the Massachusetts Bay in New England, and by them were parcelled out to individuals, and, at a later period, to bodies of proprietors, as tenants in common." *Porter vs. Sullivan,* 7 Gray 441, 443 (1856)

4. *Power of General Court*

"But in 1634 the general court, consisting of the governor, deputy governor, and assistants, and all the freemen of the company, (which had power by the charter to make laws and ordinances for the government of the colony and its inhabitants), declared that none but the general court had power to choose and admit freemen, to make laws, to elect or remove high officers and define their powers and duties, to raise moneys and taxes, or 'to dispose of lands, viz. to give and confirm proprieties.' 1 Mass. Col. Rec. 117." *Boston vs. Richardson,* 13 Allen 146, 149 (1866)

5. *Incorporation of Towns*

"Of course, all the early records of the colony are filled with acts prescribing the bounds of each township; but such acts, intended solely to fix the limits of jurisdiction, are never used as instruments for granting land." *Porter vs. Sullivan,* 7 Gray 441, 444 (1856). "This fixing of town lines merely determined the limits of municipal jurisdiction. It neither conferred any right of property nor restrained any such right. A man might well own upland in one town, and the flats appurtenant, although in another town. Rights, either private rights of property or common rights, depended on other considerations." Same, (450)

6. *Right of Town to Grant Land*

"And on March 3, 1635-6, it was 'ordered that the freemen of every town, or the major part of them, shall only have power to dispose of their own lands and woods, with all the privileges and appurtenances of the said towns, to grant lots, and make such orders as may concern the well ordering of their own towns, . . .' " *Lynn vs. Nahant,* 113 Mass. 433, 448 (1873). "The lands within the limits of a town which had not been granted by the government of the Colony either to the town or to individuals, were not held by the town as its absolute property, as a private person might hold them, but, by virtue of its establishment and existence as a municipal corporation, for public uses, with power by vote of the freemen of the town to divide them among its inhabitants, yet subject to the paramount authority of the General Court, which reserved and habitually exercised the power to grant at its discretion lands so held by the town." Same, (448)

7. *Ordinance of 1641–1647*

"This is commonly denominated the ordinance of 1641; but this date is probably a mistake. It is found in the Ancient Charters, 148, in connection with another on free fishing and fowling, and marked 1641, 47. That on free fishing, etc., is taken in terms from the 'Body of Liberties,' adopted and passed in 1641, leaving the date 1647 to apply to the other subject respecting ownership in coves, etc., about salt water. . . .

"The whole article, as it stands in the Ancient Charters and in the edition of the colony laws of 1660, is as follows:

'Sect. 2. Every inhabitant who is an householder shall have free fishing and fowling in any great ponds, bays, coves and rivers, so far as the sea ebbs and flows within the precincts of the town where they dwell, unless the freemen of the same town, or the general court, have otherwise appropriated them: provided, that no town shall appropriate to any particular person or persons, any great pond, containing more than ten acres of land, and that no man shall come upon another's propriety without their leave, otherwise than as hereafter expressed.

'The which clearly to determine; Sect. 3. It is declared, that in all creeks, coves, and other places about and upon salt water, where the sea ebbs and flows, the proprietor, or the land adjoining shall have propriety to the low water mark, where the sea doth not ebb above a hundred rods, and not more wheresoever it ebbs further: provided, that such proprietor shall not by this liberty have power to stop or hinder the passage of boats or other vessels, in or through any sea, creeks, or coves, to other men's houses or lands.

'Sect. 4. And for great ponds lying in common, though within the bounds of some town, it shall be free for any man to fish and fowl there, and may pass and repass on foot through any man's propriety for that end, so they trespass not upon any man's corn or meadow. [1641, 47.].'" *Comm. vs. Alger,* 7 Cushing 53, 67, 68 (1851)

"The views, we believe, that the courts of this state have constantly taken of the construction of the colony ordinance, are these: That it vested the property of the flats in the owner of the upland in fee, in the nature of a grant; but that it was to be held subject to a general right of the public for navigation until built upon or inclosed, and subject also to the reservation that it should not be built upon or inclosed in such manner as to impede the public right of way over it for boats and vessels." Same, (79)

B. Fresh Water Titles

1. *Rivers*

(a) Riparian Owners Own to Thread of Fresh Water Streams.

"The most satisfactory analogy would seem to be that presented by a fresh-water stream or river where the line of division between opposite proprietors is the thread of the stream . . . . In such a case each proprietor owns an equal share of the bed of the stream in proportion to his line on the margin and in front of or adjacent to his upland. . . . The principle of division between them is, as in the case of flats, that of equality, and the division is effected by drawing lines at right angles from the termini of the side lines on the shore to and at right angles with the thread of the stream. . . . By the thread of the stream is meant the centre line from one bank to the other, not when swollen by floods, or diminished by drought, but in its ordinary and natural condition. This may or may not coincide with the channel. That is immaterial. And it is immaterial also whether there is one channel or more than one. . . ." *Tappan vs. Boston Water Power Company,* 157 Mass. 24, 30, 31 (1892)

(b) Division of Flats Between Private Owners of Land on Tidal River from which Tide Fully Recedes.

". . . we think the demandants are respectively entitled to recover so much as falls within straight lines drawn from the termini on its banks of the side lines of their respective marsh lands, at the ordinary stage of the water, to and at right angles with the centre line of the stream." *Tappan vs. Boston Water Power Company,* 157 Mass. 24, 31 (1892)

2. *Great Ponds*

". . . a grant bounded by a great pond or lake which is public property extends to low water mark." *Paine vs. Woods,* 108 Mass. 160, 170 (1871)

"In the present case, it appeared that the land in question was flowed, and the pond raised to an artificial height, in winter only, and that in summer the pond was allowed to remain at its natural level. Applying to this case the rules already stated, the conclusion is that the deeds, under which the complainant claims title, bounding him 'to' and 'on the pond,' extended to low water mark of the pond in its natural state; and that the fact that the deeds were made during the season when the pond was temporarily raised by the dam cannot affect the extent of their operation." Same, (172)

C. Private Ownership in Tidal Flats

1. *Extent of Private Ownership*

"The inner and outer limits of proprietorship under the colonial ordinance and the laws of the Commonwealth are well settled. The inner line is that of high water at ordinary tides . . . . The outer line, as determined by repeated decisions of this court, is that of extreme low water, if within one hundred rods, because it is often necessary to the enjoyment of the rights granted by the ordinance to have a wharf extend to low water mark when the tide ebbs the lowest." *Wonson vs. Wonson,* 14 Allen 71, 82 (1867)

". . . a natural and original creek, in which the tide ebbed and flowed, and from which it did not ebb entirely at the time when from natural causes it ebbed the lowest, would constitute a boundary of the flats, . . ." *Attorney General vs. Boston Wharf,* 12 Gray 553, 558 (1859)

2. *Effect of Natural and Artificial Changes in Flats*

"The change came gradually from natural causes, when there were no marks or boundaries to show exactly what was the line of mean low water in 1640, or at any later date. Upon the doctrines applying to accretion and erosion and to the elevation and subsidence of land affecting the water line along the shore of the sea under conditions like these, the line of ownership follows the changing water line." *East Boston Company vs. Commonwealth,* 203 Mass. 68, 75 (1909)

"Whatever increase, therefore, happened from natural causes, or from a union of natural and artificial causes, within that distance, must be to the benefit of the owner of the upland, or of him who owned the flats to which the increase was attached. This increase is of necessity gradual and imperceptible. No man can fix a period when it began, no testimony can mark the exact margin of the channel on any given day or year. The ancestor being seised of the estate, of which all the flats now demanded are part, and having the right by law to all such additions as should be made by the gradual retiring of the waters, he must supposed to have been seised of all which now exists, for no one can show any parcel of which he was not seised." *Adams vs. Frothingham,* 3 Mass. 352, 362, 363 (1807)

D. WHARFING AND FILLING

1. *Development of the Law as to Wharfing and Filling*

(a) "The object of the ordinance of 1641, from which the right to flats originated, was to give the proprietors of land adjoining on the sea convenient wharf-privileges, to enjoy which to the best advantage, it is often necessary to extend their wharves to low-water mark at such times when the tides ebb the lowest." *Sparhawk vs. Bullard,* 1 Metcalf 95, 108 (1840)

(b) From 1647 to 1837, when the first Harbor Line was established, a private owner had the absolute right to enclose and fill his flats out to the line of his ownership established by the ordinance of 1647.

(c) Chapter 229 Acts of 1837. Established Harbor Line in Boston Harbor. Under this statute (other Harbor Line statutes were similar) the littoral owner could build wharf or could fill, only to the line of his ownership or to Harbor Line, whichever was closer to shore.

(d) From 1647 to 1866 no license from General Court or elsewhere was required for filling privately owned flats situated within the applicable Harbor Line. After Harbor Lines were established, licenses from the General Court were required (1) to fill privately owned flats outside of Harbor Line and (2) to fill Commonwealth flats whether within or outside of Harbor Line.

(e) Chapter 149 Acts of 1866. Board of Harbor Commissioners must approve method of filling and structures in tidewaters. License from General Court still required to fill privately owned flats outside Harbor Lines and for filling Commonwealth flats. Compensation for tidewater displacement to be paid for all licenses thereafter granted.

(f) April 30, 1836. Revised Statutes, Chapter 119, Section 12. First statute limiting rights of action of Commonwealth: establishes twenty year limit, for actions to recover real property. Note that adverse possession could run against Commonwealth in tidelands until May 27, 1867, Chapter 275 Acts of 1867. Thus adverse maintenance of fill, a pier or a dock for twenty uninterrupted years: from April 30, 1836 to May 27, 1867 gave rise to fee simple (or perhaps irrevocable easement).

(g) Prior to 1869 any owner who had filled in flats by virtue of a license acquired a title by legislative grant which was indefeasible, see *Treasurer and Receiver General vs. Revere Sugar Refinery,* 247 Mass. 483 (1924)

(h) Chapter 432 Acts of 1869. "All authority or license . . . to build any structure upon ground over which the tide ebbs and flows . . . shall be revocable at any time, at the discretion of the legislature, and shall expire at the end of five years from its date, except where and so far as valuable structures, fillings or inclosures, . . . shall have been actually and in good faith built or made under the same." Now embodied in G.L. (Ter. Ed.) C. 91 S. 15

(i) Chapter 236 Acts of 1872. General Court turned over to the Board of Harbor Commissioners the authority to grant licenses to fill flats whether publicly or privately owned. Also provides that licenses not recorded within one year shall be void: this provision now appears in G.L. (Ter. Ed.) C. 91 S. 18. Failure so to record renders license void. *Tilton vs. City,* 311 Mass. 572 (1942)

(j) Chapter 284 Acts of 1874. Requires payments for licenses granted to fill Commonwealth tidelands at rates fixed by Governor and Council; this payment in addition to tidewater displacement charge.

(k) From 1869 to 1941 it was the general opinion of the Bar that licenses after acted upon in good faith were irrevocable and equivalent to fee simple titles, see *8 Attorney General's Opinions* 220.

(l) *Com'rs. of Public Works v. Cities Service Oil Company,* 308 Mass. 349 (1941). The Supreme Judicial Court in a dictum suggested that licenses to fill tidewaters may be revocable by the General Court even as to valuable structures built in good faith.
"We are of opinion, however, that . . . the exception as to valuable

structures does not apply in the event that the General Court, in the exercise of its discretion, sees fit to revoke the authority or license, but, on the contrary, that it is an exception relating to the provision that the authority or license shall expire in five years from its date." *Com'rs. Public Works vs. Cities Service Oil Company,* 308 Mass. 349, 363, 364 (1941)

(m) Chapter 748, Acts of 1911, Licenses with respect to Boston Harbor cease in the event of non-use; provision now contained in G.L. (Ter. Ed.) C. 91 S. 16.

(n) Under certain legislation the owners of flats were *ordered* to fill them to abate menaces to health, see: Chapter 304 Acts of 1873, Chapter 197 Acts of 1878, Chapter 238 Acts of 1881, and Chapter 144 Acts of 1883. Would not the right to maintain such filling be indefeasible?

2. *Mechanics of Examining Titles to Wharves and Filled Land*

(a) Determine primitive mean high water and primitive extreme low water lines from ancient plans.

(b) Examine title in Registry of Deeds back to time before wharfing or filling took place.

(c) Run all owners in chain of title in Office of Counsel to Senate to ascertain existence of Legislative Acts permitting filling or wharfing. Such grants up to Chapter 432 of Acts of 1869, gave title in fee simple. *Treasurer & Receiver General vs. Revere Sugar Refinery,* 247 Mass. 483 (1924); *Bradford vs. McQuesten,* 182 Mass. 80 (1902).

(d) Run in card files in Waterways Division, Department of Public Works, all owners of upland for licenses from Harbor and Land Commissioners, Director of Port of Boston and other administrative agencies which from time to time had authority to grant licenses.

(e) Determine what Harbor Lines have been established. Note that such lines did not give title but to the contrary limited right to fill or wharf. *Commonwealth vs. Alger,* 7 Cush. 53 (1851)

(f) Ascertain extent of filling and wharfing.

I. On or before May 27, 1847: since twenty years possession up to May 27, 1867 gave title. Revised Statutes, Chapter 119 Section 12 and Chapter 275, Acts of 1867. *Nichols vs. City of Boston,* 98 Mass. 39 (1867)

II. On date first Harbor Line was established: since before Harbor Lines were established no licenses were required for filling and wharfing out to closer to shore of (a) Primitive Extreme Low Water line, or (b) 100 rod mark.

III. On April 23, 1872, since before passage of Chapter 236 Acts of 1872 no licenses were required for filling and wharfing out to the closer to shore of (a) Harbor Line, (b) Primitive Extreme Low Water Line (c) 100 rod mark.

(g) Determine whether licenses granted after April 23, 1872 (Chapter 236 Acts of 1872) have been properly recorded; otherwise they are void. Often it is necessary to run Commonwealth on the grantor schedules and to run owners on the grantee schedules to be assured whether licenses have been properly recorded.

(h) Determine whether all licenses granted after June 21, 1869 were exercised within 5 years. Otherwise, they are void. (Chapter 432 Acts of 1869).

(i) Determine whether special acts may exist under which General Court has waived its right to revoke a specific license. See Chapters 773, 774, 775 and 776 Acts of 1957; Chapters 799 and 803, Acts of 1960; Chapter 566 Acts of 1961.

(j) Plot the several lines established by the above considerations.

3. *Land Court problems with respect to wharfing and filling*

(a) Consequence of Land Court Registration to Harbor Line.
G.L. (Ter. Ed.) C. 185 S. 46 provides: "Every petitioner receiving a certificate of title in pursuance of a decree of registration, and every subsequent purchaser of registered land taking a certificate of title for value and in good faith, shall hold the same free from all encumbrances except those noted on the certificate, and any of the following encumbrances which may be existing:
"First, liens, claims or rights arising or existing under . . . the statutes of this Commonwealth which are not by law required to appear of record in the Registry of Deeds in order to be valid against subsequent purchasers or encumbrances of record. . . ."
Are not rights of Commonwealth with respect to tidelands, including rights to revoke licenses within above exception? If so, does registration give any protection to a private owner?

(b) If upland title is registered should licenses be registered or recorded. Note that license may be to fill land below low water line, title to which is not registered.

E. SIDELINES OF FLATS

1. G.L. (Ter. Ed.) C. 240 S. 19 provides: "One or more persons holding land or flats adjacent to or covered by high water may apply by petition to the land court for the settlement and determination of the lines and boundaries of their ownership therein."

2. "The underlying principle is simply the adoption of such methods of division as will give to each parcel a line at low water proportional to its line at high water." *Bodwell vs. Bradstreet*, Davis: Land Court Decisions p. 34.

3. Where the general course of shore at primitive mean high water is substantially a straight line the flats before the shore are ordinarily to be divided by straight lines at right angles to general lines of coast. See figure 1.

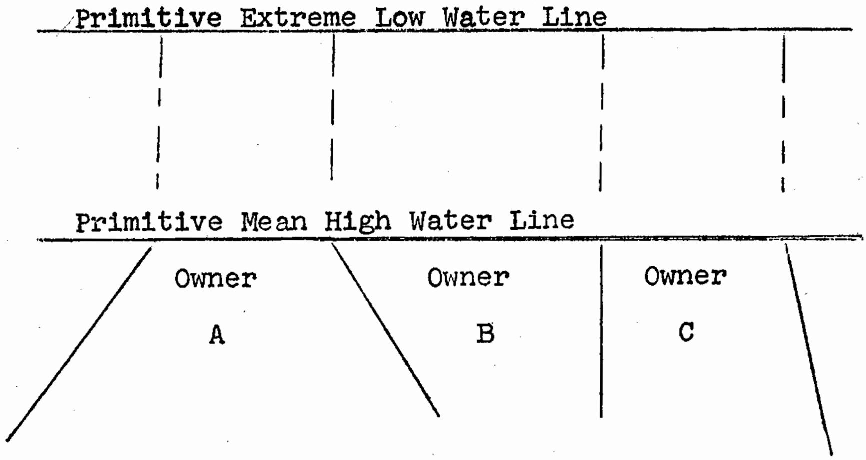

FIGURE 1

4. "On each side of the cove there is a headland, the proprietors of which are entitled to have a division of their flats by diverging lines, giving to each proprietor a greater width of flats at low water than at high water." *Gray vs. DeLuce,* 5 Cush. 9, 13 (1849). See figure 2.

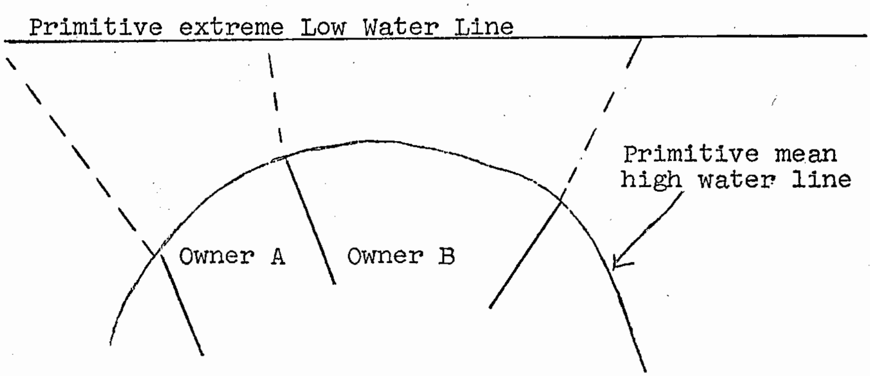

FIGURE 2

5. The flats in a shallow cove are to be divided by running a base line across the mouth of the cove, and dividing the flats by lines at right angles to these base lines. *Gray vs. DeLuce,* 5 Cush. 9, (1849). See figure 3.
6. The same rule applies to division of flats in a cove if the line of primitive extreme low water is almost entirely outside of, and nowhere more than a few feet inside the base line. *Stone vs. Boston Steel & Iron Company,* 14 Allen 230 (1867). See figure 4.
7. "Let us suppose that a line drawn across the mouth of the cove were 100

rods in length; and that the circular line of the cove at high-water mark were 200 rods in length. Then each proprietor of a lot abutting on the cove would be entitled to run his lines from the two corners of his lot in a direction to low-water mark, so as to include a piece of flats which would be at the mouth of the cove one half of the width of the lot at

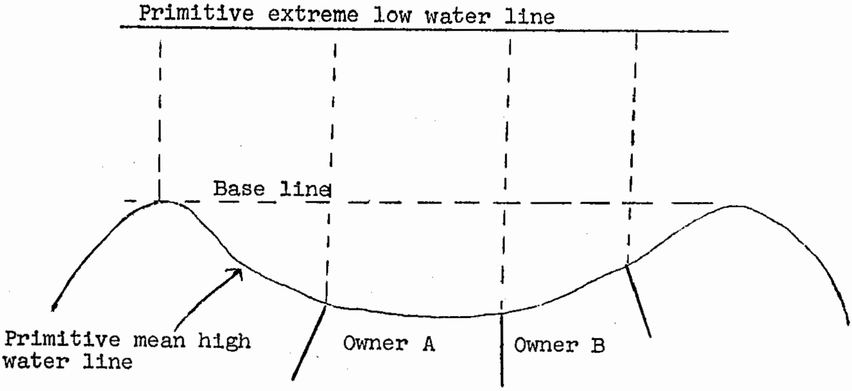

FIGURE 3

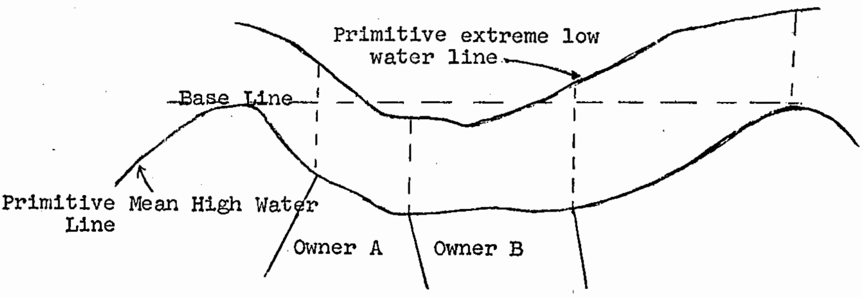

FIGURE 4

high-water mark; and thus by converging lines the whole cove might be divided without any intersecting lines." *Rust vs. Boston Mill Corporation,* 6 Pick. 158, 167, 168 (1828). See figure 5.

8. A creek from which the tide never ebbs running through a cove is a boundary, notwithstanding the foregoing rules. *Attorney General vs. Boston Wharf Company,* 12 Gray 553 (1859).
9. The owners of flats may, of course, change the foregoing boundary lines. *Adams vs. Boston Wharf Company,* 10 Gray 521 (1858).
10. "Where the mouth of the cove narrows and broadens again, a base line should be drawn across the narrowest part. Toward this base line proportionately divided as above provided the side lines should, of course, converge. Beyond the base line they should so diverge as to 'give each

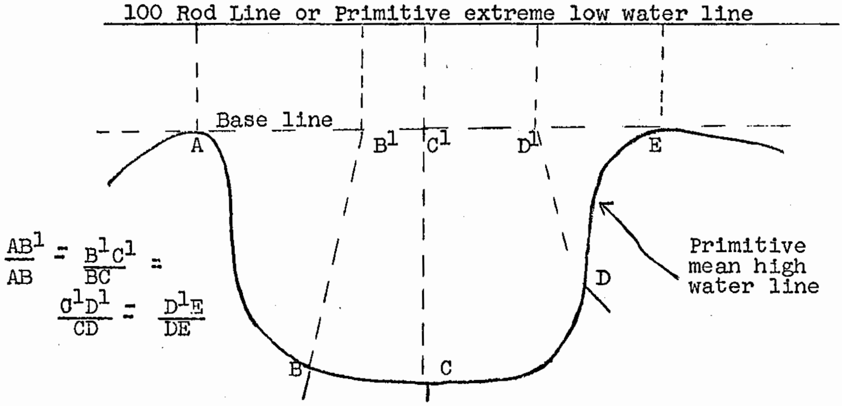

FIGURE 5

owner his due proportion.'" *Bodwell vs. Bradstreet,* Davis: Land Court Decisions 34, 36. See also division of Patten's Cove, Land Court Misc. 1061. See figure 6.

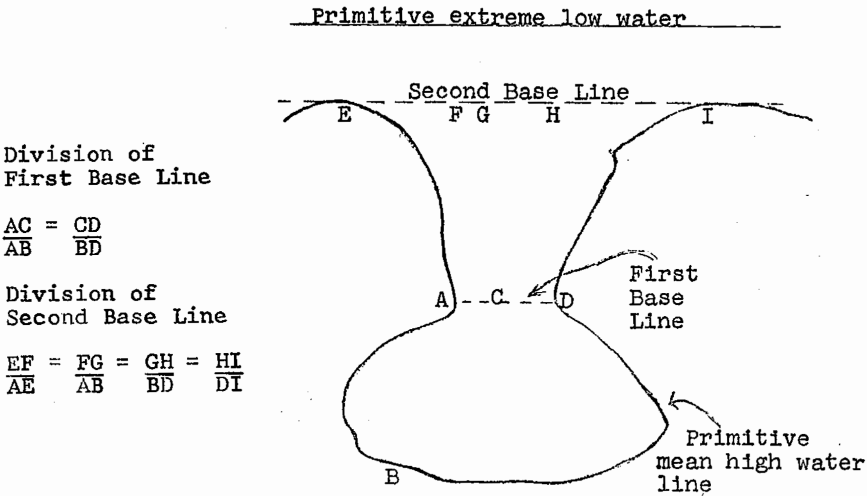

FIGURE 6

F. INTEREST OF U. S. A.

By the Submerged Lands Act, U. S. Code, Title 43, Chapter 29, enacted in 1953, the Federal Government released to the several states all its right, title and interest to the tidelands seaward to a line three geographical miles distant from the coast line of each state and, to the State's boundary at the time of its admission to the Union, if such boundary lay seaward of the three mile mark.

HYDRAULIC DESIGN OF DETENTION TANKS

By R. Stevens Kleinschmidt,* *Member*

Introduction

The use of large tanks or basins for improving the quality of water goes back at least to Roman times. Frontinus[1] reports that in the first century of the Christian era a basin was provided for the removal of suspended matter from the water of New Anio Aqueduct. In the more recent past, settling basins were often provided for control of turbidity well before the advent of modern water treatment. In present day water and sewage treatment plants such tanks are still of major importance. Their use is to provide time for suspended solids to settle, and for this reason they come under the general heading of detention tanks. However, as shown by Hazen,[2] the operation of a settling basin is primarily controlled by the surface area and flow rate and may be quite independent of depth. It is, therefore, not primarily a function of the detention time of the basin, so these will be excluded from the definition of detention tanks.

On the other hand, there are in use today tanks provided for the sole purpose of allowing time for some process other than sedimentation to take place. The name "detention tank" will be reserved for these. Chlorine contact tanks are a good example. In these, time is provided for the chlorine to kill any organisms present in the water. Another use is in the disposal of radioactive wastes with a short half-life. Here, basins are used to provide time for decay.

Another use of continuous flow tanks is for the dilution of slugs of objectionable material in order to keep their concentration below a maximum permissible level. Such a tank might well be placed in the sewer from a laboratory in order to prevent concentrated chemical solutions from entering the receiving system. These will be called integrating tanks and their ideal performance will be seen to be quite different from that of detention tanks.

The field of sedimentation tank design has been well covered in the

---

* Assoc. Professor of Civil Engineering, Northeastern University, Boston, Mass.

[1] Frontinus, "The Stratagems and the Aqueducts of Rome," Tr. Charles E. Bennett, p. 357, Putnam, 1925.

[2] Allen Hazen, "On Sedimentation," Trans. Am. Soc. Civil Engrs., 53, 63 (1904).

past[3] but a literature search disclosed an almost complete lack of information on the design of detention tanks. The present work attempts to alleviate to some degree this vacuum. The practical importance of the work can be seen by the fact that today there are large and expensive detention tanks in use in the atomic energy industry which are giving removals which might be expected of tanks of one-half the size, working near optimum efficiency.

The fundamental engineering problem is "How can the most satisfactory detention tank be built for the least money?" To answer this one must first consider to what use the tank is to be put. Next, one must determine what optimum operation would be, and follow with a look at those factors which will prevent this optimum performance from being realized. Then means must be found to minimize these detrimental factors, and finally, one must determine from the over-all picture just what will constitute the most economical design to fulfill the requirements. This is a complex problem since it involves the cost of materials, labor, excavation and available space. Moreover, the shapes of tanks that provide the greatest volume per unit cost are generally unsuitable for good detention tanks. However, if the effect of shape, velocity, baffles and inlet and outlet arrangements on performance can be evaluated and the practical limits established over which these factors may be varied, then the designer will be in a better position to design the most economical tank for a given set of conditions. As a secondary objective an attempt will be made to develop a theory for operating small scale models so that where cut and try is needed it can be done at small expense before construction of a prototype is begun.

Because of the common occurrence of exponential decay of substances to be removed from water it will be worth while to investigate tanks used to provide time for such a process to take place. Optimum performance would be realized if each particle remained in the tank for exactly one theoretical detention time. Such a situation can be visualized as follows. If a slug of dye were introduced into the inlet of such an ideal tank, the dye would spread instantly into a thin plane at right angles to the flow and this plane would move down the tank remaining undistorted until it reached the outlet where it would draw together and leave the tank as a unit. That this would give minimum concentration in the effluent will be proved following the discussion of a most powerful research tool—the flow-through curve.

[3] G. M. Fair and J. C. Geyer, Water Supply and Waste Water Disposal," Ch. 22, Wiley 1954.

THE FLOW-THROUGH CURVE

In the investigation of detention tanks, the so-called "flow-through curve" (Figure 2-1, Curve A) is a most useful analytical tool. Ideally, such a curve indicates the statistical distribution of the flow-times of individual water molecules in passage through the tank. Actual

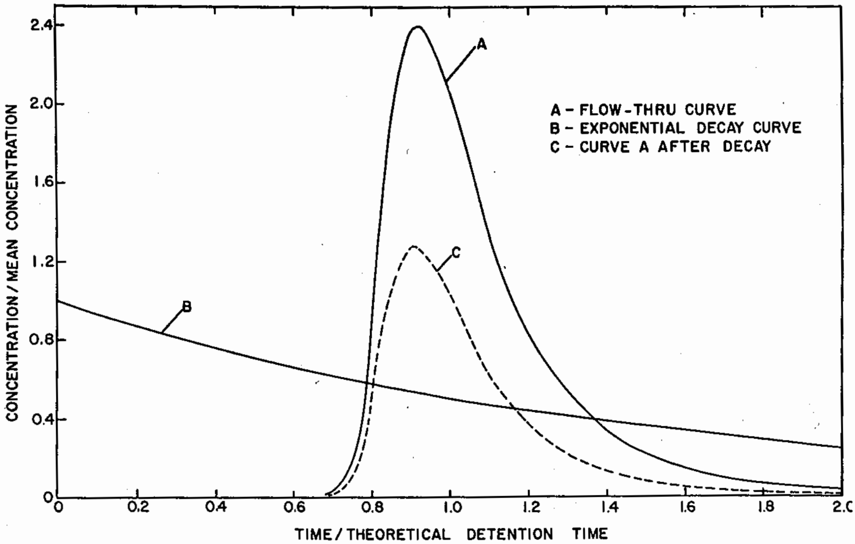

FIGURE 2-1.—TYPICAL FLOW-THRU CURVE WITH AND WITHOUT DECAY.

flow-through curves can only approximate such a distribution since the tracer molecules or ions do not necessarily follow pathlines that are identical with those of water molecules or particles of waste. Moreover, limitations are imposed by imperfections in the sensitivity and precision of the tracer detection apparatus. However, these non-ideal conditions may ordinarily be controlled so that the error is negligible. A flow-through curve obtained with a good tracer and measuring apparatus provides useful information relating to the detention characteristics of a tank. With a knowledge of the statistical distribution of flow-times of individual particles, it is possible to calculate the reduction in activity of any radioactive isotope of specified half-life in passage through the basin. To do this the decay curve for the isotope (Curve B, Figure 2-1) can be drawn starting with a concentration of one at time zero. Then each ordinate of the flow-through curve (Curve A, Figure

2-1) is multiplied by the corresponding ordinate of the decay curve giving the flow-through curve with decay (Curve C, Figure 2-1). The area under this curve represents the ratio of effluent to influent concentrations.

METHOD OF OBTAINING FLOW-THROUGH CURVES

To get a flow-through curve, a "slug" of tracer is abruptly introduced into the flow at the inlet and the concentration of tracer at the outlet measured at regular intervals. The results are plotted in dimensionless form. The observed concentration c, is divided by a mean concentration $\bar{c}$, which is defined as the amount of tracer in the slug, W, divided by the volume, V, of the tank, so that the ordinate of the flow-through curve is $\frac{cV}{W}$ or $c/\bar{c}$. Time is made dimensionless by dividing the time, t, by the theoretical mean detention time, T, which is defined as the volume of the tank divided by the discharge rate, Q. In this way the curve is made independent of detention time and size of slug and the comparison of different designs is facilitated.

FUNDAMENTAL PROPERTIES OF FLOW-THROUGH CURVES

All flow-through curves have two fundamental properties: first, the area, A, under the curve is equal to unity; and second, the center of gravity of the curve falls at $t/T = 1$. In general, both of these statements entail a small error. For example, consider a section of channel with highly turbulent flow. Some molecules of tracer will pass back and forth across the end of the section a number of times before being carried on down stream, thus, increasing the probability of their being detected. This would give an area of more than one for the flow-through curve. In tanks of the usual type which have a weir or other control producing high velocities in the downstream direction just outside the tank, it is evident that such an error is negligible.

That the area under the flow-through curve equals unity follows from the following relation:

$$A = \int_0^\infty \frac{c}{\bar{c}} \frac{1}{T} \, dt = \frac{1}{\bar{c}T} \int_0^\infty c\,dt = \frac{Q}{\bar{c}V} \int_0^\infty c\,dt = \frac{W}{\bar{c}V} = \frac{\bar{c}}{\bar{c}} = 1 \tag{2-1}$$

To show that the centroid falls at $t/T = 1$ is more difficult. By definition the centroid is:

$$T_c = \frac{1}{T} \frac{\int_0^\infty ctdt}{\int_0^\infty cdt} \tag{2-2}$$

From the definition of a perfect tracer the portion of the flow which gets through the tank in time, t, is:

$$dQ = Q \frac{dW}{W} = Q \frac{cQ}{W} dt = \frac{Q^2}{W} cdt \tag{2-3}$$

rearranging:

$$cdt = \frac{W}{Q^2} dQ$$

Substituting in equation (2-2)

$$T_c = \frac{1}{T} \frac{\int_0^\infty \frac{W}{Q^2} tdQ}{\int_0^\infty \frac{W}{Q^2} dQ} = \frac{1}{T} \frac{\int_0^\infty tdQ}{\int_0^\infty dQ} = \frac{1}{TQ} \int_0^\infty tdQ \tag{2-4}$$

$dQ = v\, dA$ and $v = \frac{L}{t}$ so that $dQ = \frac{L}{t}\, dA$ making

$$T_c = \frac{1}{TQ} \int_0^\infty t \frac{L}{t} dA = \frac{L}{TQ} \int_0^\infty dA = \frac{LA}{TQ} = \frac{V}{TQ} = \frac{T}{T} = 1 \tag{2-5}$$

Observed curves often deviate from these properties. The measured area of an observed curve is equal to the fraction of the tracer accounted for in the effluent of the tank. Any removal of tracer within the tank by adsorption, or fading in the case of dyes, or decay in the case of short-lived radioisotopes, will cause the area to be less than one. Another cause is a long tail on the curve of a low concentration below the threshold of sensitivity of the detecting instrument. This long tail also usually accounts for any discrepancy between the centroid of the curve and the theoretical detention time. Where no loss of tracer occurs within the tank, if the recovery is low and the centroid is somewhat to the left of 1, the missing tail can often be reconstructed as follows: Some simple curve, such as an hyperbola, is fitted to the existing portion of the tail. When the curve is so chosen that the area between the initial curve and infinity approximately accounts for all of the missing tracer, it is usually found that the centroid of the flow-through curve with extrapolated tail falls very close to t/T = 1.

Interpretation of Flow-Through Curves

As previously mentioned, from the flow-through curve one can compute the reduction in concentration of a radioactive isotope present in the flow, but while this is important as a measure of how well the tank is performing, the flow-through curve provides much more information. From the shape of the curve it is possible to diagnose the ills of a poorly performing tank. Let us refer to Figure 2-2. Curve A is a

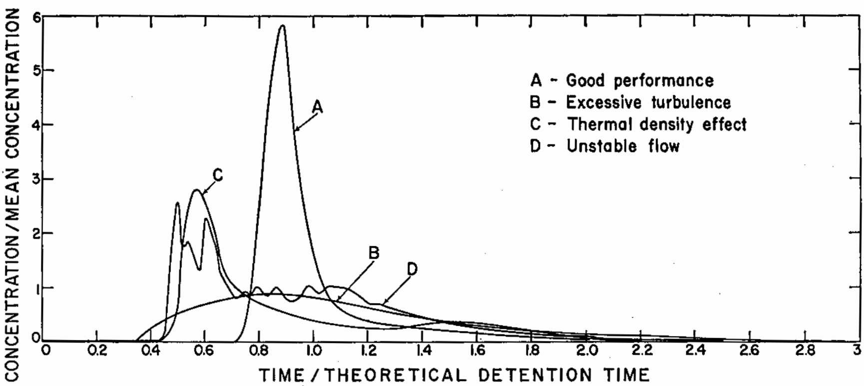

Figure 2-2.—Flow-Thru Curves for Various Modes of Tank Performance.

very good flow-through curve. The first trace efficiency (defined as the time of arrival of the first trace of tracer at the outlet divided by the theoretical detention time) is high, the peak is high and sharp and located very close to t/T = 1. Curve B presents a very different picture. The first trace efficiency is low, and the peak is very low and rounded. This indicates excessive short-circuiting of flow due to violent turbulent mixing, Curve C illustrates an even more excessive form of short-circuiting caused by extensive stagnant areas in the tank. The low first trace efficiency combined with the steep rise and relatively high peak are characteristic of this condition, as is also the thick long tail. This tail is caused by tracer being trapped in the dead areas and slowly released to the active flow over a long period of time. Such curves as C usually have rather poor tracer recovery and it appears that the centroid must be some distance to the left of 1, but if all the tracer in the long tail is accounted for, the centroid is usually found to coincide with 1 to a remarkable degree. Curve D is the result of a highly unstable flow condition where the tracer slug is not spread out in a single cloud as it travels through the tank, but is broken up into different sections which arrive more or less individually at the outlet causing the very rough curve. The performance of such a tank would be very erratic and highly undesirable.

Proof of Statement of Optimum Performance of Detention Tanks

It was stated above that for optimum performance each particle of tracer should remain in the tank for exactly one detention time, or in other words, the flow-through curve should be a sharp spike occurring at $t/T = 1$. A proof of this is now in order. Stated mathematically:

$$C_o \geqslant C_i\, e^{-KT} \qquad (2\text{-}6)$$

where

C_o is the steady state concentration in the effluent
C_i is the steady state concentration in the influent
K is the decay constant for the isotope in question
T is the theoretical detention time of the tank, volume/discharge.

From the fundamental properties, the center of gravity of the flow-through curve (Figure 2-3, Curve A) falls at $t = T$, thus from Equations 2-2 and 2-5

$$\int_0^\infty ctdt = T \int_0^\infty cdt \qquad (2\text{-}7)$$

where c is the concentration given by the flow-through curve at time, t.

Also $$W = Q \int_0^\infty cdt \quad \text{or} \int_0^\infty cdt = \frac{W}{Q} \qquad (2\text{-}8)$$

where W is the weight of tracer used in getting the flow-through curve.

Introducing c′ as the concentration with decay at time t gives

$$C_o = C_i \frac{\int_0^\infty c'dt}{\int_0^\infty cdt} = C_i \frac{Q}{W} \int_0^\infty c'dt \qquad (2\text{-}9)$$

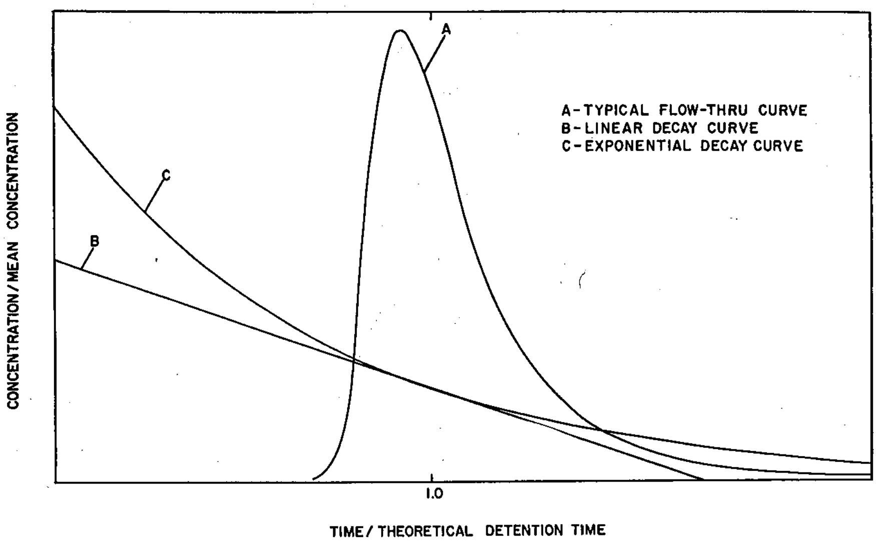

FIGURE 2-3.—TYPICAL FLOW-THRU CURVE WITH LINEAR AND EXPONENTIAL DECAY CURVES.

Now consider a linear decay function (Figure 2-3, Curve B) such that

$$c' = c\left(1 - \frac{t}{T'}\right) \qquad (2\text{-}10)$$

where T′ is the time at which $c' = 0$

Substituting in equation (2-9)

$$C_o = C_i \frac{Q}{W} \int_0^\infty c\left(1 - \frac{t}{T'}\right) dt$$

which can be written

$$C_o = C_i \frac{Q}{W} \left(\int_0^{T'} c\left(1 - \frac{t}{T'}\right) dt + \int_{T'}^{\infty} c\left(1 - \frac{t}{T'}\right) dt \right) \qquad (2\text{-}11)$$

The second part of the integral is seen to be negative and since negative values of concentration have no physical meaning

$$C_o = C_i \frac{Q}{W} \int_0^{T'} c \left(1 - \frac{t}{T'}\right) dt$$

One can also write

$$C_o \geqslant C_i \frac{Q}{W} \int_0^{\infty} c \left(1 - \frac{t}{T'}\right) dt \qquad (2\text{-}12)$$

where for the moment it is understood that the right hand side of the inequality is merely a mathematical expression.

Rewriting equation (2-12) in a different form

$$C_o \geqslant C_i \frac{Q}{W} \left(\int_0^{\infty} cdt - \frac{1}{T'} \int_0^{\infty} ctdt \right) \qquad (2\text{-}12')$$

but from equation (2-7)

$$\int_0^{\infty} ctdt = T \int_0^{\infty} cdt \qquad (2\text{-}7)$$

thus

$$C_o \geqslant C_i \frac{Q}{W} \left(\int_0^{\infty} cdt - \frac{T}{T'} \int_0^{\infty} cdt \right)$$

or

$$C_o \geqslant C_i \frac{Q}{W} \left(1 - \frac{T}{T'}\right) \int_0^{\infty} cdt \qquad (2\text{-}13)$$

and substituting from equation (2-8)

$$C_o \geqslant C_i \frac{Q}{W} \left(1 - \frac{T}{T'}\right) \frac{W}{Q} = C_i \left(1 - \frac{T}{T'}\right) \qquad (2\text{-}14)$$

Thus it is seen that for a linear decay if each particle takes just one detention time to pass through the tank, we will get optimum reduction of concentration. In this trivial case it is interesting to note that if on the flow-through curve $c = 0$ before $T = T'$ then

$$\int_0^\infty c\left(1 - \frac{t}{T'}\right) dt = \int_0^{T'} c\left(1 - \frac{t}{T'}\right) dt$$

and

$$C_o = \left(1 - \frac{T}{T'}\right) C_i \qquad (2\text{-}14')$$

In this special case, then, the shape of the flow-through curve makes no difference.

Now consider an exponential decay function (Figure 2-3, Curve C)

$$c' = ce^{-Kt} \qquad (2\text{-}15)$$

Draw a straight line (Curve B, Figure 2-3) tangent to the decay curve at $t = T$. The equation of this line is:

$$c'' = ce^{-KT}\,(1 + KT - Kt) \qquad (2\text{-}16)$$

Since the point in question has been proved for such a linear decay as equation (2-16), one needs only to show that $c' > c''$ for all $T > O$).

To prove $$e^{-Kt} \geqslant e^{-KT}\,(1 + KT - Kt) \qquad (2\text{-}17)$$

let $$t = T + t_1 \qquad (2\text{-}18)$$

then $$e^{-KT}\,e^{-Kt_1} \geqslant e^{-KT}\,(1 + KT - KT - Kt_1)$$

Cancelling and rearranging

$$1 - Kt_1 \leqslant e^{-Kt_1} \qquad (2\text{-}19)$$

if $t_1 = 0$ then the equality holds

since $1 - 0 = e^0 = 1$ and $c' = c''$

if $t_1 < 0$, it must be shown that

$$1 + Kt_1 < e^{-Kt_1} \qquad (2\text{-}19')$$

Expanding e^{-Kt_1}, we get

$$1 + Kt_1 < 1 + Kt_1 + \frac{(Kt_1)^2}{2!} + \frac{(Kt_1)^3}{3!} + \ldots\ldots$$

so that $$c' > c''$$

If $$t_1 > 0$$

then $$1 - Kt_1 < e^{-Kt_1} \tag{2-19''}$$

from this expression it can be seen that for $Kt_1 < 1$ the inequality holds since the left side will be negative and the right side always positive. By expanding it in a series:

$$1 - Kt_1 < 1 - Kt_1 + \frac{(-Kt_1)^2}{2} + \frac{(-Kt_1)^3}{3} + \ldots\ldots$$

One sees that for $Kt_1 < 1$ the terms decrease so that each positive one is greater than the next negative one. Thus, the inequality holds and we have proved that

$$C_o \geqslant C_i\, e^{-KT}$$

Relation of Decay Characteristics to Tank Removal Efficiency

In practical tanks the flow-through curve will always have more or less dispersion so that C_o will always be greater than $C_i\, e^{-KT}$, but the important question is how much. It is evident that this will depend on the shape of the flow-through curve and on the half-life of the substance. Figure 2-4 is useful in illustrating this point. Curve A is from a tank showing good performance with little dispersion, while Curve B is from a poorer tank. Both curves have areas of very close to one and their centers of gravity fall near one on the time axis. Curve C is an exponential decay curve with a half-life of one theoretical detention time, while Curve D is similar with a half-life of one tenth of a theoretical detention time. Both these decay curves have been adjusted to give a concentration of one at $t = T$, so that in a tank of optimum performance, they would each give a concentration of one in the effluent. The actual concentrations are represented by the areas under Curves AC, AD, BC, BD, and are given in the table on Figure 2-4. It is thus seen that in the removal of relatively short half-lived substances, it is important to have a tank with good flow-through characteristics. To do this, the tank must have good distribution of the flow at the inlet, uniform velocity distribution across the flow, and a good outlet. Our first major objective will be to determine how best to get these characteristics, but first a look at the experimental equipment.

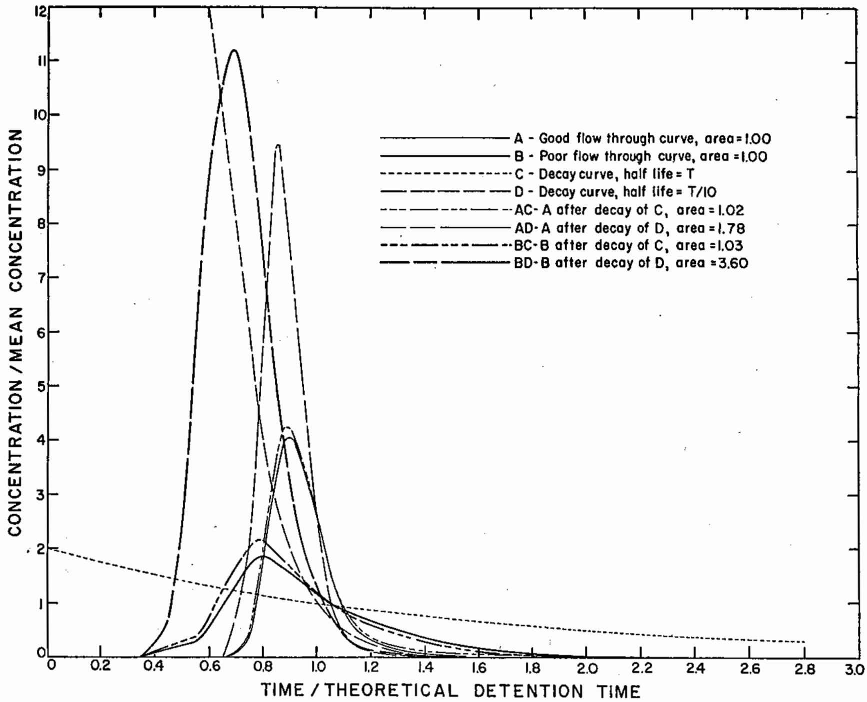

FIGURE 2-4.—COMPARISON OF PERFORMANCE OF TWO TANKS IN REMOVING SUBSTANCES WITH DIFFERENT HALF-LIVES.

EQUIPMENT

Considerable time and effort were spent on developing sensitive, precise, and reliable techniques for obtaining flow-through curves. The most important decision in this respect is the choice of a tracer. The ideal tracer should have the following properties: 1) It must not affect the flow pattern by inducing density currents; 2) it must be stable in solution and not adsorb appreciably on solid boundaries; 3) its concentration must be readily and precisely measurable with high sensitivity. It is also desirable that it be safe and easy to handle.

Radiotracers were first considered. They are excellent with respect to the first condition as the total weight of tracer need be very small. Condition (2) could be met satisfactorily by proper choice of isotope, one with a long half-life relative to the theoretical detention time, and by the addition of carrier isotopes to reduce adsorption. Condition (3) however, brought up some difficulties. Sampling of the effluent with subsequent evaporation and counting would give more than sufficient sensitivity but to get reliable and accurate results would require slow and painstaking work. For properly measuring directly in the effluent as it flowed from the tank much larger quantities of tracer would be required for each run and suitable equipment such as a special scintillation counter with rate meter was not readily available. Also, radio-tracers are expensive and safety precautions in their use are a nuisance.

While for large scale field tests radio-tracers are certainly the most satisfactory, with small scale models where flow times are short and the quantities of water are small, methylene blue dye has been found to be more convenient and sufficiently sensitive as a tracer. A photo-electric colorimeter was arranged to operate with a continuous portion of flow from the tank effluent through its sample cell. The range of the instrument with a sample cell 15 cm long was 0 to 1.0 mg/liter with a sensitivity near zero of 0.002 mg/liter/division. It is thus seen that condition (3) for a good tracer was well met. With respect to condition (1), values for $\bar{c}$ in the order of 0.20 mg/liter have been found satisfactory so that even where peak concentrations as high as $c/\bar{c} = 5$ have occurred, the dye amounts to only one part per million, a quantity which should not affect the flow pattern. As to condition (2) considerable work was done to evaluate the stability of methylene blue in Cambridge tap water. Careful tests showed that over a range of pH from 6 to 9 no appreciable change in color occurred nor was there any fading in periods of several days. Some adsorption occurred on glass after long contact, but adsorption of surfaces in the 8″ × 8″ flume was found to be negligible in the times involved. Considerable difficulty was encountered in this respect with the 1:32 scale model but although the flow through curves obtained were in error they still served to compare performance of the model under different conditions. One source of error stemmed from changes in the turbidity of the water supply but it was found that with reasonable care this could be kept down to less than one-half of a division of drift during any one run.

The Continuous Flow Colorimeter

To get accurate flow-through curves with minimum effort the continuous flow colorimeter was devised. The apparatus set up for use with the flume is shown in Figure 3-1. Here the various components are

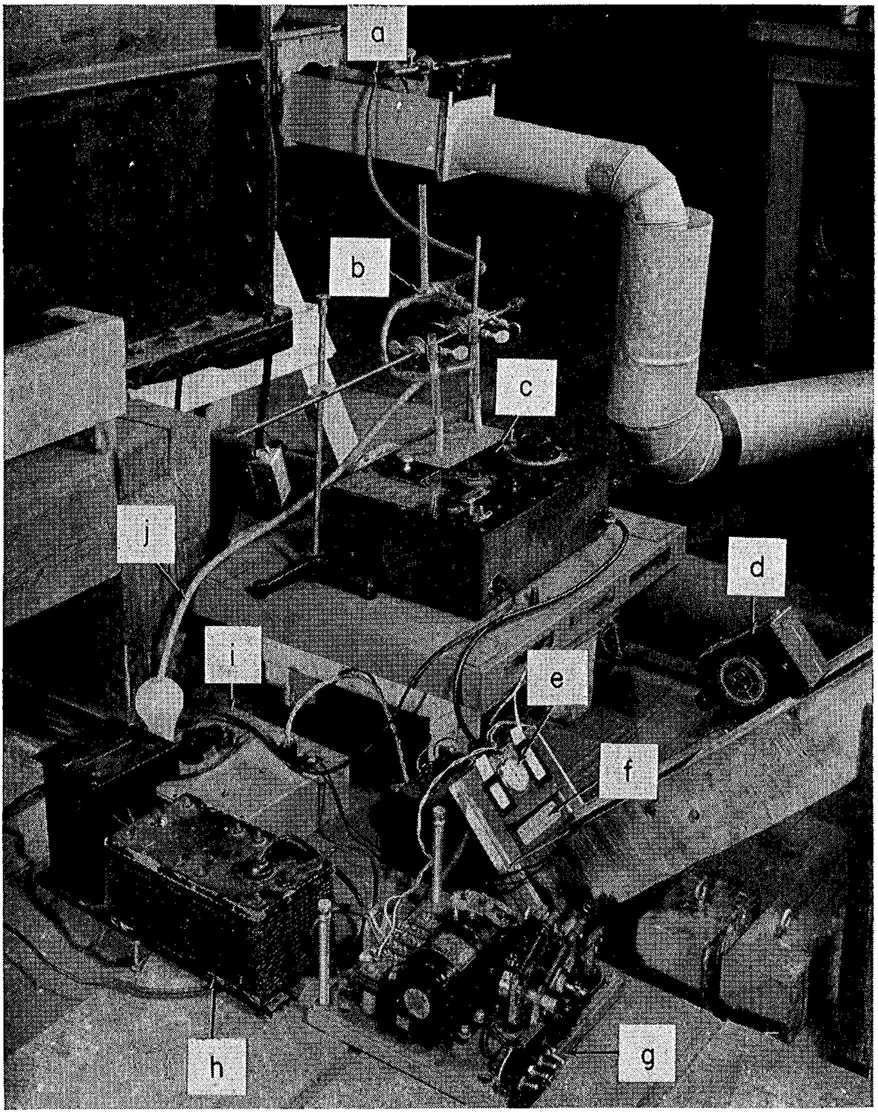

FIGURE 3-1.—CONTINUOUS FLOW COLORIMETER AS SET UP FOR USE WITH FLUME.
(SEE TEXT FOR IDENTIFICATION OF PARTS)

labeled with letters for easy reference in the text. Figure 3-2 shows the schematic diagram of the camera operating circuit.

The basic instrument was a Lumitron colorimeter model 402-E to which no alterations were made except replacement of the sample cell compartment cover with one having two holes for the inlet and outlet tubes. The sample cell was a standard Lumitron cell with cemented ends 15 cm long having a filling neck at each end to which the inlet and outlet tubes (a and j) were attached. In the sample inlet tube (a) there was a bubble trap (b) to prevent air bubbles from entering the cell. The dye concentration in the cell was indicated by the deflection of the galvanometer (f) which was recorded along with a time reading

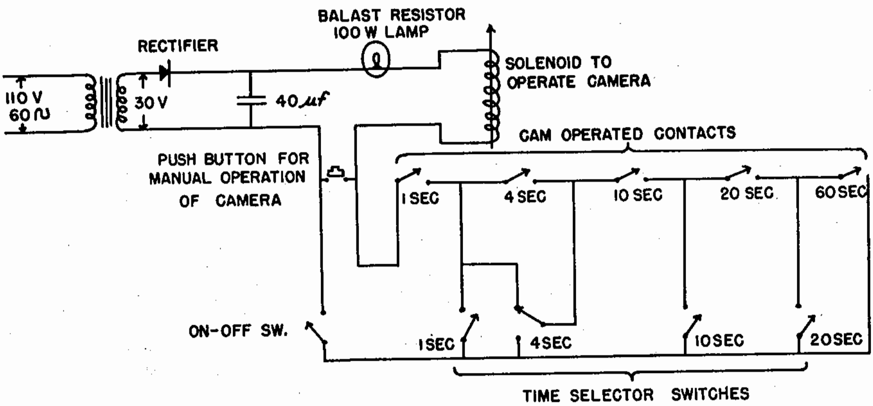

FIGURE 3-2.—SCHEMATIC DIAGRAM OF TIME SWITCH TO OPERATE CAMERA ON CONTINUOUS FLOW COLORIMETER.

from the stop watch (e) by the 16mm movie camera (d). The camera was solenoid operated to take single frames at time intervals determined by the time switch (g) which could be set to operate at time intervals of one, four, ten, twenty, and sixty seconds. The constant voltage transformer (h) was used to stabilize the operation of the colorimeter. A power supply (i) was provided to activate the camera solenoid.

The operation of the apparatus was as follows: before making a run the water was started flowing through the colorimeter cell, and the colorimeter which had been warmed up for at least one-half hour was balanced to give zero deflection on the galvanometer. Then the light beam through the sample cell was cut off and the light intensity adjusted to give full scale galvanometer deflection. After these adjustments had been checked back and forth several times and found to be accurate, the run was started. To do this the time switch was set on the sixty second interval and turned on. At the instant the first picture was taken the dye was introduced at the inlet of the tank and the stop watch started. As the run progressed the time switch was reset as necessary to provide sufficient points to give a good curve.

At the completion of a run the exposed film was removed from the camera and developed after which each frame was read and recorded and the galvanometer readings converted to concentration values by means of a calibration curve.

The Lucite Flume

The Lucite flume shown in Figure 3-3 and schematically in Figure 3-4 was a most useful research tool. The 8 inch by 8 inch transparent

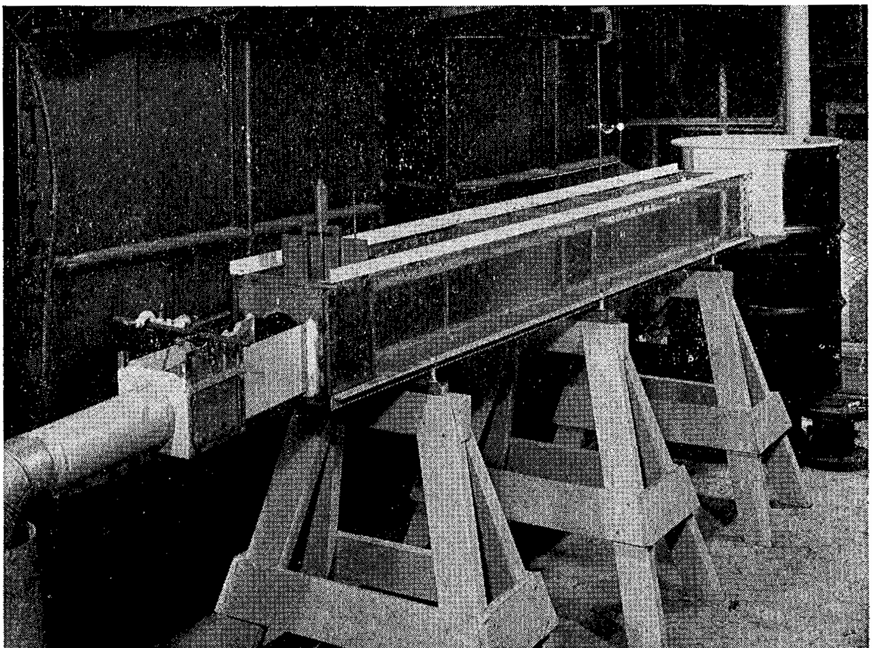

Figure 3-3.—8″ × 8″ × 8′ Lucite Flume.

section was constructed of ½ inch thick Lucite and could be used as a closed conduit by the addition of a Lucite cover. The flume was strengthened by 1 inch angle irons at the corners and was held together with brass screws which passed through the angle irons and plastic sides and were threaded into brass inserts in the bottom. The cover was mounted in a similar way by brass inserts in the upper edges of the sides. When one was operating with the flume open several ½ inch x ½ inch brass cross pieces took the place of the cover in bracing the sides.

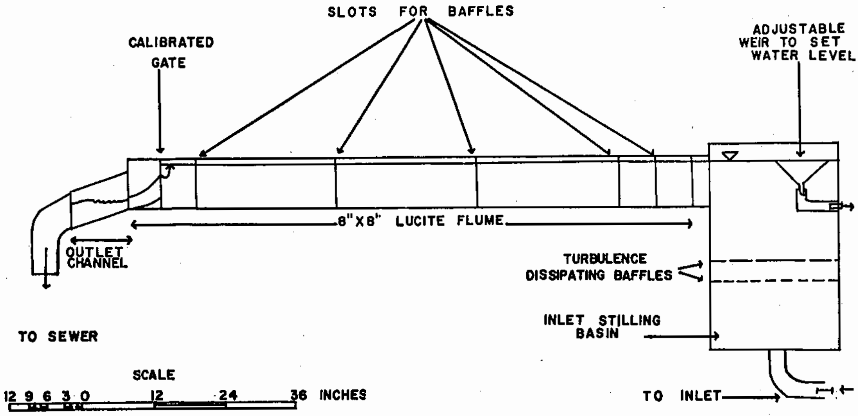

FIGURE 3-4.—SCHEMATIC DIAGRAM OF FLUME.

Two slots were provided at the inlet and two at the outlet, spaced six inches and twelve inches from the ends, to support inlet and outlet baffles and controls. Other slots dividing the remaining length into three two foot sections were provided. The entire transparent section and outlet was mounted on leveling screws on three substantial supports.

The inlet tank, which was connected to the flume with a flexible rubber joint, consisted of a 55 gallon open-end drum. The water entered at the bottom from a large storage tank through a four inch galvanized sheetmetal pipe with the joints soldered, in which there was a valve to adjust the flow. In flowing upward through the drum the water passed through two perforated baffles to dissipate turbulence. The first had small holes giving relatively high head loss and the second had large holes placed out of line with the holes below. In the top of the drum was a spillway weir consisting of a glass funnel used to hold the water level constant.

The outlet was designed to permit gravimetric measurements of discharge. As shown in Figure 3-3 the water would flow straight through the outlet channel to the sewer. For discharge measurements a tank on a platform scales was placed under the side gate and five or ten pounds weight in excess of the weight of the tank added to the scale beam. A gate was closed across the outlet (gate not shown) and the side gate opened. When sufficient water had flowed into the tank to balance the scales a stop watch was started and a convenient weight added to the beam. When the scale balanced again the watch was stopped and from the weight and time the discharge was computed.

For most work an easier means of getting discharge measurements was provided by a control gate which was placed in the last slot of the flume. It was adjusted by a micrometer screw and had a fixed hook gauge attached to the frame. A calibration curve obtained by gravimetric measurements gave discharges from micrometer readings, with the water level behind the gate at the point of the hook, to plus or minus 2 percent. By proper adjustment of the in-flow valve and the height of the weir spillway at the inlet, the level at the gate could be maintained within several thousandths of an inch of the hook point.

The flume with associated equipment proved to be indispensable in research on density effects, since it was possible to see not only the horizontal, but also the vertical distribution of flow. Figure 3-5 which shows the progress of a dye-colored density current along the bottom of the flume illustrates this point.

Other Experimental Tanks

A rectangular wooden tank 12′ × 20′ × 16″ deep was used for a number of tests. It is shown in Figure 3-6. With pumping equipment to supply about 0.5 cubic feet per second, a wide range of theoretical detention times could be obtained ranging upward from 10 minutes.

Two models were also built of existing detention tanks. Figure 3-7 shows the model of a circular tank scaled 1:100 built of galvanized sheet iron and Figure 3-8 shows a wooden model of a rectangular tank scaled 1:32.

The general arrangement of the equipment in the laboratory is shown in Figure 3-9. Water was supplied to the storage tank from the building supply. It had a capacity of some six hundred cubic feet and was used primarily for obtaining hot water. After the tank was filled steam could be run into the water through a perforated pipe to heat it

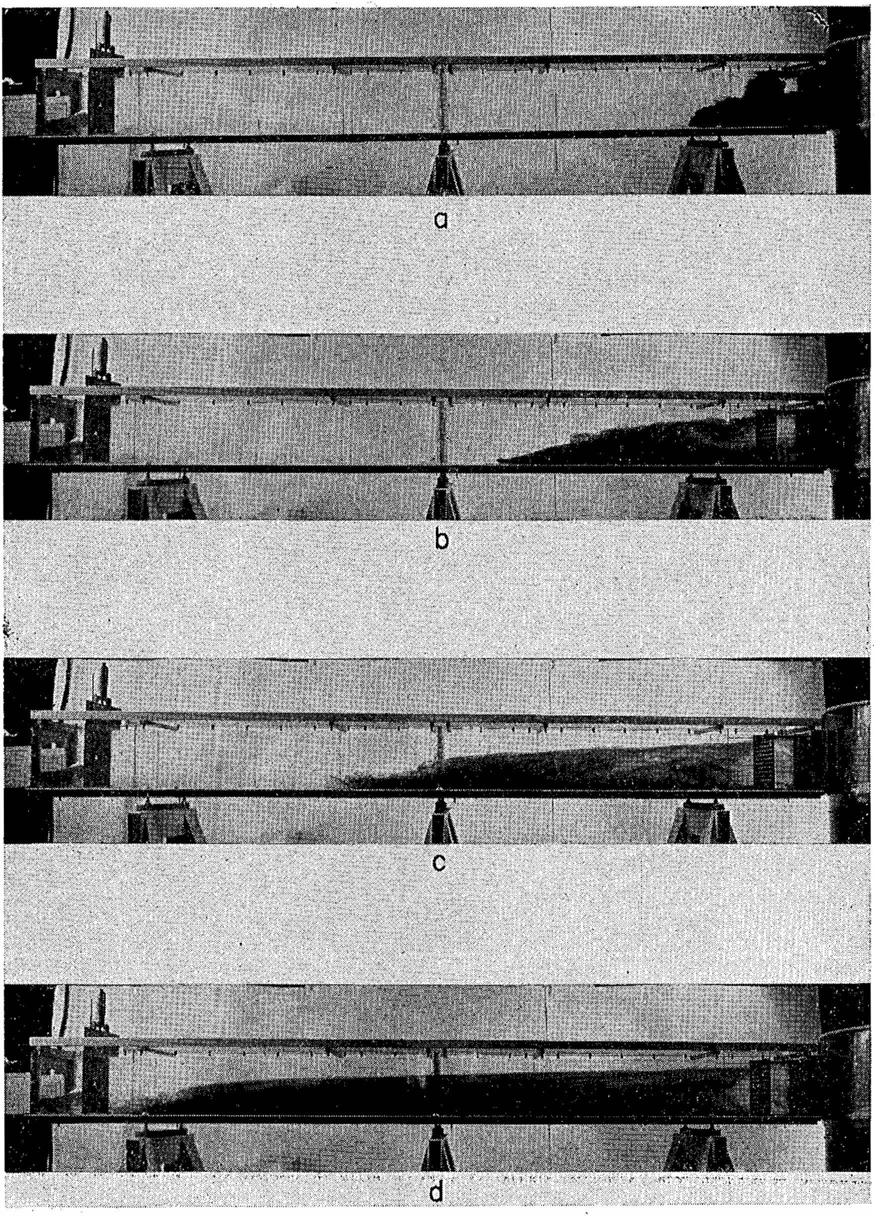

FIGURE 3-5.—THE PROGRESS OF A THERMAL DENSITY CURRENT ALONG THE BOTTOM OF THE FLUME.

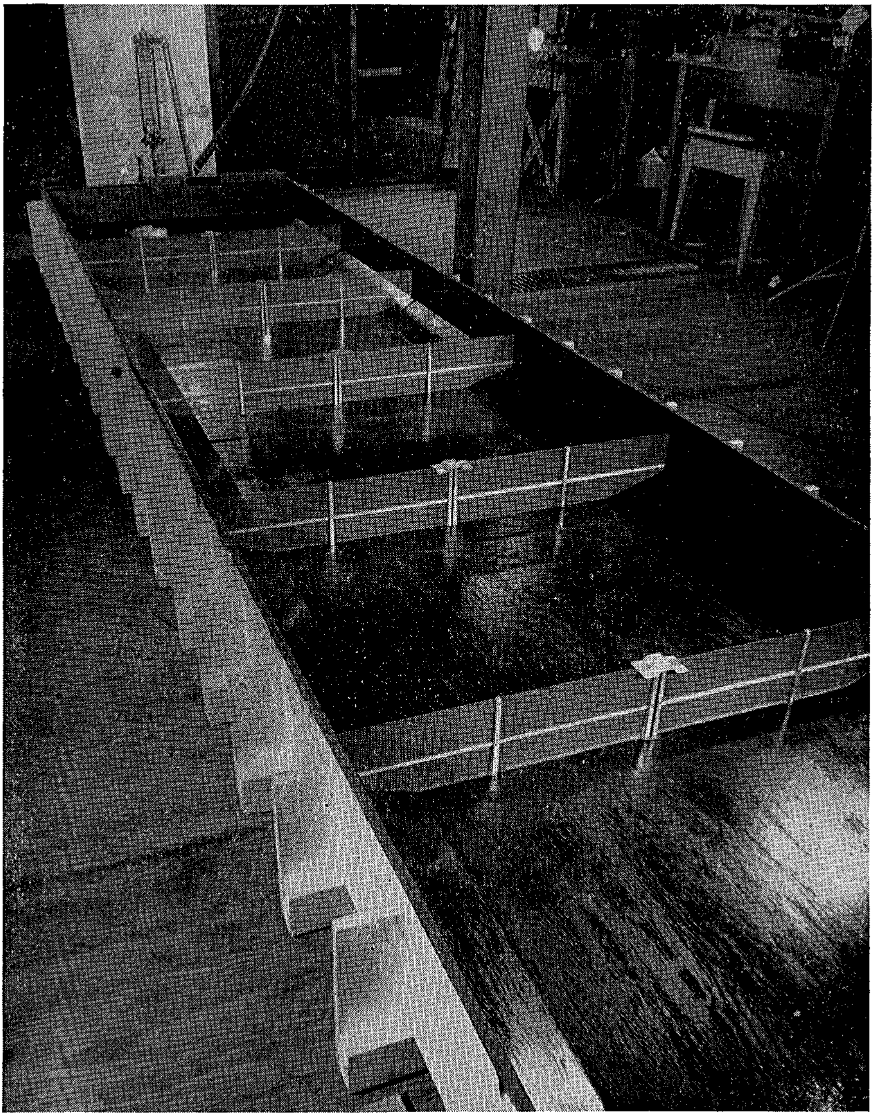

FIGURE 3-8.—SCALE MODEL 1:32 RECTANGULAR COOLING WATER DETENTION TANK.

to any desired temperature. Sufficient water could be stored and heated at one time to complete at least one test on any of the experimental models except the large rectangular wooden tank. When running the

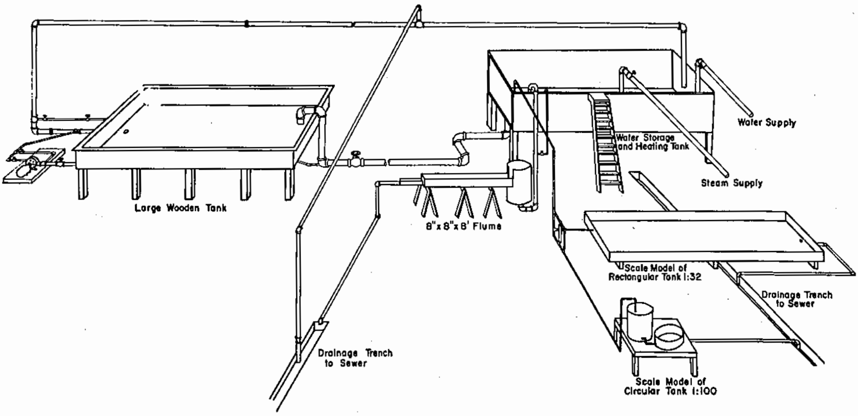

FIGURE 3-9.—ARRANGEMENT OF EQUIPMENT FOR DETENTION TANK STUDIES.

latter, the storage tank was used as a constant head tank with a steady flow from the building supply.

THE DIFFERENTIAL THERMOMETER

In studying thermal density problems a sensitive differential thermometer was used. This consisted of two copper-constantan thermocouples connected through a resistance box to a multiple reflection Rubicon galvanometer. The galvanometer had a sensitivity of 0.0015 microamperes per millimeter and the entire circuit resistance was about 500 ohms with the resistance box at zero giving a maximum voltage sensitivity of 0.75 microvolts per millimeter. With copper-constantan couples this gave a temperature sensitivity of approximately 0.03 degrees F. per millimeter at temperatures around 100 degrees F. By introducing additional resistance with the decade box the sensitivity could be reduced to give any desired range.

The time and effort spent in building this equipment was considerable, and even more was required to get it all operating properly, but it is felt that the results as shown in the following sections more than justify the expenditures.

Inlet Zone Design

The inlet to a detention tank serves two purposes. First, it distributes the flow horizontally and vertically so that velocities will be uniform across the cross section of the tank. Secondly, it dissipates any excessive energy in the incoming flow. The problem of design is thus similar to that encountered in sedimentation tanks; however, in detention tank inlets the transportation of suspended solids and the destruction of fragile flocs does not have to be considered, but the vertical distribution of the flow is more important.

Another problem in inlet design is the equal distribution of flow between tanks operating in parallel or the distribution between multiple inlets to one tank. This has been well treated by others[4] so that the present discussion will be restricted to proper handling of the flow after it leaves the influent conduits and enters the inlet zone within the tank.

To study inlets, the 8″ × 8″ Lucite flume was used. The transparent walls made it possible to observe the vertical, as well as the horizontal distribution of the flow, and the small size of the flume made it possible to use small easily built inlets for the different tests. The flume also provided an ideal flow condition with which to compare the various test inlets. Without baffles at the inlet, water entered from the relatively large inlet tank with practically no turbulence and with almost perfect distribution.

Flow through curves for this ideal inlet are shown in Figure 4-1 along with those for a one inch horizontal slot and for the slot followed by a perforated baffle with 36% open area. The slot illustrates how a poor inlet can spoil the performance of a tank; the slot with baffle shows how relatively simple measures can often provide important improvements in inlet performance.

Inlet Designs

A number of inlets are shown schematically in Figure 4-2. (A) is the simplest and poorest inlet. The jet from the inlet pipe can be expected to travel down the tank for about five times the width or depth of the tank, whichever is greater, before the velocity distribution becomes more or less uniform over the cross section. If the velocity of the jet is low enough so that the flow is laminar, an unlikely, but possible situa-

[4] G. M. Fair and J. C. Geyer, Elements of Water Supply and Waste—Water Disposal, Wiley, New York, pp. 325-326 (1958).

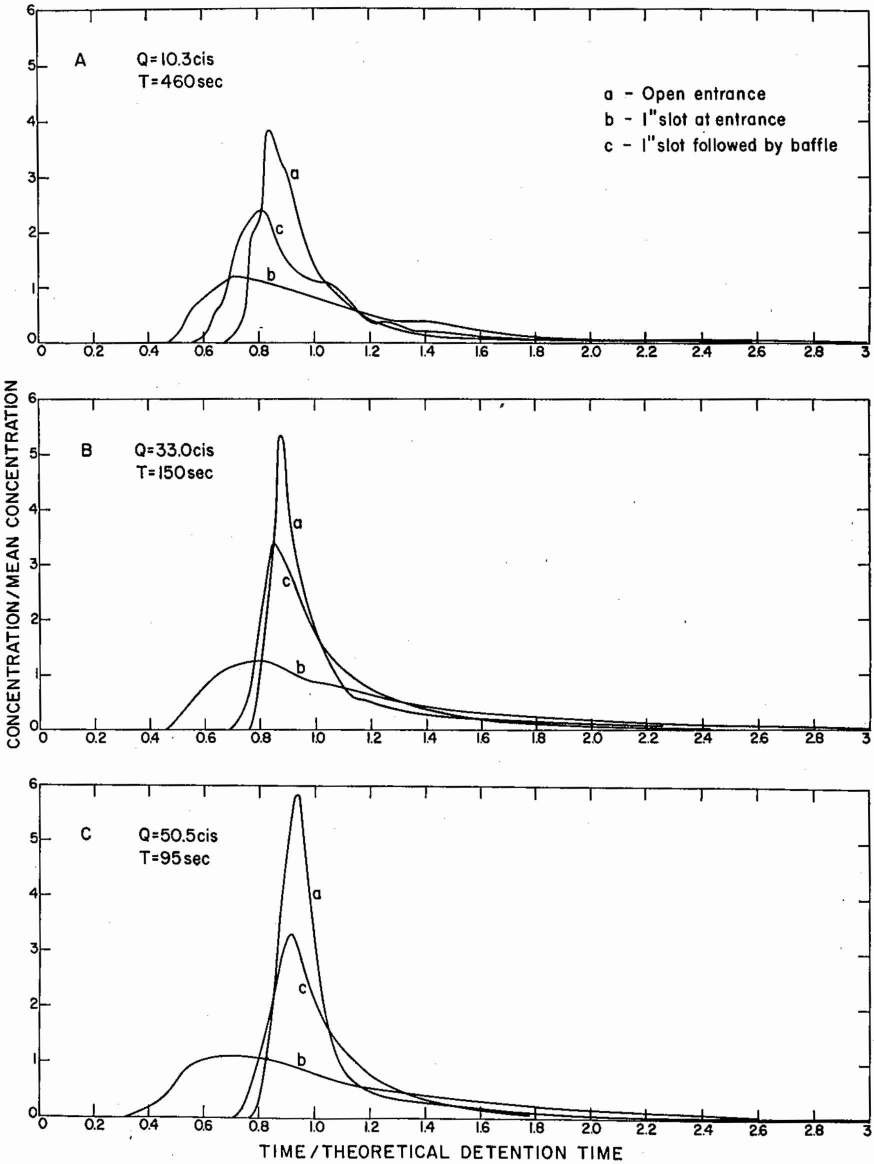

FIGURE 4-1.—EFFECT OF INLET DESIGN ON TANK PERFORMANCE.

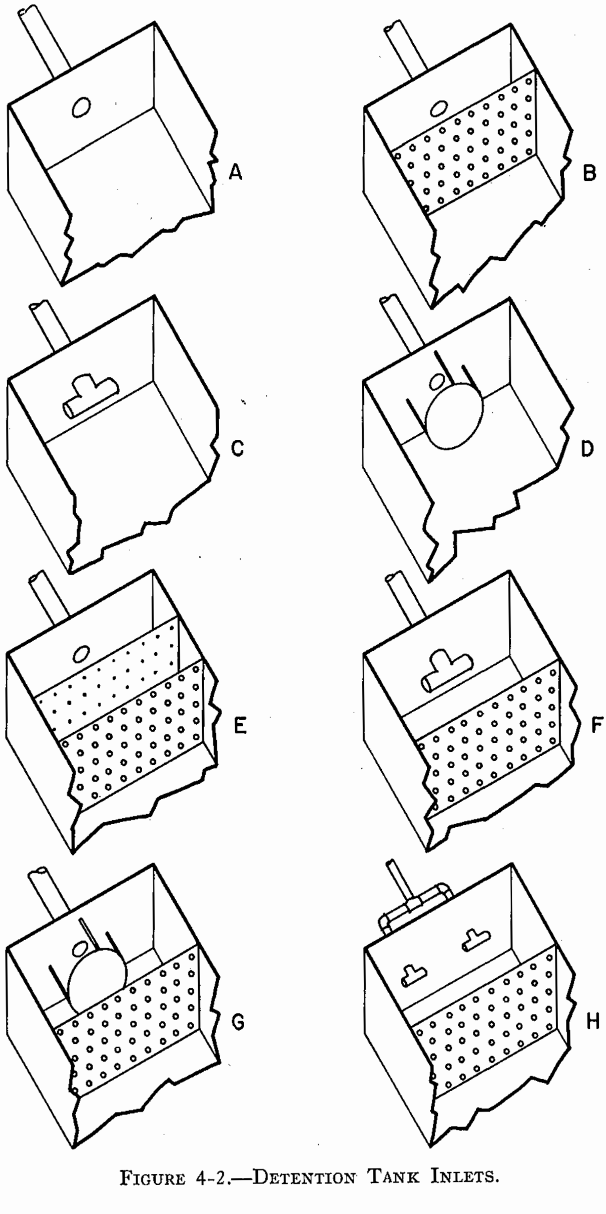

FIGURE 4-2.—DETENTION TANK INLETS.

tion, the jet will persist for a much longer distance causing even worse short circuiting. It is evident that a baffle (B) or a tee (C) or a disc (D) will break up the jet and improve the distribution of the flow. Any of these three followed by a perforated baffle, (E), (F), and (G), should give even better performance. It is logical to consider more than one inlet (H) where the tank is wider than it is deep.

The inlet using two perforated baffles is attractive as it can be designed from theoretical considerations. The first baffle should have a head loss sufficiently high to insure equal flow through each hole. Under the worst conditions, this was found to be approximately equal to the velocity head in the influent pipe, but since the velocity of a submerged jet is rapidly reduced as it moves away from its origin, the maximum velocity head which could be caused by the incoming jet on a few holes is seldom realized and a much smaller head loss will ordinarily suffice. The spacing between the two baffles should be sufficient to allow the jets from the holes in the first to expand and cover the entire cross section of the tank. Rouse[5] indicates that the expansion ratio of a jet is about one to five. From this, it is seen that the spacing between baffles should be about five times the distance between holes in the first. Experimental work confirmed this. The second baffle works in the same manner as the first but instead of having above it a velocity distribution ranging from several feet per second to zero as the first has, the second baffle should have a velocity distribution of less than two to one in the flow approaching it. Because of this, a very small head loss is required to get almost perfect distribution of the flow downstream.

Tests on Inlets

The flow pattern in inlets using tees or small baffle plates to dissipate the influent jet is not amenable to theoretical treatment: however, the performance of these devices can be studied readily in the laboratory. For tests on these, as well as the two-bafflle inlets, the flume was arranged as shown in Figure 4-3, which also shows the different influent pipe endings and baffles which were tested. Baffle Number 3 was not used as it had too high a head loss at the desired flow rate. The desired inlet was placed at section A-A and baffles, if used, were placed at sections B-B and/or C-C. A small quantity of fluorescein dye was injected with a medicine dropper just upstream from sec-

[5] H. Rouse, "Fundamental Principles of Flow" Engineering Hydraulics, Wiley, New York, pp. 95-99 (1950).

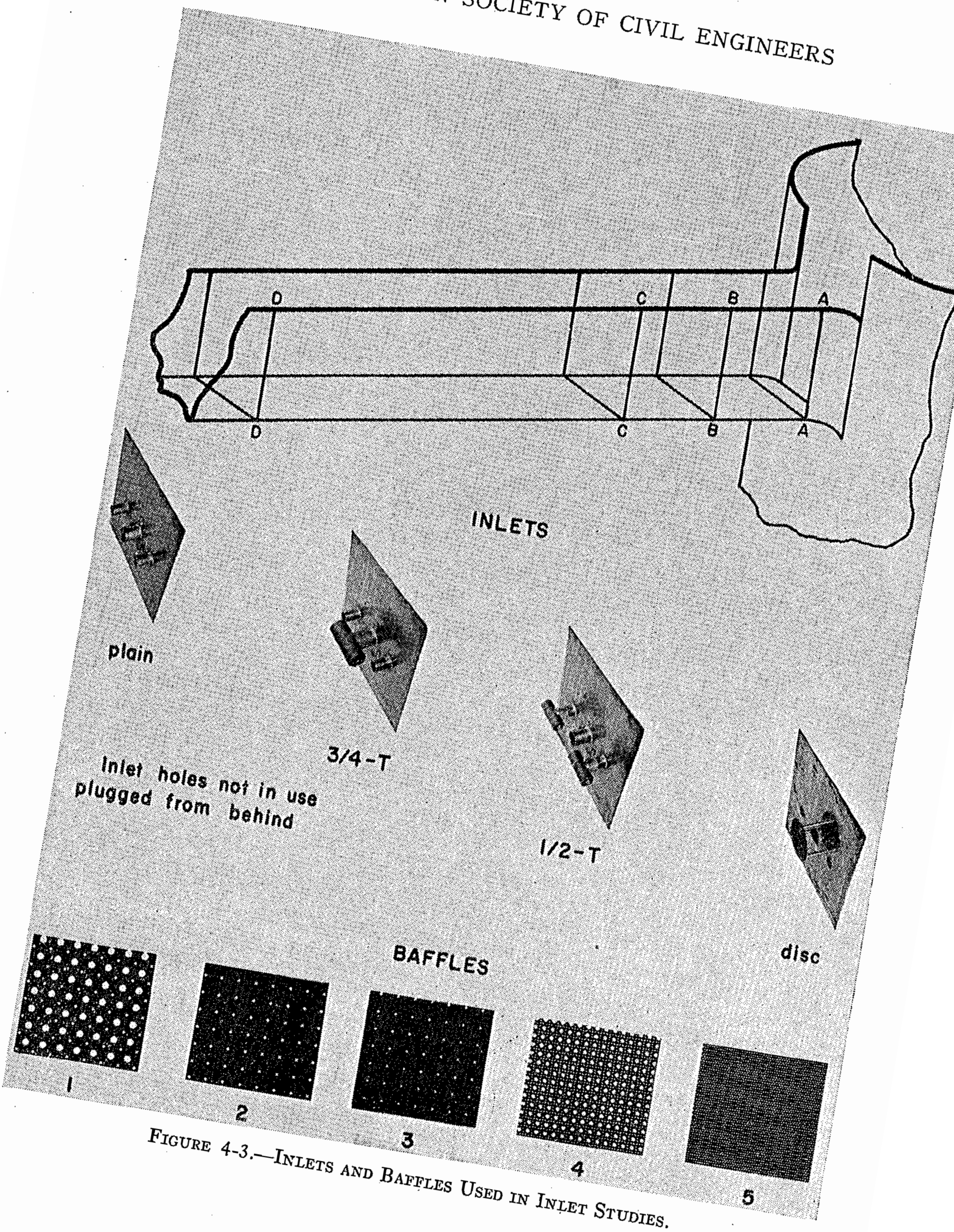

FIGURE 4-3.—INLETS AND BAFFLES USED IN INLET STUDIES.

tion A-A and the first trace time was measured from the time the first trace of dye passed section C-C until it passed section D-D. This transit time divided by the theoretical detention time for the volume between sections C-C and D-D gave the first trace efficiency. It was felt that this method of measuring first trace efficiency was better for the purpose at hand than the more usual one involving the entire volume and measuring first trace time from the introduction of the tracer, because it compared inlet designs on the basis of their ability to permit the volume outside the inlet zone to perform efficiently without masking the results with the necessarily low first trace efficiency of the inlet zone.

Table 4-1 shows the different arrangements tested along with the first trace efficiency for each. It also shows the head loss of the inlet and the loss across the different baffles. These head losses were measured with point gauges mounted on micrometer heads which could be read consistently to 0.0005 inches. Besides the obvious comparison of first trace efficiencies of the different arrangements, there are some other points which can be discovered from Table 4-1. Consider tests Number 4 and 5. Baffle 2 had a row of holes right at the bottom but the highest row through which water flowed was about a half inch below the surface. This caused an excessive velocity along the bottom which was easily observed. In test Number 5, the bottom row of holes was plugged as indicated and with the elimination of the bottom current, the significant improvement in performance was noted. This illustrates the importance of designing baffles to be symmetrical with respect to the flow. This is particularly noticeable where a baffle with relatively few holes is used.

Another point to consider is the effect of a given volume before or after the final baffle. With two exceptions, when similar tests were run with a baffle at B-B or at C-C, the efficiency was higher with the baffle at C-C. The added volume before the baffle allowed more room for excessive velocities to dissipate, thereby, reducing the difference in the effective head on each hole in the baffle. In one case where the baffle at B-B gave a higher efficiency, the baffle involved had big holes with only 20% open area, giving a coarse grained turbulence with a relatively long persistence. Here the stilling volume after the baffle was more important than a better velocity distribution upstream. In the other case, the two half inch T inlets gave such a good distribution of flow that the volume between B-B and C-C was more valuable for

TABLE 4–1

FIRST TRACE EFFICIENCY FOR VARIOUS INLET AND BAFFLE COMBINATIONS
FOR IDENTIFICATION OF INLETS AND BAFFLES SEE FIGURE 4–3. ALL TESTS RUN AT A FLOW OF 9.75 CUBIC INCHES PER SECOND

Inlet*	¾–0	¾–T	½–0	½–T	¾ D 1	¾ D ½	¾ D ¼
head loss	0.78″	0.86″	0.81″	0.85″	0.83″	0.85″	0.95″
Baffle	1	2	4	5	2′**		
head loss	0.005″	0.058″	0.002″	0.003″	0.085″		

Number	Inlet	First Baffle	Second Baffle	First Trace Eff.
1	¾–0	0	0	7.8%
2	¾–0	2	0	30.2%
3	¾–0	2	4	48.9%
4	¾–0	2	5	53.2%
5	¾–0	2′	5	58.9%
6	¾–0	4	4	41.1%
7	¾–T	0	0	49.5%
8	¾–T	4	0	65.1%
9	¾–T	0	4	69.5%
10	¾–T	5	0	56.6%
11	¾–T	0	5	64.4%
12	¾–T	1	0	48.2%
13	¾–T	0	1	54.7%
14	½–0	0	0	11.5%
15	½–0	2	4	58.1%
16	½–T	0	0	38.3%
17	½–T	0	4	63.9%
18	½–T	4	0	66.7%
19	½–T	5	0	69.2%
20	½–T	1	0	72.1%
21	¾ D 1	0	0	30.4%
22	¾ D 1	4	0	51.0%
23	¾ D 1	0	4	61.3%
24	¾ D ½	0	0	32.5%
25	¾ D ½	4	0	47.8%
26	¾ D ½	0	4	63.4%
27	¾ D ¼	0	0	36.8%
28	¾ D ¼	4	0	44.7%
29	¾ D ¼	0	4	54.4%

* ¾–0 is plain inlet using center hole.
½–0 is plain inlet using two outer holes.
¾–D 1 (½, ¼) is inlet with disc spaced 1 (½, ¼) pipe diameters from pipe end.

** 2′ is number 2 baffle with bottom row of holes plugged.

dissipating the relatively small turbulence generated by the baffle than for dissipating the inlet energy before the baffle.

The inlet with baffle plate mounted in front of the pipe gave poor results. Of the three spacings tried, the one where the plate was located downstream from the pipe, a distance equal to one-half the pipe diameter, gave the best performance. It is possible that even better performance could have been obtained with some other plate size; however the flow in the inlet zone was unstable and the efficiency was quite sensitive to flow rate. The arrangement as a whole was less satisfactory then the T inlet.

Conclusions from Inlet Tests

The following conclusions can be drawn from the research on inlets and observations on inlets of prototype and model tanks:

1. Perfect symmetry about the longitudinal axis of the tank in a horizontal plane is almost a necessity.
2. A zone should be provided for energy dissipation and distribution. Within this zone the flow should be highly turbulent, but with transverse components of velocity kept to a minimum.
3. The flow pattern in this inlet zone should be stable under all expected conditions.

The two-baffle inlet, the tee and baffle inlet, and the baffle-disc and baffle inlet can all be made to give satisfactory performance, but of the three, the tee and baffle seems to best fill the requirements for a good inlet. The number of tees to give best performance cannot be specified at present. It seems probable that one tee for each unit of width equal to the depth should prove to be good design. The baffle should have thirty to forty percent open area with relatively small holes of one thirty-second to one sixty-fourth of the smaller dimension of the baffle. Placing it downstream from the inlet, a distance equal to the depth of the tank, should provide satisfactory results. Finally, it is recommended that a model of the proposed inlet zone be built from which the number of inlets, the design of the baffle, and the spacing of the baffle from the inlet can be easily and quickly determined.

Detention Zone Design

The detention zone in a tank should be designed in such a way that tendencies toward hydraulic short circuiting are damped out rather than amplified. The tank should be such that the uniform velocities produced across the cross section by the inlet are maintained throughout the tank. Toward this end the velocities should be kept high. It is well known that laminar flow tends toward a parabolic distribution of velocity ranging from zero at the boundaries, to a maximum at the center, whereas turbulent flow tends to produce a much more uniform velocity distribution. Also, if the flow is turbulent there is less chance for density currents or layers to form. On the other hand it is possible that excessive turbulence would contribute to short circuiting by speeding a part of the flow through the tank at a velocity considerably above the mean. This is indeed true in the case of large scale turbulence generated by initial imperfect transverse distribution of the flow, or in bends or corners, or transverse perforated baffles. However, the turbulence generated by flow through a straight unobstructed channel is relatively fine grained and the resulting degree of short circuiting is more than compensated by the advantages gained with improved velocity distribution and the elimination of density layers.

Upper Limit of Velocity

The question immediately arises: Is there an upper limit to the velocity above which logitudinal dispersion becomes a serious problem? Work by Taylor[6] is illuminating on this. He has treated the case of longitudinal dispersion in pipes with turbulent flow. Using the standard deviation σ of the flow-through curve as a measure of dispersion:

$$\sigma = \frac{\sqrt{2KT}}{v} \tag{5-1}$$

where T is the theoretical detention time,
v is the mean displacement velocity,
K is the effective longitudinal diffusion coefficient,

Taylor showed that for turbulent flow:

$$K = 1.785\ d\ v\ \sqrt{f} \tag{5-2}$$

where d is the diameter of the pipe, and f is the Darcy-Weisbach friction coefficient.

Now consider two pipe reaches having the same volume. One is

[6] Sir Geoffrey Taylor, "The Dispersion of Matter in Turbulent Flow Through a Pipe," Proceedings of the Royal Society, Section A, CCXXII, December 1953.

long and small in diameter while the other is short in length and large in diameter. At the same flow rate, Q, they will both have the same theoretical detention time, T. Using subscript 1 to denote the small diameter pipe and subscript 2 to denote the large we can write:

$$\frac{K_1}{K_2} = \frac{d_1 v_1}{d_2 v_2} \tag{5-3}$$

assuming f to be constant. Also

$$Q = \frac{\pi}{4} d_1^2 v_1 = \frac{\pi}{4} d_2^2 v_2 \tag{5-4}$$

so that

$$\frac{v_1}{v_2} = \frac{d_2^2}{d_1^2} \tag{5-5}$$

making

$$\frac{K_1}{K_2} = \frac{d_2}{d_1} \tag{5-6}$$

The ratio of the two dispersions is

$$\frac{\sigma_1}{\sigma_2} = \frac{\sqrt{2K_1Tv_2}}{v_1\sqrt{2K_2T}} = \sqrt{\frac{K_1}{K_2}\,\frac{v_2}{v_1}} = \sqrt{\frac{d_2}{d_1}\,\frac{d_1^2}{d_2^2}} = \left(\frac{d_1}{d_2}\right)^{3/2} \tag{5-7}$$

It is thus seen that the small long pipe will give less dispersion of the flow-through curve than the short large one. If we consider that f is not a constant but in fact is a function of the Reynolds number, we can show that this also works in the direction of decreasing the dispersion in the smaller pipe. For moderate values of Reynolds number and smooth surfaces the Blasius formulation is

$$f = \frac{0.316}{R^{1/4}} \tag{5-8}$$

and

$$R = \frac{vd}{\nu} \tag{5-9}$$

It is evident that where Q is a constant the small pipe will have a higher Reynolds number since the velocity is inversely proportional to the square of the diameter. Thus the small pipe will have a smaller value of f giving a smaller value of K and a smaller value of σ. It is logical to assume that these results will hold true at least qualitatively for open channels used as detention tanks particularly where these are long and narrow. There appears to be, then, no upper limit on velocity as far as short-circuiting performance is concerned.

Lower Limit of Velocity

From a practical point of view where a tank must be fitted into a given space the question might well arise: How low a velocity can be used without having trouble with density layers and generally unstable conditions? This is more difficult to answer.

Theoretical analysis of the behavior of thermal density layers in a tank under steady state flow is beyond the scope of the present investigation. It involves heat transfer due to conduction through the walls, through the air-water interface, and by evaporation. Stability of the interface between layers of very slightly different density is important. In the case of open tanks wind action can be important. From a practical view all that is required is some parameter which can be used to determine whether a tank will be free of thermal density problems. Investigation showed that the Reynolds number is not suitable. A very large detention tank, the full scale prototype of the model shown in Figure 3-8, when operating at a Reynolds number of about two million, showed very serious short circuiting due to thermal density effects, while the $8'' \times 8''$ flume operating at a Reynolds number of about four thousand showed excellent performance. Flow-through curves for these two tests are shown in Figure 5-1A.

There are two conditions that can cause trouble. The first is a steady state stratification of flow due to heat transfer taking place within the tank. In the case of hot water entering, a large practically dead pool of water lying in the bottom of the tank can form. The pool is constantly being drained by entrainment at the interface with the flowing water above, by mixing at the inlet, which causes reverse flow along the bottom of the tank, and sometimes by withdrawal at the outlet. The pool is replenished by cool water formed at the surface by evaporation which flows by convection slowly down the walls. The second condition is transient, caused by a change in the temperature of the incoming water. This will cause a thermal density current to flow down as shown in Figure 3-5 causing a pool to form at the top or bot-

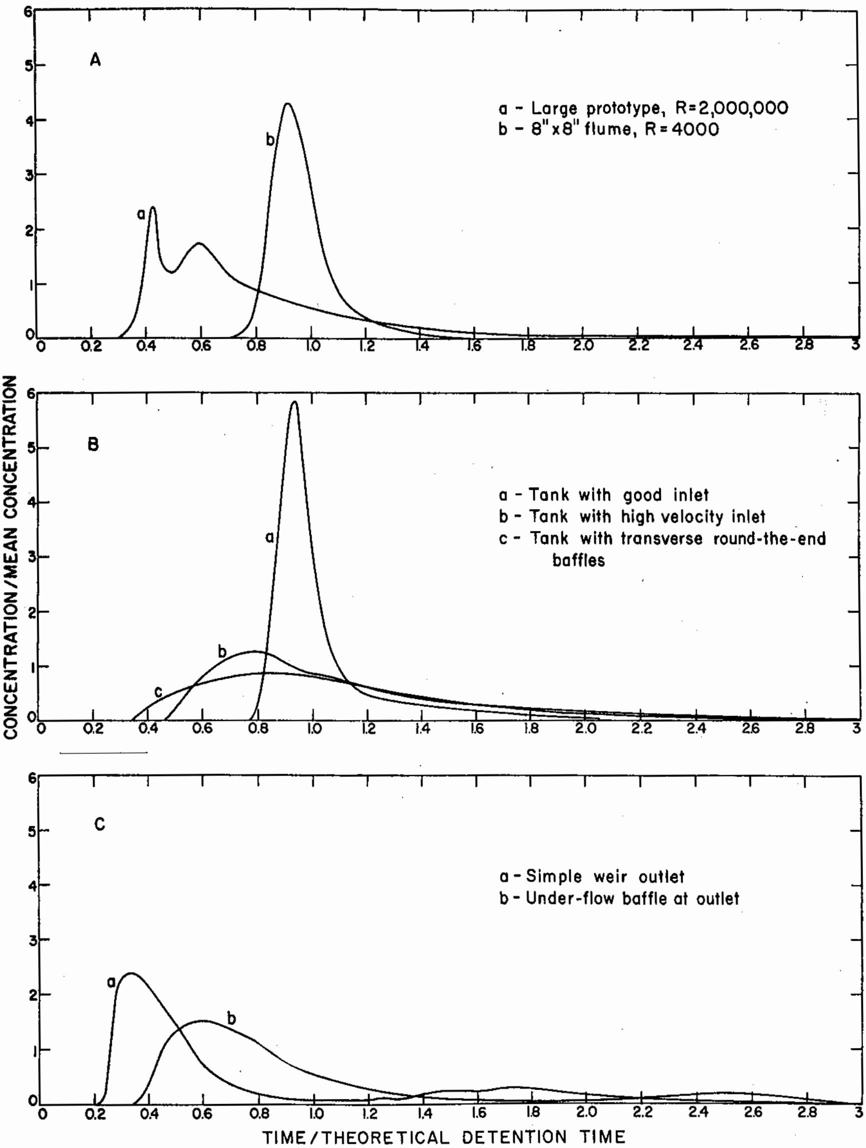

FIGURE 5-1.—FLOW-THRU CURVES ILLUSTRATING POINTS DISCUSSED IN TEXT.

tom depending on whether the incoming water is cooler or warmer than previously. Since this condition is truly transient, the pool will not be replenished so that the forces tending to drain it will in time remove it completely.

A CRITERION FOR DETENTION ZONE DESIGN; THE MEAN VELOCITY GRADIENT

For the steady state condition it is evident that there is a minimum amount of mixing that must be provided to prevent stratification in the tank. Camp[7] has shown how the performance of flocculators in water and sewage treatment plants can be evaluated in terms of the temporal mean velocity gradient within the flow. Since flocculators are primarily mixing devices this appeared to the author to be a promising parameter for detention tanks. From Camp's formulation, in open channels

$$G = \sqrt{\frac{f}{\nu}\frac{v^3}{8r}} \tag{5-10}$$

where G is the mean velocity gradient
f is the Darcy-Weisbach friction factor
v is the mean velocity ft per sec
ν is the kinematic viscosity ft^2 per sec
r is the hydraulic radius of the cross section ft

Again using the Blasius formulation

$$f = \frac{0.316}{R^{1/4}} \tag{5-8}$$

where the Reynolds number is $R = \dfrac{4vr}{\nu}$ (5-9′)

Substituting for R gives

$$f = \frac{0.224\,\nu^{1/4}}{v^{1/4}\,r^{1/4}} \tag{5-11}$$

and putting this into Equation (5-10) gives

$$G = 0.167\,\frac{v^{1.375}}{\nu^{0.375}\,r^{0.625}} \tag{5-12}$$

Equation 5-12 was derived for smooth circular tubes. However, no appreciable error is involved in extending its use to open channels.

[7] T. R. Camp and P. C. Stein, "Velocity Gradients and Internal Work in Fluid Motion" Journal Boston Society of Civil Engineers, 30, pp. 219-237, October 1943.

Also it is strictly applicable only to flow where the Reynolds number is in the range of from a few thousand to one hundred thousand. This is the range in which most laboratory detention tank models will operate. In prototype tanks the Reynolds number may be above this range. Under these conditions the Blassius formulation for f gives values that are somewhat too low but the error is not large. Consideration of equation 5-10 will show that low values of f give low values of G so that the error introduced by using equation 5-12 for full size tanks is on the safe side. Values of G were computed for many tests run on all the tanks at many different temperatures and flow rates. These were plotted against first trace efficiency as shown in Figure 5-2. Tests in which short circuiting due to poor inlet conditions or other causes of excessive turbulence was marked, were not included since the present study was concerned only with short circuiting in the detention zone due to thermal density effects. From the plot it is evident that for flow with a value of G = 0.3 per second or more the first trace efficiency was consistently high.

Method of Obtaining Required Mean Velocity Gradient

Maintaining a value of G above 0.3 per second will assure freedom from density problems, but it is important to note that the velocity gradient must be produced by friction loss in open channel flow if it is not to produce undesirable side effects. In flocculators the high values of G which are used (between 10 and 75 ft/sec/foot) are often produced by stirring mechanisms, by short round the end or over and under baffles, or by introducing the flow at high velocity. In detention tanks such means should never be employed to get the required value of G because of the serious turbulent short-circuiting which they produce (see Figure 5-1B). Relatively fine mesh perforated baffles which produce a fine grain turbulence do assist in reducing thermal density stratification, but their influence is effective for only a short distance down stream because of the rapid dissipation of their turbulence. This can be seen in the first view of Figure 3-5. Note how the cool water colored by dye used as a tracer flowed through the entire baffle in a uniform manner but almost immediately after getting through, it dropped to the bottom to form the density current. Tests run on the 1:32 scale model illustrate further the effectiveness of such baffles. The model was run at the same flow rate and same temperature with the baffles in place as shown in Figure 3-8, with no baffles except the one at the inlet, and with

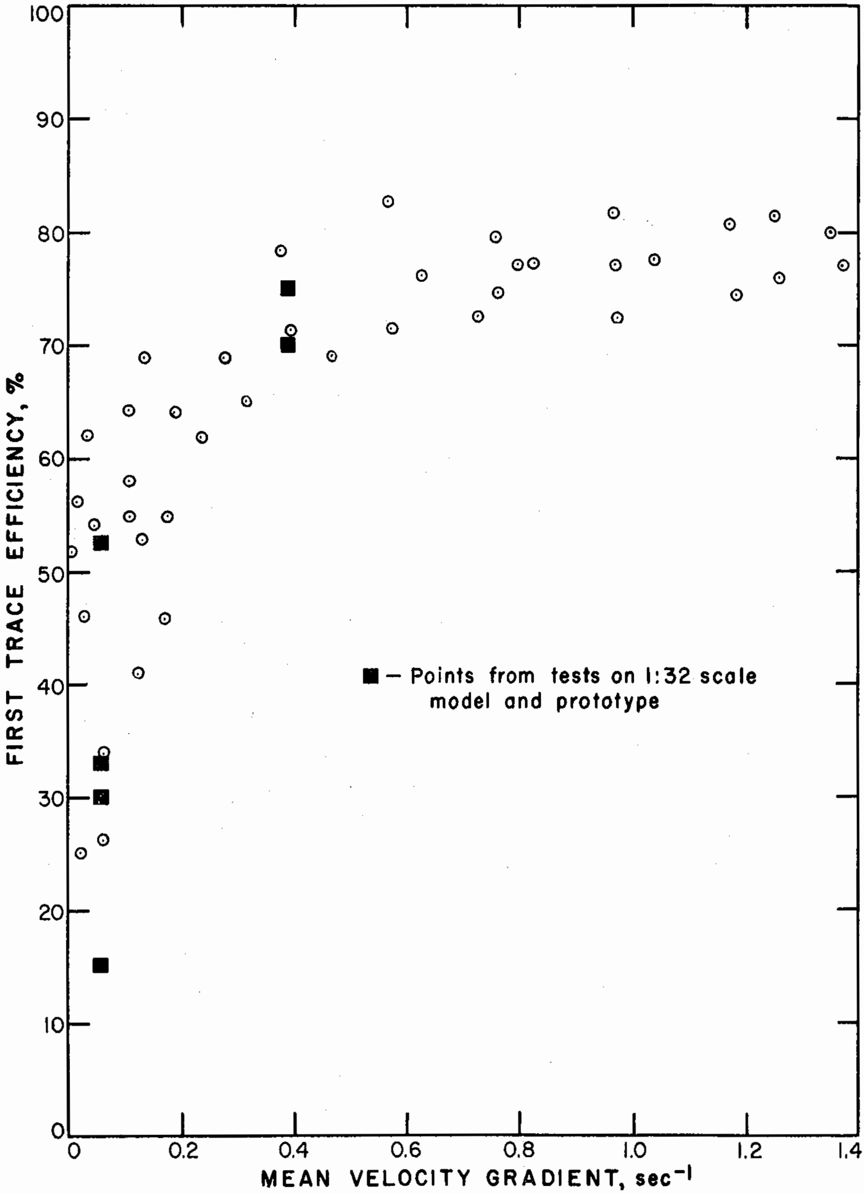

FIGURE 5-2.—FIRST TRACE EFFICIENCY VS. MEAN VELOCITY GRADIENT.

two long baffles dividing the tank into three channels in series as shown in Figure 5-3. In the first two runs, G, as computed from Equation (5-12), was 0.068 per second while in the last run, G was 0.386. Figure 5-4 shows that while the transverse baffles did some good the effectively long narrow tank with a value of G above the critical was far superior.

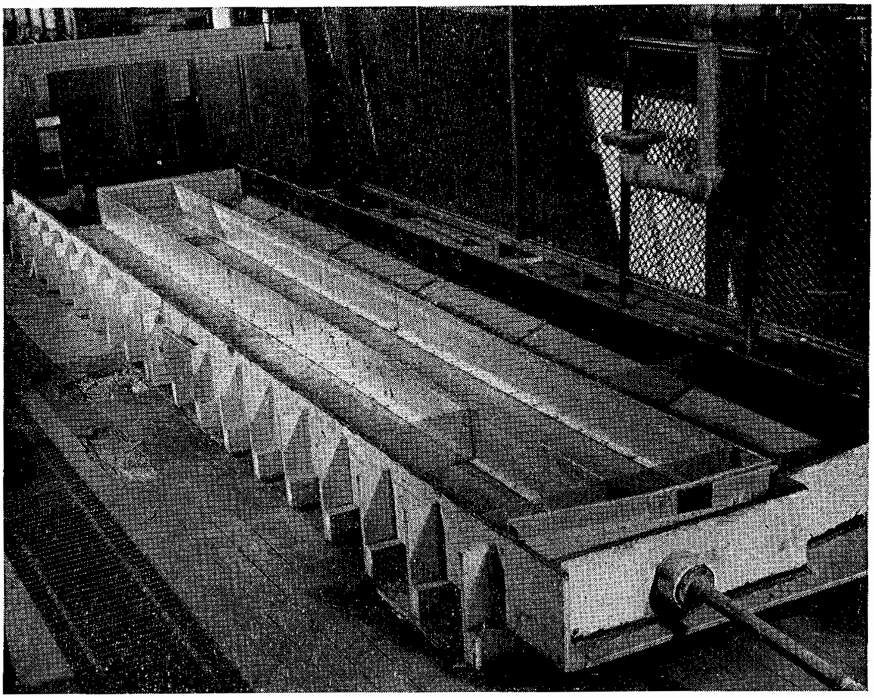

FIGURE 5-3.—1:32 SCALE MODEL MODIFIED BY INSERTING LONGITUDINAL BAFFLES.

The curves presented in Figure 5-4 do not meet the requirements of good flow-through curves. The areas under them are low, in the order of 65 percent, and for some, the center of gravity obviously falls to the right of $t/T = 1$. Adsorption of dye on the surfaces in the tank probably accounted for these conditions. This would obviously reduce the area of the curve and if dye were adsorbed during times of high concentration and some of it were released again as the concentration diminished this would shift the center of gravity of the observed curve to the right. More evidence of such adsorption is furnished in the case of Figure 5-4C by the high first trace efficiency combined with the rela-

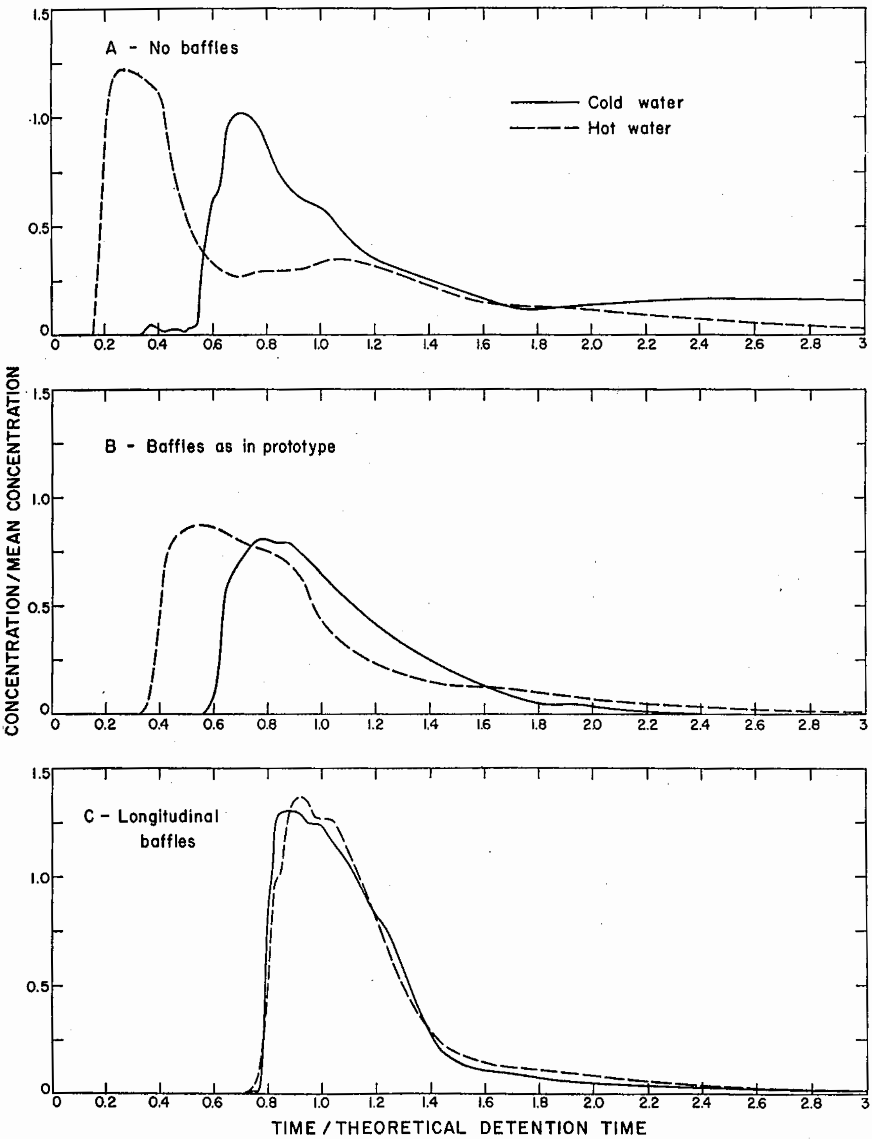

FIGURE 5-4.—FLOW-THRU CURVES FROM TESTS ON 1:32 SCALE MODEL.

tively low broad peak and not too rapid drop. The first trace time in these tests should be about the same as that expected with an ideal tracer since the first trace of dye travels through the tank with the flow of maximum velocity which would tend to stay away from the boundaries. However, with a first trace efficiency of about 80 percent one would expect a high sharp peak on the flow-through curve. Evidently the peak was adsorbed and then released again giving the distorted curve.

The transient condition caused by a change in water temperature was studied in the 8″ × 8″ flume. Numerous tests made by operating the flume with water of one temperature and then suddenly changing the temperature of the water showed that when G was above 0.3 per second the time required to remove all the water of the initial temperature was only two to three detention times under the most adverse temperature conditions while for lower values of G the stagnant layer remained almost indefinitely. This is as would be expected since for G values above 0.3 per second turbulence energy is present in sufficient quantity to cause mixing and subsequent removal of the stagnant layer.

Tests on Various Tanks

Tests on many different tank designs confirmed the importance of the mean velocity gradient in detention tank design. Tests on converging and diverging walls made in the large wooden tank showed that for G greater than 0.3 per second the performance of either was only slightly different from that of parallel walls, and was good, while for lower values of G the converging walls were superior to diverging walls but the performance in both was poor compared to that with G greater than 0.3 per second.

The 1:100 scale model of the circular detention tank illustrated in Figure 3-7 showed very poor results due to thermal density effects and non-uniform velocity distribution induced by the passage of flow around the ends of the baffles. The hydraulic design of this tank could have been improved by increasing the number of baffles so as to increase the displacement velocity, and by installing perforated baffles at the start of each straight channel to give a more uniform velocity distribution. These modifications, however, would increase the tank cost substantially.

Detention Tank Outlets

For tanks operating above the critical value of G no special outlets are required. The usual weir is satisfactory. On the other hand if a tank is operating with hot water below G = 0.3 per second a baffle wall with a slot at the bottom of the tank just before the outlet weir will help in drawing off cool water thereby reducing the size of the stagnant pool in the bottom of the tank. Figure 5-1C.

Tank Performance at Less than Design Flows

Performance of a detention tank at flows below that for which it was designed is of interest. It is evident that such reduced flow might well have a mean velocity gradient below the critical value. It is also true that the theoretical detention time for such flow would be longer than the design value. Results from many tests have shown that almost invariably with decreased flow rates the first trace time increases despite the fact that the first trace efficiency is reduced. The one exception which was noted occurred as follows: During a group of runs in the 8 inch by 8 inch flume at very low flows a certain run was made using baffle number 3 (see Figure 4-3) which had forty-nine one-quarter inch holes. The jets from the holes in the baffle traveled a few inches down stream and then broke up in turbulence. The next run was made with the same baffle at a slightly lower flow rate. This time the jets did not break up but remained as laminar flow and traveled with practically undiminished velocity well over half way down the flume, carrying the first trace of dye through in less time than in the previous run. This exception to the general rule is only of academic interest since no prototype baffle would be designed to operate at such a low velocity as is required for such laminar jets to exist.

Model Similitude

In operating models of detention tanks maintaining G the same in the model and prototype appears to be suitable for obtaining satisfactory similitude as far as the detention zone is concerned. Since

$$G = 0.167 \frac{V^{11/8}}{\nu^{3/8}\, r^{5/8}} \qquad (5\text{-}12)$$

we can write

$$\frac{V_r^{11/8}}{\nu_r^{3/8}\, r_r^{5/8}} = 1 \qquad (5\text{-}13)$$

where the subscript r denotes the ratio of the quantity between the model and prototype. If v_r is assumed to be 1 as would be the case if the same water temperature were used in the model and prototype then

$$V_r = L_r^{5/11} \tag{5-14}$$

It is of interest to compare this model operating criterion with the Froude criterion which would be normally used for operating inlet models. For free surface flow where viscous forces are not important

$$F = \frac{V}{\sqrt{gL}} \tag{5-15}$$

gives

$$V_r = L_r^{1/2} \tag{5-16}$$

as compared with $L_r^{5/11}$ when using G. Observations indicate that the flow patterns and velocity distributions in inlets are not sensitive to changes in flow expected in normal operation so that if the inlet is also designed and operated according to the velocity gradient criterion, satisfactory results will be obtained.

Figure 5-5 shows a comparison of flow-through curves obtained

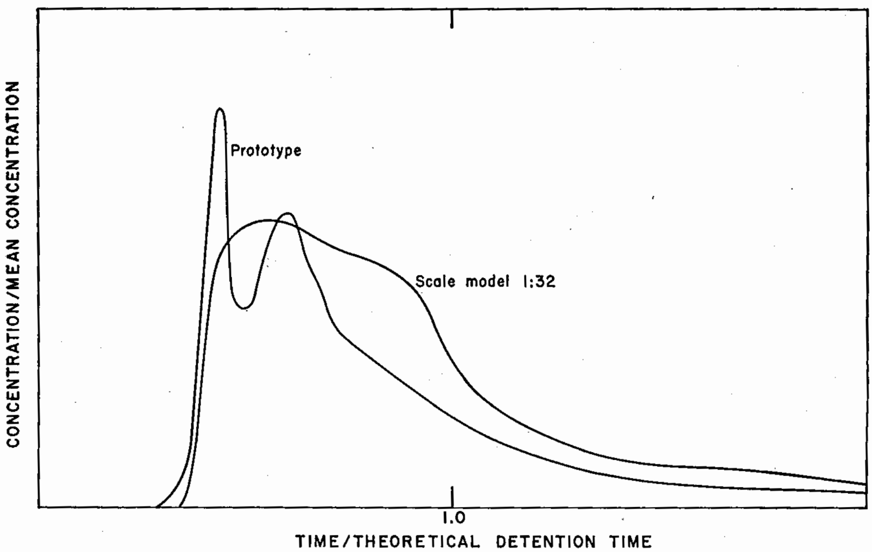

FIGURE 5-5.—FLOW-THRU CURVES FOR MODEL AND PROTOTYPE OPERATING AT SAME VALUE OF G.

from the 1:32 model and its prototype operating at the same value of G. Note the good fit as far as the time axis is concerned. The discrepancies in the concentration values are due to experimental difficulties. The prototype test was run under adverse conditions. For this reason the absolute values of concentration are in doubt. Also the baffles in the prototype were in bad condition and wind action was a factor, both of which could cause discrepancies between the model and prototype. Further tests run on other tanks and their models would be useful in proving the value of G as an operating criterion for such studies.

Integrating Tanks

Integrating tanks are used to reduce the concentration of slugs of toxic or otherwise undesirable substances introduced upstream from the tank. Their use is to provide dilution rather than detention. In a tank with perfect mixing a slug entering the tank will be instantly diluted with one tank volume of water. In terms of the flow-through curve at time zero $c/\bar{c}$ will be equal to one and as time progresses its value will drop exponentially to zero. Thomas and McKee[8] show that with two perfectly mixed tanks in series somewhat better dilution can be had. For such an arrangement $c/\bar{c}$ will have values somewhat less than 0.8. No special experiments were run on integrating tank design but it is evident from the results that some of those arrangements which gave very poor results as detention tanks would function well as integrating tanks. With the flume arranged with short round-the-end baffles the peak value of $c/\bar{c}$ was about 0.8. Apparently successful design of integrating tanks calls for good but not necessarily perfect mixing.

Conclusions

In previous sections the principles of detention tank design have been investigated. Now what remains is to indicate how these principles can be applied in practical situations. At this stage economics enters the picture, since the aim of every engineer is to provide a suitable structure at minimum cost.

Canals as Detention Basins

In the atomic energy industry it seems that in cases where installations are built at considerable distances from metropolitan areas,

[8] H. A. Thomas, Jr. and J. E. McKee, "Longitudinal Mixing in Aeration Tanks," Sewage Works Journal, 16, 1 p. 48 (January 1944).

long narrow basins in the form of canals would prove economical. They should be designed so that at design flows as computed from Equation (5-12) the mean velocity gradient would be at least 0.3 and preferably 0.5 per second. The inlet to such a basin should be simple, such as the T-inlet with one transverse baffle. The possibility of omitting the baffle and adding slightly to the length of the basin would be worth considering. Such basins would be particularly attractive in areas where irrigation canals are common since equipment and technique for their excavation and lining would be readily available.

Rectangular Tanks

In situations where space is limited it becomes necessary to "fold up" the long narrow tank to fit the available area. Under these circumstances the longest possible tank should be built and divided into channels with longitudinal baffle walls in such a way that G for the channels connected in series is at least 0.3 and preferably 0.5 per second. Such a tank must have a larger total volume for a given flow time than the long straight canal because the volume taken up by the return bends can not be considered as part of the detention zone. The T-inlet with baffle should be centered on the first channel. Some baffling at the return bends may be worth while to maintain a uniform velocity distribution.

Circular Tanks

Another design which shows promise is that of a circular tank with a spiral baffle giving a long spiral channel. A small scale model of such a tank was set up and visual observations indicated good performance. A spiral baffle has a number of inherent advantages. First, circular tanks and baffles are economical to build. Second, the flow can be introduced at the center so that in the case of radiation hazard from radioactive isotopes the most dangerous radiation zone would be farthest from the walls of the tank. Third, the secondary spiral flow induced by the continuous bend should help in keeping thermal density currents from forming. Fourth, there are no ineffective areas in the detention zone such as occur in a rectangular tank with return bends at the ends of longitudinal baffles. More experimentation would be worthwhile on such tanks to determine optimum width to depth ratios and to find what velocities should be used for best results. In the absence of further information, designs using the same values of G as recommended for straight channels would most probably give satisfactory results.

Circular tanks without baffles with the flow entering at the center and leaving over a peripheral weir such as are used for settling tanks should never be used as detention tanks. The low velocities and diverging pattern of flow both tend to cause instability and extreme short-circuiting.

Model Studies

In the design of long canal type basins without return bends a model study would appear to be unnecessary; a satisfactory design could be based on the theoretical principles embodied in Equation (5-12). A small scale model of the inlet zone would be helpful in determining an effective and economical design, but in all but the largest projects it might well prove cheaper to increase slightly the length of the canal as a factor of safety, and to dispense with the model tests. On the other hand if a tank with longitudinal baffles and return bends is contemplated, a model study is almost a necessity. Experimentation on a model will often show that marked improvements can be made in the return bend zones by the proper placing of small baffle walls or deflectors to reduce high velocities and increase the homogeneity of the turbulence. In many instances such inexpensive measures may permit the omission of more costly perforated transverse baffles with little or no sacrifice in performance. As for spiral tanks it is thought that when some experience has been gained in their design and operation good performance will be achieved without the necessity of making a model study. However, on the basis of present knowledge it would be unwise to proceed with the construction of a circular prototype tank without first undertaking a model study.

Economic Implications of Hydraulic Design of Detention Tanks

Cost estimates on three tanks which give the same performance show that good hydraulic design is economically worthwhile. The 1:32 scale model modified by inserting longitudinal baffles (Figure 5-3) was used as the "standard" of performance. The flow-through curves (Figure 5-4) indicate that in order to get the same performance from the tank with transverse perforated baffles as shown in (Figure 3-8), the length would have to be increased. The exact amount of increase would depend on the decay characteristics of the substances to be removed by the tank. If the half-lives of these are very short it would be necessary almost to double the tank length so that the leading edge of the flow-through curve would come through in the same absolute time as in the standard. To be conservative it was assumed that a fifty percent increase in length would be sufficient. The following additional assumptions pertaining to construction were made: 1) External walls and bottom 12 inches thick; 2) Internal walls 8 inches thick; 3) Transverse perforated baffles made of standard 8 inch concrete blocks with holes, reinforced with concrete beams and buttresses were taken to have the same cost per unit area as 8-inch thick concrete longitudinal baffles; 4) Concrete cost, including reinforcing steel, \$40.00 per cubic yard in place; 5) Excavation cost \$0.50 per cubic yard; 6) Flow line level with ground surface. On this basis the standard tank would cost approximately \$240,000 while the longer tank would be approximately \$295,000. On the same basis the prototype tank as built in Figure 3-8 would cost only \$200,000; however, its performance is well below that of the other two. The cost of a canal to give a performance equivalent to that of the standard tank was also calculated. The cross-section selected had a bottom width of 24 feet, a depth of 17 feet including 1 foot of freeboard and side slopes of one on one. The required length was 1870 feet. When operating at design flow the canal would have a mean velocity gradient of 0.47 per second, somewhat higher than that in the tank. This canal excavated to a depth of 16 feet below the natural ground level at a cost of \$0.50 per cubic yard and lined with four inches of concrete at \$40.00 per cubic yard, would cost about \$90,000. The high cost for concrete was used in this case to cover the expense of providing unusally effective water stops at the expansion joints and superior concrete work throughout to insure a water-tight lining.

The design of detention tanks will always require the services of a competent hydraulic engineer. His judgment is necessary in selecting the best arrangement for any given situation. It is hoped that the results of this research will aid him in providing a suitable, efficient, and economical design.

Acknowledgements

Many people have contributed to the successful completion of the research described in the present work. The author is indebted to Mr. Arthur E. Gorman, formerly Sanitary Engineer with the Atomic Energy

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The author is also indebted to all his fellow workers on the Atomic Energy Commission project and particularly to Mrs. Jane Maclachlan and Mr. Gerald Parker for their invaluable help in carrying out the experimental work and to Mrs. Lena N. Young for her help in preparing the manuscript. The successful operation of the equipment was in large part due to the skill of Mr. Kenneth C. Hanna and his helpers in the machine shop in building it. Mr. Chester Clarke and his men deserve special mention for the nice work they did in building the 1:32 scale model and the large rectangular wooden tank.

The authors final thanks are to his sister-in-law, Mrs. Olga Horoschak, who contributed her time and skill in typing the final draft and to his wife, Mrs. Elizabeth Kleinschmidt, without whose aid and encouragement this thesis would not have been written.

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USE OF THE McILROY ELECTRIC ANALYZER FOR PIPELINE NETWORK ANALYSIS

By Stephen E. Dore, Jr.*

(Presented at a joint meeting of the Hydraulic and Sanitary Section, B.S.C.E., held on February 3, 1960.)

It is indeed a pleasure to be here this evening and present to you my experiences in the use of the direct-reading electric analyzer for the solution of complex pipeline network analyses. Although my experiences have been somewhat limited they will, I believe, acquaint you with the McIlroy analyzer and its potential value to the water supply engineer.

Those of you in the water supply field have made, or are at least aware of, the lengthy computations which are required to determine the division of flow, the loss of head, and the resulting water pressures throughout a complex water distribution system. These computations are laborious and time consuming, even though several shortcuts and improvements in the methods of computations have been devised to somewhat reduce the time required.

In 1946 Malcolm S. McIlroy conducted research and studies of pipeline network analysis and established basic design for a direct-reading electric analyzer which determines the division of flow and the loss in head in a pipeline network based on the analogous behavior of electric current in an electrical circuit and the flow of water in a pipeline. This research and studies were conducted for his doctorate at the Massachusetts Institute of Technology, where he became associated with many men eminent in the field of hydraulic and electrical engineering including several who had already devoted much time and thought to the analogies of electric current flow and the flow of water in pipeline networks and to the construction of a working analyzer.

Subsequently, as Professor of Electrical Engineering at Cornell University and as special consultant to the Standard Electric Time Co. of Springfield, Mass., Mr. McIlroy successfully constructed a test model of a direct-reading electric analyzer. This analyzer is presently located at Cornell University and has been used in the analyses of many

* Coffin & Richardson, Inc., Boston, Mass.

gas and water distribution networks by both municipal, industrial and consulting engineers. The death of Mr. McIlroy in 1956 cut short his work at a time when his analyzer was rapidly gaining recognition and acceptance throughout the country.

Description of Analyzer

The direct-reading electric analyzer is based on the electrical analogy method of pipeline network analysis and eliminates all calculations beyond those necessary for the compilation of the basic data concerning the pipeline network and the conversion of this data to electrical equivalents. The electrical-analogy method of pipeline network analysis is based on five basic analogies as follows:

1. That the flow of current in an electric circuit is similar to the flow of water in a pipeline. The quantity of current is measured by placing an ammeter in series with the conductor just as the quantity of water is measured by placing a flowmeter in the pipeline.

2. That the drop in voltage in an electrical circuit is similar to friction head loss in a pipeline. When electrical current passes through a conductor or resistor a drop in the voltage occurs. This drop in voltage is measured by connecting a voltmeter across the extremities of the conductor or the terminals of the resistor. In a water pipeline the loss of head along the line is measured by connecting a differential pressure gage to the ends of the pipeline.

3. That the law of balanced flow at the junction of a pipeline network is analogous to the flow of electric current at a junction of an electrical circuit. That is, the flow into a junction or intersection equals the flow out of that junction or intersection.

4. That the sum of the voltage drops in a clockwise direction in any loop of an electric circuit equals the sum of the voltage drops in a counter-clockwise direction around that loop, just as the sum of the clockwise head losses equals the sum of the counter-clockwise head losses in any loop of a pipeline network.

5. The fifth and very important analogy for the use of the direct-reading analyzer concerns the voltage drop across any resistor. Whereas the current and the voltage drop across any resistor are analogous to the rate of flow and the head loss in a pipeline, they are not necessarily proportional. In the case of water flow the head loss varies nearly as the square of the flow. In electric current flow according to Ohm's law the drop in voltage is proportional to the flow and the resistance; however,

the resistance is a constant. Special non-linear resistors also called fluistors were developed for use in the analyzer in which the voltage varies nearly as the square of the current and satisfies this basic analogy.

The direct-reading electric analyzer consists of a sheet steel panel on which is attached a number of permanently wired, four sided electrical circuits with sockets for inserting non-linear resistors on each of the four sides. This panel is usually mounted in an upright position with access to both sides. The non-linear resistors represent the pipelines and by inserting short-circuiting plugs or by leaving sockets empty or by using electrical jumpers between intersection of these sides the electrical circuits can be arranged to conform to the pattern of the pipeline network under study. The non-linear resistors which represent the pipelines resemble ordinary showcase bulbs having tungsten filaments supported on heavy nickel leads. The filament and leads are designed so that voltage varies to the 1.85 power of the current through them to correspond with the exponent of flow in a pipeline as determined by the Hazen and Williams formula. Such a condition is possible because of the variable diameters of tungsten available and the stability, long life and peculiar resistance property of this metal. The voltage along a tungsten filament varies nearly as the 1.63 power of the current through it, if the filament is protected from the atmosphere and from the cooling effects of adjacent material. Therefore, by placing the filament in a vacuum and designing the leads to result in some cooling effect, the desired exponential value of 1.85 of the current flow is obtained for the voltage drop across the non-linear resistor.

It was the development of this non-linear resistor that eliminated the trial and error runs of previously constructed electrical analyzers using linear resistors.

The withdrawals from a pipeline network are represented on the analyzer by load currents which are independent of the voltage available at the load circuit terminals. The load currents are constant current ballast lamps which operate over a fairly wide range of voltage, and an adjustable rheostat is placed in series with the lamp to assure its operation in the desired range. These lamps are demountable and are selected according to the quantity of current required to represent the withdrawal from the pipeline network.

The lamps are placed in the load panel section of the analyzer and are connected to the circuits representing the pipeline network by electrical jumpers to permanently wired jacks connected to intersections of those circuits.

The source of power required to operate the analyzer can be any source of steady direct voltage such as a battery or a motor-generator set. By means of manually adjusted rheostats the desired differences of source voltages can be maintained to correspond with the known differences of head between any pair of sources, or between a single source and any other known point in the system.

Preparation for and Use of the Analyzer

The consulting engineering firm with which I am associated has used the McIlroy Analyzer located at Cornell University to analyze several water pipeline networks.

These distribution systems were not unusually large as to the number of major pipelines; however, several sources of supply at different heads were involved which added considerably to the complexity of the analyses. In all instances the field and office procedure prior to the use of the analyzer was practically identical and was as follows:

1. All of the necessary data was obtained from records for the particular water distribution system under study. This data included the size, length, age, and type of all mains in the distribution network. Field tests were conducted to determine representative "C" values of the water mains. From this basic data a head loss coefficient for each main in the network was computed which reflected the pipeline length, its size, type, and the relative roughness of the pipe interior. The ground elevations at all hydrants with reference to points of supply and storage were also obtained so that pressures throughout the system could be readily determined.

2. The water requirements and their distribution throughout the system were determined using meter and pumping records, and records of the variations of elevation in all storage facilities. From these records the maximum, the average, and the minimum water demands of the system were determined. In addition to the domestic and industrial demands for water, the quantities and location of water requirements for fire-fighting purposes were obtained from the New England Fire Rating Association. Studies were also conducted to estimate future water supply requirements of the system together with the estimated distribution of these future requirements throughout the existing network and at possible future additions to the network.

3. Due to the impracticability of determining the flow and pressure in each and every main within the distribution system, a trunk main system was established which comprised the larger sized mains of the system together with those smaller sized mains which formed loops with the trunk main system and those known to carry large flows.

Skeletonizing a system undoubtedly leads to inaccurate results as to the actual conditions, however, by careful reduction of the mains to equivalent mains and by a judical location of water demands on the network these inaccuracies can be held to a magnitude where they will not affect the final conclusions to be made from the analyses.

However, the McIlroy analyzer is so conceived that if it were practical and economically justifiable to do so, one could be constructed which would provide for almost an unlimited number of pipelines, sources, and loads.

At the several times when we utilized the Cornell Analyzer the capacity of this anlyzer was limited to approximately 90 pipelines 7 sources and 32 loads.

By way of comparison, the McIlroy Analyzer recently installed at Tufts University in Medford, Mass., has provisions for 710 pipelines, 225 electronically regulated loads, and 36 sources. Thus, the magnitude of the particular analyzer to be used as well as the practicability of the results to be obtained from the analyses is a factor that must be considered in skeletonizing the distribution network.

4. After all basic data regarding the physical and hydraulic characteristics of the system were determined, these fluid data were converted into electrical equivalents. Conversion to electrical equivalents involves the determination of three factors:

1. B factor, which is $\frac{V}{H}$ = volts per unit head loss.

2. G factor, which is $\frac{I}{Q}$ = amperes per unit flow rate.

3. O factor, which is the ration $\frac{k}{k_p}$, k = resistor coefficient and k_p = head-loss coefficient.

The selection of these factors depends upon the approximate total head loss and the range of flows expected in the system. The B factor is selected so that the average voltage per resistor is approximately 2.5 volts and the G factor is selected to give load currents suitable for the ratings of the ballast lamps which vary between .1 ampere and 1.0 ampere. The determination of the B and G factors establish the O factor for the resistor coefficients.

5. Following the conversion of the fluid quantities to electrical equivalents, an electric circuit diagram was prepared, using a standard form for the particular analyzer to be used. This diagram depicts the pipeline network with sources, load points and pipelines appropriately shown and numbered. For our first use of the analyzer this diagram was prepared by the technician operating the analyzer; however, for subsequent use these diagrams were prepared by our own personnel.

6. After the electrical data and circuit diagrams were completed, the next step was to set up the analyzer ready for use. This involved the insertion of the proper non-linear resistors in the panel circuits to represent the pipeline network, installation of constant-current lamps at the designated load points, and adjustment of the input current at the sources to the desired rates corresponding to actual pumping rates of the particular source or to any assumed rates of flow for purposes of study. Using the previously prepared electric circuit diagram, the operation of placing the distribution network on the electric analyzer was accomplished in a matter of only several hours time.

7. After the pipeline network and the demand loads were placed on the analyzer, the system was gradually energized. This was necessary to prevent the possible burning out of the highly loaded fluistors. The fluistors should not glow brighter than a cherry red color and any of those which were apt to glow brighter or burn out under full load were replaced by equivalent series or parallel combinations of fluistors. These combinations were mounted in separate banks of sockets and temporarily connected into the fluistor socket on the panel. All ballast lamps representing the loads on the system were also checked to ascertain that the correct currents were being drawn. Any variations indicated were corrected by adjustment of the manually operated rheostats at each load point.

Finally with the source voltage up to full load and all fluistors and ballast lamps operating properly, the analyzer was ready to use.

It is at this time that the direct-reading electric analyzer becomes a real asset to the Engineer. Up to this point, his work has consisted of compilation and programming of data and setting up the analyzer. The

Engineer may now measure and record the flow rate and head loss in each and every main in the network or he may only investigate certain portions of the network dependent upon the particular problem involved. A glance at the fluistors tells him where the trouble spots in the distribution system are located. The degree of brightness of the fluistors is indicative of the magnitude of the head loss in each section. However, those fluistors which were replaced by series or parallel equivalents may not necessarily glow brightly but the need for replacement is usually sufficient indication that a high head loss exists in that section.

To obtain the rate of flow in any pipeline of the network it is merely necessary to connect the panel mounted ammeter in series with the fluistor representing that pipeline and read the flow converted to gallons per minute. To obtain the head loss in that pipeline, the panel mounted voltmeter is connected in parallel with the fluistor representing that pipeline and the voltage drop or head loss in feet is read on the voltmeter. Special demountable scales are available for use on both the ammeter and voltmeter which allow flow readings to be made directly in gallons per minute and head loss read directly in feet.

These readings can be quickly taken as connections for the ammeter and voltmeter are made by inserting jumpers into built-in jacks, which are located at every pipeline intersection and for every pipeline. By noting the placement of the voltmeter leads, the direction of flow in each pipeline is determined.

In less than one hour's time, two persons can measure, read, and record the rate of flow and head loss for each section of a system approximating 90-100 pipes and 25 loops. A mathematical check on the analyzer can be made by adding the head losses around each loop and the flows in and out of each intersection. This can be done by the recorder simultaneously with the taking of the readings. When compared with the many hours required for trial and error computations of a network of this magnitude, the saving in time is considerable. Once the network is placed on the analyzer little time is required for the actual measuring, reading and recording of the data. Changes in the existing network such as the addition of new mains to form loops or to reinforce the existing mains, or perhaps a new source point or increased load points can be made very quickly and the resulting flows and head losses measured. Because this analyzer can be rapidly adjusted to reflect the results of possible changes in the distribution system, it is invaluable to the Engineer in that, for relatively little work, and in a very short time much information can be obtained concerning any proposed change in the network.

The performance records for the analyzer indicate that an overall error of less than 3% of the total head loss is obtained which is within satisfactory engineering accuracy for distribution network analysis. This error is on the conservative side and tends to show a slightly larger loss than would be obtained by using the Hazen and Williams formula. The accuracy of the results obtained from the analyzer is of course consistent with the accuracy of the input data relative to the distribution system.

In closing, I would like to thank Dorothy W. McIlroy, widow of the late Dr. McIlroy for her invaluable assistance rendered to representatives of our firm during the programming and operation phases of our work on the analyzer at Cornell University.

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SMALL ACTIVATED SLUDGE TREATMENT PLANT DESIGN

By Richard M. Power*

(Presented at a meeting of the Sanitary Section, B.S.C.E., held on March 1, 1961).

At the present time there are in the state of Massachusetts, 19 activated sludge treatment plants having a design capacity ranging from 5,000 to 100,000 gallons per day. These plants have no primary settling or primary sludge handling facilities. The raw sewage enters directly into the aeration tank and thence flows to a settling tank. The sludge from the settling tank is returned rapidly back to the aeration tank. In most cases chlorination is also provided. This type of treatment plant is called many names by the different manufacturers, some of which are Oxigest, complete mix, wet burning, Rated aeration, Aerobic digestion and Sparjair. In all cases, however, the unit is essentially an activated sludge treatment plant without primary treatment.

This paper will deal with the design of the treatment units, the influent works and the receiving stream.

What is Expected of the Treatment Units

Normally, the effluent is discharged to a small brook or stream which has a down-stream water use either presently or in the future of Class C which requires a minimum dissolved oxygen of 5.0 mg/1. Most of the small streams or brooks will reaerate rapidly but do not have the essential biota for pollution assimilation. Normally, the treatment unit is expected to produce better than 85% treatment. Most raw wastes being treated at the small plants have a biochemical oxygen demand (BOD) greater than 200 mg/1. Fifteen percent of a BOD of 200 mg/1 is 30 mg/1.

Let's assume an effluent BOD of 30 mg/1. What is the minimum stream flow capable of receiving this effluent? Using a general stream loading equation the maximum allowable BOD of the stream below the point of discharge varies between 3.3 and 15.8 mg/1, for a minimum dissolved oxygen of 5 and 2 mg/1 respectively. Since the receiving

* Associate Sanitary Engineer, Mass. Dept. Public Health.

waters are in a Class C condition, we would use the minimum dissolved oxygen of 5 mg/1 and a maximum allowable BOD after dilution of 3.3 mg/1. Under these conditions, the stream should provide a dilution of 11.6 to 1. This dilution should be available during the time when the effluent is coming out of the plant. For a flow occurring during 16 hours, the dilution factor should be raised to 17.4 to 1. For example, a plant receiving 10,000 gallons per day in 16 hours would need a stream flow of 174,000 gallons per day. This would require a drainage area of at least 3 square miles. For a BOD of 20 mg/1 in the effluent, the stream must provide 109,000 gpd or about 2 square miles of drainage area. This all adds up to finding a good stream and not trying to put the effluent into a small brook which flows only during wet weather. It points out the need for a good field survey of the source or watershed of the stream and also the down-stream water use.

The BOD in the effluent is what we are primarily interested in. After determining this, we would then work out the percent of treatment required to attain the effluent concentration. The treatment unit then must reduce the BOD to at least 30 mg/1 and better yet 20 mg/1. The effluent must be very low in settleable solids and also visible solids.

The Influent Works

It is desirable that the influent sewers be laid at a slope which will be self cleansing. It cannot be assumed that high rates of flow will occur occasionally to remove deposits. For 10,000 gpd occurring in 16 hours, the rate of flow is 15,000 gpd. In order to provide self cleansing velocity at these low rates of flow, it would be necessary to lay a six-inch sewer at a slope of 10.5 feet per 1000 feet. In many areas, this slope is not available and it may be necessary to periodically flush the lines in order to prevent the accumulation of deposits. The necessity for flushing will be determined in large part by the amount of heavy foreign matter which enters the system. At some locations, only light organic material is carried and flushing may not be necessary while at other locations, industries, for example, there may be a small amount of heavier materials which find their way into the collection system.

Comminutor and Pumps

A pump is required only where gravity head is not sufficient. The pumping rate in large part controls the addition of organic material to the aeration tank and the overflow rate on the settling tank. It should operate as often as is feasible and at as low a rate as is practical. A 15,000 gpd rate of flow is 10.2 gpm and a 300% variation of this is 30 gpm.

The smallest pump which is in the normal production for raw sewage is a 50 gpm unit. As pointed out above, this pumping rate is greater than necessary for small collection systems. One way of using the 50 gpm pump and not overloading the treatment unit is to pump into a flow splitterbox. In this box, there is an adjustable fin which diverts part of the flow and returns it back to the wet well. By trial and error, this fin can be adjusted so that the design flow is directed through the treatment unit. The splitterbox has worked very effectively at some plants.

A comminutor is needed upstream from the wet well or at the inlet to the aeration tank, whenever it is felt that a shredder is necessary. It is questionable whether this expensive ($1800) piece of equipment is necessary at all plants. There are several plants operating well without one. At these latter, it is necessary to prevent the use of high wet strength brown paper towels etc. If a shredding device or a coarse screen is used, there should be an ample approach velocity, in order to provide sufficient energy to push the soft organic matter through the fine openings. Provision should be made for a free drop into the aeration tank while being aerated.

Design of the Treatment Unit

Aeration tank—the determination of the size of the aeration tank is one of the most controversial subjects in sanitary engineering today. The following is a list of some of the criteria presently in use: lbs of BOD/1000 cubic feet, lbs of BOD/lb of mixed liquor solids, time of aeration, sludge age, Eckenfelder's equations and lbs of BOD per 1000 lbs of solids per hour of aeration. The time of aeration, while an important individual factor, does not take into account the amount of organic material being added to the biological unit, thus it can be ruled out as the complete criterion. Sludge age is a measure of some of the many factors and seems to be useful at some plants but not at others. In Eckenfelder's formulation, it is necessary to know or assume a solids loading factor, the concentration of mixed liquor solids and the concentration of the return sludge. It is normally advisable to determine these by the operation of a pilot plant. If the mixed liquor solids concentration is known or has been assumed, the lbs of BOD/1000 cubic feet can be expressed as lbs of BOD/lb mixed liquor solids. In using the last two criteria and the lbs of BOD per 1000 lbs of solids per hour of aeration, it is necessary to assume a mixed liquor concentration.

Let's backtrack a little and see what the criterion should include. Since this is a biological treatment unit, it should include lbs of BOD or amount of organic matter being added per day or per unit of time. It has been stated that [1] "(a) area of contact surface or film and (b) opportunity for contact" are the controlling factors in biological sewage treatment. Area of contact surface or film can be indirectly expressed as lbs of BOD per 1000 lbs of mixed liquor solids and opportunity for contact can be expressed as hours of aeration.

The criterion lbs of BOD per 1000 lbs of suspended solids per hour of aeration has been correlated and formulated relative to efficiency. The curve for this formulation is shown on page 723 of "Water Supply and Waste Water Disposal" by Fair and Geyer. The efficiency equation is

$$P_2 = \frac{100}{1 + 0.03 \dfrac{(Y_0)}{(Wt)}^{0.42}}$$

By changing the units of expression, the $\dfrac{Y_0}{Wt}$ can be equated as follows:

$$Y_0 = \frac{2400\ (U_0)}{(mg/1 \text{ of ss})\ t^2}$$

therefore

$$t^2 = \frac{2400\ (U_0)}{(mg/1 \text{ of ss})\ (X)}$$

U_0 is BOD_{-5} raw in mg/1
X is value on X-axis—from curve
t is time of aeration in hours

Solving for t we can substitute it into

$$T = \frac{V}{Q}$$

$$V = \frac{Qt}{24}$$

[1] Fair & Geyer, Water Supply and Waste Water Disposal.

Where —Q is the raw waste flow
—T is detention time in days
t is detention time in hours
V is the volume of aeration tank

The curve is based on a voluminous number of sample results at plants throughout North America.

Let's compare this last criterion with lbs of BOD/lb of solids. (This may be expressed as per lbs of solids, per 100 or per 1,000 lbs of solids.) Take two wastes each requiring 90% treatment efficiency. The comparison of treatment plants using the two criteria is as follows:

TABLE I

				Based on Curve			Based on lbs BOD/lb of s.s.		
Waste	BOD raw mg/1	Flow mgd. Q	# BOD day	Effl. mg/1 90%	V for 90% mgd.	t hrs.	% removal Both Tanks 0.87	Effl. mg/1	t
A	400	1	3330	40	0.61	14.7	92.5	30	20.8
B	200	2	3330	20	0.87	10.4	90.0	20	10.4

Based on the curve, tank A. is 0.61 million gallons and tank B is 0.87 million gallons. Using lbs per lb of suspended solids both tanks would be the same size since there is the same weight of BOD and we assumed the same suspended solids concentration. If tank A were made 0.87 million gallons, the efficiency would increase to 92.5% but only 90% is needed. This would mean that tank A is 260,000 gallons over-designed or 0.26/0.61 = 42.5% too large. This is a considerable waste.

The efficiency formula has been compared with the results of a one week composite sampling at the Bedford nike site. The actual overall efficiency during the one week of around the clock sampling was 96.2%. The formula worked out to 97.7%.

At the present time, there is no efficiency formula which is extremely accurate for all wastes at all types of activated sludge plants. In the writer's opinion, the Fair-Thomas formula has the proper ingredients and with more data in the high percentage range is the type which should be used for the design of the small high efficiency units.

Suppose we assume that these criteria are fine guides but that the aeration tank can be made larger and that efficient operation will be more assured. This assumption has been shown to be in error. The oversized unit produces high nitrates and nitrites. In doing this, it breaks down nitrogenous material and particularly ammonia. The combination of these chemicals plus the low alkalinity in the sewage in this part of the country causes the pH to lower considerably, sometimes to 4.5. At the lower pH values, the desirable bacteria do not function well and a different biota is established. The solids do not settle well and the efficiency falls off appreciably. This condition may also occur to some extent in a properly designed highly efficient unit. The intermittent or continuous addition of ground limestone, calcium carbonate, has remedied the condition.

The Aeration Equipment

This equipment performs two functions, namely, oxygenation and mixing. The mixing should be sufficient to provide intimate contact between the biological floc, the organic matter and the oxygen. The mixing should also assure that there be no appreciable accumulation of solids on the bottom of the tank. The usual rule of thumb velocity to prevent accumulation is 1.0 to 1.5 fps. At most of the small plants this velocity is greatly exceeded. At one plant the excessive mixing, turbulence, which gave rise to voluminous foaming was reduced with a resultant decrease in foaming and a considerable increase in settleability. The control of mixing and turbulence is a matter which should be borne in the designer's mind and one which will likely be the subject of research in the future.

In these small high efficiency plants the amount of oxygen to be supplied is approximately equal to the ultimate BOD. The oxidizable COD is similar to the ultimate BOD and since oxidizable COD takes less time in the laboratory, it is frequently used as the basis for air supply. If the BOD_{-5} to oxidizable COD ratio is determined, the air supply can then be computed relative to the BOD_{-5}. For example: with normal domestic sewage, a 5% transfer efficiency of the aeration equipment, 15 pounds of oxygen per 1,000 cubic feet of air, and a BOD_{-5} to oxidizable COD ratio of 0.66 the amount of air is 1.4 cfm per pound of BOD_{-5} per day.

The amount of oxygen in solution and available for the biochemical reactions is dependent on many factors; some of which are—transfer efficiency of the diffusor device, dissolved oxygen in the tank, geometry of the tank and surface active chemicals in the solution. The design of the diffusor equipment can be based on the cfm per pound of BOD_{-5} figure. The air blower chosen should supply this output at about 2/3 of maximum speed. If the blower unit is equipped with a standard adjustable motor base and spring loaded adjustable sheave, the speed of the blower can be controlled by means of a hand wheel and screw over a range of 3 to 1. With this inexpensive variable control, the errors due to unmeasured variables can be compensated.

With mechanical aerating devices, the manufacturer's rated capacity is used. The manufacturer also recommends the dimensions of the aeration tank for a particular size of unit. Variability is accomplished by intermittent operation which is controlled by a time clock. It is to be noted that when the unit is off, mixing as well as oxygenation is stopped. The operational results of plants employing this type of control show a good effluent so continuous mixing is evidently not required at these plants. This may also indicate that less mixing is required in the aeration tank of any plant.

Settling Tank

The function of the settling tank is to separate the active solids from the liquid. The overall efficiency of the treatment plant is determined in large part by the settleability of the solids as they enter the tank and on the ability of the tank to remove the solids from the liquid. In the smaller treatment units the possibility of shortcircuiting through the tank is much higher than in larger settling tanks. Experience has shown that standard proven settling tank designs should be used. Novel or over-simplified settling tanks have proven very unsatisfactory. Because the tank is very small, the inlet zone and outlet zone are sometimes so close that there is little or no settling zone. Standard rectangular or circular tanks are recommended and any variation from normal design practices must be critically reviewed.

The surface area load on the settling tank should be less than 800 gpd per square foot. With the proper overflow rate, inlet arrangement and tank geometry, the detention time usually works out to 4 hours. It is normal practice to include the volume of the upper 1/3 of the sludge hopper when determining the detention time. The slope of the hopper of a nonmechanically cleaned settling tank should be at least 1.5 to 1. It has been found that the sludge tends to stick to the sides of the settling tank and that some method of periodically cleaning the sides is necessary. One company has provided a chain on the end of an off-set shaft which when manually rotated will drag along the surface of the hopper. This device is similar to what is used in cleaning the sloped sides of the settling chamber of an Imhoff Tank. The settling tank should also be equipped with a device for skimming. Since these treatment plants do not have primary settling, some of the solids which adhere to the mixed liquor are lighter than water and will cause the mixed liquor particles to float rather than to settle in the settling tank. The skimming can be accomplished with several devices; some of which are an air lift pump attached to a skimming funnel or pipe, scum trough which discharges back to the pump wet well or in the case of the down draft mechanical unit, a funnel and pipe which connects into the center of the aeration unit at a point above the level of the vortex. The tank should also be equipped with a scum baffle, standard adjustable v-notched weir plate and inlet baffle.

Some features which have not proven successful are two sludge hoppers in series, effluent weir trough which is adjustable only on one end. Vertical settling tanks integrally connected with the aeration tank and with or without positive sludge return have not proven at all satisfactory. As a matter of fact, vertical settling tanks should be designed on the basis of 500 gpd per square foot. It is difficult to return the sludge from a normal settling tank by the use of a sludge return pump and gravity sludge return is extremely difficult to accomplish. In the case of the gravity return settling tanks, the sludge accumulates until it becomes septic and then floats to the surface because of the release of nitrogen gas. The sludge cannot be effectively skimmed by shovel or scoop from the surface since it is so fragile, and the operators tend to let it accumulate until it becomes septic and very odorous.

Sludge Return Pumps

At all of the existing plants sludge return is accomplished by use of an air lift pump. An air lift pump consists of a 2 ½ or 3 inch pipe extending down into the sludge hopper with air being defused through a footpiece at a point near the lower end of the intake pipe. The air lift pump has been a source of trouble at almost every plant since it frequently clogs. In addition, the capacity of the pump is normally about 50 gpm. At some plants, particularly those with two sludge hoppers, the sludge return rate is 1,000% of the raw sewage flow. The rate of flow of an air lift pump can only be reduced approximately 15%. Reduction of the air supply beyond this point shuts off the pumping action. Operating the pump intermittently seems to correct the clogging at some plants. Where it is possible to place a pump inside a heated building, the use of a diaphragm pump or small pump having a strong suction would have considerable advantages over the air lift pump. The sludge return pump should be designed and/or chosen so that the rate can be varied and frequency of operation can be controlled by a time clock. The capacity of the pump should be approximately 100% of the raw sewage influent rate. It should be noted here that the sludge return rate determines in part the concentration of mixed liquor solids. The sludge pumping rate also affects the flow pattern in the settling tank.

Miscellaneous

All of the treatment plants produce excess sludge. The build up of excess solids is determined in large part by the efficiency of the settling tank. Under equilibrium conditions of food to metabolism, the rate of sludge build up will be approximately 11% of the amount of BOD removed. Therefore, there will be an increase in build up of solids in the treatment unit. At most plants it is felt that removing excess solids by means of a septic tank truck is the best method of wasting. It is also possible to pump the solids to a septic tank or an anaerobic holding tank. One manufacturer provides an aerobic digester. Since the prevention of odors in the vicinity of the plant is important, the use of sludge drying beds is discouraged.

The control of bacteria in the waste is almost always required. Because the unchlorinated effluent contains excessive bacteria, chlorination is necessary. The chlorinator and contact chamber should be capable of providing a 1 mg/1 (flash test) chlorine residual after 15 minutes contact time. The contact chamber should be designed to reduce short circuiting. In the author's opinion the flow pattern through the usual baffled contact chamber does not reduce short circuiting but may actually induce it and therefore the design should be critically reviewed.

Most of these plants are called package plants. The proper design of a plant, however, involves considerably more than merely looking in a catalogue and picking out a package. Each component part must be tailored to the strength, volume and variations of the particular waste. For this reason the author suggests that the plants be called Small Activated Sludge Plants and that the word package be abandoned.

Conclusions

1. The efficiency of the treatment unit is determined by the pollution assimilative capacity of the receiving stream.

2. The treatment plant should be designed to produce an effluent BOD of less than 30 mg/1 and preferably 20 mg/1. The effluent should be low in suspended and visible solids.

3. Provision should be made in the design of influent works for proper operation at the low flow rates which occur at small plants.

4. There is no present efficiency formula for the design of aeration tanks which is extremely accurate for all plants receiving any or all wastes. The Fair-Thomas formula has the proper ingredients and is accurate over a wide range of application. With more data in the high efficiency range the formula should become more accurate.

5. The air supply equipment should be carefully designed to provide sufficient oxygen in solution, sufficient mixing and be variable to provide for fluctuation in operation and raw waste.

6. The settling tank at the small plants in large part determines the effluent concentrations and must be carefully designed. Standard proven tank designs should be used. The surface area loading should be less than 800 gpd/square foot.

7. The sludge return pumps at existing plants have been a source of trouble. The pump should provide variable capacity, at least 100% of raw waste flow and intermittent operation. The use of presently designed air lift pumps should be reviewed.

8. Chlorination is required at almost all plants.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

SEPTEMBER 27, 1961.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the United Community Services Building, 14 Somerset Street, Boston, Mass., and was called to order by Vice-President Charles H. Norris, at 7:00 P.M.

Vice-President Norris stated that the Minutes of the previous meeting held May 17, 1961 would be published in a forthcoming issue of the Journal and that the reading of those Minutes would be waived unless there was objection.

Vice-President Norris announced the death of the following members:—

Grant H. Potter, who was elected a member May 19, 1952, and who died April 15, 1961.

Alfred Colletti, who was elected a member February 15, 1939, and who died June 20, 1960.

Wilbur S. Colby, who was elected a member February 15, 1928, and who died September 17, 1961.

Hugh P. Duffill, who was elected a member November 26, 1941, and who died September 22, 1961.

The Secretary announced the names of applicants for membership in the Society and that the following had been elected to membership on June 21, 1961:—

Grade of Member—Robert E. Barrett, Jr., Henry W. Buck, Myron B. Fiering, Russell C. Holt, Athanasior A. Vulgaropulos, Janis Zagarins.

Grade of Junior—Gerald R. Cichy, Robert F. Daylor.

Vice-President Norris announced Meeting of the Society with ASCE and ASTM to be held November 2, 1961, and that Student Night Meeting would be held at M.I.T. on October 25, 1961.

Vice-President Norris introduced speaker of the evening Stanley M. Dore, Chief Engineer, Board of Water Supply, City of New York, who gave a most interesting talk on "Investigations and Studies for the Richmond Water Tunnel under New York Harbor." The talk was illustrated with slides and sound movies.

The meeting was preceded by a dinner and 55 members and guests attended the dinner. 70 members and guests attended the meeting.

The meeting adjourned at 9:10 P.M.

CHARLES O. BAIRD, JR., *Secretary*

OCTOBER 25, 1961.—A Joint Meeting of the Boston Society of Civil Engineers with the Massachusetts Section of the American Society of Civil Engineers was held this evening at Massachusetts Institute of Technology, Cambridge, Mass. The Student Chapters of the New England Colleges were especially urged to attend.

At 4:00 P.M., a Coffee Hour was held in the Spofford Room at M.I.T., after which a tour of the Engineering Department and Laboratories was made. At 6:00 P.M., dinner was served in the Graduate House Campus Room. Student delegates from Mass. Institute of Technology, Northeastern University, Tufts University and University of Rhode Island were present.

The meeting was held in the Graduate House Campus Room, Mass. Inst. Technology and was called to order at 7:00 P.M., by President James F. Brittain.

President Brittain extended a cordial welcome to the students and expressed appreciation of the cooperation of the officers of the student organizations and the faculty for making this event so successful.

The Secretary announced the names of applicants for membership in the BSCE and that the following had been elected to membership October 23, 1961:—

Grade of Member—Ronald E. Bucknam, Edward F. Delaney, Kenneth E. MacWilliams, George H. Mathews, Walter M. Newman, Joseph A. Standley.

Grade of Junior—John D. Morrissey.

President Brittain introduced John H. Fullerton, President of Massachusetts Section, ASCE, and asked him to conduct any necessary business of ASCE at this time.

President Brittain then introduced the speaker of the evening, Mr. Thomas H. E. Quimby, Director of Recruitment Peace Corps, Washington, D.C., who gave a most interesting talk on "The Opportunity for Junior Professional Personnel in the Peace Corps."

A question period followed the talk.

One hundred ten members and guests attended the dinner and meeting.

The meeting adjourned at 9:30 P.M.

CHARLES O. BAIRD, JR., *Secretary*

Structural Section

OCTOBER 11, 1961. Chairman Myle Holley called the meeting to order at 7:05 P.M. After noting, with pleasure, the presence of Mr. James F. Brittain, President of the Society, the chairman introduced the speaker of the evening, Mr. James Carey.

Mr. Carey, Senior Technologist, Shell Chemical Company spoke on "Prospects for Epoxy Resins in Structural Engineering." The presentation included a sound film describing the use of epoxy resins to provide a thin, skid-resistant, re-surfacing coat for highway pavements. Other present (and potential) applications were illustrated by slides. The unusual characteristics of the material, as a protective coating, an adhesive, a matrix for terrazos, etc., were described and explained.

Mr. Carey responded to questions from the floor. He acknowledged that successful field applications require care on the part of the user, but emphasized his belief that epoxy resins represent a valuable construction material having almost limitless potential.

The chairman thanked the speaker and urged him to submit his remarks in written form suitable for publication in the Society Journal.

The chairman announced that the next meeting of the section would be held on Wednesday, November 8, 1961, with the speaker and subject to be announced in an early issue of the ESNE Bulletin. There being no further business the meeting adjourned. Attendance 50.

P. S. RICE, *Clerk*

Surveying and Mapping Section

OCTOBER 18, 1961.—The October meeting of the Surveying and Mapping Section was called to order by Chairman Roy L. Wooldridge at 7:25 P.M. Due to the absence of the clerk, the report of the previous meeting was omitted. The Chairman announced that the future meetings of the Section would be held on January 17, 1962 and May 16, 1962.

Chairman Wooldridge then introduced the speaker of the evening, Mr. U. M. Schiavone, Newton City Engineer, who spoke on "Conversion of a Former Metropolitan Water Works Aqueduct to the Use of a Gravity Flow Sewer." Mr. Schiavone was assisted in his presentation by Mr. Edmund Bolduc, who gave a running commentary on a film of the project.

After a brief question and answer period, the meeting was adjourned at 8:20 P.M.

The meeting was attended by 17 members and guests.

ROY L. WOOLDRIDGE, *Chairman*

ADDITIONS

Ronald E. Bucknam, University of Illinois, 92-101 W. Green St., Urbana, Ill.

Edward F. Delaney, Gov't Center Comm., 80 Boylston St., Boston 16, Mass.

Joseph A. Standley, Brask Engrg. Co., 177 State Street, Boston, Mass.

Walter M. Newman, 215 Newbury Street, Boston 16, Mass.

George M. Mathews, 114 Main St., Fairhaven, Mass.

JUNIORS

Daniel Eagan, Jr., 36 Clinton St., Clinton Hghts., Renssalaer, New York.

DEATHS

Wilbur S. Colby, September 17, 1961

Hugh P. Duffill, September 22, 1961

Albert E. Lochridge, October 3, 1961

John C. Moses, October 26, 1961

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