THE DESIGN OF A SEWERAGE SYSTEM FOR STERLING-MONROE STATE PARK

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE
J. S. Orton
1943

THESIS

THE DESIGN OF A SEWERAGE SYSTEM
FOR
STERLING-MONROE STATE PARK

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

OF

AGRICULTURE AND APPLIED SCIENCE

by

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Candidate for the Degree of

Bachelor of Science

June, 1943

THESIS

ACKNOWLEDGEMENT

I wish to express the deepest appreciation to the members of the Michigan State Parks Department office staff for their kind indulgence and coopera-tion, throughout this writing. They have been most helpful in offering advice, suggestions and use of their facilities.

I also tender a vote of thanks to Professor F. R. Theroux of the Civil Engineering staff of Michigan State College for his many helpful suggestions.

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Introduction

Sterling-Monroe State Park is the Conservation Department's answer to the problem of furnishing recreational areas to the inhabitants of the Detroit area. It is located on the northern outskirts of the City of Monroe along the shore of Lake Erie and is within easy reach of both Detroit and Toledo. The existing development is composed of three buildings, a few acres of land and a bathing beach. It is difficult to approach from any well travelled highway and is resultantly poorly patronized.

The proposed expansion program consists, in general, of the purchase of additional adjacent land from the Monroe Piers Land Company, the entire area to be graded and landscaped according to the features of Plate I. The excavated earth from the lagoon area, plus some necessary additional fill material, is to be used to build up the area on the lake side of the lagoons. This region is now of a swampy nature and is drained, rather extensively, by means of canals.

The park is to be highly developed because of the rather large congregation of people expected. These developments will consist of twenty buildings of various sizes and uses, a bathing beach, a camping area, picnic area and a playground area as well as the boating lagoons. The Jones Beach State Park, located on Long Island in New York, has been used as a comparative development in the design of the Sterling-Monroe State Park.

An estimated capacity crowd to make use of the facilities in one day would be nearly 75,000 persons, with at least a third of that number in the park at one time. A crowd of this size, in estimating the quantity of sewage that would require treatment, would be equivalent to a city of nearly 5,000 population,

Other important factors that have a decided bearing on the project are: (1) The largest portion of the crowd would be expected during the daylight hours, (2) The greatest crowding would occur during the week-ends and on Holidays, (3) During nine months of the year the park has few if any visitors and consequently little or no sewage to dispose of.

Methods of Disposal

Judging by the tight concentration of facilities of the proposed plan we can assume that every available inch of the park would be used during a peak load. This leaves us no recourse but to use as little area as possible in the disposal of sewage. The lack of area will naturally eliminate some readily apparent methods. Even so, several possibilities still present themselves.

The first of these would be to connect the outfall directly to the City of Monroe system. This would require a pressure main and large pumps to lift the sewage into position to flow into the Monroe's treatment plant. The cost of this would probably just about balance the cost for any other type system but, in addition, we must figure the cost to the City of Monroe for the disposal of the park wastes.

This would probably be paid on a monthly or annual basis. Further, the entire system would have to be laid at once to insure treatment for the sewage from each building as it goes into operation. This is important, since the building plan for the park is to be spread over a number of years, with money appropriated accordingly.

A second possibility would be the use of individual septic tanks for each building: the effluent to be pumped to a chlorination contact chamber and thence to the River Raisin. Thus the sewage sludge would be digested in the tank to a certain extent. Each tank would require some attention and possibly a periodic cleanout. The initial expenditure for this type system could be figured right along with the cost of each building as it was being built and is an excellent possibility. However, for a group of buildings as numerous as will be in existence here, the number of tanks that will require attention is too much for a normal sized park staff to handle. There are altogether too many things that can go wrong in a series of septic tanks, especially a group of this size.

An alternate to this plan of disposal would be the use of ordinary sand filters along with the septic tanks, for the simple aeration of the effluent. However, the condition of the soil and the nearness of the water table to the surface inviolates all the practical sides to this method. In addition, the sand filters would require nearly an acre of very valuable land and almost constant attention to operate efficiently.

Another design, and the onewhich we shall adopt as being

most feasible would be a method of primary treatment with chlorination. This would include a sedimentation tank, a method of sludge disposal and the previously mentioned chlorination chamber.

The sedimentation tank would be, preferably, of rectangular shape covered and sodded to ease the sharpness of the landscaping. A sludge removal devise of the endless chain scraper type would be installed to remove the sludge. Detention time should be at least two hours preferably three. The sludge would be trapped in a hopper at one end of the tank and pumped when a sufficient amount had accumulated, to a waiting truck and carried off to the Sludge Digestion Tank of the City of Monroe. The effluent would be chlorinated and pumped into the river Raisin at a point near its mouth. The length of pressure pipe carrying the effluent would allow enough detention to sufficiently sterilize it.

Preliminary Design

After careful consideration of the best available features of the previously discussed proposals we have decided to develop our problem around the last mentioned: The primary treatment with post chlorination. Regardless of the type of treatment we have selected our first problem is the design of the sewers to transport the park wastes to the point of disposal. This is one of the most important parts of the design and cannot be passed over lightly.

From a study of the proposed grading plan the route of the sewers themselves can be followed and traced to the southern tip of the park. Shortest routes were selected in all cases be-

cause there was no need to follow streets or stay on any particular section of property. In accordance with accepted practice manholes have been placed not farther apart than 350 feet, and over 300 feet in but one or two instances. They have also been placed at all changes of direction of the sewer lines to facilitate cleaning. They will be of brick construction with a concrete floor. A metal ladder will be provided in all manholes to ease descent for any reason whatsoever. They should also be provided with small cast iron lids heavy enough to resist pulling by any but grown men.

The sewers themselves will be of vitrified clay and should not be less than eight inches in diameter. Joints should be made of cement mortar or of some accepted bituminous material. Grades shall be not less than 0.4% at any point on the line to the sedimentation tank and should preferably be much greater than that. This is important since we will be handling raw domestic sewage almost entirely. But, because of cracks that are bound to occur and leeching of the sewers there will be some grits seep into the lines. In other words, infiltration becomes an important factor in the design. This discussion will serve as an introduction to the subject of minimum velocities in sewers. For sanitary sewers, such as we will design, the least scouring velocity that is practical to use is two feet per second. Since most sewers are designed for full flowing conditions, which is very near to the maximum flow, we do have some leeway. From hydraulic studies it has been dtermined that the maximum velocity occurs when the sewer is a little better than two thirds full. However, on a project such as this one, where minimum flows will be in evidence more than with a system which would have continuous normal flows, we must take extra precautions. These will be simply the steepening of the grades throughout the system.

Of even more importance, perhaps, is the basis for computing the volume of the flow in the sewer. We have used 300 gallons per day per fixture as a good average value. The fixtures being the ordinary plumbing fixtures, such as toilets, urinals, wash basins, and showers. The value used is one that has been used successfully by the Parks department for years in their park utility designs. For the maximum flow that would be expected to occur on the heaviest day we could jump to 300 percent of this and still find that the facilities would be overburdened. However, the days during the year that a heavy load such as this would occur could be counted on the fingers of one hand. It would not be feasible to design for such a hugh overload and then only get it but a few times during the summer months. It will not be harmful for the system to be overloaded for a few hours at infrequent intervals, especially because of the extra precautions that will be taken elsewhere in the design. Of the many empirical formulas available for the determination of the sizes of sewers we have selected Kutter's as best for our purpose, using "n" as equal to 0.015.

Force Mains

The force mains, or pressure lines, are necessary to the carrying off of wastes in this project because of the relatively flat terrain that is planned for this park. In addition the position of the water table, which subject to the rise and fall of Lake Erie, prevents the laying of sewers to any great depth. The danger of flotation of the sewers after they are laid and indeed, the difficulty even laying them, enters into the picture here.

It appears that we will need something like six pressure lines of varying lengths but probably of uniform size. These should be of cast iron and of the bell and spigot construction. No particular attention need be paid to grades since they have no bearing on the quantity or velocity of flow. The losses of head incurred while traversing from the wet well to the succeeding manhole should be carefully computed to determined the size of pump needed.

At one point, where the sewage from the west size of the lagoon must cross to the east side, the pressure main is a substitute for an inverted syphon. This was the logical step to follow because of the great distance to travel to the next man hole, and the consequent necessary fall.

The outfall from the treatment plant to the diluting body, the River Raisin, has not been designed because of the incomplete data available on the condition and extent of the area between it and the park.

Pump Units

The pumping system, necessarily, must be a complete one. They must be carefully specified both for quantity of sewage to be handled and for the amount of head to over come. Static head, entrance and exit losses, as well as friction head loss should be investigated thoroughly before the pump can be selected.

They will be of the vertical centrifugal type, with the pump submerged in the wet well directly. The driving motor will be mounted a foot or so above the inlet sewer on a solid platform. They should be electrically driven and protected from weathering and meddlers by being placed in an underground vault. They will be float controlled with an automatic alternating device to shift the load from one pump to the other, there being two pumps in each unit.

The wet well will be of the bevelled bottom type of construction with the impeller constantly submerged at the point of least width. There should be a clearance from the bottom of at least three inches. It will be of concrete construction made accessible to inspection or repairs by means of a manhole in the roof. The wet well should also be made accessible to inspection and cleanout by means of a manhole in the motor mount platform.

The sedimentation tank effluent pump unit shall be of the flat bottom construction, since it will have no sewage solids to pump. The size and capacity of these pumps will have to be arbitrarily chosen because of the lack of information of the necessary head required to pump to the River Raisin. Included in this unit will be the chlorination dosing apparatus.

The sludge pumps will be two in number and shall be manually controlled according to whenever the hoppers will have to be emptied. They will also be of the vertical centrifugal type and shall be mounted underground to remove unsightliness and for protection. They shall be selected according to the frequency of use and the head to overcome.

Sedimentation tank

The sedimentation tank is the basic tool of primary treatment of ment of sewage. We shallchoose the rectangular continuous flow type because of its simplicity of operation. The tank will actually be two tanks operating as a complete unit by themselves if for any reason one of them should break down. The distribution box shall be designed so that either one of the tanks may be put out of operation at any time. It shall be sloped slightly so as to insure uniform flow into the outlet holes at all times. The effluent shall be caused to flow over a wier into an effluent channel sloped toward the effluent pumping unit and the chlorination dosing chamber.

Sludge will be removed by means of a mechanical scraper mounted on an endless chain conveyor. It will be scraped into a sludge hopper placed at the upper end of the tank from which it will be pumped into waiting trucks for disposal elsewhere. A suggestion from Professor F. R. Therous of the Michigan State College Civil Engineering Staff helped me arrive at this solution. He further suggested that an arrangement be made with the City of Monroe to dispose of it in their digestion tank.

Scum, which collects on the surface at the lower end of the tank, due to its being pushed there by the sludge scraper on its return journey, shall be removed by means of another and smaller endless chain conveyor. The scum trough will span the width of the tank and should be sloped up toward the surface on its downstream side. It will be caused to flow back into the wet well preceding the sedimentation tank and allowed to be retreated.

The tank itself will be of reinforced concrete construction with a bottom sloped toward the sludge hopper. It will be covered with a concrete roof and a small amount of earth on which to plant sod to cover as much of its unsightliness as possible. The effluent pump unit will have a small structure atop it in which to house the chlorination apparatus and the pumps. It will also have room for the storage of any maintenance equipment needed to operate the plant.

DESIGN OF SEWERS

Line No.	From Manhole Bldg.	To Man hole	Length Ft.	Infiltration Increment	Infiltration Total	Max Flow GPD	Max Flow CFS
1	B-17	1	280	1060	1060	12460	.02
3	B-16	2	160	600	600	23800	.0366
4	2	3	300	1140	2800	37400	.0575
5	3	4	300	1140	3940	38540	.0584
6	4	5	300	1140	5080	43620	.0610
7	B-15	6	90	1340	340	15340	.0236
8	6	9	190	725	1065	16065	.0247
9	B-14	7	90	340	340	15340	.0236
10	7	8	225	855	1195	16195	.0299
11	8	9	240	910	2105	17105	.0264
12	9	10	300	1140	4310	34310	.0528
13	B-13	10	120	455	455	11855	.0183
14	10	5	90	340	5105	46505	.0762
16	B-12	11	80	300	300	44700	.0687
17	11	12	325	1240	1540	45940	.0706
18	B-12	12	80	300	300	44700	.0687
19	12	13	125	475	2315	91115	.140
20	13	14	300	1140	8560	173360	.267
21	14	15	300	1140	9700	174500	.268
22	15	16	300	1140	10840	175640	.270
24	B-9	17	80	300	300	15300	.0236
25	17	20	240	910	1210	16210	.0250
26	B-10	18	80	300	300	15300	.0236
27	18	19	200	760	1060	16060	.0247
28	19	20	200	760	1820	16820	.0259

29	20	21	300	1140	4170	34170	.0525
30	B-11	21	60	228	228	11628	.0178
31	21	16	90	340	4738	46138	.0710
35	B-18	25	250	950	950	6950	.010
36	25	26	350	1330	2280	17880	.027
37	B-19	26	200	760	760	3760	.006
39	B-8	27	210	800	800	23600	.0364
40	27	28	320	1220	21770	279370	.430
41	28	29	310	1180	22950	280550	.432
42	29	30	300	1140	24090	281690	.434
43	B-7	30	90	342	342	11742	.018
45	B-5	31	220	840	840	24240	.037
46	31	32	270	1030	26302	318702	.490
47	32	33	270	1030	27332	319732	.491
48	B-6	33	90	342	342	11742	.0181
49	33	34	290	1100	28774	332574	.513
50	B-1	34	340	1440	1440	16440	.025
51	34	36	200	760	30974	349774	.540
52	B-4	35	190	724	724	13924	.021
53	B-3	35	30	114	114	11514	.0176
54	35	36	310	1180	2018	26618	.041
55	B-2	36	30	114	114	11514	.176
56	36	37	200	760	33866	388666	.600

Line No.	Ground Elevations Upper	Ground Elevations Lower	Dia. of Pipe in.	Grade Ft.	Fall Ft.	Invert Elevations Upper	Invert Elevations Lower
1	578.00	576.20	8	0.010	2.80	575.00	572.20
3	580.00	578.00	8	0.010	1.60	577.00	575.40
4	578.00	578.00	8	0.008	2.40	575.40	573.00
5	578.00	578.00	8	0.008	2.40	573.00	570.60
6	578.00	578.00	8	0.008	2.40	570.60	568.40
7	578.20	577.50	8	0.010	0.90	575.00	574.10
8	577.50	577.40	8	0.010	1.90	574.10	572.20
9	577.50	577.50	8	0.008	0.72	575.00	574.28
10	577.50	577.50	8	0.008	1.80	574.28	572.48
11	577.50	577.40	8	0.008	1.92	572.48	570.56
12	577.40	578.00	8	0.008	2.40	570.56	568.16
13	578.30	578.00	8	0.010	1.20	575.00	573.80
14	578.00	578.00	8	0.010	0.90	568.16	567.26
16	580.00	579.00	8	0.008	0.64	577.00	576.36
17	579.00	579.00	8	0.008	2.60	576.36	573.76
18	580.00	579.00	8	0.032	2.57	576.33	573.76
19	579.00	578.80	8	0.008	1.00	573.76	572.76
20	578.80	577.80	8	0.006	1.80	572.76	570.96
21	577.80	577.00	8	0.006	1.80	570.96	569.16
22	577.50	577.30	8	0.006	1.80	569.16	567.36
24	578.00	577.80	8	0.010	0.80	575.00	574.20
25	577.80	577.40	8	0.010	2.40	574.20	511.80
26	578.00	577.80	8	0.010	0.80	575.00	574.20
27	577.80	577.60	8	0.010	2.00	574.20	572.20
28	577.60	577.20	8	0.010	2.00	572.20	570.20

29	577.20	578.00	8	0.008	2.40	570.20	567.80
30	578.60	578.00	8	0.010	0.60	575.00	574.40
31	578.00	577.30	8	0.008	0.72	567.80	567.08
35	578.00	579.00	8	0.010	2.50	575.00	572.50
36	579.00	575.00	8	0.010	3.50	572.50	569.00
37	578.50	575.00	8	0.010	2.00	575.00	573.00
39	579.00	579.00	8	0.012	2.50	576.00	573.50
40	579.00	577.80	8	0.006	1.92	574.00	572.08
41	577.80	577.20	8	0.006	1.86	572.08	570.22
42	577.20	577.20	8	0.006	1.80	570.22	568.42
43	575.80	577.20	8	0.010	0.90	575.00	574.10
45	580.00	577.50	8	0.010	2.20	576.50	574.30
46	577.50	577.50	8	0.006	1.42	574.30	572.88
47	577.50	577.10	8	0.006	1.42	572.80	571.46
48	577.00	577.10	8	0.017	1.50	573.00	571.50
49	577.10	577.00	8	0.006	1.74	571.46	569.72
50	578.00	577.00	8	0.010	3.40	573.40	570.00
51	577.00	576.90	8	0.006	1.20	569.72	568.52
52	578.00	577.30	8	0.01	1.90	575.50	574.60
53	577.30	577.30	8	0.01	0.30	575.00	574.70
54	577.30	577.30	8	0.019	5.90	574.60	568.70
55	577.20	577.30	8	0.10	3.00	572.00	569.00
56	576.90	577.00	8	0.006	1.20	568.52	567.32

DESIGN OF PUMP STATIONS AND FORCE MAINS

Pump No. 1 - Manhole 1

Maximum Influent 12460 GPD

9 GPM

Assume 30 minute accumulation

9 x 30 - 270 gal.

- 37 cu.ft.

Assume 2 - 50 GPM Pumps of Vert. Control

Centrifugal type with float

Wet Well Design

Assume 4.5 x 4.5 square concrete tank

with bottom sloped toward the center

at 100%

Depth of tank -

Ground Elevation	-	576.20
Sewer Invert Elevation	-	572.20
Difference		4.00
Depth from sewer invert to start of sloped bottom	-	2.00
Depth of hopper bottom	-	1.75
Total		7.75
Depth between trip head and shut off head		2.00

Design of Force Main

Length of Line - 980 ft. of 4" pipe

Head Losses

Static Head	5.50
Friction Loss	3.50

Velocity	-	1.30 ft./SEC	
Entry Loss	-	$\frac{V^2}{2g}$ -	0.03
Exit Loss	-	$\frac{V^2}{2g}$ -	0.03
		Total Head Loss	9.06

Adopt 2 - 50 GPM 1 HP Pump

place one check valve and one gate valve on pressure side of each pump.

Pump No. 5 - Manhole 26

Maximum Influent	-	21640 GPD
	-	15 GPM
Assume 30 minutes accumulation	-	450 gal.
	-	60 cu.ft.

Try 2 50 GPM Vert. Centrifugal pumps with float control

Wet Well Design

Assume 4 x 5 tank with Hopper Bottom sloped at 100%

Depth of Tank -	Ground Elevation	575.00
	Sewer Invert Elevation	569.00
	DIFFERENCE	6.00
Depth of Sewer Invert to Start of	Sloping bottom	3.00
Depth of Hopper Bottom		2.00
	Total	11.00
Depth between trip head and shut off head		3.00

Design of Force Main

Length of Line 1700 ft. of 4" C.I. Pipe

Head Losses -	Static Loss	9.00
	Friction Loss	6.00
Velocity - 1.30 ft. per sec.		
Entry Loss -		0.03
Exit Loss -		0.03
	Total	15.06

Adopt 2 50 GPM 1 H P Pump, place one gate valve and one check valve on pressure side of each pump.

Sedimentation Tank Design

Maximum Daily Flow - 400000 GPD

Maximum Hourly Flow 2400 Cu.ft.

Design for two tanks assuming flow divided equally, and tanks identical in shape and size.

Assume 3 hour detention

then needed volume - 1200 x 3 - 3600 cu.ft.

Assume average depth - 6 ft.

Then area of tank - $\frac{3600}{6}$ - 600 sq.ft.

Assume each tank 12 ft. wide

Then length of tank - 50 ft.

L/W Ratio - 4 ≠ : 1

Velocity of Flow - .103 in. / sec.

Inlet Details

Flow for one tank - 0.66 C.F.S.

Assume channel 1.5 ft. wide by 1 ft. deep

$V - C \sqrt{RS}$ Chezy Formula

$C - 100$ Assumed

$R - \frac{A}{WP} - \frac{1.5}{3.5} - .43$

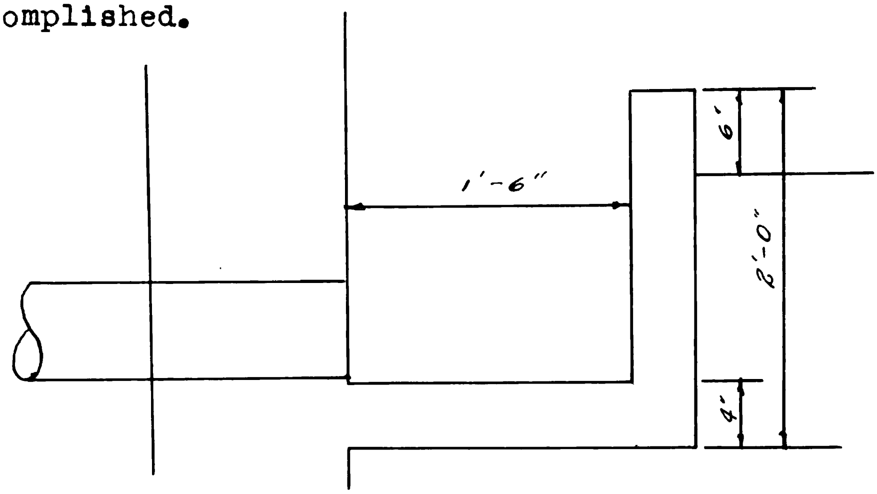
$S - \frac{0.08}{6} - 0.013$

$V - 100 \sqrt{.43 \times 0.13}$

$- 6.8$ ft/ sec.

Channel to be constructed of concrete 4" thick spanning both tanks completely. It will be inside the tank and be fed from a six inch pipe at the center of the east wall of the tank. There shall be a six inch projection of the channel wall above the surface of the liquid to prevent its backing up into tank.

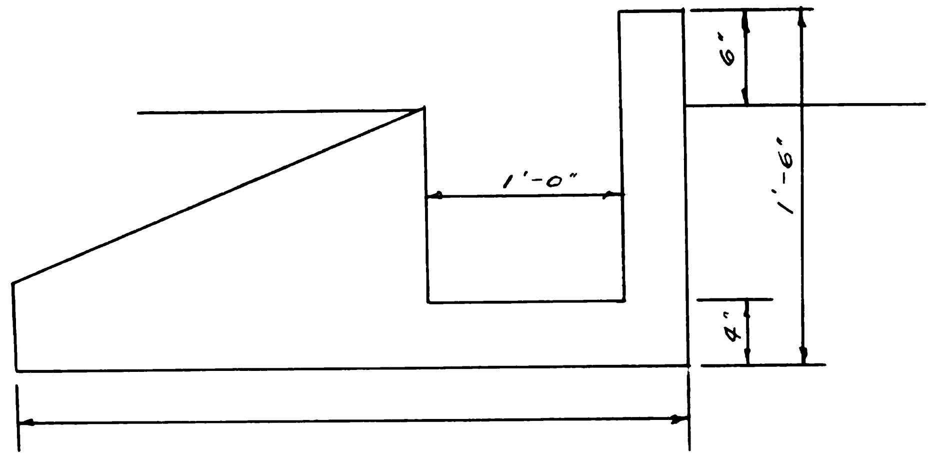
The outlet from the channel will be from the bottom and will consist of six, 6-inch openings spaced three inches on centers and naturally in a submerged position. By directing the influent downward at a low velocity the forward velocity is broken up and sedimentation is more readily accomplished.



Scum Trough Details

The scum trough will consist of a concrete channel with a wier on its back side to prevent the scum from entering the effluent channel. The downstream face shall be slightly above the elevation of the water surface. It will be sloped at the rate of 30% for a distance of 1'9". We will make it a foot deep and sloped toward the outer edge of the tank. The removal devise will consist of a small endless chain scraper that will push the scum along with small amounts of liquid into the trough. The trough will empty into a steeply graded 6" sewer pipe which leads back to the wet well along the outside of the sedimentation tank.

The trough will be of concrete four inches thick and completely spanning each individual tank.



Effluent Channel Design

We will assume dimensions of a one foot square channel with the lip placed slightly below the sewage surface to serve as a wier. The bottom of the channel will be sloped toward the inside wall where it will enter the effluent wet well by means of an eight inch pipe. The fall for the entire distance is to be three inches.

Hydraulics -

Flow of tank - 0.66 c.f.s.

Channel dimensions - 1' x 1'

Assuming channel flowing half full

$R - \frac{.5}{2} - \frac{1}{4} - 0.25$

$V - C\sqrt{RS}$ Chezy Formula

$C - 100$

$S - \frac{0.08}{6} - 0.013$

$V - 100\sqrt{(0.25)(.013)}$

$V - 5.7$ ft./ sec.

Design of Reinforcing Steel for Sedimentation Tank

Design of tank roof:

Assume: Roof 9 inches thick

Fill on top 2 ft. thick

Section of slab one foot wide

Length of clear span	13 ft.
Weight of slab	115 lb./ft. of width
Weight of earth	200 lb./ft. of width

Maximum bending movement - $WL^2/12$

$$M - \frac{(315)\ (13)^2}{12}$$

$$M - 4436 \text{ ft. lbs.}$$

Assume K - 104

$$d^2 - \frac{M}{K6} - \frac{4436 \times 12}{104 \times 12}$$

$$d - 6.5 \text{ say } 7''$$

$$D - 7 + 3 - 10''$$

Weight of slab 125 lb./ ft.

$$M - \frac{(325)(13)^2}{12}$$

$$- 4577$$

$$d^2 - \frac{4577 \times 12}{104 \times 12}$$

$$d - 6.7 \text{ say } 7$$

$$D - 7 + 3 - 10 \quad \text{O.K.}$$

Test for Shear

$$V - \frac{WL}{2}$$

$$- \frac{315 \times 13}{2}$$

$$- 2047 \text{ lbs.}$$

$$v - \frac{V}{bjd}$$

Assume j - 7/8

$$v - \frac{2047 \times 8}{12 \times 7 \times 7} - 28 \text{ lb.sq.in.}$$

allowable - 50 lb.sq.in.

Steel ratio - p - .0052

A_s - .0052 x 12 x 7 - 0.47 sq.in.

Try 5/8" round bars @ $8\frac{1}{2}$ C - C

$$\text{Test Bond} - U - \frac{V}{\text{sum } o\,jd} - \frac{2047 \times 8 \times 8.5}{2 \times 12 \times 7 \times 7}$$

- 118 allowable 120

Use deformed bars.

Temp. Steel - Area - .3% of area of concrete

- .003 x 12 x 7

- .252 sq. in.

Use 3/8 round bars at 9" C-C on top

Use 1/4" round bars at 9" C-C on bottom

Wall - Condition No fill tank full

Wall 7.5 ft. high

Assume 12" thick

Water pressure exerted as a single force at 2.5 ft. up from bottom

$$P - \frac{wh^2}{2} - \frac{(62.4)(49)}{2} - 1529 \text{ lb.}$$

M - 1529 x 2.5

- 3822 lbs.

K - 104

$$d^2 - \frac{3822 \times 12}{104 \times 12}$$

d - 6 + say 6

D - 6 + 3 - 9

Test Shear $V - 1529$

$$v - \frac{1529 \times 8}{12 \times 7 \times 6} - 24.3 \text{ allowable } 50$$

$$a_s - .0052 \times 12 \times 7 - .47 \text{ sq. in.}$$

Try 5/8" round bars @ 8 ½ C-C

Test Bond $$u - \frac{1530 \times 8 \times 8.5}{1.96 \times 12 \times 7 \times 7} - 101 \text{ allowable } 120$$

Use deformed bars -

Place same amount of steel on inside as on outside

Retain 12 inch wall as assumed.

Temperature Steel

$$A_s - .003 \times 12 \times 7$$

$$- .252 \text{ sq. in.}$$

Use 3/8" round bars at 9" C-C on both sides

$A - .30$ sq. in.

Bottom Slab	Weight of Walls	- (1x 7.5 x 1) 150 x 3 -	3375
	Weight of roof	- 125 x 27	3375
	Weight of fill	- 200 x 27	5400
		Total	121500

Length of clear span - 13

Weight per foot 12150/31 - 392 s2. ft.

Weight of 10" bottom slab - 125 lb/ ft.

Total weight per foot - 267 lb./ ft.

$$M - \frac{wl^2}{2} - \frac{267 \times 169}{12} - 5253 \text{ ft. lbs.}$$

$$d^2 - \frac{3760 \times 12}{12 \times 104} - 36.5$$

$$d - 6 + \text{ say } 7$$

$$D - 7 + 3 - 10$$

$$A_s - .0052 \times 12 \times 7 - .47 \text{ s2. in.}$$

Use 5/8 inch round bars @ 8½ c-c

Temp Steel A_t .003 x 12 x 7 - .253 place half on top and half on bottom so place 3/8 round bars @ 9" c-c

Sludge Hopper Design

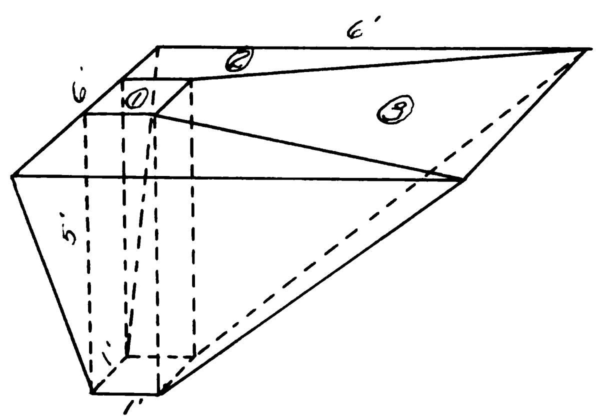
Assume 2500 Gal. Sludge / 1,000,000 gals. sewage

- 1000 gal. / day
- 134 cu.ft. / day

Assume tank to be emptied every two days

- 268 cu.ft.
- size of tank

Try 4 hoppers, 2 in each tank with sides sloped toward a one foot square bottom. They will be at the influent end of the tank where the scraper ends its upstream trip. Try a six foot square tank five feet deep.



Segment 1	Volume	-	(5)(1)(1)	-	5 cu.ft.
Segment 2	Volume	-	(2.5)(6/2)(5)	-	37.5 cu.ft.
Segment 3	Volume	-	$\frac{(17.5)}{2}$ 5	-	43.75
	Area	-	(7/2)(5)	-	17.5

Total volume - 66.25 cu.ft.

For 4 hoppers - 265.00 cu.ft.

Sludge Pumps: Two needed

Try 50 GPM sludge pumps

Volume to be pumped 993.75 gal.

Time for Pumping 993.25 / 50 - 2o minutes

Head Losses - Static Head - 22.33

Miscellaneous losses - 1.00

Total Head 23.33

Adopt 2 50 GPM pumps with a 4" suction

Use 2 H P Motor

Effluent Pump Unit -

Maximum flow - 400,000 GPD

- 278 GPM

Assume 10 minute accumulation - 2780 gal.

- 371 cu.ft.

Try 2 200 GPM Vertical Centrifugal pumps with 6 inch suction and float control.

Wet Well -

Assume 9 x 9' sq. concrete tank

Depth of Tank

Depth to surface of Sewage - 3.5

Depth of Sewage - 4.5

Depth of Pump coverage - 1.5

Total depth - 9.5

Tank is to have square bottom

Head Losses - Static Head 9.5

Friction losses unknown

Assume total head needed 80

Adopt 2 200 GPM Pumps with 10 H P Motor - Intall chlorinator in pump house on top of effluent wet well so that it injects automatically as the pumps begin to discharge.

Cost Estimate Concrete @ 6.50 / cu.yd

Item	Length ft.	Width ft.	Depth ft.	Volume cu.yds.	Cost
Pump House #1	4.5	4.5	7.75	7.2	$52.00
Pump House #2	4.0	5.0	17.50	15.2	98.80
Pump House #3	7.0	7.0	16.22	25.0	162.50
Pump House #4	4.5	4.5	5.60	6.5	42.25
Pump House #5	4.0	5.0	11.00	11.6	75.40
Pump House #6	7.0	7.0	16.28	25.0	162.50
Pump House #7	9.0	9.0	16.18	29.3	190.50
Slude Pump House	25.0	8.0	14.00	39.6	257.40
Sedimentation Tank	25.0	50.0	6.00	154.0	1000.00
Effluent Pump House	9.0	9.0	18.00	31.3	221.00
		Total		344.7	2262.35

Steel @ $2\frac{1}{2}$ lb./sq.ft. concrete and @ $0.04 / lb.

Area of Sedimentation Tank	-	4473 sq.ft.
Weight of Steel	-	11182.5 lbs.
Cost of Steel	-	$447.31

Forms @ .12 / sq.ft.

	Form Area	Cost
Pump House #1	280	$32.40
Pump House #2	630	75.60
Pump House #3	908	109.00
Pump House #4	200	24.00
Pump House #5	396	47.50
Pump House #6	930	111.40
Pump House #7	1165	140.00
Sludge Pump House	1498	180.00
Sedimentation Tank	2950	354.00

Effluent Pump Room	1286	$154.60
Total	10243 sq.ft.	$1228.50

Cost Iron Pipe @ $0.51 / ft.

Length of Pipe - 7468.50 ft.

Cost - $3810.00

Cost Clay Pipe @ $.20 / ft.

Length of Pipe - 9645 ft.

Cost - $1939.00

Manholes @ 14 bricks / sq. ft. @ $20.00/1000

Manhole	Depth	Diam.	Area	No. Bricks	Cost
2	3	3	27.26	382	$ 7.64
3	5	4	62.80	879	17.60
4	8	6	151.72	2120	42.40
6	33	3	27.26	382	7.64
9	7	6	131.88	1842	36.90
7	3	3	27.26	382	7.64
8	5	4	62.80	879	17.60
10	10	6	188.40	2638	52.20
11	3	3	27.26	382	7.64
12	5	4	62.80	879	17.60
13	6	6	113.10	1583	31.66
14	7	6	131.88	1842	36.90
15	8	6	151.72	2120	42.40
17	3	3	27.26	382	7.64
18	3	3	27.26	382	7.64
19	5	4	62.80	879	17.60
20	7	6	131.88	1842	36.90
21	10	6	188.40	2638	52.20

25	7	6	131.88	1842	36.90
26	6	6	113.10	1583	31.66
28	6	6	113.10	1583	31.66
28	6	6	113.10	1583	31.66
29	7	6	131.88	1842	36.90
31	3	3	27.26	382	7.64
32	5	4	62.80	879	17.60
33	6	6	113.10	1583	41.66
34	8	6	151.72	2120	42.40
35	3	3	27.26	382	7.64
36	9	6	169.82	2378	47.50
	Total Cost of Brick				$ 771.02

Manhole Lids and Frames -

47 @ 200#/each @ $0.10 /# - $ 940

Earth Work -

Machine Excavated and Backfilled @ .30 cu.yd.

Sewer Trench - Length	-	17113 ft.
Average Depth	-	5 ft.
Maximum width	-	1.5 ft.
Average area of a section of trench	-	7.5 sq.ft.
Volume	-	128347 cu.ft.
	-	4753 cu.yd.
Cost @ .30 cu.yd.	-	$1425.90

Manhole Excavation -

Average Diameter	-	5 ft.
Average depth	-	6 ft.
Number of manholes	-	29
Volume of average hole	-	117.75 cu.ft.

Plus 20% for construction	-	24.25
	-	142.00 cu.ft.
Average volume of all manholes	-	4118 cu.ft.
	-	152 cu.yd.
Cost @ .65 cu. yd.	-	$98.80

Pump Well Excavation -

Average Depth	-	12.26
Average Area	-	37.07
Number of Wells	-	7
Volume of wet wells	-	3181 cu.ft.
Plus 20% for construction	-	636 cu.ft.
Total	-	3817 cu.ft.
	-	141 cu.yds.
Cost @ .65 cu. yd.	-	$91.65

Sedimentation Tank -

Volume	-	13725 cu.ft.
	-	508 cu.yds.
Cost @ .20 cu.yd.	-	$101.60

Equipment Estimate

4	- 50	GPM	Pumps	@	$200	-	$800
2	- 75	GPM	Pumps	@	250	-	500
2	- 100	GPM	Pumps	@	300	-	600
2	- 150	GPM	Pumps	@	350	-	700
6	- 200	GPM	Pumps	@	400	-	24000
2	- 50	GPM	Sludge Pumps	@	750	-	1500
1			Sludge Remover			-	1400
1			Scum Remover			-	200
1			Chlorinator			-	1200

14	Gate Valves	@	$40	$560
14	Check Valves	@	25	350
	Total Cost			$10,210

[redacted]

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Pocket has!
Plate no.
1,2,3,3a,
5,6,7,8,9

Plate no. 2