A DESIGN FOR A SEWAGE TREATMENT PLANT FOR THE CITY OF GRANDVILLE, MICHIGAN

Thesis for the Degree of B. S.

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For the City of Grandville, Michigan

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THESIS

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A DESIGN FOR A SEWAGE TREATMENT PLANT
FOR THE CITY OF GRANDVILLE, MICHIGAN

At present the sewage of the city of Grandville, Michigan, runs into a large septic tank near Grand River. This plant has no mechanical equipment for removing the sludge from the two tanks. The practice has been to remove the sludge from the tanks about twice a year. The city does not own equipment for doing this work, so the work is let on contract. The sludge is pumped out with a diaphragm pump into a tank truck. It is then removed to a nearby farm for use as fertilizer.

The present plant is unsatisfactory for several reasons. Due to the fact that the sludge is removed so seldom, at times sludge is carried over into the river with the plant effluent. The accumulation of sludge in the tanks cuts down the detention period to almost nothing, which is a very undesirable condition. The plant is so nearly of the same elevation as the average level of the Grand River that a slight rise of the river floods out the plant and backs up in the sewers for a considerable distance.

Due to the fact that the city of

Grand Rapids is using only primary sedimentation for its sewage, it does not seem necessary to provide any more treatment than this for the sewage of the city of Grandville. A primary-sedimentation separate-sludge-digestion plant, therefore, will be used, with provision for the chlorination of the plant effluent. Due consideration will be given to the possibility of future expansion and more complete treatment in the design and layout of the plant. In this case activated sludge treatment seems to be the most logical choice for the anticipated future completion of the plant.

POPULATION STATISTICS

Year	Population*
1820	799
1830	1,345
1840	1,5[illegible]
1850	1,910
1860	2,[illegible]73
1870	2,3[illegible]1
1880	2,[illegible]79
1890	[illegible],030

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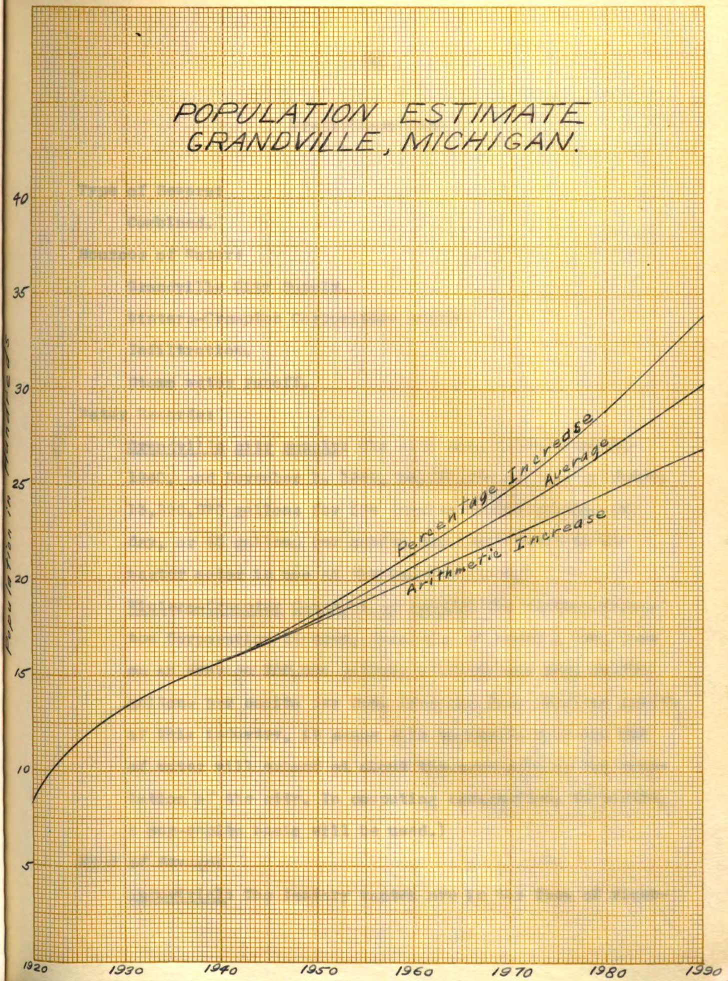
* Populations given for the years 1820, 1830, and 1840 are based on the U.S. Census; populations given for subsequent years are based on estimates.


POPULATION ESTIMATE
GRANDVILLE, MICHIGAN.
Percentage Increase
Average
Arithmetic Increase
Population in Hundreds
40
35
30
25
20
15
10
5
1920
1930
1940
1950
1960
1970
1980
1990

Preliminary Data

Type of Sewers:

Combined.

Sources of Water:

Grandville City Supply.

Winters-Crampton Corporation supply.

Infiltration.

Storm water runoff.

Water Records:

Grandville city supply: The city sold between May 1, 1940, and November 1, 1940, 13,250,970 gallons of water. 13,250,970 gallons for 184 days is 72,016 gallons per day, or 45 gallons per capita per day. (There is no master meter in use at the pumping plant.)

Winters-Crampton Corporation supply: The Winters-Crampton Corporation factory, operating 24 hours a day, uses on an average 300,000 gallons of water per day, or 300 gallons per capita per day. (Judging from previous growth of this industry, it seems safe to assume that its use of water will expand at about the same rate as the population of the city. In computing sewage-flow, therefore, a per-capita basis will be used.)

Kind of Sewage:

Industrial: The factory wastes are in the form of rincewaters from plating rooms. No very noticeable turbidity could be seen at any time during the inspection of the factory. Acid wastes, however, such as plating wastes, even though without turbidity may cause difficulties in sewage-plant operation. In case difficulties are encountered in plant operation at Grandville because of these wastes, the Winters-Crampton Corporation should be compelled to treat their plating wastes before discharging them into the sewers.

Household: The household wastes are of the usual character.

Infiltration: Infiltration is probably not very great except in sewers near the river. Calculating industrial wastes on a per-capita basis will give enough leeway to provide for such infiltration as occurs.

Storm water: Storm water is usually quite clear, but it probably carries some sand and grit into the sewers, particularly during very heavy rains, for some of the streets are not paved yet. There are catch basins at the curb inlets where the heavier grit should settle out.

Estimated flow of sewage:

(Because the river at this time is high and backed up into the sewers, no sewage flow measurements could be taken.)

Estimated normal flow:

Industrial: 200 gallons per capita per day.

Household: 45 gallons per capita per day.

Estimated minimum flow: 50% of normal flow.

Estimated maximum flow: 150% of normal flow.

Estimated storm flow: 100% of maximum flow.

(There does not appear to be any possibility of providing over-flow structures to take care of excessive storm flow, and therefore the entire flow must be pumped even to by-pass it; the storm flow, therefore, must be assumed to be 100% of the maximum flow. Some storage will be provided, moreover, in the wet well.)

ESTIMATED SEWAGE FLOW

Year:	Minimum:	Average:	Maximum:	Maximum plus Storm:
1940	186,000 G.D.	372,000 G.D.	558,000 G.D.	1,116,000 G.D.
	130 G.P.M.	260 G.P.M.	390 G.P.M.	780 G.P.M.
	.29 c.f.s.	.58 c.f.s.	.86 c.f.s.	1.73 c.f.s.
1970*	289,750 G.D.	579,500 G.D.	867,750 G.D.	1,738,500 G.D.
	200 G.P.M.	400 G.P.M.	600 G.P.M.	1,200 G.P.M.
	.45 c.f.s.	.90 c.f.s.	1.35 c.f.s.	2.70 c.f.s.

* The population as of 1970 is taken as the basis of design because it is most economical to do so.

Flood Conditions:

(Data obtained from Grand Rapids Engineering Office)

The nearest flood gauge is located at the south-east end of the Fulton Street Bridge, in the city of Grand

Rapids, at station 130 plus 20 on the river profile (note 160-foot stations). The maximum elevation of the water surface at this point during flood is 19.50 feet (zero is U.S.G.S. Elevation 589.01 feet), the worst flood on record having been that of March, 1904. At station 132 plus 50, 4,230 feet down-river, the maximum elevation of the water surface during flood is 18.50 feet, so that the drop of the water surface between stations is 1 foot.

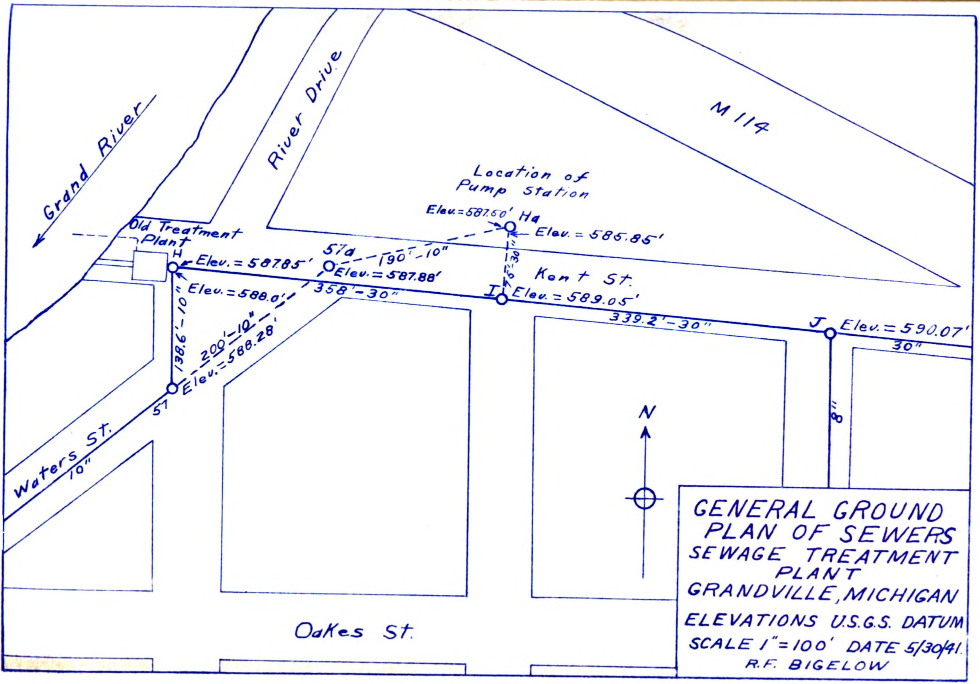
The distance from the gauge to the proposed plant site is 6 miles, or 31,680 feet, as measured by driving along the river-bank road in an automobile and checked by measurement on a map. The drop of the water surface between the gauge and the proposed plant site, therefore, is 31,680/4,230 or 7.5 feet. The maximum elevation above sea level of the water surface at the gauge is 589.01 (U.S.G.S. Elevation) plus 19.50, or 608.51 feet. The maximum at the proposed plant site, therefore, is 608.51 minus 7.5, or 601.01 feet. A topographic map of the proposed site is not available, but the ground is fairly level, and approximately 595 feet above sea level, or 6.01 feet below the maximum elevation of the water surface during flood.

This will necessitate filling in around the structures to prevent flooding, and an elevated road will also have to be built. After filling, the ground level will be 603.00 feet above sea level.

Design of the Plant

In order to have room for the proposed plant, and also to provide for future expansion, the location had to be changed, as shown on the general ground plan. The old plant will be abandoned, along with 350 feet of 30-inch sewer, from manhole I to manhole H, and 133.8 feet of 10-inch sewer, from manhole 57 to manhole H.

New sewers are to be laid as shown on the general ground plan. These new sewers will be: 200 feet of 10-inch sewer from manhole 57 to manhole 57a, 100 feet of 10-inch sewer from manhole 57a to manhole Ha, and 75 feet of 30-inch sewer from manhole I to manhole Ha. Manhole Ha is just at the entrance to the pump station. The elevation at this point of the 10-inch sewer is 577.50 feet; of the 30-inch sewer, 5[illegible].[illegible]5 feet.


GENERAL GROUND
PLAN OF SEWERS
SEWAGE TREATMENT
PLANT
GRANDVILLE, MICHIGAN
ELEVATIONS U.S.G.S. DATUM
SCALE 1" = 100' DATE 5/30/41.
R.F. BIGELOW
Grand River
River Drive
M 114
Location of
Pump Station
Elev. = 587.60' Ha
Elev. = 585.85'
Old Treatment
Plant
H
Elev. = 587.85'
57a
190'-10"
Elev. = 587.88'
Kent St.
358'-30"
Elev. = 588.0'
I
Elev. = 589.05'
339.2'-30"
J
Elev. = 590.07'
30"
8"
138.6'-10"
200'-10"
Elev. = 588.28'
57
Waters St.
10"
Oakes St.
N

Design of Grit Chamber:

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

Maximum Q = 1,728,000 gallons per day

= 2.70 c.f.s.

Cross-section = 2.7 square feet

Use one chamber 60 feet long, 1 foot 6 inches wide, 4 feet 6 inches deep, composed of 2-foot freeboard, 2-foot water depth, and 6 inches for grit storage. Grit will be removed with a mechanical remover to be used intermittently, so that space must be provided for grit storage.

A by-pass conduit will be provided for use in case work must be done on the grit-removal mechanism. This conduit will be 1 foot wide, 2 feet deep, and 60 feet long.

<u>Design of Primary Sedimentation Tanks:</u>

Average Q = .90 c.f.s.

Use detention period of 4 hours for average discharge.

Volume = .90 x 4 x 60 x 60 = 12,960 cu. ft.

Use two tanks 10 feet deep, 14 feet wide, with 2-foot freeboard.

$$\text{Length} = \frac{12,960}{2 \times 10 \times 14} = 46 \text{ feet.}$$

Use an effluent channel of reinforced concrete 8 inches deep and 8 inches wide, with an adjustable metal wier on one side, allowing a total adjustment of 3 inches. A skimming trough to carry away the scum should be provided, to be 1 foot wide and 8 inches deep, with a concrete incline of 30 degrees up which the scum can 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 pocket at the inlet end of the tank.

<u>Design of Chlorination Basin:</u>

Detention period of 20 minutes for average sewage flow

$$\text{Volume} = \frac{20}{1440} \times 578{,}000 = 8{,}030 \text{ gallons}$$

$$= 1{,}070 \text{ cu. ft.}$$

Use baffled basin 9 feet deep, 12 foot square, with 2-foot freeboard; the baffles should be spaced 3 feet apart with 3-foot end-around clearance.

A by-pass will be provided around the chlorination basin.

A direct-feed chlorinator, supplied by Wallace & Tiernan Company, will be used; it will be housed in a special well-ventilated room on the ground floor of the pump station, with a tube-connection to the diffuser in the chlorination basin.

<u>Sludge-Digestion Tank</u>:

Flow = .5725, or .6 M.G.D.

4 hours settling

Assume 300 p.p.m. suspended solids

200 p.p.m. organic solids

Temperature of digestion tank 70 degrees F.

Storage period of six months

64% of suspended solids removed (see Metcalf and Eddy, <u>Sewerage and Sewage Disposal</u>, fig. 175, p. 534)

.64 x 300 x 8.34 x .6 = 960 lb. per day solids removed.

960 x 30 = 28,800 lb. per month solids removed.

Assume 3 months for digestion at 60 degrees F.

35% of time at 60 degrees F. required for digestion at 70 degrees F. (see Metcalf and Eddy, <u>op. cit.</u>, fig. 158, p. 474)

Use one month, therefore, as period for digestion at 70 degrees F.

Assume that 50% of the organic solids are digestible.

Organic solids = 2/3 x 28,800 = 19,200 lb. per month

Digestible organic solids = .5 x 19,200 = 9,600 lb. per month.

From first five months' solids subtract solids remaining at the end of six months:

5 x 28,800 - 5 x 9,600 = 96,000 lb.

From [illegible] months' solids subtract the solids remaining at the end of [illegible] months:

[illegible]8,800 − ½ x 8,[illegible] = 34,000 lb.

Total = 93,[illegible] lb. + 3[illegible],[illegible] lb. = 1[illegible],[illegible] lb.

Assume sludge to be 94% moisture, with specific gravity of 1.0[illegible]

$$\text{Volume} = \frac{[illegible]}{.[illegible] \times 8.3[illegible] \times 1.[illegible]} = 1[illegible]8,0[illegible]0 \text{ gallons}$$

$$= 3[illegible],1[illegible] \text{ cu. ft.}$$

Use depth of [illegible] feet, diameter of 35 feet. Provide 4 feet of depth for supernatant liquid, giving a total depth of [illegible] feet at the side of the tank. The tank is 2½ feet deeper at the center.

Pipes will be provided for the withdrawal of the supernatant liquid at four different levels. Sludge-sampling pipes will also be provided at different levels.

The tank will be provided with hot-water coils on the inside of the tank walls.

A Downes floating cover will be used to provide for storage of gas. The gas from the digestion will be used to heat the water for the digester coils and also to heat the pump station building.

The supernatant liquid from the digester will be returned to the inlet of the primary tanks. The digested sludge will be drawn off on to under-drained drying beds.

Design of Sludge-Drying Beds:

Basis of design 1 square foot of bed area per capita, based on the true population plus that represented by the industrial wastes. On this basis, the total area will be about 6,000 square feet.

The beds are to be 20 feet wide, in order to facilitate cleaning.

Total length of bed is 6000 divided by 20 or 300 feet. Use 6 beds 50 feet long, grouped two long and three wide, making a rectangular area of beds 60 feet wide by 100 feet long.

The drying beds will consist of 12 inches of coarse sand underlain by 12 inches of graded gravel ranging in size from 1/8 inch to 1½ inch. The bottoms will slope slightly toward the underdrains of [illegible]-inch drain tile. The underdrains are to be 20 feet apart from center to center, and will run down the center of each bed, lengthwise of the beds. The underdrains will empty into the wet well.

The sidewalls will be of concrete 15 inches above ground, with planks that drop into grooves to permit easy entrance of a truck for removal of the dry sludge.

Approximate Head Losses and Elevations:

Pumps and Piping to Inlet of Grit Chamber:

Three feet will be allowed for head losses in pumps and piping to the inlet of the Grit Chamber.

Grit Chamber:

Velocity = 1 ft. per second

$n = .013$

$$R = \frac{\text{Area}}{\text{wetted perimeter}} = \frac{2.0 \times 1.5}{2 \times 2 + 1.5} = 0.545$$

(See Metcalf and Eddy, op. cit., fig. 37, after p. 130)

Slope = .19 ft. per 1000 feet.

Length = 60 ft. Head loss = .01 ft.

Allow 0.03 ft. total head loss from grit chamber entrance to outlet weir of primary tank.

Primary tank:

Conditions with maximum discharge equal to 2.7 c.f.s.

Effluent and Channel Loss:

$$\text{Velocity} = \frac{2.7 \times 144}{8 \times 9} = 5.4 \text{ ft. per second.}$$

$$R = \frac{72}{2 \times 8 - 9} = 2.88 \qquad n = .013$$

s = .58 ft. / 1000 ft. Fall = .0[illegible] x .[illegible]8 = .01[illegible] ft.

Effluent Wier loss:

Assume length of wier to be 24 ft.

Head for maximum discharge of 2.70 c.f.s.

$$\text{Discharge per foot of wier} = \frac{2.70}{24} = .113 \text{ c.f.s. per ft.}$$

(See Schoder and Dawson, Hydraulics, fig. 90, p. 175)

Head = .125 ft.

<u>Total Losses:</u>

Grit Chamber	.06 ft.
Primary Tank	.18 ft.
Total	.24 ft.

In times of floodchlorination would be discontinued, so it would not be necessary to have the chlorination basin above the flood level.

Three feet will be allowed for head needed in case of future expansion and more complete treatment. This seems to be a fair allowance for an activated sludge plant.

Elevation of water surface in Primary Tank is 601.01 plus 3.00, or 604.01 ft. Elevation of sewage at head of Grit Chamber is 604.01 plus .24, or 604.25 ft. Minimum Elevation of sewage in wet well is 583.85 ft. The head pumped against, therefore, is 604.25 minus 583.85, or 20.4 ft. Allow 3.00 for friction in pipes and pumps, so that the total head pumped against is 23.4.

Selection of Pumps:

Use non-clog pumps.

Maximum discharge - 1,200 G.P.M.

1 pump @ 200 G.P.M.

1 pump @ 400 G.P.M.

1 pump @ 600 G.P.M.

Pumps will be supplied by the Chicago Pump Company as follows:

200 G.P.M. Sewage Pump (Horizontal)

NC-4 4-inch suction, 4-inch discharge, 200 G.P.M. rating with 25-foot head. Motor 2 H.P. at 1150 R.P.M.

400 G.P.M. Sewage Pump (Horizontal)

NC-4 4-inch suction, 4-inch discharge, 400 G.P.M. rating with 25-foot head. Motor 3 H.P. at 1150 R.P.M.

600 G.P.M. Sewage Pump (Horizontal)

M-4 5-inch suction, 4-inch discharge, 600 G.P.M. rating with 25-foot head. Motor 5 H.P. at 1150 R.P.M.

Gate valves will be used on the suction and discharge sides of the sewage pumps and check valves will be used on the discharge side.

75 G.P.M. Sludge Pump, Single Plunger Belt-driven

[illegible]33 4-inch suction, 4-inch discharge. Capacity of 75 G.P.M. against a total dynamic head of [illegible] feet. Motor 3 H.P.

Variable capacity can be obtained by an adjustable eccentric that changes the stroke of the plunger.

An air chamber 8 inches in diameter and 50 inches long will be used on the suction side of the sludge pump, and another of the same size on the discharge side. Gate valves will be installed on the suction and discharge sides of this pump, and a check valve on the discharge side.

Pump Station Building:

Bar Rack:

Sewage will pass from manhole Ha through an opening 30 inches square that can be closed by a control-gate in case repairs are necessary on the rack. A Traverse City Iron Works Rectangular Sluice Gate will be used, with Rectangular Slide-Non-Rising Stem, A-2[illegible], Flat Spigot Frame, Size: 30 inches x 30 inches.

Clear spacing between bars will be 1½ inches. They will be inclined at an angle of 45 degrees to the horizontal. The rack channel will be 4 feet square.

Building:

The pump station building will be 30 feet wide and 40 feet long. Housed in it will be the following: Bar Rack; Wet well; Sewage Pumps; Raw Sludge Pump; Sump Pump; Chlorination Equipment; Sewage Meter; Heating Plant; Electrical controls; Laboratory; and Lavatories, etc.

Design of Concrete and Steel Construction:

Grit Chamber:

f_c = 1000 lb. per sq. in.

f_s = 18,000 lb. per sq. in.

Inner Face of Wall:

Assume maximum depth of water in the chamber as 3 ft.

$$p = w\frac{(h)^2}{2}$$

$$= 62.4 \times \frac{(3)^2}{2} = 280 \text{ lb.}$$

M = 280 x 1 = 280 ft.lb. = 3,360 in.lb.

For n = 12 in American Concrete Institute, Reinforced Concrete Design Handbook, p. 62, K = 173

$M = Kbd^2 \qquad d^2 = \frac{M}{Kb}$

$d^2 = \frac{280 \times 12}{173 \times 12} = 1.60$ d = 1.3 in. D = 4 in. (too small)

Use 10-inch wall. d = 8 in.

$$A_s = \frac{M}{jdfs} = \frac{3,360 \times 12}{.9 \times 8 \times 18,000} = .03 \text{ sq. in.}$$

From Table 3 in American Concrete Institute, op. cit., p. 66:

¼-inch circular bars at 11 inches Area = .05 sq. in.

Outer Face of Wall:

Assume earth piled to top of the walls, giving height of earth = 4.5 feet

$$E.P. = C_e \times \frac{wh^2}{2}$$

$$= .27 \times \frac{100 \times (4.5)^2}{2} = 274 \text{ lb.}$$

M = 274 x 1.5 = 411 ft.lb. = 4,930 in.lb.

$A_s = \dfrac{4,830}{.9 \times 8 \times 18,000} = .04$ sq.in.

Use of $\frac{1}{4}$-inch circular bars at 11 inches

Shear and Bond:

$v = \dfrac{280}{.9 \times 12 \times 8} = 5$ lb. per sq. in. (allowable 50 lb.)

$u = \dfrac{280}{.9 \times 8 \times .5} = 78$ lb. per sq. in. (allowable 100 lb.)

Footing and Tank Floor:

Assume shoulder is 9 inches

Depth of footing is 12 inches

Weight of concrete:

Wt.c = 2.5 x 1 x 150 + 4.5 x 1 x 150

= 375 + 675 = 1,050 lb.

Weight of water:

Wt.w = $\frac{3}{4}$ x 3 x 62.4 = 140 lb.

Point of application of vertical forces:

$\bar{y} = \dfrac{375 \times 1.25 + 675 \times 1.35 - 140 \times 2.13}{1,190} = 1.61$ ft.

x/280 = 2/1,190   x = .47 ft.

Toe distance = 1.61 - .47 = 1.14 ft.

e = 1.25 - 1.14 = .11 ft.

Soil pressure $= \dfrac{1,190}{2.5}\left(1 \pm \dfrac{6 \times .11}{2.5}\right) =$ 600 lb. per sq. in. / 350 lb. per sq. in.

Variation of Soil Pressure is 100 lb. per sq. in. per ft.

Shear at junction of wall and shoulder:

$V = \dfrac{600 + 525}{2} \times 1 - 9/12 \times 150 \times 1 = 450$ lb.

$v = \dfrac{450}{12 \times .9 \times 10} = 4$ lb. per sq. in.

Shear at junction of wall and tank floor:

V = 9/12 x 387.5 - 9/12 x 1 x 150 - 140 = 39 lb.

$v = \dfrac{39}{12 \times .9 \times 10} = .4$ lb. per sq. in.

Moment at the wall and tank floor:

$$M = 140 \times 9/24 - 150 \times 1 \times 9/12 \times 9/24 - 350 \times 9/12 \times 9/24 - 75/2 \times 3/12 \times 9/12$$

$$= 52.5 - 42.2 - 98.4 - 7.0$$

$$= 10.7 \text{ ft.lb.}$$

$d^2 = \frac{10.7 \times 12}{175 \times 12} = .0685$ $d = .25$ inches $D = 2.25$ inches

Use 12-inch D for safety; then d is 10 inches

$$A_s = \frac{10.7}{.9 \times 10 \times 18{,}000} = .00009 \text{ sq. in.}$$

It will be left up to the steel company supplying the steel to determine what is best suited for the bottom slab reinforcing.

<u>Primary Sedimentation Tanks:</u>

f_c = 1,000 lb. per sq. in.

f_s = 18,000 lb. per sq. in.

Inner Face of Wall:

Assume maximum depth of water in the tanks to be 10 ft.

$$p = \frac{w(h)^2}{2}$$

$$= 62.4 \times \frac{(10)^2}{2} = 3,120 \text{ lb.}$$

M = 3,120 x 10/3 = 10,400 ft.lb.

For n = 12 in. American Concrete Institute, *op. cit.*, p. 62, K = 173

$$M = Kbd^2 \qquad d^2 = \frac{M}{Kb}$$

$$d = \sqrt{\frac{10,400 \times 12}{173 \times 12}} = 7.8 \text{ in.; say 8 in. } D = 10 \text{ in.}$$

Use for safety D = 12 inches, d = 10 in.

$$A_s = \frac{M}{jdf_s} = \frac{10,400 \times 12}{.9 \times 8 \times 18,000} = .97 \text{ sq. in.}$$

From Table 3 in American Concrete Institute, *op. cit.*, p. 62:

Use 7/8" circular bars at 7-inches Area is 1.03 sq. in.

$$v = \frac{3,120}{.9 \times 12 \times 10} = 29 \text{ lb. per sq. in.}$$

$$u = \frac{3,120}{.9 \times 10 \times 4.7} = 74 \text{ lb. per sq. in.}$$

Use 7/8" circular bars at 7 inches in both sides of the dividing wall between the two tanks.

Outer Face of Wall:

Assume earth piled to top of the walls giving height of earth as 10 feet.

$E.P. = c_e \times \frac{wh^2}{2}$

$= .27 \times \frac{100 \times (12)^2}{2} - 1,945$ lb.

$M = 1,945 \times 12/3 = 7,720$ ft.lb.

$A_s = \frac{7,720 \times 12}{.9 \times 10 \times 18,000} = .58$ sq. in.

Use $\frac{3}{4}$-inch circular bars at 9 inches Area is .59 sq. in.

Shear and Bond:

$v = \frac{1,1945}{.9 \times 12 \times 10} = 18$ lb. per sq. in.

$u = \frac{1,1945}{.9 \times 10 \times 3.1} = 70$ lb. per sq. in.

Footing and Tank Floor:

Assume shoulder to be 12 inches

Depth of footing is 12 inches

Weight of Concrete:

Wt.c = 12 x 1x150 + 9 x 1 x 150

= 1,800 - 1,350 = 3,150 lb.

Weight of water:

Wt.w = 10 x 7 x 62.4 = 4,370 lb.

Point of application of vertical forces:

$\bar{y} = \frac{1,800 \times 1.5 - 1,350 \times 4.5 - 4,370 \times 5.5}{7,520} = 4.36$ ft.

$x/3,120 = 10/\ 3 \times 7,520 \qquad x = 1.39$ ft.

Toe distance = 4.36 - 1.39 = 2.97 ft.

e = 4.50 - 2.97 = 1.53 ft.

Soil pressure $= \frac{7,520}{9}(1 \pm \frac{6 \times 1.53}{9}) =$ 1,685 lb. per sq. in.
0 lb. per sq. in.

Variation of soil pressure is 185 lb. per ft.

Shear at junction of wall and shoulder:

$$V = \frac{1,825 - 1,400}{2} \times 1 - 1 \times 1 \times 150 = 1,483 \text{ lb.}$$

$$v = \frac{1,483}{12 \times .9 \times 10} = 13 \text{ lb. per sq. in.}$$

Shear at junction of wall and tank floor:

$$Av.P. = \frac{1,225 - (2 \times 175)}{2} = 850 \text{ lb. per ft.}$$

$$V = 850 \times 7 - 7 \times 1 \times 150 - 4,370$$

$$= 870 \text{ lb.}$$

$$v = \frac{870}{12 \times .9 \times 10} = 8 \text{ lb. per sq. in.}$$

Moment at junction of wall and tank floor:

$$M = 4,370 \times 7/2 - 150 \times (7)^2/2 - 1,225/2 \times 7 \times 7/3$$

$$= 15,300 - 2,450 - 10,300 = 7,150 \text{ ft.lb.}$$

$$d^2 = \frac{7,150 \times 12}{173 \times 12} = 41.35 \quad d = 6.5" \quad D = 8.5 \text{ inches}$$

Use D = 12 inches for safety; then d = 10 inches.

$$A_s = \frac{7,150}{.9 \times 10 \times 18,000} \doteq .044 \text{ sq. in.}$$

Use ¼-inch circular bars at 11 inches Area is .05

Design of Recommended Future Additions to Provide for Completion of Treatment by the Activated Sludge Method:

Aeration Tanks:

For this size plant mechanical aerators are best suited. A 6-hour detention period with 25% of activated sludge and average sewage flow will be used as the basis for design.

Volume needed = 6/24 x 578,500 x 1.25

= 181,000 gallons

= 24,100 cu. ft.

Use four tanks, each 12 ft. deep and 22.5 feet square. Volume of each is 6,100 cu. ft., giving a total volume of 24,400 cu. ft.

Final Settling Tanks:

Use a detention period of 2 hours, based upon 125% of the average sewage flow.

Volume needed = 2/24 x 578,500 x 1.25

= 60,300 gallons

= 8,030 cu. ft.

Use two tanks, each 10 feet deep and 22.6 ft. in diameter.

Another sludge-digestion tank will have to be built to provide for the activated sludge produced. The sludge drying-beds would also have to be enlarged. There is also the possibility that some other method of sludge-drying might be used, such as vacuum filtration with an Oliver rotary filter. Space for this expansion is provided north of the present sludge beds.

THE END

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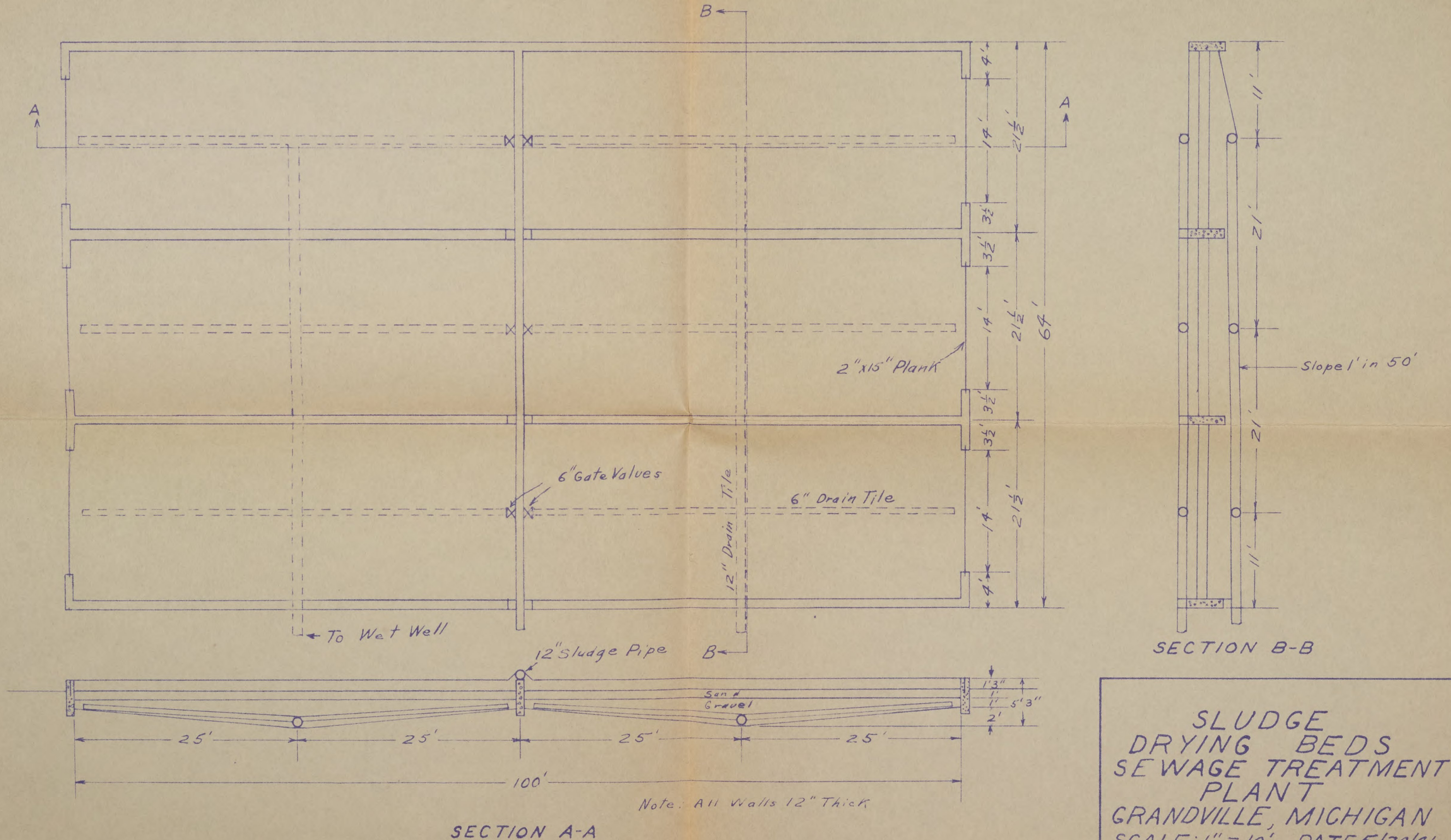
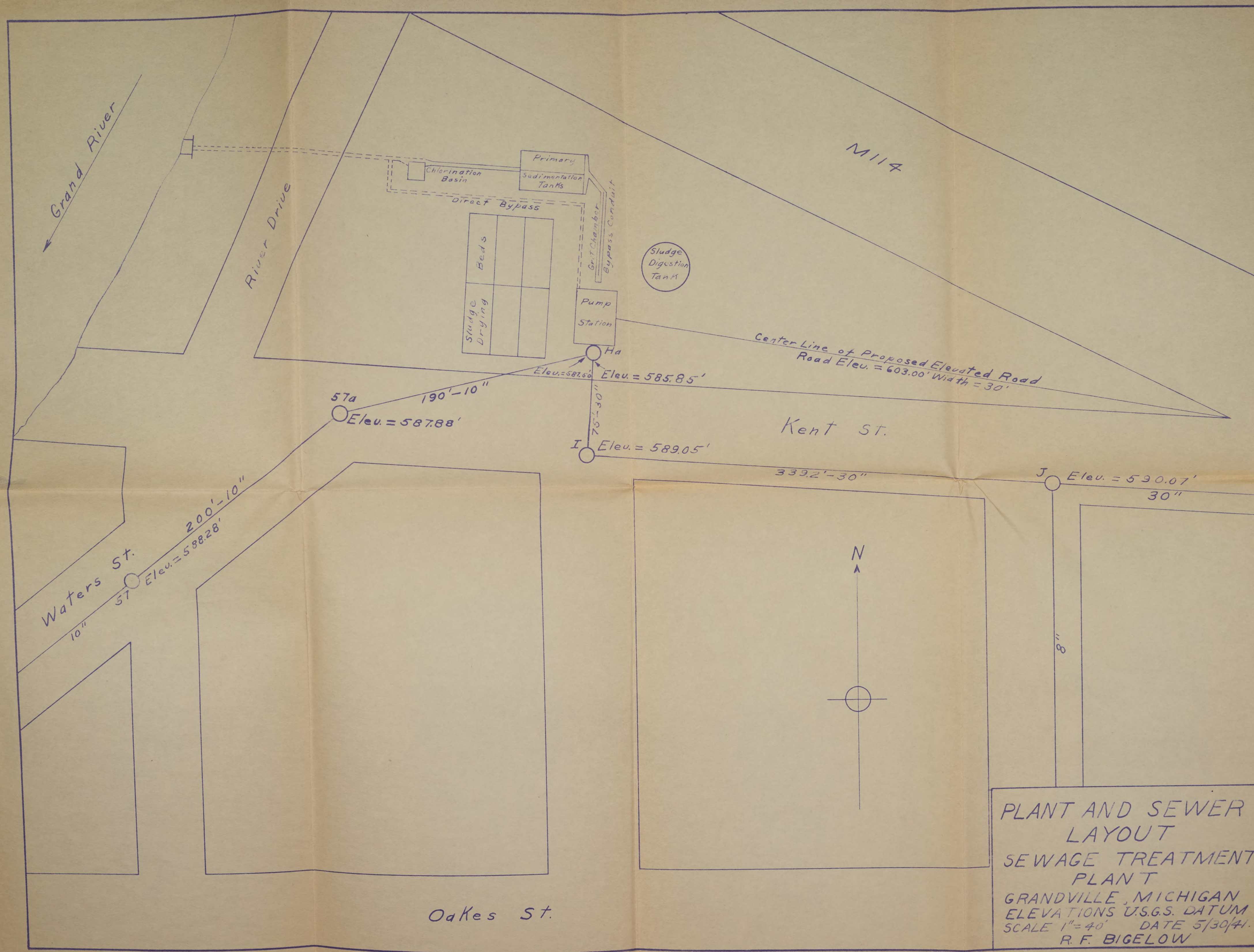
ROOM USE ONLY



SLUDGE DRYING BEDS
SEWAGE TREATMENT PLANT
GRANDVILLE, MICHIGAN

SCALE: 1" = 10' DATE 5/30/41

R.F. BIGELOW



Grand River
River Drive
M114
Chlorination Basin
Primary
Sedimentation Tanks
Direct Bypass
Grit Chamber
Bypass Conduit
Sludge Drying Beds
Sludge Digestion Tank
Pump Station
Ha
Elev. = 587.50
Elev. = 585.85'
Center Line of Proposed Elevated Road
Road Elev. = 603.00' Width = 30'
57a
190'-10"
Elev. = 587.88'
75'-30"
I
Elev. = 589.05'
Kent St.
339.2'-30"
J
Elev. = 590.07'
30"
8"
N
Waters St.
200'-10"
57
Elev. = 588.28'
10"
Oakes St.
PLANT AND SEWER
LAYOUT
SEWAGE TREATMENT
PLANT
GRANDVILLE, MICHIGAN
ELEVATIONS U.S.G.S. DATUM
SCALE 1" = 40' DATE 5/30/41.
R.F. BIGELOW