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A few notes from the sanitary engineering field

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"A FEW NOTES FROM THE SANITARY ENGINEERING FIELD"

BY

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A

THESIS

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Approved by____________________________________

Professor of Civil Engineering

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SYNOPSIS

This paper, it is felt, may appear to be a somewhat disconnected discussion of a few points that have come to the writer's attention from his limited experience and practice in the field of Sanitary Engineering. It has not been the intention, however, to provide a discussion of continuity which would take the reader from the ground up through the process involved in the design and construction of any particular structure, system, or plant; but rather is it to be considered the writer's attempt - although probably a somewhat feeble one - to set forth in a note-book, as it were, a few observations he has made together with some of the data he has utilized in connection with his work in the design, construction, and operation of small municipal waterworks and sanitary sewerage systems in the states of Missouri, Illinois, and Indiana. By no means is the paper offered in the light of research, nor are any particularly new ideas or observations claimed by the writer from its contents.

The paper has been divided into four sections. Section I is a brief attempt at one or two observations on some of the difficulties to be encountered by the engineer in the promotion of municipal projects today. Section II contains a condensed discussion concerning the interests of some of the state and other institutions in the design and construction of municipal work, and endeavors to bring out some of the merits of their requirements with the urge to co-operation instead of antagonism towards them on the part of the engineer. Sections III and IV contain some notebook details on waterworks and sanitary sewerage system design, and are given particularly as illustrative answers to the matter listed under Section II. These details apply particularly to the writer's design of a waterworks and sanitary sewerage system for the city of Osgood, Indiana, population 1280, but it is to be noted that they apply equally well to other systems and plants of like nature.

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I. A FEW OBSERVATIONS ON PROMOTION

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It would seem to the writer that the problems the engineer of to-day must encounter as he endeavors to secure a contract for the engineering of a contemplated small municipal project must indeed differ vastly from those of "days of old". That whereas formerly the main problems of the engineer in promotion work consisted of the education of the community to its needs for the better and more hygienic standard of living to be attained through the media of a pure water supply and sanitary method of sewage disposal, the satisfactory recognition of his ability as an engineer, and the intelligent advising of the community in matters of municipal finance, in order to stand a reasonable chance of securing a contract for his services, such are far from the main obstacles to be encountered to-day. Few, indeed, but the engineer devoting much time and effort in the field of promotion probably realize the additional factors to be confronted. To assert that many of the ethics of the profession of former days are fastly disappearing would, the writer believes, be no exaggeration; and, sad to say, in most cases there may be some necessity for it.

In the first place, whereas formerly the field of sanitary engineering was comparatively limited, to-day a dozen or more engineers are to be found at a gathering in some community of probably not more than a thousand population each endeavoring to obtain the same engineering contract, and it indeed may be considered a "survival of the fittest" as to whom the contract will eventually be awarded. Each of the twelve being given his chance in turn to appear before the city council to present his qualifications and terms for engineering service, it often happens that the man with the most flowery language, promises, and lowest price wins out and is awarded the job. Engineering ability may count for little, for copious letters of recommendation always appear from somewhere for each applicant. Many city councils could be told that water ordinarily flows up grade without disputing the matter, and often do not take too kindly to the engineer of well-educated bearing and speech who appears before them, but prefer the "engineer" of the practical workman type. In the State of Indiana, and undoubtedly in other states where registration of qualified professional engineers is compulsory, part of the above situation has probably been somewhat improved, although by no means altogether due to laxness in the registration requirements themselves. Nevertheless the prevalent practice to-day of the engineer bidding in his services to a considerable extent cannot be denied, insofar has it been the writer's observation on municipal projects, in each instance of which engineering firms both large and small were represented.

Secondly, the influence "material" men have been able to exert over the members of the city council, often long before the arrival of the engineer who is endeavoring to secure a contract for professional services, must be mentioned. It would seem that there are few companies to-day, who may be selling anything from a screw bolt on up to a steam shovel, who have not representatives traveling continually from one town to another endeavoring to promote some phase of municipal improvement in which their particular product may be used. Such endeavors are, for the most part, undoubtedly commendable, inasmuch as interest in municipal betterment is often thus created in communities where the engineer might never have prospected. The drawback, however, often is that whether the material man's particular apparatus or material should be used in any resulting municipal project from an engineering standpoint matters not, as he is out to sell his wares regardless of methods to be resorted to to do so.

From this it can be seen that much friction between the engineer, who has finally obtained a contract for his services, and the city council, who have been previously "wined and dined" by the material man, may at times result. Inasmuch as most material men also possess some technical knowledge as applied to the use of their own particular product, this, in combination with the above good fellowship stunt, has often served to exert undue influence over at least several members of the city council, and the engineer in such cases has no course but to yield to the wishes and dictates of the city officials over his own opinions and recommendations for the use of materials if he would hold his job. This must not be taken as general practice, however, but it would seem to occur far too frequently for the benefit of the profession.

On the other hand, quite frequently material men deserve far more credit than they obtain for their efforts and expense for municipal promotion before an engineer is engaged. Many have succeeded in creating a feeling for municipal improvements where none had ever existed before, and probably never would have existed were it not for their efforts. Many do play the game squarely, and if they find the engineer averse to the use of their product for reasons quite explainable and apparent, they withdraw taking their losses without murmur. These men, the writer believes, deserve the utmost consideration from the engineer who has obtained his contract primarily through their efforts, if he finds it at all possible that he can approve their materials, and at the same time in his own judgement serve the interests of the community to the utmost advantage.

Such are a few instances only of many positions in which the engineer may find himself situated. Whether there may be a remedy whereby the ethics of the engineering profession of old may be recovered and at the same time insure the bona-fide engineer a living commensurate with that of other professional men the writer is at a loss to say, but it is his frank opinion that a few more efforts for the welfare and uplifting of the profession as a whole by the several national engineering societies could well be exerted, coupled with some system of national or state registration free from politics.

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II. CO-OPERATION WITH STATE AND OTHER INSTITUTIONS

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It would seem worth while that a brief mention be made concerning the part that state and other institutions wish to take in the design of municipal waterworks and sanitary sewerage systems. Hardly need it be said that it is to the advantage of every engineer - and should be his earnest desire also - to meet the requirements of these institutions, and to solicit their co-operation whenever possible. In some cases such co-operation is necessitated by state law, yet in any event the writer, in his work in the three states of Missouri, Illinois, and Indiana, has yet to encounter any recommendations or requirements of these institutions that were not in accord with best engineering practice, whether the matter concern features of design, construction, or operation. The engineer should further bear in mind that every community is proud of the state in which it is located, and with the knowledge that its municipal improvements are being made with full approval of interested departments of state, its confidence and trust in his ability become great from the very beginning.

Briefly, these institutions, with their interests and jurisdiction, are as follow.

STATE BOARD OF HEALTH

The interests of this department of state are, of course, primarily concerned with the safeguard of public health through safety and sanitation in accordance with common practices of medical hygiene. Some of these have been intelligently extended to influence certain details of design of municipal waterworks and sanitary sewerage systems, and it is from this point of view that the state board of health requires that all plans and specifications be first submitted for their approval in order that none of these details may have been overlooked, and that maximum sanitary operation of the finished system or plant may be anticipated from the design. The engineer, in most cases, is further asked to submit a report embodying the following details in so far as may be possible.

<u>Waterworks</u>

1. Source of Supply

(a) Surface Supply -

A full discussion of the nature and extent of the watershed, with special reference to its sanitary condition, together with any proposed methods of regulation of pollution, should be given. Storage capacity, average depth, width, rate of flow, nature and area of reservoir, if any, character of raw water, etc. should also be discussed. If a river supply is under consideration, the course above stream within five miles of the proposed intake should be shown, together with a list of all cities on the watershed.

(b) Ground Supply -

If the water supply is to be taken from wells or infiltration galleries, the number, depth, size, and proposed method of construction of these should be given, together with any geological features of the region in which the well is to be drilled; the type of strainers to be used, probable capacity of the wells, and size of pumps should also be specified.

2. Purification Plant

If purification of the supply is proposed, information should be given showing the arrangement, size and construction of sedimentation basins, together with the method of introducing and mixing chemicals for coagulation, and the method of sterilization. If filters are proposed, details should be given showing size, type, arrangement, and capacity of units, together with methods of control and operation. Details of any provisions for laboratory control and testing of water are essential.

Sanitary Sewerage Systems

1. System -

Adequate information, from the basis of design, is asked on the following: population to be served, both present and future, estimated for twenty five years; estimated daily flow of sewage per capita, and total estimated daily flow; allowance for ground water infiltration; probable character of sewage; minimum and maximum grades of sewers of each size; provisions for flushing sewers.

2. Disposal Plant -

The following information is requested: location of plant, together with character and flow of stream receiving effluent, and nearest location of water supplies being taken from stream above and below plant outlet, if any; general method of disposal proposed, and basis of design upon which plant is expected to operate; method of disposal of sludge; provisions for secondary treatment of effluent, if any.

STATE PUBLIC SERVICE COMMISSION

The interests and jurisdiction of this second department of state in the matter of municipal waterworks and sanitary sewerage system construction varies in intensity in each of the three states heretofore mentioned, being probably most intense of the three in the State of Indiana.

The chief concerns of this department may be said to be that the interests of the community are properly taken care of in the awarding of the construction contract. To attain this end, all plans and specifications are gone over to see that no manufacturer's products have been unjustly discriminated against in the requirements of the engineer. By so doing, it is hoped that competitive bidding as unlimited as possible may be assured at the letting, thereby encouraging the fairest price possible to the community. It must not be understood from this that the engineer is prevented in any way from specifying the types of materials and apparatus he wishes to use, but that the practice of helping to swing the job for some particular manufacturer alone is to some extent thereby prevented. Such practice, the writer regrets to say, has at times seemed far too apparent to him on the part of a few "would-be" members of the profession. In addition, the engineer's estimate of materials and quantities for the job is also requested, as an aid in the checking up of bids submitted and the possible detecting of any "frame-up" of bids on the part of the contractors, and to insure that the contract may have been truly awarded to the lowest responsible bidder.

STATE INSPECTION BUREAU

An inspection bureau, whose business it is to see that the requirements of the National Board of Fire Insurance Underwriters are met for adequate fire protection of a community contemplating the installation of a waterworks system, is maintained by the fire insurance companies in each of the three states heretofore mentioned. Naturally the maximum possible fire protection and reduction in existing insurance rates is one of the biggest "talking points" and objectives for any community endeavoring to obtain the approval of its citizens for financing the cost of a proposed waterworks installation. Needless to say, therefore, it is to the interest of the engineer to co-operate with this bureau in its requirements in every way possible.

The requirements of this bureau being considerably more definite than those of other interested institutions the following are given in detail for the obtaining of a very desirable fire insurance rating, known as a "Class 4-1/2 (N.B. 8)" rating in the State of Missouri, It may be remarked that it has been the writer's practice to meet these requirements in design with equal rigidity in the States of Illinois and Indiana.

Waterworks

1. Source of Supply -

The source of supply shall be adequate and reliable at all times.

2. Capacity of Supply Works -

The total capacity, in excess of the maximum daily domestic consumption rate, shall be able to deliver a fire flow of not less than the following rate and duration, and at the pressures specified under Item 3, "Pressure", below.

Population	Fire Flow (Gals. per min.)	Duration
Up to 750	400	2 hours
750 to 1500	400	3 hours
1500 to 2500	400	4 hours
2500 to 3500	500	5 hours

For places larger than 3500 and up to 10,000 population, a fire flow of 600 gallons per minute for a 6 to 10 hour period is required, in excess of the maximum daily domestic consumption rate.

3. Pressure -

The flows specified under "Capacity of Supply Works" are to be maintained at not less than 50 pounds flowing pressure from fire hydrants in business and manufacturing sections in towns of less than 2500 population. In towns of more than 2500 population, the fire flows in these sections are to be maintained at not less than 60 pounds flowing pressure. In residential sections of single family dwellings, half the above flows are to be maintained at 50 pounds flowing pressure. Where a fire engine pumper is provided in the fire department, having a capacity of at least 300 gallons per minute in towns up to 5,000 population, and at least 500 gallons per minute capacity in towns over 5000 and up to 10,000 population, these flowing pressures may be reduced to 40 pounds in all cases.

4. Power -

To be adequate to operate all pumps at all times; transformers serving pump motors to be equal to total combined motor capacities, and to serve pump motors only.

5. Pump House -

To be constructed of brick or tile, with fire-proof roof covering.

6. Pipe Distribution System -

The pipes are to be adequate to deliver the flows at pressures above specified, but must be 6 inches or larger to supply fire hydrants in business and manufacturing sections, and not less than 4 inches in dwelling sections. In general, 4 inch dead-end mains should not exceed 600 feet in length, or 4 inch single loops exceed 1500 feet in length. If such adverse conditions obtain generally over town, at least a 300 gallon per minute fire engine pumper must be provided in the fire department. Gate valves shall be suitably located throughout the system so that parts of the system may be blocked off at any time.

7. Fire Hydrants -

All hydrants must have at least two 2-1/2 inch nozzles, threaded to the "National Standard" gauge; 4 inch or larger foot valves; 6 inch pipe connections to 6 inch or larger mains, and 4 inch connections to 4 inch mains. Fire hydrants shall be located not to exceed 300 feet apart in business and manufacturing sections, and not to exceed 600 feet apart in residential sections.

STATE GEOLOGICAL SURVEY

The interests of this department concern the obtaining of accurate log records for the purpose of compiling geological correlation of the underlying strata from all wells or test holes drilled within the state. As a result of the records so accumulated by the State Geological Survey of Missouri, much information is often at hand to guide the engineer as to the possibilities of obtaining a municipal well water supply of sufficient quantity and desirable quality in the particular location in which he may be interested within the State of Missouri. This service is freely and cheerfully given, and the engineer should reciprocate whenever possible by forwarding accurate and detailed drilling log records and cutting samples to this department whenever he may be in contact with such work. This department co-operates with the state board of health in its endeavors to insure all water supplies as free from contamination as possible, and it is well that its specifications covering the drilling, casing, and sealing of wells, and the testing of water be adopted whenever a well supply is under contemplation.

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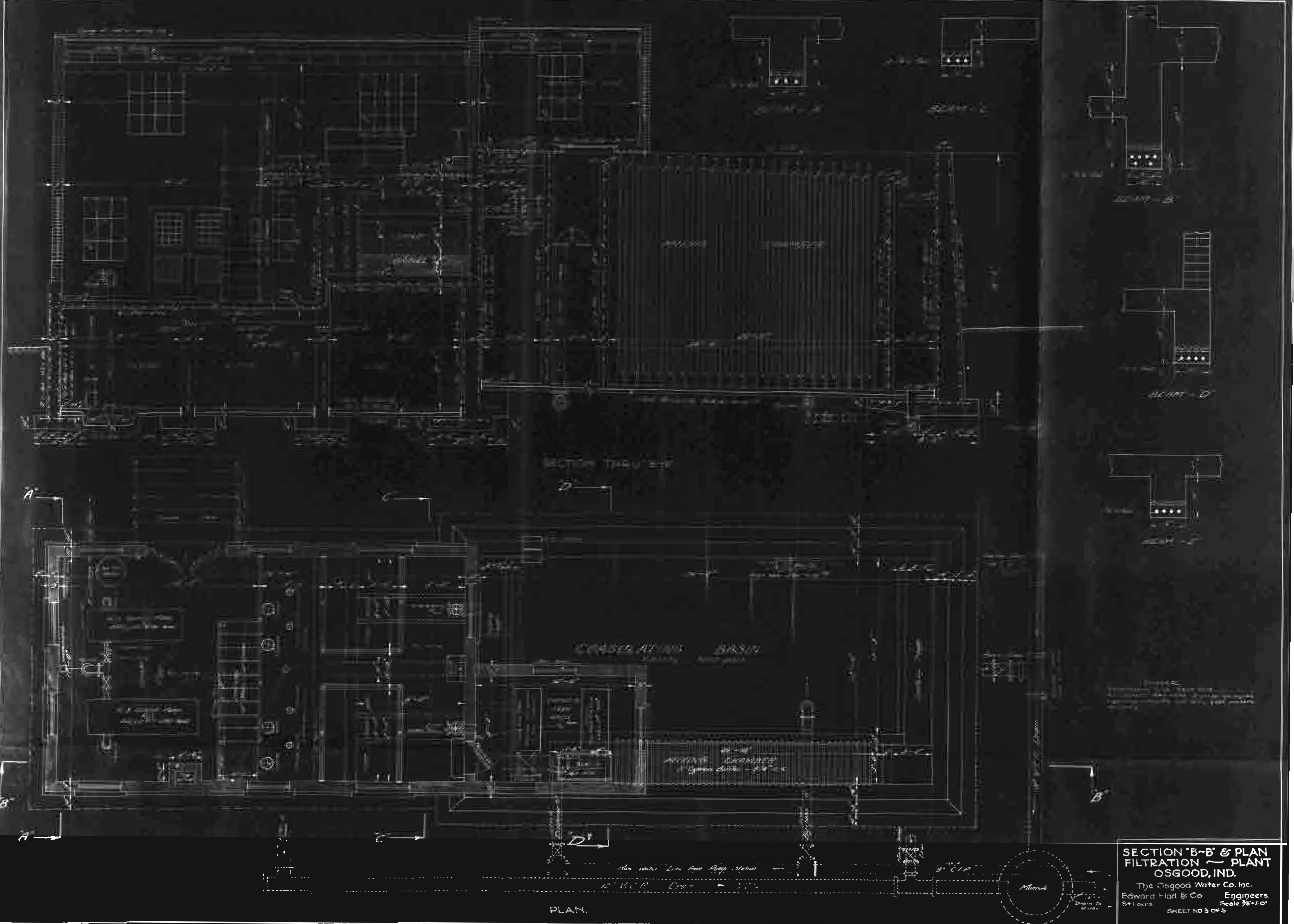
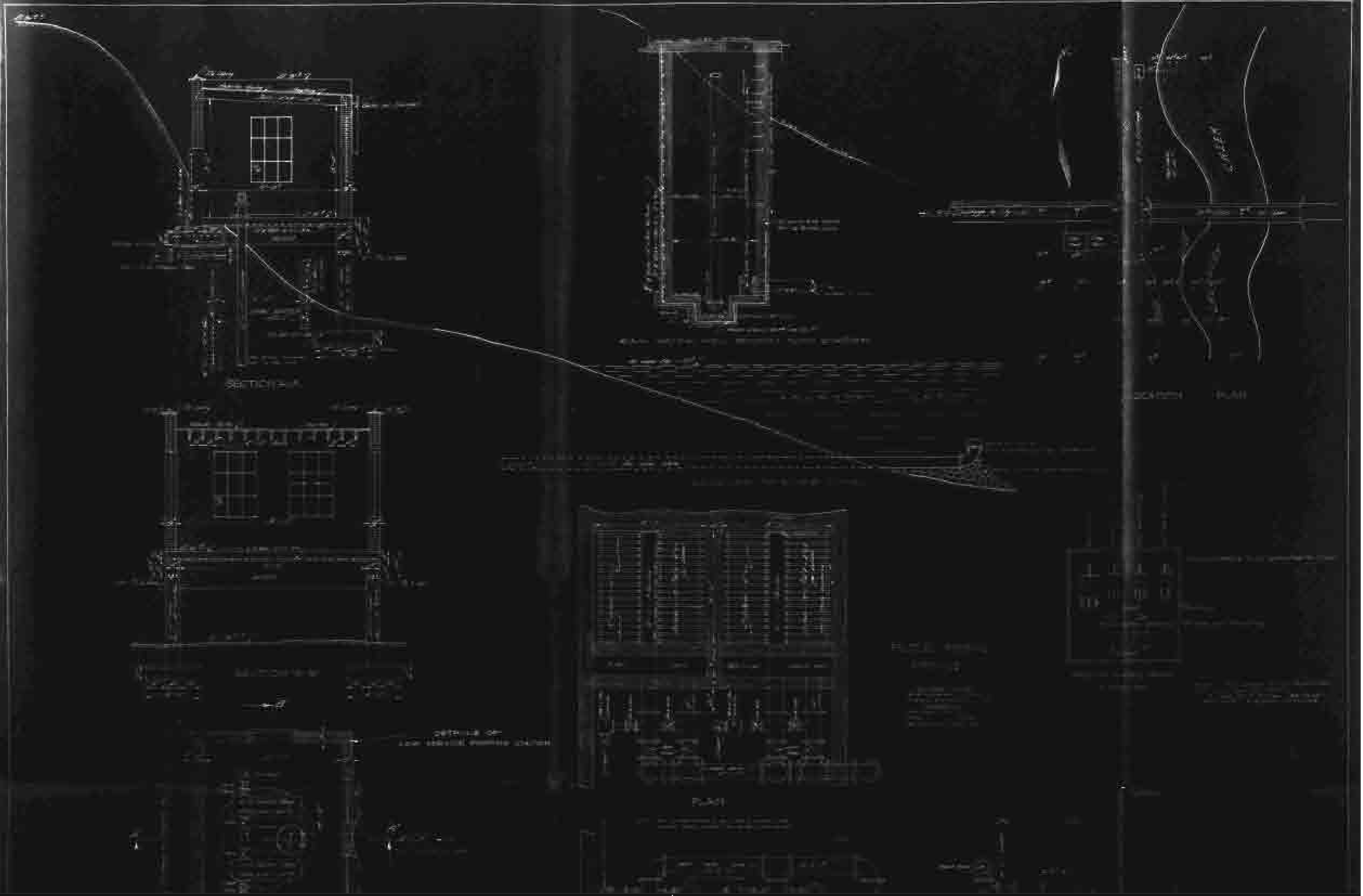


SECTION "B-B" & PLAN
FILTRATION — PLANT
OSGOOD, IND.

The Osgood Water Co. Inc.

Edward Flad & Co. Engineers.
St Louis
Scale 3/8"=1'-0"

SHEET NO 3 OF 5


SECTION A-A
DETAILS OF LOW SERVICE PUMPING STATION
PLAN

III. NOTES ON WATERWORKS DESIGN

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General

On August 29th, 1927, Edward Flad & Company, Consulting Engineers, St. Louis, Mo., received a contract from the Osgood Water Company for the complete engineering work comprising investigations, surveys, design, and supervision incidental to the construction of a complete waterworks system for the City of Osgood, Indiana. The writer, as associate engineer with the above company of engineers, had charge of the design, and submits herewith in brief, from his notebook as it were, some of the features upon which the design was based; with particular reference, however, to those conforming with the requirements of the various interested institutions discussed in Section II of this paper, "Co-operation with State and Other Institutions". By no means are the notes which follow submitted as representing particularly original or out of the ordinary points of design, the intent being merely to present the basis and data utilized in a particular instance which met the approval of the above mentioned interested institutions.

The City of Osgood, a community of some 1280 inhabitants, is located in the farming region of southeastern Indiana. Through it runs the main eastern line of the Baltimore and Ohio Railroad from St. Louis, and in addition to its farming interests, a wooden plug factory in its midst gives additional employment to many of its citizens. The topography of the city itself is generally flat, although located in the highest part of the surrounding country with the area dropping away sharply to the northeast where it soon becomes quite hilly. The top soil, consisting of a glacial sandy clay averaging some nine feet in depth, overlies a hard limestone formation which is approximately fifteen feet thick in this area. Underlying this, in layers averaging from four to six feet in thickness each, limestone strata alternating with a hard argillaceous shale, locally termed "Bluestone", are to be encountered.

Cost

The cost of the entire waterworks project, including all incidentals, amounted to approximately $75,000. Part of this amount was obtained from a waterworks fund already in existence through proceeds from the sale of a former light and power plant, the balance being raised through the sale of stock in the Osgood Water Company, a specially created municipal corporation, in accordance with the provisions therefor of the state laws of Indiana.

Source of Supply

It was decided that a surface supply could be utilized and be obtained from Laughery Creek, a stream located approximately two miles northeast of the city limits and flowing in a general direction from north to south. The sources of the creek are a natural watershed extending some twenty miles north of the located point of intake at Osgood, while from here the stream flows some twenty miles further due south, thence turning sharply to the east for another twenty miles discharges directly into the Ohio River.

The discharge of Laughery Creek in the vicinity of Osgood varies considerably. It is reported that in one very unusual period of drought the flow dropped to some probable fifteen second feet as a minimum, but that the average is many times greater than this, being some five thousand second feet to many times more during certain periods of the rainy season. There being no other communities located near the creek for at least ten miles in either direction from Osgood, possible pollution from sewage discharge into the stream could therefore be placed at a probable absolute minimum. A very low degree of turbidity of the water in this vicinity was also reported.

Purification Plant

General

The general operation of this plant provides for coagulation, sedimentation, filtration, and sterilization of the raw water. This is supplied to the plant from a low service pump house, located on the bank of Laughery Creek some 400 feet distant, by either of two centrifugal pumping units, having a capacity each of 250 gallons per minute, which takes the raw water directly from a "wet well" beneath the pump house, being fed through a ten inch cast iron pipe gravity line from the creek intake. Design and equipment of the low service pump house will be discussed later.

Capacity of Plant

The plant was designed for a maximum capacity of 300,000 gallons per 24 hour period, or 209 gallons per minute, under theoretical conditions of ideal operation. That this capacity could be increased to 250 gallons per minute without particularly affecting the quality of thetreated water will be brought out below. As may readily be seen, these capacity amounts are considerably in excess of any conceivable daily domestic consumption rate for a city the size of Osgood, even were a future population of 2000 inhabitants with a daily consumption of 75 gallons per capita - which would be exceptionally high - plus a 200% fluctuation factor during the period of peak consumption assumed. It is believed, however, that the above designed-for capacity, although considerably on the side of safety in almost any event, is not unnecessarily extravagant for a city of this size from the point of view of providing adequate fire protection in addition to maximum domestic consumption. This will be brought out later under a discussion on the distribution system.

Mixing Chamber

The raw water, upon reaching the plant from the low service pump house, is discharged upward through a small rectangular compartment where it receives a properly proportioned dose of lime and alum from dry feed machines above, and from here is passed through the mixing chamber. A discussion on the proportioning of these chemicals will be found later.

The mixing chamber consists of a series of cypress wood vertical baffles, two inches in thickness, so spaced and designed as to allow the desired quantity of water to flow and be mixed by passing between them - over one baffle and under the next - in a vertical direction. The design of the mixing chamber itself and of the spacing of the baffles was made in accordance with the following provisions:

1. Retention period for passing through chamber to be 15 minutes
2. Velocity in passing through chamber to be between 30 and 40 feet per minute
3. Maximum quantity to be designed for to be 250 gallons per minute

(a) Number of Baffles Required

Maximum length of path of travel, from above,

$$40 \times 15 \quad - \quad 600 \text{ lineal feet}$$

The depth of the chamber averaging 17.7 feet, (from design of coagulating basin), and the minimum length of vertical path of flow between baffles being 15.0 feet, the minimum number of baffles required is

$$\frac{600}{15} \text{ plus } 1 \quad - \quad 41 \text{ baffles}$$

(b) Spacing between Baffles

The width of the baffles being 3.0 feet (arbitrarily used), and assuming a passage of 250 gallons per minute, or 33.5 cubic feet, between baffles, the allowance per foot width of baffle becomes 11.2 cubic feet per minute.

From $Q = av$, with $Q = 11.2$, $v = 40$, $a = s \times 1$

spacing necessary, $s = \frac{11.2}{40 \times 1} = 0.28$ feet.

The baffles were actually spaced 3-1/4 inches apart in construction, or on 5-1/4 inch centers.

Coagulating or Sedimentation Basin

From the mixing chamber the treated water is allowed to spill quietly over a weir, 13.25 feet in length, at the extreme end of an open sedimentation basin or reservoir. This basin, 34.0 feet long by 13.25 feet wide by 17.8 feet average depth, allows a maximum quantity of 50,000 gallons of treated water to undergo sedimentation. This capacity is based on the allowance of a minimum retention period of 4 hours for the total plant capacity of 300,000 gallons per 24 hours. The correctness of such an assumption from the point of design may, of course, be somewhat open to question, owing to such variable factors as the possibility of short circuits and currents through the basin due to shape of basin, non-uniform weir flows, different specific gravity at various depths, and other factors impossible of determination. However, with the possibility of the plant operating to capacity being confined to rare occasions only, it is probable that the size of the basin as designed would provide a general retention period considerably in excess of the desired 4 hours. The basin in this case, however, was designed, and the weir from the mixing chamber so located, that a minimum amount of short circuiting is anticipated, a point very essential from the standpoint ofhighly successful operation.

Filter Units

From the coagulating or sedimentation basin the treated water passes over a second weir into a trough at the other extremity of the basin from which the water may flow by gravity through two separate valve intakes into the wash troughs of two independent rapid sand filter units. These units may be operated one at a time or both together, depending upon necessity; the intention being, however, that as a general rule one unit at a time only should be used.

(a) Size of Filter Units

The minimum desirable size of the units was obtained from an allowance of 2 gallons of filtered water per minute per square foot of filter area, plus a small allowance for wash water, or a total assumption of 365 square feet of filter area per million gallons of water filtered per 24 hours. Allowing each unit to be capable of filtering half the maximum plant capacity, or 150,000 gallons per 24 hours, the minimum area required per unit becomes 52.5 square feet. Inasmuch as it appears that there is a limit to the desirable minimum size for the construction of a filter unit, mainly from the standpoints of providing adequate working space for cleaning operations and other manipulations, it was decided to construct each unit 8 feet square; thus providing a filter area of 64.0 square feet per unit, which, of course, was well on the side of safety.

(b) Sand and Gravel

A depth of filter sand of 30 inches covering a layer of graded gravel 18 inches in depth was specified for each unit.

Specifications covering the sand provided that it be a hard, resistant, quartz sand, having not more than 2% of foreign material by weight. The effective grain size was limited between 0.35 and 0.45 millimeters, with no grain size over 0.45 millimeters being allowed; and a uniformity coefficient of less than 1.6 was specified. The effective size being that size which is coarser than 10% of the sand grains by weight; the uniformity coefficient being the ratio between the size such that 60% of the sand is finer than it and the effective size.

Specifications covering gravel provided that it be free from fine material and uniform in size to avoid disturbance in the washing process, and that it be carefully graded and laid in strict accordance with the following:

Bottom	layer	- 6	inches	deep	-	gravel	size	1 to 2 inches
Second	"	- 3	"	"	-	"	"	1/2 to 1 "
Third	"	- 3	"	"	-	"	"	1/4 to 1/2 "
Fourth	"	- 3	"	"	-	"	"	1/8 to 1/4 "
Top	"	- 3	"	"	-	"	"	1/8 "

(c) Wash Troughs

One wash trough, having a cross section 20 inches wide by 12 inches deep with an additional 3 inch depth of grooved channel, constructed of reinforced concrete, was provided for each filter unit; so suspended that the spillway edge was exactly 24 inches above the top of the sand. Allowing a maximum wash water rate of 15 gallons per minute per square foot of filter area, this would produce a crest 1-1/2 inches in depth of wash water over each of the two spillway edges of each trough. The maximum horizontal path of travel for any particle of wash water being one half the filter width less one half the width of the wash trough, or 3.22 feet in this case, the maximum recommended allowance of 3.5 feet was not exceeded, which has been so suggested in order to aid in counteracting the formation of mud balls in washing as much as possible.

(d) Filter Underdrain System

Beneath the gravel layer the filtered water is collected by a system of underdrains consisting of a series of perforated cast iron pipes, or "orificed laterals", spaced on 6 inch centers, which connect to "Tee" projections similarly spaced on each side of a larger central cast iron pipe, or collecting manifold, running parallel to and directly beneath the wash trough above, the whole resting horizontally on the concrete bottom of each unit. The design of this underdrain system is based on the following factors:

1. Number of Laterals Required

An arbitrary spacing of 6 inches between each lateral was taken. There being one lateral on each side of the manifold for each 6 inch spacing, and the length of each unit being 8.0 feet, the total number of laterals required for each filter unit becomes $\frac{8 \times 2}{0.5} = 32$

2. Length of Laterals

Assuming a 12 inch diameter manifold, the calculations for which are shown below, and allowing for pipe thickness and lateral clearance, a length per lateral of 3.25 feet was specified.

3. Number and Size of Lateral Orifices

It seems general practice now that the total orifice area per unit be taken as from 0.20% to 0.30% of the filter area, and from this a factor of 0.25% was chosen:

$$0.25\% \text{ of } 64.0 \text{ sq. ft.} = 0.16 \text{ sq. ft.}$$
$$= 23.04 \text{ sq. ins.}$$

The total orifice area per single lateral then becomes

$$\frac{23.04}{32} = 0.72 \text{ sq. ins.}$$

19 orifices, spaced on 2 inch centers, each 7/32 inches in diameter, giving a total area of 0.718 square inches, were, therefore, specified for each lateral.

4. Size of Laterals

General practice recommends that the area of each lateral be at least 3 times the area of its orifices;

$3 \times 0.718 =$ an area of 2.154 square inches per lateral. The nearest theoretical size would, therefore, be a 1-11/16 inch diameter cast iron pipe, whose area equals 2.237 square inches. Inasmuch as this is not a standard size in cast iron pipe, it was decided to use a 2 inch diameter pipe, area = 3.14 square inches, which, of course, would be well on the side of safety.

5. Size of Manifold

The recommended size for a filter manifold is that it be at least 1.5 times the total theoretical area of the laterals in the unit. In this case this would be -

$$2.154 \times 32 \times 1.5 = 103.2 \text{ square inches}$$

A 12 inch diameter cast iron pipe manifold was, therefore, specified, the area being 113.1 square inches.

Pipe Gallery

From the filters the water passes out through the manifolds into the pipe gallery, where it runs through rate of flow controllers to the inverted double elbow discharge pipes into the clear water well underlying the main body of the filter plant, the well having a total storage capacity for 28,400 gallons of filtered water. A detailed description of the piping and valve arrangements of the gallery will not be attempted, but the following accessories may be noted:

1. Rate of Flow Controllers

A rate of flow controller of the Venturi type, with limits between 100,000 and 200,000 gallons per 24 hours and set at 150,000 gallons, was provided for each unit.

2. Loss of Head Gauges

A manometer type loss of head gauge, reading from 0 to 10 feet, for determining the limiting loss of head in the passage of the water through the filters before washing becomes necessary was installed for each unit.

3. Depth Gauge

A depth gauge for indicating the depth of filtered water in the clear water well at all times was also installed.

Provisions for Washing

For reversing the flow through the filters in order to remove the accumulated silt and other foreign matter out of each filter, no special wash water pump was deemed necessary in this case. It was decided instead to wash directly from connection to the 8 inch city service main running from the plant, operating at the same time the 500 g.p.m. high service pumping unit. In this way adequate capacity and pressure for washing could be assured at any time.

Assuming a maximum wash water requirement at the rate of 15 gallons per square foot of filter area per minute, or a total of 960 gallons per minute per unit - which would give the desirable vertical velocity of 2 feet per minute through each filter - it was estimated that this total flow could be thus obtained from the main at a pressure as high as 100 pounds per square inch if necessary. Such a pressure is, of course, far too excessive for filter washing, but it is evident that the source is satisfactory, and by means of careful valve manipulation could be limited very satisfactorily to the desired quantity and pressure.

Adequate provisions were made, of course, for the removal of wash water from each unit through a special pipe drain emptying into a lower part of Laughery Creek.

Laboratory Control and Testing of Water

Explanatory

By no means is the following to be considered an attempt on the writer's part to set forth the many methods of analysis and theory for the proper treatment of water with chemicals in filtration plant operation. The few tests and tables which follow have been compiled in part by the writer, and have been successfully used by him in the field where the use of the chemicals, alum, lime, and chlorine alone was concerned.

General

By treatment of the raw water as it enters the plant with the chemicals lime and alum, a way is found to collect together by coagulation the particles of sand, clay, and organic matter which render the raw water muddy or turbid. The above particles, being for the most part in a state of suspension, are collected together in the process to form masses or larger particles of such size that they will readily settle to the bottom of the sedimentation basin during the given detention period of four hours. To remove the finest of these suspended particles, the chemicals are further responsible for the presence of a gelatinous covering and mass on and within the sand filters themselves which prevents the passage of these particles through the filters and further aids in the production of a clear grade of water.

Notes on Theory of Coagulation

$$\underset{\text{Alum}}{Al_2(SO_4)_3} + \underset{\text{Lime}}{3Ca(OH)_2} = \underset{\text{"Floc"}}{Al_2(OH)_6} + 3CaSO_4$$

A full explanation of the factors influencing the proper coagulation of the suspended matter in raw water by the introduction of lime and alum would require a lengthy discussion involving much chemical theory, and will not be attempted here. The following may give a fair idea of the reactions involved, however.

When aluminum sulphate, commonly termed "alum", is added to water having a sufficient degree of alkalinity, a jelly-like invisible mass or precipitate is formed. By means of thorough mixing and agitating, this mass may be broken up into fine "pin-head" flakes constituting a coagulum or "floc". This "floc" has the property of causing small particles of clay, silt, organic matter, etc., with which it may come in contact in the raw water, to coagulate or collect together, and precipitate or settle. The maximum precipitation possible of these particles in the sedimentation basin before the raw water reaches the sand filters is the end to be constantly sought after for the successful operation of any filtration plant, and the final obtaining of a good clear water. The eventual presence of the "floc" in the sand of the sand filters is also highly essential, as explained above, and the success of the operation of the filters depends almost entirely upon this presence.

Tests

In the attempt to obtain the above mentioned maximum possible "floc", it is of course necessary that certain tests be made from time to time to determine the character and certain constituents of the raw water which will influence the proportioning of the lime and alum quantities. The following tests have been used by the writer, and are obtained from "Standard Methods of Water Analysis", 1925 edition, American Public Health Association.

1. Degree of Turbidity

This is determined by means of a turbidimeter, and is expressed in parts per million, as fully described on page 6, "Standard Methods of Water Analysis".

2. Degree of Alkalinity

This is determined by titrating 100 c.c. of the sample water with N/50 sulphuric acid, using

4 drops of methyl orange as an indicator; the indicated alkalinity being equal to the number of c.c. of the acid multiplied by 10, expressing the result in parts per million. The procedure is fully described on page 33, "Standard Methods of Water Analysis".

(Note: Titration, using phenolphthalein as an indicator, was omitted by the writer inasmuch the presence of bicarbonates only was indicated in the raw water on each occasion of his experience.)

3. Free Carbon Dioxide Content

This is determined by titrating 100 c.c. of the sample raw water with N/44 sodium hydroxide, using 10 drops of phenolphthalein as an indicator; the indicated free CO_2 being equal to the number of c.c. of sodium hydroxide multiplied, expressing the result in parts per million. The procedure is fully described on page 36, "Standard Methods of Water Analysis".

Chemicals

1. Alum

The amount of alum to be used depends almost entirely upon the degree of turbidity of the raw water, and it is very essential that a turbidity test of the raw water from the low service pump discharge be taken at least daily to determine this desired amount. Not only is the daily regulation of the amount of alum being used necessary from the standpoint of proper filter operation, and the obtaining of a good grade of water, but it will be found to have considerable bearing on the economic cost of plant operation. The table given later shows the suggested amounts of alum to be used for water of varying degrees of turbidity. It may be noted here, that the amounts suggested in this table may seem considerably in excess of those suggested by the various text book authors, but it is the writer's opinion that they are far from excessive for a plant of this nature, and in this opinion he has the approval of others, amongst whom might be mentioned Mr. E. S. Clark, Chief Chemist and Bacteriologist of the State Board of Health of Illinois.

2. <u>Lime</u>

The success of the amount of alum being used to produce the maximum possible "floc" depends also to a considerable extent upon the degree of alkalinity of the raw water. It may be estimated that for each grain of alum per gallon, an alkalinity of 10 parts per million is necessary. Should the second chemical test above, the alkalinity test, show that the raw water is deficient in alkalinity content, lime must be added to the water to make up this deficiency. The amount of lime to be added is given in the tables below for various amounts of alum and alkalinity content.

The presence of free carbon dioxide (CO_2) in the raw water is particularly objectionable inasmuch as not only does it affect the process of coagulation but it is responsible for considerable corrosive action within the service pipes. Free CO_2 can be eliminated by the addition of an additional amount of lime. Having determined the amount of free CO_2 present, by means of the third chemical test given above, the additional quantity of lime to be added may also be determined from the tables below.

Chemical Feed Machines

In the installation at Osgood two chemical feed machines of the dry feed type were used; one machine for feeding alum, another for feeding lime. The chemical from each machine being then automatically put into solution with pure water is carried through a small pipe to the mixing chamber, where it encounters the incoming raw water as previously described. The feeding from these machines is automatically and accurately controlled in each case through regulation of the speed of revolution of a small cog wheel. The wheel being actuated by a pawl for a definite number of times each minute, the number of cogs or "notches" on the wheel over which the pawl is allowed to climb for each actuation determines its speed of revolution. Assuming, therefore, an increase in the setting of the machine of one "notch", this would mean that the length of travel for each actuation has been increased by the space of one cog, resulting in an increase of travel of the wheel during one minute equal to the number of actuations per minute times one notch. Most machines having from ten to fifteen actuations per minute, an increase of one notch would represent an additional space of travel for the wheel of from ten to fifteen notches per minute.

The above explanation is given in detail to show that the fineness of regulation of chemicals with these machines is by no means all that may be desirable, an increase in the setting of one notch in each case being responsible for an increase of more than one grain of chemical per gallon of water. It was necessary, therefore, in the writer's preparation of the following tables for use at Osgood to specify settings of the machines in each instance which would at least insure the use of the minimum desirable amount of chemical. For instance, it will be noted in the alum table that for raw water with a turbidity of 200, a setting of the machine at 3 notches is specified, whereby a dose of 3.70 grains of alum per gallon is obtained. The writer ordinarily would like to limit the necessary dose to 3.0 grains per gallon, but a setting of 2 notches would not allow this so that the higher setting must be used.

Tables

I. Alum Table

Turbidity of Raw Water (parts per million)	Alum Dose (grs. per gallon)	Setting of Machine (based on 250 gpm raw water discharge)	Alum (lbs. per 1000 gals.)
Less than 100	1.23	1 notch	0.176
100 - 150	2.46	2 notches	0.351
150 - 300	3.69	3 "	0.527
300 - 400	4.93	4 "	0.704
400 - 500	6.17	5 "	0.881
500 - 600	7.39	6 "	1.056
600 - 1000	8.64	7 "	1.234

II. Lime Tables

(a) Desirable Alkalinity for Various Alum Doses

Alum Dose (grs. per gallon)	Desirable Alkalinity (parts per million)
1.23	22
2.46	34
3.69	47
4.93	59
6.17	72
7.39	84
8.64	97

(b) Quantity of Lime for Alkalinity Deficiency

Alkalinity Deficiency (parts per million)	Lime (grs. per gal.)
10	0.35
15	0.53
20	0.70
25	0.88
30	1.05
35	1.23
40	1.40
45	1.58
50	1.75
55	1.93
60	2.10
65	2.28
70	2.45
75	2.63
80	2.80
85	2.98
90	3.15
95	3.33
100	3.50
105	3.68
110	3.85

(c) Additional Lime to Neutralize Free CO_2 Content

CO_2 (parts per million)	Lime (grs. per gal.)
1	0.05
2	0.10
3	0.15
4	0.20
5	0.25
6	0.30
7	0.35
8	0.40

(d) Setting of Lime Machine

Setting of Machine (based on 250 g.p.m. raw water discharge)	Lime Used (grs. per gal.)	Lime Used (lbs. per 1000 gal.)
1 notch	1.055	0.150
2 notches	2.095	0.299
3 "	3.150	0.450
4 "	4.185	0.598
5 "	5.250	0.750
6 "	6.300	0.9000
7 "	7.340	1.048
8 "	8.400	1.200

Example in Use of Tables

Assuming water analysis as follows:

Turbidity - - - 400 p.p.m.

Alkalinity - - 25 p.p.m.

Free CO_2 - - - 5 p.p.m.

what should be the setting of the dry feed machines?

Alum Machine

Table I, Alum Table, shows -

turbidity 400, alum dose 6.17 grains per gallon;

set machine at 5 notches

Lime Machine

Under Table II, find -

alum dose, 6.17 grains per gallon requires

alkalinity of 72

Above analysis shows alkalinity present 25

Therefore, alkalinity deficiency

72 - 25 47

Under Table II, (b), for alkalinity deficiency of 47, by interpolation we find lime required equals 1.65 grains per gallon

Under Table II, (c), for CO_2 content of 5 p.p.m., we find additional lime needed equals 0.25 grains per gallon

Therefore, total lime necessary becomes

1.65 0.25 1.80 grains per gallon

Table II, (d), shows machine set at 2 notches feeds 2.10 grains per gallon which is satisfactory.

Sterilization of Water

General

Sterilization of the filtered water is provided for by means of a properly operated and automatically controlled "chlorinator" which injects a solution of chlorine gas into the suction side of the high service pumps. Inasmuch as chlorine enters into such reaction with the water that pathogenic bacteria present are killed in a very short time, it can be applied to the mains very shortly before the water reaches the consumer.

Quantity of Chlorine Necessary

The amount of chlorine necessary for sterilization depends upon the character of the water. However, in general, it must be said that not less than 0.22 parts per million is required to insure safe and satisfactory sterilization at all times. This is equivalent to .00184 lbs. per 1000 gallons, or .0127 grains per gallon.

Assuming 250 gallons per minute are being pumped into the mains, the chlorinator pulsations should be adjusted by the chart to be delivering 250 x .0127 = 3.175 grains of chlorine per minute.

Determination of Chlorine Content

The proper application of chlorine for sterilization of water is far more essential than any other operation at the plant and it is very advisable to add to the three chemical tests already mentioned one more, which is exceedingly simple, for the determination of the amount of chlorine being injected into the service mains. In this way a check is obtained on the action of the chlorinator.

To 100 c.c. of filtered water obtained from the laboratory faucet add 1 c.c. of ortho-tolidine solution; let stand for 5 minutes and then compare the resulting color of the water with the standard bottled chlorine comparator colored solutions. The bottles are labeled ".15", ".2", ".25", ".3", ".4", ".5", and these numbers represent residual chlorine content in parts per million.

Supposing the above 100 c.c. sample when 1 c.c. of ortho-tolidine is added shows a yellowish color exactly the same as that in the bottle labeled ".25", that would mean a residual chlorine content of .25 parts per million was present. The above dose of .00184 lbs. of chlorine per thousand gallons, or .22 parts per million, should compare between the bottles marked ".2" and ".25" in the comparator set.

Pumping Equipment

General

Four electrically driven centrifugal pump units, more particularly described below, were installed in connection with the filtration plant. Two of these are for "low service" use, supplying raw water from the intake "wet" well to the head of the mixing chamber; two for "high service" use, taking filtered water out of the clear water well beneath the filtration plant for the distribution system supply.

Before proceeding further, the writer is taking the liberty of a slight digression with the following remarks concerning the selection of the pump units in this particular case. That many very estimable makes of centrifugal pumps are to be found on the market these days is a fact that cannot be disputed; and assuming adequacy as to capacity, size, and reliability of operation, the final selection as far as the engineer is concerned simmers down to the question of greatest economy of operation under the given conditions of operation. In this latter respect, no two manufacturer's pumps are to be found entirely alike; one bettering the other under certain conditions of capacity and head, and vice versa under those slightly different. Nevertheless it is regrettable that selections are made far too often on the basis of first cost and the personal elements involved, rather than from the viewpoint of maximum efficiency of operation, assuming there is no choice as far as the mechanical construction of the pump is concerned.

That such influences prevailed in the final selection of the pump units for Osgood was very much apparent, inasmuch as personal motives only against the manufacturer whose units showed the highest guaranteed efficiencies for the required conditions of operation were responsible for the rejection of his equipment by the Osgood Water Company, all efforts and advice of their engineers being to the contrary and of no avail. The efficiencies of the pump units finally selected were particularly low, being on the average some thirteen per cent lower than those of the equipment recommended.

Pump Units

Low Service

Duplicate units each consisting of a 250 g.p.m. single stage centrifugal pump, operating against a maximum total head of 52.7 feet with a guaranteed efficiency of 72% overall at full load, having a 3 inch discharge, directly connected to a 7.5 H.P. squirrel cage motor operating at a speed of 1750 R.P.M. under alternating current characteristics of 440 volts, 60 cycles, 3 phase, with magnetic switch push button starters, were installed. These units were housed in a small pump house, 10.0 feet by 14.0 feet floor space, of brick and concrete construction, located some 300 feet from the filtration plant on the bank of Laughery Creek, and took their supply from a reinforced concrete "wet" well, 6.0 feet in diameter, located directly beneath, which was fed by gravity from a cast iron pipe, 10 inches in diameter, extending some 50 feet out through rock to the river.

High Service

One unit consisting of a 500 g.p.m., 5 inch discharge, double stage, centrifugal pump, operating against a total maximum head of 390 feet - 247 feet static, 133 feet friction, 10 feet suction - having a guaranteed overall efficiency at full load of 62%, and being directly connected to a 100 H.P. squirrel cage motor operating at a speed of 1750 R.P.M. under alternating current characteristics of 440 volts, 60 cycles, 3 phase, with manual control compensator, was installed to meet fire protection requirements and as an auxiliary unit to the one below upon certain occasions only.

A second unit consisting of a 250 g.p.m., 3 inch discharge, triple stage, centrifugal pump, operating against a total maximum head of 296 feet - 247 feet static, 39 feet friction, 10 feet suction - having a guaranteed overall efficiency at full load of 55%, and being directly connected to a 40 H.P. squirrel cage motor operating at a speed of 1750 R.P.M. under alternating current characteristics of 440 volts, 60 cycles, 3 phase, with manual control compensator, was installed for everyday domestic consumption requirements.

It should be noted here that the system was designed for the operation of the high service pumps as separate units only, and upon no occasion whatever were they to be operated together.

Power

Power, obtained from the local public utilities company, was supplied to the plant at 2300 volts, where it was stepped down through three 37.5 K.V.A. transformers to 440 volts; this transformer capacity being adequate to take care of the maximum combination load, based upon the following data:

Two 250 g.p.m. L.S. pumps, B.H.P. 5.7 each
One 250 g.p.m. H.S. pump, B.H.P. 34.0
One 500 g.p.m. H.S. pump, B.H.P. 79.4
Two dry feed machine motors, 0.25 B.H.P. each
Three 3000 watt coil heaters, 220 volts
Eleven 60 watt, 110 volts, lamps

Distribution System

General

From the filtration plant an 8 inch cast iron pipe service main was run to the northeast corner of the city limits, a distance of 12,700 feet, where it was connected to a network or "gridiron" distribution system of 6 inch and 4 inch cast iron pipe serving the entire city. The system was laid out in strict accordance with the suggestions of the National Board of Underwriters, previously discussed under Section II of this paper. Additional loops of 2 inch cast iron pipe for domestic supply connections only were provided for those consumers located within fire protection limits but at some slight distance from the mains. Pressure was maintained and a source of supply provided by means of a 75,000 gallon hemispherical bottom steel tank, elevated on a steel tower 100 feet above ground to the balcony surrounding the tank - a height of 116 feet above ground to the top of the tank - the base of the tower being located on the highest ground elevation within the city, and approximately in the center of the distribution system. Fire hydrants were located and conformed in detail of construction to the specifications of the National Board of Underwriters mentioned above. Valves were provided throughout the system so that any part of the network could be cut off separately at any time. The system as laid out provided for service of the entire city with the exception of approximately 12 residences widely scattered along extreme boundaries of the city limits.

Pipe Lines

All pipe lines were of cast iron pipe with bell and spigot ends having pre-caulked lead joints, and consisted of 8 inch, 6 inch, 4 inch and 2 inch sizes. A working pressure of 150 lbs. on all pipe was guaranteed by the manufacturers, with the exception of 10,000 feet of 8 inch pipe from the filtration plant which was of heavier construction and guaranteed to withstand a working pressure of 250 lbs.

Throughout the system four "dead end" lines only were laid, each of these being a 4 inch line terminating at a fire hydrant, the lengths being 450, 350, 310 and 300 feet respectively from the main.

Fire Hydrants

Fire hydrants were located not to exceed 300 feet apart in business sections and not to exceed 600 feet in residence sections.

The following two hydrants were estimated as giving minimum pressure conditions, under the assumption of water supply from the elevated tank only.

1. Business District

A fire hydrant fed from a 6 inch main located at the intersection of Ripley and Buckeye Streets and being by chance the "business district" hydrant most remote from the elevated tank, was estimated to have a flowing pressure of 48.5 lbs. for a flow of 400 gallons per minute at the hydrant. A friction head loss between tank and hydrant of 9.5 feet was included for a supply to be obtained in both north and south directions from the tank through all intermediate lines and loops available.

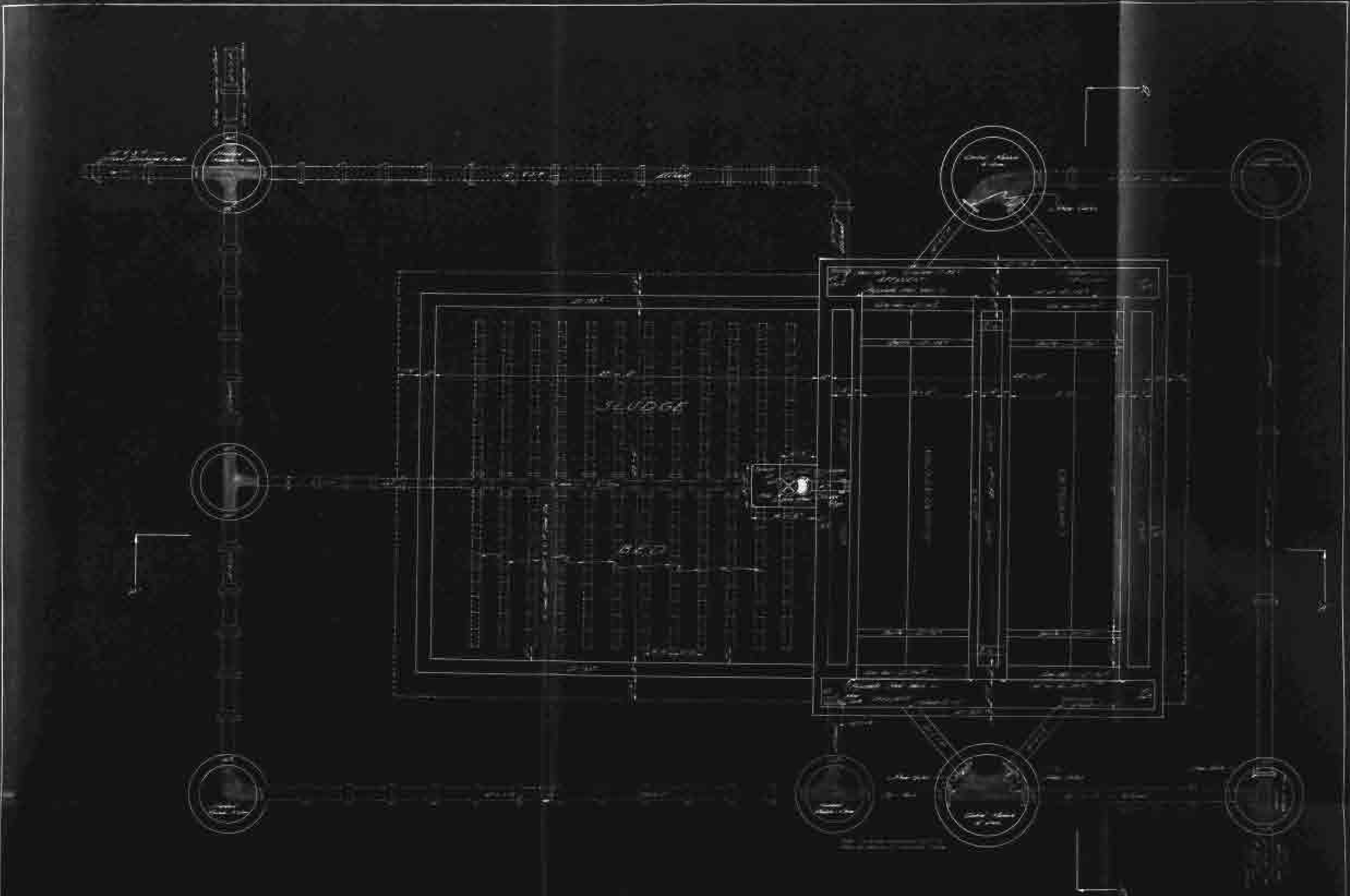
2. Residence District

A fire hydrant "dead-ended" from a 4 inch main located the farthest east on Ripley Street, and being by chance the "residence district" hydrant most remote from the elevated tank, was estimated to have a flowing pressure of 48.2 lbs. for a flow of 200 gallons per minute at the hydrant. A friction head loss of 20 feet was included for a supply to be obtained in both north and south directions from the tank through all intermediate lines and loops available.

Elevations

It might be of some interest that the following few ground elevations, from an assumed datum, be noted herewith.

Normal water in Laughery Creek	-	Elevation	70.7
Raw water intake in Laughery Creek	-	"	63.0
Ground at filtration plant, average	-	"	98.5
Ground at tank and tower site in city	-	"	225.0
Ground at above "business district" hydrant	-	"	220.0
Ground at above "residence district" hydrant	-	"	210.0



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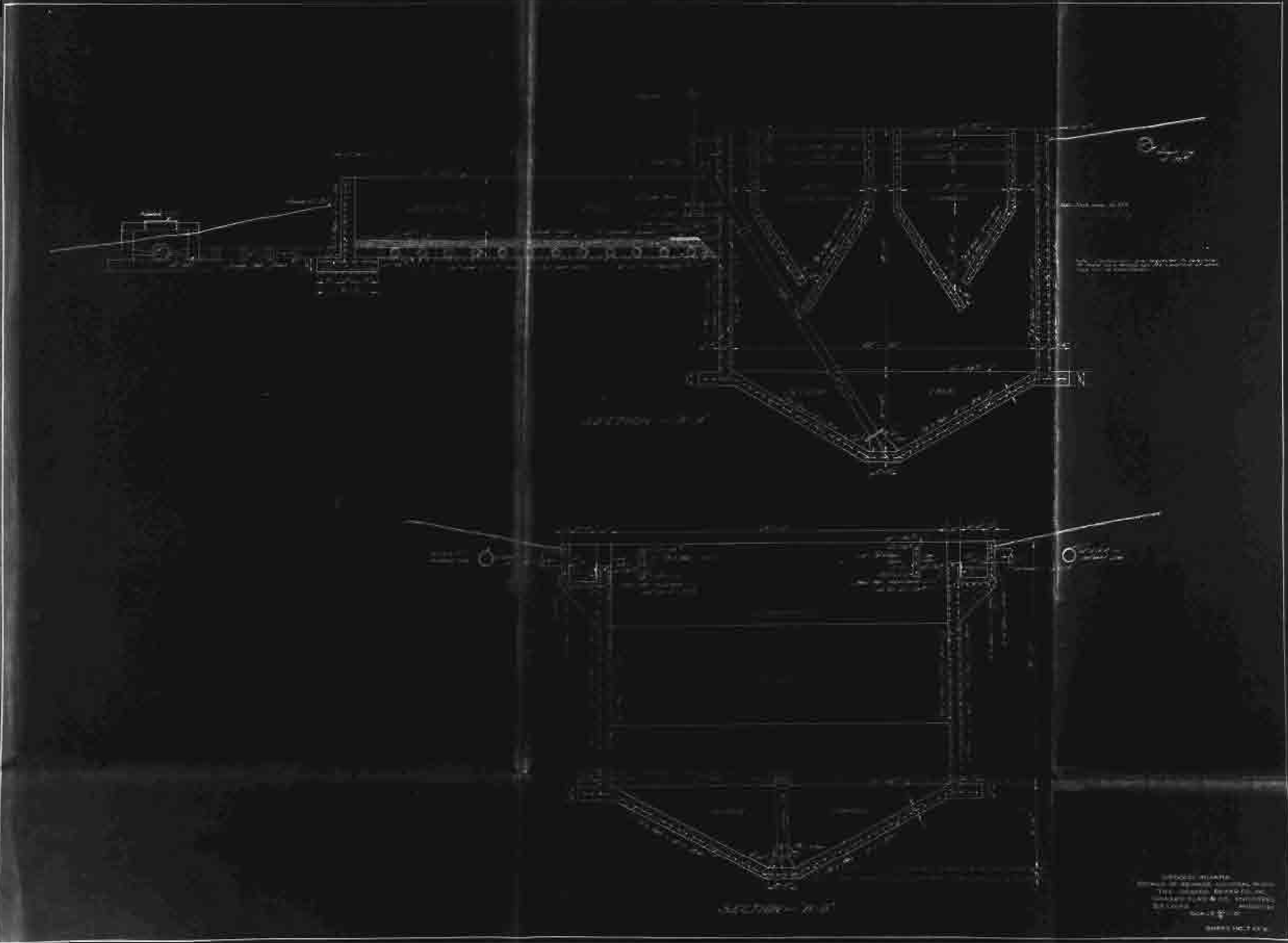

IV. NOTES ON SANITARY SEWERAGE SYSTEM DESIGN

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General

In December, 1927, Edward Flad & Company received a contract from the Osgood Sewer Company, a second municipal corporation for this city specially created under the provisions of the state laws of Indiana therefor, for the complete engineering work comprising investigations, surveys, design, and supervision incidental to the construction of a complete sanitary sewerage system for the City of Osgood, Indiana. The writer had charge of the design, and it is hoped that the following brief notes submitted in connection therewith may not seem to be entirely too elementary as to prove uninteresting. It might be mentioned that many of these notes were incorporated in the writer's report to the State Board of Health of Indiana upon completion of the design, and that the whole received full approval of the board. The system as a whole may be considered as representing a simple problem in sanitary sewerage system design with no particularly complicated methods of sewage discharge or disposal involved.

Cost

The cost of the entire sanitary sewerage system project, including all incidentals, is estimated at $40,000.00. This has been raised through the sale of stock in the Osgood Sewer Company.

Design of Sewers

Population to be Served

The present population of Osgood is estimated at 1280 inhabitants. The system has been designed to be capable of serving a population of 2000 to 2500 inhabitants if necessary.

Estimated per Capita daily Flow of Sewage

The per capita daily flow of sewage was assumed as a maximum possibility equal to a maximum daily discharge of 75 gallons per day. Such an assumption is undoubtedly extremely high for a city of this size.

Estimated Flow of Sewage for Design

It is undoubtedly apparent that were the total estimated daily flow of sewage from the entire area under consideration reduced to gallons per minute or cubic feet per second without taking into consideration additional factors such as maximum periods of fluctuation of sewage discharge, or allowance for ground water infiltration, much error in design of size of sewers might easily result. Considerable speculation and formulae as to the possible limits of these factors have been advanced by various authors for the purpose of safe design, resulting in a great variance of suggestions for the guidance of the engineer. It may be noted here that inasmuch as the waterworks for Osgood were not completed at this time no observations on periods of maximum water consumption, which might have been used to advantage in estimating possible maximum sewage discharge, were possible.

In view of such possible variations in the periods of maximum sewage discharge, the writer presents with some timidity the following table he has prepared for use in design and which to him has seemed reasonably well on the side of safety from all observations he has been able to make. By no means, however, is this table intended as a substitution for more accurate data to be obtained from actual observations in the field whenever possible.

Table 1. Quantity of Sewage for Design

Maximum quantities of sewage to be provided for due to periods of fluctuation, varying from 50 times the average estimated discharge for 10 persons or less to twice the amount for population of 1000 or more. Ground water infiltration not included.

Based on average Sewage Discharge of 50 gallons per capita per day

Population	- Maximum Discharge to be provided for -	
	Gallons per 24 hours	Gallons per minute
10 or less	25,000	17.4
50	28,000	19.5
100	31,000	21.5
150	34,000	23.6
200	37,000	25.7
250	40,000	27.8
300	43,000	29.9
350	46,000	31.9
400	49,000	34.1
450	52,000	36.1
500	55,000	38.2
550	58,000	40.3
600	61,000	42.3
650	65,000	45.2
700	70,000	48.6
750	75,000	52.1
800	80,000	55.6
850	85,000	59.0
900	90,000	62.5
950	95,000	66.0
1000	100,000	69.3

Based on average Sewage Discharge of 75 gallons per capita per day

Population	Maximum Discharge to be provided for	
	Gallons per 24 hours	Gallons per minute
10 or less	37,500	25.3
50	42,000	29.1
100	46,500	32.3
150	51,000	35.4
200	55,500	38.5

Population	Maximum discharge to be provided for	
	Gallons per 24 hours	Gallons per minute
250	60,000	41.7
300	64,500	44.8
350	69,000	47.9
400	73,500	51.1
450	78,000	54.2
500	82,500	57.3
550	87,000	60.4
600	91,500	63.6
650	97,500	67.7
700	105,000	72.9
750	112,500	78.2
800	120,000	83.3
850	127,500	88.5
900	135,000	93.8
950	142,500	99.0
1000	150,000	104.2

For population up to 3000, use 200% of estimated daily discharge

For population over 3000, use 175% of estimated daily discharge

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Allowance for Infiltration of Ground Water

Specifications covering the construction of joints for vitrified sewer pipe used in this work required that the following ground water infiltrations be not exceeded in the final testing of the system:

Size of Pipe	Allowable Infiltration
8 inch	1500 gallons per mile per 24 hours
10 inch	2000 " " " " "
12 inch	2500 " " " " "

From these figures it was estimated that the maximum allowable flow due to ground water infiltration over the entire system as designed for Osgood should not exceed 5.09 gallons per minute.

It is realized, however, that the ground water infiltration after a few years service may be increased to many times the initial allowable amount, and in view of this an assumption of a probable increase to 10 times the above amount, or 51 gallons per minute, was used. Whereas it is quite possible that this assumption of 10 times the initial allowable amount may be greatly exceeded at some future date, however, it would seem that some limit from the standpoint of design should be drawn somewhere.

The writer would venture to express an opinion of his own here that the impossibility of attempting to apply mathematical formulae for the infiltration of ground water into sewer joints cannot be overstated. The many factors involved, particularly the topographic, geologic, and climatic conditions of the area, together with methods of joint construction, and materials used, are undoubtedly variable beyond any mathematical comprehension. Nevertheless, the writer has noted several instances, one amongst the engineers of a state board of health in particular, in which the above factors were considered to be of minor importance, and ground water infiltration deemed to be in proportion of the number of inhabitants served. He can see little or no reason whatever for such a basis for design, and believes that in view of the importance of the four factors already mentioned, the only sensible basis seems to be that of infiltration in terms of length and size of the sewer line itself.

Grades of Sewers

All sewer lines, with the exception of those laterals on the end of which flushtanks are located, were designed so that the minimum velocity of the sewage for pipes flowing not less than half full shall be 2 feet per second. The following minimum grades were used as a basis for design -

Table II. Minimum Grades for Vitrified Pipe Sewers

		With Flushtanks	
Size of Pipe	Grade	Capacity (flowing full)	Velocity in feet/second
8 inch	0.25%	250 gallons/minute	1.6
10 inch	0.23%	450 " "	1.8
12 inch	0.20%	680 " "	1.96

		Without Flushtanks	
Size of Pipe	Grade	Capacity (flowing full)	Velocity in feet/second
8 inch	0.40%	310 gallons/minute	2.0
10 inch	0.29%	500 " "	2.0
12 inch	0.22%	700 " "	2.0

Capacities of Sewers

In determining the limiting capacities for the design of vitrified pipe sewers, laid as lateral lines, branch lines, and trunk lines, the following factors of safety were used by the writer in addition to the extra provisions for fluctuation of flow and infiltration of ground water already mentioned.

Laterals - An assumed maximum capacity of flow was taken as but 1/2 of the "flowing full" capacities given in Table II.

Branches - An assumed maximum capacity of flow was taken as but 1/1.75 of the "flowing full" capacities given in Table II.

Trunks - An assumed maximum capacity of flow was taken as but 1/1.5 of the "flowing full" capacities given in Table II.

Table III. Maximum Allowable Capacities for Design

Type of Sewer Line	Capacity Factor	With Flushtanks - Grades in Table II		
		8 inch pipe	10 inch pipe	12 inch pipe
Laterals	2	125 g.p.m.	225 g.p.m.	340 g.p.m.
Branches	1.75	143 g.p.m.	257 g.p.m.	389 g.p.m.
Trunks	1.5	167 g.p.m.	300 g.p.m.	453 g.p.m.

Type of Sewer Line	Capacity Factor	Without Flushtanks - Grades in Table II		
		8 inch pipe	10 inch pipe	12 inch pipe
Laterals	2	155 g.p.m.	250 g.p.m.	350 g.p.m.
Branches	1.75	177 g.p.m.	286 g.p.m.	400 g.p.m.
Trunks	1.5	207 g.p.m.	333 g.p.m.	467 g.p.m.

Allowable Population to be Served

The following table is an attempt to set forth the maximum allowable number of people to be served by 8 inch, 10 inch, and 12 inch vitrified pipe sewers, based upon the fluctuation factors provided for in Table I, the minimum pipe line grades given in Table II, and the additional safety factors for capacity of Table III.

Table IV. Maximum Allowable Population to be Served

(Neglecting provisions for Ground Water Infiltration)

Based on Maximum Sewage Discharge of 50 gallons per capita per day - -

Type of Sewers	With Flush Tanks			Without Flush Tanks		
	8 inch	10 inch	12 inch	8 inch	10 inch	12 inch
Laterals	1800 -	3240 -	4900	2230 -	3600 -	5030
Branches	2060 -	3700 -	5600	2550 -	4120 -	5760
Trunks	2400 -	4320 -	6530	2980 -	4800 -	6720

Based on Maximum Sewage Discharge of 75 gallons per capita per day - -

Type of Sewers	With Flush Tanks			Without Flush Tanks		
	8 inch	10 inch	12 inch	8 inch	10 inch	12 inch
Laterals	1200 -	2160 -	3260	1490 -	2400 -	3360
Branches	1370 -	2470 -	3730	1700 -	2750 -	3840
Trunks	1600 -	2880 -	4340	1990 -	3190 -	4480

Method of Flushing Sewers

Automatic siphon flushtanks were located on the extremities of lateral sewers wherever it was found necessary to lower the percentage of line grade below the minimum for a velocity of 2 feet per second.

Lampholes, Manholes

Lampholes were provided at the dead ends of all lateral sewers where flushtanks were not necessary. Manholes were provided at the junction points of all sewers, and spaced not more than 400 feet apart along all lines throughout the system.

Character of Sewage

The sewage is to be considered as that resulting from domestic sources entirely.

Disposal Plant

General

The sewage disposal plant designed for Osgood provides for primary treatment of sewage only, and consists of an Imhoff tank with sludge bed, the effluent from the tank being discharged directly into a small creek. It may be noted, however, that the plant was so located and arranged as to allow for the construction of sprinkling filters and additional sedimentation, or humus, tank as an addition thereto for secondary treatment at a future date if necessary.

The disposal plant consists essentially of a double sedimentation chamber Imhoff tank, designed for reversal of flow operation, and a connecting sludge bed for the removal of sludge.

Data for Design of Imhoff Tank

Capacity

The tank was designed to serve an ultimate population of 2000.

Capacity of Sludge Chamber

Of three formulae commonly used for calculation of sludge chamber capacity, namely the Imhoff formula, the Allen formula, and the Metcalf and Eddy formula, the writer chose the last named. Whereas the second and third of these give results reasonably close to one another, the Imhoff formula seems to be considered to give results entirely too low for American practice. In fact it is suggested by numerous authors that if the Imhoff formula be considered, an additional 50% of the result should be added for safety.

(a) Imhoff Formula -

This provides for the capacity

of the sludge chamber equal to a volume of sludge that would accumulate over a period of 6 months at the rate of 0.0035 cubic feet per capita per day.

(b) Allen formula -

(This formula is based on sludge having an average moisture content of 80%)

$$V = 5.25\ PD$$

V = volume of sludge chamber in cubic feet

P = population to be provided for, in thousands

D = number of days storage to be provided for

(c) Metcalf and Eddy formula -

(This formula is based on sludge having an average moisture content of 85%, and assumes a volume of sludge of 2.48 cubic yards per million gallons of domestic sewage. It is considered to be a formula very much on the side of safety for American practice)

$$V = \frac{2.48 \times 27 \times P \times G \times D}{1{,}000{,}000}$$

V = volume of sludge chamber in cubic feet

G = assumed sewage flow per capita per day

P = population to be provided for, actual number

D = number of days storage to be provided for

In the use of the above formula for Osgood, the following figures were used:

$P = 2000$ $G = 75$ $D = 210$

resulting in a required sludge chamber capacity of 2117 cubic feet.

Additional Miscellaneous Data upon which Design of Imhoff Tank was based

1. Detention period for passage through sedimentation chambers to be from 2 to 3 hours. Length to be sufficient to limit velocity to 25 feet per hour, where possible.

2. Gas vent area to be 17% of the total area of the tank.

3. Minimum velocity for effluent channels after passage through sedimentation chambers to be 2.5 feet per second.

4. Stilling baffles to be provided at entrance to sedimentation chamber; scum boards to be provided in front of sedimentation chamber weir outlets.

5. Slope for sedimentation baffles to be 1.5 vertical to 1 horizontal. Slot at bottom of chamber between baffles to be from 6 to 12 inches wide. Horizontal overlap of baffles below slot to be at least 8 inches.

6. Surface of sludge in sludge tank to be at least 18 inches from slot in sedimentation chamber when sludge tank is filled to capacity.

7. Depth of sludge tank to be not less than from 16 to 20 feet.

8. Use sludge pipe at least 8 inches in diameter for removal of sludge, the pipe outlet to sludge bed to be at least 4 to 6 feet below sewage level in sedimentation chamber.

Data for Design of Sludge Bed

1. Sludge bed to be porous and well underdrained at a depth of from 24 to 30 inches with from 4 to 6 inch open joint tile.

2. Area of sludge bed to be calculated on the basis of 3 persons per square foot of area. The writer would like to remark here that there seems to be a growing opinion that the above heretofore generally accepted allowance for sludge bed area is too small and that the basis should be changed to not more than 1 person per square foot of area.

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