DESIGN OF A SEWAGE
TREATMENT PLANT FOR
HOLT, MICHIGAN

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE
William E. Sayles
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Design of a Sewage Treatment Plant

for

Holt, Michigan

A Thesis Submitted to

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of

AGRICULTURE AND APPLIED SCIENCE

by

William E. Sayles

Candidate for the Degree of

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A C K N O W L E D G M E N T

The author wishes to express his appreciation for the kind help and good advice given him by Mr. Frank R. Theroux, Mr. Chester L. Allen and Mr. John Fay, Delhi Township Clerk.

PRELIMINARY INVESTIGATIONS.

Holt, Michigan is a small agricultural town of about twenty three hundred, located six miles south of Lansing, Michigan, on U. S. 127. The town was founded in Eighteen-hundred forty two and is not incorporated, thus it has no specific town government. It is, however, under the jurisdiction of a township board made up of a supervisor, a township clerk, a treasurer and two justices of the peace. It has no sewer system at present with the exception of county drains.

There are no industries of any size in Holt. There are, however, two clothespin factories, five garages, repair and welding shops and the Holt Manufacturing Products Company, makers of machine parts, located there. Due to the fact that a great percentage of the residents are factory workers who work in Lansing, it is not expected that other industries will spring up in Holt in the near future. Thus a negligible amount of industrial wastes may be expected in sewage flow. Some grease, oil and sand, however, may be expected from the Holt Products Company and from the various garages located there.

In order to properly design a treatment plant of any kind the primary consideration is that of estimating the quantity of sewage flow. This sewage flow must be predicted and this prediction will be based on population data.

Due to the fact that no population figures are obtainable on Holt a count of houses, multiplied by a factor

of three and one half should give a fairly accurate estimate. The result of this house count is as follows -

690 Houses

690 x 3.5 = 2400 population for 1947

For estimates of past population for Holt the method suggested by Mr. John Fay, Delhi Township Clerk will be used. The past population figures for the township of Delhi will be multiplied by a factor of one-third, for in the past, the population of Holt has been determined to be approximately one-third that of Delhi Township. Using this method as a check for accuracy of the house count and visa versa the following results are obtained

Population of Delhi Township for 1940

6723

Estimated population this date

7100

7100/3 = 2370 population of Holt for 1947.

This checks very accurately with the figure arrived at from the house count. Thence in determination of past population figures for the town of Holt this method will be used. However, we must take into account one other item. The town of Holt is at present growing rapidly. This rapid growth may be attributed to the fact that Lansing is becoming rather filled up and that due to the short distance between Holt and Lansing workers may get from factory to home in Holt just as easily as from factory to home in Lansing. Due to Holt's uncrowded condition, many factory workers are re-

siding or building homes in Holt at present.

To predict population with any degree of accuracy good judgment must be exercised on the part of the engineer. He should know the history of the town, its location as to markets and its transportational facilities. After considering all these factors affecting future population a method of predicting future population which best suits the town in question must be selected.

We choose the graphical comparison method as the one which best suits a prediction of future population for Holt for the following reasons. The towns with which Holt will be compared are all located in the vicinity of Lansing and have all undergone a normal increase in growth rate. They have all had, in the near past, populations similar to that of Holt and thus baring abnormal future conditions, their population growth characteristics, may be assumed to apply in the prediction of future population for Holt.

POPULATION FIGURES

GRAND LEDGE -

1940 - 3899

1930 - 3572

1920 - 3043

1910 - 2893

1900 - 2161

HOWELL -

1940 - 3740

1930 - 3615

1920 - 2951

1910 - 2338

1900 - 2518

ZEELAND -

1940 - 3007

1930 - 2850

1920 - 2275

1910 - 1982

1900 - 1326

Figures on past population for Delhi Township -

1940 - 6723

1930 - 4507

1920 - 1729

1910 - 1412

1900 - 1467

These figures multiplied by a factor of one-third give the following approximate population figures for Holt, Michigan -

1940 - 6723/3 = 2240

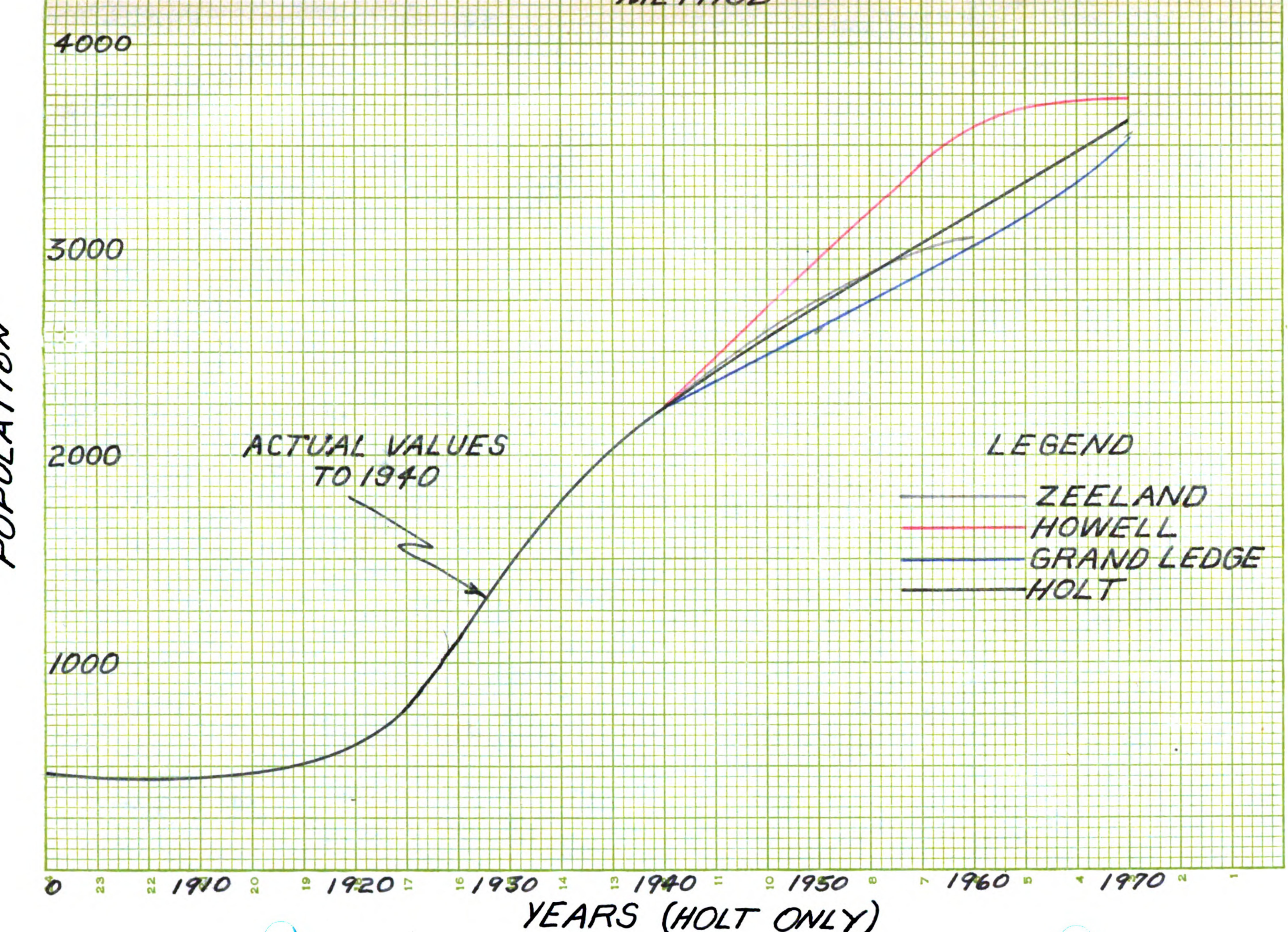
1930 - 4507/3 = 1500

1920 - 1729/3 = 575

1910 - 1412/3 = 470

1900 - 1467/3 = 488

METHOD



4000
3000
2000
1000
0
POPULATION
ACTUAL VALUES
TO 1940
LEGEND
ZEELAND
HOWELL
GRAND LEDGE
HOLT
1910
1920
1930
1940
1950
1960
1970
YEARS (HOLT ONLY)

As a check, however, and to further assure ourselves that the foregoing method is accurate, we have predicted future population by the geometric method. The results obtained by employing this method (3590) compares most favorably with those obtained from the graphical comparison method (3600).

The estimated population for 1970 allowing 5 per cent deviation then will be 3600.

Due to the fact that Holt is unmetered it will be impossible to check it accurately for water consumption. A value of one hundred gallons per capita per day is a figure often used and a check of similar Michigan agricultural towns of about Holt's size and location show it to be an average one.

Cass City -	57	Gallons	per	capita	per	day
Cassapolis-	117	"	"	"	"	"
Clare -	167	"	"	"	"	"
Decatur -	63	"	"	"	"	"
Frankfort-	102	"	"	"	"	"

Average value = 101.3 Gallons per capita per day

The average sewage flow for Holt in 1970 will then be -

$$\frac{100 \times 1.00 \times 3600}{7.48 \times 60 \times 60 \times 24} = .556 \text{ c.f.s.}$$

Infiltration has been assumed to be 10,000 gallons per mile per day. If a sewer were laid in every street in Holt there would be 16.6 miles of sewer line there. Hence -

16.6 x 10,000 = 166,000 gallons per day for infiltration,

or .248 c.f.s.

The total average flow will then be equal to

.248 + .556 or .802 c.f.s.

.802 c.f.s. = 520,000 gallons per day.

The maximum rate of flow will be assumed to be one hundred and seventy five per cent of the average flow. Hence a value of -

$$\frac{100 \times 1.75 \times 3600}{7.48 \times 60 \times 60 \times 24} = .975 \text{ c.f.s.}$$

The total maximum flow will be -

.975 + .248 = 1.23 c.f.s.

or 775,000 gallons per day.

The minimum flow will be assumed as fifty per cent of the average flow.

$$\frac{100 \times .50 \times 3600}{7.48 \times 60 \times 60 \times 24} = .278 \text{ c.f.s.}$$

The total minimum flow is -

.278 + .248 = .526 c.f.s.

or 340,000 gallons per day.

Due to the fact that it will be impossible, due to the non-existance of sewers in Holt, to actually determine the physical characteristics of the sewage by actual test, a table of average values for physical characteristics for a small town of this size has been compiled by Metcalf & Eddy, and these values will be used.

They are as follows:

5 Day B.O.D.	143	P.P.M.
Chlorides	47	"
Total Solids	603	"
Total Volatile Solids	393	"
Total Fixed Solids	210	"
Total Suspended Solids	342	"
Volatile Suspended Solids	82	"
Total Disolved Solids	261	"
Volatile Disolved Solids	128	"
Fixed Disolved Solids	133	"
Total Fats - Indeterminate.		

Next the method of treatment must be decided on. One common method of primary treatment is the plain sedimentation method. Using this method we may expect a sixty per cent reduction in suspended solids and a thirty per cent reduction in B.O.D. The cost of this form of treatment is approximately four dollars per million gallons treated. Methods of additional treatment are; (1) Filtration using the intermittent sand filter, (2) the trickling filter, or (3) the activated sludge process. The sand filter being outmoded will not be considered. The trickling filter does deserve to be considered and weighed carefully. It, however, requires a higher initial cost than does the activated sludge process. It requires high head and odors may be expected to arise during spraying.

The activated sludge process is moderately economical. However, it does form a plant which is difficult to

operate and requires an operator with some degree of skill. Recent data has disproven the theory that the activated sludge process is not applicable to small cities of Holt's size. Employing the activated sludge process, we may expect a reduction of 90 - 100 per cent in B.O.D. and a reduction of 95 per cent in suspended solids.

Considering and weighing carefully the factors discussed above, we choose the activated sludge process because of its high efficiency and its cheaper initial cost.

SERVICE BUILDING:

The service building should be of pleasing architectural appearance and at the same time be very functional. The building should have two basements.

A sub-basement; containing a wet well, sewage pumps, a sump pump to provide drainage and a ventilator.

A basement; containing a screen room, heating plant and sludge pumps.

The main floor should contain a laboratory, a chlorination room, an office, a lavatory and a store room.

Walls should be built of brick and a two inch glazed tile wall interior provided.

An electric control panel should be provided to control operations of the plant and safety features for the chlorination room. The chlorination room should be well ventilated and a mechanical ventilator should be provided as a safety measure. Gas masks should be provided.

As the operator of the plant may know little or nothing about the technical side of operation, the laboratory should be equipped only to run certain basic tests such as five day B.O.D., suspended solids, settleable solids, volatile solids, etc.

GRIT CHAMBER -

Due to the fact that this plant is being designed to receive sewage from a separate system of sewers, the grit chamber is non-essential. However, a rough design follows should this unit ever be deemed necessary.

Use a velocity of one foot per second and a detention period of one minute.

Maximum Q = 775,000 gallons per day.

or 1.23 c.f.s.

Assumed Removal - 4 ft^3 per million gallons treated.

$$\frac{775{,}000}{24 \times 60} = 540 \text{ gallons per minute.}$$

Volume Required - 540/7.48 = 72 ft^3

Use one channel 30' long 1' - 6" wide and 3' - 6" deep, composed of 1' freeboard, 2' water depth and 6" for grit storage. The grit will be removed with a mechanical type remover and will be used intermittently so that it is necessary to provide space for grit storage.

In the event that repair work will have to be done on either the mechanical remover or the tank itself a bypass conduit will be provided. This conduit to be one foot wide, two feet deep and thirty feet long.

PRIMARY SETTLING TANK -

A 60 per cent removal of total suspended solids is anticipated.

A 2 hour detention period will be used.

Average Q = .802 c.f.s.

= 520,000 gallons per day.

Volume required - .802 x 60 x 60 x 2 = 5,800 ft^3

Two identical units will be provided for continuity of operation in case of break down.

A tank 10 feet deep will be assumed; this 10 feet to be composed of 2 feet of freeboard and 8' actual depth.

Use two tanks 10 x 10 x 60, a ratio of 6 - 1 length to width was assumed.

Volume required = 5800 ft^3

Volume provided per tank = 6000 ft^3

A skimming trough to carry away the scum is provided. This trough to be 1' wide, 8" deep and to have a concrete apron inclined 30 degrees up which scum may be raked into the trough.

Sludge removal equipment provided in the sedimentation tanks is of the drag-chain wood-flight type. The returning flights act as skimmers. The flights move the sludge to a collecting hopper located at the inlet end of the tanks. A 3 per cent bottom slope in tanks will be used.

HOPPER FOR PRIMARY TANK -

Using a value of 342 P.P.M. for suspended solids, as suggested by Metcalf & Eddy, for a town of this size, an anticipated removal of 60 per cent and a 90 per cent moisture content the volume of sludge to be expected at the end of a twelve hour period is -

$$\frac{342 \times 3.35 \times 60}{.06 \times 2} = 5730 \text{ gallons}$$

$$5730/7.48 = 765 \text{ ft}^3$$

A hopper six feet across the top, three feet across the bottom and three feet deep with side slopes of 60 per cent will be used.

$$\text{Volume supplied} = 60 \times 3 \times 4.5 = 810 \text{ ft}^3$$

$$\text{Volume required} = 765 \text{ ft}^3$$

INLET -

The inlet will consist of an eight inch pipe discharging horizontally into a channel one foot wide and eighteen inches in depth running perpendicular to the inlet and across the width of the tank. This channel to have openings at various intervals on its discharge side. A baffle will be provided across the width of the tank and in front of these openings or perforations. This baffle to extend at least one foot below said openings or perforations in the channel.

OUTLET -

$Q = 1.23$ c.f.s.

or 775,000 gallons per day.

For velocity of one foot 1 second - Area required - 1.23 ft^2

Use a channel of reinforced concrete 1 foot deep and one and one half feet wide with an adjustable weir.

Area supplied = $1 \times 1.5 = 1.5$ ft^2

Area required = 1.23 ft^2

DIGESTION TANK -

Assume - 340 P.P.M. Suspended Solids

90 per cent removal

90 per cent moisture content

30 per cent digestion

75 per cent volitale solids (Susp.)

3' allowed for supernatent

1.02 = Specific gravity in primary tank

1.005 = Specific gravity in final tank

340 x.9 = 296 P.P.M. Removal

Average Q = .802 c.f.s.

or 520,000 gallons per day.

.52 x 8.34 x 296 = 1280 pounds per day.

296 - 296 (.3) (.75) = 228 PPM new each day.

$$\text{Volume required} = \frac{228 \times .802 \times 8.34 \times 120}{1.02 \times 62.4 \times 1} = 28{,}700 \text{ ft.}^3$$

$$\frac{28{,}700}{3600} = 8 \text{ ft.}^3 \text{ per capita}$$

An eight cubic foot space per capita will be provided in the digestion tank. This figure is well above the minimum value of five cubic feet per capita set up by The Michigan State Health Department.

28,700 ft.^3 required

Use a depth of 16 ft = h

$28{,}700 = d^2 h$

$28{,}700 = 3.14 \times 16 \times d^2$

$d^2 = 570$

d = 25 feet

Use one tank 16 feet in height and 25 feet in diameter.

Volume Required = 28,700 ft^3

Volume Provided = $(25)^2 \times 16 \times 3.14$ = 31,400 $ft.^3$

Thus a total volume of 31,400 $ft.^3$ will be provided for sludge storage. A three foot depth will be provided for the supernatent liquor. This gives a total of 19 ft. at the side of the tank. The tank is two and one-half feet deeper at the center.

Pipes will be provided for withdrawing the supernatent at three different levels. Hot water coils will be provided on the inside of the tank walls in order to keep the temperature constant at 70 degreesF.

Assuming a gas production of one cubic foot per capita per day -

3600 x 1 = 3600 $ft.^3$ of storage space needed for gas.

A floating cover will be provided for the storage of this gas. The gas produced will be used to heat the pump station building and the digestion tank.

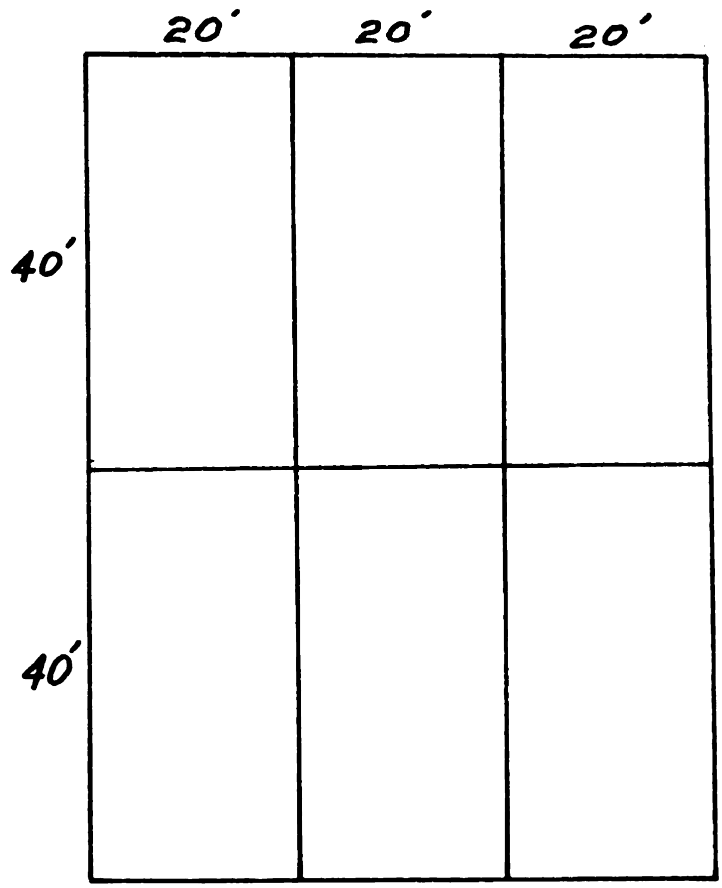
The sludge tank supernatent liquor will be returned to the inlet of the aeration tank and the digested sludge will be drawn off on to underdrained drying beds.

SLUDGE DRYING BEDS:

The design of the sludge drying beds will be based on the figure of one and one quarter square feet per capita.

Area required = 3600 x 1.25 = 4500 ft^2

Use six beds 40 feet long and 20 feet wide, spaced thusly -



This gives a total of 4800 ft^2 or about 1.33 ft^2 per capita.

The sludge drying beds will consist of 9" of coarse sand underlain with layers of graded gravel ranging in size from 1/8" to 1/4" at the top to 3/4" to 1½" at the bottom, with a total thickness of about 1'. The bottom is of natural earth graded slightly to the underdrains.

The underdrains will be of 4 inch open joint drain tile placed in trenches.

The divisions between beds will consist of batter board placed end to end and joined by short pieces of board driven into the ground. Planks dropping into grooves in order to permit easy entrance of trucks for removal of the dried sludge will be provided.

WET WELL -

Average flow will be the basis for design.

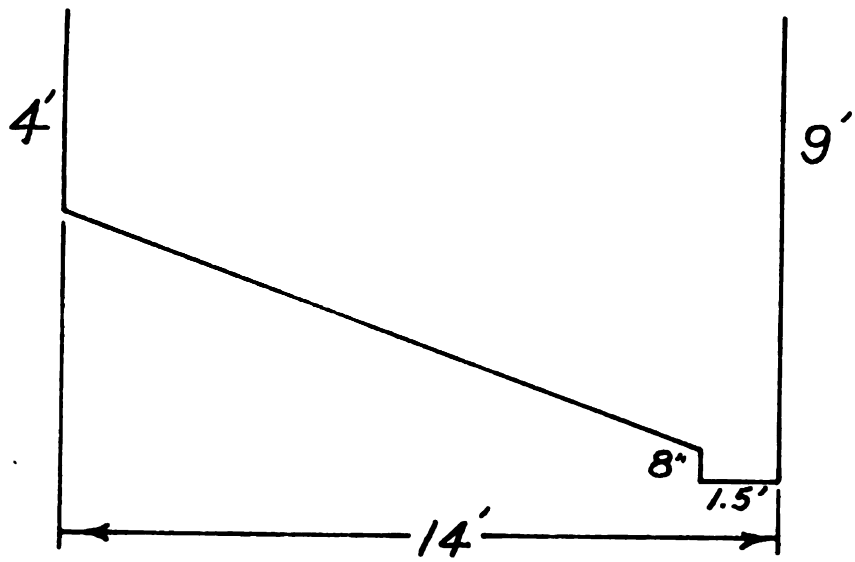
Average Q - .802 c.f.s.

Provide a ten minute storage.

10 x 60 x .802 = 480 $ft.^3$ needed.

or 3700 gallons capacity.

Using a 14' Tank.



12.5 x 5/2   14 x 4 = 90.5 $ft.^3$

Using a 6' width of tank - 540.5 $ft.^3$ are provided. Allow 6" for safety and a 6" freeboard to bottom of screen room floor.

PUMPS -

Three pumps will be used for flexability of operation as well as safety. A pump must empty the wet well plus the amount of sewage running in, in a given time. This unit will be provided along with others to be used in case of breakdown.

Non Clog Centrifugal Pumps will be used.

3700 gallon tank

Flow = .802 c.f.s. or 360 gallons per minute.

Time required to empty tank is:

3700/600   3700/600 x 360 = 9.7 minutes.

USE - One 600 G.P.M. Sewage Pump

One 400 G.P.M. Sewage Pump

One 200 G.P.M. Sewage Pump

This array of pumps will permit flexability of operation. One of the smaller pumps will suffice for minimum sewage flow. Reserve capacity has been provided should breakdown occur. The pumps will be arranged to start and stop as the sewage level in the wet well flucuates.

SLUDGE PUMP -

A 75 G.P.M. sludge pump, single plunger, belt driven, 4" suction and discharge, 75 G.P.M. Capacity against a total dynamic head of 20', 1 H.P. motor pump will be used.

Air chambers will be provided on both the suction and discharge side of the pump. These air chambers to be 6" in diameter and 52" long. Gate valves will be installed on both sides with a check valve on the discharge side.

AERATION TANKS -

8 hour detention period.

An updraft type aerator will be used. One cubic foot of air will be used per gallon of sewage treated.

Q = .802 c.f.s. or 520,000 gallons per day.

Allowing 30 per cent for returned activated sludge.

.802 x 1.30 = 1.04 c.f.s.

8 x 60 x 60 x 1.04 = 30,000 $ft.^3$ of tank must be provided.

Surface area to be served by one aerator should be approximately 25 feet in diameter.

Assume tanks 12 ft. deep and 21 ft wide.

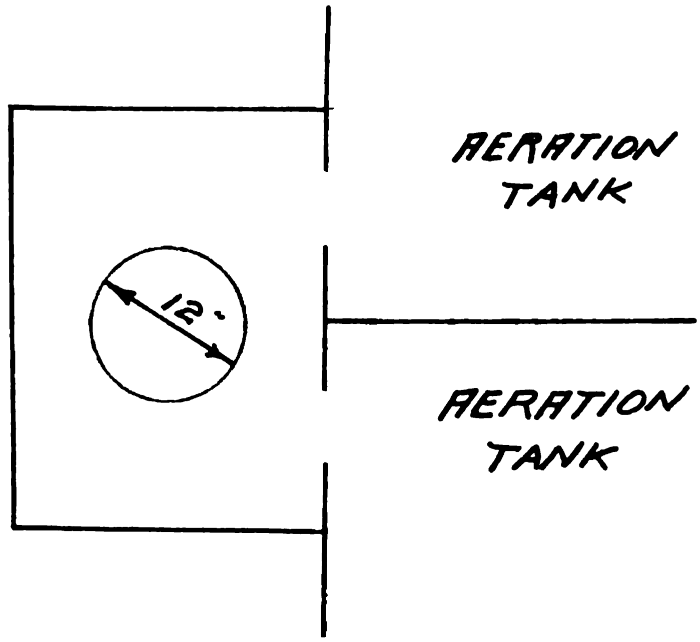
$$L = \frac{30,000}{12 \times 21} = 119 \text{ ft.}$$

Use five tanks 21 feet wide and 12 feet deep. This arrangement fulfills all requirements. The bottom edges of these tanks will be rounded to aid circulation.

An updraft aerator which carries the manufacturer's guarantee to furnish at least 28,000 $ft.^3$ of air per hour will be used. The assumption has been made that one $ft.^3$ of air will be used for each gallon of sewage treated.

INLET -

The inlet will consist of an eight inch pipe discharging vertically into a box provided as shown on the following page.



OUTLET -

Use a simple weir adjustable to ± 0.1' and emptying into a channel. Use a .15' fall from end to end of channel.

RACKS & SCREENS -

Using a value of 12 square feet per million gallons per day as suggested by Mr. Frank R. Theroux -

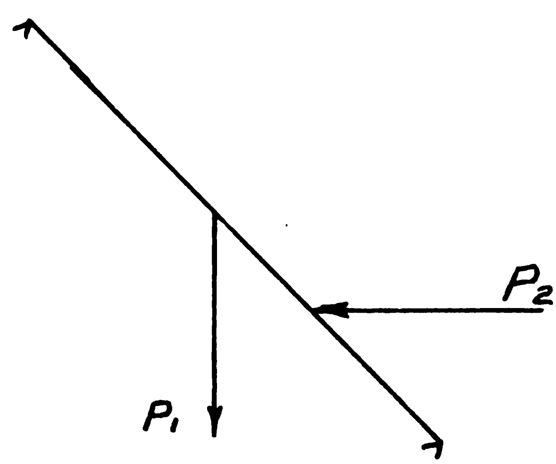
.802 x 12 = 9.6 ft.^{2} of rack area is required.

A two foot rack will be used with 6 inches of clearance provided above the sewage surface.

Assume the use of bars one and one-half feet long and 2" x 3/8" in cross section.

$$\text{Wt. of bar} = 2 \text{x} 3/8 \text{x} 1.5 \text{x} 12 \text{x} \frac{1}{1728} \text{x} 490 = 3.85 \text{ pounds}$$

applied at the center of gravity or 3.85/2.24 = 1.73 pounds applied as shown. (P 1)



$$\text{Water pressure} = \frac{62.4}{12} \text{ x } 3/8 \text{ x } \frac{.75 \text{x} .75}{2} = 55 \#$$

applied 1/3 of the distance to the top of the span as shown (P 2)

Shear decrease = 1.15 pounds per foot.

$$X = \frac{.97}{1.15} = .84 \text{ ft. from right end}$$

Area = .97/2 x .84 x 12 = 4.9

$$S = \frac{Mc}{I} = \frac{4.9 \text{x} 1}{1/12 \text{x} 3/8 \text{x} 2^3} = 78.5 \# \text{ per sq. inch,}$$

which is small for steel.

A hose with a powerful nozzle will be used at various intervals of time for breaking up screenings collected on the screen and forcing them through. This will naturally reduce the volume of screenings which must be removed.

Containers, however, will be provided with adequate means for their removal from the building; such as, hoists, trap doors, etc. A common figure used in estimating amounts of screenings is five feet^3 per million gallons of sewage; however, this figure will be much reduced with the use of the aforementioned hose arrangement.

CHLORINATION BASIN -

A detention period of twenty minutes will be used for normal sewage flow.

Q = .802 c.f.s. or 520,000 gallons per day.

$$\text{Volume Required} = \frac{20 \times 520{,}000}{24 \times 60} = 7{,}200 \text{ Gal.}$$

$$\frac{7200}{7.48} = 963 \text{ ft.}^3$$

Use a baffle basin 7 feet deep, 12 feet square and with a 2 foot freeboard. The baffles will be placed at 3 ft. intervals with a 3 foot clearance around end.

A bypass around this basin will be provided.

A direct feed chlorinator will be used. It will be housed in a special, well-ventilated room on the ground floor of the pump station and provided with a tube connection to the diffuser in the chlorination basin.

SLUDGE DIVISION BOX -

This box must be so designed as to distribute the sludge from the final tank to either the wet well, aerator or primary tank in most any desired portion. Details of design will be found in the detailed drawings.

FINAL TANK -

A two and one-half hour detention period will be used in determining the tank size.

$Qav. = .802$ c.f.s.

$= 520,000$ gallons per day.

$.802 \times 1.30 = 1.04$ c.f.s.

$520,000 \times 1.30 = 680,000$ gallons per day

Volume required -

$680,000 / 24 = 28,400$ gallons per hour.

$28,400 \times 2.5 = 7,000$ gallons per 2.5 hours.

$71,000 / 7.48 = 9,500 \text{ ft.}^3$

Try one tank, circular in shape, 14 feet deep and 30 feet in diameter.

$$V = \frac{14 \times (30)^2 \times 3.14}{4} = 9,900 \text{ ft.}^3$$

Volume required $= 9,500 \text{ ft.}^3$

Volume provided $= 9,900 \text{ ft.}^3$

Use one tank, circular in shape, 14 ft. deep and 30 ft. in diameter.

INLET -

The Inlet will consist of a pipe in the center discharging vertically and surrounded by a perforated baffle extending 4 feet below the water surface. A one foot freeboard will be provided for wind current protection.

OUTLET -

MAXIMUM Q = 1.23 c.f.s.

$1.23 = d^2$

$d = 1.07$ ft - 13 sq. inches

The effluent velocity will be 1.5 feet per second.

Required cross section = 169 sq. inches.

$13 \times 13 = 169$

Use a depth of 19 inches and a width of 9 inches.

Cross Section provided = $19 \times 9 = 171$ sq. inches.

The outlet will consist of an adjustable metal weir around the circumference of the tank. The effluent will be collected in this channel of uniform size.

HEAD LOSSES -

Entrance Losses = $2 \frac{v^2}{2g}$

Pipe Friction = Slope (decimal) x length.

Loss in 90 degree turn = $\frac{v^2}{2g}$

A one and one-half foot head loss will be allowed in pumps and piping to the grit chamber.

GRIT CHAMBER -

Slope = .19' per 1000

Length = 30'

Head Loss = .008'

Allow a .05 ft head loss through the grit chamber and to the entrance of the primary tank.

PRIMARY TANK -

Velocity = $\frac{1.22 \times 144}{8 \times 9}$ = 2.5. ft. per sec.

Head loss over weir = $h^{2/3} = \frac{Q}{3.33\ h}$

h .10 Q = 1.22 c.f.s.

$h^{2/3} = \frac{1.22}{33.3 \times 10} = .0367$

h = .005'

Loss through channel = .20'

A 2.5 ft. head loss will be allowed from outlet of primary tank through aeration tank and to the final tank inlet.

SECONDARY TANK -

$L = 21'$

$Q = 1.23$

$$h = \left(\frac{1.23}{3.33 \times 21}\right)^{2/3} = .005 \text{ head loss over weir.}$$

Allow a .1' head loss through the outlet channel.

Loss in 100' of 8" pipe = .25'

Loss in 2 90 degree elbows = .20'

Loss in inlet channel = .20'

The losses over outlet weirs are so small that adjustment provided in the weir will take care of any variation which may arise.

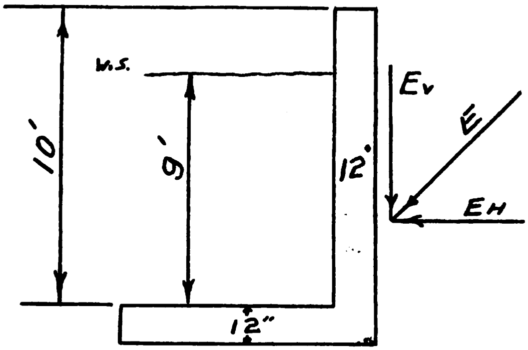
Allowing a .05' loss through the channel and a .08' loss through the chlorinator the total head loss through the entire plant is then -

5.018'

DESIGN OF A WALL -

PRIMARY SETTLING TANK.

CANTILEVER METHOD -



$E = \frac{100x(10)^2}{2}$ x.3 =1500 # per ft.

Ev = 1500x sin 30 degrees = 750 # Per foot.

Eh = 1500xcos 30 degrees = 1300 # Per foot.

Maximum Moment at Base (Tank Empty)

M = 1300x10/3x12 = 52,000[1] pounds

Use fs = 16,000 pounds per sq. inch. K = 146

$d \left(\frac{M}{kh}\right)^{1/2} = \left(\frac{52,000}{146x12}\right)^{1/2}$ - 5.45 inches.

Use d = 6 inches minimum t = 12 inches.

Shear at Base -

$v = \frac{V}{b\ j\ d} = \frac{1300}{12x7/8x6}$ = 20.63 pounds per sq. inch

Allowable = 40 pounds per square inch

Area of Steel -

$As = \frac{M}{f\ s\ j\ d} = \frac{52000}{16000x7/8x6}$ = .62 Sq. inch

Use 3/4" round bars at 8" spacing.

Area supplied = .66 sq. inches.

BOND -

$$U = \frac{V}{\Sigma o j d} = \frac{1300}{12/8x2.36x7/8x6} = 70.0\# \text{ per sq in.}$$

Allowable - 100# per sq. in.

Length of Steel Bars -

3 of every 4 bars will be cut off.

Remaining area then is 1/4 x .62 = .155 sq. in.

h = 6' - 6" anchorage.

Anchorage -

40 D = 40 x 3/4 = 30"

h = 6' - 6" - 30 = 4'

One-half remaining bars will be cut off.

1/2 x .155 = .0775

h = 5' - 1" - 3" = 2' - 7"

Maximum Moment at Base (Tank Full)

$$\text{Water Pressure} = \frac{62.5 \times (10)^2}{2} = 3100\#$$

M = 3100 x 10/3 = 103,000

Difference in Moments -

103,000 - 52,000 = 51,000

$$d = \sqrt{\frac{M}{K b}} = \sqrt{\frac{51,000}{146x12}} = 5.40 \text{ use } d = 6"$$

$$6 = \sqrt{\frac{M}{146 \, x12}}$$

M - 43,000' pounds.

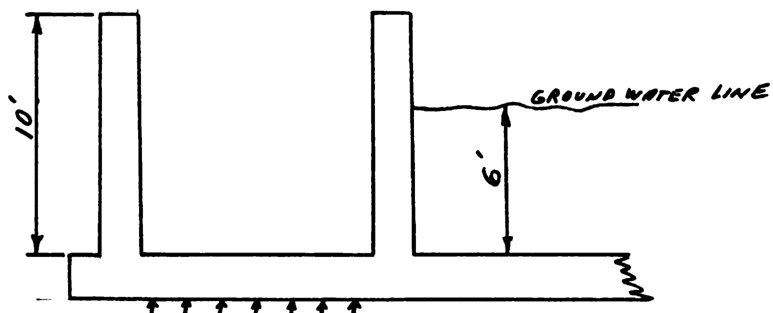
Area of Steel -

$$As = \frac{43,000}{15,000 \times 7/8 \times 5} = .615$$

Use 3/4" round rods spaced at 8"

Area supplied = .66 sq. in.

Design of Base Slab -



Drainage - 6' deep assume t = 10"

Upward pressure = $10 - \frac{6}{2}$ = 4' head

P = 62.4 x 4 = 250 # ft

Total P = 250 x 64 = 11,200 #

Slab Wt = $\frac{10}{12}$ x 1 x 1 x 150 = 125# per foot

Total Sw = 125 x 60 = 7,500 #

Wt. of Walls = 10 x 150 x 1 = 1500 # per ft.

Total W_w = 1500 x 4 = 6,000 #

Force up = 11,200

Force down = 14,000

W = 250 - 125 = 125 pounds per foot.

Moment Maximum -

$$M = \frac{W\,l^2}{8} = \frac{125 \times (10)^2}{8} \times 12 = 38{,}000'' \#$$

$$d = \sqrt{\frac{38{,}000}{146 \times 12}} = 3.3''$$

t = 12" mimimum

Area of Steel -

$$As = \frac{M}{f\,s\,j\,d} = \frac{38{,}000}{16{,}000 \times 7/8 \times 6.5} = .42 \text{ sq. in.}$$

Use 3/4" round rods spaced at 12"

Area supplied = .44 sq. in.

Steel run in both directions

BIBLIOGRAPHY.

Metcalf & Eddy - "Sewage Design"

Steele - "Water Supply and Sewage"

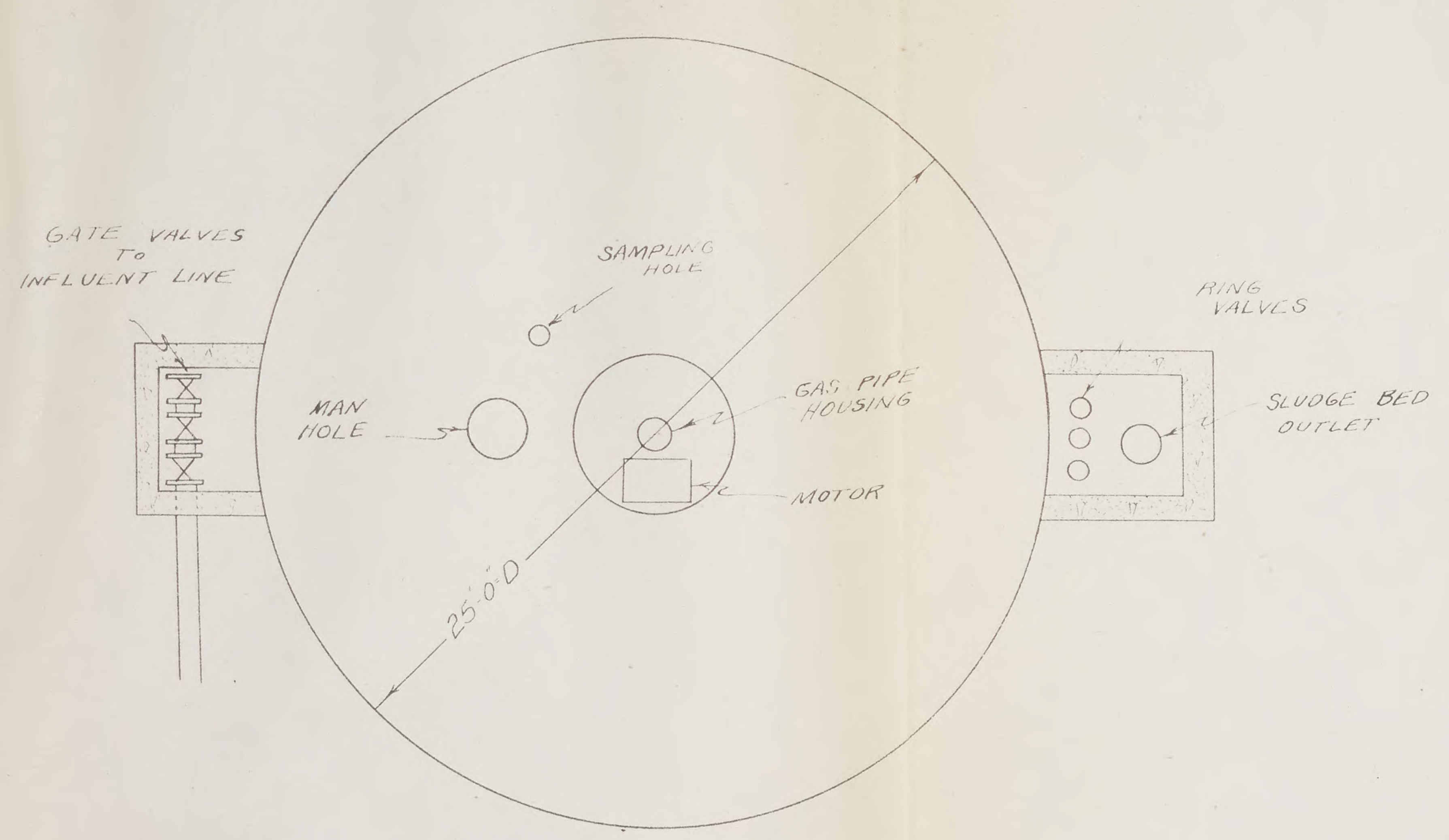
Schoder & Dawson - "Hydraulics"

Professor Frank R. Theroux - Lecture Notes & Interview

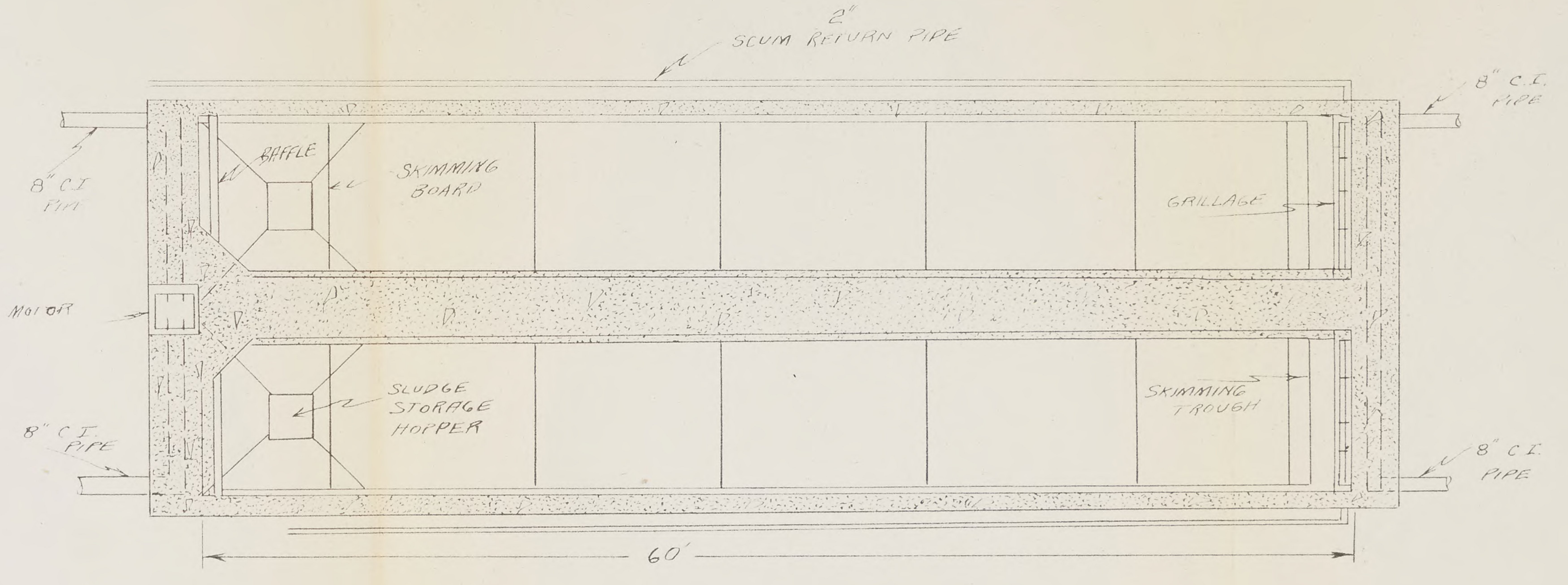
Chicago Pump Co. - Bulletin 195 - "Chicago Plunger Sludge Pumps"

American Concrete Institute - "Reinforced Concrete Design Handbook"

[redacted]

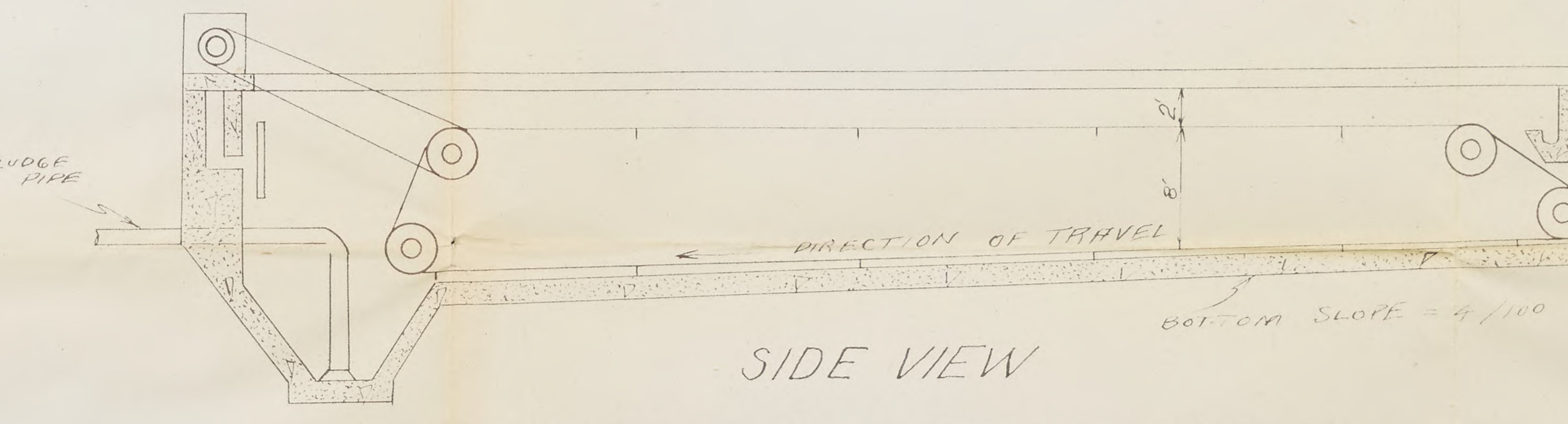


PLAN VIEW



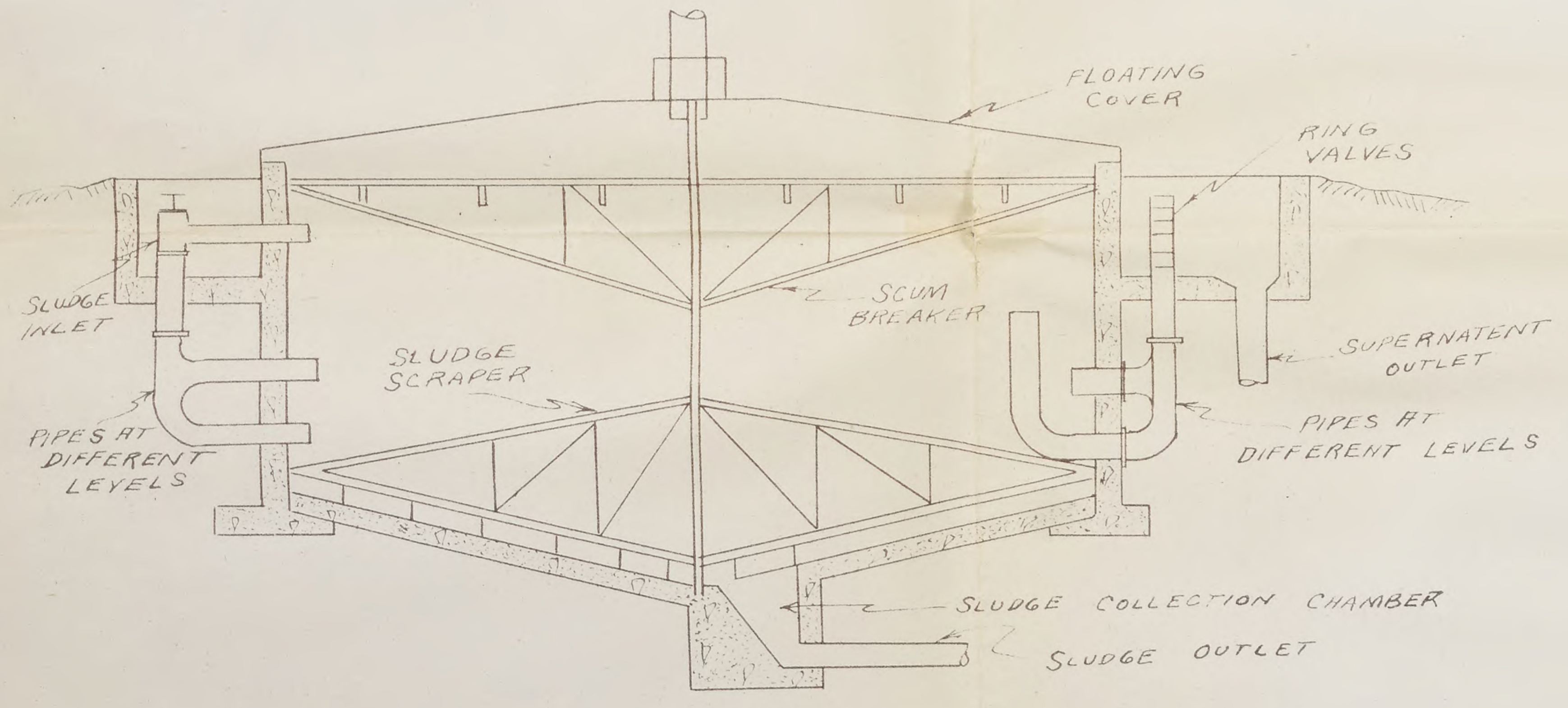
PLAN VIEW

PRIMARY TANK 1" = 7'



SIDE VIEW

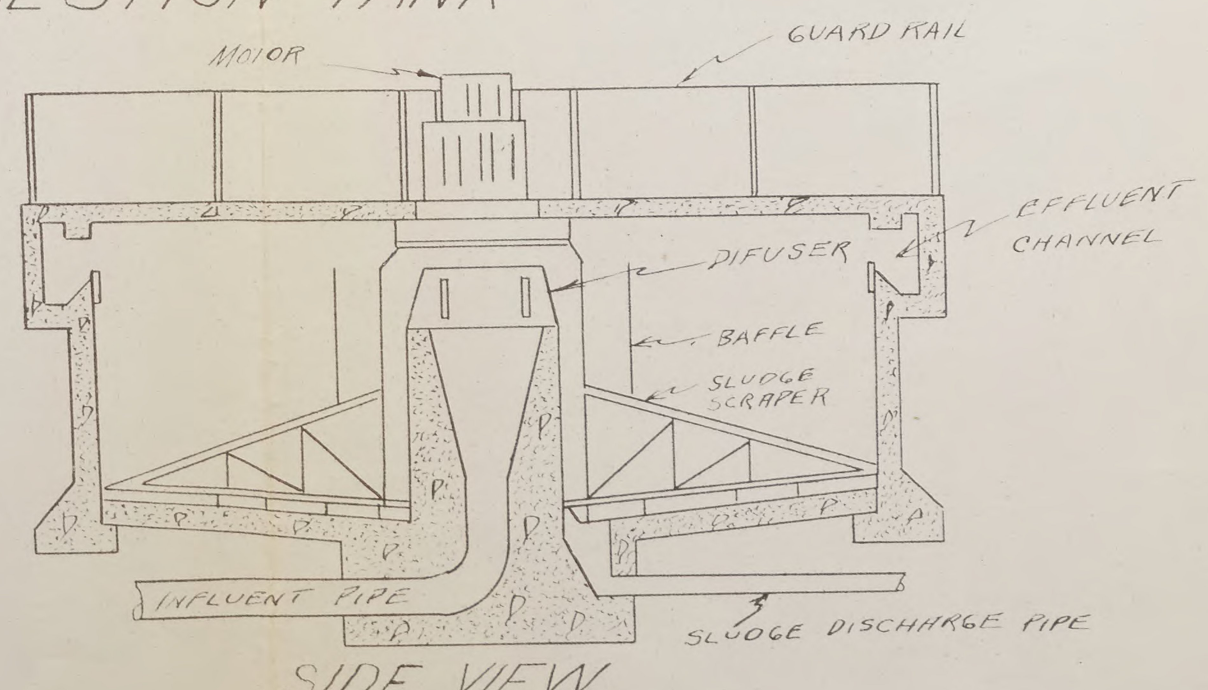
PRIMARY SETTLING TANK



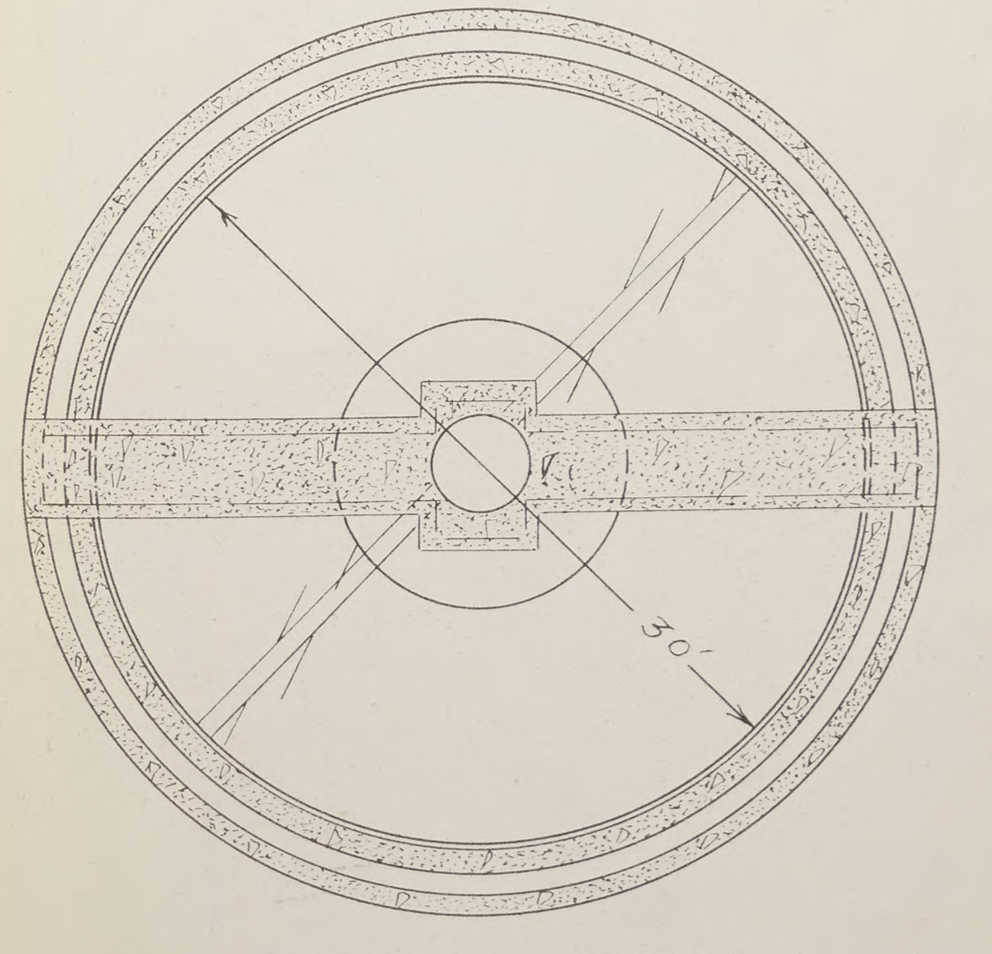
SIDE VIEW

SLUDGE DIGESTION TANK

SCALE ALL TANKS 1" = ~~10~~ 7'

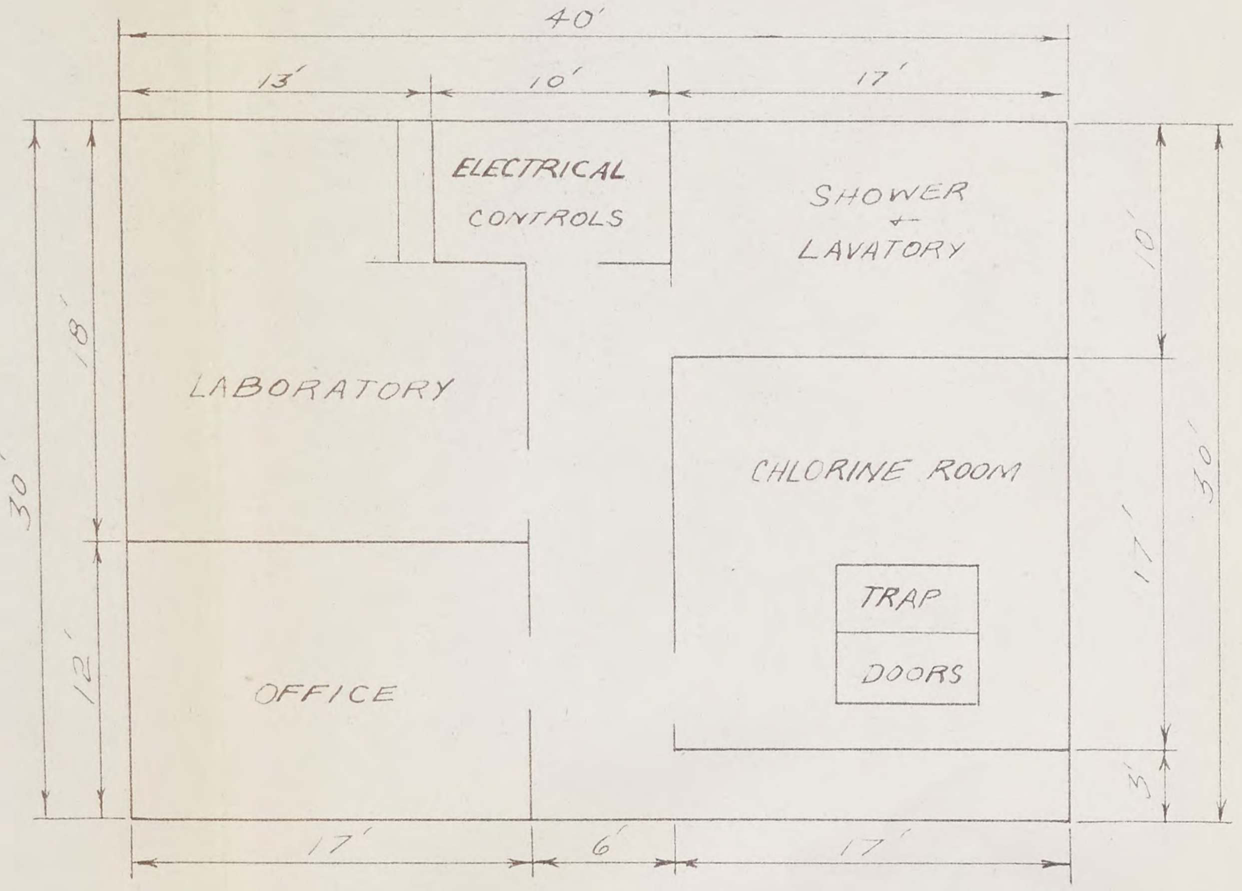


SIDE VIEW

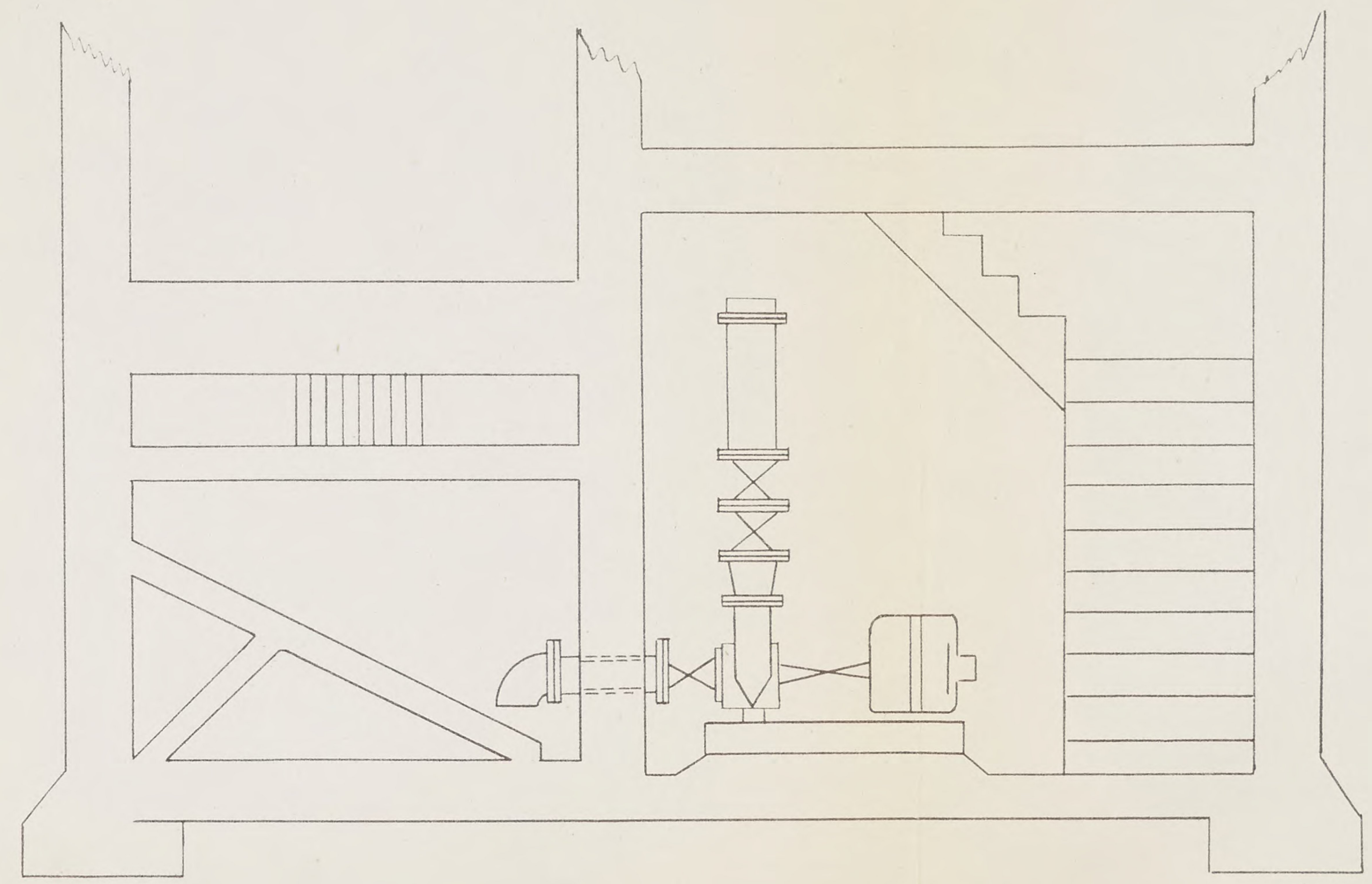


PLAN VIEW

FINAL SETTLING TANK

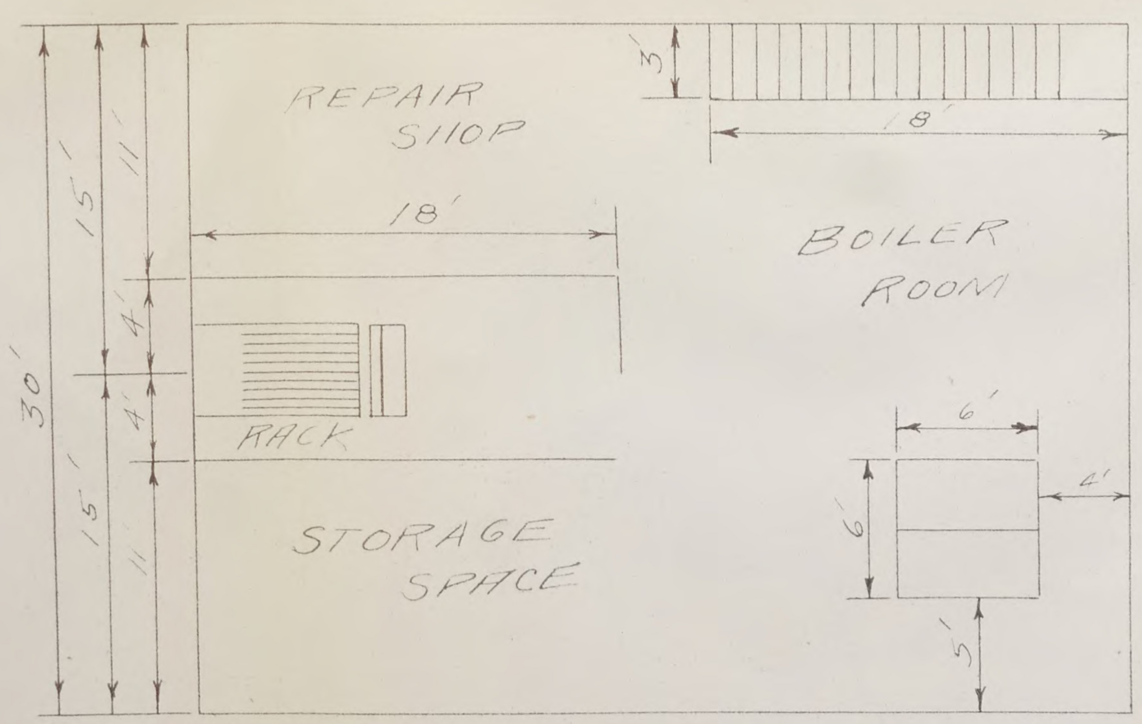


MAIN FLOOR

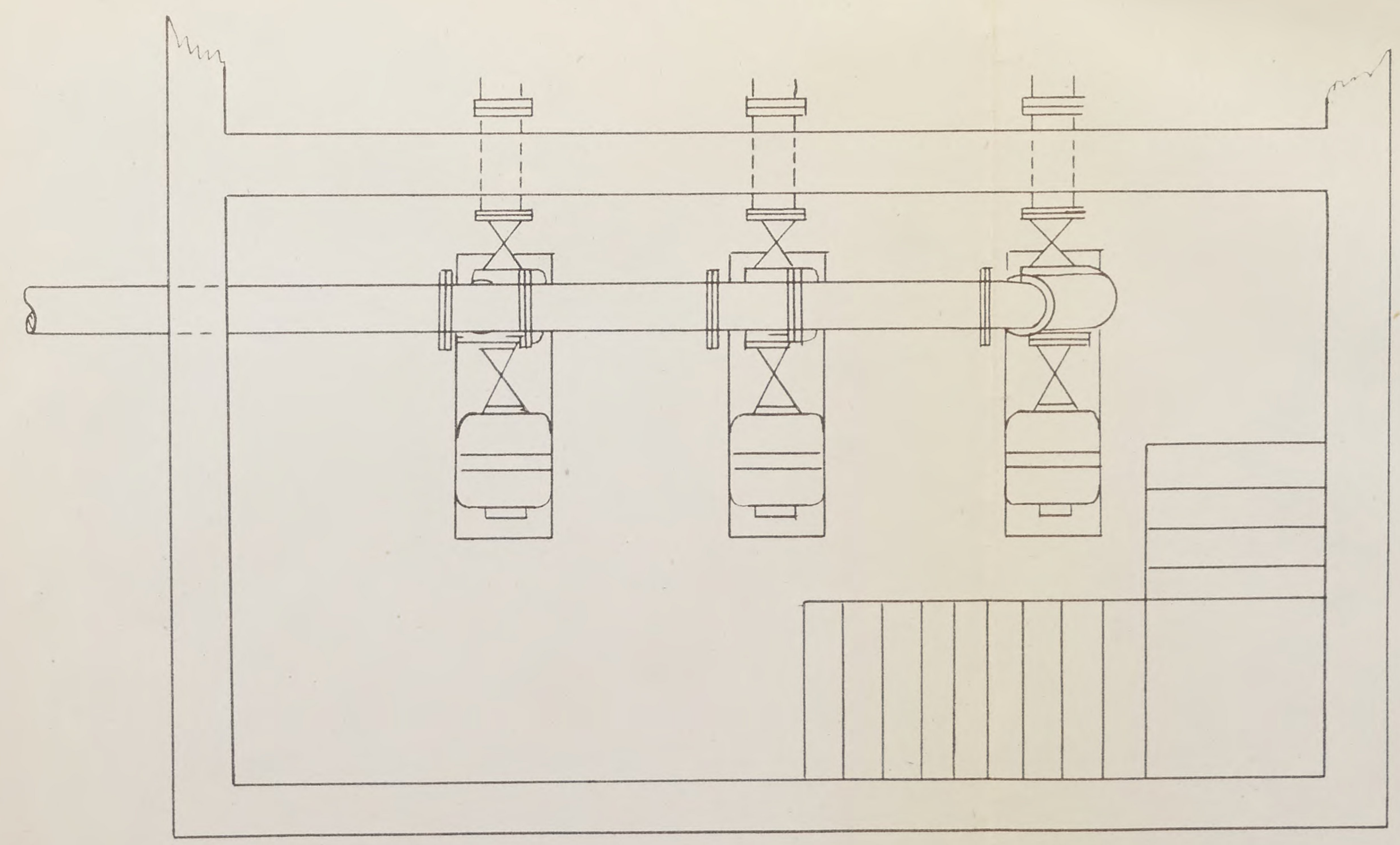


WET WELL + PUMP ROOM

SCALE $\frac{1}{2}$" = 1'



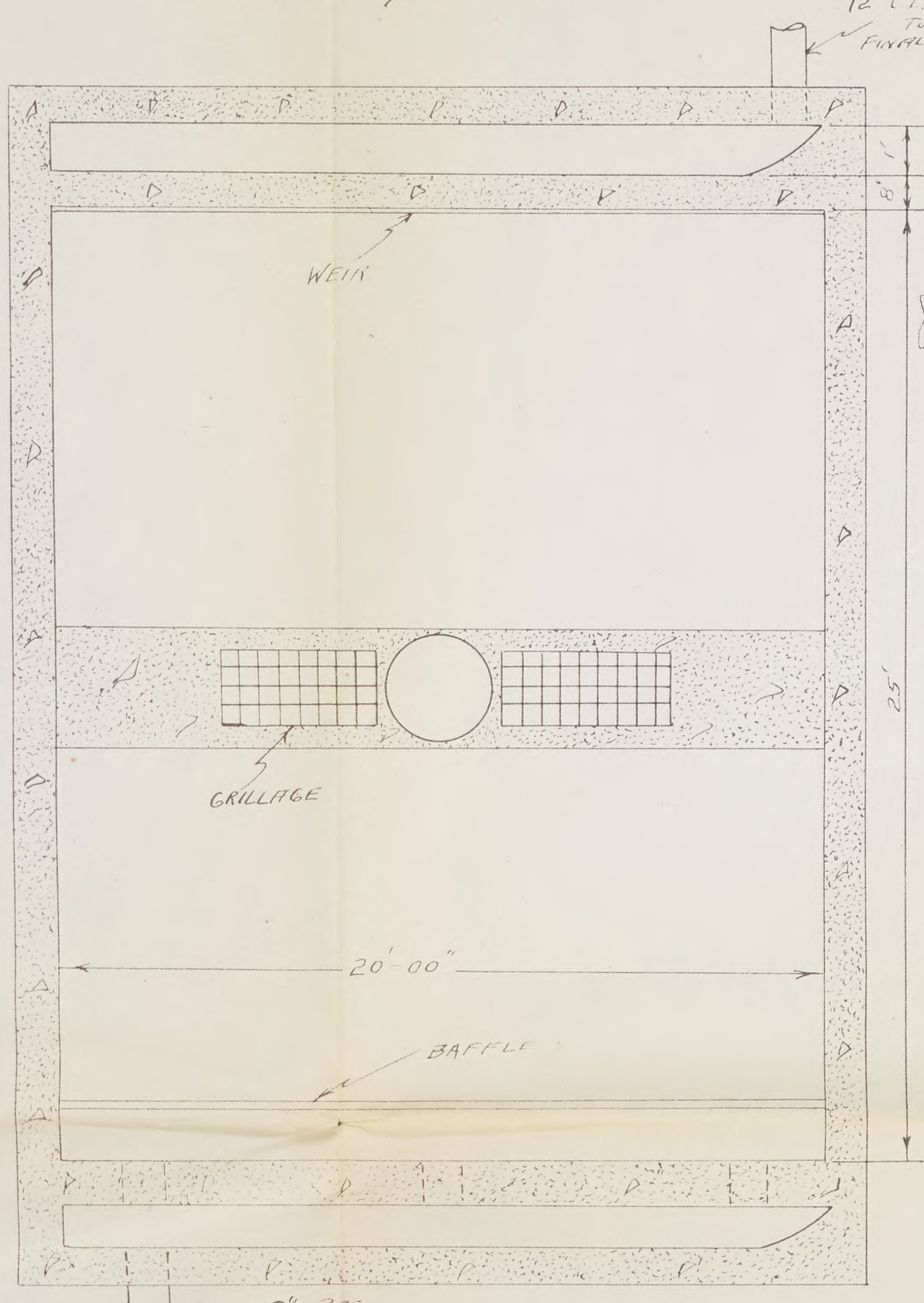
BOILER ROOM FLOOR



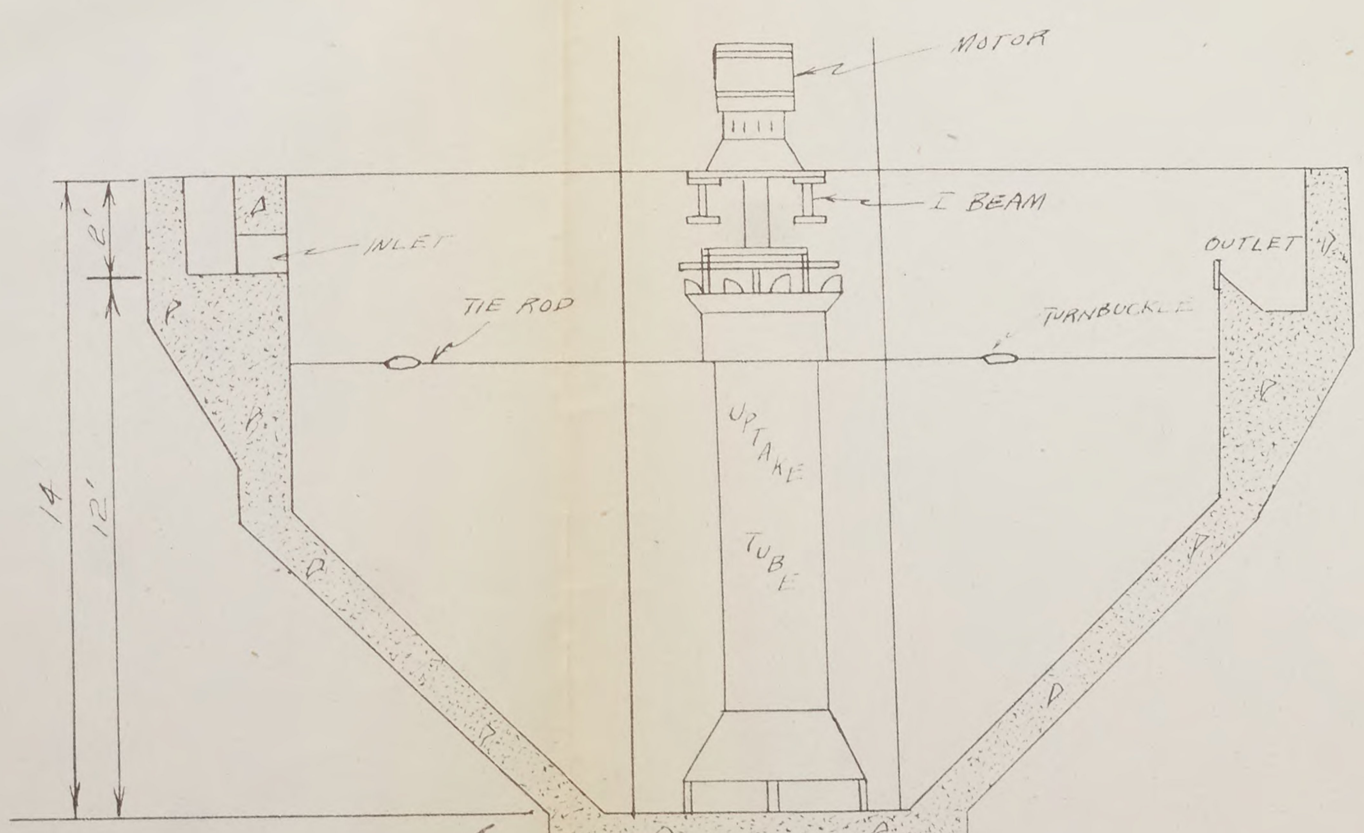
PUMP ROOM - PLAN VIEW

SERVICE BUILDING - HOLT MICHIGAN PLANT

SCALE 1/4" = 1'

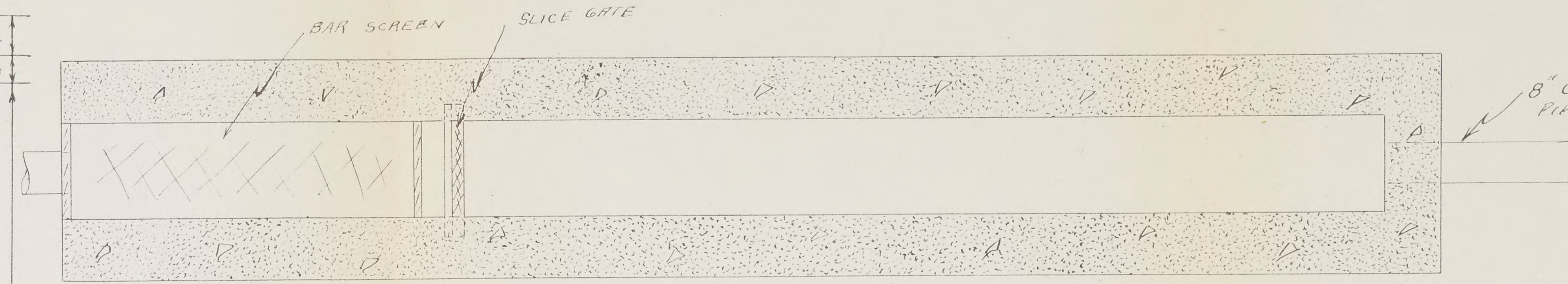


PLAN VIEW



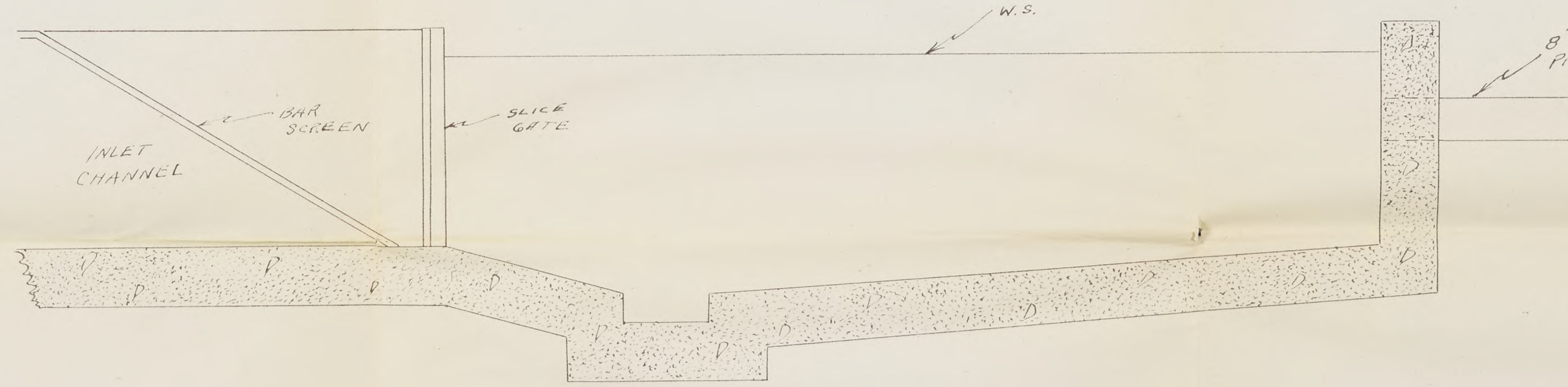
SIDE VIEW

AERATION TANK



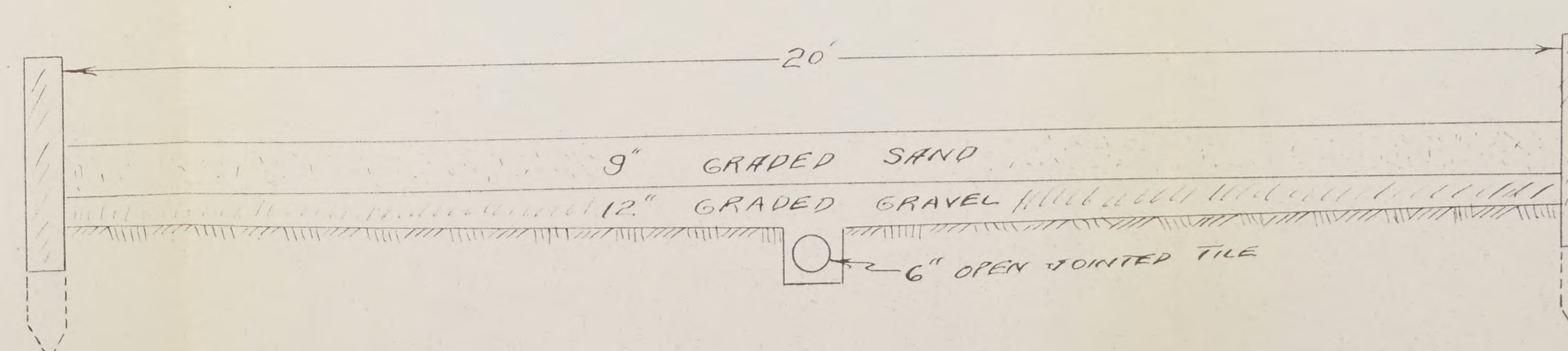
PLAN VIEW

HORIZONTAL SCALE - 1" = 4'
VERTICAL SCALE - 1" = 2'

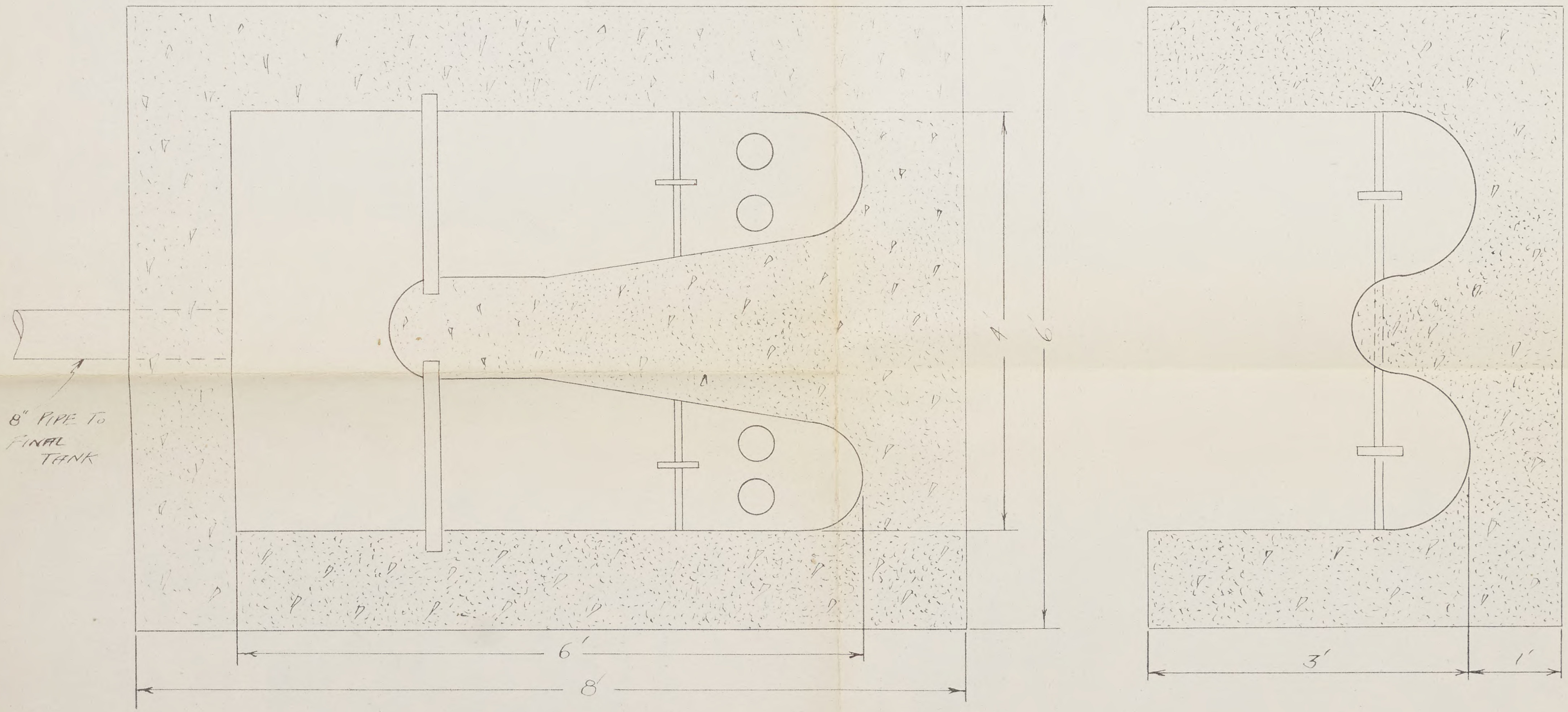


SIDE VIEW

GRIT CHAMBER

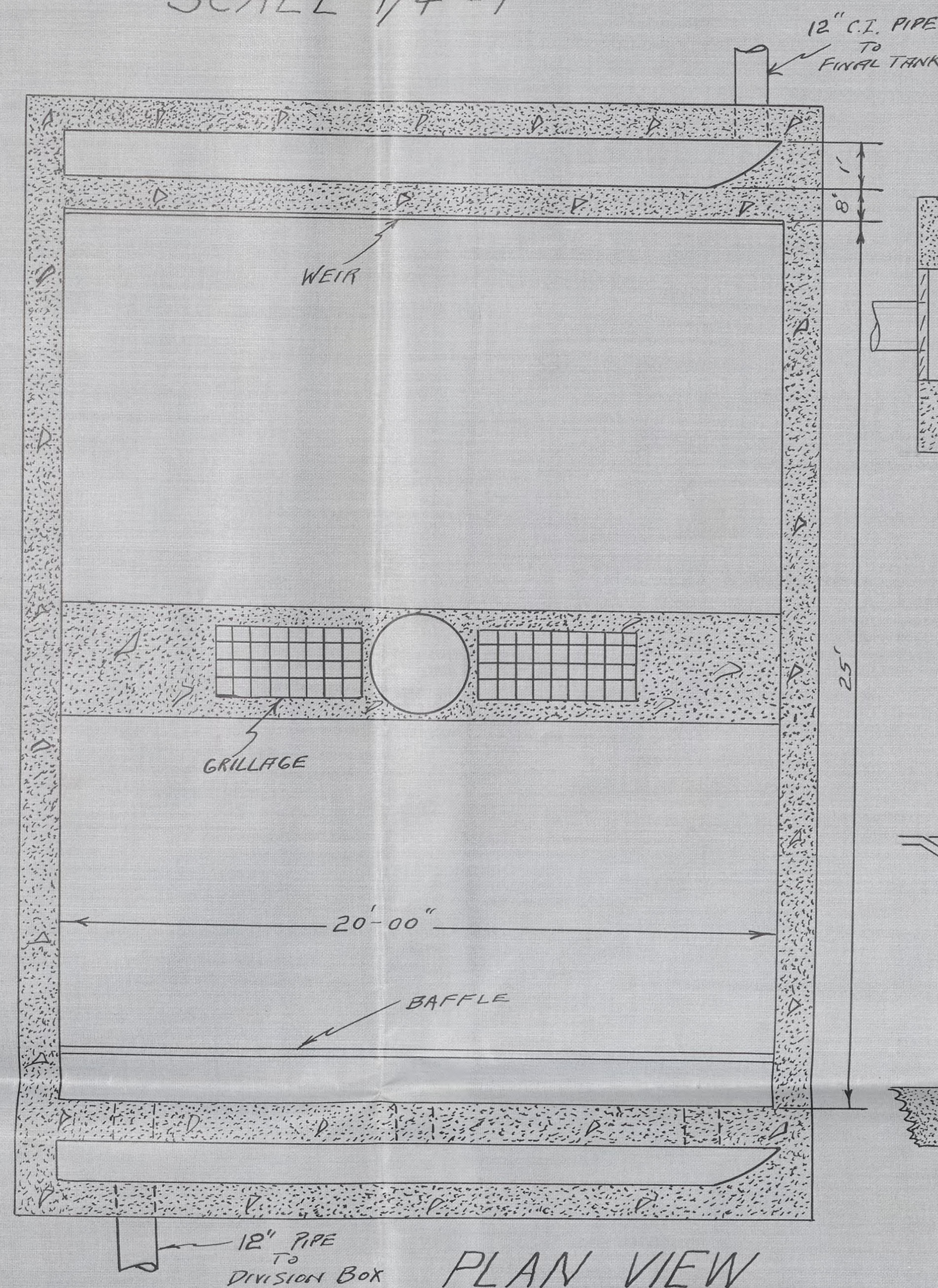


SLUDGE DRYING BED 1" = 2'

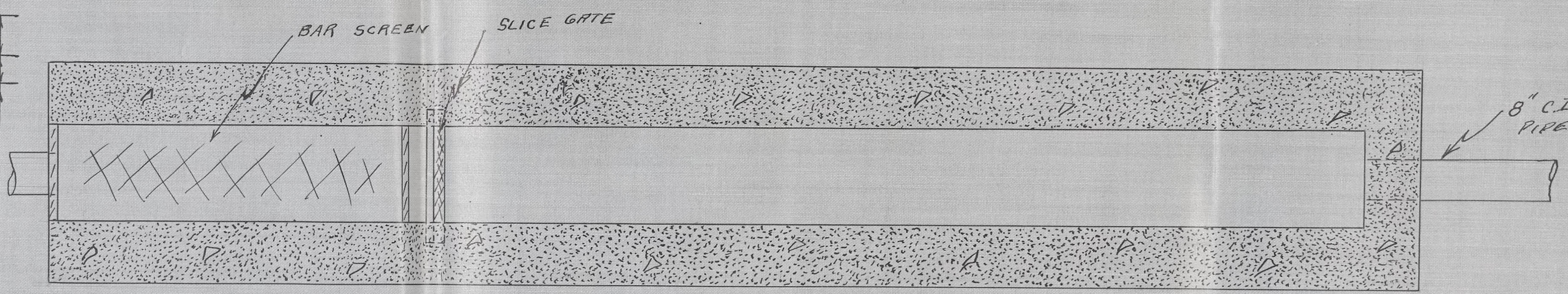


SLUDGE DIVISION BOX

SCALE 1/4" = 1'

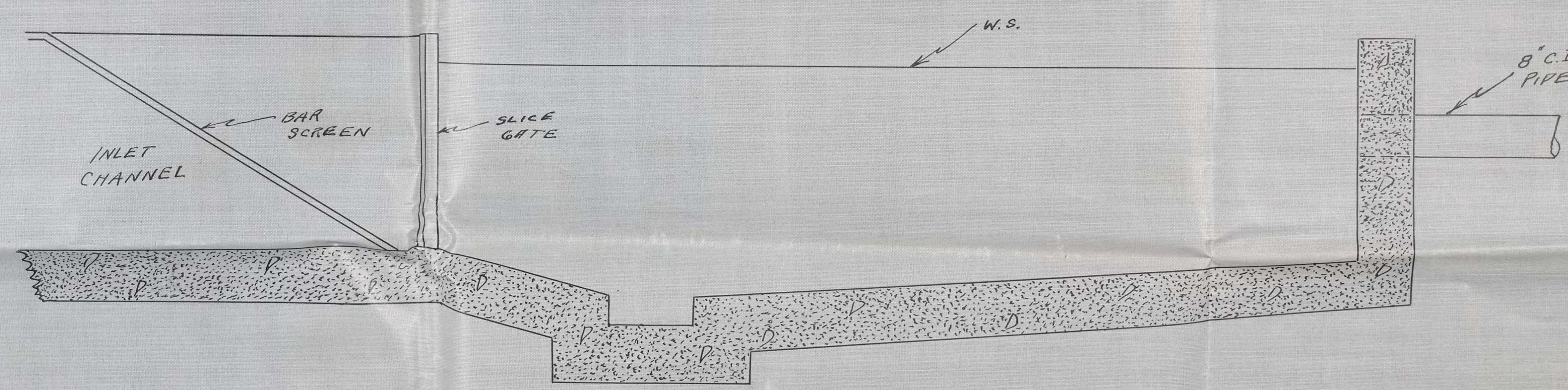


PLAN VIEW



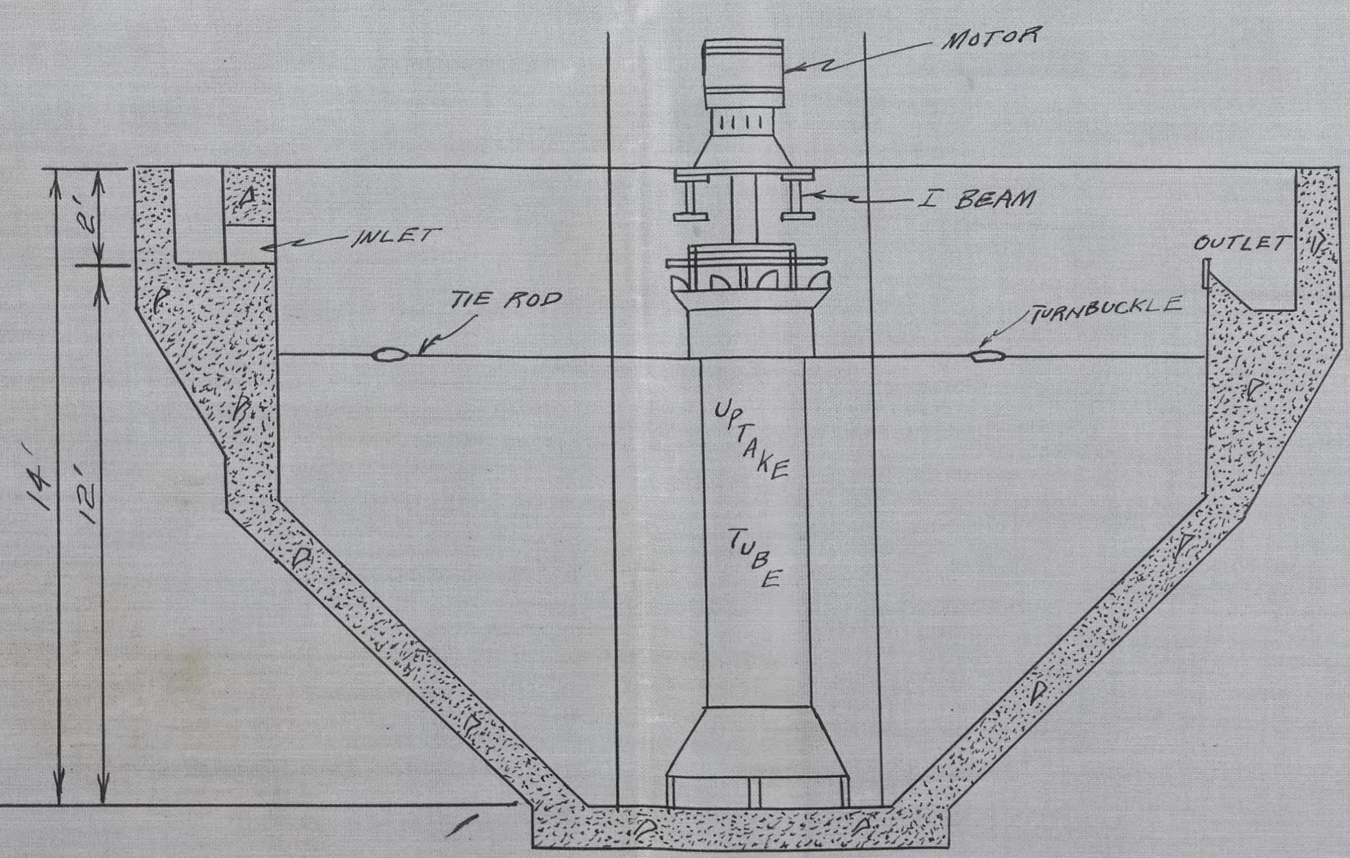
PLAN VIEW

HORIZONTAL SCALE - 1" = 4'
VERTICAL SCALE - 1" = 2'



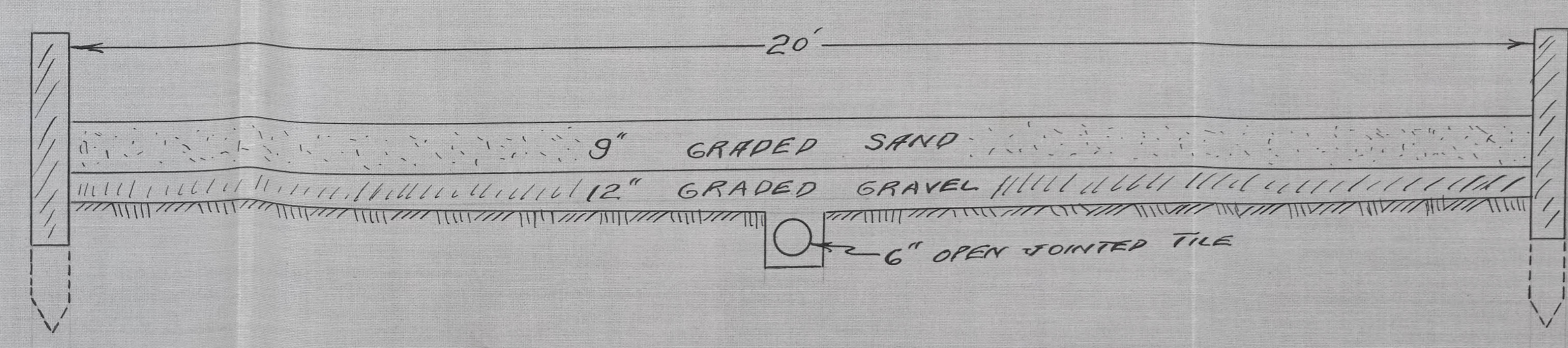
SIDE VIEW

GRIT CHAMBER

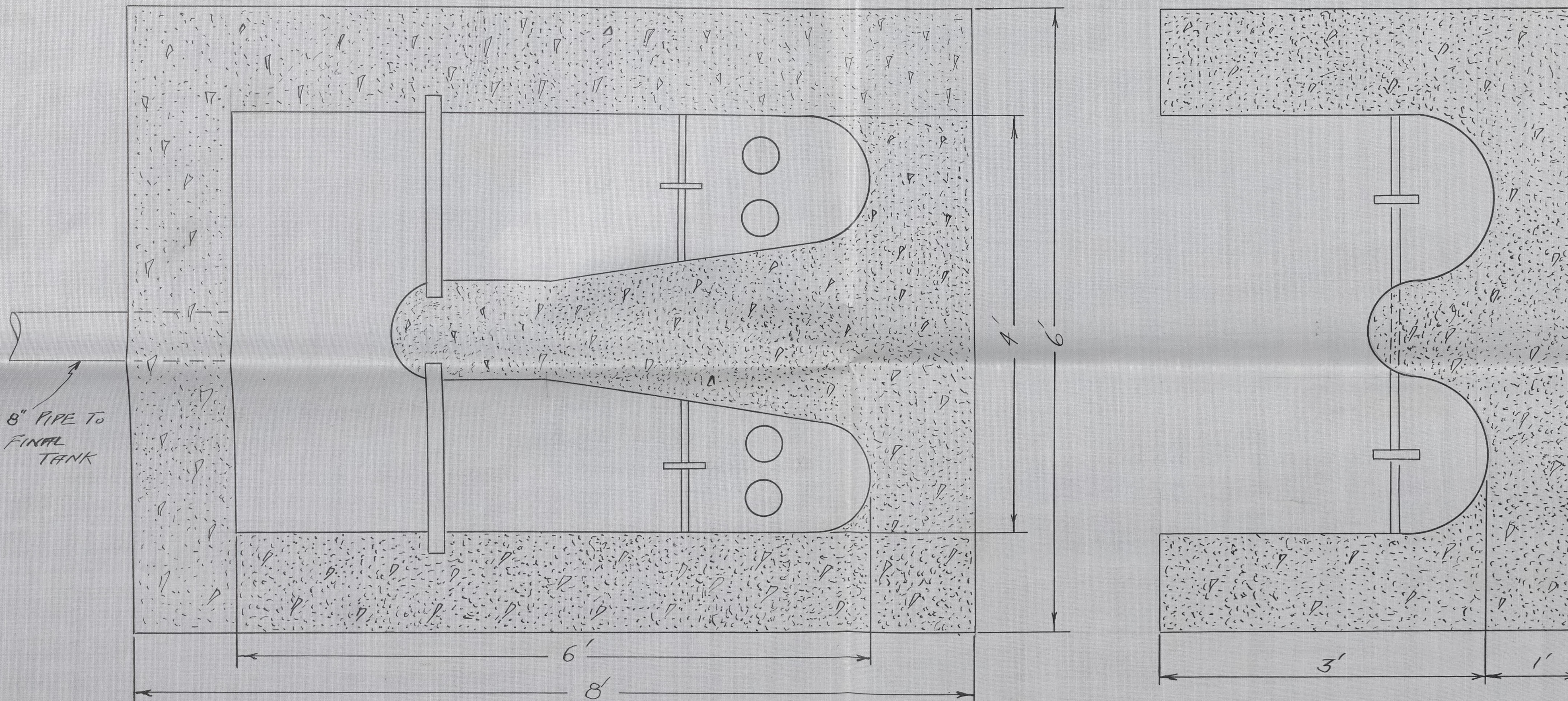


SIDE VIEW

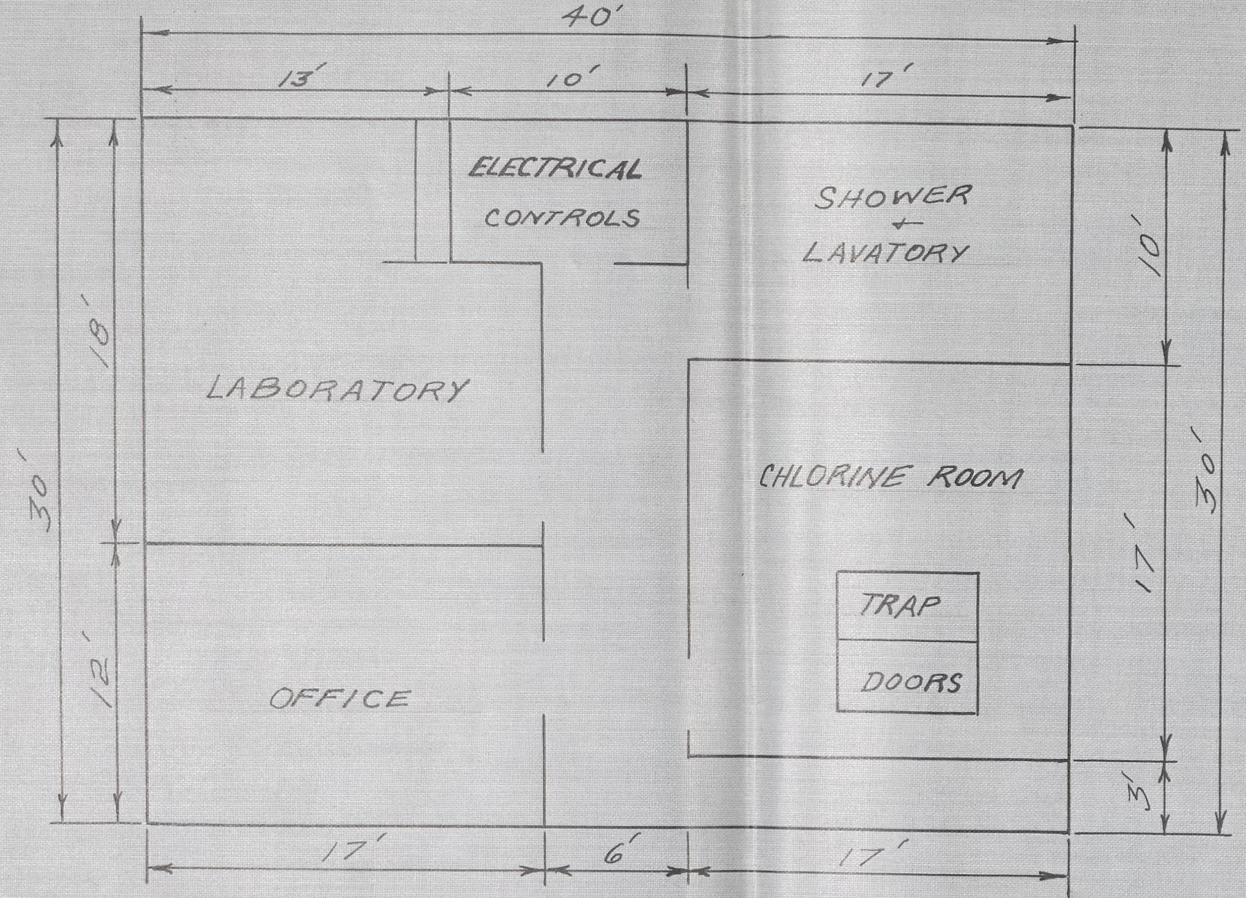
AERATION TANK



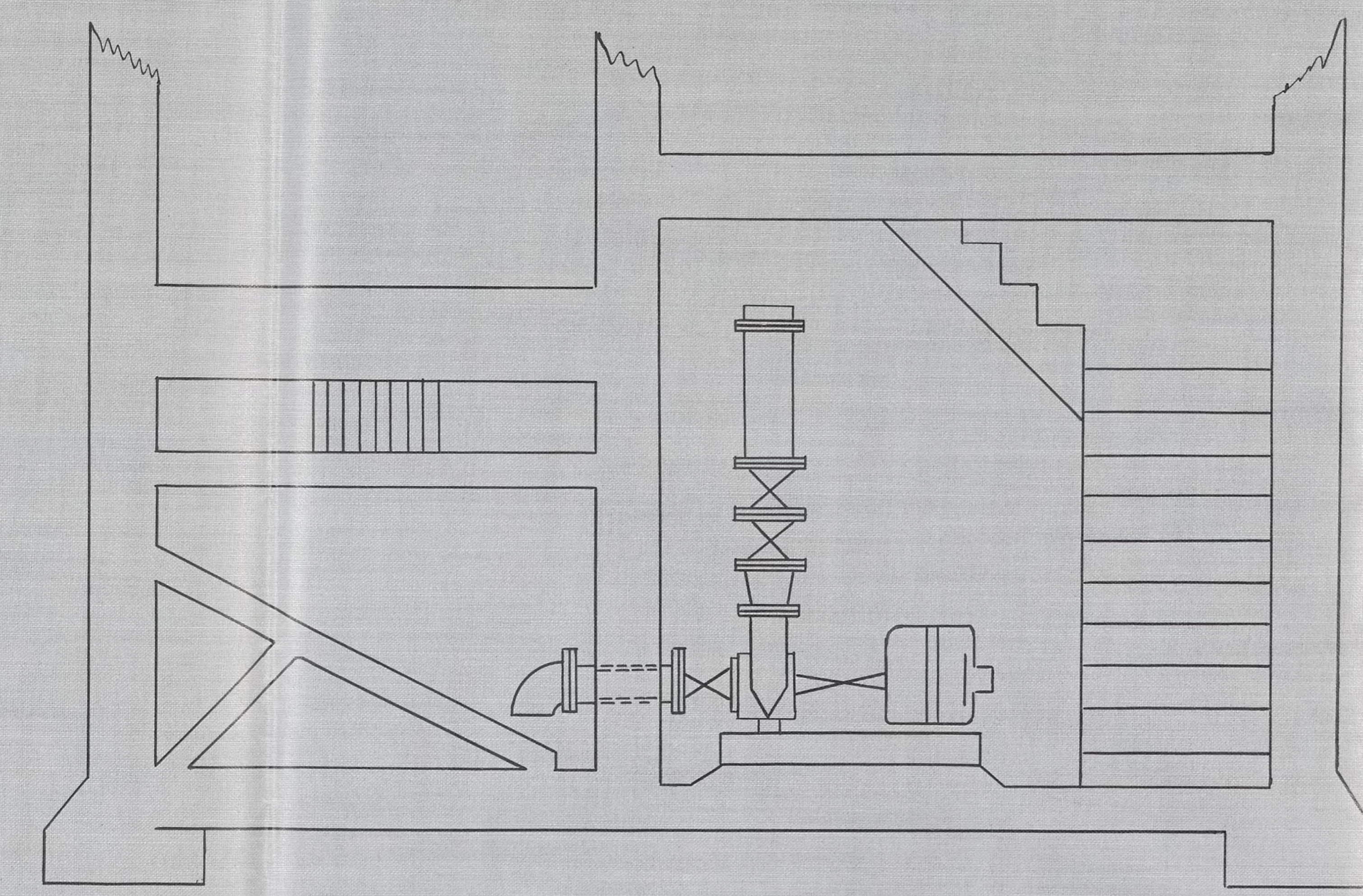
SLUDGE DRYING BED 1" = 2'



SLUDGE DIVISION BOX

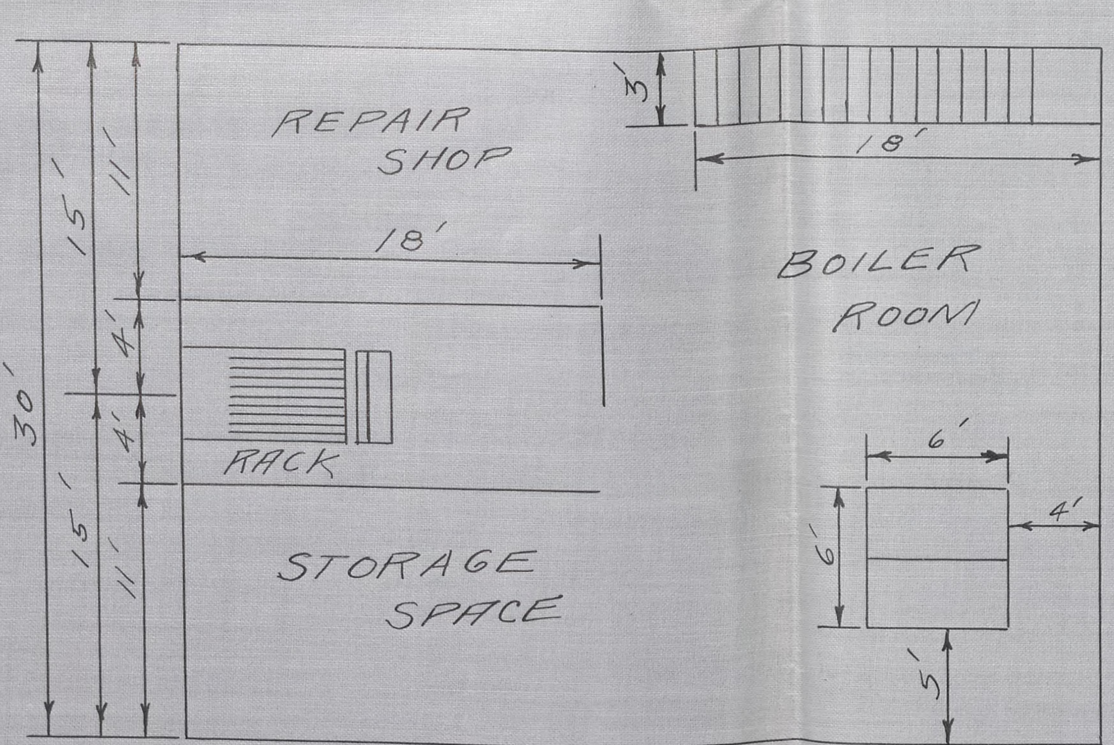


MAIN FLOOR

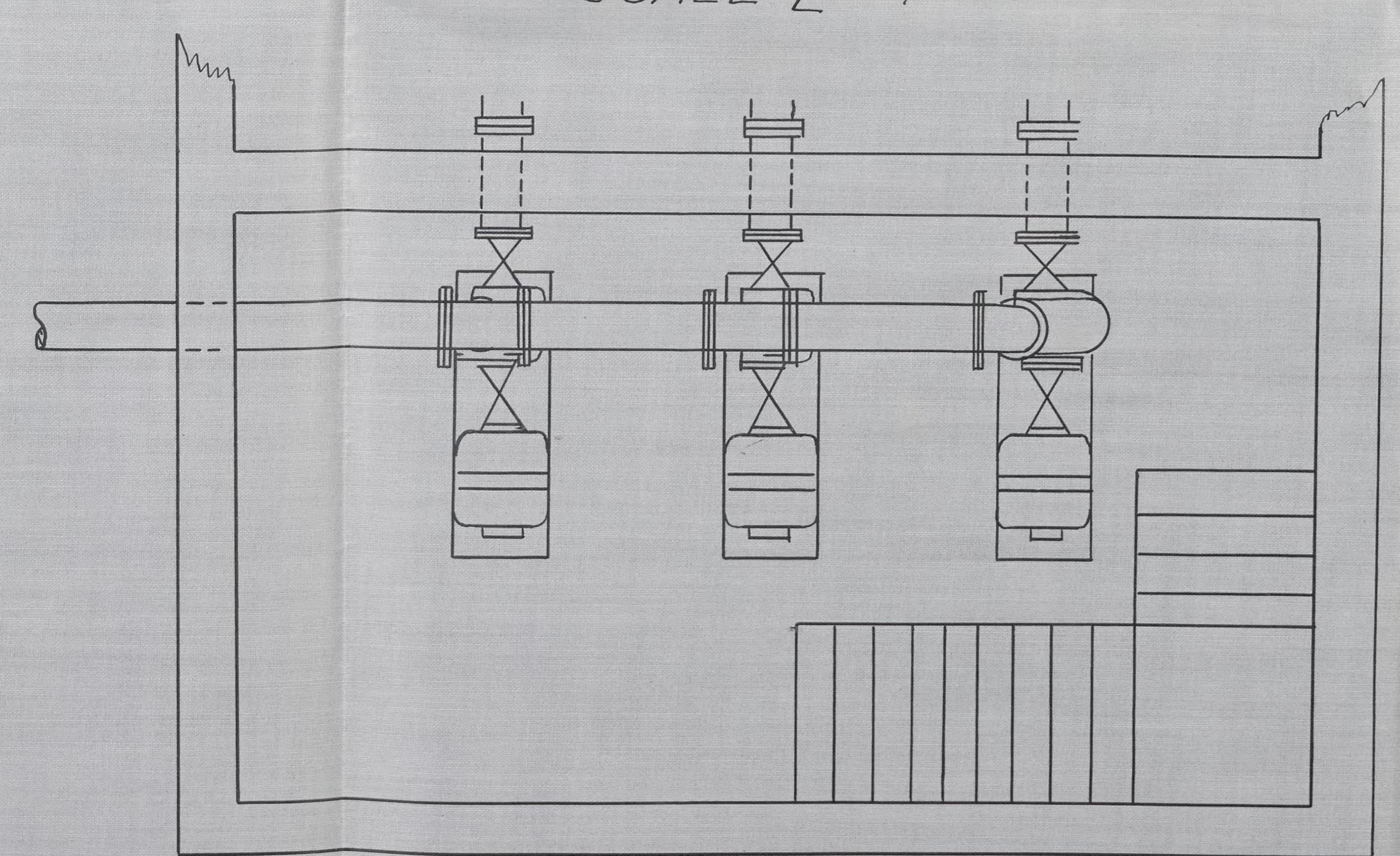


WET WELL + PUMP ROOM

SCALE 1/2" = 1'

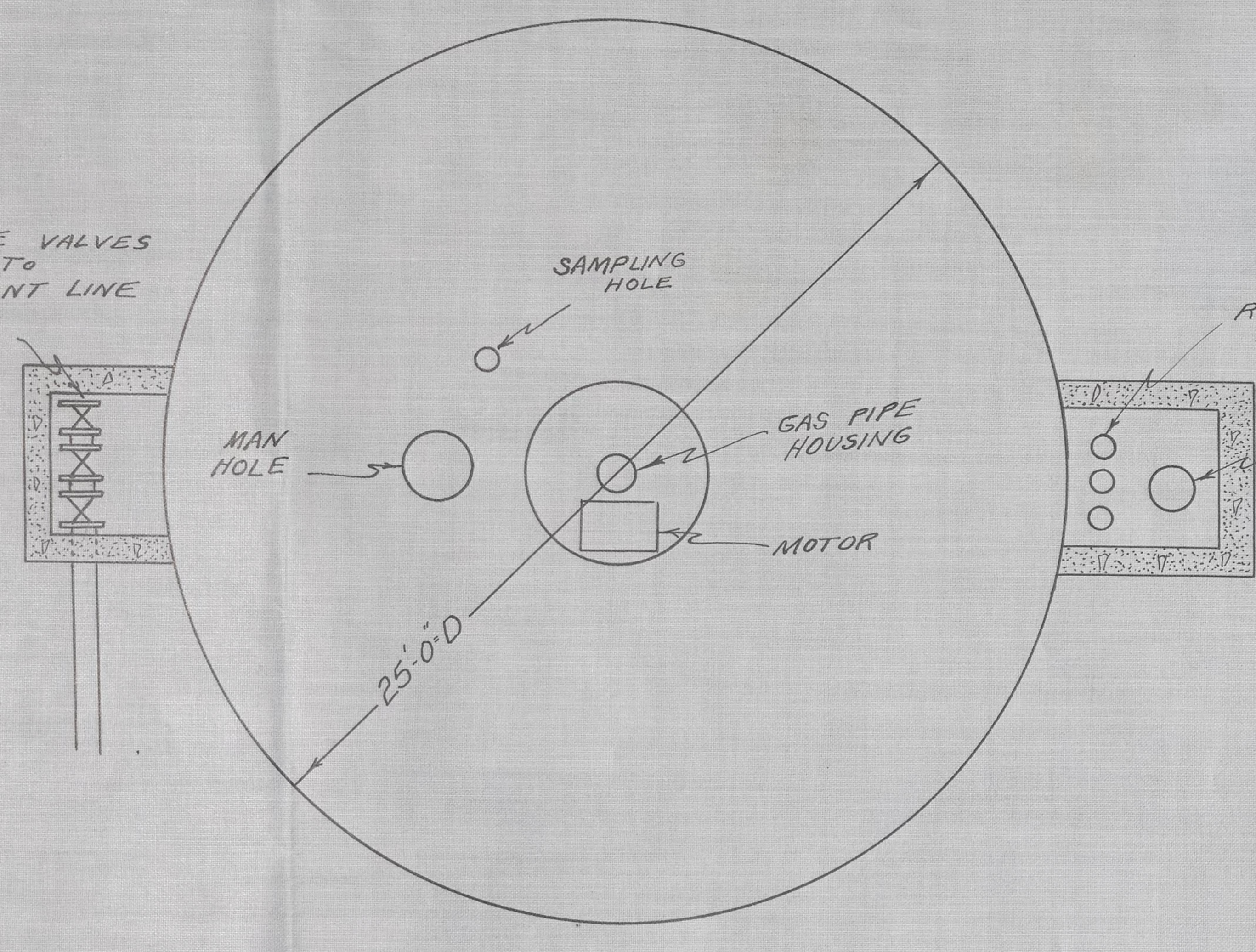


BOILER ROOM FLOOR

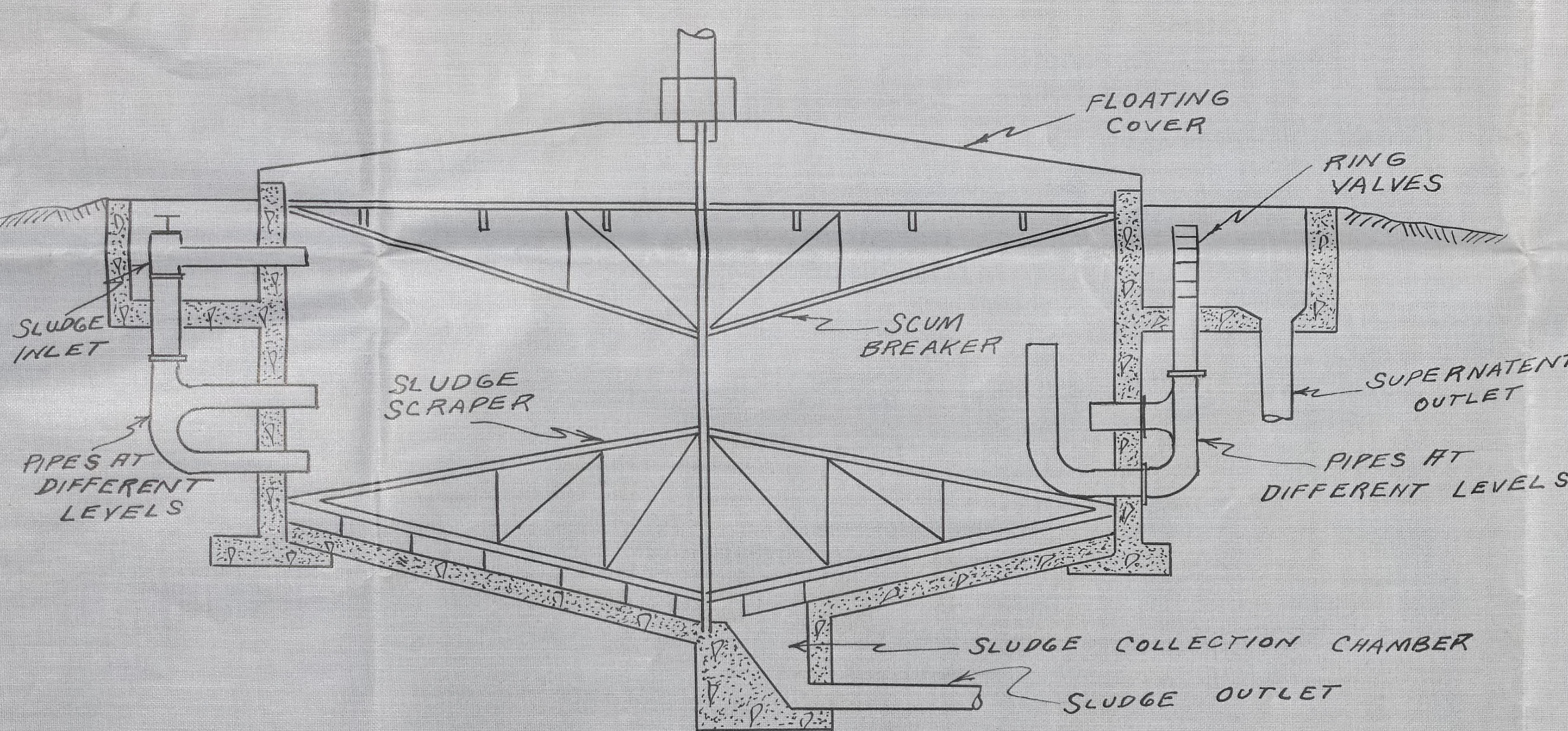


PUMP ROOM - PLAN VIEW

SERVICE BUILDING - HOLT MICHIGAN PLANT

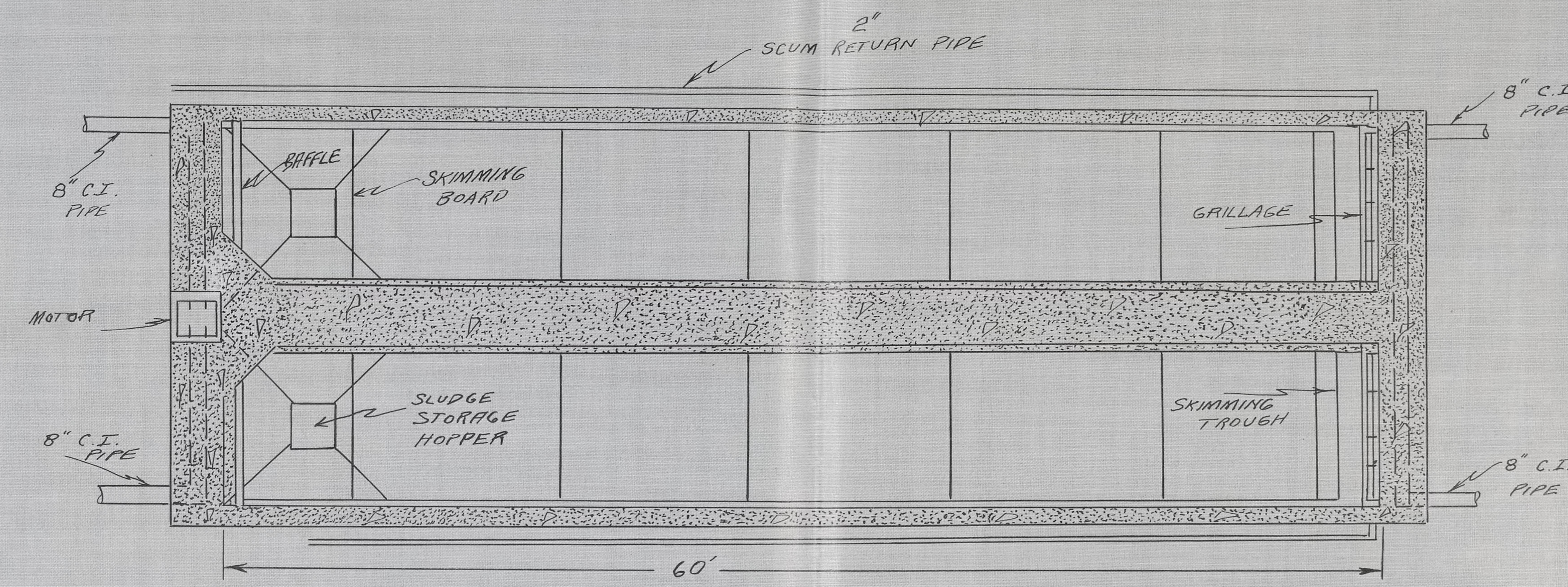


PLAN VIEW



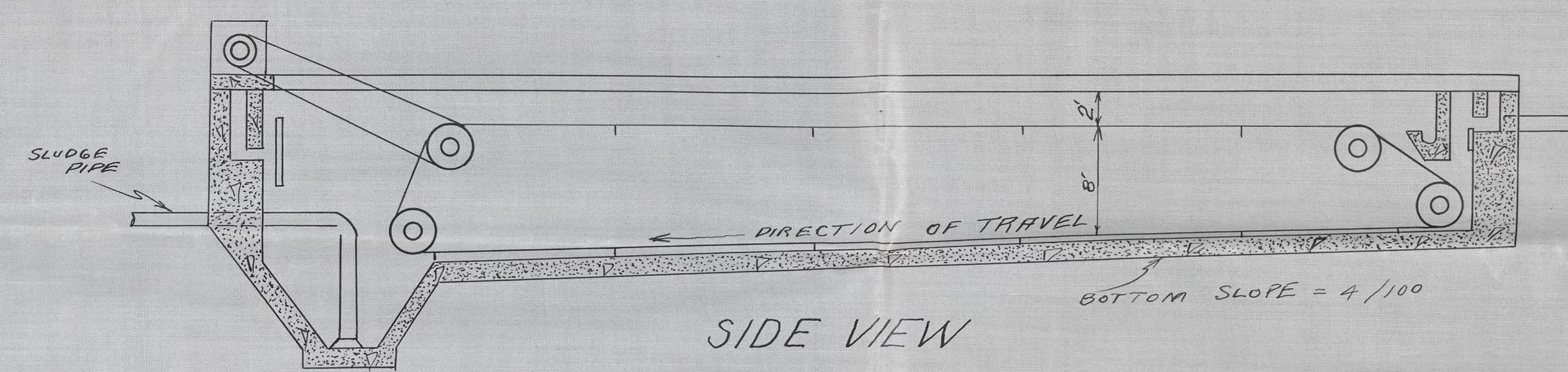
SIDE VIEW

SLUDGE DIGESTION TANK



PLAN VIEW

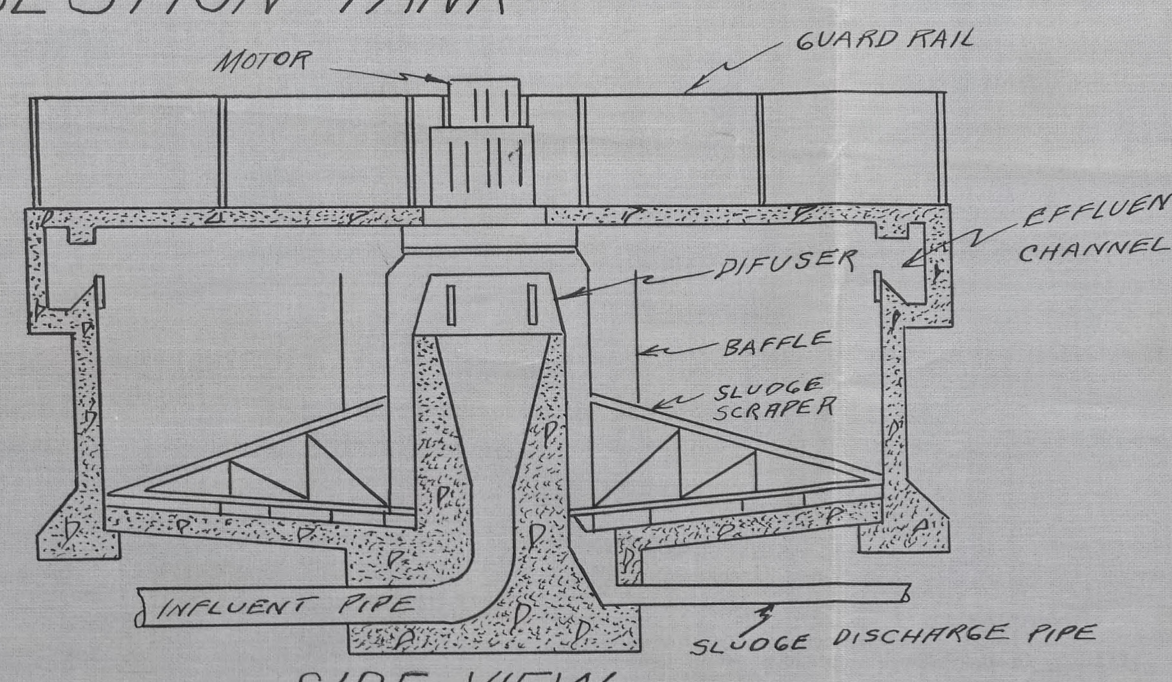
PRIMARY TANK 1" = 7'



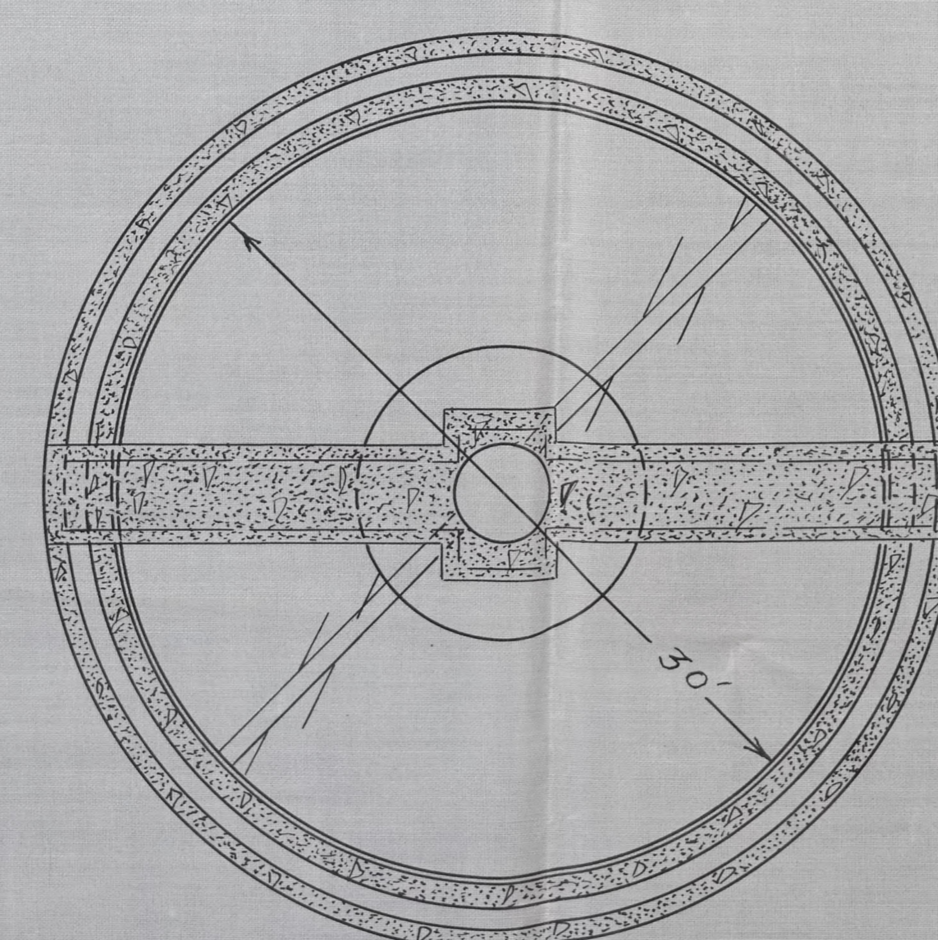
SIDE VIEW

PRIMARY SETTLING TANK

SCALE ALL TANKS 1" = ~~#~~ 7'



SIDE VIEW



PLAN VIEW

FINAL SETTLING TANK

Pocket has: 8 Plans