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(*See Page 1031*)

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FEDERATION OF SEWAGE WORKS ASSOCIATIONS

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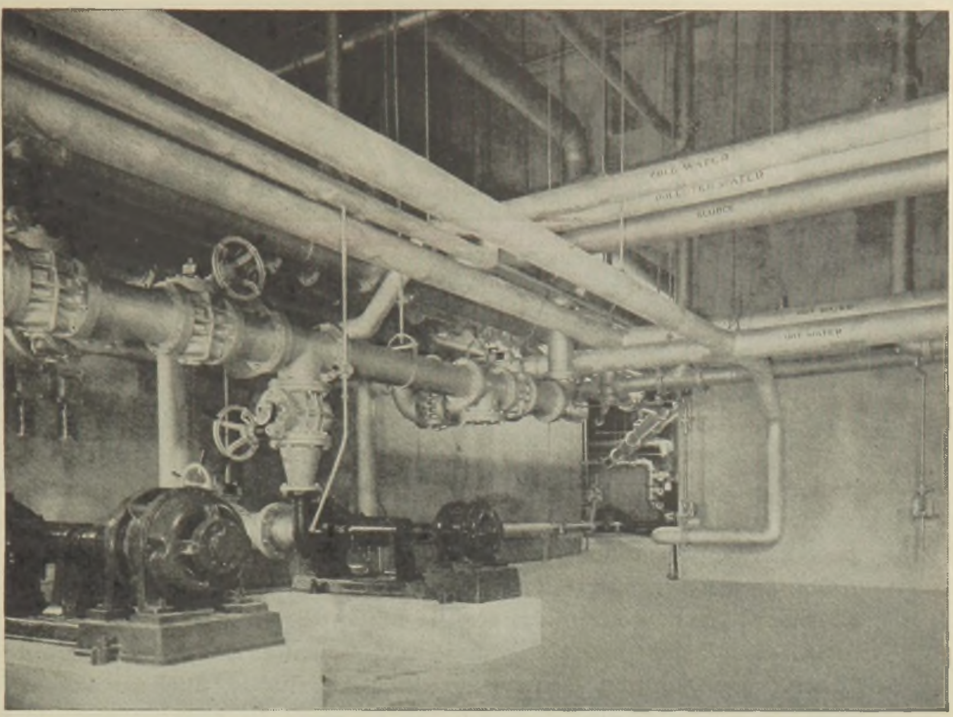

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SEWAGE TREATMENT AT MILITARY INSTALLATIONS

REPORT OF THE

SUBCOMMITTEE ON SEWAGE TREATMENT

of the

COMMITTEE ON SANITARY ENGINEERING

NATIONAL RESEARCH COUNCIL

DIVISION OF MEDICAL SCIENCE

WASHINGTON, D. C.

★

Subcommittee on Sewage Treatment in Military Installations

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May, 1946

This presentation of the basic data and the preliminary conclusions drawn by the Committee are issued as the first section of a study on the subject. Comment and constructive criticism will be welcomed and reviewed for incorporation in a final summary chapter to be presented in a future issue, with the objective of developing a helpful guide in this particular type of sewage treatment plant.

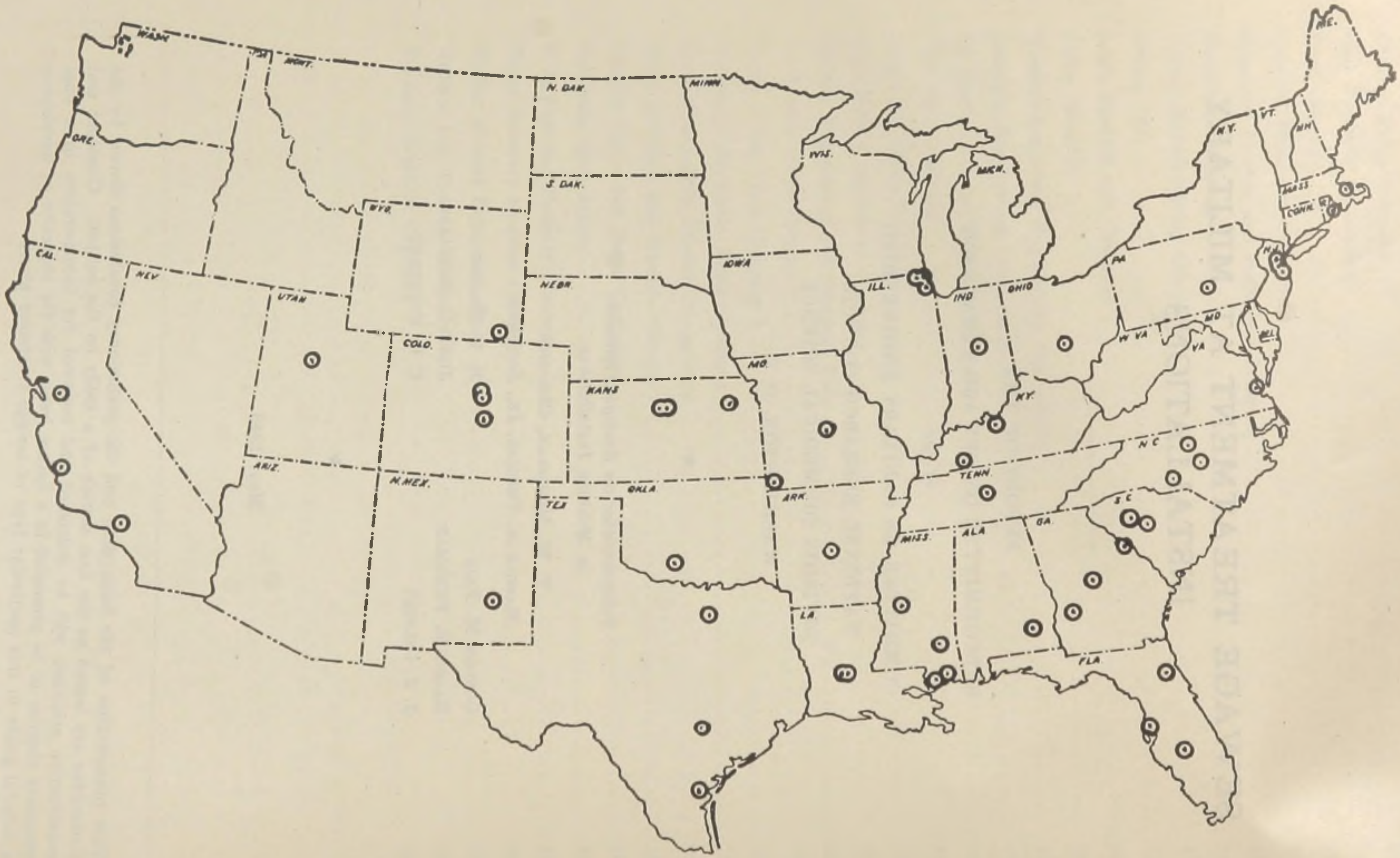

FIGURE 1.—Distribution of plants on survey list. Complete study groups only are shown. Numerous other plants contributing additional data are not indicated.

SEWAGE TREATMENT AT MILITARY INSTALLATIONS

PREFACE

The expansion of military installations in the United States under the exigencies of global war during the period 1940–44 brought an unprecedented opportunity to record operational experience with different processes of sewage treatment. In order to take advantage of the situation, the Committee on Sanitary Engineering of the National Research Council, Division of Medical Sciences, under the chairmanship of Dr. Abel Wolman, organized a Temporary Subcommittee on Sewage Treatment, with Dr. F. W. Mohlman as Chairman.

At the first meeting on May 22, 1944, the objectives of the Subcommittee were set forth as follows: (1) To conduct a critical study and review of design and operation of sewage treatment plants in military installations; (2) To establish useful norms to guide in the solution of future problems of a similar nature such as: (a) flow and character of sewage; (b) methods of control and analysis; (c) training and performance of operators. The investigation would be nationwide in scope and would be directed toward an evaluation of performance of effective sewage treatment processes under different climatic conditions, yet under unified control. The study would serve the future in the design of projected military and other government installations and in the postwar construction of municipal works.

It was believed that the situation in the field presented a favorable juncture of circumstances for the realization of the foregoing broad objectives. In full operation at that time were several hundred plants, all designed in accordance with a standardized code, all treating sewage of domestic character, and all using identical methods of analysis and control. Heretofore, surveys of similar scope have been precluded by the obscuring influence of varied industrial wastes and by the lack of uniform methods of analysis. Furthermore, as has been pointed out by many, operational data from sewage plants of World War I were extremely scarce; such paucity rendered difficult the problem of design during the recent emergency.

The subcommittee felt that the services of a special investigator should be secured to carry out the details of the study in its various ramifications. Dr. H. A. Thomas, Jr., Assistant Professor in Sanitary Engineering in the Harvard Graduate School of Engineering was selected to serve in this capacity. Also actively engaged in the detailed work of the investigation at Harvard were several officers of the Sanitary Corps under the supervision of Captain R. E. Heacox.

With regard to the mechanism of the investigation, it was decided that in view of available personnel and time the survey could best be conducted by means of a detailed questionnaire, which was sent through Army Service Commands and Naval Districts to individual installations for the purpose of obtaining specific information. In addition, many data relating to design were obtained from the Branch of Engineering and Development, Office of the Chief of Engineers, War Department, Washington, D. C. Much information from previous field investigations was made available to the subcommittee by the Office of the Surgeon General and the Division of Repairs and Utilities, Office of the Chief of Engineers.

With the main bulk of data assembled in the organized form of the questionnaire, members of the subcommittee visited a representative portion of plants on military installations, becoming acquainted with problems of operation in military establishments, and with operating personnel. The work of the subcommittee was resolved into an analysis having three phases, as follows: (1) detailed description of a number of plants of each type and size that could be considered characteristic of the program as a whole; (2) statistical treatment of the data collected in order to delineate characteristics of design and operational results; and (3) specific investigations of special topics of interest.

The survey, which was limited to Army and Navy posts in the continental United States, included new plants as well as modified existing installations.

In the matter of obtaining special data of unusual and important nature, the USPHS laboratory and personnel at Cincinnati under the direction of Mr. H. W. Streeter rendered valuable service. Special investigations relating to the biochemical oxygen demand of military sewage were conducted at the Washington, D. C. sewage treatment plant by Mr. R. E. Fuhrman and Mr. P. McNamee.

It is manifest that the success of a survey of this character is predicated upon the cooperation of officials from whom the basic data were obtained. It is desired to express the appreciation for the assistance and courtesies accorded the subcommittee by all these officials. It is not possible to make individual acknowledgment of everyone who contributed to the fulfillment of the objectives of the survey. The following, however, have substantially participated in the survey and merit personal recognition:

For the U. S. Army:

First Service Command:
Lt. Colonel R. N. Clark, SnC.
Dr. J. R. Snell

Second Service Command:
Major W. M. Culley, SnC.
Captain J. Bethel, CE.

Third Service Command:
Major E. S. Hopkins, SnC.
Major T. H. Stout, CE.
Mr. F. L. Vermette

Fourth Service Command:
Colonel M. J. Blew, SnC.
Major L. H. Scott, CE.
Mr. J. E. Kuruzo

Fifth Service Command:
Major F. D. Stewart, SnC.
Mr. Ray H. Fuller

Sixth Service Command:
Captain P. W. Reed, SnC.
Major A. J. Steffen, CE.

Seventh Service Command:
Major J. E. Kiker, SnC.
Captain C. Obma, CE.
Mr. J. E. Franks

Eighth Service Command:
Major C. W. Klassen, SnC.
Major R. M. Dixon, CE.

Ninth Service Command:
Lt. Colonel F. M. Stover, SnC.
Major R. Eliassen, CE.

For the U. S. Navy:

Area I—Lt. Cdr. K. M. Clark
Area II—Comdr. F. H. Dechant
Lt. Cdr. W. R. Benford
Area III—Lt. R. Hazen
Area IV—Lt. H. E. Heck
Area V—Capt. H. O. Lord
Area VI—Lt. Cdr. P. L. McLaughlin
Area VII—Lt. (jg) E. A. Bell

In addition to the foregoing officials it is the desire of the subcommittee to acknowledge particularly the cooperation extended by Col. W. A. Hardenbergh, Sanitary Corps, and Major B. A. Poole, Corps of Engineers, Washington, D. C., who have done much to facilitate the survey.

Finally, recognition should be accorded those officers of the Sanitary Corps who were assigned to the project at Cambridge: Lt. J. H. Butler, Lt. C.

Calmon, Capt. H. L. Dabney, Capt. J. A. Drake, Lt. C. A. Gibbons, Lt. W. G. Hamlin, Lt. H. G. Luley, Capt. R. F. Portman, Capt. J. A. Salvato and Capt. J. L. Sorbel. It was due to the diligent effort of these men in carrying forward the tedious work of statistical analysis that full realization of the objectives of the survey was possible.

The technical literature has been freely used, and much material has been obtained from the numerous reports. Authorship is credited at appropriate places in the body of the report. A list of abbreviations used in the report is given in Chapter I.

National Research Council, Committee on Sanitary Engineering

ABEL WOLMAN, *Chairman*
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This presentation of the basic data and the preliminary conclusions drawn by the Committee are issued as the first section of a study on the subject. Comment and constructive criticism will be welcomed and reviewed for incorporation in a final summary chapter to be presented in a future issue, with the objective of developing a helpful guide in this particular type of sewage treatment plant.

CHAPTER I

GENERAL CONSIDERATIONS

Sewage Treatment in World War I

Perspective as to the overall problem of treatment of military sewage may be had at the hand of a brief resume of sewage plants of World War I. Treatment at Army installations during 1917–18 varied from septic sedimentation at National Guard Camps to complete biological oxidation at certain aviation fields.

Sewer systems consisted of tile, cast iron and concrete pipe in sizes ranging from 4 to 30 inches in diameter. House connections were 4- to 6-inch terra cotta pipe. Capacities and grades were determined by maintenance of a minimum velocity of approximately 2¼ fps. Square concrete manholes with wooden covers at intervals of 350 feet were used on the sewer lines. In some instances it was necessary to provide locks for manhole covers due to dumping of garbage and refuse into sewers.

Design was predicated upon an envisaged maximum rate of flow of between 2.5 to 3 times the average flow. Observations at Camp Meade, Md. (36,590 men) in May, 1918 revealed an average ratio of maximum to average flow of 1.87 and a ratio of minimum to average flow of 0.26. Water consumption during May, 1918 averaged 55 gcd; sewage flow amounted to 82 per cent of this. Corresponding data for 23 aviation fields show per capita water and sewage flow figures ranging from 42 to 274 gallons daily. Sewage flow ranged from 34 to 166 percent of water consumption.

In general, camp sewage was found to be more concentrated than municipal sewage. Camp Meade sewage for example, had a suspended solids content of 276 ppm as based upon a 24-hour composite sample. Usually, aviation camp sewage was more dilute than that at other cantonments. In many instances the organic constituents were relatively much higher than in municipal sewage. Greeley and Chase (26), after a review of available information, reported that the following data were suggested for World War I sewage:

	ppm	lb/cap/day
BOD	460	0.27
Suspended solids	290	0.17
Ether-soluble matter	150	0.09

The treatment units that were utilized include single-story septic tanks, Imhoff tanks, trickling filters, contact beds, and intermittent sand filters. Bar screens, grease traps, detritus chambers, and sludge beds were commonly provided; grit chambers were also installed at a number of posts. Bar screens with clear spaces of 1 to 1½ inches were successful, and removed up to 6 to 10 cf per mg of screenings. Detritus, consisting largely of putrescible matter, was collected in amounts of about 6 to 8 cf per mg. Grease traps had a holding capacity of ⅔ gallon per capita and retained as high as 95 percent of the grease.

A special type of septic tank known as the ''construction division'' septic tank (or ''Doten'' tank) was used to a considerable extent. It was a horizontal flow, multiple-compartment tank. Hopper bottoms in each compartment were equipped with sludge drawn-off pipes. Flow from one compartment to the next was of the ''under and over'' pattern, around appropriately placed baffles. The original design capacity of the ''construction division'' septic tank was 10 gallons per capita with an expected minimum detention period of 2 hours. It was discovered, however, that rapid accumulation of scum materially reduced the detention period and necessitated frequent removal of malodorous sludge. As a result of a number of complaints and investigations, the design allowance was increased to 20 gallons per capita. Subsequent operation was improved, in some instances, with addition of grease traps, screens and detritus tanks. On the whole, however, the "construction division" septic tank was generally unsatisfactory. Separation of sludge proceeded on a "flotation" rather than on a "sedimentation" principle. Sludge was voluminous, greasy, undigested, and slow drying.

A considerable number of Imhoff tanks were used, particularly at aviation camps. These were efficient and gave relatively little operating trouble. Sludge obtained from them was well digested and dried rapidly.

Sludge beds were designed on a basis of 0.5 sf per capita. This frequently proved to be insufficient, especially for septic tank sludge; often it was necessary to resort to disposal by burial.

Trickling filters, as designed by the construction division, had areas based on 30,000 persons per acre. Depths were of the order of 5 to 6 ft. Filtering material was "hard, durable, crushed rock or slag" capable of passing a screen with 2½-inch openings. Flow distribution was accomplished by means of fixed nozzles. These units proved to be generally satisfactory. A high degree of nitrification in the effluent was the rule. Not all trickling filter plants were provided with final settling tanks.

Intermittent sand filters were used at three installations. These were: Camp Devens, 20 acres; Camp Upton, 22 acres; and Camp Mills, 14 acres. Some difficulty was experienced from clogging of these units when loaded to capacity, due to excessive amounts of grease.

Both automatic and manually controlled chlorination units were used. Chlorine was employed with varying amounts of success as a substitute for secondary treatment. Dosages of 6 to 10 ppm were common. The automatic chlorinators were often unsatisfactory, and manually controlled machines were generally favored.

Administration of the Army Sewage Treatment Program in World War II

It is pertinent to set forth concisely the structure and responsibilities of the organizations connected with the program of sewage treatment at military installations. These subdivisions of the Army Service Forces were, respectively, the Corps of Engineers and the Sanitary Corps. In order to delimit succinctly the points of tangency as to function, it is useful to consider the normal field of operation of each as regards administration and control of utilities at Army posts.

On December 16, 1941, by an Act of Congress, the Office of the Chief of Engineers took over the Construction Division of the Office of the Quartermaster General. Under AR 100–70 and change No. 1 thereto, and AR 100–80 and changes 1 to 7 inclusive thereto, the Chief of Engineers, under the authority of The Commanding General, Army Service Forces, and the Secretary of War, was charged with the direction of all work pertaining to the design and construction of building, structures, and utilities for the Army (27). The work included:

(a) Design and initial erection or installation of any building, structure, plant, ground facility, utility system, or other real property for the several agencies of the Army;

(b) Material additions to or material alteration of any existing structure or facility;
(c) Repairs to any existing structure or facility.
(d) Preparation of plans and estimates, budgeting and allocation of funds, insurance of directives and provisions of means for the construction, and
(e) Setting up of criteria of construction, preparation of technical manuals and guide specifications for various component parts of the work.

The program of Military Construction of the Office of the Chief of Engineers was carried out through three principal echelons:

(1) The Director of Military Construction, Office of the Chief of Engineers, Washington, D. C.
(2) The Offices of the Division Engineers in the several Divisions, and
(3) The Offices of the District Engineers in the Districts of each Division.

The Director of Military Construction was responsible for the initiation, planning, design and construction of sanitary sewers and sewage treatment plants in connection with military engineering projects under the jurisdiction of the Chief of Engineers. These duties and responsibilities were carried out by the three echelons in a certain predetermined manner as stated in the following paragraphs.

The District Engineers had the authority to initiate projects of a certain magnitude upon the approval of the Division Engineer and the Director of Military Construction, Office of the Chief of Engineers. The initiation of the sanitary sewerage projects generally took place upon requests or recommendations of the Army Air Forces, Army Ground Forces, or Army Service Forces. The District Engineers (1) made the necessary field surveys and investigations, (2) determined the necessity for the proposed improvements, (3) prepared preliminary and final estimates, plans, and specifications either by their office forces or through outside Architect-Engineers in accordance with the technical manual and the guide specifications issued by the Office of the Chief of Engineers. After the approval of the project by the Division Engineer and the Office of the Chief of Engineers and the allocation of funds, the District Engineer invited bids on the project and, after award thereof, was responsible for the construction either directly or through the Post Engineers. The District Engineers were responsible to the Division Engineers for the work performed in their respective Districts.

The Division Engineers had the authority to initiate projects of a greater magnitude than the District Engineers. The general scheme of planning and execution of the projects by the Division Engineer was patterned after the lines pursued by the District Engineer. Under the general supervision of the Division Engineer the projects initiated in his office were carried out by the District Engineers in the Division.

The Director of Military Construction, Office of the Chief of Engineers (1) passed upon the overall necessity for the sanitary sewerage project upon the basis of reports and justifications submitted by the District and Division Engineers, (2) reviewed the various projects submitted by the District and Division Engineers, (3) authorized the allocation of funds for the project, (4) issued directives to proceed with the work and (5) exercised overall authority in connection with administration, planning and execution of the work.

Repairs and Utilities Division

The Chief of Engineers, under AR 100–80, was responsible for direction of operation and maintenance of Army

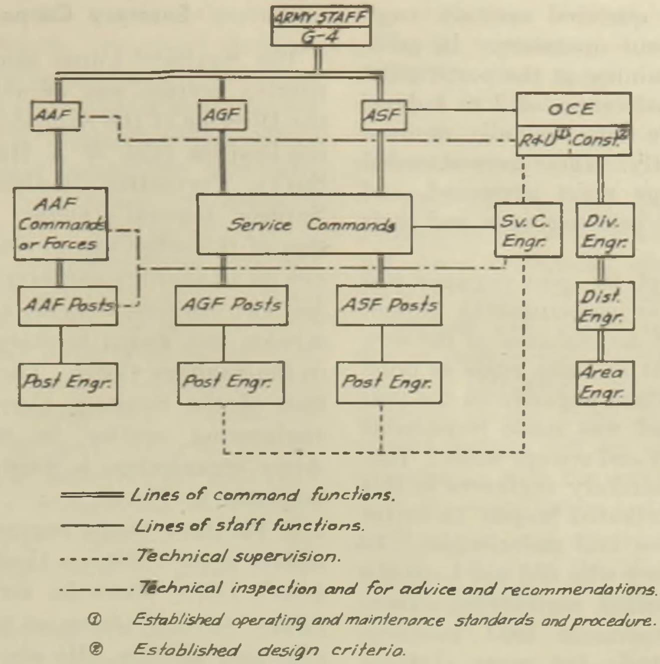


FIGURE 2.—Organization chart showing relationship of the Office of the Chief of Engineers to other subdivisions of the Army with respect to utilities.

utilities, including sewage works. This function was performed by the Repairs and Utilities Division. Field supervision and actual accomplishment were carried out through the staff organizations with the relationship shown in Fig. 2.

The Service Commands were responsible for supervision of sewage works at all Service Force and Ground Force posts in their commands, and provided technical inspection and recommendations for Air Force posts within the Service Command boundaries. The Air Force Commands were responsible for effecting recommendations so made. Each Service Command Engineer's office had a group of sanitary engineers in the Water and Sewage Section. These engineers made periodic inspections of all posts to determine adequacy of sewage collection and treatment, assist and train sewage plant operators, effect improvements in operating and maintenance practices, and recommend minor changes in plant facilities. In addition, they checked (1) operating costs, personnel required, and their salary, grades and qualifications, (2) laboratory procedures and operating reports and (3) inventory control of operating and maintenance supplies. Special visits to posts were made (a) to study stream conditions and plant operation, (b) determine need for major alterations to plants, (c) assist post engineers in design of alterations and (d) inspect such work during construction. In the office they (1) established operating budgets for each post, (2) reviewed operating reports and (3) after engineering review of project requests for alterations, additions or major repairs, recommended allotment of funds.

The recruitment and training of post operating personnel was a major function. Aid was given to post engineers in obtaining qualified sanitary engineers and plant operators. In addition to the training at the posts, short courses or conferences of 2 to 4 days' duration were held annually or even more frequently. These were attended by key sewage plant personnel, and frequently by post engineers and their assistants.

At post level, the post engineer was responsible for accomplishing proper operation and maintenance of the sewage works. At as many posts as practicable, a sanitary engineer on the post engineer's staff was made responsible for both water and sewage works. Employment of sanitary engineers in this capacity contributed largely to better plant operation and maintenance. In some cases, men who did not have the usual professional engineering education and experience held positions under this grade, but many of these had had extended plant operating experience.

Sanitary Corps

The Sanitary Corps sanitary engineering service was organized under the Director of the Sanitary Engineering Division (Col. W. A. Hardenbergh, SnC.), Preventive Medicine Service, Surgeon General's Office. The function of this office was to establish policies on all sanitary engineering matters for The Surgeon General and to commission and assign sanitary engineers in the Sanitary Corps. The incorporation of the Sanitary Corps sanitary engineering service in the overall Army organization is illustrated schematically in Fig. 3.

A Sanitary Corps engineer was assigned to each Service Command Surgeon's Office where he acted as consultant to the Surgeon on sanitary engineering matters. He also was in direct technical command of the sanitary engineers at the posts, camps and sta-

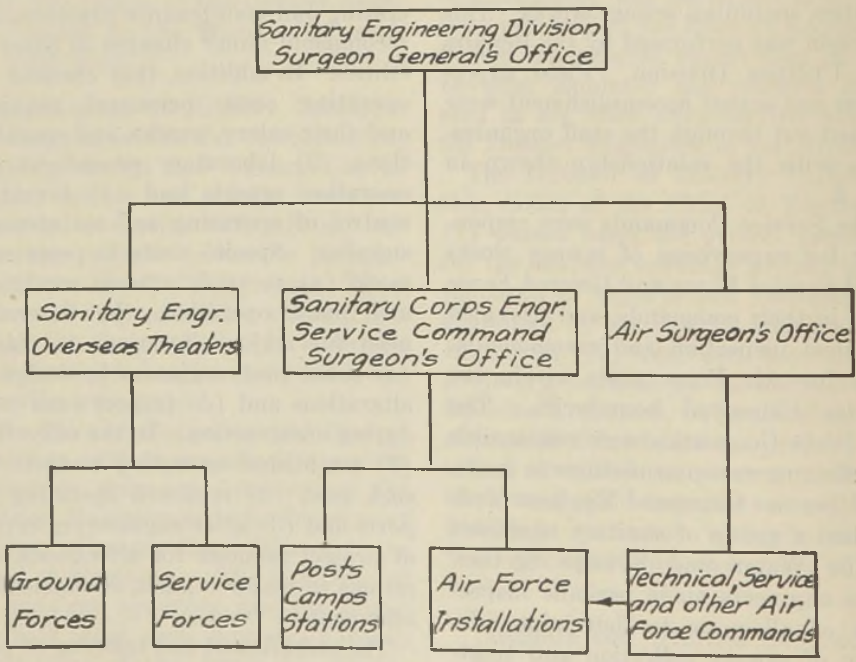


FIGURE 3.—Organization chart showing relation of the Sanitary Corps to other subdivisions of the Army.

tions within the Service Command. Since the Service Command Surgeon also was responsible for the sanitation at Air Corps installations within his command, the Sanitary Corps officer made periodic inspections and consulted with the surgeons and sanitary engineers at Air Corps installations.

At posts, camps, stations and airfields the Sanitary Corps sanitary engineer was assigned to the post surgeon's office and was usually the sanitary officer for the post. The sanitary officer's duties brought him in close contact with the post engineer and at many bases he acted for the post engineer in the training of the sewage treatment plant operator and in the technical supervision of plant operation.

In overseas theaters Sanitary Corps engineers were assigned to both the Service Forces and the Ground Forces where their duties were comparable to those in a similar assignment within the continental United States.

The Air Surgeon's Office of the Army Air Forces, through the various technical service and other air force commands, assigned sanitary engineers to the various air force installations. These officers were guided by the Sanitary Engineering Division through Service Command sanitary engineers.

In general, the functions in relation to sewerage and stream pollution of the Medical Department and Sanitary Corps engineers were established by Par. 18, AR 40–205, as follows:

"a. Sewage treatment.

"(1) The medical officer responsible for the sanitary supervision of a station or command will keep himself informed regarding the public health laws of the State or municipality where in such station or command is maintained. He will assure compliance with these laws so far as is possible by recommendations to proper authority in regard to necessary changes in operation or installation, reconstruction or replacement of the sewage treatment plant.

"(2) The Medical Department is responsible for investigating and making recommendations concerning the general types of sewage treatment equipment and appliances and their design factors with reference to their sanitary suitability before their final adoption by the Army. Investigations of this nature will be undertaken in cooperation with the service concerned with the installation and operation of such plants and appliances and said service will keep the Medical Department informed of proposed installations.

"b. Analysis of sewage.—While the Corps of Engineers is charged with the operation of all sewerage systems and sewage disposal plants in the Army, nevertheless, medical officers will familiarize themselves with the rudiments of sewage disposal and with the regulations governing the performance and interpretation of tests of sewage treatment at military stations. Where treatment plants are installed, medical officers will have sanitary supervision over the tests for Settleable Solids and for the Relative Stability of the effluent. When no competent Corps of Engineers attendant is available to make such tests, they will be made by a medical officer or a competent person designated by him. In case extended and detailed laboratory surveys are required, recommendations outlining the necessity for such investigation and request for same will be submitted to the commanding general of service command or department or other higher authority."

General Considerations Relating to Sewage Treatment at Military Posts

The War Department in October, 1940, in the face of great national urgency, undertook the largest construction program ever attempted in a short space of time. The need for cantonments was precipitated by the passage of the Selective Service Act of 1940. Architect-Engineers in private practice were retained immediately by the Construction Division, which at that time was under the Quartermaster General. The functions of the Architect-Engineers were to design and supervise the construction of military installations. Many of the more difficult problems that had to be solved at this time related to post utilities—water, sewerage, garbage disposal, and sewage treatment.

Shortly after commencement of construction it became evident that Architect-Engineers had divergent conceptions as to the degree of sewage treatment required and methods of obtaining it. Accordingly, the consulting firms of Metcalf and Eddy, Boston, and Greeley and Hansen, Chicago, were selected to make a study of a representative group of projects and to advise as to the simplest form of sewage treatment adequate to meet local conditions. The decision to establish such an "Engineering Board" of these two firms was a fortunate one; many of the more vexing problems were resolved by establishment of overall policy.

Policy of the Engineering Board

The Engineering Board prepared a report in which were embodied: (i) a review of special problems of Army sewage disposal; (ii) a best available estimate of sewage quantities and characteristics; and (iii) recommendations for appropriate loadings of sewage treatment devices.

In developing the report (References 19 and 21) the Engineering Board was guided by the following considerations: (1) security to the public health; (2) sufficient treatment to preclude serious nuisance; (3) economy; and (4) speed of design and construction.

In carrying out their instructions the Engineering Board stated that the following factors were taken into account: (a) character of the receiving body of water; (b) environment of both treatment plant and camp; (c) degree of permanence envisaged; (d) need for utmost speed of design and construction; (e) safeguarding of health of troops and nearby civilians; and (f) reasonable economy of construction.

Security to Public Health

The Engineering Board took the position that security to public health was the paramount consideration upon which there could be no compromise. In this matter initial protection stemmed from the use of separate sanitary sewers. All sewage containing infectious material was passed through treatment works. While treatment works had by-passes, these were not used except during unusual emergencies. Safeguards in this respect, the Board stated, would be (1) the adoption, wherever possible, of replicate units, both primary and secondary and (2) the use of heavy doses of chlorine while by-passing.

Public health, it was found, may be affected by any one or more of the following uses of the receiving water: (1) for public water supply; (2) for bathing and boating; (3) for maneuvers, including fording, bridge building, etc.; (4) for shellfish laying; and (5) for watering of dairy cattle. In some cases it was anticipated that it would be impossible to locate the outfall sewer at a sufficient distance from bathing and recreational areas so that self-purification could be relied upon. In these instances, it was conceived, chlorination of the effluent would be adequate to protect public health; however, in certain plants a high degree of BOD reduction would be indicated. With regard to maneuvers it was believed that all reasonable requirements could be met by an effluent that did not look bad and that also was adequately chlorinated. As to the protection of shellfish, it was stated that any effluent without objectionable odors and not unsightly may be made harmless to the layings by adequate chlorination.

Prevention of Serious Nuisance

The Board interpreted the term "serious nuisance" to connote the permissibility of "some" nuisance. It was assumed that some discoloration, some turbidity, and occasional odor would not be serious. The situation would vary, it was foreseen, depending upon the location of the outfall. In any event, the application of chlorine to the final effluent would provide considerable latitude of control over nuisance. Proper chlorination of the effluent would retard the putrefactive decomposition processes and permit maintenance of non-odorous aerobic stabilization. While the consulting engineers recognized that cases would inevitably occur that demanded a high degree of treatment, emphasis was placed upon the minimum initial expenditure that would meet the needs of the situation.

Treatment Devices

Measurement of sewage volume was strongly advocated. The use of Parshall and Palmer-Bowlus flumes, but not Venturi meters, was suggested. Since an important factor was speed, design was to be directed toward the simplification of construction and reduction of material quantities to a minimum. In order to attain this objective, any devices entailing elaborate reinforced concrete were not encouraged. In line with this policy, sedimentation tanks with mechanical removal of sludge were preferred to Imhoff tanks because of the complicated concrete work involved. Mechanical screens were discouraged because of rather intricate concrete construction required; also, the cost of these units militated against widespread adoption. Trickling filtration was preferred to the activated sludge process because of relative simplicity. High-rate filters were preferred over standard trickling filters because of the smaller amounts of materials inherent in their construction.

The Board recommended that when minimum average monthly flow was more than 4 cfs per 1,000 population, screening, sedimentation, and chlorination would be sufficient. For flows less than this amount, biological treatment was recommended. High-rate trickling filters with final chlorination were regarded as adequate even where the dilution was at times reduced to zero, unless there existed special local conditions requiring a higher degree of treatment. Chemical precipitation was not approved because of operating cost and possible lack of chemicals in a wartime economy. Untried processes or those with which experience was limited did not receive approbation.

Economy

The Engineering Board stressed three factors relating to economy:

1. A desire to expend the least *total* money necessary during the existence of the camps to protect the public health and otherwise obtain acceptable sewage disposal;

2. Recognition of the fact that relatively high *operating* costs may be incurred for the short period during which the camps may be used, as a means of minimizing *total* costs;

3. Recognition of the fact that sewage disposal is but a small factor con-

FIGURE 4.—Typical primary treatment plant, Pine Camp, New York.

FIGURE 5.—Large trickling filter plant, Camp Edwards, Mass. Units in foreground are sand beds that completely absorb effluent into ground.

tributing to the success of the overall war effort.

In line with the foregoing considerations, buildings were ordinarily to be made of wood and of simple design. Only in case of serious fire hazard was brick construction utilized. The Engineering Board favored relatively high loadings of sewage treatment devices such as the high-rate filter, with recirculation of the effluent. Final polishing treatment units were not advocated.

Policy of War Department Relating to Sewage Treatment

The Engineering Board transmitted its report in December, 1940. The recommendations appeared as a part of a technical bulletin dated April 23, 1941, by the Engineering Branch, Construction Division, Office of the Quartermaster General. The bulletin was intended primarily as a guide to Architect-Engineers engaged in design and construction of military posts. A number of changes appeared in successive editions of the bulletin, particularly after the Office of the Chief of Engineers took over the Construction Division of the Office of the Quartermaster General. At this time a substantial amount of information began to be available from the field. In June, 1941, publication in loose-leaf form of the Repairs and Utilities Manual began. This manual facilitated the work of the operator. In this report it is referred to as the "Engineering Manual."

Army policy with regard to sewage treatment evolved over a period of several months. Progressive adjustments were made, and by early 1942 broad outlines of policy had emerged in essentially their final form. Subsequent changes related to matters of detail and interpretation. Salient phases of Army policy are presented in the following paragraphs.

Type and Degree of Treatment

The Engineering Manual states: "The degree of treatment required will depend upon the amount of diluting water in the receiving stream during periods of minimum flow, the condition of the stream as regards pollution, and the use of the stream below the proposed point of discharge. Consideration should also be given to local practice with regard to the degree of treatment being provided by existing plants, and the anticipated occupancy of the project." As to partial treatment, the Manual states: "Where the effluent from the plant is to be discharged into a relatively large stream with a flow sufficient to provide a minimum dilution of about 4 cfs of stream flow per 1,000 population, sedimentation tanks with chlorination will usually be sufficient. Such partial degree of treatment may be suitable in cases where the effluent is to be discharged into tidal waters, or finally disposed by irrigation on land." For posts requiring additional treatment, the Manual states: "If a greater degree of treatment is required, consideration should be given to operating requirements, and the simplest type of plant should be used that will give the desired results. Where more than partial but less than complete treatment is required, single-stage high-capacity filters can be used to good advantage. Treatment by the activated sludge process, standard-rate trickling filters, or multi-stage high-capacity filters will provide complete treatment where required." Parenthetically it should be stated that no mention was made in the Manual of the effect of the degree of freshness of sewage upon selection of the type of treatment. Results with certain plants showing substandard performance point up the advisability of making sewage conditions one of the criteria upon which the selection of type of treatment should be based.

Critical Material Shortages

Current policies of the War Department with respect to the conservation of critical materials were to be strictly observed. In general this implied minimum use of mechanical equipment, reinforcing steel, and other similar items. Where filter stone could be secured at reasonable cost, standard filters, which required a minimum amount of mechanical equipment, were to be given preference over high-rate filters. It is to be noted that in this particular the Corps of Engineers' policy deviated from the original recommendations by the Engineering Board. The Manual states: "The use of high-rate filters . . . should be confined to localities where stone is not readily available or where space limitations and other factors require the use of such processes." Every effort was to be made to utilize equipment which could be supplied by more than one manufacturer. This facilitated delivery. Architect-Engineers were advised to investigate the ability of equipment manufacturers to make delivery within the required time.

Relations with State Health Departments

Instructions were issued to the effect that approval of all state and federal agencies should be obtained whenever required. Plants were to be designed to meet minimum requirements of local and state health departments. To this end the state sanitary engineer or director of the state department of health was to be consulted. With but a few exceptions, it was found that the dictates of good practice from a military viewpoint were in accord with state requirements. Some difficulty, however, was encountered in correlating designs of different plants in the same service command because of *slight* differences in rules set forth by various contiguous states. Such slight differences related, for example, to minimum sewer velocities.

Site Selection for Sewage Treatment Plants

Army policy was to locate plants as far as practicable from buildings, private dwellings, and traveled ways. Consideration also was to be given to the direction of the prevailing wind. Within limits set by topography and location of the receiving stream, plants were to be situated where they would not interfere with military operations and would be at a safe distance from artillery ranges. Sites, if possible, were to be located above high water; otherwise flood protection was to be provided. As it developed later, however, sites were not invariably selected with requisite attention to level of flood waters. It was necessary to install a considerable number of plants near streams for which hydrographic data were lacking. In a few cases serious operating difficulties were brought about as the result of frequent inundation. In all such instances it was possible to rectify the situation by installation of levees and other flood control devices.

Personnel

The importance of adequate operating personnel as a factor conducive to successful plant performance was recognized from the outset of the program. The policy was established of placing sanitary engineers in control of post water and sewage utilities. Officers and enlisted men were needed for military purposes, and military personnel at camps was likely to be changing as a progression of military units moved in and out of the posts. In order to preserve continuity of operation of the utilities, therefore, it became evident that sewage plants should be staffed with competent civilian personnel.

Because of the manpower demands on skilled labor and technicians concomitant with the war effort, it was found that the field from which experienced operators could be drawn was very much restricted. Civil Service requirements and the uncertainty as to the length of employment often made it difficult to attract men with requisite knowledge of the art of sewage treatment.

Certainly in the initial phases of the program operation was not all that could have been desired. Frequently no provision was made for operation until the cantonment had been completed. It was common procedure to assign operation of both water works and sewage treatment plants to the post detachment, which more often than not had no competent persons available. As a result, Architect-Engineers were themselves frequently obliged to devote a considerable amount of attention to plant operation and training of operators. In many cases strong representation was necessary in order to obtain competent operators.

Gradually, however, the personnel difficulties were resolved. An important factor in this connection was the program of operator instruction carried forward simultaneously in various service commands. In addition there were state-sponsored short courses, and "defense courses" offered by a number of universities. Criticism of plant performance was greatly reduced as the effect of the education program became manifest.

TABLE 1.—Salary Scale for Civilian Personnel for Sewage Disposal at Army Posts

Section	Designation	Class	Base Pay	Contributory Population							
				38,000 and over		12,000 to 38,000		6,000 to 12,000		1,500 to 6,000	
				No.	Annual Cost	No.	Annual Cost	No.	Annual Cost	No.	Annual Cost
Sewage and Water Supv.	Sanitary Engineer	P-4	$3,800	1	$3,800	—	—	—	—	—	—
	Assoc. Sanitary Engineer	P-3	3,200	—	—	1	$3,200	—	—	—	—
	Asst. Sanitary Engineer	P-2	2,600	—	—	—	—	1	$2,600	1	$2,600
	Class "A"										
Sewage Pumping and Treatment Plants	Asst. San. Engr.	P-2	2,600	1	2,600	1	2,600	—	—	—	—
	or Chief Oper. Engr.	Uncl.	2,600	—	—	—	—	—	—	—	—
	Jr. Chemist	P-1	2,000	1	2,000	1	2,000	—	—	—	—
	Oper. Engrs.	Uncl.	1,680	4	6,720	4	6,720	—	—	—	—
	Class "B"										
	Asst. San. Engr.	P-2	2,600	1	2,600	1	2,600	1	2,600	—	—
	or Chief Oper. Engr.	Uncl.	2,600	—	—	—	—	—	—	—	—
	Oper. Engrs.	Uncl.	1,680	4	6,720	4	6,720	4	6,720	—	—
	Class "C"										
	Principal Oper. Engr.	Uncl.	2,000	—	—	—	—	1	2,000	1	2,000
	or Jr. San. Engr.	P-1	2,000	—	—	—	—	—	—	—	—
	Jr. Oper. Engr.	Uncl.	1,500	—	—	—	—	4	6,000	4	6,000
	Class "D"										
	Oper. Engr.	Uncl.	1,680	—	—	—	—	1	1,680	1	1,680
	Sewage Disp. Plant Worker	Uncl.	1,320	—	—	—	—	1	1,320	1	1,320

Classification of Sewage Treatment Plants

Sewage Treatment Plant Types	Classification			
Post of Contributory Population	38,000 and over	12,000 to 38,000	6,000 to 12,000	1,500 to 6,000
Complete Treatment with Separate Sludge Digesters	A	A	B	C
Primary Tanks and Separate Digesters-Primary Treatment Only	A	B	C	C
Doten or Imhoff Tanks with Trickling Filter	B	B	C	D
Doten or Imhoff Tanks Only or Sand Filters Only	—	—	D	D

Salaries

The salary scale of operators and helpers suggested by the Washington office of the Division of Repairs and Utilities (Table 1) generally compared favorably with that prevailing in municipal practice. On the other hand, less attractive living conditions adjacent to military posts tended to offset salary differentials. On the whole, the turnover in civilian operators as a result of higher pay in industrial war projects was less than might have been expected. The policy was adopted of rewarding the deserving operators with progressively more important duties as larger plants were completed. This policy, together with that of providing technical libraries at each post, proved to have a salutary effect. It was found that many of the operating personnel would make a strenuous attempt at self-education in order to better equip themselves for the work. As a result, a significant improvement in plant performance on a nation-wide scale was in evidence during the years 1942 and 1943. The records would indicate that the time required to break in the average untrained operator was of the order of 12 to 24 months, although this varied widely with circumstances. It was clearly evident that a significantly shorter period of time was required to master trickling filter plant operation than was necessary in activated sludge and contact aeration plant operation.

Design Capacities of Treatment Devices

Loadings of various units are presented in outline form. Values given are from the Engineering Manual as revised in January, 1943.

A. *Average Flow* (Airfields, Camps, and Cantonments)

1. Permanent posts—100 gcd
2. Mobilization type—70 gcd
3. Theater of Operation type—50 gcd

Average 16-hour flow taken as 125 percent of average 24-hour rate

Maximum 4-hour average rate taken as 175 percent of average 24-hour rate

Minimum 4-hour average rate taken as 40 percent of average 24-hour rate

Extreme peak flows taken as 300 percent of average 24-hour rate

B. *Characteristics of Sewage*

1. Suspended solids—0.27 lb/cap/day
2. BOD (5-day)—0.20 lb/cap/day
3. Grease—0.09 lb/cap/day

Ordinarily sewage is expected to be relatively fresh and has good settling qualities; settled sewage is expected to be amenable to treatment by biological treatment processes. 0.2 lb per cap per day and 70 gal per cap per day are equivalent to sewage a concentration of 343 ppm BOD.

C. *Design Capacity*

1. *Design capacity = authorized population × capacity factor*
2. Capacity Factor Table:

Authorized Population	Capacity Factor
less than 10,000	2.00
less than 20,000	1.50
less than 30,000	1.25
less than 40,000	1.10
greater than 50,000	1.00

3. Purpose of capacity factor—to provide for the following:
   a. Reasonable increases in population that may occur after construction.
   b. Variations in sewage flows and uncertainties as to actual sewage quantities.
   c. Unusual peak flows (to prevent extremely short detention periods in smaller units).

d. A sufficient number of units to promote flexibility of operation and to permit treatment with any individual plant unit out of service. Accomplished (in plants with populations greater than 5,000) by providing the required capacity for settling, filters, etc., in at least two units each, and arranging the piping so that any individual unit can be taken out of service.

D. *Hydraulic Design*

1. Pipes, channels, hydraulic distributors, weirs, comminutors, and flumes designed for peak flows 300 percent of 24-hour average flow.
2. Capacity of various units designed for 16-hour average flow rate equal to 125 percent of 24-hour average flow rate.
3. Detention periods to be adequate for average 4-hour flows equal to 175 percent of 24-hour average flow; however, in general, detention periods designed on 24-hour average flow.

E. *Screens and Comminutors*

Screens: hand cleaned—1 in. to 1½ in.
mechanically cleaned—⅝ in. to 1 in.

Screens ordinarily placed in comminutor by-pass.

F. *Grit Chambers* (where needed)

1. Controlled velocity of 1 fps
2. Detention period of 30 to 45 sec
3. Grit to be buried.

G. *Measuring Devices*

Parshall flume; other rectangular flumes; Venturi meter; weirs (rectangular and V-notch); Kennison nozzle.

H. *Primary Sedimentation Tanks*

Type of Treatment	Detention Period (hours) Based on 24-hour Average Flow
Primary (no recirculation)	2.5
Standard trickling filter (including recirculation)	2.5
High-capacity filter (including recirculation)	2.5
Activated sludge (including recirculation)	1.5
Contact aeration (including recirculation)	2.5

I. *Trickling Filters*

In determining the amount of BOD applied to filters, the amount of BOD of the raw sewage assumed to be reduced by 35 percent in passage through primary sedimentation tanks.

1. Standard trickling filter (complete treatment): 600 lb of applied BOD per acre ft per day (for filters not greater than 6 ft deep).
2. High-capacity trickling filter (intermediate treatment): loading to depend upon degree of treatment selected, but not to exceed 3,000 lb of applied BOD per acre ft per day (for filters not greater than 6 ft deep).

Note: "The development of original designs to overcome patented processes or devices is, in general, considered undesirable . . . it is suggested . . . that manufacturers' recommendations on the required equipment and processes be followed. This will make it possible to fix responsibilities and require guarantees covered by performance bonds."

J. *Activated Sludge*

Type	Hours' Detention*
Diffused air	8.0
Mechanical aeration	12.0

* Allow 25 percent for return sludge.

Air requirements have often been in excess of 1½ cf of air per gal of sewage treated.

Performance improved by (a) recirculation of effluent and (b) reduction of detention period in primary sedimentation tanks.

K. *Contact Aeration*

1. Design of tanks: Provision for frequent sludge removal, ready access to air grids, contact plates, and sludge-removal equipment for cleaning and repairs.
2. Capacity of tanks: 156 to 175 sf of contact media surface per lb of applied BOD per day. Spacing of plates: 1½ in.
3. Air requirements: Not less than 1½ cf per gal of sewage treated for two-stage process.
4. Performance improved by: (a) recirculation of effluent; (b) corrugated plates; (c) suspension of air grid from downpipes so as to facilitate cleaning.

L. *Final Sedimentation Tanks* (required for trickling filter, activated sludge, and contact aeration processes).

1. Detention period (based on 24-hour average flow): 2.5 hours, including any recirculation of sewage or sludge.
2. Final tanks to have foregoing detention period in addition to any intermediate sedimentation between stages of biological treatment.
3. Depth of tank: 8 or 10 ft. Overflow rates less than 800 gal per day per sf of tank area (based on average daily rate).

M. *Sludge Digestion Tanks*

1. Capacities:

Type of Treatment	Capacity (cf/cap) Heated	Unheated
Primary	2.0	3.0
Standard filter	3.0	4.5
High-capacity filter	3.0	4.5
Activated sludge	4.0	6.0
Contact aeration	3.0	4.5

2. Operation of digesters in (a) series and (b) parallel.

N. *Sludge Disposal*

1. Digested and air drying on open sand beds.
2. Design of sludge beds:
   (a) 1.0 sf per cap—underdrains provided.
   (b) 2.0 to 3.0 sf per cap—without underdrains.
3. Wet sludge disposal to outlying areas by means of tank truck.

O. *Imhoff Tanks*

1. Detention periods in flowing-through compartment as specified in Part H. Displacement velocity less than 1 ft per min at *peak* flows.
2. Sludge compartment to have same capacity as unheated separate sludge-digestion tank (Part M). Sludge capacity computed from a line 18 in. below slot.

P. *Chlorination*

1. Purposes: odor control (prechlorination) and reduction of bacteria (and BOD) in effluent (postchlorination).
2. Chlorine contact tank; 15-min detention at 4-hr rate of peak flow; baffles to insure adequate mixing. Sufficient contact may in some cases be provided in the outfall line.

Q. *Oxidation Ponds* (not included in Manual, but representative of practice)

Method of secondary treatment following primary sedimentation.

(a) Design capacity: 400 to 700 persons per acre; 25 to 50 lb BOD per acre ft per day; 20 days' detention; depth 2 to 3 ft.

(b) Arrangement: ponds operated in series or parallel. Provision should be made for flexibility of operation as regards flow pattern (distributed load, etc.).

Analysis of Data

Four aspects of sewage treatment may be distinguished, namely, *design, loading, operation,* and *performance.* It is useful in analysis to envisage *performance* as a function of *loading,* with the functional relationship depending upon *design* and modified by *operation.* The *design* of a plant is considered to connote the physical dimensions, number, and arrangement of units. *Loading* involves the flow, strength, and condition of the influent sewage. *Operation* is construed to mean the manipulation on the part of the operating staff of those variables within their control. For example, regulation of volume of recirculated flow and number of times daily that sludge is pumped to digesters are items that fall in the domain of operation. *Performance* represents the quality of the plant effluent, whether considered in the absolute sense or relative to the raw sewage (percent removal).

It is pertinent to consider sewage plant operational data as belonging to two categories as follows: (a) data that relate directly to performance and efficiency (or example, *average* volumetric dosage of a filter in mgad); and (b) data that relate indirectly to performance and that may be used to explain deviations from the norm (for example, *maximum instantaneous* volumetric dosage of a filter in mgad). From a methodological viewpoint, the first, (a) is used in analysis, and the second, (b) in interpretation of variations from expected performance.

Limitations of the Survey

Insufficient data existed to permit an evaluation of the effect of the sewage treatment plant effluent upon the receiving water. While it was the general practice at military installations to perform periodically certain routine analyses upon samples from the receiving stream (DO, BOD, etc.), these were inadequate for the purpose of making a complete and integrated study. In particular, data regarding the amount of available dilution were found to be almost entirely lacking. Consequently the survey had to be limited to the problem of sewage *treatment* rather than the broader problem of sewage *disposal.* Likewise it was found that it was not possible to make a detailed comparative study of costs of installation and operation. The difficulty stemmed in part from inadequacy of data. Available information relating to cost—such as power consumption, for example—pertained, in general, to overall power costs for the treatment plant and adjoining installations, including extraneous items as well as the unit or process in question. Moreover, it was considered that in view of the abnormal condition of the national economy during the period of construction, inclusion of costs was not advisable. Contracts for construction were drawn on a cost-plus-fixed-fee basis for all post units as a group. Cost data for individual units such as sewage treatment plants could not be separated from overall post costs. Furthermore, speed was essential; the gravity of the national situation made economy a secondary consideration. The cost data at hand appeared to be of little or no value to municipalities in a peacetime economy.

Selection of Plants

In Table 2b are listed the sewage treatment plants from which questionnaires were received and the location of the posts by state and Service Command or Navy Area.

TABLE 2a.—Types of Sewage Treatment Plants Installed at Army Posts in United States by Service Commands—1940 to 1945

Service—Command	Primary Treatment			High Capacity Single Stage Filters			High Capacity Two Stage Filters			Standard Rate Filters			Intermittent Sand Filters			Activated Sludge		Contact Aeration			Totals
	No.	Sep S D	Ox'n Pond	No.	Sep S D	Ox'n Pond	No.	Sep S D	Ox'n Pond	No.	Sep S D	Ox'n Pond	No.	Sep S D	Ox'n Pond	No.	Sep S D	No.	Sep S D	Ox'n Pond	
I	3/(3)	0	0	3/(3)	2	0	0	0	0	0	0	0	2/(2)	0	0	0	0	0	0	0	8
II	4	3	0	4	4	0	1	1	0	4/(2)	3	0	7/(4)	1	0	1	1	0	0	0	21
III	8/(3)	5	0	4	4	0	1	1	0	15/(7)	4	0	0	0	0	0	0	0	0	0	28
IV	17/(2)	14	0	12	12	0	1	1	0	28/(8)	21	0	10/(1)	3	0	9	9	5	5	0	83
V	1	1	0	1	1	0	1	1	0	14/(10)	9	0	2/(1)	0	0	2	2	0	0	0	22
VI	2/(2)	0	0	0	0	0	1	1	0	9/(2)	6	0	0	0	0	1	1	0	0	0	13
VII	7/(4)	3	2	3	3	0	4	4	0	24/(11)	13	0	2/(2)	0	1	0	0	6	6	0	46
VIII	10/(5)	4	0	7/(2)	5	1	5	4	0	27/(8)	18	1	4/(2)	1	0	3	3	30	30	3	86
IX	32/(20)	12	10	20/(7)	15	5	22/(4)	21	8	13/(11)	5	3	4/(4)	0	0	0	0	0	0	0	94
Totals	84/(39)	42	12	54/(12)	46	6	36/(4)	34	8	134/(59)	79	4	31/(16)	5	1	16	16	41	41	3	399/(130)*

Notes: (1) *This total represents number of plants analyzed to date and at least 80% of plants installed.
(2) Total number of plants with separate sludge digesters—268.
(3) Total number of plants with oxidation ponds—35.
(4) Six plants incorporate standard filters as additional units—1 in VII, 4 in VIII and 1 in IX Service Command; of these, 4 are for two stage high capacity filters and 2 for contact aeration plants.
(5) Three plants incorporate intermittent filters as additional units—1 each in I, VIII and IX Service Commands; 1 of these for single stage high capacity filters and 2 for standard filters.
(6) Two plants incorporate roughing filters as additional units in VIII Service Command—1 each for standard rate filter and activated sludge.
(7) One plant incorporates a high rate filter as additional unit in a contact aeration plant in VII Service Command.
(8) Numbers in parentheses indicate the number of plants included in the total which have Imhoff tanks.
(9) In some plants where Imhoff tanks are used, separate sludge digesters are also provided.
(10) Three chemical precipitation plants, one in VI and two in IX, were installed. These are not included in the table.
Sep S D denotes Separate Sludge Digester.
Ox'n Pond denotes Oxidation Pond.

TABLE 2b.—Plants on Survey List

Name	State	S C or Navy Area	Type of Plant	Complete Study Plants (*)
(1)	(2)	(3)	(4)	(5)
Atterbury, Camp	Ind	5th	Trickling Filter	—
Avon Park AAF	Fla	4th	Contact Aeration	—
Baer Field	Ind	5th	Activated Sludge	—
Barkeley, Camp	Texas	8th	Oxidation Ponds	—
Blackland AAF	Texas	8th	Contact Aeration	—
Blanding, Camp	Fla	4th	Primary	*
Boca Raton AAF	Fla	4th	Trickling Filter	*
Bowie, Camp	Texas	8th	Trickling Filter	—
Bradley Field #1	Conn	1st	Primary	—
Bradley Field #2	Conn	1st	Primary	—
Bragg, Fort	N C	4th	Trickling Filter	*
Breckinridge, Camp	Ky	5th	Activated Sludge	—
Buckley Field	Colo	7th	Trickling Filter	*
Bushnell Gen Hosp	Utah	9th	Trickling Filter	—
Butner, Camp	N C	4th	Activated Sludge	*
Cabaniss Field	Texas	IV	Activated Sludge	—
Callan, Camp	Calif	9th	Trickling Filter	*
Campbell, Camp	Ky	5th	Trickling Filter	*
Carson, Camp	Colo	7th	Trickling Filter	*
Charles, Lake AAF	La	8th	Activated Sludge	—
Chase Field	Texas	IV	Activated Sludge	—
Cherry Point MAS	N C	III	Trickling Filter	—
Claiborne #1, Camp	La	8th	Trickling Filter & Activated Sludge	*
Claiborne #2, Camp	La	8th	Trickling Filter	*
Coffeyville AAF	Kans	7th	Contact Aeration	—
Cooke, Camp	Calif	9th	Trickling Filter	—
Corpus Christi NAS	Texas	IV	Trickling Filter	—
Crowder, Camp	Mo	7th	Trickling Filter	*
Darnall Gen Hosp	Ky	5th	Trickling Filter	—
Davisville NCTC	R I	I	Primary	*
Devens, Fort	Mass	1st	Primary	—
Dix, Fort	N J	2nd	Trickling Filter	*
Drew Field	Fla	4th	Trickling Filter	*
Edwards, Camp	Mass	1st	Trickling Filter	—
Ellis, Camp	Ill	6th	Trickling Filter	—
El Toro MAS	Calif	VI	Trickling Filter	—
Enid AAF	Okla	8th	Trickling Filter	—
Fannin, Camp	Texas	8th	Trickling Filter	—
Fitzsimons Gen Hosp	Colo	7th	Trickling Filter	*
Forrest, Camp	Tenn	4th	Trickling Filter	**
Funston, Camp	Kans	7th	Primary	*
Gillespie, Camp	Calif	VI	Trickling Filter	—
Gordon, Camp	Ga	4th	Trickling Filter	**
Great Lakes (GB) NTC	Ill	V	Activated Sludge	*
Great Lakes (Main) NTC	Ill	V	Trickling Filter	*
Greenville AAF	Miss	4th	Contact Aeration	*
Greenville AAB	S C	4th	Contact Aeration	*
Gulfport AAB	Miss	4th	Activated Sludge	*
Harrison, Benj., Fort	Ind	5th	Trickling Filter	*
Henry, Patrick, Camp	Va	3rd	Primary	—
Hondo AAF	Texas	8th	Contact Aeration	*
Hood, (South), Camp	Texas	8th	Contact Aeration & Oxidation Ponds	—
Huachuca #1, Fort	Ariz	9th	Primary	—
Huachuca #2, Fort	Ariz	9th	Primary	—
Hutchinson NAS	Kans	V	Contact Aeration	—

TABLE 2b (cont.).—Plants on Survey List

Name	State	S C or Navy Area	Type of Plant	Complete Study Plants (*)
(1)	(2)	(3)	(4)	(5)
Jackson, Fort	S C	4th	Trickling Filter	*
Johnson, Seymour Field	N C	4th	Primary	*
Kearns AAF	Utah	9th	Trickling Filter	*
Keesler Field	Miss	4th	Trickling Filter	*
Kilmer, Camp	N J	2nd	Trickling Filter	*
Kingsville Field	Texas	IV	Contact Aeration	*
Klamath Falls M B	Oreg	VIII	Trickling Filter	—
Knox, Fort	Ky	5th	Trickling Filter	*
Kohler, Camp	Calif	9th	Trickling Filter	—
Lambert Field	Mo	V	Activated Sludge	—
Langley Field	Va	3rd	Trickling Filter	—
Lee, Camp	Va	3rd	Primary	—
Livingston, Camp	La	8th	Trickling Filter	—
Lockbourne AAB	Ohio	5th	Trickling Filter	*
Mabry, Dale AAF	Fla	4th	Trickling Filter & Activated Sludge	—
March Field	Calif	9th	Trickling Filter	—
Mather Field	Calif	9th	Trickling Filter	—
Meade, George G, Fort	Md	3rd	Trickling Filter	—
Mechanicsburg NSD	Pa	II	Trickling Filter	—
Memphis NAS	Tenn	IV	Activated Sludge	—
Merced AAF	Calif	9th	Trickling Filter	—
Mitchell Field	N Y	2nd	Primary	—
Monmouth, Fort	N J	2nd	Activated Sludge	*
New Castle AAF	Del	2nd	Primary	—
New York NAS	N Y	II	Trickling Filter	—
Olathe NAS	Kans	V	Contact Aeration	—
Pantex Ordnance	Texas	8th	Trickling Filter	—
Parks, Camp	Calif	VI	Oxidation Ponds	*
Peary, Camp	Va	III	Primary	*
Pendleton #1, Camp	Calif	VI	Trickling Filter	—
Phillips, Camp	Kans	7th	Contact Aeration	—
Picatinny Arsenal	N J	2nd	Trickling Filter	—
Pickett, Camp	Va	3rd	Trickling Filter	—
Pine Camp	N Y	2nd	Primary	—
Plauche NOPOE, Camp	La	8th	Trickling Filter	—
Polk, (North), Camp	La	8th	Trickling Filter	—
Polk, (South), Camp	La	8th	Trickling Filter	—
Presque Isle AAB	Maine	1st	Primary	—
Pueblo AAF	Colo	7th	Contact Aeration	—
Reynolds, Camp	Pa	3rd	Trickling Filter	—
Richmond AAB	Va	3rd	Trickling Filter	—
Riley, Fort	Kans	7th	Trickling Filter	*
Roberts, Camp	Calif	9th	Trickling Filter	*
Robinson, Jos. T., Camp	Ark	8th	Trickling Filter	*
Rogers, Will, Field	Okla	8th	Trickling Filter	—
Roswell AAF	N Mex	8th	Contact Aeration	*
Rucker #1, Camp	Ala	4th	Activated Sludge	—
Rucker #2, Camp	Ala	4th	Contact Aeration	*
San Luis Obispo, Camp	Calif	9th	Trickling Filter	—
San Marcos AAF	Texas	8th	Trickling Filter	—
Savanna Proving Ground	Ill	6th	Trickling Filter	—
Scott Field	Ill	6th	Trickling Filter	—
Seattle Naval Hospital	Wash	VIII	Trickling Filter	—
Seattle NAS (Sand Point)	Wash	VIII	Trickling Filter	—
Selfridge Field	Mich	6th	Activated Sludge	—

TABLE 2b (cont.).—Plants on Survey List

Name	State	S C or Navy Area	Type of Plant	Complete Study Plants (*)
(1)	(2)	(3)	(4)	(5)
Shelby #1, Camp	Miss	4th	Activated Sludge	—
Shelby #2, Camp	Miss	4th	Activated Sludge	*
Sheridan, Fort	Ill	6th	Trickling Filter	*
Sill, Fort	Okla	8th	Trickling Filter	**
Smyrna AAB	Tenn	4th	Contact Aeration	—
Solomons NATB	Md	III	Activated Sludge	—
Standish, Myles C., Camp	Mass	1st	Trickling Filter	*
Stewart, Camp	Ga	4th	Trickling Filter	—
Swift, Camp	Texas	8th	Trickling Filter	*
Topeka AAB	Kans	7th	Trickling Filter	*
Totten, Fort	N Y	2nd	Trickling Filter	—
Trenton NAF	N J	II	Sand Filter	—
Turner AAF	Ga	4th	Trickling Filter	*
Van Dorn, Camp	Miss	4th	Trickling Filter	—
Waco #1 AAF	Texas	8th	Trickling Filter	—
Warren, Francis E., Fort	Wyo	7th	Trickling Filter	**
Weingarten, POW, Camp	Mo	7th	Trickling Filter	—
Westover Field	Mass	1st	Primary	—
Wheeler, Camp	Ga	4th	Primary	*
Willow Grove NAS	Pa	II	Trickling Filter	*
Wilson, Woodrow, G H	Va	3rd	Primary	—
Wood, Charles, Camp	N J	2nd	Trickling Filter	—
Wood, Leonard, Fort	Mo	7th	Trickling Filter	*

In analyzing and comparing performance of different plants of the same type it was found that the number of hours during the day that were utilized in preparing composite samples had a marked effect upon the apparent performance of a plant. Significant distortion was observed for plants employing only short sampling periods. It was therefore decided to retain for complete study only those plants with sampling periods greater than eight hours, and in which composites were formed in proportion to flow. Data from other plants were utilized when desirable to establish normal values of design, and also for special studies. It was the opinion of the subcommittee that while an eight-hour sampling period invalidated comparisons with plants employing longer sampling periods, it was not without value in routine operation at a given installation.

Classification of Survey Plants

For purposes of investigation, it was decided that four classifications of plants on the survey list (Table 2b) would be used as follows: (1) complete study; (2) partial study. Analysis would be concentrated upon the first "complete study" group. A basis of selection for this group was evolved that consisted of *minimum* requirements as to sampling procedure, flow measurement, population data, and laboratory analyses. These criteria are outlined as follows:

A. Sampling—minimum requirement:

1. Period of compositing greater than 8 hours.
2. Samples composited proportional to flow (including recirculation) at intervals not greater than 2 hours.

3. Four composite samples per month.

B. Flow:
   1. Meter and recorder or counter upon dosing tank.
   2. Reasonably reliable quantitative information as to volume of recirculation.

C. Population: Data sufficiently detailed to permit evaluation of equivalent population.

D. Operating Data—minimum requirement:
   1. Operational report for more than an 8-month period.
   2. Laboratory analyses to include both BOD and SS.

About fifty plants were found to meet the foregoing criteria; these are indicated by an asterisk on Table 2b. A few exceptions were permitted in order to obtain balanced representation in all treatment methods. Such exceptions, which are noted as they appear in the sequence of the analysis, were allowed only after it had been ascertained that comparison with other complete study plants would be valid. In Table 3 is presented a breakdown of complete study plants as to distribution, type and size.

It is apparent from Table 3 that trickling filters comprise a very substantial proportion of the total number of plants.

Filter Classification

In order to facilitate classification of filters, a set of symbols (Table 3) was evolved for the designation of various filter characteristics pertaining to design and operation. The scheme, in outline form, is as follows:

(1) *As to depth:* D, deep (over 4.5 feet); S, shallow (less than 4.5 feet).

(2) *As to recirculation ratio:* r, low (under 1.0); R, high (over 1.0); o, no recirculation.

(3) *As to number of stages:* 1, single stage; 2, two stage; etc.

(4) *As to loading:* H, high rate; L, low rate.

Thus, for example, the usual single stage standard filter would be designated by the symbol LDo–1. The defining line as to loading between low rate and high rate filters was not prescribed precisely by the subcommittee, since it is believed that a fixed limit would result in an unnatural grouping

TABLE 3.—Distribution of Plants in Complete Study Group

Group	Total No.	Location		Size		Loading		
		North	South	Large	Small	Under	Normal	Over
Primary settling	6	2	4	5	1	4	2	0
Filter Do-1	10	8	2	6	4	6	2	2
Filter Dr-1	6	2	4	3	3	4	1	1
Filter DR-1	5	1	4	4	1	2	3	0
Filter Sr-1	1	0	1	1	0	0	0	1
Filter SR-1	7	5	2	4	3	4	2	1
Two-stage filter	9	4	5	5	4	6	2	1
Contact aeration	5	0	5	1	4	1	1	2
Activated sludge	5	2	3	3	2	4	1	0
Oxidation pond	1	0	1	1	0	0	1	0
Total	56	24	32	33	23	32	16	8

D = Deep (over 4.5 ft.).
S = Shallow (under 4.5 ft.).
o = No Recirculation.
r = Low Recirculation Ratio (under 1).
R = High Recirculation Ratio (over 1).
1 = Single Stage.

of plants without due regard to the intent of the designer.

The geographical distribution as shown in Fig. 1 indicates approximately the same number of plants in the North as in the South. The South in this connection was defined as the Fourth, Eighth, and, in part, Ninth Army Service Commands.

Navy Sewage Treatment Plants

Twenty-five Navy sewage treatment plants were included in the survey as shown in Table 2b. In general it was found that the Navy program of sewage treatment was sufficiently similar to that of the Army with respect to design and operation to warrant inclusion of a number of Navy installations in the complete study group. A number of factors, however, operated against inclusion of more Navy plants. These factors are the following: (1) Naval installations were very often on the periphery of a large body of water —with adequate dilution such that partial treatment only met the needs of the situation; (2) the Naval sewage treatment program was instituted at a somewhat later date than the Army program, with the result that available operating data did not generally extend over a period of time comparable to that of the Army; and (3) the policy was prevalent in some Naval Areas of utilizing a central laboratory where samples from several plants were analyzed. While such procedure was no doubt quite satisfactory from the viewpoint of controlling plant operation, the time lag entailed in transmitting samples to the central laboratory impaired the comparative value of the data with respect to the data of plants utilizing their own laboratories. On the whole, as shown by an inspection of results of plants deemed suitable for inclusion in the complete study group, the performance of Navy plants was similar to that noted for Army installations.

Partial Treatment Plants

A paucity of data was noted in the category of primary treatment plants, particularly for smaller installations. The situation arose naturally from the fact that the simpler and smaller plants required less skilled operation and fewer analytical determinations for control. However, six primary treatment plants of varying sizes were found to have adequate data. No primary plants employing Imhoff tanks are included in the complete study group. A number of plants with biological treatment in this group, however, used Imhoff tanks as primary units.

Operating Period Studied

A number of factors led to the selection of the interval between January, 1943 and September, 1944 as the period of operation upon which survey findings would be based. While the program was essentially in full swing in 1942, it was decided not to include this year since it represented more or less a "tuning up" period for both plants and operators. Moreover, a substantial number of enlargements and other modifications were being carried forward during 1942. By 1943, however, significantly fewer alterations were in progress, and performance at individual plants had become stabilized. It was decided to terminate the period of study with August, 1944, because of the fact that post populations were becoming smaller at an accelerated rate during the fall of 1944, as the overseas troop movement reached a maximum in preparation for the great Allied offensive of 1945. The period selected—January, 1943 through August, 1944—constituted a 20-month period including two summers, and was considered adequate for the survey. However, it was not possible to utilize the full period for all plants because of such factors as: (1) physical alterations in progress; (2) discontinuities in operation or laboratory analysis; and (3) units out of service. In the event that only a few months of continuous operation were available, every effort was made to ascertain whether or not the results represented characteristic performance before inclusion in the complete study group (Table 3).

In order to obtain additional data relating to nitrification, special determinations were carried out in the field at a number of selected posts during the months of September, October, November, and December, 1944. The period of operation studied for each plant is noted at the appropriate place in the report.

Size of Plants

Two additional items remain to be explained in connection with Table 3, namely, the columns referring to size and to loading.

The line of division between "large" and "small" plants was arbitrarily set at 1.0 mgd. By municipal standards, of course, all military plants studied were relatively small; the foregoing division was entirely a matter of convenience in the selection of plants of a given type in order to obtain, as nearly as possible, a representative group for complete analysis.

Relative Loading

The column in Table 3 entitled "Loading" refers to the number of plants in each category that were underloaded, normally loaded and overloaded with respect to the design criteria as set forth in the foregoing section. A standardized design capacity of each plant in the complete study group was calculated, as based upon the principles set forth in the following section. The relatively large number of underloaded plants, as noted in Table 3, was the consequence of the use of the "capacity factor" in calculating the population for which plant capacity was provided. Normal loading was taken to be in the range of 75 percent to 133 percent of design capacity.

Standard Design Capacity

Design capacities submitted with completed questionnaires from plants were not comparable, it was found, since they were calculated in accordance with rather widely divergent criteria. Hence a standard method of calculation of design capacity was evolved. In establishing a uniform and comprehensive, yet simple, method of evaluation of the capacity of an existing military sewage treatment plant, the following principles were considered to be satisfactory and to meet the needs of the situation:

1. Adherence, in so far as possible, to the code given in the Engineering Manual, Chapter VII, as revised in January, 1943.
2. Use of the dimensions of the principal unit as a basis of evaluating overall capacity. In complete (and intermediate) treatment, the principal unit was construed to be the biological unit. Settling and digestion tank sizes provided ancillary measures of plant capacity.
3. Use of both population and flow criteria wherever feasible.
4. Definition of "Percent Design" as the following ratio:

$$\frac{\text{Actual population (or flow) per unit volume (or area)}}{\text{Design population (or flow) per unit volume (or area)}}$$

The design population in the denominator in this ratio is called the "factor" and was calculated in accordance with loadings specified by the Engineering Manual.

Example: To find the "factor" for a low rate filter. Assume 600 lb BOD per af, 35 percent removal in primary settling, and 0.20 lb BOD per cap per day.

Factor = 600/0.20(1–0.35)
= 4,620 persons per acre foot

A list of factors is given in the following section. In order to provide a uniform relation between the factors for population and factors for flow, 0.2 lb BOD per cap per day and 70 gcd are predicated.

Percentages of Design Capacities Utilized During Loading Periods

I. Filters, Low and High Rate.

A. On Basis of Population:

Percent Design

$$= \frac{\text{Average Population}}{\text{Acre Feet} \times \text{Factor}} \times 100$$

B. On Basis of Average Sewage Flow:

Percent Design

$$= \frac{\text{Average Sewage Flow, mgd}}{\text{Acres} \times \text{Factor}} \times 100$$

Low-Rate Filters

Depth of Filter (ft)	Population	Flow Factor
3.0	4,620	0.969
3.5	4,620	1.130
4.0	4,620	1.290
4.5	4,620	1.450
5.0	4,620	1.620
5.5	4,620	1.780
6.0	4,620	1.940
6.5	4,620	2.100
7.0	4,620	2.260
8.0	4,620	2.590

High-Rate Filters

Depth of Filter (ft)	Population	Flow Factor
3.0	23,100	4.850
3.5	23,100	5.650
4.0	23,100	6.460
4.5	23,100	7.270
5.0	23,100	8.080
5.5	23,100	8.880
6.0	23,100	9.700
6.5	23,100	10.500
7.0	23,100	11.300
8.0	23,100	12.900

II. Activated Sludge.

A. On Basis of Actual Sewage Flow:

Percent Design

$$= \frac{\text{Average Tank Influent (gal)}}{\text{Volume of Tank (cf)} \times \text{Factor}} \times 100$$

Detention	Flow Factor
8.0 hrs	22.5
12.0 hrs	15.0

III. Contact Aeration.

A. On Basis of Population:

Percent Design

$$= \frac{\text{Average Population}}{\text{Contact Surface (sf)} \times \text{Factor}} \times 100$$

Location	Population Factor
South	0.0493
North	0.0440

IV. Settling Tanks.

A. On Basis of Actual Sewage Flow:

Percent Design

$$= \frac{\text{Av Tank Infl incl Recir (gal)}}{\text{Volume of Tank (cf)} \times \text{Factor}} \times 100$$

Detention	Flow Factor
1.5 hrs	120.00
2.5 hrs	72.00

V. Sludge Digestion Tanks.

A. On Basis of Population:

Percent Design

$$= \frac{\text{Average Population}}{\text{Volume of Tank (cf)} \times \text{Factor}} \times 100$$

Type of Sludge and Digester	Population Factor
Plain settling, unheated	0.333
Plain settling, heated	0.500
Activated sludge, unheated	0.167
Activated sludge, heated	0.250
All others, unheated	0.222
All others, heated	0.333

Types of Posts

A variety of types of military posts was included in the survey. In accordance with expectation, some differences in distribution of flow and characteristics of sewage were observed. The type of post is noted for each plant in the body of the report. Greeley and Chase (26) have summarized the salient characteristics of different types of posts as follows:

Cantonments for housing troops varied in capacity from a few thousand up to 60,000, with 35,000 more or less as an average.

Armored division posts differed from cantonments in that mechanized artillery units were provided for as well as personnel. Anti-aircraft firing centers were included in this group.

Base hospitals were of various sizes and in some cases were adjacent to communities whose existing sewage facilities were utilized. Large variations in flow and violent fluctuations in the concentration of sewage occurred at some plants in this group.

Air fields, although larger in area, were in general relatively small as regards personnel. Consequently their sanitation involved small structures. A common arrangement existed in which several small fields were grouped around a central field at distances of several miles. At such installations a single laboratory would often analyze samples from all plants.

Munitions depots were staffed with a personnel essentially civilian. Moreover, such posts were occupied for the most part only during daylight hours. In a few cases it was believed that unusual ingredients, chemicals and byproducts found their way into the sewers. Consequently it was not surprising that results from munition depots exhibited a significant deviation from those observed at more typical installations. The following compilation lists the more important types of posts encountered.

Post Types

1. AGF Divisional Training
2. AGF Armoured, Mechanized Cavalry, and Tank Destroyer Training
3. AGF Replacement Training
4. AGF Harbor Defenses
5. AGF Artillery Training and Anti-Aircraft Training
6. AAF Tactical Fields and Flying Training
7. AAF Technical Training Schools
8. AAF Replacement Training and Reclassification Centers
9. ASF Reception Centers
10. ASF Service Troop Training
    (a) Engineer
    (b) Quartermaster
    (c) Signal
11. ASF Staging Areas and Overseas Discharge and Replacement Depots or Centers (troops, not material)
12. ASF General Hospitals
13. ASF Ordnance Department Industrial Facilities
14. ASF Ordnance Department Industrial Facilities, not including industrial wastes (housing only)
15. ASF Internment Facilities (Prisoner of War)

Population Data

Population at a post was subdivided into four classifications for the purpose of investigation. These were: (1) resident military; (2) resident civilian; (3) nonresident military and (4) nonresident civilian. Of these the first was invariably predominant. The number of civilian residents was usually negligibly small. Considerable difficulty was encountered in obtaining the number of nonresident military personnel; for the most part, available information represented estimates only. Nonresident civilian personnel included day workers, office workers, etc. From the numbers in each of the four classifications, an equivalent population for the post was

TABLE 3a.—Distribution of Population

Post	Type	Av Equiv Pop'n	Relative Population				Equiv Pop'n
			Res Milit	Res Civil	Nonres Milit	Nonres Civil	
Callan	Repl Trg	10,100	100	0.9	9	9	106
Kearns	Repl Trg	8,940	100	6	5	*	102
Claiborne	Div Trg	21,800	100	*	9	5	105
Knox	Arm MC and TD Trg	52,200	100	12	8	4	108
Crowder	Serv Trp Trg	29,200	100	0.7	18	8	109
Bragg	Art and AA Trg	48,500	100	2.4	19	18	112
Dix	Stg Area and Rec Ctr	40,700	100	1.5	0	6	102
Drew Field	Tact Fields and Fly Trg	18,700	100	12	8	4	108
S Johnson Field	Tech Trg	18,900	100	*	5	16	107
Gulfport Field	Tact Fields and Fly Trg	17,100	100	14	21	16	112
Boca Raton Field	Tech Trg Sch	9,000	100	0	11	15	109
Keesler Field	Tech Trg Sch	32,000	100	0.4	7	10	106

* Negligible.

computed. Equivalent population was defined as the resident plus one-third of the nonresident population. The validity of this assumption is considered in Chapter II in light of operating results.

In order to set forth the approximate distribution of population in various classifications, Table 3a is presented. It should be noted that in many of the plants, population estimates by post engineering personnel had to be used in lieu of actual enumeration. In order to facilitate comparison, the figures have been made relative to a resident military population of 100.

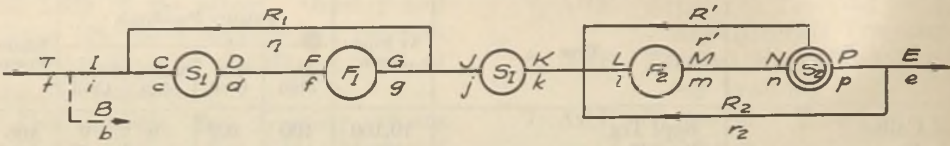
While a considerable range was manifest, the following distribution of the total population would appear to be typical: (1) resident military, 75 percent; (2) resident civilian, 5 percent; (3) nonresident military, 10 percent; and (4) nonresident civilian, 10 percent. The equivalent population was therefore on the average about 8 percent larger than the resident population. Available information for general hospitals indicated a somewhat larger proportion of nonresidents. At Fitzsimon GH, the nonresident personnel constituted about 23 percent of the total; at Bushnell GH, nonresidents constituted 37 percent of the total.

Names, Symbols, and Abbreviations

The work of analysis of the survey was facilitated by adoption of a standardized nomenclature for various quantities and terms pertaining to loading and performance of a sewage plant. A rational scheme of flow designation has been much needed since the advent of recirculation, which introduces a number of complications. The system of names and symbols is presented with the aid of the diagram and accompanying list of Table 4. The following conventions were adopted: (1) The designation of the various fluid flows at different points in sewage plants would be based upon: (a) plant unit in proximity; (b) inflow or outflow, *e.g.*, primary settling tank effluent; (c) in the case of recirculated flow, the origin of the fluid, e.g. secondary filter recirculation. (2) "Plant influent" would be defined as "total flow" minus "by-passed flow."

A standardized set of abbreviations was worked out by the subcommittee to apply to the more common words used in connection with the analysis

TABLE 4.—Symbols, Names and Abbreviations of Flows, Concentrations and Weights of Sewage at Various Points in Sewage Treatment Plants



Symbol (mgd)	Symbol (lb)	Name	Abbreviation
T	*t*	Total Flow	Total Flow
I	*i*	Plant Influent	Plant Inf
C	*c*	Primary Settling Tank Influent	Pri Inf
D	*d*	Primary Settling Tank Effluent	Pri Eff
F	*f*	Primary Filter (or Aerator) Influent	Pri Fil (or Aer) Inf
G	*g*	Primary Filter (or Aerator) Effluent	Pri Fil (or Aer) Eff
R_1	r_1	Primary Filter (or Aerator) Recirculated Flow	Pri Fil (or Aer) Recir
J	*j*	Intermediate Settling Tank Influent	Int Set Inf
K	*k*	Intermediate Settling Tank Effluent	Int Set Eff
L	*l*	Secondary Filter (or Aerator) Influent	Sec Fil (or Aer) Inf
M	*m*	Secondary Filter (or Aerator) Effluent	Sec Fil (or Aer) Eff
R_2	r_2	Secondary Filter (or Aerator) Recirculated Flow	Sec Fil (or Aer) Recir
N	*n*	Secondary Settling Tank Influent	Sec Set Inf
R'	*r'*	Secondary Settling Tank Launder Recirculated Flow	Sec Set Launder Recir
P	*p*	Secondary Settling Tank Effluent	Sec Set Eff
R_2	r_2	Secondary Settling Tank Recirculated Flow	Sec Set Recir
E	*e*	Plant Effluent	Plant Eff
B	*b*	By-passed Flow	By-pass

of sewage treatment plant performance. These abbreviations are presented in the following compilation.

Abbreviations

a . . . . acre
af . . . . acre-feet
AT . . . . aeration tank
av . . . . average
BOD . . . . . . biochemical oxygen demand (in 5 days at 20° C unless otherwise stated)
C . . . . Centigrade
cap . . . . capita
cf . . . . cubic foot
cf/gal . . . . . . cubic feet per gallon
cfm . . . . cubic feet per minute
cfs . . . . cubic feet per second
Cl_2 . . . . chlorine
col . . . . column
cu . . . . cubic
cy . . . . cubic yard
d . . . . day; daily
dia . . . . diameter
diff . . . . diffused
DO . . . . dissolved oxygen
dp . . . . detention period
dv . . . . displacement velocity
eff . . . . efficiency
effl . . . . effluent
elev . . . . elevation
F . . . . Fahrenheit
Fig . . . . Figure
fil . . . . filter
fin . . . . final
ft . . . . foot
g . . . . gram
gal . . . . gallon
gcd . . . . gallons per capita daily
gpd . . . . gallons per day
gpd/sf . . . . . . gallons per day per square foot
gpm . . . . gallons per minute
hori . . . . horizontal
hp . . . . horsepower
hp-hr . . . . horsepower-hour
hr . . . . hour
in . . . . inch
inf . . . . influent
kw . . . . kilowatt
kw-hr . . . . kilowatt-hour
lb . . . . pound
lin . . . . lineal
max . . . . maximum
mech . . . . mechanical
mg . . . . million gallons
mgd . . . . million gallons daily
mgad . . . . million gallons per acre daily

min........minimum; minute
ml.........milliliter
mo.........month; monthly
no..........number
NH_3-N......Ammonia Nitrogen
NO_3-N......Nitrate Nitrogen
NO_2-N......Nitrite Nitrogen
p...........page, per (in connection with other abbreviations)
pop'n.......population
pp.........pages
ppm........parts per million
pri.........primary
psi.........pounds per square inch
recir........recirculation
rem........removal
rpm........revolutions per minute
RS.........relative stability
sec.........second; secondary
set.........settling
sew.........sewage
sf..........square foot
sg..........specific gravity
sol.........solids
sq..........square
SS.........suspended solids
ssr.........surface settling rate
sy..........square yard
temp.......temperature
tot.........total
vel.........velocity
vert........vertical
vol.........volume
vola........volatile
wt.........weight
yd.........yard

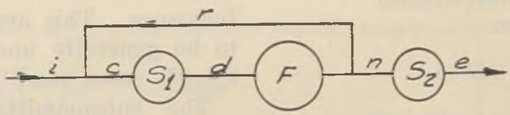
Units Relating to Area and Volume

In the matter of expressing loading and performance, particularly with reference to filter plants, the subcommittee adopted the following resolutions as regards type of dimensional units to be employed: (1) area would be measured in acres, and volume in acre-feet; (2) loading of a filter would be expressed (a) on a velocity basis, mgad, and (b) on a weight (BOD, SS) basis, lb/a and lb/af. It was the consensus of opinion that the use of the "acre" and the "gallon" in filter technology for nearly a century provided sufficient justification to warrant their retention in modern practice. The use of the "acre" and the "acre-foot" as measures of area and volume has been common in the allied fields of applied hydraulics; adoption of a new set of units would constitute an entirely unnecessary break with tradition. The argument has been advanced in some quarters that the use of other dimensional units, *e.g.* the cubic yard, results in more conveniently sized numbers indicative of loading and performance. This argument was found to be generally unconvincing and, in specific cases, readily confuted.

The subcommittee recognizes the value of universal acceptance of standard units, and recommends that the following principles be applied when it becomes necessary to devise new units in connection with technological innovations in sewage treatment: (1) The new measures should be compounded in so far as possible from existing fundamental units (feet, acres, pounds, etc.) commonly employed in such a way that continuity with established usage is preserved. (2) A careful distinction between *direct* and *inverse* measures should be made so as to accord with established custom in analogous quantities. For example, in describing loading, a term such as lb BOD/af would be decidedly superior to an inverse measure such as cf stone/lb BOD. (3) If more than one kind of measure is required in order to express adequately loading or performance, all necessary measures should be recognized. For example, a volumetric unit such as lb BOD/af is inadequate by itself as a criterion of filter loading, since it neglects such relevant considerations as filter depth and volume of recirculated flow. All relevant measures should be retained in order to avoid inaccurate, distorted descriptions of design, loading, and performance. A rational method of expressing filter loading is presented in Chapter V.

Measures of Performance—Percent Removal and Percent Efficiency

In expressing the performance of a particular plant unit, two distinct kinds of measure were found to be useful and necessary for complete description. The two are defined at the hand of the following diagram, in which all the letters represent weight of BOD (or SS) flowing by the point in question each day.



S_1 = primary settling tank
S_2 = secondary settling tank
F = filter or aeration basin

With regard to the performance of any unit—say, for example, the primary settling tank, S_1—the results may be expressed variously as follows:

(i) *Percent Efficiency,* as based upon total load applied to unit (including that from recirculated flow):

Percent efficiency
= (100) lb removed/lb applied to unit
= $100(c\text{-}d)/c$ (1)

(ii) *Percent Removal,* as based upon total load applied to plant:

Percent Removal
= (100) lb removed/lb applied to plant
= $100(c\text{-}d)/i$ (2)

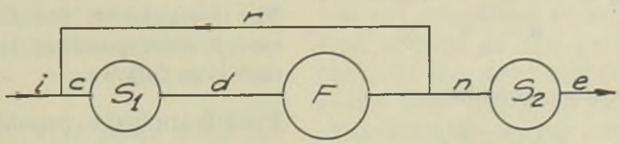
From the foregoing definitions it is evident that the per cent removals for all units of the plant are additive to 100 per cent if the percent remaining in the effluent is included. Accordingly, per cent removals are useful in comparing one plant unit with another as to relative performance. Per cent efficiency, which is based upon the load applied to the unit in question, is a criterion of performance useful in comparing similar units in *different* plants. For illustration, consider the standard filter plants, A and B, that are identical as to design and loading in every respect save that A has a larger primary settling tank. The longer detention period in this tank is conducive to greater primary settling efficiency; as a result, filter A receives a smaller load than B, and per cent removal in filter A computed in accordance with Equation (2) is seen to be less than the corresponding removal in filter B. It is evidently not correct to compare the performance of the two units *as filters* by comparing per cent removals.

Filter performance must be rated upon the basis of the load applied, and for this purpose per cent efficiency, as defined by Equation (1), is the appropriate measure. It is pertinent to note that when recirculation is introduced, a general redistribution of the loading and removals among the various units takes place. If, for example, in the foregoing illustration flow is recirculated about filter A, the applied load arbitrarily may be made equal to that of filter B. In fact, by increasing the rate of recirculation, the load may be made indefinitely large. The important point, however, is that the biochemical character of the loading changes with increased recirculation because an increasing proportion of the organic material applied has already passed through the filter one or more times. Consequently, neither the loading nor the performance can be expressed in an entirely satisfactory manner by the simplified concepts embodied in Equations (1) and (2). The introduction of recirculation makes it necessary to consider the mechanism of sewage treatment in a more fundamental way in order that performance of plants of a type may be compared upon an equitable basis. Perception of the problem is facilitated by precise distinction between primary and secondary treatment.

Primary and Secondary Sections of Sewage Plants

In analyses of performance it is desirable to subdivide the units of a plant into a primary treatment section and a secondary treatment section. Subdivision is necessary in order that performance of units such as filters and contact aerators may be evaluated in the light of the load applied. With some schemes of recirculation no difficulty is presented in finding a natural division between the primary and secondary sections of the plant. However, when flow is recirculated through the primary settling tank, it becomes impossible to assess accurately that portion of the BOD (or SS) removed in the primary settling tank coming from the raw sewage as distinct from the recirculated flow. Assessment of the portion attributed to raw sewage must be based upon an assumption. Various assumptions with differing degrees of arbitrariness may be made. Some insight into the problem may be had at the hand of the following diagram which delineates the two treatment sections.

After studying several alternate schemes, the subcommittee decided that the following assumption was in general the most satisfactory with regard to analysis of military sewage plants: When flow is recirculated from filter effluent to primary settling tank inlet, the *percent* of BOD removed from the recirculated flow by the primary settling tank is assumed to be the same as that actually occurring in the *secondary* settling tank. With this assumption, the problem of dividing the plant into the two sections is resolved. In the interest of lucidity, the following formulation is presented, in which the symbols represent weight of BOD (or SS) and are defined by the following diagram:



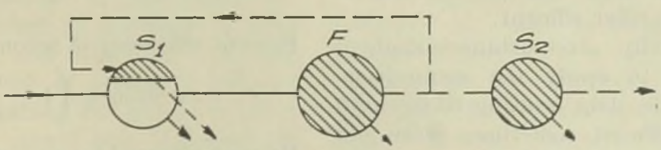
Pounds applied to primary section $= i$

Pounds removed by primary section

$$= i + r - d - \frac{(n-e)}{n} r$$
$$= i - d + \frac{e}{n} r \quad (3a)$$

Pounds applied to secondary section

$$= i - \left(i - d + \frac{e}{n} r\right)$$
$$= d - \frac{e}{n} r \quad (3b)$$



Primary. Section—Unshaded
Secondary Section—Shaded

Pounds removed by the secondary

$$\text{section} = i - e - \left(i - d + \frac{e}{n} r\right) \quad (3c)$$

$$= d - e - \frac{e}{n} r$$

Percent efficiency of primary

$$\text{section} = \frac{i - d + \frac{e}{n} r}{i} 100 \quad (3d)$$

Percent removal by primary

$$\text{section} = \frac{i - d + \frac{e}{n} r}{i} 100 \quad (3e)$$

Percent efficiency of secondary

$$\text{section} = \frac{d - e - \frac{e}{n} r}{d - \frac{e}{n} r} 100 \quad (3f)$$

Percent removal by secondary

$$\text{section} = \frac{d - e - \frac{e}{n} r}{i} 100 \quad (3g)$$

It is noted that in accordance with the foregoing assumption, percent efficiency and percent removal are the same in the primary section, although they may be considerably different in the secondary section.

With other schemes of recirculation the formulations vary slightly but are obtained readily from application of the underlying assumption. The assumption may be stated in various ways. It is tantamount to computing the weight of BOD (or SS) in the settled secondary effluent from the flow through the filter and the strength of the final clarifier effluent.

Occasionally, circumstances made it impossible to apply the assumption. For example, data relating to strength of filter effluent sometimes were not available for a number of months during the operating period studied; this precluded application of the assumption and made it necessary to use another scheme for accrediting overall performance to primary and secondary sections. Moreover, in the case of a few of the two stage filters and activated sludge plants employing effluent recirculation, lack of sampling points made it desirable to adopt an alternate basis for computation of loading and performance. Accordingly, when the usual scheme could not be applied, the following alternate assumption was made: The percent of BOD (or SS) removal from the recirculated flow was assumed to be the same as that actually occurring in the *primary* settling tank. This is equivalent to the assumption that the percent of BOD (or SS) removal from the raw sewage was the same as the percent removal of combined flow in the primary settling tank. The formulation for loading and removal corresponding to this assumption is as follows:

Pounds applied to primary section $= i$

Pounds removed in primary

$$\text{section} = \left(1 - \frac{d}{c}\right) i \quad (4a)$$

Pounds applied to secondary

$$\text{section} = \frac{d}{c} i \quad (4b)$$

Pounds removed in secondary

$$\text{section} = \frac{d}{c} i - e \quad (4c)$$

Percent efficiency of primary

$$\text{section} = \left(1 - \frac{d}{c}\right) 100 \quad (4d)$$

Percent removal by primary

$$\text{section} = \left(1 - \frac{d}{c}\right) 100 \quad (4e)$$

Percent efficiency of secondary

$$\text{section} = \left(1 - \frac{ec}{di}\right) 100 \quad (4f)$$

Percent removal by secondary

$$\text{section} = \left(\frac{d}{c} - \frac{e}{i}\right) 100 \quad (4g)$$

Tables of plant performance denote which of the two assumptions as embodied in Equations (3) and (4) was employed in calculating the data. On the whole, an excellent agreement was noted between results for plants at which an assumption had to be made and those of the same type at which an assumption was unnecessary. Such agreement pointed up the validity of the assumptions.

An attempt was made to utilize standard primary settling BOD reduction curves obtained from plants without recirculation as a basis for calculation of the load of settled raw sewage applied to the filter. However, use of standard removal curves in this manner proved to be unsatisfactory, since only small variations in reduction occurred with differences in detention period. The fact that detention periods were in general quite long accounted for the lack of interrelationship.

Other factors were much more important in determining the settling efficiency and a wide range of performance was noted. Chief of these factors were (1) condition of sewage and (2) condition of tank. The condition of sewage entering the primary tank was determined by characteristics of the sewerage system and by the amount of recirculated flow. The condition of tank reflected (a) design of inlet and outlet arrangements and (b) schedule of operation of sludge collection and removal equipment.

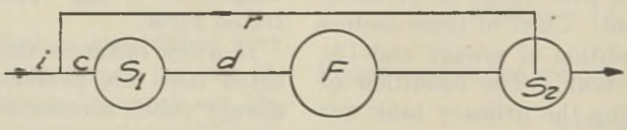
Graphs (Fig. 67) in which percent removal was plotted against detention period were flat (showing an approximately constant 35 percent removal), and the influence of the important variables relating to condition of tank and sewage was obscured. Consequently, the use of standard removal curves, as obtained for plants without recirculation, would have involved the assigning of a nearly constant percentage removal to plants with recirculation, an assumption at variance with actuality. Consequently it was decided that the assumption to be used in calculating the otherwise indeterminate amount of removal from the recirculated flow in the primary tank should involve actual settling performance in the secondary tank. While such a procedure was no doubt arbitrary, it did bring into play the paramount variable affecting settling efficiency—the condition of the sewage.

Influence of Secondary Sludge upon Primary Section Performance

It was the usual operating procedure in the majority of plants surveyed to pump sludge from the final and intermediate settling tanks to the primary tanks, from which it would be pumped to the digesters along with primary sludge. Secondary sludge, however, was not considered to be a part of the load applied to the primary settling tank. It was presumed that having already been deposited once, the secondary sludge would ordinarily settle again in the primary tanks, particularly since it was applied in concentrated form.

In a few instances the evidence indicated that this presumption was not always valid; occasionally the organic content of the primary effluent would be unduly large, indicating that secondary sludge was passing through the primary tank. Application of the foregoing presumption in these instances, therefore, resulted in a value for primary efficiency that was too low and a filter loading that was too high. Usually, however, secondary sludge resettled readily and did not pass through the primary settling tanks.

In a number of plants it was the practice to run secondary sludge pumps continuously so as to provide recirculation that minimized septicity, especially during low plant flows at night. In some plants such recirculation amounted to a substantial proportion of the sewage flow, and the effect upon primary sedimentation efficiency could not be ignored. Therefore, in plants in which the proportion of flow from secondary sludge pumps exceeded ten percent of the plant influent, the data were analyzed in accordance with the scheme adopted for plants designed for normal recirculation (Equation 3). The total weight of BOD (or SS) applied to the primary tank, c, was considered to be the sum of that in the plant flow, i, plus that in the recirculated flow, r. The latter was calculated from the volume of the recirculated flow and concentration of the final effluent. In this way secondary sludge itself was not included in the loading of the primary tank. Since the recirculated flow, r, exclusive of the sludge was taken to be equal in strength to the settled final effluent, it was assumed that no deposition from the recirculated flow would take place in the primary tank. Accordingly, in calculating the load applied to secondary units, the quantity r (see diagram) would be deducted from the weight of BOD (or SS) in the primary effluent, d. For example, in installations where recirculation from secondary sludge pumps constituted the sole recirculation for the plant, the load applied to secondary units would be d minus r.



Calculation of Detention Periods

Because of the large variation in flow throughout the day that was found almost invariably to occur at military sewage plants, it was necessary to adopt some standardized method of calculating detention period. Two definitions of average 24-hour detention period are suggested by the following equations:

$$DP_1 = \frac{24V}{\int_0^1 q\,dt} \qquad (5)$$

$$DP_2 = 24V \int_0^1 \frac{1}{q}\,dt \qquad (6)$$

where V = volume of tank, gallons

q = instantaneous rate of flow, gallons per day

DP = detention period, hours

Equation (5) represents the average time required by individual particles of fluid to traverse the tank. Equation (6) represents the mean value of the "instantaneous detention period" occurring throughout the day. While each definition has its own particular advantages, it was decided that the first (Equation 5) was more suitable as a measure expressive of loading. Moreover, detention periods are easily evaluated in accordance with Equation (5), being simply the quotient of tank volume divided by average daily flow.

The flow used in calculation of detention period was invariably taken as *actual* flow into the unit, including recirculation, if any. Equation (5) was also applied to tanks with two or more normal flow addition or removal points. For example, the detention period of a final settling tank with a central launder for removing recirculated flow was taken to be the ratio of the total tank volume to the average flow, as in Equation (5). It is true that flow removed through the launder is not held in the tank as long as that passing over the effluent weir; nevertheless, the validity of the definition (Equation 5) of detention period thus calculated is retained; the result gives the average time of passage for *all* particles entering the tank.

It is evident that detention periods as calculated by Equation (5) do not take into account the possibility of "short circuiting" or "dead spaces" within the tank. Consequently, the values obtained represent "theoretical" rather than "actual" detention. While the latter quantity would undoubtedly be more closely correlated with tank performance, the necessary data for its evaluation were not available.

Surface Settling Rate

As an ancillary measure of loading of settling tanks, surface settling rates were believed to provide sufficient information in addition to that furnished by detention period to warrant calculation. The surface settling or "overflow" rate in gallons per day per square foot of tank area represents the average upward velocity against which suspended particles must settle. It was calculated by dividing the total flow through the tank (including recirculation, if any) by the gross horizontal tank area (including surface area covered by weir troughs, etc.). An equivalent method of evaluating surface settling rate which has conceptual value is the following:

$$\text{gpd/sf} = 180 \times (\text{Mean Depth, ft})/(\text{Detention Period, hr}) \qquad (7)$$

Statistical Methods

The work of analysis consisted of the following steps: (1) assembling and cataloguing data pertaining to each treatment method; (2) establishing the period of study for each plant; (3) consolidating and summarizing characteristics of the raw sewage; (4) calculating and checking pertinent dimensions, areas and volumes, and other design details; (5) analyzing performance of each unit and of plant as a whole with respect to the load applied.

The period of study selected for each plant was a time interval during which the loading was reasonably uniform as to contributory population and flow, and the operation consistently followed a more or less fixed pattern (with respect to recirculation, units in service, etc.). Ordinarily, there was but one period of study for each plant; in a few plants, however, two periods of study were obtained.

In consolidation and analysis, certain statistical parameters were consistently used, and it is pertinent to explain methods by which they were computed.

Calculation of Organic Loading

Monthly operating data were averaged over the period of study by use of the arithmetic mean, the result for each month being weighted in accordance with the amount of organic matter as computed from strength (concentration) and flow. For example, the overall percent removal of BOD for a plant was computed by the following equation:

$$\text{Percent removal BOD} = \frac{\sum QC_i \times 8.34 - \sum QC_e \times 8.34}{\sum QC_i \times 8.34} \times 100 \qquad (8)$$

where

Q = average daily flow for month, mgd

C_i = ppm BOD in raw sewage

C_e = ppm BOD in final effluent

$\sum$ denotes summation for all months in the period of study

Some of the less important analytical determinations such as Relative Stability were analyzed by simple unweighted averages over the period of study.

The median, defined as the middle item of a series arranged in order of size, was used to advantage in a number of applications in which application of the arithmetic mean would have led to a distorted result.

Sewage plant data characteristically exhibit considerable dispersion about average values. In the quantitative description of the "scatter" the usual statistical measures—standard deviation, quartile deviation, average deviation—were employed.

In general, three methods of curve fitting were employed, depending upon the importance attached to the generalization at hand. These were, in order of theoretical validity: (1) the method of least squares; (2) the method of moments; (3) the method of averages applied to group medians. These methods are described in standard statistical textbooks.

Reliability of Data

There are in general four types of errors involved in sewage plant data. These comprise errors in estimation of (1) population, (2) flow, (3) strength of sewage (BOD and SS), and (4) physical dimensions. The first two of these are usually simple compensating errors whose effect is non-cumulative when many similar plants are investigated. Errors in measurement of strength are of two kinds, those made in sampling and those made within the laboratory as a result of inaccurate analysis. Sampling errors may be classified further as due to (a) method of collection, (b) place of collection, and (c) amount collected (compositing error). Such errors are not invariably of the compensating kind, and their effect may be cumulative and thereby impart error to averages and norms. The final source of error, that due to imperfect measurement of physical dimensions, is generally small and unimportant. There are instances, however, in which estimation of effective areas and volume is difficult, such as in the evaluation of the effective settling volume of a multi-compartment tank with hopper bottoms for sludge draw-off.

Overall error in measurement of loading depends upon the magnitude of error in each of the foregoing sources. Error in the measurement of efficiency stems from only two sources, namely, flow and strength. While overall error reflects errors of different types, it is usually true that the largest portion comes from one controlling source that limits the precision obtainable.

As a general estimate of the error entailed in the survey, the following approximate probable errors are indicated: (1) population, 7.5 percent; (2) flow, 3 percent; (3) concentration (BOD and SS), 2 percent; (4) dimensions, 2 percent. These figures apply to overall averages for the entire period of study at each plant, and not to the data for any particular day or month.

It is evident that the relatively large error involved in estimation of population controls the error in derived quantities indicative of loading and performance. For example, error in pounds of BOD per capita daily in raw sewage may be calculated by assuming the following typical data for a plant: concentration of raw sewage, 343 ppm; flow 1 mgd; population, 14,300.

Probable error in concentration

$$= 0.02\,(343) = 6.8 \text{ ppm}$$

Probable error in flow

$$= 0.03\,(1) = 0.03 \text{ mgd}$$

Probable error in population

$$= 0.075\,(14{,}300) = 1{,}070 \text{ persons}$$

lb BOD/cap/day

$$= \frac{8.34\,(1 \pm 0.03)\,(343 \pm 6.8)}{14{,}300 \pm 1{,}070} = 0.20 \pm 0.025$$

A probable error of 0.025, or 12.5 percent results from the combination of errors in concentration flow and population. More than half of the combined error comes from imperfect population data.

In derived quantities that do not involve population, a greater precision obtains. For example, assume in the foregoing illustration that the BOD in the final effluent is 34 ppm, with a corresponding probable error of 0.68 ppm. Then the overall percent reduction in BOD may be evaluated together with its probable error as follows:

Percent reduction in BOD

$$= 100\frac{(343 \pm 6.8) - (34 \pm 0.68)}{343 \pm 6.8}$$

$$= 90 \pm 3.8$$

A probable error of about 4 percent is seen to be introduced into the estimate of overall performance of an individual plant. However, when a dozen plants are surveyed, the probable error involved in the estimate of their performance as a group is $4 \div \sqrt{12}$, or only a little over 1 percent.

CHAPTER II

CHARACTERISTICS OF MILITARY SEWAGE

Environmental conditions at military establishments led to production of sewage that differed significantly from municipal sewage in a number of respects. The chief differences were that the sewage (i) tended to be more concentrated; (ii) had a greater proportion of volatile matter relative to the total solids; (iii) contained a higher proportion of grease; and (iv) exhibited larger and more rapid variations in diurnal flow.

With minor exceptions, there were no industrial wastes in the sewage nor extraneous matter that interfered with treatment; sewage in military installations was essentially of a domestic character. Separate sewer systems were the rule.

The differences between military and municipal sewage stemmed primarily from the military regime—the systematized pattern of sleeping, working and eating.

The high concentration of military sewage was due to the following factors: (a) absence of diluting water from industrial sources (cooling and air conditioning); (b) institution of water conservation programs that involved reasonable disciplinary restrictions as to water consumption, together with installation of additional plumbing fixtures; (c) greater organic waste per capita from predominantly male population fed on a high caloric diet.

The relatively low amount of mineral matter may be attributed to the sanitary sewage system and absence of industrial waste. Laundry effluent often was a source of difficulty. In amount it was sometimes found to be excessive relative to comparable civilian installations. Often one post did laundry work for several adjacent posts. The disproportionately large volume of laundry waste produced interfered with normal sewage treatment, particularly when the waste came to the plant in "slugs." Differences in laundering practice resulted in some variation in sewage characteristics observed at certain army and navy installations. At some navy posts, for example, the custom of requiring the individual to do his own laundry, resulted in relatively large consumption of water and soap.

Flow of Sewage

The diurnal distribution of sewage on the whole was found to differ only slightly with the type of post. In typical installations, such as divisional training and flying training schools, the chief characteristic was the rapidity with which the flow would increase during the early morning hours. This phenomenon was noted in other types of installations, although in plants at staging areas and general hospitals a wide range of flow distribution was experienced and an abrupt flow acceleration did not always occur. Some large plants treated sewage from adjacent civilian areas and flow distribution approached that characteristic of municipalities.

Basis of Design as to Diurnal Flow Distribution

The Engineering Manual of the Corps of Engineers stipulated that the following flows were to be envisaged in design of sewerage systems and sewage treatment plants: (1) an average 16-hr flow of 125 percent of the daily average flow; (2) a maximum 4-hr average flow of 175 percent of the daily average flow; (3) a minimum 4-hr flow of 40 percent of the daily average flow; (4) occasional extreme peak flows of three times the daily average rate. It was noted in the manual that design of the various elements of the sewage treatment plant was to be based primarily upon the average daily rate of flow. At installations in which nonresident employees worked, the flow allowance was prorated in accordance with the proportion of time spent at the post.

Flow Measurement

It was the policy to install flow measuring devices in all sewage plants. In a number of plants serving a population less than 5,000 recording and totalizing equipment was not provided. It was recommended at those posts where sewage was pumped that the measuring device to be placed ahead of the pumping station in order to record hourly variation in sewage flow. At many plants, however, this practice would have been inconvenient from the standpoint of operation. Consequently measuring devices were, as a rule, placed within the bounds of the treatment plant proper.

The following devices were found to be satisfactory for measurement of the flow of sewage: Parshall flume; Palmer-Bowlus flume (and other rectangular flumes); Venturi meter; Kennison nozzle; and weirs of various sizes and shapes. The Parshall flume, when properly designed and installed, was eminently satisfactory in every respect. It proved to be reliable under a variety of operating conditions.

Due to hasty construction occasioned by the urgency of the program, flumes were not always installed with requisite attention to details of design and construction. Adjustments of dimension and alignment were necessary at some posts. One common difficulty noted was that flumes were generally too large in proportion to the volume of flow handled. This over-design could probably be attributed, in many cases, to application of the "capacity factor" in design. While underloading of a plant could usually be handled, in so far as treatment was concerned, by removal of units from service, this did not apply to flow measurement, since duplicate flumes were not provided. As a consequence, the water level at the throat of many of the flumes was low even during peak flow; this condition resulted in a sacrifice of precision of flow measurement that could have been attained with a smaller flume.

In typical installations flumes were located ahead of primary settling tanks; with this arrangement the following design considerations are involved: (i) elevation of the flume floor to be not so high as to cause drowning of the trunk sewer to the plant, yet not so low as to occasion excessive turbulence by a drop toward the flume; (ii) throat to be sufficiently narrow to obtain adequate flow regulation necessary for precision of measurement, yet not so narrow as to cause overtopping of flume walls during peak flows or to introduce large velocities and accompanying turbulence in the tailwater at the settling tank entrance. Tailwater depth should not be greater than about ⅔ of the headwater depth in order that the flume function as a hydraulic control. On the other hand, the tailwater should not be too low so as to cause high velocities at the settling basin inlet. The foregoing considerations may be incorporated in an analysis. The rating equation for the Parshall flume may be written as follows:

$$Q = 4bh^{3/2} \qquad (9)$$

where Q = instantaneous flow, cfs

h = head, ft

b = width of throat, ft

Taking into account the extremes of flow

$$b = \frac{1}{4}\left[\frac{Q_{\max}^{2/3} - Q_{\min}^{2/3}}{h_{\max} - h_{\min}}\right]^{3/2} \qquad (10)$$

For example, assume $Q_{max} \div Q_{min} = 8$, $h_{max} - h_{min} = 0.75$ ft

$Q_{max} = 3 \times$ average daily flow $= 3Q_{av}$

Then substituting in Equation (10)

$$b = 1.0Q_{av}$$

Thus with the foregoing assumptions a 12-in. flume is indicated for a 1-cfs average flow. Other factors such as arrangement of units and relative elevations must also be considered in flume design.

Venturi meters were not widely used in sewage treatment plant flow measurement, although they gave satisfactory service particularly when installed in plant effluent lines. Certain advantages accrued to measuring devices when installed on the effluent line, namely less tendency toward clogging and less rapid fluctuation in rate of flow from pumping and other causes as a result of the hydraulic damping effect of treatment units.

The Kennison nozzle, while not used in a large number of plants, proved to have decided advantages. Its space requirements are modest and the fact that it may be attached to the end of a pipe or conduit made installation possible at a number of locations not originally equipped for flow measurement.

In 126 plants at which flow measurement data were investigated, 72 had Parshall flumes and the remaining 54 were equipped with rectangular flumes and weirs. Of the 56 plants in the

TABLE 5.—Hourly Flow Study, Size and Type of Post

Post	Flow (mgd)	Date	Type of Station	Location	Composite Graph
(1)	(2)	(3)	(4)	(5)	(6)
Camp Blanding	3.700	Aug 44	AGF Divisional Training	Fla	Fig 6
Camp Claiborne #1	4.305	Aug 44	AGF Divisional Training	La	Fig 6
Camp Claiborne #2	1.309	Sep 43	AGF Divisional Training	La	Fig 6
Camp Breckinridge	2.280	Aug 44	AGF Divisional Training	Ky	Fig 6
Camp Rucker #1	1.340	Jan 44	AGF Divisional Training	Ala	Fig 6
Fort Leonard Wood	2.185	Aug 44	AGF Divisional Training	Mo	Fig 7
Fort George G Meade	1.235	Sep 43	AGF Divisional Training	Md	Fig 7
Camp Carson	2.290	Jul 44	AGF Divisional Training	Colo	Fig 8
Fort Jackson	3.560	May 44	AGF Divisional Training	S C	Fig 8
Fort Huachuca #1	0.208	Jul 43	AGF Divisional Training	Ariz	Fig 9
Fort Huachuca #2	0.636	Jan 44	AGF Divisional Training	Ariz	Fig 9
Camp Edwards	1.587	Dec 43	AGF Divisional Training	Mass	Fig 9
Camp Swift	1.672	Sep 44	AGF Divisional Training	Tex	Fig 10
Camp Van Dorn	2.402	Jul 43	AGF Divisional Training	Miss	Fig 10
Camp Atterbury	2.050	Aug 44	AGF Divisional Training	Ind	Fig 11
Camp Butner	2.337	Aug 44	AGF Divisional Training	N C	Fig 11
Camp Forrest	2.735	Jul 43	AGF Divisional Training	Tenn	Fig 11
Camp Pickett	2.200	Dec 43	AGF Divisional Training	Va	Fig 11
Camp Philips	1.690	Mar 44	AGF Divisional Training	Kans	Fig 12
Woodrow Wilson G H	0.140	Aug 44	ASF General Hospital	Va	Fig 13
Seymour Johnson Field	0.855	Sep 44	AAF Tactical Field & Flying Trg	N C	Fig 14
Drew Field	1.515	Aug 44	AAF Tactical Field & Flying Trg	Fla	Fig 14
Roswell AAF	0.416	—	AAF Tactical Field & Flying Trg	N Mex	Fig 14

TABLE 5 (cont.).—Hourly Flow Study, Size and Type of Post

Post	Flow (mgd)	Date	Type of Station	Location	Composite Graph
(1)	(2)	(3)	(4)	(5)	(6)
Turner AAF	0.436	Sep 43	AAF Tactical Field & Flying Trg	Ga	Fig 15
Hondo AAF	0.560	Jun 43	AAF Tactical Field & Flying Trg	Tex	Fig 15
Enid AAF	0.240	May 44	AAF Tactical Field & Flying Trg	Okla	Fig 15
March Field	0.367	Sep 42	AAF Tactical Field & Flying Trg	Calif	Fig 15
Fort Knox	2.997	Aug 44	AGF Armoured, Mechanical	Ky	Fig 16
Camp Hood (South)	3.160	Sep 44	Cavalry & Tank Destroyer	Tex	Fig 16
Camp Campbell	1.800	Nov 43	Training	Ky	Fig 16
Camp Fannin	1.173	Aug 44	AGF Replacement Training	Tex	Fig 17
Camp Gordon	2.020	May 43	AGF Replacement Training	Ga	Fig 17
Camp Roberts	1.923	Oct 43	AGF Replacement Training	Calif	Fig 17
Camp Joseph T Robinson	0.767	Sep 44	AGF Replacement Training	Ark	Fig 17
Camp Wheeler	1.980	Aug 44	AGF Replacement Training	Ga	Fig 17
Fort Bragg	4.528	Jul 43	AGF Artillery Training & Anti-	N C	Fig 18
Fort Sill	2.840	Nov 43	Aircraft Training	Okla	Fig 18
Buckley Field	0.878	Jan 44	AAF Technical Training School	Colo	Fig 19
Kearns AAF	0.647	Feb 44	AAF Repl Trg & Recl Center	Utah	Fig 19
Fort Dix	2.911	Aug 44	ASF Reception Center	N J	Fig 20
Camp Reynolds	1.400	Jun 44	ASF Reception Center	Pa	Fig 20
Bushnell G H	0.243	Sep 44	ASF General Hospital	Utah	Fig 21
Darnall G H	0.107	Mar 44	ASF General Hospital	Ky	Fig 21
Fitzsimons G H	0.535	Aug 44	ASF General Hospital	Colo	Fig 21
Camp Cooke	1.330	Jan 44	AGF Divisional Training	Calif	Fig 22
Savanna Proving Grds	0.284	Jul 44	ASF Ordinance Dept (housing)	Ill	Fig 23
Camp Myles Standish	2.073	Aug 44	ASF Stg Area & Disch & Repl Ctr	Mass	Fig 24
North Camp Polk	0.764	Jan 44	AGF Arm, M C & Tank Dest Trg	La	Fig 25

complete study group shown in Table 3, 39 were equipped with Parshall flumes.

Hourly Flow Variation

Each plant upon the survey list that was equipped with flow recording apparatus submitted a typical chart indicating variation in flow throughout the day. Considerable differences in flow distribution were manifest. However, it was possible to distinguish certain types of distribution by which the flow at most plants could be classified. Accordingly, hourly flow data were generalized by superimposing distribution curves for individual plants of a given type in order to obtain characteristic composite curves. The various groups from which composite curves were formed are presented in Table 5, together with pertinent information relating to plant type, size and location. Certain plants experienced unusual distributions of flow not amenable to compositing; (for example, Figs. 13 and 25) these have been presented to indicate the range of conditions observed. All curves of hourly flow distribution, composite and individual, are shown in Figs. 6 to 25. These data are based upon average

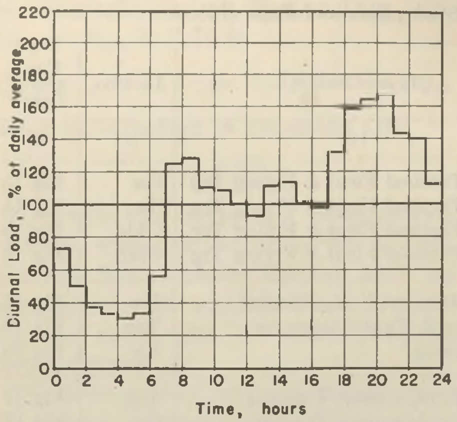


FIGURE 6.—Typical hourly flow variation. Composite graph of Camps Blanding, Claiborne No. 1, Claiborne No. 2, Breckinridge and Rucker No. 1.

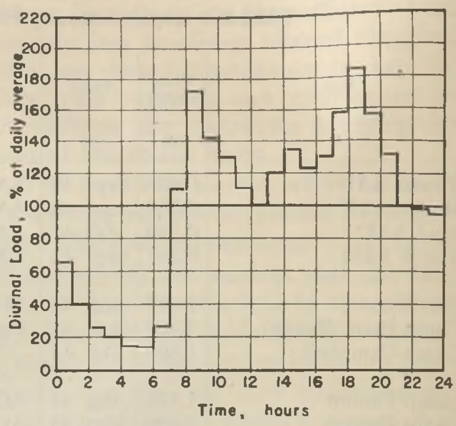


FIGURE 7.—Typical hourly flow variation. Composite graph of Fort Leonard Wood and Fort George G Meade.

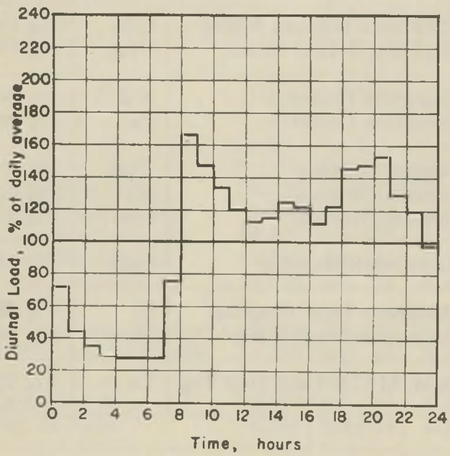


FIGURE 8.—Typical hourly flow variation. Composite graph of Camp Carson and Fort Jackson.

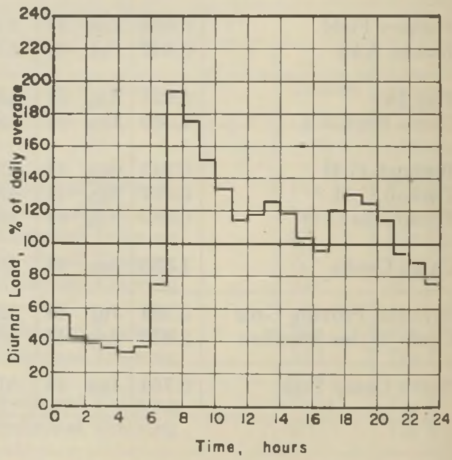


FIGURE 9.—Typical hourly flow variation. Composite graph of Fort Huachuca No. 1, Fort Huachuca No. 2 and Camp Edwards.

hourly flows; they do not indicate short fluctuations nor instantaneous peaks.

The horizontal time axis for composite curves does not invariably correspond to time of day as indicated by the clock. Differences between solar time and sidereal time, as well as differences in camp routine, resulted in differences among posts as to position of the flow curve with respect to the time axis. Compositing was accomplished by superimposing salient features of the individual curves, such as maxima and minima. Usually the times at which the curves crossed the axis representing 100 percent of daily average flow during the morning rise, were made to coincide. This usually involved a shift of only an hour or so for perhaps one or two plants in a compositing group; in the remaining plants identity with clock-time was preserved.

Of the types of hourly flow distribution exhibited in Figs. 6 to 25, the following are the most common:

(1) Monomodal (Figs. 13 and 21, General Hospitals).

(2) Bimodal:

(a) With evening maximum predominant (Figs. 6 and 20).

(b) With morning maximum predominant (Fig. 24).

(3) Trimodal (Fig. 15, Airfields, and Figs. 8 and 9).

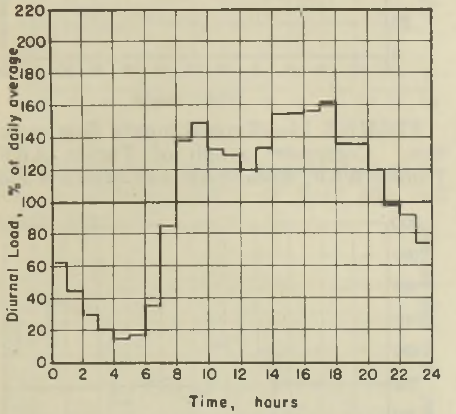


FIGURE 10.—Typical hourly flow variation. Composite graph of Camp Swift and Camp Van Dorn.

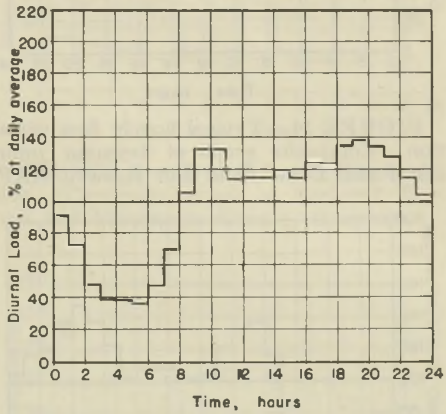


FIGURE 11.—Typical hourly flow variation. Composite graph of Camp Atterbury, Butner, Forrest and Pickett.

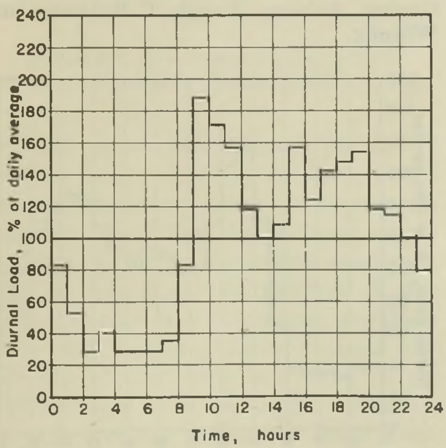


FIGURE 12.—Typical hourly flow variation at Camp Philips.

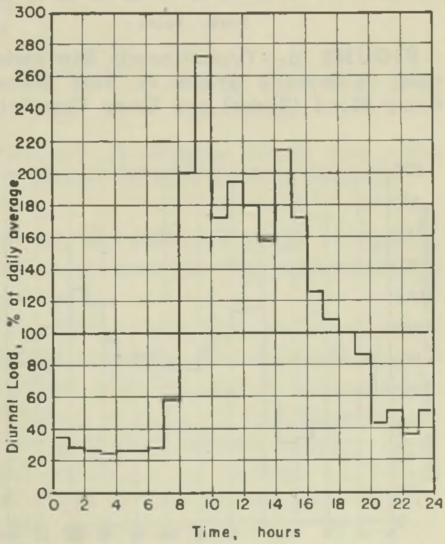


FIGURE 13.—Typical hourly flow variation at Woodrow Wilson General Hospital.

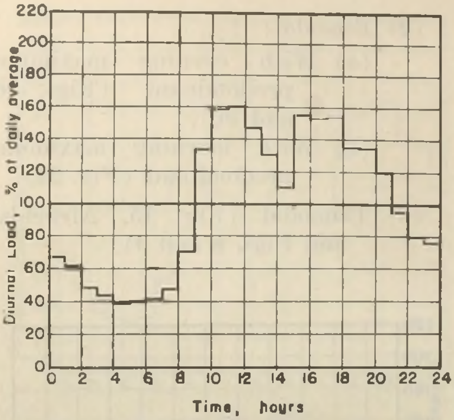


FIGURE 14.—Typical hourly flow variation. Composite graph of Seymour Johnson Field, Drew Field and Roswell AAF.

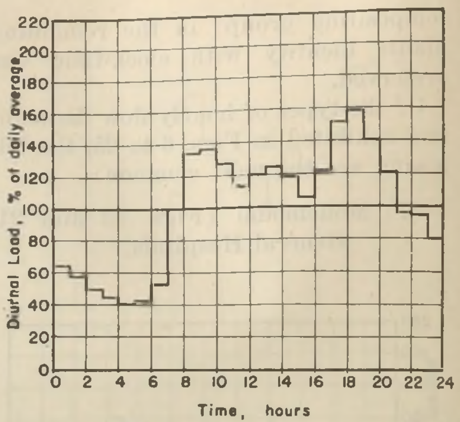


FIGURE 15.—Typical hourly flow variation. Composite graph of Turner AAF, Hondo AAF, Enid AAF and March Field.

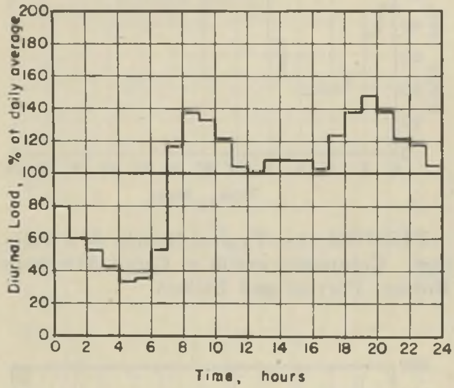


FIGURE 16.—Typical hourly flow variation. Composite graph of Fort Knox, Camp Hood (South) and Camp Campbell.

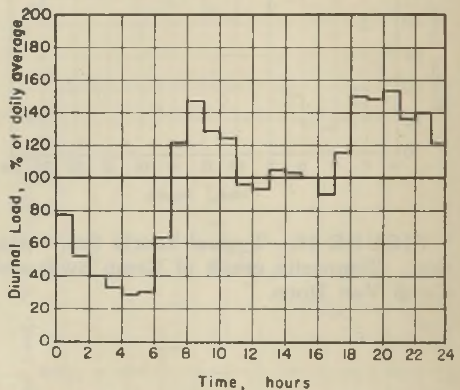


FIGURE 17.—Typical hourly flow variation. Composite graph of Camps Fannin, Gordon, Roberts, Joseph T Robinson and Wheeler.

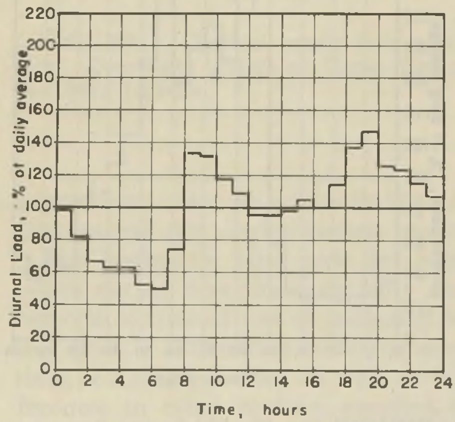


FIGURE 18.—Typical hourly flow variation. Composite graph of Fort Bragg and Fort Sill.

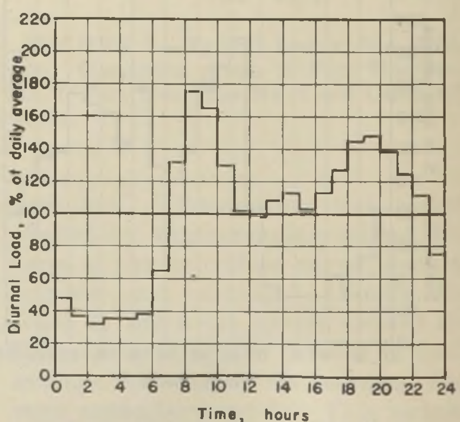


FIGURE 19.—Typical hourly flow variation. Composite graph of Buckley Field and Kearns AAF.

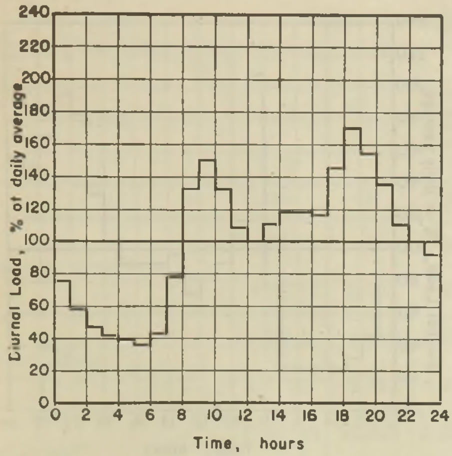


FIGURE 20.—Typical hourly flow variation. Composite graph of Fort Dix and Camp Reynolds.

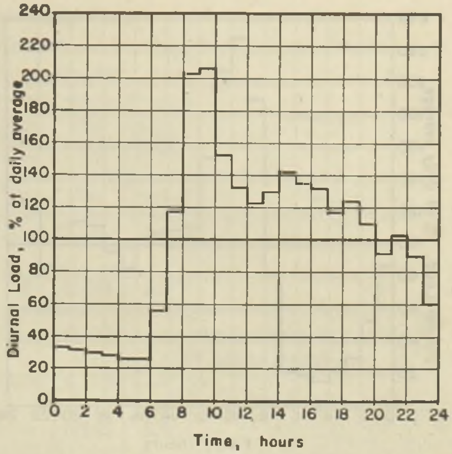


FIGURE 21.—Typical hourly flow variation. Composite graph of Bushnell, Darnall and Fitzsimons General Hospitals.

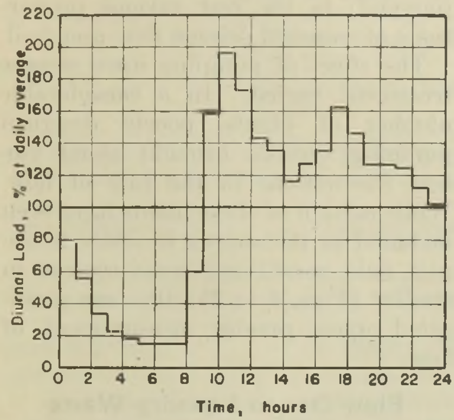


FIGURE 22.—Typical hourly flow variation at Camp Cooke.

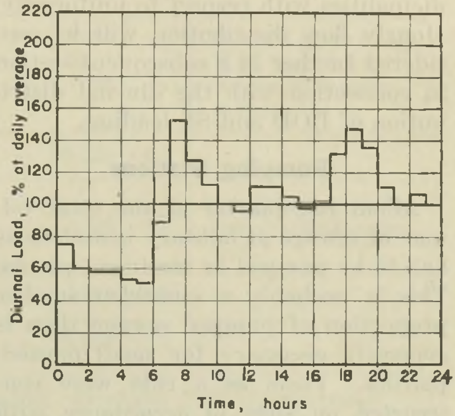


FIGURE 23.—Typical hourly flow variation at Savanna Proving Grounds.

Most flow distributions exhibit a greater range of variation than would normally occur in towns of comparable size. In five of the distributions shown in Figs. 6 to 25, the night minimum falls to less than 30 percent of the daily average flow; in eleven distributions the morning peak exceeds 160 percent of the daily average; in six distributions the evening maximum exceeds 160 percent of the daily average; in seven distributions the flow at midday falls below the daily average. In about one quarter of the individual plants, the ratio of range of flow to the daily average flow exceeded 200 percent.

In general, the flow distribution reflected promptly the sequence of post routine affecting water consumption. While a variety of distributions occurred, typical patterns emerge. Normal conditions for airfields are shown in Figs. 14 and 15, and for ground force cantonments in Figs. 16 and 17. In some of the larger posts, such as Forts Bragg and Sill (Figure 18), flow distribution approached that of mu-

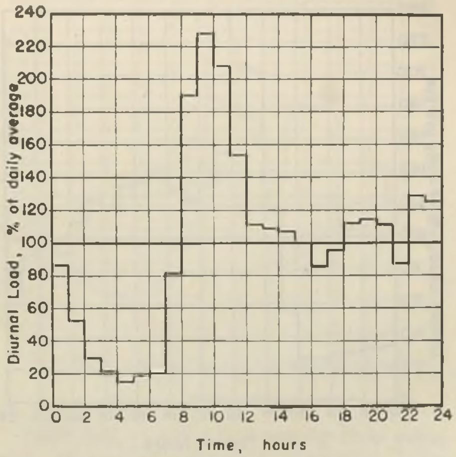


FIGURE 24.—Typical hourly flow variation at Camp Myles Standish.

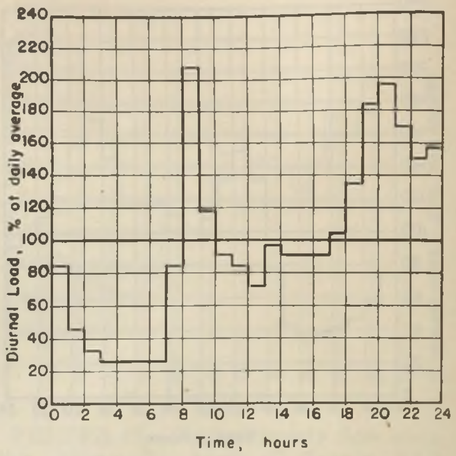


FIGURE 25.—Typical hourly flow variation at North Camp Polk.

nicipalities with respect to uniformity. Hourly flow distribution will be considered further in a subsequent section in connection with the diurnal distribution of BOD and SS loading.

Pumping Stations

About one-quarter of the total volume of sewage at military installations had to be pumped to treatment plants. This is probably a somewhat smaller proportion of pumped sewage than is generally necessary for small municipalities. Posts as a rule were constructed on sites in accordance with overall plans that, among other considerations, took into account efficiency of the various utilities. However, topographical conditions were not always favorable. At Fort Dix, for example, 13 sewage lift stations were required and at Fort Monmouth 10 lift stations. A breakdown of pumping station data is presented in the following compilation, as based upon information from 110 plants:

Number of Plants	53	14	11	9	6	4	13
Percent of Sewage Pumped	0	0–10	10–25	25–50	50–75	75–100	100

It is seen that in about one-half the posts the flow was by gravity; and in about 10 percent the flow was entirely pumped; in the rest various proportions of pumped sewage flow occurred.

The effect of pumping upon sewage treatment varied. In a considerable number of plants, poorly designed pumping systems brought about violent fluctuations in the rate of flow. While certain of these plants have been included in the survey in order to depict field conditions, none appear in studies (Figs. 6 to 25) that are predicated upon precise measurement of flow.

Flow Due to Laundry Waste

Volumes of laundry flow experienced at nine posts are presented in Table 6, together with other relevant data. The volume of laundry waste amounted to about 5 gcd or about 7 percent of the total sewage flow. This figure is in line with a corresponding estimate of 6.5 percent by Gehm, based upon flow data collected from four small municipalities.

Despite the fact that laundry waste constituted a relatively low proportion of the overall volume treated, it sometimes occasioned operating difficulties

TABLE 6.—Laundry Waste Flow

Post	Date	Pop'n (a)	Sewage Flow (mgd)	Laundry			Remarks
				Hours Operated	Flow (mgd) (b)	Flow (gcd)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
South Camp Polk	1/25/45	14,266	1.44	16	0.155	10.9	
	1/31/45	13,757	1.30	16	0.124	9.0	
Fort Riley	1/23/45	4,340	0.425	Cont	0.110	—	Laundry serves several other stations.
	2/2/45	3,142	0.454	Cont	0.118	—	
Fort Sill	1/23/45	31,399	2.250	—	0.182	5.8	
	1/31/45	31,594	2.100	—	0.144	4.6	
Camp Roberts	1/16/45	25,913	1.353	7½	0.112	4.3	
	1/25/45	25,631	1.572	7½	0.121	4.7	
Fort Knox	1/10/45	39,582	3.83	16	0.036	—	Officers' Laundry only, E.M. Laundry not connected to sewer system.
	1/19/45	39,531	3.93	16	0.036	—	
Fort Jackson	—	53,800	3.21	—	0.220	4.1	
Camp Crowder	—	32,700	2.06	—	0.139	3.7	Laundry served additional 5,000 population.
Fort Leonard Wood	—	41,400	1.814	—	0.163	3.9	
Camp Atterbury	—	21,600	1.77	—	0.124	5.7	

(a) Military population—resident and non-resident. (Included POW in some cases.)
(b) Actual reported or computed from data given.

by interfering with biochemical processes. This was to be attributed largely to the fact that laundry waste was often discharged in "slugs" at times when treatment units were already carrying peak loads.

The problem was rendered more difficult at a number of posts at which laundry work was done for several nearby installations. The laundry at Fort Riley, for example, served several other stations and the waste constituted more than 25 percent of the total volume of flow. At Camp Forrest, Tenn., the rate of flow from the laundry varied throughout the day from a mere trickle to 450 gpm. The laundry was operated 24 hours a day, but during the night shift only one-third of the full capacity was utilized. Average flow for day shifts was approximately 240 gpm and 120 gpm during the night shift.

The following compilation is indicative of distribution of laundry flow:

Camp	Pop'n (approx)	Av Flow Laundry Waste (mgd)	Ratio Peak to Av Flow
A	15,000	0.09	2.3
B	14,000	0.07	3.2
C	56,000	0.30	2.5

An approximate figure on volume of waste appears to have been 5 gallons per lb of clothes laundered. In warm months about 1.2 lb of clothes per capita were washed daily; in winter the amount dropped to about 0.6 lb.

In general, laundry waste constituted more of a problem in the smaller installations. However, by January, 1943, many of the more flagrant instances of interference with sewage treatment had been ameliorated. In a number of cases adjustments were made in laundry practice; in others, holding tanks were provided in order to obtain a more uniform distribution of flow to the plant. At Fort Sill, for example, the problem of distribution was met by installation of a holding basin. In order to keep the raw sewage peak flow within pumping capacity and to keep the second stage filters turning continuously at low loads, a holding basin was constructed at the laundry to impound the waste. The flow from the laundry was allowed to accumulate during the day and then released over a 6-hr period starting at midnight.

Characteristics of Sewerage System Affecting Flow

The length and slope of the sewers carrying sewage to a plant determine to a certain extent the variation of flow. The action of the sewerage system is particularly important with respect to fluctuations of duration of the order of one hour. The extent to which minor discontinuities in flow entering the system are damped out in passage, depends primarily upon the length and time of travel. A parameter providing an approximate measure of the influence of sewers is ''lineal ft of sewers per mgd (average daily flow).'' This quantity is roughly proportional to the detention period in sewers and, as may be seen from Fig. 26, reflects the damping action.

A definite reduction in flow variation, as measured by the ratio of the range in flow to the daily average, is observed to occur with high values of the parameter. The scatter of points on Fig. 26 is to be attributed to the fact that the slope factor was not taken into account. Quantitative data relating to slope were lacking. It is not expected that sewers in small cities and towns would be capable of exerting so marked a reduction in peak flow. The damping of the flow was rather to be attributed to the abruptness of the morning rise, which takes place within an hour or two—a range of time in which sewer storage effect can be appreciable.

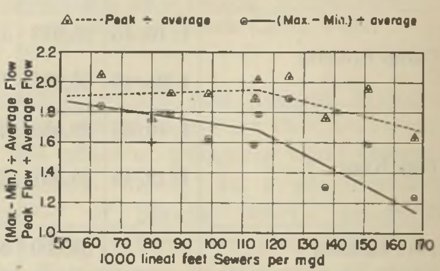


FIGURE 26.—Relation between length of sewers and variation of plant flow. Data are taken from ten typical posts. Sewer lengths do not include house connections. Curves are drawn through group medians.

It is to be emphasized that the parameter, lineal feet per mgd of flow, does not reflect the size of the installation. Very little correlation between the size of post and flow fluctuation was exhibited. Figure 27, with data from 25 posts, shows a nearly constant average ratio of peak to average flow of about 2.0 for a population range of about 30 to 1. On Fig. 27, as a matter of reference, is included the relationship between peak flow and population that is specified for sewer design in the Engineering Manual. In view of the damping effect of sewers, it is to be expected that higher peak flows will occur in upstream portions of sewerage systems than at sewage treatment plants. The difference in the two curves of Fig. 27 reflects the influence

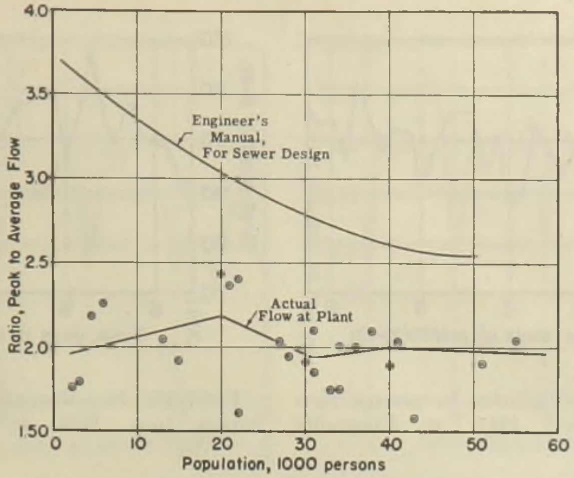


FIGURE 27.—Relation between post population and variation of flow. Data for lower curve are taken from twenty-five typical posts based on flow at plant. Curve is drawn through group medians. Data for upper curve are taken from OCE Engineering Manual, for sewer design. Difference between curves reflects in part the dampening effect of sewerage system.

of the sewerage system and, in part, a factor of safety usual in conservative hydraulic practice.

The reversal of direction in the lower curve does not imply a significant trend in the points; it denotes merely sampling fluctuations characteristic of limited data. The method of fitting that was employed, in which the curve was drawn through group medians, has provided simple and objective results. Accordingly, it has been used with a number of graphs wherein application of the tedious method of least squares appeared to be unwarranted. The method does contain an element of subjectivity in subdivision of points into groups of arbitrary size. The selection of group size, to a limited extent, determines the trend of the resulting curve. This feature, however, can be turned to advantage. By trying groups of different sizes in succession, the range in location of the median curve can be ascertained and an idea formed as to the degree of precision inherent in the data.

Daily Flow Variation

Having considered the diurnal distribution of flow, it is pertinent to take up the matter of variation in flow from day to day throughout the week and month.

In Figs. 28 to 33 are presented data indicating flow variation at Kingsville Field, Fort Jackson and Woodrow Wilson General Hospital for two months each. Differences in the daily distribution between two months at the same plant are a sign of seasonal effects and differences in post routine. During June, 1944, at Kingsville Field, for example, sewage flow on Sundays fell below the monthly average. During March, 1943, however, Sunday flow sometimes exceeded the monthly average. During both months, the weekly maximum usually occurred on Wednesday, Thursday or Friday.

At Fort Jackson, Sunday minima were pronounced, particularly during the summer months, amounting to about 80 percent of monthly average flow. Maxima occurred usually on Mondays and Fridays. It should be observed that despite the fact that Fort Jackson, a divisional training post, is about ten times larger than Kingsville Field, a small naval air station, it exhibits a variation in daily flow during the month that is consider-

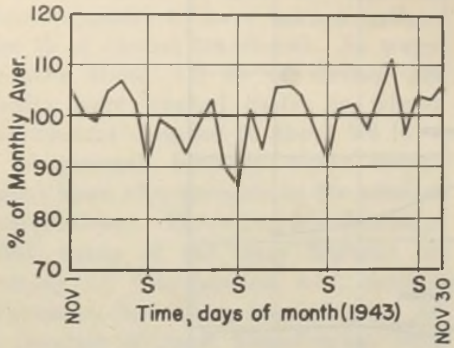


FIGURE 28.—Variation in sewage flow during November, 1943, at Kingsville Field.

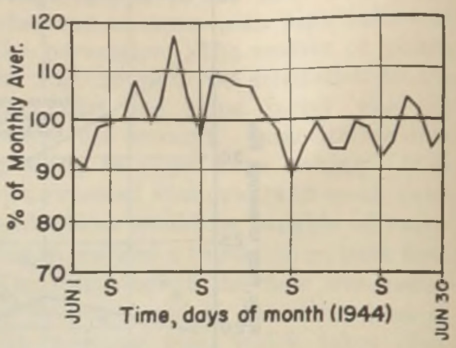


FIGURE 29.—Variation in sewage flow during June, 1944, at Kingsville Field.

ably larger than the latter. In military cantonments little correlation was shown between size and variability of population. In addition to the marked weekly fluctuation in population at Fort Jackson during August, 1943, a downward trend indicating troop movement is evident throughout the month. Trends upward and downward were common in posts surveyed.

At Woodrow Wilson General Hospital an extreme variation in flow may be noted. This reflects hospital routine primarily rather than large population changes. During Sundays the staff was largely inoperative. Large fluctuations of flow at the General Hospitals were usual and account in part for erratic and substandard performance sometimes noted.

Relationship Between Water Consumption and Sewage Flow

On Figs. 34 to 37 are shown the distribution of daily flow of water and sewage at Camp Edwards during the months of May and February, 1943. While the distributions followed the same trends, certain differences are noted. During May, the variation in water consumption was significantly greater than in sewage flow. Sewage flow did not exhibit the marked Sunday minima of the water consumption distribution. Water consumption was relatively large during the first and second weeks and then declined during the last half of the month; the sewage flow, on the other hand, showed a slight but persistent increase throughout May, 1943. In February, the water flow increased throughout the month, whereas the average weekly sewage flow remained nearly constant. The variation in rate of flow of the two was about the same, however, an abnormally high sewage flow peak occurred during the first week of February that had no counterpart on the water consumption distribution.

At Camp Edwards, the sewer system comprised about 62,000 ft of mains, 120,000 ft of branches, and 66,000 ft of building connections for a total of about 47 miles of sewers. The sewers varied from 6 to 24 inches in diameter. Gradients were generally moderate to flat. While infiltration was negligible, surface water sometimes entered the sewer system through unauthorized removal of manhole covers to drain low areas.

Figures 38 and 39 indicate the variation in water and sewage flow during the week at Camp Edwards, as based upon a twelve-month record. The two distributions are quite similar, with

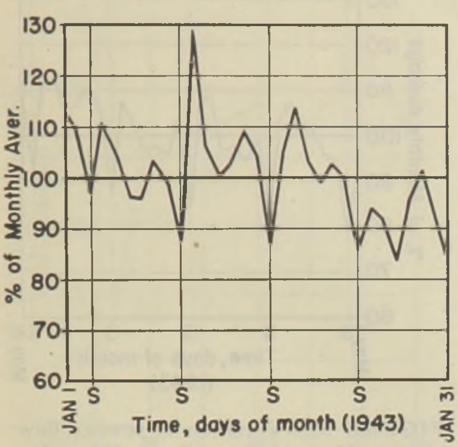


FIGURE 30.—Variation in sewage flow during January, 1943, at Fort Jackson.

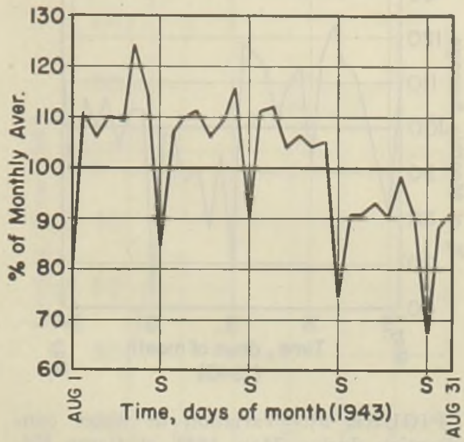


FIGURE 31.—Variation in sewage flow during August, 1943, at Fort Jackson.

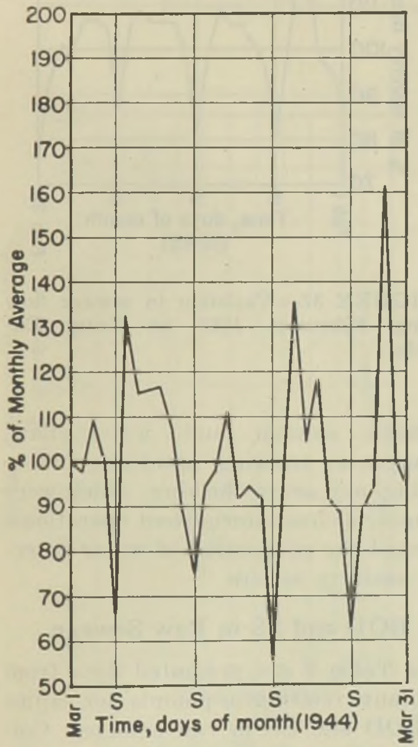


FIGURE 32.

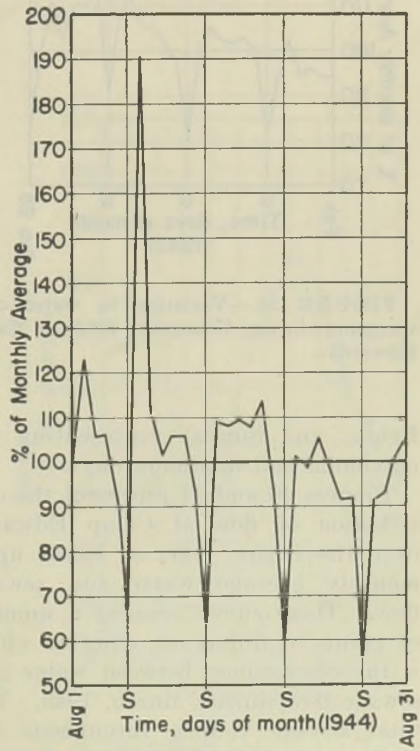


FIGURE 33.

FIGURES 32 and 33.—Variation in sewage flow during March and August, 1944, at Woodrow Wilson General Hospital.

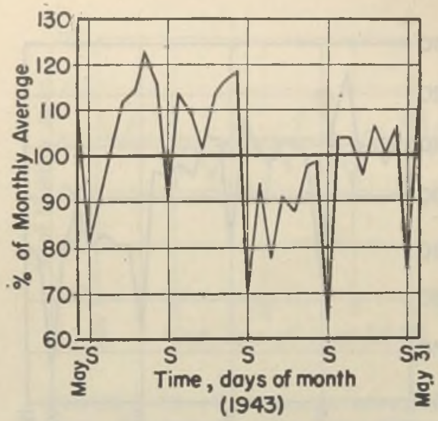


FIGURE 34.—Variation in water consumption during May, 1943, at Camp Edwards.

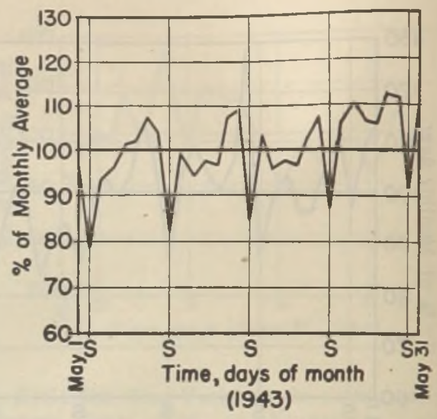


FIGURE 36.—Variation in sewage flow during May, 1943, at Camp Edwards.

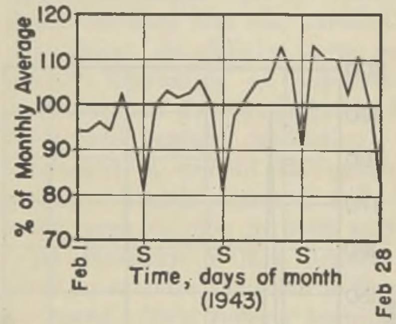


FIGURE 35.—Variation in water consumption during February, 1943, at Camp Edwards.

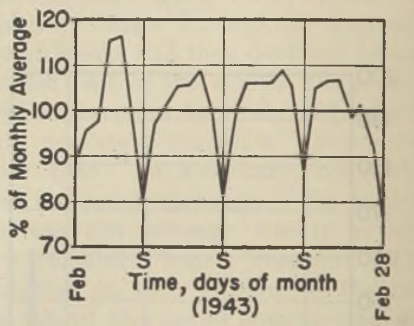


FIGURE 37.—Variation in sewage flow during February, 1943, at Camp Edwards.

Friday and Sunday representing the maximum and minimum days.

Figures 40 and 41 represent the distribution of flow at Camp Edwards over the entire year, as based upon monthly average water and sewage flow. These curves display a number of points of difference, chief of which is the discrepancy between water and sewage flow during March, 1943. The total sewage volume throughout the year amounted to 86 percent of water volume. The 14 percent difference in the two volumes was sufficient to account for the fairly large temporary disparities noted from time to time between sewage and water flow. Changes in training schedules, bivouacking, and sewer flushing, which were all more or less intermittent operations, affected the proportion of water entering sanitary sewers.

BOD and SS in Raw Sewage

In Table 7 are presented data from 44 plants relating to pounds per capita of BOD and SS in raw sewage. Columns (2) and (5) in Table 7 give mean values as averaged over entire period of study. These quantities were averaged by weighting the figures for each post according to the total BOD

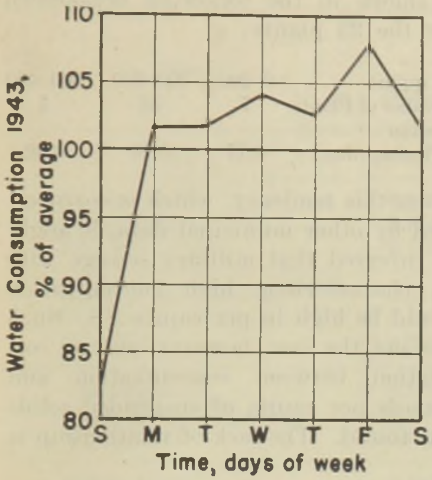


FIGURE 38.

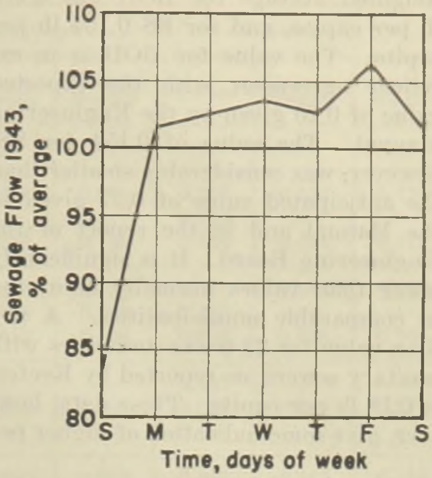


FIGURE 39.

FIGURES 38 and 39.—Variation in water and sewage flow during week at Camp Edwards. Curves represent average daily values for 1943.

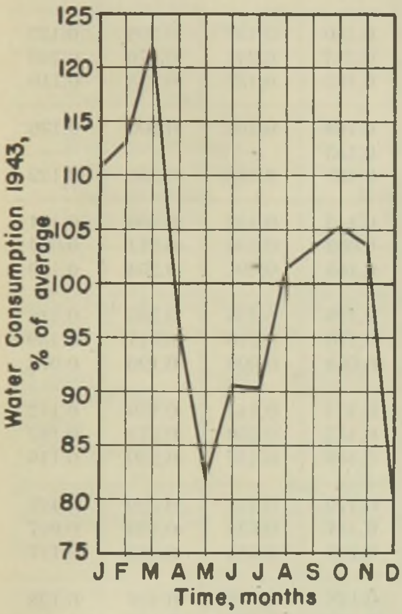


FIGURE 40.

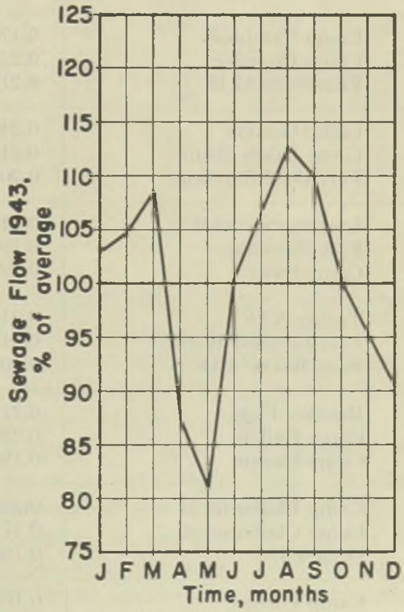


FIGURE 41.

FIGURES 40 and 41.—Variation in water and sewage flow during 1943 at Camp Edwards. Total sewage flow during year was 86 percent of total water consumption.

(or SS) produced. The overall weighted average for BOD was 0.192 lb per capita, and for SS 0.154 lb per capita. The value for BOD is in excellent agreement with the expected value of 0.20 given by the Engineering Manual. The value of 0.154 for SS, however, was considerably smaller than the anticipated value of 0.27 given in the Manual and in the report of the Engineering Board. It is significantly lower than values normally occurring in comparable municipalities. A median value for 23 towns and cities with sanitary sewers, as reported by Keefer, is 0.18 lb per capita. These data, however, give some indication of higher per capita SS in concentrated sewage, as is shown in the following breakdown for the 23 plants:

SS (ppm)	0–200	200–300	300–400
Number of Plants	7	13	3
Median (lb/cap/day)	0.17	0.18	0.22

From this tendency, which is corroborated by other municipal data, it might be inferred that military sewage with its characteristic high concentration would be high in per capita SS. Such was not the case, however, and no correlation between concentration and pounds per capita of suspended solids was found. The lack of relationship is

TABLE 7.—BOD and SS, Pounds per Capita per Day in Raw Sewage

Post	BOD (lb/cap/day)			SS (lb/cap/day)		
	Average	Maximum, Monthly	Minimum, Monthly	Average	Maximum, Monthly	Minimum, Monthly
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Camp Campbell	0.179	0.226	0.150	0.157	0.201	0.123
Camp Crowder	0.244	0.292	0.213	0.241	0.276	0.210
Fitzsimons G H	0.217	0.282	0.163	0.137	0.185	0.110
Camp Gordon	0.218	0.270	0.168	0.166	0.203	0.136
Great Lakes (Main)	0.214	0.279	0.153	—	—	—
Fort Benj Harrison	0.201	0.301	0.129	0.188	0.351	0.122
Lockbourne AAB	0.215	0.276	0.143	0.157	0.206	0.121
Fort Sheridan	0.190	0.283	0.092	0.198	0.311	0.084
Camp Swift	0.228	0.276	0.165	0.201	0.254	0.160
Turner AAF	0.211	0.290	0.128	0.195	0.335	0.118
Fort Leonard Wood	0.215	0.270	0.176	0.210	0.311	0.159
Boca Raton AAF	0.103	0.119	0.084	0.099	0.120	0.078
Buckley Field	0.213	0.279	0.154	0.144	0.189	0.112
Camp Callan	0.230	0.280	0.155	0.250	0.274	0.182
Camp Carson	0.180	0.300	0.146	0.144	0.220	0.116
Camp Claiborne #1	0.262	0.398	0.179	0.197	0.329	0.121
Camp Claiborne #2	0.172	0.261	0.151	0.124	0.163	0.097
Drew Field	0.186	0.258	0.108	0.186	0.280	0.117
Camp Forrest	0.195	0.374	0.128	0.186	0.412	0.128
Fort Jackson	0.183	0.212	0.128	0.150	0.198	0.124
Fort Knox	0.170	0.212	0.115	0.159	0.200	0.109
Fort Bragg	0.225	0.310	0.178	0.157	0.208	0.124
Fort Dix	0.167	0.239	0.113	0.174	0.250	0.121
Kearns AAF	0.198	—	—	0.250	—	—

TABLE 7 (cont.).—BOD and SS, Pounds per Capita per Day in Raw Sewage

Post	BOD (lb/cap/day)			SS (lb/cap/day)		
	Average	Maximum, Monthly	Minimum, Monthly	Average	Maximum, Monthly	Minimum, Monthly
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Keesler AAF	0.148	0.206	0.099	0.213	0.338	0.129
Camp Kilmer	0.164	—	—	0.185	—	—
Fort Sill	0.179	0.216	0.135	0.160	0.194	0.113
Camp Myles Standish	0.281	0.398	0.159	0.110	0.162	0.069
Fort Francis E Warren	0.264	0.386	0.169	0.166	—	—
Fort Riley	0.209	0.318	0.138	0.292	0.496	0.191
Camp Roberts	0.178	—	—	0.141	—	—
Camp Joseph T Robinson	0.175	0.241	0.144	0.097	0.123	0.071
Topeka AAB	0.155	—	—	0.183	—	—
Willow Grove	0.137	0.220	0.093	0.182	0.312	0.126
Camp Butner	0.175	0.251	0.144	0.155	0.204	0.108
Fort Monmouth	0.107	—	—	0.110	—	—
Camp Shelby #2	0.171	0.249	0.086	0.148	0.222	0.082
Great Lakes (Green Bay)	0.209	0.281	0.142	—	—	—
Camp Blanding	0.086	0.104	0.076	0.139	0.192	0.114
Davisville	0.143	0.254	0.107	0.187	0.241	0.139
Camp Funston	0.185	0.391	0.120	0.216	—	—
Seymour Johnson Field	0.189	0.284	0.144	0.151	0.226	0.109
Camp Peary	0.260	0.354	0.133	0.209	0.250	0.137
Camp Wheeler	0.190	0.252	0.119	0.131	0.155	0.084
Weighted Average	0.192	—	—	0.154	—	—
Arithmetic Average	—	0.277	0.138	—	0.245	0.121
Upper Quartile	0.214	—	—	0.197	—	—
Lower Quartile	0.171	—	—	0.144	—	—

indicated in the following compilation for 34 plants at military installations:

SS (ppm)	0–200	200–300	300–400
Number of Plants	2	19	11
Median (lb/cap/day)	0.19	0.16	0.185

A similar breakdown for BOD from 36 plants on the survey list is as follows:

BOD (ppm)	200–300	300–400	400–500
Number of Plants	11	17	7
Median (lb/cap/day)	0.20	0.185	0.23

The semi-interquartile range for BOD was 0.022 or about 12 percent of the average value. Half the values fell within range 0.172 to 0.214. The dispersion of SS is somewhat larger; the semi-interquatrile range is .026 or 17 percent of the average.

Column (3) gives the amount of BOD on a per capita basis for each plant during the maximum month of the period of study. The figures in columns (4), (6) and (7) were calculated in an analogous way and represent the range of condition experienced at each plant. This range runs from about 80 percent to 160 percent of the medians of BOD and SS.

The variation in BOD and SS exhibited in Table 7 may be attributed to the following factors:

(1) Training schedules that involved bivouacking to a significant degree.
(2) Laundry wastes, particularly at installations that did laundry work for adjacent posts.
(3) Imperfections inherent in evaluating population, flow and concentration.
(4) Differences in efficiency of grease recovery and trap maintenance programs.

Little, if any, correlation between BOD and SS was observed. Low values of BOD were not necessarily associated with low values of SS. The correlation diagram between BOD and SS (Figure 45) exhibits wide dispersion. Of the 42 plants, each represented by a point on Figure 45, only 13 have a higher SS concentration than BOD.

Figures 42, 43 and 44 are scatter diagrams showing the relationship existing between BOD and SS at Fort Knox, Buckley Field and Camp Campbell. Each point on these graphs represents a monthly average. Additional pertinent information is presented in the following compilation:

Moreover, no significant correlation with BOD could be detected at this post. Data from most plants, however, accorded more closely with that from Buckley, Knox and Campbell. It is evident from Figures 42 to 45, that while BOD and SS could exhibit good correlation in a particular plant, little relationship between BOD and SS was manifest for the entire group of plants.

The correlation of BOD with SS, when considered in connection with the dispersion of each, had interpretative value. In forming an estimate of conditions obtaining at a particular military installation, the following inferences could be made:

(a) High standard deviations and fair correlation: considerable bivouacking, excellent analytical work.
(b) High standard deviations and poor correlation: considerable bivouacking, poor analytical work or inadequate sampling.
(c) High or low average values, or inverse correlation: unusual conditions operative, post atypical.

Correlation Between BOD and SS in Pounds per Capita

Post	Flow (mgd)	Arithmetic Mean and Standard Deviation		Coefficient of Correlation* and Probable Error
		BOD (lb/cap/day)	SS (lb/cap/day)	
Buckley	0.907	0.214±0.032	0.145±0.023	0.640±0.097
Knox	3.200	0.180±0.031	0.158±0.031	0.210±0.160
Riley	0.504	0.206±0.059	0.300±0.117	0.031±0.240

* For *perfect* correlation, coefficient of correlation = 1.0; for *perfect* inverse correlation, coefficient of correlation = − 1.0; if *no relationship exists*, coefficient of correlation = 0.0.

Standard deviations reflect moderate dispersion of monthly averages about the mean for the overall period of study. Thus, during about ⅔ of the months the per capita weight of BOD at Buckley Field fell in the range 0.18 to 0.25. A large variation in production of SS occurred at Fort Riley.

Sewage Volume Per Capita

In 48 plants, the average volume of sewage amounted to 74.7 gallons per capita daily. This was in reasonable agreement with an anticipated amount of 70 gcd stated in the Engineering Manual for airfields, camps, canton-

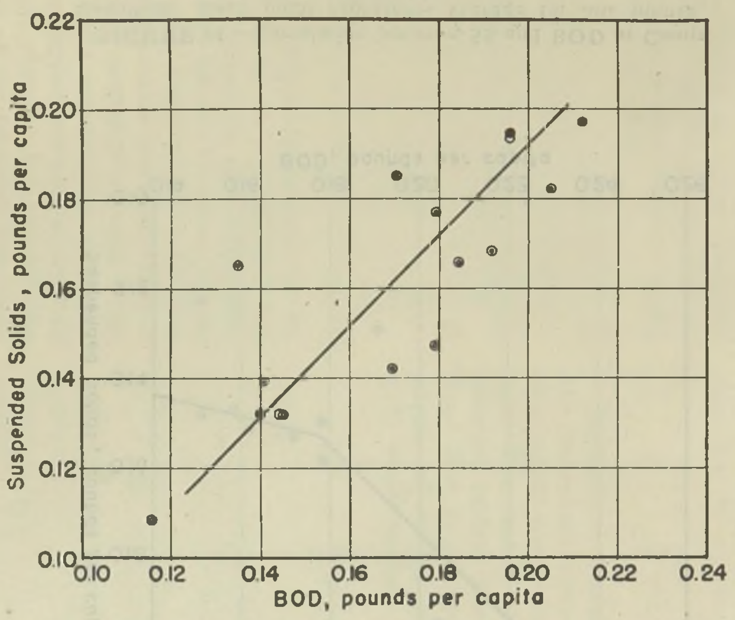


FIGURE 42.—Correlation between SS and BOD at Fort Knox. Each point represents average for one month. Curve drawn through group medians.

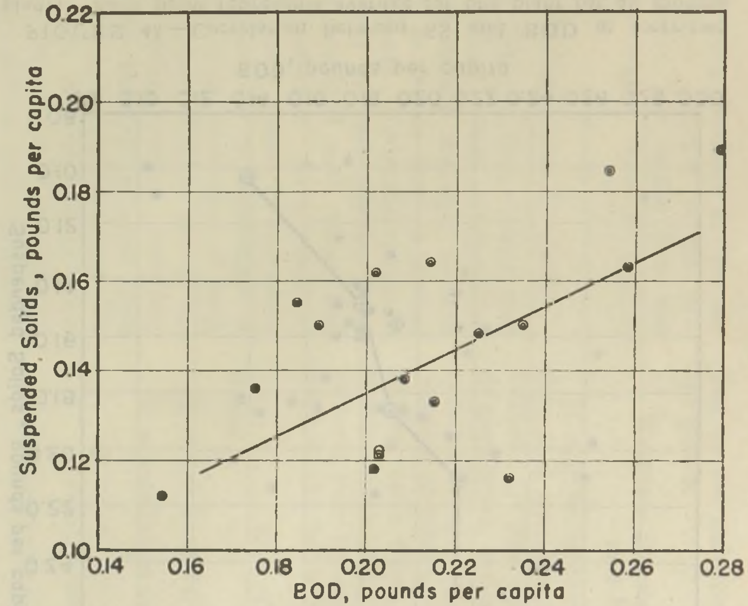


FIGURE 43.—Correlation between SS and BOD at Buckley Field. Each point represents average for one month. Curve drawn through group medians.

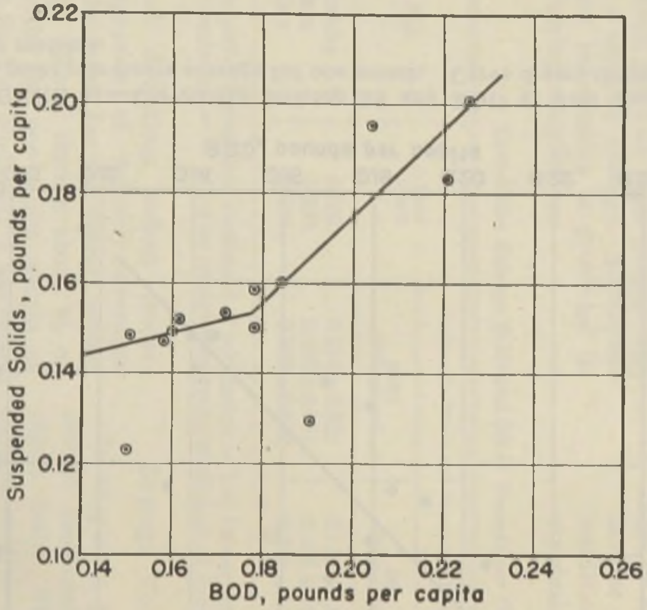


FIGURE 44.—Correlation between SS and BOD at Camp Campbell. Each point represents average for one month. Curve drawn through group medians.

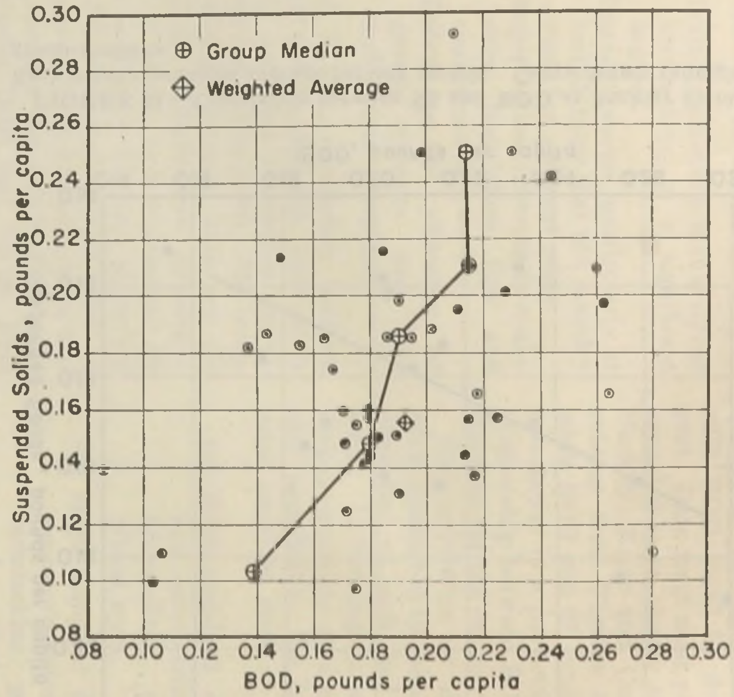


FIGURE 45.—Correlation between SS and BOD at forty-two plants. Each point represents average for one plant for all months during operating period studied. Curve drawn through group medians.

ments and troop facilities of the mobilization type. Earlier in the program, however, per capita flow was significantly higher than 70 gcd in many plants.

Water Conservation Program

In July 1941, it was evident that steps to reduce water consumption would be necessary not only to save water, but to reduce flow at sewage treatment plants. Hydraulic engineers were retained by each Service Command and provided with water leak detection apparatus. As a result of surveys of water waste, recommendations were made as to installation of urinal flush valves, correction of leaky valves and more efficient use of refrigeration cooling water. Ancillary supplies were installed at some posts for vehicle and airplane washing. Restrictions were instituted as to water use, such as shortening of the time for shower baths during peak occupancy. These modifications brought about a notable reduction in water consumption. Kessler and Norgaard (27) reported results in four posts as shown in the following table:

Post	Median Flow for Three-Month Periods (gcd)			
	July–Sept, 1941	Oct–Dec, 1941	Jan–Mar, 1942	Apr–June, 1942
A	119	87	41	40
B	116	—	60	46
C	100	72	78	44
D	50	38	39	40

These data reflect the amount of reduction that can be accomplished, but are not in themselves sufficiently comprehensive to delimit the consequences of the overall water conservation program. Conservation was most effective when initial consumption was high; with moderate consumption, effects of water consumption were often obscured by seasonal changes, but nevertheless the success of the program was manifest in the reduction of flow load at the plants.

Relation Between Population and Flow

It may be assumed that the sewage flow at a post has two components as follows: (i) flow directly proportional to population, reflecting water use by personnel; and (ii) base flow, independent of population, depending more or less upon fixed water-use requirements, infiltration, etc. These components may be expressed mathematically in the following equation:

$$Q = cP + k \qquad (11)$$

where

Q = flow in gallons per day

P = population (actual)

c = constant of proportionality of component (i)

k = component (ii), gallons per day

With this assumption, the per capita flow may be written,

$$\frac{Q}{P} = c + \frac{k}{P} \qquad (12)$$

Normally it would be expected that the component independent of population would not be high, and that sewage flow would follow population changes rather closely. Thus, if the fixed flow represented 20 gallons per capita of *design* capacity, Equation (12) would become

$$\frac{Q}{P} = c + \frac{20}{p}$$

where

$$p = \frac{\text{actual population}}{\text{design population}}$$

c = 30 to 50 under typical conditions in cantonments.

While the foregoing analysis held approximately for most plants, in others a rather surprising lack of dependence of flow upon population was

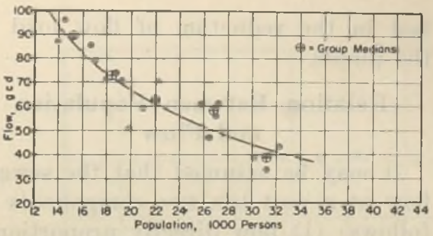


FIGURE 46.—Relation between population and daily flow per capita at Drew Field. Equation of curve, gcd = 1,320,000 ÷ population.

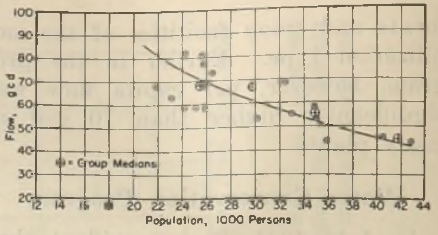


FIGURE 47.—Relation between population and daily flow per capita at Camp Gordon. Equation of curve, gcd = 1,800,000 ÷ population.

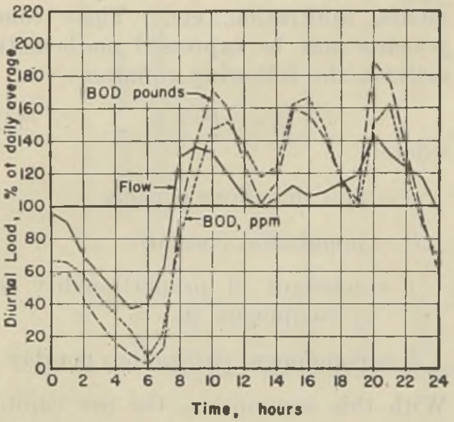


FIGURE 48.—Typical hourly variation in flow and BOD. Data represent average rate for each hour. Composite—Camp Joseph T Robinson, Camp Hood (South) and Fort Sill.

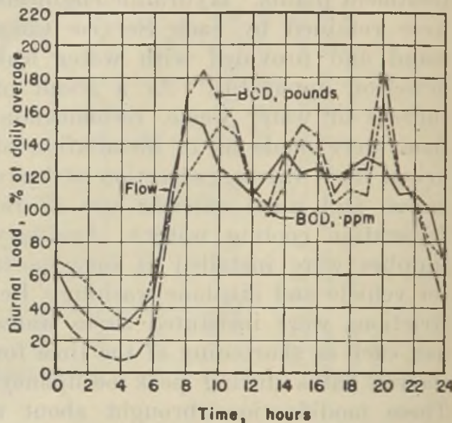


FIGURE 49.—Typical hourly variation in flow and BOD. Data represent average rate for each hour. Composite—Fort Leonard Wood, Camp Campbell, Camp Roberts and Camp Swift.

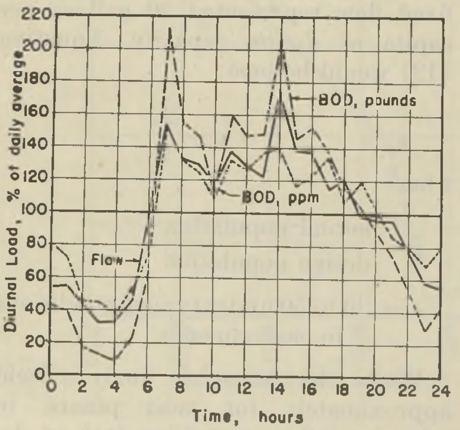


FIGURE 50.—Typical hourly variation in flow and BOD. Data represent average rate for each hour. Composite—Enid AAF, Lake Charles AAF and Will Rogers Field.

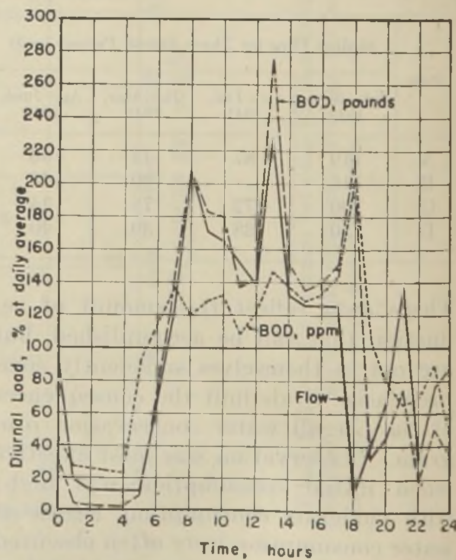


FIGURE 51.—Typical hourly variation in flow and BOD. Data represent average rate for each hour. Fitzsimons General Hospital.

observed. At Drew Field and Camp Gordon, for example, fixed flow was found to be predominant, and an excellent agreement with Equation (12) obtained if c was zero. The data for these posts are plotted on Figs. 46 and 47 in which the curves are hyperbolas representing Equation (12) with $c = 0$. Both posts were located in the southern part of the country, hence the data were not obscured by seasonal variations in water consumption to such a degree as occurred in more northerly plants. The goodness of fit obtained clearly indicates a tendency for flow at these posts to be constant, regardless of population shift. Corroborating evidence appeared at other posts; Greeley and Chase (26) presented data showing a drop in water pumpage (in gcd) with increase in troop concentration that are in accord with data of Figs. 46 and 47. Additional data pertaining to the inter-relationship of population, flow and concentration are presented in loading and

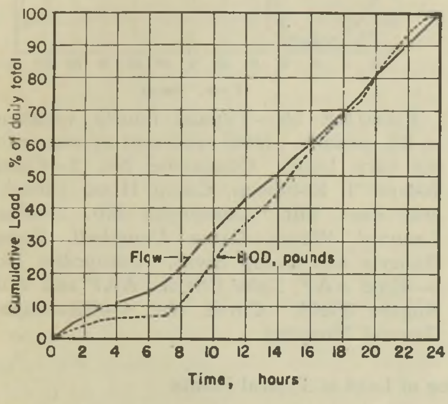


FIGURE 52.—Typical cumulative hourly load. Flow and BOD, pounds shown as percent of daily total. Composite—Camp Joseph T Robinson, Camp Hood (South) and Fort Sill.

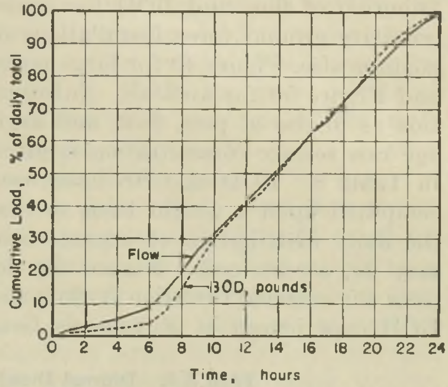


FIGURE 53.—Typical cumulative hourly load. Flow and BOD, pounds shown as percent of daily total. Composite—Fort Leonard Wood, Camp Campbell, Camp Roberts and Camp Swift.

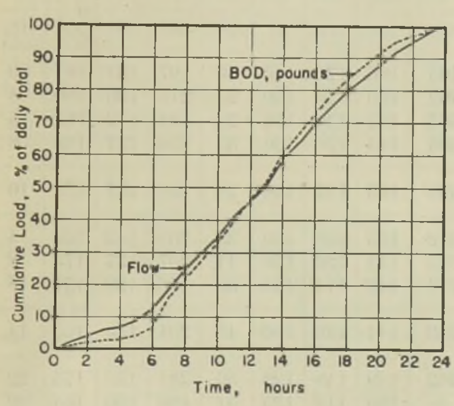


FIGURE 54.—Typical cumulative hourly load. Flow and BOD, pounds shown as percent of daily total. Composite—Enid AAF, Lake Charles AAF and Will Rogers Field.

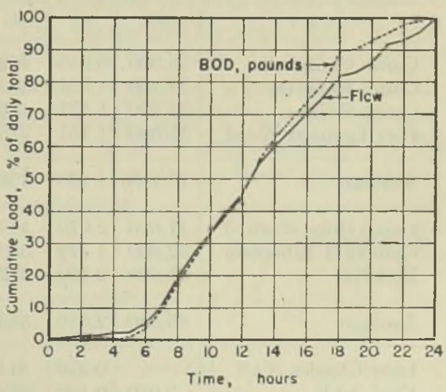


FIGURE 55.—Typical cumulative hourly load. Flow and BOD, pounds shown as percent of daily total. Fitzsimons General Hospital.

performance charts (Figs. 73, 74, etc.) at numerous representative plants.

Hourly Variation in Composition of Raw Sewage

Thirty-two plants submitted data indicating flow and concentration (BOD and SS) in raw sewage, as obtained from analyses of samples collected every hour during typical 24-hr periods. The results were generalized by forming composite curves (Figures 48 to 56) for posts of similar type and size. Figure 48 indicates hourly distribution of flow and BOD for representative ground force installations of medium size, Figure 49 for large posts, and Figure 50 for airfields. Information as to size of post, flow, and average raw sewage concentration is given in Table 8. BOD and SS have been computed upon a weight basis so that the daily distribution of organic load may be ascertained. Figure 51 depicts the extreme variation in flow and BOD experienced at Fitzsimons Gen-

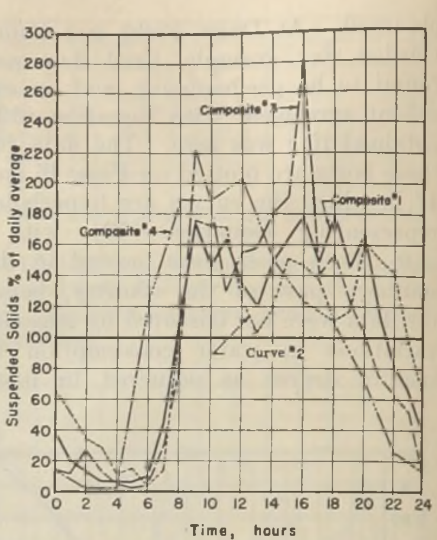


FIGURE 56.—Typical hourly variation in SS, pounds. Data represent average rate for each hour. Composite No. 1—Camp Joseph T Robinson, Camp Hood (South) and Fort Sill. Composite No. 2—Fort Leonard Wood, Camp Campbell, Camp Roberts and Camp Swift. Composite No. 3—Enid AAF, Lake Charles AAF and Will Rogers Field. Curve No. 4—Fitzsimons General Hospital.

TABLE 8.—Diurnal Distribution of Load of Typical Plants

Post	Pop'n	Flow (mgd)	BOD (ppm)	SS (ppm)	Flow (% of Average)				BOD (% of Average lb/hr)			
					Max 1-hr Av	Max 16-hr Av	Max 4-hr Av	Min 4-hr Av	Max 1-hr Av	Max 16-hr Av	Max 4-hr Av	Min 4-hr Av
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Camp Campbell	15,900	0.946	442	405	166	126	146	35	197	134	181	20
Camp Roberts	24,100	1.353	392	402	180	126	150	25	216	136	180	8
Camp Swift	21,300	1.175	473	365	162	130	151	20	194	142	165	5
Fort Leonard Wood	15,000	1.151	273	303	144	126	136	32	228	137	168	8
Average	19,100	1.156	395	369	163	127	146	28	209	137	173	10
Camp Hood (South)	41,000	2.079	419	319	153	126	139	35	209	143	160	4
Camp J T Robinson	32,800	1.779	359	285	141	120	156	41	207	137	175	8
Fort Sill	31,600	2.100	312	276	142	114	124	53	186	129	151	16
Average	35,500	2.019	363	293	144	120	140	43	201	136	162	11
Lake Charles AAF	—	0.226	314	262	149	118	128	56	281	135	175	22
Enid AAF	2,040	0.190	669	—	139	118	123	51	186	130	164	22
Will Rogers Field	4,220	0.181	418	297	279	142	192	7	283	144	198	3
Average	3,130	0.199	467	280	189	126	148	38	250	136	179	15
Fitzsimons G H	5,415	0.380	419	294	227	137	174	13	277	143	177	3

eral Hospital. Figures 52 to 55 present the data of the previous graphs calculated on a cumulative basis in the form of mass diagrams. This kind of plot is particularly useful in exhibiting the lag of the organic load behind the flow.

The graphs display in a striking way the rapid increase in flow and BOD during the morning. In all posts the distribution curve of BOD in pounds was observed to ''outswing'' that of the flow. The bulk of organic load was applied in a significantly shorter time than was the peak volumetric load. An analysis of the data of Figures 48 to 56 is presented in Table 8. The maximum 1-hr average flow varies from 144 percent of the daily average flow in the large posts to 189 percent in a small air force installations. Similar trends may be noted in the maximum 4-hr and 16-hr flows and in the minimum 4-hr flow. The small plants experienced a markedly greater concentration of diurnal load. In the following compilation, results are compared with design expectation as set forth in the Engineering Manual.

Flow, Percent of Average Daily Flow

	Max 1-hr	Max 4-hr	Max 16-hr	Min 4-hr
Large Posts	144	140	120	43
Medial Posts	163	146	127	28
Small Posts (airfields)	189	148	126	38
General Hospital	227	174	137	13
Engineering Manual	—	175	125	40

With respect to the maximum 4-hr flow, the design figure of 175 percent is seen to have been somewhat conservative. In a number of plants, however, the hourly flow distributions during a substantial proportion of days were so non-uniform as to closely approach the design criterion pertaining to maximum 4-hr flow. This criterion, it should be observed, was conceived primarily as a guide for designing units in which adequate detention period was essential. The maximum 16-hr flows experienced were about in line with design expectation. Minimum 4-hr average flows in many posts, however, fell below 40 percent of the daily average. In a number of plants, anaerobic conditions in settling tanks developed as a result of low night flow and, because of effects concomitant with septicity, remedial measures had to be taken.

The non-uniformity of flow and organic load distribution has been analyzed with respect to the minimum time required for various portions of the daily average load to arrive at the plant. The results, which are presented in Table 8a, were calculated from the steep portions of the mass diagrams (Figures 52 to 55). One-quarter of the total BOD load (in pounds) arrived in only 3 to 4 hr; three-quarters arrived in 11 to 13 hr. Suspended solids loading exhibited a significantly greater concentration than that of BOD in most of the smaller plants; in larger installations, however, the distributions were quite similar. From Table 8a it is seen that 10 percent of the total daily flow at small plants arrived in a little over an hour. Half the total volumetric load was applied in times varying from 6 to 10 hr, depending upon the size of the post.

The characteristic rapid rate of flow increase in the morning may be quantitatively stated in terms of the maximum hourly rate of increase of flow (gcd). With a daily average of 70 gcd, the following flow accelerations were typical:

	Hourly Rate of Increase in Flow (gcd/hr)
Large posts	45
Medial posts	50
Small posts (airfields)	77
General hospital	78

The increase in flow during the early morning hours was so rapid that a series of traveling waves was created in the sewer system. Wave velocities

TABLE 8a.—Diurnal Distribution of Load, Composites of Groups of Plants

Per-cent of Load	Minimum Time of Arrival of Various Proportions of Total Daily Flow, BOD and SS Load (hr)											
	Curve 1 (a)			Curve 2 (b)			Curve 3 (c)			Curve 4 (d)		
	Flow	BOD	SS	Flow	BOD	SS	Flow	BOD	SS	Flow	BOD	SS
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
10	1.8	1.3	1.4	1.5	1.3	1.4	1.2	1.2	0.9	1.1	0.9	1.2
25	5.0	4.0	3.7	4.5	3.5	3.6	4.2	4.4	2.7	3.5	3.2	3.0
50	10.4	8.2	7.6	9.3	8.0	8.2	8.8	7.6	6.6	6.1	5.8	6.3
75	14.3	12.6	11.6	14.2	12.5	12.7	14.0	11.5	10.0	13.8	10.5	10.8
90	21.5	15.8	14.7	17.8	16.0	15.7	18.5	16.0	14.0	16.8	14.2	13.9

(a) Camp Campbell, Camp Roberts, Camp Swift & Fort Leonard Wood.
(b) Camp Hood (South), Camp J T Robinson & Fort Sill.
(c) Lake Charles AAF, Enid AAF & Will Rogers Field.
(d) Fitzsimons General Hospital.

in general exceeded the mean velocity of flow, consequently a flow increase at the plant was experienced before the first significant increase in organic load became manifest. The first part of the new flow was relatively weak sewage; significant BOD increase occurred about an hour after flow increase.

Coefficient of Variation of Load

The various ways of expressing the non-uniformity of flow distribution heretofore discussed are all useful only in connection with particular phases of design. Maximum 4-hr flow, for example, is an essential factor in providing for adequate detention period; maximum 16-hr flow must be considered in design of sludge hoppers, scum pits and other units that are operated on a day-to-day schedule. Each of these measures, however, pertains only to a particular aspect of the overall load distribution. None of them completely characterizes the diurnal variation, since each is calculated upon a fixed time base that bears no especial relation to camp routine.

It is worthwhile to introduce another measure of load variation that is based upon all 24 hours of record at a plant, and therefore is representative of the flow distribution as a whole. The coefficient of variation, C_v, which is popular among statisticians, is suited to the purpose at hand. It is calculated in accordance with the following equation:

$$C_v^2 = \frac{1}{24}\sum_{i=1}^{24} (X_i - 100)^2 \qquad (13)$$

where

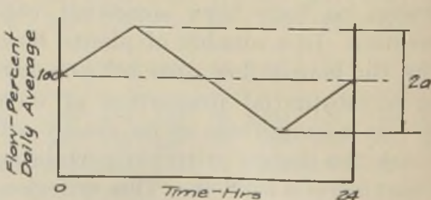
X = the flow (or BOD), expressed as a percentage of the daily average;

C_v = coefficient of variation, expressed as a percentage of the mean daily flow.

It is evident from Equation (13) that the more non-uniform the flow, the greater the value of C_v. Only for the case of a uniform (rectangular) distribution does C_v become zero.

In order to implement the interpretative value of the coefficient of variation the following idealized load distributions, with corresponding values of C_v, are presented:

I. Triangular Flow Distribution:

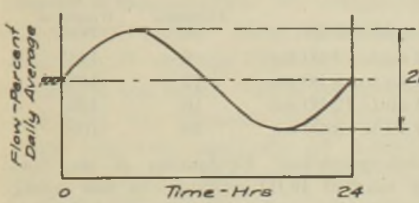


$$C_v = \frac{a}{\sqrt{3}}$$

Maximum C_v when $a = 100$,

$$C_v = 57.7\%$$

II. Sinusoidal Flow Distribution:



$$C_v = \frac{a}{\sqrt{2}}$$

Maximum C_v when $a = 100$,

$$C_v = 70.7\%$$

Results reported in subsequent sections indicate that the effect of diurnal load variation upon plant performance was marked. Load distribution accordingly becomes a primary design consideration. Values of the coefficient of variation for flow and BOD (pounds) represented in Figs. 48 and 51 are given in the following table:

	Figure	Coefficient of Variation as Percent of Daily Average	
		Flow	BOD
Large posts	49	31	57
Medial posts	48	40	58
Small posts (airfields)	50	40	62
General hospital	51	66	80

The more pronounced variation in daily load at smaller installations is quantitatively expressed in the foregoing table; also the relative non-uniformity of BOD as compared with flow is indicated.

Presuming that normal * frequency distribution of loading occurs, the following interpretation regarding C_v may be made—about two-thirds of the time the flow (or BOD) lies within the range $(100\text{-}C_v)$ to $(100 + C_v)$.

* In the statistical sense.

Equivalent Population

The proportion of the total load arriving at the plant between 10 A.M. and 6 P.M.† provides a basis for estimating the sewage contribution of non-resident population, which, for design purposes (in the Engineering Manual) was taken as one-third relative to resident population. The proportion of load arriving between 10 A.M. and 6 P.M., for various sized plants as calculated from Figs. 52 to 55, is presented in the following compilation:

	Percent of Daily Load Arriving between 10AM and 6PM	
	Flow	BOD
Large plants	36	44
Medial plants	40	43
Small plants (airfields)	47	51
General hospital	49	56

These data would indicate that the flow factor of one-third for personnel working on the daytime 8-hr shift is somewhat low. While it is probable that the assumptions underlying the foregoing analysis tend to result in an over-estimation of the contribution of non-resident population, it is fairly certain that a flow factor of one-third for eight working hours cannot be regarded as entirely conservative in smaller posts. The Engineering Manual indicated an allowance for BOD and SS for personnel on an 8-hr shift equivalent to one-half that of resident personnel. The foregoing table indicates that this allowance was somewhat high, particularly in medium sized and larger plants.

The figures in the compilation are of course somewhat higher than they would have been if non-resident personnel were excluded. As shown in Table 3a, non-resident population averaged about one-sixth of the total.

† These times follow from the assumption that sewage would require approximately an hour in passage through sewers to the treatment plant.

Consequently a more precise estimate of the contribution of non-resident population may be obtained by reducing the figures in the foregoing compilation by about 17 percent.

The following factors for estimation of equivalent population appear to be indicated:

Size of Post	Flow	BOD
Greater than 5,000	8/24	9/24
5,000 to 15,000	9/24	9/24
Less than 5,000	10/24	12/24
Average	9/24	10/24

Influence of Length of Sampling Period

The data shown in Figs. 48 to 56 make possible an evaluation of errors inherent in compositing periods less than 24 hours that come as a result of non-uniformity of flow. Thus a composite of eight samples collected every hour starting, say, at 8 A.M. would ordinarily be more concentrated than if the compositing period had extended over a 24-hr period. Analysis of the flow and organic load distribution for 17 plants yielded the following average results for raw sewage BOD (weight) when short sampling periods were utilized:

Sampling Period	Duration (hr)	Percent of 24-hr Average Weight of BOD
8:00 AM to 4:00 PM	8	144*
8:00 AM to 8:00 PM	12	140
8:00 AM to 12:00 PM	16	135
8:00 AM to 8:00 AM	24	100

* Interpretation: Evaluation of the total daily load of BOD (pounds) in raw sewage based upon a composite sample formed of hourly increments between 8:00 AM and 4:00 PM would be too high by a factor of 44 percent.

The foregoing table pertains to results when the composite sample is formed by hourly increments, the volumes of which are proportioned in accordance with rate of flow. In some plants (not included on the survey list) it was the practice to sample with hourly increments of constant volume without regard to flow. This procedure, while simple, does of course entail some error, as shown in Table 8b, which presents results of tests at 18 plants.

TABLE 8b.—Comparison of Samples of Raw Sewage BOD Composited Proportional to Flow and by Equal Portions

Post	Flow (mgd)	BOD, 24-Hr Av by Equal Portions (ppm)	BOD, 24-Hr Av by Flow (ppm)	Percent Difference
Camp Campbell	0.946	406	443	− 8.3
Lake Charles AAF	0.226	281	314	−10.5
Enid AAF	0.190	622	669	− 7.0
Camp Fannin	0.571	355	409	−13.2
Fitzsimons Gen Hosp	0.380	348	419	−16.9
South Camp Hood	2.079	352	419	−15.8
Fort Knox	3.832	196	213	− 8.0
Fort Monmouth	0.542	224	248	− 9.7
Camp Roberts	1.353	348	392	−11.2
Camp J T Robinson	1.779	314	359	−12.5
Fort Sheriden	0.490	457	480	− 4.8
Fort Sill	2.100	291	312	− 6.7
Camp Myles Standish	1.350	425	477	−10.8
Camp Swift	1.175	399	473	−15.6
Fort F E Warren	0.397	437	529	−17.3
Will Rogers Field	0.181	368	417	−11.8
Camp Wood	0.136	372	404	− 7.8
Fort Leonard Wood	1.151	240	273	−12.1
				Av = 11.1%

It is seen that sampling by equal portions result in an estimte of BOD concentration that averages about 11 per cent too low. Such an undervaluation is a consequence of including a disproportionately large amount of weak night sewage that dilutes the composite too much. It is pertinent to note that the greatest disparities between the two methods of sampling were not always confined to small posts. At Forts Sill and Warren, for instance, an error greater than 15 percent in evaluation of raw sewage BOD would have resulted, if it had been the practice at these posts to form composite samples from equal portions of sewage. However, with samples composited in proportion to flow, 8-hr sampling in small plants was found to introduce an appreciably greater error than was noted for a corresponding sampling period in large plants. A comparison of the effect of different sampling periods in plants of various sizes is shown in the following table:

Sampling Period	Percent of 24-hr Average BOD (Weight)			
	0 to 0.5 mgd	0.5 to 1.0 mgd	1.0 to 2.0 mgd	2.0 to 4.0 mgd
8:00 AM to 4:00 PM	162	149	142	127
8:00 AM to 8:00 PM	150	146	139	128
8:00 AM to 12:00 PM	134	136	137	138

The foregoing discussion pertains to influence of sampling periods upon evaluation of BOD of raw sewage. Insufficient data were available to make a corresponding analysis for the other usual sampling points, such as final effluent. Data reported by Caster and Hamilton, based upon tests at Fort Knox, are indicative of the influence of sampling period upon evaluation of plant performance. Results in ppm, are as follows:

Sampling Period	Av Flow (mgd)	Raw Sewage		Plant Effluent	
		BOD	SS	BOD	SS
8:00 AM to 4:00 PM	4.71	378	376	31	42
8:00 AM to 12:00 PM	4.81	334	337	38	44
8:00 AM to 8:00 AM	4.03	268	286	35	46

The BOD results on raw sewage for the 8-hr period were 41 percent higher than for the 24-hr period; the SS were 32 percent higher. On final effluent the 8-hr period BOD results were 12.5 percent lower than for the 24-hr period and the SS were 9 percent lower. It is evident therefore that short sampling periods have the effect of exaggerating plant performance, not only from over-estimation of raw sewage strength but also from under-estimation of final effluent concentration. The latter effect, however, was relatively a minor one at Fort Knox since recirculation operated to maintain the final effluent at a fairly uniform strength throughout the day.

An investigation similar to the one made at Fort Knox was carried out on June 17, 1944 at Camp Myles Standish by Snell and Donnelly. The 24-hr composite sampled in proportion to flow gave a BOD of 670 ppm and SS of 207 ppm; results in ppm for other sampling procedures are given in the following table:

Sampling Period	Raw Sewage		Plant Effluent	
	BOD	SS	BOD	SS
8:00 AM to 4:00 PM (every hr)	782	355	57	26
6:00 AM to 6:00 PM (every hr)	703	295	58	27
8:00 AM to 8:00 AM (every 2 hrs)	492	165	59	26
8:00 AM to 8:00 AM (every hr)	670	207	65	26

In this table, the figures pertain to samples composited in proportion to flow. Effluent concentration at Myles Standish tended to be fairly uniform as a result of rather heavy recirculation. Despite this circumstance, analysis indicates that a substantial discrepancy obtained between the 8-hr and 24-hr sampling periods.

The hourly data for the various plants were examined to ascertain whether any short sampling procedure could be devised that would give results more nearly in accord with the 24-hr compositing period. However, the flow was so variable during the day, and from day to day at military installations, that no consistently successful short procedure could be found.

In addition to short period compositing, other sampling practices were noted at various plants that tended to give a distorted representation of loading and performance. These may be listed as follows: (1) formation of composites at primary effluent (and filter effluent) in proportion to sewage flow instead of *sewage plus recirculated* flow; (2) neglecting the collection of composite samples of recirculated flow. It was the tacit assumption that the average concentration of, say, recirculated secondary settling tank effluent would be the same as the average concentration of the secondary tank effluent. This is an erroneous assumption, however, for in order to obtain a correct evaluation of the average concentration of recirculated flow, it is necessary to form a composite sample in proportion to *recirculated flow,* not sewage flow. An analysis of the errors entailed in the foregoing practices indicated that the first (1) was of the order of 2 to 5 percent, while the second (2) was of the order of 6 to 12 percent. The first (1) was not often encountered, whereas the second (2) was almost universal.

Grease in Raw Sewage

In Table 9 are shown results of grease analyses in raw sewage at seven representative plants. The concentration of ether soluble matter is seen to vary from 33 to 255 ppm—a variation of about 8 to 1. Occasional peak concentrations noted in grab samples ran as high as 1,000 ppm. The low of 33 ppm at Camp Kilmer resulted from a high flow during the time of sampling of 155 gcd due to infiltration of ground water. The average concentration for the seven plants was 164 ppm, and the median was 151 ppm. On a per capita basis, values ranged from 0.043 to 0.139 lb daily; the mean value was .08 lb/cap/day. This value is in excellent agreement with the value of 0.09 lb/cap/day of ether soluble material used as a design criterion.

Variations in amounts of grease in sewage at different posts were substantial, reflecting differences in efficacy of grease recovery programs and trap

TABLE 9.—Grease in Raw Sewage

Post	Number of Analyses	Average Pop'n	Average Flow (mgd)	Grease	
				ppm (a)	lb/capita/day
(1)	(2)	(3)	(4)	(5)	(6)
Camp Blanding	18(b)	41,950	2.38	155	0.074
Camp Campbell	37	40,600	2.16	160	0.071
Fort Dix	5	52,100	3.39	255	0.139
Fort Jackson	3	45,400	3.08	137	0.078
Camp Kilmer	2	15,100	2.35	33	0.043
Fort Knox	14(b)	46,500	3.31	127	0.075
Fort Monmouth	44	19,200	0.66	151	0.043

(a) Weighted. (b) Monthly composites.

maintenance. Grease retention characteristics of sewerage systems also were a factor; grease adhering to sewer walls would, at some posts, slough off at irregular intervals and enter treatment units in slugs."

Excessive amounts of grease in raw sewage caused operating difficulties at a number of plants, particularly in the early part of the war. Tests by the Water and Sewer Unit of the Repairs and Utilities Branch indicated that grease traps outside mess halls, which were designed by the Engineering Branch and R & U, were not satisfactory. In addition, it was found that they were not cleaned properly and were in general insanitary. Cast-iron traps in kitchen sinks had unsatisfactory hydraulic characteristics; they accumulated grease and discharged it in a concentrated load to the sewer. High peak discharges from sinks were reduced by installation of cast-iron cleanout control type tees (one per sink) at the end of the drain pipes. This appurtenance successfully ironed out flow fluctuations, yet still permitted of reasonable times of discharge.

A grease trap testing program at the University of Iowa resulted in development of a recommended cast-iron trap of 25 gpm flow rate, with 50 lb of grease retention capacity at 90 per cent efficiency. Later this model, which was installed at many posts, was made of ceramic material in order to conserve critical material. Hydraulic characteristics, however, were not changed.

A number of reasons are advanced to explain the high grease content so characteristic of military sewage. In part, the large amounts of grease may be attributed to consumption of more meat by a male population in a physically rigorous mode of life. Many fried foods were eaten; a fryer was standard equipment in each kitchen. These circumstances were conducive to high grease concentration in dishwater. Another factor was the use of a considerable quantity of soap per capita. The physical nature of the training activities, together with the stress placed upon cleanliness, resulted in a substantially greater number of shower baths per capita daily than is usual in a civilian population. Sodium and calcium stearate and other substances characteristic of shower effluent collected on primary treatment units, forming a scum of spent lather mixed with grease. At the hydrogen-ion concentrations prevalent, some hydrolysis of waste soap occurred that resulted in the formation of ether-soluble derivatives of fatty acids. Consequently high values obtained in the usual grease determination stemmed in part from high soap consumption. Large laundries that were installed at various cantonments contributed substantial quantities of grease in laundry waste as well as spent soap.

Part of the grease in military sewage came from airplane and auto service shops. Washing and cleaning activities, particularly with gasoline, resulted in the carrying of grease and oil into sewage. At a number of posts wash racks had to be disconnected from the sanitary sewer system.

An investigation of the grease removal efficiency of various devices has been reported by Eliassen and Schulhoff (47). A paper discussing measures taken to reduce grease load at and near the source has been presented by McCaskey and Vermette (50).

Establishment of the grease recovery program by the Salvage Branch of the Office of the Quartermaster General was a significant factor in the eventual reduction of grease content in sewage at many posts. The Salvage Branch collected approximately six million pounds of grease each month. This included interceptor grease as well as frying fats and other kitchen wastes.

Composition of Military Sewage

An investigation of characteristics of raw sewage at Sheppard Field, Texas,

TABLE 9a.—Characteristics of Raw Sewage at Sheppard Field, Texas

Date	Day	BOD (ppm)	Total SS (ppm)	Fixed SS (ppm)	Total Solids (ppm)	Fixed Solids (ppm)	Dissolved Solids* (ppm)	Oxygen Consumed (ppm)	NH_3-N (ppm)	Alkalinity (ppm as $CaCO_3$)	pH	Chlorides (ppm)	Population	Av Flow (mgd)
1/26/43	Tue	422	504	116	1,488	666	984	168	45	375	8.2	310	31,347	1.30
1/27/43	Wed	520	356	—	1,368	736	1,012	285	50	350	8.1	455	32,086	1.33
1/28/43	Thur	397	—	—	1,568	796	1,264	220	56	340	8.5	450	32,685	1.35
1/29/43	Fri	522	304	20	1,524	612	1,220	215	32	310	8.1	450	33,127	1.41
1/30/43	Sat	439	384	60	1,540	688	1,156	310	—	370	8.3	390	37,711	1.35
1/31/43	Sun	539	352	48	1,572	796	1,220	260	46	380	8.2	370	35,016	1.21
2/1/43	Mon	403	380	48	1,416	804	1,036	290	58	360	8.2	330	36,392	1.38
2/2/43	Tue	562	384	56	1,732	1,036	1,348	340	52	310	8.0	360	36,837	1.38
Average		476	380	60	1,526	767	1,152	274	48	350	8.2	389	34,400	1.34

* Calculated from total solids and suspended solids.

by Cox and Fix gave results as presented in Table 9a. Each of the figures represent the result obtained from a 24-hr composite sample made up of increments proportioned according to flow and stored in a refrigerator. No nitrates, nitrites nor dissolved oxygen were present in the raw sewage. Dissolved oxygen tests were made hourly on grab samples. Gooch crucibles were used for all tests except on the first day when alundum crucibles were used. Ammonia nitrogen was obtained by direct Nesslerization nitrate was determined by the reduction method. Bicarbonate dilution water was used in the BOD tests.

BOD to oxygen consumed at Sheppard Field was 1.7. This is a considerably lower ratio than is usual in municipal sewage where it is of the magnitude of 2.5 to 3.0. The ratio of BOD to ammonia-nitrogen at Sheppard Field was about 10. In a group of 14 representative military posts this ratio was about 13. In this respect military sewage appears not to differ materially from municipal sewage in cities with separate sewers; there the ratio of BOD to ammonia is of the order of 10.

Three grab samples collected at Fort Meade were analyzed at the Washington, D. C. sewage treatment plant. Results are given in the following table:

Date	Time	BOD (ppm)		SS (ppm)	Fats (ppm)	NH_3-N (ppm)	Fe (ppm)	Total Solids (ppm)
		5-Day	20-Day					
4/9/45	12:45 P.M.	490	714	266	153	35	0.1	702
4/19/45	9:10 A.M.	610	1168	558	165	60	2.8	876
4/23/45	8:45 A.M.	488	1128	548	—	68	1.9	900

The low per capita flow at Sheppard Field was the result of the water conservation program, replacement of automatic flush valves with manual flush valves on urinals, and water discipline training by the Army. The ratio of

A composite sample of sewage collected at Floyd Bennett Field, USNAS, on April 17–18, 1944, and tested by the New York State Department of Health, gave results as follows: color, 3; odor, 3; turbidity, 450 ppm; NH_3N, 18 ppm;

organic nitrogen, 24 ppm; oxygen consumed, 288 ppm; BOD, 292 ppm; chlorides, 140 ppm; alkalinity, 252 ppm; pH, 7.4; grease, 106 ppm; total solids, 815 ppm; fixed solids, 482 ppm; dissolved solids, 575 ppm; suspended solids, 240 ppm; and fixed suspended solids, 46 ppm.

The physical condition of the solids in military sewage, as based upon the tests at Sheppard Field and Floyd Bennett Field, are compared in the following table with average municipal sewage as estimated by Metcalf and Eddy:

Proportion of Suspended, Dissolved, Fixed and Volatile Constituents *

	Sheppard Field	Floyd Bennett Field	Average Municipal
Total Solids	1,000	1,000	1,000
Suspended Solids	250	290	370
Fixed Suspended Solids	40	55	120
Volatile Suspended Solids	210	235	250
Dissolved Solids	750	710	630
Fixed Dissolved Solids	460	170	370
Volatile Dissolved Solids	290	540	240
Fixed Solids	500	225	500
Volatile Solids	500	775	500

* Relative to a total solids value of 1,000.

A striking difference is noted in the proportion of fixed solids (mineral content) in the two military samples as compared with the municipal. Raw military sludge on the basis of these data would be expected to have a much higher volatile content that would affect its settling qualities. Other differences in the table are noted; these are to be attributed in part to differences in composition of water supply.

pH of Raw Sewage

At 22 plants the pH in the raw sewage ranged from 7.0 to 8.6 as based upon values averaged over the entire period of study. The median value was 7.4.

The pH of the water supply of 50 posts ranged from 6.4 to 9.2, with an average value of 7.6.

Data pertaining to raw sewage characteristics, as measured by the usual determinations, are presented in subsequent sections in connection with detailed analyses of performance of individual plants.

Laundry Waste Analysis

Analytical data on laundry waste of Camp Forrest during the fall of 1943, as based upon 27 determinations made on composited samples, are presented in the following table:

	pH	Alkalinity	SS	BOD	Total Fats
Av Camp Forrest	9.2	227	356	459	250*
Max Camp Forrest	9.7	278	556	695	484*
Min Camp Forrest	8.7	178	200	277	144*
Commercial Laundrys	10.3	511	—	1,860	554

* Approximately 10% free fats and remainder fatty acids.

Average laundry flow at Camp Forrest at the period under consideration was about 0.20 mgd. The laundry used each day approximately 350 lb of alkali, 500 lb of laundry soap and variable amounts of starch, sour, bleach and bluing.

For purposes of comparison, results for commercial laundry in municipalities as reported by Gehm have been included in the foregoing table. These figures, which are average values for three commercial establishments, indicate a much stronger waste than occurred at Camp Forrest.

BOD Characteristics of Raw Sewage

In Figures 57 to 65 are shown curves indicating the cumulative biochemical oxygen demand exerted by raw sewage grab samples at various plants. The curves represent the unimolecular equation for the first stage of biochemical oxidation. The nitrification stage, it is seen, began after 10 to 12 days of incubation. The data are analyzed in Table 10, in which parameters of the unimolecular equation,

$$y = L\ (1 - 10^{-kt}) \qquad (14)$$

are shown. The values of L, representing the ultimate first stage demand, vary from 204 to 997 ppm with an average of 517 ppm. The average 5-day BOD is 432 ppm which amounts to 84 percent of the ultimate, *L*. The average value of the one-day BOD is 165, which is 32 percent of the ultimate, *L*.

The commonly accepted value of the reaction velocity constant *k* for municipal sewage is 0.10 per day. The average *k* value observed at the 16 military posts listed in Table 10 was 0.18 per day. The implication is that military sewage is more rapidly oxidized than municipal. Quantitatively, the difference is shown in the following table:

	Percent of Completion of Ultimate First Stage BOD		
	In 1 Day	In 5 Days	Ultimate
With $k = 0.10$	20.5	68.0	100
With $k = 0.18$	34.0	87.4	100

The variation of *k* values in Table 10 is considerable, ranging from 0.10 to

TABLE 10.—BOD Characteristics of Raw Sewage. Parameters of Unimolecular Equation Fitted to Data

Post	Figure Number	SS (ppm)	pH	Ammonia, as N (ppm)	1-Day BOD (a) (ppm)	5-Day BOD (a) (ppm)	Ultimate First Stage BOD, *L* (ppm)	Reaction Velocity Constant, *k*, per day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Buckley Field	63	311	8.5	52	93	256	303	0.16
Camp Claiborne #1	—	210	8.5	24	122	230	243	0.30
Camp Crowder	64	—	—	—	127	358	437	0.15
Enid AAF	—	319	8.0	—	205	530	605	0.18
Fitzsimons Gen Hosp	59	—	—	—	157	485	647	0.12
Camp Hood	—	436	7.0	28	235	620	721	0.17
Lockbourne AAF	61	—	—	—	80	185	205	0.21
Fort Logan	62	210	7.8	—	124	328	381	0.17
Fort George G Meade	57	558	—	60	207	618	798	0.13
Camp Joseph T Robinson	—	238	7.1	—	281	648	704	0.22
Will Rogers Field	—	280	7.6	—	84	189	204	0.23
Fort Sill	—	331	7.1	—	117	268	292	0.22
Sioux Falls	58	732	7.6	—	216	555	635	0.18
Camp Swift	—	602	7.3	35	280	680	755	0.20
Waco AAF	65	187	—	17	103	287	351	0.15
Fort Francis E Warren	60	726	8.0	15	205	677	997	0.10

(a) Based on theoretical curve.

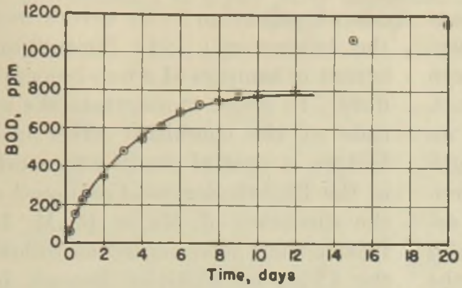


FIGURE 57.—Fort George G Meade.

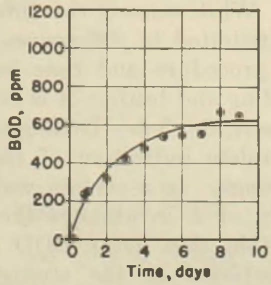


FIGURE 58.—Sioux Falls.

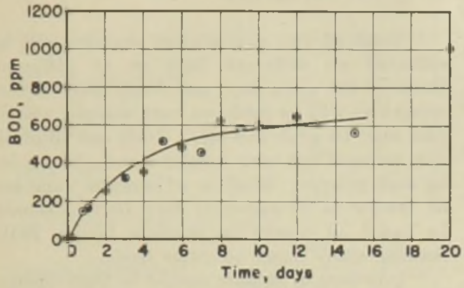


FIGURE 59.—Fitzsimons General Hospital.

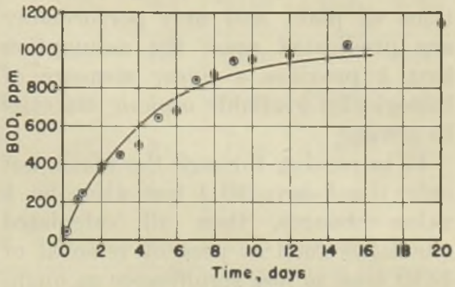


FIGURE 60.—Fort Francis E Warren.

FIGURES 57 to 60.—Course of biochemical oxidation of raw sewage. Grab samples from various posts. Curves represent unimolecular equation for the first stage BOD.

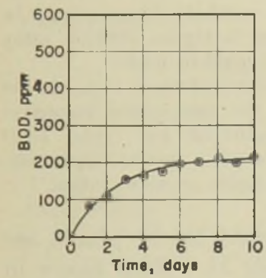


FIGURE 61.—Lockbourne AAB.

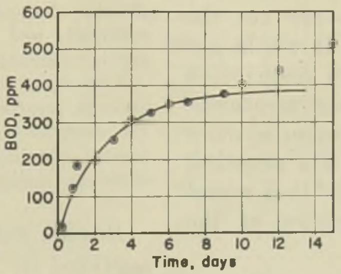


FIGURE 62.—Fort Logan.

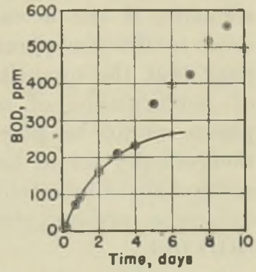


FIGURE 63.—Buckley Field.

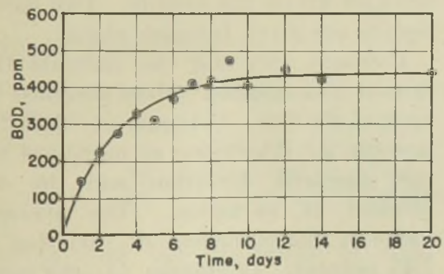


FIGURE 64.—Camp Crowder.

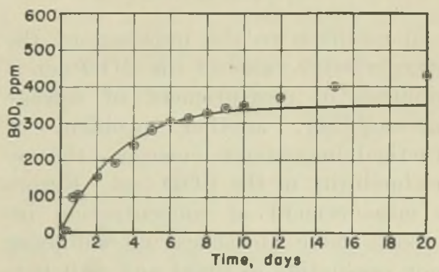


FIGURE 65.—Waco AAF.

FIGURES 61 to 65.—Course of biochemical oxidation of raw sewage. Grab samples from various posts. Curves represent unimolecular equation for the first stage BOD.

0.30. While some of the difference may be attributed to differences in laboratory procedure and time lag between sampling and testing, it is evident that, by itself, the 5-day BOD gives only an incomplete indication of the strength of sewage. In a certain way the variability of k invalidates the usual assumption that 5-day BOD is *directly proportional* to the strength of the sewage. This is only true if k is a constant. The veracity of all computations of plant and unit performance are predicated upon the assumption that k provides a *linear* measure of biologically available organic material in sewage.

If in passing through the plant, not only the 5-day BOD but also the k value changes, then all calculated quantities such as percent removal of BOD tend to lose significance as quantitative measures of performance. Thus two plants both having, say, a BOD removal of 90 percent when treating sewage of 300 ppm initial BOD may have considerably different effluents, if the k values are not the same in the two plants. It would appear that the quantitative description of performance of sewage treatment plants would be placed upon a more theoretically sound basis if a practical method could be established that would overcome present deficiencies of the BOD test.

Precision of the Five-Day BOD Determination

In addition to the question of the interpretative value of the BOD determination in measurement of sewage putrescibility, another problem of practical importance emerges—the reproducibility of the BOD test. Errors in measurement of concentration inevitably have an obscuring influence upon evaluation of plant and unit performance. As has been stated, errors in measurement of concentration may be attributed to: (i) improper methods of collection; (ii) errors made in the laboratory; (iii) fluctuations inherent in samples of a non-homogeneous fluid. In order to ascertain the magnitude of the combined effect of these factors, a special study was conducted in the Eighth Service Command under the direction of Major R. M. Dixon. Instructions were issued as follows, by the Chief of Utilities Branch in the Office of the Eighth Service Command, Command Engineer:

"Total of two raw sewage samples will be collected on different days or at different times of the same day, and 5-day BOD determinations will be made on each sample. Samples may be grab samples. Only one dilution will be used but nine bottles are to be set up on each sample. Dilution of samples (amount of sample to be added to each bottle) should be based on results in previous hourly BOD loading study made on 24-hr basis.

"Dilutions of sample should be high enough to prevent any sample bottle being totally depleted of oxygen but set up within the dilution range so that 2 ppm of oxygen may reasonably be expected to remain in the bottle at the end of the incubation period. If any one of the nine tests made shows a total depletion of oxygen, another sample should be collected; and using a higher dilution ratio, another incubation should be made.

"In reporting results of these tests, do not average results of the nine bottles incubated for each sample collected, but report BOD results obtained on each bottle separately. Only raw sewage samples are to be taken."

Results submitted by 11 plants are analyzed in Table 11. Each figure in column (4) of Table 11 represents the arithmetic mean of 9 independent BOD tests set up on the sample. Two days' results are given for each plant.

Columns (5) and (6) indicate the largest and smallest values obtained in each of the sets. Columns (8) and (9) present the dispersion as measured by the standard deviation and the coefficient of variation. The average value of the coefficient of variation is 7.45 percent. In column (9) the magnitude of the range relative to the standard deviation is given. The average value of this ratio is about 3.2, sug-

TABLE 11.—Precision of 5-Day BOD Determinations, Military Sewage Treatment Plants. Single Dilution, Nine Bottles

Name	Flow (mgd)	Dilution (%)	BOD (ppm)			S (a)	C_v (b)	(Max-Min) ÷ S
			Mean	Max	Min			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Camp Hood (North)	2.75	1	353	373	338	13.53	0.0384	2.59
	2.85	1	369	385	360	8.03	0.0218	3.11
Enid AAF	—	0.67	367	420	300	41.85	0.114	2.87
	—	0.67	335	495	195	79.37	0.2369	3.78
Will Rogers Field	—	1	272	310	210	29.06	0.1068	3.44
	—	1	366	540	210	108.86	0.2974	3.03
Fort Sill	2.90	1	422	445	400	12.28	0.0291	3.66
	2.60	1	439	495	410	31.10	0.0709	2.73
South Camp Polk	1.30	1.5	367	413	320	25.52	0.0695	3.65
	1.50	1.4	388	393	371	7.29	0.0188	3.02
Camp J T Robinson	—	0.73	472	510	430	28.63	0.0607	2.79
	—	0.73	582	630	550	24.01	0.0413	3.33
Camp Swift	—	1	373	440	340	30.01	0.0804	3.33
	—	1	240	250	230	6.46	0.0269	3.10
Camp Claiborne #1	—	1	207	220	180	14.15	0.0684	2.83
	—	1	227	240	200	17.32	0.0763	2.33
Camp Fannin	1.50	1	533	580	490	26.46	0.0496	3.41
	1.00	1	585	625	550	24.01	0.0410	3.12
Lake Charles AAF	—	1	234	252	222	5.41	0.0231	5.55
	—	1	336	362	312	15.09	0.0449	3.32
Camp Plauche NOPOE	—	—	436	470	400	28.48	0.0653	2.46
	—	—	324	350	300	18.79	0.0580	2.66
Average	—	—	—	—	—	27.08	0.0745	3.19

(a) S = Standard Deviation.
(b) C_v = Coefficient of Variation = Standard Deviation ÷ Mean.

gesting that the dispersion of individual BOD values about the mean of each set takes place approximately in accordance with the normal curve of error. Thus the following interpretation may be made regarding the result shown on Table 11—two-thirds of BOD determinations, on the average, may

TABLE 12.—Precision of 5-Day BOD Determinations, Municipal Sewage Treatment Plants, Sanitary District of Chicago

Sample Number (1945)	Dilution (%)	Single Dilution, Nine Bottles					Nine Dilutions, Original Sample				
		BOD (ppm)			S (a)	C_v (b)	BOD (ppm)			S (a)	C_v (b)
		Mean	Max	Min			Mean	Max	Min		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
S W Jul 25	2	172	180	160	7.1	0.041	193	200	185	4.7	0.024
S W Aug 9	1	181	190	170	5.3	0.029	181	190	170	5.3	0.029
S W Jul 31	1	193	200	180	6.8	0.034	197	210	180	8.1	0.048
S W Jul 26	1	222	270	200	20.4	0.092	221	240	200	11.0	0.049
S W Jul 22	1	237	260	220	11.5	0.049	253	270	240	13.1	0.054
S W Jul 22	1	260	290	240	16.3	0.063	248	260	240	7.9	0.031
S W Aug 8	1	246	250	240	4.9	0.021	236	250	220	8.2	0.034
Joliet Aug 27	1	373	440	310	47.9	0.128	420	520	380	40.9	0.097
Joliet Aug 3	1	487	540	420	37.0	0.076	445	495	370	31.5	0.071
Joliet Aug 1	1	501	600	410	54.3	0.108	436	490	295	55.2	0.127
Average	—	—	—	—	21.15	0.064	—	—	—	18.59	0.056

(a) S = Standard Deviation.
(b) C_v = Coefficient of Variation = Standard Deviation ÷ Mean.

be expected to fall within about 7.5 per cent of the correct value.

In order to compare the precision attained in measurement of BOD in military plants with that in large municipal plants, tests were made at two laboratories of The Sanitary District of Chicago and also at the Washington, D. C. treatment plant. Results are shown in Tables 12 and 13, which are arranged in a manner similar to Table 11. The average values of the coefficient of variation obtained are somewhat lower than those of Table 11, indicating that a somewhat higher degree of precision was attained in the municipal laboratories. Another test of Camp Crowder sewage, made by personnel of the USPHS on nine bottles each of two dilutions, 1 percent and ½ percent, gave an average coefficient of variation of 3.25 percent.

The precision tests at The Sanitary District of Chicago were conducted using two different techniques in making the dilutions, as shown in Table 12.

TABLE 13.—Precision of 5 Day BOD Determinations, Municipal Sewage Treatment Plants, Washington D. C. Single Dilution, Nine Bottles

Sample	Dilution (%)	BOD (ppm)			S (a)	C_v (b)	(Max-Min) ÷ S
		Mean	Max	Min			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
19 Jul 45	2	76.5	83	70	3.93	0.0514	3.31
26 Jul 45	2	110	122	104	5.37	0.0489	3.35
22 Jul 45	4	91.8	94	39	1.856	0.0202	2.69
Average	—	—	—	—	3.718	0.0402	3.12

(a) S = Standard Deviation.
(b) C_v = Coefficient of Variation = Standard Deviation ÷ Mean.

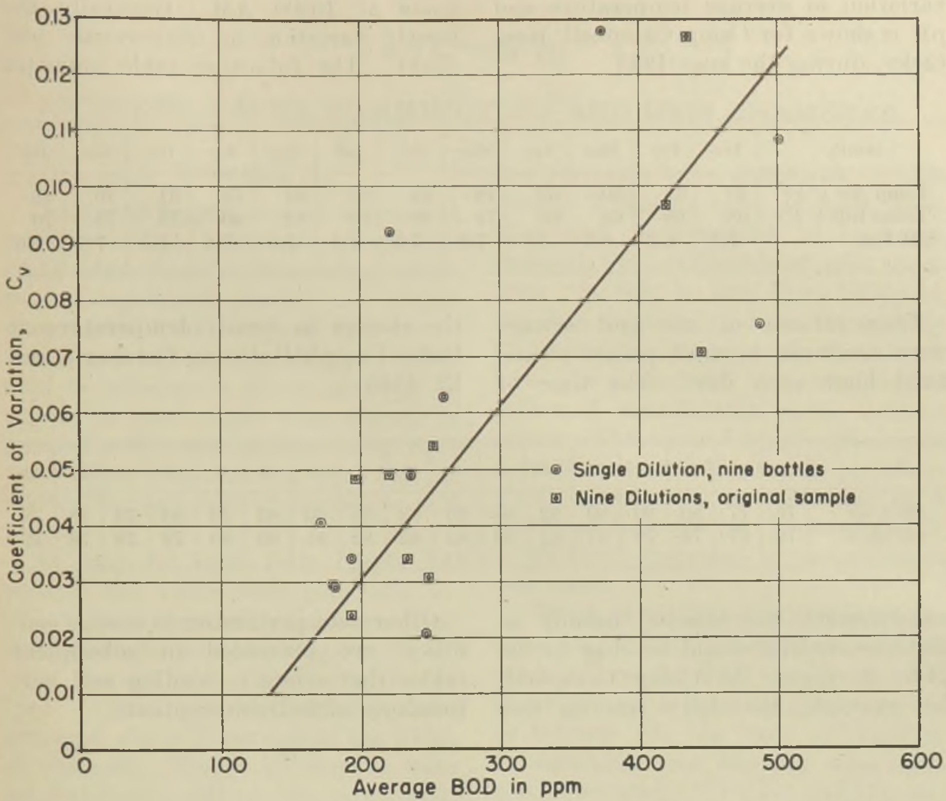


FIGURE 66.—Relation between precision of BOD determination and strength of sewage at Main and Joliet laboratories, Sanitary District of Chicago. Each point represents nine replicate determinations of 5-day BOD at 20° C. Curve is drawn through group medians.

The results by the two methods were similar. An interesting relation, shown on Fig. 66, was found to exist between the strength of sewage (average BOD) and the precision attained. High concentrations of BOD are seen to be associated with low precision of measurement. It may be inferred from this that *weak* sewage is also more *homogeneous* sewage. No similar relation was found to apply to the data from the Eighth Service Command plants of Table 11. The implication would appear to be that the flat grades and large diameters of the Chicago trunk sewers bring about an appreciable storage of sewage in the sewer system. During periods of low flow, heavier particles tend to settle; during peak flow, which consists of strong sewage, these particles are again transported toward the plant.

Condition of Sewage

Military sewage normally arrived at treatment plants in a fresh and uncomminuted condition. Odor generally was slight. Color was fairly light, and black constituents indicative of the formation of sulfides were not usually present. Oxygen content in raw sewage normally was low and usually zero during moderate and peak flow periods, except at plants where surface water entered sewers.

In the following table the monthly

variation in average temperature and pH is shown for Camp Campbell, Kentucky, during the year 1943:

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Temp Air (° F)	40	47	50	62	78	88	84	84	73	61	46	38
Temp Infl (° F)	68	68	68	69	74	80	82	82	80	78	73	70
pH Infl	7.8	8.2	8.3	7.6	7.6	7.6	7.6	7.6	7.6	7.5	7.6	7.6

Temperatures of air and sewage were measured in most plants at one fixed hour each day. The time of measurement was selected usually so that the reading would be close to the 24-hr average. At Camp Campbell, for example, the daily reading was made at 10:00 AM. Generally the hourly variation in temperature was slight. The following table indicates the change in hourly temperature at Camp Campbell during October 4 and 12, 1945:

	8 AM	9	10	11	12	1 PM	2	3	4	5	6	7	8	9	10	11	12
10/4/45	76	77	80	80	80	82	80	80	82	80	80	80	79	80	78	78	78
10/12/45	76	77	78	79	81	82	82	82	82	82	81	80	80	79	78	78	77

Other data pertaining to sewage condition are presented in subsequent tables that relate to loading and performance of individual plants.

CHAPTER III

SCREENS, RACKS, COMMINUTORS AND GRIT CHAMBERS

Screening

Design and operation of screens at military sewage treatment plants differed little from corresponding practice at municipal plants. All plants investigated were equipped with either coarse screens or comminutors. A common installation is shown in Plate 3, where inclined racks were placed in parallel with a comminutor; racks were used when comminutors were out of service for purposes of inspection and repair.

As may be seen from Table 14, screens and racks were generally inclined 45 degrees with the horizontal. Bar openings varied from ¾ to 1¾ in. with the average somewhat over an inch. Clear openings between bars averaged about 75 percent of the width of channel. Nearly all screens were hand-cleaned; only a few mechanical screen installations were made. Average amount of screening in 18 plants was 3.2 cf/mg of sewage with a range of 0.8 to 9.0 cf/mg. This constituted a significantly greater amount of screening than is usual at municipal plants utilizing racks of comparable size. For example, 29 plants with screens with 1- to 1⅝-in. openings, according to Keefer, gave a median removal of 1.6 cf/mg. The difference is to be attributed primarily to freshness of military sewage.

Comminutors

A high proportion of plants was equipped with comminutors. The satisfaction of the operator with this equipment was nearly universal. In view of the heterogeneous condition of the raw sewage, the comminutor performed an especially valuable service at military installations. Maintenance problems were relatively minor; only in a few instances were difficulties encountered. Excessive rates of tooth abrasion were, in a number of instances, lessened by installation of grit chambers. As may be seen from Table 14, the flow depth ranged on the average 25 inches (maximum minius minimum head) as a result of hydraulic control action of comminutors in the influent stream. This control-range varied from a maximum of 41 inches, noted at Camp Gordon, to a minimum of 8 inches at Boca Raton. Continuous 12- to 24-hour operation of comminutors was usual.

Claims as to the effectiveness of the comminutor that appear to have been corroborated by field survey and reports of over a hundred operators are as follows: (1) the labor of cleaning, transporting and burying screenings was eliminated; (2) odor problems associated with screenings were obviated; (3) clogging of piping check valves and pumps by primary sludge was substantially reduced; and (4) pulsating flow through primary settling tanks with concomitant impairment of settling efficiency due to intermittent build-up and release of head between screen cleanings, was done away with by the comminutor.

Grit Chambers

At a number of posts, grit chambers were installed after the treatment plant had been in operation. Much benefit was derived from grit chambers at those cantonments using old sewers from World War I days. Such old sewers often had to be practically reconstructed. It was frequently discovered that large quantities of grit had entered the system prior to reconstruction, presumably through man-

PLATE 1.—Parshall flume and racks at Buckley Field, Colo.

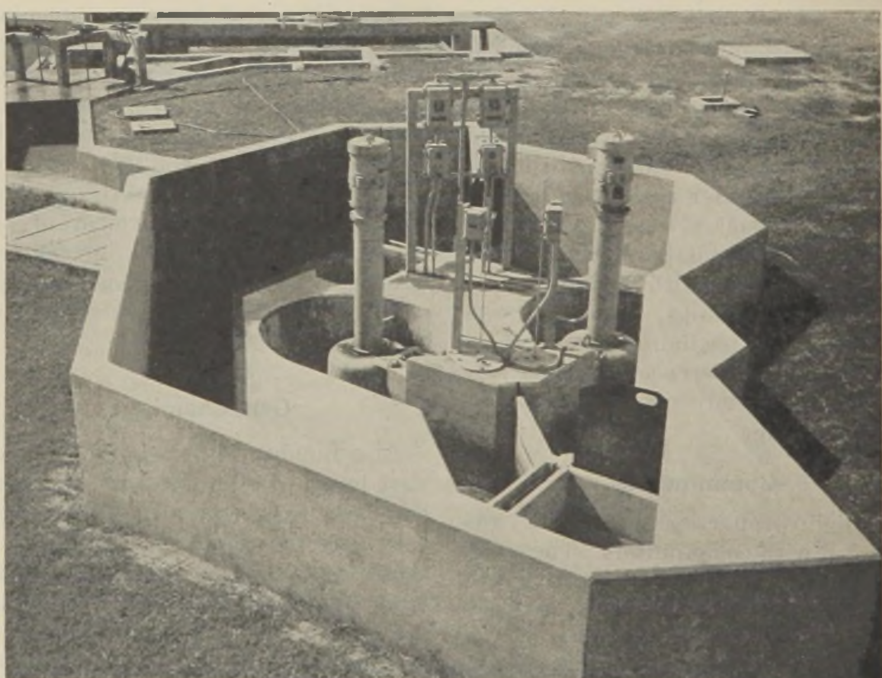


PLATE 2.—Comminutors at main plant, Camp Claiborne, La.

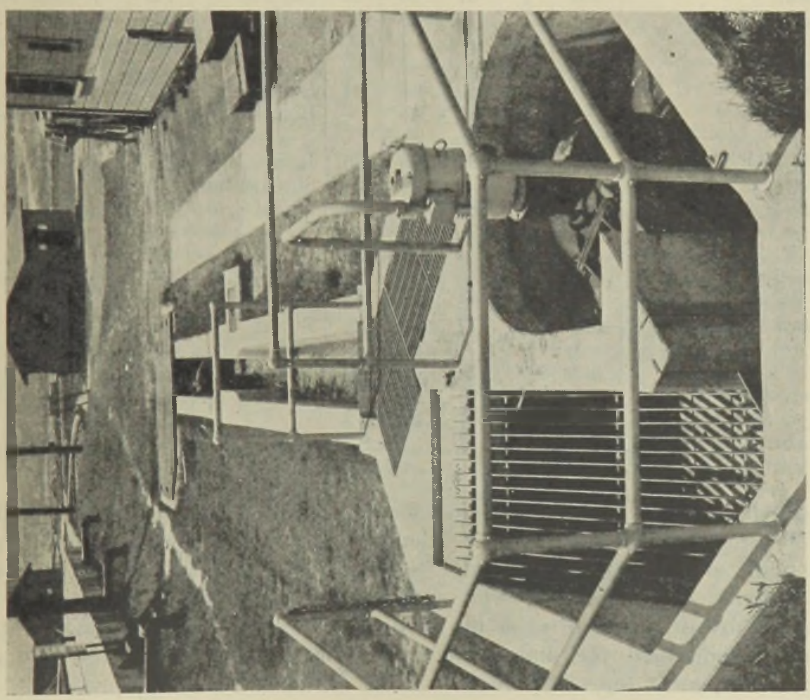


PLATE 3.—Comminutor, rack and measuring flume, Camp Edwards, Mass.

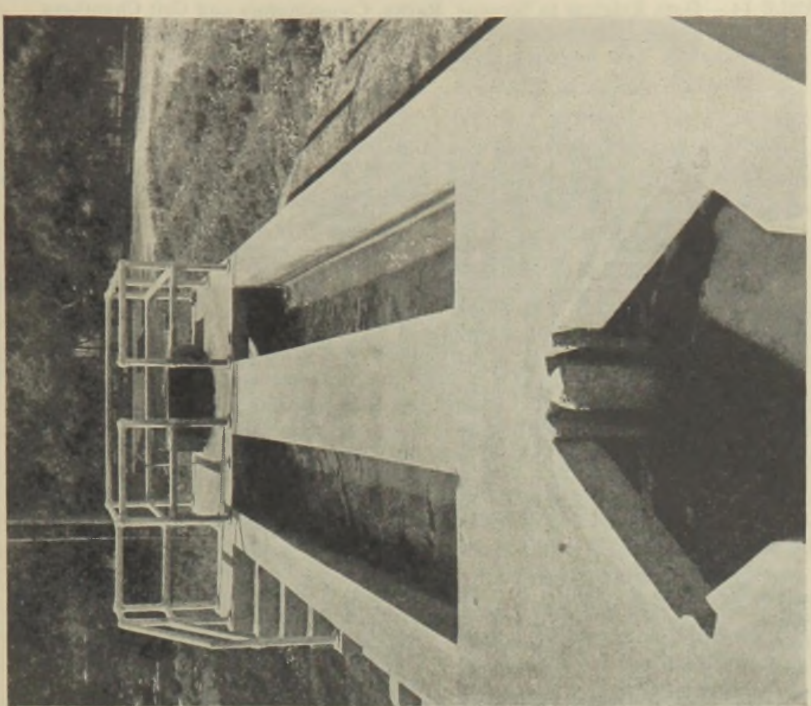


PLATE 4.—Grit chambers at Fort Sill, Okla.

TABLE 14.—Data Relating to Screens, Racks, Comminutors and Grit Chambers

Post	Screens, Racks and Comminutors						Grit Chambers		
	Screenings Removed (cf/mg)	Size of Bar Openings (in.)	Percent Opening	Angle with Horizontal	Comminutors Range of Flow Depth (in.)	Screening Disposal	Grit Removed (cf/mg)	Av No. Days Between Cleanings	Method of Disposal
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Camp Blanding	—	$1\frac{5}{8}$	75	90	—	Burial	4.5	7	Fill
Boca Raton AAF	2.0	$\frac{3}{4}$	37	40	8	Burial	—	—	—
Buckley Field	1.1	4	91	45	27	San Fill	3.2	1	San Fill
Camp Campbell	—	1	80	40	28	Burial	—	—	—
Camp Carson	1.0	$1\frac{3}{8}$	78	53	36	San Fill	1.3	7	Fill
Camp Claiborne #2	2.0	$\frac{3}{4}$	75	30	33	Burial	—	—	—
Camp Crowder	1.5	$1\frac{3}{4}$	88	22	15	Fill	—	—	—
Davisville	—	$1\frac{1}{2}$	68	45	27	Burial	—	—	—
Drew Field	—	$1\frac{1}{2}$	80	60	26	—	—	—	—
Fitzsimons Gen Hosp	0.8	$\frac{3}{4}$	60	85	21	Incin	4.3	3	Incin
Camp Funston	—	—	—	—	24	Grind	—	—	—
Camp Gordon	—	$\frac{3}{4}$	76	23	41	Incin	1.1	3	Burial
Fort Benjamin Harrison	—	1	80	30	32	Compost	—	1	—
Camp Patrick Henry	3.9	$1\frac{1}{4}$	62	—	12	Burial	0.6	7	Burial
Fort Huachuca #1	3.0	$1\frac{1}{2}$	85	45	28	Incin	—	—	Inc & Fill
Fort Huachuca #2	3.0	$1\frac{1}{2}$	85	45	—	Incin	2.0	7	Fill
Fort Jackson	1.0	$\frac{3}{4}$	67	30	18	San Fill	1.0	7	San Fill
Seymour Johnson Field	—	$1\frac{1}{4}$	76	45	—	Burial	0.9	14	Burial
Fort Knox	2.8	—	—	—	—	Incin	—	—	—
Camp Lee	9.0	1	80	45	24	Burial	2.3	3	Burial
Mitchell Field	—	$1\frac{1}{2}$	85	45	—	Burial	4.0	7	Burial
New Castle AAF	6.0	$1\frac{1}{2}$	80	45	36	Incin	2.5	7	Burial
Camp Peary	5.8	$1\frac{1}{4}$	—	60	20	Shred & Inc	2.0	7	Burial
Pine Camp	1.0	$1\frac{1}{2}$	85	45	24	Burial	1.0	Cont	Burial
Turner AAF	—	—	—	—	—	—	1.0	1	Fill
Camp Wheeler	2.4	1	80	22	22	Burning	1.2	2	Burial
Woodrow Wilson Gen Hosp	8.8	$1\frac{1}{4}$	85	45	30	Incin	—	—	—
Fort Leonard Wood	2.8	$1\frac{1}{4}$	77	54	21	Burial	—	—	—
Average	3.2	1.33	76	45	25		2.1	5	
Maximum	9.0	1.75	91	90	41		4.5	14	
Minimum	0.8	0.75	37	22	8		0.6	1	

holes originally equipped with wooden covers of 1917 construction.

In the newly constructed posts with separate sewers, grit chambers were generally not installed and in most instances results justified the omission. However, some plants, especially those located on sandy soil, experienced difficulty with grit. One troublesome source of grit was ashes entering through floor drains in kitchens. In other plants poor sewer construction, stemming in part from the urgency and haste of the building program, resulted in much grit being in the system at the time of acceptance. In such instances it was necessary to install grit chambers in order to reduce operating difficulties in primary settling tanks and digesters, and excessive wear on comminutors. On the whole, the quantity of grit in both new and reconstructed lines decreased to an equilibrium level in approximately an 18-month period. Earlier troubles were reduced even in plants in which no grit chambers were installed.

Formidable operating problems were encountered at certain posts, particularly on the West Coast, attributed to rock disintegration. At one plant where deep grit chambers had been installed between the filters and secondary settling tanks, rock debris had to be cleaned out every month. Subsequent investigation revealed, however, that the quality of rock was good. Confirmed cases of rock disintegration were found at Camp Haan, Camp San Luis Obispo and Gowan Field. Fines brought in at the time of construction were responsible for this condition in the balance of the cases.

The rectangular grit chamber with velocity of about 1 fps was the design generally favored. Sutro weirs, Parshall and Palmer-Bowlus flumes were employed for velocity control; a number of installations were made, however, with no provision for regulation of velocity. Grit generally contained a substantial amount of organic matter and, as may be seen from Table 14, burial was a common method of disposal. The average amount of grit was 2.1 cf/mg. The median amount of grit collected at 40 municipal plants as reported by Keefer was 3.0 cf/mg. At military posts, as indicated on Table 14, the average number of days between cleanings was five.

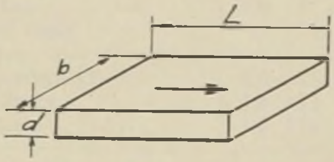
CHAPTER IV

SEDIMENTATION

Basis of Design

Detention period was made the primary criterion of design for settling tanks at military installations. As shown previously, the detention period for all primary and secondary tanks, as based upon average flow including recirculation, if any, was to be 2.5 hours, except for activated sludge primary tanks, where a figure of 1.5 hours was stipulated. Also, it was specified that final tanks should be designed to provide overflow rates of less than 800 gpd/sf as based upon average daily flow, and that depths were to be of the order of 8 to 10 ft deep.

In addition to these prescribed yardsticks of design, other rules for obtaining satisfactory dimensions were observed that more or less determined the size and shape of settling tanks at military posts. With reference to a rectangular tank of dimensions b × d × L,



the criteria applied are listed in the following schedule:

In design, it was sometimes found expedient to violate certain of the foregoing criteria that have a less direct bearing upon settling efficiency. Usually, however, adherence to the requirement for detention period and maintenance of a reasonable ratio of width to depth resulted in dimensions that met other requirements.

It is interesting to observe that with rectangular final tanks compliance with the foregoing criteria with respect to detention period, depth, displacement velocity, and overflow rate results in a fixed design admitting of no variation. The following equation may be derived from the foregoing relationships between the variables:

$$b = q/(v_{max}d)$$

$$L = v_{max}d/r$$

Example:

If $q = 1.0 \text{ mgd} = 92.8 \text{ cfm}$

$v_{max} = 0.50 \text{ f/m}$

$t = 2.5 \text{ hrs} = 150 \text{ min}$

$r = 800 \text{ gpd/sf} = 0.074 \text{ fpm}$

$d = 9 \text{ ft}$

then

$$b = 92.8/(0.50)(10) = \mathit{20.7\ ft}$$

$$L = (0.50)(10)/(0.074) = \mathit{67.5\ ft}$$

Criterion	Formulation	Primary Tanks	Secondary Tanks
As to detention period	$Lbd/q = t$	$t = 2.5$ hrs	$t = 2.5$ hrs
As to depth	$d_{min} < d < d_{max}$	$d_{min} > 8$ ft d_{max} determined by foundation condition, etc., usually < 15 ft	8 ft $< d <$ 10 ft
As to displacement velocity	$L/t = v_{max}$	0.5 fpm $< v_{max} <$ 1.5 fpm	0.5 fpm $< v_{max} <$ 1.5 fpm
As to overflow rate	$q/bL = r$	$r <$ 1,500 gpd/sf	$r <$ 800 gpd/sf
As to weir capacity	$\frac{q}{L_w} < k$	$k <$ 50,000 gpd/sf	$k <$ 25,000 gpd/sf

In rectangular tanks application of set values of detention period, depth, displacement velocity and overflow rate results in a variable ratio of width to length, depending upon flow through each tank. The greater the flow per tank (detention period, etc., being fixed) the greater the ratio of width to length necessary in order to satisfy *all* the criteria. With the values of the design criteria assumed in the example, the flow cannot exceed about 1.5 mgd per tank if the width is to be maintained no greater than 50 precent of the length.

In circular tanks some of the criteria applied to rectangular tanks lose significance and others must be substituted. The foregoing requirement as to weir capacity is easily met in the circular tank with circumferential weirs, but because of the short length of travel across the tank, it is necessary that a *lower* value be attained. The displacement velocity criterion loses some significance due to the rapid diminution of velocities as particles proceed radially from the center of the tank. Nevertheless, the mean displacement velocity, calculated as the ratio of the tank radius divided by the mean detention period, has been found to have some bearing upon tank performance. Accordingly, it has been computed in the tables that follow.

Settling Tank Performance

In Tables 15, 16, and 17 are presented data pertaining to design, loading and performance of settling tanks at 18 typical posts. Both primary and secondary tanks with and without recirculation are included. The number of tanks in each classification is indi-

TABLE 15.—Design and Operation of Settling Tanks

Post	No. of Tanks	Type of Tank	Dimensions (ft)	Volume (cf)	Ratio		Av Displacement Velocity (fpm) (a)	1000 Gal Daily per Foot of Weir (a)	Square Feet per Foot of Weir
					Lgth to Width	Lgth to Depth			
(1)	(2)	(3)	(4)	(5)	(7)	(8)	(10)	(11)	(12)
Blanding	2	Pri	70d x 11.3	94,600	—	—	0.125	8.3	18
Davisville	4	Pri	36 x 12 x 8	13,800	3.0	4.5	0.264	22.8	36
Funston	2	Pri	59 x 28.5 x 9.5(b)	23,700	2.1	6.2	0.182	18.7	59
S Johnson Field	4	Pri	58 x 12 x 9.75	27,000	4.8	6.0	0.242	25.5	58
Peary	4	Pri	116 x 16 x 10	76,000	7.3	11.6	0.575	62.0	116
Wheeler	2	Pri	74 x 16 x 11	26,000	4.6	6.7	0.420	49.8	74
Campbell	3	Sec	55d x 8.5	44,000	—	—	0.112	5.7	14
Crowder	1	Sec	45d x 9.67	49,000	—	—	0.085	4.7	11
Fitzsimons G H	1	Sec	35d x 10	9,620	—	—	0.074	4.0	9
Gordon	2	Sec	72 x 28.5 x 10	41,400	2.5	7.2	0.287	31.0	72
Callan	2	Pri	40.5 x 12 x 12	11,660	3.4	3.4	0.508	—	—
Drew Field	3	Pri	54 x 14 x 10	22,700	3.9	5.4	0.346	7.7	11
Knox	3	Pri	97 x 30 x 12	105,000	3.3	8.1	0.728	67.3	104
Bragg	4	Pri	74d x 10	172,000	—	—	0.214	12.7	18
Callan	1	Sec	35d x 11.1	10,700	—	—	0.211	14.2	9
Claiborne #1	2	Sec	21.3 x 6.2 x 8	2,150	3.4	2.7	2.370	165.5	17
Drew Field	3	Sec	52 x 14 x 10	22,700	3.7	5.4	0.519	8.5	7
Bragg	4	Sec	74d x 10	172,000	—	—	0.236	11.5	18

(a) Based on actual flow. (b) Dimensions given for larger tank only.

TABLE 16.—Operation of Settling Tanks

Post	No. Tanks in Service	Schedule Sludge Removal—No. per day	Detention Period (hr)			Surface Settling Rate (gpd/sf)		
			At Av Flow	At Max Flow	At Min Flow	At Av Flow	At Max Flow	At Min Flow
(1)	(2)	(3)	(6)	(7)	(8)	(9)	(10)	(11)
Blanding	1(a)	8	4.65	1.81	27.20	435	1,125	75
Davisville	4	3	2.26	1.69	3.10	638	850	465
Funston	1	9	5.38	2.16	—	317	791	—
S Johnson Field	4(a)	6	3.96	2.15	10.30	444	817	170
Peary	4(a)	3	3.45	2.92	4.08	522	615	442
Wheeler	2	8	2.92	1.26	6.80	678	1,570	292
Campbell	2	3	4.08	2.39	16.10	408	694	103
Crowder	3	6	4.42	2.03	19.70	418	912	94
Fitzsimons G H	1	—	3.92	1.68	18.40	459	1,072	98
Gordon	2	—	4.22	1.91	16.20	429	946	112
Callan	2	6	1.51	1.06	2.26	1,430	2,040	957
Drew Field	3	2	2.59	1.52	6.67	693	1,184	270
Knox	2	4	2.20	1.40	3.12	648	1,050	462
Bragg	4	2	2.87	2.31	3.81	627	782	473
Callan	1	cont	1.38	0.98	2.06	1,450	2,040	970
Claiborne #1	2	—	0.15	0.094	0.33	9,600	15,300	4,370
Drew	3	cont	1.67	1.15	2.76	1,080	1,570	652
Bragg	4	cont	2.61	2.14	3.36	690	842	537

(a) Variable.

TABLE 17.—Loading and Performance of Settling Tanks

Post	Tank Influent				Tank Effluent		Reduction (%)		Sludge Removed	
	Total (mgd)	Recir Ratio	BOD (ppm)	SS (ppm)	BOD (ppm)	SS (ppm)	BOD	SS	Gal per MG Sewage	Vola Sol (% of Tot Sol)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(12)	(13)
Blanding	1.820	0	195	311	141	125	27.3	60.0	4,100	81.0
Davisville	1.100	0	207	271	153	155	26.0	43.0	7,460	74.7
Funston	0.520	0	291	335	156	148	46.4	55.7	7,980	64.8
S Johnson Field	1.220	0	349	279	207	152	40.6	45.5	17,700	75.0
Peary	3.960	0	363	291	144	128	60.4	56.2	—	—
Wheeler	1.600	0	391	270	259	115	33.8	57.2	7,180	76.1
Campbell	1.940	0	53	75	22	27	59.4	64.0	—	—
Crowder	2.000	0	44	90	29	32	33.0	64.5	—	—
Fitzsimons G H	0.440	0	118	87	68	37	42.7	57.6	—	—
Gordon	1.760	0	108	86	70	34	35.6	60.9	—	—
Callan	1.390	0.85	235	240	197	217	16.1	9.5	10,250	78.3
Drew Field	1.570	0.16	265	266	132	117	50.2	56.9	6,870	66.4
Knox	5.660	0.77	217	208	134	107	38.2	48.3	6,760	69.9
Bragg	10.740	1.93	169	126	147	87	13.0	31.0	8,440	79.4
Callan	1.390	0.85	118	278	77	83	34.7	70.2	—	—
Claiborne #1	2.650	0.24	—	—	71	74	—	—	—	—
Drew Field	2.650	0.79	48	63	34	35	29.2	44.6	—	—
Bragg	11.820	2.21	70	62	61	45	12.8	27.4	—	—

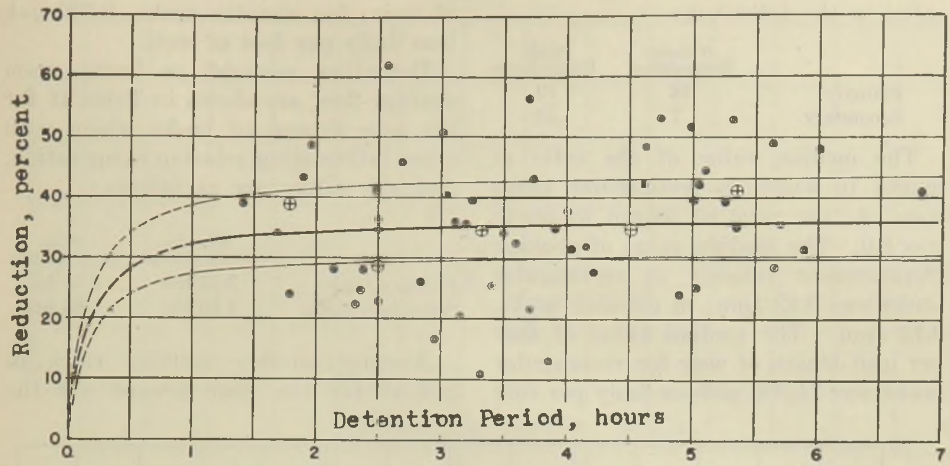


FIGURE 67.—BOD reduction in primary settling tanks without recirculation. BOD over 350 ppm.

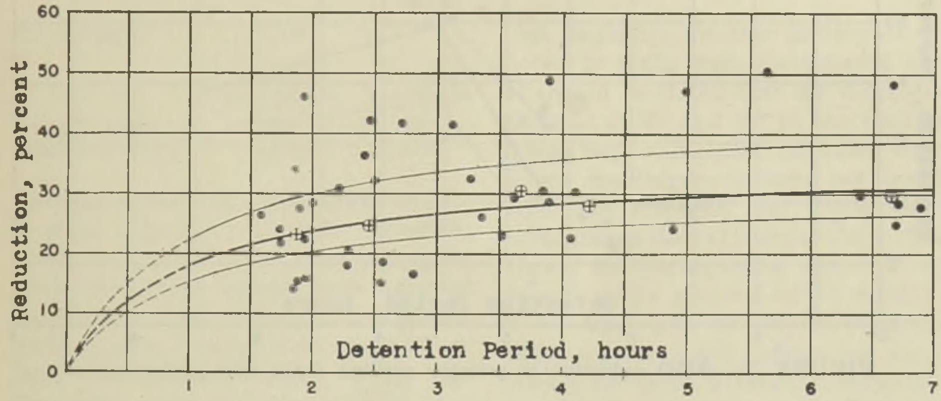


FIGURE 68.—BOD reduction in primary settling tanks without recirculation. BOD under 350 ppm.

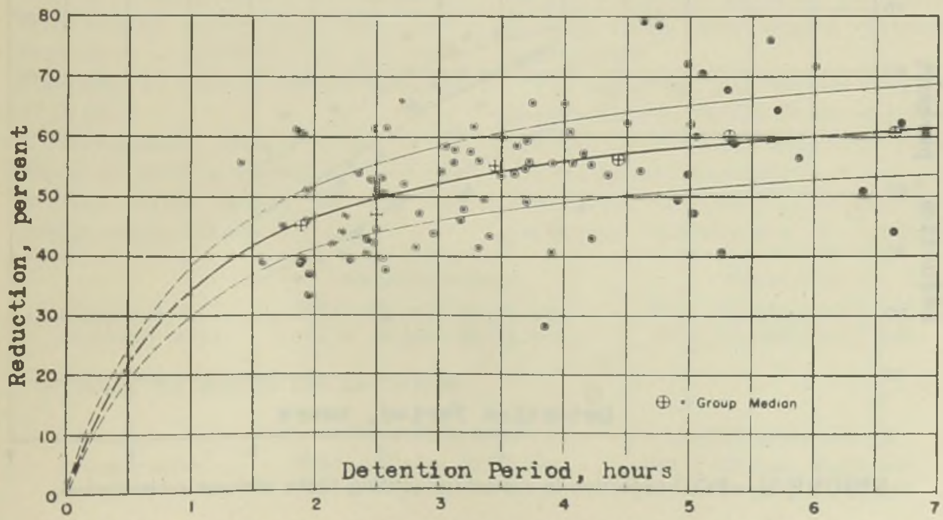


FIGURE 69.—SS reduction in primary settling tanks without recirculation.

cated in the following:

	Without Recirculation	With Recirculation
Primary	18	12
Secondary	7	10

The median value of the ratio of length to width in rectangular tanks was 3.4; the ratio of length to depth was 6.0. The median value of average displacement velocity in rectangular tanks was 0.42 fpm; in circular tanks, 0.12 fpm. The median value of flow per unit length of weir for rectangular tanks was 24,000 gallons daily per foot of weir; for circular tanks, 8,300 gallons daily per foot of weir.

Detention periods, as based upon average flow, are shown in Table 16 for the four groups of tanks, along with other information relating to operation. Average values are as follows:

	Without Recirculation	With Recirculation
Primary tanks	3.77 hrs	2.58 hrs
Secondary tanks	4.16 hrs	1.88 hrs

Average surface settling rates in gpd/sf for the four groups are the

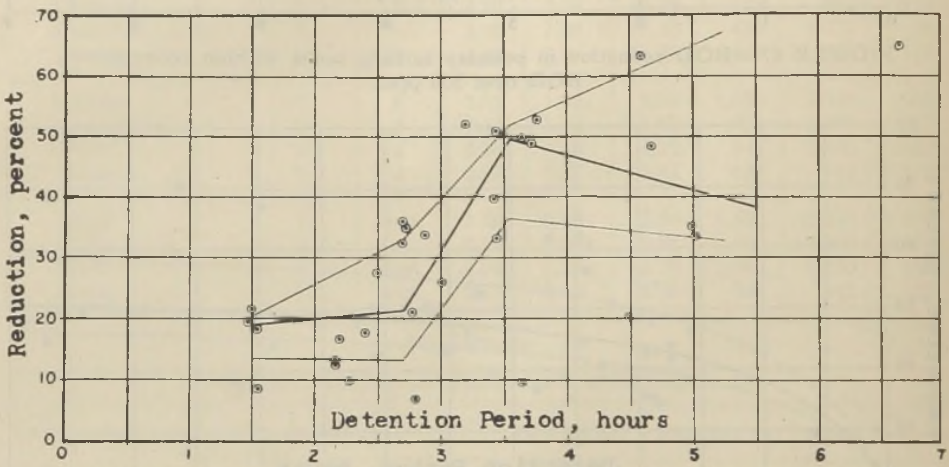


FIGURE 70.—BOD reduction in primary settling tanks with recirculation.

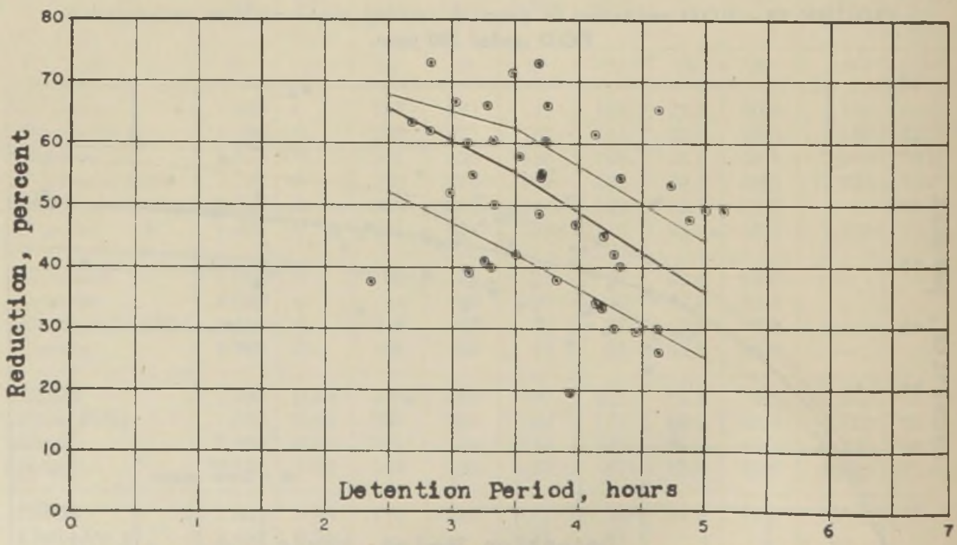


FIGURE 71.—BOD reduction in secondary settling tanks without recirculation.

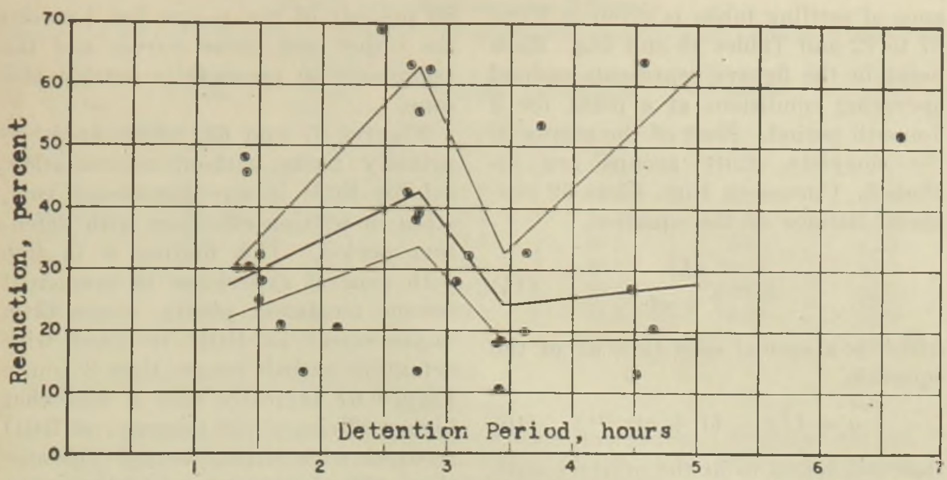


FIGURE 72.—BOD reduction in secondary settling tanks with recirculation.

following:

	Without Recirculation	With Recirculation
Primary tanks	506	850
Secondary tanks	429	1,072

In this breakdown, the Claiborne No. 1 plant data have not been included; these two small tanks, which precede activated sludge basins, were atypical in design. They do indicate, however, performance attainable in short-detention intermediate units. From the foregoing average values it is seen that of the tanks under consideration, those with recirculation were somewhat more heavily loaded as to flow. However, such a condition did not occur generally, as may be seen from Figs. 67 to 72.

Performance with respect to removal of BOD is shown on Table 17. Average BOD results for the four groups are as follows:

A distinctly inferior performance is noted in tanks with recirculated flow. It should be stated that the concentrations of BOD and SS in the tank influent were calculated (in cases where direct measurements were not made) from the strength and discharge of *both* sewage flow and recirculated flow. Hence the performance reported represents actual removal in the tanks.

Average value of volume of sludge produced was 8,880 gallons per mg of sewage for tanks without recirculation, and 8,080 gallons per mg of sewage in tanks with recirculation. Since secondary sludge was normally pumped to primary tanks, these figures represent total sludge.

The foregoing analysis for 18 posts as presented in Tables 15, 16 and 17 is primarily intended to delimit the range of loading and operation in representative plants. A more detailed and comprehensive investigation of perform-

	Without Recirculation	With Recirculation
Primary tanks	299 to 177 ppm, 39.1% rem	222 to 153 ppm, 29.4% rem
Secondary tanks	81 to 47 ppm, 42.7% rem	79 to 61 ppm, 25.6% rem

Average SS results are as follows:

	Without Recirculation	With Recirculation
Primary tanks	293 to 137 ppm, 52.9% rem	210 to 132 ppm, 36.4% rem
Secondary tanks	85 to 33 ppm, 61.8% rem	134 to 59 ppm, 47.4% rem

ance of settling tanks is given in Figs. 67 to 72 and Tables 18 and 18a. Each point in the figures represents normal operating conditions at a plant for a 3-month period. Most of the plants in the complete study groups are included. Curves on Figs. 67 to 69 represent fittings of the equation,

$$y = \frac{aLt}{1 + at} \qquad (15)$$

which is a special case ($k = a$) of the equation,

$$y = L[1 - (1 + at)^{-k/a}] \qquad (16)$$

that was found to fit the military sedimentation data best. Theoretical considerations justify the application of this equation to settling efficiency data. The terms in the equation have the following significance:

y = proportion of BOD (or SS) removed at any detention period t.

L = the ultimate proportion of BOD (or SS) removable by sedimentation.

k/a = factor reflecting influence of turbulence, shape of tank, and hydraulic conditions of entrance and exit.

Values of L and a, for the curves of best fit, are given in Table 18. The upper and lower curves on each graph pass approximately through the quartiles of the dispersion of points. Thus 50 percent of the points fall between the upper and lower curves and the remaining 50 percent lie outside this zone.

Figures 67 and 68, which apply to primary tanks without recirculation, exhibit little if any significant variation in settling efficiency with detention period. This finding is in line with general experience in municipal sewage treatment plants, where little improvement in BOD is noted with detention periods longer than 3 hours. Figure 67 indicates that a somewhat higher efficiency (35 percent) of BOD removal with strong sewage (greater than 350 ppm) occurred than with more diluted sewage (less than 350 ppm) as shown in Fig. 68 (30 percent). This tendency toward greater settling efficiency with strong sewage is also in line with municipal experience.

Removals of SS (50 to 60 percent), were significantly greater than BOD. Moreover, some increase in SS removal efficiency was noted with detention periods greater than 3 hours. Detention periods were computed on a basis of average daily flow.

Figure 70 presents results obtained in primary tanks with recirculation. Detention periods in these units were based upon a discharge that included recirculated flow. Generally lower efficiencies were noted in tanks with recirculation when detention periods were less than about 3 hours. However, the overall median removal efficiency for tanks with recirculation

TABLE 18.—Sedimentation Efficiency. Parameters of Fitted Curves

Figure	Description	L = Ultimate Percent Reduction	a = Rate Constant per Day
(1)	(2)	(3)	(4)
67	BOD Reduction in Primary Settling Tanks without Recirculation. BOD over 350 ppm	36.74	6.604
68	BOD Reduction in Primary Settling Tanks without Recirculation. BOD under 350 ppm	35.25	1.015
69	SS Reduction in Primary Settling Tanks without Recirculation.	70.70	0.944

TABLE 18a.—Sedimentation Efficiency

Figure	n = Number of Points	Overall Median, Percent Reduction	LQ = Lower Quartile	UQ = Upper Quartile	Q = Semi Interquartile Range	Standard Deviation of Median (a)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
67(b)	60	35.0	31.7	38.3	3.30	0.79
68(c)	45	28.5	22.5	32.5	5.00	1.38
69(d)	89	57.3	44.5	60.0	7.75	1.53
70(e)	31	32.4	18.0	46.0	14.00	4.67
71(f)	43	49.3	40.0	60.5	10.25	2.90
72(g)	31	30.5	20.5	44.5	12.00	4.00

(a) Standard Deviation of Median $= \dfrac{1.25Q}{(0.6745)\sqrt{n}}$.

(b) BOD Reduction in Primary Settling Tanks without Recirculation. BOD over 350 ppm
(c) BOD Reduction in Primary Settling Tanks without Recirculation. BOD under 350 ppm
(d) SS Reduction in Primary Settling Tanks without Recirculation.
(e) BOD Reduction in Primary Settling Tanks with Recirculation.
(f) BOD Reduction in Secondary Settling Tanks without Recirculation.
(g) BOD Reduction in Secondary Settling Tanks with Recirculation.

(Figure 70) of 32.4 percent, as shown in Table 18a, is about the same as that for plants without recirculation.

The BOD removal in secondary tanks without recirculation, as shown in Fig. 71, is fairly high and uniform at all detention periods. The curve drawn through group medians, however, would indicate some reduction in efficiency at excessively long detention periods, but this effect may in part be attributed to vagaries of random sampling fluctuations. The overall median BOD reduction in secondary tanks without recirculation is 49.3 percent, which is notably higher than that observed in primary tanks. A considerable dispersion of results for BOD removal efficiency in secondary tanks with recirculation is shown in Fig. 72. Again little, if any, correlation is exhibited between reduction efficiency and detention period. Median BOD removal for all tanks with recirculation is 30.5 percent, a value definitely lower than that of secondary tanks without recirculation.

Table 18a presents information bearing upon the dispersion of the observations and the reliability of the median values that are given in column (3). It is seen that the dispersion of the data pertaining to percent reduction in Figs. 67 to 69, as measured by the semi-interquartile range (column 6), is considerably smaller than in Figs. 70 to 72. Consequently, the median values of the first three graphs are more precisely established than the last three. Quantitative expression of the reliability of the median is given by the standard deviation of the median for which an estimate is given in the last column.

Data for surface settling rates versus BOD and SS removal were plotted in a form analogous to Figs. 69 to 72. A great dispersion was observed and little correlation between surface settling rate and removal efficiency could be detected. It was inferred that surface settling rate was inferior to detention period as a measure of performance in settling tanks at military plants. This might not have been the case if flows per unit volume of tank had been larger.

Additional data relating to settling tanks are given in subsequent sections dealing with various treatment methods and devices.

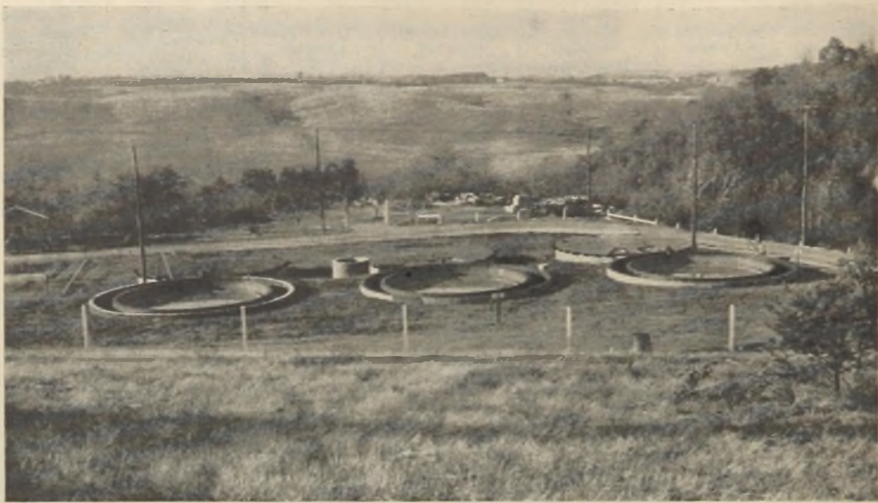


PLATE 5.—Settling tanks at Woodrow Wilson G.H., Va. Sludge is removed by gravity through hopper bottoms to digesters directly behind settling tanks.

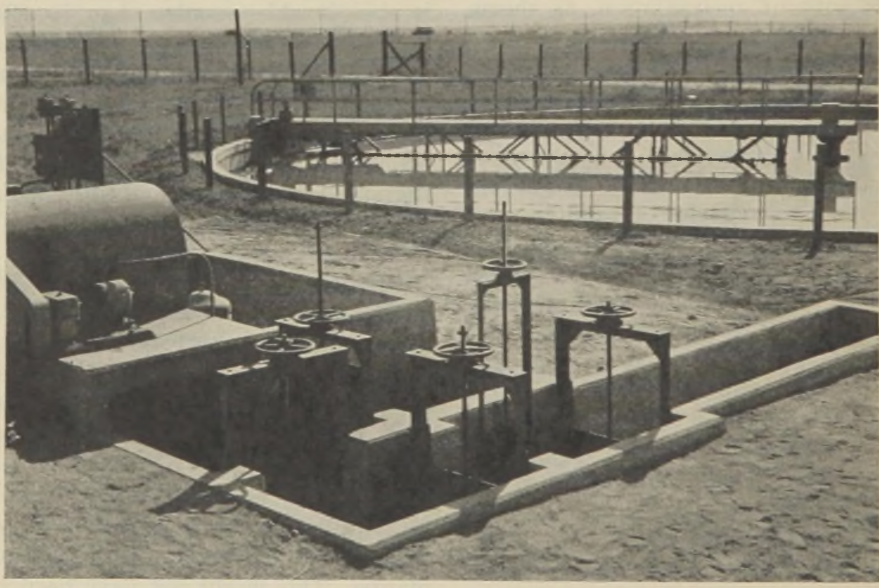


PLATE 6.—Primary settling tank at Buckley Field, Colo. Helical scum removal mechanism may be seen at top of sludge removal arm. Mechanical racks are shown in foreground.

Comparison of Sedimentation Performance with Municipal Plants

Despite the circumstance that detention periods in military plants were longer than those usual in municipal practice, performance as measured by removal of BOD was distinctly inferior. Keefer reported results for 26 municipal plants in which the average removal of BOD in primary settling tanks was 35.1 percent. In these plants the average strength of raw sewage (BOD) was 234 ppm. However, in military plants with raw sewage less than 350 ppm BOD the median removal was only 28.5 percent. In municipal plants with raw BOD in excess of 300 ppm removals of BOD of 40 percent are normal; such performance excels that noted in military plants treating concentrated sewage.

In seeking a reason for the disparity as to settling performance between military and municipal installations, the hypothesis that detention periods in the former were *too* long must be rejected. No significant downward trend in data, Figs. 67 to 69, is apparent with increasing detention periods. On the contrary the points indicate a small but definite increase with detention periods longer than 2 hrs.

The effect of secondary sludge, which was not accredited as part of the primary tank, might in some cases reduce the removal percentage somewhat since secondary sludge would perhaps not resettle completely. In any event, however, this effect was not large and, moreover, secondary sludge wasted in primary tanks would affect primary removal in municipal plants in a similar way.

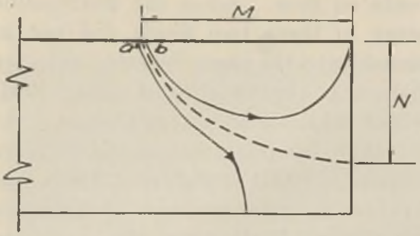
In primary tanks with recirculated flow another explanation for lower efficiencies that pertains to vagaries of the BOD test may be advanced. Primary loading was calculated as the sum of the BOD weight in the raw sewage plus the BOD weight in the recirculated flow. Since the putrescible matter in these two fluids did not in general have the same reaction velocity constant k, the weights of 5-day BOD are not addable in a strict sense. On the other hand no assumption as to addition of BOD of different fluids was involved in measurement of primary tank effluent BOD, since this quantity was measured directly. Therefore influent and effluent BOD values were evaluated in essentially different ways.

In a few plants analyses were made of BOD not only in raw sewage and recirculated flow but also in the mixture comprising the primary tank influent. Usually in these plants the weight of BOD in the mixture did not differ greatly from that of the raw sewage plus recirculated flow, especially when nitrification did not occur. Consequently no large amount of error in measurement of primary tank performance could be attributed to inadequacy of the BOD test.

It is believed that one factor, more than any other, was responsible for the differences in settling efficiency observed between military and civilian plants. This was non-uniformity of flow. Flow at military installations, as has been stated, tended to be less uniform than that in municipalities of corresponding size, as a result of the following two factors: (1) regimented mode of life at cantonments inherent in military training and (2) improper pump sizes and float switch adjustments at many posts that were conducive to pulsating flow. An analysis indicates that the effect of irregularity of flow distribution upon settling efficiency may be given quantitative expression.

Influence of Flow Variation upon Sedimentation Efficiency

Consider the region in a rectilinear tank in the vicinity of the effluent weir, which is located at the end of the tank:

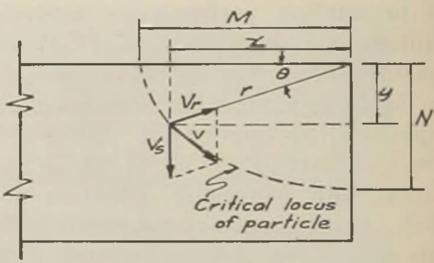


With a fixed rate of flow per unit length of weir q, and with particles of constant settling velocity v_s, there exists a critical point on the water surface a distance M from the weir. A particle a, with settling velocity v_s, released at distance slightly greater than M, will describe under the influence of gravity and tank flow a path with reversed curvature (see diagram) and settle upon the bottom of the tank.* On the other hand, a particle b, with the same settling velocity, released at a distance slightly less than M, will describe a simple curved path and be carried over the weir in the effluent.

In case of particle a, the effect of the settling velocity transcends that of the radial velocity of flow toward the weir. With particle b the radial velocity overcomes the settling velocity. Between the two paths lies a critical locus (dashed line on sketch) representing the path of a particle of settling velocity v_s in which the effect of the settling velocity is exactly balanced by that of the radial velocity. Such a particle would approach the end of the tank at a distance N below the weir.

The locus evidently delimits a zone near the weir that has no utility in sedimentation; all particles of settling velocity v_s inside the zone (indicated by the dashed line) will be carried out over the weir in the effluent. All other particles of same settling velocity will be removed in the sludge. The effective volume of the tank, therefore, is the gross volume minus that within the critical locus. For particles of smaller settling velocity, the critical zone will be larger. Indeed, with slowly settling particles, M may exceed the length of the tank, and N the tank depth. Evidently such particles cannot be removed by settling.

In addition to settling velocity, flow per unit length of weir determines the size of the ineffective zone. If flow per unit length of weir is increased, the radial component of velocity becomes stronger relative to the settling velocity. Accordingly, particles at greater distance will be carried over the weir and the critical locus moves farther away from the weir crest, thereby reducing the effective volume of the tank. It is worthwhile to ascertain the volume of the ineffective zone for particles of size and density occurring in sewage. This may be done with the aid of the following diagram:



With reference to the diagram, it is seen that the velocity of the particle v may be resolved into horizontal and vertical components as follows:

$$\frac{dx}{dt} = v_r \cos \theta$$

$$\frac{dy}{dt} = v_r \sin \theta + v_s$$

The radial velocity v_r varies inversely as the distance r from the weir.

$$v_r = -\frac{2q}{\pi r}$$

where

q = flow per unit length of weir.

* Neglecting currents induced by density differences, wind, and sludge removal mechanism, the direction of flow is taken radially upward toward the weir.

The slope of the path of the particle is the ratio of the vertical and horizontal components of velocity which is given in the following expression:

$$\frac{dy}{dx} = \tan\theta + \frac{v_s}{v_r}\sec\theta \quad (17)$$

The relations between the coordinates are as follows:

$$y = r\sin\theta$$

$$x = r\cos\theta$$

$$dy = r\cos\theta\, d\theta + \sin\theta\, dr$$

$$dx = -r\sin\theta\, d\theta + \cos\theta\, dr$$

Eliminating dy and dx from Equation (17), and reducing to simplest form, the following relation is obtained:

$$\frac{dr}{d\theta} - (\tan\theta)\, r = -\frac{2q}{v_s\pi}\sec\theta \quad (18)$$

The distance N at which the critical locus intersects the end of the tank is located at a depth such that the upward velocity of flow exactly equals the settling velocity of the group of particles under consideration.

$$v_s = \frac{2q}{\pi N}$$

$$N = \frac{2q}{\pi v_s}$$

Thus, the coefficient on the right hand side of Equation (18) is seen to be the dimension N. Integrating this equation, a final relation is obtained for the coordinates of the critical locus.

$$r = \frac{q}{v_s}\left[1 - \frac{2}{\pi}\theta\right]\sec\theta \quad (19)$$

when

$$\theta = 0, \quad r = M = \frac{q}{v_s}$$

Hence, the essential dimensions of the ineffective zone are:

$$N = 2q/\pi v_s \quad \text{and} \quad M = q/v_s$$

The volume per unit length of weir bounded by the critical locus is obtained from Equation (19) by further integration:

$$V = \tfrac{1}{2}\int_0^{\pi/2} r^2 d\theta = \left(\frac{2}{\pi}\log_e 2\right)\frac{q^2}{v_s^2} \quad (20)$$

$$V = 0.441\, q^2/v_s^2$$

In general, the ineffective volume for geometrical conditions other than those assumed for purposes of illustration (circular tanks, weir troughs, etc.) will be given by an equation similar to (20) of the form:

$$V = kq^2/v_s^2 \quad (21)$$

where

$$k = \text{constant, } 0.35 \text{ to } 0.55$$

Equations 20 and 21 indicate that the volume of a settling tank that is ineffective for settling particles of a given size range is a *quadratic* function of the ratio of the flow intensity and the settling velocity. This is a significant fact that has an important bearing upon the influence of irregularity of flow upon settling efficiency. If the flow per unit length of weir is doubled, then the ineffective volume for particles of a particular size is quadrupled. The same is true for all settleable particles—an increase in flow intensity reduces effective settling volume at a disproportionate rate. Quantitative aspects of the foregoing analysis are set forth in an example.

Example: To determine the ineffective volume in a rectangular settling tank for sewage particles of average settling velocity due to upward flow toward weir.

Assume:

q = 35,000 gal/day/lineal ft (common value during day)
= 195 cf/hr/lineal ft

v_s = 3 to 7 ft/hr, say 5 ft/hr

Ineffective volume from Equation 20

$V = 0.441\,(q/v_s)^2 = 0.441\,(195/5)^2$
= 670 cf for each lineal foot of weir.

With the conditions assumed, the critical zone would extend from a distance of about 40 ft from the weir and all the way to the bottom of the tank. It is seen therefore that the ineffective volume may constitute a substantial proportion of the total. At lower flows and for more rapidly settling particles, the ineffective volume is of course smaller. With slowly settling particles such as the light pinpoint floc sometimes noted in the vicinity of effluent weirs in activated sludge secondary tanks, the ineffective volume would be even larger than that estimated in the foregoing example, which pertains to average conditions.

The effect of flow irregularity is quantitatively indicated by considering the following two cases:

I. Flow is assumed to be intermittent due to the action of pumps; 50 percent of the time no flow occurs and 50 percent of the flow is twice the average, that is, 390 cf/hr/lin ft. The period of the cycle is of the order of 20 minutes. The average value of the ineffective volume then is

$$V = \tfrac{1}{2}[0 + 4 \times 670] = 1{,}340 \text{ cf.}$$

With this pattern of flow distribution, therefore, the ineffective volume becomes *twice* as large even though the average flow remains the same. The effective volume is reduced disproportionately and settling efficiency is impaired.

II. Flow is assumed to be sinusoidal during the 24-hr period reflecting water use variation with post routine.

$$q = \bar{q} + a \sin 2\pi t$$

where

q = instantaneous rate of flow

$\bar{q}$ = average daily flow

$2a$ = max flow minus min flow

t = time

Average value of the ineffective volume,

$$V = \frac{0.441}{v_s^2}\int_0^1 q^2 dt$$

$$= \frac{0.441}{v_s^2}\int_0^1 (\bar{q} + a \sin 2\pi t)^2 dt$$

$$V = \frac{0.441}{v_s^2}\left(\bar{q}^2 + \frac{a^2}{2}\right) \qquad (22)$$

If the flow ranges from 25 percent to 175 percent of the daily average, then the value of a, the half-range will be 75 percent of the average. With numerical values assumed in the example, the ineffective volume will be

$$V = \frac{0.441}{5^2}[(195)^2 + (0.5)(0.75 \times 195)^2] = 860 \text{ cf.}$$

TABLE 19.—List of Primary Treatment Plants

Post	State	Sv C or Navy	Type of Station	Date Plant Completed	Date Major Additions or Alterations Completed	Operating Period Studied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Camp Blanding	Fla	4th	Div Trg	Jul 41	Dec 43 Jan 44	Mar 43 to Feb 44
Davisville	R I	I	Navy Trg	Feb 43	Mar 43	Jul 43 to Aug 44
Camp Funston	Kans	7th	Arm., MC & T D Trg	Mar 41	Aug 43	Jun 43 to Aug 44
Seymour Johnson Field	N C	4th	Tact Fields & Fly Trg	Apr 43	—	May 43 to Mar 44
Camp Peary	Va	III	Navy Trg	Mar 43	May 43	Nov 43 to Sep 44
Camp Wheeler	Ga	4th	Repl Trg	Apr 41	—	May 43 to Jun 44

TABLE 20.—General Data Relating to Primary Treatment Plants

Post	Approx Plant Elev (ft)	Range of Sewage Influent Temp (°F)	Average Sewage Influent pH	Lineal Feet of Sewers per Capita (a)	Sampling	
					Hours per Day	Average Number per Month
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Blanding	—	74–94	7.5	6.8	24	30
Davisville	24	66–73	7.1	2.5	8	5
Funston	1,045	59–76	7.3	1.5	16	25
S Johnson Field	—	65–90	7.7	3.8	22	30
Peary	25	61–90	8.4	3.0	24	5
Wheeler	—	60–89	6.8	6.4	19	23

TABLE 21.—Units in Primary Treatment Plants

Post	Plant Units							
	Screens or Racks	Commin-utors	Grit Chambers	Primary Settling Tanks	Chlorine Contact Tanks	Digestion Tanks, Heated	Digestion Tanks, Unheated	Sludge Beds
(1)	(2)	(3)	(4)	(6)	(9)	(10)	(11)	(12)
Blanding	2	2	1	2	1	2	2	16
Davisville	1	1	0	4	0	2	0	8
Funston	1	0	0	2	0	1	0	4
S Johnson Field	1	1	1	4	0	1	0	10
Peary	1	1	0	4	1	1	2	24
Wheeler	1	0	1	2	0	2	0	5

TABLE 22.—Design Capacities and Loading of Primary Treatment Plants

Post	Number of Months in Operating Period Studied	Population				Flow				
		Design Capacity (a)	Actual			Design (mgd) (a)	Actual			
			Average	Maximum	Minimum		Average (mgd)	Maximum (mgd)	Minimum (mgd)	Gal per Capita Daily
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Blanding	12	97,500	34,300	47,700	14,400	6.82	1.82	4.72	0.32	53.2
Davisville	14	14,200	13,200	15,600	11,600	0.99	1.10	1.47	0.80	82.9
Funston	15	24,400	6,890	11,900	4,210	1.71	0.52	1.33	0.00	77.4
S Johnson Field	11	27,900	18,900	22,800	15,700	1.95	1.22	2.25	0.47	64.8
Peary	11	78,100	46,200	61,300	3,180	5.47	3.96	4.68	3.35	85.7
Wheeler	14	26,700	26,600	31,100	23,600	1.87	1.60	3.70	0.69	57.8

(a) Design capacities are based chiefly on criteria in OCE Engineering Manual (1943). See Text. Figure in col (3) based upon detention period of 2.5 hours and 70 gal per capita.

TABLE 22a.—Percent of Design Capacity Utilized in Primary Treatment Plants

Post	Primary Settling Tank (%)	Sludge Digester (%)	Sludge Beds (%)
(1)	(2)	(5)	(6)
Blanding	27	72	51
Davisville	110	202	123 (a)
Funston	31	21	34
S Johnson Field	63	91	66
Peary	72	89	95
Wheeler	85	129	177

(a) Covered sludge beds.

This value is 28 percent larger than that for uniform flow. It is evident therefore that irregularity of flow, a salient characteristic of military sewage, was a principal source of difficulty in obtaining efficient settling tank performance.

Primary Treatment Plants

An analysis of six primary treatment plants with separate sludge digestion is given in Tables 19 to 25 and Figs. 73 and 74. In this analysis the viewpoint is of performance of plants as entities rather than only of settling units. Four of the primary plants (Table 19) were at Army posts and two at Navy installations. Three plants were located in the South. It should be noted that the length of sampling period (8 hours per day) at Davisville was short and hence comparison with

TABLE 23.—BOD Loading and Performance of Primary Treatment Plants

Post	Plant Influent			Detention Period, Hours			Final Eff (ppm)	Overall BOD Removal (%)
	Flow (mgd)	BOD		At Av Flow	At Max Flow	At Min Flow		
		PPM	Pounds per Capita Daily					
(1)	(3)	(4)	(5)	(16)	(17)	(18)	(36)	(40)
Blanding	1.82	195	0.086	4.65	1.80*	27.20*	141	27.3
Davisville	1.10	207	0.143	2.26	1.69	3.10	153	26.0
Funston	0.52	291	0.185	5.38	2.16	—	156	46.4
S Johnson Field	1.22	349	0.189	3.96	2.15	10.30	207	40.6
Peary	3.96	363	0.260	3.45	2.92	4.08	144	60.4
Wheeler	1.60	391	0.190	2.92	1.26	6.80	259	33.8

* One tank in service.

TABLE 23a.—SS Loading and Performance of Primary Treatment Plants

Post	Plant Influent			Final Effluent (ppm)	Overall SS Removal (%)
	Flow (mgd)	Suspended Solids			
		PPM	Pounds per Capita Daily		
(1)	(3)	(4)	(5)	(9)	(14)
Blanding	1.82	311	0.139	125	60.0
Davisville	1.10	271	0.187	155	43.0
Funston	0.52	335	0.216	148	55.7
S Johnson Field	1.22	279	0.151	152	45.5
Peary	3.96	291	0.209	128	56.2
Wheeler	1.60	270	0.131	115	57.2

TABLE 24.—Sludge Data Relating to Primary Treatment Plants

Post	Number Months in Operating Period Studied	Sludge Digester				Schedule of Sludge Loading
		Total CF per Capita (a)	Heated, Percent of Total Volume	Covered, Percent of Total Volume	Average Operating Temperature (°F)	Primary to Sludge Digesters—Times per Day
(1)	(2)	(3)	(4)	(5)	(6)	(9)
Blanding	12	5.1	45	71	85–90	8
Davisville	14	1.5	100	100	93–104	3
Funston	15	14.1	100	100	91	9
S Johnson Field	11	3.3	100	100	90	6
Peary	11	4.4	33	100	85–90	3
Wheeler	14	2.4	100	100	90	8

(a) Based on actual population, Table 22 col (4).

TABLE 24a.—Sludge Data Relating to Primary Treatment Plants

Post	Gallons Raw Sludge per MG Sewage	Volatile Solids, Percent of Total Solids		Sludge Disposal			
		Raw Sludge	Digested Sludge	SF per Capita (a)	No. Months Sludge Drawn per Year	Sludge Drawn Yearly from Digesters (cf/cap)	Sludge Removed Yearly from Beds (cf/cap)
(1)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Blanding	4,100	81.0	58.0	1.95	—	1.2	—
Davisville	7,460	74.7	47.6	0.81(b)	12	3.2	—
Funston	7,980	64.8	—	2.91	8	—	—
S Johnson Field	17,700	75.0	—	1.53	—	—	—
Peary	—	—	42.5	1.05	8	1.6	—
Wheeler	7,180	76.1	59.0	0.54	12	1.5	0.5

(a) Based on actual population, Table 22 col (4).
(b) Covered sludge beds.

TABLE 25.—Chlorination and Effluent Disposal in Primary Treatment Plants

Post	Chlorination		Receiving Water Body	
	Points of Application Used	Residual Chlorine (ppm)	Name	Type
(1)	(2)	(3)	(4)	(5)
Blanding	—	—	—	Flowing Stream
Davisville	Pre	0.6	Atlantic Ocean	Tidewater
Funston	—	—	Kansas River	Flowing Stream
S Johnson Field	—	—	Neuse River	Flowing Stream
Peary	Post	0.6	York River	Tidewater
Wheeler	—	—	—	Flowing Stream

the other plants is somewhat impaired. Aside from this limitation, records of analytical work and operation at Davisville were exceptionally complete and representative of performance of primary plants under conditions existing in a military regime.

The size of plants, as measured by design flow, varied from 0.99 mgd at Davisville to 6.82 mgd at Blanding. The design capacities were calculated as being that flow which in passing through the tank would have experienced a detention period of 2.5 hrs. Design population was calculated on the presumption that per capita flow would be 70 gal. Actual per capita flow (column 11, Table 22) averaged 70.3 gcd. On this basis, which was similar to that stipulated by the Army

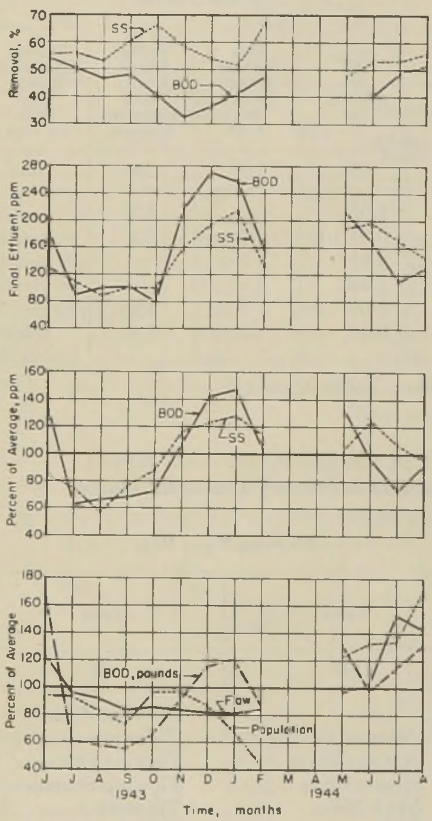


FIGURE 73.—Loading and performance of primary sedimentation units at Camp Funston.

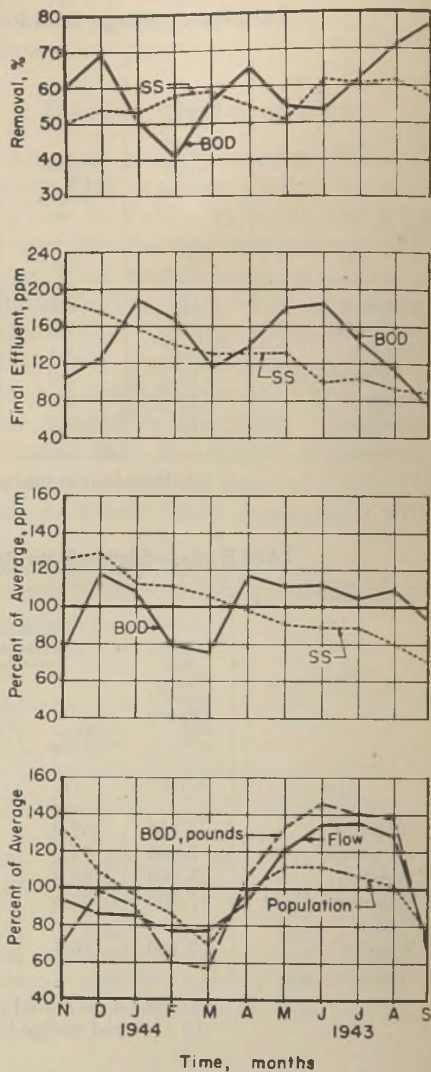


FIGURE 74.—Loading and performance of primary sedimentation units at Camp Peary.

Engineering Manual, all plants were underloaded with respect to contributory population. With respect to flow, all but one of the plants were underloaded. The ratio (stated as a percent) of actual load to design load, varied from 27 percent at Camp Blanding to 110 percent at Davisville; average percent of design capacity utilized was 65 percent. This general condition of underloading was met in some of the

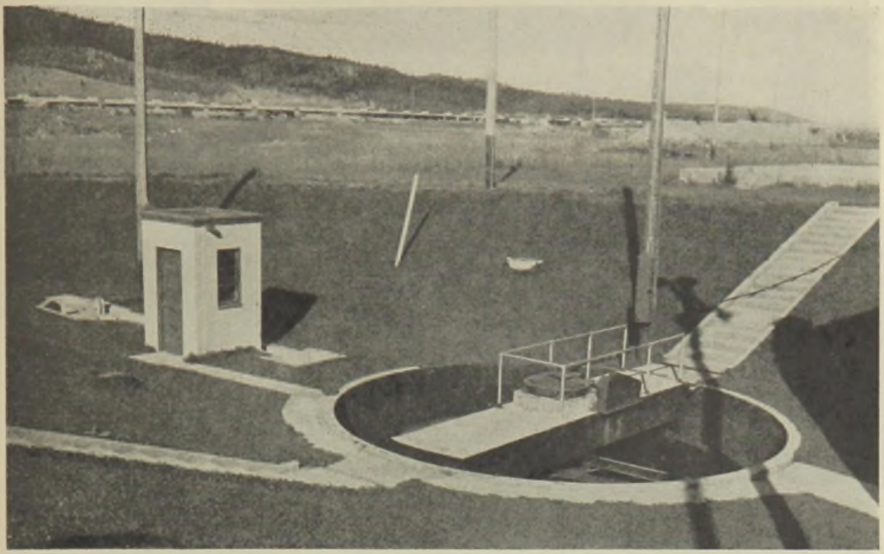


PLATE 7.—Small settling tank at Naval Supply Depot, Spokane, Wash.

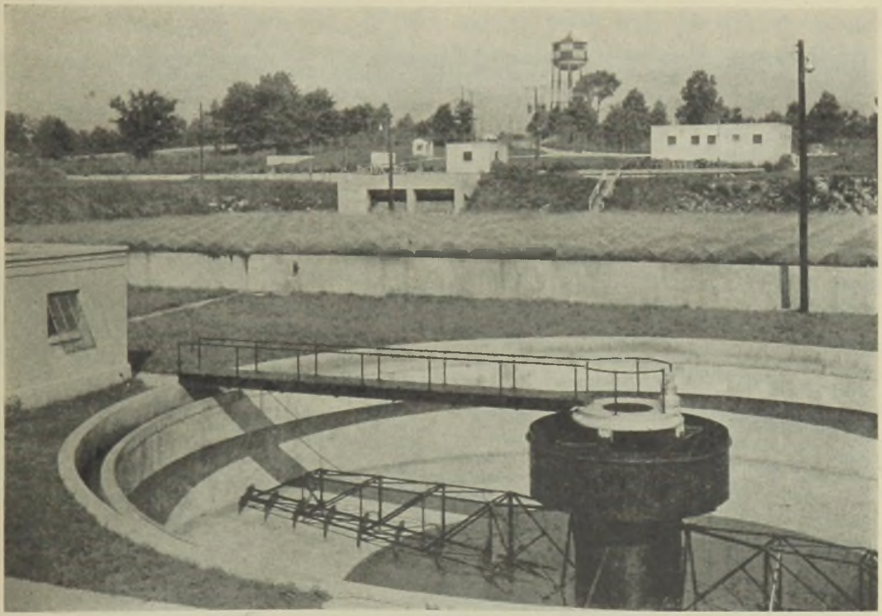


PLATE 8.—Secondary settling tank at Fort Knox, Ky.

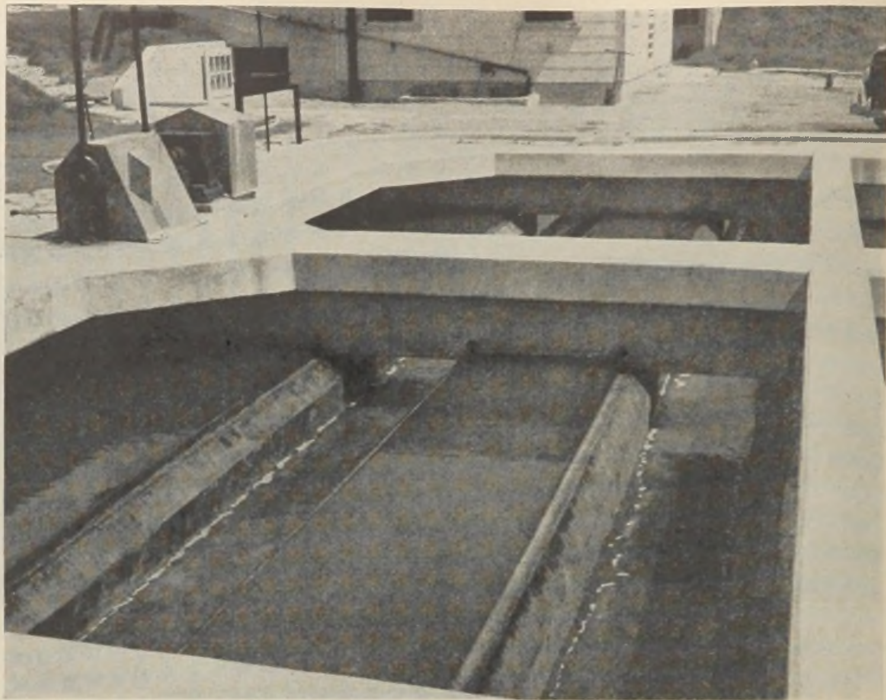


PLATE 9.—Effluent weirs at Camp Hood, Texas.

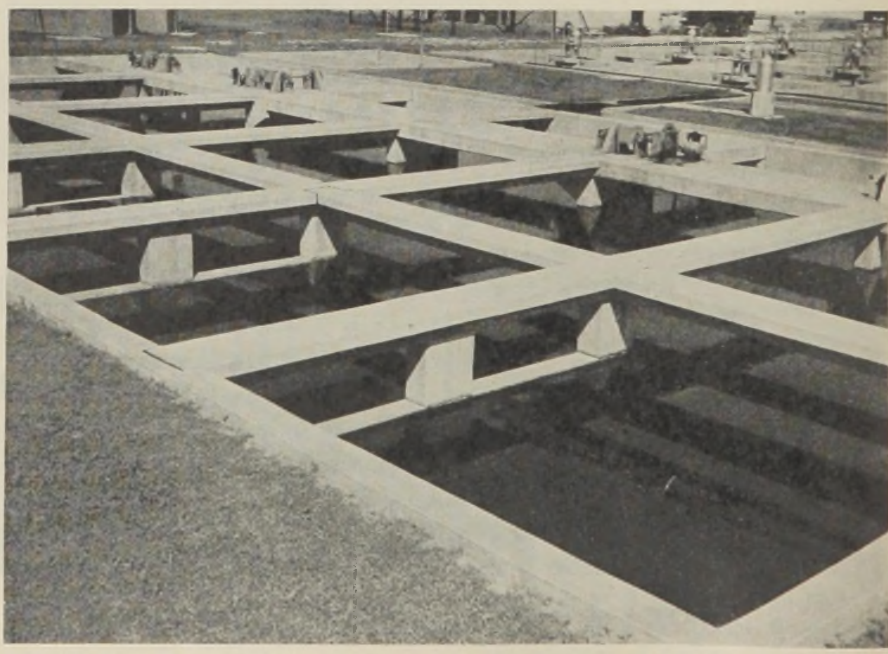


PLATE 10.—Final settling tank at Main Plant, Camp Claiborne, La.

posts by removal of units from service in order to avoid excessive detention periods. Davisville and Camp Wheeler were overloaded as to sludge digestion and drying facilities as indicated in Table 22a.

Columns (5), (6), (9) and (10) in Table 22 refer to extremes of population and flow, the highest and lowest monthly average during the period studied. Columns (17) and (18) in Table 23 are based upon average values of daily maximum and minimum flows.

Concentration of raw sewage varied from 195 ppm BOD at Blanding to 391 ppm at Wheeler. The former post had an exceptionally low per capita BOD, which stemmed in part from training activities that involved bivouacking at points removed from the sewerage system. At Blanding, infiltration was estimated to be 10,000 gallons per mile per day. Grease was excessive and laundry waste constituted about 9.5 percent of the flow. At Davisville, grease was reported as excessive during the period of operation studied. The relatively lower BOD performances at Blanding and Davisville are chiefly to be attributed to grease. Prechlorination at Davisville was successful in retarding septic action and aided in maintaining the tank surface clear. Sludge hoppers had to be reconstructed with steeper sides in order to concentrate sludge at the outlet pipe. Before alteration, it had been difficult to remove the entire sludge content without pumping large quantities of sewage. The Davisville plant was one of few with glass-covered sludge beds.

Digester capacity at Davisville was small relative to other primary plants. Temperatures as high as 104° F had to be maintained in order to ameliorate conditions brought about by grease. Scum and grease accumulation under digester covers constituted an operating difficulty. Lime was added, depending upon pH. Sludge was well digested.

Performance of the plant at Camp Funston is representative of small military primary plants. As may be seen from Fig. 73, loading and removal exhibited considerable variation over the period of study. It is pertinent to note, however, that perfect inverse relation between loading and performance did not occur. Bivouacking influenced to a marked degree the relation between population and BOD load (lb.). During the fall and winter of 1943, population at Funston dropped rapidly, yet the weight of BOD in the influent exhibited a continued rise. Performance during this period was relatively poor.

Laundry wastes at Seymour Johnson Field amounted to about 25 percent of the flow during the period of operation studied. This accounted in part for the relatively low removal of SS relative to BOD. Grease was not a problem; no unusual operating difficulties were encountered as to operation. Lime added to raw sewage was found to be of value in odor control. Diurnal variation in flow was marked; long detention periods occurred. Subsequent to the period of operation studied, flow decreased and one tank only was retained in service. Sludge was removed by gravity into a well from which it was pumped. It may be seen from Table 24a that this arrangement did not produce a concentrated raw sludge. However, no difficulties with supernatant occurred; the liquor, which was quite clear, was returned to the plant through the influent sewer line.

A notable removal of BOD was reported at Camp Peary. Raw sewage had a high pH (Table 20). At the time of inspection, a distinct floc was noticeable in the plant influent. An unusually effective inlet arrangement induced even distribution of flow. Settling tank effluent was chlorinated with an average dosage of 107 lb per mg; an average residual of 0.6 ppm was maintained. Oyster beds in the York River constituted the reason for chlorination. Contact of about one hour at average flow was obtained in the 36-in. effluent line that extended 2,500 ft into the river.

On Fig. 74 are presented loading and performance curves for Camp Peary. A fairly distinct inverse proportionality is evident between loading and performance. Concentration of BOD in the effluent followed population changes. Fluctuations in BOD were greater than for SS; this was noted generally in primary tank performance.

At Camp Wheeler performance was normal with grease causing the principal operating difficulties. An arrangement was devised whereby scum was skimmed off into a manhole and grease extracted by heating in a gas-fired oven. The spent material was then burned. Extracted grease was turned into the general collection for making explosives. The plant effluent was discharged into a swift creek with a flow of 3 to 5 mgd.

CHAPTER V

TRICKLING FILTERS

Selection of Plants

More than one-half of sewage treatment plants installed at military posts during World War II included trickling filters. From an approximate total of about 300 filter plants, 34 were selected for complete study. It is pertinent to note that the selection of filter plants was based upon completeness and reliability of design and operational data; it was not based upon performance. The attempt was to emphasize *normal* rather than exceptional experience. Consequently, the results depict what actually occurred in filter plants as a group, without special emphasis upon a few plants that happened to operate under unusually propitious circumstances conducive to outstanding results. It is sufficient to say that with every type of trickling filter installed there were instances of notable performance as well as of substandard performance.

In selecting the plants for complete study, attention was given to size, geographical location, and relative numbers of each type.

Classification

In order to record performance, it was desirable to establish a scheme of classification. The usual classification of filters into two categories, standard and high capacity, was not suitable for military installations. It was found that any fixed limit as to load, arbitrarily selected to differentiate low capacity from high capacity units, resulted in unsatisfactory and unnatural groupings of filters without regard to actual design. A number of filters operated with loads in the zone between the usual low capacity and high capacity loading; designation of these as high capacity or standard units would have been an arbitrary and unjustifiable procedure.

Rate vs Capacity

Another objection to classification as to capacity is that the term is misleading. Within broad limits, set by size of media and provisions for ventilation, all filters have the same *capacity*—the real distinguishing variable is the *rate of application of load.* Aside from ancillary aspects, such as ventilation and hydraulic capacity, a standard filter is potentially a high capacity filter—it would perform as such if loaded accordingly. Consequently, *rate* is a more apposite term than *capacity*. The terms high rate and low rate filter are used to connote the total volume treated daily; they do not pertain to rates obtaining over short periods of time.

In the opinion of the subcommittee, there has been an exaggeration of the importance of continuity of flow applied to filters when intervals between dosage at a particular point are measurable in seconds. It is to be pointed out that discontinuities in flow are most marked at the surface. The filter bed itself interposes a detaining action that tends to distribute the percolating flow uniformly with respect to time. Tests indicate that the more irregular the rate of flow at the surface, the more efficient the damping and distributing action of the filter. A high instantaneous rate at the top of the filter becomes substantially reduced as it percolates under the retarding influence of the biological gels on the media. This effect is manifest at all filters whether dosed with rotary distributors or by means of nozzles. At Fort Sheridan, for example, it was observed that while the filter was dosed for 4 minutes out of 20, effluent continued to flow from the filter for 16 minutes out of 20.

Distribution of flow with respect to area appeared to have a more direct relation to efficiency than momentary flow irregularities at the surface. Again, however, with respect to areal distribution, it is essential to differentiate between distribution at the surface and distribution across a horizontal plane below the surface. Percolating flow applied in a non-uniform manner acquires a certain amount of horizontal motion that tends to produce a more uniform distribution. Fluid applied at a point on the surface of granular material travels downward approximately in a 15° cone.

Classification of Filters

Three attributes of design and operation of filters provided a logical basis of classification, which is outlined as follows:

(a) As to depth. Deep or shallow. Average depth of media greater or less than 4.5 ft.
(b) As to amount of recirculation. Recirculation ratio through filter greater or less than 100 per cent.
(c) As to number of stages. Single or multi-stage. Operation of filters in series.

On the basis of this method of grouping, the 34 plants are subdivided into the various categories. The number of plants in each division is indicated in the following:

Division I: Deep filters without recirculation—10 plants.

Division II: Deep filters with recirculation—11 plants.

(a) Low recirculation ratio—6 plants.
(b) High recirculation ratio—5 plants.

Division III: Shallow filters with recirculation—8 plants.

(a) Low recirculation ratio—1 plant.
(b) High recirculation ratio—7 plants.

Division IV: Two-stage filters—9 plants.

In the two-stage plants at Forts Sill and Warren, it was possible to analyze separately the operating results for the first-stage filtration including intermediate settling. Accordingly, these plants appear in two divisions. At Camps Forrest and Gordon, both single-stage and two-stage filtration were utilized during different periods. These plants also appear in two divisions. In discussing operation, the four divisions are examined in the foregoing order with the view of setting forth conditions conducive to deviations of performance from the normal. Finally, a discussion and generalization of filter plant performance as a whole is presented, including the effect of organic load, volumetric loading, recirculation ratio, and plant dimensions (absolute and relative size of the various units).

I. Deep Filters Without Recirculation

In Tables 26 to 38 is presented an analysis of the design, loading, operation and performance of ten filter plants that did not employ recirculation. Two of the posts, Camp Swift and Turner AAF, were in the South; the others were located in middle latitudes of the nation. Nine plants were at Army posts; one was at a naval installation, the Great Lakes Naval Training Center. The period of operation investigated at each plant varied from 12 to 20 months. All but two of the plants (Great Lakes and Lockbourne) may be considered as standard rate plants with load concentration varying within the usual range.

In order to facilitate comparison of the size of the plants relative to loading, a design population factor of 4,620 persons per acre-foot was utilized. This factor was in accord with loadings for standard rate filters as specified by the Army Engineering Manual. Most of the plants in the division were actually designed upon criteria that were similar to this figure. The plant at Lockbourne, however, was actually designed for a population of 12,000 persons per acre-foot, a factor intermediate between the standard and high rate range. At Great Lakes, the primary section of the main plant was actually designed for 45,000 persons, whereas the filters were designed for only 17,000. The entire primary effluent,

TABLE 26.—List of Deep Filter Plants Without Recirculation

Post	State	Sv C or Navy	Type of Station	Date Plant Completed	Date Major Additions or Alterations Completed	Operating Period Studied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Camp Campbell	Ky	5th	Arm, MC & TD Trg	Oct 42	Jul 43	Aug 43 to Aug 44
Camp Crowder	Mo	7th	Serv Trp Trg	May 42	Jul 43	Jul 43 to Aug 44
Fitzsimons Gen Hosp	Colo	7th	Gen Hosp	Sep 42	May 43	Jan 43 to Aug 44
Great Lakes (Main)	Ill	V	Navy Trg	Aug 42	Oct 44	Mar 43 to Oct 44
Fort Benjamin Harrison	Ind	5th	Rep Trg	Nov 41	Apr 43	Jan 43 to Aug 44
Lockbourne AAB	Ohio	5th	Tact Fields & Fly Trg	Jul 42	—	Jan 43 to Aug 44(a)
Fort Sheridan	Ill	6th	Rec Ctr	Oct 41	Dec 42	Jan 43 to Aug 44
Camp Swift	Texas	8th	Div Trg	Jun 41	Dec 42	Mar 43 to Aug 44
Turner AAF	Ga	4th	Tact Fields & Fly Trg	Sep 41	—	Jul 43 to Sep 44
Leonard Wood	Mo	7th	Div Trg	Apr 41	Aug 43	Jan 43 to Aug 44

(a) Oct 43 to May 44 omitted.

TABLE 27.—General Data Relating to Deep Filter Plants Without Recirculation

Post	Approx Plant Elev (Ft)	Range of Sewage Influent Temp (°F)	Average Sewage Influent pH	Lineal Feet of Sewers per Capita (a)	Sampling	
					Hours per Day	Average Number per Month
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Campbell	430	67–84	7.5	5.6	24	25
Crowder	1,200	54–96	7.5	5.3	24	8
Fitzsimons G H	5,340	70–98	7.4	1.9	24	10
Gt Lakes (Main)	—	—	—	3.2	24	25
Benj Harrison	760	40–80	7.4	9.0	24	8
Lockbourne AAB	—	54–78	8.0	—	16	25
Sheridan	660	47–80	7.7	5.4	14	6
Swift	—	71–93	7.5	2.6	24	30
Turner AAF	—	67–89	7.3	—	24	10
Leonard Wood	910	38–88	7.7	8.2	24	8

(a) Based on design population, Table 29, col (3).

TABLE 28.—Units in Deep Filter Plants Without Recirculation

Post	Plant Units											
	Screens or Racks	Comminutors	Grit Chambers	Imhoff Tanks	Primary Settling Tanks	Filters	Secondary Settling Tanks	Chlorine Contact Tanks	Digestion Tanks—Heated	Digestion Tanks—Unheated	Sludge Beds	Other Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Campbell	1	1	0	0	3	4	2	1	2	1	11	(a)
Crowder	1	0	0	0	2	5	3	1	2	1	22	(b)
Fitzsimons G H	1	2	1	0	1	4	1	1	2	0	4	(c)
Gt Lakes (Main)	0	2	0	0	4	2	2	1	2	1	9	—
Benj Harrison	2	2	0	2	0	1	2	2	0	0	6	(d)
Lockbourne AAB	1	1	0	2	0	2	1	0	0	0	4	(e)
Sheridan	1	0	2	0	2	1	1	2	2	0	11	—
Swift	1	1	0	0	3	4	3	0	0	2	22	—
Turner AAF	0	1	1	0	2	2	2	1	0	1	2	—
Leonard Wood	1	0	0	0	2	5	2	1	2	0	14	—

(a) Step aerator on effluent.
(b) Grease interceptor.
(c) 1.0-mg storage reservoir for effluent.
(d) Grease skimming tank.
(e) Preaeration for grease flotation.

TABLE 29.—Design Capacities and Loading of Deep Filter Plants Without Recirculation

Post	Number of Months in Operating Period Studied	Population				Flow				
		Design Capacity (a)	Actual			Design (mgd) (a)	Actual			
			Average	Maximum	Minimum		Average (mgd)	Maximum (mgd)	Minimum (mgd)	Gal per Capita Daily
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Campbell	13	54,700	38,700	46,800	34,400	3.830	1.940	3.30	0.49	50
Crowder	14	57,700	29,200	35,600	23,500	4.040	2.000	4.35	0.45	68
Fitzsimmons G H	20	15,200	6,600	8,400	5,300	1.060	0.440	1.03	0.09	67
Gt Lakes (Main)	20	21,000	57,800	65,900	45,800	1.470	3.790	6.10	2.31	66
Benj Harrison	20	15,500	12,400	15,700	9,200	1.090	1.170	1.96	0.42	94
Lockbourne AAB	12	2,300	4,120	4,600	3,700	0.161	0.372	0.81	0.18	90
Sheridan	20	13,900	7,100	10,500	3,800	0.973	0.557	0.84	0.40	78
Swift	18	55,900	31,800	41,800	20,800	3.910	1.830	3.66	0.30	58
Turner AAF	15	8,500	3,130	4,000	2,700	0.595	0.374	0.73	0.12	119
Leonard Wood	20	51,400	38,500	48,700	32,300	3.600	1.810	3.45	0.48	47

(a) Design capacities are based chiefly on criteria in OCE Engineering Manual (1943). See text.

however, was applied to the filters, except during heavy rains. It is important to point out therefore, that application of a single design factor of 4,620 persons per acre foot is warranted only because variations in the intensity of loading are brought forth in a succinct manner so as to facilitate comparison.

Excellent sampling procedures were employed in plants of this division. The 11 plants of Table 27 were selected from a much larger number of similarly designed plants, and sampling was a primary consideration in selection. Twenty-four-hour, on the hour, sampling was the rule at most of the plants in the group. An average of 15 composite samples were analyzed each month at each plant.

Average population varied from 3,130 at Turner AAF, to 57,800 at the Great Lakes USNTC. Flows varied from 0.372 mgd to 3.79 mgd in the division. Flow expressed on a per capita basis varied from 47 to 119 gallons daily with an average of 74.

Median values for percent of design capacity utilized, as calculated from Table 30, are as follows:

(i) Primary settling tanks, 60 percent.
(ii) Filter units, 64 percent.
(iii) Secondary settling tanks, 73 percent.
(iv) Sludge digesters, 74 percent.
(v) Sludge beds, 63 percent.

It is to be noted that the use of the median in this connection precludes the

TABLE 30.—Percent of Design Capacity Utilized in Deep Filter Plants Without Recirculation

Post	Primary Settling Tank (%)	Filter (%)	Secondary Settling Tank (%)	Sludge Digester (%)	Sludge Beds (%)
(1)	(2)	(3)	(4)	(5)	(6)
Campbell	50	71	61	77	70
Crowder	67	51	57	60	60
Fitzsimons G H	64	43	64	66	66
Gt Lakes (Main)	97	276	122	117	250(a)
Benj Harrison	92(b)	80	77	89(b)	69
Lockbourne AAB	97(b)	179	72	96(b)	46
Sheridan	25	51	73	47	47
Swift	55	57	64	70	53
Turner AAF	52	37	77	62	49
Leonard Wood	49	75	73	96	79

(a) Tank truck also used for disposal of liquid sludge. (b) Imhoff tank.

TABLE 31.—Design and Description of Filters in Deep Filter Plants Without Recirculation

Post	Number of Filters	Dimensions—Length by Width or Diameter (ft)	Area (acres)	Volume (af)	Filter Media		
					Depth (ft)	Type	Size (in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Campbell	4	1 at 150d 3 at 154d	1.69	11.83	7	Limestone	1½–4
Crowder	5	152d	2.08	12.48	6	Limestone	1½–3
Fitzsimons G H	4	1 at 125d 3 at 70d	0.55	3.30	6	Trap Rock & Granite	1½–3
Gt Lakes (Main)	2	136d	0.668	4.54	6.8	Limestone	2½–4
Benj Harrison	1	148x125	0.42	3.36	8	Limestone	2½
Lockbourne AAB	2	48d	0.083	0.50	6	Slag	2½–3½
Sheridan	1	206x106	0.50	3.00	6	Limestone & Granite	2½–4
Swift	4	155d	1.73	12.11	7	Limestone & Crushed Gravel	3
Turner AAF	2	80d	0.23	1.84	8	Slag	1½–3
Leonard Wood	5	142d	1.82	11.13	6.12	Limestone	2½–4 4–6

TABLE 32.—BOD Loading and Performance of Deep Filter Plants Without Recirculation

Post	Plant Influent			Primary Treatment Section					
	Flow (mgd)	BOD		Detention Period (hr)			BOD		
		PPM	Pounds per Capita Daily	At Av Flow	At Max Flow	At Min Flow	Inf (ppm)	Eff (ppm)	Removal (%)
(1)	(3)	(4)	(5)	(16)	(17)	(18)	(19)	(20)	(21)
Campbell	1.940	427	0.179	5.0	3.0	19.8	427	265	38.0
Crowder	2.000	430	0.244	3.7	1.7	16.5	430	283	34.2
Fitzsimons G H	0.440	390	0.217	3.9	1.7	18.4	390	312	20.0
Gt Lakes (Main)	3.790	390	0.214	2.6	1.6	4.2	390	306	21.6
Benj Harrison	1.170	257	0.201	2.7(a)	1.6(a)	7.5(a)	257(a)	148(a)	42.5(a)
Lockbourne AAB	0.372	285	0.214	2.6(a)	1.2(a)	5.3(a)	285(a)	145(a)	49.1(a)
Sheridan	0.557	291	0.190	9.8	6.5	13.7	291	174	40.2
Swift	1.830	473	0.228	4.6	2.3	28.1	473	234	50.6
Turner AAF	0.374	212	0.211	4.8	2.5	15.5	212	149	29.7
Leonard Wood	1.810	547	0.215	5.1	2.7	19.3	547	277	49.3

(a) Imhoff tank.

TABLE 33.—BOD Loading and Performance of Deep Filter Plants Without Recirculation

Post	Secondary Treatment Section									
	Filter Loading						Filter Only—Performance			
	Flow		Pop'n per Acre-Foot	BOD			BOD			
	MGD	MGAD		Inf (ppm)	LB/AFD	LB/AD	Eff (ppm)	LB/AFD Removed	Efficiency (%)	Removal (%)
(1)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)
Campbell	1.940	1.15	3,270	265	363	2,540	53	290	80.0	49.7
Crowder	2.000	0.96	2,340	283	377	2,260	44	319	84.5	55.6
Fitzsimons G H	0.440	0.80	4,140	312	720	4,330	118	449	62.2	49.7
Gt Lakes (Main)	3.790	5.66	12,730	306	2,130	14,500	—	—	—	—
Benj Harrison	1.170	2.78	3,690	148	429	3,420	61	252	58.8	33.8
Lockbourne AAB	0.372	4.48	8,240	145	903	5,420	61	523	57.9	29.5
Sheridan	0.557	1.11	2,370	174	269	1,610	79	148	54.6	32.7
Swift	1.830	1.06	2,620	234	295	2,060	54	227	76.8	38.0
Turner AAF	0.374	1.63	1,700	149	253	2,020	22	216	85.4	59.9
Leonard Wood	1.810	1.00	3,460	277	377	2,310	71	281	74.5	37.7

distorting influence of high values at Great Lakes and Lockbourne, which are atypical as previously explained. The foregoing summary makes it evident that the plants generally were loaded less than three-quarters of design capacity as given in the Engineering Manual. Such a degree of underloading is to be attributed to a large extent to application of the "capacity factor."

With the exception of the plants at Fort Sheridan and Fort Benjamin Harrison, all filter plants in Division I were equipped with rotary distributors. Largest diameter of filters was

155 ft at Camp Swift. Most filter beds had an average depth of 6 ft. Use of limestone for media was common. Media size ranged from 1½ to 4 inches, with most of the stone ranging from 2 to 3½ inches.

The median concentration of BOD in the plant influent was 390 ppm; on a per capita basis the BOD amounted to 0.211 lb daily.

Average detention periods varied from 2.6 to 9.8 hours. The latter figure, which obtained at Fort Sheridan, was subsequently reduced by installation of

TABLE 34.—BOD Loading and Performance of Deep Filter Plants Without Recirculation

Post	Secondary Treatment Section (Cont) — Filter and Secondary Settling Performance — Detention Period (hr) — Settling Tank — At Av Flow	At Max Flow	At Min Flow	Contact Tank at Av Flow	BOD — Final (ppm)	LB/AFD Removed	Efficiency (%)	Removal (%)	Overall Plant BOD Removal (%)	Final Effluent — Dissolved Oxygen (ppm)	Relative Stability (%)
(1)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(43)	(44)
Campbell	4.1	2.4	16.1	0.54	22	333	91.7	57.0	95.0	7.3	—
Crowder	4.4	2.0	19.7	0.80	29	338	89.7	59.0	93.2	3.0	—
Fitzsimons G H	3.9	1.7	18.4	0.33	68	573	78.2	62.6	82.6	1.5	21
Gt Lakes (Main)	2.0	1.3	3.3	—	89	1,510	71.0	55.6	77.2	—	—
Benj Harrison	3.2	1.9	9.0	0.23	22	365	85.2	49.0	91.5	7.4	—
Lockbourne AAB	3.5	1.6	7.0	—	32	700	77.8	39.6	88.7	2.7	—
Sheridan	3.4	2.3	4.8	1.35	49	194	72.0	43.0	83.2	7.0	46
Swift	3.9	2.0	24.1	—	27	261	88.4	43.7	94.3	1.8	82
Turner AAF	3.2	1.7	10.4	0.55	14	229	91.6	63.7	93.4	3.3	94
Leonard Wood	3.4	1.8	12.9	0.34	28	339	90.0	45.6	94.9	3.0	72

TABLE 35.—SS Loading and Performance of Deep Filter Plants Without Recirculation

Post	Suspended Solids — Plant Influent — PPM	Pounds per Capita Daily	Primary Treatment Section — Effluent (ppm)	Removal (%)	Secondary Treatment Section — Final Eff (ppm)	Efficiency (%)	Removal (%)	Overall Plant Removal (%)
(1)	(2)	(3)	(4)	(5)	(9)	(12)	(13)	(14)
Campbell	374	0.157	145	61.2	27	81.4	31.5	92.7
Crowder	423	0.241	188	55.6	32	82.9	36.9	92.5
Fitzsimons G H	246	0.137	135	45.2	37	72.6	39.8	85.0
Gt Lakes (Main)	—	—	—	—	—	—	—	—
Benj Harrison	233	0.183	131	43.7	34	74.1	41.7	85.4
Lockbourne AAB	208	0.157	96	53.8	23	76.1	35.1	88.9
Sheridan	301	0.198	112	62.8	41	63.4	23.6	86.4
Swift	418	0.201	117	72.0	19	83.8	23.5	95.5
Turner AAF	196	0.195	89	54.6	13	85.4	38.9	93.5
Leonard Wood	523	0.205	157	70.0	22	86.0	25.8	95.8

TABLE 36.—Sludge Data Relating to Deep Filter Plants Without Recirculation

Post	Number Months in Operating Period Studied	Sludge Digester					Schedule of Sludge Loading	
		Total CF per Capita (a)	Heated, Percent of Total Volume	Covered, Percent of Total Volume	Average Operating Temperature (°F)	Source of Sludge	Secondary to Primary Settling Tanks—Times per Day	Primary to Sludge Digesters—Times per Day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Campbell	12	4.2	72	100	90	Pri(b)	3	2
Crowder	20	3.6	66	100	91	Pri(b)	6	12
Fitzsimons G H	20	4.5	100	100	98	Pri(b)	—	96
Gt Lakes (Main)	—	2.8	79	100	98	Pri(b)	Cont	6
Benj Harrison(c)	20	5.1	0	—	—	—	—	—
Lockbourne AAB(c)	12	4.7	0	—	—	—	—	—
Sheridan	20	6.4	100	100	92	Pri(b)	—	2–4
Swift	24	6.4	0	100	—	Pri(b)	—	—
Turner AAF	15	7.2	0	100	82	Pri(b)	4	6
Leonard Wood	20	3.1	100	100	93	Pri(b)	3	8

(a) Based on actual population, Table 29, col (4). (b) Includes scum. (c) Imhoff tanks.

TABLE 37.—Sludge Data Relating to Deep Filter Plants Without Recirculation

Post	Gallons Raw Sludge per MG Sewage	Volatile Solids, Percent of Total Solids		Sludge Disposal			
		Raw Sludge	Digested Sludge	SF per Capita (a)	No. Months Sludge Drawn per Year	Sludge Drawn Yearly from Digesters (cf/cap)	Sludge Removed Yearly from Beds (cf/cap)
(1)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Campbell	3,885	69.2	46.7	1.42	10–12	4.5	0.9
Crowder	4,780	74.0	53.4	1.65	12	7.0	1.7
Fitzsimons G H	5,640	80.3	61.8	1.51	10	6.5	1.4
Gt Lakes (Main)	—	—	—	0.40	—	—	—
Benj Harrison	—	—	52.4	1.45	10–12	6.3	0.9
Lockbourne AAB	—	—	56.9	2.18	9–11	—	—
Sheridan	12,700	70.0	64.0	2.12	10–12	6.3	1.6
Swift	10,400	65.2	46.9	1.90	12	4.6	1.5
Turner AAF	16,840	82.0	46.1	2.04	—	4.3	1.2
Leonard Wood	9,060	71.0	52.4	1.26	12	7.2	1.6

(a) Based on actual population, Table 29, col (4).

recirculation facilities. The median value of BOD removal in primary settling was 40 percent. The median value of concentration of primary effluent was 250 ppm.

In Table 32, it may be noted that columns (4) and (19) are identical. In plants without recirculation, plant influent and primary settling tank influent are identical; this is not the case in plants with recirculation through primary tanks. In order to maintain a similarity of arrangement of tables in the various filter divisions, numbers of columns in corresponding tables are made the same so that comparison is rendered easy.

On Table 33 are presented data indicative of volumetric and BOD loading. Median value of mgad is 1.13; median value of the lb BOD/af is 377; and lb BOD/acre is 2,420. Median concentration of filter effluent was 61 ppm BOD, corresponding to a removal of 44 percent of initial BOD in the filter alone. Percent efficiency * on the average was 70.5 percent. That is, the filters of Division I removed 70.5 percent of the BOD load applied to them.

A comparison of the foregoing performance with that attained in municipal practice is instructive. In an investigation of sewage plant performance by the USPHS in 1923 (Public Health Bulletin 132), the following average results for seven trickling filter plants were reported:

Raw Sewage BOD, 118 ppm
Filter Influent BOD, 95 ppm
Percent BOD Removal, Filter only, 45%
Percent BOD Efficiency, Filter only, 80%.

It is evident that with these relatively dilute municipal sewages the filter removal was about the same as that in military installations. However, the percent efficiency in the municipal filters is significantly higher than in the military. This is in line with the intensity of organic load applied per unit volume in the two groups. The municipal filters received a smaller weight of BOD per acre-foot daily and, accordingly, efficiencies were higher.

Hatch presented data pertaining to loading and performance of eight Ohio trickling filter plants for the period 1927–31 as follows:

Average BOD Settling Tank Effluent, 129 ppm
Average Filter Loading, 188 lb BOD/acre-foot
Percent Efficiency of Filter,* 76.5 percent.

A lower load intensity and correspondingly higher efficiency in these municipal plants again is exhibited as compared to the trickling filters at military installations.

In Table 34 the additional removal of BOD by secondary settling is indicated. Median value of detention period in secondary tanks was 3.4 hrs. Average percent removal of initial BOD in raw sewage that occurred in

* For definitions of percent removal and efficiency see Chapter I.

* Not including removal in secondary settling tanks.

TABLE 38.—Chlorination and Effluent Disposal in Deep Filter Plants Without Recirculation

Post	Chlorination		Receiving Water Body	
	Points of Application Used	Residual Chlorine (ppm)	Name	Type
(1)	(2)	(3)	(4)	(5)
Campbell	Post	0.3	Little West Fork Creek	Flowing Stream
Crowder	Post	0.2	Buffalo Creek	Dry Run (2 Mi) to Flowing Stream
Fitzsimons G H	Pre & Post	0.2	Toll Gate Creek	Flowing Stream
Gt Lakes (Main)	Pre & Post	—	Lake Michigan	Lake
Benj Harrison	Post	1.0	Fall Creek	Flowing Stream
Lockbourne AAB	Post	—	Little Walnut Creek	Flowing Stream
Sheridan	Post	0.3	Lake Michigan	Lake
Swift	None	—	Piney Creek	Flowing Stream
Turner AAF	None	—	Flint River	Flowing Stream
Leonard Wood	Post	0.2	Big Piney River	Dry Run (4½ Mi) to Flowing Stream

filter and secondary settling was 51.9 percent. This figure includes removal due to oxidation and settling in chlorine contact chambers, if any. Generally, however, removal in chlorine contact chambers was negligible. Use of chlorine had in general been minimized at the time of investigation by orders emanating from Service Command and Navy Superintending Civil Engineer Offices. Consequently, the difference in median removals of columns (39) and (31) ($51.9 - 44.0 = 7.9\%$) is attributed to secondary tanks.

The average value of percent efficiency of the secondary treatment section (column 38) was 83.6 percent. The average concentration of final effluent of plants, Division I, was 38 ppm. If the results for Lockbourne and Great Lakes are excluded, the average concentration in plant effluent was 32 ppm, corresponding to a secondary section removal of 336 lb/af. Average overall plant removal of BOD was 89.4 percent. Omitting results for Lockbourne and Great Lakes, the average overall plant BOD removal for plants in Division I was 91.4 percent, a value that compares favorably with the best municipal trickling filter performance. Average performance of plants in Division I, with respect to removal of SS, is summarized in the following:

Raw Sewage, median SS, 337 ppm
Raw Sewage, median SS, 0.196 lb/cap/day
Primary Section Effluent, median SS, 140 ppm
Primary Section, median removal, 58.4%
Plant Effluent, mean SS, 28 ppm
Secondary Section, mean SS efficiency, 78.4%
Secondary Section, mean SS removal, 33.0%
Overall Plant, mean SS removal, 90.6%

It is evident that removal of SS in filter plants without recirculation is high and in line with BOD performance. It is pertinent to note that the average suspended solids loading of plants of Division I, as expressed upon a per capita basis, was somewhat higher than usual in military plants (Table 35).

Data relating to sludge digestion and disposal are given in Tables 36 and 37. Additional sludge data for plants at Camps Campbell and Crowder and Fitzsimmons General Hospital are analyzed in detail in a subsequent section on sludge digestion.

Performance of Individual Plants of Division I

Camp Campbell, Ky.

At Camp Campbell, the operating period investigated extended from August, 1943 through August, 1944. An additional filter and digester were completed in July, 1943 to provide for an expected increase in post population. These units resulted in improved performance; overall plant removal of BOD during March, April and May, 1943, was 89.1 percent, whereas during the corresponding period in 1944 a 95.0 percent overall BOD removal occurred. Grease was excessive in the plant influent but was largely removed in the primary tanks; scum removed from these units amounted to about 250 gpd. Laundry flow constituted as high as 10 percent of the total flow. The sewer system consisted of 190,500 ft of sewers of diameters of 6 to 30 inches; house laterals totaled 114,100 ft of 4- to 6-inch pipe. The final effluent at Campbell passed over a step aerator made up of a series of five falls; channel width was 8 ft. The receiving stream afforded little dilution, however, the degree of treatment attained was sufficient to permit of bridge-building training activity downstream from the plant.

As may be seen from Fig. 75, the variation of population and flow at Camp Campbell was of the order of

10 to 20 percent during the period investigated. Performance was uniformly high; BOD removal varied from 90.5 percent to 97.5 percent, and SS from 84.9 percent to 96.5 percent. A relatively heavy BOD load during October and November, 1943 was handled without impairment of final effluent. Effluent dissolved oxygen during the summer months was about 6.5 ppm and during winter averaged about 8.5 ppm. No particular difficulties with sludge digestion or supernatant disposal occurred.

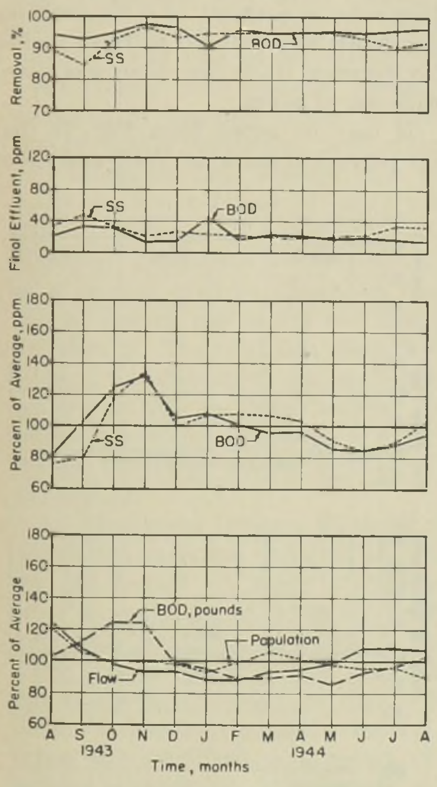


FIGURE 75.—Loading and performance of trickling filters at Camp Campbell.

Camp Crowder, Mo.

The low rate filter at Camp Crowder as originally constructed was placed in operation in May, 1942. In July, 1943 an extension to the plant was installed, consisting of an additional filter, a secondary settling tank and a storage digester. A chlorine contact tank was placed in operation in August, 1944; a grease interceptor utilizing diffused air was installed in December, 1943.

Monthly averages of ether-soluble material at different points throughout the plant during 1944 are indicated in the following table:

Month	Raw (ppm)	Primary Effluent (ppm)	Filter Effluent (ppm)	Final Effluent (ppm)
Jan	1,136	623	243	123
Feb	608	439	209	109
Mar	605	354	210	127
Apr	700	450	154	122
May	1,040	594	332	251
Jun	900	—	—	—
Jul	620	331	59	48
Aug	583	278	103	52

As may be seen, the grease content was substantial; this was reflected in high per capita production of BOD (Table 32).

Considerable improvement in the skimming of indoor grease traps was accomplished through an educational program started in June, 1944. The grease intercepter at the plant brought about better grease removal, but due to the turbulence engendered by diffused air, only a small percentage of grease was recovered at the intercepter; the bulk appeared on the settling tanks. Laundry wastes constituted about 7 percent of the flow.

The chief operating difficulty stemmed from faulty performance of the sludge removal mechanism in one of the primary settling tanks. This unit failed a number of times, particularly in cold weather, necessitating removal of the tank from service for several days. Many minor improvements were made by the operator that contributed to efficiency. A list of these is of interest as being indicative of the type of modification found to be useful by an enterprising operator of a World War II sewage treatment plant—one that was generally conceded to be originally well designed and constructed:

(a) Installation of a sludge cleanout connection in sludge piping.
(b) Construction of a crane for handling grease from the intercepter.
(c) Installation of drain lines from the drip traps.
(d) Construction of chemical feed tank for applying alum and borax to the sludge.
(e) Installation of an extra waste gas burner.
(f) Decking over all open flumes.
(g) Installation of hot water tank for cleaning and washing of units and equipment.
(h) Construction of wooden sluice gates at dosing chambers.
(i) Addition of handrails and concrete walks around units for safety.
(j) Provision for stream of sewage to impinge upon edge of filter bed in order to wash *Psychoda* larvae down wall.
(k) Piping of hot water to laboratory.
(l) Addition of cleanout hand-hole on suction side of sewage pumps at lift station.

Media size in the filters at Camp Crowder was 1½ to 3 inches, which was the smallest recorded in low rate filters. Filters using stone of this size —Crowder, Fitzsimons and Turner AAF—gave an average removal of 55 percent (not including that in secondary settling), which was significantly higher than the overall average filter removal of 44 percent for the division.

The filters at Crowder were equipped with dosing tanks. The cycle consisted of 5- to 8-minute dosages with 8- to 11-minute rest intervals. Filters were not by-passed during peak flows, but occasionally during extremely heavy rain the entire plant was by-passed.

Digestion of sludge interposed no serious operating difficulties. Scum was not a problem; grease was troublesome only in raw sludge piping. Supernatant pH was 7.2, and solids ranged from 0.54 to 1.13 percent, of which the volatile content varied from 60 to 79 percent. The average amount of supernatant was about 160,000 gal per month. Although the plant was designed for supernatant return to plant influent, most supernatant was actually discharged to sludge beds.

Sludge from digesters was originally applied to beds to a depth of six inches. However, as a result of a fly problem, the depth was reduced to three inches. No decrease in drying time was noted with use of alum.

It may be noted from Fig. 76 that the percent removal of BOD and SS at

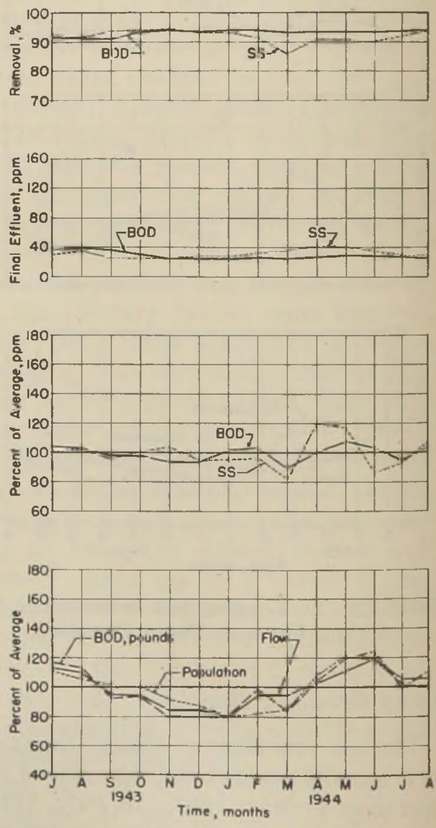


FIGURE 76.—Loading and performance of trickling filters at Camp Crowder.

Camp Crowder was not appreciably affected by changes in organic and volumetric loading of the order of 20 percent, the average for the period of study. No significant seasonal change in performance was evident.

Fitzsimons General Hospital, Colo.

The data for Fitzsimons are of considerable interest since they pertain to sewage treatment at a general hospital. The chief differences between military hospitals and other posts of comparable size were as follows: (i) composition of sewage; (ii) areal concentration of population, affecting sewage condition and rate of arrival at treatment plants. The inherent nature of activities at hospitals led to introduction of certain substances, both organic and inorganic, into sewage that normally were not found at other posts (blood plasma, disinfectants, etc.). Moreover, population was concentrated in a smaller area at hospitals; lineal feet of sewers per capita was usually less than 2. The relatively short sewer system, together with the regimented pattern of life in these hospitals, resulted in extreme variations of flow at sewage treatment plants (Figs. 21, 32, 33, and 51).

At Fitzsimons General Hospital the population variation was gradual; during the period of study it changed from a minimum of 5,300 in March, 1944 to a maximum of 8,230 in May, 1943. The sewage was fresh and warm due to the short length of run to the plant. Grease was normal; removal at traps was adequate as a result of a regular schedule of maintenance.

Four rotary filters were installed at Fitzsimons (Table 31). Only the three small filters were used in normal operation during the period of investigation. The large filter was installed to handle expected additional population and laundry waste. As may be seen from Table 33, the organic load was higher than normal for low rate filters and performance was about in line with loading.

All plant effluent flowed through a reservoir of approximately 1 mg capacity. Water from the reservoir was used for irrigation as needed. The reservoir was not considered a plant unit (Table 34). The storage provided did not contribute materially to removal of organic material; in fact, some samples indicated an increase in BOD in passage through the reservoir. Increase in BOD in storage reservoirs and ponds was not an uncommon occurrence in military posts where such units existed. These anomalous observations are to be attributed to fixation of organic matter by photosynthetic processes and to irregular release of anaerobic decomposition products from benthal deposits.

Great Lakes Naval Training Station, Ill.

The main plant at the Great Lakes Naval Training Center affords an interesting example of performance of a heavily loaded filter that was originally designed for a low rate of load application. As explained previously, the primary and secondary sections of this plant were designed for different populations; in actual operation, however, all sewage was routed through the filters. These units had fairly coarse media and were able to handle the load with fair efficiency. High degree of treatment was not considered necessary in view of dilution afforded the effluent by Lake Michigan.

Primary settling performance was substandard; three settling tanks were rebuilt in order to improve settling and to eliminate conditions conducive to septicity. Preaeration equipment at the primary settling tank influent was installed subsequent to the period of operation investigated.

Thin sludge together with relatively limited digestion and sludge bed capacity made it necessary at Great

Lakes to make provision for hauling liquid digested sludge from the plant. This was accomplished with a 1,200-gal tank truck that was capable of hauling the equivalent of one bed of sludge in two days. Much of the liquid sludge carried by truck was deposited upon low-lying waste land in the vicinity of the post; some was applied to lawns. All areas dosed were heavily grassed and no dust nuisance occurred. Odor did not constitute a problem.

Fort Benjamin Harrison, Ind.

Operation at Fort Benjamin Harrison was normal; sewage did not contain excessive amounts of grease and laundry waste constituted only 5 per cent of the flow. Infiltration, however, was excessive and caused the concentration of the raw sewage to be considerably less than normal. This did not have an adverse effect upon settling. Fixed nozzles distributed sewage upon filters from twin dosing chambers with an average dosing cycle of about ten minutes. During heavy precipitation, flow had to be by-passed around the filters. The filters unloaded twice each year, usually in March and November, without introducing difficulty in operation. A grease skimming tank employing diffused air was installed, but the unit was not particularly effective in removal of grease, and, moreover, permitted of disposition of organic solids with grit.

Lockbourne AAB, Ohio

The treatment plant at Lockbourne AAB, as previously explained, was designed for a population loading of 12,000 persons per acre-foot. Sewer gradients were flat and infiltration was large. Grease content was normal; traps in all mess halls were cleaned at least twice a week. A grease flotation unit installed ahead of primary settling tanks operated successfully, and was in part responsible for notable BOD removal in primary settling in the Imhoff tanks. Filters were dosed with sewage from motor-driven rotary distributor arms. Difficulty due to breakage of a distributor arm drive resulted in one of the filters being out of service during the winter of 1943–44, during which period performance dropped considerably. This period is omitted in the analysis (Tables 33, 34, and 35).

Fort Sheridan, Ill.

At Fort Sheridan infiltration and grease were not excessive; no laundry waste entered sewers except a small amount from officers' quarters. As previously stated, plans were in effect at the time of survey for installation of recirculation facilities that would reduce detention periods in settling tanks. Poor workmanship was the cause of considerable leakage in an inverted siphon connecting the Parshall flume and filter dosing chambers. This structure was subsequently replaced with an open channel of reinforced concrete.

An unusually low efficiency in filtration occurred (Tables 33 and 34). It is believed that this is to be attributed to the following factors: (i) excessive primary detention with septic filter influent; (ii) clogging of media—pooling occurred at various times and control measures such as resting and chlorination were instituted; and (iii) high application rates as a result of intermittent dosing. With discharge through nozzles taking place for only 4.5 minutes out of 25, a rate of about 9 mgad occurred during the peak flow period—a value much larger than that in other plants of the division.

Wet sludge was disposed of by tank truck at Fort Sheridan, as well as upon drying beds.

Camp Swift, Texas

The sewage plant at Camp Swift is an example of a large low rate trickling filter plant operating under typical military conditions and giving excellent performance. Raw sewage was concentrated and, rather large population fluctuations occurred during the period of study; despite these circumstances the plant effluent most of the time had a BOD of less than 35 ppm. During one-half the months studied, the plant removed more than 95 percent of the raw sewage BOD. Suspended solids removal was equally effective. Unloading periods were not discernible.

Prolonged resting periods were possible through rotating the use of three of four available filters. Some grit trouble in the secondary sludge return line was caused by filter rock debris.

No unusual difficulties were encountered in connection with sludge digestion and disposal. However, additional sludge beds would have eased operation during the rainy season.

Turner AAF, Ga.

The plant at Turner AAF afforded a good example of performance of a relatively small standard rate filter plant. As may be seen from Table 30, the plant was loaded to less than 50 percent capacity insofar as filter units were concerned.

Grease was excessive in the raw sewage, but was negligible in the primary settling tank effluent. A grease trap was installed ahead of the grit chamber. No laundry waste was included in the sewage.

A moderate amount of pooling upon the filters was controlled by gouging the pooled area with metal rods. Unloading occurred in fall and spring. A notable removal of BOD was attained by the filters (column 31, Table 33). Media size was relatively small.

Sludge digestion and disposal units operated in a satisfactory manner despite the fact that an unusually thin raw sludge was produced.

As may be seen from Fig. 77, a substantial variation in loading occurred during the operating period investigated. The average daily weight of BOD treated during July, August, and September, 1943 was 540 lb, while during this same period in 1944 it was 942 lb. Overall plant removal of BOD for the summer months in 1943 was 93.9 percent; and with the marked increase in load during the summer of 1944 removal dropped to only 89.6 percent. This is indicative of the ability of the trickling filter to withstand load variation without serious deterioration of performance.

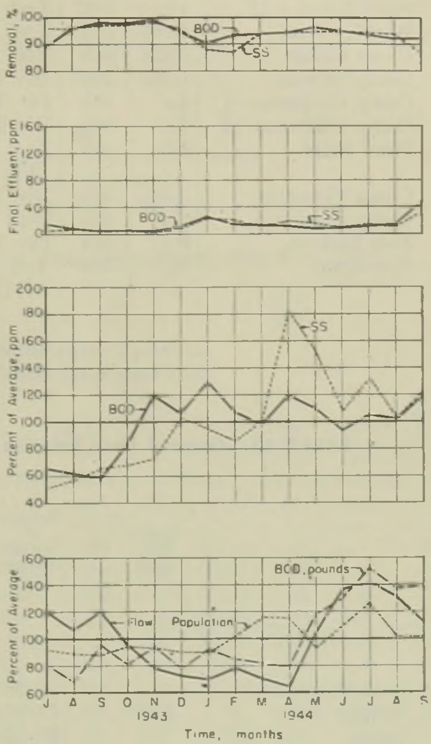


FIGURE 77.—Loading and performance of trickling filters at Turner AAF.

Fort Leonard Wood, Mo.

No unusual operating problems were encountered at Fort Leonard Wood. The plant did not have a comminutor and some difficulty was experienced with clogging of filter nozzles. The plant influent was unusually concentrated; BOD in the raw sewage was 547 ppm. Grease was not excessive; laundry waste constituted 9 percent of the flow.

The four original filter distributors were remodeled by changing the braces and adding wheels to the end of each distributor arm that ran along the filter bed walls.

A thick layer of scum cut down capacity in the digesters. Supernatant from the primary digester contained 2,500 to 7,000 ppm of suspended solids and that from the secondary digester

TABLE 39.—List of Deep Filter Plants With Recirculation

Post	State	Sv C or Navy Area	Type of Station	Date Plant Completed	Date Major Additions or Alterations Completed	Operating Period Studied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Boca Raton AAF	Fla	4th	Tech Trg Sch	Apr 43	—	Aug 43 to Jul 44
Buckley Field	Colo	7th	Tech Trg Sch	Dec 42	—	Apr 43 to Aug 44
Camp Callan	Calif	9th	Repl Trg	Apr 41	Jun 44	Jan 43 to Dec 43
Camp Carson	Colo	7th	Div Trg	Jun 42	Jun 43	Aug 43 to Aug 44
Camp Claiborne #1	La	8th	Div Trg	Mar 41	Apr 43	May 43 to Aug 44
Camp Claiborne #2	La	8th	Div Trg	Apr 43	—	Apr 43 to Aug 44
Drew Field	Fla	4th	Tact Fields & Fly Trg	Feb 43	Nov 43	Sept 43 to Aug 44
Camp Forrest(a)	Tenn	4th	Div Trg	Oct 42	Sept 43	Jan 43 to Aug 43
Camp Gordon(b)	Ga	4th	Repl Trg	Jan 42	Jun 43	Jan 43 to Dec 43
Fort Jackson	S C	4th	Div Trg	Aug 42	Sept 43	Apr 43 to Aug 44
Fort Knox	Ky	5th	Arm M C & T D Trg	Apr 42	—	Jan 43 to May 44

(a) Single stage filtration prior to August, 1943.
(b) Single stage filtration prior to December, 1943.

TABLE 40.—General Data Relating to Deep Filter Plants With Recirculation

Post	Approx Plant Elev (Ft)	Range of Sewage Influent Temp (°F)	Average Sewage Influent pH	Lineal Feet of Sewers per Capita (a)	Sampling	
					Hours per Day	Average Number per Month
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Boca Raton AAF	—	74–88	7.4	4.1	24	10
Buckley Field	5,480	74–92	8.6	2.9	24	25
Callan	—	66–83	7.4	11.0	24	10
Carson	—	63–77	7.2	4.1	24	12
Claiborne #1	105	70–96	7.4	3.9	24	9
Claiborne #2	90	61–84	7.4	5.8	24	8
Drew Field	16	72–101	7.4	—	16	25
Forrest	—	50–86	7.2	5.4	24	30
Gordon	—	56–86	7.1	7.1	24	30
Jackson	160	67–91	7.0	9.6	24	30
Knox	640	52–80	7.3	4.4	24	25

(a) Based on design population, Table 42, col (3).

TABLE 41.—Units in Deep Filter Plants With Recirculation

Post	Plant Units											
	Screens or Racks	Comminutors	Grit Chambers	Imhoff Tanks	Primary Settling Tanks	Filters	Secondary Settling Tanks	Chlorine Contact Tanks	Digestion Tanks—Heated	Digestion Tanks—Unheated	Sludge Beds	Other Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Boca Raton AAF	1	0	0	0	2	2	2	1	0	2	10	
Buckley Field	1	1	1	0	1	2	1	1	1	0	8	
Callan	1	0	0	0	2	1	1	1	2	0	5	
Carson	1	0	1	0	3	4	3	0	2	1	6	
Claiborne #1	0	2	0	0	2	1	2	1	2	1	18	(b)
Claiborne #2	1	0	0	0	2	2	2	1	2	0	13	(b)
Drew Field	2	4	0	3(c)	3	2	3	1	2(d)	0	16	
Forrest	1	0	0	0	3	3	4	1	2	1	18	
Gordon	2	1	1	0	2	4	2	1	2	1	5	
Jackson	2	0	2	0	2	4	3	0	2	2	16	
Knox	1	0	0	0	3	1	2	1	2	0	0	(e) (f)

(b) Grease flotation tanks.
(c) Imhoff tanks not used.
(d) Two-story digestion tanks, upper story only heated.
(e) Lagoons; 4 in number; 1.44 mcf, 10 ft deep.
(f) Step aerator on discharge and 150-ft splash falls.

TABLE 42.—Design Capacities and Loading of Deep Filter Plants With Recirculation

Post	Number of Months in Operating Period Studied	Population				Flow				
		Design Capacity (a)	Actual			Design (mgd) (a)	Actual			
			Average	Maximum	Minimum		Average (mgd)	Maximum (mgd)	Minimum (mgd)	Gal per Capita Daily
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Boca Raton AAF	12	21,800	9,000	10,700	7,600	1.53	0.510	2.08	—	57
Buckley Field	17	22,200	11,600	15,700	7,400	1.55	0.907	1.36	0.35	78
Callan	12	7,600	10,100	10,800	9,000	0.53	0.753	1.33	0.29	75
Carson	13	64,200	26,400	34,200	14,300	4.50	1.540	3.28	0.61	58
Claiborne #1	16	70,000	25,000	31,000	10,500	4.90	2.130	3.60	0.65	85
Claiborne #2	17	31,000	21,800	33,800	14,900	2.17	0.986	1.70	0.44	45
Drew Field	12	19,800	18,700	25,700	14,000	1.39	1.360	2.48	0.40	73
Forrest	8	39,800	32,100	41,100	22,700	2.79	2.150	4.83	0.83	67
Gordon	12	33,800	32,100	43,000	21,700	2.37	1.760	3.88	0.46	55
Jackson	17	47,100	47,000	58,200	30,900	3.30	3.220	5.96	1.00	68
Knox	17	86,400	52,200	58,100	47,200	6.05	3.200	6.69	1.57	61

(a) Design capacities are based chiefly on criteria in OCE Engineering Manual (1943). See text.

varied from 250 to 2,000 ppm. Supernatant from both digesters was returned to the primary settling tank.

Subsequent to the period of investigation, recirculation facilities (final settling tank to plant and/or filter influent) were installed to reduce long detention periods in the settling tanks.

II. Deep Filters with Recirculation

An analysis of performance of sewage treatment plants with deep filters and utilizing recirculation is presented on Tables 39 to 52. Of the eleven posts represented, seven were located in the South and the remainder were in the central part of the country. All plants were at Army posts. Three of the plants—Buckley Field, Camps Claiborne #2 and Gordon—were loaded in the normal range of standard rate filter plants. Other plants had higher load concentrations. A number of these were loaded in the so-called "intermediate zone" between high rate and standard rate loading.

Comparison of filter size relative to loading is facilitated by application of a single design factor of 23,100 persons per acre-foot. This factor is five times larger than that used in plants of Division I. Actual loading, population and volumetric, is compared with design population in Tables 42 and 43.

As based upon actual population, post sizes varied from 9,000 at Boca Raton AAF to 52,200 at Fort Knox; corresponding flow ranged from 0.51

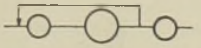
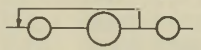
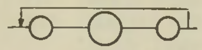
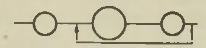
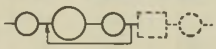
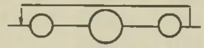
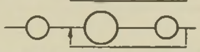
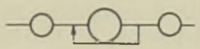
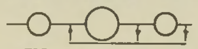
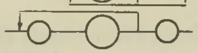
TABLE 43.—Percent of Design Capacity Utilized in Deep Filter Plants With Recirculation

Post	Primary Settling Tank (%)	Filter (%)	Secondary Settling Tank (%)	Sludge Digester (%)	Sludge Beds (%)
(1)	(2)	(3)	(4)	(5)	(6)
Boca Raton AAF	116	41	25	47	51
Buckley Field	60	52	39	73	39
Callan	165	134	180	48	32
Carson	36	41	75	54	49
Claiborne #1	82	36	1,710	36	42
Claiborne #2	64	71	64	68	51
Drew Field	97	94	149	95	38
Forrest	66	81	169	88	74
Gordon	59	95	59	71	64
Jackson	70	99	—	94	105
Knox	74	60	46	99	—

TABLE 44.—Design and Description of Filters in Deep Filter Plants With Recirculation

Post	Number of Filters	Dimensions—Length by Width or Diameter (ft)	Area (acres)	Volume (af)	Filter Media		
					Depth (ft)	Type	Size (in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Boca Raton AAF	2	66d	0.157	0.942	6	Slag	Max of 4
Buckley Field	2	145d	0.758	4.80	6.33	Gravel	1½–2½
Callan	1	55d	0.054	0.327	6	Granite	2–3
Carson	4	73d	0.384	2.78	7.25	Granite	2–4
Claiborne #1	1	155d	0.433	3.03	7	Granite	3–4
Claiborne #2	2	155d	0.866	6.64	7.67	Granite	1½–3
Drew Field	2	63d	0.143	0.858	6	—	2–4
Forrest	3	70d	0.265	1.66	6.25	—	2½–4
Gordon	4	130d	1.22	7.32	6	Granite	2½–4
Jackson	4	75d	0.407	2.04	5	Granite	2–3
Knox	1	209 x 134	0.642	3.74	5.83	Limestone	1½–4½

TABLE 45.—BOD Loading and Performance of Deep Filter Plants With Recirculation

Post	Diagram	Plant Influent			Recirculation		
		Flow (mgd)	BOD		R_1 (mgd) (a)	R_2 (mgd) (b)	Ratio Total R to Plant Inf
			PPM	Pounds per Capita Daily			
(1)	(2)	(3)	(4)	(5)	(10)	(12)	(14)
Boca Raton AAF		0.510	217	0.103	1.830	0	3.58
Buckley Field		0.907	325	0.213	0.464	0	0.51
Callan		0.753	369	0.230	0.636	0	0.85
Carson		1.540	368	0.180	0	2.387	1.55
Claiborne #1		2.134	368	0.262	0	0.516	0.24
Claiborne #2		0.986	456	0.172	0.329	0	0.33
Drew Field		1.358	302	0.186	0.216	1.080	0.95
Forrest		2.149	321	0.180	0	3.690	1.72
Gordon		1.760	450	0.206	2.880(c)	—(c)	1.64(c)
Jackson		3.217	319	0.183	0	4.135	1.28
Knox		3.202	332	0.170	2.457	0	0.77

(a) R_1 = Recirculation to primary treatment section.
(b) R_2 = Recirculation to secondary treatment section.
(c) Recirculation filter effluent to filter influent.

TABLE 46.—BOD Loading and Performance of Deep Filter Plants With Recirculation

Post	Primary Treatment Section						
	Flow (mgd)	Detention Period (hr)			BOD		
		At Average Flow	At Maximum Flow	At Minimum Flow	Influent (ppm)	Effluent (ppm)	Removal (b) (%)
(1)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
Boca Raton AAF	2.340	1.1	0.64	—	69	—	—
Buckley Field	1.371	4.2	3.1	7.0	233	168	25.7
Callan	1.389	1.5	1.1	2.3	235	197	18.8
Carson	1.540	4.7	2.2	11.7	368	255	30.8
Claiborne #1	2.134	3.1	1.8	10.0	368	211	42.8
Claiborne #2	1.317	3.9	2.5	6.7	355	156	56.8
Drew Field	1.574	2.6	1.5	6.7	265	132	51.0
Forrest	2.149	3.8	1.7	9.8	321	234	27.1
Gordon	1.764	4.2(a)	1.9	16.2	450	275	38.8
Jackson	3.217	3.6	1.9	11.5	319	218	31.7
Knox	5.659	2.2	1.4	3.1	217	134	34.0

(a) Based upon both primary settling tanks being in service.
(b) Calculated in accordance with Equation (3e), Chapter I.

TABLE 47.—BOD Loading and Performance of Deep Filter Plants With Recirculation

Post	Secondary Treatment Section									
	Filter Loading						Filter Only—Performance			
	Flow to Filter (a)		Pop'n per Acre-Foot	BOD			BOD			
	MGD	MGAD		Inf (ppm) (b)	LB/AFD (c)	LB/AD (c)	Eff (ppm)	LB/AFD Removed	Efficiency (%) (c)	Removal (%)
(1)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)
Boca Raton AAF	2.340	14.90	9,550	68	1,170	7,000	27	852	—	—
Buckley Field	1.371	1.809	2,418	168	382	2,410	52	276	72.4	53.7
Callan	1.389	25.70	30,900	197	5,750	34,900	118	2,820	48.8	39.6
Carson	3.927	10.23	9,500	126	1,180	8,520	—	—	—	—
Claiborne #1	2.650	6.13	8,240	183	1,240	8,660	—	—	—	—
Claiborne #2	1.317	1.521	3,285	156	245	1,880	56	164	67.4	29.1
Drew Field	2.654	18.55	21,800	93	1,950	11,700	48	1,148	58.7	28.8
Forrest	5.839	22.10	18,650	129	2,530	15,900	91	1,110	43.9	32.0
Gordon	4.64	3.81	4,390	—	555	3,330	109	337	60.7	37.1
Jackson	7.349	18.10	23,000	—	2,870	14,400	—	—	—	—
Knox	5.659	8.82	13,960	134	1,560	9,120	67	845	54.0	35.7

(a) Includes recirculation.
(b) Weighted average including BOD of recirculated flow.
(c) Calculated in accordance with Equation (3), Chapter I.

TABLE 48.—BOD Loading and Performance of Deep Filter Plants With Recirculation

Post	Secondary Treatment Section (Cont)								Over-all Plant BOD Removal (%)	Final Effluent	
	Filter and Secondary Settling Performance (c)										
	Detention Period (hr)				BOD						
	Settling Tank			Contact Tank at Av Flow	Final (ppm)	LB/AFD Removed	Efficiency (%)	Removal (%)		Dissolved Oxygen (ppm)	Relative Stability (%)
	At Av Flow	At Max Flow	At Min Flow								
(1)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(43)	(44)
Boca Raton AAF	9.9	2.4	—	—	16	1,095	—	—	92.8	3.7	—
Buckley Field	6.3	4.2	16.2	—	23	346	90.4	67.1	92.8	4.7	—
Callan	1.4	1.0	2.0	0.36	77	4,280	74.5	60.5	79.3	1.0	23
Carson	3.3	2.3	4.4	—	45	972	82.3	57.0	87.8	3.9	53
Claiborne #1	0.15(a)	0.09(a)	0.33(a)	—	71	823	66.3	38.0	80.8	2.7	—
Claiborne #2	3.9	2.5	6.7	0.92	31	206	84.2	36.5	93.3	4.6	—
Drew Field	1.6	1.2	2.8	0.43	34	1,505	77.0	37.7	88.7	2.8	30
Forrest	1.5	1.0	1.9	0.36	70	1,780	70.2	51.2	78.3	3.2	—
Gordon	4.2	1.9	16.2	0.30	70	413	74.6	45.7	84.5	4.7	42
Jackson	2.8(b)	—(b)	—(b)	—(b)	73	1,910	66.3	45.5	77.2	2.1	—
Knox	5.4	2.6	11.0	0.43	24	1,390	89.2	58.7	92.7	7.1	82

(a) Small intermediate settling tank.
(b) Overall average; variable number of tanks in service.
(c) Including contact tank, if any.

TABLE 49.—SS Loading and Performance of Deep Filter Plants With Recirculation

Post	Suspended Solids							
	Plant Influent		Primary Treatment Section		Secondary Treatment Section			Overall Plant Removal (%)
	PPM	Pounds per Capita Daily	Effluent (ppm)	Removal (%)	Final Effluent (ppm)	Efficiency (%)	Removal (%)	
(1)	(2)	(3)	(4)	(5)	(9)	(12)	(13)	(14)
Boca Raton AAF	209	0.100	74	—	16	—	—	92.5
Buckley Field	220	0.144	62	61.1	14	83.6	32.5	93.6
Callan	371	0.250	217	9.6	83	74.7	67.8	77.4
Carson	296	0.144	121	59.2	33	72.6	29.6	88.8
Claiborne #1	276	0.197	112	59.5	74	34.3	13.9	73.4
Claiborne #2	329	0.124	110	56.7	30	78.9	34.2	90.9
Drew Field	302	0.186	117	56.2	35	73.3	32.2	88.4
Forrest	285	0.160	129	54.8	50	61.0	27.5	82.3
Gordon	365	0.167	134	63.3	34	74.6	27.4	90.7
Jackson	260	0.150	112	57.0	44	62.8	27.1	84.0
Knox	310	0.159	107	46.3	29	82.7	44.4	90.7

TABLE 50.—Sludge Data Relating to Deep Filter Plants With Recirculation

Post	Number Months in Operating Period Studied	Sludge Digester					Schedule of Sludge Loading	
		Total CF per Capita (a)	Heated, Percent of Total Volume	Covered, Percent of Total Volume	Average Operating Temperature (°F)	Source of Sludge	Secondary to Primary Settling Tanks—Times per Day	Primary to Sludge Digesters—Times per Day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Boca Raton AAF	12	9.5	0	100	87	Pri(b)	6	12
Buckley Field	17	4.1	100	100	92	Pri(b)	24	48
Callan	12	6.2	100	100	90	Pri(b)	Cont	6
Carson	13	6.3	61	—	82–90	Pri(b)	—	—
Claiborne #1	16	9.0	79	100	88	(b) (c)	—	24
Claiborne #2	17	4.4	100	100	90	Pri(b)	8	12
Drew Field	12	3.8	50(d)	100	88	Pri(b)	Cont	2
Forrest	8	4.0	56	—	70–92	Pri(b)	3	6
Gordon	20	4.8	67	100	90	Pri(b)	8	8
Jackson	17	3.9	47	100	85	Pri(b)	12	12
Knox	17	3.0	100	100	97	Pit(e)	Cont	4

(a) Based on actual population, Table 42, col (4).
(b) Includes scum.
(c) Sludge to digestion tanks from primary and secondary settling tanks and from wasted activated sludge tank.
(d) Two-story tanks.
(e) Sludge from primary settling tanks was drawn to sludge pit for liming, observation and sampling. (Scum was also drawn to sludge pit.) Sludge and scum from sludge pit was pumped to digestion tanks *four* times per day.

TABLE 51.—Sludge Data Relating to Deep Filter Plants With Recirculation

Post	Gallons Raw Sludge per MG Sewage	Volatile Solids, Percent of Total Solids		Sludge Disposal			
		Raw Sludge	Digested Sludge	SF per Capita (a)	No. Months Sludge Drawn per Year	Sludge Drawn Yearly from Digesters (cf/cap)	Sludge Removed Yearly from Beds (cf/cap)
(1)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Boca Raton AAF	15,100	72.0	—	1.96	—	—	—
Buckley Field	7,830	73.8	48.0	2.58	10	4.1	0.4
Callan	10,250	78.3	70.0	3.16	—	—	—
Carson	7,180	79.5	54.4	2.05	10	3.3	0.6
Claiborne #1	2,120	70.0	47.0	2.37	12	9.1	1.3
Claiborne #2	16,580	60.0	—	1.96	—	—	—
Drew Field	6,870	66.4	—	2.61	—	—	—
Forrest	6,350	67.5	50.8	1.35	11	5.3	1.8
Gordon	13,400	79.6	63.5	1.56	12	6.9	1.6
Jackson	16,200	78.6	53.5	0.95	11	—	—
Knox	6,760	69.9	48.0	(b)	—	8.1	—

(a) Based on actual population, Table 42, col (4).
(b) Lagoons only—4 in number; 4 acres total surface; 1.44 mcf volume. Supernatant liquor discharged to lagoon.

TABLE 52.—Chlorination and Effluent Disposal in Deep Filter Plants With Recirculation

Post	Chlorination		Receiving Water Body	
	Points of Application Used	Residual Chlorine (ppm)	Name	Type
(1)	(2)	(3)	(4)	(5)
Boca Raton AAF	—	—	—	Drainage Canal (¼ Mi) to Tide Water
Buckley Field	Post	0.2	Sand Creek	Flowing Stream Dry at Times
Callan	Post	0.5	Creek in Sorrento—Soledad Valley	Dry Run (¾ Mi) to Flowing Stream
Carson	Post	0.14	Fountain River	Flowing Stream (1½ Mi) to River
Claiborne #1	—	—	Barber Creek	Dry Run
Claiborne #2	—	—	Barber Creek	Dry Run
Drew Field	—	—	Tampa Bay	Tide Water
Forrest	Post	0.3	—	Dry Run
Gordon	—	—	Spirit Creek	Flowing Stream
Jackson	—	—	Gills Creek	Flowing Stream
Knox	Post	0.3	Mill Creek	Flowing Stream

mgd to 3.20 mgd. Average per capita sewage flow was 66 gallons. Median values of percent of design utilized (Table 43) may be summarized as follows:

(i) Primary settling tanks, 64 percent.

(ii) Filter units, 71 percent.

(iii) Secondary settling tanks, 64 percent.

(iv) Sludge digesters, 71 percent.

(v) Sludge beds, 50 percent.

It is evident that this group of plants was loaded at less than 75 percent of design capacity as set forth in the Engineering Manual. All filters in the group were equipped with rotary distributors, with the exception of the one at Fort Knox where fixed nozzles were utilized. Granite media were characteristic; size ranged from 1½ to 4 inches.

Median concentration of plant influent BOD was 332 ppm, a value somewhat smaller than occurred in plants of Division I. Per capita production of BOD amounted to 0.183 lb daily. Recirculation ratios were varied considerably (column 14, Table 45). Mean value of recirculation ratio was 1.18. Primary tank detention periods varied from 1.1 to 4.7 hours, and were generally somewhat lower than occurred in filter plants without recirculation.

As may be seen from column (19), the primary settling tank influent concentration was significantly reduced in plants employing recirculation through the primary section of the plant. Median removal in the primary section was 32.8 percent. It is pertinent to call attention to the fact that percent removal figures in column (20) of Table 46 were calculated on the basis of estimated removal from *raw sewage*, not including BOD of recirculated flow. The approach has been explained in detail in Chapter I.

A wide range of volumetric dosing rates was utilized, as may be seen from Table 47. Filter influent was much more dilute than in the plants of Division I in which there was no recirculation. Median value of filter influent BOD was 134 ppm—about one-half that for plants of Division I.

Average efficiency, as calculated from column (30) for filters, was 58 percent; average removal was 36.6 percent. Average BOD of final effluent was 44 ppm. Efficiency of the secondary treatment (filtration plus sedimentation) was 77.5 percent; removal was 49.8 percent. These two items, which are presented in columns (38) and (39), are based upon performance in (i) filters; (ii) secondary settling tanks and (iii) primary settling tanks, in cases where flow was recirculated through these units. Average value of overall plant removal of BOD was 86.2 percent. Values of dissolved oxygen in plant effluent varied from 1.0 to 7.1 ppm, with an average value of 3.7 ppm.

Average loading and performance data with respect to SS may be summarized as follows:

Raw Sewage, median SS, 296 ppm
Raw Sewage, median SS, 0.159 lb/cap/day
Primary Section Effluent, mean SS, 118 ppm
Primary Section, mean removal, 57.1 percent
Plant Effluent, mean SS, 42 ppm
Secondary Section, mean SS efficiency, 69.9 percent
Secondary Section, mean SS removal, 33.7 percent.
Overall Plant, mean SS removal, 86.6 percent.

Data pertaining to sludge digestion and disposal are given in Tables 50 and 51.

Performance of Individual Plants of Division II

Boca Raton AAF, Fla.

At Boca Raton AAF an unusually low per capita production of BOD was observed. This may in part be attributed to sampling difficulties with intermittent flow due to irregular pump operation. Samples were collected every 2 hours. Moreover, the plant was not equipped with a comminutor and a non-uniform distribution of organic load came as a result of intermittent release of accumulated solids with rack cleaning.

Grease was normal; no laundry waste was discharged to sewers. Overall plant performance was high considering the rate of load application. Unfortunately, sufficient data were not available from this plant to assess the performance of individual units.

Buckley Field, Colo.

Grease and laundry waste did not cause operational difficulties at Buckley Field. Attention should be directed to the design of the settling tanks (Plate 6) at this post. These were made quite shallow on the theory that sedimentation efficiency is a function of surface area, not depth. The side water depth of the tanks was 5 ft.

The original inlet baffle was open at the bottom and local high velocities of the inflowing sewage created turbulence that interfered with settling. Installation of a bottom baffle rectified this condition.

Considerable grit appeared in the influent during rainy weather. While a grit chamber was not included in the original plans, the forebay of the Parshall flume was operated as a sand trap, from which as much 3.2 cf of grit was removed per mg treated.

Both filters gave a little trouble from ponding, but this condition was readily controlled by rodding. Media size was somewhat smaller than usual for this type of filter.

Filter performance was excellent; these units removed 53.7 percent of BOD applied. In combination with secondary sedimentation, the filters removed 67.1 percent of applied BOD. Removal of SS in primary settling was considerably more efficient than BOD; overall plant removal of SS was in line with that of BOD. As may be seen from Fig. 78, the weight of BOD in the plant influent varied from 60 percent to 142 percent of the average during the period of study; percent removal of BOD, however, did not decrease significantly with this increase in load. In general the loading and performance of the plant at Buckley Field were in the range of those of a standard rate filter.

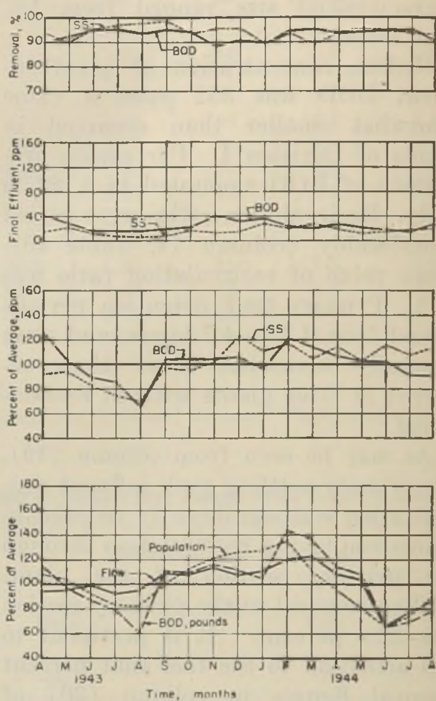


FIGURE 78.—Loading and performance of trickling filters at Buckley Field.

Camp Callan, Calif.

The plant at Camp Callan afforded an excellent example of performance of a heavily loaded filter utilizing a relatively low recirculation ratio. Sewage was pumped to the plant intermittently. Grease in the influent was negligible and no laundry waste was present. Excess capacity at one of the lift stations caused frequent periodic surges of flow at the plant. This excess capacity was also conducive to septicity, which in part accounts for comparatively low BOD removal in primary sedimentation. The filter at Callan was loaded more heavily than any other filter in the group (column 26), yet its removal, together with that of secondary settling, was comparatively higher. Excellent performance at Callan, in part, may be attributed to the warm equable climate of southern California.

From a study of results at Callan, it may be concluded that under favorable circumstances a high rate filter plant with typical military sewage can handle almost twice the load specified by the Engineering Manual for high rate filters and yet give a removal of 75 percent to 80 percent. It is pertinent to note, however, that with military sewage removals of this magnitude are not sufficient to produce good effluent. The effluent at Callan was poor, having a BOD of 77 ppm.

Camp Carson, Colo.

Grease content in the sewage at Camp Carson at the time of the survey was normal and it was practically all removed in the primary settling tanks. Laundry waste formed a substantial part of the flow (15%); the camp laundry at Carson furnished laundry service for all nearby army installations.

The original plant at Carson, consisting of screen, flume, two primary settling tanks, two filters, two final settling tanks, recirculating pumps, two digesters, and four sludge drying beds was placed in operation in June, 1942. In June, 1943 additions to the plant were made, consisting of one primary tank, two filters, one secondary tank, one sludge storage tank and two sludge drying beds. The additions were necessary because of increase in post population.

In view of altitude and meteorological factors in central Colorado, it was of interest, in connection with analysis of performance of the plant at Camp Carson, to ascertain whether a seasonal influence upon performance existed. The data pertaining to overall plant removal of BOD and SS are summarized in the following table:

Period	Wt in Plant Influent (lb)		Percent Overall Plant Removal	
	BOD	SS	BOD	SS
Jan to Mar 1944	3,090	2,412	88.2	88.3
Jun to Aug 1944	5,773	4,449	89.3	88.3

It is probable on the basis of these results that an appreciable seasonal influence exists; the filters would appear to have a significantly greater capacity for removal during the warm months.

Data pertaining to loading and performance at Camp Carson are presented in Figure 79. Load varied from 55 to 140 percent of the average of period studied. Seasonal effects, how-

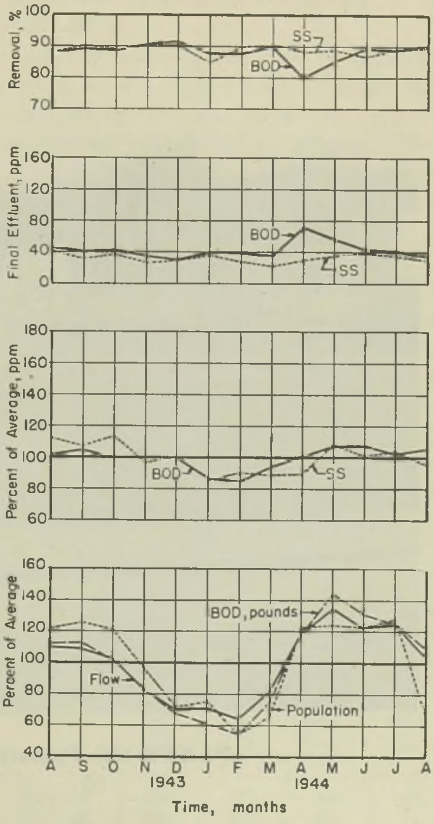


FIGURE 79.—Loading and performance of trickling filters at Camp Carson.

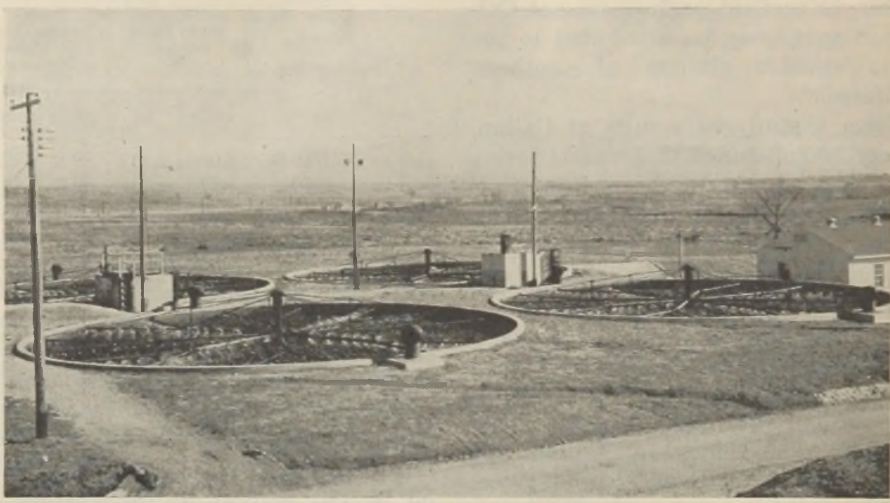


PLATE 11.—Trickling filters at Camp Carson, Colo.

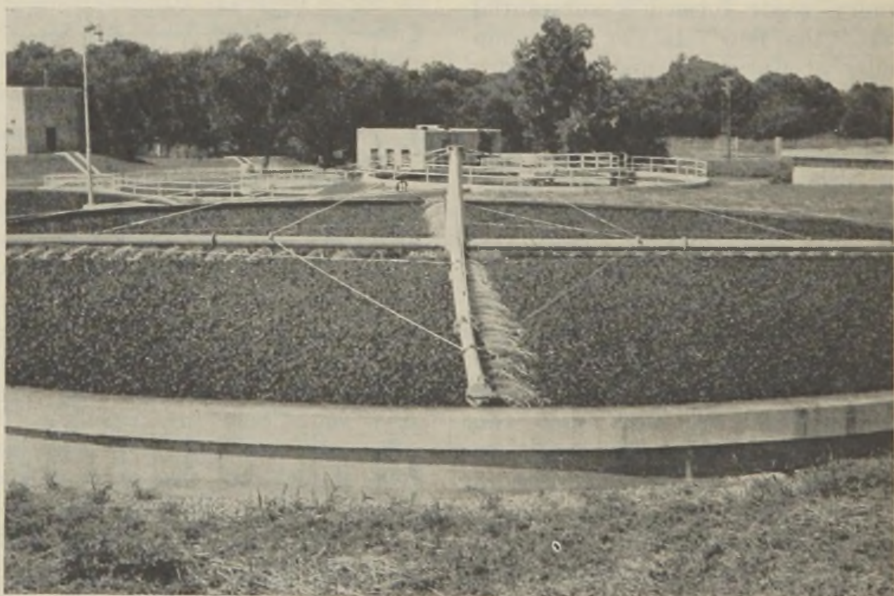


PLATE 12.—Primary filter at Fort Sill, Okla.

ever, tended to compensate the difference in load so that effluent quality remained about the same.

Few difficulties were encountered in operation of sludge digesters, however, no provision was made for recirculation of sludge in the storage tank, with the result that the bottom sludge at times became too dense and viscous to move freely, therefore line stoppage occurred. Dried sludge was used as a fertilizer base upon various areas of the military reservation; some dried sludge was sold to a manufacturer of commercial fertilizer.

Camp Claiborne, La.

Two sewage plants were operated at Camp Claiborne; the main plant, No. 1, had about twice the capacity of the west plant, No. 2. It was difficult to separate the camp population precisely into two contributory groups. The split was made on the basis of population reports from the Army Service Force Training Center, which group was presumed to be largely in the west camp. Consequently, the populations reported in column (4) of Table 42 must be taken as approximate estimates.

The main plant (No. 1) was placed in operation in March, 1941, but with subsequent population increase it became necessary to expand sewage treatment facilities at the post. Accordingly, the original plant was enlarged. Due to topographical conditions, it was necessary to make provision for part of the increased population by construction of a second plant (No. 2). The new units were placed in operation in April, 1943.

The original construction at the main plant consisted of 2 comminutors, 1 Parshall flume, 1 grease flotation unit, 1 primary settling tank, 4 aeration bays for mechanical activated sludge aeration, 1 secondary settling tank, 1 chlorine contact tank, 2 digesters, and 8 drying beds.

The main plant enlargement consisted of a grease flotation tank, a primary settling tank, a high rate filter, an intermediate settling tank, an aeration tank with several bays, a final settling tank, a sludge digester and sludge beds.

In the main plant about 70 percent of the sewage came from lift stations, and flow was largely intermittent due to the action of pumps. The sewer system consisted of 52 miles of sewers; gradients were very flat and sewage entered the plant in a stale condition with a strong odor. Grease content was not excessive at the time of the survey; laundry waste comprised 10 percent of the flow. The difference in per capita sewage flow at the two plants may in part be attributed to the fact that such establishments as the post laundry and bakery which served both camps were connected to sewers of the main plant.

As was usually the case in southern plants, it was found that optimum settling efficiency was attained only by continuous operation of sludge collection mechanisms. In this way the settling tank bottoms were kept free from accumulation of anaerobic deposits of raw sludge.

As indicated on the diagram in Table 45, the primary effluent was routed to the filter. The analysis of the main plant in Tables 42 to 50 pertains only to the filtration stage of treatment and does not include performance of activated sludge units. Consequently, removals attained are those of first-stage treatment and do not constitute overall plant removal. In order to indicate the BOD removals that occurred in various sections of the plant, the following table is presented:

	Raw Sewage (ppm)	Pri Eff. (ppm)	Intermediate Set. Eff. (ppm)	Plant Eff. (ppm)
Sep 43	338	197	48	16
Oct 43	377	192	89	14
Nov 43	392	217	77	19
Dec 43	403	238	68	21

Overall average removal of BOD during the 16-month period studied for the filtration stage, including primary and intermediate settling, was 80.8 percent; for the entire plant the average was 96.4 percent. As may be seen from Table 49, SS removal by the filter was abnormally low. Further data pertaining to performance of the main plant at Camp Claiborne are presented in a subsequent section relating to nitrification.

The west plant at Camp Claiborne, which treated rather concentrated sewage, gave typical performance as a low rate filter with low recirculation ratio. A notable removal of BOD and SS occurred in primary sedimentation (Table 46 and 49).

Drew Field, Fla.

The plant at Drew Field was placed in operation in February, 1943. A battery of three Imhoff tanks was added in November, 1943. The population at the post, however, was not increased as had been expected, and the Imhoff tanks were not used as regular treatment units during the period of the survey. Sewage arrived at the plant intermittently from 3 lift stations.

An excessive amount of grease occurred in the raw sewage, which was largely removed in the primary settling tanks. Nozzles on the rotary filter distributor were of the centrifugal type with removable cover. They were connected to distributor arms through a half union, 90° street ell, short nipple, and pipe clamp. Ventilation was provided consisting of two forced-draft blowers for each of the two filters. Air was withdrawn from the filters by the blowers through a central well.

The Drew Field plant provided another example of a filter with a loading that fell into the intermediate zone between the usual ranges for standard and high rate plants. Performance, too, was intermediate between that of standard and high rate filters. An unusually large removal occurred in the primary tanks.

Camp Gordon, Ga.

Some difficulty with grease was encountered at Camp Gordon; this was attributed to unsatisfactory design of grease interceptors at mess halls. Scum boxes at the effluent of primary settling tanks operated successfully and grease content in primary effluent was small. It was estimated that 40 to 50 gallons of scum were removed daily. Difficulty also was experienced with sand that entered the sewer system through low manholes and broken sewer lines. The grit accumulated in the primary settling tank and the problem was met by installation of a sand trap ahead of the comminutor, and also by repairing broken lines and raising manholes.

Recirculation at Gordon was around filters only. However, at the time of the survey, consideration was being given to recirculation of final effluent through the primary settling tanks because of low night flow and septic conditions. The filter was loaded in the usual standard rate range, but BOD removal was somewhat lower than normal. SS removal was about in line with loading. In January, 1944 distribution of flow to the four filters was modified so that, in effect, two-stage filtration without intermediate settling occurred. Accordingly, performance at Gordon subsequent to January, 1944 has been analyzed in the section on two-stage filter plants (Tables 67 to 84).

Fort Jackson, S. C.

Results at the plant at Fort Jackson are of particular interest in view of the fact that filter loading approximated that speecified as a design criterion in the Engineering Manual.

The history of expansion of sewage treatment facilities at Fort Jackson is typical of divisional training posts. A

2.0-mgd plant (biological treatment, single-stage digester) was placed in operation in February, 1941; in the construction of this plant, the original 1917 plant was incorporated as an integral part. In August, 1942, additional construction was placed in operation that increased capacity to 5.0 mgd. Two second-stage digesters were added in September, 1943. Raw sewage was typical as to concentration, and diurnal variation in flow was about average for cantonments of the same size. Pumped sewage constituted about 25 percent of the total.

Grease content during October, 1943 was 120 ppm (see also Table 9); subsequent corrective measures brought about a reduction. Laundry wastes were discharged into a creek during the period of the survey and did not enter the sewer system.

Some operating difficulties were experienced as a result of stale sewage from the north camp. This section of the post was connected to the system by a 1.5-mile force main, 12 to 16 in. in diameter, in which velocities often became very low. The distance from the farthest point served in the north camp to the plant was 5.25 miles.

Effects of excessive detention periods at night were largely overcome by augmenting nocturnal flows with water from a small stream, Wildcat Creek. This was accomplished by installation of control gates on a manhole near the stream. The gates were operated manually so as to divert 100,000 gallons for 8 hours during the night into the sewer system. The point of diversion was about 1.5 miles from the plant.

Nuisance from filter flies was successfully abated by turning nozzles on distributor arms straight down so as to utilize more effectively the kinetic energy of flow in scouring the upper surfaces of the media. This modification, together with augmentation of nocturnal flow, largely eliminated the *Psychoda* problem.

Removal of organic matter in primary settling at Fort Jackson was about average, however, the secondary section gave performance somewhat lower than average for the loading. This is probably to be attributed to the septic plant influent and to the effect of supernatant which was returned from digesters. A significant improvement in performance was noted subsequent to installation for additional digester capacity in September, 1943.

Fort Knox, Ky.

The record of operation of the plant at Fort Knox affords an outstanding example of the value of skilled and intelligent operation in the attainment of effective treatment. Diurnal distribution and concentration of plant influent were normal. Three small lift stations operated intermittently to discharge about 0.65 mgd of sewage into the gravity system. This constituted about 20 percent of the average volume of sewage treated. Infiltration was somewhat greater than average, and during the rainy season affected the distribution of flow. Data pertaining to grease content are presented in the following table:

	Jun '43	Jul	Aug	Sep	Oct	Nov	Dec	Jan '44	Feb	Mar	Apr	May	Jun	Jul
Raw Sewage (ppm)	161	133	206	170	129	199	126	94	91	114	90	98	80	85
Pri Eff. (ppm)	—	38	60	52	39	57	89	39	50	67	42	50	44	40

A small dose of alum (2.8 ppm) was applied to the primary settling tank influent; the operators were convinced that alum improved grease removal during settling.

The following difficulties in operation were experienced:

(i) Material dropping from fine screens caused clogging of fixed nozzles. It required about 6 to 8 man-hours a day to keep nozzles in working condition. Additional fine screens of ¼-in. hardware cloth were installed directly behind the original screens, which modification effectively protected the nozzles and resulted in a saving of about 6 man-hours a day.

(ii) Cycles of operation in the two dosing tanks failed to follow each other closely enough, so that there was an excessively long period in which there was no flow into either tank. During this period the flume leading to the dosing tanks would fill up and primary effluent would spill over into the by-pass channel. This condition was corrected by shortening blow pipes and making other adjustments in the dosing tanks.

(iii) Partial blocking of sewage flow into the primary settling tanks occurred by improper introduction of recirculated flow at this point. Recirculated trickling filter effluent entered the influent header channel at an off-center point. Due to turbulence, this created an unbalanced distribution of load in the primary settling tanks with the result that the amounts of sludge removed from each tank varied. Moreover, the unbalanced distribution partially defeated the purpose of recirculation since some of the sewage did not receive enough filter effluent to freshen it properly. The relatively low removal of suspended solids (46.3 percent) in primary settling may in part be attributed to unequal distribution of load in the primary tanks. A change in recirculation of secondary sludge from four one-hour periods daily to continuous operation, resulted in improved settling in the primary settling tanks and a reduction of approximately one-third of the amount of chlorine required to maintain a satisfactory residual in the plant effluent. This modification, which materially altered the recirculation ratio, was instituted subsequent to the period of operation analyzed in the tables. Performance of filters and secondary settling units at Knox was excellent in relation to the load applied.

No unusual problems of operation were encountered at Fort Knox in connection with sludge digestion. Lime was added to sludge twice daily at the sludge well; 50 pounds were used at each application. The supernatant had the following analysis:

Average BOD	17,000 ppm
pH	7.4
Total solids	0.85%
Volatile solids	61.2%

The supernatant was not returned to the plant but was routed to a lagoon. Total volume of supernatant during a 20-month period amounted to 40 percent of the volume of the raw sludge pumped.

Four lagoons at Knox augmented digester capacity and took the place of sludge drying beds. The lagoons had dimensions as follows:

Lagoon	Top Area (sf)	Effective Volume (cf)
1*	49,400	389,000
2	41,000	350,000
3	53,000	459,000
4	27,000	240,000

* No. 1 Lagoon used for supernatant liquor.

Sludge applied to lagoons was well digested; total solids amounted to 6.8 percent, volatile solids 48.0 percent. Analysis of sludge removed from lagoons was as follows:

Ash	54.9%
Total organic nitrogen	2.1%
Total ammonia	2.6%
Available phosphoric acid	1.50%
Citrate insoluble phosphoric acid	0.24%

No nuisances, flies or odors, were reported in connection with the operation of the lagoons.

An aeration ladder, including a 30-ft section providing step aeration was installed (Plate 16) in the effluent

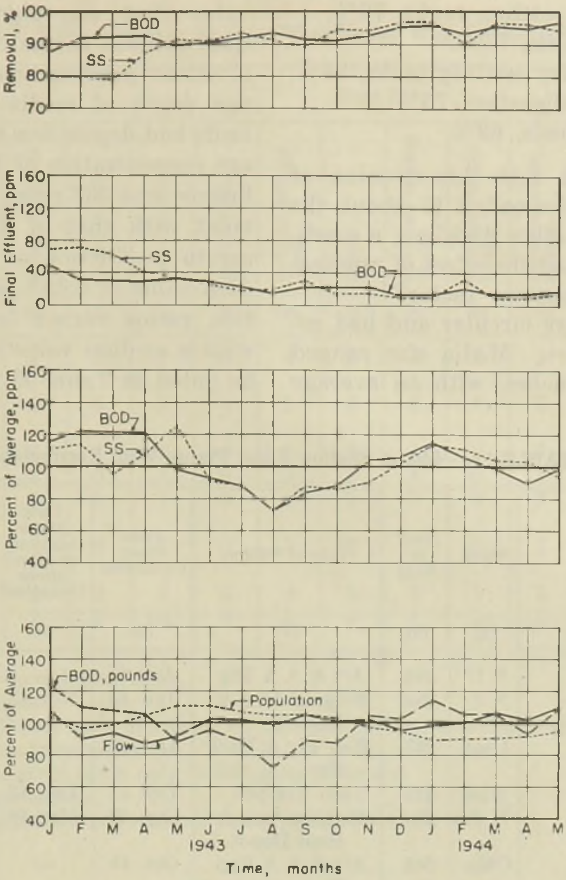


FIGURE 80.—Loading and performance of trickling filters at Fort Knox.

channel. There was an additional drop of 150 ft in the natural channel before the effluent reached a small stream.

III. Shallow Filters With Recirculation

The eight plants with shallow filters that are analyzed in Tables 53 to 66 form a fairly homogeneous group from the standpoint of design. Three of the posts, Forts Bragg and Sill and Keesler AAF, were located in the South. All plants in Division III were at army installations; all were designed as high-rate filters. The period of operation surveyed at each post varied from 11 to 20 months.

A design population factor of 23,100 persons per acre-foot was utilized as a design criterion in order to facilitate comparison. It is noted that the same factor was employed with plants of Division II pertaining to deep filters with recirculation. Actual loading is compared with design loading as calculated with the foregoing criterion on Tables 56 and 57.

As based upon actual population, post sizes varied from 8,940 at Kearns to 48,500 at Bragg. Flows ranged from 0.76 mgd at Warren to 3.67 mgd at Bragg. Average per capita sewage flow was 76 gallons. Median values of percent of design utilized (Table 57) may be summarized as follows:

(i) Primary settling tanks, 79%.
(ii) Filter units, 73%.
(iii) Secondary settling tanks, 94%.
(iv) Sludge digesters, 76%.
(v) Sludge beds, 62%.

It is evident that this division of plants was underloaded to about the same degree as other divisions, a condition that reflected the effect of application of the "capacity factor."

All filters were circular and had rotary distributors. Media size ranged from ¾ to 4 inches, with an average value that was significantly smaller than utilized in the deep filters of the divisions previously examined. Average depth of media was 3.35 ft; all units had depths less than 4.0 ft. Average concentration of BOD of plant influence was 333 ppm, a value in agreement with that of Division II. Per capita production of BOD had a median value of 0.188 pound. Recirculation ratios varied from 0.51 to 10.5 with a median value of 2.80. As may be noted in Table 59, primary recircu-

TABLE 53.—List of Shallow Filter Plants With Recirculation

Post	State	Sv C or Navy	Type of Station	Date Plant Completed	Date Major Additions or Alterations Completed	Operating Period Studied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Fort Bragg	N C	4th	Art & A A Trg	Aug 41	—	May 43 to Aug 44
Fort Dix	N J	2nd	Staging Area & Rec Ctr	Jun 43	Nov 43	Aug 43 to Jul 44
Kearns AAF	Utah	9th	Rep Trg & Recl Ctr	Dec 42	—	Aug 43 to Jun 44
Keesler AAF	Miss	4th	Tech Trg Sch	Dec 41	Dec 42	Jan 43 to Aug 44
Camp Kilmer	N J	2nd	Staging Area & Repl Depot	Oct 42	Mar 44	Aug 43 to Jul 44
Fort Sill	Okla	8th	Art & A A Rep Trg	Oct 41	—	Jan 43 to Aug 44
Camp Myles Standish	Mass	1st	Staging Area & Repl Depot	Jun 43	—	Sep 43 to Aug 44
Fort Francis E Warren	Wyo	7th	Serv Trp Trg	Jan 42	Jan 43	Jan 43 to Aug 44

TABLE 54.—General Data Relating to Shallow Filter Plants With Recirculation

Post	Approx Plant Elev (ft)	Range of Sewage Influent Temp (°F)	Average Sewage Influent pH	Lineal Feet of Sewers per Capita (a)	Sampling	
					Hours per Day	Average Number per Month
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Bragg	—	56–85	7.0	12.3	24	30
Dix	—	36–80	7.0	4.9	24	4
Kearns AAF	—	57–82	7.7	2.6	24	25
Keesler AAF	—	72–91	7.3	—	24	25
Kilmer	—	44–82	7.4	—	24	22
Sill	—	61–85	7.5	—	24	9
Myles Standish	—	33–78	7.4	3.9	24	30
F E Warren	6100	52–71	7.6	—	24	9

(a) Based on design population, Table 56, col (3).

TABLE 55.—Units in Shallow Filter Plants With Recirculation

Post	Plant Units											
	Screens or Racks	Comminutors	Grit Chambers	Imhoff Tanks	Primary Settling Tanks	Filters	Secondary Settling Tanks	Chlorine Contact Tanks	Digestion Tanks, Heated	Digestion Tanks, Unheated	Sludge Beds	Other Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Bragg	1	2	1	0	4	2	4	1	2	0	10	(a)
Dix	2	0	2	0	2	2	2	1	2	0	36	(b) (c) (d)
Kearns AAF	1	1	1	0	1	1	1	0	2	0	22	
Keesler AAF	1	0	1	0	3	3	3	0	3	0	17	
Kilmer	2	0	1	0	2	2	2	0	2	1	40	
Sill	1	0	1	0	1	2	1	0	1	0	10	(b) (d) (e)
Myles Standish	2	2	2	0	2	2	2	1	2	0	10	(f)
F E Warren	2	1	1(g)	0	1	2	1	1	2	0	6	(e)

(a) Two flocculation tanks.
(b) Triturator.
(c) Old Doten tank, 82,500 cu. ft. Used for grease, scum and supernatant.
(d) Grease flotation tank.
(e) Units given apply only to first-stage filtration.
(f) Sludge lagoon.
(g) Combination grit removal and grease flotation by air.

TABLE 56.—Design Capacities and Loading of Shallow Filter Plants With Recirculation

Post	Number of Months in Operating Period Studied	Population				Flow				
		Design Capacity (a)	Actual			Design (mgd) (a)	Actual			
			Average	Maximum	Minimum		Average (mgd)	Maximum (mgd)	Minimum (mgd)	Gal per Capita Daily
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Bragg	16	51,300	48,500	56,400	37,200	3.59	3.670	6.270	1.030	76
Dix	12	53,000	40,700	52,100	31,300	3.71	2.340	4.050	0.790	58
Kearns AAF	11	34,900	8,940	16,000	4,480	2.44	0.780	1.690	0.118	87
Keesler AAF	20	47,100	32,000	41,300	20,500	3.30	2.590	5.970	0.700	81
Kilmer	12	39,000	24,000	34,000	16,300	2.73	2.340	3.490	1.170	98
Sill	20	19,500(b)	40,200	51,700	33,000	1.37(b)	2.820	4.200	1.300	70
Myles Standish	12	25,000	14,600	18,900	9,920	1.75	0.920	—	—	63
F E Warren	20	6,240(b)	10,300	14,300	6,250	0.44(b)	0.760	1.420	0.353	74

(a) Design capacities are based chiefly on criteria in OCE Engineering Manual (1943). See text.
(b) Based on first-stage filters only.

TABLE 57.—Percent of Design Capacity Utilized in Shallow Filter Plants With Recirculation

Post	Primary Settling Tanks (%)	Filter (%)	Secondary Settling Tank (%)	Sludge Digester (%)	Sludge Beds (%)
(1)	(2)	(3)	(4)	(5)	(6)
Bragg	87	95	96	73	75
Dix	92	77	92	79	81
Kearns AAF	87	26	150	30	8(a)
Keesler AAF	52	68	71	107	86
Kilmer	60	61	65	79	40
Sill	148	206(b)	157	144	110
Myles Standish	71	58	71	49	49
F E Warren	62	165(b)	145	52	46

(a) Natural sand beds.
(b) Based on first-stage filters only.

TABLE 58.—Design and Description of Filters in Shallow Filter Plants With Recirculation

Post	Number of Filters	Dimensions—Length by Width or Diameter (ft)	Area (acres)	Volume (af)	Filter Media		
					Depth (ft)	Type	Size (in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Bragg	2	134d	0.648	2.22	3.42	—	1½–2½
Dix	2	130d	0.610	2.29	3.75	—	1½–2½
Kearns AAF	1	155d	0.430	1.51	3.50	Gravel	¾–4
Keesler AAF	3	2–85d 1–120d	0.522	2.04	3.90	2-Granite 1-Slag	2 4
Kilmer	2	125d	0.564	1.69	3.00	Limestone	1–2½
Sill(a)	2	85d	0.260	0.845	3.25	—	1–3
Myles Standish	2	100d	0.361	1.08	3.00	Granite	1–2½
F E Warren(a)	2	50d	0.09	0.27	3.00	Gravel	1½–2½

(a) First-stage filters only.

lation (column 10) was generally somewhat smaller than secondary recirculation (column 12).

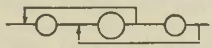
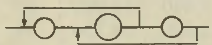
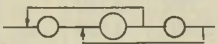
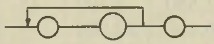
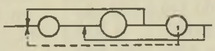
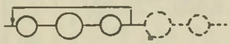
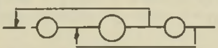
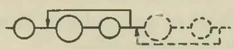
As mentioned previously, it was possible to analyze separately the performance of the first stage of filtration at Forts Sill and Warren; the data in Tables 55 to 63 pertain to first-stage filtration only. Overall plant performance at these two posts is presented in the following section on two-stage filters. The first-stage filters at Sill and Warren were heavily loaded and, as expected, partial treatment only was attained.

Detention periods, which were so calculated as to include effect of recirculated flow, varied from 1.68 to 4.75 hrs with a median value of 3.19 hrs. Primary settling tank influent was considerably diluted by primary recirculation. Average primary influent BOD concentration in plants utilizing primary recirculation (R_1) was 175 ppm, representing an effective dilution of almost two to one. Average removal in primary settling tanks was 27.2 percent, a value somewhat lower than the average for all filter plants.

With the relatively high recirculation ratios of many of the plants, volumetric dosage rates were high, varying from 11.1 to 31.9 mgad.

Concentration of filter influent va-

TABLE 59.—BOD Loading and Performance of Shallow Filter Plants With Recirculation

Post	Diagram	Plant Influent			Recirculation		
		Flow (mgd)	BOD				
			PPM	Pounds per Capita Daily	R_1 (mgd) (a)	R_2 (mgd) (b)	Ratio Total R to Plant Inf
(1)	(2)	(3)	(4)	(5)	(10)	(12)	(14)
Bragg		3.670	356	0.225	7.074	8.145	4.15
Dix		2.343	348	0.167	3.231	3.235	2.82
Kearns AAF		0.777	273	0.198	2.787	5.382	10.50
Keesler AAF		2.592	218	0.148	3.194	—	1.26
Kilmer		2.339	201	0.164	1.724	2.066	1.86(c)
Sill(d)		2.820	305	0.179	1.440	—	0.51
Myles Standish		0.916	537	0.281	2.100	2.100	4.59
F E Warren(d)		0.759	429	0.264	—	2.108	2.77

(a) R_1 = Recirculation to primary treatment section.
(b) R_2 = Recirculation to secondary treatment section.
(c) Total R includes 0.288 mgd secondary sludge return to primary.
(d) First stage of filtration alone considered here; for analysis of entire plant see Tables 67 to 84.

TABLE 60.—BOD Loading and Performance of Shallow Filter Plants With Recirculation

Post	Primary Treatment Section						
	Flow (mgd)	Detention Period (hr)			BOD		
		At Average Flow	At Maximum Flow	At Minimum Flow	Influent (ppm) (a)	Effluent (ppm)	Removal (%) (b)
(1)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
Bragg	10.742	2.87	2.31	3.81	169	147	11.6
Dix	5.574	2.70	2.07	3.75	177	32	21.7
Kearns AAF	3.564	2.87	2.28	3.52	90	58	37.3
Keesler AAF	5.786	4.75	3.00	7.05	119	115	—
Kilmer	4.351	4.12	3.26	5.64	128	89	34.1
Sill	4.26	1.68	1.27	2.62	231	175	24.3(c)
Myles Standish	3.016	3.51	2.64	4.55	220	138	46.3
F E Warren	0.759	4.02	2.14	8.65	429	359	15.3

(a) Weighted average, calculated from plant influent and recirculation.
(b) Calculated in accordance with Equation (3).
(c) Calculated in accordance with Equation (4).

TABLE 61.—BOD Loading and Performance of Shallow Filter Plants With Recirculation

Post	Secondary Treatment Section									
	Filter Loading						Filter Only Performance			
	Flow		Pop'n per Acre-Foot	BOD			BOD			
	MGD	MGAD (a)		Inf (ppm) (b)	LB/AFD (c)	LB/AD (c)	Eff (ppm)	LB/AFD Removed	Efficiency (%) (c)	Removal (%)
(1)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)
Bragg	18.887	29.2	21,800	109	4,340	14,900	70	2,745	63.3	55.8
Dix	8.809	14.4	17,800	93	2,320	8,720	54	1,250	53.9	42.3
Kearns AAF	8.947	20.8	5,920	36	792	2,780	—	—	—	—
Keesler AAF	5.786	11.1	15,700	115	2,370	9,320	42	1,735	72.9	75.3
Kilmer	6.417	11.4	14,200	72	1,530	4,590	45	865	56.3	37.1
Sill	4.260	16.4 (e)	47,600 (e)	175	6,450 (d) (e)	20,900 (d) (e)	—	—	—	—
Myles Standish	5.116	14.2	13,550	109	2,030	6,100	81	1,085	53.3	28.5
F E Warren	2.867	31.9 (e)	38,100 (e)	—	8,520 (e)	25,500 (e)	—	—	—	—

(a) Includes recirculation.
(b) Weighted average including BOD of recirculated flow.
(c) Calculated in accordance with Equation (3).
(d) Calculated in accordance with Equation (4).
(e) Based upon first-stage filtration.

ried from 36 to 175 ppm, with a median value, 109 ppm, that was smaller than for filters considered in foregoing divisions. On a weight basis, the BOD loading varied from 792 to 8,520 lb/af daily. The items in columns 26 and 27 were calculated in accordance with the method presented in the section "Primary and Secondary Sections of Sewage Plants" (Chapter I); accordingly, the loading data pertain to BOD in raw settled sewage and do not include BOD in recirculated flow. The influence of recirculation upon performance will be considered in a subsequent section, along with a discussion of the effect of loading upon removal in trickling filter plants.

The average value of percent efficiency of filter units was 60.2 percent. This value is somewhat smaller than the corresponding figure (70.5 percent) for filters without recirculation (Table 33, column 30) that were much less heavily loaded. Nevertheless, in view of data from a number of laboratory and semi-plant scale tests that have appeared in the literature since 1935, the actual reduction of BOD by high rate filters at military installations, not including secondary sedimentation, was remarkably high. The average BOD efficiency of the secondary sections, including secondary settling (column 38), was 75.8 percent. Thus the filters alone induced about four times the removal that occurred in secondary settling units. Hence the concept that high rate filters act essentially as biological coagulating agents would not appear to be applicable to heavily loaded filters at military posts. The contribution to overall removal by secondary settling was relatively small.

The average value of removal in filters plus secondary settling was 58.4 percent, which was somewhat over twice the removal occurring in primary settling in the plants of Division III. Overall plant removal of BOD aver-

TABLE 62.—BOD Loading and Performance of Shallow Filter Plants With Recirculation

Post	Secondary Treatment Section (Cont) — Filter & Secondary Settling Performance (a) — Detention Period (hr) — Settling Tank — At Av Flow	At Max Flow	At Min Flow	Contact Tank at Av Flow	BOD — Final (ppm)	LB/AFD Removed (f)	Efficiency (%) (f)	Removal (%) (f)	Overall Plant BOD Removal (%)	Final Effluent — Dissolved Oxygen (ppm)	Relative Stability (%)
(1)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(43)	(44)
Bragg	2.61	2.14	3.36	0.34	61	3,500	80.7	71.3	82.9	1.8	12
Dix	2.70	2.07	3.74	0.57	28	2,080	89.9	70.3	92.0	3.1	—
Kearns AAF	1.66	1.45	1.86	—	21	705	88.8	60.0	92.3	3.4	60
Keesler AAF	3.53	1.53	9.80	—	30	2,060	86.4	89.5	86.0	1.8	—
Kilmer	3.83	3.07	5.08	—	44	1,030	67.0	44.2	78.2	4.5	49
Sill	1.05 (c)	.80 (c)	1.64 (c)	—	85 (b)	4,060 (d) (g)	63.0 (d) (g)	47.9 (d) (g)	72.2 (d)	—	—
Myles Standish	3.52	2.68	4.54	1.12	54	1,660	81.3	42.6	90.0	3.5	—
F E Warren	1.06 (c)	.86 (c)	1.24 (c)	0.60 (e)	173 (b)	4,300 (d)	50.5 (d)	42.7 (d)	58.0 (d)	—	—

(a) Including chlorine contact tank, if any.
(b) Intermediate settling tank effluent.
(c) Intermediate settling tank.
(d) First-stage filtration only.
(e) Including detention time in weir chamber.
(f) Calculated in accordance with Equation (3).
(g) Calculated in accordance with Equation (4).

TABLE 63.—SS Loading and Performance of Shallow Filter Plants With Recirculation

Post	Suspended Solids — Plant Influent — PPM	Pounds per Capita Daily	Primary Treatment Section — Effluent (ppm)	Removal (%) (a)	Secondary Treatment Section — Final Effluent (ppm)	Efficiency (%) (a)	Removal (%) (a)	Overall Plant Removal (%)
(1)	(2)	(3)	(4)	(5)	(9)	(12)	(13)	(14)
Bragg	249	0.157	87	32.0	45	73.7	50.2	82.2
Dix	364	0.174	118	42.4	51	75.8	43.8	86.2
Kearns AAF	345	0.250	73	34.2	30	87.0	57.2	91.4
Keesler AAF	314	0.213	122	32.1	52	75.5	51.2	83.3
Kilmer	228	0.185	83	46.3	44	64.1	34.4	80.7
Sill	273	0.160	95	51.7 (b)	55 (c)	58.6 (b) (c)	28.3 (b) (c)	80.0 (c)
Myles Standish	211	0.110	63	30.9 (b)	21	85.7	59.2	90.1
F E Warren	269	0.166	210	22.0	96 (c)	54.2 (c)	42.2 (c)	64.2 (c)

(a) Calculated in accordance with Equation (3).
(b) Calculated in accordance with Equation (4).
(c) First-stage filtration only.

TABLE 64.—Sludge Data Relating to Shallow Filter Plants With Recirculation

Post	Number Months in Operating Period Studied	Sludge Digester					Schedule of Sludge Loading	
		Total CF per Capita (a)	Heated, Percent of Total Volume	Covered, Percent of Total Volume	Average Operating Temperature (°F)	Source of Sludge	Secondary to Primary Settling Tanks—Times per Day	Primary to Sludge Digesters—Times per Day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Bragg	16	4.1	100	100	98–85(c)	Pri(b)	Cont	2
Dix	12	3.8	100	100	86–76	Pri	—	4
Kearns AAF	11	10.1	100	100	95–100	Pri(b)	6 or more	6
Keesler AAF	20	2.7	100	100	85	Pri(b)	—	5
Kilmer	12	4.2	68(e)	100	90(e)	Pri(b)	Cont	1
Sill	20	2.1	100	100	88	Pri(b)	Cont	20
Myles Standish	12	6.1	100	100	90	Pri	Cont	3 or 4
F E Warren	20	5.8(d)	100	100	95–86	Pri(b)	Cont	48

(a) Based on actual population, Table 56, col (4).
(b) Includes scum from primary tank.
(c) Larger value refers to primary tank.
(d) Two-story tray-type primary digester.
(e) Two heated primary units; one unheated secondary unit.

TABLE 65.—Sludge Data Relating to Shallow Filter Plants With Recirculation

Post	Gallons Raw Sludge per MG Sewage	Volatile Solids, Percent of Total Solids		Sludge Disposal			
		Raw Sludge	Digested Sludge	SF per Capita (a)	No Months Sludge Drawn per Year	Sludge Drawn Yearly from Digesters (cf/cap)	Sludge Removed Yearly from Beds (cf/cap)
(1)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Bragg	8,440	79.4	58.8	1.34	12	4.7	—
Dix	7,460	75.6	58.0	2.09	—	—	—
Kearns AAF	3,630	79.5	—	13.30(b)	—	—	—
Keesler AAF	13,300	74.8	59.3	1.17	12	3.5	—
Kilmer	5,000	77.0	60.0	2.50	12	5.8	0.9
Sill	5,900	79.0	—	0.91	10	—	—
Myles Standish	12,500	74.9	—	2.06	—	—	—
F E Warren	4,890	84.4	76.7	2.16	10	4.1	1.1

(a) Based on actual population, Table 56, col (4).
(b) Natural sand beds.

aged 81.3 percent. Average value of overall plant removal of BOD in those plants loaded with from 2,000 to 4,500 lb/af daily (column 26) was 87.7 percent. Average value of dissolved oxygen in the plant effluent for all plants in the division was 3.0 ppm.

The average loading and performance of plants with shallow filters with respect to SS may be summarized as follows:

Raw Sewage, median SS, 271 ppm
Raw Sewage, median SS, 0.170 lb/cap/day
Primary Section Effluent, 91 ppm
Primary Section, mean removal, 33.1%

TABLE 66.—Chlorination and Effluent Disposal in Shallow Filter Plants With Recirculation

Post	Chlorination		Receiving Water Body	
	Points of Application Used	Residual Chlorine (ppm)	Name	Type
(1)	(2)	(3)	(4)	(5)
Bragg	—	—	—	Flowing Stream
Dix	Post	0.5	Tributary of Crosswick Creek	Flowing Stream
Kearns AAF	—	—	Jordan River	Flowing Stream
Keesler AAF	—	—	Back Bay of Biloxi	Tidewater
Kilmer	—	—	Raritan River	Tidewater
Sill	—	—	Cache Creek	Flowing Stream
Myles Standish	Post	0.7	Horton's Pond	Dry Run ($\frac{1}{2}$ Mi) to 6 Acre Pond
F E Warren	—	—	Crow Creek	Flowing Stream

Plant Effluent, 48 ppm
Secondary Section, mean SS efficiency, 72.0%
Secondary Section, mean SS removal, 46.0%
Overall Plant, mean SS removal, 82.3%

Data pertaining to sludge digestion and disposal for plants of Division III are presented in Tables 64 and 65. A detailed analysis of sludge digestion and drying at Fort Bragg and Camp Kilmer is included in a subsequent section on sludge digestion.

Performance of Individual Plants of Division III

Fort Bragg, N. C.

The plant at Fort Bragg was relatively large and loaded nearly to capacity. About 25 percent of the sewage flow to the plant was pumped from low areas. Infiltration and grease were normal. Sewage reached the plant in a condition somewhat less fresh than average, the sewer system being long for the population served. Gradients in sewers were moderate. The chief operating difficulty resulted from broken rubber tire treads on the wheels of the sludge removal mechanisms. Some treads lasted only 3 weeks before failure, and wheels had to be returned for retreading. The condition was attributed to the poor quality of rubber obtained during the war.

The soil is sandy at Fort Bragg, and with the old sewer system of this permanent post, the grit detriter served a useful function. Two large coagulating basins with stirring mechanisms were installed ahead of the primary settling tanks. The coagulating basins provided a detention period of about 2.5 hrs at average flow. No chemical coagulants were used. The basins did not improve settling performance, which was low (Tables 60 and 63). On the other hand, the aerating and mixing action of the basins appeared to have some value in maintaining high efficiency of biological treatment on the filters (Tables 61, 62 and 63). A history of loading and performance at the Fort Bragg plant is presented in Fig. 81.

The orifice plates on the filter distributors required cleaning two or three times per week to remove such obstructions as leaves and grease balls. With the exception of one time when the plant was first placed in operation, no filter clogging or pooling occurred. In this instance the condition was corrected by allowing the filter to remain out of operation for 48 hrs, with appli-

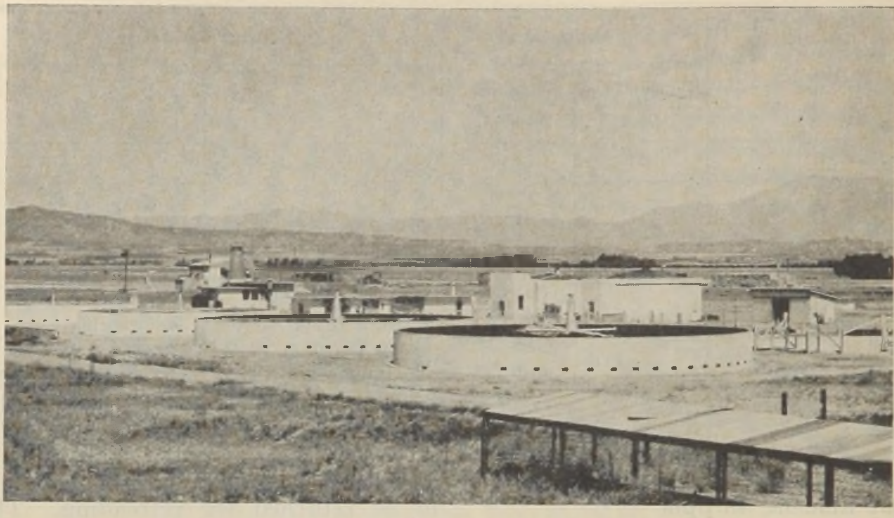


PLATE 13.—Trickling filter plant at March Field, Calif.

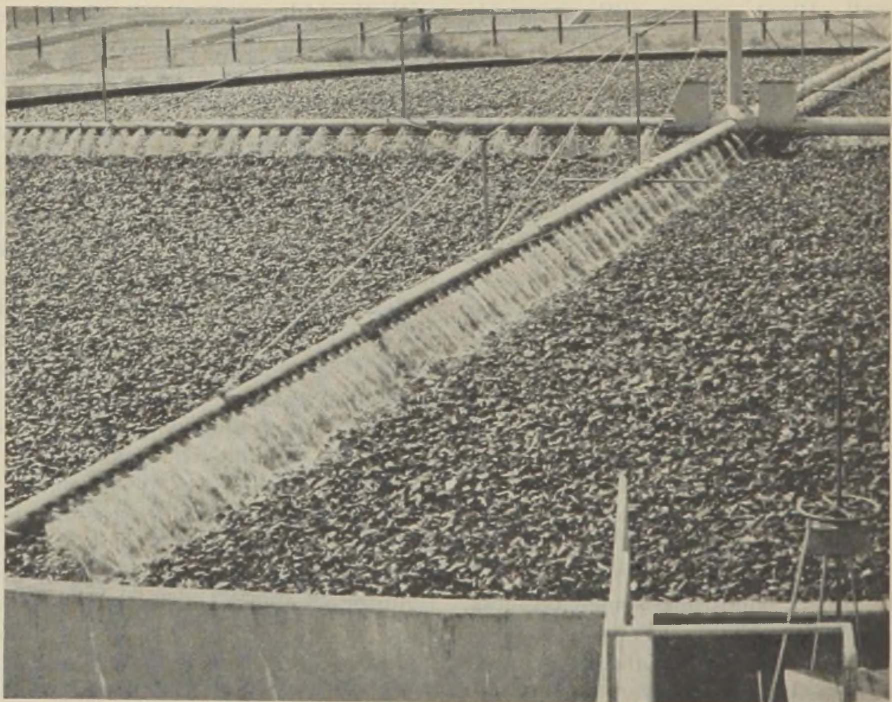


PLATE 14.—Rotary distributor on filter at Main Plant, Camp Claiborne, La.

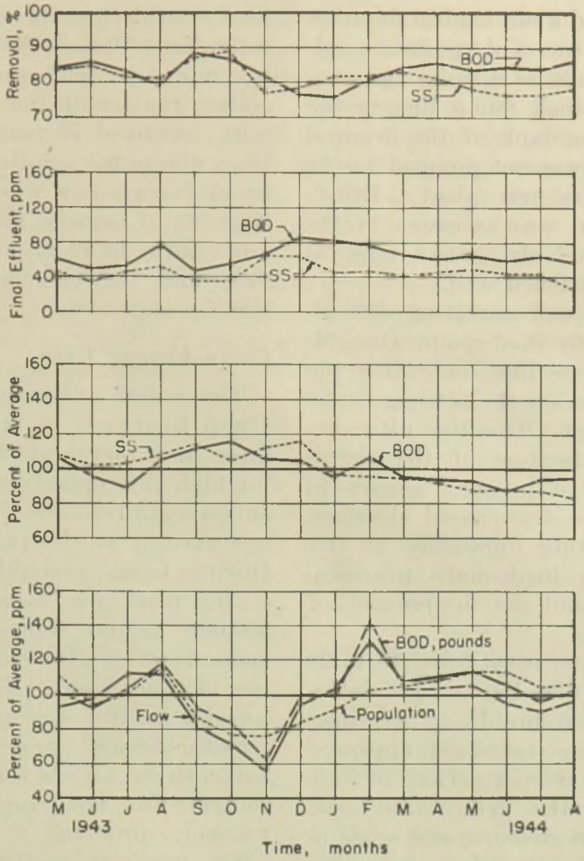


FIGURE 81.—Loading and performance of trickling filters at Fort Bragg.

cation of calcium hypochlorite to ponded areas. Unloading of filters took place in the winter months, from December through February. At the time of inspection unusually large numbers of *Achorutes viaticus* were seen on the filter surface; *Psychoda* larvae were notably absent. DDT was used at this plant for insect control.

Lime and scum formed rather thick layers in digesters, but this condition did not interfere with operation. Supernatant was generally of fair quality; it was routed to the head of plant and also to the sludge drying beds.

Fort Dix, N. J.

Fort Dix, as in World War I, was a Staging Area and unusual conditions of loading were imposed upon the sewage treatment plant. Population shifts were sudden and extreme; at certain periods, the changes took place on a semi-weekly and even daily basis. The flow might be about 2.0 mgd for several days and then increase to 3.0 mgd for a week. Variations in population were even more extreme. The new high rate plant at Fort Dix was completed in April, 1943 to supplement the old plant which was overloaded throughout 1942. The new plant, which was constructed on a plot adjacent to the original plant, also provided capacity to handle waste from a new post laundry.

In addition to the usual equipment for high rate filtration, grease removal equipment was installed. The grease removal tank had a capacity of 4,410 cf and contained a mechanical flotation unit. Nearby was a deep grease collection pit serving as a sump and connected by a 6-inch pump line to the primary settling tank of the original plant. Grease was not pumped to the digesters of the new plant. Ether-soluble material was excessive in the raw sewage, and the grease removal equipment functioned well.

One of the chief operating difficulties was due to inadequate size (6-inch) of the pipe line connecting the grease skimmers on the settling tanks to the grease pit. This line often became clogged because of the exceptional amount of grease. Leaves in the late autumn also caused clogging.

Other operating difficulties at Dix stemmed from inadequate provision for drainage and for by-passing of individual units.

Laundry wastes constituted 15 to 20 percent of the flow but did not introduce any serious problem. At times a considerable amount of grit appeared in the influent; during periods of high flow some of the grit would pass through the grit chamber and settle in grease removal units, where it was removed with difficulty.

Removal of BOD in primary tanks was rather low, but this was more than offset by the exceptionally high efficiency of filtration (Table 62). The filters at Dix received approximately four times the organic load per unit volume usually applied to low rate plants, yet BOD removal compared favorably with that attained in standard filter plants. The outstanding performance of the plant at Fort Dix pointed up the adaptability of the high rate filter to the extreme conditions of loading that occurred in staging areas.

The plant effluent at Fort Dix was discharged into a potable water supply (Crosswick Creek to Delaware River). Accordingly chlorine at the rate of 50 to 60 lb per mg was added. A smaller amount of chlorine might have been used considering the degree of treatment, but a 30-inch, 900-foot reinforced concrete pipe that connected the secondary tanks with the chlorine contact tank, remained 75 percent full all the time, due to flat gradients. The velocity at average flow was less than 1 fps. Deposits of organic matter that could now readily be removed, formed in this line, and the chlorine demand was thereby increased.

Camp Kearns, Utah

The record of sewage treatment at Camp Kearns is of interest since the filter unit there, which was designed for high rate operation, was loaded to only a small fraction of capacity. Sewage arrived at the plant by gravity. During heavy precipitation, the flow at the plant was increased by about 500,000 gallons daily. Considerable amounts of sand were removed from the grit chamber. No provision for velocity control was made in the grit removal channels, and these units were not entirely satisfactory. The grease content was low and laundry waste was not a problem.

The flow was smaller than expected and it was necessary to change the throat of the Parshall flume from 24 to 12 inches in order to increase precision of measurement. The new throat, which was made of plywood, was calibrated with a triangular weir in the grit chamber. While fluctuations during the month were marked, this nonuniformity was not reflected to a great extent in water consumption. Lawn sprinkling and other uses of water that were more or less independent of population accounted for a large proportion of flow.

The recirculation ratio, 10.5, was high; this may be attributed to fixed recirculation capacity and lower sewage flow than provided for in design. Performance in both primary and secondary sections of the plant was excellent and in line with the relatively low rate of load application as based on raw settled sewage. The wooden overflow weirs were subject to swelling, and it was necessary to plane the crests from time to time to keep them level.

The filter was set entirely above ground with ventilator holes through the wall below the grating at intervals of about 6 ft. This construction, which was common in the West (Plate 13), made it impossible to flood for fly control. However, due to good distribution and ventilation, flies never became numerous at the Kearns plant.

Sludge digester capacity was more than ample and no serious operating difficulties were encountered in digestion. Cracks in the dome of the digesters permitted gas to escape; this circumstance increased cost of operation, since fuel had to be purchased. During the last months of operation investigated, liquid digested sludge was discharged by gravity from the digesters into a tank truck and spread upon post grounds. Soil in the region was porous and odor problems did not develop.

Keesler Field, Miss.

Performance of the plant at Keesler AAF was fairly typical of performance of high rate filters at southern posts. The filter units were loaded to about two-thirds capacity. Sewer gradients were moderate, and infiltration normal; odor was slight, but with some sulfide detectable. Grease was not excessive and laundry waste constituted about 5 percent of the flow. The plant was generally well designed and operated. Some difficulty stemmed from lengthy detention periods in primary settling tanks. Installation of water nozzles on these tanks improved scum removal. Filters could not be flooded, but at this plant fly breeding was not a problem.

The chief difficulty pertained to scum formation in the secondary digester due to grease. Supernatant quality was variable, but usually contained less than 0.5 percent solids; it had a pH of 7.2. It was returned to plant influent. In the relatively warm climate of the Gulf Coast, gas was needed as a fuel for heating digesters only during the winter months. Wet digested sludge was applied to drying beds and also hauled to airfields for soil conditioning.

Use of sludge, both wet and dry, to promote growth of grass upon denuded areas near landing strips was a common practice at flying fields, particularly in the more arid regions. The amount of sand and grit carried into the air during windstorms was thereby reduced. It is impossible to evaluate precisely the enormous saving that this practice effected throughout the war in lessening the rate of deterioration of airplane engines from internal abrasion.

Camp Kilmer, N. J.

The record of sewage treatment at Camp Kilmer is indicative of the difficulties that beset operation in a plant subjected to the extreme population fluctuations that occurred in military staging areas. Loading on the average was less than two-thirds of design capacity. Average values do not, however, convey an adequate description of loading; population at Kilmer would change from, say, 5,000 to 40,000 overnight. Treatment units that had been nearly idle would be taxed to capacity within a space of a few hours. The situation demanded the utmost flexibility of operation.

About 50 percent of the sewage at Camp Kilmer was pumped intermittently to the plant from two automatic pumping stations. Flow through the plant was further influenced by the operation of a mechanical bar screen controlled by a time clock that was set to cause operation during a fixed proportion of the time. This unit, with excessive screen accumulation, caused backing up of flow in sewers and induced pulsations. Instantaneous rates occurred as high as four-thirds the average daily flow. With installations of this type, chance synchronization of the sampling schedule with the time clock may result in a distorted evaluation of loading and performance.

Infiltration into the sewers at Kilmer was considerable. When the sewer system and treatment plant were initially placed in operation, infiltration constituted 70 percent to 300 percent of the average flow. After completion of a program of surface water drainage at the post, infiltration was reduced to 15 to 150 percent of flow. Per capita flow of sewage (Table 56, column 11) was the highest of all plants in Division III.

The grit chamber at the Kilmer plant fulfilled a useful function, particularly during the earlier period of the war when the high infiltration carried considerable amounts of sand into the sewer system.

Performance of the primary section of the plant was normal; long detention periods occurred. The reduction of BOD and SS that took place in the filters and secondary settling units, however, was definitely substandard. Numerous investigations were made and it was concluded that the poor performance of the filters, which were loaded at about three-fifths the rate specified as a design criterion in the Engineering Manual, was caused by clogging resulting from a flourishing growth of sulfur bacteria, *Thiobacteriales,* principally *Beggiatoa alba.* The office of the Service Command Engineer, the Post Engineer organization, representatives of the New Jersey Agricultural Experimental Station and equipment manufacturers were concerned with the problem—one that was unique in the history of the high rate filter. It was found that the sulfur-utilizing organisms could be killed by superchlorination; but after being killed, the organisms would not slough from the media.

Only with a considerable passage of time did the dead growths wash off and pass through the filter. With the cleaning of the filters, performance improved. Excavation of pipes connecting the primary clarifier with the biofilter brought forth the fact that sulfur organisms had been thriving in underground conduits and in the surrounding soil. The water supply at the post contained only 35 ppm of sulfate (as $SO_4^{=}$); the ground water near the plant, however, had a high sulfur content. The cells of the filter growth were observed to be filled with multi-colored sulfur granules.

In addition to the clogging caused by the sulfur organisms, some odor nuisances developed; these were controlled by chlorine. Life of rubber squeegees on the skimmers of settling tanks was short; this was remedied by placing a sheet metal reinforcement behind the rubber on the squeegee frame.

No serious operating problems were encountered at Camp Kilmer in connection with sludge digestion and disposal. Digesters were provided with agitation equipment, which was operated about 3 hr each day. Scum and grease were not troublesome. Supernatant was of fairly good quality; it was a light straw color and contained 0.2 percent to 0.3 percent of solids.

Camp Myles Standish, Mass.

The Staging Area and Replacement Depot at Camp Myles Standish, like other deployment centers near the ports of embarkation, experienced a constant turnover of personnel, with large and irregular variations in post population. The high rate filter at Myles Standish performed satisfactorily under the non-uniform loading and gave an effluent that met the requirements of the receiving water and its environs. In all respects, the record at this plant reflected the skill and ability of the operating staff; the grounds about the plant were maintained in an unusually attractive condition. Until April, 1943, sewage at

Camp Myles Standish was treated in a settling lagoon and chlorinated with temporary post-made apparatus. In April, the settling units of the new plant were in operation and in June, 1943, complete biological treatment commenced.

All but 15 percent of the flow came to the plant by gravity. Pumping from two lift stations, however, was intermittent in the extreme, and occasional peak flows of stale sewage were experienced. Infiltration was negligible. Grease was excessive at the time of the survey; during some weeks as much as 16,000 lb of scum were removed. Corrective measures, particularly at mess-halls, were effective and reduced the grease load by approximately two-thirds. Grease in the primary effluent was normally less than 75 ppm. No regular laundry wastes were discharged to the sewers.

It is worthwhile to describe some of the modifications that were effected by the operating staff. These were necessitated by minor imperfections in design and construction. It is not to be inferred from the following discussion that the situation at Myles Standish was unusual; the plant was considered to be well designed. In all plants, however, with the haste and urgency of wartime construction, a number of errors of commission and omission were were inevitable. The following were typical:

1. As constructed, the No. 1 diversion chamber at the head of the plant received recirculated filter effluent and secondary sludge on one side, while on the other the supernatant was returned. Mixing was inaedquate and as a result, supernatant flowed through the east half of the plant, whereas most of the recirculated flow passed through the west side. Therefore the filters, which operated in parallel, were loaded differently. By extending the supernatant pipe to the center of the chamber and by placing a 45° sleeve on the discharge end of the recirculation line, an even distribution of the various fluids was effected.
2. In the original installation, both secondary sludge lines discharged into a common 4-inch pipe, and thence into the 12-inch recirculation line. To correct this, the sludge pipe from the west secondary settling tank was changed to deliver into the primary recirculation wet well.
3. As originally constructed, the concrete inlet wall to the recirculation wet well was somewhat too high. This resulted in effluent leaving the plant during extremely low flows instead of being returned ahead of the filters to effect a constant high rate of dosage. To remedy the situation, the wall between No. 4 diversion chamber and the recirculation well was cut through, thereby permitting full flow to the well.
4. Some trouble was experienced with tipping of distributor arms on the filters at seasons of the year when large variations occurred in diurnal temperature. The cause was unequal tension of guy rods on the arms. The only recourse was to adjust turnbuckles as frequently as required.
5. At the time of construction, steel was a critical material and plywood was substituted wherever possible. Most of the control gates throughout the plant were of ¾-inch plywood. Weirs, too, were constructed of wood. These split at the laminations within an 8-month period and had to be replaced with metal weirs. Control gates were reinforced with steel strips bolted together so as to prevent splitting.

Aluminum sulfate was added in varying amounts to digested sludge before application to sand drying beds. Alum in powder form was dosed at sludge shear gates. Optimum dosage was 400 ppm. Drying time on the average was reduced from 10 to 14 days to 4 to 5 days. Solids were buoyant and drainability was enhanced. Cracking of sludge cake was noticed sometimes as early as the second day. One difficulty experienced with alum was that if the cake was not removed immediately after drying, it became difficult to put through the grinding machine.

Activated carbon was tried on an experimental basis as an aid in reducing drying time. However, it was not found to improve appreciably the drainability of digested sludge.

In Figure 81 are presented data relating to loading and performance of the Myles Standish plant during the period of operation investigated. In general, the performance was uniform and rather independent of loading. A sharp increase in BOD in February, 1944 was handled without perceptible effect upon the quality of the effluent. However, it may be noted that the New England winter of 1943–44 induced some deterioration in performance.

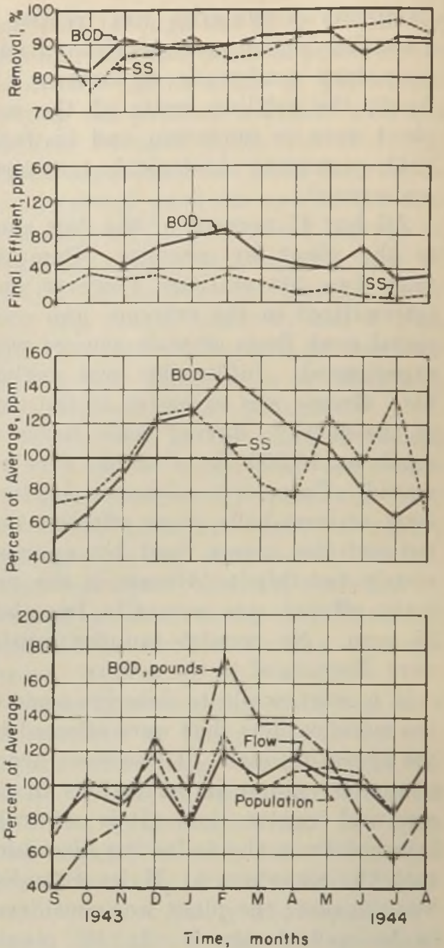


FIGURE 82.—Loading and performance of trickling filters at Camp Myles Standish.

Forts Sill and Warren

The data for the first stage of filtration at Forts Sill and Warren have been included in tables of Division III for purposes of comparison. Detailed discussion of the plants at these posts is presented in the following section.

IV. Two-Stage Filter Plants

In Tables 67 to 84 an analysis of loading and performance at nine two-stage trickling filter plants is presented. These installations could not be treated as a homogeneous group since considerable differences in design are exhibited. Almost every conceivable scheme of recirculation (Table 74) was represented. Loading varied over a wide range. Some plants utilized intermediate settling between the first and second stage of filtration, others did not. Indeed, the only feature of design common to all plants was the arrangement of filter units in series.

No attempt was made *à priori* to set up design criteria to establish the capacity of filter units. Other units, including intermediate settling tanks, were rated in accordance with design

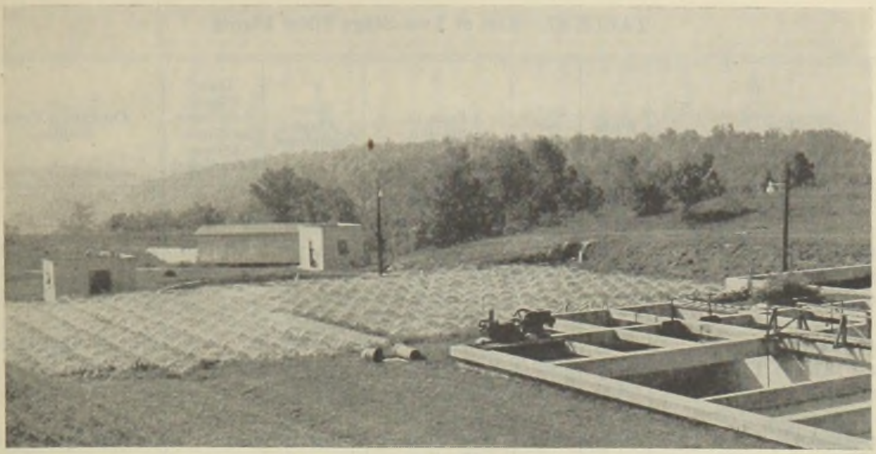


PLATE 15.—Trickling filter at Fort Knox, Ky.

PLATE 16.—Aeration ladder for plant effluent, Fort Knox, Ky.

TABLE 67.—List of Two-Stage Filter Plants

Post	State	Sv C or Navy	Type of Station	Date Plant Completed	Date Major Additions or Alterations Completed	Operating Period Studied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Camp Forrest(a)	Tenn	4th	Div Trg	Oct 42	Sep 43	Sep 43 to Aug 44
Camp Gordon(b)	Ga	4th	Rep Trg	Jan 42	Jun 43	Jan 44 to Aug 44
Fort Riley	Kans	7th	Arm, Mc & TD Trg	Sep 43	—	Sep 43 to Aug 44
Camp Roberts	Calif	9th	Rep Trg	Mar 41	—	Jan 43 to Aug 44(c)
Camp Joseph T Robinson	Ark	8th	Rep Trg	Oct 40	Mar 43	Jun 43 to Sep 44
Fort Sill	Okla	8th	Art, AA & Rep Trg	Oct 41	—	Jan 43 to Aug 44
Topeka AAB	Kans	7th	Tact Field & Fly Trg	Nov 42	—	Apr 43 to Dec 43
Fort Francis E Warren	Wyo	7th	Serv Trp Trg	Jan 42	Jan 43	Jan 43 to Aug 44
Willow Grove	Pa	II	—	Oct 42	—	Jul 43 to Aug 44

(a) Two-stage filtration after August, 1943.
(b) Two-stage filtration after December, 1943.
(c) Omitting March and April, 1943.

TABLE 68.—General Data Relating to Two-Stage Filter Plants

Post	Approx Plant Elev (Ft)	Range of Sewage Influent Temp (°F)	Average Sewage Influent pH	Sampling	
				Hours per Day	Average Number per Month
(1)	(2)	(3)	(4)	(6)	(7)
Forrest	—	50–86	7.2	24	30
Gordon	—	56–86	7.1	24	30
Riley	—	57–71	7.4	24	8
Roberts	—	66–94	7.9	24	30
J T Robinson	460	58–85	—	24	—
Sill	—	61–85	7.5	24	9
Topeka AAB	—	50–80	7.4	24	10
F E Warren	6,100	52–71	7.6	24	9
Willow Grove	320	43–88	8.0	24	30

TABLE 69.—Units in Two-Stage Filter Plants

Post	Plant Units											
	Screens or Racks	Comminutors	Grit Chambers	Imhoff Tanks	Primary Settling Tanks	Filters	Secondary Settling Tanks	Chlorine Contact Tanks	Digestion Tanks (Heated)	Digestion Tanks (Unheated)	Sludge Beds	Other Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Forrest	1	0	0	0	3	5(b)	3	1	2	1	18	(a)
Gordon	2	1	1	0	2	4(c)	2	1	2	1	5	
Riley	1	1	0	0	1	2(d)	1	0	1	0	6	
Roberts	1	0	0	0	1	2(d)	1	1	1	1	8	(e)
J T Robinson	2	0	0	0	3	2(d)	2	0	1	1	18	(f)
Sill	1	0	1	0	1	4(c)	1	0	1	0	10	(g) (h)
Topeka AAB	1	0	0	0	1	2(d)	1	1	1	0	4	
F E Warren	2	1	1(i)	0	1	3(j)	1	1	2	0	6	(h)
Willow Grove	1	0	0	0	2	2(d)	1	1	1	0	6	

(a) Four intermediate settling tanks.
(b) Three 1st-stage filters and two second-stage.
(c) Two 1st-stage filters and two second-stage.
(d) One 1st-stage filter and one second-stage.
(e) Step aerator for final effluent.
(f) Two aeration tanks for grease removal.
(g) Triturator and grease flotation tank.
(h) One intermediate settling tank.
(i) Combination grit removal and grease flotation by air.
(j) Two 1st-stage filters and one second-stage.

TABLE 70.—Capacities and Loading of Two-Stage Filter Plants

Post	Number of Months in Operating Period Studied	Population			Flow			
		Actual			Actual			
		Average	Maximum	Minimum	Average (mgd)	Maximum (mgd)	Minimum (mgd)	Gal per Capita Daily
(1)	(2)	(4)	(5)	(6)	(8)	(9)	(10)	(11)
Forrest	12	18,400	30,800	10,100	2.200	4.240	0.990	115
Gordon	8	25,400	26,500	24,300	1.800	3.580	0.460	71
Riley	12	4,900	5,920	3,980	0.504	0.940	0.240	102
Roberts	18	33,200	38,400	28,600	1.880	4.050	0.420	57
J T Robinson	16	28,600	39,900	22,300	1.680	—	—	59
Sill	20	40,200	51,700	33,000	2.820	4.20	1.30	70
Topeka AAB	9	4,200	5,250	3,570	0.367	0.495	0.156	88
F E Warren	20	10,300	14,300	6,250	0.760	1.420	0.350	74
Willow Grove	14	1,080	1,320	810	0.123	—	—	114

TABLE 71.—Percent of Design Capacity Utilized in Two-Stage Filter Plants

Post	Primary Settling Tank (%)	Intermediate Settling Tank (%)	Secondary Settling Tank (%)	Sludge Digester (%)	Sludge Beds (%)
(1)	(2)	(4)	(4a)	(5)	(6)
Forrest	68	146	69	51	43
Gordon	60	—	60	56	51
Riley	56	—	56	29	33
Roberts	119	—	123	130	32
J T Robinson	64	—	44	62	69
Sill	148	236	157	144	110
Topeka AAB	106	—	96	59	44
F E Warren	62	235	145	52	46
Willow Grove	61	—	76	26	45

TABLE 72.—Design and Description of *First-Stage Filters* in Two-Stage Filter Plants

Post	Number of Filters	Dimensions—Length by Width or Diameter (ft)	Area (acres)	Volume (af)	Filter Media		
					Depth (ft)	Type	Size (in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Forrest	3	70d	0.265	1.66	6.25	—	2¼–4
Gordon	2	130d	0.608	3.65	6.0	Granite	2¼–4
Riley	1	75d	0.101	0.404	4.0	Limestone	1½–3
Roberts	1	90d	0.146	0.438	3.0	Trap Rock	1–3½
J T Robinson	1	135d	0.329	1.40	4.25	Quartzite	1½–6
Sill	2	85d	0.260	0.845	3.25	—	1–3
Topeka AAB	1	51d	0.047	0.188	4.00	Limestone	1½–2½
F E Warren	2	50d	0.09	0.27	3.00	River Gravel	1½–2½
Willow Grove	1	38d	0.026	0.078	3.00	Granite	1–2½

TABLE 73.—Design and Description of *Second-Stage Filters* in Two-Stage Filter Plants

Post	Number of Filters	Dimensions—Length by Width or Diameter (ft)	Area (acres)	Volume (af)	Filter Media		
					Depth (ft)	Type	Size (in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Forrest	2	163d	0.960	6.00	6.25	—	2¼–4
Gordon	2	130d	0.608	3.65	6.00	Granite	2½–4
Riley	1	75d	0.101	0.404	4.00	Limestone	1½–3
Roberts	1	90d	0.146	0.438	3.00	Trap Rock	1–3½
J T Robinson	1	145 x 295	0.982	5.89	6.00	Quartzite	1½–6
Sill	2	130d	0.608	3.05	5.00	—	1–3
Topeka AAB	1	51d	0.047	0.188	4.00	Limestone	1½–2½
F E Warren	1	108d	0.210	1.680	8.00	Granite	2–3
Willow Grove	1	38d	0.026	0.078	3.00	Granite	1–2½

TABLE 74.—BOD Loading and Performance of Two-Stage Filter Plants

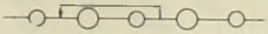
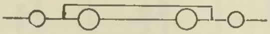
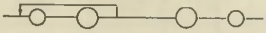
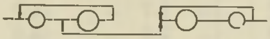
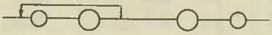
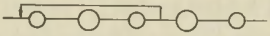
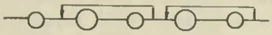
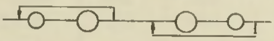
Post	Diagram	Plant Influent		
		Flow (mgd)	BOD	
			PPM	Pounds per Capita Daily
(1)	(2)	(3)	(4)	(5)
Forrest		2.197	215	0.214
Gordon		1.796	397	0.235
Riley		0.502	244	0.209
Roberts		1.878	378	0.178
J T Robinson		1.680	358	0.175
Sill		2.820	305	0.179
Topeka AAB		0.367	210	0.154
F E Warren		0.759	429	0.264
Willow Grove		0.123	144	0.137

TABLE 75.—BOD Loading and Performance of Two-Stage Filter Plants

Post	Pop'n per Acre-Foot	Ratio First-Stage Filter to Total Filter		Ratio Second-Stage Filter to Total Filter		Recirculation			
		Area	Volume	Area	Volume	R_1 (mgd) (a)	Ratio R_1 to Plant Inf	R_2 (mgd) (b)	Ratio R_2 to Plant Inf
(1)	(24)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Forrest	2,400	22	22	78	78	2.858	1.30	—	—
Gordon	3,470	50	50	50	50	2.88(c)	1.60(c)	—	—
Riley	6,070	50	50	50	50	1.180	2.35	1.180	2.35
Roberts	37,900	50	50	50	50	4.538	2.41	4.773	2.54
J T Robinson	3,930	25	19	75	81	1.28	0.76	—	—
Sill	10,300	30	22	70	78	1.44	0.51	—	—
Topeka AAB	11,200	50	50	50	50	0.450	1.22	0.492	1.34
F E Warren	5,280	30	14	70	86	2.108	2.78	0.492	0.65
Willow Grove	6,930	50	50	50	50	0.259	2.10	0.299	2.43

(a) First-stage recirculation.
(b) Second-stage recirculation.
(c) Recirculation of filter effluent to filter influent.

TABLE 76.—BOD Loading and Performance of Two-Stage Filter Plants

Post	Primary Treatment Section					
	Flow (mgd)	Detention Period (hr)			BOD	
		At Average Flow	At Maximum Flow	At Minimum Flow	Influent (ppm)	Removal (%) (a)
(1)	(15)	(16)	(17)	(18)	(19)	(21)
Forrest	2.197	3.7	1.9	8.2	215	25.2
Gordon	1.796	4.2	2.07	8.1	397	35.1
Riley	1.682	4.5	3.5	5.3	121	25.9
Roberts	6.416	2.1	1.6	2.7	169	9.3
J T Robinson	2.960	3.9	—	—	239	27.8
Sill	4.260	1.7	1.7	1.7	231	24.3(b)
Topeka AAB	0.817	2.8	2.5	3.8	126	20.2
F E Warren	0.760	4.0	2.1	8.6	429	15.3
Willow Grove	0.382	4.1	—	—	73	48.0

(a) Calculated in accordance with Equation (3).
(b) Calculated in accordance with Equation (4).

TABLE 77.—BOD Loading and Performance of Two-Stage Filter Plants

Post	Secondary Treatment Section — First Stage Filtration and Settling												
	Filter Loading					Performance							
	Flow to Filter		BOD			Detention Period (hr)			BOD				
	MGD	MGAD	Inf (ppm)	LB/AFD (a)	LB/AD (a)	At Av Flow	At Max Flow	At Min Flow	Eff (ppm)	LB/AFD Removed	Efficiency (%) (a)	Removal (%)
(1)	(22a)	(23a)	(25a)	(26a)	(27a)	(32a)	(33a)	(34a)	(36a)	(37a)	(38a)	(39a)
Forrest	5.055	19.06	—	1,780	11,500	1.70	1.21	2.24	—	—	—	—
Gordon	1.796	2.96	258	1,060	6,370	—	—	—	—	—	—	—
Riley	1.682	16.66	92	1,880	7,520	—	—	—	69	1,165	62.0	45.9
Roberts	4.538	31.10	149	8,680	26,100	—	—	—	148	6,980	80.4	51.7
J T Robinson	2.960	8.98	167	2,600	11,030	—	—	—	81	1,780	68.8	49.8
Sill	4.260	16.38	175	6,440 (b)	20,900 (b)	1.06	1.06	1.06	85	4,070	63.0 (b)	47.8
Topeka AAB	0.817	17.36	86	2,730	10,900	—	—	—	57	1,794	65.8	52.4
F E Warren	2.867	31.9	221	8,520	25,500	1.06	0.86	1.24	181	4,300	50.5	42.7
Willow Grove	0.382	14.70	40	988	2,960	—	—	—	41	448	45.5	23.7

(a) Calculated in accordance with Equation (3).
(b) Calculated in accordance with Equation (4).

TABLE 78.—BOD Loading and Performance of Two-Stage Filter Plants

Post	Secondary Treatment Section								
	Second Stage Filtration and Settling								
	Filter Loading					Performance			
	Flow to Filter		BOD			Detention Period (hr)			
						Settling Tank			Contact Tank at Av Flow
	MGD	MGAD	Inf (ppm)	LB/AFD (a)	LB/AD (a)	At Av Flow	At Max Flow	At Min Flow	
(1)	(22b)	(23b)	(25b)	(26b)	(27b)	(32b)	(33b)	(34b)	(35)
Forrest	2.200	2.29	—	—	—	3.60	1.86	7.99	0.35
Gordon	4.680	7.69	—	—	—	4.14	2.08	8.1	0.29
Riley	1.684	16.7	45	714	2,850	4.48	3.54	5.29	—
Roberts	6.653(b)	45.6(b)	70(b)	5,280(b)	15,800	2.02	1.52	2.59	0.55
J T Robinson	1.680	1.71	81	192	1,150	5.64	—	—	—
Sill	2.820	4.64	86	658	3,290	1.59	1.07	3.45	—
Topeka AAB	0.859	18.3	33	930	3,720	2.72	2.37	3.64	0.39
F E Warren	1.252	5.97	133	678	5,430	1.72	1.13	2.55	0.60
Willow Grove	0.422	16.2	20	910	2,730	3.27	—	—	1.14

(a) Based upon effluent of first-stage filtration.
(b) Includes settled raw sewage.

TABLE 79.—BOD Loading and Performance of Two-Stage Filter Plants

Post	Secondary Treatment Section							Over-all Plant Removal (%)	Final Effluent	
	Second Stage Filtration and Settling				Combined First and Second Stage				Final Dissolved Oxygen (ppm)	Relative Stability (%)
	Performance				Performance					
	Final Eff (ppm)	LB/AFD Removed	Efficiency (%)	Removal (%)	LB/AFD Removed (a)	Efficiency (%) (a)	Removal (%) (a)			
(1)	(36)	(37b)	(38b)	(39b)	(37)	(38)	(39)	(40)	(43)	(44)
Forrest	15	—	—	—	349	90.8	68.1	93.3	2.9	93
Gordon	48	—	—	—	432	81.7	53.1	88.2	5.3	—
Riley	34	364	51.1	14.3	762	81.6	60.2	86.1	5.4	—
Roberts	53	3,370(c)	64.0	24.8	5,170	84.5	76.5	85.8	1.2	—
J T Robinson	27	128	67.1	15.0	447	89.8	64.8	92.6	2.7	90
Sill	26	455	69.5	19.4	1,232(b)	88.8(b)	67.2(b)	91.5	3.2	91
Topeka AAB	15	687	73.8	20.1	1,240	91.1	72.6	92.8	5.4	94
F E Warren	66	431	63.6	26.8	967	82.0	69.5	84.8	4.0	—
Willow Grove	11	397	73.8	20.9	423	85.7	44.6	92.6	—	—

(a) Calculated in accordance with Equation (3).
(b) Calculated in accordance with Equation (4).
(c) Influent includes settled raw sewage. (See design flow).

TABLE 80.—SS Loading and Performance of Two-Stage Filter Plants

Post	Plant Influent		Primary Treatment Section		Secondary Treatment Section — First Stage Filtration and Settling		
	PPM	Pounds per Capita Daily	Effluent (ppm)	Removal (%) (a)	Effluent (ppm)	Efficiency (%) (a)	Removal (%) (a)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Forrest	217	0.216	102	52.8	—	—	—
Gordon	278	0.166	120	56.8	—	—	—
Riley	340	0.292	85	56.3	89	39.9	17.4
Roberts	299	0.141	109	32.9	108	65.3	31.1
J T Robinson	197	0.097	86	34.5	61	52.7	34.6
Sill	273	0.160	95	51.7(b)	55	58.6(b)	28.3(b)
Topeka AAB	—	—	—	—	—	—	—
F E Warren	269	0.166	210	22.0	96	54.2	42.2
Willow Grove	192	0.182	—	—	—	—	—

(a) Calculated in accordance with Equation (3).
(b) Calculated in accordance with Equation (4).

TABLE 81.—SS Loading and Performance of Two-Stage Filter Plants

Post	Secondary Treatment Section — Second Stage Filtration and Settling			Secondary Treatment Section — Combined First and Second Stage		Overall Plant Removal (%)
	Final Eff (ppm)	Efficiency (%) (a)	Removal (%) (a)	Efficiency (%) (b)	Removal (%) (b)	
(1)	(9)	(10)	(11)	(12)	(13)	(14)
Forrest	23	—	—	84.3	39.7	92.5
Gordon	16	—	—	80.9	34.9	91.7
Riley	45	49.6	13.0	69.8	30.4	86.7
Roberts	54	49.8	17.9	73.3	49.1	81.9
J T Robinson	21	65.7	20.2	83.8	54.8	89.3
Sill	21	60.6	12.1	83.7(c)	40.4(c)	92.1
Topeka AAB	20	—	—	—	—	92.7
F E Warren	36	62.9	22.5	82.9	64.7	86.7
Willow Grove	10	—	—	88.8	40.1	94.9

(a) Based upon effluent of first-stage filtration.
(b) Calculated in accordance with Equation (3).
(c) Calculated in accordance with Equation (4).

TABLE 82.—Sludge Data Relating to Two-Stage Filter Plants

Post	Number Months in Operating Period Studied	Sludge Digester					Schedule of Sludge Loading	
		Total CF per Capita (a)	Heated, Percent of Total Volume	Covered, Percent of Total Volume	Average Operating Temperature (°F)	Source of Sludge	Secondary to Primary Settling Tanks—Times per Day	Primary to Sludge Digesters—Times per Day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Forrest	12	7.0	56	56	75–91	Pri(b) Inter Set.	3(c)	6
Gordon	8	6.0	67	100(d)	88–90	Pri	—	8
Riley	10	10.2	100	100	88	Pri(b)	Cont	12
Roberts	20	2.8	50	100	95	Pri(b)	Cont	148
J T Robinson	16	5.8	52	100(e)	85–90	Pri(b)	Cont	48
Sill	20	2.1	100	100	88	Pri(b)	Cont	20
Topeka AAB	9	5.0	100	100	90–92	Pri	Cont	12
F E Warren	20	5.8	100	100(f)	86–95	Pri(b)	Cont	48
Willow Grove	14	11.5	100	100	85–95	Pri(b)	12	12

(a) Based on actual population, Table 70, col (4).
(b) Includes scum and grease.
(c) Sludge pumped to intermediate settling tank.
(d) Two floating covers and one fixed wooden cover.
(e) One floating and one fixed wooden cover.
(f) Tray digester.

TABLE 83.—Sludge Data Relating to Two-Stage Filter Plants

Post	Gallons Raw Sludge per MG Sewage	Volatile Solids, Percent of Total Solids		Sludge Disposal			
		Raw Sludge	Digested Sludge	SF per Capita (a)	No Months Sludge Drawn per Year	Sludge Drawn Yearly from Digesters (cf/cap)	Sludge Removed Yearly from Beds (cf/cap)
(1)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Forrest	6,580	65.1	51.5	2.34	11	6.9	2.2
Gordon	12,000	79.5	65.0	1.97	12	5.5	2.4
Riley	4,660	54.9	—	3.20	—	—	—
Roberts	6,280	75.3	60.8	3.13	12	7.6	1.4
J T Robinson	10,000	74.0	—	1.50	12	4.6	1.7
Sill	5,900	79.0	—	—	—	—	—
Topeka AAB	5,450	73.6	46.0	2.29	9	5.7	1.2
F E Warren	4,880	84.4	76.7	2.17	10	4.1	1.1
Willow Grove	—	—	—	2.24	—	—	—

(a) Based on actual population, Table 70, col (4).

TABLE 84.—Chlorination and Effluent Disposal in Two-Stage Filter Plants

Post	Chlorination		Receiving Water Body	
	Points of Application Used	Residual Chlorine (ppm)	Name	Type
(1)	(2)	(3)	(4)	(5)
Forrest	—	—	—	Dry Run
Gordon	—	—	Spirit Creek	Flowing Stream
Riley	—	—	Kansas River	River
Roberts	Post	0.06	Salinas River	Flowing Stream
J T Robinson	—	—	Five Mile Creek	Dry Run
Sill	—	—	Cache Creek	Flowing Stream
Topeka AAB	Post	0.40	—	Dry Run
F E Warren	—	—	Crow Creek	Flowing Stream
Willow Grove	Post	0.36	Park Creek	Flowing Stream

factors used in pervious sections. The percent of design capacity of these units is presented on Table 71.

In order to establish the general distribution of load among the various units of two-stage filter plants, the

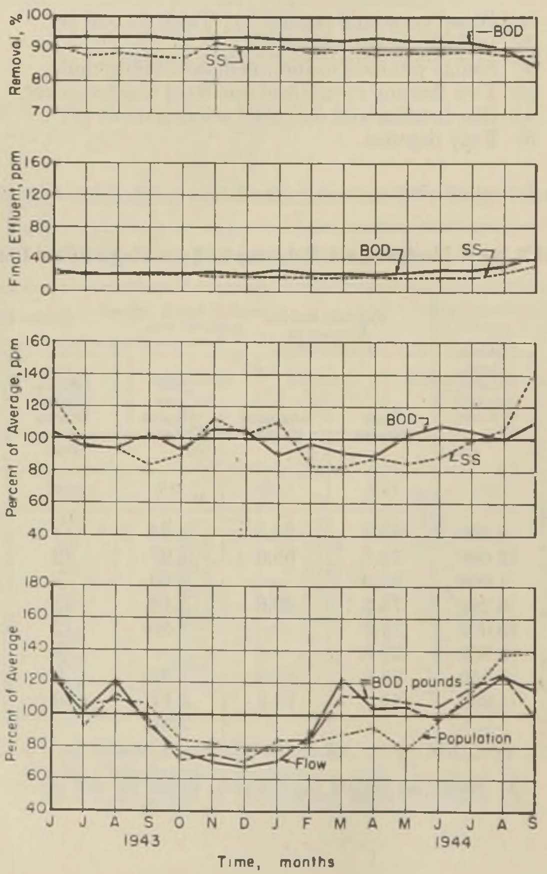


FIGURE 83.—Loading and performance of two-stage filters at Camp Joseph T Robinson.

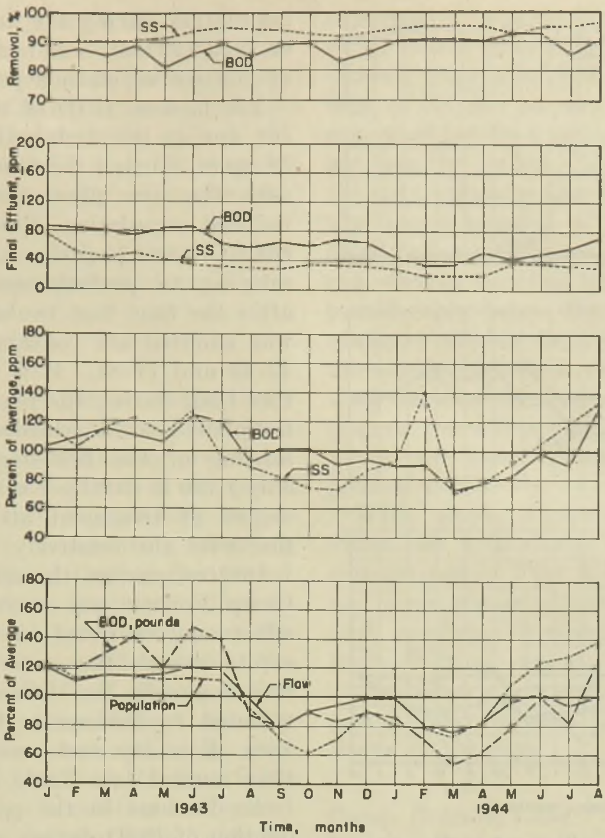


FIGURE 84.—Loading and performance of two-stage filters at Fort Francis E Warren.

following table is presented:

	Average Percent Efficiency		Average Percent Removal	
	BOD	SS	BOD	SS
Primary Section	26	44	26	44
First-Stage Filtration*	62	54	44	31
Second-Stage Filtration*	66	58	20	17
Combined First and Second-Stage Filtration*	86	81	64	44
Overall Plant Removal	90	90	90	90

* Including subsequent settling, if any.

It is evident that a high degree of treatment was attained in two-stage filter plants. The quality of the effluent from these plants was comparable to that from standard rate filters and was distinctly superior to that from single-stage high rate filters.

It will be noted that the average BOD and SS removed in the first stage of filtration was about twice that removed in the second stage. The average removal in the primary section was about in line with that observed in other single-stage filter plants employing recirculation. In general, the percent removals of the various units should be additive to the overall plant removal in accordance with the definition of percent removal, as defined in Equation (2). Owing to the fact, however, that the averages in the foregoing table were computed from groups containing different numbers of plants (in some plants the performance of each stage of filtration could not be computed separately), the additive rule does not hold exactly.

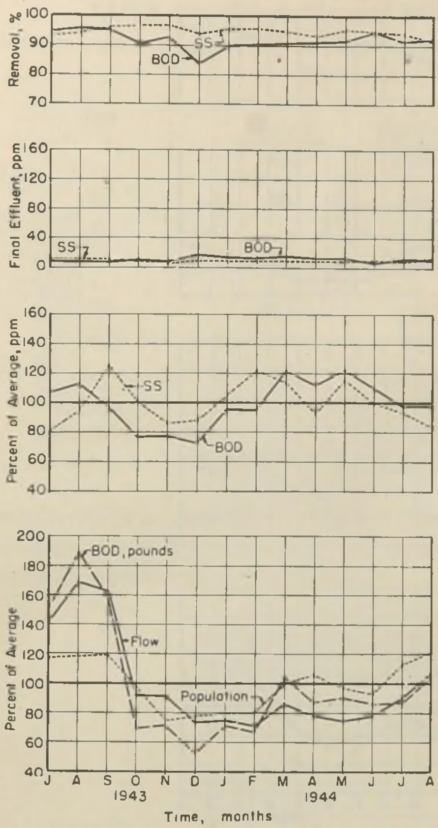


FIGURE 85.—Loading and performance of two-stage filters at Willow Grove NAS.

Performance of Individual Plants of Division IV

Camp Forrest, Tenn.

A considerable variation of population occurred at Camp Forrest during the twelve months of operation investigated. The plant at this post was enlarged two times. The first addition, which was placed in operation in March, 1941, consisted of one primary settling tank, one high rate filter, and two secondary settling tanks. A second addition consisting of two standard filters, three final settling tanks, a secondary digester, and nine sludge drying beds was completed in August, 1943. The expected increase in population for which plant additions were installed did not materialize; in fact the population decreased. The median population during the first six months of 1943 (29,000) was nearly twice that of the corresponding period of 1944.

The median BOD of the plant effluent during the first half of 1943 was 58 ppm. During the first half of 1944, with the new plant units and with reduced population, the median effluent BOD was 14 ppm. Additional data relating to performance before and after the time that two-stage filtration was adopted are contained on Tables 47–49 and 77–81. It is evident therefore that, during the period of survey, the plant was underloaded; the organic loading of the first-stage filters was only 1,780 lb BOD per acre-ft. The high degree of treatment attained was in line with the relatively light loading.

Infiltration into the sewer system at Camp Forrest was excessive and the effect may be noted in the large per capita sewage flow. Grease at times was excessive, the difficulty being attributed to inadequate trap maintenance. Laundry wastes also were larger than normal (see Table 70).

An increase in the per capita production of BOD during winter months was evident at Camp Forrest. This phenomenon, which was noted at a number of other divisional training posts, was attributed to decreased amounts of bivouacking during colder months. Lack of analytical data unfortunately made it impossible to analyze completely the performance of first- and second-stage filters.

A number of schemes of recirculation were used in connection with the first stage of filtration. These included recirculation (i) from effluent of secondary settling tank to filter influent; (ii) from filter effluent to filter influent; (iii) from filter effluent to primary settling tank influent. The flow diagram indicated in Table 74 indicates but one of the recirculation schemes used. No recirculation was used about the second-stage filters.

Despite a relatively large digester capacity at Camp Forrest, it was necessary occasionally to lime the sludge when the pH fell below 7.0. The quality of supernatant, however, was generally satisfactory; it ran 2,000 to 3,000 ppm of total solids with a pH in the range 6.8 to 7.5 from the primary digester and 8.0 to 11.0 from the secondary digester.

The operation of the plant at Camp Gordon has been discussed in a previous section.

Fort Riley, Kan.

Performance of the two-stage trickling filter plant at Fort Riley was erratic during the operating period investigated. While grease in the plant influent was negligible, laundry wastes were excessive, constituting about a third of the flow. During the period of 12 months from September, 1943 to September, 1944, the population at the post declined from 5,302 to 4,237; the ppm BOD of the final effluent varied from 78 to 12 as based upon monthly averages. The settling tanks and sludge handling units at the Fort Riley plant were large in relation to the load applied, as may be seen from Table 71. Detention periods were long, and removals in settling were only fair.

While the filter medium was, according to specifications, to be 1½ in. to 3 in. in size, an inspection indicated that some smaller sizes of stone were present. It was not determined whether the smaller aggregate was due to disintegration. However, clogging and pooling occurred on three occasions during the summer of 1944. On each occasion flooding remedied the difficulty. Unloading of filters was noted on one occasion in April, 1944. The end gates of the filter distributor arms were opened daily and a hose inserted in order to clean the orifices.

The two filters at Riley, which were operated in series, were identical as to design and construction. Removal of BOD in the primary filter (45.9%) was more than three times that in the secondary filter (14.3%). As would be expected, a much smaller disparity was noted in the efficiencies of these units. Overall removal of BOD was 86.1 percent, of SS 86.7 percent. This figure is somewhat low for a two-stage filter plant not heavily loaded. A breakdown of the data indicates that during much of the time a removal of BOD in excess of 90 percent occurred. However, during certain months performance fell considerably below the average. For example, during October, 1943, when the sludge line from the primary settling tank to the sludge pump became clogged for a few days with sand and weeds, the plant effluent averaged 78 ppm of BOD.

With ample digester capacity, no difficulties with sludge digestion were encountered at Fort Riley except during initial phases of operation when an acid condition required addition of lime. Dried digested sludge was ground and used as fertilizer at the post nursery and on flower beds and lawns on the post.

Camp Roberts, Calif.

The two-stage filter plant at Camp Roberts accomplished an extraordinary amount of removal of organic matter in filters that were heavily loaded. Infiltration was moderate, occurring principally during the rainy season. Grease was not excessive at the time of the survey. A program of instruction for mess-hall personnel was effective in improving maintenance of grease traps. Laundry wastes were present but not excessive. Sewage temperature at Camp Roberts, which was situated in southern California, was higher than at other two-stage filter plants, and it is likely that this was a significant factor in the attainment of high efficiency in the biological units. The population loading of the plant during the 18 months of operation investigated was fairly uniform, but diurnal variation in flow at the plant was marked. The per capita flow was low and sewage was somewhat more concentrated than at most two-stage filter plants.

Settling tanks were loaded in excess of capacity specified by the Engineering Manual, having detention periods only slightly more than two hours. Performance of primary settling tanks (Table 76) was poor, particularly with respect to the removal of BOD.

Filter loading at Camp Roberts was the highest of all trickling filter plants surveyed. Nearly three times the BOD load specified in the Engineering Manual was applied to the primary filter. It should be noted that the recirculation scheme utilized at this plant (Table 74) placed the two filters partly in series and partly in parallel, the ratio depending upon plant flow. Thus, during periods of peak flow, the secondary filter received a considerable proportion of settled raw sewage. Theoretically, this recirculation scheme is particularly effective in bringing about an optimum distribution of the daily organic load between the first- and second-stage filter in a two-stage plant. Results corroborate this conclusion. The filters were of the same size and shape. The recirculation ratio was about the same for both filters. The first-stage filter, however, effected about twice as much BOD removal as the second.

As may be seen from Tables 80 and 81, both filters were considerably less effective in removal of SS as compared to BOD. Dissolved oxygen in the effluent of this heavily loaded plant averaged 1.2 ppm.

Prechlorination for odor control was practiced. An ammoniacal odor was noticeable from the filter units. The trap-rock media in the filter had the following size distribution (by weight): 5 percent retained on 3 in. passing through 3½-in. mesh; 1 percent (not more than) passing through 1-in. mesh.

No serious operating difficulties pertaining to sludge digestion or disposal were encountered at Camp Roberts. The primary digester was maintained at 95° F and the secondary digester, which was unheated, attained a summer temperature of 86° F. Agitation equipment operated 24 hours daily. Lime was used only during the initial phases of operation. Digested sludge was disposed of upon drying beds. Lagoons, originally provided, were converted into sludge drying beds by filling in with sand without provision for drainage. This arrangement proved to be unsatisfactory and tile underdrains were installed. The effluent discharged over a step-aerator into Salinas River, which was virtually dry during four or five months of the year.

Camp Robinson, Ark.

At Camp Joseph T. Robinson, a World War I standard trickling filter without final settling was modernized to form a two-stage plant. Primary settling tanks consisted of (1) two old rectangular Doten tanks, revamped and equipped with sludge collecting mechanism and (2) a modern, circular tank with usual appurtenances. The first-stage filter was a new shallow high rate unit with a rotary distributor. Media consisted of blue quartzite with an admixture of slate and ranged in size from 1½ in. to 6 in. The second-stage low rate filter, which dated from World War I days, was rectangular in plan and was dosed with fixed nozzles. This unit was divided into two equal sections, each supplied by a twin alternating dosing tank. At 2.0 mgd the dosing tank filled in 8 minutes and discharged in 3 minutes. The medium was similar to that of the first-stage filter. As indicated on the diagram of Table 74, recirculation was provided only at the first-stage filter.

Practically all of the sewage reached the plant by gravity. Sewer gradients were moderate and infiltration not excessive. The sewage contained an excessive amount of grease. Laundry waste comprised about 5 percent of the total flow. Grease was removed in two separation tanks with mechanical down-draft aerators. The detention period was about 20 minutes, as based upon average flow. The units performed satisfactorily, removing a high percentage of the grease.

The settling tanks were underloaded with respect to criteria stated in the Engineering Manual. Removal in primary settling was only fair. Efficiency of the first-stage filter was 68.8 percent, as based upon reductions occurring in the filter itself as well as that portion of the primary settling tank considered as part of the secondary treatment section. The efficiency of the first stage of filtration would, of course, have been notably improved if an intermediate settling tank had been provided. As based upon actual performance of the final settling tanks, it was estimated that the efficiency of the first stage of filtration with intermediate settling would have been 82.5 percent.

The second-stage low rate filter at Camp Robinson effected the reduction of 67.1 percent of the applied load of BOD. This high efficiency, however, must in part be attributed to action of the first-stage filter, since there was no intermediate settling to remove solids agglomerated by the initial unit. The efficiency of the combined first- and second-stage filters for BOD was 89.8 percent and was 83.8 percent for SS. The final effluent had an average value of 27 ppm BOD and 21 ppm SS.

In Figure 83 are presented data pertaining to loading and performance at Camp Robinson during the period of operation investigated. Performance was remarkably uniform despite fluctuations in loading.

The efficiency of the plant at Camp Robinson was impaired by poor digester supernatant which was returned to the head of the plant. The volume of supernatant returned averaged 17,000 gpd. A heavy scum layer in the first-stage digester was controlled by raising the temperature sometimes as high as 100° F. At the time of visit, a line to carry supernatant from the first-stage to the second-stage digester was being installed, together with modifications that would permit settling of supernatant in one-half of the second-stage digester before return to the plant inlet. Due to an excessive organic content of liquor draining from sludge drying beds it was necessary to install means for returning it to the raw sewage.

Fort Sill, Okla.

The two-stage trickling filter plant at Fort Sill had a remarkable record of performance. It successfully treated sewage from a considerably larger population than was provided for in the original design. Sewer gradients were moderate and infiltration normal; sewage arrived at the plant in a fresh condition. At the time of the survey, the amount of grease in the raw sewage was small. Laundry waste constituted about 9 percent of the total sewage flow. A holding basin for laundry waste was installed so that it could be discharged to the plant at a uniform rate over a six-hour period starting at midnight. This flow aided in maintaining continuous operation of the trickling filters.

One defect in the data pertaining to the plant at Fort Sill should be noted, although it would not appear to be important. No regular samples were taken of raw sewage, but rather the strength of plant influent was assumed to be that of the primary influent. Numerous composited samples of raw sewage and primary tank influent indicated that differences in concentration at these sampling points were so small as to be negligible. Accordingly, for convenience, only samples of primary plant influent were taken regularly.

The per capita production of BOD and SS based upon the concentration of samples of primary influent and actual plant flow was normal (Tables

74 and 80); hence no large error in estimation of strength of raw sewage was entailed in the procedure adopted at the plant. Performance of the primary settling tank was fair. Some improvement in primary performance resulted from installation of a pump to control recirculation from the final clarifier to the raw sewage wet well. Difficulty was experienced with the original design of the sludge recirculation return line from the final settling tank during peak flow because of lack of head. During the peak loads no return occurred, resulting in line stoppage. Installation of a 200 gpm pump in the line rectified the difficulty.

The first-stage filters at Fort Sill received a BOD load of 6,440 lb per acre-ft as based upon settled raw sewage. These filters, together with intermediate settling, removed 63.0 percent of this applied load, corresponding to a removal of nearly one-half (47.8%) of the entire BOD load of the plant. This high degree of efficiency is to be attributed to the following factors: (i) warm temperature without extreme seasonal variations; (ii) rather small size of media with large contact area; (iii) absence of grease in excessive amounts; (iv) uniform application of flow to filters, which operated at a constant rate of 16.4 mgad regardless of the rate of flow into the plant; and (v) absence of returned supernatant, which was routed to lagoons.

Performance of the first-stage filters with respect to removal of SS was less effective than with respect to BOD; nevertheless, a considerable reduction occurred relative to the size of the units.

No samples of filter effluent were regularly collected at Fort Sill. As a consequence it was not possible to evaluate the distribution of load between primary and secondary sections of the plant in accordance with Equation (3) as developed in Chapter I. Instead it was necessary to base the distribution upon actual performance in the *primary* settling tank in accordance with Equation (4).

The second-stage filters at Fort Sill were constructed over an area that included the old World War I fixed nozzle filter bed. The filter media and drainage arrangement in the old beds were not disturbed. The remaining area of the new filters was constructed over a new underdrainage system. Ventilation in the new construction area consisted of 8-in. vitrified tile risers with a 3.0-ft spacing around the circumference. There was some pooling of water in the old portion of bed where the medium was not disturbed during construction. This condition was improved somewhat by agitation of the media to a depth of 18 inches with an air-hammer.

These filters were loaded with 658 lb BOD/af; they operated with an efficiency of 69.5 percent. The loading was in the range of standard filters, but removal was only about three-quarters of that usual in standard filters. The reason was that "treatability" of a first-stage filter effluent is significantly less than that of raw settled sewage, with the result that efficiencies of secondary filters are less than those of primary filters for equal loadings. This phase of performance will be considered quantitatively in a subsequent section of this report.

Some unloading occurred in the second-stage filters at Fort Sill during abrupt weather changes in the early spring and late fall. The rotary distributors of the second-stage filters were dosed from two dosing tanks. Flow was usually continuous; dosing cycles, when they occurred, had a period of about 15 minutes.

Efficiency of the combined first and second stage of filtration was 88.8 percent. The overall loading for both filters based upon raw settled sewage was about 1,400 lb per acre-ft—a value less than one-half that stipulated by the Engineering Manual for single-stage high rate filters. As calculated on this basis, therefore, the loading of the plant was not at all heavy—indeed it was considerably lighter than that of many single-stage filter plants. Overall plant performance at Fort Sill, however, was high; BOD removal was 91.5 percent and SS removal was 92.1 percent. Relative stability was high; concentration of plant effluent was 26 ppm of BOD.

Little trouble was experienced at Fort Sill from scum accumulation in the sludge digester during the period of operation reviewed. Excess grease caused considerable foaming during the breaking-in period. Subsequent to this time, however, grease was largely removed from a scum chamber near the primary settling tank. As may be seen from Table 82, the digester at Fort Sill was more heavily loaded than digesters at other two-stage filter plants. The digester did not produce a returnable sludge liquor. Lagoons were constructed to receive the supernatant liquor, which was in effect a thin sludge. Occasionally supernatant was returned to sand drying beds. The sludge was well digested and no odor problems developed in connection with disposal.

Topeka AAB, Kan.

Results of the trickling filter plant at Topeka AAB are of interest in that they indicate a rather typical performance of a small two-stage plant under conditions operative at military establishments. The sewer system was flat. The sewers were fairly tight, but infiltration was somewhat more than normal due to the circumstance that a large part of the sewer system had to be laid in rock trenches. Ground water drained into these trenches, the only outlet of which was through the sewers. While no rapid rise in flow occurred immediately after rain, the volume of fluid reaching the plant was greater than average over an extended period following the storm. Grease in the plant influent was negligible and no laundry wastes were present.

As may be seen from the diagram of Table 74, no intermediate settling was provided at Topeka AAB; the rate of recirculation was about the same for both filters. On some occasions it was necessary to discontinue recirculation temporarily in order to accommodate unusually high peak flows.

Flow measuring equipment was not available at the plant until October, 1943, at which time a flow-meter was installed in a Parshall flume built when the plant was constructed in 1942. Figures for flow prior to October, therefore, are estimates. However, internal evidence would indicate that the estimates were reliable. Sufficient data pertaining to SS were not available to permit of an adequate analysis.

Performance of primary settling at Topeka AAB with respect to removal of BOD was somewhat low. The first- and second-stage filters were similar as to size and shape. Some trouble was experienced with filter clogging due to undersized rock and improper media placing. The size range specified in design was 1½ to 2½ in. It was necessary to dig up and relay parts of the beds to eliminate clogging. Chlorination also aided in reducing the clogging. The first-stage filter received a BOD load of 2,730 lb/af, which was not far from the load stipulated in the Engineering Manual for single-stage high rate filters (3,000 lb/af). A 65.8 percent reduction was effected in this filter. With intermediate settling, it was estimated that the first-stage filtration efficiency would have been about 72 percent. The second-stage filter received a BOD loading about one-third that of the first-stage filter, and its efficiency was accordingly higher (73.8 percent). The efficiency of the combined first- and second-stage filters was 91.1 percent. The overall loading calculated as the quotient of the pounds of BOD of raw settled sewage divided by the total volume of both filters in acre-ft was 1,360—a value intermediate between the usual loadings of standard and high-rate filters.

The sludge digestion tank at Topeka AAB was equipped with an agitation device which was operated continuously. Scum and grease were never troublesome. Supernatant was discharged to the raw sewage at the head of the plant. The effluent from the plant was discharged into a dry run; chlorine, about 10 lb per day, was used to prevent nuisance downstream.

Fort Francis E. Warren, Wyo.

The two-stage filter plant at Fort Francis E. Warren did not give as high a degree of treatment as other two-stage plants of similar design and loading. The plant was constructed in two major phases. Work on the original plant was started in January, 1941, and it began to function as a complete unit in May, 1941. At this time the plant consisted of two clarifiers, two high rate filters, and a sludge digester.

The performance of the plant from June, 1941 to January, 1942 was poor, although it was not loaded to design capacity. The final effluent averaged 169 ppm of BOD and 106 ppm of SS. The difficulties were attributed to large amounts of grease in the sewage. The grease also contributed to the somewhat erratic behavior of the digester during the initial stages of operation.

The digester was of the tray type with the lower, secondary section built underground and the upper or primary section above ground and covered with a veneer of hollow tile. The cover was of the fixed type. Only the upper section was heated. The contents of this section were agitated with mixing and scum-breaking equipment. Gas production was irregular and considerable quantities of lime were required. During this early phase of operation the digester capacity, including both upper and lower sections, amounted to 3.2 cf/cap. One difficulty stemmed from inability to seal cracks around the dome of the digester. The supernatant during this period averaged 1.47 percent solids, and difficulty was experienced with clogging of the supernatant return line. Sludge drying was retarded due to lack of underdrains in the sand beds.

In view of operational experience and expected increase in post strength, the plant at Fort Warren was enlarged and revised. The additions included a grease and grit chamber, a final filter and a final clarifier. Additional digester capacity also was provided and the new digester, a rectangular, earth-covered tank, was placed in operation in January, 1943. Provision was made for heating this unit but heat was not utilized until the middle of 1943.

During the operating period at the Fort Warren plant investigated in this report (January, 1943 to August, 1944), two-stage filtration was utilized. In this period the grease content was normal; grease traps serving mess-halls were cleaned regularly. No laundry wastes entered the plant; separate laundry waste treatment facilities were operated.

The population at the post was approximately 50 percent larger than during the first year of operation. The primary settling tank was loaded to 62 percent of capacity; intermediate and final tanks, however, had significantly shorter detention periods. Sludge handling capacity was about twice that stipulated by the Engineering Manual for a population of the size actually present at the post.

The sewage at Fort Warren was more concentrated (429 ppm BOD) than at any other two-stage filter plant studied. BOD removal in the primary settling tank (15.3%) was considerably lower than average; performance with respect to SS was also poor. The first-stage filters were heavily loaded (8,520 lb BOD/af); performance, however, was distinctly inferior compared to similarly loaded filters at other plants.

The effluent from the intermediate settling tank averaged 181 ppm of BOD and 96 ppm of SS. BOD efficiency of the first stage of filtration, not including reduction in primary settling, was 50.5 percent. The media in the first-stage filters were small ($1\frac{1}{2}$ to $2\frac{1}{2}$ in.) but no clogging or pooling occurred. Recirculation took place at a constant rate. Flow to the filters was composed of raw settled sewage plus a sufficient volume of settled sewage from the intermediate settling tank to keep the pumps operating at a constant rate throughout the day. The magnitude of this constant rate of application was altered from time to time, with an average value of 2.11 mgd.

The final filter had a volume more than six times as large as the combined volume of the first-stage filters. Media size in this unit was 2 to 3 inches, and media depth was 8 ft. The rate of volumetric dosage upon this filter was about 20 percent of that upon the high rate filters. The efficiency of the final filter was low as compared to final filters in other similarly loaded two-stage plants. Flow (including recirculation) to the secondary filter at Warren was maintained at a constant rate. Unloading periods were observed.

The BOD loading as computed upon an overall basis (raw settled sewage BOD per acre-ft of total filter volume) was 1,180. This value was about twice the usual design load for standard filters. The Warren plant, however, failed to turn in a performance comparable to heavily loaded plants in the standard filter range. The overall BOD efficiency of the combined first- and second-stage filters was 82 percent (Table 79); overall plant removal was 84.8 percent. In connection with the interpretation of operating results at Fort Warren, attention should be directed to the fact that the mean annual temperature in Wyoming was notably lower than that at the majority of the two-stage filter plants investigated.

In Figure 84 are shown data pertaining to the variations in loading and performance at Fort Warren throughout the period of investigation. In particular, it may be noticed that performance with respect to the removal of BOD was erratic. Variations in loading were of the order of 80 percent to 100 percent of the average.

Even with the additional digester capacity provided in January, 1943, sludge digestion was not satisfactory. Sludge as drawn from the secondary digester was incompletely digested; it had a volatile content that averaged 72 percent. Difficulty was sometimes experienced with the supernatant drawoff device, which was difficult to regulate. Supernatant was not of good quality; it was returned to the head of the plant. Sludge from intermediate and final settling tanks was also returned to the grease and grit chamber. Regular composite samples were taken from this chamber and it is possible to estimate the weight of organic matter returned to the head of the plant in the form of supernatant and secondary sludge. Total weight of BOD in the plant influent averaged 2,716 lb per day, whereas the grease and grit chamber effluent averaged 3,592 lb of BOD per day. Therefore the weight of BOD returned to the head of the plant averaged 776 lb, or 28 percent of the weight of BOD in the raw sewage.

Willow Grove Naval Air Station, Pa.

Results of operation of the plant at Willow Grove Naval Air Station are indicative of performance of a two-stage filter plant not heavily loaded. Sewer gradients were moderate and infiltration was not excessive. No laundry wastes were handled by the plant during the period of the survey. Grease traps at mess-halls were cleaned regularly and grease in the raw sewage was not excessive. Water consumption at the post was relatively large and the raw sewage was less concentrated than that at any other two-stage plant.

Percent removal of BOD in primary settling (48%) was greater than that of other plants in the division. The efficiency of the first-stage filter was 45.5 percent, while the efficiency of the second-stage filter was 73.8 percent. The disparity between the two efficiencies does not stem so much from the difference in rate of load application as from the fact that no intermediate settling was provided. Part of the settleable organic matter agglomerated in the first filter was deposited in the final settling tank.

The filter media at Willow Grove NAS were small, and not over 5 percent passed through 1-in. mesh. No ponding occurred. A high degree of treatment was attained. The BOD efficiency of the combined first- and second-stage filters was 85.7 percent, and the overall plant BOD removal was 92.6 percent. SS overall removal was 94.9 percent. It may be noted in Figure 83 that plant performance was rather independent of load variations; some seasonal influence, however, appeared to be manifest.

Digester capacity relative to actual population at Willow Grove NAS was unusually large and no difficulties with sludge digestion were experienced. Scum and grease were not troublesome.

The plant effluent was discharged into a waterway that was used for a public water supply. Accordingly, chlorine was added to the plant effluent; residual tests were made every two hours.

Generalization of Results of Filter Performance

The data pertaining to trickling filters at military installations, presented heretofore, cover a wide range of conditions. In Figures 86 and 87 are shown the relationships between BOD loading and removal for single-stage filters. A general decrease in efficiency may be observed with higher loadings. Figure 86 includes most of the low rate filters; Figure 87 includes most of the high rate filters. The form of diagram used in these figures is not suited to a precise evaluation of the decrease of efficiency with increased organic loading; moreover, it does not bring out the effect of other important variables such as recirculation and filter depth upon performance. The figures do, however, indicate a salient general trend.

It is evident that the relationship between loading and performance of trickling filters is not a simple one, due

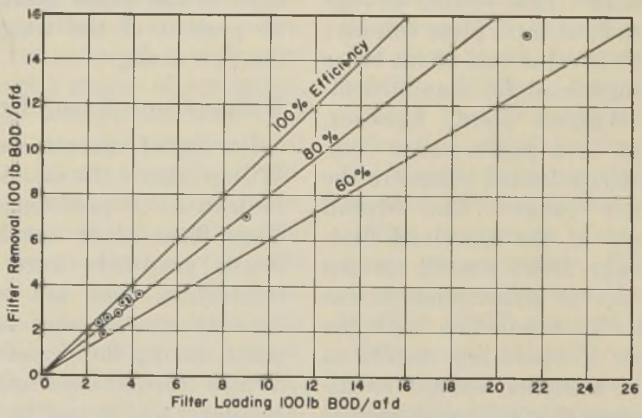


FIGURE 86.—Relation between loading and performance in plants without recirculation. Loading is based upon raw settled sewage. Removal includes that in settling tanks subsequent to filtration. Each point represents average for a single-stage plant for period of study.

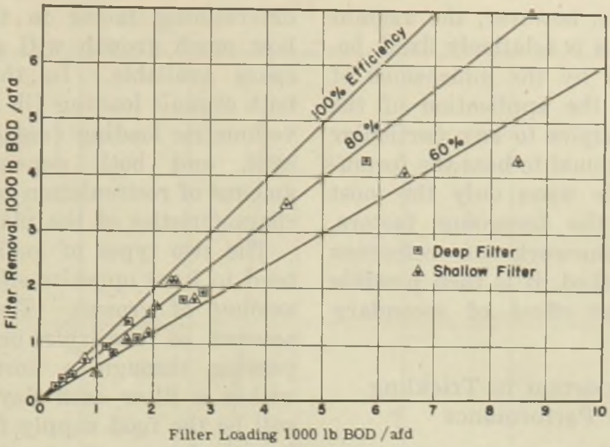


FIGURE 87.—Relation between loading and performance in plants with recirculation. Loading is based upon raw settled sewage. Removal includes that in settling tanks subsequent to filtration. Some points represent first stage filtration in two-stage plants.

to the many variables that are involved. Elucidation of the relationship has been obstructed in the past by employment of a number of different methods of rating of the loading and performance.

After considerable investigation and study, the subcommittee has come to the belief that the simplest and most direct measure of filter loading is the average weight of BOD in the settled sewage applied to the filter per day. This does not include recirculated BOD. To be sure, recirculation has an important bearing upon performance, but this can better be taken into account in another fashion. Results indicate that recirculated BOD has a "treatability" that is essentially different from the BOD in raw settled sewage. Consequently, the two substances cannot be indiscriminately combined to form a useful measure of loading. A more logical and useful method for evaluation of the influence of recirculation is set forth in subsequent paragraphs.

Mechanism of Biological Sewage Treatment

It is worthwhile to view the process of biological treatment of sewage from a broad objective view that embraces all treatment methods including filters, submerged contact surfaces, and activated sludge. While it might appear that the various treatment methods are essentially different, it is readily shown that they all have certain fundamental points of similarity. In general it may be stated that the degree of treatment attained in each will depend upon the following factors: (1) magnitude of the organic load treated per unit of time; (2) amount of biologically active growth, whether sessile or free-floating; (3) adequacy of air-liquid interface; (4) time of contact between organic load and biological growth; (5) degree of agitation and turbulence at the interface of growth and sewage and (6) provision made for settling of agglomerated material and detached excessive growths. The importance of each of these factors may vary in the different methods of treatment. Moreover, the degree of control over each factor that may be exercised will not be the same in different processes. For example, in the activated sludge process, the amount of biologically active growth at a given plant can be varied over wide limits by regulating the rate of sludge waste and return. In a trickling filter plant, however, the amount of active growth is relatively fixed, being determined by the dimensions of the filter. In the application of the foregoing principles to any particular method it is rational to base the formulation primarily upon only the most important of the foregoing factors. Within a framework of reference thereby established, it is then possible to evaluate the effect of secondary variables.

Factors Important in Trickling Filter Performance

With regard to treatment upon trickling filters it is evident from Figures 86 and 87 that the magnitude of the BOD load, relative to the volume of the filter, is more or less directly related to efficiency. The amount of biologically active growth is dependent primarily upon the dimensions of the filter, including those of the media.

On the basis of the results at hand, it would appear to be an oversimplification to presume a simple direct proportionality between the surface area of the contact media and the volume of growth. Such a condition would only obtain if the growth were a thin film confined to the surface of the stones. This is not the case, since in some filters growths of large dimensions were found in some of the interstices.

It would appear that the volume of growth in a biological filter is proportional to the diameter of the media raised to an exponent between one and two. Only in the case of a very thin film would the exponent approach two. If the growth completely filled the interstices, the exponent would be zero. Moreover, it is not possible to resolve the matter merely upon the question of the *volume* of growth, since the important thing is the volume of *active* growth.

While the size of the filter and the media establishes the amount of space available for biologically active growths, the rate of application of load is the determining factor in the matter of how much growth will appear in the space available. In this connection both organic loading (lb BOD/af) and volumetric loading (mgad) are significant, and both depend upon the amount of recirculation as well as upon characteristics of the plant influent.

The two types of loading, however, tend to have opposite effects upon the amount of growth. The greater the amount of available organic matter passing through a horizontal plane within a filter each day, the greater will be the food supply for the organisms, and hence the greater the growth in the plane. In this respect high loadings and high recirculated flows favor growth of active biological matrices. On the other hand, high rates of dosage per unit area increase the scouring action throughout the bed; projecting growths of loosely attached material are continuously detached and carried into the settling tanks. Such flushing tends to limit the amount of growth in the filter, which may or may not be desirable.

Of the two types of loading, organic and volumetric, the former has the greater influence upon efficiency, as may be seen from a comparison of Figures 86 and 87 with Figure 89. In these figures, filters with media depths less than 4.5 ft are considered to be shallow filters and those more than 4.5 ft deep are considered to be deep filters. Almost no relation between volumetric rate of dosing (mgad) and efficiency is exhibited in Figure 89. In this plot a number of shallow filters with large rates of volumetric dosing may be observed to have high efficiencies. This performance was obtained only with use of high recirculation ratios and does not imply a particular advantage pertaining to shallow filters. In the plants studied, it happened that the shallow filters installed usually had relatively large recirculation ratios. Deep filters usually were operated with smaller amounts of recirculation.

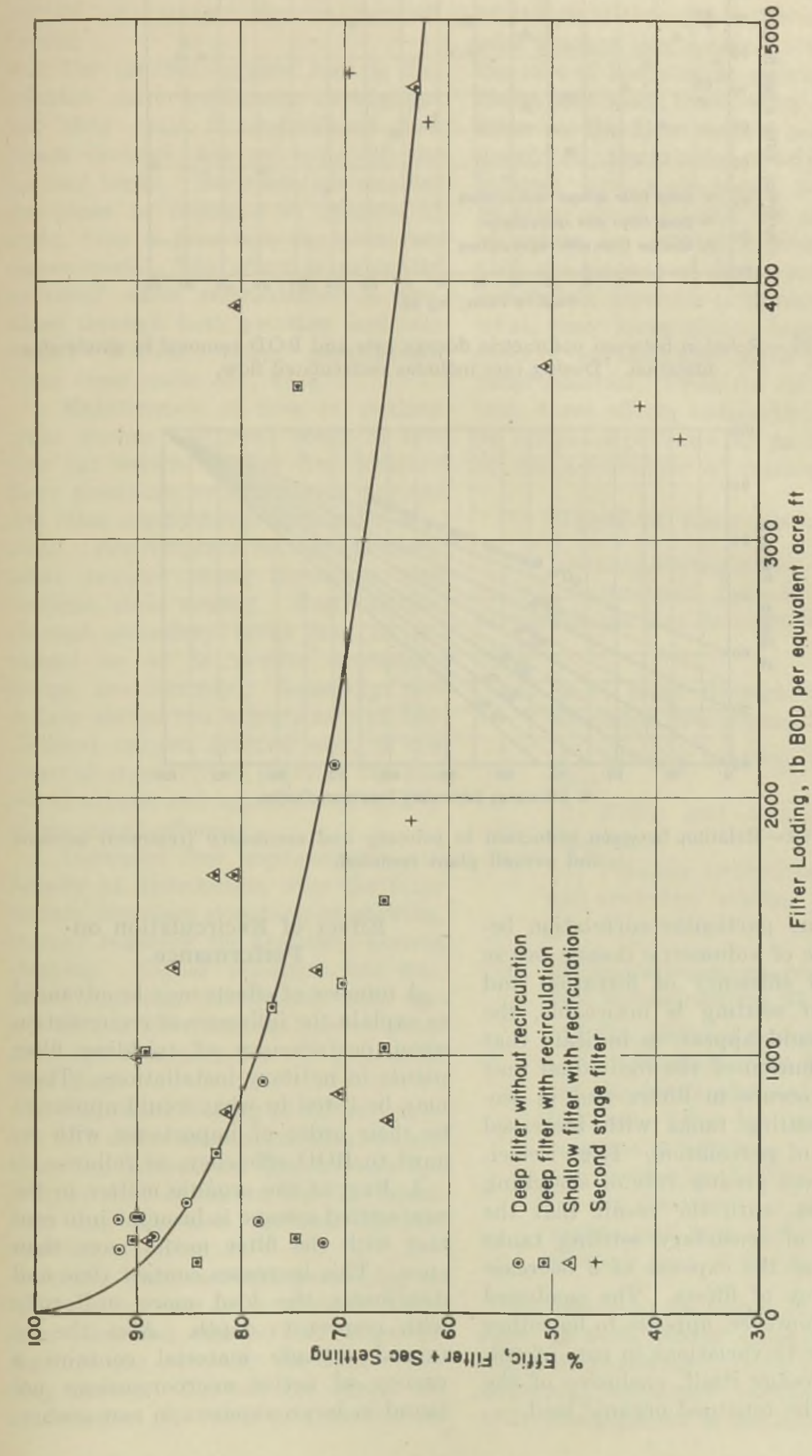


FIGURE 88.—Relation between filter loading and efficiency for all filter plants. Loading is expressed upon an equivalent basis in accordance with filter efficiency formulation. Points representing second-stage filter performance are plotted as equivalent first-stage points, but were not considered in establishing curve.

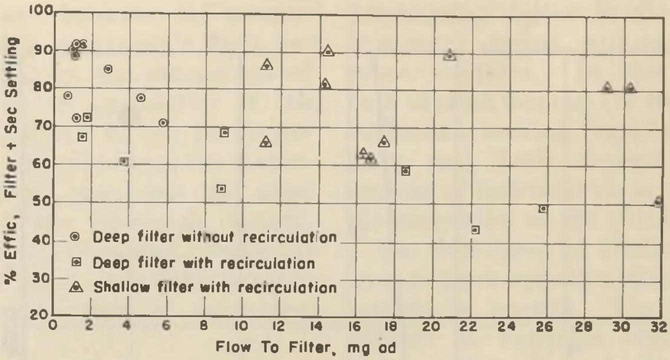


FIGURE 89.—Relation between volumetric dosage rate and BOD removal in single-stage filtration. Dosing rate includes recirculated flow.

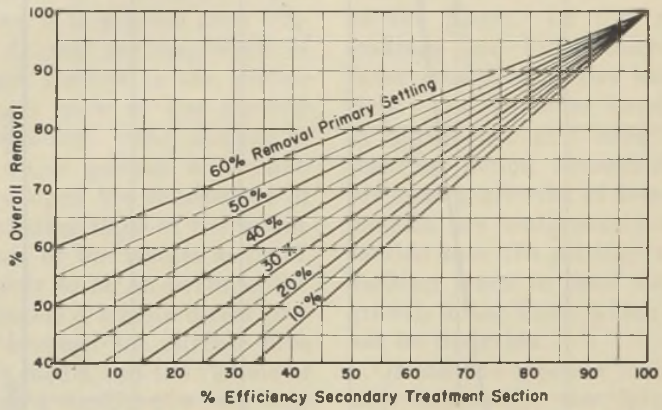


FIGURE 90.—Relation between reduction in primary and secondary treatment sections and overall plant removal.

While no particular correlation between rate of volumetric dosage *per se* and BOD efficiency of filtration and subsequent settling is noticeable, the results would appear to indicate that a redistribution of the individual unit removals occurs in filters and in secondary settling tanks with increased velocities of percolation. High velocities induce a greater rate of sloughing of growths, with the result that the efficiency of secondary settling tanks increases at the expense of a decrease in efficiency of filters. The combined removal, however, appears to be rather insensitive to variations in rate of volumetric dosage itself, exclusive of the effect of the returned organic load.

Effect of Recirculation on Performance

A number of effects may be advanced to explain the influence of recirculation upon performance of trickling filter plants in military installations. These may be listed in what would appear to be their order of importance with regard to BOD efficiency, as follows:

1. Part of the organic matter in the raw settled sewage is brought into contact with the filter media more than once. This increases contact time and distributes the load more uniformly with respect to depth. Also, the returned organic material contains a variety of active microorganisms not found in large numbers in raw sewage,

thereby continually providing the seeding of appropriate species for all depths.

2. The diurnal organic load is distributed more uniformly throughout the 24-hr cycle if recirculated flow passes through one or more of the settling tanks. Recirculation enables the plant to continue to operate at night, even if flow into the plant becomes small. The effect is evidently enhanced when recirculation is provided through both primary and secondary settling tanks, particularly when these units are large.

3. Maintenance of flow in settling tanks during nocturnal hours of low flow (or seasons of low dry weather flow) precludes long detention periods and other conditions conducive to septicity. Recirculation through primary tanks reduces scum formation and freshens stale sewage. Recirculation through secondary tanks may be arranged so as to remove secondary sludge continuously. Generally secondary sludge has a high rate of biochemical oxygen demand and, if not removed regularly, it depletes the dissolved oxygen and impairs the stability of the plant effluent.

4. Increased flow improves the uniformity of distribution over the filter surface, increases sloughing of growths, thereby reducing the tendency toward clogging. Higher velocities and continual scouring, particularly near the top of the filter, render the environment less favorable for the growth of filter flies.

5. Continual seeding of the raw sewage with active microorganisms and enzymes in recirculated flow induces hydrolysis and oxidation in the mixed fluid and increases the rate of biochemical stabilization.

Certain disadvantages pertaining to recirculation may be mentioned (aside from increased expense due to pumping and the necessity for larger settling tanks). The adverse effects are as follows: (i) temperatures are reduced as a result of a number of passes of the fluid through the filter and in cold weather this causes a lowering of the rate of biochemical activity; in extreme conditions troublesome ice formation on the filter surface may be induced; (ii) the dilution of settling tank influent with recirculated flow tends to bring about a small but significant reduction in settling efficiency; and (iii) the amount of sludge solids to be handled in digesters is increased somewhat, since recirculation increases the ratio of agglomerated material to oxidized material. From the viewpoint of cost, these effects ordinarily are only of minor importance in an appraisal of the advantages of recirculation.

Types of Recirculation

The various arrangements for providing recirculated flow at military sewage plants may be classified as follows:

I. As to units through which recirculated flow is passed.

(1) Primary settling tank and filter.
(2) Filter and secondary settling tank.
(3) Primary settling tank, filter and secondary settling tank.
(4) Filter only.

II. As to variation in recirculated flow throughout the 24-hr cycle:

(1) Recirculated flow (R) maintained constant at all times.
(2) Recirculated flow plus plant influent (R + I) maintained constant at all times.
(3) Recirculated flow constant part of the time (e.g. during the night) and zero during the remaining part of the cycle.

No important difference in performance between plants employing primary recirculation (I −1) and those employing secondary recirculation (I − 2) was manifest in the data. Minor advantages for each type have been cited. A large number of military plants utilized both primary and secondary recirculation (I – 3), and the results indicate that these plants, on the whole, gave definitely better performance than the others. It must be pointed out, however, that most military plants had large settling tanks and night flow was ordinarily low. Consequently recirculation at many posts was necessary in order to avoid the effects accompanying septicity. In utilizing recirculation to avoid anaerobic conditions, the filter, at the same time, was made to continue its function of removing organic matter long after the plant influent had fallen to its night-flow level. Municipal plants with smaller settling tanks and fairly uniform flow during the 24-hr cycle undoubtedly would not obtain as much benefit from recirculation as did military plants.

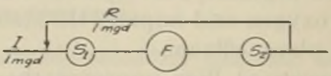
The number of military plants that utilized recirculation about the filter only (I – 4) was too small to permit assessing properly the merits of this scheme. Certainly it did not improve the freshness of primary settling tanks, nor did it bring about a more uniform distribution of the diurnal organic load. The data that are available would indicate that, in the absence of these two effects, performance suffered. The plant at Camp Gordon, for example, which employed recirculation from filter effluent to filter influent, was loaded in the standard filter range but operated with a BOD efficiency of only 81.7 percent and produced an effluent with a BOD of 48 ppm.

With regard to variation of recirculated flow throughout the 24-hr period, no outstanding advantage for any particular scheme is apparent from the data. However, a number of plants, such as those at Fort Sill and Camp Roberts that utilized "equalized" flow (II–2), had records of exceptional performance, and it would appear that some advantage may be accredited to this scheme. However, a statistical analysis of the data for all plants indicates that recirculation in nearly any form brought about an improvement in performance that was significantly larger than the differences in performance noted between different schemes of recirculation.

It is useful to formulate the general effect of recirculation upon plant performance. The most direct approach consists of a careful consideration of the most important effect of recirculation—the returning of part of the organic matter to pass through the filter again.

Relation Between Recirculation Ratio and Number of Passes Through Filter

It is possible to develop a quantitative expression for the average number of times water particles in the influent sewage pass through a filter with a given recirculation ratio. To fix the ideas, consider the following recirculation scheme in which the recirculation ratio is one:



Half of the flow leaving the secondary settling tank (S_2) is returned to the plant influent. If the progress of, say, 1 cubic foot in the plant influent is traced through the plant to the effluent, the following statements are seen to be true: (a) one-half of the flow passes through the plant and leaves with the effluent and the remaining half is returned to pass through the filter twice; (b) one-half of this remaining half (one-quarter of the original) is returned to pass through the filter three times, etc. Evidently the average number of passes made by the particles in the original unit volume is given by the sum of the following series:

$$\tfrac{1}{2}(1) + \tfrac{1}{4}(2) + \tfrac{1}{8}(3) + \tfrac{1}{16}(4) + \cdots = 2$$

Some particles, it is seen, will be returned to pass through the filter many times, and one-half of the flow will pass through but once. However, when the recirculation ratio is one, the flow on the average passes through the filter twice.

In a more general way, when the recirculation ratio is (R/I), the average number of passes is given by the sum of the following expression:

$$\frac{I}{R+I}(1)+\frac{I}{R+I}\left[\frac{R}{R+I}\right](2)+\frac{I}{R+I}\left[\frac{R}{R+I}\right]^2(3)+\cdots=1+\frac{R}{I} \quad (23)$$

Thus the average number of passes through the filter is equal to the circulation ratio plus one. If, for example, the recirculation ratio is 2.5, then the average number of passes would be 3.5.

The average number of passes made by the *incoming water* in the sewage is not to be confused with the average number of passes of the *incoming organic matter.* The two average values are not identical, for the plant removes organic matter in each pass, whereas the water particles, excepting for negligible amounts removed in sludge and by evaporation, are not affected by the plant.

The foregoing analysis for numbers of passes of water may be used as a guide in a formulation for the average number of passes of the organic matter. It may be assumed that, to a fair degree of approximation, a constant proportion of the organic material is removed with each passage. There are many data in trickling filter literature corroborating this assumption. Therefore, with each passage a diminution of the original organic matter occurs, not only because of the splitting of flow at the point of recirculation, but also because of the biochemical activity of the filter. Another fact is pertinent: not only is part of the organic matter removed in each passage, but the *availability* of the remaining organic matter is reduced, since the filter growths tend to extract the more readily transferable substances first. The diminution may be taken into account by a "weighing factor," p, which reflects the amount and availability of organic matter remaining after each passage relative to the initial amount. The average number of passes of the organic material will be given by the sum F of the following series, which is obtained by modifying Equation (23) with the "weighting factor" p:

$$F=\frac{I}{R+I}(1)(1)+\frac{I}{R+I}\left[\frac{R}{R+I}\right](2)(p)+\frac{I}{R+I}\left[\frac{R}{R+I}\right]^2(3)(p)^2+\cdots,$$

$$F=\frac{1+\frac{R}{I}}{\left[1+(1-p)\frac{R}{I}\right]^2} \quad (24)$$

The numerical value of p, on the basis of data from military trickling filter plants, appears to be about 0.90. Analysis of performance of individual plants gave values of p between 0.81 and 0.95, with an average of 0.90. The data were not sufficiently detailed and precise to permit of a precise evaluation of p for plants in the South as compared with those in the North. However, an overall value of about 0.90 would appear to be fairly well established. Thus, for example, if the recirculation ratio at a plant was 2.5, then the average number of passes of the organic material would, in accordance with Equation (24), be 2.24.

Effective Filter Volume

The amount of contact, both on an areal and a time basis, between the filter media and organic matter depends upon the filter dimensions and the number of passes: it is in fact given by the product of the volume of the filter V and the number of passes F. The greater the effective contact VF the greater will be the efficiency. On the other hand, the greater the applied load (w = weight of BOD in raw settled sewage), the lower the efficiency, since the available effective volume will be taxed more heavily and cannot develop a sufficient amount of active growth to produce as high a degree of treatment. Therefore, the quantity that primarily determines efficiency in a trickling is w/VF. The efficiency E is a function of the load per unit of effective volume (lb BOD per equivalent acre-foot).

In Figure 88 are plotted the data for all trickling filter plants, including standard filter plants without recirculation. In these plants, F is one, and actual filter volume coincides with effective filter volume. In all cases the actual volume V was taken as the gross volume, that is, the product of the surface area A and the depth of the media d.

The validity of the foregoing reasoning is substantiated by the distribution of the plotted points in Figure 88. A distinct drop in efficiency may be noted with increased values of the parameter w/VF; moreover, all divisions of filters are seen to be well distributed above and below the curve of best fit. These circumstances indicate that the parameter, lb BOD per equivalent acre-foot, retains validity for all types of filters used at military installations, including deep and shallow filters, standard filters, and filters with high as well as with low recirculation ratios.

The curve of best fit has the equation,

$$\text{Percent Efficiency} = \frac{100}{1 + 0.0085\sqrt{\frac{w}{VF}}} \qquad (25)$$

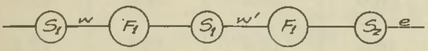
While Equation (25) is empirical, it does conform to reasonable expectation that with low loading efficiency will approach 100 percent, and that with high loading efficiency will approach zero, asymptotically. Moreover, the form of Equation (25) was found to fit sewage treatment plant data more precisely than any other equation with two arbitrary constants. The same form of equation has been found to apply in connection with the analysis of contact aeration and activated sludge plant performance.

To a large extent, the deviation of plotted points from the curve in Figure 88 may be attributed to variation in mean annual temperature. Most of the points falling above the curve correspond to southern plants, while northern plants had efficiencies that more often than not were less than those given by the curve.

Efficiency of Second-Stage Filters

The analysis leading to the formulation of Equation (25) applies to single-stage filter plants, and also to the first-stage filters in two-stage plants from which sufficient data were available to permit of a separate analysis of the two filters. The approach, however, is general and may be applied to second-stage filters.

In this application, it is necessary to take account of the fact that the BOD in the effluent of the first stage is considerably less "treatable" than that in raw settled sewage. The "treatability" depends upon the proportion of the initial organic matter removed in the first stage, because the filter growths tend to assimilate or otherwise remove the more readily available organic compounds first. Consequently, if, say, 1,000 lb of BOD from a first-stage filter were applied to a second-stage filter, the efficiency would be distinctly less than if 1,000 lb of BOD in raw settled sewage were applied to the same filter. In fact, a considerably larger weight of raw settled sewage BOD may be applied so as to bring about an efficiency equal to that with the 1,000 lb of BOD of first-stage effluent. This larger weight of BOD may be termed the "equivalent first-stage load." The "equivalent first-stage load" is defined as that weight of raw BOD which, if applied to the second-stage filter, would, in accordance with Equation (25), induce an efficiency equal to that resulting when the *actual* second-stage BOD is applied. The various loadings may be exemplified with the aid of a diagram of a two-stage filter.



where

w = pounds of BOD in the raw settled sewage, the first-stage load.

w' = pounds of BOD in the intermediate settling tank effluent, the second-stage load.

e = pounds of BOD in the plant effluent.

If E_1 represents the efficiency of the first-stage filter, then

$$w' = w(1 - E_1) \qquad (26)$$

If E_2 represents the efficiency of the second-stage filter, then

$$e = w'(1 - E_2) \qquad (27)$$

The organic matter represented by w' is not as readily available or transferable to filter growths as that represented by w. The equivalent first-stage loading of filter, F_2 is equal to a factor f multiplied by the loading w'. In accordance with the definition of equivalent loading, a quantity of raw BOD, (fw'), when applied to a filter of the dimensions of F_2, will be treated with an efficiency of E_2. The factor f evidently depends upon the "treatability" of w', which in turn depends upon E_1. By an analysis of available data for second-stage filters, it has been ascertained that the value of f may approximately be evaluated by the following expression:

$$f = \frac{1}{(1 - E_1)^2} \qquad (28)$$

With this modification, it is possible to apply Equation (25) to second-stage filters; for this purpose it is written in the following form:

$$E_2 = \frac{100}{1 + 0.0085\sqrt{\frac{w'}{VF(1 - E_1)^2}}} \qquad (29)$$

For convenience, the foregoing analysis of performance of trickling filters has been summarized below and arranged in systematic form with worked examples. The examples have been devised to represent military practice based upon design criteria specified by the Engineering Manual. In order to facilitate computations, a graph of the relation between reduction in primary and secondary treatment sections and overall plant removal is presented in Figure 90. The distinction between primary and secondary treatment sections is explained in Chapter I, as are the definitions of percent efficiency and percent removal.

TABLE 85.—Filter Efficiency Formulation

Load in LB BOD per Equivalent Acre-Foot (a)	Percent Efficiency of Filter (b)	Load in LB BOD per Equivalent Acre-Foot	Percent Efficiency of Filter
100	92.2	1,600	74.6
200	89.2	1,800	73.5
300	87.2	2,000	72.5
400	85.4	2,500	70.2
500	84.0	3,000	68.2
600	82.8	4,000	65.0
800	80.6	5,000	62.5
1,000	78.7	6,000	60.2
1,200	77.3	8,000	56.8
1,400	75.8	10,000	54.0

(a) $\frac{\text{lb BOD in settled raw sewage}}{\text{Filter Area} \times \text{Equivalent Depth}} = \frac{w}{A \times d \times F} = \frac{w}{VF}$

where

F = recirculation factor $= \frac{1 + R/I}{(1 + 0.1R/I)^2}$

R/I = recirculation ratio

TABLE 86.—Trickling Filter Effluent BOD With Various Loadings and Recirculation Ratios. Single-Stage Filtration

Dosing Rate* of Settled Raw Sewage (mgad)	BOD of Settled Raw Sewage (ppm)	Effective Filter Depth = Actual Depth × Recirculation Factor†					
		3	6	9	12	15	18
2.0	100	16.5	12.4	10.4	9.0	8.0	7.3
	200	44.4	33.0	28.4	24.8	22.4	20.8
	300	77.4	59.4	49.5	44.4	40.2	37.2
4.0	100	22.2	16.5	14.2	12.4	11.2	10.4
	200	57.0	44.4	37.4	33.0	30.4	28.4
	300	98.7	77.4	66.6	59.4	54.0	49.5
6.0	100	25.8	19.8	16.5	14.8	13.4	12.4
	200	65.8	51.6	44.4	39.6	36.0	33.0
	300	112.5	89.4	77.4	69.6	63.9	59.4
8.0	100	28.5	22.2	18.7	16.5	15.2	14.2
	200	72.2	57.0	49.6	44.4	40.6	37.4
	300	123.0	98.7	85.5	77.4	71.4	66.6
12.0	100	32.9	25.8	22.2	19.8	18.0	16.5
	200	82.0	65.8	57.0	51.6	47.6	44.4
	300	—	112.5	98.7	89.4	82.5	77.4
16.0	100	36.1	28.5	24.8	22.2	20.3	18.7
	200	—	72.2	63.6	57.0	52.6	49.6
	300	—	123.0	108.3	98.7	91.8	85.5
20.0	100	38.7	31.0	26.8	24.0	22.2	20.7
	200	—	77.4	68.0	62.0	57.0	53.6
	300	—	—	116.1	106.2	98.7	93.0

* Does not include recirculated flow.

† Recirculation factor $= F = \dfrac{1 + R/I}{(1 + 0.10\,R/I)^2}$

(b) *Single-Stage Filtration:*

Percent Efficiency

$$= \frac{100}{1 + 0.0085\sqrt{\dfrac{w}{VF}}}$$

Second-Stage Filtration:

Percent Efficiency

$$= \frac{100}{1 + \dfrac{0.0085}{1 - E_1}\sqrt{\dfrac{w'}{VF}}}$$

where

$100E_1$ = percent efficiency of first-stage filtration

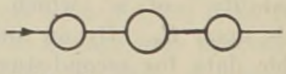
and

w' = lb BOD in effluent of first-stage filtration.

Examples of Use of Filter Formula

Assume: flow = 1 mgd; concentration of BOD in raw sewage = 308 ppm; 35 percent removal in primary settling; concentration of BOD in primary tank effluent = (1 − 0.35)308 = 200 ppm; weight of BOD in primary tank effluent = 200(1)(8.34) = 1,668 lb.

Example (i) Standard filter—no recirculation:



Given: Vol of filter = 2.78 acre-ft
Depth of filter = 6.0 ft
Area of filter = 0.463 acre

Find: Concentration of BOD in plant effluent and overall percent reduction in BOD.

Loading;

$w = 1{,}668$ lb

Recirculation factor $= F = 1$ (no recirculation)

$$\frac{w}{VF} = \frac{1{,}668}{2.78} = 600$$

Substituting in filter efficiency equation,

Percent Efficiency

$$= \frac{100}{1 + 0.0085\sqrt{600}} = 82.8\%$$

(See also Figure 88 and corresponding table of efficiencies.)

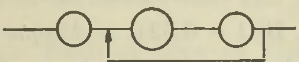
Concentration of BOD in plant effluent $= (1 - 0.828)200 =$ *34 ppm*

Percent overall reduction in BOD

$$= 100\,\frac{308 - 34}{308} = 89.0\%$$

(See also Figure 89.)

Example (ii) High rate filter—with recirculation:



Given: Vol of filter* = 0.556 acre-ft
Depth of filter = 3.0 ft
Area of filter = 0.185 acre
Recirculation ratio = 1.5

Find: Concentration of BOD in plant effluent and overall percent reduction in BOD.

Loading;

Recirculation factor

$$= F = \frac{1 + 1.5}{(1 + 0.15)^2} = 1.89$$

$$\frac{w}{VF} = \frac{1668}{(0.556)(1.89)} = 1{,}580$$

Substituting in filter efficiency equation,

* Volume of filter is 20 percent of volume of filter of Example (i).

Percent Efficiency

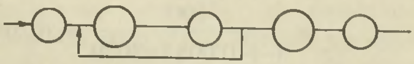
$$= \frac{100}{1 + 0.0085\sqrt{1580}} = 74.8\%$$

Concentration of BOD in plant effluent $= (1 - 0.748)200 =$ *50 ppm*

Percent overall reduction in BOD

$$= 100\,\frac{308 - 50}{308} = 83.7\%$$

Example (iii) Two-stage filter:



Given:

	First Stage	Second Stage
Volume* (acre-ft)	0.70	2.08
Depth (ft)	3.0	6.0
Area (acres)	0.233	0.347
Recirculation ratio	0.50	0.0

* Total volume same as in Example (i).

Find: Concentration of BOD in plant effluent and overall percent reduction in BOD.

First-stage filtration—loading;

Recirculation factor

$$= F = \frac{1 + 0.5}{(1 + 0.05)^2} = 1.36$$

$$\frac{w}{VF} = \frac{1668}{(0.70)(1.36)} = 1{,}750$$

Substituting in filter efficiency equation,

Percent Efficiency

$$= \frac{100}{1 + 0.0085\sqrt{1750}} = 73.7\%$$

Concentration of BOD in effluent of first-stage filtration,

$(1 - 0.737)200 =$ *53 ppm*

Weight of BOD in influent of second-stage filtration,

$53(1)(8.34) = 440$ lb

Second-stage filtration—loading;

Recirculation factor $= F = 1$ (no recirculation)

$1 - E = 1.000 - 0.737 = 0.263$ (from efficiency of first stage)

Equivalent first-stage loading,

$$\frac{w'}{VF(1-E)^2} = \frac{440}{(2.08)(1)(0.263)^2} = 3{,}060$$

Substituting in filter efficiency equation,

Percent Efficiency

$$= \frac{100}{1 + 0.0085\sqrt{3070}} = 67.9\%$$

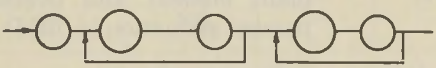
Concentration of BOD in effluent of second-stage filtration,

$53 = (1.000 - 0.679) = \mathit{17\ ppm}$

Percent overall reduction in BOD

$$= 100\,\frac{308 - 17}{308} = \mathit{94.6\%}$$

Example (iv) Two-stage filter:



Given:

	First Stage	Second Stage
Volume* (acre-ft)	0.277	0.277
Depth (ft)	3.0	3.0
Area (acres)	0.0925	0.0925
Recirculation ratio†	0.75	0.75

* Total volume same as in Example (ii).

† Total volume of recirculated flow same as in Example (ii).

Find: Concentration of BOD in plant effluent and overall percent reduction in BOD.

First-stage filtration—loading;

Recirculation factor

$$= F = \frac{1 + 0.75}{(1 + 0.075)^2} = 1.51$$

$$\frac{w}{VF} = \frac{1668}{(0.277)(1.51)} = 3{,}990$$

Substituting in filter efficiency equation,

Percent Efficiency

$$= \frac{100}{1 + 0.0085\sqrt{3990}} = 65.0\%$$

Concentration of BOD in effluent of first-stage filtration,

$(1.00 - 0.65)(200) = \mathit{70\ ppm}$

Weight of BOD in influent of second-stage filtration,

$70(1)(8.34) = 585$ lb

Second-stage filtration—loading:

Recirculation factor $= F = 1.51$

$1 - E = (1.000 - 0.650) = 0.350$

Equivalent first-stage loading:

$$\frac{w'}{VF(1-E)^2} = \frac{585}{(0.277)(1.51)(0.350)^2} = 11{,}400$$

Substituting in filter efficiency equation,

Percent Efficiency

$$= \frac{100}{1 + 0.0085\sqrt{11400}} = 52.4\%$$

Concentration of BOD in effluent of second-stage filtration,

$70(1 - 0.524) = \mathit{33\ ppm}$

Percent overall reduction in BOD

$$= 100\,\frac{308 - 33}{308} = \mathit{89.4\%}$$

Recirculation Factor

The recirculation factor (Equation 24), which has been defined as the mean number of passes of organic material through the filter, has properties that are of significance. The value of the recirculation factor F increases with the recirculation ratio R/I until a maximum value is attained. The maximum value of F and the corresponding value of R/I are given in the following expressions:

$$F_{\max} = \frac{1}{4p(1-p)} \qquad (30)$$

$$R/I]_c = \frac{2p - 1}{1 - p} \qquad (31)$$

When the constant p in Equation (24) has the value 0.90, then the following numerical relationships exist between the recirculation factor F and the recirculation ratio R/I:

Recirculation Ratio (R/I)	Recirculation Factor (F)	Recirculation Ratio (R/I)	Recirculation Factor (F)
0.0	1.00	4.0	2.55
0.5	1.36	6.0	2.73
1.0	1.65	8.0	2.78*
2.0	2.08	10.0	2.75
3.0	2.37	15.0	2.56

* Maximum.

It is seen that with $p = 0.90$, the greatest effect that recirculation through a given filter can produce is to make the effective volume 2.78 times as large as the actual volume. The increase in efficiency that results will depend upon the load intensity. The capacity of a plant is measured by the equivalent volume and the extent to which recirculation may be employed to enhance the actual volume is delimited by Equations (24) and (25).

Optimum Relative Size of Filters in Series

In the design of two-stage filters, Equations (25) and (29) may be used to ascertain the most effective volume of the second stage relative to that of the first stage. The usual design problem is the following: *given* the load w of raw settled sewage and the weight of BOD in the effluent e as calculated from the degree of treatment desired, *find* the equivalent volume of both filters, V_1F_1 and V_2F_2, such that the total equivalent volume is a minimum—that is, no larger than necessary.

A divergence of opinion as to the optimum relative sizes of filters in two-stage plants has been exhibited in designs at military posts. As may be seen from Table 75, the first- and second-stage filters at a number of installations had the same dimensions; at others cantonments, however, the second-stage units were made larger than the first. This latter design often came about from the fact that second-stage filters were added to existing single-stage plants to provide additional treatment in order to handle expected increases in post population. While the additional capacity and degree of treatment was always attained at these posts, it is pertinent to inquire whether the same overall performance might not have been attained with a *smaller* total equivalent volume of media, had it been possible to design these installations originally as two-stage plants. It is evident, though perhaps not obvious, that for a given situation a balanced design must exist such that the load w is equitably distributed between the two filters so that the desired effluent e is attained with a minimum total effective volume.

Mathematically stated, the problem is the following: *given* w and e, *find* V_1F_1 and V_2F_2, subject to the condition that the sum $V_1F_1 + V_2F_2$ is a minimum.

The overall efficiency E is set by the conditions of the problem,

$$E = (1 - E_1)(1 - E_2) = \frac{e}{w} \quad (32)$$

It is convenient to introduce three quantities t, u, and v that relate to equivalent volume per pound of raw settled sewage applied,

$$t^2 = \frac{V_1F_1 + V_2F_2}{a^2w}$$

$$u^2 = \frac{V_1F_1}{a^2w}$$

$$v^2 = \frac{V_2F_2}{a^2w}$$

where

$$a = \text{constant} = 0.0085 \quad \text{(Equation 25).}$$

If the quantities t and u are substituted in Equations (25) and (29) and the resulting expressions for E_1 and E_2 are introduced into Equation (32), the following relation between t and u may be derived:

$$E^2t^2 = \frac{1}{1+u} + E^2(1 + u + u^2) - 2E \quad (33)$$

The required minimum equivalent volume may then be evaluated by setting the derivative of t with respect to u equal to zero. This operation yields the following expression involving u:

$$(1 + 2u)(1 + u)^2 = E^{-2} \quad (34)$$

It is possible next to find the optimum values of u and t from Equations (33) and (34) for any fixed value of E. Finally, from the definitions of t and u the optimum equivalent volumes of the two stages in acre-feet may be evaluated. The result of this computation for a range of values of performance is presented in Table 86a.

TABLE 86a.—Efficiency of Two-Stage Filters of Optimum Sizes

Percent Efficiency of Combined First and Second Stages of Filtration (a)	Pounds BOD per Total Equivalent Acre-Foot (b)	Percent of Total Equivalent Volume in First Stage (c)	Percent of Total Equivalent Volume in Second Stage (d)
94	320	44	56
92	510	45	55
90	750	45	55
88	1,060	46	54
86	1,440	46	54
84	1,850	47	53
82	2,360	47	53
80	2,920	47	53

(a) $E = (w - e)/w$

where

e = lb BOD in plant effluent.

(b) $$\frac{w}{V_1F_1 + V_2F_2}$$

where

w = lb of BOD in raw settled sewage.

F_1, F_2 = recirculation factors for first and second stages (Equation 24).

V_1, V_2 = actual volumes (acre-ft) in first and second stages.

(c) Percent of total equivalent volume in first stage $= \dfrac{100V_1F_1}{V_1F_1 + V_2F_2}$

where V_1F_1, V_2F_2 = equivalent volumes of first and second stages.

(d) Percent of total equivalent volume in second stage $= \dfrac{100V_2F_2}{V_1F_1 + V_2F_2}$

The salient result of the foregoing analysis is that the minimum equivalent volume is obtained when the first- and second-stage filters are made about the same size. Although the exact solution indicates that a somewhat better balance occurs when the second filter is made slightly larger or is given more recirculation, the advantage is not sufficiently marked to offset the convenience of having duplicate units with all dimensions and pumping capacities identical.

The analysis shows the exceptionally high degree of treatment afforded by two-stage filters when the load is balanced equitably between the filters. Performance equivalent to that of standard filters may be obtained in two-stage filters with a total equivalent volume of only one-third or one-quarter of that required in standard filters.

The presentation of the principles of balanced design may be implemented with an example.

Example (v) Two-Stage Filter—Balanced Design:

Assume conditions of Example (i);

Given: w = 1,668 lb

e = 92% efficiency of combined first- and second-stage filters.

Find: The equivalent volume of first- and second-stage filters such that the total equivalent volume is a minimum.

$$e = (1 - E)w = (1 - 0.92)(1668) = 133 \text{ lb}$$

From Table 86a, for 92 percent efficiency,

$$\frac{w}{V_1F_1 + V_2F_2} = 510$$

Therefore the total equivalent volume would be

$$V_1F_1 + V_2F_2 = 1668/510 = 3.27$$

Again from Table 86a, for 92 percent efficiency, the first-stage filter should constitute 45 percent of the total, and the second-stage filter 55 percent of the total.

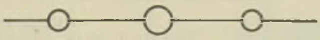
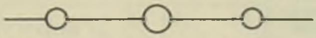
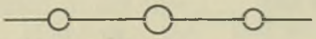
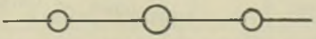
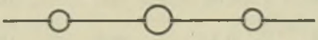
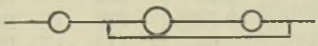
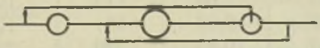
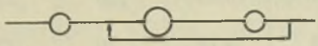
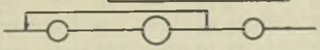
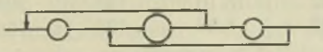
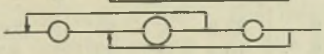
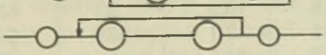
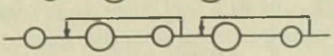
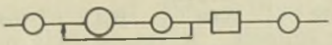
$0.45(3.27) =$ *1.47* equivalent acre-ft, first stage
$0.55(3.27) =$ *1.80* equivalent acre-ft, second stage

The BOD efficiency, however, would not differ materially from 92 percent if both filters were made of equal capacity and designed with $3.27/2 = 1.63$ equivalent acre-ft each.

It must be recognized that the foregoing analysis has been based upon a BOD rating of performance. The values of p in Equation (24) and a in Equations (25) and (29) for BOD do not necessarily apply to SS performance. Available data would appear to indicate that insofar as SS performance was concerned, a somewhat better quality of effluent was obtained when the second-stage filter had a larger volume and a smaller recirculation ratio than the first.

If the design does not include an

TABLE 87.—Nitrification Data at Various Plants

Post	Diagram	Filter Dimensions		
		Area (acres)	Depth (ft)	Volume (af)
(1)	(2)	(3)	(4)	(5)
Do-1				
Campbell		1.69	7.0	11.83
Fitzsimons G H		0.55	6.0	3.30
Benj Harrison		0.42	8.0	3.36
Swift		1.73	7.0	12.11
Leonard Wood		1.82	6.12	11.13
DR-1				
Carson		0.384	7.25	2.78
Drew Field		0.143	6.0	0.858
Jackson		0.407	5.0	2.04
Knox		0.642	5.83	3.74
SR-1				
Bragg		0.648	3.42	2.22
Kearns AAF		0.430	3.50	1.51
Two Stage				
Gordon		0.608 0.608	6.0 6.0	3.65 3.65
F E Warren		0.09 0.21	3.0 8.0	0.27 1.68
Claiborne #1*		0.433	7.0	3.03

* Trickling filters followed by activated sludge unit.

TABLE 88.—Nitrification Data at Various Plants

Post	Plant Influent		Primary Settling Tank		Filter Effluent				Final Effluent			
	Flow (mgd)	BOD (ppm)	BOD (ppm)	NH_3-N (ppm)	BOD (ppm)	NH_3-N (ppm)	NO_2-N (ppm)	NO_3-N (ppm)	BOD (ppm)	NH_3-N (ppm)	NO_2-N (ppm)	NO_3-N (ppm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Campbell	0.933	432	262	29.1	27	4.2	2.97	13.9	20	3.9	2.96	14.4
Fitzsimons G H	0.402	401	278	19.9	114	14.4	0.45	0.42	73	12.1	0.36	0.51
Benj Harrison	0.708	237	113	21.1	—	6.1	0.64	0.98	19	6.3	0.52	1.00
Swift	1.493	424	191	27.4	30	10.3	—	10.2	15	9.7	—	11.8
Leonard Wood	1.553	465	220	44.2	76	10.0	1.8	12.6	15	9.2	1.8	17.7
Carson	0.592	259	180	14.3	—	6.8	0.98	1.47	37	8.0	0.63	1.33
Drew Field	1.109	254	78	25.1	30	21.4	—	1.01	21	18.2	—	0.92
Jackson	2.835	369	252	29.0	84	23.0	0.27	0.62	61	20.4	0.33	0.47
Knox	3.106	308	138	15.7	75	14.1	0.15	0.20	23	13.4	0.94	1.00
Bragg	3.029	343	123	33.5	58	32.5	Tr	Tr	46	32.0	Tr	Tr
Kearns AAF	0.577	250	52	26.0	32	22.0	2.05	11.35	18	22.0	1.80	10.50
Gordon	1.458	342	236	22.6	88	17.4	0.71	4.71	52	16.6	0.95	4.10
F E Warren	0.559	496	410	46.1	170	44.4	0.38	0.10	55	21.9	1.20	5.75
Claiborne #1	2.129	372	177	32.0	89	24.0	0.5	0.65	13	11.3	1.0	4.8

Note: Figures are averages from Oct to Dec, 1944.

TABLE 89.—Nitrification Analysis at Various Plants

Post	Total Recircu-lation Ratio	BOD Removal—Primary Settling (%)	Nitrification Analyses				
			Filter Loading				
			MGAD	BOD		NH_3-N	
				Inf (ppm)	LB/AFD	Inf (ppm)	LB/AFD
(1)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Campbell	None	39	0.55	262	172	29.1	19.1
Fitzsimons G H	None	30	0.73	278	283	19.9	20.2
Benj Harrison	None	52	1.68	113	199	21.1	37.0
Swift	None	55	0.86	191	197	27.4	28.2
Leonard Wood	None	53	0.85	220	256	44.2	51.5
Carson	2.74	30	5.76	76	320	9.7	25.4
Drew Field	1.17	64	16.80	52	974	22.6	298.0
Jackson	1.60	32	18.10	135	2930	23.8	337.0
Knox	0.77	27	8.56	138	1563	15.7	117.0
Bragg	6.61	0	35.6	89	3900	33.0	420.0
Kearns AAF	13.30	15	19.2	32	678	23.0	142.0
Gordon	1.96	31	7.1	138	392	19.1	37.6
F E Warren	5.45	17	34.5	—	980	—	101.0
Claiborne #1 (b)	0.20	52	5.05 (a) 5.89	163	1040	31.0	188.0

(a) Flows to 1st and 2nd filters, respectively.
(b) All figures apply to 1st stage of filter plants only.

TABLE 90.—Nitrification Analysis at Various Plants

Post	Filter and Secondary Settling Performance							
	BOD			Overall BOD Removal (%)	NH_3-N		NO_2 and NO_3-N	
	LB/AFD Removed	Efficiency (%)	Removal (%)		LB/AFD Removed	Efficiency (%)	LB/AFD Produced	LB per LB of BOD Removed
(1)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)
Campbell	159	92	56	95	16.5	87	11.40	0.072
Fitzsimons G H	210	74	51	82	8.0	39	0.88	0.004
Benj Harrison	165	83	40	92	25.9	70	2.68	0.016
Swift	181	92	42	96	18.2	65	12.10	0.067
Leonard Wood	238	93	44	97	40.7	79	22.60	0.095
Carson	256	80	56	86	11.1	44	3.49	0.014
Drew Field	745	77	27	91	100.4	34	8.75	0.012
Jackson	2,220	76	52	84	101.0	30	9.20	0.004
Knox	1,402	90	65	92	23.8	20	13.42	0.010
Bragg	3,380	87	87	87	56.4	13	0.009	Tr
Kearns AAF	619	91	78	93	71.4	50	39.0	0.063
Gordon (a)	306	78	54	85	9.9	26	8.4	0.027
F E Warren (a)	848	87	72	89	57.7	52	16.6	0.020
Claiborne #1 (b)	518	50	24	76	47.0	25	7.75	0.015

(a) Two-stage filtration.
(b) All figures apply to 1st stage or filter plants only. In the activated sludge unit 0.06 lb of NO_2 and NO_3 were produced per lb of BOD removed.

intermediate settling tank, a somewhat lower degree of treatment may be expected than that indicated in the foregoing formulation. However, the analysis for balanced design retains validity if some adjustment is made for less satisfactory settling.

Nitrification

In normal operation at the majority of military sewage treatment plants routine nitrogen determinations were not made. In order to obtain information relating to nitrification occurring in various types of filters, a special study was made at a selected group of plants during the last four months of 1944. It should be stated that the scope of the special nitrogen study was limited to a determination of the *amount* of nitrification that occurred at different plants, and not to the *value* of nitrification.

The data are presented in Tables 87 and 88 and are analyzed in Tables 89 and 90. Since post strengths generally declined during September to December, 1944, the loading at many of the plants differed somewhat from that during the main period of investigation (January, 1943 to August, 1944). Accordingly, BOD data are included in the nitrification tables in order to indicate the loading of the various plants.

In plants employing recirculation, the weights of BOD and NH_3 applied to the filter (columns 18 and 20) were based upon raw settled sewage in accordance with Equation (3).

Three of the five standard filters (Division I) produced a considerable amount of nitrite and nitrate. In the standard filters at Fitzsimons General Hospital and Fort Benjamin Harrison, nitrification was small. These plants operated with a lower overall BOD removal than other standard filters.

During the autumn of 1944 no particular change in nitrifying activity could be discerned with the decrease in temperature. Data are given for three posts in the following table:

Post	1944 Data	Plant Flow (mgd)	Raw Sewage BOD (ppm)	Plant Effluent BOD (ppm)	NO_3-N in Plant Effluent (ppm)
Camp Swift	Sep	1.60	393	13	11
	Oct	1.52	411	12	12
	Nov	1.59	447	15	13
	Dec	1.27	452	23	11
Fort Leonard Wood	Oct	1.82	449	12	17
	Nov	1.66	457	16	16
	Dec	1.18	497	17	21
Fort F E Warren	Oct	0.65	451	67	3
	Nov	0.61	525	50	7
	Dec	0.42	526	45	9

None of the single-stage high rate filters that were investigated produced a marked degree of nitrification. The plant at Kearns AAF, while designed as a high rate filter, was loaded to only a small fraction of capacity, and during this period operated with a recirculation ratio of 13.3. The sum of nitrite plus nitrate at this post was 12.30 ppm—a value comparable to that of the standard filters in Division I. The two-stage plant at Camp Gordon, which was also loaded in the standard filter range, produced an effluent with 4.10 ppm of NO_3. At Fort F E Warren a concentration of 5.75 ppm of nitrate was produced with a BOD loading of 980 lb BOD per acre-ft daily.

The BOD loadings at the two-stage plants were calculated on the basis of weight of BOD in raw settled sewage per acre-ft of combined first- and second-stage filter volume. The plant at Fort Warren was the only one with filters loaded beyond the normal range of standard filters that produced a significant amount of nitrate. However, on a total filter volume basis, this plant was not loaded heavily.

Although data were not available, it is probable that nitrification did fall off during the subsequent winter months, at least in the northern plants.

Data for the plant at Camp Claiborne No. 1 are included as a matter of interest. This plant consisted of a filter followed by an activated sludge unit. The figure of 4.8 ppm of nitrate for Claiborne in Table 88 pertains to the activated sludge effluent. All figures in Tables 89 and 90, however, apply only to the filter plant, and do not indicate the action of the activated sludge unit. The filter did not induce much nitrification; results at Claiborne are in line with those at other filters that were loaded beyond the standard filter range. A moderate amount of nitrification took place in the activated sludge units.

In generalizing the nitrogen results, the most useful measure of the degree of nitrification appeared to be the BOD loading of filters on an equivalent acre-ft basis. While the filter loadings in Table 89, column 18 are based upon actual volumes, a better correlation with nitrate (and nitrite) concentration was obtained with equivalent volumes. The data would indicate that no significant degree of nitrification occurred in plants with a daily BOD load in excess of 500 lb per equivalent acre-ft. A few plants, however, such as those at Fort Benjamin Harrison and Camp Carson, did not produce a significant amount of nitrate despite low loading.

The data in Tables 89 and 90 are plotted in Figures 91 to 94. The curves have been plotted through group medians. Figure 92 indicates the variation in the efficiency of ammonia removal with various loadings. This graph is similar to Figures 86 and 87. Thirty to forty percent of the applied ammonia was removed. In high rate plants this removal was brought about by assimilation of ammonia in the synthesis of filter growths that constantly sloughed and appeared as secondary

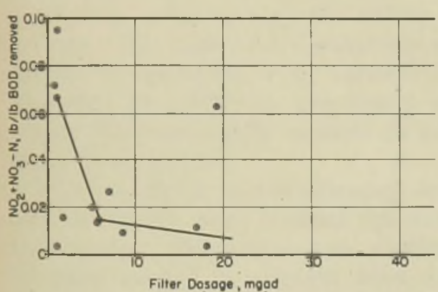


FIGURE 91.—Relation between the pounds of nitrite and nitrate nitrogen produced per pound of BOD removed and the filter dosage in mgad.

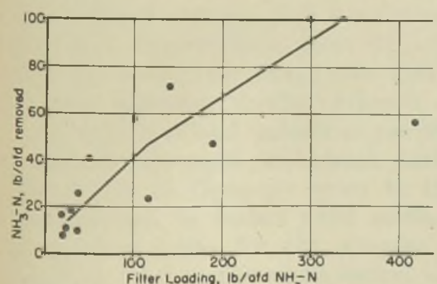


FIGURE 92.—Relation between the loading and removal of ammonia nitrogen.

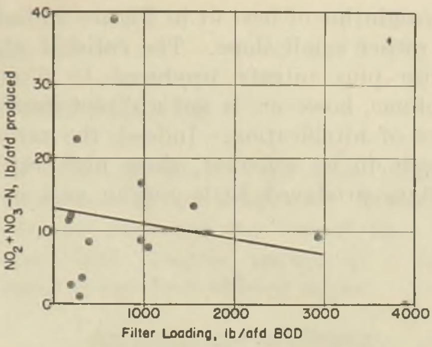


FIGURE 93.—Relation between the amount of nitrite and nitrate nitrogen produced and the filter BOD loading.

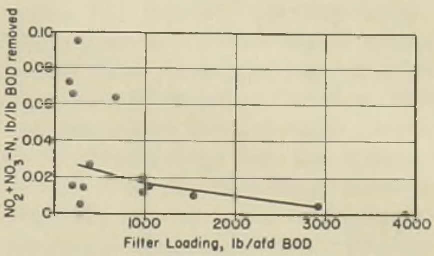


FIGURE 94.—Relation between the pounds of nitrite and nitrate nitrogen produced per pound of BOD removed and the filter BOD loading.

sludge. In some of the low rate plants, a considerable part of the applied ammonia was oxidized to nitrites and nitrates, as may be seen by comparing columns 25 and 27 in Table 90. The data in Figure 92 indicate a significantly greater relative ammonia removal with light loading than with heavy loading.

Figures 91, 93, and 94 show in various ways the decrease in nitrification with increase in rates of load application. In Figure 91, the plotted point for Kearns AAF stands apart from the configuration of the others; this is accounted for by the unusually high recirculation ratio used. The plant was loaded to only a small fraction of capacity. Figure 91 gives further evidence that the volumetric rate of dosage (mgad) was not a suitable parameter for measuring performance in plants with recirculation. The straightline of best fit in Figure 93 has a rather small slope. The ratio of nitrite plus nitrate produced to filter volume, however, is not a direct measure of nitrification. Indeed, the ratio tends to be constant, since high rate filters produced little nitrite and nitrate but at the same time had relatively small volumes. A better method of indicating variations in nitrification with loading is presented in Figure 94, which clearly shows the falling off of nitrite and nitrate production with increased rate of load application.

CHAPTER VI

CONTACT AERATION PLANTS

Selection of Plants

Contact aeration plants were constructed at about 50 Army installations and at numerous Navy installations. Except for minor variations, these plants were all of one type, but modifications or additions often had to be made in order to secure satisfactory operation. Under favorable conditions of loading and sewage concentration, good effluents were obtained, with overall removals of BOD averaging 80 percent to 95 percent in the plants selected for study. When strong stale sewage had to be treated, however, or when difficulties developed in the aeration system originally installed, effluents were unsatisfactory, odors became intense and first-aid measures had to be applied in order to keep the plants in service. By close and continuous attention to operation or by suitable remodeling or additions practically all plants were eventually enabled to continue in operation.

Five plants for which adequate data were available were selected for complete study. The raw sewage received by them ranged in strength from 186 to 357 ppm of BOD and from 233 to 265 ppm of suspended solids. These five plants were considered to be representative of the more successful plants of this type. Other plants, in which such radical changes had been necessary as to remove them from the category of contact aeration, were visited or their operating results reviewed.

In municipal and industrial practice numerous attempts had been made, principally in Germany prior to this development, to install fixed surfaces in aeration tanks for the purpose of securing the benefits of biological oxidation and precipitation inherent in both the activated sludge process and in trickling filters. All such plants, experimental or otherwise, however, had not survived the test of time in municipal practice, except in a few small towns in Southern states.

Arrangement of Plants

The type of contact aeration plant constructed at military installations comprised (1) preliminary settling tanks, (2) primary aeration units in which vertical asbestos plates are supported above horizontal pipe-grid aerators, (3) intermediate settling tanks, (4) secondary aeration units identical as to arrangement but not always as to size with the first-stage units and (5) final settling tanks. The bases of design specified in the Army Engineering Manual were:

(1) Preliminary settling—2.5 hr.
(2) Primary aeration—156 sf of surface per lb of BOD applied daily (6.4 lb of BOD applied daily per 1,000 sf).*
(3) Intermediate settling—1.5 hr.
(4) Secondary aeration—same loading as for primary aeration.
(5) Final settling—2.5 hr.

The tanks were frequently grouped on three sides of a rectangle, with the primary settler and primary aerator in series on one side, the intermediate settler at right angles on the second side and the final aerator and final settler in series on the third side, the central area being used for the housing of control devices, valves and sludge receivers.

The asbestos plates which provided the contact surface were usually flat but sometimes corrugated. They were

* As shown later in this chapter, the daily BOD loading in lb per sf-hr appears to be a more relevant measure.

about ¼ in. thick, usually 4 by 8 ft in plan and hung or supported at right angles to the direction of displacement with approximately 1.5-in. spaces between the plates. The top of the plates was submerged about 4 in. Air was admitted below the plates by a pipe grid perforated with orifices 1/16 to 1/8 in. in diameter spaced to direct the rising air bubbles between the plates, and sufficient in number to provide uniform distribution of air.

Below the air grid the tanks were constructed with hopper bottoms for sludge collection and removal. In spite of the slope of the hoppers, however, sludge removal was not always found to be complete. At certain plants (Hondo and Waco No. 1), therefore, the hopper was later filled in so that sludge could not accumulate below the contact units. In some larger plants (such as Hood) sludge collecting mechanisms were installed in troughs under the air grids.

Description of Plants

In Tables 91 to 100 is presented an analysis of the design, loading, operation and performance of the five contact aeration plants included in this study. Although this discussion is limited to these five plants, they are considered sufficient in number and variety to demonstrate the characteristics of contact aeration for the treatment of sewage at military installations. The five plants are: Greenville AAF, Miss.;

TABLE 91.—List of Contact Aeration Plants

Post	State	Sv C or Navy	Type of Station	Date Plant Completed	Date Major Additions or Alterations Completed	Operating Period Studied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Greenville AAF, Miss	Miss	4th	Tact Field & Fly Trg	Apr 42	—	Jul 44 to Nov 44
Hondo AAF	Texas	8th	Tact Field & Fly Trg	—	—	May 43 to Jun 43
Kingsville Field	Texas	IV	Naval Air Station	Feb 43	—	Apr 43 to Jul 44
Roswell AAF	N Mex	8th	Tact Field & Fly Trg	Jul 42	May 43	Jul 44 to Mar 45
Camp Rucker	Ala	4th	Div Trg	Aug 42	—	Jul 44 to Nov 44

TABLE 92.—General Data Relating to Contact Aeration Plants

Post	Approx Plant Elev (ft)	Range of Sewage Influent Temp (°F)	Average Sewage Influent pH	Lineal Feet of Sewers per Capita (a)	Sampling	
					Hours per Day	Average Number per Month
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Greenville AAF, Miss	—	82–88	7.8	—	24	30
Hondo AAF	—	—	7.6	—	24	20
Kingsville Field	—	75–91	7.5	7.2	24	8
Roswell AAF	—	73–100	7.6	21.8	24	9
Rucker	—	67–87	7.3	11.5	24	30

(a) Based on design population.

TABLE 93.—Units in Contact Aeration Plants

Post	Plant Units												
	Screens or Racks	Comminutors	Grit Chambers	Primary Settling Tanks	First Stage Aerators	Intermediate Settling Tanks	Second Stage Aerators	Final Settling Tanks	Chlorine Contact Tanks	Digestion Tanks—Heated	Digestion Tanks—Unheated	Sludge Beds	Other Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Greenville AAF, Miss	1	0	0	1	1	1	1	1	1	1	0	4	—
Hondo AAF	1	0	0	2	2	2	2	2	0	0	1	4	—
Kingsville Field	1	1	0	1	1	1	1	1	1	0	1	3	—
Roswell AAF	1	1	0	1	1	1	1	1	0	1	1	9	—
Rucker	1	0	1	2	2	2	2	3	0	2	2	11	(a)

(a) Two grease flotation tanks.

TABLE 94.—Design Capacities and Loading of Contact Aeration Plants

Post	Number of Months in Operating Period Studied	Population				Flow				
		Design Capacity (a)	Actual			Design (mgd) (a)	Actual			
			Average	Maximum	Minimum		Average (mgd)	Maximum (mgd)	Minimum (mgd)	Gal per Capita Daily
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Greenville AAF, Miss	5	3,780	3,276	4,120	2,780	0.264	0.355	0.550	0.159	108
Hondo AAF	2	5,160	7,420	—	—	0.362	0.556	—	—	75
Kingsville Field	15	6,570	3,908	4,887	2,458	0.460	0.446	—	—	114
Roswell AAF	9	4,190	5,740	6,474	5,479	0.293	0.429	0.785	0.095	75
Rucker	5	27,000	—	—	—	1.890	1.594	2.930	0.510	—

(a) As based upon area of contact plates.

Hondo AAF, Texas; Kingsville Field, *IV* Naval Area, Texas; Roswell AAF, New Mexico; and Camp Rucker, Ala.

TABLE 95.—Percent of Design Capacity Utilized in Contact Aeration Plants

Post	Primary Settling Tanks (%)	Aeration Tanks (%)	Final Settling Tanks (%)
(1)	(2)	(3)	(5)
Greenville AAF, Miss	84	87	107
Hondo AAF	105	144	55
Kingsville Field	87	60	111
Roswell AAF	82	137	105
Rucker	77	—	84

Performance of Individual Plants

Greenville AAF, Miss.

The Greenville AAF plant was completed in April, 1942. It was designed for a population of 3,780. The operating period studied extended from July through November, 1944. During this period the connected population averaged 3,276, or 87 percent of design, and the sewage flow averaged 0.355 mgd, or 108 gcd. At average flow, the primary settling tanks provided 2.98 hr of detention and reduced the BOD from 273 to 138 ppm, or 49 percent, and the suspended solids from

TABLE 96.—Design and Description of Aeration Tanks

Post	Design								Operation			
	No of Aeration Tanks	Aeration Volume (cf) (a)	Aeration Tank Surface Area (sf)	Type of Contact Plates	Total Number of Orifices	Size of Orifice Opening (in.)	No of Blowers	Total Rated Capacity (sf/min)	Appld Air (cf/gal)	CF/MIN per SF Tank Surface	CF per LB BOD Appd	CF per LB BOD Rem
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Greenville AAF, Miss	2	6,500	650	corr	1,343	1/8	4	750	1.99	0.75	1,725	2,250
Hondo AAF	2	6,650	740	plane	2,660	5/64	2	1,000	1.35	0.70	1,455	2,410
Kingsville	2	7,440	993	plane	5,440	1/8	3	600	1.75	0.55	2,250	3,170
Roswell AAF	2	8,195	826	—	—	—	4	640	2.15	0.78	1,375	1,890
Rucker	4	65,500	5,460	plane	63,320	1/8	3	7,000	4.21	0.86	2,560	2,990

(a) Total volume including contact plants.

TABLE 97.—BOD Loading and Performance

Post	Average (mgd)	Primary Settling			
		Average Detention Time (Hours)	BOD Influent (ppm)	BOD Effluent (ppm)	Removal (%)
(1)	(3)	(4)	(5)	(6)	(7)
Greenville AAF, Miss	0.355	2.98	273	138	49
Hondo AAF	0.556	2.38	215	111	48
Kingsville Field	0.446	2.86	186	93	50
Roswell AAF	0.429	3.04	310	187	40
Rucker	1.594	1.62	357	197	45

TABLE 98.—BOD Loading and Performance

Post	First Stage Aeration and Settling								
	Aeration Tank				Intermediate Settling Tank				
	LB BOD per 1000 SF Contact Surface	LB BOD per 1000 CF Aeration Volume	Av Detention Time (hr)	D.O. (ppm)	Av Detention Time (hr)	D.O. (ppm)	Eff BOD (ppm)	Eff (%)	Removal (%)
(1)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
Greenville AAF, Miss	12.9	119.0	1.49	2.4	2.34	1.2	51	63	32
Hondo AAF	12.3	155.0	0.86	0.3	1.06	0.0	77	31	16
Kingsville Field	6.8	93.0	1.18	—	1.57	0.6	55	41	21
Roswell AAF	17.1	142.0	1.72	1.4	1.16	—	107	43	26
Rucker	10.9	77.0	3.5	—	2.23	—	78	60	33

TABLE 99.—BOD Loading and Performance

Post	Second Stage Aeration and Settling									Over-all Re-moval (%)
	Aeration Tank				Final Settling Tank					
	LB BOD per 1000 SF Contact Surface	LB BOD per 1000 CF Aeration Volume	Av Detention Time (hr)	D.O. (ppm)	Av Detention Time (hr)	D.O. (ppm)	BOD Eff (ppm)	Eff (%)	Re-moval (%)	
(1)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)
Greenville AAF, Miss	5.30	48.7	1.35	1.4	2.3	1.0	32.0	37	7.0	88.0
Hondo AAF	8.55	107.0	0.86	0.9	4.6	0.0	44.0	43	15.5	79.5
Kingsville Field	3.99	54.9	1.18	—	2.2	2.1	27.0	51	15.0	86.0
Roswell AAF	9.78	110.0	0.95	4.0	2.4	3.7	51.0	52	18.0	84.0
Rucker	4.66	33.0	3.24	—	3.1	3.8	28.0	64	14.0	92.0

TABLE 100.—SS Loading and Performance

Post	Suspended Solids										
	Plant Influent		Primary Treatment Section		Secondary Treatment Section						Over-all Re-moval (%)
					Intermediate Settling Tank			Final Settling Tank			
	PPM	LBS per Capita Daily	Eff (ppm)	Re-moval (%)	Eff (ppm)	Effi-ciency (%)	Re-moval (%)	Eff (ppm)	Effi-ciency (%)	Re-moval (%)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Greenville AAF, Miss	256	0.231	90	65	33	63	22	19	42	5.5	92.5
Hondo AAF	233	0.146	65	72	50	23	6.5	31	38	8.2	86.7
Kingsville Field	257	0.244	105	59	70	33	13.6	45	36	9.7	82.3
Roswell AAF	264	0.165	106	60	54	49	19.7	30	44	9.1	88.8
Rucker	265	—	141	47	69	51	27.2	20	82	18.5	92.7

256 to 90 ppm, or 65 percent. The gross aeration volume of the contact units was 3,400 cf for the first stage and 3,100 cf for the second stage, a total of 6,500 cf. At average flows this provided a net aeration period of 1.49 hr in the first stage and 1.35 hr in the second, a total of 2.84 hr. The total gross aeration period was 3.3 hr.

The surface area of the contact plates was 24,420 sf in the first stage and 21,780 sf in the second stage, a total of 46,200 sf. Owing to the corrugations, the effective areas of contact were 20 percent to 25 percent larger than the foregoing figures. Tank surface areas were 340 sf in the first stage and 310 sf in the second stage, a total of 650 sf.

In the first stage, the applied BOD of 138 ppm represents a daily loading of 12.9 lb per 1,000 sf of contact surface, or 119.0 lb per 1,000 cf of gross aeration volume. In this stage the BOD was reduced to 51 ppm.

The daily loading of the second stage (51 ppm of BOD in 0.355 mgd) was 6.9 lb per 1,000 sf of contact surface, or 48.7 lb per 1,000 cf of gross aeration volume.

Air consumption was 1.99 cf per

gallon, 0.75 cfm per sf of tank area, 1,725 cf per lb of BOD applied and 2,250 cf per lb of BOD removed.

The final BOD was 32 ppm, giving an overall reduction of 88.0 percent. The final SS content was 19 ppm, an overall reduction of 92.5 percent.

The results obtained were satisfactory but the air requirement was quite high per lb of BOD removed as compared to activated sludge experience with domestic sewage, where values vary within the range of 500 to 1,500 cf of air per lb of BOD removed. The shallower depth employed in the contact aeration process, however, entails less power for compression of air.

Hondo AAF, Texas

The Hondo AAF plant was completed in 1942. The operating period studied ran from May through October, 1943. During this period and later, various experimental modifications were investigated in attempts to improve aeration and other operating difficulties. With the entire plant in operation, the aerators were underloaded with respect to the design criterion of 6.4 lb of applied BOD per 1,000 sf of contact surface. In order to increase the loading, flow was routed through one-half the plant. The data pertaining to Hondo AAF in Tables 95 to 100 are based upon this method of operation. Under this condition Hondo received a loading in excess of design averaging 7,420 population, or 144 percent of the design population of 5,160. The design flow was 0.362 mgd and the actual flow 0.556 mgd, or 75 gcd.

Preliminary settling reduced the BOD from 215 to 111 ppm, or 48 percent, and the suspended solids from 233 to 65 ppm, or 72 percent. This high efficiency of clarification was accomplished in 2.38 hr of detention. Aeration periods were only 0.86 hr in both stages, a total of 1.72 hr. This resulted in high volumetric and surface loadings, even though the applied BOD was only 111 ppm. The BOD was reduced to 77 ppm after intermediate settling and to 44 in the final effluent, yielding an overall removal of only 79.5 percent. Suspended solids were reduced from 233 to 31 ppm, or 86.7 percent.

Air consumption at Hondo was the lowest among the five plants studied when it was expressed as cf per gallon (1.35), but it was not low in terms of cf per lb of BOD removed (2,410).

The experimental work carried out at Hondo and the results obtained are discussed later.

Kingsville Field, Texas

The Kingsville Field plant was completed in February, 1943. The operating period selected was from April, 1943 through July, 1944. Kingsville is a Naval air station with a design population of 6,570 but having an actual connected population of only 3,908, or 60 percent of design. The flow was 0.446 mgd, or 114 gcd, resulting in the weakest sewage among the five plants surveyed. Primary settling reduced the BOD from 186 to 93 ppm, or 50 percent, and the suspended solids from 257 to 105 ppm, or 59 percent.

Surface and volume loadings were low, and the contact aeration periods were 1.18 hr in each of the two stages.

Air consumption was 1.75 cf per gallon (a large amount considering the low BOD applied) and averaging 2,250 cf per lb of BOD applied, or the excessive amount of 3,170 cf per lb of BOD removed. In spite of this large amount of air the intermediate BOD was 55 ppm, being reduced to this figure from 93 ppm in the primary effluent, and the final BOD was high (27.0 ppm), giving an overall BOD removal of 86.0 percent. The suspended solids were reduced from 257 ppm in the raw sewage to 45 ppm final, or 82.3 percent. These removals are disappointing in view of the low

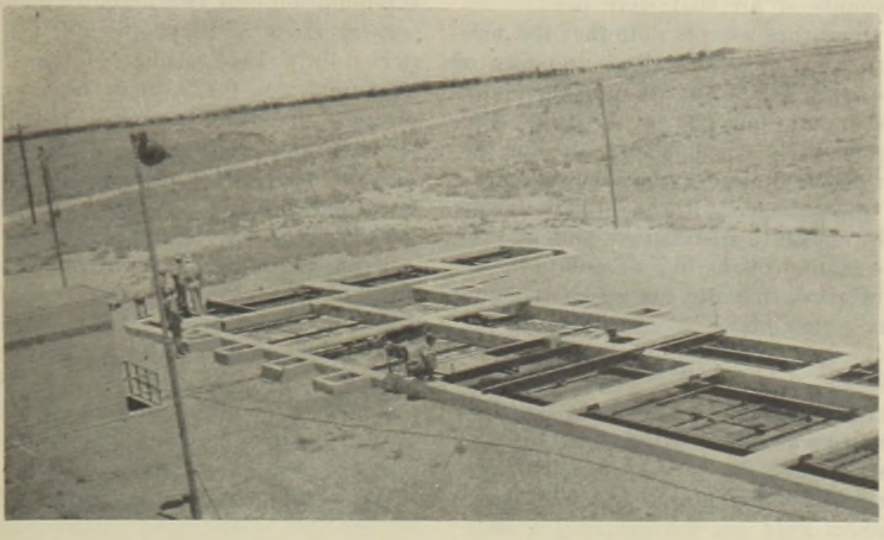


PLATE 17.—Contact aeration plant as Roswell AAF, N. Mex. Three settling tanks and four aeration basins are shown.

PLATE 18.—Contact aerators, showing plate arrangement at Hondo AAF, Texas.

loading that obtained and the excessive amounts of air that were used.

Operating records note that the aerators were cleaned daily. In spite of the large amount of air used the operator stated that one of the chief difficulties was "insufficient air for the proper oxidation of the organic matter of the sewage." Undoubtedly the air was unevenly distributed and allowed the plate growths to accumulate in certain areas that did not receive enough air.

Roswell AAF, New Mexico

The Roswell AAF plant in New Mexico was completed in July, 1942. A new blower was added in 1943, also an additional digester and five sludge drying beds. The plant, designed for a population of 4,190, actually handled sewage from 5,740 people during the period July, 1944 through March, 1945. The flow was 0.429 mgd, or 75 gcd, giving a detention period of 3.04 hr in the primary settling tank and reducing the BOD from 310 to 187 ppm, or 40 percent and the suspended solids from 264 to 106 ppm, or 60 percent.

First-stage aeration reduced the BOD of the settled sewage from 187 to 107 ppm and second-stage aeration gave a reduction to 51.0 ppm, giving an overall removal of 84.0 percent. Suspended solids were reduced from 264 (raw) to 30 ppm (final), or 88.8 percent.

Air supply was high per gallon (2.15

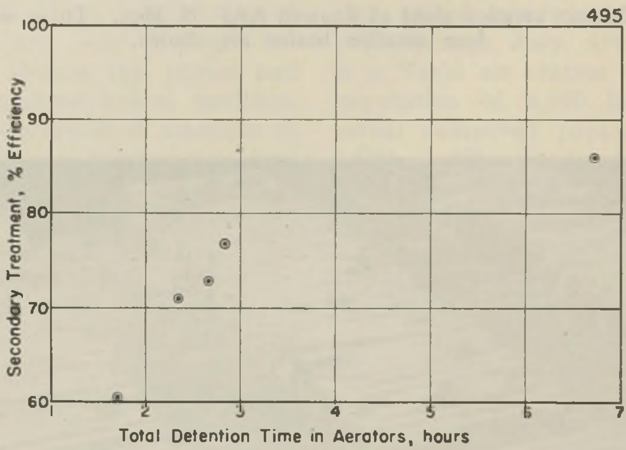


FIGURE 95.—Relation between detention period in aeration basins and BOD efficiency. BOD reduction includes that in both aeration basins, intermediate, and final settling tanks.

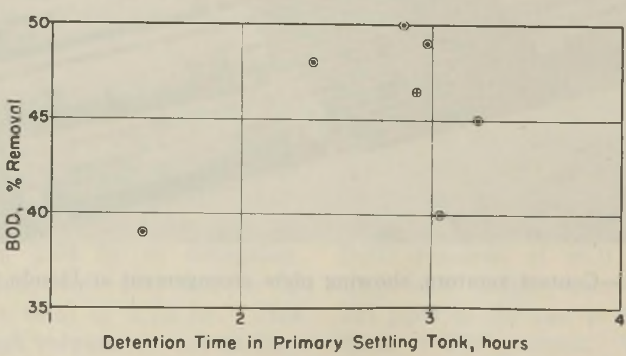


FIGURE 96.—BOD reduction in primary settling tanks of contact aeration plants.

cf) but it was the lowest among the five plants per lb of BOD applied and removed (1,375 and 1,890 cf, respectively).

Camp Rucker, Ala.

Camp Rucker was a large training camp in Alabama for which a contact aeration plant was designed and installed in August, 1942. The results of operation were unsatisfactory, with a poor effluent in 1942 and 1943. An activated sludge plant was accordingly designed and installed in 1944, practically doubling the treatment facilities and providing for an expected population of 59,000 which, however, was not reached. The population actually declined to around 21,500 during the period of this survey, July through November, 1944. Consequently the contact aeration plant was required to handle a population below the design figure of 27,000, and all units were lightly loaded. The flow was 1,594 mgd, compared with the design flow of 1.890 mgd.

Primary settling reduced the BOD from 357 to 197 ppm, first-stage contact aeration to 78 ppm, and second-stage aeration and final settling to 28.0 ppm, giving an overall reduction of 92 percent. Suspended solids were reduced from 265 in the raw sewage to 20 in the final effluent, a reduction of 92.7 percent.

A large amount of air was available from one 5,000-cfm and two 1,000-cfm blowers. Consequently the air consumption averaged 4.21 cf per gallon, or 2,560 cf per lb of BOD applied daily and 2,990 cf per lb of BOD removed. These are excessive quantities of air.

Discussion of Results

As shown in Figure 96, the efficiency of primary settling was relatively high in the plants studied. Comparison may be had with Figure 68, since all but one of the sewages had a BOD below 350 ppm.

If the detention periods provided in three settling tanks of the process are totalled, it is found that they aggregate 5.0 to 8.0 hr, while the combined detention periods in the two aeration tanks varied from 1.7 to 6.7 hr. Relative to the settling allowance, therefore, the aeration period was brief, thus accounting for the high loadings of BOD per 1,000 cf of net aeration volume and suggesting that contact aeration, as practiced, was in the nature of high rate operation with resulting inferior effluents.

The question arises as to what is the governing design or operating factor of contact aeration units. Is it the amount of air used, or is it the aeration period or the contact surface or a combination of the last two items? The individual relations to BOD efficiency, based on settled sewage, were as follows:

Post	BOD Efficiency (%)	Air Supply		Net Aeration Period (hr)	Contact Surface (lb of BOD applied per 1,000 sf-hr)
		CF per Gallon	CF per LB of BOD Removed		
Hondo	60.4	1.35	2,410	1.72	3.56
Kingsville	70.8	1.75	3,170	2.36	1.44
Roswell	72.8	2.15	1,890	2.67	3.21
Greenville	76.8	1.99	2,250	2.84	2.40
Rucker	85.8	4.21	2,990	6.74	0.84

As shown in Figure 95, the efficiency of the plants studied increased apparently systematically as the overall aeration period was lengthened. A similar relationship can be demonstrated for overall air supply (cf per gallon), but no similar systematic trend could be shown with the other individual design or operating factors. It stands to reason, however, that the extent of contact surface must play a part, and it is shown in the following schedule and in Figure 97 that it does in fact, if it is recognized that it is the repeated sweeping of the sewage past the growths on the contact plates that must be the basis of contact aeration as a biological sewage treatment process.

Post	BOD Efficiency (%)			BOD Applied per 1,000 SF-HR of Contact Aeration		
	First Stage	Second Stage	Over-all	First Stage	Second Stage	Over-all
Hondo	31	43	60.4	14.2	9.9	3.56
Kingsville	41	51	70.8	5.8	3.4	1.44
Roswell	43	52	72.8	10.0	10.3	3.21
Greenville	63	37	76.8	8.6	4.0	2.40
Rucker	60	65	85.8	3.1	1.4	0.84

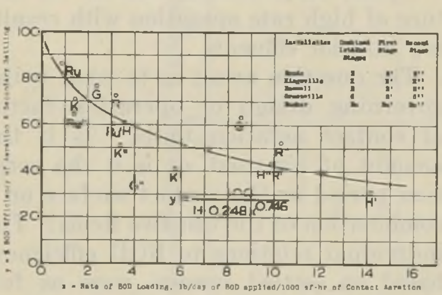
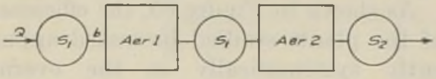


FIGURE 97.—Efficiency of contact aerators. Plotted points indicate performance of first stage, second stage and combined first and second stages at five contact aeration plants.

The validity of the parameter, BOD applied daily per 1,000 sf of contact surface per hour of aeration, may be explained with the aid of the following diagram of a contact aeration plant:



where

Q = flow.

A_1, A_2 = area of contact plates in first-stage and second-stage aerators, respectively.

V_1, V_2 = vol (net) of first-stage and second-stage aerators, respectively.

T_1, T_2 = detention period in the first-stage and second-stage aerators, respectively.

b = wt BOD in settled sewage applied to first-stage aerator.

It is evident that the efficiency of the process is dependent upon (1) the total amount of biological film, which is proportional to the area of contact, and (2) the time of contact of the sewage with the contact surface, as measured by the detention period in the aeration basins. The maintenance of aerobic conditions in the flowing sewage is, however, a prerequisite.

With the foregoing notation, the efficiency, e_1, of BOD reduction in the first aeration basin and intermediate settling tank would be a function of the quantity b/A_1T_1, where $T_1 = V_1/Q$. In the second stage, the efficiency, e_2, is measured by the quantity $b(1 - e_1)/A_2T_2$, where $T_2 = V_2/Q$. Also, for the over-all secondary treatment section, the efficiency would be determined by the quantity $b/(A_1 + A_2)(T_1 + T_2)$. In a sense, the quantity $A \times T$ may be termed "surface-hours;" when it is large relative to the load, high efficiencies may be expected. On the other hand, if the loading is high relative to the surface-hours available in the plant, low efficiencies will result.

Consider the effect upon the efficiency of the first stage of aeration at a plant with the quantities b and A_1 fixed and the volume V_1 (conceived of as being) variable. If V_1 is compressed to, say, half its initial value so that the spacing of plates is reduced 50 percent, the parameter b/A_1 remains unchanged. The efficiency, however, will surely be reduced since the average particle of sewage must now pass through the tank in half the time formerly available for biochemical activity.

An interesting alternate interpretation of the parameter "weight per area-time" may be derived from the definition of detention period, $T = V/A$, as follows:

$$\frac{b}{AT} = \frac{b}{A} \times \frac{Q}{V}$$

The parameter is resolved into two components, both of which have value as a measure of efficiency, namely: (1) weight of BOD per unit area of contact surface and (2) volume of flow applied per unit time per unit volume of aeration tank. Thus the quantity on the horizontal axis of Figure 97 is expressive of both organic and volumetric load—neither of which taken separately constitutes a wholly satisfactory measure of performance.

It is significant to note that, taken as a group, the second stage points on Figure 97 follow the trend exhibited by the first-stage points. This would indicate that no significant difference existed between the kind of organic material entering the second stage and the first. The "treatability" of the material entering the second stage is about the same as in the raw settled sewage. Accordingly, the efficiency is a function only of magnitude of the load relative to the "surface-hours" available.

This finding is at variance with the concept of this type of plant as a "condensed stream" where self-purification is carried forward by a succession of different organisms, each utilizing the waste products of the preceding species. The analogy presumed by some to exist with the metabiosis of stream purification would appear to lack factual basis in the contact aeration process. It seems more likely on the basis of the foregoing evidence that the greater part of the stabilization is attained within the zoogleal matrix where sewage matters are first absorbed from fluid, and that soluble end products, with exception of perhaps ammonia and nitrite, are relatively dilute and unavailable to downstream growths. Coagulated material and synthesized growths are also unavailable to subsequent microbe species, since this material continuously sloughs off and settles, leaving the aerobic regime entirely.

Contact aeration, when operating under favorable conditions, would appear to have a purification mechanism not greatly different *from a biological viewpoint* from other conventional methods of sewage treatment such as trickling filtration and activated sludge. From the standpoint of maintaining and controlling the physical environment (rate of transfer of dissolved gases, hydraulic turbulence, etc.) in the vicinity of the substrate, there would appear to be very real differences between contact aeration and other established methods of treatment.

Figure 97 may be used to ascertain the performance that normally would be expected with a plant designed in accordance with criteria set forth by the Army Engineering Manual. This may be shown by an example:

Assume: 2,000 population; 0.2 lb BOD/cap/day; 70 gcd, and 40 percent removal of BOD in primary settling.

Weight of BOD in raw settled sewage = 0.20 (1.00 − 0.40)(2,000) = 240 lb BOD applied. This would require $\frac{240}{6.4} \times 1{,}000 = 37{,}500$ sf of contact area.

Assuming an aeration volume of 90 cf per 1,000 sf of contact area (average value based upon a number of plants as constructed), the volume of the aeration units would be 37.5 × 90 = 3,370 cf or 25,300 gallons.

The plant flow would be $2,000 \times 70 = 140,000$ gpd.

The detention period during aeration (including both first and second stage) would be $\frac{25,300(24)}{140,000} = 4.34$ hr.

The loading of the aerators on a "surface-hour" basis would accordingly be $\frac{240}{37.5 \times 4.34} = 1.48$ lb BOD applied per 1,000 sf-hr.

From Figure 97 the efficiency of the aerators would be about 75 percent. The weight of BOD passing into the effluent would amount to $240(1 - 0.75) = 60$ lb, which corresponds to an overall plant removal of

$$\frac{400 - 60}{400} = 85\%.$$

The effluent of the plant would contain

$$\frac{60}{8\frac{1}{3}(0.140)} = 51 \text{ ppm BOD}.$$

Operating Problems

Many contact aeration plants required rigid attention to prevent the release of hydrogen sulfide odors. Occasionally a milky appearance was imparted to the sewage in the first-stage aerator when hydrogen sulfide was converted to free sulfur. The space between the plates had to be cleaned frequently. In some plants this was accomplished by means of an air-jetting pipe which was inserted between the plates. Sludge was freed and fell to the bottom of the tank but plate growths soon reached their original bulk again.

On the other hand some plants, such as those at Blacklands, Rucker, and Greenville, S. C., were eventually practically free from odor, and green growths were present on the plates and dissolved oxygen in the first aerator effluent. Light loadings and high water consumption (over 100 gcd) contributed mainly to this satisfactory condition whenever uniform air distribution could be maintained.

One of the major problems encountered was the maintenance of uniform air diffusion through the perforated pipe grids. Special investigations were undertaken at Hondo to accomplish this. Results were reported in May, 1944.

Orifices in the black iron pipe tended to enlarge by rusting, or else to plug up with slime or calcium carbonate. In order to get at the air grids, it was necessary to drain the tanks, remove part of the contact plates and to build a scaffolding for the workmen below the grids in the hoppers. In some plants grids had been redesigned to be turned parallel with the flow line, and removed without disturbing the asbestos plates, but this was not tried at Hondo. At Hondo, plastic plugs were inserted into the header with a 5/64-in. orifice drilled in each plug. The orifices were faced up rather than down. At first the orifices were effective in giving much better distribution of air but they were gradually clogged by biological growths so that after six weeks diffusion was as bad as before.

Next, porous tubes and porous plates were compared with the original grid. The results with porous tubes were much better than with any of the other types. The tubes were finally placed along the center flow line, giving a two-roll circulation up and over the plates. This immediately gave better results and obviated a large amount of maintenance work.

The sludge hoppers on one side of the aeration chambers had been filled in, with no adverse effect on the effluent, and gave relief from the necessity of frequent removal of sludge of high moisture content.

The air supply was split so that approximately 60 percent was used in the first aerator and 40 percent in the second. This effected improvement in the DO and reduced the odor considerably.

The intermediate settler was bypassed during one test period, but this was not found to be beneficial.

The results of these experiments at Hondo were helpful in improving the operation of the contact aerators.

In some of the larger installations

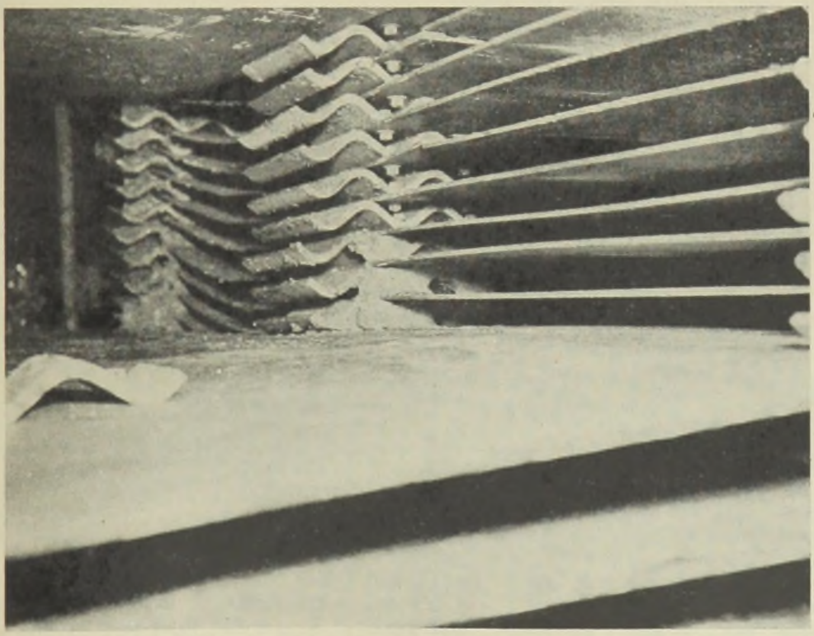


PLATE 19.—Plate holders in contact aeration basin at Camp Hood, Texas.

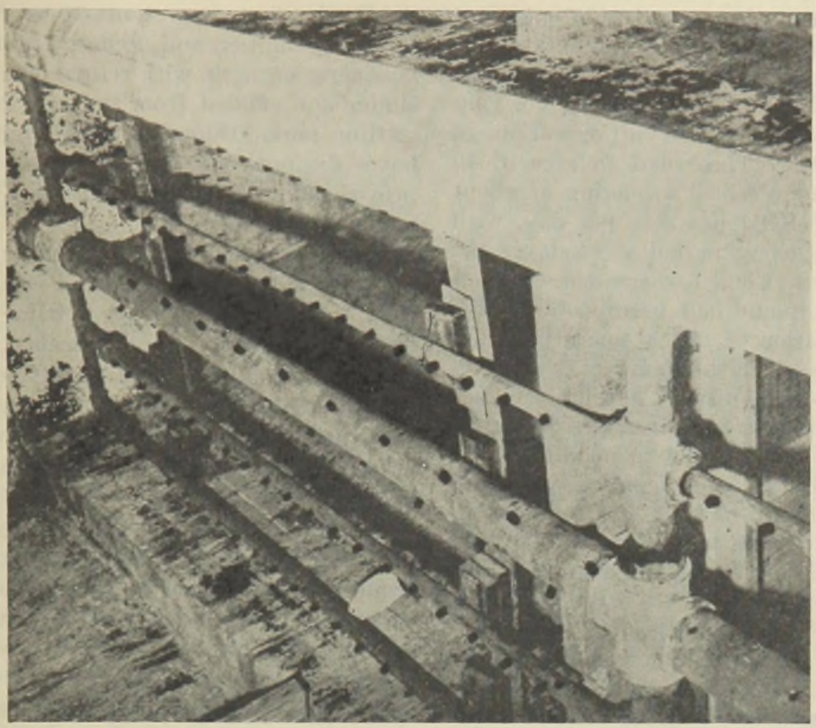


PLATE 20.—Air distribution system in contact aeration basin at Hondo AAF, Texas.

other types of treatment were added to augment the contact aeration plant or to take the load off the aerators at which odors were pronounced. At Camp Hood, effluent ponds were constructed and placed in operation in May, 1943. These had an area of 40 acres and received a loading of about 33 lb of BOD per acre per day, with a displacement period of 14 days. At Sheppard Field, where the contact aeration plant had been installed for a population of 25,000 operating difficulties such as breaking of sludge-removal mechanisms, failing contact plates, plugging of air orifices and production of odors led to installation of a trickling filter in September, 1943 to take care of half of the population, which at times was up to 44,000. At Camp Rucker an activated sludge plant was installed in 1943 to handle a population increase expected to reach 54,000. As stated before, however, the original population of 29,000 fell to 21,500 and the original contact aeration plant was again able to handle the load.

At one of the smaller plants, Waco AAF, the first-stage contact aeration tank was emptied and used as a spiral-flow aeration tank, with return of some sludge and effluent from the secondary settling tank. Odors were reported to have disappeared and effluents improved, but records are inadequate for inclusion in this report.

Frequent practice in a number of plants was to recirculate final effluent back through the primary settling or first aeration tank. No detrimental results and often improved results indicated that it was unnecessary to segregate the biological life in zones, as was originally claimed to be an element of the process.

The difficulties of operation of contact aerators—high maintenance labor requirements and more than occasional odor nuisances—indicate that contact aerators are less desirable for use in military camps than are trickling filters. No superiority of contact aeration over activated sludge treatment has been demonstrated.

CHAPTER VII

ACTIVATED SLUDGE PLANTS

Selection of Plants

Activated sludge plants were constructed at about 20 Army installations and a number at Navy installations. Although these included a number of mechanical aeration plants at smaller military camps, the data available for only the four diffused air and single combined mechanical aeration and diffused air plants listed in Table 101 met the requirements for inclusion in this survey. Of the plants selected for study, all the Army plants were of the diffused air type. The single Navy plant at Green Bay, Great Lakes, Illinois, was a mechanical aeration plant in which aeration had been supplemented with diffused air. Two of the Army plants were situated in Mississippi (Gulfport and Shelby No. 2) and one each in North Carolina (Butner) and New Jersey (Monmouth). The plants served design populations from 12,800 to 41,600.

Basis of Design

According to the Army Engineering Manual, the following displacement periods in hours based upon the 24-hr average rate of flow were used in designing units for activated sludge plants:

	Hours
Primary Settling Tanks	1.5
Aeration Tanks:	
Diffused Air Plants	8.0
Mechanical Aeration Plants	12.0
Final Settling Tanks	2.5

In designing the aeration tanks and final sedimentation tanks, an allowance of 25 percent for sludge return was included in the average rate of flow.

The sludge digestion tank capacities were 4.0 cf per cap for heated tanks and 6.0 cf per cap for unheated tanks.

Description and Performance of Plants

General Data

The general data relating to the activated sludge plants studied are listed in Table 102. The minimum sewage temperatures ranged from 51° to 67° F and the maximum temperatures from 78° to 84° F. There were 1.9 to 13.4 lineal feet of sewer per capita. The sewage had an average pH of 7.2. The sampling period covered 18 hr at Monmouth and 24 hr at the other plants. Samples were col-

TABLE 101.—List of Activated Sludge Plants

Post	State	Sv C or Navy	Type of Station	Date Plant Completed	Date Major Additions or Alterations Completed	Operating Period Studied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Camp Butner	N C	4th	Div Trg	Aug 42	Sep 43	Jan 43 to Aug 44
Gulfport AAB	Miss	4th	Tact Fields & Fly Trg	Apr 43	Oct 43	May 43 to Mar 44 Apr 44 to Nov 44
Fort Monmouth	N J	2nd	Signal Corp	Sep 40	Sep 43	Apr 43 to Jul 44
Camp Shelby #2	Miss	4th	Div Trg	Jun 43	—	Jun 43 to Sep 44
Great Lakes (Green Bay)	Ill	V	Navy Trg.	Nov 43	Aug 43	Jan 44 to Oct 44

TABLE 102.—General Data Relating to Activated Sludge Plants

Post	Approx Plant Elev (ft)	Range of Sewage Influent Temp (°F)	Average Sewage Influent pH	Lineal Feet of Sewers per Capita (a)	Sampling	
					Hours per Day	Average Number per Month
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Butner	265	53–84	7.2	10.0	24	30
Gulfport AAB	—	60–84	7.2	4.4	24	30
Monmouth	—	51–83	7.3	6.2	18	30
Shelby #2	—	28–82	7.2	13.4	24	22
Great Lakes (GB)	—	67–78	—	1.9	24	25

(a) Based on design population, Table 104, col (3).

TABLE 103.—Units in Activated Sludge Plants

Post	Plant Units										
	Screens or Racks	Comminutors	Grit Chambers	Imhoff Tanks	Primary Settling Tanks	Aeration Basins	Secondary Settling Tanks	Chlorine Contact Tanks	Digestion Tanks (Heated)	Digestion Tanks (Unheated)	Sludge Beds
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Butner	1	1	1	0	5	5	5	0	2	2	24
Gulfport AAB	1	0	1	0	2	3	2	1	0	3	8
Monmouth	1	2	0	0	2	4	2	2	1	2	6
Shelby #2	1	0	1	0	2	3	2	1	1	1	2
Great Lakes (GB)	0	1	0	0	2	4	2	0	1	1	8

lected and analyzed on 22 to 30 days each month. The plant units comprising the treatment works are listed in Table 103.

Design Capacity and Loading

Table 104 gives a comparison of design and actual populations, and of flows. At Camp Butner and Gulfport AAB the average populations contributing sewage were about 75 percent of the design values, and the average flows were 56 and 75 percent, respectively, of the design figures. At these plants, the maximum populations and flows roughly equalled the design values.

At Fort Monmouth the average population served was 73 percent greater than the design population, while the average flow was 30 percent less than the design flow. This apparent anomaly is explained by the fact that the average daily per capita sewage flow was only 36 gallons.

At Camp Shelby No. 2 the greatest difference between design and actual values of the group occurred, for the average population and flow reached only 23 percent of design values. The actual average population and flow was respectively 80 and 72 percent of design at the Green Bay plant at Great Lakes. In contrast to the low per capita flow recorded at Monmouth, three of the plants received flows very close to the 70 gcd value assumed for Army systems.

PLATE 21.—Aeration basins in activated sludge plant at Lake Charles AAF, La.

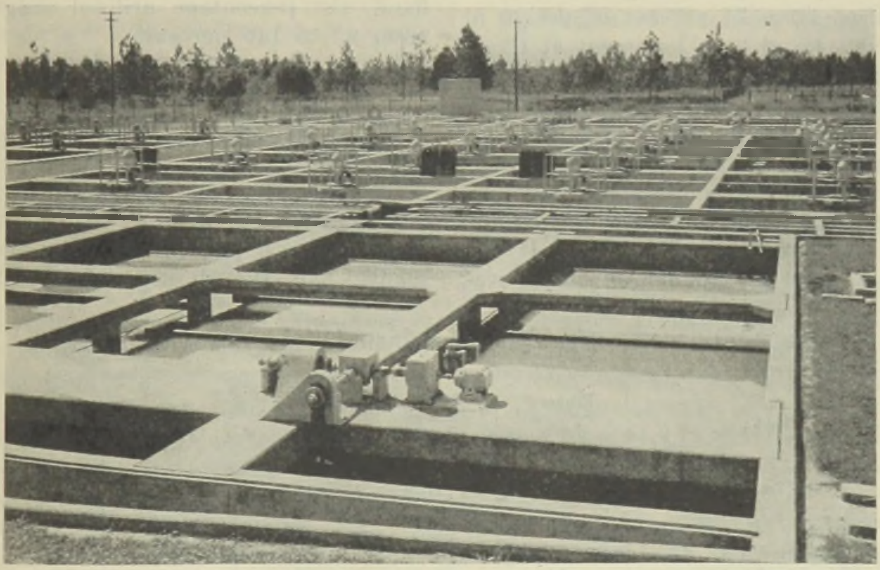


PLATE 22.—Primary settling and mechanical aerators at Main Plant, Camp Claiborne, La.

TABLE 104.—Design Capacities and Loading of Activated Sludge Plants

Post	Number of Months in Operating Period Studied	Population				Flow				
		Design Capacity (a)	Actual			Design (a) (mgd)	Actual			
			Average	Maximum	Minimum		Average (mgd)	Maximum (mgd)	Minimum (mgd)	Gal per Capita Daily
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Butner	20	41,600	30,800	43,200	19,300	2.912	1.620	2.220	1.080	53
Gulfport AAB	8	22,600	17,100	19,800	12,800	1.582	1.180	1.530	0.920	69
Monmouth	16	12,800	17,200	23,500	11,800	0.900	0.626	0.841	0.505	36
Shelby #2	16	34,700	8,000	10,900	1,652	2.429	0.563	0.840	0.160	71
Great Lakes (GB)	10	27,400	22,000	23,500	18,400	1.920	1.489	1.700	1.330	68

(a) Design capacities are based chiefly on criteria in OCE Engineering Manual (1943). See text. Column (3) capacities based upon detention period in the aeration tank.

Percent of Design Utilized

The percentages of design capacity utilized are shown in Table 105. Based on a designed aeration period of 8 hours, only 28 percent of the aeration capacity was utilized at Shelby No. 2, and only 71 to 78 percent at the other three diffused air plants.

The primary settling tanks were utilized from 24 percent of design at Shelby No. 2 to 111 percent at Great Lakes. A reasonable percentage of secondary tank design capacity was utilized at all plants except at Shelby No. 2, where utilization was only 26 percent. The sludge digester design capacity was more than ample at Shelby No. 2, Gulfport, and Butner. It was about right at Monmouth, but it was insufficient at Great Lakes. The designed sludge bed area proved to be about correct at Great Lakes and more than ample at all other plants. None of the plant units at Shelby No. 2 were anywhere near fully put to use. Only 21 to 32 percent of the design capacity of the various units was engaged. At Great Lakes, on the other hand, the percentage utilized ranged from 92 to 146 percent.

Design of Aeration Basins

Design data for aeration basins, air distribution systems and air supply blowers are given in Table 106. There was considerable variation in the ratio of tank length to tank width at these plants. Of the diffused air plants,

TABLE 105.—Percent of Design Capacity Utilized in Activated Sludge Plants

Post	Primary Settling Tanks (%)	Activated Sludge Tanks (%)	Secondary Settling Tanks (%)	Sludge Digesters (%)	Sludge Beds (%)
(1)	(2)	(3)	(4)	(5)	(6)
Butner	32	71	86	64	51
Gulfport AAB	75	78	82	58	60
Monmouth	54	73	104	95	69(a)
Shelby #2	24	28	26	32	21
Great Lakes (GB)	111	—(b)	93	146	92

(a) Covered sludge beds.
(b) Combined mechanical and diffused air.

TABLE 106.—Design and Description of Aeration Basins

Post	No of Basins	Dimensions—L x W x D (ft)	Total Volume (cf)	Air Distribution System				Air Supply Blowers			
				Tubes		Arrangement					
				No.	Dia (in.)	No of Rows	Spacing (ft)	No.	HP	Total Capacity (cfm)	PSI
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Butner	5	200 x 15 x 10.7	160,000	—	—	2(b)	7.5(b)	4	60	6,000	6
Gulfport AAB	3	122 x 20 x 12	88,000	114	3	2	0.8	4	90	2,500	6
Monmouth	4	58.5 x 19.5 x 11	50,000	124	3	4	2.0	2	40	2,400	5
Shelby #2	3	100 x 30 x 15	135,000	366	5	2	—	3	75	4,500	8
Great Lakes (GB)	4(a)	36 x 36 x 10.3	160,000	192	—	–(c)	—	1(d)	50	15,000	3

(a) Twelve aerator cells arranged 3 in a row in 4 continuous aeration bays.
(b) One-sf plate held in vertical position in single longitudinal row of double plate holders traversing center of basin; total plate area 1800 sf.
(c) Side wall of each entrance cell.
(d) Additional blower installed fall of 1944.

Butner had the highest (13.3 to 1) and Monmouth the lowest (3 to 1). Butner was the only plant that used diffuser plates. A total of 1,800 plates was provided in a single longitudinal row of vertical, double-plate holders. The individual plates were 1 ft square and the total plate area was 12 percent of the surface area. In the other plants, from 114 to 366 aeration tubes, 3 or 5 in. diameter, were arranged in rows of two to four tubes. The tube spacing was 0.8 ft at Gulfport and 2.0 ft at Monmouth. The depth to the diffuser tubes was 10.7 ft at Monmouth.

The diffused air plants were provided with two to four blowers, rated at 20 to 75 hp each, and capable of delivering 2,400 to 6,000 cfm of free air at pressures of 3 to 8 psi. The cubic feet of air supplied per gallon of sewage was as follows: Gulfport 1.95, Butner 2.89, Monmouth 2.90, and Shelby No. 2 4.0. These values are greatly in excess of those normally employed in municipal practice.

The mechanical aerator at Great Lakes included 12 aerator cells arranged in three rows of four continuous aeration bays. Each aerator required 5 hp at 860 rpm. These units failed to maintain dissolved oxygen in the mixed liquor, and it was necessary to add diffused air by installing 192 teardrop diffuser tubes of 80 permeability, 6 ft below the surface, along the side walls in the first cells. The air thus supplied amounted to 1.52 cf per gallon.

Operation of Aeration Basins

Figures for total flow, aeration period and air supply are shown in Table 107. The total flow to the aeration basins, including returned sludge varied from 0.82 mgd at Monmouth to 2.55 mgd at Butner. At average flows, all plants except Shelby provided aeration periods between 9 hr (Butner) and 12 hr (Great Lakes). At maximum flow these aeration periods were reduced to between 5.5 hr (Monmouth) and 9.3 hr (Great Lakes). At Shelby No. 2 the aeration periods were excessive, ranging from 11.8 hr at maximum flow to 56.8 hr at minimum flow, with 28.9 hr at average flow.

The hourly air supply per sf of tank surface was 6.1 at Great Lakes and varied between 10.4 (Shelby No. 2) and 13.0 (Butner) for the diffused air plants.

Excepting Shelby No. 2, the volume of air supplied per lb of applied BOD

TABLE 107.—Operation of Aeration Basins

Post	Total Flow to Basin (a) (mgd)	Aeration Period (hr)			Air Supply				
		At Av Flow	At Max Flow	At Min Flow	1000 CF Daily	CF per SF Tank Surface Hourly	CF per Gal Sewage	CF/LB BOD Applied	CF/LB BOD Removed
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Butner	2.55	9.0(c)	6.5	17.9	4,687	13.0	2.89	1,300	1,590
Gulfport AAB	1.67	10.3	6.2	16.8	1,902	10.8	1.95	1,330	1,620
Monmouth	0.82	10.9	5.5	33.4	1,814	16.6	2.90	1,230	1,585
Shelby #2	0.84	28.9	11.8	56.8	2,247	10.4	4.00	2,470	2,650
Great Lakes (GB)	2.39	12.0	9.3	18.7	2,265(b)	6.1(b)	1.52(b)	605(b)	690(b)

(a) Total flow includes plant influent flow and returned sludge.
(b) Combined mechanical and diffused air.
(c) Variable number of tanks in service.

TABLE 107 cont.—Operation of Aeration Basins

Post	Returned Sludge				Wasted Sludge				Mixed Liquor	
	1000 Gal Daily	Percent of Sewage Flow	SS (ppm)	30-Min Settle-able Solids (ml/l)	Sludge Index	1000 Gal Daily	Percent of Sewage Flow	No of Passes	SS (ppm)	30-Min Settle-able Solids (ml/l)
(1)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Butner	927	57.0	2,465	750	381	29.1	1.8	32	1,143	398
Gulfport AAB	490	50.1	1,191	740	628	18.0	1.8	28	592	375
Monmouth	199	31.9	3,848	790	584	17.8	2.9	11	1,497	390
Shelby #2	277	49.2	1,855	—	280	28.2	5.0	10	889	260
Great Lakes (GB)	750	50.2	2,560	—	1,026	150.0	10.0	5	845	830

varied from 1,230 to 1,330 cf at the diffused air plants. These values are in line with those given by Greeley (54) for the Easterly Cleveland, Ohio, and Pasadena, Calif. municipal plants. At Shelby, 2,470 cf of air was supplied per lb of applied BOD. This is almost three times the average value for municipal plants studied by Greeley (54). The ratios of air volumes per lb of BOD removed relative to those per lb of BOD applied were 1.22, 1.22, and 1.29 at Butner, Gulfport and Monmouth, respectively. The corresponding ratios were 1.07 for Shelby No. 2 and 1.14 for Great Lakes.

Returned Sludge

At Fort Monmouth, where the most concentrated returned sludge was obtained (3,848 ppm of suspended solids), this sludge equalled 31.9 percent of the total flow. At the other plants, the returned sludges contained only 1,191 to 2,560 ppm of suspended solids, and constituted between 49.2 and 57 percent of the total flow. These returned sludges yielded between 740 and 790 ml per liter of settleable solids in 30 minutes, as shown in Table 107.

Data on the amounts and character of the sludge wasted and on the character of the mixed liquor are also given in Table 107. It will be noted that the percentage of sludge wasted was very low, at only 1.8 to 5.0 percent of the sewage flow. As a result, the number of passages of the sludge through the diffused air plants averaged between 10 and 32. At Great Lakes, the wasted sludge amounted to 10 percent of the sewage flow and the sludge passed through the units 5 times.

The suspended solids in the mixed liquor was relatively low, ranging from 592 ppm at Gulfport to 1,497 ppm at Monmouth. The average settleable solids in the mixed liquor ranged from 260 to 830 ml.

The sludge volume index (ml per gram) for the mixed liquor was as follows for the average, maximum, and minimum months:

	Average Month	Maximum Month	Minimum Month
Butner	381	959	120
Gulfport AAB	467	1,021	45
Monmouth	586	1,855	62
Shelby No. 2	279	486	103
Great Lakes (GB)	1,026	1,159	893

The very high average monthly indices suggest that the mixed liquor at all plants was bulking a good part of the time. At Butner, for instance, the indices were below 100 only in July and August, 1943 and for parts of the months of April and May, 1944. Throughout the remainder of the survey period the sludge was bulking badly. During the last four months of the study period, the monthly sludge index at Gulfport was maintained below 100, but this was the only plant that showed such improvement. Lime was added to the returned sludge. By contrast, the indices at Shelby No. 2 were over 400 during the last four months of the study period. These high values explain why it was impossible to carry more suspended solids in the mixed liquor and returned sludge of these plants. Maximum individual sludge indices varying from 1,660 to 3,846 were observed at all of these plants.

The mean DO values found in the influent and effluent of the aeration basin and in the final effluent were as follows:

	Aeration Tank		Final Effluent (ppm)
	Influent (ppm)	Effluent (ppm)	
Butner	0.0	2.9	0.8
Gulfport AAB	1.3	3.0	1.1
Monmouth	—	2.3	2.1
Shelby No. 2	—	3.5	2.3
Great Lakes (GB)	1.4*	0.8*	0.7

* Average of three grab samples during study by U. S. Public Health Service in November, 1944.

These data suggest clearly that, except for Gulfport and perhaps Shelby No. 2, the dissolved oxygen at the influent end of the aeration tank was rather low and frequently inadequate. The values for the aeration tank at Great Lakes were obtained when both the mechanical and diffused air aerators were in operation.

BOD and SS Loadings and Performance

The BOD loadings and performance of the primary settling tanks are shown in Table 108. The average BOD of the raw sewage at these plants ranged from 224 ppm at Gulfport to 397 ppm at Butner. At the sewage flows indicated, the daily per capita BOD contribution ranged from 0.107 lb at Gulfport to 0.207 lb at Great Lakes. The average BOD was reduced by primary settling to 175 ppm at Gulfport, 194 at Shelby, 266 at Butner, 280 at Great Lakes, and 284 at Monmouth. The poorest percentage removal of BOD by primary settling was obtained at Great Lakes (16.7%), at an average detention time of 1.6 hr. With detention times of 2 to 3 hr, percentage removals of BOD were 19.7, 21.9, and 32.9 at Monmouth, Gulfport and Shelby No. 2, respectively. Butner attained 33.0 percent removal in the primary tank, the highest performance in this group. As a whole, BOD re-

TABLE 108.—BOD Loading and Performance of Activated Sludge Plants

Post	Plant Influent			Primary Settling Tank					
	Flow (mgd)	BOD		Detention Period (hr)			BOD		
		PPM	Pounds per Capita Daily	At Av Flow	At Max Flow	At Min Flow	Inf (ppm)	Eff (ppm)	Removal (%)
(1)	(3)	(4)	(5)	(16)	(17)	(18)	(19)	(20)	(21)
Butner	1.62	397	0.175	—(a)	—(a)	—(a)	397	266	33.0
Gulfport AAB	0.98	224	0.107	2.0	1.0	5.2	224	175	21.9
Monmouth	0.62	354	0.108	2.8	1.2	24.7	354	284	19.7
Shelby #2	0.56	289	0.171	3.1	1.0	10.0	289	194	32.9
Great Lakes (GB)	1.49	363	0.207	1.6	1.2	3.5	335(b)	280	16.7

(a) Variable number of tanks in service.
(b) Calculated from BOD in raw sewage and BOD in returned sludge flow exclusive of BOD of sludge itself.

TABLE 108 cont.—BOD Loading and Performance of Activated Sludge Plants

Post	Aeration Basin Loading					
	Flow (mgd)				BOD	
	Plant Influent	Returned Sludge	Total Flow	1,000 GPD per Foot Tank Lgth for One Hour (d)	PPM (c)	Pounds per 1,000 CF Aeration Basin Volume (a)
(1)	(3)	(12a)	(22)	(23a)	(25)	(26a)
Butner	1.62	0.93	2.55	18.3	188	24.9
Gulfport AAB	0.98	0.49	1.47	28.8	127	17.6
Monmouth	0.62	0.20	0.82	29.2	231	31.5
Shelby #2	0.56	0.28	0.84	54.2	134	7.0
Great Lakes (GB)	1.49	0.75	2.24	44.5	217	25.3

(c) Calculated from BOD of primary settling tank effluent and BOD in returned sludge exclusive of BOD of sludge itself.
(d) Area of cross-section aeration basin × 7.5 × 24 ÷ (1 + proportion returned sludge).

movals were relatively low (Figures 67 and 68).

As shown in Table 109, the concentration of suspended solids in the plant influents varied from 172 ppm at Gulfport to 376 ppm at Great Lakes. These values represent from 0.082 to 0.237 lb of suspended solids per capita per day. The removal of suspended solids by primary settling was relatively low, ranging from 34.3 percent at Gulfport to 48.8 percent at Butner.

Aeration basin loadings are presented in Table 108. The BOD of the mixed liquor influent varied from 127 ppm at Gulfport to 217 ppm at Great Lakes. These values were calculated from the measured BOD values for the primary effluent by adding the measured BOD of the final effluent as that of the returned sludge. On this basis, the quantity in pounds of BOD applied per 1,000 cf of aeration basin volume ranged from 7.0 for Shelby No. 2 to 31.5 for Monmouth. The 7.0-lb load at Shelby amounts to only about one-third of the load recorded at most municipal installations. All other BOD loading values are in line with municipal experience.

Table 108 shows the detention periods for average, maximum, and minimum flows through the secondary settling tanks. The longest detention periods obtained at Shelby No. 2 were 4.86, 1.99 and 9.5 hr for the average, maximum and minimum flows. At the other plants the detention periods were more in line with municipal practice and varied at average flows from 2.31 hr at Butner to 3.17 hr at Gulfport. The greatest spread in detention time (1.2 to 7.28 hr) occurred at Monmouth.

The combined performance of the aeration and secondary settling units is shown in Tables 108 and 109. Calculation of the suspended solids values for the influent to the aeration basins, given in column 4 of Table 109, followed the procedure previously as that outlined for BOD.

Shelby No. 2 produced an effluent with an average BOD of 13 ppm and

TABLE 108 cont.—BOD Loading and Performance of Activated Sludge Plants

Post	Performance of Aeration Basin and Secondary Settling Tank							Over-all BOD Removal (%)	Dissolved Oxygen (ppm)		
	Secondary Settling Tank Detention Period (hr)			BOD							
	At Av Flow	At Max Flow	At Min Flow	Final Eff (ppm)	LB Removed per 1000 CF Basin Vol	Efficiency (%)	Removal (%)		Basin Inf	Basin Eff	Final Eff
(1)	(32)	(33)	(34)	(36)	(37a)	(38)	(39)	(40)	(41)	(42)	(43)
Butner	2.31	1.67	4.57	49	18.3	81.3	54.7	87.7	0.0	2.9	0.8
Gulfport AAB	3.17	1.89	5.37	31	13.4	82.4	64.2	86.1	1.3	3.0	1.1
Monmouth	2.39	1.20	7.28	63	22.9	77.7	62.3	82.0	2.3	—	2.1
Shelby #2	4.86	1.99	9.50	13	6.3	93.3	62.7	95.6	—	3.5	2.3
Great Lakes (GB)	2.60	1.88	3.80	37	20.7	87.8	73.0	89.7	—	—	—

TABLE 109.—SS Loading and Performance in Activated Sludge Plants

Post	Suspended Solids								
	Plant Influent		Primary Settling Tank		Aeration Basin and Secondary Settling Tank				Overall Plant Removal (%)
	PPM	Pounds per Capita Daily	Effluent (ppm)	Removal (%)	Influent (ppm)	Effluent (ppm)	Efficiency (%)	Removal (%)	
(1)	(2)	(3)	(4)	(5)	(4a)	(9)	(12)	(13)	(14)
Butner	350	0.154	179	48.8	125	30	83.3	42.6	91.4
Gulfport AAB	172	0.082	113	34.3	89	40	64.6	42.4	76.7
Monmouth	360	0.110	208	42.2	190	131	37.0	21.4	63.6
Shelby #2	250	0.148	130	48.1	92	15	88.4	46.0	94.1
Great Lakes (GB)	376(a)	0.237(a)	195(a)	44.3(a)	147(a)	35(a)	83.4(a)	46.4(a)	90.7(a)

(a) Based upon only 2 months operation; daily composite samples during September and October, 1944.

a suspended solids content of 15 ppm. This was the only plant in this group that produced an effluent comparable to that from a large, well operated, municipal activated sludge plant. This plant, with the lowest loading in the group, gave an aeration basin and secondary tank efficiency of 93.3 percent for BOD and 88.4 percent for suspended solids. The percentage removal of BOD and suspended solids by these units was 62.7 and 46.0 percent, respectively. The overall removals for Shelby No. 2 were 95.6 percent for BOD and 94.1 percent for suspended solids.

Great Lakes gave the next best performance with overall plant removals of 89.7 and 90.7 percent for BOD and suspended solids, respectively. The performance at Butner was slightly lower for BOD (87.7%) and slightly higher for suspended solids (91.4%). The loading and performance curves for Butner, as shown in Figure 98, indicate that the performance was relatively uniform during the study period except for the month of December, 1943, when the greatest effluent deterioration took place at a time when population, BOD and suspended solids loads were below average.

At Gulfport, the average overall BOD removal fell to 86.1 percent and the suspended solids removal performance deteriorated to 76.7 percent. The average suspended solids content of the effluent was 40 ppm because of long

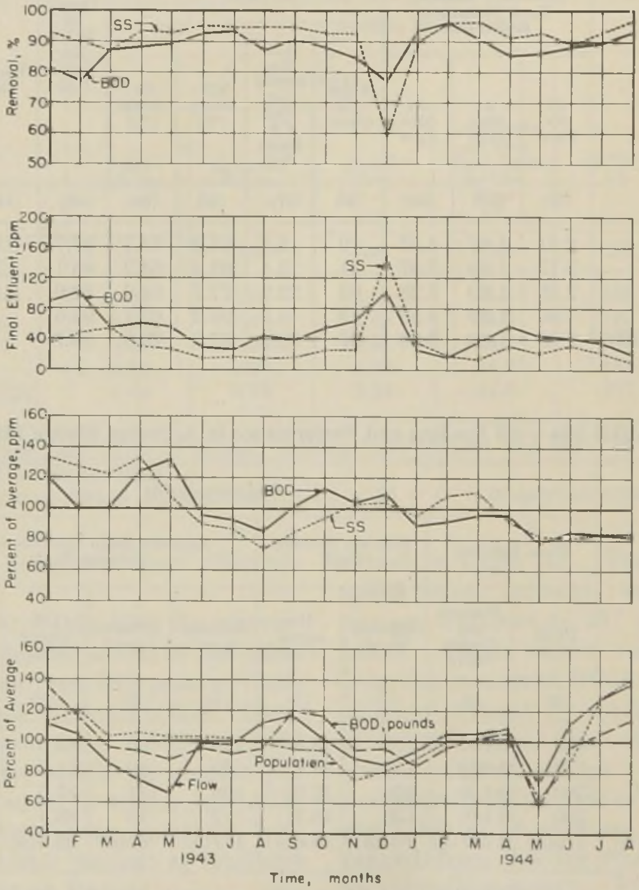


FIGURE 98.—Loading and performance of activated sludge plant at Camp Butner.

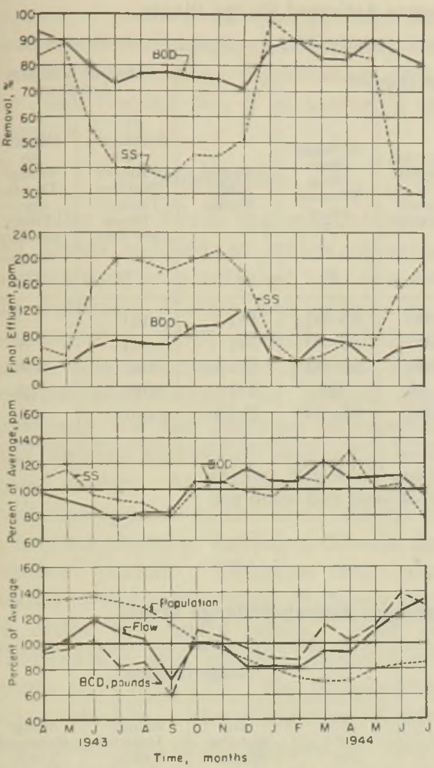


FIGURE 99.—Loading and performance of activated sludge plant at Fort Monmouth.

periods of poor operation associated with bulking sludge.

The poorest performance of all is that recorded for Monmouth, where the overall BOD and suspended solids removals were only 82.0 and 63.6 percent, respectively. At this plant 42.2 percent of the suspended solids were removed by the primary tank in contrast with only 21.4 percent by the aeration basin and secondary tank. This indicates a definite failure of the secondary part of the treatment process at Monmouth, which may have been caused, in part at least, by the discharge of photographic wastes into the sewage reaching this plant. Loading and performance curves for 16 months of operation at Fort Monmouth are shown in Figure 99. These curves show that with a slowly falling population load the plant did not perform satisfactorily from June to December, 1943. During most of this time, the final effluent averaged between 40 and 120 ppm of BOD and 160 ppm or more of suspended solids. From July until October, 1943, iron salts were added to the aeration tank as in the Guggenheim aeration process. The mean BOD and suspended solids contents of the effluent during this period were 75 and 201 ppm, respectively. The results obtained during this summer period of operation indicated, therefore, that addition of iron salts did not improve the activated sludge process at Fort Monmouth.

Sludge Digestion and Drying

The waste activated sludge at all the plants studied was digested with primary sludge in separate sludge digestion tanks. Two to four digestion tanks were provided at each plant for digestion in two stages. Pertinent design data and the schedule of sludge loading are presented in Table 110. One or two tanks at Butner, Monmouth, and Great Lakes were equipped with floating covers. One tank at Shelby No. 2 was open. The remaining tanks had fixed covers. Mechanical stirring was provided only at Monmouth. A 300-gpm pump was installed on a tank cover at Great Lakes for recirculation. At least one tank was heated at each plant, except at Gulfport, where no heat was provided. The average temperature of heated tanks was 90 to 95° F.

The plants studied were provided with sludge drying beds, all of which were open except at Monmouth, where they were glass covered. Figures for per capita area and data on sludge disposal are given in Table 110.

Operators at all plants except Monmouth reported that the sludge drawn was well digested. Operating difficulties were reported for Monmouth and for Gulfport. The unheated tanks at Gulfport gave poor digestion in winter. At all plants, sludge gas was burned

TABLE 110.—Sludge Data Relating to Activated Sludge Plants

Post	Number Months in Operating Period Studied	Sludge Digester					Schedule of Sludge Loading	
		Total CF per Capita (a)	Heated, Percent of Total Volume	Covered, Percent of Total Volume	Average Operating Temperature (°F)	Source of Sludge	Secondary to Primary Settling Tanks—Times per Day	Primary to Sludge Digesters—Times per Day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Butner	20	7.5	50	100	85–90	Pri	cont	—
Gulfport AAB	12	10.3	0	100	50–90	Pri	cont	8
Monmouth	12	5.6	21	100	85–90	Pri (b)	cont (c)	3–6
Shelby #2	15	14.8	54	46	90	Pri	cont	12
Great Lakes (GB)	10	5.4	50	100	95–100	Pri (b)	cont	6

(a) Based on actual population, Table 104, col (4).
(b) Also from vacuum flotation unit and scum pit.
(c) 25% to aeration basin.
(d) Covered sludge beds.

TABLE 110 cont.—Sludge Data Relating to Activated Sludge Plants

Post	Gallons Raw Sludge per MG Sewage	Volatile Solids, Percent of Total Solids		Sludge Disposal			
		Raw Sludge	Digested Sludge	SF per Capita (a)	No Months Sludge Drawn per Year	Sludge Drawn Yearly from Digesters (cf/cap)	Sludge Removed Yearly from Beds (cf/cap)
(1)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Butner	23,260	73.6	53.4	1.95	10	5.6	1.5
Gulfport AAB	8,490	72.1	51.7	1.67	—	—	—
Monmouth	6,870	75.6	57.1	0.39 (d)	5	—	1.7
Shelby #2	7,920	—	50.7	4.80	5	9.2	0.5
Great Lakes (GB)	18,700	83.6	—	1.09	—	—	—

TABLE 111.—Chlorination and Effluent Disposal in Activated Sludge Plants

Post	Chlorination		Receiving Water Body	
	Points of Application Used	Residual Chlorine (ppm)	Name	Type
(1)	(2)	(3)	(4)	(5)
Butner	Ret Sludge (a)	—	Knapp of Reeds Creek	Flowing Stream
Gulfport AAB	Ret Sludge	—	Bayou Barnard	Tidewater
Monmouth	Ret Sludge (b)	—	Tributary to Shrewsbury River	Tidewater
Shelby #2	None	—	Davis Creek	Flowing Stream
Great Lakes (GB)	Ret Sludge	—	—	Flowing Stream

(a) Occasionally to control bulking.
(b) Also pre- and postchlorination.

to heat digesters, offices, laboratories, or control buildings. Additional information on sludge digestion at Butner and Shelby No. 2 is included in Chapter VIII.

Chlorination

Shelby No. 2 was the only plant of the group studied at which chlorine was not used at some point. At Butner the returned sludge was chlorinated occasionally. At Monmouth and Great Lakes the returned sludge was chlorinated continuously or almost continuously for the purpose of controlling bulking and conditioning the activated sludge. At Monmouth, the chlorine was applied to the final effluent in a contact chamber having a 30-min detention period. Chlorine was also added to the raw sewage at Monmouth in order to control odors. Data on chlorination and effluent disposal are given in Table 111. No information was available on the quantities of chlorine used, the doses applied or residuals found.

Operating Problems

No difficulties with diffuser clogging were experienced at Butner, Gulfport or Monmouth. At Shelby No. 2 the tubes became clogged after the blowers had been shut down for a period. The tubes were therefore removed and cleaned with caustic. At Great Lakes, although no trouble was noticed, the tubes were removed and cleaned with acid every 10 months.

Tapered aeration was practiced at all plants except Monmouth, but the method of obtaining it was different at each plant. At Butner the air enters the diffuser plate at the influent end of the aeration tank and, due to friction loss and the passage of some air through the plates, tapered aeration resulted. At Shelby No. 2 tapered aeration was obtained by the arrangement of the tubes, while at Gulfport it was accomplished by reduction of the header sizes. The mechanical aerator at Great Lakes was supplemented by the addition of diffused air through tubes in the first one-third of the aeration bays, thus producing tapered aeration. Although tapered aeration undoubtedly reduced aeration costs, there seems to be a reasonable doubt that aeration was sufficient for most satisfactory operation at some of the plants. This was certainly true at Great Lakes.

Control Tests

The tests used in the control of plant operation were as follows:

On the mixed liquor: (1) dissolved oxygen; (2) suspended solid; (3) sludge volume (30-min) and (4) sludge index.

On the return sludge: (1) suspended solids; and (2) sludge volume.

It was the practice at the better operated plants to determine the dissolved oxygen content and the sludge volume of the mixed liquor at two or more points twice daily. The suspended solids contents were determined from a composite sample of mixed liquor and the sludge indices were calculated from these data. The usual tests to determine plant performance were also made.

For the purposes of this study it was unfortunate that the percentage of ash in the mixed liquor solids was not determined regularly so that a correlation of this variable with the sludge index could have been made for military sewage. Although the control tests used may not be as adequate as desired, they gave a reasonable indication of operation trends and suggested to an alert experienced operator what changes should be made in operation to attain and maintain best plant performance.

It was suggested by one operator that nitrite and nitrate nitrogen determinations should be made on the mixed liquor for control purposes. These tests certainly should be added as control tests if the plant is loaded and operated in a manner conducive to nitrification.

Sludge Bulking and Associated Symptoms of Substandard Performance

Operators gave various answers to the question of frequency and cause of sludge bulking. The report from Shelby No. 2 stated that very little bulking had been experienced and the report from Gulfport that no bulking had been experienced during the last eight months of operation. In contrast, Monmouth reported continuous and persistent bulking. Great Lakes reported occasional bulking and Butner reported that the sludge index was usually around 300, but that bulking with loss of sludge over the weirs did not occur until the index reached 400 or more. These reports indicate that there was no uniform opinion as to what constitutes a bulking sludge. In municipal sewage works practice, a mixed liquor with an index of 150 or more may be considered to represent a bulking sludge. The percentages of the time that the average sludge index for the month was over various limits are listed below for each of the Army plants:

Post	No. of Months of Operation	Percentage of Time During Which the Average Sludge Index was Above the Stated Value			
		150	200	300	400
Butner	20	80	75	65	35
Gulfport AAB	16	75	69	56	56
Monmouth	16	69	62	56	56
Shelby No. 2	16	69	62	50	25

These data indicate that all these plants produced bulking sludge during a considerable portion of the time, if bulking limits are set by municipal standards.

The causes of bulking may be attributed to differences in composition of military and domestic sewage, to poor operation and control of the plant or to faulty design and construction of the plant.

Excessive detention periods in the primary tank are sometimes considered a cause of bulking. In the group of plants studied, Shelby No. 2 had the longest detention period (3.1 hr) in the primary tanks at average flows and this was increased to 10 hr at minimum flow. Yet Shelby No. 2 had the least bulking of all plants. Gulfport and Great Lakes, which both had shorter and more normal primary detention periods, experienced much more severe bulking. Monmouth had an unusually long primary detention period at minimum flows and experienced bulking conditions very similar to those at Gulfport. On the basis of this evidence, long primary detention periods can hardly be considered a primary cause for bulking at military installation.

The differences between municipal and military sewage which should be considered as possible factors include: (a) irregular distribution of flow; (b) excessive grease content; (c) higher BOD values and rates of biochemical oxidation and (d) higher volatile content. The indications are that all of these factors are implicated in the phenomenon of bulking. The poor operation of the activated sludge process as a result of shock loads and excessive grease content has been observed in municipal practice. However, there is evidence (55) that the correlation between grease content of the returned sludge and the sludge index at Army plants is low.

The feeding of a strong, easily oxidized sewage to returned sludge of low adsorptive power produces conditions conducive to the growth of nonzoogleal organisms such as *Sphaerotilus natans* and other free swimming organisms. Once these conditions are allowed to occur, an undesirable sludge is produced having a relatively lower content of dense zoogleal masses and a higher content of loose diffuse sludge particles and organisms. Although claims have been made that there is a difference in the biological activity and oxidizing quality of good sludge and bulking sludge, these have not been proved.

The BOD load per day per pound of suspended solids in the mixed liquor and secondary tanks is worth examining. At Gulfport, the monthly average load expressed in this way was decreased from 0.38 to 0.41 in the summer of 1943 to 0.07 in the fall of 1944. This was accomplished by a reduction in the BOD load and an increase in the suspended solids content of the mixed liquor. It was accompanied by an increase in plant efficiency and a decrease in sludge index. On the basis of the Gulfport results, the load should be kept under 0.2 lb of BOD per pound of suspended solids in the aeration and secondary settling tanks if proper sludge indices and efficient plant operation are to be maintained.

In one example of bulking at Army plants, it has been shown (55) that high volatile content of the sludge had no correlation with sludge index. However, the mineral constituents or ash of the sludge have the highest specific gravity of the sludge constituents and contribute to its settling qualities. Sludges with ash contents of 20 percent or more are not as likely to give trouble as those of lower ash content. In the case just referred to, the ash varied between 23 and 32 percent, and it was shown that a factor other than ash content, which factor could be controlled by chlorination, was of much more importance in bulking.

It has also been shown experimentally in a pilot plant (56) that the BOD load per pound of suspended solids in the aeration tank is more important than the ash content. Nevertheless, activated sludges containing only 10 to 15 percent of ash do not settle well. Twenty-one activated sludge samples from Great Lakes averaged 13.0 percent ash and eight from Baer Field averaged 14.4 percent. Sludges of such low ash content have low settling rates even when they are not bulking. Analyses of 87 samples of return sludge at Monmouth before chemicals were added to the aeration tank showed a mean ash content of 15.6 percent.

Methods of Eliminating Bulking

It is interesting to note that at Shelby No. 2, where an amount of air far in excess of that normal to municipal plants was used, where the BOD load was lowest and where the longest final settling periods were available, the least bulking occurred and no method to control bulking had to be instituted. At all other plants, various methods—including leveling off the flow, addition of chemicals, and chlorination—were tried to control bulking.

During the months when the Butner plant had ample aeration capacity, one aeration tank was used as a reservoir for primary effluent. It was filled during the morning peak flow, aerated during the day and pumped to the plant influent during the night. This procedure reduced the variation in flow as well as the variation in BOD and nitrogenous loading. This method was successful but it could not be used when the plant carried the full design load. There was also the danger of clogging the plates each time the tank was pumped out.

Addition of Chemicals

At Gulfport milk of lime solution was added at a rate of 600 lb per day to the returned sludge for a period of a week and the rate of sludge return was increased. The addition of lime was continued throughout the last four months of the study when the best operating results were obtained at Gulfport. At Monmouth, where continuous bulking difficulties occurred, attempts at control were made by the addition of calcite to the primary influent and by the addition of ferrous sulfate to the aeration tank, but these practices were abandoned. It is believed that a considerable part of the difficulties of operation at Monmouth stemmed from photographic wastes that entered the sewer system. Despite attempts to prevent this, the data would indicate that these toxic wastes had a material effect upon performance.

Chlorination

As stated previously, chlorination of returned sludge was used at four of these plants to control sludge bulking. The dose used varied from 7.0 to 10.0 ppm. The period of chlorination varied from intermittent application for a day or two to continuous application. This reduced the sludge indices very materially and permitted an increased rate of sludge return. This, in turn, reduced the BOD load per unit of activated sludge in the aeration tanks as already mentioned for Gulfport. Experience with chlorination of returned sludge at three of these plants confirmed the report of Tapleshay (55).

Recirculation of Plant Effluent

Results of a study of the effect of recirculation of plant effluent have been presented by Koruzo (51) for plants in the Fourth Service Command. During 1945 recirculation of final chlorinated effluent to the head of primary tanks was practiced at Fort Monmouth. This was done at night, on weekdays and all day on Sundays. The recirculation ratio was about 0.35 not including recirculated activated sludge. The effect on plant effluent was immediate, with bulking conditions considerably ameliorated.

Bulking at Monmouth had been nearly continuous since 1941. Recirculation was considered to reduce but not eliminate bulking. During the period January to October, 1945, the median population was 9,300 and the median ppm SS in the plant effluent was 32. However, during February, 1945, an average of 129 ppm SS occurred, indicating that bulking occurred. Air at the rate of about 1,200 cf per lb raw BOD was applied during the period recirculation was practiced.

Improvement of performance with recirculation of plant effluent was believed to result from the elimination of septic conditions occurring in settling tanks during the night when flow was low. Dissolved oxygen in the plant effluent during January to October, 1943 averaged 3.2 ppm, which is significantly higher than that noted in Table 108. Recirculation also effected a more uniform distribution of the diurnal organic load. At Monmouth, in assessing the value of recirculation, account must be taken of the reduced plant load during the period recirculation was used.

Generalization and Analysis

To determine whether there was any change in the relationship between the quantity of BOD removed and that applied with change in sludge index, the data available for Gulfport were analyzed. To this purpose the daily data for all months in which neither chlorine nor coagulants were added to the returned sludge or mixed liquor were selected for study. The daily data for two typical months when lime was added daily were also examined. The results of this study are summarized in Table 112. It will be observed that the coefficient of correlation r between BOD load and BOD removal was high for every month of study.

An examination of the sludge indices and the slopes a of the curves represented in Table 112 indicates that there was no correlation between these parameters. Neither could correlation be found between the sludge index and the intercept b of the curves (Table 112) for the period when special treatment was not used to control the index.

TABLE 112.—Gulfport AAB.—Correlation Between BOD Removal and BOD Load and the Relation of the Sludge Index to this Correlation

Month	Treatment to Control Bulking	Sludge Index (ml/gm)	Number of Observations	Value of Correlation Coefficient (r)	Constants for Linear Curve of Best Fit	
					a*	b†
1943						
Jun	None	527	24	0.87	0.95	540
Jul	None	579	11	1.0	0.82	870
Aug	None	659	27	0.93	1.11	240
Sep	None	695	30	0.85	0.84	480
Dec	None	1,021	30	0.85	0.85	400
1944						
Jan	None	906	31	0.97	1.01	170
Feb	None	695	27	0.90	0.97	370
May	Lime Tr'tment	163	31	0.99	0.99	63
Aug	Lime Tr'tment	67	31	0.99	0.95	96
Mean Jun to Feb	None	726	—	0.91	0.94	439
Mean May & Aug	Lime	115	—	0.99	0.97	79.5

* a = rate of change in removal relative to load.
† b = load at which no removal is obtained.

Post	Number of Months Included	(1) Load (X)	Average Sludge Index (Y)	Coefficient of Correlation (r)	Constants for Linear Curve of Best Fit	
					a(2)	b
Gulfport	14	0.253	436	.82	1799	−19
Monmouth	19	0.435	584	.83	1488	−93

(1) Average pounds of BOD per pound of suspended solids in the aeration and secondary tanks.
(2) Equation of curve $Y = aX + b$.

However, a great difference was noted in the mean values of b for the periods when lime was added and when it was not added. Much higher applied loads could be successfully treated with than without lime. The mean curves representing the relations between the BOD removal and loading at Gulfport could be represented by the equations

$$y = 1.10x - 484 \text{ (No lime used)}$$
$$y = 1.03x - 82.2 \text{ (Lime used)}$$

where

x = the applied BOD load in pounds,

and

y = the quantity of BOD removed in pounds.

This curve and a similar one obtained for two months when lime was used are plotted in Figure 100. The vertical distances between these curves indicate the improvement in BOD removals obtained at any applied load. It is evident that the use of straight lines in fitting data of this type is for convenience only. The equations represent actual results only in the vicinity of values of actual loading. At ex-

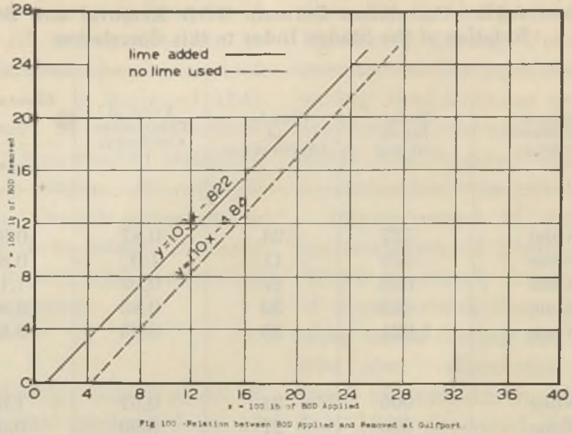


FIGURE 100.—Relation between BOD applied and removed at Gulfport.

treme loadings, the equations are not valid.

The relationship between the average BOD load per pound of suspended solids in the aeration and secondary settling tanks and the sludge index was studied at Gulfport and Monmouth. These were the only plants at which the number of aeration and settling tanks in use were known at all times. The data obtained are summarized in Table 112.

These data indicate that there was a high degree of correlation between the average BOD load per pound of suspended matter and the sludge index for the month. The equations for this relationship are:

Gulfport, $Y = 1799X - 19$
Monmouth, $Y = 1488X - 93$

These curves, together with the underlying data, are plotted in Figure 101. It should be pointed out in connection with these curves that loadings below 0.3 lb of BOD per pound of suspended solids were obtained at these plants only by special treatment to control the sludge index. This was accomplished by the addition of lime at Gulfport and by chlorination at Monmouth.

Comparison with Municipal Plants

In Figure 102 are shown data for

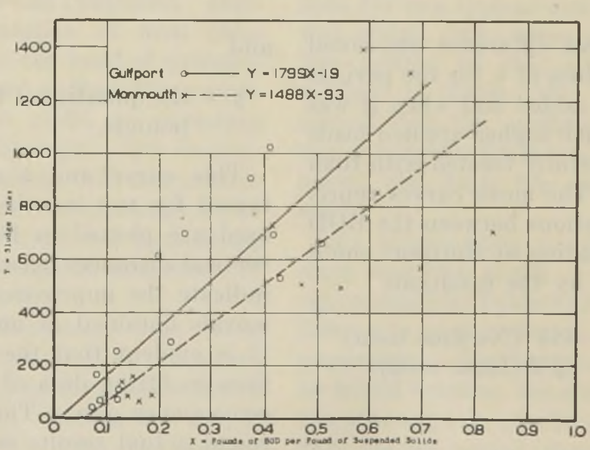
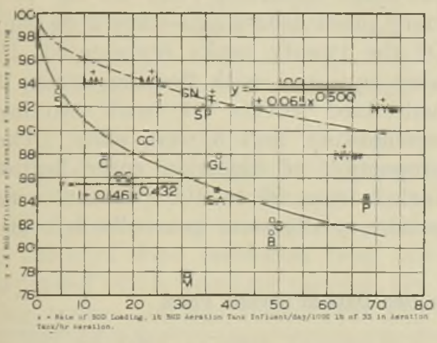


FIGURE 101.—Relation between sludge index and BOD/SS ratio.

various municipal and military activated sludge plants relating to efficiency of aeration and secondary settling, with BOD loading as expressed upon an "SS-hour" basis. This method of rating loading reflects both the amount of biologically active material and the effective time for biochemical reaction. In this respect it is similar to the parameters used in the analysis of trickling filters (Figure 88) and contact aeration plants (Figure 97). A somewhat similar parameter, called the coefficient of interfacial contact, was proposed by Harris, Cockburn, and Anderson (57), and shown to be an appropriate measure of efficiency.



+ Municipal
O Military plants
□ Plants with industrial waste
CC Chicago, Calumet
CN Chicago, Northside
C Cleveland, East
I Indianapolis
MO Milwaukee, old
MN Milwaukee, new
T Toronto
P Peoria
RC Rockville Center
SA San Antonio
Sp Springfield
NY New York, Ward's Island
B Butner
G Gulfport
M Monmouth
S Shelby
GL Great Lakes

FIGURE 102.—Efficiency of municipal and military activated sludge plants. Points representing municipal plants are based upon operating results for one-year period. Upper curve is fitted to points representing municipal plants not greatly influenced by industrial wastes. Lower curve is fitted to points representing military plants excepting Monmouth.

Two curves are fitted to the data plotted in Figure 102. The upper curve pertains to municipal plants not greatly influenced by industrial wastes. The lower curve applies to the military plants excluding Monmouth. The shape and location of the two curves bring out clearly the differences in efficiency manifest between military and municipal plants.

Discussion

An examination of the data in the foregoing tables shows that activated sludge plants in military installations did not give higher percentages of removals of BOD or SS than did a number of other types of plants. Because of the large quantities of air used per gallon of sewage treated, these activated sludge plants, furthermore, were expensive to operate.

The operating and analytical data obtained indicate that as designed at present, activated sludge plants are not sufficiently flexible to handle the variations in flow and load that are to be expected in military posts. With the best available control and operation, very light sludge indices (over 200) were obtained during the greater part of the time. To prevent the loss of sludge over effluent weirs, application of chlorine or coagulants to the returned sludge and constant vigilance on the part of the operating staff were required.

Many problems remain to be solved before smooth operation of military activated sludge plants can be assured, and much additional research is needed in the activated sludge process as applied to military sewage.

CHAPTER VIII

SLUDGE DIGESTION AND DISPOSAL

For the study of sludge treatment methods, plants considered as generally representative have been listed with pertinent data in Tables 113 to 116. The tabulations represent plants with primary treatment alone, with low and high rate trickling filters and with activated sludge treatment. The only primary treatment plant listed is also the only plant listed with glass-covered sludge drying beds.

Of the trickling filter plants, the digestion space provided averaged 4.2 cf/cap based on the actual number served during the loading period. Similar figures were 11.2 cf/cap for the activated sludge plants and 1.5 cf/cap for the primary treatment plant. The Engineering Manual (January, 1943) stipulated design figures of sludge digester volumes to be 3.0 cf/cap for trickling filter plants, 4.0 cf/cap for activated sludge and 2.0 cf/cap for primary treatment plants. These design values were 50 percent larger with unheated tanks.

All but one of the plants listed used two-stage digestion, while the one primary plant used single-stage digestion. Of the twenty tanks represented, 13 were equipped with floating covers, 4 had fixed covers (1 with stirring mechanism), and 3 were open. The proportion of digestion space heated varied from 50 to 100 percent, with secondary digesters generally left un-

TABLE 113.—Digester and Sludge Bed Data at Several Posts

Post	Digesters								Sludge Beds		
	No. of Units	Total Volume (cf)	CF per Cap	Operation	Type Cover	Stirring Mechanism	Percent of Total Vol Heated	Av Temp (°F)	No. of Beds	Total Area (sf)	SF per Cap
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Campbell(b)	2 Pri 1 Sec	164,000	4.2	Stage	2 Flo	No	72	90	11	55,000	1.4
Crowder(b)	1 Pri 2 Sec	109,000	3.7	Stage	2 Flo 1 Fix	No	66	91	22	48,400	1.7
Fitzsimons G H(b)	1 Pri 1 Sec	30,000	4.5	Stage	2 Flo	No	100	98	4	10,000	1.5
Bragg(c)	1 Pri 1 Sec	199,000	4.1	Stage	2 Flo	No	100	98	10	65,000	1.3
Kilmer(c)	2 Pri 1 Sec	102,000	4.3	Stage	2 Fix 1 Flo	Yes	68	90	40	60,000	2.5
Butner(d)	2 Pri 2 Sec	230,000	7.5	Stage	2 Flo 2 Open	No	50	90	24	60,000	1.9
Shelby #2(d)	1 Pri 1 Sec	117,700	14.8	Stage	1 Fix 1 Open	No	54	90	12	38,400	4.8
Davisville(e)	2 Pri	19,600	1.5	Parallel	2 Flo	No	100	98	8	10,700(a)	0.8(a)

(a) Glass covered drying beds.
(b) Low rate filter.
(c) High rate filter.
(d) Activated sludge.
(e) Primary sedimentation.

TABLE 114.—Sludge as Added to Digesters

Post	Raw Sludge Added to Digesters								
	GAL/MG Sewage	Total Solids				Volatile Solids			
		Percent of Wet Sludge	LB/CF per Mo	LB per 1,000 Pop'n	LB per MGD Sewage	Percent of Total Solids	LB/CF per Mo	LB per 1,000 Pop'n	LB/MG Sewage
(1)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
Campbell	3,885	7.17	0.833	115.6	2,310	69.2	0.576	80.1	1,590
Crowder	4,780	6.14	1.366	167.0	2,440	74.0	1.010	123.7	1,803
Fitzsimons G H	5,640	2.86	0.602	89.8	1,346	80.3	0.484	72.1	1,080
Bragg	8,440	2.03	0.805	108.2	1,430	79.4	0.640	86.1	1,137
Kilmer	5,000	3.06	0.893	124.2	1,274	77.0	0.687	95.8	982
Butner	14,300	1.50	0.387	94.8	1,796	73.6	0.285	69.8	1,320
Shelby #2	14,100	1.47	0.252	122.3	1,730	—	—	—	—
Davisville	7,640	1.56	1.660	80.6	974	74.7	1.240	60.1	727

Note: Cols 14, 15, 16, 17, 19, 20 and 21 computed from SS removal in settling tanks.

heated. The temperature of the digesting sludge in heated tanks ranged from 90 to 98° F, averaging 93° F.

The quantities of raw sludge solids in Table 14 were estimated from the suspended solids decrease through settling tanks. It was found that this method afforded a more precise estimate of amounts of sludge solids than direct measurement from sludge samples. From the data it would appear that the obtaining of fair sludge samples was difficult under conditions operative in military sewage plants. Poor correlation was often found to exist between estimates of quantity of sludge solids by direct measurement and by drop in SS through primary and secondary settling tanks. The following table indicates the degree of interrelation observed at various posts:

Generally the degree of correlation between the two methods of measurement at a particular post was found to be of a low order; for a group of plants, higher correlations between averages were found. The study would indicate that unless a very thorough sludge sampling procedure is followed, SS data should be used to estimate weight of sludge solids. The usual sampling of sludge resulted in an overevaluation of solids. The error pertained only to raw sludge quantities; digested sludge as drawn from digesters was more homogeneous and ordinary sampling procedures sufficed to provide reliable estimates of solids.

In the plants listed, the total solids in the raw sludge added to the digesters varied from 1.47 percent for Shelby No. 2 (activated sludge) to 7.17 per-

Post	No Months Analyzed	Av Wt of Solids in Sludge Pumped to Digesters Daily (lb per 1,000 ppm)		Coef of Correlation and Probable Error
		From SS Decrease	From Sludge Samples	
Camp Pickett	14	105	113	−0.06±0.19
Fort Leonard Wood	20	170	188	0.18±0.15
10 Trickling Filter Plants	—*	115	151	0.88±0.07

* Average of 16 months' operation at each plant. Data are based on overall averages for entire period at each post.

TABLE 115.—Gas Production; Sludge Drawn From Digesters

Post	Gas Produced			Sludge Drawn from Digesters							
						Total Solids			Volatile Solids		
	CF per Cap Daily	CF/LB Vol Added	CF/LB Vol De-stroyed	GAL/MG Sewage	pH	Per-cent of Wet Sludge	LB/MG Sewage	LB per 1,000 Pop'n	Per-cent of Total Solids	LB/MG Sewage	LB per 1,000 Pop'n
(1)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)
Campbell	0.97	12.9	17.3	1,893	7.0	5.74	881	41.2	46.7	411	19.3
Crowder	1.79	14.5	21.4	2,096	7.3	6.29	1,098	75.0	53.4	584	39.8
Fitzsimons G H	1.32	18.3	31.2	1,992	7.2	4.38	730	48.8	61.8	446	29.8
Bragg	0.93	10.8	15.5	1,256	7.4	5.60	590	44.6	58.8	347	26.2
Kilmer	1.06	11.1	14.5	1,218	7.7	4.20	428	41.7	60.0	256	25.0
Butner	1.40	19.4	29.8	2,180	7.4	4.80	867	45.7	53.4	463	24.4
Shelby #2	—	—	—	2,660	6.8	3.30	720	51.0	50.7	366	25.9
Davisville	0.62	10.4	13.9	787	7.0	5.91	388	32.1	47.6	185	15.3

Note: Cols 23 and 24 computed from SS removal in settling tanks.

TABLE 116.—Sludge—As Applied to Drying Beds

Post	Sludge Applied to Drying Beds							Sludge Removed		
					Total Solids		Volatile Solids			
	Months per Year Sludge Applied	Av Depth (in.)	CF per 1,000 Pop'n Yearly	Total Depth (in. per year)	Percent of Wet	LB/YR/SF	Percent of Total	CF/SF Yearly	CF per 1,000 Pop'n Yearly	Total Depth (in. per year)
(1)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42
Campbell	10	9	4,205	37.6	5.7	11.2	46.7	0.61	868	7.8
Crowder	12	6	6,780	51.0	6.3	16.5	53.4	0.98	1,620	12.2
Fitzsimons G H	10	9	6,500	43.0	4.4	11.8	61.8	0.91	1,380	10.9
Bragg	10	14	4,650	41.6	5.6	14.5	58.8	0.72	969	8.7
Kilmer	12	7	5,800	27.8	4.2	6.1	60.0	0.34	860	4.1
Butner	10	7	5,626	34.7	4.8	8.7	53.4	0.76	1,492	9.2
Shelby #2	5	—	9,230	23.0	3.3	3.9	50.7	0.12	548	1.4
Davisville	12	6	3,180	47.0	8.7	21.2(a)	—	—	—	—

(a) Covered sludge beds.

cent for Campbell (low rate filter). Ordinarily the wettest sludge would be expected in activated sludge plants, while that with the greatest percentage of solids would be expected in primary treatment. Probably the low figure of 1.56 percent total solids for Davisville resulted from sludge pumping beyond the minimum requirements as shown by column (13). The raw volatile solids in the plants listed fall in the upper part of the range which is expected in separate municipal systems. As shown in the tabulation, this figure varied from 69.2 percent to 80.3 percent, averaging 75.5 percent.

Of the eight plants listed, sludge gas production figures were available at seven. Mean production of the six complete treatment plants was 1.25 cf/cap daily, while at the single primary treatment plant the corresponding figure was 0.62. The seven plants gave gas yields of 13.9 cf/lb of volatile

PLATE 23.—Sludge digesters at Camp Plauche, NOPOE, La.

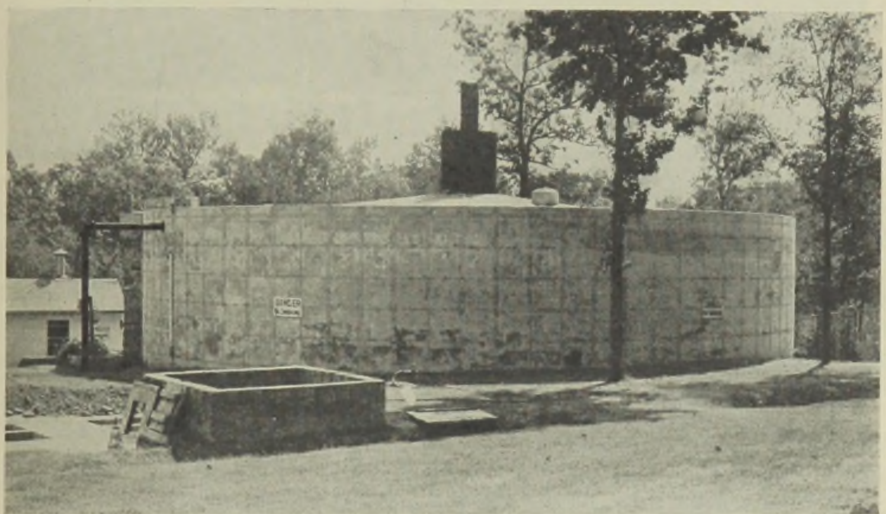


PLATE 24.—First-stage digester at Camp Joseph T Robinson, Ark.

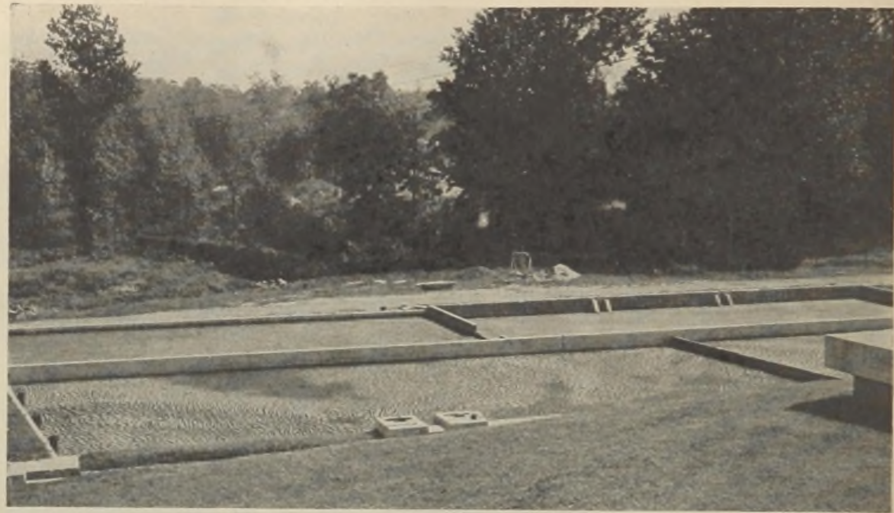


PLATE 25.—Sludge beds at Camp Joseph T Robinson, Ark.

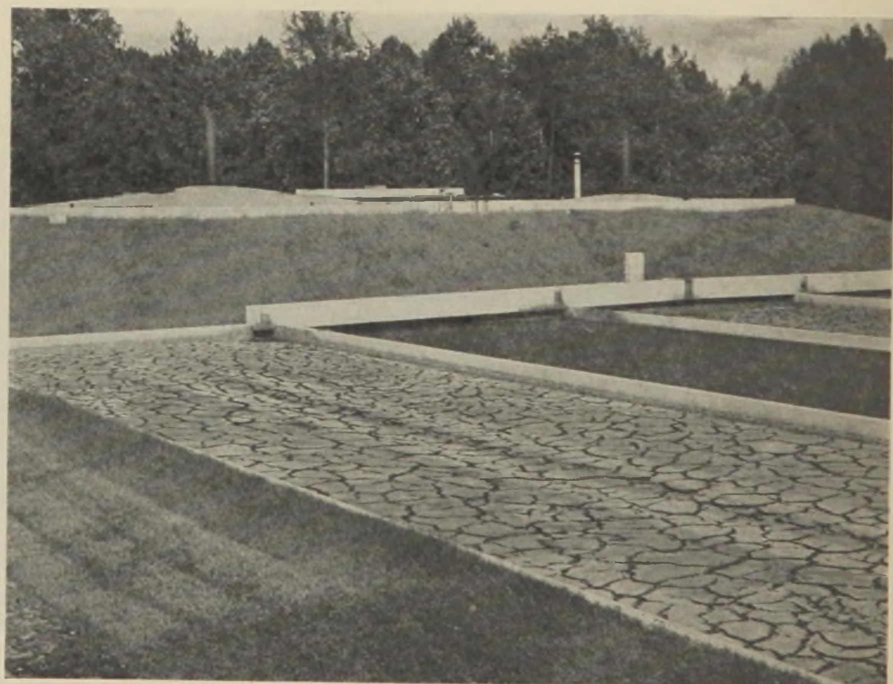


PLATE 26.—Sludge beds at Camp Wheeler, Ga.

matter added and 20.5 cf/lb of volatile matter destroyed. Both of these figures are slightly higher than is usual in municipal experience. The average destruction of solids added to the digesters of the listed plants was 69.0 percent of the volatile solids and 57.1 percent of the total solids.

The digested sludge of the eight plants averaged 7.2 in pH, 5.0 percent total solids and 54.1 percent volatile solids. The latter figure, while higher than general municipal practice, reflects the higher volatile solids content of military sewage.

All digested sludge was dried on conventional drying beds. The sludge beds were open with the exception of Davisville, the primary treatment plant, which had glass-covered beds. As indicated in the table, loadings were generally light, amounting to only 10.4 lb of dry solids per sf annually in the complete treatment plants and 21.2 lb of dry solids per sf annually in the primary treatment plant with covered sludge beds. Accepted figures for municipal practice are 25 to 40 lb per sf annually.

Of the open drying beds, the average sludge production in inches per year was 40.0. The corresponding depth for dried sludge was 7.6 inches per year. The digested sludge data on Table 115 were based on operating data, representing twelve-month periods in order to avoid distortion due to seasonal influences.

The general selection of sludge drying beds over other means of sludge dewatering gave the plants an inexpensive method of sludge dewatering which in general was satisfactory. The success of the method was predicated upon adequate underdrainage, which was lacking in a few instances. The installation of mechanical dewatering devices would not have been justified because of the general availability of land at the plant sites and the availability of labor in camps. The lack of skilled operation which vacuum filters would have required and the saving of critical materials in construction and chemicals in operation also were factors. Many of the plants serving military installations were too small for overall economical justification of mechanical dewatering devices. The location of many camps in portions of the country where favorable weather occurred was a contributing factor to the successful operation of sludge drying beds.

At a number of plants, liquid digested sludge was distributed over land areas in the cantonment. This method of disposal was tried initially on an experimental basis at several posts in the Third Service Command, the Great Lakes Naval Training Station and a number of other posts. The results appeared to justify the method from an economic standpoint under conditions existing at some military establishments.

The medical aspects of the problem were in general recognized by official personnel in Service Command and Navy Area headquarters. The danger inherent in an unrestricted program of wet sludge disposal without regard to such factors as (a) latitude, (b) soil condition (perviousness), (c) type of sludge (moisture and degree of digestion) and (d) wind direction and velocity were emphasized by wartime investigations demonstrating existence of parasitic organisms, such as *Ascaris lumbricoides* and *Tricuris trichiura* in military sewage sludge. Wet sludge disposal at military hospitals might under unfavorable conditions be an exceedingly undesirable practice.

Aside from questions of medical significance, it is pertinent to point out that economic conditions favoring wet sludge disposal at certain military posts may not be operative in a post war civilian economy. Plants at military posts were often located near extensive tracts of comparatively open uninhabited land. Moreover, the effect upon the perviousness of the soil of continual dosing over a period of several years has not been established. The cost of obtaining and maintaining the requisite number of tank trucks at a cantonment for motorized divisions would undoubtedly be different from that in a municipality. The availability and cost of labor in the post-war era may be expected in general to differ markedly from availability and cost of labor during World War II.

CHAPTER IX

OXIDATION PONDS

A number of plants at military installations utilized oxidation ponds to attain secondary treatment. These units consisted of large open basins, 2 to 3 feet deep, into which settled raw sewage was introduced. The ponds were usually connected in series. Design capacities were in the range of 400 to 700 persons per acre (25 to 50 lb BOD/af). Detention periods were of the order of 20 to 30 days.

While the idea of oxidation ponds is not new, recent developments have tended to make them more advantageous in military installations. The introduction of the modern bulldozer and other heavy earth-handling equipment into construction practice has made the excavation and the cleaning of large basins more practicable. The required area was available at many posts. In arid regions the recharging of ground water that may be concomitant with sewage treatment in oxidation ponds also constitutes a consideration in connection with their use.

Data were examined from three large posts where oxidation ponds were installed. Two of these were in Texas at army installations—Camp Hood and Camp Barkeley. The third was at a US Naval Station at Shoemaker, California. Exceptionally complete data were available from this plant. The following description is a condensation of a report by Caldwell and Ackerman of the Twelfth Naval District.

Oxidation Ponds at USNS, Shoemaker, Calif.

The Shoemaker facilities consisted of primary settling with separate sludge digestion, oxidation ponds and a final chlorine contact pond. The plant effluent was discharged into Alamo Creek, a tributary of Alameda Creek which was used for recreational purposes. The four oxidation ponds were connected in series and had dimensions as indicated in the following schedule:

Pond No.	Area (acres)	W. S. Elev. (ft)		Bottom Elev (ft)
		Maximum	Normal	
1	4	327.5	327.0	324.5
2	26	327.5	327.0	324.0
3	25	327.5	327.0	324.5
4	46	326.3	325.8	322.5

The overall volume of the oxidation ponds including the chlorine contact pond was about 300 acre-feet.

Sampling

Samples of raw sewage and primary effluent were composited hourly using equal volumes. The plant effluent composite was made up from four equal spot samples collected at six-hour intervals. Diurnal variation of organic matter in the plant effluent was not marked because of the equalizing action of the ponds. Therefore samples of effluent taken at six-hour intervals were adequate. All composites were refrigerated during the collection period and analyzed immediately thereafter. Samples for bacteriological analysis were collected each day. Plantings were made from one to twenty-four hours after collection.

Operating results for a twelve-month period, starting in August, 1944, are presented in the following table:

	Pop'n	Sewage Flow (mgd)	Algae Final Eff* (ml/liter)	Coliform Bacteria** (organisms/100 ml)		Dissolved Oxygen Final Eff (ppm)
				Pri Eff	Final Eff	
Av	40,000	3.71	0.13	10^7	10^2	8
Max	51,000	4.13	0.55	10^8	10^3	15
Min	29,000	2.92	Tr	10^6	10^1	0

* Based upon 9 months of operation.
** Median value of coliform index.

	SS (ppm)			BOD (ppm)		
	Raw Sewage	Pri Eff	Final Eff	Raw Sewage	Pri Eff	Final Eff
Av	254	95	54	236	143	34
Max	310	115	140	260	180	50
Min	205	70	25	180	100	20

The average per capita flow of sewage was 93 gal daily; the per capita production of BOD was 0.18 lb daily, of SS 0.19 lb daily. Removal in primary settling was 63 percent for SS and 39 percent for BOD. Efficiency of the oxidation ponds was 43 percent for SS and 76 percent for BOD. Removal in the oxidation ponds was 16 percent for SS and 46 percent for BOD. Overall removal was 79 percent for SS and 85 percent for BOD.

SS in oxidation ponds increased during the summer by the growth of algae that imparted an emerald green color to the sewage. There was evidence that most algal cells started to decompose during the 5-day incubation period and therefore increased BOD values. When algal metabolism was low, the BOD in plant effluent fell as low as 10 ppm. The monthly average for February to May, 1945 was less than 20 ppm.

Algae

The total volume of algae in the effluent was determined in an Imhoff cone by killing the algae with formaldehyde and allowing them to settle overnight. The tests were not started until November, 1944 and the values in the above table were based upon a 9-month period. Samples collected from the receiving stream indicated a marked diminution in the algae concentration in the downstream samples. At no time, however, were odors of decaying algae noted. Odors at the plant were not particularly objectionable, but were noticeable during certain periods. Microscopic examinations were made several times to determine the species of algae present. *Euglena* was normally predominant; *Pandorina* was also found; on one occasion *Chlorella* was present. Relatively few crustaceans were found.

Bacterial Efficiency

The ponds had a remarkable ability to remove coliform organisms. Individual samples very often would show no positive presumptive tubes in 10-ml portions. No chlorine was used during the period reported. Bacterial density was not invariably low; occasionally samples would contain as high as 10,000 coliform organisms per 100 ml. The average drop in bacterial concentration through the ponds, however, was about 10^7 organisms per milliliter. It was not precisely established what element in the environment was responsible for the notable bacterial efficiency; pH values, though often high, were not responsible. The most plausible explanation is that other saprophytic organisms in the heterogeneous, teeming flora and fauna of the ponds were better adapted to survival in the competition for food.

Effect of Chlorine

During August 1944, experiments were carried out to determine the effect of chlorine upon algae and bacteria in the plant effluent. Chlorine dosages varied from 12 to 15 ppm. Algae were partially killed and settled out in the chlorine contact pond, which had an area of 3.2 acres. The algae reduction was shown by a corresponding reduction in turbidity in the effluent. A distinct fishy odor developed at the point of chlorination.

Coliform bacteria increased somewhat despite chlorination. When chlorination was stopped, coliform bacteria density decreased. Chlorine definitely altered the aquatic environment in favor of the coliform organisms.

Alkalinity

Algal activity during daylight hours often was such that carbon dioxide and the bicarbonate ion concentration were reduced to the extent that phenolphthalein alkalinity became as high as 100 ppm (as $CaCO_3$). Total alkalinity remained nearly constant at 400 ppm (as $CaCO_3$). During September, 1944 an average of 200 ppm of carbonate alkalinity was transformed. This represented a CO_2 consumption of at least 3,000 lb/day. This figure does not include free CO_2 produced by bacterial action. Periods of high phenolphthalein alkalinity corresponded with periods of high pH and turbidity in the plant effluent.

Dissolved Oxygen

Algal action induced a high degree of supersaturation of dissolved oxygen in the oxidation ponds. No special apparatus was used in collection of samples; hence results for supersaturated samples may be slightly low. Reagents were added immediately after sample collection. The average DO in the effluent varied from a trace to 15 ppm. Results for individual days varied from 0 to 25 ppm. Low values occurred in winter when photosynthetic activity was low.

Nitrogen Tests

Ammonia nitrogen averaged 20 to 30 ppm in the primary effluent, while nitrites and nitrates were not present in appreciable concentrations at this point. Reduction of ammonia in the oxidation ponds was greatest during the summer; less than 1 ppm of NH_3-N was found in the plant effluent during September, 1944. Accompanying the decrease in ammonia was an increase in nitrate and nitrite. The increase in nitrite and nitrate was less than the reduction in ammonia; the difference went to synthesis of algae and other microorganisms. The highest monthly average concentration of nitrite in the plant effluent observed was 0.4 ppm. Nitrate concentration also was relatively low. This, however, does not imply that nitrifying bacteria were not active.

Nitrates constitute one of the most available forms of nitrogen for plankton that absorb their nourishment through the cell membrane. Such organisms are stimulated in their growth by even small concentrations of nitrate when, as was the case at the Shoemaker ponds, the source from which nitrate is produced is abundant, namely, ammonia.

Diurnal Variation

On August 23 and 24, 1944, a 24-hr test was made of the Shoemaker ponds. DO, pH and alkalinity were determined approximately every two hours, at seven sampling stations. At each station composite samples were collected in proportion to flow. These samples were analyzed for BOD, SS, NH_3-N, NO_2-N, NO_3-N, etc. Results are indicated in the table at top of page 1026.

It is of interest to note that the quality of the effluent of Pond No. 2, as measured by the BOD and SS tests, was better than that of any other pond except the chlorine contact pond. The increase in BOD and SS in the last ponds was due to a proliferous growth of algae. The algal growth was accompanied by a reduction in ammonia with first an increase and then a decrease in nitrification. After chlorination, bacterial aftergrowths produced carbon dioxide at a rate greater than algae could use it, and pH decreased. The reduction in algae, after chlorination, was evidenced by the sharp reduction in turbidity, SS and BOD.

The sewage flow during the 24-hr period was 3.75 mg. Dissolved oxygen

Sample	NH_3-N (ppm)	NO_2-N (ppm)	NO_3-N (ppm)	pH	Turbidity (ppm)	SS (ppm)	BOD (ppm)
Raw Sewage	17	0.08	0	7.4	200	145	170
Settled Sewage	15	0.08	0	7.5	140	80	95
Eff.-Pond 1	15	0.08	0	7.8	130	54	60
Eff.-Pond 2	12	0.12	0.04	8.4	80	64	40
Eff.-Pond 3	6	0.16	0.28	9.0	180	115	45
Eff.-Pond 4	1	0.12	0.04	9.1	170	110	45
Eff.-Chlorine Contact Pond*	4	0.12	0.02	8.1	50	35	25

* Chlorine added.

values varied considerably during the day. The ranges observed at various sampling points are indicated in the following schedule:

Sampling Point	Dissolved Oxygen (ppm)	
	Maximum	Minimum
Raw Sewage	4.0	1.0
Pond No. 2	15.0	2.0
Pond No. 3	25.0	7.5
Pond No. 4	14.0	4.0

Maintenance of Ponds

No mosquito nuisance from the ponds developed; at the time of visitation, no larvae were found in the littoral regions. Water plant growth was not marked. The surface of the downstream ponds was free from scum and no evidences of sludge deposits were detected. At the time of inspection, the ponds had been operating continuously since January, 1943 when they were installed. No sludge had been removed from the ponds.

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Editorials

THE NRC REPORT

Each year since 1941 a special effort has been made to assemble outstanding editorial material for the September Convention Number of THIS JOURNAL. Consequently, there was considerable satisfaction in the completion of arrangements to publish in this issue the valuable contribution that has come from the Subcommittee on Sewage Treatment of the Committee on Sanitary Engineering, National Research Council.

But there is another reason that the NRC report is appropriately featured at this time. The 1946 Convention Number commemorates the first annual meeting of the Federation since the conclusion of World War II; it is only fitting that it also perpetuates the work of the hundreds of sanitary engineers, chemists and others who designed, built, operated and administered the sewage works that served the Army and Navy so well during the great conflict. The record of these men honors our profession.

Particular credit for the report is owing to the Repairs and Utilities Division of the Office of the Chief of Engineers and to the Sanitary Corps of the Army. These agencies were mainly responsible for the overall supervision given military sewage works, for the training of operators and for the complete records that were compiled. Without these things the accomplishment of the subcommittee would have been impossible.

This report represents probably the most comprehensive analysis of mass sewage works operation data that has ever been attempted. It is a statistical masterpiece and will be referred to many times as an example of proper and thorough statistical analysis. Moreover, it is an exceedingly practical piece of work in that it holds to the fundamental purposes of the various sewage treatment units and concentrates the huge volume of original data into values of loading and performance that are easy to understand and apply.

Many readers will note that the regular JOURNAL style sheet was cast aside in the publication of this report. The writer feels that the abbreviations are somewhat extreme although he favors in general the "periodless" abbreviations presently recommended by the American Standards Association. It is also believed that some of the tables might have been consolidated to advantage. Editorial concessions were made, however, because of the apparent desire of the subcommittee to present the report in this form and because a limited time was available for preparing the manuscript for the printer.

The report is of such general technical excellence that it is not easy to select "highlights" for specific attention; it is more than likely that the interest of each reader will center upon different parts. The writer is particularly impressed with the chapter on trickling filters, which contains information of great and timely value on loadings and performance of conventional and high rate trickling filters, and on the benefits and limitations of recirculation. He is also intrigued by the mathematical analysis of the influence on sedimentation efficiency of variation in weir overflow rates at settling tanks. A third item of personal interest is the care used throughout the study in selecting proper reporting units; this is especially noteworthy in the discussions on contact aeration and activated sludge where the parameters "weight (load) per contact area-time" and "weight (load) per weight (mixed liquor solids)-time," respectively, are employed to interesting advantage. There are many other points that might have been given similar mention as "highlights."

The subcommittee has been painfully careful, and properly so, to confine all conclusions to the conditions of sewage flow and character encountered in military sewage works. This does not mean, however, that the report has no value beyond its usefulness in the event that extraordinary numbers of military installations ever again become necessary. Practically every finding in the study is adaptable to municipal sewage works practice, if proper adjustment is made, of course, for the differences in sewage characteristics and flow rates and for the divers industrial wastes that are endowing each municipal sewage works with specific individual traits. The work should be a significant technical reference for the sewage works field in the impending years of intense activity.

The Federation takes pride in its selection as the vehicle to carry the NRC report to those who will be able to use it to best advantage.

20TH ANNUAL MEETING IN 1947 TO BE CONCURRENT WITH AWWA CONFERENCE AT SAN FRANCISCO

As this issue goes to press, the Board of Control of the Federation has overwhelmingly approved, by letter ballot, a recommendation by the Meeting Place and Executive Committees that the Twentieth Annual Meeting of the Federation be held at San Francisco during the week of July 21, 1947, the meeting to be held separately but concurrently with the 1947 Conference of the American Water Works Association. This two-fold attraction will undoubtedly draw the largest gathering of sanitary engineers and others engaged in the water and/or sewage works fields that has ever been assembled.

Neither of the international organizations consider the 1947 arrangement to be a precedent for their future annual meetings. With AWWA scheduled to meet in California in 1947 and the Federation largely committed to hold its first postwar meeting there, it was deemed impracticable to hold an entirely separate Federation meeting on the Pacific Coast only three months after the AWWA Conference at San Francisco. The decision was clinched by the fact that ample hotel accommodations and meeting facilities have been guaranteed for the concurrent meetings in July. Conditions are such at this time that a year is all too short to reserve hotel and other accommodations for a national technical conference.

Except for the joint manufacturers exhibit, the meetings are planned to be conducted separately, in so far as possible. The Municipal Auditorium at San Francisco is arranged admirably for the purpose. Separate and non-conflicting social and entertainment events are also be to arranged.

Further details will be forthcoming later.

W. H. W.

Federation Affairs

A PREVIEW OF THE NINETEENTH ANNUAL MEETING

Royal York Hotel, Toronto, Can., October 7–9, 1946

★

IN CONJUNCTION WITH
THIRTEENTH ANNUAL CONVENTION
THE CANADIAN INSTITUTE ON SEWAGE AND SANITATION

★

Canada, in her autumnal splendor, is ready to welcome those who will attend the Nineteenth Annual Meeting of the Federation at Toronto's beautiful Royal York Hotel on October 7–9, 1946. The Canadian Institute on Sewage and Sanitation is renowned for the excellence of its own meetings; on this occasion, the Institute performs its functions as host to the Federation through Dr. A. E. Berry and his several convention arrangement committees, all of which have worked long and hard to make the first international meeting in Canada a memorable one. Technically and socially, the Nineteenth Annual Meeting is certain to be a huge success!

THE PROGRAM

SUNDAY EVENING—OCTOBER 6

7:00 **Pre-Registration**—Convention Floor Foyer
Inspection of Exhibits—Banquet Hall and Hall A

8:30 **Choral Concert**—Concert Hall
The Leslie-Bell Singers

MONDAY MORNING—OCTOBER 7

9:00 **Registration**—Convention Floor Foyer
Inspection of Exhibits—Banquet Hall and Hall A

10:30 **Nineteenth Annual Meeting Called to Order** by President J. K. Hoskins—Tudor Room
General Business Session

11:00 **Sewage Treatment at Toronto**
William Storrie, Partner, Gore and Storrie, Consulting Engineers, Toronto, Canada
Discussion
Almon L. Fales, Metcalf and Eddy, Boston, Mass.

12:00 **The Clubroom**—Roof Garden

12:30 **FEDERATION LUNCHEON**—Concert Hall
Greetings to the Federation
Hon. R. H. Saunders, Mayor, Toronto, Canada
Environmental Sanitation in the Field of Public Health
Dr. J. T. Phair, *Deputy Minister of Health of Ontario,* Toronto, Canada

MONDAY AFTERNOON—OCTOBER 7

Ball Room

Presiding: Francis S. Friel, *Vice-President,* FSWA

2:30 **Characteristics and Methods for Treatment of De-inking Wastes**
Dr. W. W. Hodge, *Senior Fellow,* Mellon Institute of Industrial Research, Pittsburgh, Pa., and
Philip F. Morgan, *Research Engineer,* National Council for Stream Improvement (of the Pulp, Paper and Paperboard Industry), Kalamazoo, Mich.

Discussion
William S. Wise, *Chief Engineer,* Connecticut State Water Commission, Hartford, Conn.

3:15 **Industrial Alcohol Production from Sulfite Waste Liquor**
H. G. Joseph, *Alcohol Plant Superintendent,* Ontario Paper Co., Thorold, Ontario, Canada

Discussion
Prof. R. G. Tyler, University of Washington, Seattle, Wash.

4:00 **Standards of Stream Sanitation**
H. W. Streeter, *Sanitary Engineer Director,* Water and Sanitation Investigations, U. S. Public Health Service, Cincinnati, Ohio

Discussion
Milton P. Adams, *Executive Secretary-Engineer,* Michigan Stream Control Commission, Lansing, Mich.

4:45 **The Hypochlorite Process for Treatment of Wool Scouring Wastes and the Recovery of Wool Grease**
Harry A. Faber, *Research Chemist,* The Chlorine Institute, Inc., New York City.

Discussion
Joseph L. Campanella, *Chemical Engineer,* Fields Point Mfg. Co., Providence, R. I.

5:30 **Adjourn Session**

MONDAY EVENING—OCTOBER 7

5:30 **The Clubroom**—Roof Garden

7:15 **DINNER AND ENTERTAINMENT**—Concert Hall
Address
Hon. Russell T. Kelley, Minister of Health, Province of Ontario

10:00 **The Clubroom**—Roof Garden

TUESDAY MORNING—OCTOBER 8

Concert Hall

Presiding: H. S. Nicklin, *President,* CISS

9:00 **Scum Control in Digesters**
H. S. Rankin, The Dorr Company, Inc., Engineers, New York City

Discussion
Harry E. Schlenz, *Vice-President*, The Pacific Flush Tank Company, Chicago, Ill.

Incineration Problems
George J. Schroepfer, *Professor of Sanitary Engineering,* University of Minnesota, Minneapolis, Minn.

Discussion
Mark B. Owen, *Partner,* Moore and Owen, Consulting Engineers, Indianapolis, Ind.

Sludge Heating
Henry J. Miles, *Associate Professor of Civil Engineering,* University of Southern California, Los Angeles, Calif.

Discussion
Willem Rudolfs, *Chief,* Dept. of Water and Sewage Research, New Jersey Agricultural Experiment Station, New Brunswick, N. J.

Present Status of Sludge Gas Utilization
Richard H. Gould, *Director,* Division of Engineering and Architecture, Department of Public Works, New York City

11:45 **The Clubroom**—Roof Garden

12:30 **Canadian Institute Business Luncheon**—Crystal Ball Room, 17th Floor, King Edward Hotel

TUESDAY AFTERNOON—OCTOBER 8

Ball Room

Presiding: J. K. Hoskins, *President,* FSWA

2:30 **Financing Sewage Systems and Sewage Treatment**
Francis S. Friel, *Partner,* Albright and Friel, Inc., Consulting Engineers, Philadelphia, Pa.; *Vice-President,* FSWA

Discussion
F. M. Veatch, *Partner,* Black and Veatch, Consulting Engineers, Kansas City, Mo.

3:30 **British Developments in Sewage Purification Practice**
John Hurley, *Manager and Chemist,* Sewage Disposal Dept., Tettenhall, Wolverhampton, England; *President,* Institute of Sewage Purification

Discussion
F. W. Mohlman, *Director of Laboratories,* The Sanitary District of Chicago, Chicago, Ill.; *Advisory Editor,* FSWA

4:30 The Army Sewage Treatment Program

Harold A. Thomas, Jr., *Faculty Instructor in Sanitary Engineering,* Graduate School of Engineering, Harvard University, Cambridge, Mass.

Discussion

Rolf Eliassen, *Professor of Sanitary Engineering,* New York University, New York City

5:30 Adjourn Session

TUESDAY EVENING—OCTOBER 8

5:30 The Clubroom—Roof Garden

7:15 ANNUAL DINNER AND DANCE—Concert Hall and Ball Room

9:00 The Clubroom—Roof Garden

WEDNESDAY MORNING—OCTOBER 9

Concert Hall

Presiding: H. S. Nicklin, *President,* CISS

9:00 The Operators Forum

Activated Sludge Problems
Trickling Filter Problems
Sludge Disposal Problems
Sewer Regulations
Sewage Chlorination Problems
Combined Sewer Overflows

12:00 Award of Convention Attendance Cup
Adjournment of Nineteenth Annual Meeting

TECHNICAL PROGRAM COMMENTS

It will be evident from the foregoing that Program Chairman F. W. Gilcreas and his committee have done another commendable job in assembling a technical program of diversified and timely papers, to be presented and discussed by acknowledged leaders in their fields. New developments, design and operation practices, industrial waste problems, British trends, financing and other topics are represented—all of them of great current interest and value.

The Operator's Forum on Wednesday morning, a tried and proved feature of Federation programs, will be conducted under the leadership of the Canadian Institute. The discussion subjects that have been selected are certain to provoke active and productive forums.

There is something worthwhile here for everyone in the field!

EXHIBITS

The exhibit of sewage works equipment and supplies will be particularly complete and interesting because exhibitors will include member companies of the Canadian Sanitation Equipment Manufacturers Assn. in addition to the Water and Sewage Works Manufacturers Assn. The Royal York offers extraordinary facilities for a show of this kind and those attending the meeting will be impressed with the exhibit of postwar products that has been developed under the supervision of Secretary-Manager Arthur T. Clark of the WSWMA.

SOCIAL FUNCTIONS

A departure from former meetings is the replacement of the stag smoker with the Canadian Institute Dinner and Entertainment on Monday evening, October 7. The Hon. Russell T. Kelley, Provincial Minister of Health of Ontario, known throughout Canada as an entertaining speaker, will give a short address at the dinner. Ladies are invited to this function as well as the Federation Dinner-Dance on Tuesday evening, October 8.

Another feature new to annual meetings of the Federation will be the "Club Room" sessions which will precede and follow both of the evening events. Refreshments will be served and fellowship will be the prevailing theme. The Club Rooms are made possible through the joint courtesy of the Canadian Sanitation Equipment Manufacturers Assn. and the Water and Sewage Works Manufacturers Assn.

LADIES ENTERTAINMENT

The Canadian Institute is reputed for the caliber of entertainment provided at its conventions for the ladies. The Ladies Entertainment Committee, headed by Mrs. J. B. Kinney, has planned a series of events that will more than uphold this tradition.

Beginning with the choral concert on Sunday evening, the ladies' program gets into full swing on Monday. Following the Federation Luncheon, they will be taken by bus to the Ontario Provincial Parliament Buildings for a tour and reception by The Hon. Albert Matthews, Lieutenant Governor of Ontario, and Mrs. Matthews. The Monday program closes with the Dinner-Entertainment and Club Room in the evening.

On Tuesday, the ladies will enjoy a luncheon at the "Old Mill," transportation by bus being furnished. An afternoon bridge party and the evening Dinner-Dance and Club Room will complete another delightful day. Wednesday is left open for shopping and sight-seeing.

Ladies headquarters at the Royal York will be in the Library, on the Main Mezzanine, one of the most interesting rooms in the hotel.

INSPECTION TRIPS

Provision will be made as desired for those wishing to visit the 10-m.g.d. activated sludge plant at North Toronto; the new primary sewage treatment plant, now under construction, which will serve Toronto proper; the municipal incineration plants and the municipal water works. No arrangements are being made to handle large groups on these trips.

In addition to the above, Toronto boasts many historical and other points of interest. A tour of the city will be well worth the time required.

SUGGESTIONS ON ENTERING CANADA

No passport is required of those entering or leaving Canada, but proof of identification may be requested upon re-entry to the United States. An old passport, birth certificate, driver's license, Selective Service registration card or similar document will be acceptable for such proof. Naturalized citizens should carry their naturalization certificates.

Gasoline and oil sufficient for 300 miles of driving, 50 cigars, 200 cigarettes and two pounds of tobacco may be taken into Canada duty free. Automobile travelers should carry their state license cards. Gasoline is more expensive in

Canada but purchasers should note that the Imperial Gallon used in Canada is 20 per cent larger than the U. S. gallon.

Visitors from the United States who have been in Canada 48 hours or longer may take back duty free articles aggregating up to $100 in value, if such articles are for personal or household use, souvenirs or curios not for sale. The exemptions for members of the same family may be grouped.

Until recently, there was a 10 per cent premium on U. S. money when converted to Canadian funds. The Canadian and U. S. dollars are now adjusted to equal value, however, and the two currencies are equally acceptable throughout Canada.

HOTEL ACCOMMODATIONS

A bustling industrial city and an attractive haven for vacationists, Toronto has a number of fine hotels. Occupancy has been very heavy in recent months, however, and Dr. Berry's Local Arrangements Committee has expended considerable effort in securing allocations of rooms for the use of those attending the Federation meeting.

Accommodations have been reserved for about 600 persons in six Toronto hotels, and these are being made available first to those attending from the United States. Members of the Canadian Institute are generously fulfilling their obligations as hosts by securing accommodations wherever and however they may be found. The total registration for this meeting is expected to reach a record-breaking 800 or more.

Those who do not already have a letter or card from Dr. Berry advising that accommodations have been assigned should write or wire immediately for reservations. *Requests should be addressed to Dr. A. E. Berry, Sanitary Engineering Division, Ontario Department of Health, Toronto 8, Ontario, Canada.* Room-sharing arrangements utilizing double rooms to the greatest possible extent will be necessary to accommodate the heavy anticipated registration.

TORONTO BECKONS IN OCTOBER!

SEWAGE WORKS EQUIPMENT AND SUPPLIES*

THE AMERICAN BRASS COMPANY

General Offices: WATERBURY 88, CONNECTICUT

COPPER-SILICON ALLOYS FOR SEWAGE TREATMENT EQUIPMENT

This strong, highly corrosion-resistant group of alloys, typified by The American Brass Company's Everdur Metal, has given superior service in the sewage equipment field for nearly two decades.

Everdur, made in several types, consists essentially of copper and silicon, with controlled additions of other elements. The tensile strength, in wrought form, ranges from a minimum of 50,000 p.s.i. for annealed tank plates to approximately 100,000 p.s.i. in cold worked forms. Everdur alloys all possess high fatigue resistance and resistance to corrosion equal to or better than pure copper. There are Everdur alloys adjusted for hot working and for cold work, and, with two exceptions, these metals are readily welded by either the oxy-acetylene torch or the carbon arc.

Applications

The principal applications of Everdur for sewage treatment are as follows: coarse and fine screens, swing gates, built-up sluice gates, coarse bar rack aprons, effluent weirs and scum weirs, scum baffles and brackets, troughs, screen hoppers, orifices, baskets, anchors, ladders, float gage chain, valve springs, manhole steps, walkways, bars and plates, bolts and nuts, electrical metallic tubing and rigid conduit.

FIGURE 1.—Air filter of secondary room of the Ward's Island sewage treatment plant. Framework of filter structure and filter holders is of welded and bolted Everdur sheet and strip. Retaining mesh for filter is perforated Everdur sheet. Fabricated by American Filter Co., Inc.

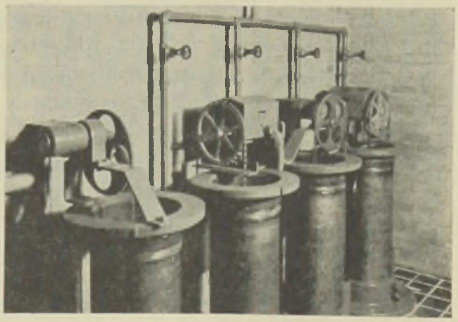


FIGURE 2.—Control mechanism of 12-in. diameter Everdur float tubes. Designed by Krajewski-Pesant Mfg. Corp. for the Ward's Island sewage treatment plant, New York City.

Advantages

Because of high strength, corrosion-resistance and weldability, copper-silicon alloys have effected considerable economies in equipment previously made of heavy iron castings. Relatively lightweight wrought sheets of Everdur, assembled by welding, provide much lighter, more easily operated and more durable equipment.

These copper alloys cost less than most high strength corrosion-resistant alloys and not only meet many of the definitely determined corrosion problems of sewage treatment, but also provide the necessary additional protection where the forces of corrosion are variable.

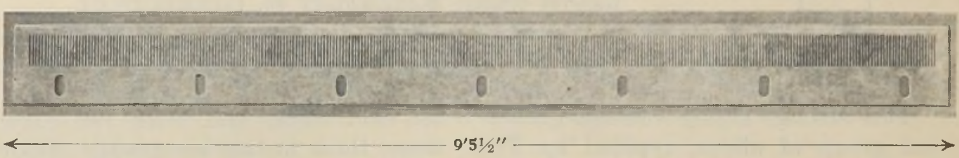


FIGURE 3.—More than 100 of these Everdur weir plates, 3/16 in. thick, were installed in the concrete tanks of the Coney Island sewage treatment works. The 3/16-in. slots are 4 in. long, milled on 3/8-in. centers and relieved on the underside. Plates are anchored with 1-in. by 2-1/2-in. Everdur bolts.

AMERICAN CAR AND FOUNDRY COMPANY

30 Church St., New York 8, N. Y.

The ACF Lubricated Plug Valve, produced by American Car and Foundry Company, has been proved eminently successful in sewage treatment plant applications. This valve is constructed with a cylindrical type plug by means of which full pipe area is obtained as well as the elimination of possible hard turning that might be due to the wedge effect of a taper.

The full pipe area feature is especially desirable in handling sewage and sewage sludge, which materials are usually flowed under low pressures and in heavy consistency. Materials in suspension are sheared off when operating the valve and there are no pockets or recesses in which foreign particles may lodge.

As compared with other types the ACF Lubricated Plug Valve is quick operating, and compact in construction, lending itself readily to close installations. There are no wearing surfaces exposed to the lading when in the open position and on flow. All vital seating surfaces are sealed away from the fluid passageway when in the open position.

ACF Lubricated Plug Valves can be supplied in the regular standard rectangular port type having full pipe area, or they can be supplied with full round port having port area cross-section the same as standard pipe in any given size. Full area valves can be supplied in sizes up to and including 16 in.

The Company has available, upon request, a descriptive bulletin No. S–2M–2–46 giving suggested specifications for their inclusion in sewage works designs. This bulletin also contains a list of representative sewage treatment plants that are equipped with these valves handling such services as raw sewage, sludge, by-product gas, compressed air, caustics, hot and cold water, and acids.

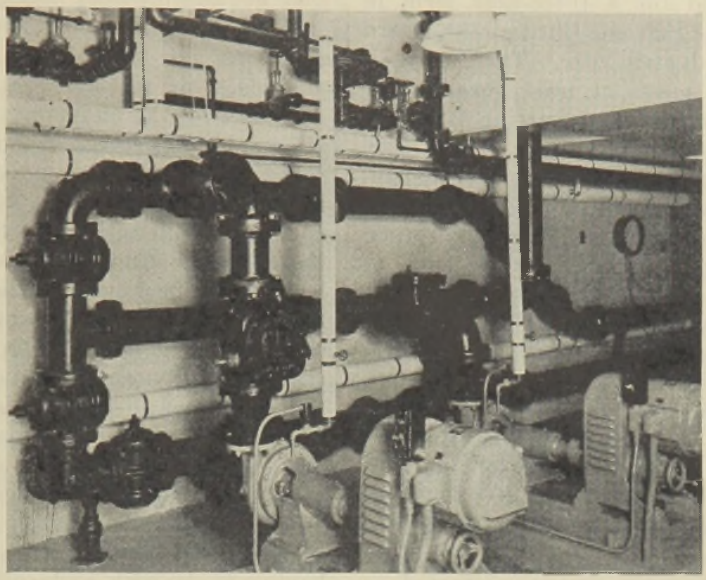


FIGURE 1.—ACF Lubricated Plug Valves in Sludge Pump Room of sewage treatment plant at Peru, Illinois.

THE AMERICAN CONCRETE PIPE ASSOCIATION

228 North LaSalle St., Chicago 1, Ill.

Formed in 1907 at Ames, Iowa, the Interstate Cement Tile Association changed its name to The American Concrete Pipe Association in 1912, at which time national offices were established on West Washington Street, Chicago. In 1916 the association was reorganized as a branch of the Portland Cement Association with special emphasis on investigation, research and the preparation of technical material. By 1945 the industry had grown to such an extent that the association established offices independent of the Portland Cement Association at 228 North LaSalle Street, Chicago, with activites expanded to include all phases of a trade association.

The American Concrete Pipe Association is the national authoritative spokesman for the concrete pipe industry in the United States and to a large extent throughout the world. It is made up of 150 member companies in the United States, with active members in Canada, Mexico, South America, British Isles, Sweden, Holland, France, Switzerland, Egypt, South Africa, India, Australia, New Zealand and the Hawaiian Islands.

The objects of the association include the advancement of the art and science of making concrete pipe; the encouragement of intercourse between men engaged in the manufacture of concrete pipe and the establishment of a central point of reference and union for those engaged and interested in the concrete pipe industry.

The affairs of the American Concrete Pipe Association are managed by a Board of Direction representative of the entire industry, members of which are elected for a three-year term in such order that one-third of the Board is changed each year. The Board of Direction meets at least once a year and part of the Board, acting as an Executive Committee, meets once a year. This Board has entire charge of managing the affairs of the association, including assessment of dues, determination of policies and employment of staff. The Board is elected by a nominating committee which in turn is elected by the President and the term of office of the President is one year.

The headquarters of the industry association are located in the Builders Building in Chicago and include a library of the most authoritative information on concrete pipe assembled anywhere in the world. The activities of the association are concentrated on national advertising, direct promotion and the preparation of authoritative technical literature having to do with the concrete pipe industry. Because of the fact that concrete pipe is used on all types of public improvements the promotional activities are directed to informing the officials and engineers of the federal, state and municipal governments regarding the qualities and performance of concrete pipe. The members of the industry are apprised weekly, by means of a weekly letter from the Managing Director, of the up-to-the-minute news items pertinent to the concrete pipe business. From time to time bulletins having to do with technical phases of manufacturing and laying concrete pipe have been prepared for distribution by the members of the industry. Because of the demand for these bulletins it is very difficult to keep them in print. Nevertheless one bulletin having to do with the irrigation and drainage phase of the industry is available to the members at this time, as well as a 275-page manual "Concrete Pipe Lines," a new reprint of which has just been obtained from the printers. The demand for these bulletins is so widespread that they are continuously being sent to all states of the Union and many foreign countries.

Formed as a loose partnership of members of the industry, the association is in the process of being incorporated as a not-for-profit corporation in the state of Illinois. Its expenses and cost of operation are financed by the members after a schedule of dues has been determined by the Board of Direction yearly. Sufficient funds are retained in the bank to run the association for the succeeding year and all the income of the association is spent in the direct interest of the industry. The national advertising is pro-rated among the leading national technical and engineering journals. Moneys are appropriated for engineering research having to do with hydraulics and the performance of concrete pipe. The direct promotion is carried on by staff members in Washington and, as time and money permits, in every state of the Union and many of the large cities.

During the war a headquarters was maintained in Washington with a paid staff member whose duty it was to inform government engineers of the engineering qualifications of concrete pipe and to keep the industry informed of affairs in the governmental bureaus which would influence in any way the concrete pipe industry.

Annually the association holds a convention which is attended by many of the leaders of the industry. This convention is held each year in a different city of the United States. The program of the meeting contains up-to-the-minute technical presentations, speeches by figures of national prominence in the field of business or general news interest and a dinner and entertainment for members, guests and their wives. The meeting is preceded by a Board of Direction meeting at which policy matters are determined. Following the Board meeting there is an open meeting of the association at which every member company has the right and the duty to approve or disapprove the actions of the Board and to propose new items for approval or discussion by the entire industry. Because the industry is small, it is possible for each member company to take an active and important part in the affairs of its trade association.

Concrete pipe was used by the Romans in the year 80 for building a water supply system for what is the present city of Cologne. It has been used continuously since then with ever increasing adaptability to more and wider fields. The standardization of specifications, improvement in manufacturing equipment and the application of many new methods to the manufacture and installation of concrete pipe have opened up ever-increasing fields, especially during and since the first World War. The industry produces a product which is permanent in every sense of the word. It is so durable that many projects using concrete pipe are being designed with a life expectancy exceeding 100 years. It is made of local materials which are in abundant supply. It is made with local labor under extremely competitive conditions, thus insuring a very economical product. Since its original use in water supply systems, the use of concrete pipe has grown with the growth in population until it occupies a very special sphere being employed universally in culvert construction, all types of drainage work, for irrigation, for city sewers and for pressure water supply systems.

HOWARD F. PECKWORTH,
Managing Director

BUILDERS–PROVIDENCE, INC.

DIVISION OF BUILDERS IRON FOUNDRY

28 Codding St., Providence 1, R. I.

RECENT METERING DEVELOPMENTS FOR SEWAGE WORKS

Propellor Type Meter for Clear Effluent

For totalizing the flow of water and clear effluent, Builders-Providence, Inc., Providence, R. I., offers the Propeloflo Meter (Figure 1) whose unique Venturi design creates uniform velocity distribution where the flow meets the propeller. This greatly increases the accuracy of this type of meter over an unusually wide range of flow. Loss of head is reduced to a minimum by streamlining the propeller bracket, hub and nose, resulting in definite savings in pumping costs.

The Propeloflo Meter is complete in itself and may be installed in any position, from horizontal to vertical, as easily as a corresponding length of pipe. The totalizer, protected by a clear plastic bonnet, reads directly in gallons or cubic feet, and the Propeloflo has ample power to drive secondary instruments or proportioning devices. For intermittent flow it is ideal as it resumes operation without attention when the drained line is refilled. Low first cost, simple maintenance, and long life are added qualities of this improved meter.

Meter for Primary Sludge

To insure continuous operation of Venturi meters handling primary sludge and liquids that are especially difficult to meter, Builders-Providence, Inc., has developed a simple and ingenious flushing arrangement known

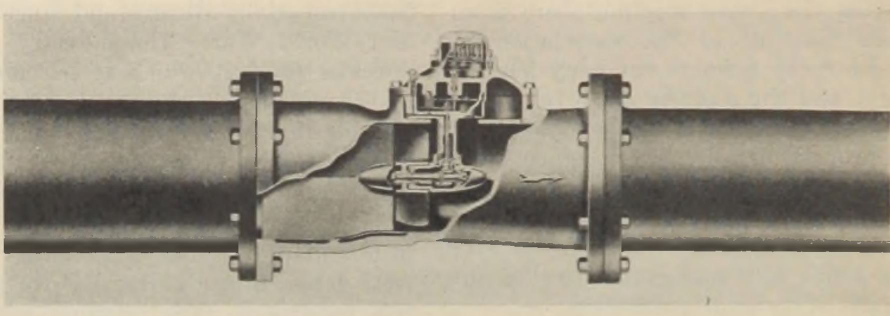


FIGURE 1.—Cutaway view of Propeloflo meter.

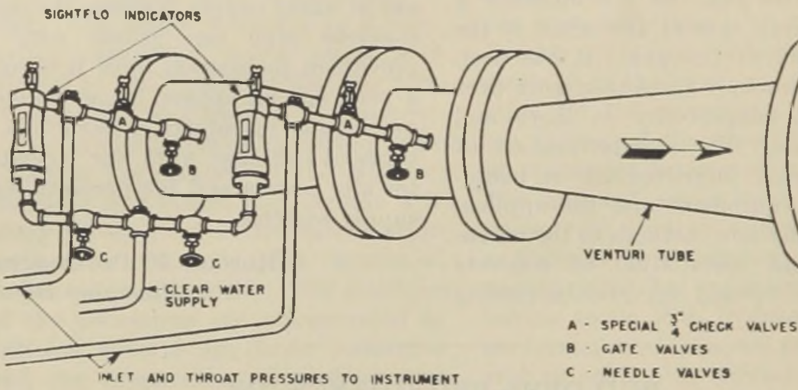


FIGURE 2.—Flush check system for primary sludge meters.

as the Flush-Check System (Figure 2). Since the annular chambers of the standard Venturi tube would require frequent cleaning under such operating conditions, the usual chambers in the tube are omitted and single large vents are substituted at inlet and throat. A small flow of clear water is injected continuously into the Venturi tube through Sightflo Flushing Indicators and special check valves, effectively preventing solids from clogging the vents and small piping to the instrument. This system, requiring a minimum of attention, has solved the problem of measuring primary sludge in numerous sewage treatment plants.

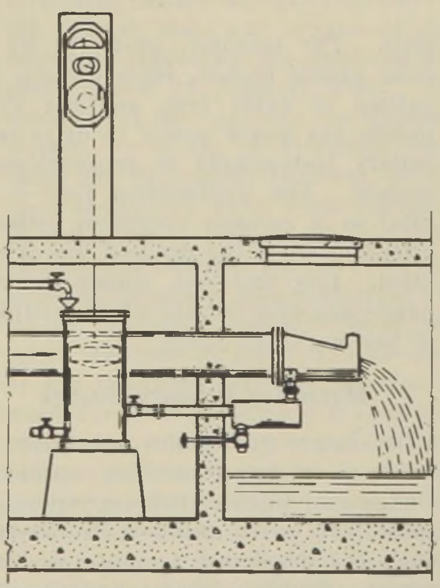


FIGURE 3.—The Kennison nozzle.

Meter for Open Flow

The accurate metering of raw sewage, sludge, and trade waste in partially filled pipes and channels has long been a difficult problem. The Kennison Nozzle, made by Builders-Providence, Inc., meets the metering needs of the small sewage treatment plant under such conditions. It consists of an open pipe contracted to a special shape at the outlet end; low head, wide flow range, suspended solids and debris no longer interfere with accurate metering.

A typical Kennison Nozzle meter installation is shown in Figure 3. The nozzle casting is bolted to the pipe through which the sewage flows with free discharge at the shaped end. Leveling lugs are provided on the nozzle and the top is left open for easy inspection or cleaning. At the bottom, near the inlet flange, a relatively large pressure pipe connects, through a sediment chamber, to the open float well of the recording instrument. This instrument may be Builders Type M or Flo-Watch, and for distance transmission the Chronoflo Telemeter is available. Kennison Nozzles are furnished for pipe sizes of 6 in. to 36 in.

THE CARBORUNDUM COMPANY

Refractories Division

Perth Amboy, N. J.

"ALOXITE" AIR DIFFUSER TUBES AND PLATES

The value of "Aloxite" ceramically bonded aluminum oxide porous diffusers for aeration purposes has been well established over many years through installation in almost all phases of primary and secondary sewage treatment.

Uniform air diffusion with "Aloxite" tubes and plates (Fig. 1) is assured by the product uniformity achieved in exact manufacturing methods. High structural strength is built into "Aloxite" porous products, imparting to them the ability to withstand reasonable installation and handling conditions. Another factor promoting operating economy is the low wet pressure loss for any given permeability of porous diffuser.

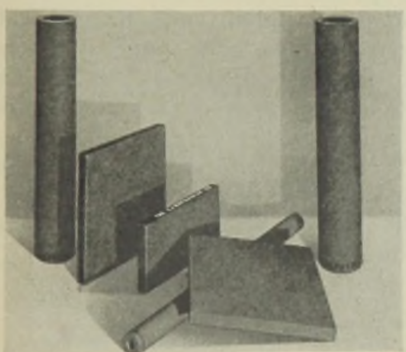


FIGURE 1.—"Aloxite" porous tube and plate diffusers.

Resistance to corrosion or attack by solutions or gases encountered in sewage treatment is inherent in "Aloxite" plates and tubes. They have proved both physically and chemically inert in this kind of service.

Diffuser tubes are available in sizes necessary to fit various types of assembly equipment and to conform to aeration tank requirements. The use of tubes has become increasingly popular because of lower overall cost and simplicity of installation and removal for servicing under many conditions.

Where aeration by the use of diffuser plates is specified, standard size "Aloxite" plates 12 in. square by 1 in. thick are generally used. In addition, special shapes can be made to fit all types of tank designs.

The Carborundum Company does not supply complete assembly equipment or plate and tube holders. Many sewage works equipment manufacturers, however, will gladly quote on and furnish improved types of equipment using "Aloxite" diffusers.

For further information on "Aloxite" porous diffuser tubes and plates, write for Engineering Bulletin No. 1.

CAST IRON PIPE RESEARCH ASSOCIATION

Peoples Gas Bldg., Chicago 3, Ill.

Cast iron pipe is used extensively in the activated sludge process of sewage treatment for influent, effluent and distribution conduits; air mains and headers; sludge drawoff lines and drains—all of which must necessarily be absolutely tight to insure against leaks which result in power losses or nuisances. It is the standard material for these purposes since more than 90 per cent of the pipe used in all types of treatment plants is cast iron pipe.

In recent years pressure sewers have become more common, partly due to the improvement in the manufacturing of pumping equipment and partly because of the realization that pressures may be safely maintained in cast iron sewers. An important feature of a pressure sewer is that it can follow the contour of the ground surface. Thus the initial cost of a sewer is reduced due to the size factor and to the fact that a shallower trench than is usually required for gravity sewers, is possible.

In 1928 a 14-in. cast iron pipe waste sludge line was placed in operation for The Sanitary District of Chicago. The waste sludge from the activated sludge process at the North Side sewage treatment works is pumped at a pressure of 90 p.s.i. through this line some 87,950 ft. for disposal at the West Side sewage treatment works. This line was tested for leakage at a pressure of 175 p.s.i. for 90 min. The allowable rate of leakage was 2,400 gal. per 24 hr. per mile of pipe. The leakage found was 1167 gal. per 24 hr. per mile of pipe. Friction tests were made on the line immediately after it was placed in operation and again after 2½ yr. of operation. The first test indicated a value of C in the Williams and Hazen formula of 141. The second test indicated no signs of clogging or increased roughness.

Cast iron pipe is an economical material for sewers and sewage treatment plants. The test of time has demonstrated the ability of cast iron pipe to resist effectively the corrosive action of sewage liquids and gases.

A permanently tight sewer or sludge line is obtainable with cast iron pipe. No infiltration of water to overload sewage capacities of sewers and sewage treatment works. No leaks of putrescible matter, since the joints of a cast iron sewer are tight and stay tight.

The ability of cast iron pipe to meet service stresses is due to great beam strength, compressive strength and impact resistance. These factors, plus effective resistance to corrosion, keep maintenance costs down to a negligible minimum.

When installations should be as nearly permanent as men and materials can make them, the long life and great structural strength of cast iron pipe assure the lowest cost per service year. If mains are abandoned or relocated, cast iron pipe has a definite salvage value.

Pipe can be furnished with bell and spigot ends, plain ends, flanged ends, threaded ends, with integral stuffing box type mechanical joints and also with ends prepared for special types of couplings.

If specific information concerning special applications of cast iron pipe is desired, we invite you to write us for additional information.

CHAIN BELT CO.

Milwaukee, Wis.

During the past year, the Chain Belt Company has completed the field testing, under full scale operation, of two new items of Rex sanitation equipment. The test data secured has proven their value and demonstrated their soundness of design. These units are the Rex Verti-Flo Clarifier, a highly efficient sedimentation unit, and the Rex Pressurator, a grease removal unit.

The Rex Verti-Flo Clarifier

The Rex Verti-Flo Clarifier (Figure 1) employs a unique exclusive cellular construction obtained by dividing a conventional sedimentation basin with proper weirs and baffles. A unit of excellent hydraulic characteristics is thus provided that has many outstanding advantages. The system of baffling insures that all parts of the settling basin are used, with the result that the actual detention period is very close to the theoretical. This simply means that accurate flow distribution has been obtained and short circuiting eliminated.

A second advantage is the extremely long weir length obtained by dividing the settling tank into cells. The adjustable V-notched weirs allow close regulation of the quantity of effluent withdrawn from each cell and provide for very low uniform upward velocities. Effluents of exceptionally low turbidities are assured.

The Verti-Flo construction will permit the application of a greater load to a smaller tank, and also the handling of a varying rate of flow, without performance suffering to any great extent. Tests of a unit now in full time operation prove that a Verti-Flo Clarifier having a 1.5-hr. detention will perform better than a conventional settling basin having a 6-hr. detention period.

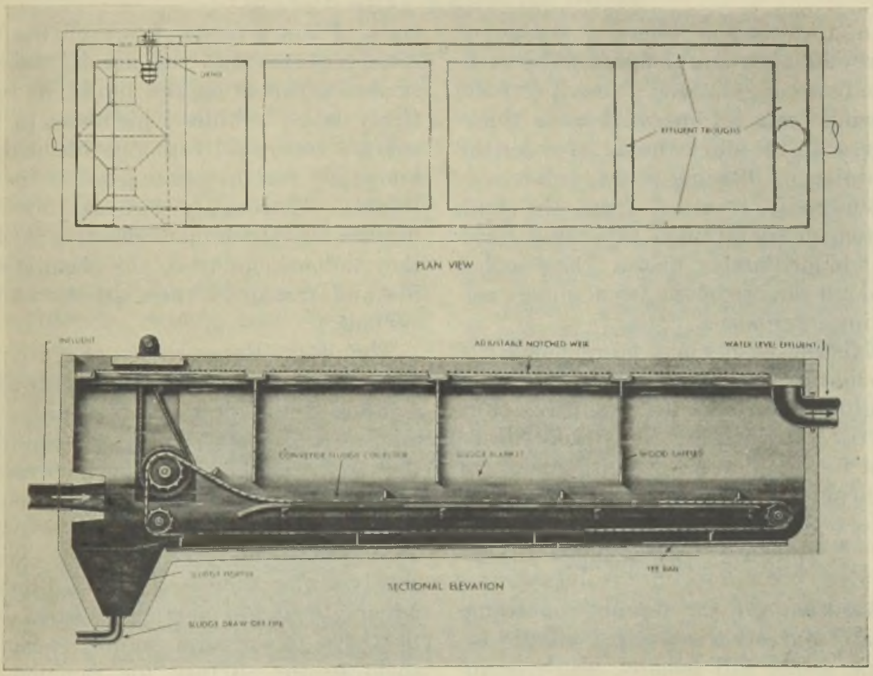


FIGURE 1.—Plan and sectional elevation of Rex Verti-Flo Clarifier.

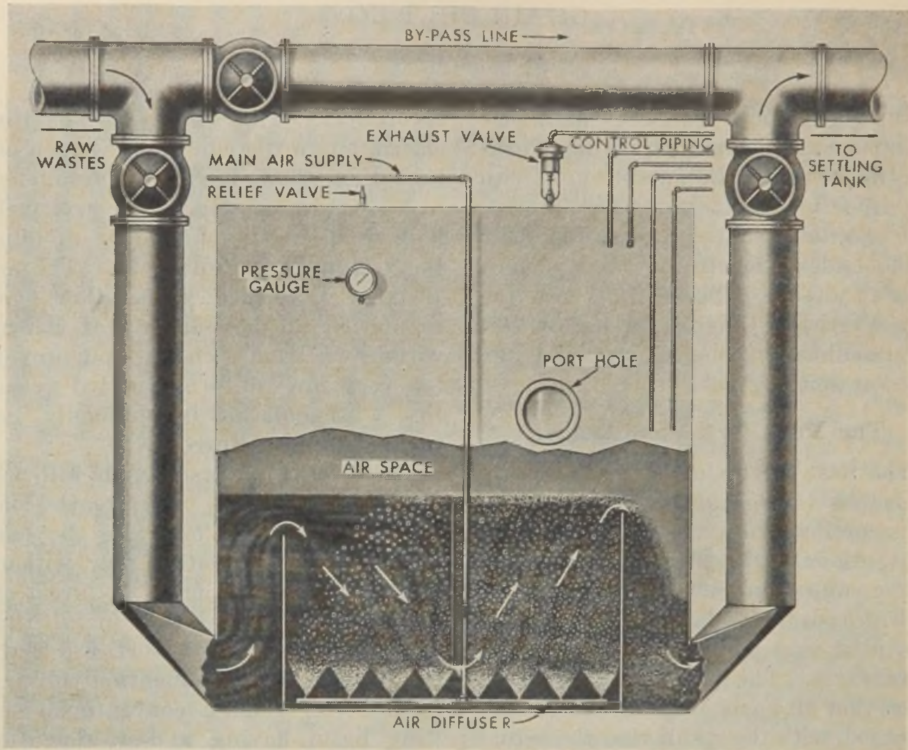


FIGURE 2.—Vertical section of Rex Pressurator.

In applications where a sludge or slurry blanket can be maintained without fear of septicity, the Verti-Flo Clarifier can be operated as a thickener, with the slurry being used for the straining or filtering of fine solids.

Sludge is removed from the tank bottom or maintained as a fluid blanket immediately below the baffles, through the use of the usual sludge collecting equipment.

Existing basins may be remodeled to provide greater capacity or improved results by using the exclusive Rex Verti-Flo principle. New tanks can be kept to a minimum size without loss of efficiency.

The Rex Pressurator

Aeration of a liquid containing grease and other minute floatable organic solids will usually result in increased removal of the floatable materials. Plain aeration, however, has the disadvantage that the air is momentarily entrained in the liquid in relatively large bubbles which rise to the surface too rapidly to provide proper buoyancy for the organic solids in the liquid. Thoroughly diffused fine air bubbles are much more effective in that they adhere closely to the organic solids and transport them slowly to the surface.

The Rex Pressurator (Figure 2) provides a means for supersaturating a liquid under pressure with dissolved and entrained air. When the liquid is then restored to atmospheric pressure in a conventional sedimentation basin, the excess air will come out of solution in a multitude of extremely small slowly-rising bubbles. These bubbles adhere to grease particles and minute particles of organic solids, elevating them to the surface for removal by skimming. The amount of air required averages approximately 0.02 cu. ft. per min. per gal. of waste treated.

The Rex Pressurator is entirely automatic in operation and is provided with complete control and safety features.

Two automatic valves are provided, one of which causes an excess of air to be exhausted to the atmosphere in the event that the volume of waste should decrease, the other of which causes the air supply to be cut off in the event the waste flow should cease entirely. In addition, a relief valve is built into the Pressurator.

A low working pressure of 5 p.s.i. has proved entirely adequate and the tank need provide only about one minute detention for thorough saturation of the liquid.

The unit can be installed readily in front of existing flotation tanks and will greatly improve grease removals with a minimum of initial investment and operating expense. Tests of a full-scale unit handling packinghouse wastes indicate that the revenue obtained from increased grease recovery will pay the equipment cost within two year's time.

Chain Belt Company engineers are available for consultation regarding the application of either of the above described devices to existing problems.

CHICAGO PUMP COMPANY

2314 Wolfram St., Chicago 18, Ill.

"PACKAGE" SEWAGE PLANTS, WIDE BAND AIR DIFFUSION SYSTEM WITH SWING DIFFUSERS, COMMINUTORS FOR SCREENING AND CUTTING COARSE SEWAGE SOLIDS, SCRU-PELLER PRIMARY SLUDGE PUMPS, FLUSH-KLEEN SEWAGE LIFT STATIONS, RAW SEWAGE PUMPS, PLUNGER SLUDGE PUMPS, WATER SEAL PUMPING UNITS

Chicago Pump Company has made the following exclusive contributions of specialized equipment that solve specific problems in the handling and treatment of sewage and industrial wastes: "Package" plants for complete sewage treatment serving small communities, airports, housing projects, and industrial plants; Chicago Wide Band Air Diffusion System with Swing Diffusers for uninterrupted flexible aeration to meet variable operating conditions of sewage works serving populations of 5,000 upward; Comminutors for the elimination of sewage screenings handling and disposal and the prevention of pump and pipe line clogging; Flush-Kleens for the prevention of sewage pump clogging; and Scru-Pellers for primary sludge pumping without clogging trouble.

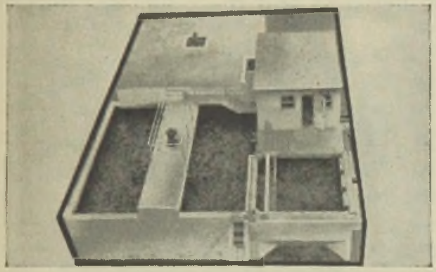


FIGURE 1.—"Package" plant developed by Chicago Pump Co. for complete treatment of sewage of small communities.

Chicago "Package" plants (Figure 1) give effective, low-cost, nuisance-free complete sewage treatment for populations up to 5,000. They are compact, tailored to requirements, simple to operate, odorless, and may be located near dwellings. They were developed twelve years ago specifically to handle the characteristic small community sewage flow and strength conditions, with semi-automatic operation.

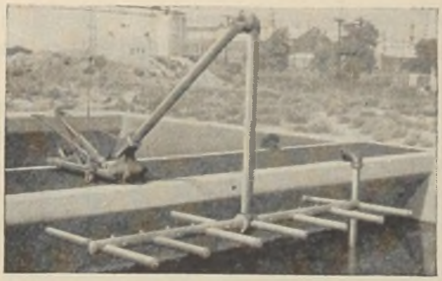


FIGURE 2.—"Chicago" Wide Band Air Diffusion System with Swing Diffusers.

More than 100 Chicago "Package" plants are in successful operation.

The Chicago Wide Band Air Diffusion System with Swing Diffusers (Figure 2) was developed more than ten years ago to increase the overall efficiency of activated sludge plants. Swing Diffusers can be raised to the tank walk for tube cleaning and for simple re-arrangement of diffuser tube spacing to conform with oxygen demand, without dewatering the tank or interrupting operation. The wide band of diffused air provides greater oxygen absorption, eliminates center

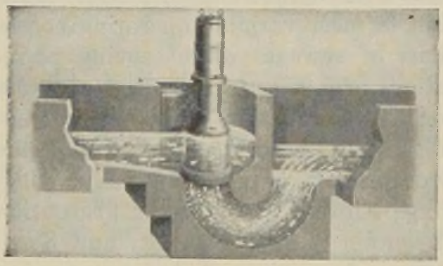


FIGURE 3.—Comminutors eliminate the nuisance of screenings handling and disposal.

coring, requires less air for optimum circulation and effects power economy through the reduced hydrostatic head on the elevated diffuser tubes. More than 100 plants are equipped with Chicago Wide Band Swing Diffusers. Many more are installed in industrial waste treatment plants and in chemical processing plants.

Comminutors (Figure 3) were developed twelve years ago to eliminate the nuisance of sewage screenings handling and disposal. Installed in the raw sewage channel, they provide subsurface automatic screening and cutting of sewage solids without removal from the channel. Comminuted particles settle readily in the primary tank and are pumped to the digester. Installed in lift stations ahead of raw sewage pumps they prevent pump clog-

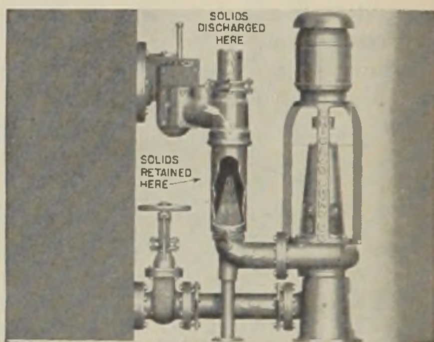


FIGURE 4.—Flush-Kleen Clog-Proof Sewage Pump for lift stations.

ging by cutting sewage solids before they reach the pumps. More than 1,200 Comminutors have been installed.

Flush-Kleen Clog-Proof Sewage Pumps (Figure 4) were developed in 1924. More than 3,000 units have since been installed in lift stations, buildings and ships. The Flush-Kleen system is clog-proof because a screen in the inlet keeps sewage solids from entering the pump. The screen is automatically backwashed and the solids are flushed

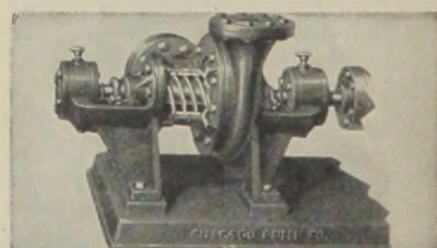


FIGURE 5.—Scru-Peller Primary Sludge Pump.

out the discharge pipe. Sewage enters the wet basin through the pump when it is idle. Operation is automatically controlled by float.

Scru-Pellers (Figure 5) were developed fourteen years ago especially for pumping primary sludge. Since that time more than 1,000 have been installed. The pump has a screw-feed and continuous multiple shearing actions all the way through the pump, which cut solids that clog ordinary pumps. There are ball bearings on both sides of the impeller that keep the pump in perfect balance and prevent vibration and strain on the shaft under heavy loads. The pump is built sturdy enough for severe shocks.

CLAY PRODUCTS ASSOCIATION

111 West Washington St., Chicago 2, Ill.

ClaPipe Jointing Compound for Use with Clay Pipe Sewers

ClaPipe Jointing Compound is one of the most revolutionary forward steps in hot-pour jointing compounds in the past 25 years. It was developed by the Armour Research Foundation for the Clay Products Association, expressly for use with vitrified clay pipe.

Specifications were for a jointing compound that would form a leakproof bond with the surface of clay pipe, a joint which would not dry out, shrink, crack, or pull away from the pipe, and which was both alkali and acid resistant.

Hundreds of laboratory and field tests using vitrified clay pipe prove that ClaPipe Jointing Compound meets all the above requirements and many others.

ClaPipe is the ideal compound for making root-proof joints with clay pipe. It forms a strong and lasting bond with the glazed surface of the pipe. It really holds tight. Neither time nor sewage content affects its grip.

But, in addition to this exceptional bond, ClaPipe retains its plasticity. It never completely hardens. When set it feels hard and looks hard. You can lift several sections of jointed pipe into the trench without breaking or bending ClaPipe joints. Should there be a slight settlement or shift in the pipe line, however, a ClaPipe joint will slowly give—yet remain watertight! *And any sewer line that remains watertight also remains completely root-proof.*

Impartial tests of ClaPipe Jointing Compound made by the Pittsburgh Testing Laboratory show the following performance:

Adhesion Tests

Glazed vitrified clay briquettes having 1-in. square surfaces were jointed together with ClaPipe Jointing Com-

FIGURE 1.—Racks of this kind make it simple to pour several ClaPipe joints at a time in advance of installation. Pipe is kept in true alignment while pouring.

pound and tested in tension at the rate of 600 lb. per min. The average strength was 214 lb. per sq. in.

Hydrostatic Tests

Pressure tests were conducted on 6-in. glazed vitrified clay sewer pipe. Pipe were jointed by packing with jute and pouring the ClaPipe Jointing Compound in vertical position (Figure 1). The pipe was plugged in such a manner that the point under test was unrestricted. Results:

(a) Pipe *unscored either end:*

Pressure accepted—36 lb. per sq. in.

Failure—at joint in jointing compound.

(b) Pipe *scored both ends:*

Pressure accepted—60 lb. per sq. in.

Failure—at joint and weep holes in barrel simultaneously.

Resistance to Acid and Alkali

Specimens of ClaPipe Jointing Compound were submerged in 0.1N HCL, 0.1N HNO_3, 0.1N H_2SO_4 and 0.01N NH_4OH for 35 days.

There was no loss in weight and no disintegration of the specimens.

Plastic Flow

Two-in. cubes were cast of ClaPipe Jointing Compound and placed under compression at room temperature with the following results:

(*a*) Maximum load accepted, at loading rate of 0.10 in. per min. was 1,540 p.s.i. The compound was found to deform without fracturing under load due to plastic flow. The jointing compound plastic flow was such that supporting strength was gradually released to equilibrium in 2 min., 20 sec.

(b) Comprehensive literature on the use and performance of ClaPipe Jointing Compound will be gladly furnished including the complete Pittsburgh Testing Laboratory report.

THE DORR COMPANY

570 Lexington Avenue, New York 22, N. Y.

New Type MA Digester

Both mixing and gas storage are combined in a single tank in a new type of digester now offered by The Dorr Company. Known as the Type MA, it is designed for smaller plants where sizes from 20 ft. to 40 ft. diam. are required. The mixer is mounted in the center of the gas holder crown and rises and falls with the movement of the holder.

The mixing capacity is much greater than formerly used and is sufficient to turn over the entire contents of the tank in about 30 minutes. In operation, only an hour or so will be necessary for daily mixing of raw and digesting sludge as well as submergence of any scum. Supernatant can be drawn just prior to mixer operation.

The gas storage provided is sufficient to hold about 30 to 40 per cent of the average daily output, which should iron out any hourly fluctuations. Supernatant draw-off is provided with an adjustable sleeve valve operated by handwheel, thus eliminating the troublesome regulation by overflow rings. Provision is made for drawing down the water level in the digester as much as 5 ft.

All safety devices have been furnished including pressure-vacuum relief devices, with flame trap and explosion proof motor.

Dorr Clarifiers

An important feature provided on all Dorr primary clarifiers is the automatic scum skimmer. The entire surface of the circular tank is swept by the skimming arm and the accumulated scum is trapped in a recess formed by the arm at the periphery of the tank. As the mechanism slowly revolves, the floating scum trapped in the recess passes along a submerged inclined shelf. The sloping shelf rises until it is out of the sewage, where it terminates in a scum trough. The trapped material is deposited in this trough and passes through an opening in the tank wall into a scum sump. Neoprene wipers effectively prevent escape of the scum once trapped and the whole device assures complete automatic removal of scum from the tank surface. Manual removal or operation is unnecessary.

Repair and replacement costs on Type S Dorr Clarifiers have been extremely low, according to a survey recently conducted among users who have operated them for ten years or more. Fifty-one users of a total of 95 units said they had spent a total of $2934 for repairs, or $30.90 per unit. This represents 72/100 of one per cent of the original purchase price of the equipment. Assuming these installations to average ten years old, the *annual* cost of repairs and replacements is 0.072 per cent of the original cost.

GALE OIL SEPARATOR COMPANY, INC.

52 Vanderbilt Ave., New York 17, N. Y.

WASTE ACID, OIL AND GREASE POLLUTION PREVENTION SYSTEMS

During the war, little attention was paid to the question of the pollution of the rivers of this country, as all thoughts and efforts were given to the development of instruments of war so that it would come to an end in the shortest period of time. Some of the new plants which were constructed, however, did make provision in their plans for oil and grease removal systems. A few made provision for waste acid treatment systems.

The engineers of this company installed in the Buick Company plant at Flint, Mich. two separator systems which removed the oil and grease from the plant wastes and prevented it from getting into the Flint River. Other systems were installed in the Pratt and Whitney Aircraft Co. plant at East Hartford, Conn., and others in the Revere Copper and Brass plant in Chicago. In the latter plant, the systems were so designed that the cooling water coming from the rolls was cleansed of its oil, grease and sediment, and this water passed through heat exchangers where the temperature was reduced and the water used over and over again on the rolls. This made it possible to eliminate construction of a large sewer main at that plant and effected a saving not only in water costs but also in plant investment.

In a government war plant at Halethorpe, Md., operated by the Revere Copper and Brass Co., Inc., the engineers of this company designed and installed an acid neutralization system. This industry manufactured magnesium tubes by a process involving use of such acids as sulfuric, nitric and muriatic. The Gale Acid Neutralization System received the contents of all of these acid tanks as they were dumped, and neutralized them before disposal to the sewer. The system was operated continuously to the close of the war.

The Gale Acid Neutralization System has since been improved so that it is now possible, where large volumes of acid wastes are handled, to effect some recovery from the residue. Thus, part of the operating expense is returned in usable by-products. The system is more economical than is neutralization by slaked lime, since lime treatment produces a large volume of sludge, the disposal of which is difficult and costly.

A neutralization system was also installed in a plastics manufacturing plant for the treatment of highly acid wash waters. The neutralized water is heated in boilers and recirculated for continuous use. This plant has thus saved many thousands of dollars in water costs and, most important, has been able to avoid discharge of any polluting wastes to the nearby fishing stream which would otherwise have had to take such wastes.

Among the other new developments being studied by the chemists and engineers of the Gale Oil Separator Co. are methods of reclaiming oils that are removed from waste waters.

GRUENDLER CRUSHER AND PULVERIZER COMPANY

2915–21 N. Market St., St. Louis, Mo.

Sanitary engineers of municipalities have found that Gruendler equipment eliminates the nuisance of undesirable material and waste products clogging sewage and sludge valves and piping. New patented features make possible uninterrupted flow notwithstanding frequent quantities of prevailing rag stock. The Gruendler Shredder is of the swing hammer design with double-end cutting hammers revolving inside a cylindrical shredder plate at 1,200

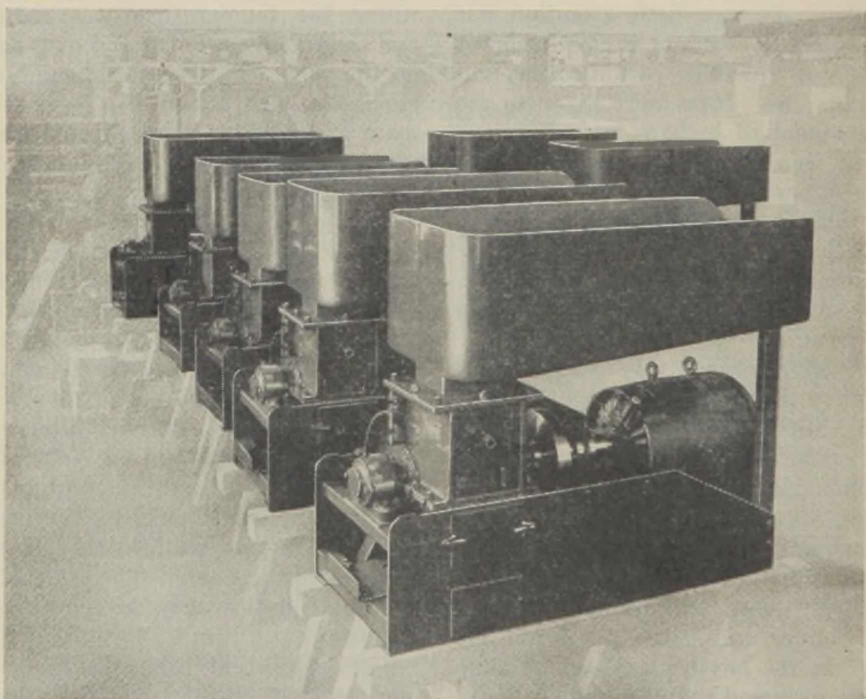


FIGURE 1.—Lineup of Greundler Garbage Shredder units ready for crates and shipment to the U. S. Navy for installation on battleships and merchant marine; hundreds of these shredders were employed in the war effort.

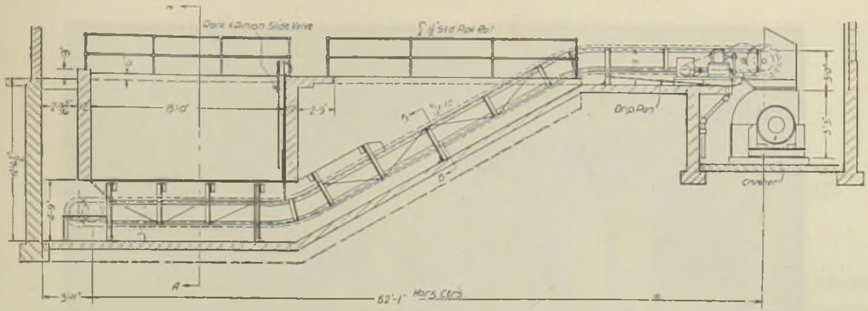


Fig. 371-61—Gruendler Garbage Shredder with Hopper and Apron Conveyor, used in connection with Sewage Disposal Plant.

FIGURE 2.—Diagram showing Gruendler Garbage Shredder with hopper and apron conveyor, used in connection with sewage treatment plant.

to 1,800 r.p.m. Capacities are 500 to 8,000 lb. per hour. Motors are direct connected, and range from 5 to 70 h.p.

The Gruendler Shredder is outstanding because of its patented features such as Non Clogging Grate Bars, Safety Tramp Iron Catchers and Improved Water Spray Flushers.

The Gruendler Sewage Screenings Shredder also makes an ideal connection with an incinerator—by grinding, mixing, and preparation in one operation, it prepares product for most efficient combustion and proper feeding.

Gruendler Shredders are dependable and of sturdy construction with sealed bearings to withstand continuous operation. The safety features of the Gruendler Tramp Iron Catcher is noteworthy, since foreign matter such as cans or metal pieces, which is ungrindable, is easily removed through the access door provided. The shredders are built in many sizes and can be fitted in most difficult plant arrangements, taking a minimum of space, and the many designs meet sewage works requirements for higher efficiency. The grinding of sludge to uniform sizes for commercial fertilizer is another operation performed by Gruendler's Hammer-Type Cake Breakers.

Hundreds of Gruendler Shredders have been selected by the U. S. Navy as standard equipment on battleships and merchant vessels.

Blueprints and sewage works layouts, together with specifications, will be gladly furnished municipal engineers, plant superintendents or plant designers by the Gruendler Crusher and Pulverizer Company. Address Plant and General Offices in St. Louis 6, Mo.

HARDINGE COMPANY, INC.

York, Pennsylvania

The sanitation equipment manufactured by Hardinge Company, Incorporated, of York, Pennsylvania, includes Circular and Rectangular Tank Clarifiers, Digesters, and Automatic Backwash Rapid Sand Filters.

Hardinge Clarifiers

Hardinge Clarifiers are used for sludge collection, sludge concentration, and skimming in plants treating domestic and industrial sewage, and in municipal water treatment plants. The Hardinge Clarifier consists primarily of a mechanically operated slow-moving scraper in the bottom of the tank, which scraper moves settled solids to a discharge hopper, concentrating them during the operation.

When used in circular tanks, the col-

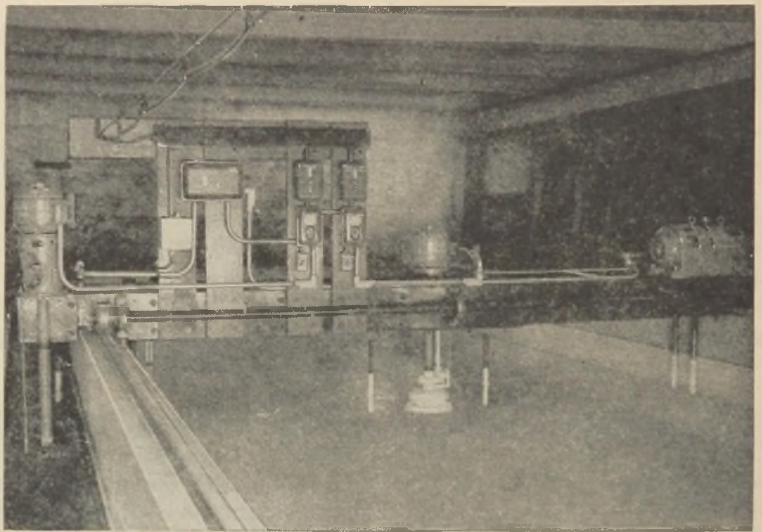


FIGURE 1.—The Hardinge Automatic Backwash Rapid Sand Filter.

lecting scrapers of the Hardinge Clarifiers are of the rotating type, while on rectangular tanks, Hardinge employs a reciprocating design.

Hardinge Digesters

The Hardinge Digester mechanism, which is used in sewage sludge digestion tanks, consists of sludge scrapers, rotating scum-breaker arms, and a gas dome in the tank top for collection and removal of liberated gas.

Hardinge Automatic Backwash Rapid Sand Filter

The newest product is the Hardinge Automatic Backwash Rapid Sand Filter (Figure 1), which is used for the treatment of sewage, industrial wastes, and water supplies. The filter consists of a compartmented sand bed with underdrains, together with a backwash and cleaning mechanism traveling on the sidewalls of the tank.

Tests recently completed at the Newton Falls Paper Mill, Newton Falls, N. Y., show that the Hardinge Automatic Backwash Sand Filter which was put into operation on January 1, 1946 is removing 97.4 per cent of the suspended solids from the raw reservoir water which is being filtered without the use of any chemicals. The ABW Filter has been in continuous operation since the installation was completed. It has not been necessary to add any sand to the filter bed or to change the sand.

The filter automatically backwashes the sand bed as the head builds up. The backwash mechanism on the Newton Falls installation operates four times a day, cleaning a portion of the bed in each operation. The filter media consists of a sand bed 10½ in. deep supported on 1¼-in. Aloxite porous plates.

HERSEY MANUFACTURING CO.

DRYING MACHINERY DIVISION

"E" and Second Sts., South Boston 27, Mass.

HERSEY DIRECT HEAT DRYERS FOR SEWAGE SLUDGE AND INDUSTRIAL WASTE BY-PRODUCT RECOVERY

Heat dried activated sludge is generally accepted as a safe fertilizer of good value as a plant food. Undigested primary sewage sludge is also well-endowed with plant nutritional elements and may be rendered hygienically safe for fertilizer utilization by heat drying. In some cases it has proved profitable to heat-dry digested sewage sludge for sale as fertilizer.

Hersey Direct Fired Rotary Dryers (Figures 1 and 2) afford an efficient and economical method of drying sewage sludge to a low moisture content while destroying any bacteria and other forms of life that might create a health hazard in the use of the material for fertilizer. Hersey Dryers may also be used, to increase overall efficiency, for partial removal of moisture as a preliminary to incineration.

Many industrial wastes responsible for stream pollution problems contain by-products of good market value. Recovery of such by-products is often accomplished by removal of the solids from the wastes and by heat-drying them to a low moisture content. Hersey Dryers afford a practical means of obtaining saleable by-products from

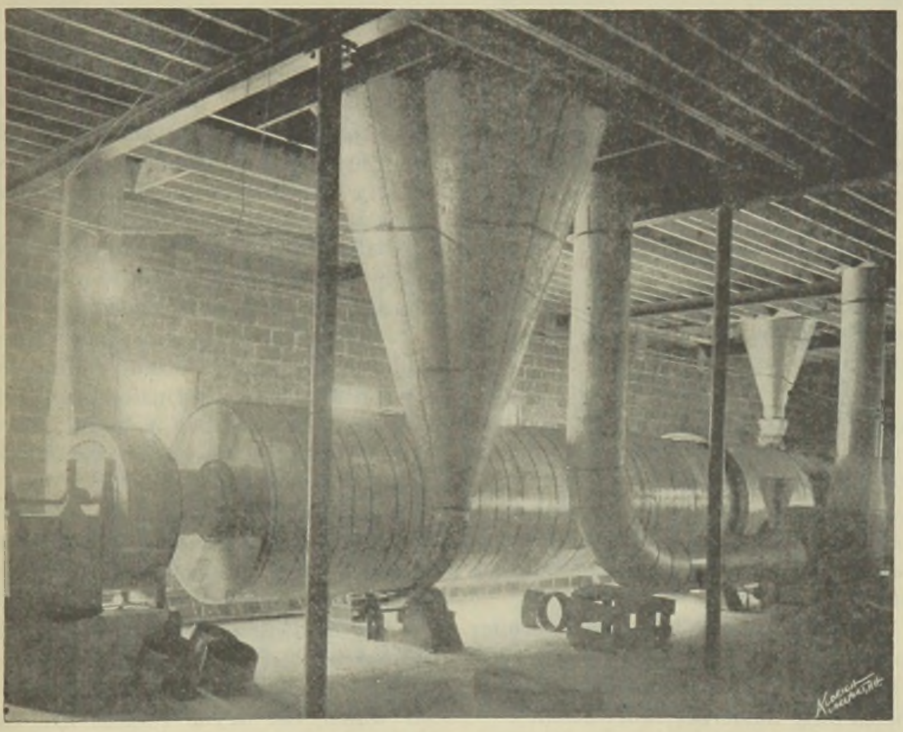


FIGURE 1.—Hersey Direct Fired, Double Intermediate Feed Drying System, applicable to drying of sewage sludge and industrial wastes.

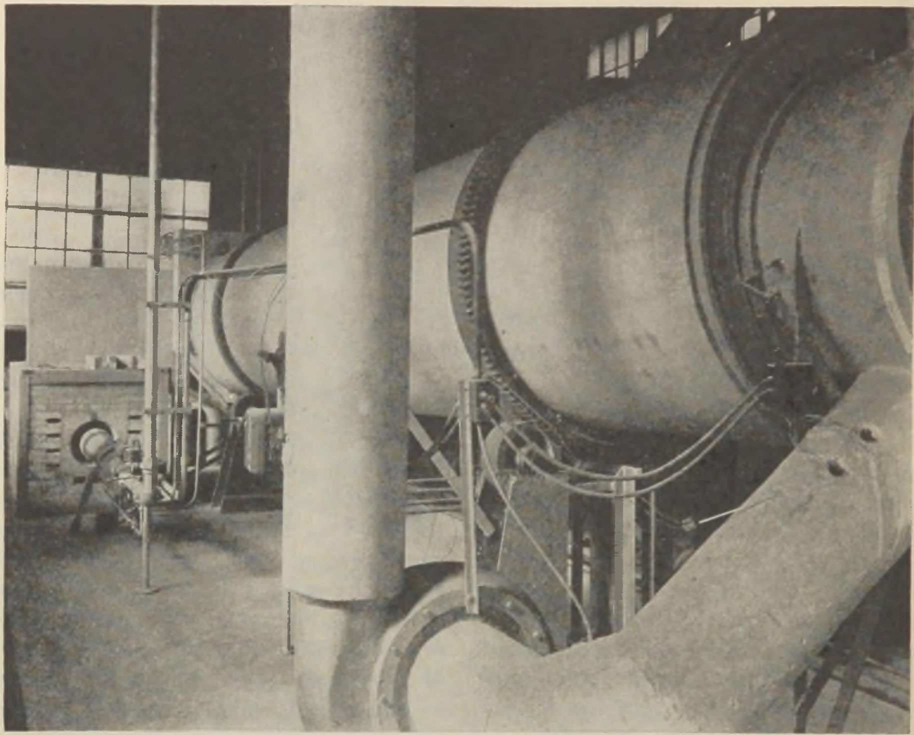


FIGURE 2.—The Hersey Direct Fired Dryer in a straight counter-current system. This unit is gas fired.

cannery wastes, distillery wastes, fish scrap, fruit pomace and similar industrial residues.

All of the following advantages may be secured in Hersey Dryers:

1. Full thermostatic control equipment for automatic adjustment of feed rate variations.

2. Hersey saw-tooth buckets, which expose a maximum surface of the material to the air or gases, giving high evaporating efficiency.

3. Improved seals, either metal-to-metal or nonmetal-to-metal, prevent leakage between the stationary and rotating members.

4. Equipment throughout with roller and ball bearings gives long life with a minimum power requirement and involves only moderate lubrication.

5. Correct and permanent alignment of the shell driving gears is insured by mounting the ring gear on one of the running rings so that it is not adversely affected by shell temperature variations, etc.

6. Conveyor plates at the feed end carry the material away promptly, eliminating spillage at the curb.

7. The conical lower thrust wheel and running ring give true rolling and long life. Ample adjustment is provided to compensate for wear.

8. A variable speed feeder can be furnished, if desired.

9. Arc welded construction is used wherever practicable to give increased strength and improved appearance. The shell is butt welded and the buckets are continuous fillet welded. The supporting strands are of welded structural steel and the truck wheels on which the dryer rotates are cast alloy steel. Fabricated steel running rings are optional.

10. Hersey Dryers may be constructed from practically any material to suit the needs of the product to be dried; mild or stainless steel, monel, copper, nickel clad steel and wood are some of the materials available.

Hersey Dryers are the result of competent engineering backed by the accumulated knowledge of over 60 years of widely varied drying experience. Booklet and "Requirements Sheet" gladly furnished on request.

INFILCO INC.

325 West 25th Place, Chicago 16, Ill.

ACCELO–FILTERS, ACCELO BIOX PROCESS, ACTIVATORS, CLARIFIERS, SKIMMERS, ROTARY DISTRIBUTORS, AUTOMATIC DOSING SIPHONS, AUTOMATIC PROPORTIONERS, ACCELATORS, CHEMICAL FEEDERS, MIXERS, GRIDUCTORS, COAGULATORS, VENTURI TUBES AND CONTROLLERS

New Developments

Since the war's end, research and development personnel have been able to concentrate on new and improved equipment for water, sewage and trade waste treatment. Keynoting this program is development of a completely new unit for trade waste treatment. This unit will also ease the problems at municipal plants which handle troublesome industrial wastes along with domestic sewage. Progress also continues in the development of a new grit removal mechanism.

In existing equipment, improvements include a modified PD Clarifier Drive, changes in the Hydraulic Skimmer and the Griductor.

PD Clarifier Drive

Friction drive, incorporated in the newly modified PD Clarifier, has been substituted for the older rack-and-pinion type of assembly. The new drive mechanism is impelled by a vertical motor reducer, which is driven by a weather-proof ball bearing motor. Keyed with the output shaft of the motor reducer, there is a drive wheel with a tire of synthetic rubber vulcanized to the wheel rim. This tire is acid-resistant and of proper durometer hardness. Tire contact with the tank sidewall is below the liquor surface.

Hydraulic Skimmer

A change has been made in the Hydraulic Skimmer that eliminates a centrifugal pump, formerly connected with the suction pipe. A quick-opening automatic scum valve has been substituted.

As used with both PD and WS Clarifiers, this automatic scum valve is operated by a lever from a cam fixed to the outer end of the skimmer arm. This design assures quick opening and closing of a rubber-covered ball-type valve. The design also includes provision for a non-operating position, so that if desired, the skimmer arm cam may pass without opening the scum discharge valve.

This new scum removal design continues to include all of the advantages of the former method. Elusive scum is not removed by mechanical means. With the Hydraulic Skimmer, scum is removed positively and efficiently by surface currents and skimmer plates that direct material to the removal pipe.

Griductor

Teeth of the Griductor have been changed in design, and the rotating discs, in which the teeth are inserted, have been extended beyond the stationary grid. These modifications in design result in double screening action.

The Griductor comprises a semi-circular stationary screen grid, with slots of sufficient width to permit mounting a series of rotatable circular screen cutter plates or discs in such a manner that the assemblies form screen slots both above and below each cutter plate. The discs extend outward beyond the face of the grid and form, in effect, a coarse screen. As these screen discs with the cutter teeth are rotated, the intercepted screenings are dislodged from the screen element and moved to an adjacent stationary cutting bar or comb. Reduction of the size of the solids is accomplished by the engagement of the cutting comb with the teeth fixed to the edge faces of the rotating screen cutter plate (Figure 1), thus causing a shearing action. The teeth are staggered on the cutter plate in such a manner that only one tooth at a time can transmit a cutting load.

Set screws hold each tooth firmly in a deep slot in the discs. This assures positive holding action, and as the teeth are set in deep recessions, the cutting load stress is transmitted to a substantial area of the plate. The tooth and the holders are also designed in such a way that radial adjustment is possible. Thus proper cutting clearance can be easily maintained.

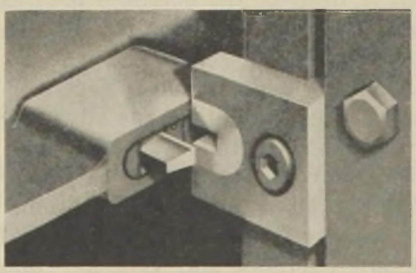


FIGURE 1.—Close-up view of precision-made Griductor tooth, mounted in rotating disc.

The two cutting edges of the teeth and the edges of the stationary combs are made of the most durable material available for service of this nature. Both the teeth and the cutter combs may be removed easily, which facilitates occasional sharpening. Complete extra sets of cutting bars are available with each Griductor installation.

The rotating assembly of screen cutter plates is driven by a vertical motorized reducer through a flexible coupling. The motor is suitable for outdoor, all-weather service. Two anti-friction type bearings, one placed at both the upper and lower ends of the removable shaft housing, take both vertical thrust and horizontal alignment loads. The rotating screen cutter plates are mounted on and keyed to the vertical shaft so that the entire assembly may be removed by disconnecting the flexible coupling and removing four capscrews.

In the standard assembly of the Griductor, the unit is mounted on a plate frame, set in the concrete of the screen channel and the vertical motorized reducer is mounted on the stationary grid. However, when necessary, the Griductor is furnished with an extended shaft so that the motorized reducer may be mounted on a floor or other structural support any reasonable distance above the channel bottom. It is also possible to position the motorized reducer above the Griductor as desired by inserting a flanged spacer piece between the top of the grid frame and the base of the reducer.

Sewage solids too large to pass through the slots between the stationary screen grid and the rotatable screen cutter plates are intercepted and intermittently subjected to the shearing and cutting action of the cutter teeth and combs, until they are reduced to pieces sufficiently small in size to pass through the screen slots.

Griductors are available for either

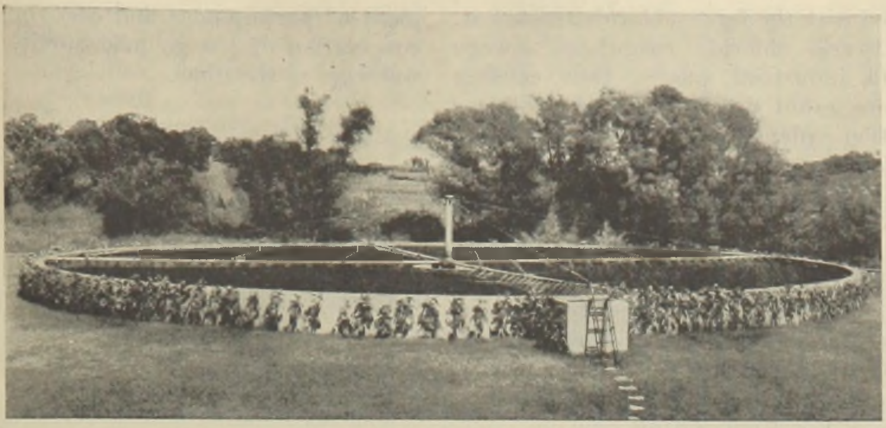


FIGURE 2.—Accelo-Filter installation which includes the effective principle of Direct Recirculation.

right-hand or left-hand operation. This permits the use of duplicate units in individual channels for standby and peak load services. Dual units may be installed also in a single channel where required by high sewage flow rates or by the screening and cutting demand of an abnormal sewage or industrial waste.

The Griductor motor control may be either manual or automatic. If automatic, intermittent service is preferred, the starter may be operated either by an adjustable timing device or by an actuating mechanism responsible to the actual screening and cutting demand as measured by a head differential. Irrespective of the type of control used, the Griductor motor will be started and stopped independently of the rate of sewage flow.

Direct Recirculation and Accelerated Biological Oxidation

Benefits of immediate and direct return of aerobic organisms to an aerobic process have been demonstrated by the steadily increasing number of successful Accelo-Filter installations (Figure 2). Many of these plants, too, have been built at a saving in cost, through the advantage of being able to use smaller sedimentation tanks.

Concrete Dome-Supported Digesters

There has also been steady growth in the numbers of Infilco Digester installations. These are available for reinforced concrete dome covers, supported either in the vortex circulator type or the sludge scraper-scum breaker type, for either single stage or two stage operation.

INNIS, SPEIDEN AND CO.

117 Liberty St., New York, N. Y.

Ferric Chloride as a Coagulant for Sewage and Industrial Wastes

The chemical treatment of sewage and industrial wastes is an effective and economical method of increasing the efficiency of primary treatment plants. Such treatment requires but little plant construction or capital investment. It provides a high overload capacity and flexibility in proportioning to produce the degree of purification required. Seasonal loads are handled with the ferric chloride treatment.

Ferric chloride coagulates sewage and industrial wastes, thus causing more rapid precipitation in sedimentation plants and faster filtering through vacuum filtration systems. It is most effective in the pretreatment of industrial wastes which offer change in characteristics almost hourly.

Experience has shown that ferric chloride is also an economical coagulant for sewage and a wide range of industrial wastes.

For Coagulating Sewage

Ferric chloride provides an inexpensive and efficient method of dealing with plant overloads and reduces original plant investment by increasing the capacity of the sedimentation plant.

Amount of Ferric Chloride Required

The accepted procedure in determining the dose of Ferri-Clor required to coagulate a given sewage is to set up "jar tests" with the sewage to be tested. Two-qt. mason jars and stock solutions of ferric chloride and lime, consisting of 10 grams per liter of distilled water, are used. Place one liter of sewage to be tested in each of several mason jars. Add to the sewage in each jar successively larger doses of ferric chloride stock solution. Stir all samples thoroughly for several minutes and let stand for 1 hour. Observation of the several samples will determine the best dosage. If lime is required to provide sufficient alkalinity for satisfactory coagulation, follow the same procedure outlined for ferric chloride. The amount in pounds of ferric chloride or lime required equals the number of ml. used in the selected test times 10 times total sewage flow (m.g.d.) times 8.33.

For Sludge Conditioning

Ferric chloride is accepted as the agent most efficient from the standpoint of performance and cost for the coagulation of sludge prior to its dewatering in the filter.

Amount of Ferric Chloride Required

Using the Büchner funnel test, the time periods required for the vacuum to break with varying quantities of coagulant are recorded. Two to three minutes is satisfactory and 1 to 1½ minutes is very good and should be used for plants where the sludge is held, as in batch mixing.

Various combinations of ferric chloride and lime should be tested until the relative quantities for optimum operation are determined.

The approximate quantities of chemical to be used in the plant can be determined from the laboratory tests, provided, of course, that the same strength chemicals are used in the plant. The quantity of ferric chloride solution in gallons per 1,000 gallons of sludge in the test and the quantity of lime in pounds per 1,000 gallons of sludge will be 42 times the grams used per 200 ml. of sludge.

Preparation of Ferric Chloride Solution

The main difficulty in dissolving ferric chloride arises from the fact that the saturated solution is heavier than water. This means that the liquid in a simple container may have a layer of pure water at the top, even with undissolved crystals at the bottom. To overcome this difficulty, three methods have been developed:

1. Automatic gravity circulation by placing the salt on a wooden grid, elevated or suspended one-half way in the tank.
2. Mechanical mixing by (a) stirring, (b) by air agitation, and (c) liquid recirculation.
3. Melting: Ferric chloride melts at 98.6 to 102.2° F. and so can read-

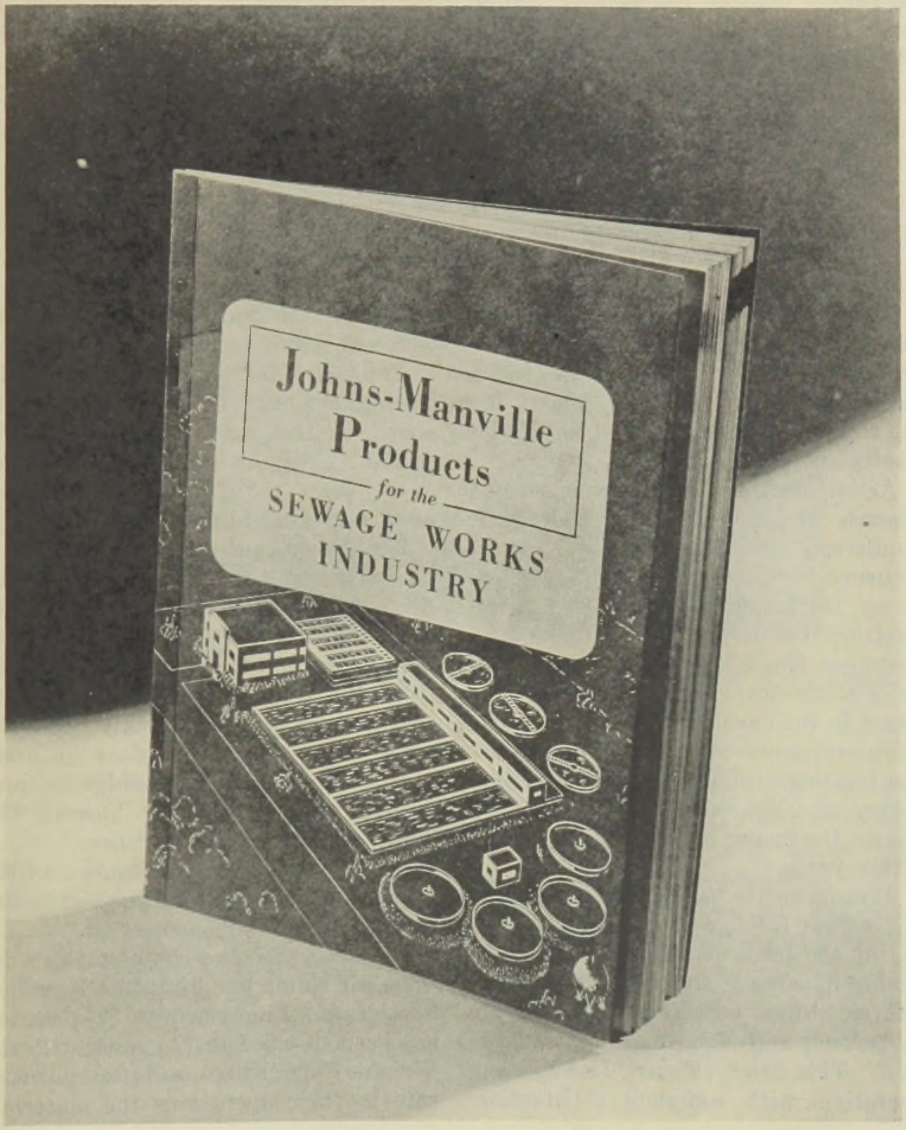
ily be liquefied by relatively small amounts of heat from exhaust or live steam, flue gases, oil, gas or electricity. The dissolved salts can be poured into the water.

FERRI-CL.OR is Innis, Speiden and Company's registered name for ferric chloride developed and manufactured especially for sewage and water treatment use.

JOHNS-MANVILLE

22 East 40th St., New York 16, N. Y.

Johns-Manville has recently published a 78-page brochure entitled "Products for the Sewage Works Industry." It contains over 100 photographs of actual installations of the many Johns-Manville products in sewer



systems and sewage treatment plants throughout the country.

Typical of the applications included are: insulation of digesters, boilers, steam and hot water lines, sludge incinerators and furnaces; packings for sewage and sludge pumps, etc.; gasketing for chlorinator piping; Transite ventilators on digester control vaults; Transite Sewer Pipe for gravity lines; Transite Pressure Pipe for force mains; Transite Industrial Vent Pipe for venting gases, vapors, etc.; Asbestos-Ebony control panels and instrument boards in treatment plants; Chemstone for gates in settling tanks and for table tops in laboratories; Transite Sheets for building construction; acoustical materials for noise control in laboratories, offices and pump rooms; roofing and shingles for administration buildings, digesters, pumping stations, etc.; waterproofing materials for sludge tanks; refractories for sludge incinerators and air preheaters; and many other products for use in the construction, operation and maintenance of sewage works.

A copy of this brochure may be obtained by writing Johns-Manville, 22 E. 40th Street, New York 16, New York.

KOMLINE–SANDERSON ENGINEERING CORPORATION

Box 444, Ridgewood, New Jersey

SEWAGE SLUDGE DRIERS AND FURNACES

The Komline-Sanderson Engineering Corporation has been formed for the purpose of developing and marketing machinery and equipment for the treatment of sewage and industrial wastes.

Active partners in the concern are Thomas R. Komline and Walter H. Sanderson. Both men have recently returned from service in the U. S. Army, the former having served in the Sanitary Corps and the latter in the Airborne Engineers. For eight years prior to the war, Mr. Komline was engaged in the development of the sludge drier and other equipment at the sewage treatment plant at Plainfield, New Jersey, and Mr. Sanderson has been active in the heavy construction field for fifteen years.

Arrangements have been made with the Instant Drying Corporation for the use of the basic patent on the spray-drying of sewage sludge, and this has been combined with the work of John R. Downes and Mr. Komline at Plainfield. This drier (Figure 1) has been operating with complete satisfaction since 1938 on a digested sludge of 15 per cent solids content.

Thickened sludge is sprayed into the drying chamber by the special high-speed centrifugal atomizer and subjected to the drying effect of the hot gases from the furnace. The dried material settles to the floor, from which it is removed continuously by the rake, while the spent gases are vented by the fan. The action of the atomizer in breaking the solid sludge into droplets permits rapid evaporation of moisture from the solid particles. This rapid evaporation in turn quickly cools the gases so that the sludge does not overheat. When burning sludge, a pulverizing fan is used for blowing the dried sludge into the furnace.

The apparatus will be known in the sanitary field as the Komline-Sanderson Sludge Drier and Furnace.

In order to make complete installations for solids handling, the Komline-Sanderson Engineering Corporation has been licensed to (1) work with the Wright Cord Filter and (2) incorporate in their operations the materials

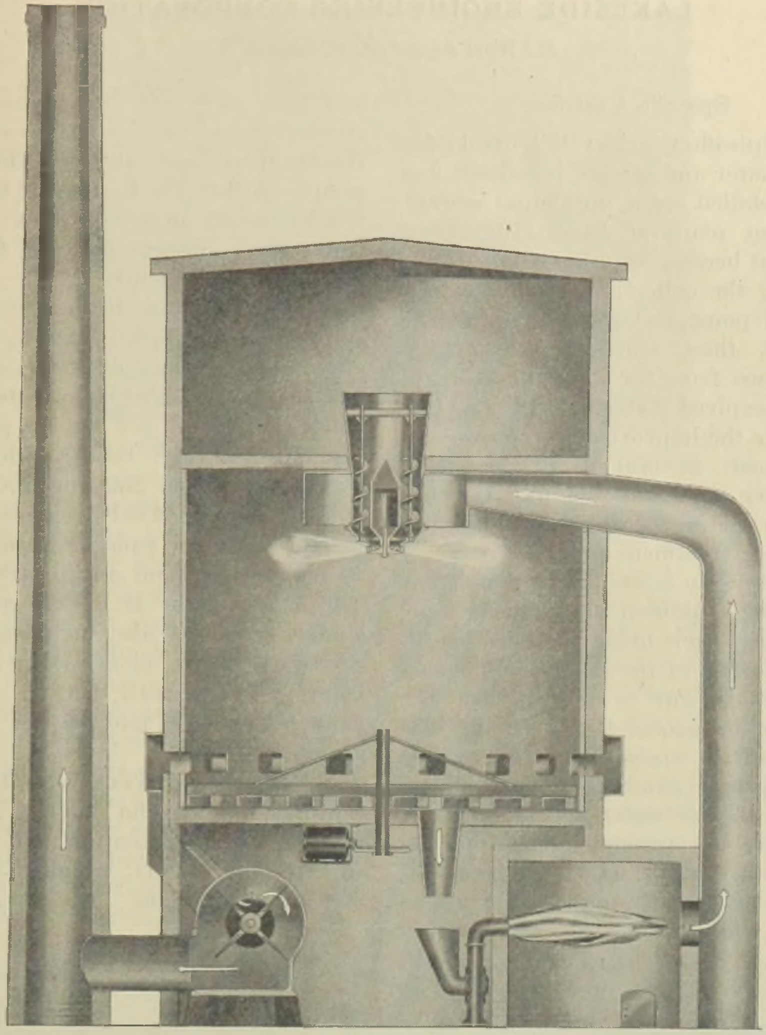


FIGURE 1.—Sectional view of the Komline-Sanderson Sludge Drier and Furnace.

handling equipment manufactured by the Gifford-Wood Company, Hudson, New York.

The Cord Filter installed at Perth Amboy, N. J., has demonstrated its ability to filter sewage sludge with no blinding, clogging, or scale formation, with high, sustained solids output. A Wright Filter is now being installed at the Plainfield plant, with the purpose of obtaining factual data on the performance of the drier on filter cake.

The Gifford-Wood Company was established in 1814, and builds a complete line of equipment for the conveying and handling of grit, sludge, ash, chemicals and other materials.

The Komline-Sanderson Engineering Corporation offers complete equipment for the treatment of municipal and industrial wastes. Inquiries are invited from interested consultants and sanitarians.

LAKESIDE ENGINEERING CORPORATION

222 West Adams St., Chicago 6, Ill.

Spiraflo Clarifier

The Spiraflo Clarifier (Figure 1) for use in water and sewage treatment was first installed in a municipal sewage treatment plant at Flora, Ill., about 1935, but because of an existing patent covering the collecting of sludge to a common point and pumping it to the digester, these clarifiers had to be withdrawn from the market until said patent expired (about 1942). After that date the improved Spiraflo was reintroduced. Several installations have since been made and this unit is being incorporated in the design of a number of sewage treatment plants.

The Spiraflo is unique in that it has both a long influent and effluent weir, the influent weir being the total length of the bottom of the skirt which is suspended adjacent to the wall of the round or hexagonal tank. In the case of clarifiers equipped with scraper mechanisms (non-hopper bottomed), tanks with side water depths of about 9 ft. are recommended in order to provide a deep skirt. A 2-hour detention has been found to be ample and this may later be reduced to a shorter period in view of the excellent results produced at the Flora, Ill. plant. A 9-ft s.w.d. and a 2-hour detention result in an overflow rate, at the average design flow, of 800 gal. per sq. ft. in 24 hours.

At Flora, Ill., with an average flow slightly in excess of 0.6 m.g.d., the primary Spiraflo has a detention of about 1.65 hours and an overflow rate a little above 1,000 gal. per sq. ft. per day. The raw sewage B.O.D. (domestic) averages between 200 and 250 p.p.m. and the average B.O.D. removal by the primary clarifier runs between 55 and 60 per cent without any pretreatment. The Flora plant is of the activated sludge type and also includes a final Spiraflo clarifier identical to the primary. The overall B.O.D. removal runs between 96 and 98 per cent as an average.

At Brownwood, Texas a 56-ft. diameter primary Spiraflo handles 2 m.g.d. with a 2-hour detention and with a raw sewage B.O.D. ranging between 300 and 350 p.p.m. The average re-

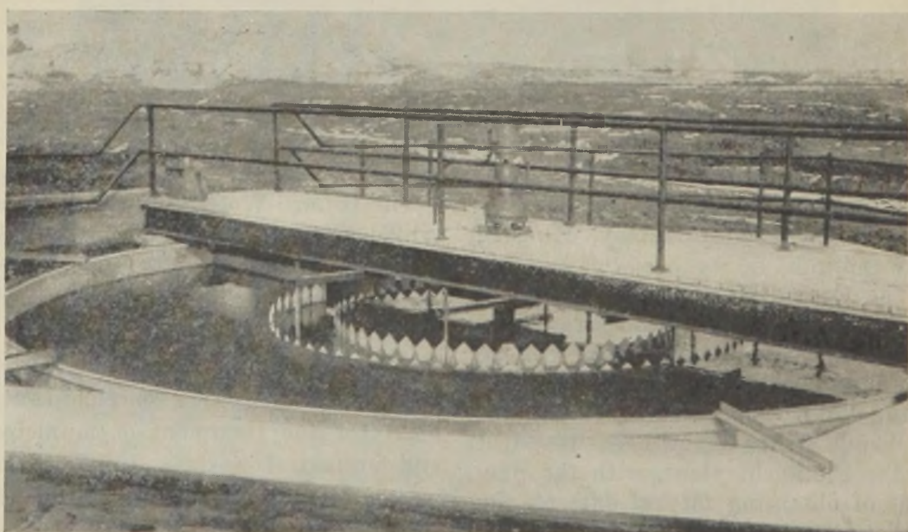


FIGURE 1.—The Lakeside Spiraflo Clarifier.

FIGURE 2.—The Lakeside Trickling Filter Distributor.

moval by the clarifier runs about 65 per cent, with an average suspended solids removal of between 75 and 80 per cent. This clarifier does not have any pretreatment ahead of it.

The Spiraflo affords excellent grease, oil and scum removal; it gives actual detentions which run much nearer the theoretical than is usually the case with conventional types. If, in the case of stale sewages, it is desired to preaerate the sewage before settling, this can be accomplished in the race of the Spiraflo without requiring the construction of separate preaeration units.

The use of the Spiraflo means a smaller load to be handled by secondary treatment; hence, the better the plant effluent, with a lower recirculation requirement for trickling filters and a lower air requirement in the case of activated sludge.

Information regarding these units will be gladly supplied upon request.

Filter Distributor

The Lakeside trickling filter distributor (Figure 2) is now simplified to the use of straight arms and may be obtained for high capacity filters or for conventional filters. Both types are equipped with centrifugal nozzles of an improved design. The nozzles have covers equipped with lugs so that they can be easily removed and replaced without in any way disturbing the orifice. The cover has an angle of discharge which prevents matches and solids up to 7/8 in. in diameter from lodging in the nozzle. The connection between the nozzle and the distributor arm has a means whereby the throat of the nozzle can be cleaned without removing the nozzle.

The roller bearing in the top of the column, where it is away from possible sewage contamination, has been increased in size. The several roller guide bearings near the base of the column are large and adjustable. A person sitting on an arm of the distributor does not prevent its rotation.

A specially designed mercury catch basin is provided at the base of the column so that even though the mercury blows from the seal, it is positively caught in the compartment and can easily be recovered. The mercury well is deep and is designed for operating heads of 7 or 8 ft.

Sewage Samplers

The Trebler Sampler is now available in three types—one with a 3/8-in. scoop width for sampling creamery or other wastes which do not carry large solids, while the other two types have a scoop width of 3/4 in. for handling sewage containing relatively large solids. One of the latter is for use with a 90 degree V-notched weir and the other is for straight weirs. The scoops are interchangeable on the machine.

These machines are low in cost, easy to move from one location to another, but they are all designed to be used in connection with weirs. Later they will likely be available for use with Parshall flumes. The machines are time clock controlled so that the sampling period can be set at will.

LAMOTTE CHEMICAL PRODUCTS COMPANY

Towson 4, Md.

LaMotte Chlorine Control Units (New Series)

The LaMotte Research Department has completed a thorough survey of the improved application of chlorine in the form of free chlorine, hypochlorites and chloramines as sterilizing agents in the treatment of water, sewage and a host of industrial uses. As a result of these studies a complete new line of LaMotte Chlorine Control Units has been made available, paralleling the recent advances in the more effective chlorination procedures. This new equipment embodies the latest developments in the use of the O-Tolidine method for the determination of active chlorine.

The factors contributing to the accepted standardization of the new method, such as color development, pH and composition of the o-tolidine reagent have been incorporated in the LaMotte procedure along with a new series of interchangeable 15 mm. chlorine color standards embracing the entire useful range. These achievements simplify the actual tests, as well as the apparatus, since the standards are not only of uniform dimensions for the complete range but are interchangeable in all LaMotte Chlorine Comparators as well as in the standard LaMotte pH comparators.

The LaMotte Chlorine Block Comparator (Standard Model) (Fig. 1) consists of 10 chlorine color standards (15 mm.), values 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9 and 1.0 p.p.m. chlorine*; 1 ampoule of distilled water; three 10 ml. test tubes; one 0.5 ml. pipette with nipple and 50 ml. of o-tolidine reagent. This equipment is con-

* The above selection of standards is taken from the following list of standards which are available for inclusion in the Standard Chlorine Comparator in selected sets of 10 color standards. Values 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.5, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0 and 10.0 p.p.m.

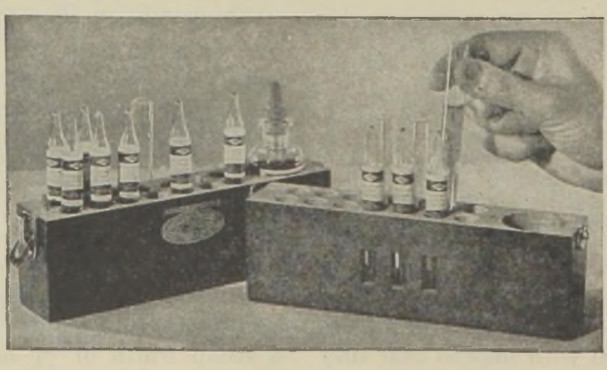


FIGURE 1.—The LaMotte Chlorine Block Comparator (Standard Model).

tained in a polished wooden case, the top of which serves as a comparator for color readings. The arrangement of tubes in the comparator eliminates any errors due to color or turbidity of the sample, such as may be encountered in raw water, sewage and industrial wastes.

The LaMotte Roulette Chlorine Comparator (Fig. 2) was designed as a permanent installation in plants or control laboratories. It covers the whole useful chlorine range from 0.1 p.p.m. to 10.0 p.p.m. Standards are always in place in a revolving drum and readings may be made under any light conditions since the unit is equipped with a special internal dalite reading arrangement. Accuracy and speed in making chlorine readings are assured with the use of this unit. It is sold complete with color standards, ortho tolidine reagent and all necessary calibrated glassware. The accessory box containing the reagent, test tubes and pipette may also be used for readings not obtainable near the original unit.

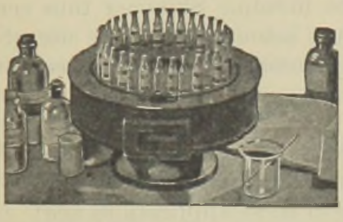


FIGURE 2.—The LaMotte Roulette Chlorine Comparator—A Wide-Range Unit.

LINK-BELT COMPANY

Sanitary Engineering Division

Chicago—San Francisco—Philadelphia

Straightline Scum Breakers

Link-Belt Straightline Scum Breakers for circular or rectangular sludge digestion tanks have been in successful operation for the past 16 years. One of the most recent installations has been made at the sewage treatment plant of the Radford, Virginia, Ordnance Plant of the Hercules Powder Company.

This scum breaker (Figure 1) has been installed in a digestion tank of improved design so that the equipment and tank have the following distinctive

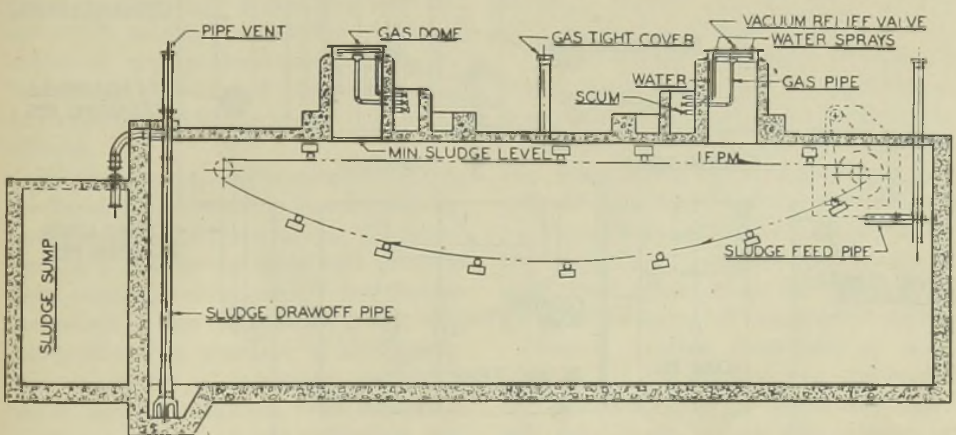


FIGURE 1.—Link-Belt Scum Breaker at the Radford, Virginia, Ordnance sewage treatment plant.

features:

(a) Positive prevention of scum mats.
(b) Rugged construction resulting in long life.
(c) A supernatant liquor with low solid content.
(d) A low digested sludge withdrawal rate which results in no gas loss and the elimination of the possibility of explosive mixtures due to carelessness.
(e) Low first cost of construction and low maintenance costs on mechanical equipment.

The Straightline Scum Breaker is operated at the slow speed of 1 f.p.m. Scum is broken up or conveyed to one of the tanks where it is submerged by the action of the paddles. Supernatant liquor or digested sludge is discharged from the tank by pumping raw sludge into the digester. The digested sludge flows into a sludge sump where it is stored until sufficient quantity is accumulated to fill one sludge drying bed, thus preventing the loss of sludge gas. Safety devices such as an emergency overflow, pressure relief and vacuum relief valves are provided.

Rotoline Skimmer

The Link-Belt Rotoline Skimmer automatically removes the scum from the surface of settling tanks and separates the scum from the excess water in a scum trap. The scum may be pumped to the digester and the excess water is returned by gravity to the effluent channel of the settling tank.

The Rotoline Skimmer thus removes a dense scum without the use of electrical motors or mechanical equipment.

Air Diffuser Units

Many applications have been found for the Link-Belt Air Diffuser Unit. Some of these applications are:

(a) Activated sludge aeration tanks.
(b) Pre-aeration tanks.
(c) Grease aeration and flotation tanks.
(d) Chemical mixing tanks.
(e) Carbon dioxide addition to excess lime-softened water.
(f) Hayes Process contact tanks.

These units are very effective, economical and simple in operation because they provide the desired aeration or mixing at low pressure and are easily removed for cleaning.

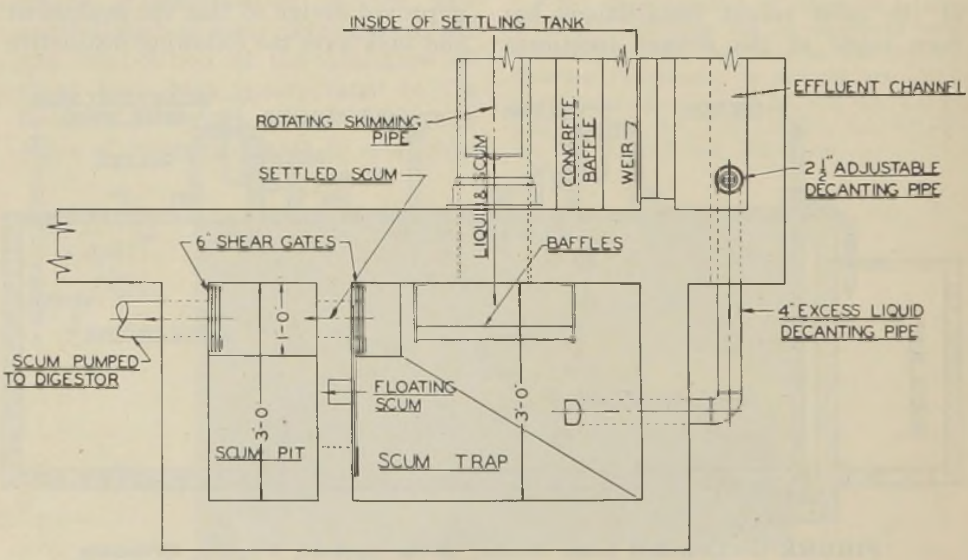


FIGURE 2.—Link-Belt Rotoline Skimmer.

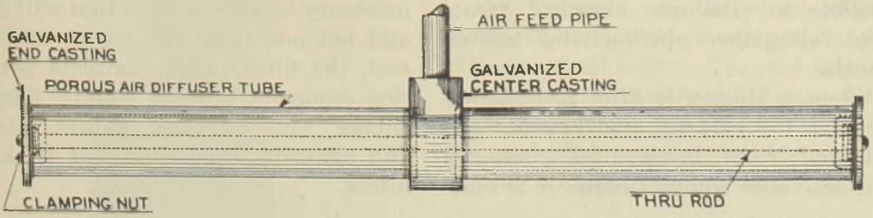


FIGURE 3.—Link-Belt Air Diffuser Unit.

The units consist of two end and one center casting which supports two 3-in. inside diameter by 24-in. long porous tubes as shown in Figure 3.

Link-Belt Spray Nozzles

The highly efficient Link-Belt spray nozzles are finding wide use wherever water sprays are required. Two of the uses in sewage treatment plants are the wetting down of scum in the gas vents and the removal of the surface scum from the settling compartments of Imhoff tanks, as shown in Figure 4.

These spray nozzles are also used to clean Link-Belt Revolving Drum Screens, Water Intake Screens and Vibrating Screens.

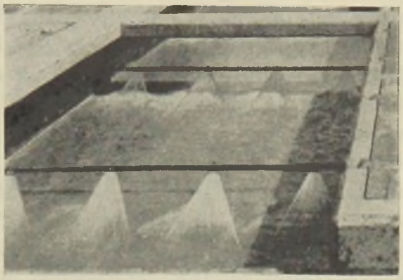


FIGURE 4.—Link-Belt Spray Nozzles removing scum from an Imhoff tank.

MAGNETITE FILTER CORPORATION

10 East 40th St., New York 16, N. Y.

The Magnetite Filter Corporation offers a completely automatic sand filter which makes possible the modernization of sewage treatment plants by the simple addition of this single unit to strain filterable solids from the plant effluent.

To the sewage works overloaded by increased flows or made inadequate by new sanitation requirements, the addition of a Magnetite filter will provide both capacity enlargement and better treatment. The Magnetite filter will out-perform an increase of sedimentation capacity in removing settleable solids and, in addition, will remove a good percentage of non-settleable dispersed solids.

In general, filtration is the most economical method of removing solids normally passing through the final settling tank of a treatment plant.

Many primary sewage treatment plants, burdened with new problems, may approach chemical treatment performance without the use of chemicals. The installation of a Magnetite filter will account for removals of 40 to 50 per cent of the suspended solids reaching the filter, thus usually giving a total plant efficiency on the order of 80 per cent removal of suspended solids.

Where chemical treatment or chlorination of the effluent are employed, the installation of a Magnetite filter will very much reduce the amount of chemicals or chlorination required. In fact, in some localities it has been possible to eliminate chemical treatment altogether during the winter months.

When a Magnetite filter is installed to treat the effluent, two benefits are obtained: *First,* in chemical treatment and activated sludge plants, it is only necessary to produce floc that will filter and not one that will settle; and *second,* the filter offers complete protection against "bulking" and "floating sludge" in activated sludge plants, and against "unloading" of trickling filters.

MINE SAFETY APPLIANCES COMPANY

Pittsburgh 8, Pa.

For the sewage works field Mine Safety Appliances Company provides a variety of equipment especially adapted to the particular safety requirements of the men employed in sewage service operations. MSA is the oldest and largest manufacturer of safety equipment in the world. Its products have become the standard for worker protection in every industry. Here is some of the company's outstanding protective equipment for sewage works safety.

Chemox Oxygen Breathing Apparatus

The MSA Chemox Oxygen Breathing Apparatus (Figure 1) provides complete respiratory protection where insufficient oxygen or contamination by toxic gases renders air unbreathable. The Chemox is self-generating and completely self-contained. The wearer simply breathes into the easily-replaceable Chemox Chemical Canister and pure oxygen is immediately evolved in a quantity to meet any breathing requirement.

Before the canister is exhausted, a pre-set alarm bell rings as a warning to withdraw to fresh air. The MSA Chemox is a product of years of research and its effective performance has been proved under the most exacting battle conditions in the Navy throughout the war. Each Chemox Chemical Canister supplies pure oxygen to the wearer for one hour. Simply inserting a fresh canister prepares the Chemox for instant use.

FIGURE 1.—MSA Chemox Oxygen Breathing Apparatus.

All-Service Gas Mask

When sufficient oxygen is available to sustain life but is contaminated by toxic gases usually encountered around sewage treatment plants, such as chlorine, hydrogen sulfide, sulfur dioxide, ammonia—singly or in combination, including carbon monoxide—the MSA All-Service Gas Mask (Figure 2) provides dependable respiratory protection, approved by the U. S. Bureau of Mines. The chemical canister of the All-Service Mask can be easily and

FIGURE 2.—MSA All-Service Gas Mask.

quickly replaced when required. The lightweight harness permits complete freedom of movement.

Demand Mask

Complete respiratory protection for sewage works personnel who must enter unbreathable atmospheres is also provided by the MSA Demand Mask (Figure 3). The Demand Mask fur-

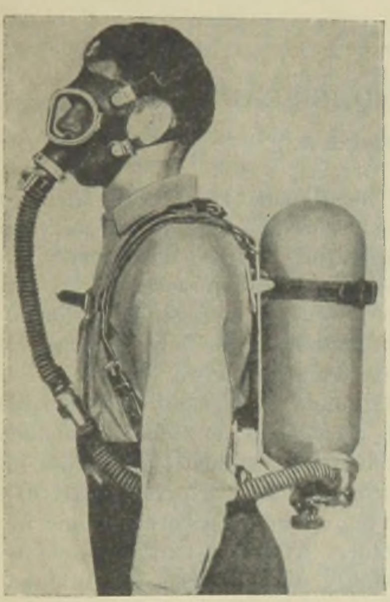


FIGURE 3.—MSA Demand Mask.

nishes oxygen or certified pure air in a refillable cylinder for a 30-min. period of hard exertion. Flow of the breathing gas is automatically regulated by the wearer's breathing, and a pressure gauge mounted on the chest fitting shows the amount of oxygen in the cylinder at all times. The cool inhaled gas keeps the mask's All-Vision face-piece completely fog-free. The MSA Demand Mask is quickly and easily put on and is comfortable to wear.

Combination Hose Mask

The MSA Combination Hose Mask features a safety belt whereby a man may be safely removed from a deep pit, tank or sewer manhole. This mask furnishes the wearer fresh air from outside the danger zone by means of a blower, thus giving protection from all gases, smokes and fumes and oxygen deficient atmospheres.

Rubberized Protection Suit

Especially adapted to the requirements of sewage works operations is the MSA Rubberized Protection Suit. The gas-proof, moisture-proof suit is made of heavy rubberized double-texture black material which is hand doubled, stitched, cemented, strapped, and vulcanized at all seams. Boots and gloves are permanently attached, and the suit is adaptable for wearing with either an All-Service Gas Mask or Oxygen Breathing Apparatus.

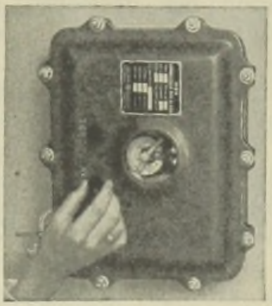


FIGURE 4.—MSA Combustible Gas Alarm.

Combustible Gas Alarm

For 24-hour safety in modern sewage works, Mine Safety Appliances Company provides the MSA Combustible Gas Alarm (Figure 4), which continuously samples confined atmospheres to determine concentration of combustible gases in the air. The Combustible Gas Alarm combines extreme sensitivity with precise accuracy in operation. When the concentration of combustible gases in the air reaches a predetermined limit the alarm gives instant audible and visible warning.

Explosimeter

Quick, accurate testing for gas explosion hazards on the job is provided by the MSA Explosimeter (Figure 5). Compact, light in weight, the Explosimeter is a portable combustible gas indicator which can easily be operated by any workman. It is sturdily constructed and simply designed to withstand hard everyday usage in sewage plant service.

FIGURE 5.—MSA Explosimeter.

First Aid Materials

Complete, up-to-date first aid equipment is a vital requirement for all sewage works operations, inside and out. Mine Safety Appliances Company provides a variety of first aid kits for every emergency—containing unit-packaged sterile bandages of all sizes and types and individual, hermetically-sealed ampoules of stimulants and antiseptics for fast, economical administration. MSA's Type D filler system provides groups of first aid fills in quantities of 10 to 36 units. The units fit like blocks into the kits so that they are instantly accessible and easily replaced and re-ordered.

NICHOLS ENGINEERING AND RESEARCH CORP.

60 Wall Tower, New York 5, N. Y.

The Nichols Duohearth Incinerator

Nichols Engineering and Research Corporation has developed and installed during the current year a combination unit for the disposal of sewage sludge along with municipal garbage and rubbish. The equipment is known as the Nichols Duohearth Incinerator. There has long been a need for a single unit in which all classes of municipal wastes could be disposed of in a simple and economic manner. Previous attempts to dispose of wet sludge or screenings in a conventional, manually stoked incinerator have all been unsuccessful. Any operator who has attempted this will bear witness to the impracticability of incinerating wet sludge along with garbage and rubbish in the conventional type incinerator.

The Nichols Duohearth Incinerator consists of a mechanically stoked garbage and refuse incinerator, over which is superimposed a drying hearth for predrying the sludge prior to its combustion in the lower refuse incinerator compartment.

Garbage and combustible rubbish are burned in a Nichols Monohearth mechanically stoked type refuse incinerator, which is a cylindrical furnace having a slowly revolving cone in the center. Attached to the cone are adjustably pitched arms which stoke the refuse while it is being dried and burned. In the annular ring surrounding the cone, and under the area swept by the arms, a solid cast iron drying hearth is used. Surrounding this in the annual area out to the periphery of the furnace walls are a series of dumping grates through which supplementary air is blown for combustion of the rubbish.

The cone and arms are perforated and are cooled by means of air which then enters the mass of refuse piled over the cone in the center. In this manner efficient drying and intimate contact of air with the burning material is assured.

Garbage and rubbish are charged through the large hollow rotating shaft at the top center of the incinerator.

Superimposed over the Monohearth refuse incinerator compartment are one or more conical firebrick hearths on which the sewage sludge is dehydrated by utilizing some of the waste hot gases from the Monohearth refuse incinerator compartment. The amount of hot gases from the refuse incinerator which is drawn across the upper sludge drying hearth is controlled by a refractory damper, and the balance of the refuse incinerator gases pass directly to the combustion chamber and stack.

The hollow shaft in the sludge drier compartment is rotated by a separate driving mechanism. Rabble arms with teeth or plows are attached to the shaft and the teeth plow and stir the sludge during its passage across the upper drying hearth. In this way intimate contact of the wet sludge with the hot gases from the refuse incinerator is assured. The dried sludge falls through a center discharge opening in the drying hearth so that it lands upon the top of the pile of burning refuse where final combustion of the sludge takes place.

Gases from drying and burning are positively deodorized by mixing with the high temperature gases from the refuse incinerator.

The Nichols Duohearth fills a long felt need in the economic disposal of all municipal sludge, screenings, grit, garbage and combustible rubbish in a single unit.

A Nichols Duohearth combined rubbish and sludge incinerator unit was placed in operation during the early part of 1946 at a plant in Pearl River, New York and the operation of this unit has been of particular interest because of the great variation of characteristics and quantities of sludge, filter cake, industrial wastes, molds, slurries, etc., which it had been called upon to dry and incinerate.

During the past year two new Nichols Multiple Hearth sewage sludge incinerator installations were placed in operation. The plant at East Chicago, Ind. is handling a filter cake from a biochemical process of sewage treatment. The plant at the city of Dearborn, Mich. includes an additional incinerator unit which was required to supplement the capacity of the original Nichols Multiple Hearth Incinerator. Greatly increased sludge quantities at Dearborn, due to the war and to growth from industrial activities, required increased sludge incinerator capacity, and after 12 years of satisfactory operation of the first Nichols Multiple Hearth sludge incinerator the city installed a duplicate unit. Both incinerator units at Dearborn are operated when sludge quantities require.

NORDSTROM VALVE COMPANY

A Division of Rockwell Manufacturing Company

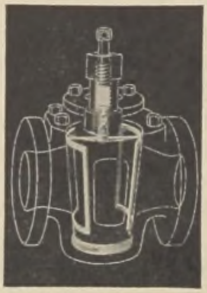
Pittsburgh, Pa.

Nordstrom Lubricated Plug Valves

Nordstrom Valves are well adapted to the requirements of Sanitation Departments for use in sewage disposal plants because of their leak-resistant, non-sticking and positive shut-off construction. Made in a wide range of sizes and for various pressures.

"Sealdport" Lubrication

The "Sealdport" method of valve lubrication forces "Nordcoseal" Lubricant under high pressure between the

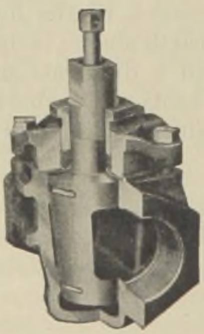


Lubricant System in Nordstrom Valves.

plug and body, making the valve leak-resistant and freeing the plug to assure easy operation. Simply turning a screw at the top of the valve completely covers both seat and plug surfaces with a film of lubricant.

Lubricant grooves in the valve body and plug are so arranged that the lubricant cannot be blown from the valve by the fluid or pressure in the line. The Nordstrom method of "Sealdport" lubrication has made practical the use of plug cock valves as large as 30 inches in size and assures the most satisfactory service for sewage disposal plants.

In the phantom view shown, the lubricant grooves on the plug and in

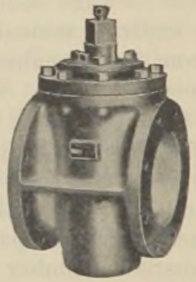


Cut-away view, Nordstrom Valve.

the valve body may be clearly seen. These grooves form an interconnecting system which conducts the lubricant to the lower end of the plug in either open or closed positons of the valve.

The principle of high pressure "Sealdport" valve lubrication may be easily traced through the entire valve in the phantom view.

Also is shown plug assembled within valve body and with the especially designed gland and yoke in place. Should neglect to lubricate occur, valve will remain tightly seated because all unbalanced pressure is in a downward



Nordstrom Valve with standard face-to-face dimensions.

direction and can be readily freed by inserting a stick of Nordstrom Lubricant and turning down the screw.

Standard Dimensions

Made in one pattern are Nordstrom Valves that have standard gate valve face-to-face dimensions and embody all the advantages inherent to the general Nordstrom line, including the patented "Sealdport" method of lubrication and plug lifting features that make operation easy and positive at all times. It is now economical to replace valves of other types with these lubricated plug valves since they can be installed without altering existing pipe dimensions.

PACIFIC FLUSH TANK CO.

4241 Ravenswood Ave., Chicago, Ill.

Heater and Heat Exchanger with Automatic Gas and Oil Firing

In connection with the digestion of solids separated from sewage in a digestion tank, it is necessary to maintain the temperature of the digesting mass within rather close limits.

Heretofore the heating of digestion tanks has been largely accomplished by the burning of the gas generated in the digestion process and complete reliance has normally been placed upon sufficient gas being available to be burned to maintain the digester temperature.

It has been observed, however, that when undue cold weather is encountered, there may not be sufficient gas to maintain the digester temperature. With a drop in the digester temperature the rate of gas production drops, further reducing the amount of gas available for heating. If this vicious cycle continues, the lowered tank temperature will cause the digestion rate to drop to such a low point that the digestion process may have to be started anew, with dependence on auxiliary fuel for heating.

The combination gas and oil firing features of the digester heater and heat exchanger unit makes possible the maintenance of digester temperatures within the desired limits without manual control by the operator. When sufficient gas is available, the unit operates entirely on gas as a source of fuel supply and continues to use this gas to the extent that heat is required in the digester system. When the supply of gaseous fuel drops, which results in a reduction in the pressure at the gas supply main, a liquid fuel such as oil is fed automatically to the burner of the unit.

The smallest unit manufactured by Pacific Flush Tank Co. is shown in Fig. 1.

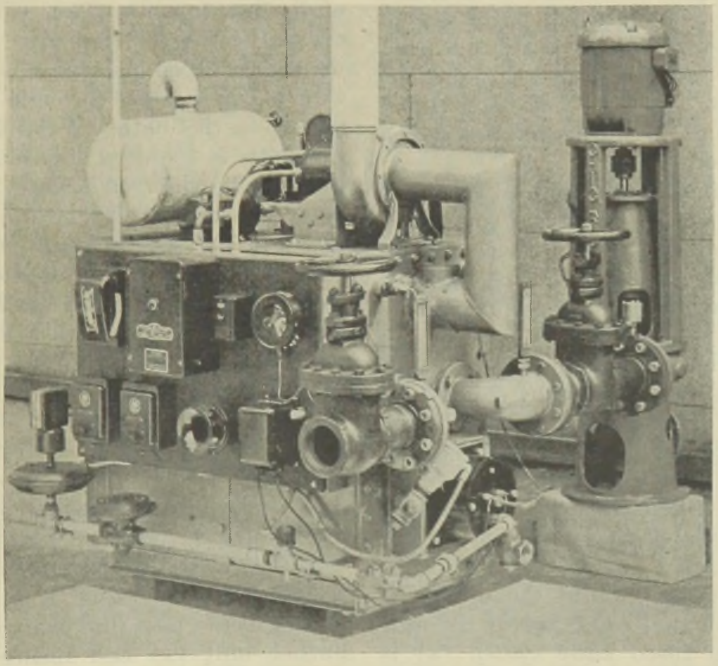


FIGURE 1.—Automatic gas and oil firing type of heater and heat exchanger.

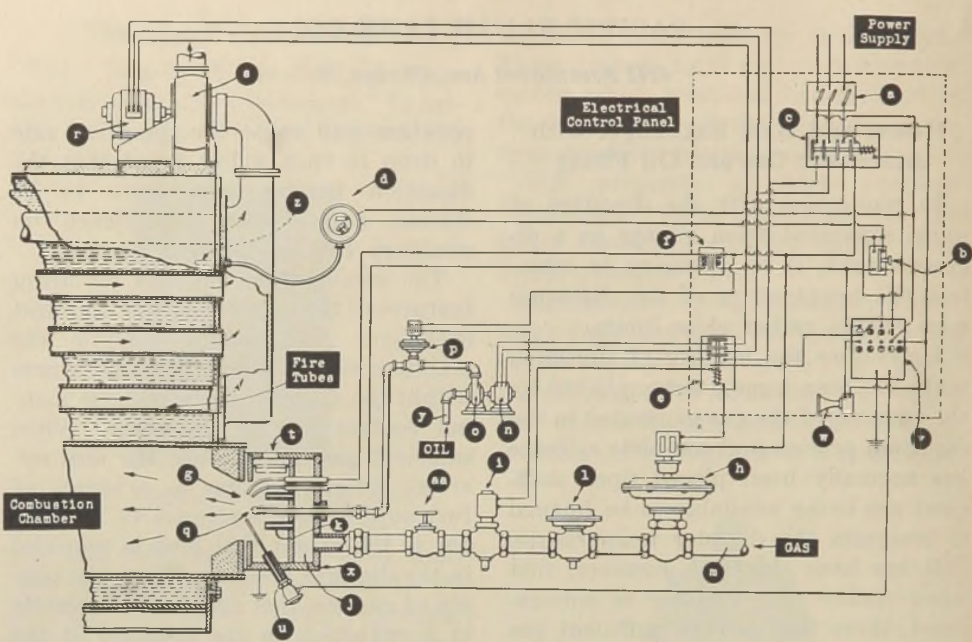


FIGURE 2.—Automatic gas and oil firing for P.F.T. heater and heat exchanger.

Description of Controls and Control Circuits

One method employed for accomplishing the automatic control of the firing of a gaseous fuel or a liquid fuel is indicated in Figure 2. A longitudinal cross-section of the heater and heat exchanger unit is taken through the combustion chamber and fire tubes. The gas and oil burner is also shown in cross-section.

A combination of standard electrical and pressure control devices are shown in diagrammatic form and are used in the automatic control of the burning of the two different types of fuel as described hereafter.

The initial starting or stopping of the unit is accomplished by completing the power circuit at main fused knife switch (a) which serves to energize magnetic starter (c) which, in turn, completes the circuit, allowing the unit to come into operation if the system requires heat.

Push button (b) in the circuit is used for manual restarting of the unit after main fused knife switch (a) is in closed position, in case of any failure of the unit to start initially or in later sequences.

Thermostatic mercury switch (d) may be set to maintain any desired range of temperature in the water bath of the heater unit. The requirements of the heating of the water bath are dependent upon passage of the material to be heated through coils immersed in this bath. When heating of the bath is required, thermostatically controlled switch (d) causes the magnetic starter or relay (e) to be energized if the gas pressure is low, or causes gas valve (i) to be energized if sufficient gas pressure is available, and at the same time causes a current to pass through the ignition coil (f), which creates a high tension spark at spark gap (g) for a period of approximately 25 seconds (or continuously, if desired).

If sufficient gas is available to exert a pressure at pressure switch (h), a circuit is closed which allows the electrically operated valve (i) to open, allowing gas to pass into chamber (j) of burner unit (x) and thence through nozzles (k) into combustion chamber where it is ignited by spark gap (g) previously referred to. Pressure regulator (l) is inserted in the line for the purpose of controlling the pressure of the gas fed into the burner chamber regardless of the varying pressure existing in the gas line at point (m).

In the event that there is not sufficient gas available for burning, the pressure at switch (h) drops to a low setting point which, in turn, starts the motor (n) of oil pump (o). When a pre-set pressure of, say, 90 p.s.i. is obtained, pressure switch (p) opens, allowing gas valve (i) to close off the gas supply completely. The unit is then operating, using a liquid fuel.

During the oil cycle, oil is spread out through nozzle (q) and is ignited by spark gap (g) or by the combustion of gas in the combustion chamber which has not been completely shut off in the previous cycle.

When the quantity and pressure of the gas increases, switch (h) again comes into operation to shut off the oil pump (o) and motor (n). As soon as the pressure in the oil line set at pressure switch (p) drops to some predetermined setting, say, 70 p.s.i., electrically operated gas valve (i) is allowed to open, resuming the burning of the gaseous fuel.

During the time that the unit is calling for heat either by the burning of liquid or gaseous fuels, fan motor (r) is in operation, causing an exhaust by means of fan (s), which draws in combustion air through ports (t).

The fan is started initially when main fused switch (a) is closed, and is in operation approximately 10 seconds before either of the fuel lines controlled by valve (i) or motor (n) of oil pump (o) may allow fuel to enter the burner unit (x).

When the heating of the unit is satisfied, mercoid switch (d) shuts off the supply of gas or oil fuel and also the draft induced by fan (s), by stopping motor (r).

In the system flame safety devices are provided of a standard type which depend upon the electrical conductivity of a flame. A circuit is completed from the ground circuit of the unit through the flame to flame rod (u) which acts upon Flame-o-trol unit (v) so as to allow the various operations previously indicated to take place as long as there is the presence of a flame. In the event that the unit should be calling for heat and the flame goes out, Flame-o-trol unit (v) is actuated in such a manner as to close off all fuel supplies of gas from (m) or oil from (y) and exhaust fan (s), and cause alarm horn (w) to signal a warning. Prior to giving this warning, however, the unit in a cyclic manner attempts to relight the gases by automatically energizing the spark coil (f) and allowing one of the fuel lines to open momentarily, the gas fuel line opening at valve (i) first if gas pressure is available at switch (h). If the flame does not satisfy the completion of the circuit as controlled by Flame-o-trol unit (v), the entire system is shut off and remains in the warning state, requiring the operator to make manual adjustments before the unit can be restarted. Such manual starting is attempted by depressing push button (b), after a flame failure shut-off.

Although only one general method of operation is indicated by this circuit, connections are possible by means of additional settings on a dual type mercoid switch similar to (d) to allow the liquid fuel to be supplied to the burner unit (x) at nozzle (q) at the same time that gaseous fuel is supplied at gas nozzles (k), in case there is sufficient gaseous fuel at supply line (m) to allow proper burning but the gaseous fuel is not sufficient to satisfy heat requirements of the unit as determined by a predetermined low temperature setting of mercoid switch (d) reflecting the temperature of the water bath by bulb (z). The same general circuit shown is used under this latter control condition. Manually operated gas valve (aa) is used to shut off the gas supply if desired by the operator.

PITTSBURGH EQUITABLE METER DIVISION

ROCKWELL MANUFACTURING COMPANY

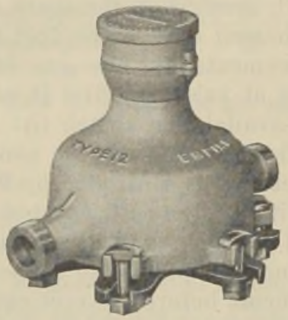
Pittsburgh, Pa.

Pittsburgh-Empire Water Meters

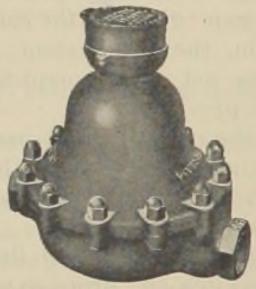
Pittsburgh-Empire Water Meters have for years been used by large and small municipalities alike throughout the country. Their excellent construction, maintained accuracy, and long life have been thoroughly proven on hundreds of thousands of installations. In sewage treatment plants they are recommended for measuring the heated water circulated through digestion tanks, and all cold water piped for various uses throughout the plant. Hot water meters are of a special construction to operate under high temperatures.

Emco Meters for Sewage Gas

It has become common practice in the design and operation of sewage disposal plants to utilize the gas generated during sludge digestion as a source of heat and power. Since modern engineering methods have demonstrated that accurate measurement is



Empire Oscillating Piston Water Meter.



Pittsburgh Disc Type Hot Water Meter.

Specifications of Emco Meters for Sludge Gas

Meter No.	Size of Conns. (in.)	Length (in.)	Width (in.)	Height (in.)	Weight (lb.)	Capacities—Cu. Ft. per hour of 0.6 Specific Gravity Gas	
						½-inch Pressure Absorption	2-inch Pressure Absorption
0	¾, 1 or 1¼	9 7/16	8¾	11⅜	38	175	350
1	1, 1¼ or 1½	11⅛	9⅛	14¼	46	275	550
2	1¼ or 1½	12¾	10⅛	16⅝	67	350	700
2½	2	20⅜	14	25⅛	227	850	1,800
3	2 or 3	22½	16½	27 13/16	305	1,200	2,500
4	3 or 4	27¼	19¾	32⅞	506	1,700	3,800
5	4	40	20¾	51⅛	1,000	5,000	10,000

Note: Capacity ratings of gas meters are usually based on a gas of 0.6 specific gravity. To correct for gas having a higher or lower gravity, the following formula should be used:

$$R_g = R \frac{\sqrt{0.6}}{\sqrt{G}}$$ where: R_g = rating for gas having specific gravity G

G = specific gravity of gas

R = rating in table above

Emco Sludge Gas Meter.

a necessary part of the control of any process, the metering of this gas is an important matter and the choice of meter, one that should be given careful consideration.

Experience has proven that the conventional meter used for measuring manufactured and natural gas will not operate satisfactorily on this type of service due to the destructive effect which sewage gas has on materials commonly employed in the construction of gas meters. For this reason Emco meters for sewage gas are especially constructed from materials which laboratory tests and years of field experience have proven resistant to the action of this gas. The outer case is of heavy cast iron with interior coated with a special preparation. A special tannage and treatment is given to the diaphragms.

ROOTS-CONNERSVILLE BLOWER CORP.

Connersville, Ind.

While the Roots-Connersville Blower Corp. of Connersville, Ind., is best known as the pioneer builder of rotary positive blowers, this company, established in 1854, has built a great many centrifugal units during recent years, some of which have found application in the large sewage treatment plants over the country.

Roots-Connersville is currently stressing what it describes as its "dual-ability" to furnish either type of blower on a completely unbiased basis, depending upon the requirements of the particular application for which the blower is to be used.

Illustrated in Figures 1 and 2 are two typical Roots-Connersville installations, one showing the centrifugal type of blower and the other the rotary positive type. The centrifugal installation (Figure 1) is located at the Jamaica sewage treatment works in New York City, and comprises three 3-stage units each having a capacity of 15,000 c.f.m. at 7¾ p.s.i., all direct coupled to 700 hp. motors.

Figure 2 shows two dual-impeller blowers installed in the sewage treatment plant at Cranston, R. I., each unit being rated at 1,500/2,500/4,000 c.f.m., respectively, operating at a speed of 600 r.p.m. at a pressure of 6½ p.s.i. These dual-impeller blowers are direct connected to 150 hp. motors. Due to the dual-impeller design, a plant is able to regulate its air supply to the varying load demands by simply selecting any one of the three capacities available. This is done by bypassing the chamber not required for the particular cycle of operation, thus giving 1,500 c.f.m. or 2,500 c.f.m. by utilizing only the smaller or larger chambers, or a total combined output of 4,000 c.f.m. when both of the chambers are used at full capacity.

Blowers built by Roots-Connersville have found wide application in activated sludge type plants of all sizes. They are also used for the trickling filter process, or wherever air is required in large quantities, produced as economically as possible.

In recent years there has been a distinct trend to the use of digester gas for fuel in engines designed to burn

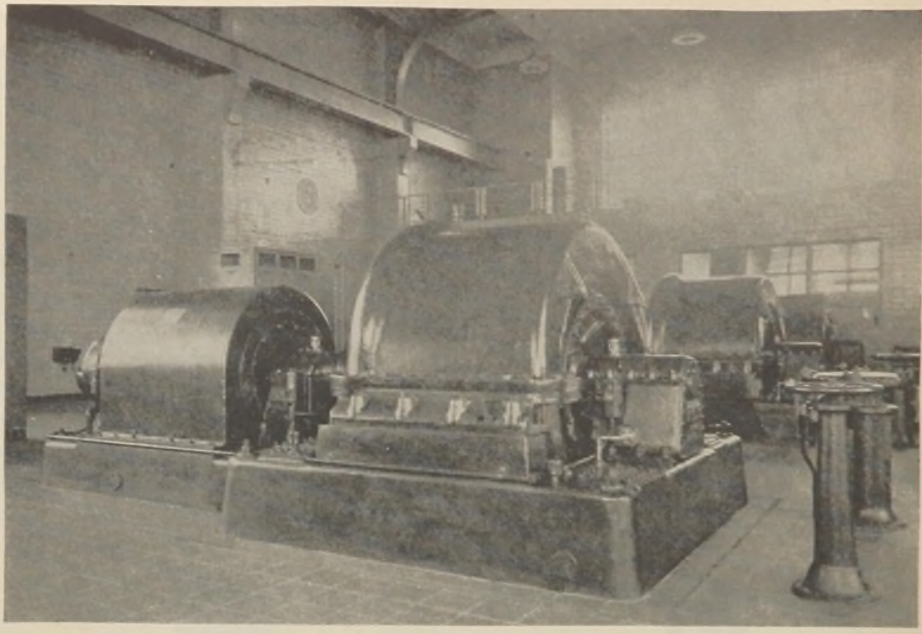


FIGURE 1.—Three 3-stage centrifugal blowers installed in Jamaica sewage treatment plant, New York City.

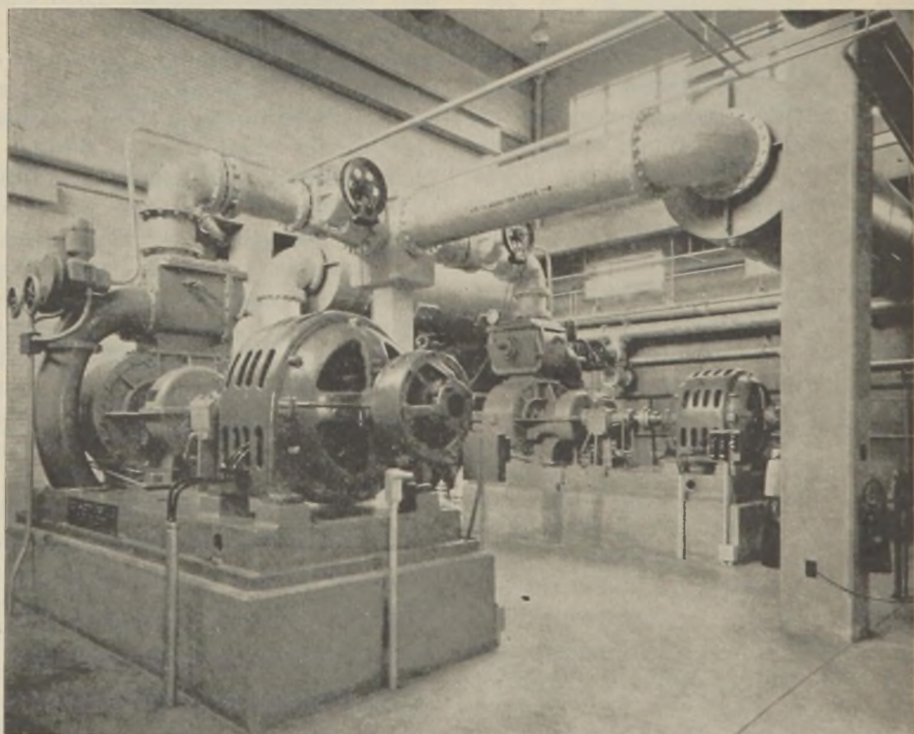


FIGURE 2.—Two dual-impeller rotary positive blowers installed at the Cranston, R. I., sewage treatment plant.

it, and for this type of drive the Roots-Connersville rotary positive blower has been found to possess inherent characteristics which make it the choice of many plant operators so utilizing gas. Records are on file which definitely establish the economies to be gained by utilizing sludge gas as engine fuel, one such showing a saving of $6,000 per year over the cost of purchased electric power.

While the installations shown here illustrate larger size plants, Roots-Connersville builds blowers ranging from 10 c.f.m. to 50,000 c.f.m., up to pressures of 30 p.s.i. in standard designs, and can also offer equipment for higher pressures and larger volumes. In addition to its line of blowers, Roots-Connersville also builds a comprehensive line of meters for measuring digester gas.

ROYER FOUNDRY AND MACHINE CO.

Kingston, Pa.

Demand for Sludge Fertilizer Continues to Grow

Fertilizer made from sewage sludge was used to a much greater extent during the war years than ever before. While the fertilizing values in sewage sludge have long been known, it took war's unprecedented demand for greater food production to bring it into its own. In addition, hundreds of naval and military establishments installed Royers and used the sewage product to beautify lawns at their bases and camps.

Sewage treatment plants with proper facilities for converting sludge into an applicable fertilizer report that they could have sold much larger quantities than they were able to produce—a volume which is, of course, limited to the amount of sludge available.

These same sewage works also report that there is no falling off this year in the demand for sludge fertilizer—in fact, it has increased. This is due to two causes. Sludge fertilizer proved its ability to increase food crops during recent years more than ever before. Its value has become known to more people, from Victory gardeners to the largest commercial growers. Also, more American-produced food than ever is needed overseas—to combat stark starvation in a number of lands. As one sewage works superintendent puts it: "I'd sure feel heartless if we were burning or burying our sludge now."

The sewage sludge that comes from the drying beds, caked and full of lumps, is hardly in condition for use as a fertilizer, although it has been used to a limited extent. Screening and other primitive methods of breaking up the sludge cake are laborious and fall far short of making the fertilizing elements in the sludge fully effective. However, sludge cake can be converted into a ready-to-use and marketable fertilizer—at a cost no greater than that of burying or incinerating the sludge. A growing host of sewage

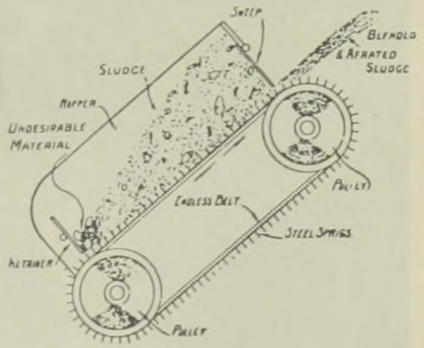


FIGURE 1.—Showing the action of the "combing belt" in disintegrating sludge cake in the hopper of the Royer.

treatment plants, in cities large and small, have eliminated the expense of sludge disposal, and in its place are enjoying a steady revenue by means of the Royer Sludge Disintegrator.

The Royer is a compact, ruggedly constructed, portable or stationary machine for the rapid, economical disintegration of sewage sludge from the drying beds. Its operation is shown in Figure 1. Sludge cake is shovelled by one or more men into the hopper.

The machine employs an endless belt of tough, resilient composition, upon which are mounted rows of steel teeth or sprigs to accomplish the thorough shredding, mixing and aerating of the sludge cake. The belt travels at an angle of 45 deg., and the teeth are so shaped and the rows so closely spaced that caked, lumpy or matted material deposited upon the belt cascades, rolling over and over on the belt as it is combed by the teeth. The material is reduced to pea size and carried to the discharge end, where it is automatically discharged onto truck or stock pile.

Sticks, stones and other trash gravitate to the bottom of the belt, where they are periodically removed by means of a gate. The Royer reduces the moisture of the sludge, which is aerated twice—on the belt and again on the discharge. The machine turns out an effective and ready-to-use fertilizer. A high strength fertilizer may be obtained by mixing enriching chemicals with the sludge in the machine. Fertilizer produced with the Royer is

FIGURE 2.—Royer Sludge Disintegrator converting sludge into fertilizer at Charlotte, N. C.

being sold under a multitude of trade names by sewage works throughout the land (Figure 2).

The Royer is available in twelve stationary and portable models; electric motor, gasoline engine or belt-to-tractor driven, with a size for every sewage treatment works.

SIMPLEX VALVE AND METER COMPANY

6725 Upland St., Philadelphia 42, Pa.

Simplex Air Release Valve

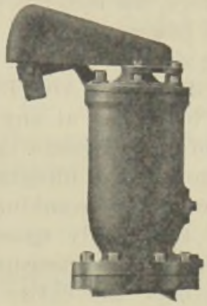
The Simplex Air Release Valve automatically vents air obstructions that might otherwise cause loss of operating efficiency or damage to pipe lines and

pumps. Simple, rugged construction assures dependable service. One lever movement operates a tightseating needle valve, actuated by a heavy glass ball float. Types available for pressures up to 1,250 lbs. per sq. in—tested to several times the service working pressure.

Type "B" Sewage Air Release Valves are especially designed to meet the particular conditions of sewage service and eliminate shutdowns caused by trapped air and gases from decomposing organic matter. A permanent hose connection makes it possible to accomplish routine periodic backflushing merely by opening the water valve to allow clear water to fill the release valve and the system. The flush is quickly and easily completed without dismantling any part of the installation.

Simplex Air Inlet Valve

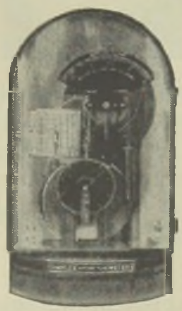
Eliminates the possibility of collapse of pipe line walls due to sudden drop in pressure in gravity flow line. The non-collapsible float is buoyed by the water in the line until a condition arises which tends to create a vac-



uum—then the buoying effect of the water is lost, the float drops from its seat, to admit air and destroy the vacuum. The Air Inlet Valve serves another useful purpose in allowing escape of air from summits while filling lines after shutdown.

Type MO Meter

Indicates, records and totalizes fluid flow rates. Consistent high accuracy over wide flow range (up to 32.5 to 1);



extreme sensitivity to every change in flow; simplicity of design, trouble-free operation, minimum maintenance cost.

Evenly graduated, direct reading flow scale and chart. Floor stand, wall bracket or panel board mounting.

Air Differential Flow Meter

For Liquids Containing Suspended Solids

Corrosive fluids are measured without directly touching this highly accurate instrument, since the differential head is measured directly from the primary device and then transmitted to the meter through the medium of air. Therefore, the meter is unaffected by the fluid being measured, and gives trouble-free service year in and year out. The Simplex Air Differential Meter may be placed at any elevation regardless of the pressure in the line. It may be installed to integrate, record and totalize in any combination; the chart is of the evenly spaced, direct reading type; flow measurement is easily read over the entire range of the meter.

Type MS Flow Rate Meter

Accurately indicates, records and totalizes flow rate. Bell-shaped float automatically extracts the square root

of the law of flow and makes possible the direct reading chart—evenly graduated from 10 to maximum. The arm of the recording pen connects directly with the float for inevitable accuracy of transcription and sensitivity to flow change.

Simplex Manometers

Simplex Manometers are used with Venturi tubes, flow nozzles, and orifice plates for flow measurement of water, steam, air, etc. They are available in fixed types (Type MAB) for permanent installation or portable models (Type MAC) for field use. Extra high-accuracy units, Type MAP, have been designed for the Simplex W–K method of turbine flow measurement and for laboratory testing. There are also special models (Type MAD) for filter plant use to measure both rate of flow and loss of head, especially where low head conditions exist and vacuums may be present.

Simplex Air Operated Filter Gauges

These operate on the same principle as the air differential flow meter and can be placed at any elevation since the differentials produced at the primary devices are transmitted to the gauges through the medium of air. Both loss of head and rate of flow may be measured by application of this system. Added advantage is the freedom of the mercury cylinders from contact with the liquid being measured. Extreme accuracy of measurement by this means may be obtained.

Simplex Parabolic Flume Meter

A new form of measuring device designated specifically for installation in pipes flowing part full or sewers or channels where low head conditions exist. The flume section permits gradually increasing velocity without materially decreasing the depth of liquid at measuring section, tending to provide self-scouring features. It is applicable to measurement of water, sewage or industrial wastes and operates in conjunction with a float operated Type MSF meter which can be arranged for indoor, wall or floor mounting, or outdoor mounting in weatherproof case. The meter is capable of measurement over a long range of flow with consistent accuracy. Mechanical connections between flume and meter, or distance electrical transmission, can be provided as desired.

Simplex Air Meters

Designed generally along the same lines of the MS mercury cylinder instrument, these meters are specifically adapted to the measurement of air flowing through pipelines under such conditions that warrant the use of primary devices such as Venturi tubes, flow nozzles on orifice plates. The standard form is designed to operate with maximum differential of 6 in. of water, but cylinders for other differentials can be furnished. Meters employ circular, evenly spaced daily or weekly charts and are of indicating, recording, or totalizing types or variations thereof. Floor stand or wall bracket mounting is optional.

Simplex Sewage and Sludge Control Equipment

Specially designed equipment utilizing two Venturi tubes or rate controllers as primary devices operating in conjunction with a proportional control setting, which permits the flow to be varied through both primary devices over a predetermined range with an equal flow through each unit. The control feature is such that the flow through one primary device can also be proportioned against flow through the second primary device in a ratio of 25 to 70 or 70 to 25, or any intermediate setting. This equipment is adaptable for installations in filter plants, sewage treatment plants or industrial installations.

TENNESSEE CORPORATION

619–27 Grand Bldg., Atlanta 1, Ga.

FEEDING FERRI–FLOC

FERRI-FLOC, a commercial ferric sulfate sold under that trade name, has found wide application in sewage and industrial waste treatment and water purification plants over the United States in recent years. A great deal has been written on various types of treatment and applications of the chemical, but very little has been said about the method of feeding and/or putting the material into solution. This article is devoted to the proper feeding and solution equipment, with particular attention paid to the smaller plants.

Iron salts in one form or another have always been well-known in the sewage treatment field. Up until the last six or seven years the only available forms of iron were ferrous sulfate and ferric chloride. These two chemicals were available at a price that was not prohibitive for use as a coagulant. Ferric sulfate at that time had not been developed in a commercial form.

About five years ago the Tennessee Corporation introduced a form of ferric sulfate (under the trade name "FERRI-FLOC") that could be marketed at a price that made it competitive with other forms of coagulants. This material is a partially hydrated ferric sulfate. It is a stable free-flowing salt, granular in shape and can be fed through any standard dry feed equipment. As it is partially hydrated it is easy to handle, safe to ship in bulk cars, and can be stored in open bins over a considerable period of time. Furthermore, being partially hydrated it does not pass through a gummy stage when going into solution. *Highly important, it does not require careful proportioning of water to the salt.* The chemical contains approximately the same amount of heat of solution as other partially hydrated salts. This small amount of heat aids in putting the material into solution.

Probably the most important requirement in feeding any chemical in sewage or water treatment plants is getting the material into *complete* solution. This is overlooked in many plants, both large and small. It is even more important with the use of ferric sulfate. In a number of plants where coagulants other than iron salts are used, the chemicals are more or less flushed into the main volume of water. In this manner a large portion of the chemical immediately settles to the bottom of the tank and is lost as far as coagulating value is concerned. This is true with aluminum salts as well as iron salts.

As stated before, FERRI-FLOC can be used in any standard dry feed equipment, with only a few modifications. The feeder should be equipped with a dissolving pot constructed of stainless steel, lead, rubber-lined, or earthenware, and equipped with some means of mechanical agitation. The size of this pot depends upon the maximum dosage required to treat the water and should afford 20 to 30 minutes retention time.

In order to obtain optimum coagulating value, the FERRI-FLOC solution should be diluted as much as possible before being applied to the water. This dilution should take place *after* complete solution has been reached. Complete solution will be indicated when the solution has a dark reddish-brown color. An arrangement such as a box or a funnel should be fitted to the overflow of the solution tank, where additional water can be added for the necessary dilution.

Another type of solution tank can be constructed from an earthenware crock of the required size, depending on the necessary dosage, and a hole sandblasted into this of sufficient size to accommodate a rubber hose to convey the solution to the point of application. A wooden baffle can be placed in the crock so as to prevent short-circuiting of the solution and to insure the full retention time.

A LIGHTNING mixer, equipped with a stainless steel shaft and propeller, has proved to be the most efficient means of agitation. However, a number of plants have constructed their own mixer by using a small ball-bearing motor and fitting it with a stainless steel shaft of desired length.

At the present time there are on the market a number of highly efficient chemical feed machines that have been designed to meet the demand for an accurate and fool-proof feeder. These devices are usually equipped with a lead-lined solution tank with a stainless steel agitator and a meter suitable for measuring the solution water flow. These machines are available at a moderate cost.

Care should be taken to handle the chemical in stainless steel, lead, rubber, or earthenware *while it is in solution.* It is not necessary to take any special precautions while the material is in the dry state. It can be handled as any dry chemical and stored in steel or concrete bulk hoppers regardless of climatic conditions.

The Tennessee Corporation maintains a staff of technically trained field men, with the sole object of giving users and potential users of FERRI-FLOC assistance in their water and sewage treatment problems. These men are available without any obligation. Inquiries will be welcomed for any additional information. A copy of a 24-page booklet containing information of interest to both water and sewage works personnel will gladly be furnished upon request.

UNITED STATES PIPE AND FOUNDRY COMPANY

General Offices: Burlington, N. J.

Cast Iron Pipe for Sewers

The use of cast iron pipe for sewers is steadily increasing throughout the country. Many new sewage treatment plants are being built, some in conjunction with new sewer systems and others to augment existing systems. Furthermore, major revisions are being made in large numbers of existing treatment plants, not only to increase their capacity but to provide more complete treatment that will produce an effluent of better quality. Treatment costs money; the amount of money is almost directly proportional to the volume treated. Logically, therefore, every gallon of water that seeps into the mains through leaky joints, cracked pipe or from other sources not only crowds the sewer but costs good money to convey it through the various treatment stages.

Cast iron pipe possesses three outstanding properties that are most important for present day sewer construction:

1. Tight pressure type joints that prevent infiltration, thereby reducing treatment costs and incidentally eliminating trouble with tree roots.

2. Compressive strength to resist heavy earth loads due to deep fills over the pipe. Beam strength to withstand stresses caused by earth movement. Bursting strength to resist internal pressure when used as a force main or when pressure might occur temporarily due to floods.

3. Long life that has been proven by years of satisfactory service, conse-

FIGURE 1.—Installing 14-in. and 18-in. flexible joint cast iron pipe for dual line sewer siphon across the Passaic River, Fair Lawn, N. J.

quently permitting a low annual amortization charge on the original investment.

There are many instances in various parts of the country where cast iron pipe has been installed in sewage works for flow mains, force mains, stream crossings (Figure 1), outfall sewers and in treatment plants.

Cast iron flow lines are generally installed where the pipe is laid in water saturated soil, to eliminate troublesome and costly infiltration.

Cast iron force mains are being installed at a steadily increasing rate due to the growing demand for sewage treatment. Treatment plants are usually located some distance away from the more densely populated sections of a community. Frequently, it is not possible to have gravity flow to the plant. In those cases the sewage flows into a sump and is then pumped through a force main to the treatment plant.

Cast iron outfall sewers have been widely and satisfactorily used to convey sewage, in most cases treated and in some cases untreated, out into a body of water for final disposal.

Cast iron pipe and fittings are used extensively to transport the sewage through the various treatment stages.

THE VAPOR RECOVERY SYSTEMS COMPANY

Compton, Calif.

EFFICIENT POSTWAR DESIGN OF SEWAGE GAS CONTROL

''Know-how'' and experience gained through the closest co-operation with sanitary engineers, Army engineers in the installation of over 3,000 sewage treatment plants are now incorporated in all ''Varec'' approved equipment. The strictest and most rigid conformity to exacting designs and specifications has developed higher quality, efficiency, and craftsmanship now available to the sewage works field.

''Varec'' Relief Valve, Fig. No. 78 (Figure A), is designed for use in water, oil, air, steam, or gas service. It may be installed on the discharge side of every pump and on pressure gas holder domes as safety valves. An inside spring is housed in an air-tight bonnet. The smooth-lifting valve plug is of the regrinding type. It is a sturdy, high capacity, top-guided valve featuring an accurate spring action, easy adjustment, low percentage of ''blow-down'' and low maintenance costs. It seats sealtight for long extended periods of time. Adaptations and accessories make the Fig. No. 78 (inside spring) and Fig. No. 79 (outside spring) valves (Figures A and B, respectively) efficiently suited to any demands in the sewage treatment plant.

Further research, on the ''Varec'' approved Pressure Relief and Vacuum Breaker Valve, Fig. No. 20C, gives greater service through the use of HYCAR in the seats of the hyperbolic inner valves. A synthetic rubber product, Hycar remains resilient at temperatures below zero, is highly resistant to heat and is non-soluble in sewage service. ''Varec'' approved Drip Ring eliminates condensate on the seats. Fig. No. 20C with the ''Varec'' approved Flame Arrester, Fig. No. 50A, form unit Fig. No. 58C, the ''Guardian of Sewage Gas Control'' in the treatment plant.

''Varec'' approved Pressure Relief and Flame Trap Assembly, Fig. No. 440, consists of a diaphragm operated regulator, flame trap, and thermal shut-off valve. It maintains a predetermined back pressure, passing sur-

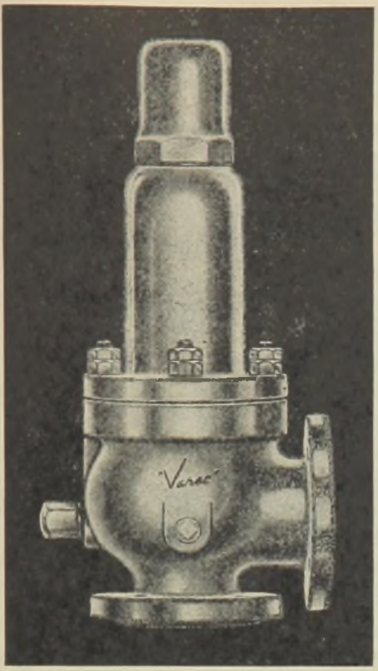


FIGURE A.—"Varec" Relief Valve, Fig. No. 78 (inside spring).

plus gas to burner and stops flame propagation. The patented telescopic flame trap element simplifies inspection and maintenance.

"Varec" approved Flame Trap Assembly consists of Flame Trap and Thermal Shut-off Valve. Installed in all gas lines supplying gas utilization equipment, it arrests flame propagation. Non-corrosive and stainless steel construction makes inspection and maintenance simple.

"Varec" Sensitive Pressure (Reducing) Regulator (single-port), Fig. No. 387, maintains upstream or downstream pressure to within 0.5 in. of water of predetermined pressure.

"Varec" Super Sensitive Pressure (Reducing) Regulator (double-port), Fig. No. 187, maintains upstream and downstream pressure to within 0.2 in. of water of predetermined pressure. Standard working parts are 18–8 S.S. with synthetic rubber diaphragm. Non-corrosive.

"Varec" Back Pressure Check Valve, Fig. No. 211A, is installed in relatively low pressure gas line. Prevents back-flow through meters. Non-chattering, non-pulsating, non-corrosive and non-sparking.

"Varec" Sampling Hatch Covers are installed on digester domes. Available in several combinations of materials, non-corrosive, gas-tight, self-closing, spark-proof; Fig. No. 42A Flanged and Fig. No. 48 Screwed.

"Varec" Sediment Trap and Condensate Drip Trap Assembly, Fig. No. 232D, are of cast iron construction, 18–8 S.S. working parts. Hand operated or automatic.

"Varec" Manometers, Fig. No. 216A, are accurate, with aluminum housing, bronze fittings. Pyrex glass. Automatic—fireproof—safe. Available in single or triple tubes, open or push-button control type.

"Varec" Manhole Covers, Fig. No. 220A, are installed on digester and gas

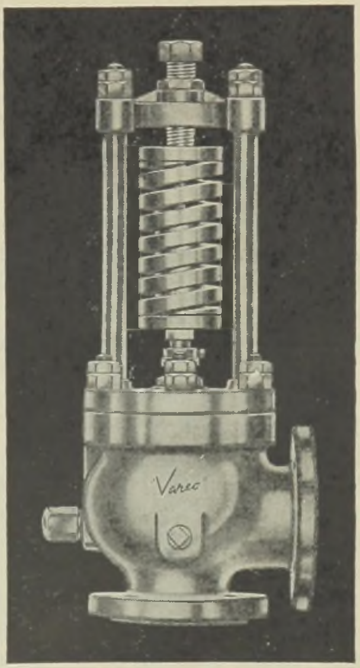


FIGURE B.—"Varec" Relief Valve, Fig. No. 79 (outside spring).

holder domes and afford quick and easy access. Equipped with square, graphite-impregnated, fire-proof gasket, gas-tight and non-sparking.

"Varec" Waste Gas Burners, Fig. No. 236, are installed wherever disposal of surplus gas is a problem. The unit has adjustable air intake in the venturi tube, and pilot valve adjustable from the outside. Long, heavy cast iron draft stack insures proper draft and complete combustion.

"Varec" Flame Check, Fig. No. 51A, is a positive flame stop for small lines, having five 40-mesh fire screens, is acid resisting, and affords easy inspection and maintenance.

The "Varec" Testing Laboratory includes what is probably the largest and most up-to-date capacity finding apparatus in existence. It accurately determines pressure loss and flow capacities up to 500,000 cu. ft. per hr.

Physical constants and characteristics of most gases are generally known, but fixed characteristics change when a fixed gas becomes mixed with foreign elements. Their reaction—even to corrosive qualities—can be determined only by thorough analyses and test investigation, because percentages by volume and varying rates of flow are important controlling factors.

All "Varec" equipment is tested to exact field application, whether for pipe line or tank. A large vessel reduces velocity head to a true static head which is important in testing for true capacity or pressure loss.

Use of this "Varec" flow testing apparatus is offered free of charge to our customers and friends at any time for flow testing or for checking flow curves of any device. Its rated capacity is 500,000 cu. ft. per hr. of air (specific gravity 1.0) at 8 ounces (13.864 inches of water) per sq. in. pressure.

WALKER PROCESS EQUIPMENT, INC.

33 Hoyt Place, Aurora, Illinois

IMPINGEMENT AERATION

For the Activated Sludge Process Offers Many Advantages

Walker Process Equipment, Inc., introduces to the sewage and waste treatment field for the activated sludge process a new aeration unit, "The Impingement Aerator." This revolutionary design replaces the diffuser tube and plate systems accepted for years as the standard universal means for diffused air aeration.

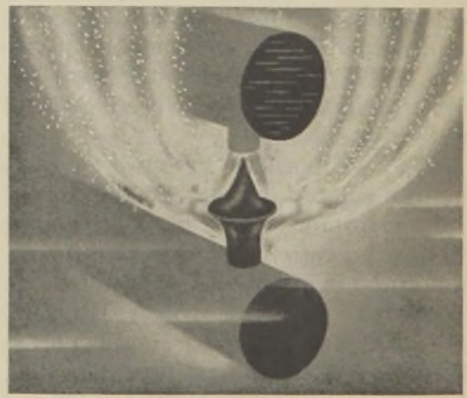


FIGURE 1.—Double cone of the Impingement Aerator showing how the sudden dissipation of energy at the air-water convergent point sets up impingement contact.

The principle involved consists of opposing water and air jet streams discharging against a double cone which causes physical contact of the two masses at the periphery. The sudden dissipation of energy at this convergent point sets up multitudinous shearing planes which reduce the air to minute particles. This impingement contact at the convergent point can be likened to a semi-colloidal mill action (Figure 1).

Functioning of Unit

The functioning of the unit is simple. Settled sewage or aeration tank liquor is pumped under a pressure of approximately 5 to 10 p.s.i. into the water manifold of the assembly. Air is compressed sufficiently to overcome the leg of water and is discharged into the air manifold of the assembly. Pressures in excess of the water leg are required by jet aerator units. This, however, is not true of the Impingement Aerator. The compressed air is entrained by the opposing water action and the entire mixture is diffused through the aeration tank, causing circulation and aeration of the tank contents.

The presence of thousands of small pin-point bubbles is characteristic of a well-operating aeration unit. This is physically observed when this unit is operating in a tank. When the aerator is shut off, these pin-point bubbles continue to break at the surface for an extended period.

The volume of air required to operate this unit is less than 3 c.f.m. per 1,000 cu. ft. of tank volume. This is approximately equal to 0.15 cu. ft. per gal. of sewage treated. As is standard practice in all diffused aeration systems, standby blower capacity is recommended. For the Impingement Aerator we recommend the use of sufficient blowers to provide 10 c.f.m. per 1,000 cu. ft. of tank volume. This is approximately equal to 0.5 cu. ft. per gal. of sewage treated. With this arrangement the amount of air or oxygenation rate can then be varied from less than 0.15 to 0.5 cu. ft. per gal. of sewage treated, in order to suit the job requirements. This can be accomplished through valving and variation in blower capacities.

The source of diffusing liquid for supplying the aeration units in activated sludge plants is normally the aeration tank. In some cases it may be the final settling tank or final effluent. For preaeration the primary effluent is used. Standard sewage pumps are used to circulate these liquids through the Impingement Aerator.

20% Oxygen Absorption

In contrast to diffuser tubes and plates, which cause an oxygen absorption of approximately 3 to 5 per cent,

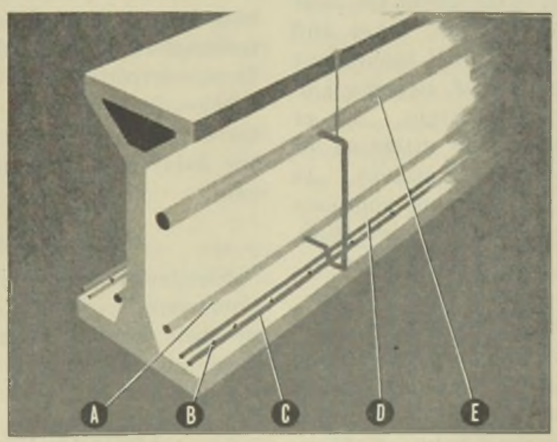


FIGURE 2.—Complete Impingement Aeration installation showing relationship and position of equipment. Legend: A, Water Header; B, Impinger; C, Air Manifold; D, Water Manifold; E, Air Header.

the Impingement Aerator causes an absorption above 20 per cent. It can be said conservatively that to achieve oxygenation comparable to that of diffused tube and plate systems, only a sixth as much air is required.

The maintenance problem resulting from clogged pores and diffuser plates is eliminated with the Impingement Aerator. Since the air discharge holes are large, they do not clog and do not require periodic cleaning. Because of the large discharge orifices, there is no accumulated back pressure on the blowers, as is characteristic of the pressure build-up with porous tubes and plates. The non-clogging feature of the Impingement Aerator (Figure 2) makes it unnecessary to use either air filters or liquid strainers.

The Impingement Aerator need not be installed at dead level in the tank since it is not critical to variations in mounting. The standard assembly arrangement for the unit is in a fixed position near the bottom of the tank, with the water manifold located above the air manifold. To meet individual job requirements, other positions can be used.

Existing installations of porous tubes and plates can be be revamped by replacement with Impingement Aerators. Existing air lines and blowers can be utilized.

Complete information on Impingement Aeration will be furnished on request.

WALLACE AND TIERNAN CO., INC.

Newark 1, New Jersey

Disinfection of Sewage

The need for sewage disinfection cannot today be overlooked. With increasing population the growing demands on water supplies necessitate using sources of supply hitherto considered unsuitable. The increasing popularity of swimming has led to the use of the most available bathing beaches, lakes and watercourses. The national resentment against the pollution of streams and river is gaining momentum. All of these have centered the attention of sanitarians on sewage treatment. As a final measure of securing complete health protection, sewage disinfection, with chlorine, is accepted practice.

Odor Control

The control of obnoxious odors from sewage treatment works, long outfall sewers and other disposal plant appurtenances is distinctly a field for chlorination. Neutralization of hydrogen sulfide (the gas responsible for these obnoxious odors), or prevention of its formation by controlled application of chlorine, is not only quite practicable but also satisfactory in results and economy of operation.

B.O.D. Reduction

Treatment plants discharging effluents into watercourses having extreme seasonal variation in flow have profited from controlled chlorination to reduce biochemical oxygen demand. Field tests have often shown a reduction in the 5-day B.O.D. of more than 30 per cent.

Other Advantages

Similarly, chlorine prevents filter "ponding." The filters are unloaded of their organic accumulations. Surface algae growths are killed and washed out.

Technical Information

Wallace & Tiernan representatives will gladly discuss any specific sewage chlorination problem. Technical bulletins on the many phases of sewage chlorination and W & T Chlorinators will be sent free on request. Address Wallace & Tiernan Company, Inc., Newark 1, N. J.

YEOMANS BROTHERS COMPANY

1411 Dayton Street, Chicago 2, Ill.

SEWAGE PUMPS, PNEUMATIC SEWAGE EJECTORS, ROTARY AIR COMPRESSORS, VACUUM PUMPS, SEWAGE TREATMENT EQUIPMENT

Yeomans Pneumatic Screenings and Sludge Ejectors

During the past three years Yeomans engineers have been studying and conducting experiments on the problem of handling screenings in the smaller sewage treatment plants. Screenings handling is probably the most bothersome problem confronting the average plant operator. Hand raking screens and placing screened material in trash cans, or wheelbarrows, for removal and burial has been a nuisance which is no longer necessary. Continuous or intermittent operation of sewage grinders can also be dispensed with by the use of Yeomans Brothers Company Screenings Ejector.

Yeomans' efforts have been directed toward perfection of a device to relieve the operator of the screenings handling problem, and at the same time perform other highly important functions about the plant. After exhaustive experiment and test the pneumatic screenings and sludge ejector (Fig. 1), an entirely mechanically controlled unit whose versatility is practically without limit, has been perfected.

Raw sewage entering the screen channel is conducted to the underside of a horizontal, submerged bar screen from where it must flow upward through the bars, and all screenable material is intercepted on the underside of the screen and retained in a small hopper in the bottom of the channel.

To clean the screen the operator merely opens a gate in the inspection well and the head of water in the channel backwashes the screen and hopper into the inspection well from which it flows by gravity into the ejector which, when filled, discharges its contents directly to the digestion tanks, thus by-passing pumps, settling tanks and other structures.

Sludge piping in the primary settling tank is arranged to discharge to the inspection well, where the operator can observe the consistency of the material being drawn and prevent excessive amounts of water from being sent to the digestion tank, resulting in a reduction in the amount of supernatant liquor to be handled by the plant. All sludge drawn from the primary tank flows by gravity from the inspection well to the pneumatic ejector, whence it is automatically discharged to the digester.

The skimming pipe on the primary tank is also connected to the inspection well so that when the tank is skimmed, the scum is discharged to the well, then to the ejector and finally to the digester. The sludge withdrawal pipe in the digester also has a connection to the ejector so that sludge may be withdrawn from the bottom of the tank and discharged by the ejector to the upper regions of the digester for recirculation.

The ejector can also be used for dewatering any of the tanks in the plant and is especially effective in handling settled activated sludge in the event that the aeration unit is down. The

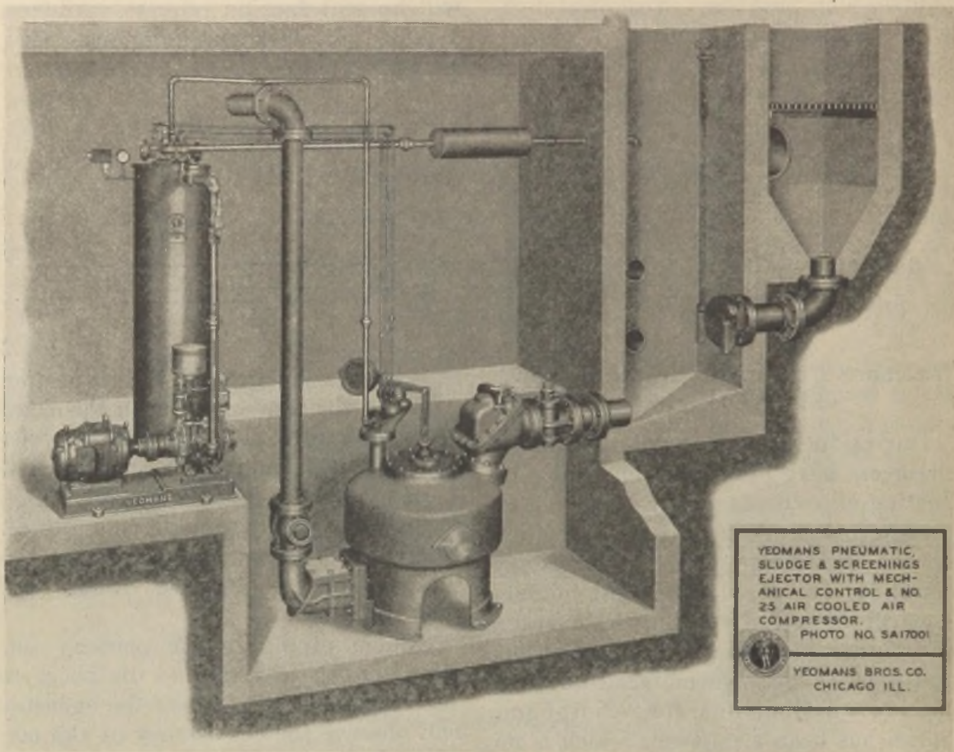


FIGURE 1.—Yeoman Type SSS Pneumatic Screenings and Sludge Ejector.

unit provides automatic screenings handling and at the same time performs all of the duties generally assigned to the plunger type sludge pump. The screenings ejector is of the mechanically controlled type for safe operation in hazardous locations. The special drop type inlet valve is sealed during the discharge cycle and the minimum size is six inches.

Yeomans "Package" Aerifier

Continued refinement of this unit has developed it to a point where it has been accepted for installation in small activated sludge plants all over the world. The economy of the combination aeration tank and final settling tank in one concrete structure is well demonstrated by numerous recent installations of this type of treatment in communities of from 500 to 4,000 population.

The Yeomans mechanical surface aerator, which is the heart of the "Aerifier," has been known and appreciated by sanitary engineers for many years. The unusual ability of this machine to maintain a spiraling circulation of the entire aeration tank contents, and at the same time distribute the mixed liquor over the surface of the tank in thin layers of finely divided particles, has long been an outstanding feature.

Pumping of enormous quantities of liquor is not required with this machine as non-settling velocities are maintained in all portions of the tank through the unique design of the aeration cone. Intimate mixing of the activated sludge and settled sewage is assured and a consistently high dissolved oxygen content in the aeration compartment is maintained with ease.

The "Aerifier in conjunction with

the "Streamline" sludge collecting mechanism in the primary tank and the pneumatic screenings and sludge ejector provides an inexpensive "Package" activated sludge plant that includes all of the advantages of the large plant, yet through compact arrangement, the cost of these advantages has been held to a minimum. The "Package Aerifier" plant produces remarkable results, reductions in B.O.D. averaging 95 per cent and suspended solids 90 per cent, and operating costs are always in the lower brackets.

Original Woodcut by Lynd Ward

DIRECTORY OF ENGINEERS

(Continued on pages 36 and 37)

DIRECTORY OF ENGINEERS

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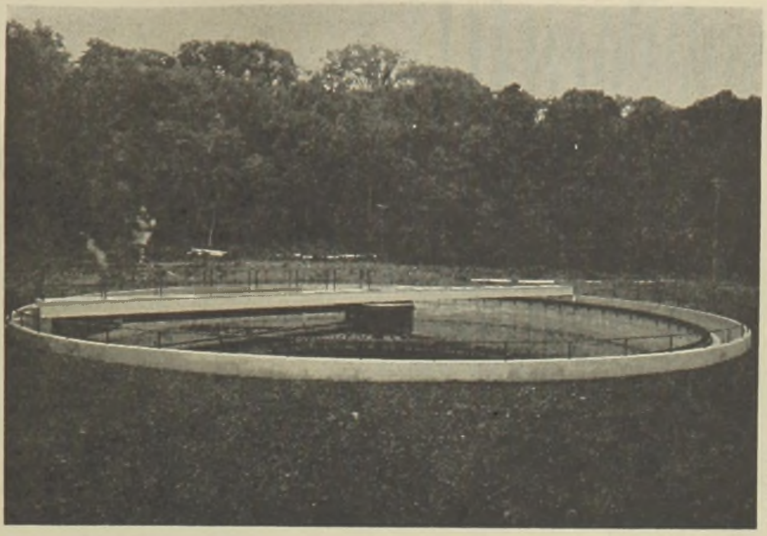
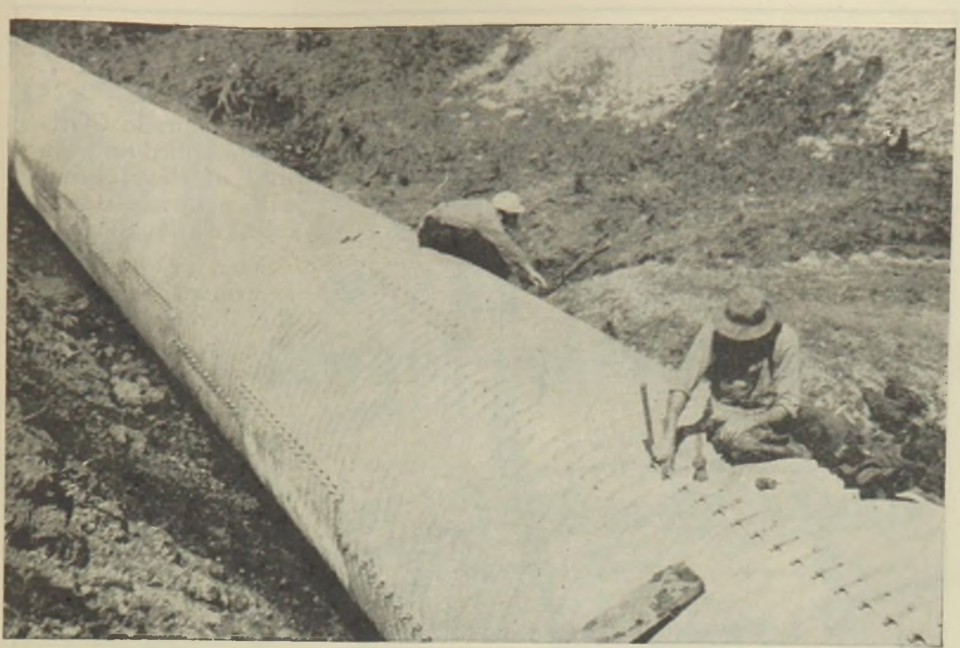
4-POINT CONTACT
MUELLER CO.
CHATTANOOGA, TENN.
4-POINT CONTACT





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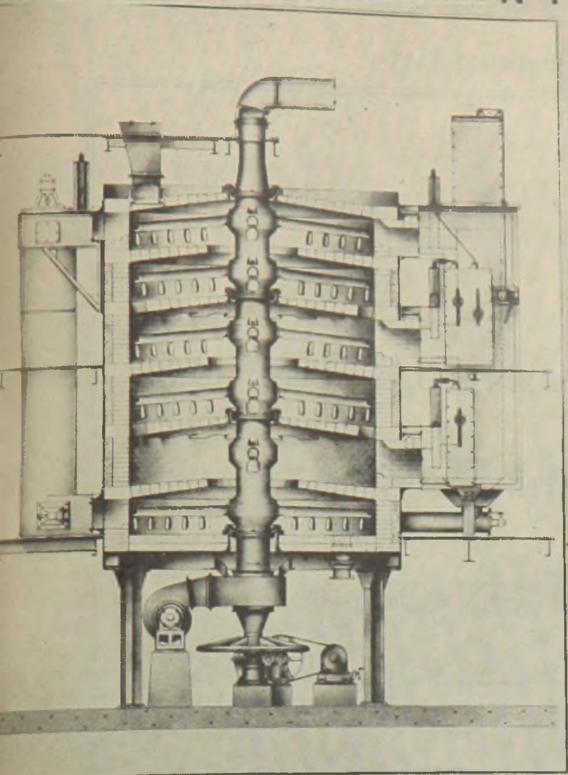
MARTIN *Multi-zone* FURNACE



★ **DRYING ZONE—Hearths 1 and 2, dries the sludge to 20% Moisture.**

★ **BURNING ZONE—Hearths 3 and 4, incinerates the dried sludge.**

★ **FINAL COMBUSTION ZONE—Hearths 5 and 6, includes destruction of odors created in drying zone.**

★ **ZONE CONTROL—Full control by balanced drafts.**

This zone-controlled incineration of sewage sludge, combined with economic destruction of odors before gases pass to the stack, sets the Martin Multi-Zone Furnace apart from others. It gives the Multi-Zone Furnace added values for the community wishing to completely sterilize its sewage wastes in a modern, automatic, economical way. Raw sewage sludge cake with as high as a 75% moisture content may be incinerated without auxiliary fuel. Digested sludge, green or digested ground garbage, screenings and grit can also be readily handled.

Special furnace arrangements can be made to dehydrate the sludge without scorching for producing high-grade fertilizer. The unit also operates without objectionable odors. The product is of a granular nature and free from dust.

The Martin Multi-Zone Furnace has broad application. It is designed to dehydrate or incinerate many kinds of industrial and municipal wastes in a continuous automatic manner with complete control of the entire operation.

Other Morse Boulger units of interest to Municipal, County and State Officials include: Stationary and Rotary Hearth Garbage and Trash Incinerators; Incinerators for Institutions and Office Buildings; Liquid Chemical Proportioning Pumps.

MORSE BOULGER DESTRUCTOR CO.

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