

THESIS

SUPPLEMENTARY
MATERIAL
IN BACK OF BOOK

Design of a Sewage Treatment Plant
for East Jordan, Michigan

A Thesis Submitted to

The Faculty of
MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

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THESIS

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East Jordan is a small town located at the southern extremity of Pine Lake, fifteen miles south of Charlevoix. This small town is one of the leaders of the district, especially in the summer when there is a large amount of tourist trade.

East Jordan was established in 1874. Its primary reason for existence was the large lumber mills established there, the first mill locating in 1877. The city was incorporated in 1903. In 1911 the neighboring village of South Arm was annexed. In 1920 the forest resources had been depleted and the population started to decline.

East Jordan has always been progressive, as is seen by the fact that the city has owned and operated its own water plant since 1896. The city government is up to date, consisting of three commissioners, each being the head of a department, namely:

1. Public Utilities
2. Streets and Sewers
3. Finances, Health and Sanitation

The city is now served by an inadequate sewer system. A separate sewer system was designed for the city in the fall of 1942. In the thesis covering this design the sources of sewage were given as:

1. Domestic Sewage
2. Trade Wastes...stores, garages, etc.

There is a cannery here but it has its own disposal system so does not contribute to the city sewage.

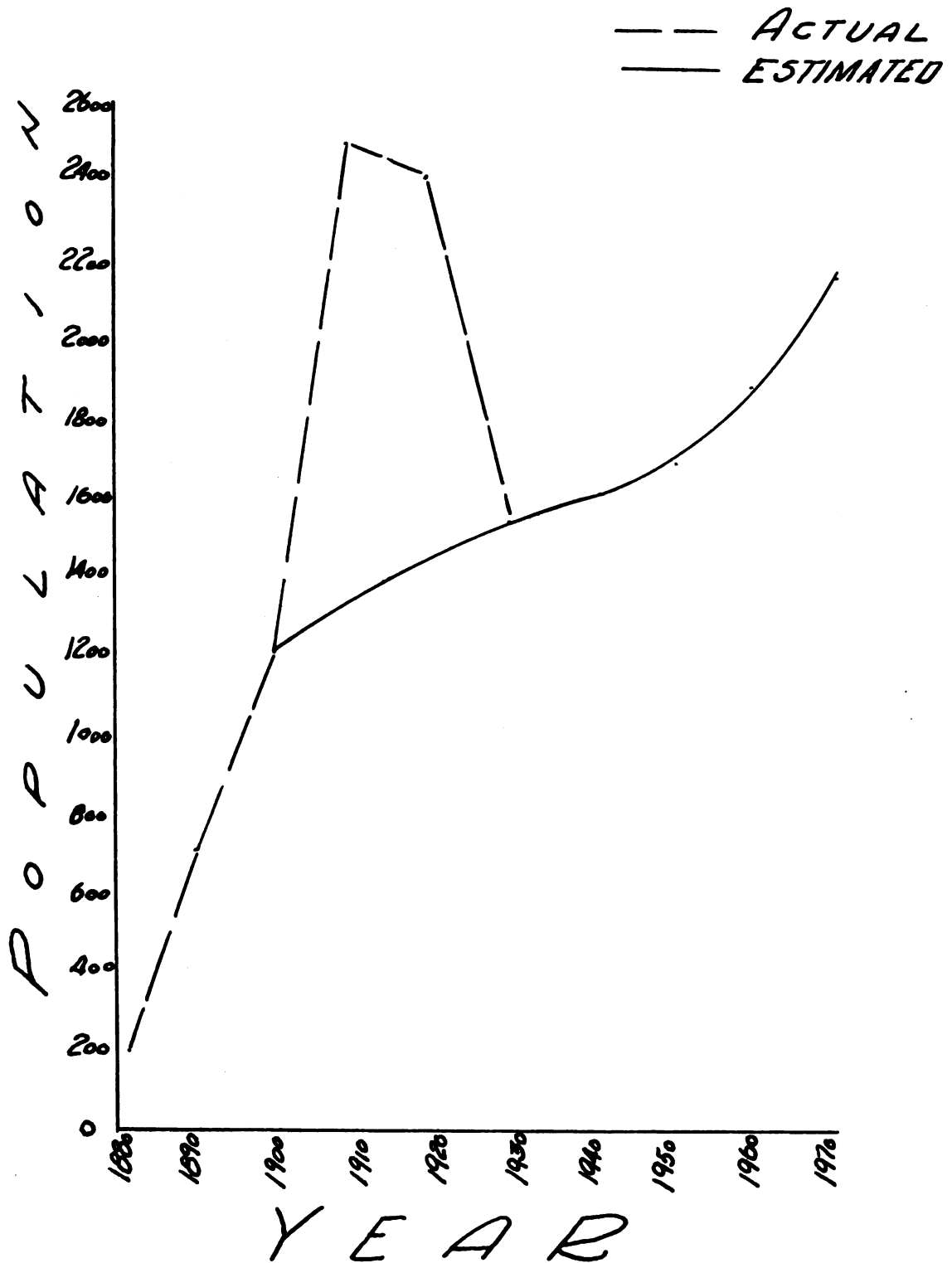
It is the purpose of this thesis to design an outfall from the end of the proposed sewer line and to design a plant to adequately serve the city's needs.

The first important consideration in any design is the amount of sewage to be treated. The flow to be expected must be predicted, based on population data. The best source of past population data is the United States Bureau of Census. For the years subsequent to a census it is common to use a city directory as giving the correct population. Another method is to use the ratio between the number of children in school for the year desired to the number of children in school for the census year as the ratio of population of the two years; or do the same thing with the number of telephone services and use the ratio obtained to determine the present population of the city.

A far more difficult problem is the prediction of a city's population in the future. To do this with any degree of accuracy good judgment must be used on the part of the engineer. The engineer should know the history of the town, its resources and location as to markets and transportation facilities. After a careful consideration of all the factors effecting future population a method of prediction must be selected which fits the conditions of the city under consideration. There are several of these methods, the most popular being the arithmetical, the uniform percent rate of growth and the curvilinear rate of growth.

The method selected for East Jordan was the uniform percent rate of growth with slight modification. An

POPULATION CURVE FOR EAST JORDAN, MICHIGAN



PREDICTED BY H.L. FROST
CONFIRMED BY W.W. JOY

observation of the population data shows a uniform rate of growth except for the period when the lumbering industry was located in East Jordan. If this condition is considered as abnormal and the values for this period ignored, a uniform percent rate of growth will apply to the period from 1882-1930 and so could be assumed to apply in the future, barring unforeseen developments. The usual period of design is 20 years. Therefore, the population will be predicted for 1963.

Population Data

Year	1882	1890	1900	1910	1920	1930
Population	200	731	1205	2516	2428	1523

Computations

$$\frac{\text{Population 1930} - \text{Population 1900}}{3} = \text{Increase per Decade}$$

$$\frac{1523 - 1205}{3} = \frac{318}{3} = 106 \text{ Persons per Decade}$$

$$\frac{106}{1523} \times 100\% = 70 \text{ Persons Increase}$$

$$1940 = 1523 + 70 = 1593$$

$$\frac{106}{1523} \times 1593 = 110$$

$$110 + 1593 = 1703 \text{ Persons in 1950}$$

$$\frac{106}{1523} \times 1703 = 202$$

$$1703 + 202 = 1905 \text{ Persons in 1960}$$

$$1905 \times \frac{106}{1523} = 340$$

$$1905 + 340 = 2245 \text{ Persons in 1970}$$

Allowing a deviation of approximately 100 persons, the estimated population in 1963 will be 2200, confirming the value obtained by H. L. Frost.

The best way to predict sewage quantity is to check water consumption. East Jordan is unmetered. Therefore, some other method of prediction must be used. For a city of this type 100 gal./day/capita is an average value often assumed. A check of similar Michigan cities reveals this to be a good value for East Jordan. The following data was considered:

City	Population	Consumption/gal./day
Cass City	1261	57.0
Cassopolis	1448	117.0
Clare	1491	167.0
Decatur	1582	63.2
Frankfort	1468	102.2
Average		101.3

The infiltration as computed by H. L. Frost was equal to .1823 C.F.S.

The quantity of sewage is not as important as the rate. The maximum rate is assumed to be 175% of the average flow. This gives a value of $\frac{100 \times 1.75 (2200)}{7.48 \times 60 \times 60 \times 24} = .5985$ C.F.S. giving a total maximum flow of $.5985 + .1823 = .7808$ C.F.S. The minimum rate of flow is important in determining the capacity of the pumps and the wet well capacities, or actually the longest period of detention. The minimum flow will be assumed to be 50% of the average or $\frac{100 \times 2200}{2 \times 60 \times 60 \times 24 \times 7.48} = .1710$ C.F.S. The minimum flow will therefore be $.1710 + .1823 = .3533$ C.F.S. The values of maximum and minimum flow were used based on a statement by Steel in his book "Water Supply and Sewage".

Flow Data in Outfall Sewer

From M.H.	To M.H.	Length Line '	C.F.S. Max.	Ground Elev Upper	Elev Lower	Drain Pipe	Grade	Fall'
55	Plant	78.0	.7808	107.0	106.6	10"	.0077	0.6
Velocity Flowing Full			Capacity Flowing Full	Upper Invert		Lower Invert		
3.58'/sec.			1.82 C.F.S.	91.4		90.8		

Flow Data in Sewer Above Outfall

Population	Aver. Flow	Min. Flow	Max. Flow	Infiltration	Total
2209	.3420	0.3533	0.5985	0.1823	0.7807

Values of flow used:

Minimum 0.3533 C.F.S. = 157 G.P.M.

Average 0.5243 = 232 G.P.M.

Maximum 0.7808 = 351 G.P.M.

The above values being based on the maximum value including infiltration.

The best way to determine the physical characteristics is to obtain samples and analyze the samples in the laboratory. The distance to East Jordan makes this method impossible. A table of average values for typical types of American cities compiled by Metcalf and Eddy gives the following values for a small, residential, agricultural town:

B. O. D. (5 day)	143.0 P.P.M.
Chlorides	47.0 P.P.M.
Solids - Total	603.0 P.P.M.
Volatile	393.0 P.P.M.
Fixed	210.0 P.P.M.
Suspended Solids - Total	342.0 P.P.M.
Volatile	260.0 P.P.M.
Fixed	82.0 P.P.M.
Dissolved Solids - Total	261.0 P.P.M.
Fixed	133.0 P.P.M.
Volatile	128.0 P.P.M.
Fats	Variable

The first important question to be decided is the method of treatment to be used in the proposed plant. A common primary method of treatment is the plain sedimentation method. The results are 90% reduction in suspended solids and 85% reduction in B.O.D. The cost of treatment averages \$4.00/ million gallons treated. This high cost makes it an uneconomical method of treatment, especially since there are no industrial wastes to be treated. The second method of treatment is filtration, either the intermittent sand filter or the trickling filter. The sand filter is now outmoded, so

it will not be considered. The trickling filter does come in for consideration. The trickling filter gives 85% reduction in suspended solids. The disadvantages are the high head required and the odors arising during spraying.

The third and final method of treatment to be considered is the activated sludge method. This method is a method which is moderately economical. It requires a complicated plant which is difficult to operate and requires an operator with some degree of skill. The former idea connected with this type of treatment was that it was not applicable to small cities. Recent data has disproven this theory and the expiration of patent rights has done much to further the activated sludge method. The results are reduction of bacteria 90%, reduction of suspended solids 95%, and stability by methylene blue is 72 hours.

The important consideration at East Jordan is the purification of the lake and stream. This leaves but one selection. The activated sludge method is the most applicable due to its higher efficiency and economical operating costs, so the activated sludge type of plant will be designed.

In any project an item of primary importance is the method of financing. The region north of the east west line from Bay City to Lake Michigan is known as "The Sticks", and any project in this district must have very good security in order to obtain financing from any of the commercial sources. This means that a city must be in very good financial condition to consider a project as large as a sewage treatment plant. East Jordan has its own water plant and no other

recent improvements have been made, so the city is in a good position to issue bonds. These bonds should be of the revenue type to insure a good market for them. The investigation of the financing of a project is a subject which requires considerable research. An exact knowledge of the financial condition of a city is something the city regards as purely personal and will not divulge this information unless an engineer were actually hired, so it will suffice just to mention the generally known facts and theorize on the probable course of action.

Outfall Design:

Ground elev. at M.H. 55 107.0

Invert elev. at M.H. 55 91.4

Depth 15.6

Q = .7808 C.F.S - Maximum Flow

Length is 100 '

Use a 10" pipe as that is the largest pipe used in the system before reaching the plant site. Using values obtained from Page 379 "Water and Sewage" by Steele, $N = .013$. Assume slope of .0077. $Q = 1.85$ C.F.S. at a velocity of 3.54'/sec. $\frac{0.7808}{1.85} = 0.424$ as ratio of capacity when flowing full and when actual maximum flow is flowing. Now, using Figure 149, Page 379, Steele, pipe is flowing 44% full or $.44 \times 10" = 4.4"$ deep and a velocity of 0.95 giving $0.95(3.54) = 3.44'/\text{sec}$. This is a little high, but pipe in system was at approximately this grade, so to be consistent this grade must be maintained.

Racks:

Use a design figure of 20 sq. ft. of rack area per million gallons per day. Use 2" C-C spacing and 3/8" bars as the first assumption. With a limiting velocity through the rack of 1.5'/sec. $\frac{.7808}{1.54}$ (1,000,000) = .51(20) = 10 sq. ft. of rack area, which is high in comparison to areas used in plants already constructed. Use Prof. F. R. Theroux's value of 12 sq. ft./million gal./day and obtain .51(12) = 6.12 sq. ft. of rack area.

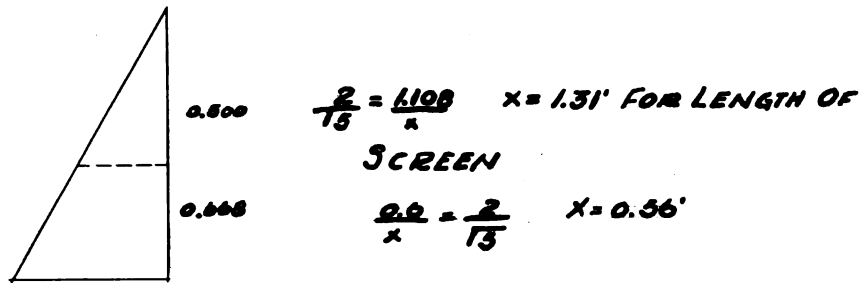
The common method is to have the rack on a 1-2 slope, so revise the first assumption to give velocity of 1.0'/sec. through the rack and 12 sq. ft./million gal/day.

First consider the head created, using a formula given by Metcalf and Eddy in their book "Sewage Design" Page 491. $h = \frac{V^2 - v^2}{2g} \times \frac{1}{0.7}$ Where "V" is velocity through rack and "v" is velocity above rack in channel, substituting in the values obtained gives $\frac{(3.44)^2 - (1)^2}{64.4 \times 0.7} = .244'$ increase in head, which will not even fill the 10" pipe, so this assumption of velocity is O.K. The manner in which this increase in head may be overcome is to increase the depth of the channel behind the screen by 0.25'. So now the total depth of sewage is .244 + .424 = .668'.

Checking Area of Screen:

Allow 6" clearance between sewage level and bottom of screen room floor. $D = .668 + .5 = 1.168'$ clearance

space for racks.



$1.31 - .56 = 0.75'$ of screen in sewage per foot length.

$\frac{6.12}{0.75} = 9'$ which is too long, so use other basis of design of 1'/sec. through screen and change to 1" clear. Area required is .7808 sq. ft. of screen to give the desired velocity.

Clear area/ft. length = $9(12 - (\frac{12}{1} - 1)3/8) = 71$ sq. in./ft. length. Say .5' length $\frac{.7808}{.5} = 1.53$ feet long, which is a little too narrow for easy cleaning. So use a 2' rack with 6" clearance above sewage surface.

Testing the Bars:

Assume 1.5' long and 2" x 3/8" in size.

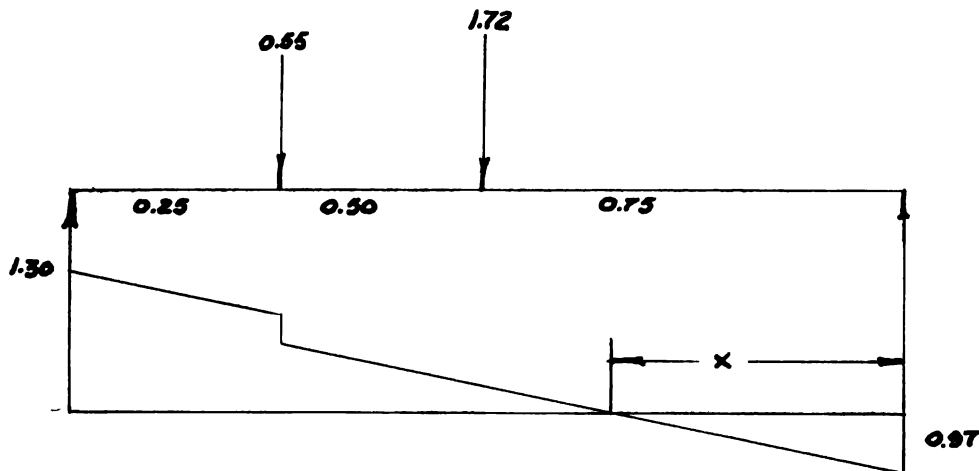
Wt. of bar = $2 \times 3/8 \times 1.5 \times 12 \times \frac{1}{1728} \times 490 = 3.85\#$
per bar applied at the center of gravity of the bar.

$X = \frac{3.85}{2.24} = 1.72\#$ applied perpendicular to the bar.

Water Pressure:

$\frac{62.4}{12} \times \frac{3}{8} \times \frac{.75 \times .75}{2} = .55\#$ applied .25' from

the bottom of tank. Now consider bar as a horizontal beam.



**NOTE - SHEAR DIAGRAM
NOT DETERMINED FROM LOADS AS SHOWN.**

Shear decrease per foot is $1.15^{\#}$ per foot

$$X = \frac{.97}{1.15} = .84' \text{ from rt. end.}$$

Bending moment = area of shear diagram.

$$\text{Area} = \frac{.97}{2} \times .84 \times 12 = 6 \times 0.84 \times 0.97$$

$$S = \frac{Mc}{I} = \frac{6 \times .84 \times .97 \times 1}{\frac{1}{12} \times \frac{3}{8} \times 2^3} = 78^{\#}/\text{sq. in.}$$

which is very small for steel. So the thickness is merely for rigidity and cleaning with a rake.

Screenings:

Use an estimate of 5 C.F./million gallons so provide enough containers to care for the screenings with daily emptying of containers.

Grit Chamber:

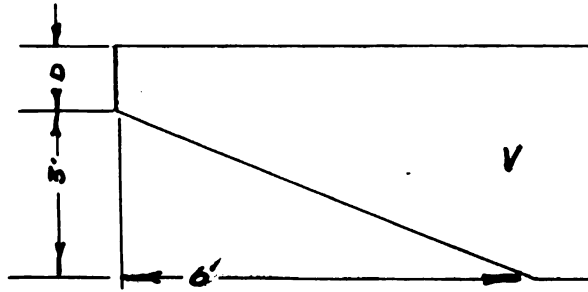
A grit chamber is not needed or required in this plant due to the fact that only separate sewage is being treated.

Wet Well:

1970 minimum flow will be a basis of design. Minimum flow = .3533 C.F.S. provide a 10 min. storage.

$$10 \times 60 \times .3533 = 214 \text{ C.F.} = 1600 \text{ gallon storage.}$$

Using a typical cross section in a 20' building use a 7' tank.



$$18 \left(3 \left(\frac{7+1}{2} \right) + 7 \right) \text{ d.} = 214$$

$$12 + 7 \text{ d.} = 12$$

$$D = 0$$

So allow 6" for safety and 6" free board to bottom of screen room floor.

Pumps:

Three pumps must be used for flexibility of operation and safety. When a pump turns on, it must empty the well plus the amount of sewage which will run in during the emptying period. Provide this unit and provide others for economy and safety in case of a breakdown.

1600 gallon tank 250 G.P.M. pump.

The time required to empty the tank in minutes is

$$t = \left(\frac{1600}{250} + \frac{1600 (.5243 \times 60)}{250} \right) = 6.4 + .80 = 7.2 \text{ min.,}$$

which is alright. This time of emptying will not

require pumps to be run continuously.

A 150 G.P.M. and a 200 G.P.M. pump should be provided to care for flows other than the average.

Primary Settling Tank:

To obtain 50% removal of sludge a $1\frac{1}{2}$ hour detention period is required, as determined by diagram Fig. 175, Page 534, Metcalf and Eddy.

Assume a sustained flow of $1.5 \times 232 = 348$ G.P.M.
Vol. = $1.50 \times 60 \times 348 = 31,400$ gallon capacity
 $\frac{31,400}{7.48} = 4180$ C.F.

Two identical units should be provided for continuity of service in case of a breakdown.

Assume a 6' depth and ratio of 1 : 6 for width to length.

$$6 \times W \times 6W = 4180$$

$W^2 = 11.6'$ Use a 10' tank, giving $L = \frac{4180}{60} = 70'$ long. Provide two tanks with an 8' depth, 2' being allowed for free board.

Velocity Through Tank:

$$.5243 \times 1.5 = .79 \text{ C.F.S.}$$

$Q = AV$ $V = \frac{Q}{A} = \frac{.79}{10 \times 6} = .013'/\text{sec.} = .156"/\text{sec.},$
which agrees favorably with a velocity of .15 - .50"/sec. given in Metcalf and Eddy, Page 536.

Storage Space:

Specific gravity = 1.03

Sludge moisture content of .95

Average flow of 232 G.P.M.

24 hours time between pumping

50% removal

342 P.P.M. suspended solids

.50 x 342 = 171 P.P.M. removal design on a basis
of 200 P.P.M. removal.

$$\frac{232}{449} \times 1,000,000 = 515,000 \text{ gallons per day}$$

$$\frac{515,000 \times 8.34 \times 200}{1,000,000} = 860^{\#} \text{ of dry sludge per day.}$$

Vol. = $\frac{860}{8.34 \times .05 \times 1.03} = 2000$ gallons of sludge
storage must be provided. This is $\frac{2000}{7.48} = 268$ cu.ft.
storage.

Provide a hopper at inlet end.

Try a 10' x 10' x 2' x 2' x 4' hopper.

$\frac{10 \times 10 + 2 \times 2}{2} (4) = 208$ cu.ft. of storage, which
is not quite enough.

Try a 10' x 10' x 2.5' x 2.5' x 5' hopper.

$\frac{10 \times 10 + 2.5 \times 2.5}{2} (5) = 270$ C.F. storage, which
is alright.

Use a mechanical skimmer of the link belt type
and use a 3% bottom slope, as recommended by Metcalf
and Eddy. $.03 \times 70 = 2.1'$, depth to bottom of
hopper. Invert elev. = $6 + 2 + 2.1 + 5 = 15.1'$ to
bottom of hopper.

A scum trough should be provided at the outlet
end. The standard type is a lip of concrete
projecting down in front of the effluent channel
with the lip a little above the water surface. A
detail of the tank with fittings is shown on an
accompanying sheet.

The design of the scum trough necessitates hand

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removal of scum. The purpose of the lip projecting down in front of the effluent channel is to prevent short circuits.

Sludge Pumps:

Use a minimum size of 8" as recommended by Metcalf and Eddy, Page 552.

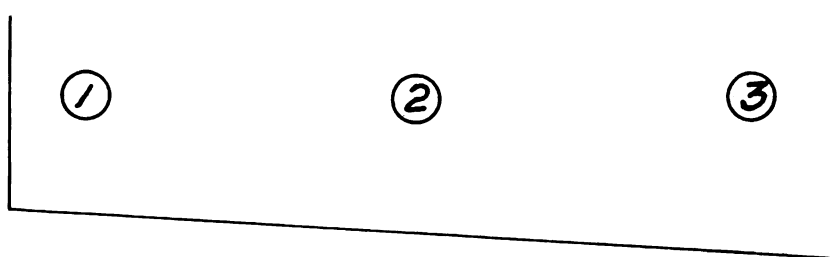
A low velocity is recommended, .5 - 1'/sec.

$.7854\left(\frac{64}{144}\right) = .35$ sq.ft. cross sectional area of a 8" pipe.

$.35$ sq.ft. $\times .75 = .26$ cu.ft./sec., so use a 100 gal/min. pump giving $\frac{100}{7.5 \times 60} = .24$ cu.ft. per sec.

Inlet Channel:

Tank is 10' wide, use 3 inlets...one at each end and one in the center. End outlets to be 1' from outside wall.



Velocity through openings to be 2'/sec.

Assume sustained flow of 348 G.P.M. = .7808 C.F.S.
Each inlet will be $\frac{.7808}{3 \times 2} = .13$ sq.ft. in cross section.

$$\frac{3.14 d^2}{4} = .13 \quad d = 5" \quad \text{Area} = .136 \text{ sq.ft.}$$

Channel Design:

Velocity of 3'/sec. in all sections of tank. Area

at (1) $\frac{.775}{3} = .26$ sq.ft., use a 6" x 6" section.
 Width at (2) $\frac{.775 - .26}{3} = .17$, use a 5" x 5"
 section and continue on to end as a 5" channel,
 due to the fact that if a smaller size is used a
 stoppage is apt to occur.

Grade of Channel Bottom:

$$n = .012$$

$$R(1 - 2) = \frac{A}{W.P.} = \frac{.5}{1.5} + \frac{1.72}{1.25} = .238, \text{ using}$$

a diagram. $S = .004$ or $.016'$ fall in the 4 ft. of
 channel.

$$R(2 - 3) = .138 \quad \text{Slope} = .010 \text{ or } .04' \text{ fall.}$$

$.04 + .16 = \frac{.20}{8} = .025$ average slope. So use $\frac{1}{4}"$
 fall/ft. of channel.

Use a 1.5' submergence as recommended by Prof.
 F. R. Theroux. A baffle in front of each inlet
 should be provided; or since there is a lip of the
 scum trough extending down, use it for a baffle.

Outlet:

The outlet should be of the Wier type with a
 baffle in front of it extending to a depth of 2'
 below the surface. Velocity must be relatively
 low and $\frac{0.775}{10} = .0775$ if 1' deep, but maximum depth
 is about 2". Therefore, velocity is $.0775 \times \frac{12}{2} =$
 $0.45'$ /sec. At this velocity particles of ordinary
 size will not be carried over. Therefore, this
 Wier is alright. Make adjustable to $\pm .1'$ from
 elevation to be determined later.

Aeration Tank:

Detention period of 6 - 10 hours.

Use an updraft type of aerator.

Use 1.0 cu.ft. of air per cu.ft. of sewage treated, as recommended by Steele, Page 567.

Circulation to be provided by aerator is 2 - 3 times per hour.

Flow of 351 G.P.M. or .7808 C.F.S.

Also returned activated sludge must be accounted for.

Assume 25% return. $1.25 \times .7808 = .98$ C.F.S.

$8 \times 60 \times 60 \times .98 = 23,000$ C.F. of tank must be provided.

Assume 12' deep tanks 20' wide.

$L = \frac{23000}{12 \times 20} = 117.5'$ long. So if the area which can be served by an aerator is about 25' in diameter and maintains a rectangular shape, five 25' tanks, 20' wide and 12' deep will fulfill the requirements. Round the bottom edges up to aid circulation.

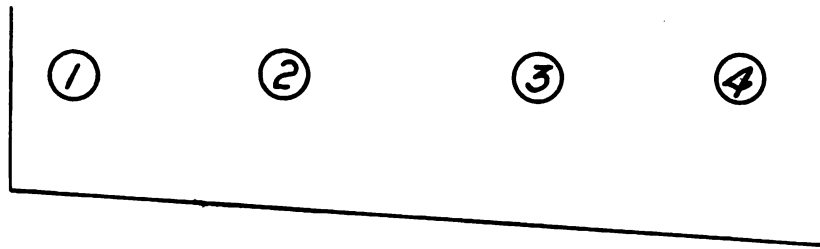
Use an aerator of the updraft type which the manufacturer guarantees to furnish 3600 cu.ft. of air and able to provide at least 18000 cu.ft. of circulation per hour.

Inlet:

The inlet can be a channel from the pipe flowing into the tank from the primary tank. The influent channel should be well baffled with a perforated plate containing $\frac{1}{4}"$ circular holes.

The slope of the channel will be determined by the following method: $Q = .98$ C.F.S. Assume four

openings in the tank wall, the end openings to be 1.0' in from the ends. Assume the sewage enters from one end.



$$N = .013$$

Diagram Page 130, Metcalf & Eddy

Assume 2'/sec. velocity.

Area $= \frac{Q}{v} = \frac{.98}{2} = 1$ sq.ft. in cross section, so use a 12" square channel.

$$\text{H.R. } 1 - 2 = \frac{A}{\text{W.P.}} = \frac{1}{3} = .33 \quad S = .0013$$

Area 2 - 3 = .75 Use 12" deep and 9" wide.

$$\text{H.R. } 2 - 3 = .33 + \frac{.75}{2.5} = .33 + .30 = .63 \quad S = .0014$$

Area 3 - 4 = .5 12" deep gives 6" width. Keep this same width the remainder of the length for ease in cleaning. Round the end of the channel in order to obtain smooth flow into the last port.

$$\text{Depth (3) to (4) is } \frac{.5}{.5} = 12" \text{ deep. W.P.} = 2(1) + .5 = 2.5 \quad \text{Area} = .5 \quad \text{H.R.} = \frac{.5}{2.5} = .20 \quad S = .031$$

$$\text{Depth (4) to (5) is } \frac{.25}{.5} = .5 \text{ is 6" deep.}$$

$$\text{So W.P.} = 2(.5) + .5 = 1.5$$

$$\text{H.R.} = \frac{.25}{1.5} = 0.016 \quad S = .0041$$

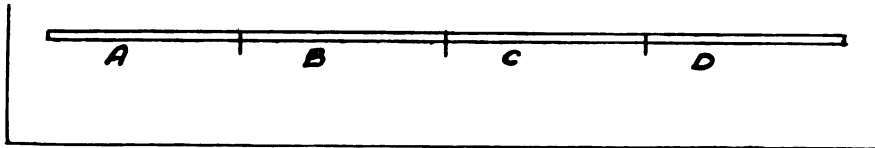
$$\text{Average slope} = \frac{.0041 + .0031 + .0013 + .0014}{4} = .0022$$

$$0.0022 \times 20 = .044' \text{ total.}$$

Outlet:

The outlet can be a simple Wier adjustable to $\pm 0.1'$ and emptying into a channel.

Design of channel



Assume an even flow over the Wier. Design in 4 parts and use average slope. Use a minimum width of 4" to obtain ease of cleaning. Velocity in channel to be $1\frac{1}{2}'/\text{sec.}$ at a Vol. = .25 C.F.S.

$$Q = Av \quad a = \frac{Q}{V} = \frac{.25}{1.5} = .167 \text{ sq.ft.} \quad .167^{\frac{1}{2}} = .41'$$

$$\text{or } 5". \quad \text{Av H.R. (0) to (A)} = \frac{\frac{16}{12} + \frac{25}{15}}{2} = \frac{1.3 + 1.4}{2} = 1.35. \quad S = .00015$$

$$\text{Vol. At B} = .5 \text{ cu.ft.} \quad A = \frac{.5}{1.5} = .33$$

$$.33^{\frac{1}{2}} = 6" \text{ wide}$$

$$\text{H.R. (A) to (B)} = \frac{1.4 + \frac{36}{18}}{2} = 1.70 \quad S = .00012$$

$$\text{Vol. at C} = .75 \text{ cu.ft.} \quad A = \frac{.75}{1.5} = .50 \quad .50^{\frac{1}{2}} = .707$$

Use 9" depth

$$\text{H.R. (B) to (C)} = \frac{2.0 + \frac{81}{27}}{2} = 2.5 \quad S = 0.001$$

$$\text{Vol. at D} = 1.0 \text{ cu.ft.}$$

$$\text{Depth} = \left(\frac{1.0}{1.5}\right)^{\frac{1}{2}} = .807 \quad \text{Use } 10"$$

$$\text{H.R. (C) to (D)} = \frac{3.0 + \frac{100}{30}}{2} = 3.17 \quad S = 0.00005$$

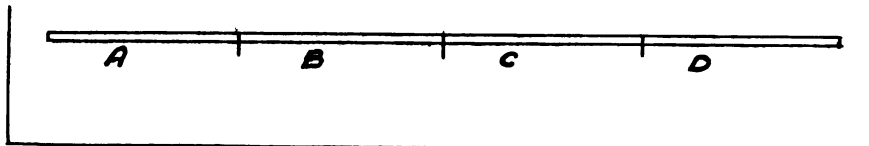
$$\text{Average slope} = \frac{.00005 + .0001 + .0001 + .00015}{4} =$$

$$0.00011 \quad .00011 \times 20 = \frac{1}{8}" \text{ fall from end to end of channel.}$$

Outlet:

The outlet can be a simple Wier adjustable to $\pm 0.1'$ and emptying into a channel.

Design of channel



Assume an even flow over the Wier. Design in 4 parts and use average slope. Use a minimum width of 4" to obtain ease of cleaning. Velocity in channel to be $1\frac{1}{2}'/\text{sec.}$ at a Vol. = .25 C.F.S.

$$Q = Av \quad a = \frac{Q}{V} = \frac{.25}{1.5} = .167 \text{ sq.ft.} \quad .167^{\frac{1}{2}} = .41' \text{ or } 5".$$

$$\text{Av H.R. (0) to (A)} = \frac{\frac{16}{12} + \frac{25}{15}}{2} = \frac{1.3 + 1.4}{2} = 1.35.$$

$$S = .00015$$

$$\text{Vol. At B} = .5 \text{ cu.ft.} \quad A = \frac{.5}{1.5} = .33$$

$$.33^{\frac{1}{2}} = 6" \text{ wide}$$

$$\text{H.R. (A) to (B)} = \frac{1.4 + \frac{36}{18}}{2} = 1.70 \quad S = .00012$$

$$\text{Vol. at C} = .75 \text{ cu.ft.} \quad A = \frac{.75}{1.5} = .50 \quad .50^{\frac{1}{2}} = .707$$

Use 9" depth

$$\text{H.R. (B) to (C)} = \frac{2.0 + \frac{81}{27}}{2} = 2.5 \quad S = 0.001$$

$$\text{Vol. at D} = 1.0 \text{ cu.ft.}$$

$$\text{Depth} = \left(\frac{1.0}{1.5}\right)^{\frac{1}{2}} = .807 \quad \text{Use } 10"$$

$$\text{H.R. (C) to (D)} = \frac{3.0 + \frac{100}{30}}{2} = 3.17 \quad S = 0.00005$$

$$\text{Average slope} = \frac{.00005 + .0001 + .0001 + .00015}{4} =$$

$$0.00011 \quad .00011 \times 20 = \frac{1}{8}'' \text{ fall from end to end of channel.}$$

Sludge Division Box:

The sludge division box must be so designed so as to distribute sludge from the final tank to the wet well aerator on primary tank in any desired proportion.

The details of the division box are shown on the tank details plate.

Final Settling Tank:

Use a $2\frac{1}{2}$ hr. detention period.

Use a circular tank.

Use as first assumption 1600 gal/sq.ft. of tank surface. .7808 C.F.S. = 506,000 G.P.D.

Surface area = $\frac{506000}{1600} = 316$ sq.ft. of area.

Volume of tank = $2.5 \times 60 \times 60 \times .7808 = 7020$ cu.ft.

$\frac{7020}{316} = 20^+$ ft. which is very much too deep, so try a value of 850 gal/sq.ft. of surface area based on average flow as given in Metcalf and Eddy.

$\frac{.7808}{1.5} \times 646300 = 336,000$ gal/day.

Surface area = 396 sq.ft.

Volume required is $\frac{.7808 \times 3600 \times 2.5}{1.5} = 4630$ C.F.

$\frac{4630}{316} = 12^+$ ft. deep, which is very close to the desirable depth. Assume a 10' depth for convenience.

Vol. = 11,300 based on maximum flow.

$\frac{10 \times d^2 \times 3.14}{4} = 11,300 \times = 34'$ in diameter the tank should be to give the desired volume.

Final Settling Tank Storage

Storage must be provided for fluctuations in flow. The fluctuations in flow will be the maximum and minimum values. Assume a maximum occurs immediately after a

minimum and exists in length at least one flowing through period.

150 - 50 = 100% fluctuation of flow flowing through period.

Primary $1\frac{1}{2}$ hr.

Aerator 8 hr.

Total $9\frac{1}{2}$ hr.

An inspection of flow diagrams for various cities shows that the maximum flow is approximately 14 hours duration. So assuming this condition the volume of sewage flowing through the tank is $.7808 \times 9.5 \times 3600 = 26,676$ cu.ft. of sewage, or assuming 95% moisture and 150 P.P.M. removal the volume of sludge is

$$\frac{26,676 \times 7.48 \times 8.34 \times 150 \times 1.02}{1,000,000} = 252 \text{ cu.ft., which}$$

may be conveniently cared for in the bottom of the tank if placed on the conventional one on twelve slope.

$$r = 17 \quad \frac{17}{12} = 1.42' \quad (\text{Consider as } 1' \text{ deep})$$

$$\text{Vol.} = A^2 3.14 \times 1/3 = \frac{17 \times 17 \times 3.14}{3} = 300 \text{ cu.ft.}$$

of storage which is o.k.

Inlet:

The inlet will be a pipe in the center discharging vertically and surrounded by a perforated baffle extending two feet below the water surface. Provide a 12" free board in this tank to provide protection from wind currents.

Outlet:

The outlet will be an adjustable Wier around the circumference of the tank. Collect the effluent in a



channel of uniform size. The effluent to have a velocity of 1'/sec. or $.7808 = d^2$ $d = 10''$ wide and 10" deep, but to obtain a better depth make 6" wide and 12" deep, giving a velocity of 1.5'/sec.

Chlorinator:

Use a straight through tank 15 min. detention maximum flow of 351 G.P.M. = .7808 C.F.S.

$351 \times 15 = 5250$ gal. or 700 C.F. capacity.

Use a 5' wide tank 6' deep.

$L = \frac{700}{30} = 23.3'$ long, which is a long tank, so use two 12' tanks placed side by side. The flow going around the end acts as a baffle. Baffles should also be placed alternately rising to within 2' of the surface and going down to within 2' of the bottom. Use four baffles. An investigation of the head losses will now be made.

At end baffle the head loss is $\frac{v^2}{2g}$

At vertical baffles the head loss is $\frac{v^2}{2g}$

Total head loss due to baffles is $\frac{5v^2}{2g}$

Besides this loss there is also the loss in the channel. Use $N = .013$

$H.R. = \frac{A}{W.P.} = \frac{30}{16} = 1.87$ From chart page 130, Metcalf and Eddy, the problem is unsolvable, so solve by the Manning formula. $V = \frac{.7808}{30} = .026'/\text{sec.}$

$V = KR \frac{S^{1/2}}{2}$ $K = \frac{1.486}{0.013} = 11.4$ $S = \left(\frac{.026}{11.4 \times 1.52} \right)^2 = .00002'$. So total loss in this tank is

$\frac{5v^2}{2g} + 24(.00002) = \frac{5(.026)^2}{64.4} + 24(.00002) = .076$

So use a 1" fall and a cascading Wier to the river as

an outlet.

Sludge Digester:

340 P.P.M. suspended solids

50% removal in primary tank. S.G. = 1.02 92% moisture

40% removal in final tank. S.G. = 1.005 98% moisture

75% volatile

30% digestible

Primary Tank:

$340 \times .5 = 170$ P.P.M. Removal.

$170 - 170 (.75)(.3) = 132$ P.P.M. per day.

$\frac{132}{1,000,000} \times 1,000,000 \times 8.34 = 1100^\#/\text{million gallons}$

$.505 \times 1100 = 550^\#/\text{day}$

$\text{Vol.} = \frac{550}{.08 \times 1.02 \times 62.4} = 108$ cu.ft. of storage per day needed.

Assuming 120 day detention,

$\text{Vol.} = 108.0 \text{ cu.ft.} \times 120 = 12,800$

Final Tank:

$40 \times 340 = 136$ P.P.M. removed

$136 - 136 (.75)(.3) = 105$ P.P.M.

$\text{Vol} = \frac{405 \times 1,000,000 \times 8.33 \times .505 \times 50}{1,000,000 \times 1.005 \times 62.4} = 352 \text{ C.F./}$

day.

$\text{Vol} = 352 \times 120 = 42,500$ cu.ft. of storage, which is obviously a mistake in reasoning, so assume

90% removal

90% moisture content

30% digestible

75% volatile

3' for supernatant

$$340 \times 9 = 296.0 \text{ P.P.M. removal}$$

$$296.0 - 296 (.3)(.75) = 228 \text{ P.P.M. new each day.}$$

$$\text{Vol} = \frac{228 \times .505 \times 1,000,000 \times 8.34}{1,000,000 \times 1.02 \times 62.4 \times .1} = 140 \text{ cu.ft./day}$$

$$140 \times 120 = 17,400 \text{ cu.ft./day}$$

$$\frac{17400}{2200} = 7.9 \text{ cu.ft. storage per capita, which is well}$$

above the minimum of 50 cu.ft./capita standard set up

by the Michigan State Health Department.

Dimensions of tank:

Assume a 25' tank

$$3.14 r^2 d = 17,400 \quad d = \frac{17,400}{3.14 \times 157} = 35' \text{ deep,}$$

which is unreasonable. So assume a 15' tank with

3' of space allowed for supernatant.

$$15 \left(\frac{3.14 d^2}{4} \right) = 17400 \quad d^2 = \frac{17400 \times 4}{3.14 \times 15} = 1480$$

$$d = 38.5'$$

Use a 39' tank 15' deep with a 3' provision for supernatant.

Use valves of the ring type as used in East Lansing with provisions for drawing the sludge from three different levels. Use a floating bottom with a safety valve which will go off under a 2' head of water.

Design of Valve:

Assume an 8" relieve hole, 1' of water = $\frac{62.4}{144} = .433\#/\text{sq.in.}$

$$2 \times .433 = .866\#/\text{sq.in.}$$

$$\text{Total } P = .866 \times 144 \left(\frac{.7854 \times 64}{144} \right) = 43.7\# \text{ acting up.}$$

$$3.14 r^2 t d = 43.7\# \quad \text{Use C. I. } d = 490\#/\text{cu.ft.}$$

$$t = \frac{43.7}{3.14 \times 16 \times \frac{490}{1728}} = 3" \text{ thick. Use a 3" plate with}$$

guides which are easy running and prevent binding.

Gas Chamber:

Assume a gas production of 1 cu.ft./day/capita.

$2200 \times 1 = 2200$ cu.ft. of gas storage must be provided.

General operating head is 8" - 9" of water.

8" water = .3464#/sq.in.

$\frac{14.7}{15.04} \times 2200 = 2100$ cu.ft. of storage must be provided to care for the gas. This means a chamber

$$\frac{3.14 \times 39^2}{4} d = 2200 \quad d = 2$$

Provide a waste vent for burning the excess gas.

Sludge Beds:

Use 1 sq.ft. of sludge bed per capita.

$1 \times 2200 = 2200$ sq.ft. of bed required.

Use beds 20' wide.

$\frac{2200}{20} = 110'$ long, so use 2 - 55' beds 20' wide. The beds are to be of top 6" graded sand and 15" of graded gravel.

Design of a wall:

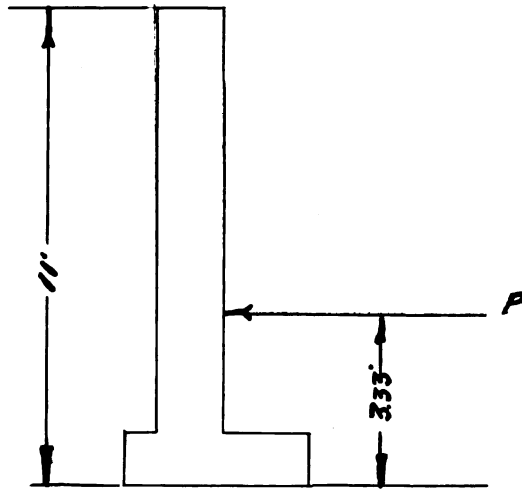
Proving the assumption and empirical rule that minimum wall thickness be 12" correct.

Design of final settling tank wall. Design when full and empty. 10' sewage and 2' free board, total $h = 12'$

$P = \frac{WL^2}{2} = \frac{62.4 \times 10 \times 10}{2} = 3120\#$ applied 3.33' from bottom of tank

$$Z M_a = 3.33 \times 3120 = 10,400 \#'$$

$$M = bKd^2$$



$$S_o = 750$$

$$F_s = 18000\#$$

$$N = 15$$

$$K = 127$$

$$d = \left(\frac{M}{Kd}\right)^{\frac{1}{2}} = \left(\frac{10400 \times 12}{12 \times 127}\right)^{\frac{1}{2}} = 8.99 \text{ effective depth}$$

$$+ 3" \text{ protective covering} = 12.00"$$

$$A_s = \frac{M}{f_s j d} = \frac{10400 \times 12}{18000 \times \frac{7}{8} \times 8.99} = .88 \text{ sq.in. of steel/ft.}$$

check when tank is empty.

$$P = \frac{C_e W L^2}{2} \quad \text{Use 1' footing} \quad \frac{0.27 \times 100 \times 11 \times 11}{2} = 1630\#$$

applied at 3.66' from bottom of tank, which gives

a smaller moment than the water. Steel must be placed

in the exterior face due to this cause, so

$$d^2 = \frac{1630 \times 3.66 \times 12}{12 \times 127} \quad d = 6.8" \quad \text{Use a 7", but we are}$$

$$\text{using a 9" effective depth so } A_s = \frac{M}{f_s j d} = \frac{1630 \times 3.66}{18000 \times \frac{7}{8} \times 9} =$$

.041 sq.in. So use $\frac{1}{4}"$ steel rods spaced 12" on center.

Check bond.

$$u = \frac{V}{Z_o f d} = \frac{1630}{\frac{7}{8} \times \frac{3.14}{4} \times 9} = 262\#/\text{sq.in.}, \text{ which is too high.}$$

$$\text{So, } Z_o = \frac{1630}{125 \times \frac{7}{8} \times 9} = 1.66 \quad 3.14 = 1.66 \quad d = .53$$

Use 7/8" circular rods on front face spaced 12" on centers to give corrent bond.

Check shear.

$$S = \frac{V}{bjd} = \frac{1630}{\frac{7}{8} \times 12 \times 9} = 18\#/sq.in., \text{ which is very safe.}$$

Now to return to the steel resisting the water pressure.

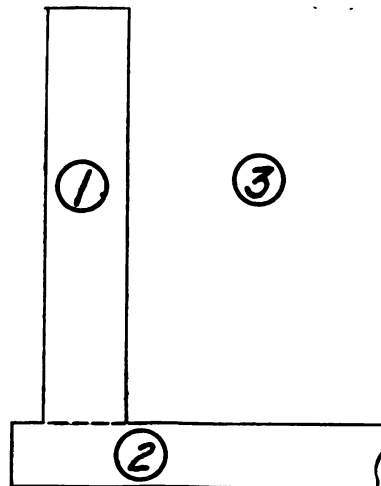
$A_s = .88 \text{ sq.in.}$ Try 1" sq.rds. 11" on centers, giving

$$u = \frac{3120}{\frac{12}{11} \times 4 \times \frac{7}{8} \times 9} = 92\#/sq.in., \text{ which is o.k.}$$

$$v = \frac{3120}{12 \times \frac{7}{8} \times 9} = 33\#/sq.in., \text{ which is safe.}$$

Design of the footing:

Tank is 10' wide so use $\frac{W}{2} = 10'$ for extension of footing. Front to have a 6" extension.



$$W_1 = 12 \times 1 \times 150 = 1800$$

$$W_2 = 1 \times 11.5 \times 150 = 1730$$

$$W_3 = 62.4 \times 10 \times 10 = 6240$$

$$\text{Total} \quad 9770\#$$

$$ZM_o = 0$$

$$\frac{1800 (1.0) + 1730 (5.75) + 6240 (6.5)}{9770} = 5.34' \text{ resultant}$$

acts from toe. Vertically, reaction cuts base at

$$x = \frac{3120 \times 4.33}{9770} = 1.375$$

$$5.34 - 1.38 = 3.96' \text{ from toe.}$$

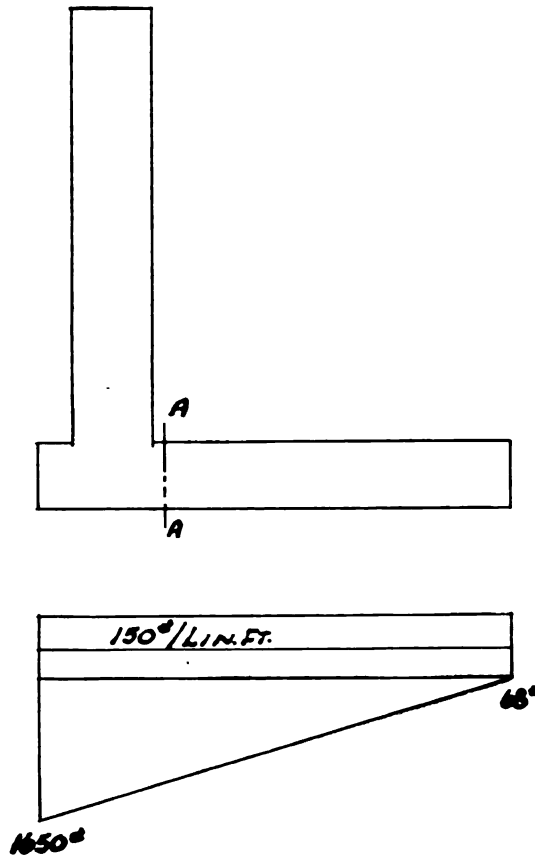
$$e = 5.75 - 3.96 = 1.76 \quad \frac{11.5}{6} = 1.92 \text{ allowable.}$$

$$P = \frac{V}{b} \left(1 \pm \frac{6e}{b}\right) = \frac{9770}{11.5} \left(1 \pm \frac{6(1.79)}{11.5}\right) = 68^{\#} \text{ and } 1650 \text{ sq.}$$

ft. which is none too good and should be corrected.

Maximum bending moment occurs at section A - A.

B.M. = Area of shear diagram.



$$B.M. = \frac{(1650 - 68) 10 \times 10 \times 10}{11.5 \times 2 \times 3} + \frac{68 \times 10 \times 10}{2} - 6240 \times 5 - 150 (10) \left(\frac{10}{2}\right) = 12,300^{\#}$$

$$D = \left(\frac{12300}{127}\right)^{\frac{1}{2}} = 9.8" + 3" \text{ cover} = 13" \text{ deep. So increase depth to } 13".$$

The footing could be redesigned, but there is only one flexible dimension and that is the thickness of the footing, so the new thickness would have to do. The footing is designed with a consideration that

1. e is too large
2. Footing thickness is not quite thick enough
3. Variation of pressure is too great

Design of Tank When Empty:

Consider a 5' head of water operating on the tank. The uniform load up is $5 \times 62.4 = 150$ (1) = 162# acting up per lineal foot.

Consider as a continuous beam $B.M. = \frac{WL^2}{10} = \frac{162 \times 400}{10} = 6480\#$, which is less than B.M. tank was designed, and so for speed and convenience put the same amount of steel in top and bottom of footing.

$$A_s = \frac{12300 \times 12}{18000 \times \frac{7}{8} \times 10} = 0.96 \text{ sq.in. per lineal ft.}$$

Try 1" sq. rds. 12" on centers.

$$u = \frac{9770}{4 \times \frac{7}{8} \times 10} = 250\#/\text{sq.in.}, \text{ which is too great.}$$

Use $\frac{1}{2}$ " rods 3" on centers.

$$u = \frac{9770}{4 \times 2 \times \frac{7}{8} \times 10} = 125\#/\text{sq.in.}, \text{ which is o.k.}$$

$$v = \frac{V}{bjd} = \frac{9770}{12 \times \frac{7}{8} \times 10} = 93\#/\text{sq.in.}, \text{ which is too}$$

high and should be corrected to a new depth of $\frac{93}{50} \times 10 = 18.5"$ to insure safety. If this increase is made the area of the steel may be cut down to

$$A_s = \frac{12300}{\frac{7}{8} \times 18.5 \times 12} = .53 \text{ sq.in. per lineal foot.}$$

So try $\frac{1}{2}$ " sq. bars - 6" on centers, giving .50 sq.in.

of steel per lineal foot. $u = \frac{9770}{4 \times \frac{7}{8} \times 18} = 156\#/\text{sq.in.}$, which is too high.

So use $\frac{5}{16}$ " sq. bars placed 3" on centers giving
 $u = \frac{9770}{4 + \frac{20}{16} \times \frac{7}{8} \times 18} = 122\#/\text{sq. in.}$, which is o.k.

Place temperature steel in wall to .3 of 1% of wall area, 66% in front on exposed wall (tank interior) and 33% in rear wall.

Service Building:

The service building should be of good pleasing architectural appearance and yet be very functional.

The building should have two basements containing:

1. Sub basement - wet well, sewage pumps, slump to provide drainage and a ventilator.
2. Basement - screen room, heating plant, sludge pumps.

The main floor should have a good floor plan and contain a laboratory, a chlorination room, an office, a lavatory and storage space for mops, brooms, shovels, etc.

The walls should be brick exterior and 2" glazed wall tile interior.

There should be an electrical control panel to cover all the operations of the plant and many safety features for the chlorination room. The light switches should be on the outside of the room. The doors should swing out from the room and for this reason all of the doors are made to open outward instead of inward. The outside door to the chlorinating room should be of the ventilating type with a mechanical ventilator provided as an

additional safety measure. A gas mask should also be provided.

The laboratory should be equipped to run chemical analysis and B.O.D. tests, but not too extensively equipped, as the operator will probably be the type of man who knows very little or nothing about the technical side of sewage plant operation.

Design of the Heating Unit:

Factors to be used.

K_0 is conductivity of material in B.T.U./sq.ft./hr./°dff. F.

f_1 is the transmissibility coefficient of inside surface, which will be omitted in the case of a liquid in contact with a surface.

f_0 is the transmissibility coefficient of the internal surface.

K is the transmissibility coefficient.

$$K = \frac{1}{\frac{1}{f_1} + \frac{t}{K} + \frac{1}{f_0}}$$

Allowance should be made for wind direction and exposure.

Sludge Tank:

Use a 12" wall and 6" bottom and top for design purposes.

6" concrete, $K = 0.77$

12" concrete, $K = 0.48$

Assume 2' of cinder cover and complete submergence of tank in ground and 1' of air and gas space in the top of the tank.

$$K = \frac{1}{\frac{12}{3} + \left(\frac{1}{1.9} + \frac{6}{7.8} + \frac{1}{1.9}\right) + \frac{24}{1.0} + 1} = .0319$$

Top loss = $.0319 \times 1160 = 370$ B.T.U. per hr.

Bottom, $K = 35$ from lecture notes by F.R. Theroux

$35 \times 1160 = 40,500$ B.T.U./hr.

Side Walls:

$K = 20.8$ from notes on heat, lecture by Prof.

F. R. Theroux.

Area = $18 \times 3.14 \times 39 = 2200$ sq.ft.

$2200 \times 20.8 = 46000$ B.T.U.

Sludge itself:

Flow of 2080 gal./hr.

$\frac{2080 \times 1.02 \times 8.34 \times 342 \times .95}{1,000,000} = 58\#/\text{hr. of dry}$

sludge solids at 90% moisture. $10 \times 58 = 580\#$ of
sludge per hr.

Assume 40° F. for temperature of incoming sludge
and 85° F. temperature of sludge tank.

$(85-40) \times 1 \times 580 = 25,800$ B.T.U./hr.

Total B.T.U. lost in tank:

Top 370

Bottom 40,500

Sides 46,000

Sludge 25,800

Total 112,670 B.T.U./hr must be provided.

The Service Building:

The top floor only will be heated.

Area = $8.5 \times 20 \times 20 = 3400$ cu.ft.

Exposure factors:

North 1.30

East 1.15

South 1.0

West 1.15

1000

Item	Area in sq.ft.	Coef.	Exp.Factor	Temp.Diff. ° F.	B.T.U./ hr.
Windows					
N	28	1.13	1.30	70	2600
E	31	1.13	1.15	70	2720
S	40	1.13	1.00	70	3160
W	21	1.13	1.15	70	1920
Doors					
S	21	.75	1.00	70	1100
E	21	.75	1.15	70	1260
W	21	.75	1.30	70	1425
Walls					
N	150	.46	1.30	70	6300
E	148	.46	1.15	70	5500
S	139	.46	1.00	70	4800
W	179	.46	1.15	70	7150
Floor					
Top	400	.56	1.00	35	7800
Bottom	400	.62	1.00	20	<u>4500</u>
Total					50335

Typical computation - a north window

$$28 \times 1.13 \times 1.30 \times 70 = 2600 \text{ B.T.U./hr.}$$

Leakage loss = two complete air changes per hour.

$$2 \times 3400 = 6800 \text{ cu.ft./hr.}$$

$$.24 \times .075 \times 1 \times 70 \times 6800 = 8600 \text{ B.T.U./hr.}$$

Building total = 65,000 B.T.U./hr.

$$\text{Total B.T.U. required} = 65000 + 112670 = 177,670$$

Piping in Tank:

Use a 2.5'/sec. velocity of flow.

1 sq. ft. of pipe is equivalent to 15 B.T.U./hr/°F

Assume a 30° F. drop in temperature.

$$Q = \frac{112,670}{15 (125-95)} = \frac{112,670}{15 \times 30} = 250 \text{ sq.ft.}$$

Assume a 2" pipe.

$$3.14 d = \frac{3.14 \times 2}{12} = \frac{3.14}{6} = .502 \text{ sq.ft./lineal ft. of pipe.}$$

$\frac{250}{.502} = 500'$ of pipe, so $\frac{500}{3.14 d} = \frac{500}{3.14 \times 39} = 4.5$ times around tank, so go around tank 5 times on a 32' diameter.

Heating of Service Building:

Total B.T.U./hr required = 50,000 B.T.U./hr.

$$\text{Radiation} = \frac{\text{B.T.U. required}}{K (T_1 - T_2)} \quad \begin{matrix} T_1 = 135 \\ T_2 = 70 \end{matrix} \quad T_1 - T_2 = 65$$

K = 1.9 - 1.4, use 1.7

$\frac{50000}{1.7 \times 65} = 460$ sq.ft. of radiation are required, provide 500 sq.ft.

Gas Pipe Size:

Maximum production assuming 1 C.F./cap/day = 2200 C.F./day. 2200 C.F. of gas can be carried in a 3" pipe on a 0.2" drop in pressure.

Determination of Pumping Head & Tank Elevations:

Invert elevation of outfall = 90.63

Ground elevation at primary tank = 105.8 - 2.0 =

103.8 as elevation of inlet wier.

Head pumped against = 103.8 - 90.6 = 13.6 static head.

Head Losses Pump to Inlet Wier:

$$\text{Loss in a 5" check valve } 1.0 \frac{v^2}{2g} = 0.25$$

$$\text{Loss in a 5" gate valve } 0.5 \frac{v^2}{2g} = 0.12$$

$$\text{Loss in a 90" elbow } .25 \frac{v^2}{2g} = 0.06$$

Loss in 20' of 5" pipe (250 GPM)	= 0.40
Loss in 60' of 8" pipe	= 0.15
Loss in 2, 8", 90° elbows	$0.25 \frac{V^2}{2g} = 0.02$
Loss in 1 - 8" tee	$1.50 \frac{V^2}{2g} = 0.06$
Loss in 1 - 8" gate valve	$0.5 \frac{V^2}{2g} = 0.02$
Loss in channel computed	= 0.20
Loss thru inlet wier	$\frac{1.5 V^2}{2g} = 0.06$
Total	1.34

All losses are computed according to Schoder & Dawson, Page 179.

Pipe losses determined by diagram cover page, Babbitt and Dolan.

$$\text{Total head pumped against} = 13.2 + 1.34 = 13.54.$$

Most manufacturers recommend a pump which will deliver against a 14' head at the desired capacity. Elevation of inlet = $103.8 - 0.2 = 103.6$

Loss through Primary Tank:

$$R = \frac{60}{22} = 2.79 \quad V = 0.013' / \text{sec.}$$

$S = 0.0000001$ from diagram, Page 307, Schoder and Dawson, which is too small to be considered. So this loss will not be computed for the other tanks.

Loss of head over Wier in primary tank using Francis formula $\frac{h^2}{3} = \frac{Q}{3.33h}$ $h = 10'$ $Q = .7808 \text{ C.F.S.}$

$$\frac{h^2}{3} = \frac{0.7808}{3.33 \times 10} = 0.0235 \quad h = (0.0235)^{\frac{3}{2}} \quad h = .003'$$

head is required for this discharge, allowing 0.5' depth of channel and a 0.20' loss through the channel, giving an elevation of $103.6 - 0.5 - 0.2 = 102.9$ as the elevation of the outlet pipe to the primary tank.

Loss in pipe between primary & aerator = 0.15
 Loss in 2 - 90° - 8" elbows = 0.02
 Loss in 1 - 8" check valve = 0.02
 Loss in channel (computed) = 0.044
 Loss in entry port = 0.06

Total 0.294

Elevation of inlet port invent = 102.9 - 0.30 = 102.6

The loss through the aerator will be considered negligible.

Loss over Outlet Wier:

$L = 20$ $Q = 0.98$ C.F.S.

$$h = \left(\frac{Q}{3.33L} \right)^3 = \left(\frac{0.98}{3.33 \times 20} \right)^3 = 0.002'$$
 head loss over the wier.

Allowing an 8" depth of outlet channel and 0.1' loss through the outlet pipe gives 102.6 - 0.64 - 0.1 = 102.6 - 0.74 = 101.86 as the elevation of the outlet to the aeration tank.

Loss in 100' - 8" pipe = 0.25

Loss in 2 - 90° - 8" elbows = 0.02

Total friction loss = 0.27

Loss in inlet channel = 0.20

Loss in inlet port $\frac{v^2}{2g}$ = 0.04

Total loss = 0.51

Elevation of inlet to final tank = 101.86 - 0.51 = 101.35. Loss over outlet Wier has been shown in the

previous tanks to be so small that the $\pm 0.1'$ adjustment provided in the wier will easily take care of any variation which arises.

Using a 10" channel and 0.05 loss in the channel the elevation of the outlet to the final tank which is also the inlet to the chlorinator is:

$$101.35 - 0.81 - 0.05 = 100.49$$

The head loss through the chlorination chamber has been computed to be 0.08', so elevation of outlet to chlorinator is

$$100.49 - 0.08 = 100.40.$$

Total loss through plant is $103.8 - 100.4 = 3.4'$ which is consistent with a good economy of head.

Estimate of Costs:

The estimate of cost will be in two parts. The first will be an estimate of concrete work and service building. The second part will be an estimate of the mechanical equipment, such as pumps, piping, skimmers, etc.

The cost of concrete and forms is based on a value obtained from Arthur Sargent, estimator for the Michigan State Parks Division. The cost of mechanical equipment is based on values obtained from Professor F. R. Theroux.

Excavation:

Item	Ground Elev.-corners				Aver- age	Base Elev.	Diff.	Size	Vol. cu. yd.	Cost
	N.E.	S.W.	S.E.	N.W.						
Service Bldg.	106.2	106.5	106.4	106.3	106.4	89.4	17.0	20x20	252	215.00
Primary Tank	105.1	105.8	105.7	105.2	105.5	97.0	8.5	10x140	52	44.30
Aeration Tank	104.3	104.7	104.6	104.5	104.5	90.5	14.0	20x120	89	76.00
Final Tank	104.2	104.4	104.3	104.3	104.3	91.8	12.5	34'	422	358.00
Chlor- inator	104.3	104.3	104.3	104.3	104.3	97.3	7.0	12x12	38	32.50
Dist. Box	104.8	104.7	104.7	104.8	104.7	101.7	3.0	4x6	1	0.85
Sludge Dig.	104.9	105.0	105.1	104.9	105.0	86.5	19.5	20'	230	196.00
Sludge Drying Beds	103.2	103.8	103.9	103.3	103.0	103.0	0.6	20x110	5	4.25
Ditching	2200 lineal ft. at an average depth of 6'±								490	415.00
Total									\$1,340.90	

Based on a value of \$0.85/cu.yd. as advised by
A. L. Sargent.

Service building figured on a basis of \$1.00/cu.ft.
8 x 20 x 20 = \$3400 for building and laboratory
equipment such as shelves, racks, special sinks,
ovens, etc.

Concrete:

Concrete based on \$6.50/cu.yd.

Item	Length'	Width'	Depth'	Vol.in cu.yd.	Cost
Service Bldg.					
1. 3 floors	20	20	0.5	22.2	\$145.00
2. Walls	20(5)	18.5	1.0	69.0	445.00

Item	Length'	Width'	Depth'	Vol.in cu.yd.	Cost
3. Footings	80	3	1.0	8.8	\$ 56.10
4. Wet Well	60	10	0.5	11.1	71.80
Primary Tank					
1. Walls	3(70)	8.5	1.0	66.0	429.00
2. End Walls	2(23)	8.5	1.0	14.5	94.50
3. Bottom	10	140	0.5	25.80	167.50
4. Footings	3	256	1.0	28.5	186.00
5. Hoppers	5	5	0.5	2.0	13.00
Aerator					
1. Walls	60(2)	14	1.0	62	404.00
2. Walls	40	14	1.0	21	136.50
3. Walls	40(3)	14	1.0	63	410.00
4. Wall	20	14	1.0	11	71.50
5. Bottom	20	120	0.5	46	300.00
6. Footings	260	3	1.0	29	188.50
Final Tank					
1. Wall	107	12.5	1.0	50	325.00
2. Bottom (17 x 3.14 ²)			0.5	17	110.50
3. Footings	107	3.0	1.0	12	78.00
Chlorinators					
1. Walls	12(3)	7.0	1.0	9	58.50
2. Walls	10(3)	5.0	1.0	2	13.00
3. Cascade	10	5	1.0	2	13.00
4. Footings	46	3	1.0	5	32.50
Sludge Division Box					
1. Walls	6(2)	3	1.0	1	6.50
2. Walls	4(2)	3	1.0	1	6.50
3. Bottom	4	6	1.0	1	6.50

Item	Length'	Width'	Depth'	Vol.in cu.yd.	Cost
4. Footings	20	3	1.0	3	\$ 19.50
Sludge Digester					
1. Wall	63	19.5	1.0	45	293.00
2. Cover (10 ²)	3.14		0.5	6	39.00
3. Bottom (10 ²)	3.14		0.5	6	39.00
4. Footing	63	3.0	1.0	7	45.50
Sludge Drying Beds					
1. Walls	110(3)	1.0	0.5	6	39.00
2. Footings	330	1.0	0.5	6	39.00
Total for concrete					4,180.80

Steel:

Steel to be computed at $3\frac{1}{2}\phi$ per pound in place. The weight of steel to be $2\#$ /sq.ft. of area as recommended by Arthur Sargent.

Area of concrete = 13312.0 sq.ft. + area of tank.

Bottom = 6800 Total area = 20,120 sq.ft.

$20,120 \times 2 = 40,240\#$ of steel to be used.

$40,240 \times 3.5 = \$1,408.40$, cost of steel in place.

Form Work at 10ϕ per sq.ft.:

Item	Length	Depth	Area sq.ft.(2 sides)	Cost
Service Bldg.				
1. Walls	20(5)	18.5	3700.0	\$370.00
2. Wet Well (3 sides 2 walls)	20	10	600.0	60.00
3. Footings	80	1.0	160.00	16.00
Primary Tank				
1. 3 walls	70	8.5	3580	358.00

Item	Length	Depth	Area sq.ft.(2 sides)	Cost
2. 2 Walls	20	8.5	680	\$ 68.00
3. Footings	250	1.0	500	50.00
4. Inlet & Outlet	40	1.0	80	8.00
Aerator				
1. Walls	2(60)	14	3360	336.00
2. Wall	40	14	1120	112.00
3. Wall	40	14	3360	536.00
4. Wall	20	14	560	56.00
5. Footings	260	1	520	52.00
6. Outlet & Inlet	80	1	160	16.00
Final Settling Tank				
1. Wall	107	12.5	2680	268.00
2. Footings	214	1.0	214	21.40
3. Motor Mount	4	10	160	16.00
4. Outlet & Inlet	107	1.0	214	21.40
Chlorinator				
1. Walls	12(3)	7	504	50.40
2. Walls	10(3)	5	300	30.00
3. Cascade	70	1.0	70	7.00
4. Footings	46	1.0	92	9.20
5. Outlet & Inlet	10	1.0	20	2.00
Sludge Division Box				
1. Walls	6	3	72	7.20
2. Walls	4	3	48	4.80
3. Footing	40	1.0	40	4.00

Item	Length	Depth	Area sq.ft.(2 sides)	Cost
Sludge Digester				
1. Wall	63.0	19.5	2520	\$252.00
2. Cover	314		344	31.40
3. Footings	126	1.0	126	12.60
4. Control Box	20	1.0	40	4.00
Sludge Beds				
1. 3 Walls	330	1.0	660	66.00
2. 3 Footings	330	1.0	660	66.00
Total				\$2,662.40

Mechanical Equipment Required:

Service Bldg.

250 G.P.M. pump to deliver against a 14' head	\$	350.00
200 G.P.M. pump to deliver against a 14' head		400.00
150 G.P.M. pump to deliver against a 14' head		400.00
100' - 10" pipe @ \$.25/#		390.00
3 - 4" bend @ 10¢/#		24.60
3 - 30" pieces 4" pipe @ 6¢/#		8.40
3 - 4" gate valves @ \$20.00		60.00
3 - 5" gate valves @ \$20.00		60.00
3 - 5" check valves @ \$35.00		105.00
3 - 30" pieces 5" pipe @ 6¢		10.50
4 - 5" bends @ 10¢/#		35.00
2 - 5" tees @ 6¢/#		24.00
120' - 5" pipe @ 6¢/#		225.00
2 - 5" gate valves @ \$20.00		40.00
2 - mechanical sludge removers 70' x 10' x 8' @ \$1400		2800.00
150' - 8" pipe @ 5¢/#		300.00
2 - 25 G.P.M. sludge pump @ \$750.00		1500.00

6 - 8" bends @ 10¢/#	\$ 120.00
200' 8" pipe @ 5¢/#	400.00
2 - 8" tees @ 10¢/#	60.00
2 - 8" crate valves @ \$50.00	100.00
200' - 4" pipe @ 6¢/#	174.00
4 - 90° - 4" bends @ 10¢/#	32.00
5 - aerators, 18000 C.F. circulation @ \$1000.00	5000.00
100' - 8" pipe @ 5¢/#	200.00
2 - 8" bends @ 10¢/#	40.00
1 - 34' mechanical sludge remover	800.00
100' - 8" pipe @ 5¢/#	200.00
300' - 8" pipe @ 6¢/#	180.00
50' - 8" pipe @ 5¢/#	100.00
500' - 2" pipe @ 6¢/#	300.00
1000' - 8" pipe @ 5¢/#	2000.00
4 - 8" bends @ 10¢/#	80.00
1 - 8" tee @ 10¢/#	30.00
1 - 10 G.P.M. hot water pump	30.00
Cover, sludge digester	1500.00
1 Boiler (150,000 B.T.U. Cap)	600.00
Flame Trap, Pressure Relief Valves, etc.	200.00
1 automatic flow recorder	350.00
1 chlorinator & connections	<u>1200.00</u>
Total	\$18,338.50

Total Cost of Plant:

Service Building	\$ 3,400.00
Concrete including steel	5,650.80
Form work	2,662.40

Excavation	\$ 1,340.80
Mechanical Equipment	<u>18,338.50</u>
	\$31,392.50
Engineer's fees (design, supervision, etc.)	<u>3,139.25</u>
Total	\$34,531.75

Capitalized Cost:

Page 20, Babbitt & Dolan

$$C.C. = C + \frac{O}{r} + D$$

$$C = \text{first cost} = \$34,500$$

$$O = \text{operating cost} = \$8/\text{million gal.} = 0.503 \times 360 \times 8 = \$1,440/\text{year.}$$

$$r = \text{rate of interest per period} = 3\%$$

$$D = \text{depreciation} = \frac{C}{(1+r)^n - 1} = \frac{34500}{(1.03)^{30} - 1} = \frac{34500}{1.62} = 21300$$

$$C.C. = 21300 + 1440 + 34500 = \$57,240$$

Annual Expense:

$$A_p = Cr + O + \frac{Cr}{(1+r)^n - 1}$$

$$A_p = 1440 + 1032 + 638 = \$3110 \text{ annual expense.}$$

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Schoder & Dawson - "Hydraulics"

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Steele - "Water Supply & Sewage"

Babbitt & Dolan - "Water Supply Engineering"

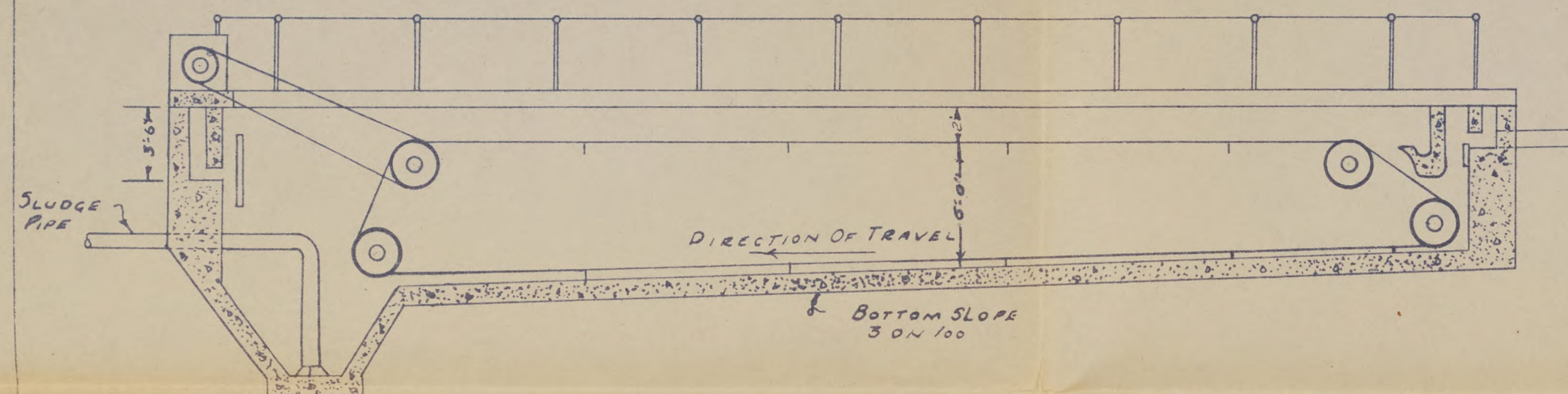
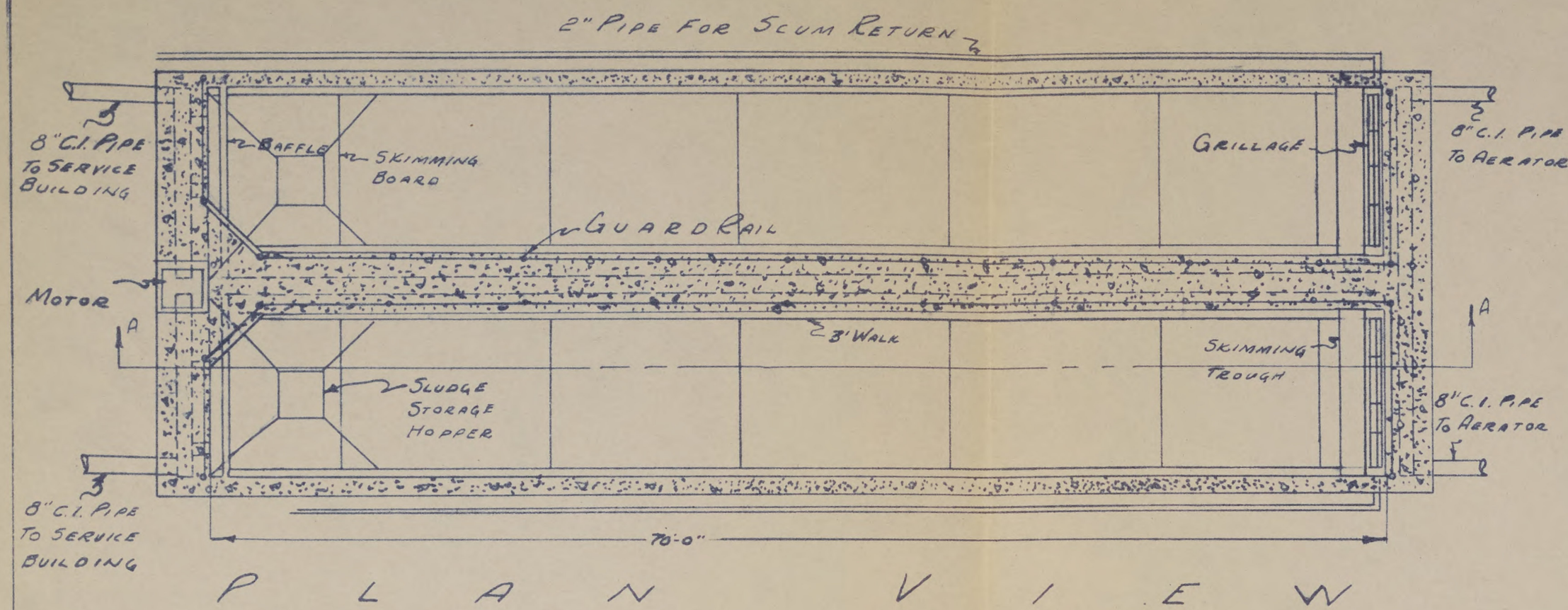
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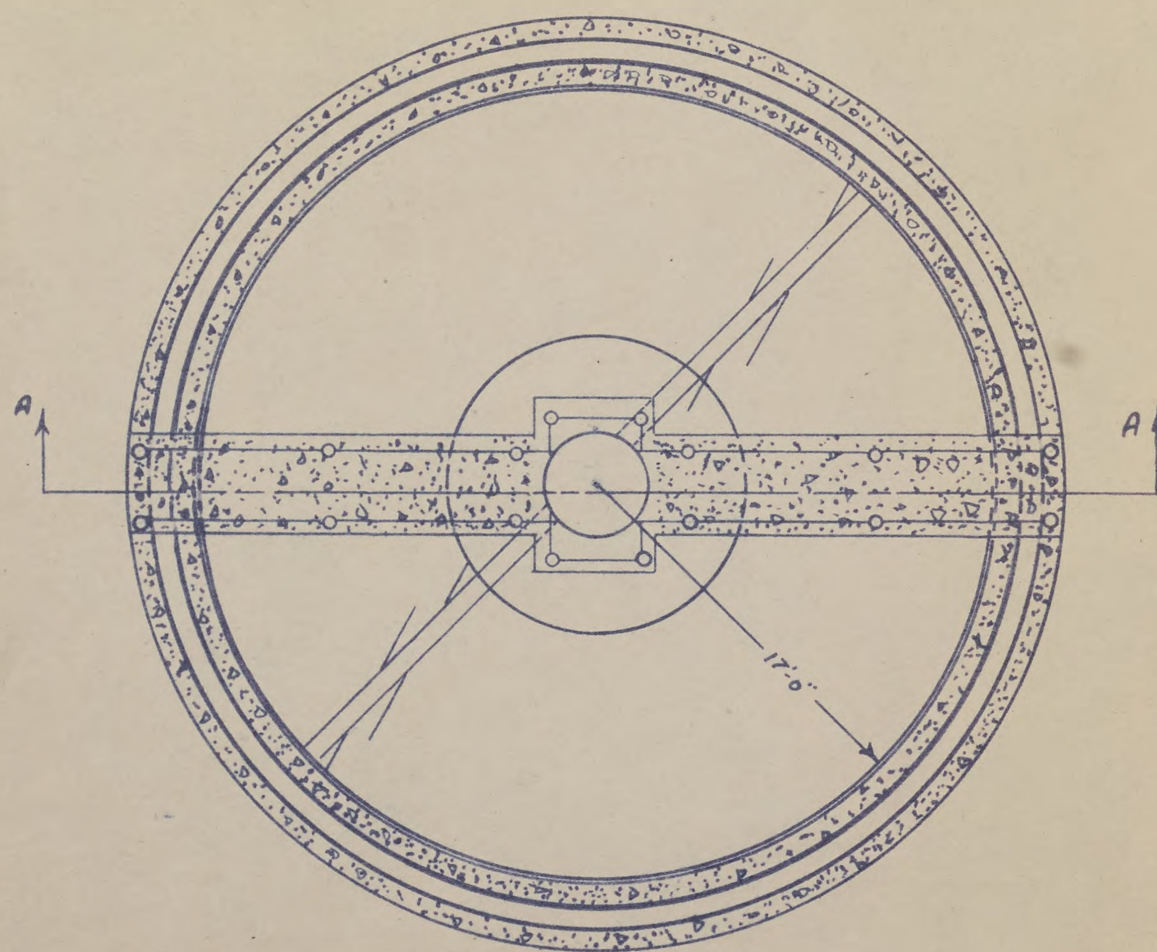
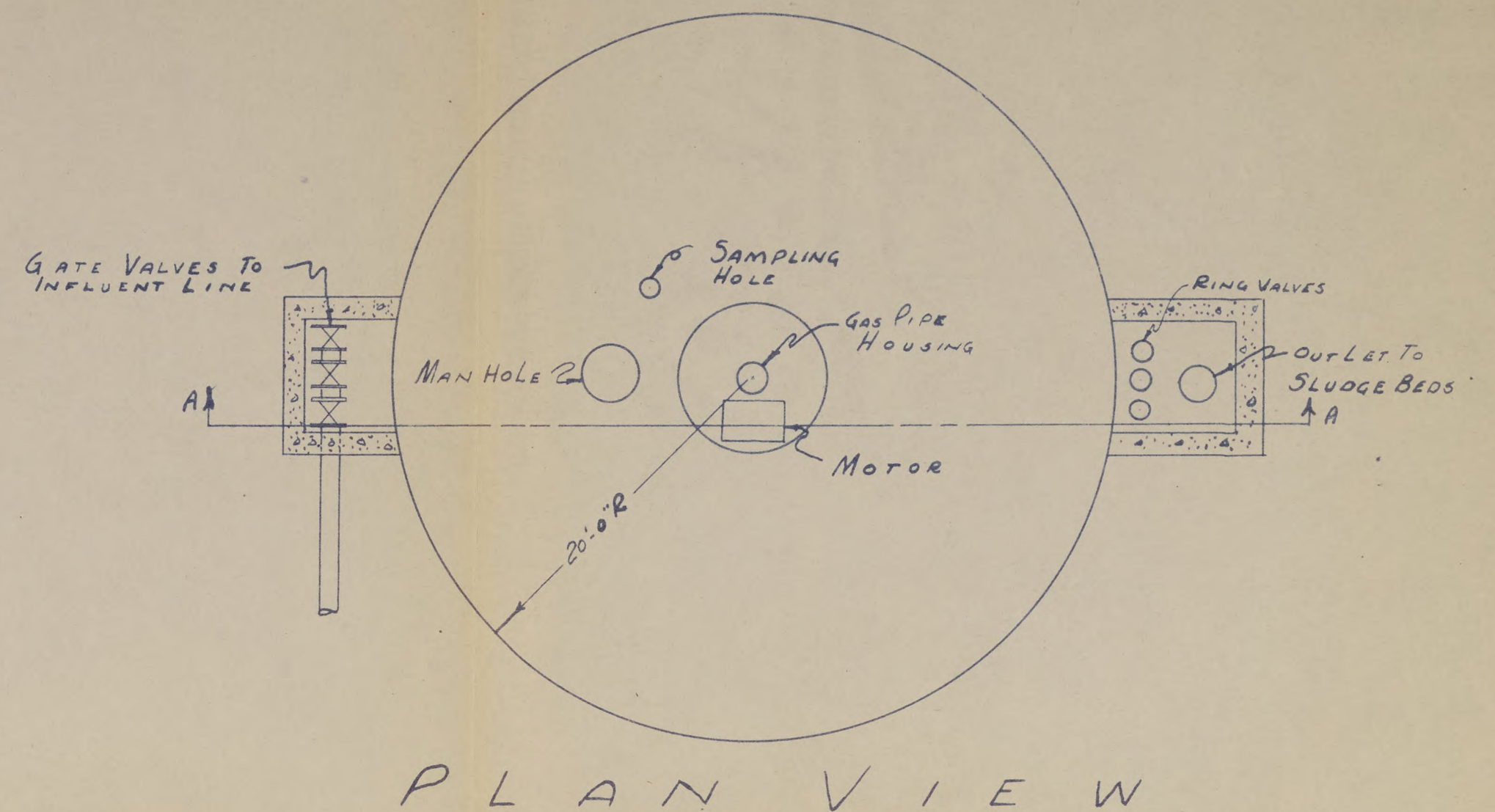
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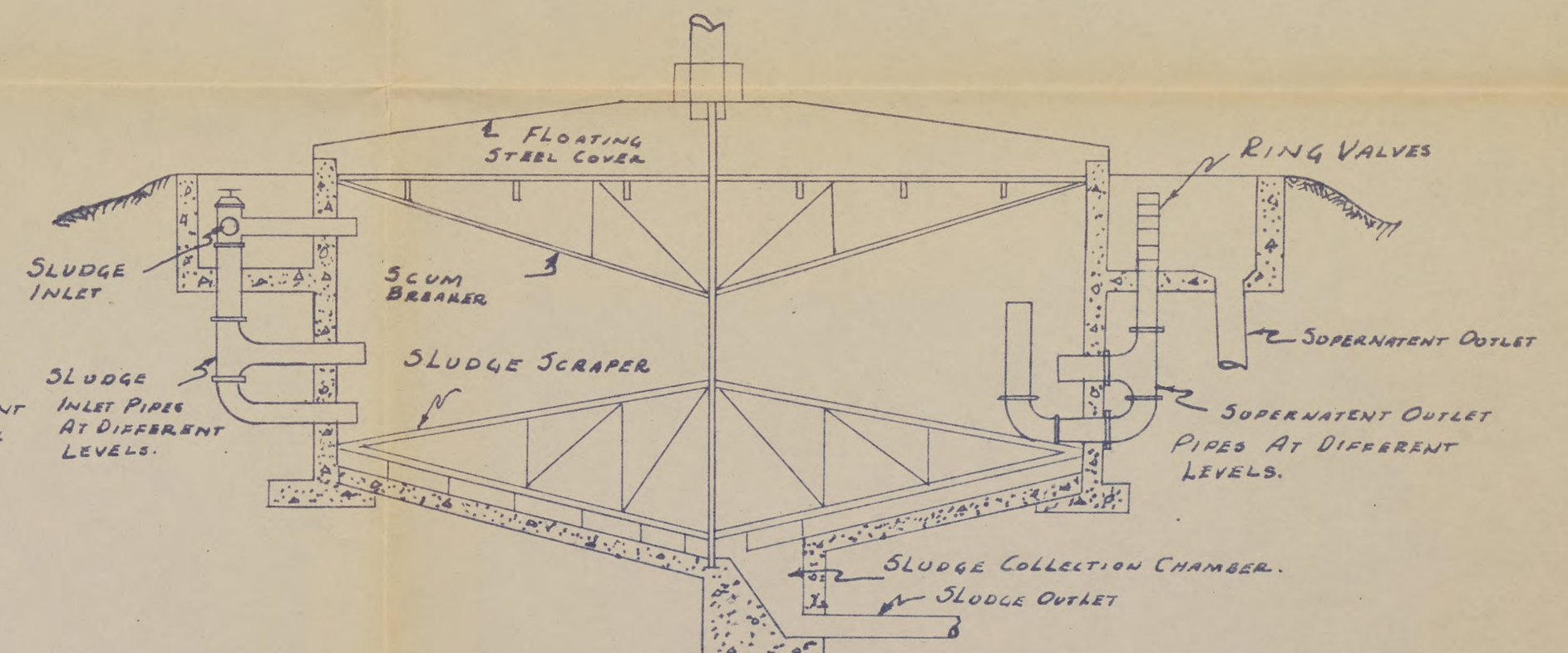
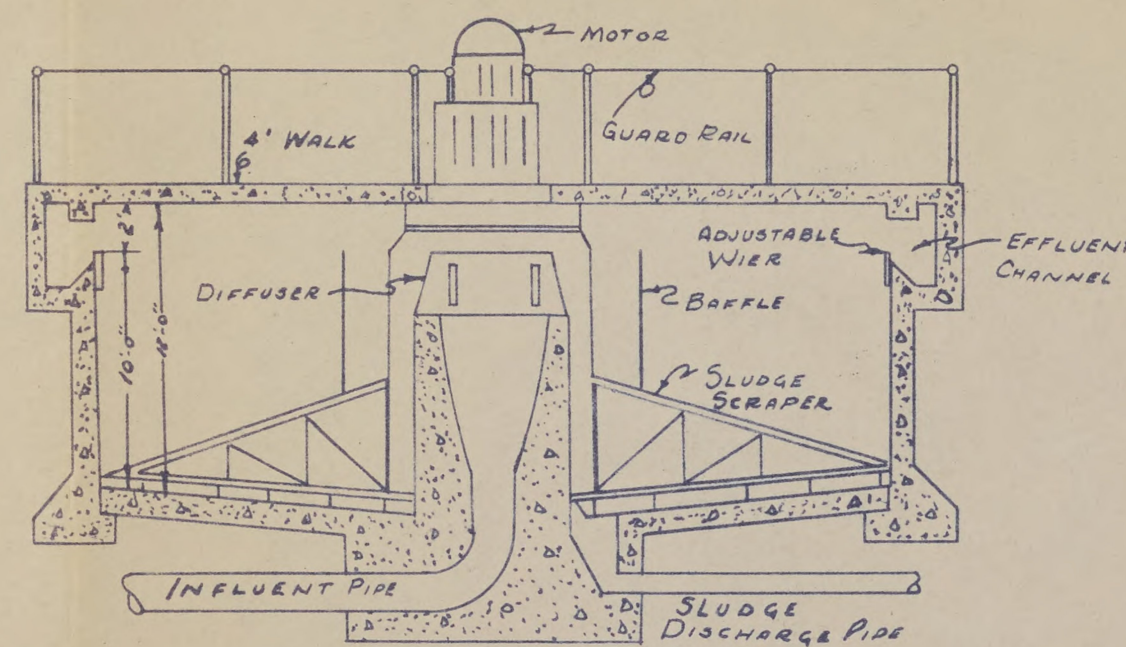
PLATE 1 DETAILS OF TANKS AND CHAMBERS



PRIMARY SETTLING TANK

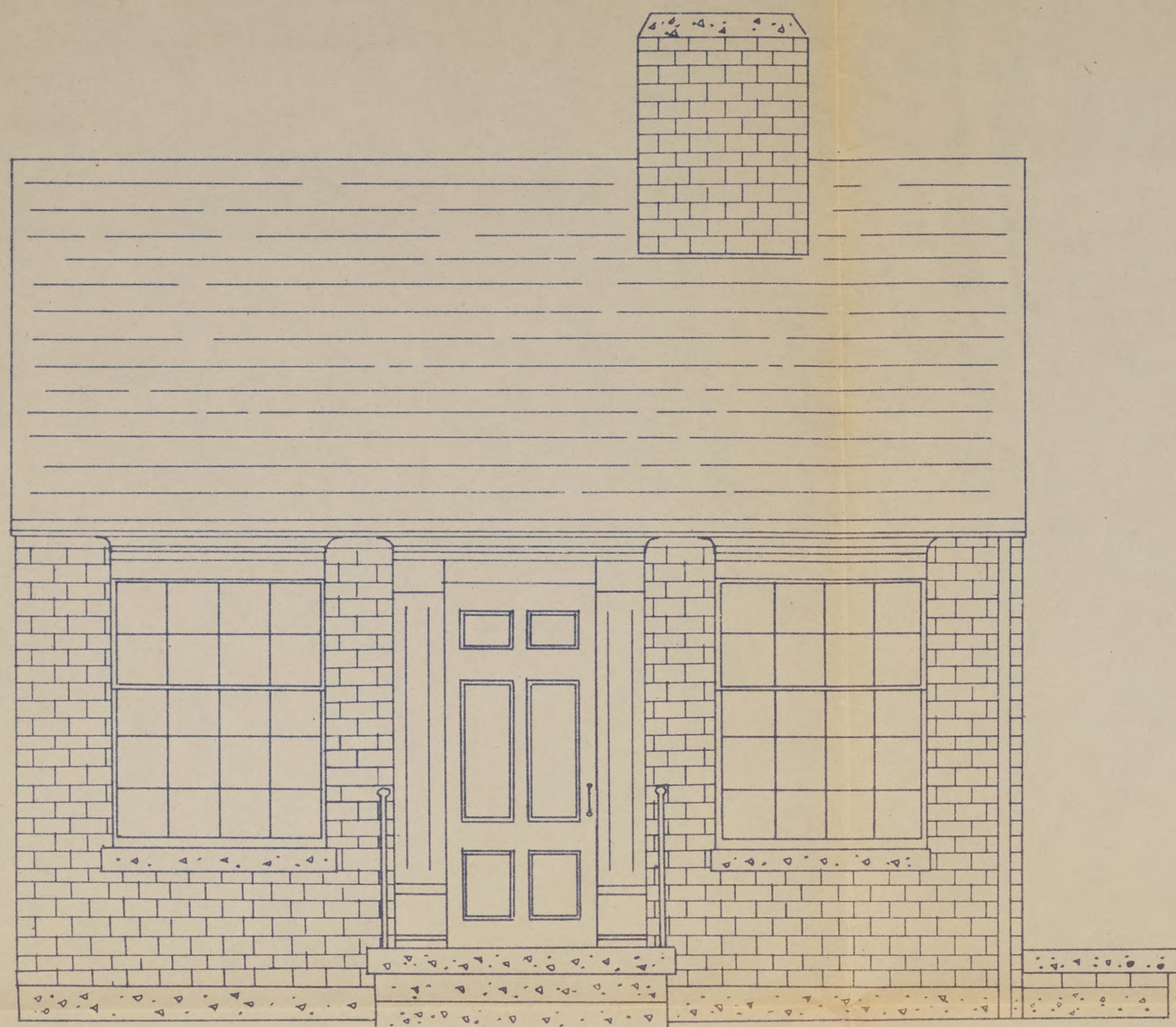


FINAL SETTLING TANK

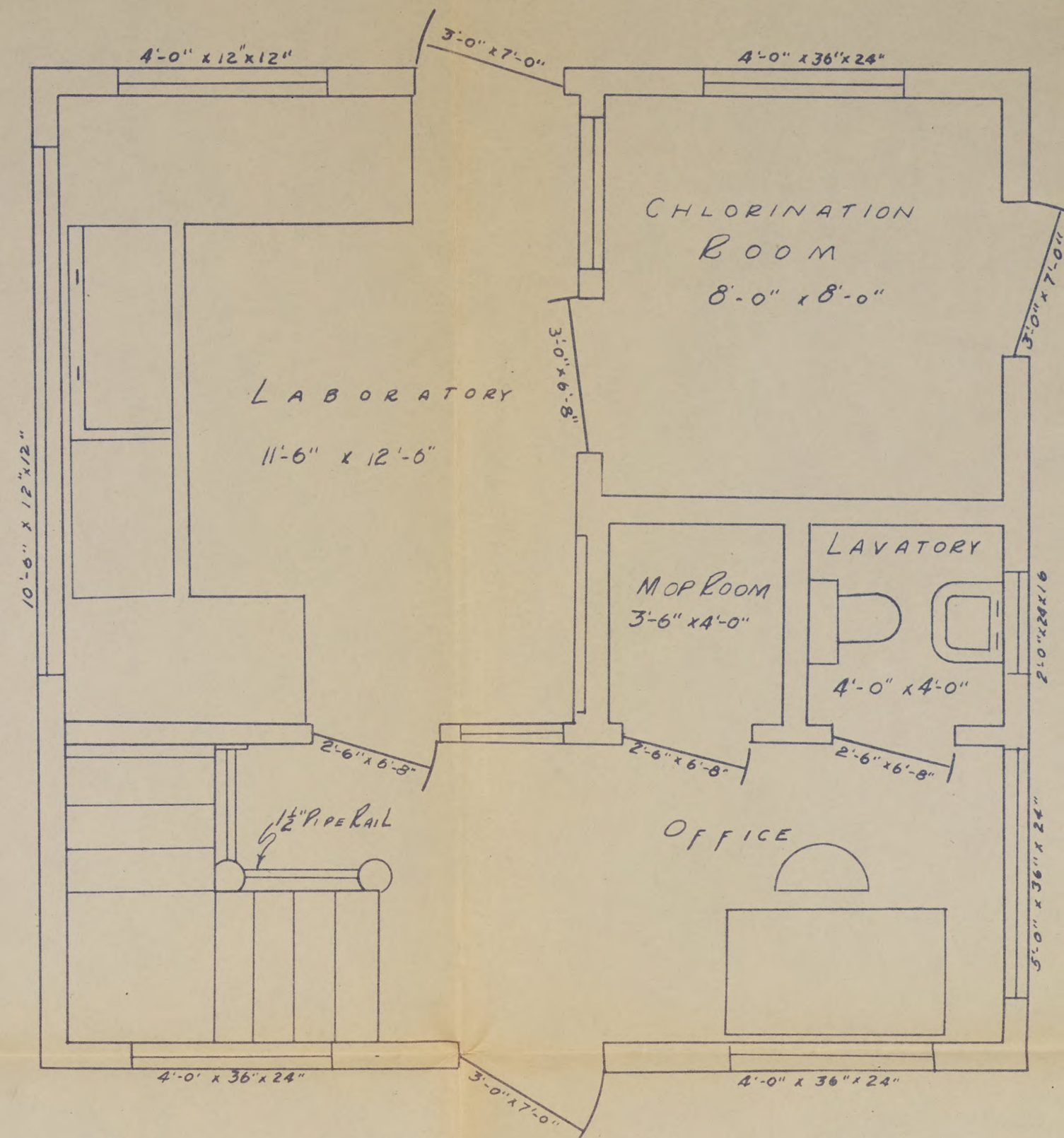


SLUDGE DIGESTION TANK

SCALE ON ALL TANKS 1 INCH = 8 FEET

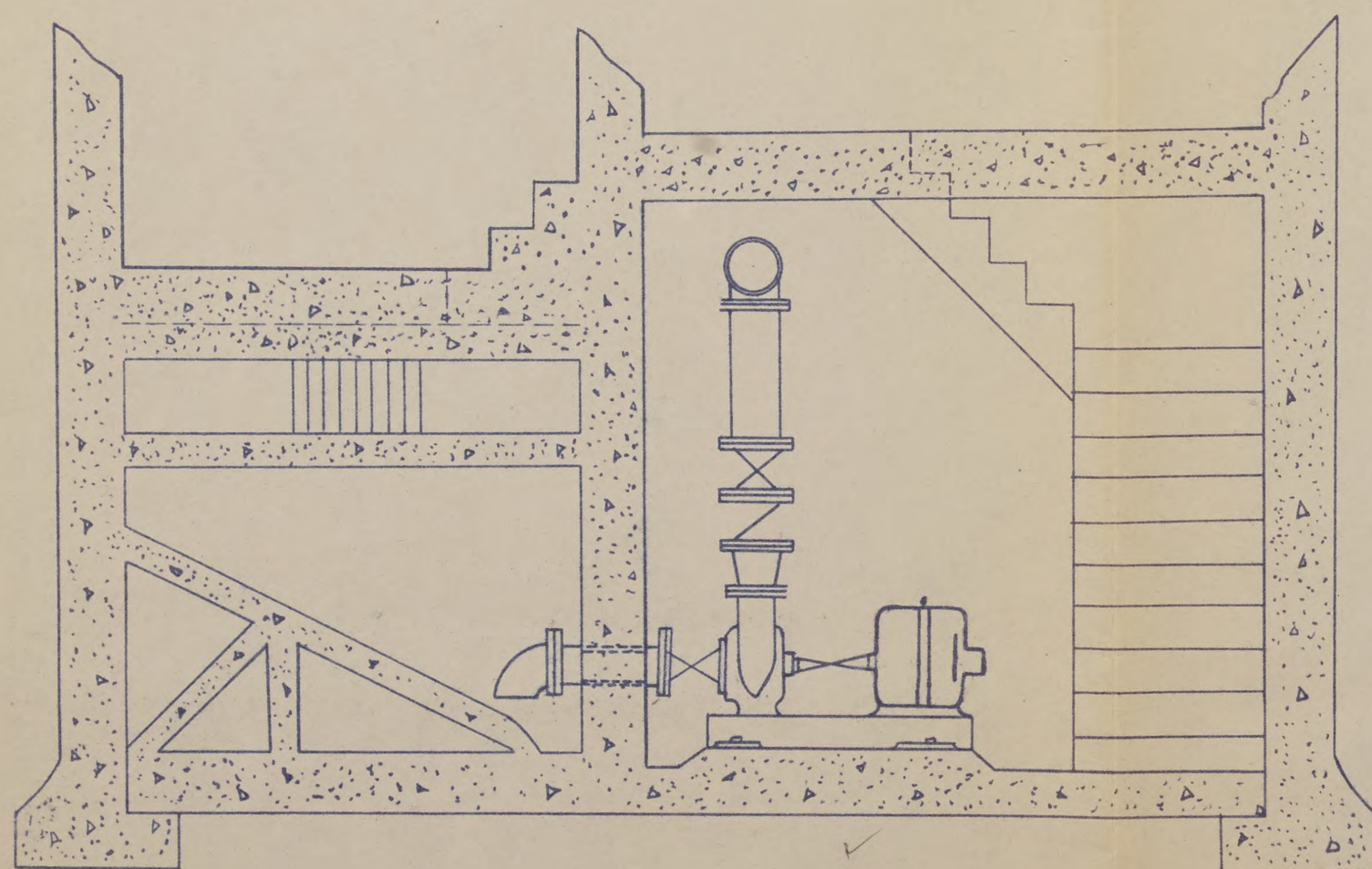


FRONT ELEVATION

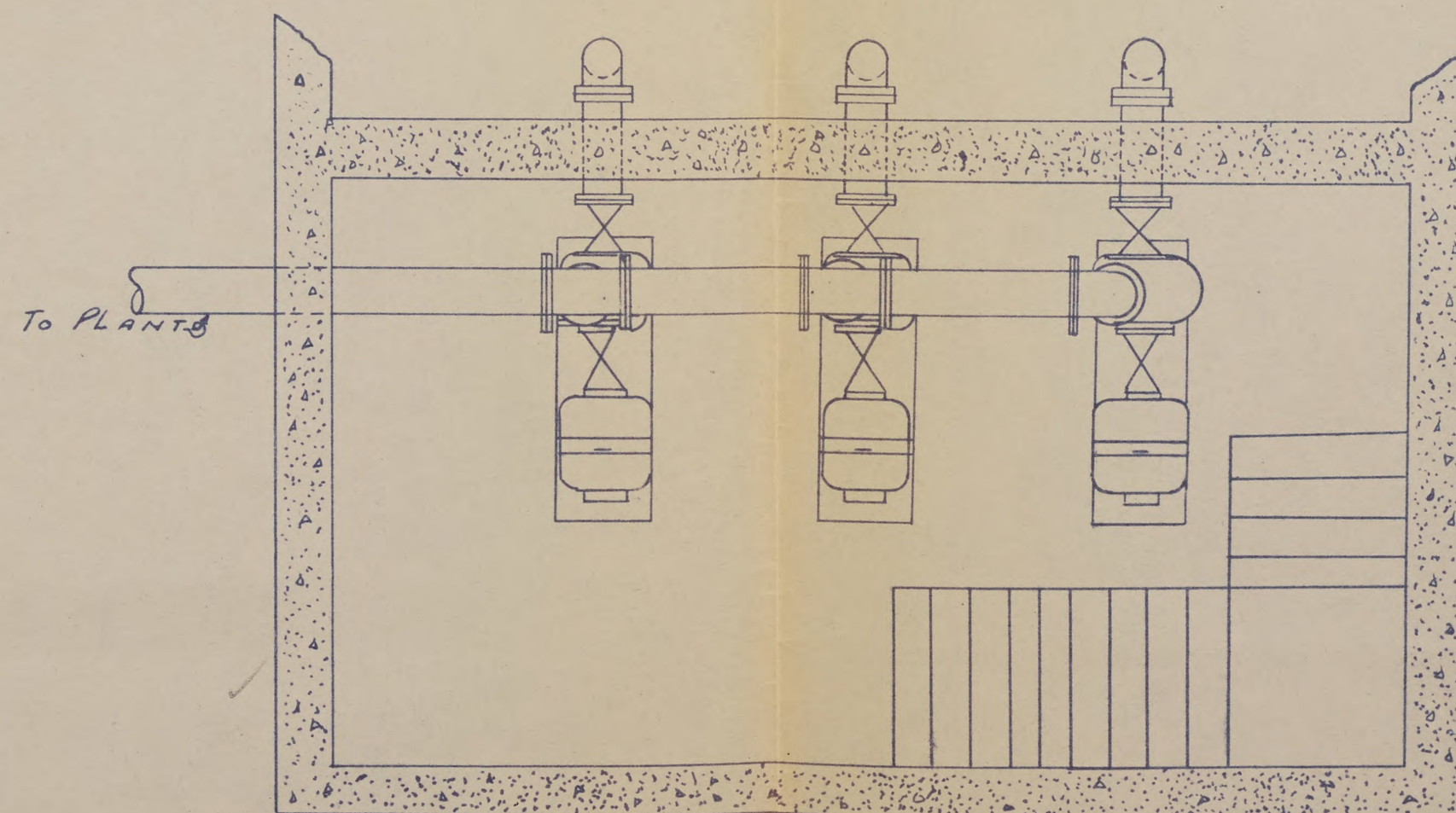


FIRST FLOOR PLAN VIEW

SCALE $\frac{3}{8}'' = 1'$



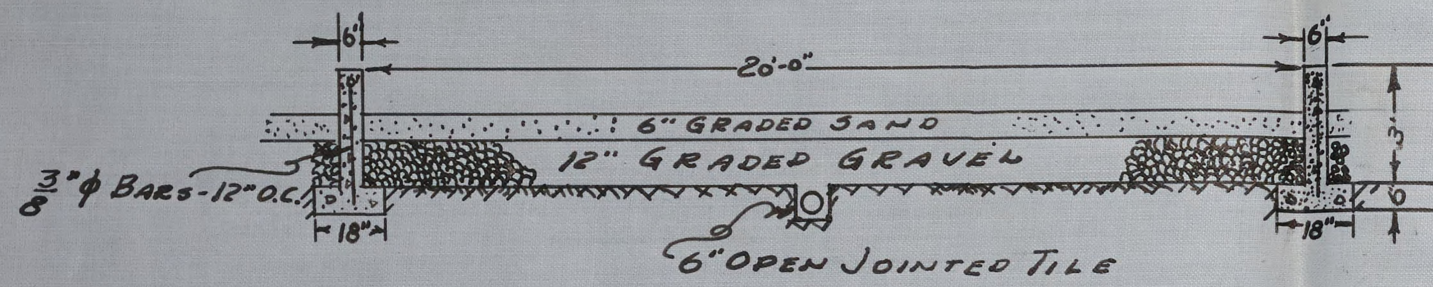
WET WELL & PUMP ROOM



PUMP ROOM PLAN VIEW

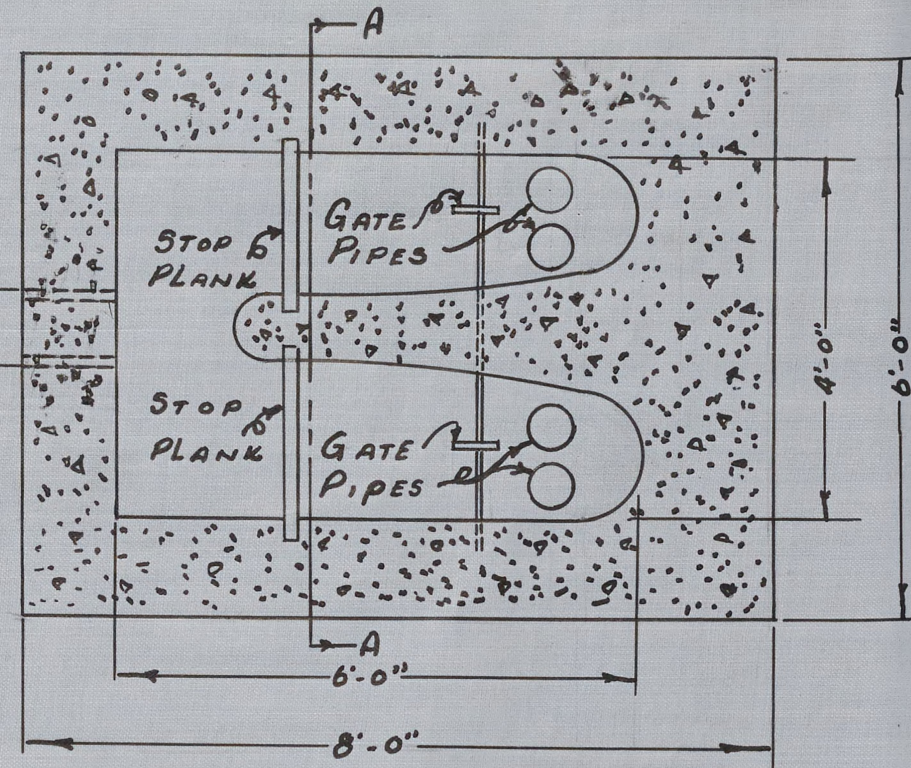
SERVICE BUILDING, SEWAGE TREATMENT PLANT EAST JORDAN, MICHIGAN

PLATE 2 DETAILS OF TANKS AND CHAMBERS

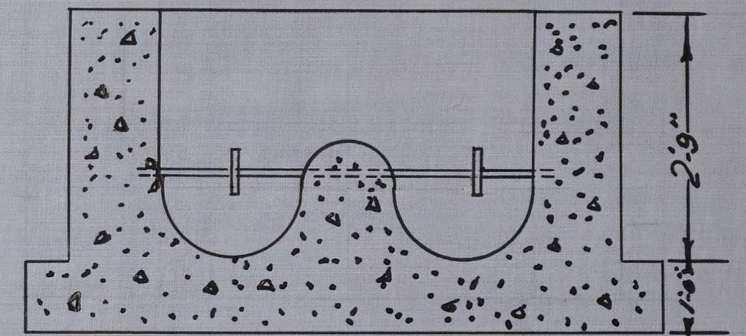


SECTION OF SLUDGE DRYING BED
SCALE $\frac{1}{4}'' = 1'$

8" C.I. PIPE
TO FINAL TANK

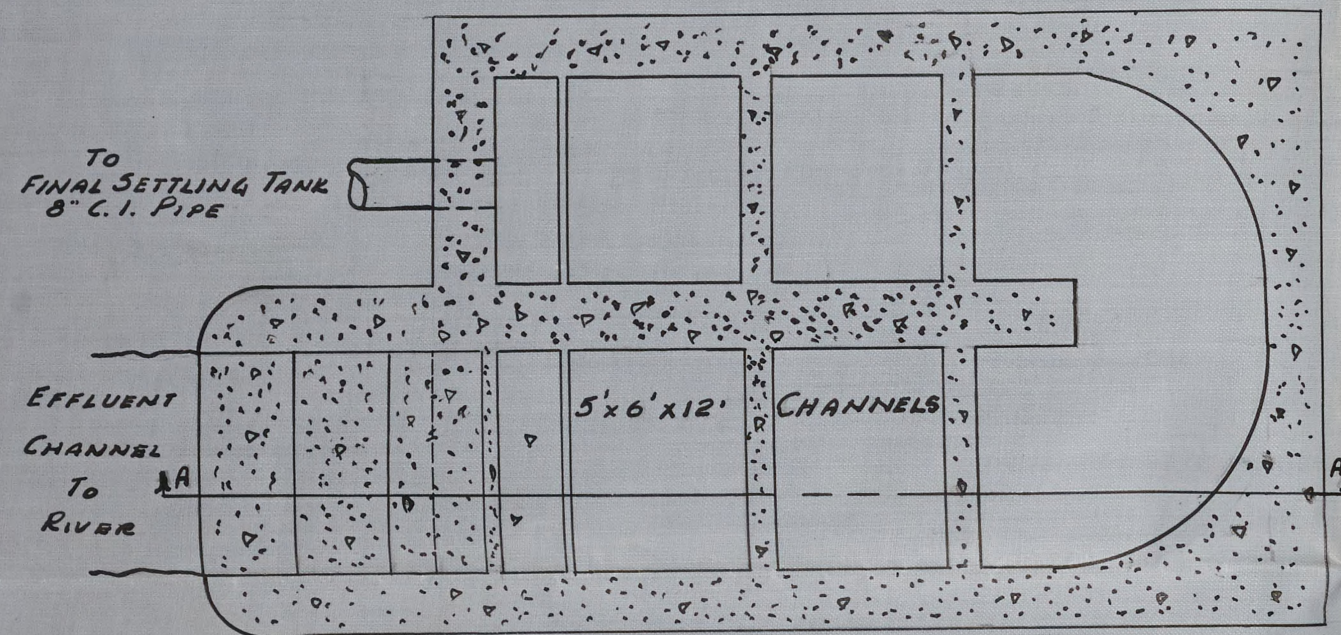


NOTE ON DIVISION
BOX CONSTRUCTION
THE RIDGE IN THE CHANNEL
DIVIDES THE FLOW EVENLY.
EACH SUB-CHANNEL HAS A
STOP PLANK IN IT, SO THAT
FLOW THRU IT MAY BE STOPPED.
EACH SET OF PIPES
SHOULD HAVE A GATE OF
THE TYPE USED IN THE
EAST LANSING PLANT

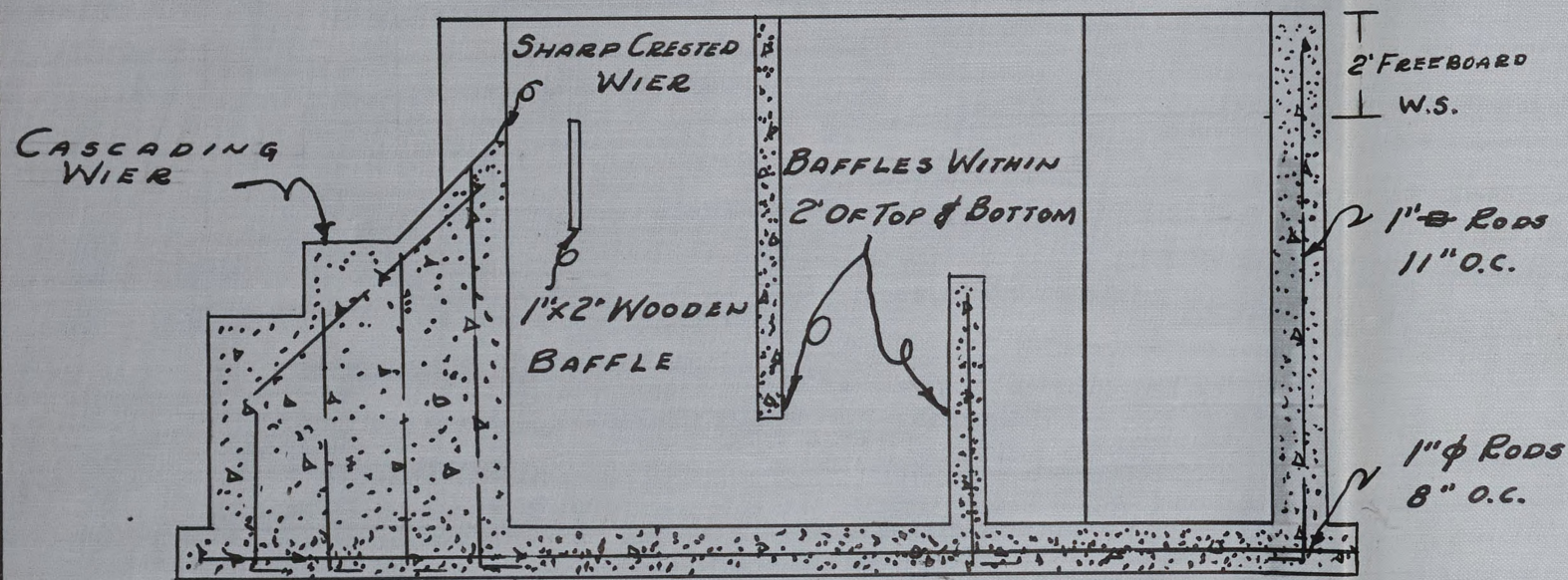


SECTION A-A

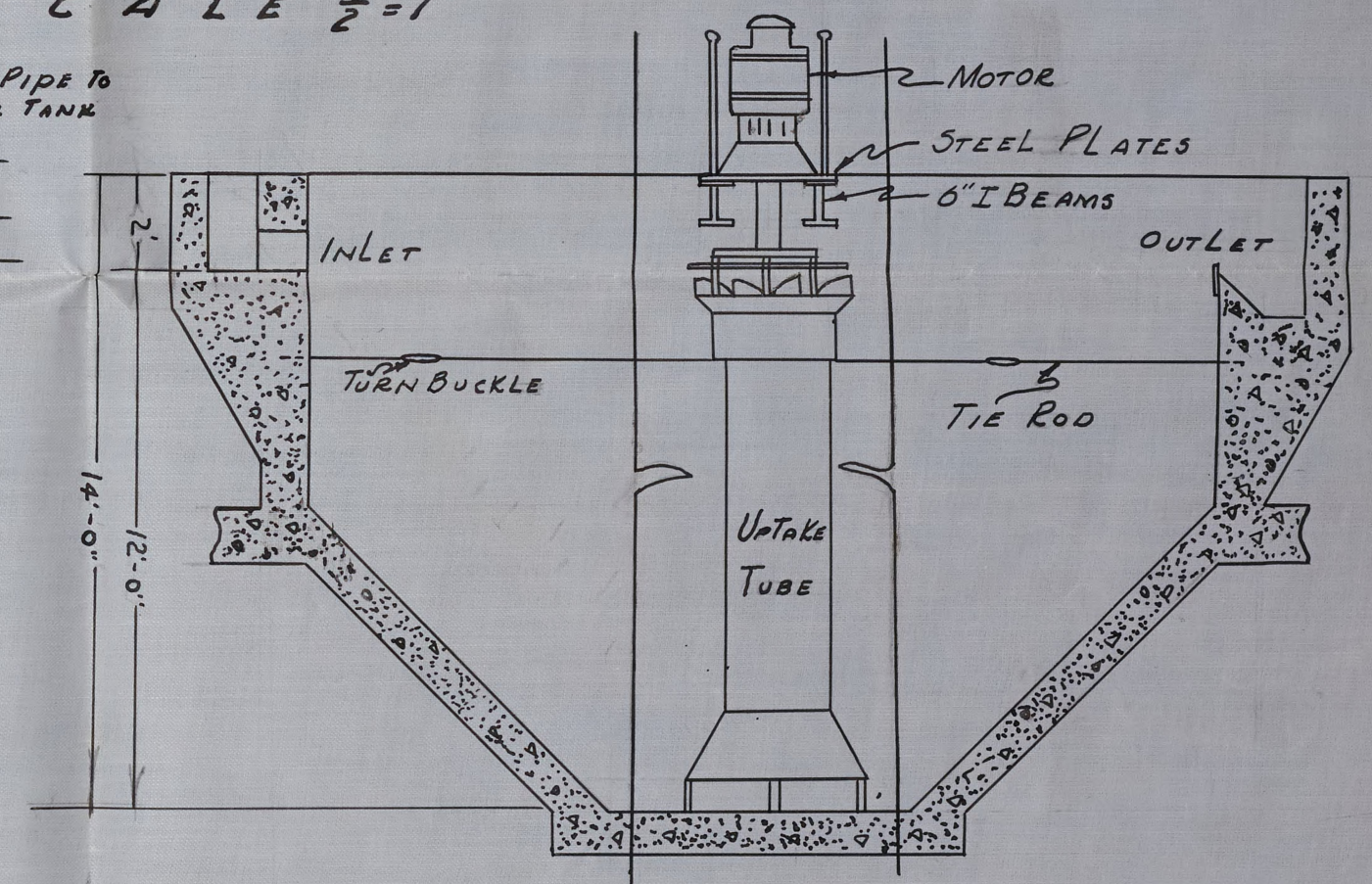
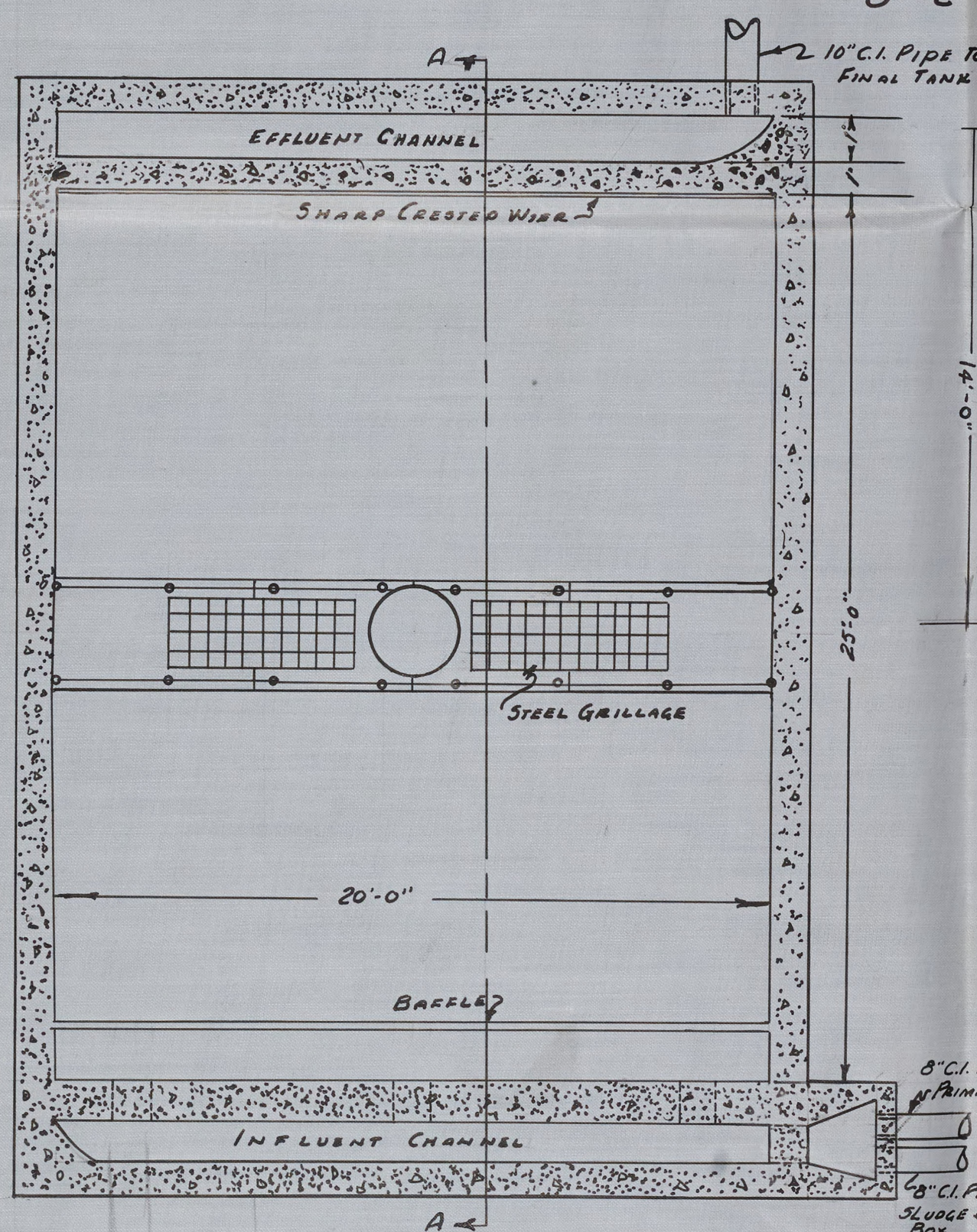
SLUDGE DIVISION BOX
SCALE $\frac{1}{2}'' = 1'$



PLAN VIEW

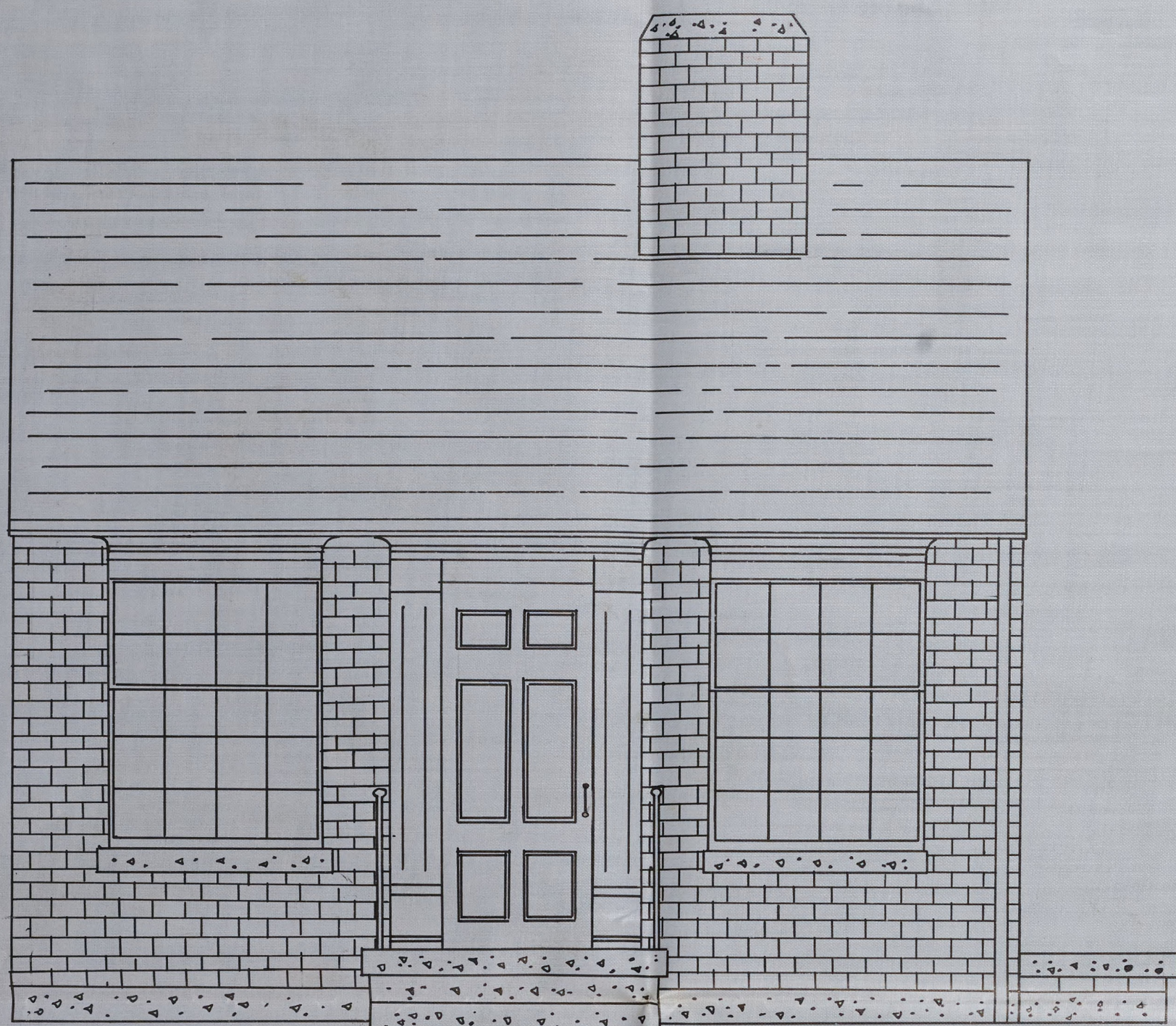


SECTION A-A
CHLORINATION TANK
SCALE $\frac{3}{8}'' = 1'$

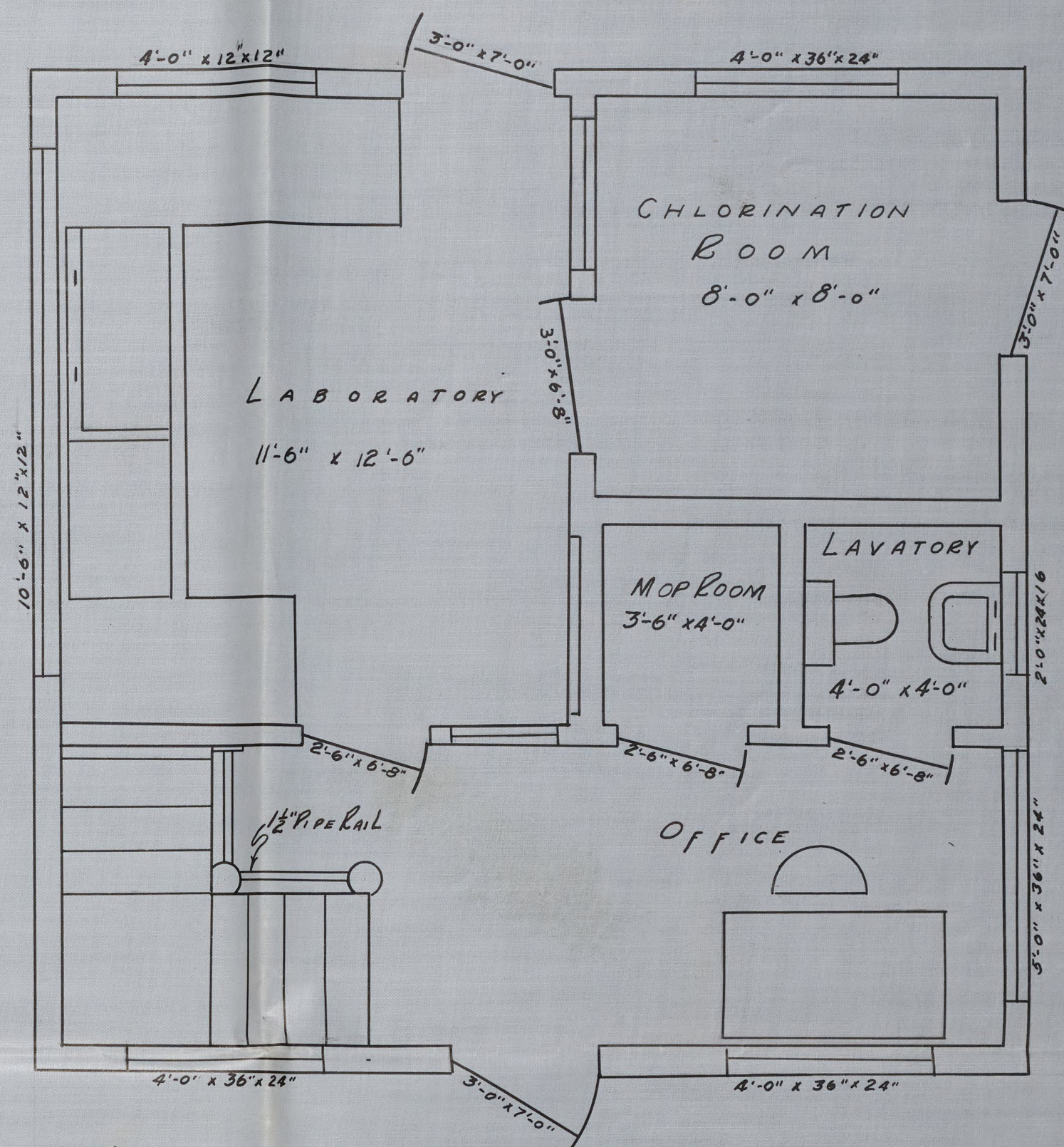


SECTION A-A
ACTUAL LENGTH FORE-SHORTENED
SCALE $\frac{1}{4}'' = 1'$

AERATION TANK

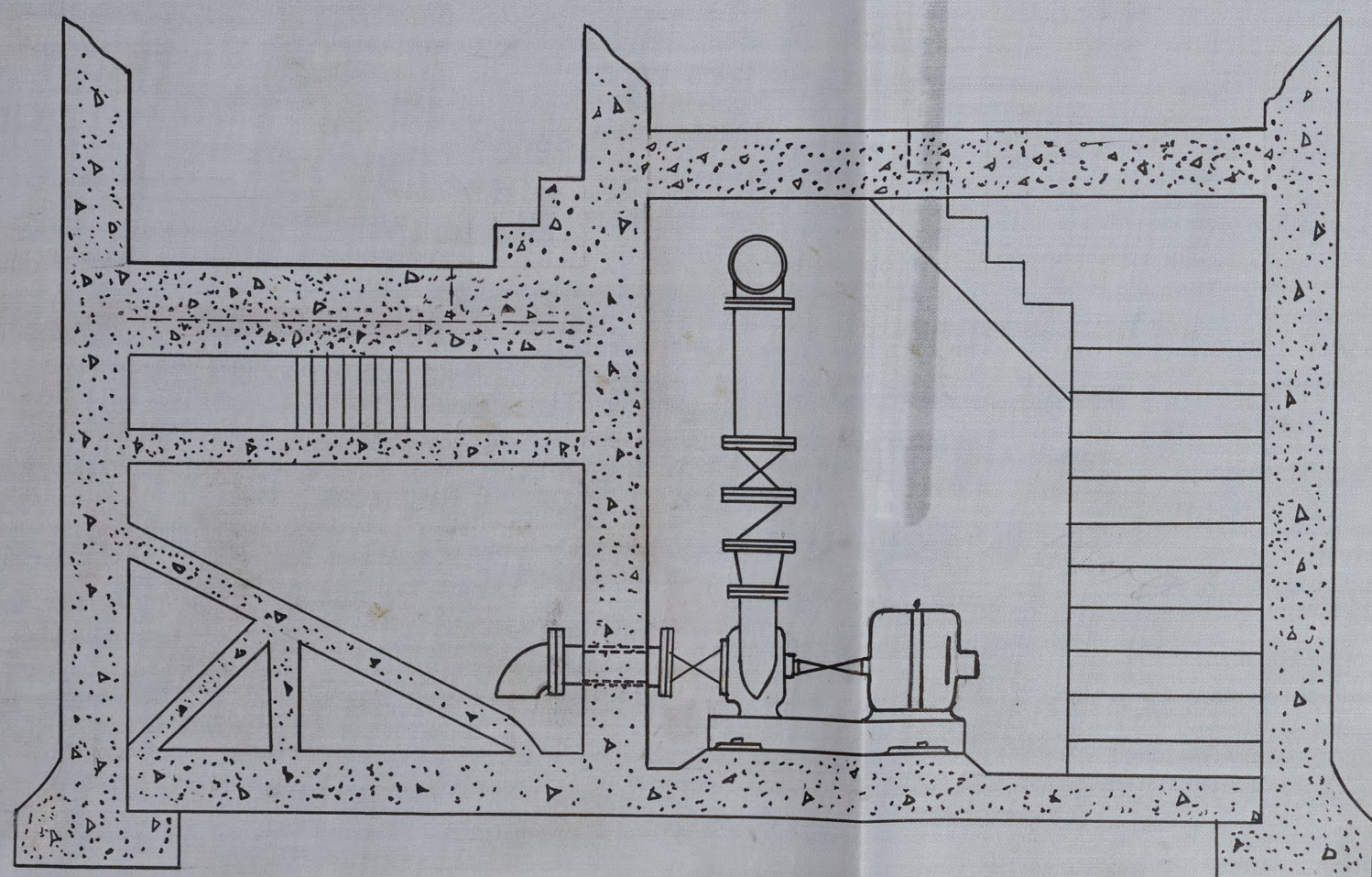


FRONT ELEVATION

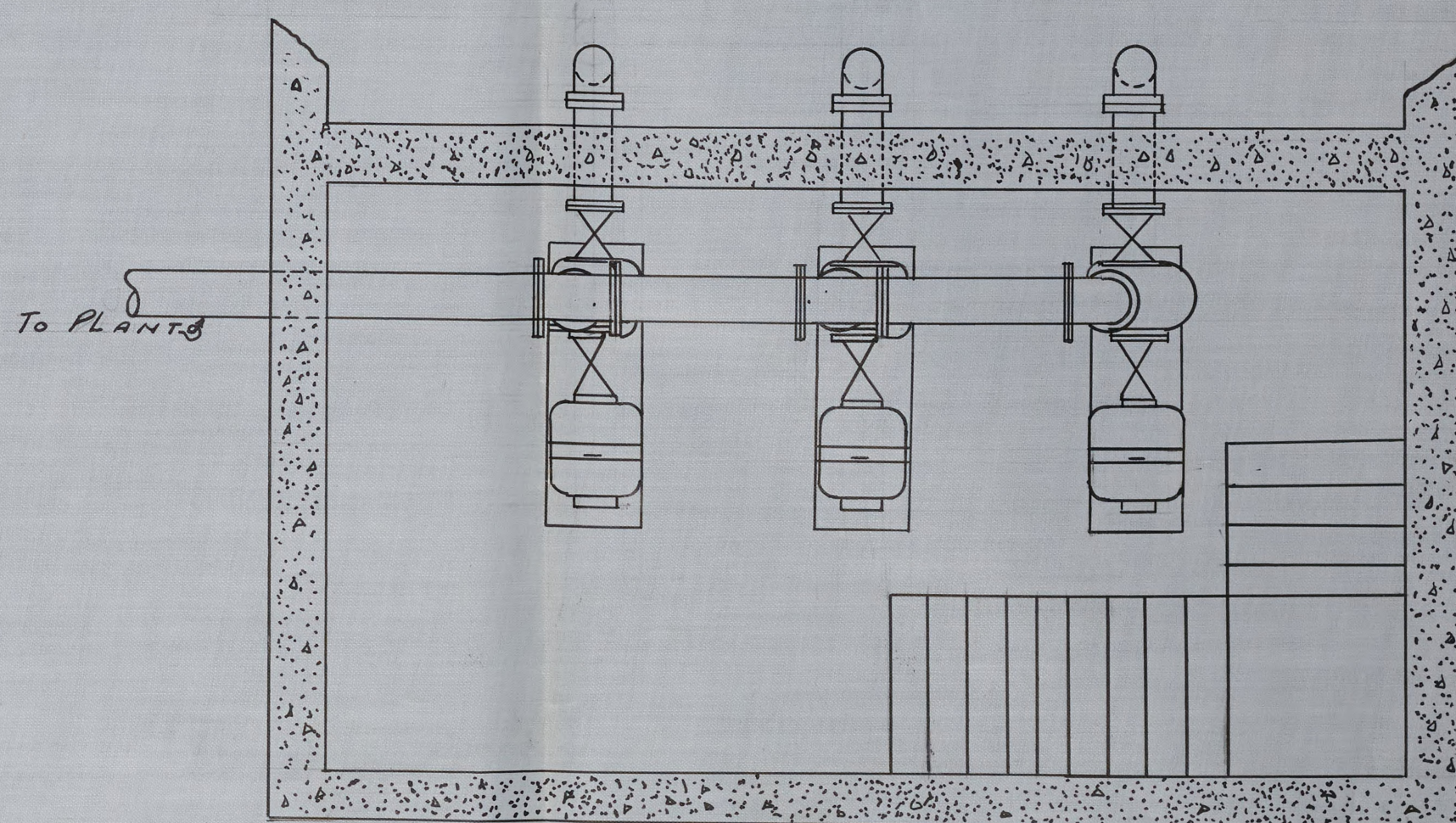


FIRST FLOOR PLAN VIEW

SCALE $\frac{3}{8}'' = 1'$

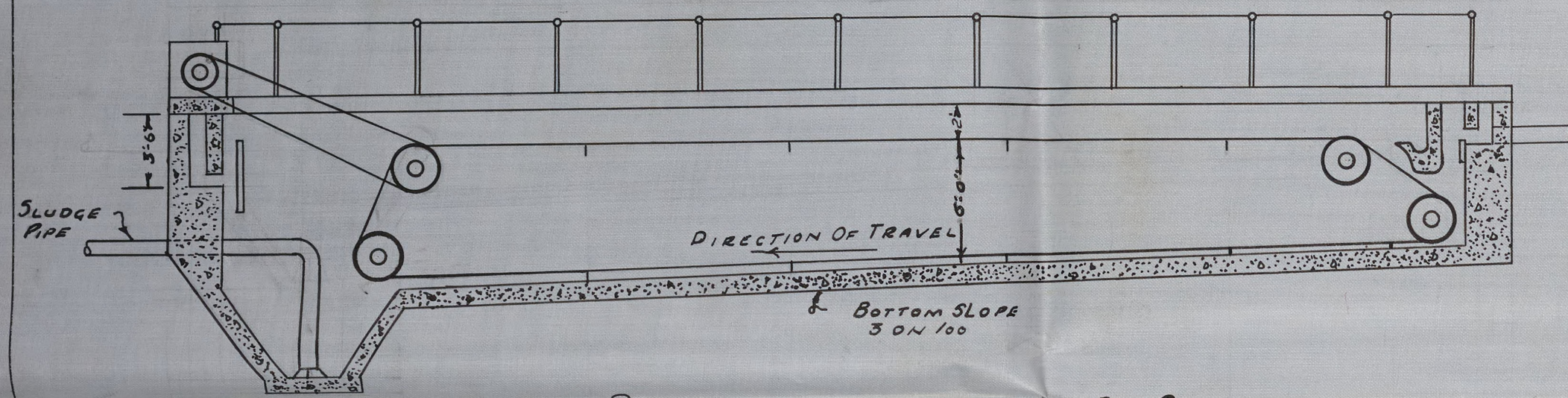
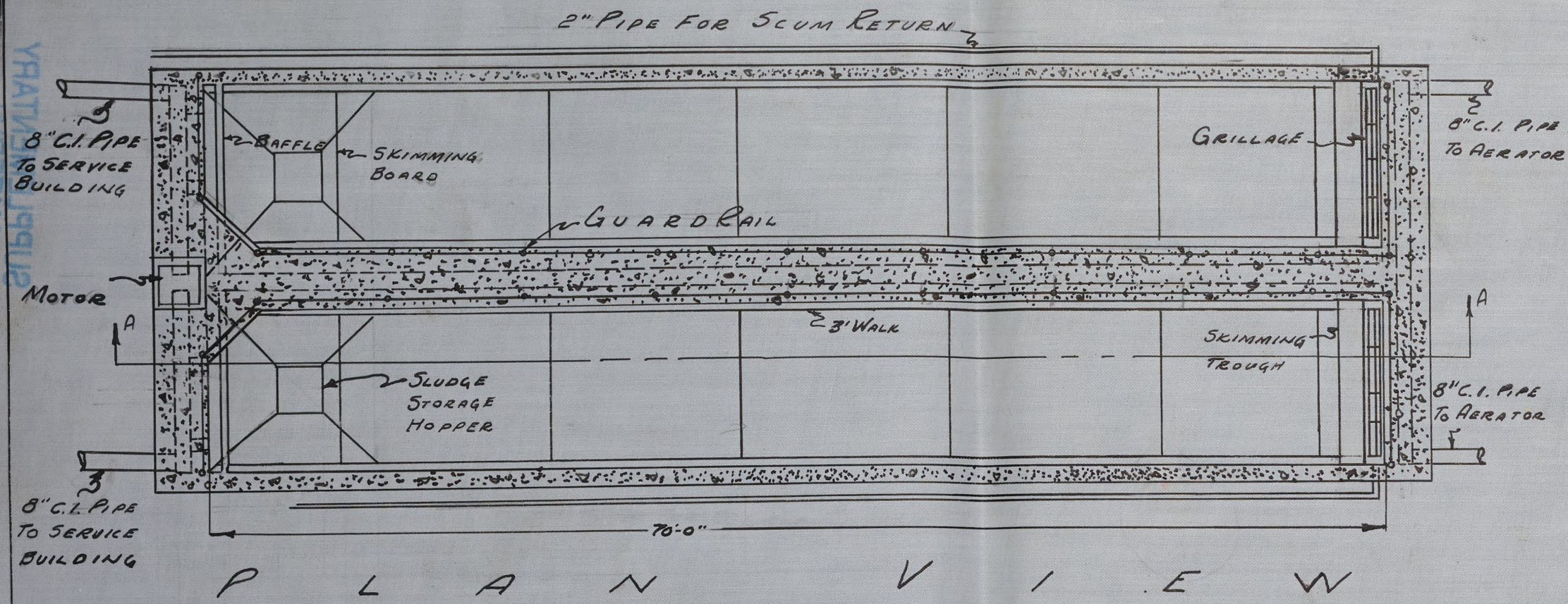


WET WELL & PUMP ROOM
SERVICE BUILDING, SEWAGE TREATMENT PLANT EAST JORDAN, MICHIGAN

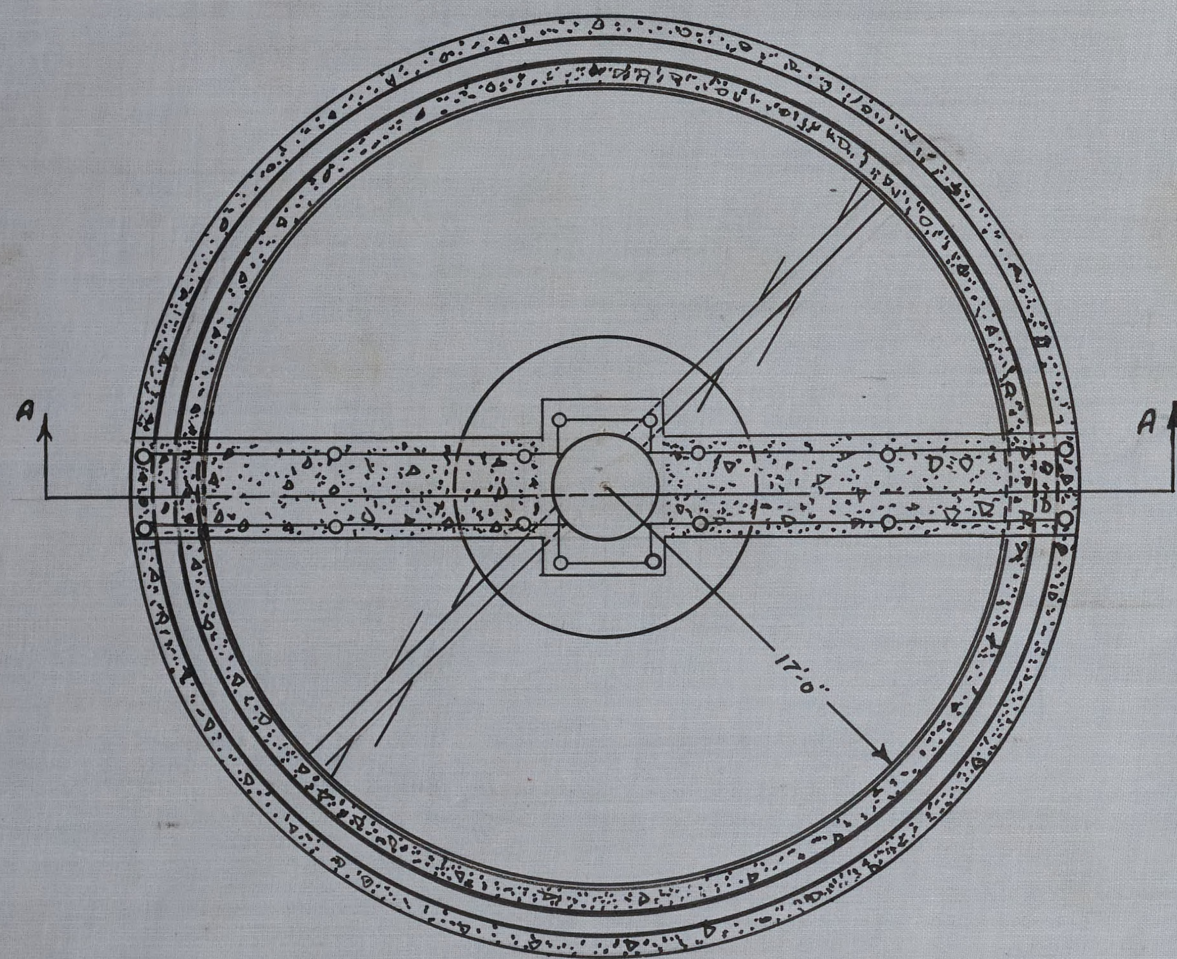


PUMP ROOM PLAN VIEW

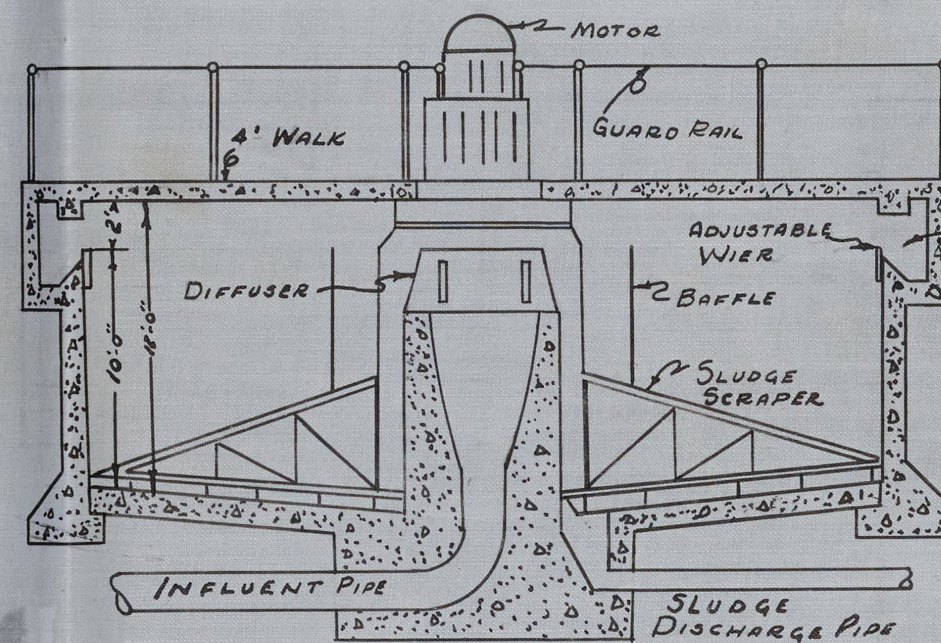
PLATE 1 DETAILS OF TANKS AND CHAMBERS



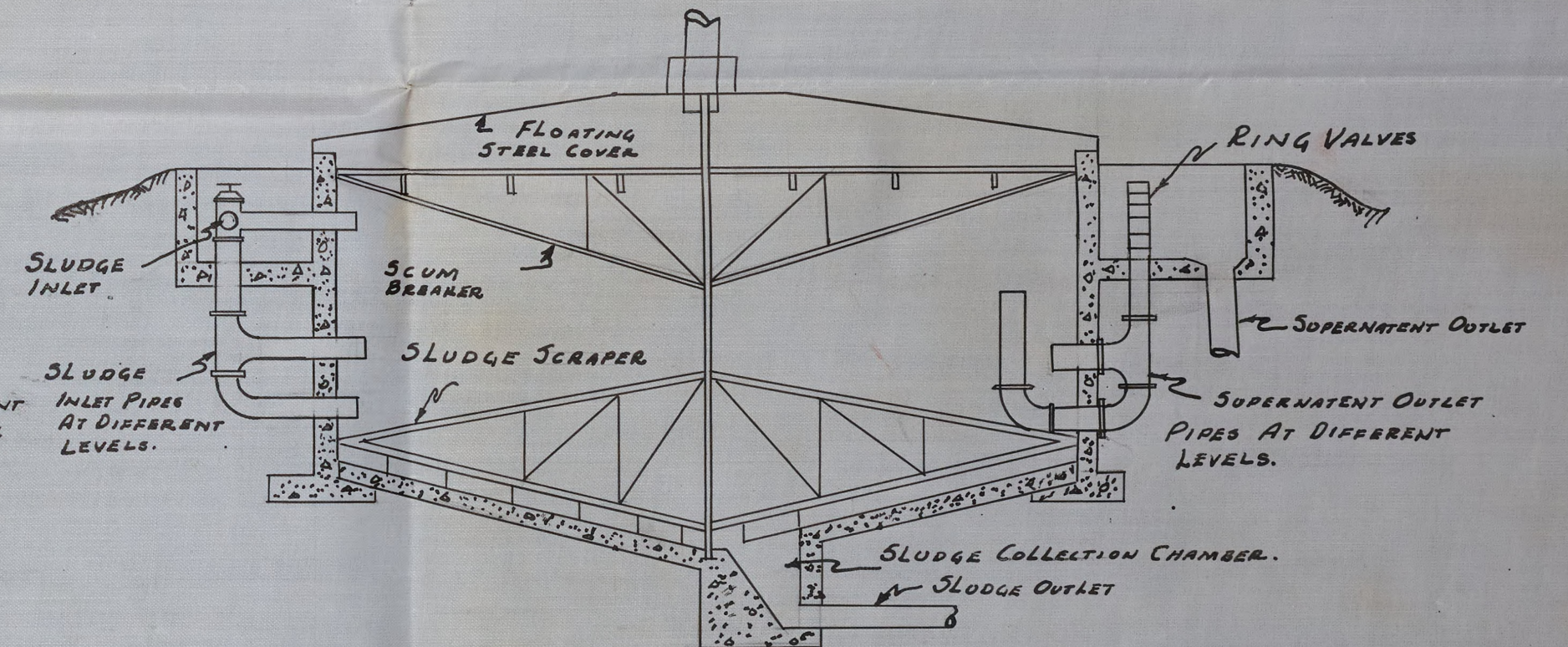
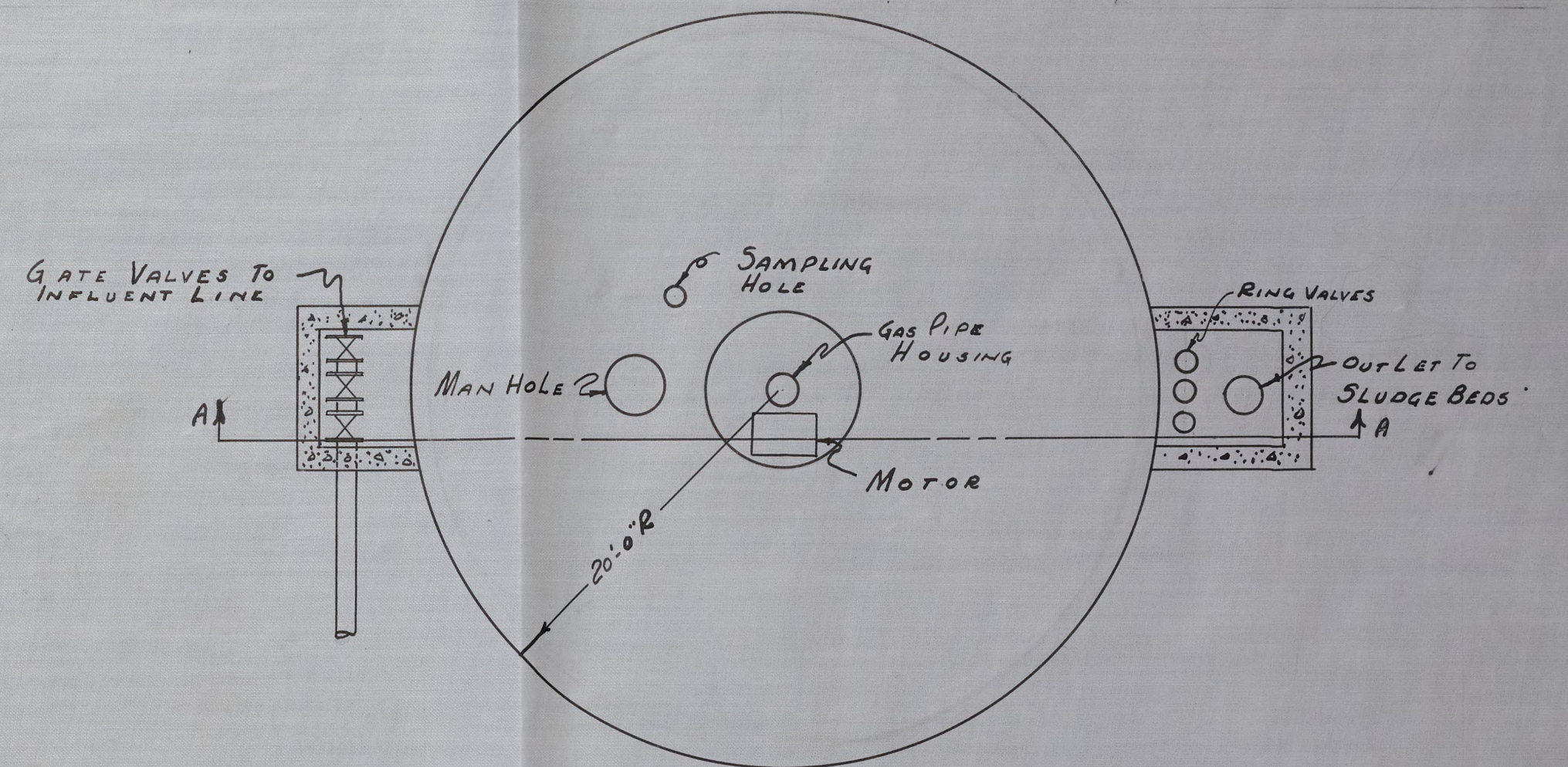
PRIMARY SETTLING TANK



FINAL SETTLING TANK



SECTION A-A

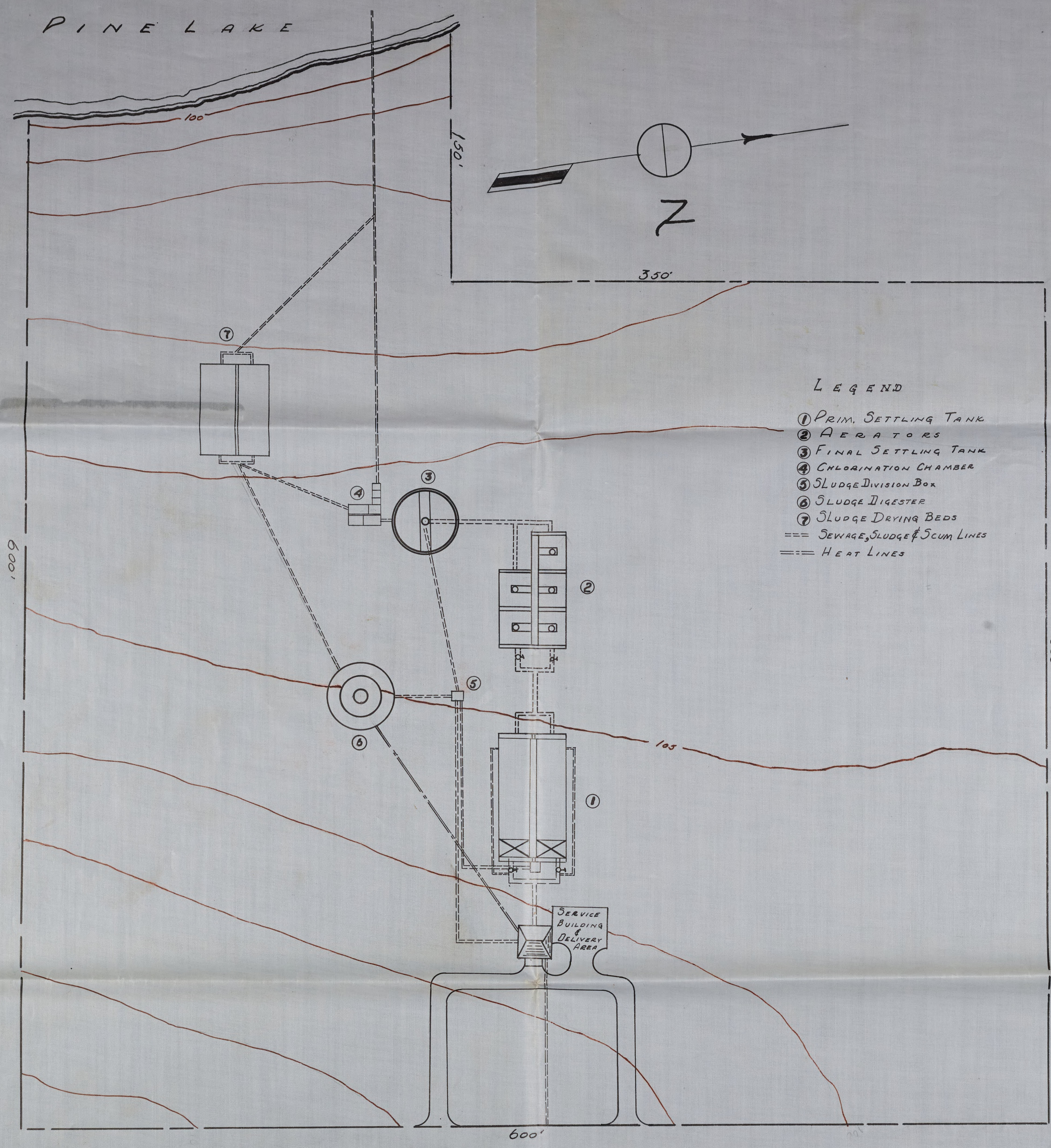


SECTION A-A

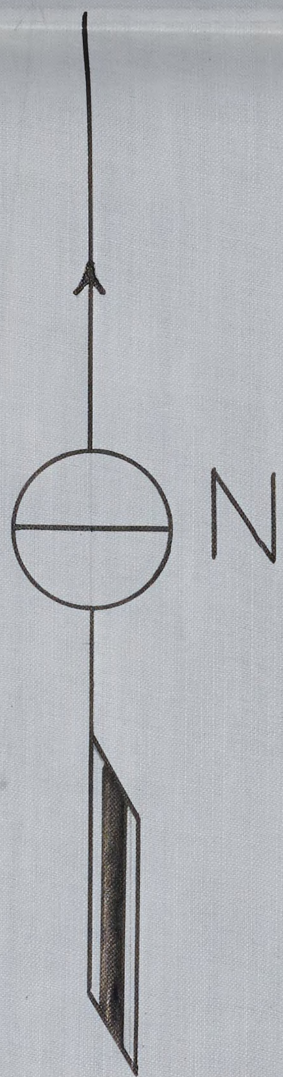
SLUDGE DIGESTION TANK

SCALE ON ALL TANKS 1 INCH = 8 FEET

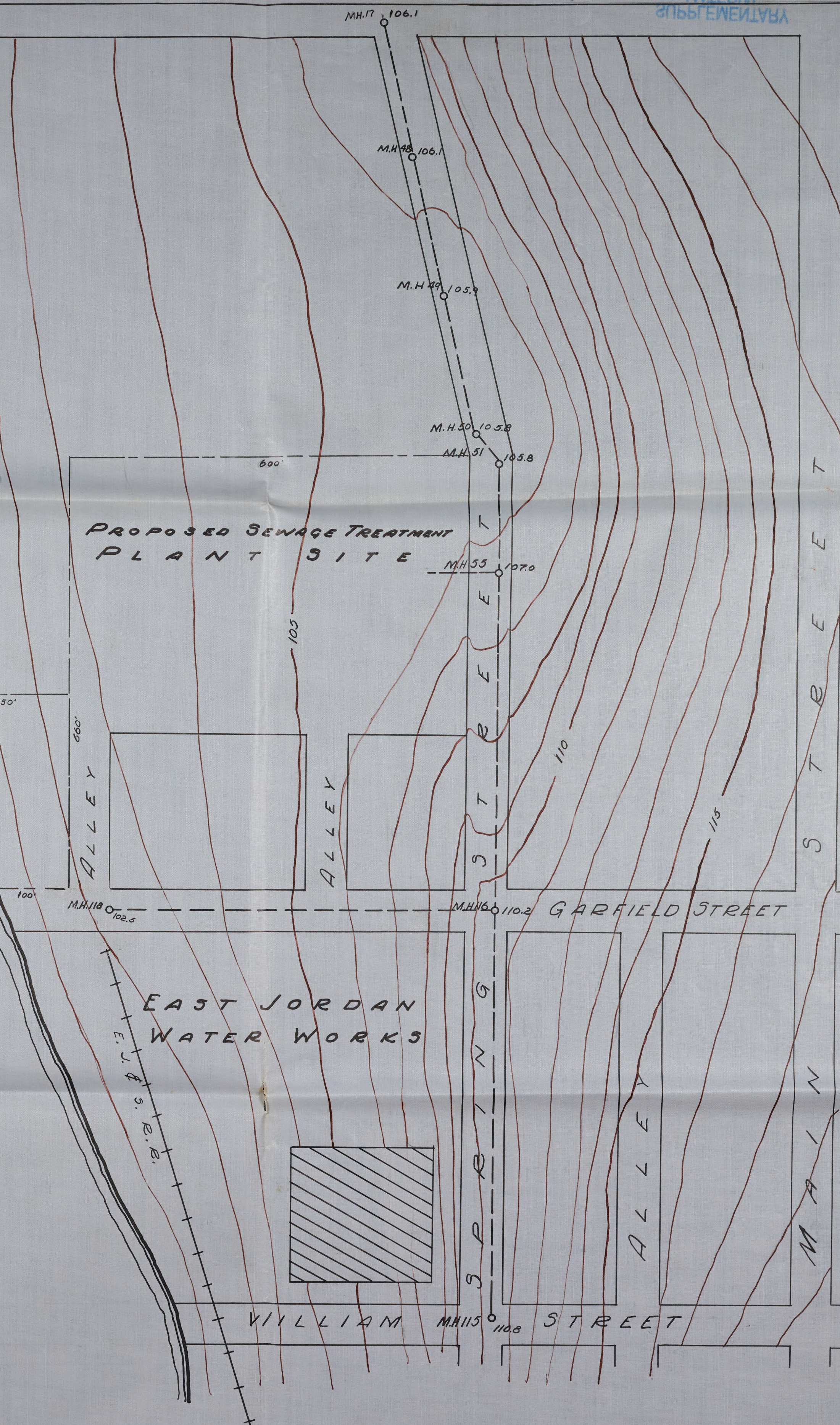
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PLAN VIEW OF PROPOSED SEWAGE TREATMENT PLANT
EAST JORDAN, MICHIGAN
SCALE 1"=40'

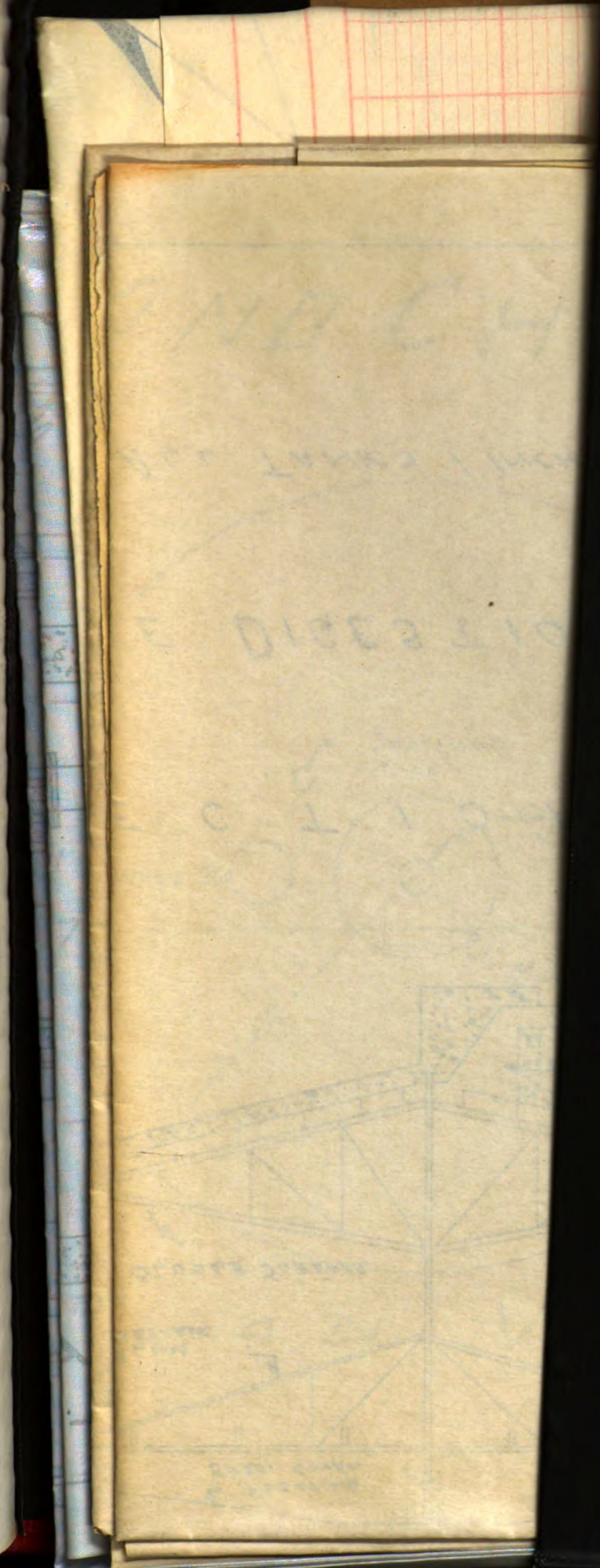


PINE LAKE
HIGH WATER ELEV. 100.3



PROPOSED SEWAGE PLANT SITE, EAST JORDAN MICHIGAN
SCALE 1"=100'

Pocket No. 8 Plans



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