DESIGN DEVELOPMENT PHASE

DESIGN DEVELOPMENT REPORT

WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX

STATE OF TENNESSEE
DEPARTMENT OF GENERAL SERVICES
ON BEHALF OF
DEPARTMENT OF CORRECTION

PIKEVILLE, BLEDSOE COUNTY, TENNESSEE

SBC PROJECT NO. 142/013-01-2013-06

JAMES NEAL HALL
No. 15895

James Neal Hall
03/30/18

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CTI N16003

CTI Engineers, Inc.
3354 Perimeter Hill Drive, Suite 140
Nashville, TN 37211
(615) 834-8300

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BLEDSOE COUNTY CORRECTIONAL COMPLEX

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BACKGROUND, PURPOSE, AND SCOPE

The Tennessee Department of Correction constructed a new penal facility adjacent to the existing Southeast Regional Correctional Facility and re-named the entire development the Bledsoe County Correctional Complex (BCCX). BCCX includes the newer, more recently constructed facility (BCCX-1), the original facilities (BCCX-2), and the former Taft Youth Center (BCCX-3), which is currently vacant. The wastewater treatment plant (WWTP) which served the original facility was removed from service and demolished. A new WWTP was designed and constructed to serve all of the expanded prison facilities. The Department of Correction plans to construct additions to the current prison facilities and also to activate BCCX-3 (Taft) as a populated facility.

The purpose and scope of this document is to examine the operation of the new WWTP under its current flows and to develop the design conditions for expansion and continued operation after the planned additions are made. This Design Development Phase Report will also be used for review of the design concept and calculations by the Tennessee Department of Environment and Conservation (TDEC), Division of Water Resources. Detailed design information has been developed containing all pertinent design calculations and providing the Opinion of Probable Construction Cost based on this preliminary design information. Further refinement and revisions may be necessary as construction documents are generated following the completion of the Design Development Phase.

CURRENT AND FUTURE FLOW INFORMATION

The Tennessee Department of Correction (TDOC) has made plans to expand BCCX with a 512 inmate addition, including an intake facility, and it also plans to populate and place the currently unoccupied BCCX-3 (Taft) back into operation. In an effort to quantify the current and future flows, the following Table 1, Current Wastewater Usage and Flow Projections, has been developed. Population information was provided and confirmed by STREAM (State of Tennessee Real Estate Asset Management) and TDOC. Staff and visitors for the expansion and for Taft were kept proportional to those reported for the existing facility. The wastewater usage amounts were taken from TDEC Design Criteria, Appendix 2-4.2, where available. Filter Reject flow was reported by the operators and it was confirmed by the Parkson training information. Belt filter press underflow and WWTP sewage flows were estimated. Digester decant flow assumed the reported design conditions of 6 feet of decant over a period of 15 – 21 days. Future WWTP flows were estimated to be the same as for the existing WWTP.

Table 1
BCCX WWTP
Current Wastewater Usage and Flow Projections

Users	**Number**	**Low Usage (gpd)**	**Low Flow (gpd)**	**Middle Usage (gpd)**	**Middle Flow (gpd)**	**High Usage (gpd)**	**High Flow (gpd)**
CURRENT CONDITIONS							
Inmates	2,547	80	203,760	120	305,640	150	382,050
State Staff	740	5	3,700	10	7,400	15	11,100
Contract Staff	161	5	805	10	1,610	15	2,415
Visitors (Max)	400	3	1,200	5	2,000	7	2,800
WWTP							
Filter Reject (12 gpm x 3)			51,840		51,840		51,840
Belt Press Underflow (10%, 12%, 14% of 45 gpm)			6,480		7,776		9,072
Sanitary, Lab, Washdown (Estimated)			1,000		1,500		2,000
Digester Decant			2,860		3,335		4,000
	Subtotals		**271,645**		**381,101**		**465,277**
PLANNED ADDITIONS							
Inmates	512	80	40,960	120	61,440	150	76,800
State Staff (30%)	154	5	770	10	1,540	15	2,310
Contract Staff (6%)	31	5	155	10	310	15	465
Visitors (16%)	82	3	246	5	410	7	574
BCCX-3 (TAFT)							
Inmates	346	80	27,680	120	41,520	150	51,900
State Staff (30%)	104	5	520	10	1,040	15	1,560
Visitors (16%)	56	3	168	5	280	7	392
	Subtotals		**70,499**		**106,540**		**134,001**
FUTURE WWTP							
Filter Reject, Belt Press Underflow, Sanitary, Lab, Washdown, and Digester Decant			62,180		64,451		66,912
	TOTALS		**404,324**		**552,092**		**666,190**

WWTP records indicate an average influent flow of 355,000 gallons per day (gpd) over the period of operation with an average maximum day flow of 515,000 gpd. Projecting flows for wastewater is far from an exact science; however, the recorded flows appear to correlate with the middle and high range flow projections in Table 1. This apparent correlation lends weight to the projected flows for the planned additions.

The flow projections do not include any allowance for infiltration and inflow (I/I). It is reported that I/I is present at BCCX although the quantities have not yet been determined. As BCCX attempts to identify and eliminate the sources of excessive I/I, the reality is that some level of I/I will always be present following major rainfall events. These extraneous flows require the same level of treatment as the remainder of the wastewater; however, they do not impact the biological loading of the WWTP, only the hydraulic loading.

Influent flow to the WWTP consists of three (3) components:
1) Wastewater,
2) Infiltration/Inflow, and
3) WWTP Return Flow.

The WWTP return flow is comprised of filter reject, belt press underflow, digester decant, and WWTP wastewater (sanitary, laboratory, washdown, etc.). The WWTP return flow is the difference between the influent flow and the effluent flow. Over the time the plant has been in operation, the average WWTP return flow is 0.067 mgd (67,000 gpd), which correlates with the amount estimated in Table 1 and represents approximately 19% of influent flow. Similarly, the maximum day return flow has averaged approximately 0.096 mgd (96,000 gpd) or approximately 19% of influent flow.

Water purchase records for BCCX were obtained from the City of Pikeville for the period from April 2013 through August 2017 to attempt to identify the volume of infiltration/inflow (I/I). The usage in both 2013 and 2016 were excluded since the BCCX Water Treatment Plant (WTP) also supplied water to the prison during portions of both those years. During 2014, 2015, and the first eight (8) months of 2017, WWTP effluent flows were higher than the amounts of water purchased. This has been the case over the history of the facility except for four (4) months. The comparison between water purchased and reported WWTP flows is shown in Table 2.

In order to use this information to attempt to calculate (I/I), at least two (2) assumptions, which might or might not be valid, must be made:

BLEDSOE COUNTY CORRECTIONAL COMPLEX
WASTEWATER TREATMENT PLANT
SBC 142/013-01-2013-06 CTI N16003

TABLE 2
WATER PURCHASED AND REPORTED WWTP FLOWS

Month/Year	Total Water Purchased (Gallons)	Average Water Purchased (MGD)	WWTP Total Influent	WWTP Average Influent (MGD)	WWTP Total Effluent	WWTP Average Effluent (MGD)	
04/13	1,573,000	0.0524	11,331,000	0.3777	8,010,000	0.2670	
05/13	2,515,000	0.0811	10,483,000	0.3382	8,742,000	0.2820	
06/13	3,794,000	0.1265	10,109,000	0.3370	8,079,000	0.2693	
07/13	3,202,000	0.1033	5,314,000	0.3126	7,057,000	0.2276	Influent data missing
08/13	3,718,000	0.1199			9,112,000	0.2939	Influent data missing
09/13	3,093,000	0.1031			6,608,000	0.2203	Influent data missing
10/13	3,844,000	0.1240			4,157,000	0.1341	Influent data missing
11/13	4,057,000	0.1352			4,463,000	0.1488	Influent data missing
12/13	5,324,000	0.1717	7,852,000	0.3926	8,582,000	0.2770	
2013	31,120,000	0.1130			64,810,000	0.2356	WTP in use 2013
01/14	5,255,000	0.1695	10,426,000	0.3363	9,674,000	0.3121	
02/14	5,930,000	0.2118	10,101,000	0.3608	8,614,000	0.3076	
03/14	5,073,000	0.1636	10,971,000	0.3539	9,351,000	0.3016	
04/14	5,313,000	0.1771	10,763,000	0.3588	9,424,000	0.3141	
05/14	6,702,000	0.2162	11,913,000	0.3843	10,047,000	0.3241	
06/14	12,497,000	0.4166	10,386,000	0.3462	9,864,000	0.3288	
07/14	9,708,000	0.3132	11,075,000	0.3573	10,386,000	0.3350	
08/14	8,865,000	0.2860	11,165,000	0.3602	10,712,000	0.3455	
09/14	8,381,000	0.2794	12,480,000	0.4160	8,619,000	0.2873	
10/14	8,340,000	0.2690	13,448,000	0.4338	8,452,000	0.2726	
11/14	7,986,000	0.2662	9,663,000	0.3221	7,290,000	0.2430	
12/14	8,313,000	0.2682	9,467,000	0.3054	8,018,000	0.2586	
2014	92,363,000	0.2531	131,858,000	0.3612	110,451,000	0.3025	
01/15	8,193,000	0.2643	11,212,000	0.3617	7,898,000	0.2548	
02/15	7,782,000	0.2779	9,982,000	0.3565	7,305,000	0.2609	
03/15	8,220,000	0.2652	11,091,000	0.3578	8,260,000	0.2665	
04/15	7,970,000	0.2656	10,650,000	0.3550	10,106,000	0.3369	
05/15	8,405,000	0.2711		0.3326	8,698,000	0.2806	Influent data missing
06/15	8,741,000	0.2914			9,473,000	0.3158	Influent data missing
07/15	8,573,000	0.2765			10,695,000	0.3450	Influent data missing
08/15	8,772,000	0.2830		0.4545	10,253,000	0.3307	Influent data missing
09/15	8,732,000	0.2911			9,533,000	0.3178	Influent data missing
10/15	8,798,000	0.2838			10,367,000	0.3344	Influent data missing
11/15	8,302,000	0.2767		0.4090	9,860,000	0.3287	Influent data missing
12/15	9,814,000	0.3166	15,170,000	0.4894	11,190,000	0.3610	
2015	102,302,000	0.2803			113,638,000	0.3111	
01/16	10,283,000	0.3317	11,527,000	0.3718	10,337,000	0.3335	
02/16	8,194,000	0.2826	11,515,000	0.3971	10,277,000	0.3544	
03/16	8,635,000	0.2785	10,898,000	0.3515	9,763,000	0.3149	
04/16	8,366,000	0.2789	10,260,000	0.3420	9,101,000	0.3030	
05/16	8,892,000	0.2868	10,909,000	0.3520	9,957,000	0.3210	
06/16	8,868,000	0.2956	10,970,000	0.3660	9,494,000	0.3160	
07/16	9,238,000	0.2980	11,383,000	0.3670	9,675,000	0.3120	
08/16	3,518,000	0.1135	11,113,000	0.3580	9,382,000	0.3030	
09/16	3,066,000	0.1022	11,141,000	0.3710	9,426,000	0.3140	
10/16	4,368,000	0.1409	10,902,000	0.3520	9,809,000	0.3160	
11/16	5,641,000	0.1880	7,773,000	0.2580	6,619,000	0.2210	
12/16	6,966,000	0.2247	10,878,000	0.3510	8,856,000	0.2860	
2016	86,035,000	0.2351	129,269,000	0.3531	112,696,000	0.3079	WTP in use 2016
01/17	7,986,000	0.2576	10,795,000	0.3480	9,587,000	0.3090	
02/17	6,517,000	0.2328	9,120,000	0.3260	7,863,000	0.2810	
03/17	7,146,000	0.2305	10,419,000	0.3360	9,005,000	0.2900	
04/17	6,831,000	0.2277	10,267,000	0.3420	8,880,000	0.2960	
05/17	7,231,000	0.2333	10,617,000	0.3420	9,018,000	0.2910	
06/17	6,717,000	0.2239	13,313,000	0.4440	7,974,000	0.2660	
07/17	7,401,000	0.2387	9,606,000	0.3100	8,767,000	0.2830	
08/17	6,851,000	0.2210	7,921,000	0.2560	8,128,000	0.2620	
2017	56,680,000	0.2332	82,058,000	0.3380	69,222,000	0.2848	

1) All purchased water is returned to the sewer. In a typical municipal system, this is not the case, usually it is about 80-85% return; however, this facility is not a typical municipal system.

2) There are no other sources of water except the City of Pikeville and I/I.

In 2014, approximately 18,088,000 gallons (0.05 mgd) were discharged into Mill Creek that were not purchased from the City of Pikeville. That is approximately 16.4% of the effluent flow. Similarly, approximately 11,336,000 gallons (0.03 mgd) and 12,542,000 gallons (0.05 mgd) were discharged but not purchased in 2015 and 2017 (partial year). These reflect approximately 10% and 18% of the effluent flow, respectively. If the assumptions are valid, an average of approximately 15% of the effluent flow is attributable to I/I. Of course, the actual volume of I/I is heavily dependent on weather conditions and the accuracy of the assumptions. If all purchased water does not enter the sewer, then the amounts of inflow and infiltration are greater than the estimated amounts. The volume can be reduced by diligent efforts towards collection system rehabilitation.

PROPOSED WWTP DESIGN CAPACITY

The existing WWTP design capacity of 0.315 million gallons per day (mgd) is less than desirable at the current flows with an average influent of 0.355 mgd. When the planned additions, the activation of BCCX-3, infiltration and inflow, and the natural variations in sewage flow are considered, it becomes apparent that a significant increase in capacity is necessary. The current WWTP design lends itself to design increments of 50%. However, an increase in capacity of 50% to 0.473 mgd is not sufficient for the projected flows. It is therefore recommended that the design capacity be doubled to 0.630 MGD. The allowable peak flow will increase from 0.425 mgd to 0.850 mgd. This capacity should be adequate to allow the facility to operate within its design capacity and to provide the opportunity to meet the stringent nutrient limits that have been established. Another significant addition to the WWTP should be an aerated influent flow equalization basin to buffer the daily flow variations and to allow the treatment processes to operate in a steady state to further enhance the level of treatment obtained. In addition to the influent flow equalization basin, the post equalization basin is too small. The proposed expansion will incorporate an oversized post equalization basin to provide sufficient total volume to allow the filters to operate with a steady throughput, which will improve nutrient removal.

NPDES PERMIT

The new BCCX WWTP was assigned NPDES Permit Number TN0056626, which was issued effective March 1, 2013, with an expiration date of February 28, 2018. The permit provides for a design capacity of 0.315 MGD to discharge into Mill Creek at Mile 1.0 above its confluence with Glade Creek at Mile 3.8. It should be noted that Glade Creek flows into Bee Creek, which has been designated an Exceptional Tennessee Water (ETW) based on habitat for a particular species of mussel. A full copy of the NPDES Permit is not included herein, but the following Table 3, Summary of NPDES Permit Limits, summarizes the discharge limits for pertinent constituents. A copy of the NPDES Permit is available online at the TDEC Water Resources Permits Dataviewer (http://tn.gov/environment/article/wr-water-resources-data-viewer).

Table 3
Summary of NPDES Permit Limits

Parameter	Monthly Average Loading (lb/day)	Monthly Average Concentration(mg/l)
$CBOD_5$	30	11.4
Chlorine Residual	-	0.02
E. Coli	-	126 (#/100ml)
IC_{25}	-	100%
Total Nitrogen	7.5	2.9
DO	-	6.0
Phosphorus	1.5	0.6
TSS	45	17.1
pH	-	6.5-9.0
NH_3 Nitrogen (Summer)	1.86	0.7
NH_3 Nitrogen (Winter)	3.2	1.2
Settleable Solids	-	1.0ml/L

Several of these parameters were established to limit degradation of Mill Creek by the prohibition of additional pollutant loadings to the stream. In doing so, the concentration limit for Total Nitrogen of 2.9 mg/l approaches the limit of traditional treatment technology.

Expansion of the WWTP capacity will require due consideration of the current NPDES permit, which expired in February 2018. In the application for renewal, which was submitted by Quantum Environmental and Engineering Services, LLC (QE2) in August 2017, the following permit revisions were requested:

1) Effluent nutrient limits (nitrogen and phosphorus) be based on loading (pounds) only, not on concentration (mg/l); and
2) Nutrient loading be based on a rolling annual average, not on a monthly average.

These revisions to the current NPDES permit were suggested by the TDEC Division of Water Resources to allow the WWTP some additional operational flexibility while continuing to apply appropriate protection to the receiving stream (Mill Creek) and to the two (2) Exceptional Tennessee Waters downstream (Glade Creek and Bee Creek). However, the Division of Water Resources has also indicated that the permit limits for nutrients cannot be increased for either concentration or load over those contained in the current permit, except as noted previously. Therefore, in order to double the capacity of the wastewater treatment system, an alternative method of effluent disposal must be developed for one-half of the total future flow.

It is recommended to treat up to 315,000 gpd using the existing WWTP. In order to fulfill this objective, several improvements are recommended as briefly mentioned previously. The remaining one-half of the proposed capacity or 315,000 gpd will require both treatment and disposal and the options available for disposal from this facility are quite limited. No suitable stream is available within a reasonable distance for disposal. No municipal system is available within a reasonable distance for treatment and disposal. The remaining one-half of the proposed capacity should be land-applied by spray irrigation following treatment. Spray irrigation is preferable to the other available alternative of drip dispersal. Both the initial (capital) cost and the operation and maintenance costs are less for spray irrigation than for drip dispersal. Wastewater to be land-applied will receive the same level of treatment as wastewater to be discharged to Mill Creek. Piping connections between the treatment trains will allow units to be removed from service and the discharge directed as needed.

For the portion of flow to be land applied, an NPDES permit is not required. Land application requires a State Operating Permit (SOP) which is issued only after a Plan of Operation and Management (POM) is developed and submitted for the land treatment system.

EXISTING WASTEWATER TREATMENT PLANT DESIGN AND OPERATION

Construction was completed and the WWTP has been in operation since early 2013. Its stated design capacity is an average daily flow of 0.315 MGD with a peak daily flow of 0.425 MGD. The

WWTP process design includes influent head works with screening and metering, influent pump station, flow splitter, two (2) parallel sequencing batch reactors (SBRs), post flow equalization basin, effluent filter pumps, three (3) effluent filters, ultraviolet disinfection, flow measurement, post aeration, and discharge to Mill Creek. Sludge from the SBRs is discharged to a two-cell aerobic digester and, after thickening, is then processed by a belt filter press and subsequently land applied to areas adjacent to the WWTP. Ancillary facilities include a non-potable water system; chemical feed systems for caustic, acetic acid, sodium aluminate, and polymer; an instrumentation and control system; and a laboratory. A Process Flow Diagram for the existing WWTP is included as Figure 1 and the Hydraulic Profile is included as Figures 2A, 2B, and 2C.

An Operations Summary from Discharge Monitoring Reports for the period from January 2013 through August 2017 (Table 4) is attached as pages 13 and 14 of this document. The average daily influent flow has been approximately 355,000 gpd and the average monthly maximum day flow has been approximately 515,000 gpd. During the period represented, only two (2) effluent violations are noted other than Total Nitrogen and Phosphorus. The WWTP has failed to meet the discharge requirements for Total Nitrogen and Phosphorus a significant portion of the time.

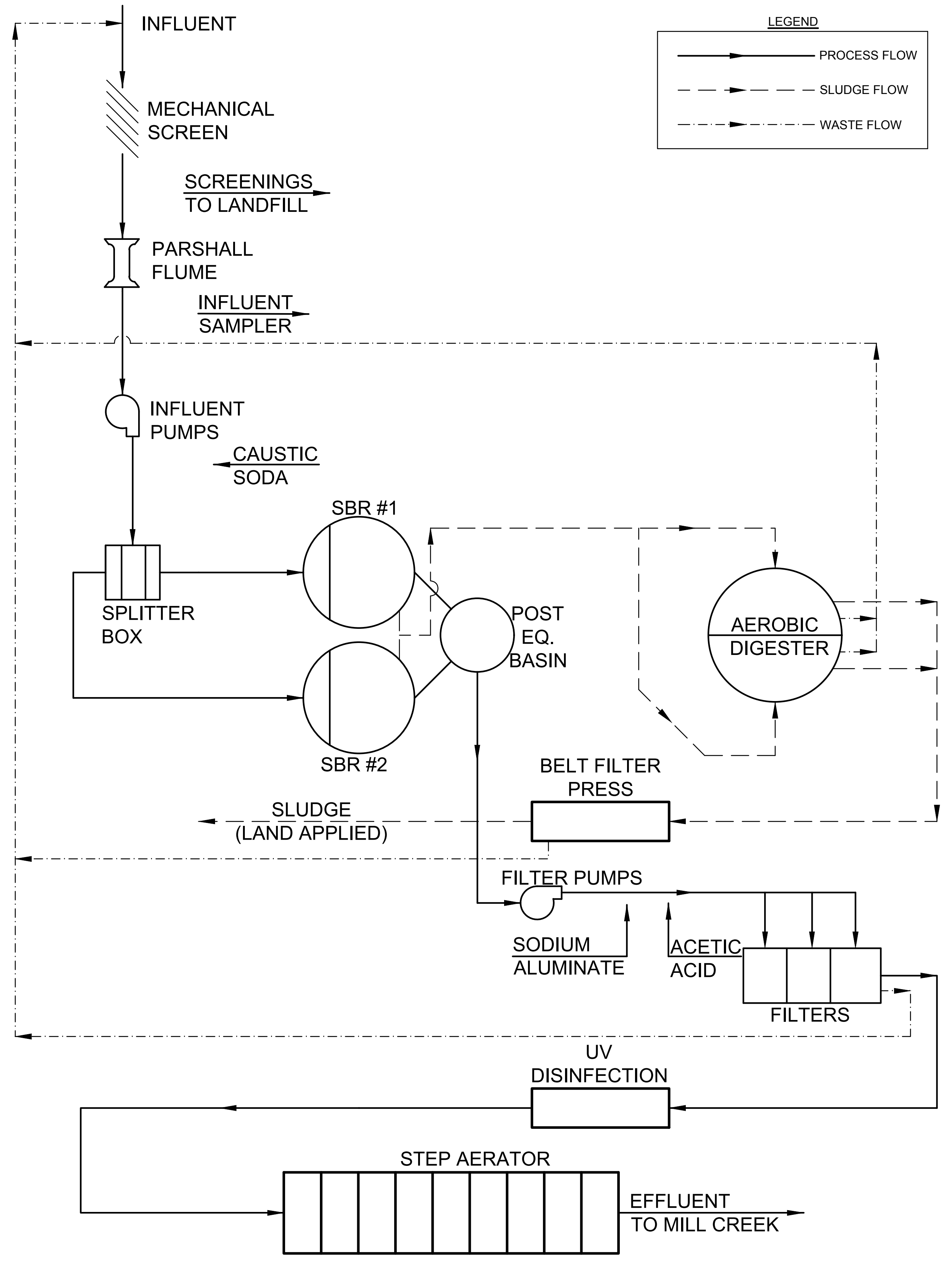
The removal of Total Nitrogen to the permit limit of 2.9 mg/l is approaching the limits of technology for traditional wastewater treatment; however, under appropriate design conditions, the existing process should be capable of meeting the permit limits for both of the regulated nutrients, nitrogen and phosphorus. Several factors contributed to these violations:

1) The sodium aluminate feed system was removed from service and the chemical solidified in the pumps and plumbing. This problem has now been corrected and a flushing system to remove the salt from the system on shutdown has been installed.

2) Influent flows are usually greater than the average day design capacity. Successful treatment of Total Nitrogen and Phosphorus requires flows below the design capacity.

3) Flows to the effluent filters vary by a significant amount as the filter pumps empty the post-flow equalization basin. The pumps will stop while the basin fills and then pump until the basin is empty. For the best results from the filter operation, the filters should operate in a steady state with minimal variation. The post-flow equalization basin is undersized. The expansion will provide another post-flow equalization basin sized appropriately and connected to the existing basin so that the total volume will allow steady state operation of the effluent filters.

4) Flow patterns to the WWTP indicate approximately 85% of the daily flow occurs over approximately 65% of the 24-hour day (day and early evening) with very low flows (approximately



CTi ENGINEERS, INC.

3354 PERIMETER HILL DRIVE,
SUITE 140
NASHVILLE, TN 37211
615-834-8300

BLEDSOE COUNTY CORRECTIONAL COMPLEX
WASTEWATER TREATMENT PLANT EXPANSION

FIGURE 1
EXISTING PROCESS FLOW DIAGRAM

DATE: MARCH 27, 2018

JOB NO. N16003

CTI PROJECT: N16003-01 Bledsoe County Correctional Complex (TN Dept. General Services TDEC), DRAWING: FIGURE 2 - WWTP HYDRAULIC PROFILE (547835v7/3/29/18 2:09PM), LAYOUT: WATER 2A

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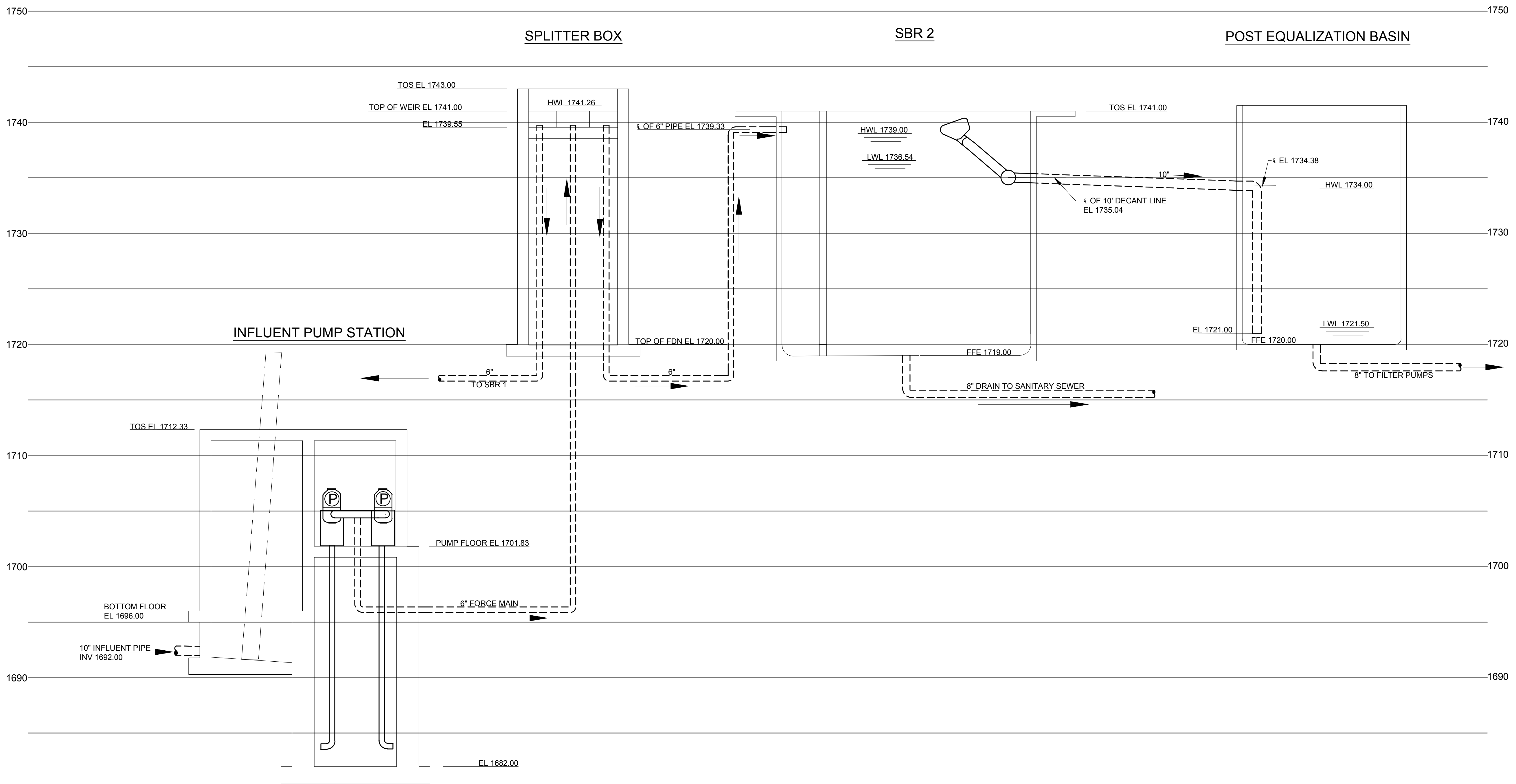

REVISIONS

NO.	DESCRIPTIONS	DATE	BY	APP'D

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WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX

PIKEVILLE, BLEDSOE COUNTY, TENNESSEE SBC PROJECT 142/013-01-2013-06

FIGURE 2A
EXISTING PROCESS HYDRAULIC PROFILE
1 OF 2

CTI ENGINEERS, INC.
3354 PERIMETER HILL DRIVE, SUITE 140
NASHVILLE, TN 37211
615-834-8300

JOB NO.
N16003

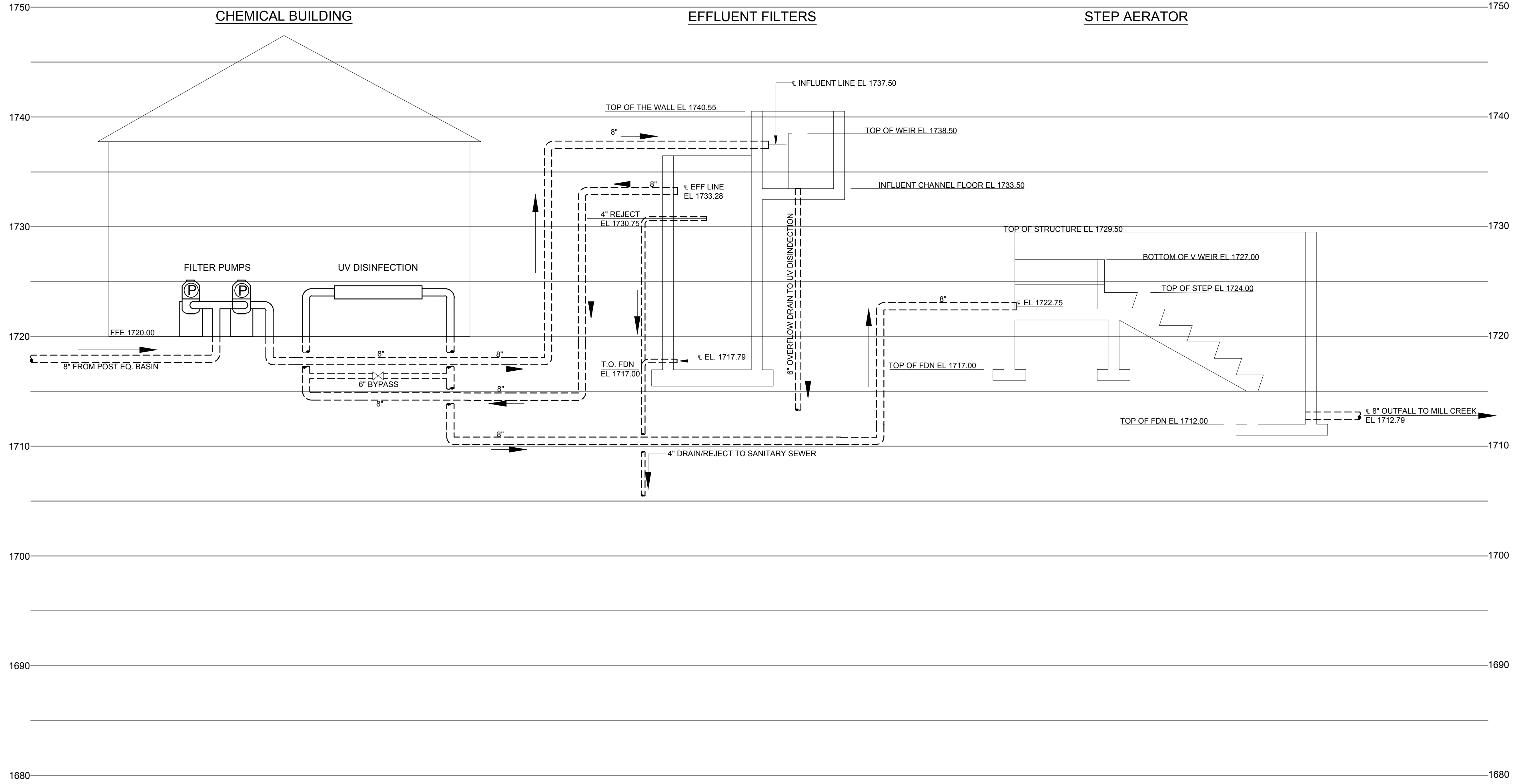
ISSUE DATE
MARCH 2018

SCALE
AS NOTED

FIGURE NO.
FIGURE 2A

CTI PROJECT: N16003-01 Bledsoe County Correctional Complex (TN Dept. General Services TDEC), DRAWING: FIGURE 2 - WWTP HYDRAULIC PROFILE (547835v7/3/29/18 2:09PM), LAYOUT: WATER 2B

DATE OF PRINT: 3/29/2018 3:15 PM



REVISIONS

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APPROVED APR

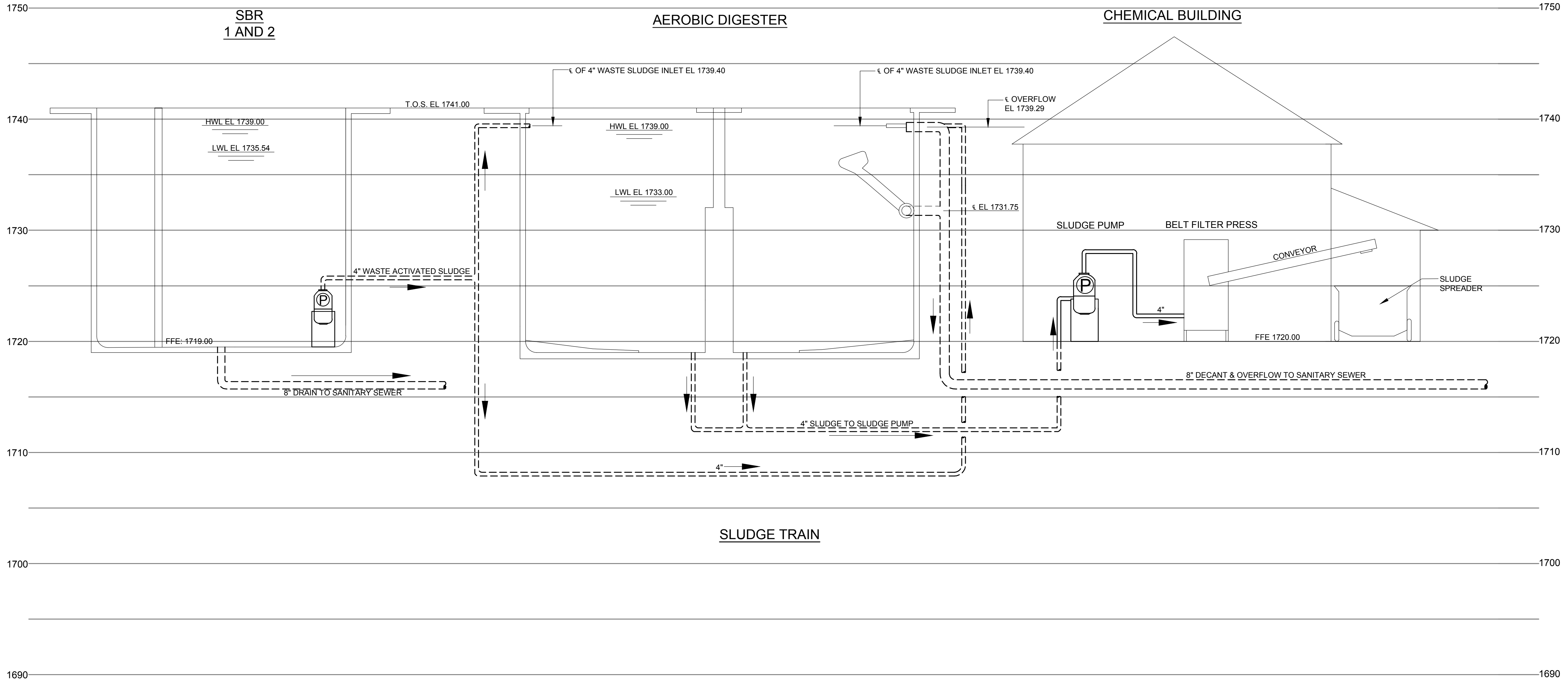
WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX
PIKEVILLE, BLEDSOE COUNTY, TENNESSEE SBC PROJECT 142/013-01-2013-06

FIGURE 2B
EXISTING PROCESS HYDRAULIC PROFILE
2 OF 2

CTI ENGINEERS, INC.
3354 PERIMETER HILL DRIVE, SUITE 140
NASHVILLE, TN 37211
615-834-8300

JOB NO.
N16003
ISSUE DATE
MARCH 2018
SCALE
AS NOTED
DRAWING NO.
FIGURE 2B

CTI PROJECT: N16003-01 Bledsoe County Correctional Complex (TN Dept. General Services TDEC), DRAWING: FIGURE 2 - WWTP HYDRAULIC PROFILE (547835v7/3/29/18 2:09PM), LAYOUT: WASTE 2C



SLUDGE TRAIN

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REVISIONS

NO.	DESCRIPTIONS	DATE	BY	APP'D

DESIGN
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APPROVED
APR

WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX

PIKEVILLE, BLEDSOECOUNTY, TENNESSEE SBC PROJECT 142/013-01-2013-06

FIGURE 2C
EXISTING SLUDGE HYDRAULIC PROFILE

CTI ENGINEERS, INC.
3354 PERIMETER HILL DRIVE, SUITE 140
NASHVILLE, TN 37211
615-834-8300

JOB NO.
N16003

ISSUE DATE
MARCH 2018

SCALE
AS NOTED

DRAWING NO.
FIGURE 2C

DATE OF PRINT: 3/29/2018 3:15 PM

BLEDSOE COUNTY CORRECTIONAL COMPLEX
WASTEWATER TREATMENT PLANT
SBC 142/013-01-2013-06 CTI N16003

TABLE 4 OPERATIONS SUMMARY FROM MONTHLY OPERATION REPORTS

🟧	Result Unknown
🟨	Violation

			Design				Permit				Permit				Permit				Permit				Permit	
			0.315	0.425			11.4	17.1			17.1	25.9			0.7 (5-10) 1.2 (11-4)	1.4 (5-10) 2.4 (11-4)			2.9				0.6	
	Influent Flow (MGD)		Effluent Flow (MGD)		CBOD Influent (mg/l)		CBOD Effluent (mg/l)		TSS Influent (mg/l)		TSS Effluent (mg/l)		NH3-N Influent (mg/l)		NH3-N Effluent (mg/l)		Total N Influent (mg/l)		Total N Effluent (mg/l)		Total P Influent (mg/l)		Total P Effluent (mg/l)	
Month/Year	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day
01/13	0.266	0.443	0.242	0.391	174.000	362.000	1.600	4.000	174.000	335.000	2.100	10.000	5.100	11.000	0.400	1.100			🟧				🟧	
02/13	0.294	0.328	0.225	0.269	169.000	268.000	1.000	1.000					4.700	7.300	0.200	0.400			🟧				🟧	
03/13	0.324	0.395	0.222	0.295	193.000	337.000	1.200	3.000	261.000	485.000	1.000	1.000	9.000	15.000	0.400	1.100			🟧				🟧	
04/13	0.378	0.490	0.267	0.349	131.000	350.000	2.000	7.000	197.000	660.000	3.000	22.000	13.600	20.000	0.200	0.600			🟧				🟧	
05/13	0.338	0.511	0.282	0.437	169.000	294.000	3.000	9.000	224.000	505.000	3.000	16.000					40.000	55.000	2.100	2.500	5.800	11.000	0.230	0.330
06/13	0.337	0.403	0.269	0.341	203.000	260.000	4.100	9.000	323.000	725.000	8.000	14.000					26.000	33.000	1.767	2.200	5.900	8.500	0.200	0.200
07/13	0.313	0.453	0.228	0.324	245.000	440.000	3.900	12.000	278.000	565.000	14.600	37.000					32.300	42.000	1.600	2.700	4.100	4.700	0.000	0.000
08/13			0.294	0.338	203.000	420.000	2.400	6.000	204.000	380.000	7.900	22.000	19.000	19.000	0.100	0.100	39.000	43.000	1.900	2.200	5.650	6.100	🟧	
09/13			0.220	0.340	220.000	480.000	1.500	3.000	180.000	330.000	3.200	5.000	17.000	23.000	0.100	0.100	40.000	54.000	2.200	2.400	4.925	11.000	0.300	0.300
10/13			0.134	0.158	235.000	440.000	1.900	4.000	297.000	540.000	8.900	19.000	18.000	24.000	0.100	0.100	30.000	32.000	1.800	2.300	5.640	7.000	0.200	0.300
11/13			0.149	0.256	237.000	339.000	3.200	6.000	325.000	500.000	12.800	24.000	14.000	22.000	0.100	0.400			🟧				🟧	
12/13	0.393	0.508	0.277	0.398	236.000	320.000	2.900	6.000	342.000	565.000	9.200	18.000	12.000	14.000	0.300	0.800			🟧				🟧	
01/14	0.336	0.393	0.312	0.364	315.000	440.000	4.700	10.000	309.000	545.000	7.200	16.000	23.000	35.000	0.600	1.300			🟧				🟧	
02/14	0.361	0.515	0.308	0.446	276.000	452.000	6.300	14.000	268.000	535.000	11.300	22.000	19.000	23.000	1.100	2.400			🟧				🟧	
03/14	0.354	0.507	0.302	0.413	233.000	400.000	4.200	9.000	296.000	570.000	12.900	24.000	21.000	26.000	0.500	0.800			🟧				🟧	
04/14	0.359	0.453	0.314	0.469	252.000	404.000	3.800	7.000	327.000	620.000	8.000	15.000	18.000	30.000	0.300	0.500			🟧				🟧	
05/14	0.384	0.468	0.324	0.465	224.000	400.000	2.000	5.000	224.000	340.000	5.100	10.000	16.000	18.000	0.100	0.600	27.000	34.000	7.300	10.000	5.075	7.300	2.600	3.700
06/14	0.346	0.425	0.329	0.523	253.000	630.000	6.300	13.000	240.000	370.000	10.200	22.000	20.000	24.000	0.500	1.200	27.000	31.000	10.200	13.000	4.725	6.000	1.300	1.600
07/14	0.357	0.396	0.335	0.421	281.000	530.000	3.100	8.000	201.000	500.000	3.100	9.000	18.000	28.000	0.100	0.300	28.000	34.000	3.300	3.700	5.100	7.100	1.300	1.800
08/14	0.360	0.432	0.346	0.407	215.000	330.000	2.100	5.200	183.000	240.000	2.400	9.200	16.000	21.000	0.100	0.200	28.000	30.000	4.700	6.700	5.575	7.300	2.200	4.300
09/14	0.416	0.463	0.287	0.375	246.000	360.000	1.600	3.000	240.000	400.000	4.900	9.400	15.000	23.000	0.100	0.400	25.000	33.000	4.300	5.200	5.400	9.700	1.500	2.800
10/14	0.434	0.573	0.273	0.519	221.000	430.000	1.800	8.000	284.000	410.000	5.600	14.000	16.000	25.000	0.100	0.100	25.000	29.000	4.900	7.900	6.760	11.000	4.000	6.400
11/14	0.322	0.427	0.243	0.423	199.000	350.000	1.300	2.800	221.000	360.000	2.800	5.600	23.000	31.000	0.100	0.200			🟧				🟧	
12/14	0.305	0.404	0.259	0.641	248.000	360.000	2.400	7.000	197.000	330.000	2.700	10.800	19.000	24.000	0.300	2.400			🟧				🟧	
01/15	0.362	0.475	0.255	0.609	178.000	330.000	4.100	2.100	186.000	390.000	8.000	17.500	17.000	23.000	0.700	2.000	23.000	27.000	2.700	3.300	4.100	4.900	0.500	0.900
02/15	0.357	0.466	0.261	0.809	196.000	350.000	3.300	8.600	231.000	485.000	8.100	25.000	19.000	26.000	0.500	2.400	29.000	34.000	2.700	5.400	5.025	7.600	0.300	0.500
03/15	0.358	0.466	0.266	0.419	223.000	310.000	3.700	11.800	292.000	680.000	8.400	20.000	16.000	23.000	0.200	0.400	29.000	40.000	2.400	2.600	3.675	5.800	0.400	0.500
04/15	0.355	0.476	0.337	0.649	199.000	280.000	2.800	4.400	305.000	525.000	5.600	13.000	18.000	25.000	0.100	0.300	35.000	43.000	2.300	3.300	5.380	6.400	0.600	1.300
05/15	0.333	0.378	0.281	0.338	209.000	390.000	1.600	4.300	330.000	545.000	3.000	10.000	18.000	30.000	0.100	0.200	31.000	40.000	2.800	3.100	8.400	11.000	3.900	5.300
06/15			0.316	0.369	255.000	530.000	2.000	9.000	372.000	540.000	4.000	20.000	18.000	25.000	0.400	1.300	30.000	35.000	2.700	4.200	6.575	8.600	3.400	7.200
07/15			0.345	0.448	236.000	380.000	1.500	4.000	355.000	665.000	1.000	6.400	17.000	23.000	0.200	0.600	33.000	42.000	2.800	4.100	7.940	12.000	3.100	4.800
08/15	0.455	1.158	0.331	0.402	184.000	270.000	1.000	1.000	287.000	390.000	2.200	18.600	14.000	20.000	0.100	0.200	33.000	48.000	3.200	4.700	8.738	14.000	4.400	5.800
09/15			0.318	0.399	223.000	470.000	1.000	1.000	414.000	770.000	1.200	5.400	15.000	27.000	0.100	0.100	34.000	39.000	2.900	3.300	6.988	9.900	3.500	5.700
10/15			0.334	0.450	160.000	220.000	1.000	1.000	239.000	440.000	1.600	3.000	14.000	17.000	0.100	0.300	23.000	29.000	2.300	3.100	4.866	6.400	1.400	1.900
11/15	0.409	0.689	0.329	0.530	211.000	370.000	3.300	6.200	322.000	660.000	4.000	9.700	19.000	31.000	0.400	0.900	25.000	32.000	6.900	8.800	4.295	5.900	1.800	2.700
12/15	0.489	1.003	0.361	0.503	209.000	360.000	2.100	5.800	339.000	545.000	5.200	18.000	14.000	17.000	0.100	0.300	26.000	29.000	4.100	5.400	4.884	6.000	1.600	3.900

BLEDSOE COUNTY CORRECTIONAL COMPLEX
WASTEWATER TREATMENT PLANT
SBC 142/013-01-2013-06 CTI N16003

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															1.2 (11-4)	2.4 (11-4)								
	Influent Flow (MGD)		Effluent Flow (MGD)		CBOD Influent (mg/l)		CBOD Effluent (mg/l)		TSS Influent (mg/l)		TSS Effluent (mg/l)		NH3-N Influent (mg/l)		NH3-N Effluent (mg/l)		Total N Influent (mg/l)		Total N Effluent (mg/l)		Total P Influent (mg/l)		Total P Effluent (mg/l)	
Month/Year	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day	Avg	Max day
01/16	0.372	0.501	0.333	0.435	219.000	380.000	6.000	11.000	273.000	495.000	14.700	23.000	19.000	30.000	0.700	2.200	23.000	30.000	4.910	7.920	6.978	12.000	2.200	3.000
02/16	0.397	0.654	0.354	0.550	196.000	330.000	4.100	11.200	212.000	370.000	9.600	19.000	20.000	24.000	0.400	1.700	31.000	35.000	4.900	7.400	5.040	6.300	1.300	2.100
03/16	0.352	0.518	0.315	0.393	252.000	460.000	7.200	14.200	290.000	390.000	8.600	23.000	17.000	24.000	0.100	0.400	35.000	48.000	2.200	3.400	7.190	9.100	2.600	3.900
04/16	0.342	0.453	0.303	0.359	212.000	329.000	5.600	9.900	274.000	495.000	13.700	25.000	25.000	34.000	0.190	0.490	37.000	52.000	3.500	6.600	5.950	7.600	2.600	3.800
05/16	0.352	0.544	0.321	0.388	203.000	320.000	4.000	8.000	315.000	530.000	13.000	28.000	21.000	27.000	0.110	0.240	27.000	33.000	2.100	2.400	6.780	8.300	2.600	3.700
06/16	0.366	0.517	0.316	0.392	224.000	410.000	1.000	3.000	322.000	460.000	1.100	3.200	24.000	27.000	0.130	0.420	32.000	36.000	3.400	4.000	4.995	5.600	3.000	3.600
07/16	0.367	0.510	0.312	0.423	296.000	660.000	2.000	7.000	282.000	435.000	1.800	13.000	23.000	27.000	0.080	0.220	28.000	32.000	2.900	3.800	6.468	8.800	2.100	3.400
8/16	0.358	0.440	0.303	0.374	175.000	320.000	1.000	3.000	224.000	450.000	1.400	6.200	25.000	48.000	0.060	0.080	34.000	50.000	2.500	4.900	6.370	9.900	2.600	3.800
9/16	0.371	0.640	0.314	0.514	175.000	280.000	1.000	3.000	166.000	675.000	2.300	5.000	32.000	74.000	0.260	1.400	25.000	27.000	1.400	1.500	4.248	4.800	2.100	4.000
10/16	0.352	0.439	0.316	0.427	173.000	240.000	2.000	7.000	179.000	360.000	5.000	20.000	17.000	21.000	0.100	0.300	26.000	31.000	1.800	2.100	5.450	5.800	3.500	4.700
11/16	0.258	0.419	0.221	0.342	264.000	520.000	1.000	3.000	302.000	515.000	3.000	5.000	20.000	36.000	0.100	0.300	38.000	41.000	3.200	6.500	6.500	7.300	3.200	4.200
12/16	0.351	0.576	0.286	0.410	336.000	720.000	4.000	10.000	264.000	675.000	9.000	24.000	16.000	28.000	0.200	0.600	31.000	46.000	2.700	3.200	5.610	7.200	2.100	2.900
01/17	0.348	0.489	0.309	0.416	261.000	610.000	2.000	4.000	164.000	275.000	3.000	11.000	12.000	19.000	0.200	0.700	28.000	31.000	2.600	2.900	5.460	6.500	2.300	4.100
02/17	0.326	0.412	0.281	0.377	237.000	470.000	2.000	5.000	182.000	270.000	2.000	5.000	15.000	22.000	0.100	0.300	28.000	31.000	3.200	3.800	5.548	6.200	1.600	2.500
03/17	0.336	0.462	0.290	0.361	257.000	540.000	2.000	6.000	223.000	390.000	2.000	4.000	19.000	23.000	0.100	0.400	24.000	34.000	2.500	3.200	3.888	6.700	2.400	3.200
04/17	0.342	0.574	0.296	0.465	214.000	410.000	2.000	3.000	210.000	495.000	2.000	5.000	22.000	27.000	0.200	0.700	32.000	38.000	2.100	2.500	5.448	6.400	2.200	2.900
05/17	0.342	0.486	0.291	0.416	282.000	573.000	1.000	3.000	350.000	675.000	2.000	7.000	17.000	27.000	0.300	0.600	32.000	40.000	2.200	2.500	5.902	8.100	2.000	4.000
06/17	0.444	1.217	0.266	0.384	218.000	400.000	2.000	5.000	229.000	605.000	2.000	6.000	21.000	26.000	0.300	0.900	36.000	47.000	5.000	7.300	6.133	6.800	3.200	4.000
07/17	0.310	0.458	0.283	0.417	196.000	370.000	3.000	7.000	180.000	425.000	2.000	7.000	19.000	24.000	0.400	0.700	35.000	41.000	6.700	8.600	6.468	8.500	3.200	3.700
08/17	0.256	0.322	0.262	0.325	229.000	410.000	2.000	5.000	231.000	470.000	2.000	9.000	19.000	23.000	0.400	1.000	39.000	47.000	6.100	8.300	6.604	7.800	3.500	4.300
AVERAGE	0.355	0.515	0.288	0.419	222.857	400.143	2.689	6.241	260.709	489.091	5.498	14.000	17.706	25.308	0.250	0.712	30.439	37.682	3.404	4.657	5.740	7.839	2.070	3.117

15% of the total) during the remainder of the day (night and early morning). When higher influent flows are detected, the SBRs automatically enter storm or maintenance mode, which reduces the process cycle times by 50%. The reduced cycle times inhibit both Total Nitrogen and Phosphorus removal.

In addition to the expanded capacity and both the influent and post flow equalization basins required, the existing facility has several other problems which require correction. The following issues will be addressed during this expansion project:

A) Correct the non-potable water supply issues. During the night time low flow periods, the non-potable water system loses its source of supply.
B) Correct sampling, communication, and control issues between the Chemscan unit and the Parkson controller for the chemical feed pumps.
C) Air leaks in the air diffuser piping and/or air valves in the SBRs should be repaired.
D) Control adjustments are required to allow the influent pumps to vary flow more readily and to allow the filter influent pumps to operate in a steady state. The expansion of the post-flow equalization basin will correct the filter influent feed cycles.
E) The SBRs are heavily impacted by filamentous bacteria, which leads to sludge bulking and the inability to successfully thicken the sludge. This problem should be investigated further during design and remedial action designed and implemented during construction. As a minimum, the SBRs will be retrofitted with scum removal systems.
F) The dissolved oxygen (DO) levels in the filter influent flow are too high and inhibit nitrogen removal. The cause for the DO levels appears to be twofold. First, the drop pipe from the SBR to the post equalization basin may entrain air and increase the DO level. Second, the filter influent pump pulls air into the flow stream when the post equalization basin approaches empty.
G) The influent mechanical bar screen allows small solids to enter the WWTP. These consist of individual serving packets for condiments, which are used extensively in the facility, as well as other small items of inorganic composition. Many of these items are passed through the treatment processes and are then discharged into Mill Creek. In addition to the discharge violations, which have not yet been noted, these items also have the potential to damage the operation of existing equipment such as the air lifts within the effluent filters.

PROPOSED IMPROVEMENTS AND EXPANSION

The existing WWTP will be expanded to double its design capacity to 0.630 mgd average daily flow and to 0.850 mgd peak daily flow. The operational issues previously presented will be corrected and the process components duplicated to achieve the required design capacity. One half of the expanded capacity will be discharged to Mill Creek and the other half will be land applied by spray irrigation. The Proposed Process Flow Diagram is presented as Figure 3 and the Proposed Hydraulic Profile is included as Figures 4A, 4B, and 4C.

1. Operational Improvements and Repairs to the Existing Wastewater Treatment Plant

A) Due to low flows overnight, the non-potable water system cannot supply sufficient water under the low-flow condition. Multiple non-potable supply pumps have failed due to a lack of available effluent water during the night time hours. Subsequently, the operators have installed hoses to provide potable water to essential items of equipment. Re-connect essential equipment that requires water to the non-potable water system to eliminate the use of exposed hoses. The exposed hoses create safety issues and are subject to freezing. Relocate the source of supply for the non-potable water system to the proposed effluent storage tank to be addressed later in this document.

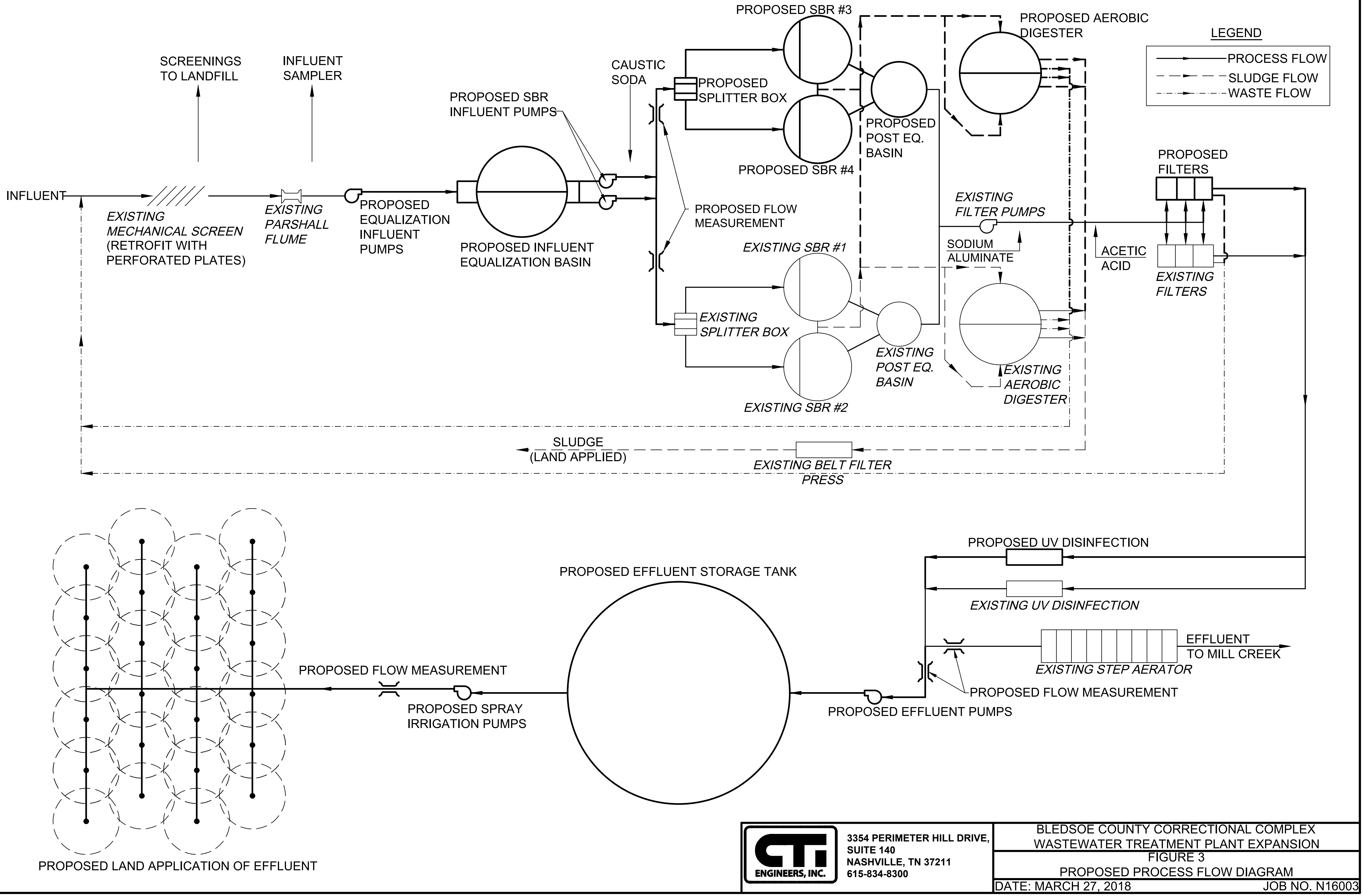
B) Correct sampling, communication, and control issues between the Chemscan unit and the Parkson controller for the chemical feed pumps. Much of this problem is the result of the inability of the controller and chemical feed pumps to dose at a sufficiently low rate during the night time low flows. This issue will be eliminated with the proposed influent equalization basin providing a more stable throughput over the course of the day.

C) Repair air leaks in the piping and valves in the SBR air diffuser system. These leaks allow air to enter the basins when “no air” is the treatment requirement and this leakage disrupts the required treatment sequence.

D) Control adjustments are required to allow the influent pumps to vary flow more readily and to allow the filter influent pumps to operate in a steady state. The influent pumps will be replaced and the control system will be revised as part of the WWTP expansion. The expansion of the post-flow equalization basin in the WWTP expansion will correct the filter influent feed cycles.

CTI PROJECT: N16003-01 Bledsoe County Correctional Complex (TN Dept. General Services TDEC), DRAWING: FIGURE 3 - PROPOSED PROCESS FLOW DIAGRAM (530933v11/3/28/18 1:25PM), LAYOUT: Layout1

DATE OF PRINT: 3/28/2018 4:35 PM



CTi ENGINEERS, INC.

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NASHVILLE, TN 37211
615-834-8300

BLEDSOE COUNTY CORRECTIONAL COMPLEX
WASTEWATER TREATMENT PLANT EXPANSION

FIGURE 3
PROPOSED PROCESS FLOW DIAGRAM

DATE: MARCH 27, 2018
JOB NO. N16003

CTI PROJECT: N16003-01 Bledsoe County Correctional Complex (TN Dept. General Services TDEC), DRAWING: FIGURE 4 - WWTP HYDRAULIC PROFILE (547836v5/3/29/18 10:31AM), LAYOUT: WATER 4A

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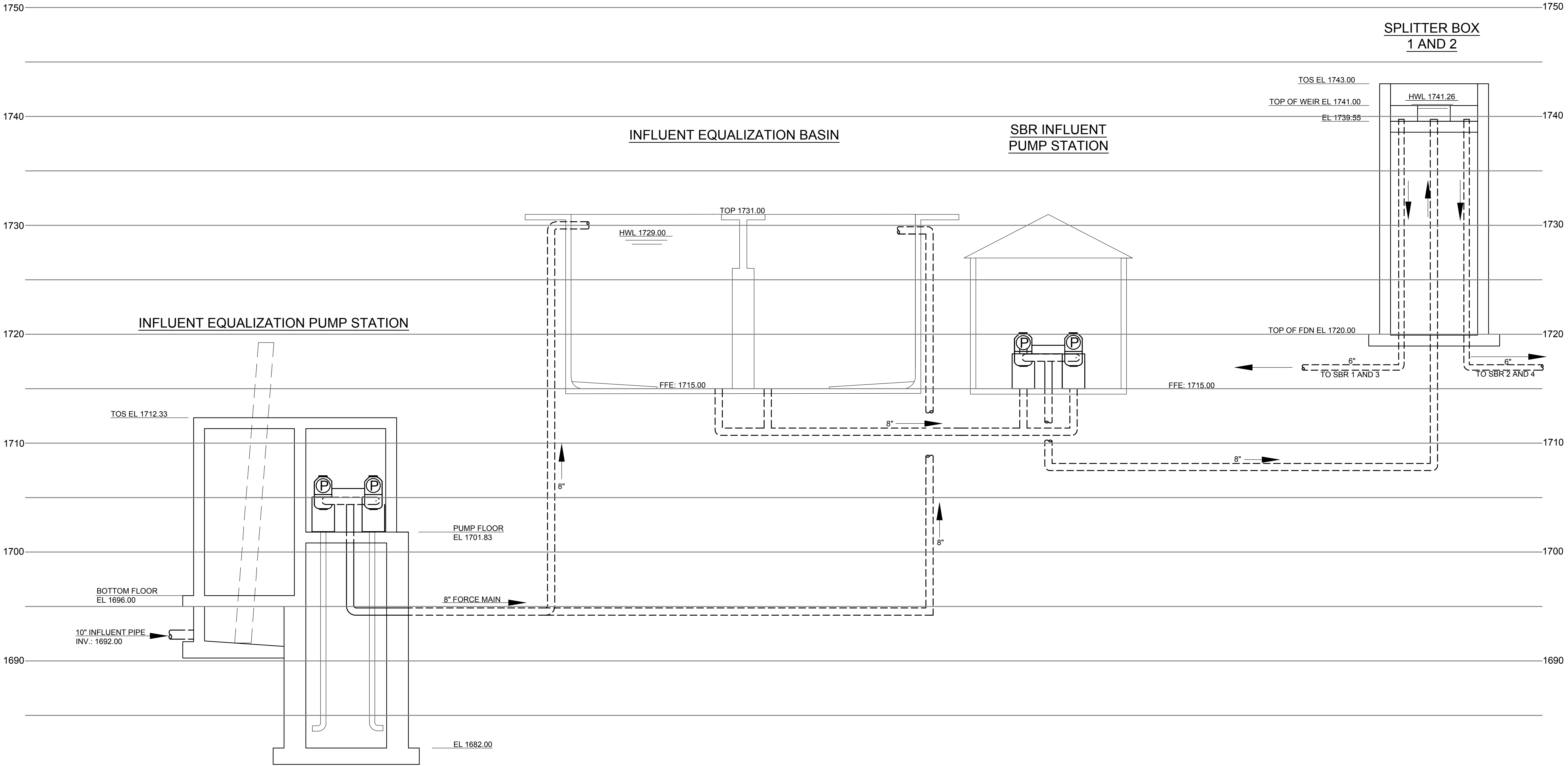

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WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX
PIKEVILLE, BLEDSOE COUNTY, TENNESSEE SBC PROJECT 142/013-01-2013-06

FIGURE 4A
PROPOSED PROCESS HYDRAULIC PROFILE
1 OF 3

ENGINEERS, INC.
3354 PERIMETER HILL DRIVE, SUITE 140
NASHVILLE, TN 37211
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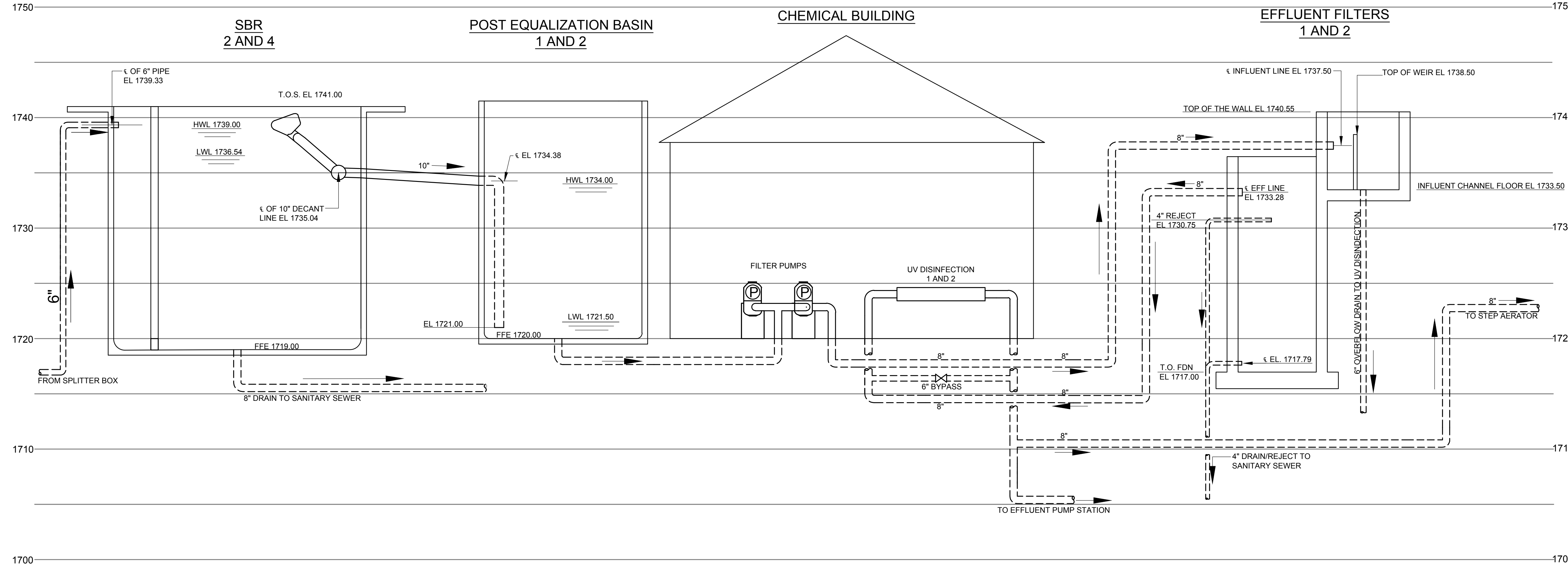
FIGURE NO.
FIGURE 4A



STEP AERATOR

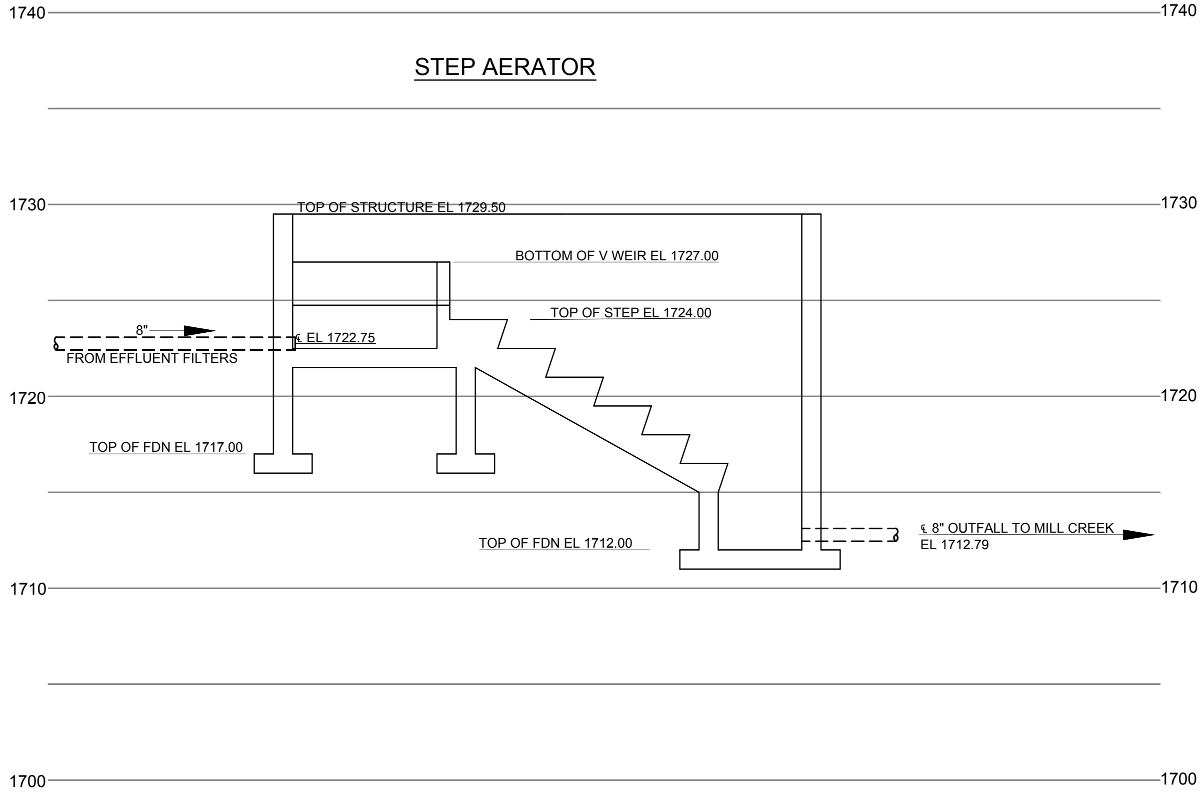


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FIGURE 4B
PROPOSED PROCESS HYDRAULIC PROFILE
2 OF 3

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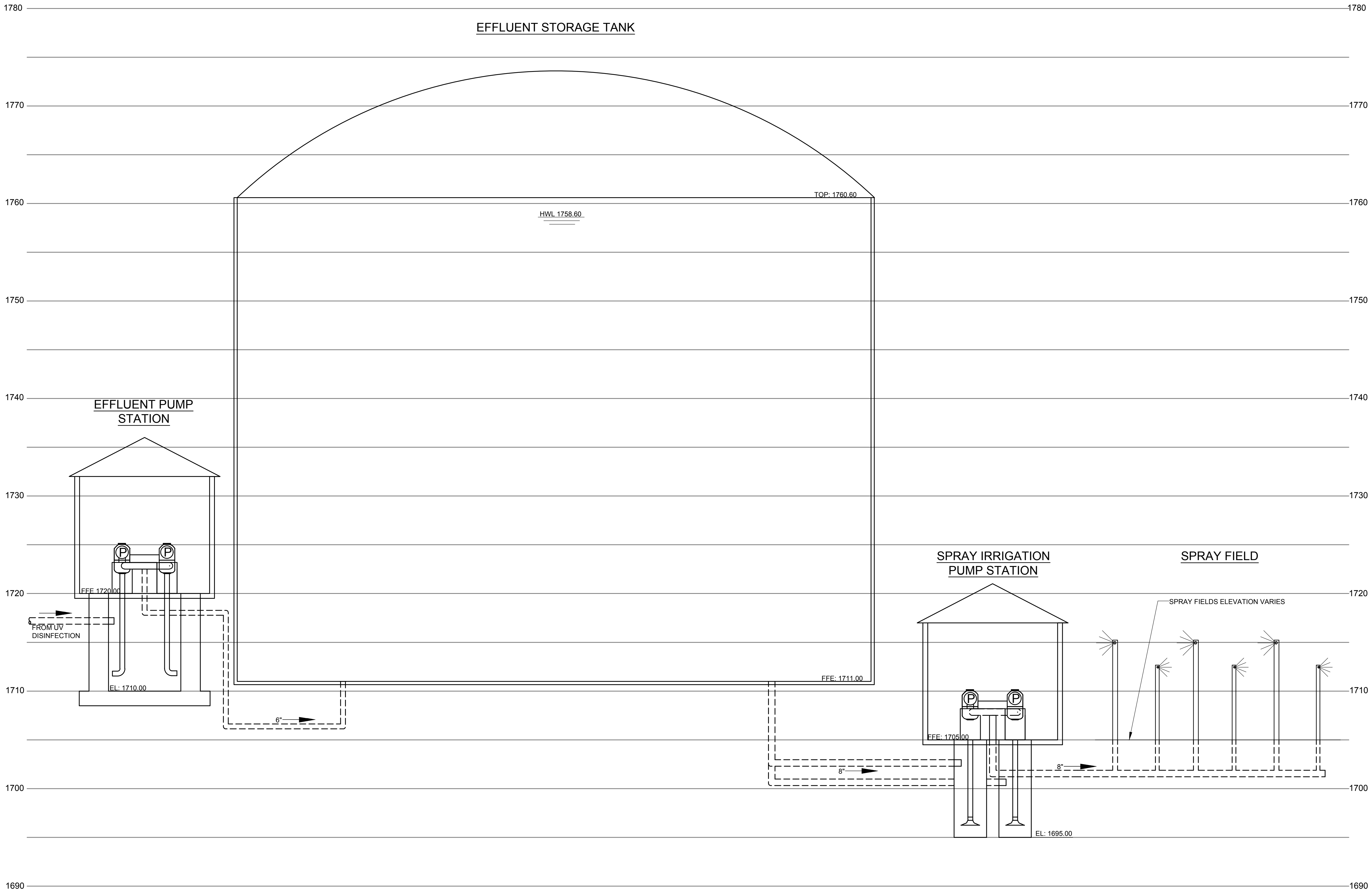
FIGURE 4B



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WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX
PIKEVILLE, BLEDSOECOUNTY, TENNESSEE SBC PROJECT 142/013-01-2013-06

FIGURE 4C
PROPOSED SLUDGE HYDRAULIC PROFILE
3 OF 3

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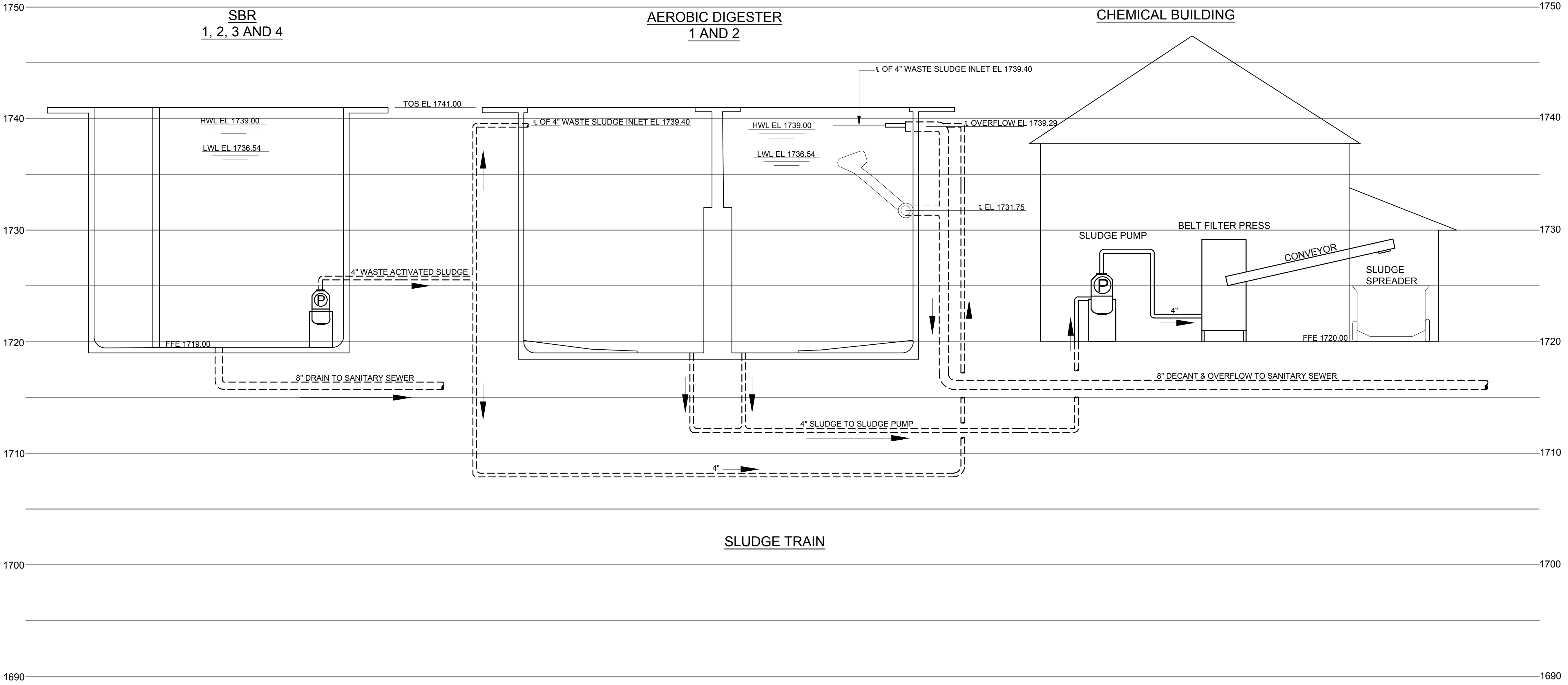
FIGURE 4C



SLUDGE TRAIN

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WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX
PIKEVILLE, BLEDSOECOUNTY, TENNESSEE SBC PROJECT 142/013-01-2013-06

FIGURE 4D
PROPOSED SLUDGE HYDRAULIC PROFILE

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FIGURE 4D

E) Install scum removal systems in both SBRs. Filamentous bacteria has been problematic in the basins, which then disrupt the digester operation. The operators have made significant headway in eliminating the bacteria; however, the scum removal systems will assist further if or when this problem develops again.

F) The dissolved oxygen (DO) levels in the filter influent flow are too high and inhibit nitrogen removal. The cause for the DO levels appears to be twofold. First, the drop pipe from the SBR to the post equalization basin may entrain air and increase the DO level. Second, the filter influent pump pulls air into the flow stream when the post equalization basin approaches empty. These corrections will be incorporated into the WWTP expansion.

G) The influent mechanical bar screen allows small solids to enter the WWTP. These consist of individual serving packets for condiments, which are used extensively in the facility, as well as other small items of inorganic composition. If the effluent filters back-up, many of these items are passed through the overflow and are then discharged into Mill Creek. In addition to the discharge violations, which have not yet been noted, these items also have the potential to damage the operation of existing equipment such as the air lifts within the effluent filters. These fines sometimes block the sand cleaners in the filters and cause back-ups, which then overflow to the bypass. During the expansion of the WWTP, it is recommended that the existing mechanical bar screen be converted to a perforated plate mechanical screen to capture the fine materials that enter the WWTP. The additional solids to be removed will necessitate the replacement of the screenings compactor and conveyor.

2. Influent Flow Equalization

The existing WWTP was designed and constructed based on an average daily flow (ADF) of 315,000 gallons. Since the initiation of operation, the average daily flow has been 355,000 gallons. This discrepancy between design flow and actual flow is further exacerbated by the influent flow pattern of higher flows over a portion of the day followed by a period of much lower flows during the night. These diurnal fluctuations are both pronounced and regular. It is assumed that the flow patterns specific to this facility will remain the same at the expanded capacity.

Lord and Company, Inc. furnished and installed the instrumentation system for the facility and has provided further system service during the operation of the WWTP. This summer, Lord and Company was able to extract influent flow data for several weeks. This data provided the influent flow rate at intervals of 42 minutes for the period from July 9, 2017 through August 5, 2017. The data provided for the week of July 16 - July 22, 2017 appeared to have several blocks of invalid data and that week was excluded from further calculations. The flow data provided for the remaining three (3) weeks was in close agreement with the daily and weekly flows reported by the WWTP on its Monthly Operation Reports for July and August 2017.

The flow rate of 0.200 mgd appeared to be near the middle of the daily swings between higher and lower flows. Each of the weeks was analyzed for the amount of flow and time both above this rate and below this rate. All of the weeks were very similar but the data was averaged over the three (3) weeks with the following results:

85.72% of the flow occurred during 63.93% of the day (15.34 hours)
ADF existing - 0.315 mgd: 0.270 MG in 15.34 hours (0.422 mgd rate)
PDF existing - 0.425 mgd: 0.364 MG in 15.34 hours (0.570 mgd rate)
ADF proposed - 0.630 mgd: 0.540 MG in 15.34 hours (0.845 mgd rate)
PDF proposed - 0.850 mgd: 0.728MG in 15.34 hours (1.140 mgd rate)

The amount of influent equalization volume required may be calculated from the future peak daily flow. Peak daily flow rate in multiplied by the time over which it occurs minus the treatment capacity over the same period equals the minimum required influent equalization volume.
Q Peak = 0.850 mgd = 590 gpm
Q Peak Rate = 1.14 mgd = 792 gpm
Minimum volume = 792(15.34)(60) - 590(15.34)(60) = 185,921 gallons

Working through this calculation from the opposite side indicates the equalization volume may be treated during the lower flow period.
24 - 15.34 = 8.66 hours
100% - 85.72% = 14.28%
14.28% of Q Peak = 0.850(0.1428) = 0.121 mgd
Q Peak Rate Night = (0.121/8.66)24 = 0.335 mgd = 233 gpm
185,921 + 233(8.66)(60) - 590(8.66)(60) = 424 gallons (This volume above 0 is attributable to rounding errors and is negligible

These calculations indicate the minimum required influent equalization basin volume is approximately 186,000 gallons. However, the estimation of sewage flows and the rates at which those flows occur is not exact and are not so easily predictable in the real world. Operational issues may also impact the capacity of the WWTP to treat the accumulated equalization volume. Therefore, additional influent equalization volume is recommended as a buffer and as a safety factor. The recommended volume is approximately 150% of the calculated volume or 280,000 gallons.

2.A. Equalization Influent Pump Station

Replace the two (2) existing influent sewage pumps and their variable frequency drives. Install two (2) new influent sewage pumps rated at 830 gpm (1.2 mgd) due to the higher daytime flows. The estimated peak daytime flow rate was estimated at 1.14 mgd in the calculations for the influent equalization basin. If the influent flow rate exceeds 830 gpm (1.14 mgd), allow both pumps to operate in parallel. The preliminary design calculations for the new pumps and force main are contained in Appendix A. New variable frequency drives will be required to allow the pumped flow rate to vary with the actual influent flow.

2.B. Equalization Influent Force Main

The existing 6” influent force main extends from the influent pump station to the flow splitter box adjacent to the SBRs. The existing force main does not have sufficient capacity for the proposed future design flow. Install a new 8” force main from the existing influent pump station’s new pumps to the proposed flow equalization basin. Preliminary design calculations for the proposed force main are contained in Appendix A.

2.C. Influent Equalization Basin

Construct a new aerated influent flow equalization basin with two (2) compartments. The influent equalization basin will be divided into two (2) compartments by the construction of a center division wall. This will allow half the tank to be drained for cleaning or repair and allow operational flexibility with half the volume available for use. The influent and effluent piping will be installed to allow service to either or both cells. The influent equalization basin will consist of a prestressed concrete tank 59 feet in diameter with a maximum water depth of 14 feet (16 feet wall height). A jet aeration system will be installed to maintain the dissolved oxygen levels and to stir the contents. The jet aeration system will be sized greater than the TDEC design minimum requirement to maintain 1.0 mg/l dissolved oxygen. The 10-States Standards establish the minimum air requirement at 1.25 cfm/1000 gallons or 350 cfm. The blowers, pumps, and piping which comprise the jet aeration system will be installed adjacent to the basin with all piping insulated. This installation will be similar to the existing aerobic digester.

3. SBR Influent Pump Station

Construct an SBR influent pump station adjacent to the aerated flow equalization basin. The pump station will contain two (2) sewage pumps and it will pump raw sewage from the equalization basin to both the existing and proposed flow splitter boxes at the average design flow of 440 gpm (0.630 mgd) and at the peak design flow of 590 gpm (0.850 mgd). An 8" force main will be installed from the new influent pump station to the existing 6" piping which enters the existing splitter box and to the proposed splitter box. Variable frequency drives will be used to modulate flows from the pump station. A masonry building will be constructed to house the pump station and its associated electrical devices. Preliminary design calculations for the SBR Influent Pumps and Piping are presented in Appendix B.

4. Proposed Wastewater Treatment Plant

Prior to the construction of the existing wastewater treatment plant, GRW Engineers, Inc. compiled the Design Report for Southeast Regional Correctional Facility (Bledsoe County Prison) Wastewater Treatment Plant in August 2009. Several of the core process components for the proposed wastewater treatment plant will duplicate those provided in the original design. Individual unit processes to be duplicated will rely on the calculations and computations contained in that Design Report, which is included herein as Appendix C. Following is a list of the facilities to be duplicated:

- Splitter Box
- Sequencing Batch Reactors (SBRs)
- Aerobic Digester
- Effluent Sand Filters
- Disinfection System

The original Design Report also presented calculations for the Belt Filter Press and its ancillary facilities. The calculations indicate the Belt Filter Press will process the sludge for the 0.315 mgd WWTP in 8 days each month. With the WWTP capacity doubled (0.630 mgd), the Belt Filter Press will require 8 x 2 = 16 days per month. The Belt Filter Press will not be replaced in this project.

Several distinctly different components of the WWTP were previously discussed and several more will be presented following this section; however, two (2) other internal components will also be revised and are discussed in this section.

4.A. Post Equalization Basin

The existing Post Equalization Basin does have sufficient volume for the existing WWTP. In order to provide sufficient volume in the post EQ basin for the next SBR decant, the filter influent pumps completely empty the post EQ basin and shut-off until the basin refills. The existing post EQ basin has a usable volume of approximately 28,000 gallons while a single SBR decant cycle is approximately 32,000 gallons at the WWTP design flow. The filter influent pumps have not been able to be “fine tuned” in time with the SBR cycles to the point that the pumps can continuously operate. Instead, the pumps empty the post EQ basin completely and shut-off. Attempts to eliminate this manner of operation have resulted in backing-up decant in the SBRs. The effluent sand filters should operate under steady-state conditions with only nominal flow variation. With the constant cycling of the filter influent pumps, the filters cannot be expected to perform as they should to remove nutrients.

Calculations for the expanded WWTP Post Equalization Basin are included as Appendix D. The existing post EQ volume is approximately 28,000 gallons and the recommended additional volume is approximately 61,750 gallons for a total post EQ basin volume of approximately 89,750 gallons. The existing and proposed post EQ basins will be interconnected to allow the total storage to be utilized by all the SBR basins. The increased volume will allow relatively constant flow to the existing and proposed effluent filters, which will enhance nutrient removal.

The existing post EQ basin exhibits increased dissolved oxygen (DO) levels from the levels measured in the SBRs. It appears that the SBR decant piping entrains air and this mixes into the waste stream as the fall occurs into the post EQ basin and raises DO levels. Additionally, the filter pumps create vortices in the post EQ basin at low levels, which also increases air entrainment. To prevent air entrainment, the internal drop pipes will be eliminated and an upturned 90-degree flange and flare will be installed in each post equalization basin (existing and proposed) at each pipe inlet. The outlets will be equipped with an anti-vortex device and the low level shut-off elevation for the pumps will be raised.

4.B. Effluent Filter Pumps

The effluent filter pumps were designed to accommodate feed to the three (3) existing effluent sand filters. These pumps remain adequate to accommodate both the three (3) existing and the three (3) proposed effluent sand filters. The existing pumps and the associated valves and piping will be reused and will be capable of the peak daily flow of 850,000 gallons (590 gpm). Variable frequency drives will remain to allow the pumps to provide service at both the current and future flow rates. Design calculations for the effluent filter pumps are included as Appendix E.

5. Effluent Pump Station

After the WWTP effluent is disinfected utilizing ultraviolet light, the current average design flow (0.315 mgd) to current peak design flow (0.425 mgd) will exit the plant via the existing step aerator and will be discharged into Mill Creek. The remaining average design flow (0.315 mgd) to peak design flow (0.425 mgd) will be transported to the effluent storage tank. Due to the elevation differences between the effluent sand filters and the effluent storage tank, another pump station will be required. Preliminary design calculations for the effluent pumps and force main are included in Appendix F.

6. Effluent Storage Tank

Wastewater effluent disposal by spray irrigation can neither be continuous nor constant. Conditions will exist when wastewater cannot be applied or when the rate of application must be decreased. Spray irrigation cannot be performed when the ground is frozen or when the ground is saturated. Consequently, effluent storage must be provided. TDEC requires minimum effluent storage of at least 60 days at design flow. The volume required for this facility is thus:

Volume = (315,000 gallons/day) (60 days) = 18,900,000 gallons.

A prestressed concrete tank will be constructed for effluent storage. The diameter of the tank will be limited to 260 feet, which is the practical maximum diameter to accommodate a free-span dome roof. Larger diameters require flat roofs with interior columns that are more expensive than the free-span domes. An open tank is not a feasible option due to its promotion of algae growth.

Determine water depth:

$V = \pi \, (D^2/4) \, H$

(18,900,000 gallons) (ft^3/7.48 gallons) = $\pi \, [(260)^2/4] \, H$

H = 47.6 feet (Allow 2 feet of freeboard for a total depth to overflow of 49.6 feet.)

A sodium hypochlorite (bleach) disinfection system will be provided as a standby system for the effluent storage tank. This will be utilized in the event of failure by the treatment system to provide adequate treatment, UV system failure, or other catastrophe to protect the tank from excessive odors and bacteria production.

The supply source for the non-potable water system will be relocated from the step aerator to the effluent storage tank. This will greatly reduce, if not completely eliminate, the potential for the non-potable supply to be depleted.

7. Spray Irrigation Pump Station

An effluent pump station will be required to pump the effluent to the spray irrigation field(s). The pump station will be equipped with multiple vertical turbine pumps with redundant capacity, check valves and isolation valves, strainers or screens to protect the spray nozzles from any particulates, and a flow meter to track and document the amount applied. The spray irrigation pumps will be equipped with variable frequency drives to allow variable pumping rates. The effluent pump station will be housed in a building along with the required electrical devices. Design calculations for the spray irrigation pump station and for the effluent piping will be provided when the final spray field layout has been determined.

8. Land Application of Effluent

Wastewater effluent to be land applied by spray irrigation at BCCX will receive the same level of treatment as the effluent discharged into Mill Creek. Therefore, neither nitrogen loading nor organic loading will be a limiting issue in determining the amount of land required for disposal. However, calculations must be made which account for the effects of precipitation and evaporation. These calculations are presented in the following Table 5.

Table 5

Spray Field Water Balance Calculations

Month	**P_R(in/mo)**	**PET (in/mo)**	**L_{WH} (in/mo)**	**L_{WH} (gpd/ft^2)**
January	7.62	0.10	4.68	0.0941
February	6.72	0.27	5.75	0.1280
March	8.85	0.97	4.32	0.0869
April	6.59	2.30	7.91	0.1644
May	6.13	3.59	9.66	0.1942
June	5.52	4.90	11.58	0.2406
July	6.85	5.44	10.79	0.2170
August	4.73	5.00	12.47	0.2507
September	5.54	3.79	10.45	0.2171
October	4.47	1.98	9.71	0.1952
November	6.11	0.82	6.91	0.1436
December	7.55	0.27	4.92	0.0989
Totals	**76.68 in/yr**	**29.43 in/yr**	**99.15 in/yr**	

P_R = 5-year Return Monthly Precipitation (from TDEC)

PET = Potential Evapotranspiration (from TDEC)

Perc = Application Rate = 0.25 gpd/ft^2 = 146.4 in/year = 12.20 in/mo

L_{WH} = (PET + Perc) - P_R

Area = Q_Y C / L_{WD}

Q_Y = Flow, MG/year

L_{WD} = Design Hydraulic Loading Rate, in/year = ΣL_{WH}

C = 36.83 (conversion factor)

Area = (0.315 mgd) (365 days/year) (36.83) / (99.15 in/year) = 42.71 acres

While these calculations indicate that 42.71 acres are required for the effluent spray irrigation, the calculations do not include any area to compensate for spray field irregular shapes due to soils types (which create “dead” space); do not include any extra area to allow the spray areas to “rest”; and do not include any additional area to dispose of stored effluent. Since the project requires 60 days of storage volume, sufficient application area must be included to allow the disposal of the daily flow plus allow the disposal of the stored volume in a reasonable amount of time.

A preliminary soils investigation was conducted on several areas on the BCCX property to determine the most likely areas suitable for surface spray irrigation. None of the areas in the preliminary investigation contained the full amount of contiguous site that was expected to be suitable; however, the largest and apparently most suitable sites were located west of the existing WWTP both north and south of the TVA easement. Both a topographic survey and Extra High Intensity Soils Mapping have been performed on approximately 113 acres in two (2) areas with a net useable area of approximately 65.8 acres. The Area Numbers were assigned based on the areas named for the topographic survey. Area 1 was the WWTP site and Areas 2 and 3 were the areas investigated for soils. The soils areas are listed in the table following and are shown on the drawings (Figures 5 and 6) which also follow:

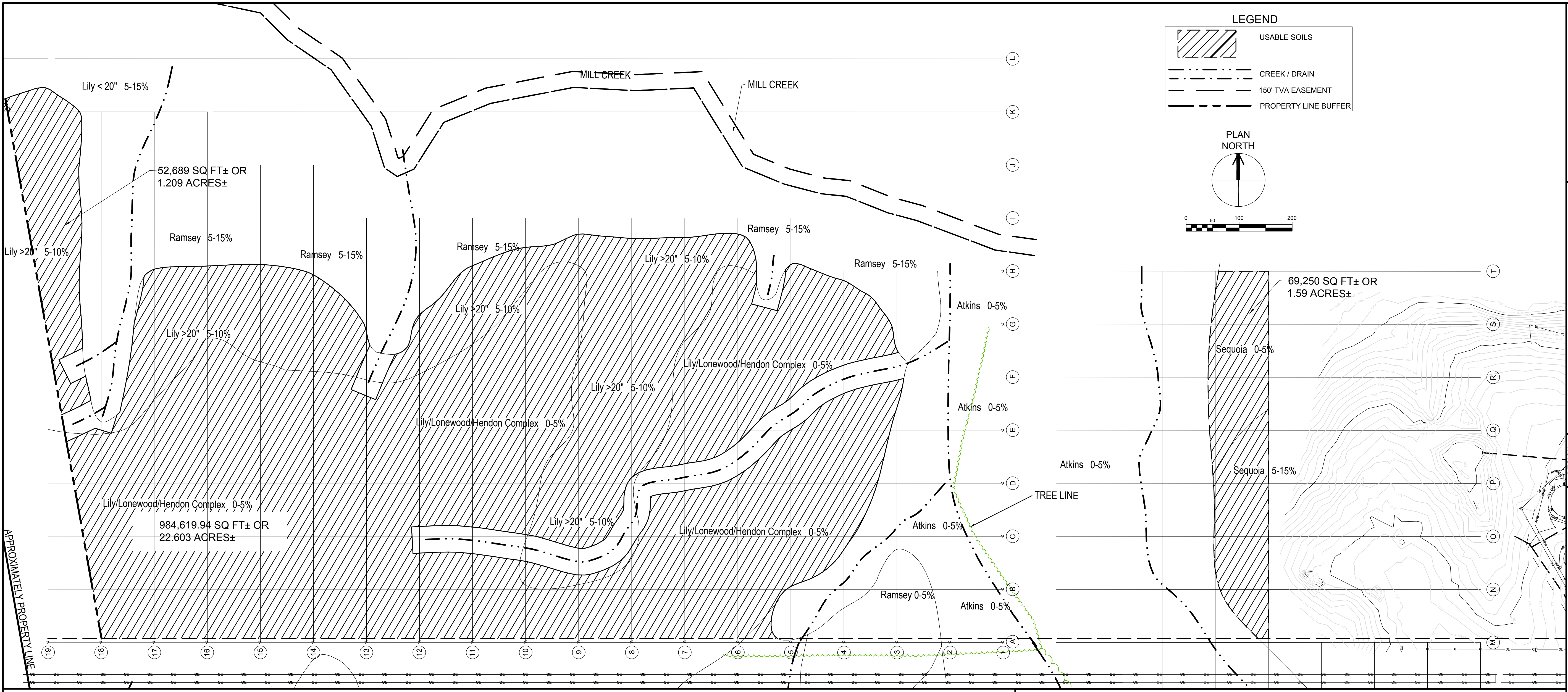
Table 6
Soils Areas Investigated

Area Number	Gross Area (Acres)	Net Area (Acres)
2	36	23.8
3	77	42.0
Totals	113	65.8

The results of the Extra High Intensity Soils Mapping for both these areas are included as Appendix G. These results were provided by the soils scientist and were confirmed in the field with TDEC as the mapping was compiled.

A supplemental soils area containing approximately 66.1 acres has been approved by the Owner for the required topographic survey and grid staking and the preparation of Extra High Intensity Soils Mapping. The required services have been procured and the survey can begin as soon as the surveyor can schedule the work to be followed by the soils investigation. It is anticipated that sufficient additional soils will be located in this area to allow a more efficient and effective layout of the spray irrigation system, provide additional area for the application of potential stored effluent, and also identify areas that may be held in reserve for future use. Detailed design information for the spray irrigation system will be provided when the final layout can be determined.

CTI PROJECT: N16003-01 Bledsoe County Correctional Complex (TN Dept. General Services TDEC), DRAWING: FIGURE 5 - EXTRA HIIGH INTENSITY SOIL MAPPING (536218v12/3/29/18 3:48PM), LAYOUT: SOIL GIRD 1



SOIL NAME SLOPE CLASS	LOADING RATE (G/D/FT^2)	DEPTH TO RESTRICTING LAYERS (INCHES)	SOIL IMPROVEMENT PRACTICES/ NOTES/ PERC STATUS
Lily, Hendon, Lonewood Complex, 0-5%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 0 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.
Lily, Hendon, Lonewood Complex, 5-10%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 5 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.

SOIL NAME SLOPE CLASS	LOADING RATE (G/D/FT^2)	DEPTH TO RESTRICTING LAYERS (INCHES)	SOIL IMPROVEMENT PRACTICES/ NOTES/ PERC STATUS
Ramsey, 0-5%	0.0	<20""	The Ramsey series consists of shallow and very shallow, somewhat excessively drained soils that formed in residuum or colluvium weathered from sandstone or quartzite. They are dominantly on plateaus and upper slopes of mountains. Runoff is medium to rapid and permeability is rapid. Slopes range from 3 to 70 percent.
Atkins, 0-5%	0.0	<20" TO REDOXIMORPHIC FEATURES	The Atkins series consists of very deep, poorly drained soils formed in acid alluvium washed from upland soils that formed in shale and sandstone. Permeability is slow to moderate. Slope ranges from 0 to 3 percent. Mean annual precipitation is about 46 inches and the mean annual air temperature is about 54 degrees F.
Sequoia, 0-15%	0.25	>20"	Sequoia soils have formed in residuum from shale and are typically 40 inches or more to auger refusal. Subsoils are clayey and somewhat poorly structured with red and yellow mottling at depths below 24 inches.

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WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX
PIKEVILLE, BLEDSOE COUNTY, TENNESSEE SBC 142/013-01-2013-06

FIGURE 5
EXTRA HIGH INTENSITY SOIL MAPPING
AREA 2

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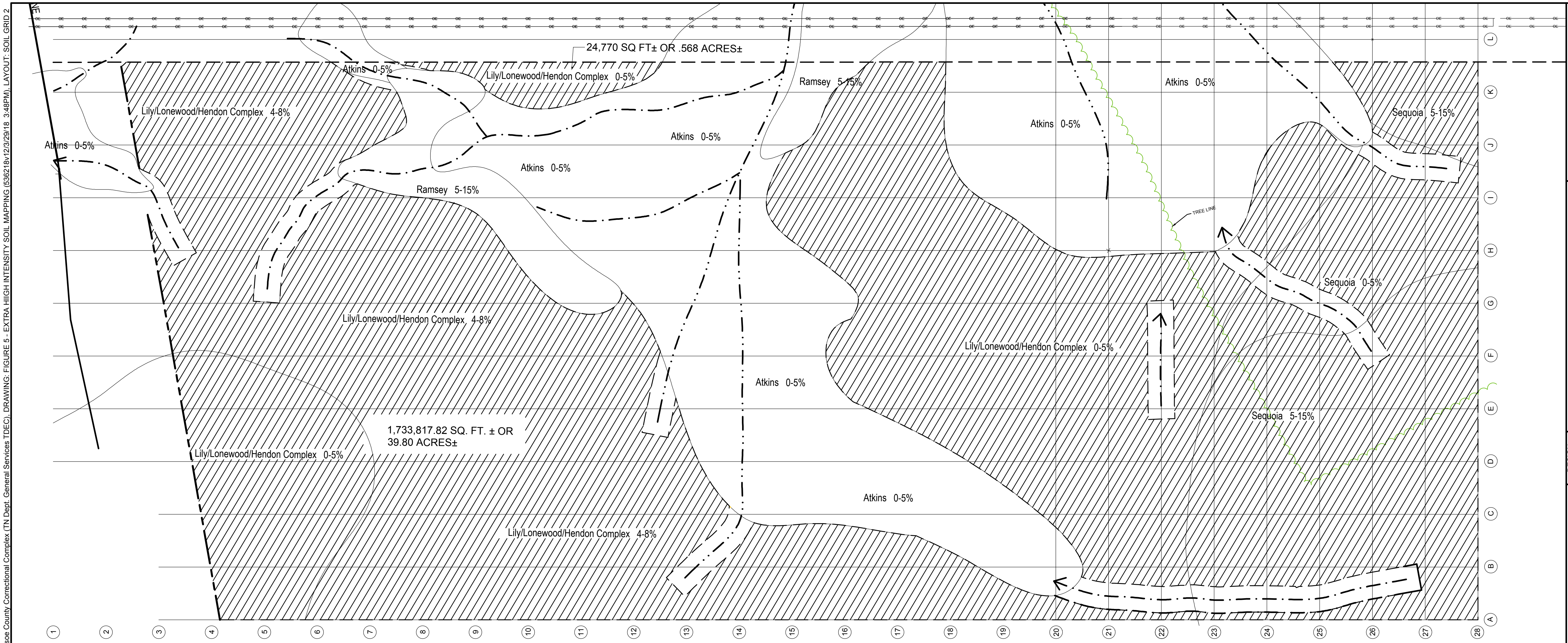
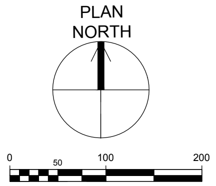
FIGURE 5



SOIL NAME SLOPE CLASS	LOADING RATE (G/D/FT^2)	DEPTH TO RESTRICTING LAYERS (INCHES)	SOIL IMPROVEMENT PRACTICES/ NOTES/ PERC STATUS
Lily, Hendon, Lonewood Complex, 0-5%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 0 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.
Lily, Hendon, Lonewood Complex, 5-10%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 5 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.

SOIL NAME SLOPE CLASS	LOADING RATE (G/D/FT^2)	DEPTH TO RESTRICTING LAYERS (INCHES)	SOIL IMPROVEMENT PRACTICES/ NOTES/ PERC STATUS
Ramsey, 0-5%	0.0	<20""	The Ramsey series consists of shallow and very shallow, somewhat excessively drained soils that formed in residuum or colluvium weathered from sandstone or quartzite. They are dominantly on plateaus and upper slopes of mountains. Runoff is medium to rapid and permeability is rapid. Slopes range from 3 to 70 percent.
Atkins, 0-5%	0.0	<20" TO REDOXIMORPHIC FEATURES	The Atkins series consists of very deep, poorly drained soils formed in acid alluvium washed from upland soils that formed in shale and sandstone. Permeability is slow to moderate. Slope ranges from 0 to 3 percent. Mean annual precipitation is about 46 inches and the mean annual air temperature is about 54 degrees F.
Sequoia, 0-15%	0.25	>20"	Sequoia soils have formed in residuum from shale and are typically 40 inches or more to auger refusal. Subsoils are clayey and somewhat poorly structured with red and yellow mottling at depths below 24 inches.



LEGEND

▨ USABLE SOILS
— · · — · · — CREEK / DRAIN
— — — 150' TVA EASEMENT
— · — · — PROPERTY LINE BUFFER

REVISIONS

NO.	DESCRIPTIONS	DATE	BY	APP'D

DESIGN NH
DRAWN JRH
CHECKED NH
APPROVED NH

WASTEWATER TREATMENT PLANT EXPANSION
BLEDSOE COUNTY CORRECTIONAL COMPLEX
PIKEVILLE, BLEDSOE COUNTY, TENNESSEE SBC 142/013-01-2013-06

FIGURE 6
EXTRA HIGH INTENSITY SOIL MAPPING
AREA 3

ENGINEERS, INC.
3354 PERIMETER HILL DRIVE, SUITE 140
NASHVILLE, TN 37211
615-834-8300

JOB NO.
N16003-01
ISSUE DATE
MARCH 2018
SCALE
AS SHOWN
DRAWING NO.
FIGURE 6

Appendix A

Proposed Influent Equalization Pumps

And

Force Main Calculations

Appendix A
Proposed Influent Equalization Pumps and Force Main Calculations

The existing influent pumps and force main will be replaced to accommodate the expanded capacity. The raw sewage will be pumped into the proposed influent equalization basin.

FLOW: Average Daily Flow = 0.630 mgd (440 gpm)
Peak Daily Flow = 0.850 mgd (590 gpm)
Peak Flow Rate from Equalization Basin Calculations = 1.14 mgd (800 gpm)
Use Peak Flow Rate = 1.20 mgd (830 gpm)
For any flows in excess of 830 gpm, allow both pumps to operate simultaneously.

MAXIMUM STATIC HEAD: EQ Basin HWL 1729.0
Wetwell Bottom <u>1682.0</u>
47.0 feet

FORCE MAIN: 335 feet of 8” (C = 120)
0.630 mgd H_f = (4.6’/1000’)(335’) = 1.54 feet
0.850 mgd H_f = (8.1’/1000’)(335’) = 2.71 feet
1.200 mgd H_f = (15.1’/1000’)(335’) = 5.06 feet

MINOR LOSSES: Assume minor losses = 10 feet (Conservative)

TOTAL DYNAMIC HEAD: 0.630 mgd 440 gpm @ 58.5 feet
0.850 mgd 590 gpm @ 59.7 feet
1.200 mgd 830 gpm @ 62.1 feet

Typical Pump Curve attached for these design conditions. 25 horsepower motor required.

Appendix B

Proposed SBR Influent Pumps

And

Piping Calculations

Appendix B
Proposed SBR Influent Pumps and Piping Calculations

The raw sewage will be pumped from the proposed influent equalization basin to the splitter boxes adjacent to the SBRs. Pumps to be used will be similar to the self priming pumps to be used for the influent equalization pumps.

FLOW: Average Daily Flow = 0.630 mgd (440 gpm)
Peak Daily Flow = 0.850 mgd (590 gpm)
If it should ever be necessary to pump in excess of 590 gpm, allow both pumps to operate simultaneously.

MAXIMUM STATIC HEAD:	Splitter Box HWL	1741.26
	EQ Basin Bottom	1715.00
		26.26 feet

Suction Piping: 75 feet ~ 8”
Discharge Piping 450 feet ~ 8” and 40 feet ~ 6” (To furthest splitter box - existing)

FRICTION LOSSES: 525 feet ~ 8” and 40 feet ~ 6” (C = 120)
0.630 mgd H_f = (4.6’/1000’)(525’) + (18.9’/1000’)(40) = 3.2 feet
0.850 mgd H_f = (8.1’/1000’)(525’) + (32.5’/1000’)(40) = 5.6 feet

MINOR LOSSES: Assume minor losses = 10 feet (Conservative)

TOTAL DYNAMIC HEAD:	0.630 mgd	440 gpm @ 39.5 feet
	0.850 mgd	590 gpm @ 41.9 feet

Typical Pump Curve attached for these design conditions. 15 horsepower motor required.

Appendix C

Design Report for Southeast Regional Correctional Facility (Bledsoe County Prison) Wastewater Treatment Plant by GRW Engineers, Inc.

DESIGN REPORT
FOR
SOUTHEAST REGIONAL CORRECTIONAL FACILITY
(BLEDSOE COUNTY PRISON)
WASTEWATER TREATMENT PLANT
SBC Project No. 142/013-02-2004-10

PREPARED FOR THE
TENNESSEE DEPARTMENT OF FINANCE AND ADMINISTRATION
ON BEHALF OF THE
TENNESSEE DEPARTMENT OF CORRECTION

AUGUST 2009

ROBERT G. THREADGILL, JR.
NO. 012645

Robert G. Threadgill Jr
10/20/2009

PREPARED BY:

GRW ENGINEERS, INC.
404 BNA DRIVE, SUITE 201
NASHVILLE TN 37217

DESIGN REPORT
FOR
SOUTHEAST REGIONAL CORRECTIONAL FACILITY
(BLEDSOE COUNTY PRISON)

I. GENERAL INFORMATION

A. Population to Be Served

The expansion of the Southeast Regional Correctional Facility (SRCF) will require expansion of the existing wastewater treatment plant. The existing treatment plant consists of 3 parallel package units. The first unit was constructed in 1979 as a 45,000 GPD package plant. The prison officially opened in 1980 and three (3) years later in 1983, two additional package units were added when the treatment facility was enlarged to the present size of 180,000 GPD. All three units have been in operation for a little over 20 years and are in need of some maintenance and/or modification. The original 2004 planned expansion of the prison was to be in the area of the existing treatment units requiring that the package units be demolished and/or relocated prior to the expansion of the correctional facility. The new 2008 plan for the prison expansion will occur with the construction of a second facility adjacent to the existing prison. The inmate population for the new prison facility is projected to increase approximately 1,440 men over the existing population of 980 for a total of 2,420 inmates.

The existing SRCF prison staff is approximately 320 persons and divided into two (2) groups; security and support. There are 3-shifts per 24 hours seven days a week. The actual number of maintenance and clerical staff (support) is reduced during weekends, holidays and nights. The existing prison has approximately 240 staff members at work during each 24-hour weekday. The staff is projected to increase to approximately 800 persons with the addition of the new prison facilities. The staff working in a 24-hour period will be approximately 530.

B. Flow Rates for Design

Discussion with Department of Correction's staff indicates that the average flow from prison inmates has been seen as high as 150 GPD per inmate (includes staff) in other facilities in Tennessee. The actual observed flow per inmate was determined to be approximately 95 GPD at the existing SRCF complex. Thus, a conservative flow rate value of 115 GPD per inmate will be used for the expansion of this facility. A flow rate of 30 GPD per staff member will be used to determine staff total flow. The projected flow based on inmate and staff average usage for the expanded correctional facility will therefore be approximately:

Staff – 530 x 30 = 15,900 GPD
Inmates - 2,420 x 115 = 278,300 GPD
Total projected flow = 294,200 GPD

The Taft Youth Center (TYC) wastewater plant is located approximately 10,000 feet from the SRCF and has a design capacity of 100,000 GPD. The TYC plant is too small to handle this flow, thus, pumping to TYC for treatment of the wastewater is not an option without increasing the capacity. The TYC WWTP is over 40 years old, thus, enlarging the facility to treat the flow from SRCF is not recommended. The construction of a new treatment plant will therefore be required to handle this flow. Due to the proximity of the TYC WWTP and the age, any new WWTP constructed in the area should include capacity to eliminate the TYC WWTP.

The TYC has capacity for approximately 156 youths (beds) and a staff of 120. The maximum staff at the facility in a 24 hour period is approximately 90. Using the same flow value for youth and staff as the adult prison the expected daily flow from this facility should be [(156 x 115) + (90 x 30)] 20,640 GPD at capacity. The daily average flow at this facility for the last few years has been approximately 27,500 GPD even though the current youth population is approximately 90. This high flow is due to an aging collection system [built in the nineteen sixties (1960's)] that allows infiltration and large amounts of inflow into the system. The TYC collection system has serious infiltration and inflow problems. The maximum recorded flow (118,000 GPD) seen at this facility is approximately 4.3 times the average daily flow of 27,500 GPD. It is assumed that storm sewers or roof drains may be connected to the sanitary sewer. Normally the average peak daily flow seen at the TYC WWTP during rain events is approximately 58,000 GPD. Thus for this project we have assumed an inflow value of (58,000 -27,500) 30,500 GPD. As part of this design, GRW Engineers has been instructed to evaluate the collection system and determine possible corrective action that need to be taken at the Taft facility to insure that I/I is removed.

Adding the design capacity flow from the TYC will increase the daily design flow to approximately (20,640 + 294,200) 314,840 GPD (use 315,000 GPD). The SRCF complex has some problems with infiltration/inflow, but they are considered minor in comparison to TYC. The annual daily flow for SRCF (in the last 12 month period) averages 167,000 GPD with peak average monthly flows of up to 201,000 GPD (approximately 21,000 GPD over the design capacity). The average peak daily flow for SRCF is approximately 243,000 GPD, thus the inflow value would be approximately (243,000 – 167,000) 76,000 GPD. TDOC has authorized GRW Engineers to have the existing sewer lines smoke tested and internally inspected to locate I/I sources. This evaluation is ongoing and rehabilitation work will be forth coming to repair and/or replace the problems found.

Based on the above discussion, flow equalization should be considered as part of any new treatment facility to allow for large hydraulic flows during the day or a large rain event. An additional hydraulic flow of (30,500 + 76,000) 110,000 GPD should be sufficient to handle I/I flow from TYC and SRCF. Sizing a new treatment plant at an organic loading rate of 315,000 GPD or 219 GPM and a hydraulic loading rate of up to 425,000 GPD (approximately 295 GPM) would require an equalization basin that could contain up to 110,000 gallons or a process unit that could handle an additional 110,000 GPD without preventing proper treatment of the organic loading. A basin 40 ft. long x 30 ft. wide x 13 ft. deep will hold the extra 110,000 gallons during a 24-hour period. This would allow a peak flow rate into the plant of up to 295 GPM for up to 24 hours with the plant operating at a flow rate of 219 GPM.

C. Proposed Effluent Standards for New WWTP

The State of Tennessee Division of Water Pollution Control established effluent standards for an increase flow of the existing treatment plant to 315,000 GPD. The standards set by the State were based on poundage per day of several parameters allowed to be discharged into Mill Creek.

Parameters	**Monthly Daily Average Loading (lbs/day)**	**Proposed Flow 315,000 GPD Concentration (mg/l)**
$CBOD_5$	30	11.4
NH_4 (summer)	2.2	1.3
NH_4 (winter)	3.2	1.8
Phosphorus (summer)	1.5	0.6
Total Nitrogen (summer)	7.5	2.9
Suspended Solids	45	17.1
Oil & Grease	---	50
pH	---	6.5 to 9.0
IC_{25}	---	100%
DO	---	6.0
Chlorine Residual	---	0.02
Settleable Solids	---	1.0 ml/l

D. Existing Environmental Conditions

Existing parameters that must be considered during the design are as follows:

1. The alkalinity of the wastewater coming into the treatment plant will be similar to the potable water used at the treatment plant. Originally the potable water to be used at SRCF was to come from Dayton, TN which has an average alkalinity of 61 mg/l. There are presently some discussions going on that the water may come from Spring City, TN which also draws water from the Tennessee River (water source for Dayton). We have assumed the alkalinity to be similar.
2. The average wastewater temperature ranges for the existing SRCF facility are:
    a. Winter – Low 12° C, High 20° C, Average 16° C
    b. Summer – Low 18° C, High 27° C, Average 24° C
3. Air temperature range 20° to 90° F
4. Elevation – 1700 feet

E. Design Influent Loading, Parameters and Flow Assumptions

1. BOD_5 loading concentration of 400mg/l
2. Suspended solids concentration of 400 mg/l (70 % volatile & 30% fixed)
3. TKN loading concentration of 60 mg/l
4. Total phosphorus concentration of 15 mg/l
5. Average Daily Flow of 315,000 GPD
6. Peak Daily Flow of 425,000 GPD

II. PLANT DESIGNED with SEQUENCING BATCH REACTORS (SBR)

A. SBR Design for Southeast Regional Correctional Facility

The effluent standards set by the State require carbonaceous oxidation, nitrification and nutrient removal. SBR treatment plants can be modified to accomplish all three types of biological removal in one tank. Typical SBR design parameters use F/M (food to microorganism) ratios ranges between 0.05 and 0.3. Since SBR units operate in batches to use the tank for clarification as well as aeration, at least two tanks are needed to allow continuous operation. The sludge volume index (SVI) design parameter is normally between 100 and 200 for a well settled activated sludge. SVI values below 75 settle too fast and leave pin flock. SVI values over 250 don't settle well and normally indicate a bulking sludge with high suspended solids in the effluent. **The empirical equations, coefficients and growth rates used in the evaluation of this design were obtained from two EPA publications, "Process Design Manual for Nitrogen Control" – October 1975 and "Nitrogen Control" - EPA/625/R- 93/010.**

The following design parameters will be used for the SRCF WWTP:

1. A two (2) tank SBR system will be designed to handle the average and peak flow conditions. Carbonaceous BOD5 removal, nitrification, some de-nitrification (nitrogen and phosphorus removal) and settling will be accomplished in the same tank.
2. Maintenance Flow (1 unit out of service) of 315,000 GPD
3. F/M ratio = 0.05 lbs. of BOD_5 / lb of MLSS / day
4. SVI – (after 30 minutes settling) = 150 ml/l
5. Deep bed sand filters will be used to accomplish additional nitrogen, phosphorus, settleable and suspended solids removal.
6. UV light radiation will be used to disinfect the wastewater to eliminate the chlorine residual limit in the effluent standards.

Total loading to the aeration basins (SBR units) will be approximately as shown below:
Lbs/day = 0.315 MGD x 8.34 x 400 mg/l
Lbs/day = 1050.84

The effluent limit established for this plant is 11.4 mg/l for the average daily flow. Using a safety factor of 2.0, the effluent should be (11.4/2.0) 5.7 mg/l. Thus, the total loading going out of the treatment plant into Mill Creek will be approximately (0.315 x 8.34 x 5.7) 14.97 lbs/day. Total BOD loading to the aeration basins will therefore be (1050.84 – 14.97) 1035.87 lbs per day. Each basin will be designed to handle half of the total loading or (1035.87/2) 517.94 lbs/day. Based on an F/M ratio of 0.05 lbs BOD5/lb-MLVSS, the biological solids content of each basin should be:

$$\frac{517.94 \text{ lbs/day}}{0.05 \text{ lbs} - \text{BOD} / \text{lb MLVSS}} = 10{,}358.7 \text{ lbs of MLVSS}$$

SBR's use settling in the same tankage as the biological reaction, thus, manufacturers normally design around the SVI index of 150. SVI is defined as the volume in milliliters occupied by one gram of activated sludge after settling 30 minutes. It is determined by dividing the milliliters of grams per liter of suspended solids (MLSS) in the activated sludge. Since 1000 mg = 1gram, the SVI of 150 has units of ml/g. Conversion of cubic feet per pound is determined as follows:

$$\frac{150 \text{ ml}}{\text{g}} \text{ x } \frac{453.524 \text{ g}}{\text{lb}} \text{ x } \frac{1 \text{ cu. ft.}}{28{,}316.8466 \text{ ml}} = \frac{2.402 \text{ cu. ft}}{\text{lb}}$$

Using the conversion value of the SVI value (2.402), the required volume in each aeration basin can be determined. As shown above the pounds of MLVSS per aeration basin was determined to be 10,358.7. Thus, the aeration basin volume needed is determined by multiplying the pounds of MLVSS by the converted SVI value as shown below:

10,358.7 lbs x 2.402 cu. ft/lb = 24,881.6 cu ft. of Biomass per aeration basin

B. Nitrogen Removal in SBR

1. Nitrification Process Discussion

Typically untreated wastewater contains little or no nitrites or nitrates, but is in the form of ammonia or organic nitrogen [both soluble and insoluble (solid) forms]. TKN is the sum of organic nitrogen; ammonia (NH_3) and ammonium (NH_4^+) in the chemical analysis of wastewater. Normally the total nitrogen content of wastewater is about 40% organic nitrogen (particulate) and 60% free ammonia. During biological treatment, most of the solid (or particulate) organic nitrogen is transformed to ammonium and other inorganic forms. Less than 30% of the total nitrogen is removed by conventional secondary treatment. Removing total nitrogen requires that the biological process include de-nitrification which can increase the removal rate to approximately 95%.

Nitrification of ammonium nitrogen is a two (2) step process involving two (2) different types of microorganisms:

1. Nitrosomonas
2. Nitrobacter

The 1^{st} step uses Nitrosomonas bacteria to convert ammonium to nitrite. The 2^{nd} step uses Nitrobacter organisms to convert nitrite to nitrate.

Once the ammonia nitrogen is reduced to nitrates, it must be converted to nitrogen gas by de-nitrification. This is done in an anoxic zone <u>or</u> stage (cycle) of a biological process. The wastewater (to be denitrified) must contain sufficient carbon (organic matter) to provide the energy source for the conversion of nitrate to nitrogen gas using a biological process. Reaching total nitrogen levels below 5.0 mg/l on a consistent basis using a biological process requires supplementing the microorganisms with another carbon source such as methanol.

The design loading of this plant assumes a TKN concentration of 60 mg/l. The ammonia nitrogen discharge limit is 1.3 mg/l in the summer and 1.8 mg/l in the winter. The State has set a total nitrogen limit of 2.9 mg/l. The total TKN concentration to be removed would be (60.0 – 1.3 – 2.9) 56 mg/l (0.315 x 56 x 8.34) or approximately 147.12 lbs/day.

The production of new heterotrophic microorganisms due to the removal of the BOD_5 loading will also remove nitrogen from the wastewater. Microorganisms in wastewater have a basic composition of:

$C_5H_7NO_2$

The atomic mass of the heterotrophic microorganism is:

C = carbon = 12 x 5 = 60
H = hydrogen = 1 x 7 = 7
N = nitrogen = 14 x 1 = 14
<u>O = oxygen = 16 x 2 = 32</u>
113

Nitrogen, therefore, makes up approximately (14/113) 0.124 x 100 = 12.4 % of the biomass produced by the growth of heterotrophic microorganisms. The observed mass of organisms formed per BOD_5 removed is dependent on the length of the mean cell-residence time (θc). The longer the residence time the lower the observed yield. The maximum yield coefficient (Y) varies dependant on the waste type. The coefficient Y has been shown to vary in a range between 0.4 to 0.8 mg-VSS/mg-BOD_5 and typically uses 0.6 VSS/mg-BOD_5 removed. The endogenous decay coefficient (Kd) for activated sludge varies between 0.025 to 0.075. The typical value used in calculations is 0.06. The empirical formula used to calculate the observed cell yield is:

$$\text{Yobs} = \frac{Y}{1 + \text{Kd}\ \theta c} = \frac{0.6}{1 + 0.06\ \theta c}$$

Typically the θc value must be greater than 7 days for nitrification to take place. GRW Engineers, Inc. will use a minimum of 20 days for the mean cell-residence time (θc) for this project. Thus the calculated cell yield (observed cell yield) will be [0.6/(1 + (0.06 x

20))] 0.273. The total nitrogen taken out of the incoming TKN concentration by the production of heterotrophic microorganisms will be approximately (assuming safety factor of 2.0 for effluent BOD_5 (11.4 / 2 = 5.7):

$$\frac{\text{lbs}}{\text{day}} = [(400 \text{ mg/l} - 5.7 \text{ mg/l}) \times 8.34 \times 0.315 \text{ MGD}] \times 0.273(\text{cell yield}) \times 0.124 \; (\%\text{N})$$

$$\frac{\text{lbs}}{\text{day}} = 35.07 \text{ (nitrogen removed by production of heterotrophic microorganisms)}$$

At the design flow of 0.315 MGD the nitrogen removed by heterotrophic microorganism growth, 35.07 lbs/day, is equivalent to 13 mg/l.

Thus, the total nitrogen loading (to be removed per day) for the plant is (147.12 – 35.07) 112.05 (use 112) lbs. Assuming that half is treated in each basin reduces the total nitrogen loading for assimilation per basin to 56 lbs. *Note: The nitrogen assimilation associated with nitrifying biomass growth is small in comparison to the heterotrophic microorganisms and therefore will not be considered in this evaluation. This biomass will be considered a safety factor in these calculations.*

The growth rate of the nitrifying organisms must be determined to calculate the amount of biomass needed to remove the nitrogen loading. The nitrification rate of wastewater requires evaluation of several parameters such as:

1. $CBOD_5$/TKN ratio,
2. Dissolved oxygen concentration,
3. Temperature (nitrification rate decreases with decrease in temperature); and
4. pH value

The growth rate of Nitrobacter organisms in wastewater is greater than Nitrosomonas, thus, the conversion of ammonium nitrogen to nitrite is consider the design limiting rate.

The average temperature during the summer months at the existing treatment plant was determined to be 24° C. The most stringent ammonia nitrogen discharge limit is 1.3 mg/l during the summer months with a total nitrogen limit of 2.9 mg/l. This evaluation makes the following assumptions during nitrification:

1. The minimum D.O. in the aeration basin will be 2.0 mg/l,
2. the minimum pH will be 7; and
3. the temperature will average 24° C in the summer and have a low of 12° C in the winter

An empirical formula used to determine the growth rate of Nitrosomonas nitrifiers (μ_{na}) adjusted for temperature, dissolved oxygen, ammonium-nitrogen concentration and pH is as follows:

$$\mu_{na} = \mu_n \times [\text{D.O.} / (K_{O2} + \text{DO})] \times [N/(K_n + N)] \times [1-0.833 \, (7.2 - \text{pH})]]$$

where;
μ_n = (maximum growth rate of Nitrosomonas over a temperature range of 5 – 30° C) $0.47e^{0.098(C°-15)}$
K_n = half saturation coefficient - typically used value is 1.0
K_{O2} = 1.0
N = ammonium-nitrogen concentration in the effluent (1.3 for summer)
pH = 7.0
DO = 2.0
T = 24° C in the summer

$$\mu_{na} = 0.47e^{0.098(24°-15)} \times [2.0 / (1.0 + 2.0)] \times [1.3/(1.0 + 1.3)] \times [1\text{-}0.833\ (7.2 - 7.0)]$$

$$\mu_{na} = 1.135 \times 0.667 \times 0.5652 \times 0.8334$$

$$\mu_{na} = 0.3566$$

μ_{na} = 0.3566 (adjusted Nitrosomonas growth rate)/day

Definitions:
Ammonium-nitrogen = NH_4-N
Nitrosomonas microorganisms (volatile suspended solids) = NVSS
q_n = nitrification rate - gram NH_4-N oxidized per gram of NVSS/day

The ammonium oxidation or nitrification rate is defined as the Nitrosomonas growth rate (μ_{na}) divided by the organism yield coefficient (Y_n – gram Nitrosomonas grown (NVSS) per gram NH_4-N removed).

$q_n = \frac{\mu_{na}}{Y_n}$ where; $Y_n = 0.1$ $q_n = \frac{0.3566}{0.1}$

q_n = 3.5661 grams NH_4-N oxidized /g NVSS/day

Since the grams cancel each other out, this same value can be used with lbs (instead of grams) that have been determined previously. The ammonium-nitrogen to be assimilated in each basin was shown above to be approximately 56 pounds per day. Thus, the total Nitrosomonas organisms needed in each basin would be:

lbs = 56 lbs/day NH_4-N x 3.5661 day = 199.7 lbs during the summer

The Nitrosomonas organisms are a very small fraction of the total VSS in each aeration basin. The fraction of nitrifiers in the aeration basin can be estimated by use of the BOD_5/TKN ratio. The empirical formula recommended determining this fraction is:

$$f = \frac{M_N}{M_N + M_C}$$

Where;
$M_N = Y_N (N_0 - N_1)$, N_0 = Influent TKN, mg/l & N_1 = Effluent TKN, mg/l
$M_C = Y_H (S_0 - S_1)$, S_0 = Influent carbon, mg/l & S_1= Effluent carbon, mg/l
Y_N = Net yield of Nitrosomonas – lbs of NVSS per lb of NH_4-N oxidized
Y_H = Net yield of heterotrophic – lbs of VSS per lb of Substrate removed

Taking into consideration the removal of nitrogen by heterotrophic organism growth, the influent N_0 is approximately (60 – 13) 47. The effluent NH_4-N has a limit of 1.3 mg/l in the summer and Y_N was taken to be 0.1; thus M_N is:

$$M_N = Y_N (N_0 - N_1) = 0.1 (47 - 1.3) \Rightarrow M_N = 4.57$$

Y_H (Net yield of heterotrophic cells) was previously determined to be approximately 0.273, thus M_C is:

$$M_C = Y_H (S_0 - S_1) = 0.273 (400 - 5.7) \Rightarrow M_C = 107.64$$

Therefore the fraction of Nitrosomonas organisms in the aeration basins would be approximately:

$$f = \frac{M_N}{M_N + M_C} = \frac{4.57}{4.57 + 107.64} \Rightarrow f = 0.04$$

The minimum mixed liquor volatile suspended solids (MLVSS) that could be acceptable for proper removal of the ammonium-nitrogen would be determined as follows:

NVSS = MLVSS x f
MLVSS = NVSS/f
MLVSS = 199.7/0.04 => MLVSS = 4,992.5 lbs

Since this is less than the proposed solids for each basin for removal of the carbon (BOD_5), there should be sufficient microorganisms for nitrification of the NH_4-N.

During the winter the temperature in the incoming wastewater will drop down to an average of 16° C and have an ammonium-nitrogen limit of 1.8. Assuming the rest of the factors (DO & pH) remain the same, the adjusted nitrifier growth rate will be:

$$\mu_{na} = 0.47e^{0.098(16^\circ-15)} \text{ x } [2.0 / (1.0 + 2.0)] \text{ x } [1.8/(1.0 + 1.8)] \text{ x } [1-0.833 (7.2 - 7.0)]]$$

$$\mu_{na} = 0.518 \text{ x } 0.667 \text{ x } 0.6429 \text{ x } 0.8334$$

$$\mu_{na} = 0.185 \text{ (adjusted Nitrosomonas growth rate)/day}$$

Therefore the ammonium oxidation or nitrification rate in the winter is:

$$q_n = \frac{\mu_{na}}{Y_n} \quad \text{where; } Y_n = 0.1 \qquad q_n = \frac{0.185}{0.1}$$

$q_n = 1.85$ days

The ammonium-nitrogen to be assimilated in each basin was shown above to be approximately 56 pounds per day. Thus, the total Nitrosomonas organisms needed in each basin would be:

lbs = 56 lbs/day NH_4-N x 1.85 days = 103.6 lbs during the winter

Since this is less than the pounds in the summer, there should be sufficient microorganisms for nitrification of the NH_4-N during the winter. The basin size should be set by the carbonaceous biological loading and removal required for the summer.

2. Ammonia-Nitrogen Assimilation and Alkalinity Consumption
   a. Nitrification

      As shown above the removal of ammonia-nitrogen will be accomplished biologically by the production of new heterotrophic microorganisms (containing approximately 12% nitrogen), the conversion of ammonia nitrogen to nitrite and finally the conversion of nitrite to nitrate. The production of heterotrophic microorganisms was shown to remove (assimilate) approximately 35.07 lbs per day of the ammonium-nitrogen content in the influent wastewater. The remaining 112 lbs per day (42.6 mg/l) must be removed by the nitrifying microorganisms (Nitrosomonas and Nitrobacter). The conversion by the microorganisms of ammonia-nitrogen to nitrite and nitrite to nitrate destroys approximately 7.14 mg for every 1 mg of ammonia-nitrogen. Thus the alkalinity destroyed by the nitrification process will be approximately (42.6 x 7.14) 304.16 mg/l. The potable water used at the new facility will have an alkalinity of approximately 60 mg/l, thus without chemical addition, all alkalinity will be destroyed (consumed). The destruction of the alkalinity in the wastewater would drop the pH levels below 7.0 which would reduce or halt the nitrification rate.

   b. De-nitrification

      If the oxygen levels in the aeration basin (SBR tanks) are reduced to near zero for a short period of time, facultative microorganisms (in the tank) will use nitrite and nitrate as an oxygen source and produce bicarbonate to recover some of the alkalinity lost by the nitrification process. Removing phosphorus from the influent wastewater stream biologically, will require a phase in the SBR cycle when the DO level is reduced to near zero. During this phase (non aeration) of the SBR cycle, both biological processes will occur (de-nitrification and biological removal of phosphorus). The basin contents are mixed by submersible propeller type mixers. The facultative microorganisms will take oxygen (needed for respiration) from nitrate and nitrite molecules. Reduction of the electron acceptors (oxygen, nitrate or nitrite) requires an electron donor (a carbon source). The donor can be organic substrate in the raw wastewater or a substrate added to the wastewater. Bicarbonate alkalinity is produced and carbonic acid concentration is reduced during de-nitrification. The theoretical stroichiometry of the bicarbonate alkalinity production is approximately 3.57 pounds of alkalinity as CaCO3 produced (recovered) for every pound of nitrate-nitrogen reduced (denitrified) to nitrogen gas.

This SBR design uses two (2) anoxic phases in each cycle to promote biological removal of total nitrogen and phosphorus. The EPA Manual on Nitrogen Control (page 284) indicates that SBR systems can achieve total nitrogen (TN) limits below 8.0 mg/l on a consistent basis, but close attention to operation is required. The proposed SBR system will be controlled by a PLC to insure proper operation of the cycles in each basin and each phase of the cycle. The manufacturer has indicated that this system should be able to produce an effluent with a maximum TN limit of approximately 5.0 mg/l.

c. Alkalinity Addition Needed

As discussed above approximately 304 mg/l of alkalinity will be destroyed each day to nitrify the ammonia nitrogen to nitrite at the design capacity of 315,000 GPD. Assuming that at least 90% of the remaining nitrite (42.6 mg/l) is converted to nitrate during the anoxic phase, the amount of alkalinity recovered during de-nitrification will be approximately:

Recovered alkalinity (RA) = 42.6 mg/l x 90% x 3.57 lb/lb
RA = 136.87 mg/l

Supplemental alkalinity required to add to the influent must be determined or calculated. Assuming the following:

Supplemental Alkalinity Required = SAR
Alkalinity consumed by nitrification = ACN = 304 mg/l
Alkalinity in raw wastewater = ARW = 60 mg/l (in potable water from supplier)
Alkalinity to remain in reserve in wastewater = ARRW = 60 mg/l

SAR = ACN – ARW – RA + ARRW
SAR = 304 – 60 – 137 + 60
SAR = 167 mg/l

The supplemental alkalinity to be feed is therefore approximately:

Lbs/day = Design flow (MGD) x 167 x 8.34
Lbs/day = 0.315 x 167 x 8.34
Lbs/day = 438.73

Using 50% sodium hydroxide (liquid caustic soda), the amount needed is determined using 0.126 gallons per 1 lb of alkalinity needed. Thus the amount needed per day is:

Volume needed = 438.73 lbs/day x 0.126 gallons/lb
Volume needed = 55.3 gallons per day

Therefore the total volume needed per month (30 days) will be:
Volume/month = 55.3 volume/day x 30 days/month
Volume/month = 1659 gallons

This is a large amount that should be delivered by tank truck. It is cheaper to purchase the caustic soda by the tank truck load (approximately 4500 gallons/tank truck), thus the storage tank should be sized to handle approximately 4,500 gallons plus another 20% for extra storage (900 gallons) to allow filling the tank prior to exhaustion of all the caustic soda supply. The storage tank should be sized for a minimum of 5,400 gallons (721.93 cubic foot). Using a 10 foot diameter tank the minimum tank height would be approximately (721.93/78.54) 9.2 feet.

C. Phosphorus Removal in SBR Basins

1. Biological Removal

This wastewater treatment facility will be required to produce an effluent with total phosphorus limits of less than 0.6 mg/l. Biological removal of phosphorus can be accomplished in SBR activated sludge plants by modifying the cycles to provide needed anaerobic and aerobic mixing periods (see Design Manual Phosphorus Removal - EPA publication 625/1-87/001, section 2.3.2). The anaerobic contact time (zone or cycle) normally ranges between 0.9 to 2.0 hours. The aerobic zone (or cycle) must follow with dissolved oxygen concentrations greater than 2.0. Sufficient aerobic time is needed for the phosphorus uptake into the biomass.

The design manual indicates that an SBR system can produce an effluent of meeting a 2.0 mg/l phosphorus limit without any chemical addition on a consistent basis. Getting the effluent down to 1.0 mg/l may require some chemical addition (metal salt) in the aeration basin to help precipitate the solids. Filtration of the effluent will be necessary to consistently meet an effluent limit of 0.6 mg/l phosphorus. Thus, the basins must also consider the additional sludge volume necessary for the phosphorus removal in the biological process.

Although phosphorus is not normally associated with the Biomass (due to the small incremental amount), it is an important element in microorganisms. It is used for energy transfer and components of the cells. The typical phosphorus content of microbial solids is 1.5 – 2 % based on dry weight (see Design Manual – section 3.1). The basic biological removal mechanism for phosphorus is as follows:

> Facultative microorganisms under anaerobic zone (cycle) produce acetate and other fermentation products. Some of the microorganisms (phosphorus removing microorganisms) prefer and readily assimilate and store these fermentation products. These same microorganisms release soluble phosphorus back into the basin during the

anaerobic cycle. Assimilation and storage of the fermentation products is aided by the energy made available from the hydrolysis of the stored polyphosphates during the anaerobic cycle. Polyhydroxybutyrate (PHB) is formed from the reactions of the cells and the presents of acetoacetate. During the aerobic cycle of the SBR process, the PHB and other stored substrate products are depleted by the microorganisms and soluble phosphorus in the basin is taken up with excess amounts stored as polyphosphates granules in the cells. Additional cells are also produced during the aerobic cycle. The amount of biological phosphorus removal achieved is directly related to the amount of substrate that can be fermented by normally occurring microorganisms in the anaerobic cycle and assimilated/stored by phosphorus-removing microorganisms.

Although there are little significant differences in the sludge production of a typical activated sludge basin and a phosphorus removal system, the storage of phosphorus in the biomass will increase the sludge some. As stated earlier the content of phosphorus in the biomass is approximately 2%. The ability of the cells to store additional polyphosphates within the cells of a phosphorus removal basin increases this value to approximately 4%.

The coefficient Y (cell yield) was previously shown to range between 0.4 and 0.8 VSS mg/mg-BOD_5 removed. The ICEAS SBR manufacturer recommends the use of 0.8 for the calculation of phosphorus removal based on their experience. The EPA design manual (published in 1987) indicated that 0.7 is typically used with a mean cell residence time of 20 days. GRW has chosen the use of 0.75 as the value for this calculation. Therefore, the total phosphorus (TP) concentration removed by biological means of the incoming concentration by the production of heterotrophic microorganisms will be approximately (assuming safety factor of 2.0 for effluent BOD_5 (11.4 / 2 = 5.7):

TP = [(400 mg/l – 5.7 mg/l) x 0.75(cell yield) x 0.04 (%P)

TP = 11.83 mg/l

Thus we can assume that with a design total influent phosphorus concentration (TP_{in}) of 15 mg/l, the total amount of phosphorus (TP_b) remaining after the biological process will be approximately:

$TP_b = TP_{in} – TP$
TP_b =15.0 – 11.83
TP_b = 3.17 mg/l (remaining in the effluent after biological treatment)

2. <u>Chemical Removal of Phosphorus</u>
Meeting the final effluent standard of 0.6 mg/l will require additional biological treatment or chemical addition to insure sufficient removal of the total phosphorus. Literature indicates that aluminum ions and iron salts can be used to combine with soluble P to form compounds that will precipitate and add to the sludge volume. The most common form of aluminum in use for the removal of phosphorus is alum (a hydrated aluminum sulfate). Alum contains about 9.1 percent soluble aluminum as Al and 17 percent soluble

aluminum as Al_2O_3 with an approximate formula of $Al_2(SO_4)_3 \bullet 14\ H_2O$. The reaction of alum with phosphate can be described as the following stoichiometric formula:

$$[Al_2(SO_4)_3 \bullet 14\ H_2O] + 2PO_4^{3-} \rightarrow 2Al\ PO_4 + 3SO4^{2-} + 14\ H2O$$

This formula indicates that one mole of alum will react with 2 moles of phosphate containing 62 g phosphorus to form 2 moles of aluminum phosphate.

The molecular weight of these compounds is as follows:

Alum (1 mole)
$Al_2(SO_4)_3 \bullet 14\ H_2O$
Al = 27 x 2 = 54
S = 32 x 3 = 96
O = 16 x 12 = 192
H = 1 x 28 = 28
O = 16 x 14 = 224
(54 + 96 +192 + 28
+ 224) = 594

Phosphate (2 moles)
$2PO_4$
P = 31
O = 16 x 4 = 64
(31 + 64) x 2 = 190

Aluminum Phosphate (2 moles)
$2Al\ PO_4$
Al = 27
P = 31
O = 16 x 4 = 64
(27 + 31 + 64) x 2 = 244

This theoretical formula indicates that the weight ratio of alum to phosphorus is 594 to 62 or 9.6:1. However, this formula does not take into consideration any other substances that would have a competing reaction with the alum (such as sulfate, sodium, clays, microorganisms, etc). A negative consideration with the use of alum is the lowering affect on the pH.

Since the alkalinity of the wastewater will be near 60 mg/l the use of alum would require additional chemicals to prevent lowering the pH. When the pH level in the aeration basin drops below 7.0 the microorganisms providing biological nitrification of the ammonium nitrogen levels, will slow and eventually stop this process. Therefore, this facility will consider the use of sodium aluminate for the chemical used to precipitate the phosphorus from the system. The reaction of sodium aluminate with phosphate can be described as the following stoichiometric formula:

$$[Na_2O \bullet Al_2O_3 \bullet 3H_2O] + 2PO_4^{3-} \rightarrow 2Al\ PO_4 + 2NaOH + 6\ OH^-$$

This formula indicates the reaction of sodium aluminate with the phosphates forms sodium hydroxide (NaOH) as the aluminum attaches to the phosphate and precipitates. Another by-product of the reaction is the hydroxyl ion (OH^-). The sodium hydroxide will help increase the pH or reduce the drop caused by nitrification. This reaction indicates that one mole of sodium aluminate will react with 2 moles of phosphate and precipitate out 2 moles of aluminum phosphate. The ratio of sodium aluminate to phosphorus is 218 to 62 or 3.6:1. Again this ratio does not consider any other substances that would compete with the sodium aluminate. Brenntag Chemicals indicates that the actual dosage of sodium aluminate (based on the concentration they recommend) to use in the removal of phosphorus is 15 to 1 (or 15 mg of sodium aluminate for ever 1 mg of phosphorus removed).

Assuming that the maximum phosphorus level after biological treatment is 3.17 mg/l, the amount of sodium aluminate required to reduce this level to 0.6 mg/l (limit set for effluent) can be approximated as follows:

3.17 – 0.6 = 2.57 mg/l - P to remove by chemical addition

Assuming a dosage rate of 15 mg of sodium aluminate for every 1 mg of phosphorus removed:

Lbs/day = [2.57 mg/l x (15 mg – Sodium Aluminate/1 mg of P)] x 8.34 (conversion factor) x 0.315 MGD

Lbs/day = 101.28 of sodium aluminate

Liquid sodium aluminate has a specify gravity of 1.55, therefore 1 gallon has a mass of (8.34 lbs of water/gal x 1.55) 12.93 lbs. The volume of chemical needed per day at the design capacity of the plant (0.315 MGD) would be approximately:

Gallons/day = 101.28 lbs/day / 12.93 lbs per gallon
Gallons/day = 7.83 of sodium aluminate

The amount needed per month would be approximately (8.00 gallons/day x 30 days/mo) 240 gallons.

The amount of sludge produced by the chemical addition of the sodium aluminate can be estimated assuming that the aluminum reacts with the phosphorus compounds first and that excess aluminum forms aluminum hydroxide. The stoichiometry of the aluminum phosphate and aluminum hydroxide is used to calculate the sludge production as follows:

$AlPO_4$ = [27 + 31 + 16(4)] = 122 atomic weight (aluminum phosphate)
$Al(OH)_3$ = [27 + (16x3) + (1x3)] = 78 atomic weight (aluminum hydroxide)
Amount of phosphorus to remove = 2.57 mg/l
Sodium aluminate dosage 15 mg/l per 1 mg/l P to remove
The atomic weight of aluminum = 27
The atomic weight of phosphorus = 31
Removal of 2.57 mg/l of phosphorus from SBR effluent by chemical precipitation
2.57/31 = 0.083 mmoles per liter of $AlPO_4$ produced
Dosage 15/27 = 0.56 mmoles of Al available
0.56 – 0.08 = 0.48 mmoles per liter of excess Al available for production of $Al(OH)_3$
$AlPO_4$ sludge: 0.08 mmoles/l x 122 = 9.76 mg/l of $AlPO_4$
$Al(OH)_3$ sludge: 0.48 mmoles/l x 78 = 37.44 mg/l $Al(OH)_3$

Total chemical sludge expected through stoichiometry = (9.76 + 37.44) 47.2 mg/l of wastewater treated. The EPA design manual (chapter 5) recommends that this calculated sludge production number be increased by 35% to allow for any unknown condition that may produce additional sludge. Thus the total design estimate of chemical sludge produced is (47.2 x 1.35%) 63.72 mg sludge/liter of wastewater treated or [63.72 x 8.34 x (0.315)] **167.4 lbs per day**.

Initially, the chemical sludge will be produced between the SBR units and the filters. The backwash waste from the filters will be wasted back to the main pump station. This sludge will return back to the SBR units and settle out in the basins. The previously discussed SBR design is based on the SVI value of 150. This number was converted too 2.4 ft^3/lb and thus the volume of chemical sludge produced each day and removed in the SBR basins due to phosphorus removal will be approximately {[167.4 (total) / 2 (basins) lbs/day] x 2.4 ft^3/lb} **200.88 ft^3/day/basin (use 201cu.ft./day/basin)**.

D. SBR BASIN SIZE

As discussed above, the volume of biomass for each basin (using two basins for the plant) should be sized with a total of 24,881.6 cu ft using the SVI as a design parameter. The ICEAS SBR system uses the same basins for biological treatment and settling. This requires the use of cycles to allow batch treatment. The SBR system design uses cycle times based on normal and peak weather flows. Since this design will only use two basins, a maintenance cycle has been used in place of the peak wet weather to allow one basin to treat the wastewater during maintenance. The cycles proposed for this project are as follows:

CYCLE	AIR-OFF	AIR-ON	SETTLE	DECANT	TOTAL Hrs.
Normal	48 min	120 min	48 min	72 min	4.8 hours
Maintenance	24 min	60 min	24 min	36 min	2.4 hours

Based on the normal cycle shown above for the design capacity, each basin will have 5 cycles per day (24 hrs/4.8 hrs. per cycle). During maintenance of one basin only one basin would be operating, thus the basin would need to increase the cycles to 10 per day.

The volume needed above the bottom water level (above decant range) must be determined to prevent hydraulically overloading the basins. Assuming that the peak flow coming into the plant was 425,000 GPD and the plant was operating in the normal mode, the total cycle time per basin would be 4.8 hours. Flow would continue to enter the basin during the entire cycle, however, decant would occur for 72 minutes (or 1.2 hrs) during the cycle. Subtracting the decant time from the cycle time, the total time water would enter each basin without any leaving would be (4.8 – 1.2) 3.6 hours. Treating half of the flow in each basin would require a total volume above the bottom water level (VABWL) of:

VABWLp = (425,000/2) GPD x (3.6 hrs per cycle/24 hrs per day) = 31,875 gallons per cycle

Converting this volume to cu. ft. gives a volume of (31,875 gal/7.48 gal/cu ft) 4,261 cu ft per basin.

Assuming maintenance needs to be performed on one of the basins during a dry period, the cycle time would be reduced increasing the cycles per day to 10. The VBWLm needed during this period would be found the same way assuming all the flow is treated in one basin. VABWLm = 315,000 GPD x [(2.4 – 0.6 hrs/cycle)/24 hrs/da] = 23,625 gallons or (23,625/7.48) 3,158 cu. ft. Since the volume needed during the peak flow condition is larger than that needed during maintenance, the 4261 cu. ft volume will be used to size the basins. This will be called the maximum volume above the bottom water level (or MVAB).

1. SBR Surface Area

Determining the surface area requires knowing the total volume required in the basins. The total basin working volume (BWV) is therefore approximately:

BWV = Biomass + Volume Above Bottom Water Level + chemical sludge volume
BWV = 24,882 + 4,261 + 201
BWV = 29,344 cu. ft.

The SBR manufacturer recommends a buffer zone (BZ) or safety factor of 3 ft to account for unknowns and sludge accumulation in basin due to lack of sludge wasting. Using the value of 20 feet for the top water level in each basin, the surface area needed can be determined by dividing the BWV value by (20 – 3) 17 to give a surface area of (29,344/17) 1,726.12 square feet.

Using a round tank for each basin, the diameter required can be calculated as shown below:

Area = $\pi D^2/4$ or
D^2 = (area x 4)/π = (1,726.12 x 4)/π
D^2 = 2,197.76 ft^2
D (diameter) = 46.88 ft (for ease of construction use 47 feet, thus area = 1735)

The expected biomass sludge depth (BSD) will therefore be approximately:

BSD = 24,882/1735 => BSD = 14.34 feet

The expected chemical sludge depth (CSD) in each basin will be approximately:

CSD = 201/1735 => CSD = 0.12 ft

Thus the total design sludge depth in each basin will be approximately (14.34 + 0.12) 14.46 feet.

2. Decant Rates

Based on the MVAB (volume) of 4,261 cu ft, the decant rates for each basin must be determined. This SBR system is being designed for continuous flow entering the basins during cycles (even during decanting phase of the cycle). Since this facility is being designed with a maintenance cycle (only one basin operating) instead of a storm cycle, the flow entering the basins will be set for the peak flow (during rain events) of 425,000 GPD for the normal cycle. The maintenance cycle will only be used during dry weather conditions so the flow will be 315,000 GPD during that cycle. Decant flow rates for the basins will be determined as follows:

Cycles	**Normal Peak Flow**	**Maintenance Flow (1 basin)**
Flow (GPD)	425,000 GPD	315,000 GPD
Flow per basin (2 basins)	212,500 GPD	315,000 GPD
GPM (1440 min/day)	147.57 GPM	218.75 GPM
Decant phase of cycle	1.2 hours (72 min)	0.6 hours (36 min)
MVAB (4,261 ft^3 x 7.48 gal/ft^3) 31,872.78 gals	442.68 GPM	885.36 GPM
Maximum Decant Rates	590.25 GPM	1,104.11 GPM

Preventing solids spilling over the decanter weir requires keeping the velocity going over the weir to a minimum. The SBR manufacturer recommends a maximum loading rate on the weir of 20 cu ft/min/ ft of weir during normal peak flow decant rate and 25 cu ft/min/ ft of weir during peak flow (maintenance flow) decant rate. These flow rates are equivalent to 150 gallons/min/ft and 187 gallons/min/ft, respectively. Thus, the decanter length during normal peak flow would need to be a minimum of (590.25/150) 3.94 feet. During the maintenance cycle of the SBR units the minimum decanter length will need to be (1,104.11/187) 5.9 feet. Based on these lengths, the **decanter weir should be 6 feet in length**.

3. Decanter Drawdown Depth (DDD)
   The previously determined MVAB for each basin was 4,261 cu ft. The decanter must be able to drawdown this much volume from the basin during each cycle. Based on the surface area of each tank, the depth of the decant zone will be approximately:

   DDD = (4261 cu ft / 1735 sq. ft)
   DDD= 2.46 ft.

4. Water Level in Basins
   This facility has been designed with a **top water level** in each basin set at **20 feet**. Based on the DDD of 2.46 ft, the **bottom water level** will be set at (20 – 2.46) **17.54 ft**. Based on a total design sludge depth of 14.46 feet, the **buffer zone** (safety factor) for each basin will be (20 – 2.46 – 14.46) **3.08 feet**, which should be sufficient with the ability to help precipitation with chemical addition and/or the filters following the basins.

5. Hydraulic Retention Time
   The hydraulic retention time (HRT) of each basin must be determined by dividing the flow into the aeration basin volume. Since the basin water depth changes due to the fill and decant phases of the cycle, the volume changes throughout the cycle. The average maximum depth of the basin must be determined by evaluating the decant drawdown depth during the normal SBR cycle at the design flow of 315,000 GPD.

Based on the design flow each basin will be required to treat 157,500 GPD or (157,500/1440) 109.4 GPM or (109.4/7.48) 14.62 cu ft/min. Each cycle will take 4.8 hours with 1.2 hours of decant, thus for 3.6 hours or (3.6 hrs x 60 min/hrs) 216 minutes per cycle, flow will be entering each basin with no withdrawal. This would indicate that approximately (14.62 cu ft/min x 216 min) 3,157.92 cu ft would enter the basin during that period. The basin surface area determined previously is 1735 sq. ft, thus, the maximum average depth in the basin will be (3,157.92 cu. ft/1735 sq. ft) 1.82 feet plus the bottom water level of 17.54 ft for a total depth of (1.82 ft + 17.54 ft) 19.36 feet. Therefore the average maximum volume in each SBR basin would be the surface area times the depth or (1,735 sq. ft x 19.4 ft) 33,659 cu ft. The HRT is therefore found as follows:

$$\text{HRT} = \frac{\text{33,659 cu ft x 7.48 gals/ cu ft}}{\text{157,500 gallons per day}} = \text{1.6 days}$$

6. MLSS Concentration in the Bottom Water Level

The MLSS concentration in each basin can be estimated using the conversion factor of 62.43 x10^{-6}. This number is conversion of 1 mg/l to lbs/cu ft. The bottom water level volume of each basin is found by multiplying the depth of the BWL (17.54 ft) with the surface area (1735 sq ft). Therefore the volume per tank is (17.54 x 1735) approximately 30,431.9 cu ft. The MLSS is the sum of biomass, chemical sludge and inorganic solids that enter the wastewater plant. The design capacity of this facility assumed an influent suspended solids concentration of 400 mg/l and that 30 % of these solids were inorganic or inert. Using the empirical equation:

Lbs/day = Flow (in MGD) x concentration (mg/l) x 8.34 (conversion factor)

The total SS loading per day at design capacity can be calculated.

Lbs/day = (0.315 x 400 x 8.34) x 30%
Lbs/day = 315.25

Each basin would receive approximately half of this loading (315.25/2), thus the **total suspended solids loading per basin per day would be 157.63 lbs**.

Lbs of MLSS = 10,358.7 lbs of MLVSS + 83.7 lbs of chemical sludge + 157.63 lbs of SS

Total lbs of MLSS = 10,600.03

Thus, the MLSS concentration is:

$$\text{MLSS} = \frac{10{,}600.03 \text{ x } (1\text{x}10^{6})}{30{,}431.9 \text{ x } 62.43}$$

MLSS = 5,579.4 mg/l (use 5,580 mg/l at design capacity)

7. Total Mass of Solids Produced

The total mass of solids produced per basin per day is determined by calculating the total biomass produced, adding the suspended solids removed from the influent and adding the chemical solids produced. Based on the design flow of 315,000 GPD (0.315 MGD) and the loadings to the plant, the sludge produced can be estimated as follows:

Biomass per basin = (Q/2) x (BODin – BODout) x (observed cell yield) x 8.34

The permit standards require an effluent concentration of 11.4 mg/l BOD_5 or less on a daily basis. Total nitrogen limits set for this facility will reduce the BODout to lower levels therefore GRW has reduced this limit to 5 mg/l for design purposes. As previously discussed, the empirical formula used to calculate the observed cell yield is:

$$Yobs = \frac{Y}{1 + Kd\,\theta c}$$

Where; Kd = cell decay coefficient – 0.06
θc = 20 days - (mean cell residence time chosen by GRW)

$$Yobs = \frac{0.6}{1 + 0.06\,\theta c} = \frac{0.6}{1 + (0.06 \times 20)} = 0.273$$

Thus,
Biomass per basin = (0.315/2) x (400 – 5) x 0.273 x 8.34
Biomass per basin = 141.65 lbs/day

The sludge produced by the suspended solids entering the treatment plant (at design capacity) as previously determined is 157.63 lbs per basin per day. The chemical sludge produced per basin was determined to be approximately 83.7 lbs/day per basin, thus the total mass of solids produced per basin should be (141.65 + 157.63 + 83.7) **382.98 lbs/day**.

8. Volume of Sludge to Waste

Maintaining a constant MLSS value in the aeration basins will require sludge wasting on a daily basis. The maximum amount of sludge to be wasted each day would occur at the design capacity loading and flow. The calculated MLSS at the design capacity was found to be approximately 5,580 mg/l in the bottom water level. Wasting of sludge occurs during the decant phase of the cycle (72 minutes during normal cycle and 36 minutes during a maintenance cycle). The decant phase is preceded by a settling phase during the cycle, thus, GRW has assumed a MLSS concentration of approximately 8,500 mg/l during the decant phase. The volume of sludge to be wasted can be calculated with this concentration assumption as follows:

$$\text{Volume to Waste (GPD)} = \frac{\text{lbs per day x } 1E^{6}}{(\text{MLSS x } 8.34)}$$

$$\text{Volume to Waste (GPD)} = \frac{382.98 \text{ lbs/day x } 1E^6}{8{,}500 \text{ x } 8.34}$$

Volume to Waste = 5,402 gallons per day

The SBR manufacturer recommends removal of this sludge on a daily basis throughout the day. This can be accomplished by pumping a small amount out during each cycle of the SBR tank. Under normal conditions each basin would have 5 cycles per day. Assuming equal portions of the sludge is wasted during each cycle, approximately (5,402/5) 1,080 gallons would need to be pumped to the digester during each cycle. Using a 100 GPM pump would require approximately 11 minutes during the decant cycle to accomplish this waste rate. If the maintenance mode of operation is used for a few days, the wasting rate would remain the same (11 minutes per cycle, but 10 cycles per day) since all loading would be going to one basin while the other unit is down for maintenance.

9. Air Requirements of SBR

The air requirement for the SBR basins is based on the loading of BOD_5 and NH_4N^- entering the basins each day. The assumed values entering the treatment plant are concentrations of 400 mg/l of BOD_5 and 60 mg/l of NH_4N^-, thus the total organic loading per basin is determined as follows:

BOD_5 lbs/day/basin = (0.315 MGD/ 2 basins) x 400 mg/l (BOD_5 influent concentration) x 8.34 (conversion factor)

BOD_5 lbs/day/basin = 525.42 lbs/day/basin

The total TKN concentration to be removed would be [60.0 – 1.3 (ammonia nitrogen limit) – 2.9 (total nitrogen limit)] 56 mg/l or approximately (0.315 x 56 x 8.34) 147.12 lbs/day.

NH_4N^- lbs/day/basin = 147.12 lbs/day/2 basins = 73.56 lbs/day/basin

The production of heterotrophic microorganisms was shown to remove (assimilate) approximately 35.07 lbs per day (or 17.54 lbs per basin) of the ammonium-nitrogen content in the influent wastewater. The remaining (73.56 – 17.54) 56 lbs per day per basin must be removed by the nitrifying microorganisms.

The oxygen (O_2) required to oxidize these organics is normally assumed to be 1.5 lbs of O_2 per lb of BOD_5 and 4.6 lbs of O_2 per lb of NH_4N^-. Thus, the oxygen needed to oxidize the BOD_5 is (525.42 x 1.5) 788 lbs/day/basin and the NH_4N^- (56 x 4.6) 257.6 lbs/day/basin. The total amount of oxygen [or Actual Oxygen Required (AOR)] needed to oxidize the organics in each basin is approximately (788 + 258 =) 1,046 lbs per day.

The actual oxygen transferred from air to the wastewater must be determined to calculate the amount of air necessary to provide the needed oxygen to oxidize the organics. The efficiency of the transfer of oxygen to the microorganisms is affected by three factors:

- The alpha factor which considers the mixing intensity and tank geometry,
- the theta factor (temperature coefficient), and
- the Beta factor is the effect of the oxygen solubility wastewater versus clean water

The alpha factor for diffused aeration normally has a range between 0.4 and 0.8. When using fine bubble aeration (which this project does), the alpha factor (α) is typically between 0.6 and 0.8. The diffuser manufacturer recommends using an alpha factor of 0.65 for the disc membrane diffusers that are designed with this project.

The temperature coefficient (theta) compares the water temperature at the plant site with standard temperature (20°C). As shown above, the average water temperature at the existing treatment plant in the summer is 24°C (high of 27°C) so the theta factor is found using the empirical equation:

Theta = $\theta^{Tsite - 20}$, where $\theta = 1.024$ for diffused aeration
Theta = $1.024^{(24-20)}$
Theta = 1.1

The Beta factor (β) value of 0.95 is commonly used as a correction factor of the oxygen-transfer rate for the oxygen solubility in wastewater versus tap water. At an elevation of 1700 feet, the atmospheric pressure will be approximately 714 mm of Hg. Oxygen saturation of water at this elevation (atmospheric pressure) and temperature of 24°C is approximately 8.0 mg/l. Assuming that the dissolved oxygen concentration of 2.0 mg/l will be maintained in the SBR basins, the effect of constituents in the wastewater can be determined as follows:

K_La (s) = β x [DO_2sat – DO in basin)/DO_2sat] = 0.95 x [(8-2)/8]
K_La (s) = 0.95 x 0.75 = 0.7125

Multiplying the different coefficients together gives an overall mass oxygen-transfer rate (K_La) of:

K_La = K_La (α) x K_La (T) x K_La (s) = 0.65 x 1.1 x 0.7125
K_La = 0.50944

The total oxygen required (TOR) in each basin is therefore
TOR = AOR/ K_La = 1,046/0.50944 = 2,053.245 lbs per day

Determining the actual process air needed in each basin requires adjusting for the air density of the proposed site and adjusting for the transfer efficiency of the diffusers used in the aeration basin. The diffusers will be mounted approximately one foot from the bottom of the tank. The operating level in the SBR tanks will vary between 17.54 and 20 feet. This is an average depth of approximately [((20 – 17.54)/2) + 17.54 =] 18.77 feet. Subtracting the diffuser disk elevation from this gives a submergence of 17.77 feet. The standard oxygen transfer efficiency (SOTE) of a fine bubble membrane disk system with a grid layout has been shown to vary between 16 – 38 % at a submergence of 15 feet. This variation is due to the loading rate of air per diffuser and the density of diffusers on the tank floor. This design uses 450 – 9" discs in each tank for a density of approximately 1 disc per 3.8 square feet. The air flow rate for each disc is designed for an average flow of 1 scfm per disc which gives an air flow of approximately 0.26 scfm/ft^2 of tank area. This layout is considered a dense grid system and would imply that the SOTE of 35% is a valid value that can be used with a submergence of 15 feet. Since the submergence is approximately 17.77 feet, the efficiency used will be a SOTE = 38% for this project.

At an elevation of 1700 feet, the atmospheric pressure will be approximately 714 mm of Hg or approximately 28.11 inches of Hg. The density of air can be determined using the empirical equation:

P = (1.325 x pressure)/Tr
Where: P = density in lbs/cu ft
Pressure = inches of Hg
Tr = absolute temperature (Rankine)

Assuming air temperatures in summer (90°F) and winter (20°F) the density can be determined as follows:

Summer - 90°F = 549.67°R
P = (1.325 x 28.11)/549.67
P = 0.068 lbs/cu ft (density of air)

Winter - 20°F = 479.67°R
P = (1.325 x 28.11)/479.67
P = 0.078 lbs/cu ft (density of air)

The amount of oxygen in 1 cubic foot of air is approximately 23.3%, thus the amount of air needed to meet the oxygen demand is found as follows:

$$\text{Volume of air} = \frac{\text{oxygen demand (lbs/day)}}{[\text{Air density (lbs/ft}^3) \times .232]}$$

Summer,

$$\text{Volume of air} = \frac{2{,}054 \text{ lbs/day}}{0.068 \times 0.232} \Rightarrow \text{Volume or air } = 130{,}197.8 \text{ ft}^3/\text{day}$$

The SBR tanks for this project are designed for aeration only 10 hours (600 minutes) each day, thus, the volume of air needed must be supplied in that short time period.

The standard total flow rate of air required (STAR) assuming a SOTE of 36% oxygen transfer efficiency is found as follows:

$$\text{STAR} = \frac{130{,}198 \text{ ft}^3/\text{day}}{[.38 \times 600 \text{ min/day}]} \Rightarrow \text{STAR} = 571 \text{ SCFM (in the summer)}$$

Winter,

$$\text{Volume of air} = \frac{2{,}054 \text{ lbs/day}}{0.078 \times 0.232} \Rightarrow \text{Volume or air} = 113{,}505.75 \text{ ft}^3\text{/day}$$

Volume of air assuming a SOTE of 38% oxygen transfer efficiency

$$\text{STAR} = \frac{113{,}506 \text{ ft}^3\text{/day}}{[.38 \times 600 \text{ min/day}]} \Rightarrow \text{STAR} = 497.83 \text{ SCFM (in winter)}$$

The maximum volume of air is needed in the summer as shown above. Using the disc diffusers mounted approximately 12 inches above the floor, the maximum submergence will be approximately 19 feet. The water pressure on the diffuser will be 0.433 psi per foot of submergence or (19 ft x 0.433 psi/ft =) 8.22 psi. Assuming a maximum headloss in the air piping of 1.25 psi, the blower should be designed to provide a flow rate of 571 SCFM at 9.47 psi.

Checking on the mixing requirements of the SBR using diffused aeration discs, the amount of air required is normally 10 to 15 CFM/1000 cu ft. The tank volume is approximately $[(47^2 \times \pi)/4] \times 20 = 34{,}700$ cu ft, therefore, at a air flow rate of 571 SCFM the amount of air available is (571/34.7 =) 16.45 CFM/1000 cu. ft. At a flow rate of 1 CFM per diffuser (450 CFM) the air available would be 12.96 CFM/1000 cu. ft. which is sufficient.

III. DIGESTER DESIGN

A. Sizing the Digester

At the design capacity flow of 315,000 GPD and an organic loading of 400 mg/l of BOD_5 and SS, approximately 5,402 GPD would be wasted to the digester(s) from each SBR basin for a total volume of 10,804 GPD. It was assumed above, that the concentration of solids being pumped from the SBR basins would be approximately 8,500 mg/l (or 0.85% solids). As the waste sludge is pumped to the digester for further reduction, a decanter in the digester allows further thickening of the solids to approximately 1.0 - 3.0% solids.

The digester for this project has been designed with a center wall that allows two separate tanks for wasting sludge. One basin will be used to contain the waste sludge for a period of 30 days. After filling the first basin, the second basin will be used for holding the waste sludge. The first basin will continue to digest the sludge for another 15 to 21 days without additional pumping into it (to prevent contamination) before starting to empty the tank by pumping the sludge to the belt filter press for dewatering.

Volume needed to hold 30 days of waste sludge at 8,500 mg/l would be:
10,804 gallons per day x 30 days = 324,120 gallons
324,120 gallons x 1 cu ft /7.48 gallons = 43,331.6 cu ft

Using a maximum working depth of 20 feet, the diameter of the tank can be determined as follows:
Area = Volume/depth = 43,332/20 => Area = 2167 square feet
Area of a circle = πr^2, thus **half of the circle would be 0.5 πr^2** (does not take into consideration thickness of center wall – considered insignificant)
2167 = 0.5 πr^2 => 2167/0.5xπ = r^2 => 1379.3 = r^2 => 37.14 feet = radius

This would require a digester tank that is 75 feet in diameter, which we considered too large. If the wasted sludge is further concentrated, by gravity thickening using a decant mechanism in the digester to 2.0% (20,000 mg/l), the volume needed in the digester would reduced. Assuming the decanter mechanism operates in a 6 foot depth, the sludge could be concentrated into a tank with an approximate diameter of:
(382.98 x 2) lbs/day x 30 days = Flow (in MGD) x 20,000 x 8.34
Flow = 22,979 lbs/(166,800)
Flow = 137,764 gallons
Volume needed = 137,764 gallons/7.48 gal per cu ft
Volume needed = 18,417.62 cu ft
Using a maximum working depth of 14 feet, the area will be (18,417.62/14)
1,315.54 sq ft
1,315.54/0.5xπ = r^2 => 837.50 = r^2 => radius = 28.94 ft

Thus, the tank would need to be a minimum of approximately 57.87 feet in diameter to contain the wasted sludge. Therefore the diameter of the digester will be designed with a **diameter of 59.0 feet** to take into consideration the center wall of the digester.

B. Air Requirements of Digester

The biomass of the sludge (needing oxygen to metabolize) has been reduced in the SBR units since the sludge age was chosen to be approximately 20 days to achieve nitrification needed to meet the effluent standards. As sludge is retained in the digester it will be further reduced in quantity by biological action. The pounds of solids sent to the digester per day were shown to be 382.98 lbs/day per SBR basin or a total of 765.96 lbs per day (which includes inorganics and chemical sludge). The total biological (or volatile) solids are approximately 283.6 lbs/day.

The air requirement for the digester design is based on the amount of volatile solids that will be stabilized in one of the basins per day. The maximum volume of biomass pumped to one side of the digester was shown to be approximately 284 lbs/day.

Using an oxygen demand of 2.0 lbs per lb of biomass stabilized (TN Design Criteria), the maximum amount of oxygen needed in one side of the digester will be (284 lbs/day x 2.0 lbs-O_2/lb of cells) 568 lbs/day at the design capacity of the plant. At an elevation of 1700 feet, the atmospheric pressure will be approximately 714 mm of Hg or approximately 28.11 inches of Hg. The density of air can be determined using the empirical equation:

P = (1.325 x pressure)/Tr
Where: P = density in lbs/cu ft
Pressure = inches of Hg
Tr = absolute temperature (Rankine)

Assuming air temperatures in summer (90°F) and winter (20°F) the density can be determined as follows:

Summer - 90°F = 549.67°R
P = (1.325 x 28.11)/549.67
P = 0.068 lbs/cu ft (density of air)

Winter - 20°F = 479.67°R
P = (1.325 x 28.11)/479.67
P = 0.078 lbs/cu ft (density of air)

The amount of oxygen in 1 cubic foot of air is approximately 23.3%, thus the amount of air needed to meet the oxygen demand is found as follows:

$$\text{Volume of air} = \frac{\text{oxygen demand (lbs/day)}}{[\text{Air density (lbs/ft}^3\text{) x .232}]}$$

Summer,

$$\text{Volume of air} = \frac{\text{568 lbs/day}}{0.068 \text{ x } 0.232} \Rightarrow \text{Volume or air} = 36{,}004.6 \text{ ft}^3\text{/day}$$

$$\text{Volume of air assuming a 10\% oxygen transfer efficiency} = \frac{36{,}004 \text{ ft}^3\text{/day}}{[.10 \text{ x } 1440 \text{ min/day}]}$$

Volume of air in summer = 250.03 ft^3/min

Winter,

$$\text{Volume of air} = \frac{\text{568 lbs/day}}{0.078 \text{ x } 0.232} \Rightarrow \text{Volume or air} = 31{,}388.15 \text{ ft}^3\text{/day}$$

$$\text{Volume of air assuming a 10\% oxygen transfer efficiency} = \frac{31{,}388 \text{ ft}^3\text{/day}}{[.10 \text{ x } 1440 \text{ min/day}]}$$

Volume of air in winter = 217.97 ft^3/min

Maximum volume of air is needed in the summer. Using jet aeration fixed at an elevation 18-inches above the floor on the digester, the maximum water depth above the jet will be 20-1.5 = 18.5 feet. Thus the water head on the jet be equal to (.433

psi/ft of water x 18.5 ft) 8.01 psi. Assuming a maximum head loss in the air piping of 1.0 psi, the blower should be able to discharge approximately 250 cfm at 9.0 psi.

C. Mixing Requirements of Digester

Determining the mixing needs of an aerobic digester normally is based on the cubic feet of air applied per 1,000 cubic feet of volume or the Hp of the mechanical mixing device. The use of jet aeration for mixing requires using Hp has the design parameter. Standard mixing requirements used by the State is 0.5 to 1.5 Hp per 1,000 cu ft of digester volume. Other sources recommend a minimum Hp of 0.75 Hp per 1,000 cu ft.

The maximum volume per tank (when full) was shown to be approximately 18,418 cubic feet, thus the mixing requirements, require at a minimum (0.5 x 18.4) 9.2 HP or (0.75 x 18.4) 13.8 HP. This project has been designed using a 1.0 HP/1,000 cu ft as a minimum, thereby, requiring a 20 HP pump motor.

IV. BELT FILTER PRESS SIZING (Sludge Dewatering)

The biological reduction of the biomass (8,508 lbs) in the digester can be calculated as follows:

Assumption – sludge in digester (without addition) for 18 days at a temperature of 15°C (winter) having a decay coefficient (K_d) of 0.05
Biomass of 8,520 lbs
Using the first order empirical equation

$$\frac{S_t}{S_o} = e^{-kd*t}$$

where; S_t = solids reduced during time t
S_o = solids initially
K_d = decay coefficient – 0.05 (for temperature of 15°C)
t = time of solids digested (in days) = 18

$S_t = e^{-(.05 \text{ x } 18)} \text{ x } 8{,}520$ lbs
$S_t = e^{-(0.9)} \text{ x } 8{,}520$ lbs
$S_t = 0.40657 \text{ x } 8{,}520$ lbs
$S_t = 3{,}456.66$ lbs reduced

Thus the remaining amount of solids in the digester would be approximately (22,979 - 3,457) **19,522 lbs of organic and inorganic solids.**

The sludge can be dewatered using a belt filter press. The solids loading (feed) rates of belt filter presses range from 400 lbs to 700 lbs per hour per meter for aerobically digested sludge. The higher loading rates are associated with the extended type belt filter press. The type of belt press recommended for this project is an extended press that uses a gravity thickener on the top level. An evaluation of loading rates (assuming 700 lbs per hour per meter) and the amount of sludge to dewater are shown below:

Belt Press Size	Loading Rates (lbs/hour)	Quantity of Sludge (lbs)	Time of Operation	Operating Days/month
0.5 meter	350	19,522	55.8 hours	8
1.0 meter	700	19,522	27.9 hours	4
2.0 meter	1400	19,522	13.9 hours	2

Assumes the belt press is operated a minimum of 7 hours a day at design capacity.

At a solids concentration of 2%, the total gallons of sludge to be pumped to the belt press would be approximately:

19,522 lbs = 20,000 mg/l x 8.34 x flow in MGD
19,522/(166,800) = Flow in MGD
117,040 gallons = sludge to dewater

Based on the evaluation of belt press sizes and the gallons to be dewatered, the pumping rates to the belt presses are as follows:

Belt Press Size	Loading Rates (lbs/hour)	Gallons of Sludge	Time of Operation (hrs)	Pumping Rates (gal/min)
0.5 meter	350	117,040	55.8 hours	34.95
1.0 meter	700	117,040	27.9 hours	69.92
2.0 meter	1400	117,040	13.9 hours	140.34

Using an 18 day minimum detention (digestion) time in the digester cell as shown above, the **0.5 meter belt press** can be used to empty the cell during the (30 – 18) 12 day period left between digestion and filling the second digester cell. This would allow the plant operation of the press to be done without working on the weekends, which is preferred.

V. POST EQUALIZATION BASIN SIZING

A. The operation of the SBR units requires a batch discharge. During normal operation of the units, the discharge will occur over a 72 minute period during every 4.8 hour cycle (288 minutes). There are seven (7) phases in each 4.8 hour cycle. The SBR basins will vary their cycles by alternating phases as shown below:

NORMAL CYCLE (4.8 hours – five cycles per 24 hours per basin)

Time (hours)

0 –0.5	1	1.5	2	2.5	3	3.5	4	4.5	5

Air-on (24 min)	Air off (24 min)	Air on (72 min)	Air-off (24 min)	Air-on (24 min)	Settle (48 min)	Decant (72 min)	Bas 1

0 24 48 120 144 168 216 288

Air-off	Air-on	Settle (48 min)	Decant (72 min)	Air-on	Air-off	Air-on (72 min)	Bas 2

0 24 48 96 168 192 216 288

As can be seen from the cycles above, there are two different time periods between decant phases. The normal cycle time is 4.8 hours or 288 minutes. Basin 2 will begin its decant phase 96 minutes into the cycle and end 72 minutes later or at 168 minutes into the cycle. In basin 1 this is the beginning of the settling phase that will last 48 minutes before the decant phase starts in this basin. Thus, there is only 48 minutes between the end of basin 2 decanting phase and the beginning of the basin 1 decant phase. The end of the decant phase in basin 1 is at 288 minutes, which is at the end of the cycle for both basins. As the cycle begins again, there is a 96 minute period before the beginning of the next decant phase (which is in basin 2). These two time periods must be taken into consideration to properly size both the equalization basin and the effluent pumping rate (filter pumps).

During maintenance of one of the basins, the remaining basin will discharge for 36 minutes during every 2.4 hour cycle. The cycle time will be reduced in half (2.4 hours - ten cycles per 24 hours in one basin). The seven (7) phases of the cycle will be similar to that shown below:

MAINTENANCE CYCLE (2.4 hours – ten cycles per 24 hours)

Time (hours)

0 – 0.5	1	1.5	2	2.5

Air-on (12 min)	Air-off (12 min)	Air on (36 min)	Air-off (12 min)	Air-on (12 min)	Settle (24 min)	Decant (36 min)

0 12 24 60 72 84 108 144

As this diagram indicates, there is a period of 108 minutes between discharges into the equalization basin. At the average design flow of 315,000 GPD, approximately 31,500 gallons would be discharged into the EQ basin every cycle. This would require pumping out of the EQ basin at a rate of (31,500/144) 224 gallon per minute.

The actual decant rates were determined previously to discharge approximately 31,500 gallons in the 72 minute decant phase of each basin cycle at the design capacity and 42,500 gallons during peak flow conditions.

	Normal (2 basins)	Peak (2 basins)	Maintenance (1 basin)
Flows	315,000 GPD	425,000 GPD	315,000 GPD
4.8 hr cycles	437.5 GPM	590.28 GPM	NA
2.4 hr cycles	NA	NA	875.0 GPM
Total volume discharged per decant phase	31,500 gals	42,500 gals	31,500 gals
Min flow to filter	219 GPM	296 GPM	219 GPM
Volume pumped to filters during decant	15,768 gals	21,312 gals	7,884 gals
Volume remaining in equalization basin after decant	15,732 gals	21,188 gals	23,616 gals
Time till next decant phase	48 minutes	48 minutes	108 minutes
Volume pumped out to filters	10,512 gals	14,208 gals	23,652 gals
Volume remaining in EQ basin	5,256 gals	6,980 gals	0
Total volume after second decant phase	20,988 gals	28,168 gals	23,616 gals
Time till next decant phase	96 minutes	96 minutes	108 minutes
Volume pumped out to filters	21,024 gals	28,416 gals	23,652 gals
Volume remaining in EQ basin	0	0	0

As shown from the above calculations, the equalization basin will be emptied during normal operation every 2 cycles. During maintenance operation, the EQ basin will empty after every cycle. The largest volume that would be in the basin after two decants would be approximately 28,168 gallons during peak flow conditions. The size of the tank should be designed for the largest volume expected with a safety factor of 10 to 30%. Looking at the maximum volume during maintenance of 23,616 gallons and using a safety factor of 30%, the tank should be sized for a capacity of 30,701 gallons. This would be a safety factor of [(30,701 - 28,168)/28,168] approximately 9% at the peak flow condition. GRW decided that the minimum safety factor should be no lower than 10% at the peak flow condition. Thus, the volume was increased from 30,701 gallons to approximately 31,000 gallons.

The decanter in each SBR unit is designed to exit the tank at an elevation approximately 16 feet (1735) from the bottom of the tank (bottom elevation 1719). Setting the bottom of the EQ basin at the same elevation as the chemical building (1720) the working depth would need to be no more than 15 feet. Allowing 1 foot of headloss to convey decant from the SBR tanks to the equalization basin would make the maximum operating depth at 14 feet.

Based on a volume of 31,000 gallons the EQ basin should be sized as follows:

[31,000 gallons/7.48 gal/cu ft] = 4144. cu ft

Assuming a depth of 14 feet for an operating depth in the tank

$$\frac{4144 \text{ cu ft}}{14 \text{ ft}} = 296 \text{ sq ft (area)}$$

$$\text{Area} = \pi D^2/4 \Rightarrow \frac{296 \times 4}{\pi} = D^2 \Rightarrow 376.9 = D^2 \Rightarrow D = 19.413 \text{ ft}$$

Therefore based on the above discussion the equalization basin will be designed with an operating water depth of 14 feet and a diameter of 19.5 feet.

VI. EFFLUENT SAND FILTERS

A. Suspended Solids Removal

All flow pumped from the EQ basin will be sent to the effluent sand filters. The sand filters will need to be capable of hydraulically handling flows from 219 to 295 GPM. These filters will also be required to not only remove suspended solids and phosphorus, but also provide some biological nitrogen removal in the sand media. Using an upflow filter will allow continuous backwashing of the filter, keeping the size and complexity down. Operating the filter at 2 gallons per square foot of surface area will require a filter with approximately (295/2) 147.5 square feet. Several manufactures use filter cells with 50 square feet of surface area per cell and a depth of approximately 40 inches (3 ft – 4 in). Thus, three (3) cells would have a total of (3 x 50) 150 square feet of surface area which would provide the needed sand surface area. If maintenance is required on one (1) of the cells, the remaining two (2) cells can be operated at 3 GPM per square foot to handle flows of up to 300 GPM. The cells are rated up to 4 GPM per square foot for maintenance purposes.

The upflow filters will be designed to remove nitrogen and phosphorus simultaneously. This type of filter process is normally referred to as an "Enhanced Nutrient Removal" (ENR) system. Removing nitrogen biologically will require a deeper bed than normal to allow sufficient space for microorganism growth. Chemical addition will be required to provide a food source for the de-nitrification organisms that will grow in the sand filter. These microorganisms will use or consume the carbon source (food) as they metabolize and remove or strip oxygen from nitrates in the wastewater being filtered leaving nitrogen gas. Since the majority of the BOD_5 loading will be removed in the SBR units, another food or carbon source (methanol) will be necessary for the de-nitrification microorganisms. Continuous monitoring of the filter influent and effluent will be necessary to control the feeding of the methanol.

Reduction of the total phosphorus to 0.6 mg/l will require the addition of a coagulant to precipitate out the phosphorus still remaining after biological treatment (see item II.C phosphorus removal above) of phosphorus. The deeper bed is also needed to help provide sufficient filtering to remove the precipitated phosphates. The continuous monitor unit will therefore also be required to check the phosphorus content of the influent and effluent and make adjustments to the chemical feed pump of the coagulant (sodium aluminate).

The ChemScan UV-4100 Analyzer can sample both the influent and effluent lines of the filters for phosphorus, nitrogen ammonia, nitrites and nitrates. The results of the analyzer will be sent to a PLC controller that will operate and control the chemical pumps used to feed the amount of coagulant and methanol needed to achieve the final effluent results.

Thus, three (3) 50 square feet continuous backwashing upflow filter cells will be used for this project. The deep bed filters will be used to provide space for microorganism growth needed for de-nitrification of the wastewater. An ENR control panel will be used in conjunction with the operation of the filters to provide control over the chemical pumps that supply the methanol and sodium aluminate to the filter influent stream.

B. <u>Total Nitrogen Removal</u>

As discussed above, denitrification will occur in the SBR tanks during the non aeration (anoxic) phases as well as nitrification of the ammonia nitrification. Denitrification of the wastewater will occur in the filters to reduce the total nitrogen limit to 2.9 mg/l. Since the SBR tanks will be used to reduce the organic matter (BOD_5) too approximately 5.7 mg/l (see II A above), the carbon source needed by microorganisms for growth will be too low. The microorganisms used to denitrify (convert the nitrates to nitrogen gas) require a food source. Normally methanol (wood alcohol) is used as the carbon source. Methanol has a simple molecular formula of CH_3OH and provides the carbon source (C) for the microorganisms and the HOH^- radical and hydrogen gas as by-products.

The SBR system should reduce the TN limits to approximately 5.0 mg/l. Meeting a limit of 2.9 mg/l will require additional biological treatment in the filters. The depth of the sand in the filter (above the inlet manifold) was increased another 40 inches for a total of 80 inches (6 ft – 8 in.) to allow sufficient media for the microorganisms to grow. This is approximately 335 cu ft of sand media per filter. A fixed film growth of heterotrophic microorganisms attach to the sand media and use a carbon source (methanol, acetic acid, etc..) for food and nitrates as an oxygen source for respiration. The nitrates (NO_3^-) are converted to nitrogen gas (N_2), carbon dioxide (CO_2), water (H_2O) and a hydroxyl ion (OH^-). The overall stoichiometric formula used to show the denitrification process is shown below:

$$6\ NO_3^- + 5\ CH_3OH \rightarrow 5\ CO_2 + 3\ N_2 + 7\ H_2O + 6\ OH^-$$

> This is 6 molecules of nitrate and 5 molecules of methanol used by the microorganisms to produce 5 molecules of carbon dioxide, 3 molecules of nitrogen gas, 7 molecules of water and 6 molecules of the hydroxyl ion.

This filter manufacturer recommends a loading rate of 0.015 to 0.12 lbs of NO_3^-N per cubic feet of media per day. This design will use a loading rate of 0.07 lbs/cu ft/day, thus, with all three filters operating they should be able to remove [0.07 x 335 x 3] 70 lbs of nitrate per day. The proposed influent flow to the filters will have a nitrate concentration of approximately 5.0 mg/l or load the filters with approximately [5 x .315 x 8.34 =] 13 lbs/day.

The water going to the filters will enter at a flow rate of 200 to 300 GPM (or a flow of 26.7 to 40.1 cu ft per min), thus, with all three (3) filters operating the flow rate entering each filter will vary between 8.9 and 13.4 cu ft per min. The filters have a total volume between the influent diffusers to the effluent weir of approximately 609 cubic feet. There is, however, a little over 6 feet of sand in this volume or about half. Assuming that half the volume is displaced by the sand, the wastewater will have a minimum retention time of approximately [609/2/13.4] 22.7 minutes in each filter. The maximum retention time is estimated at approximately [609/2/8.9] 34.2 minutes. These retention times are about double that shown for other fluidized bed denitrification units [see EPA manual, page 236, Table 7-11] and therefore provide sufficient contact time of the wastewater with the microorganisms.

VII. DISINFECTION OF WASTEWATER

Assuring the E Coli removal requirements of the discharge standards are met will require disinfection by chemical addition (normally – chlorine) or ultra-violet radiation. Adding chlorine also requires that all traces of the chlorine is removed prior to discharge. Thus, UV radiation is chosen as the disinfection process. UV radiation can be done in a channel or a closed vessel. When used outdoors problems with algae can be an issue since chlorine is not being used. A shed is normally added over the channel to help keep the algae growth down (keeps sunlight off of channel). Using a closed vessel UV unit will greatly limit the problems associated with algae. Thus a closed vessel UV unit will be used as the disinfection treatment process for this facility.

EPA Technology Fact Sheet on UV disinfection indicates the following - The optimum wavelength to effectively inactivate microorganisms is in the range of 250 to 270 nm. The intensity of the radiation emitted by the lamp dissipates as the distance from the lamp increases. Low-pressure lamps emit essentially monochromatic light at a wavelength of

253.7 nm. Standard lengths of the low-pressure lamps are 0.75 and 1.5 meters with diameters of 1.5 - 2.0 cm. The ideal lamp wall temperature is between 95 and 122EF. Medium-pressure lamps are generally used for large facilities. They have approximately 15 to 20 times the germicidal UV intensity of low-pressure lamps. The medium-pressure lamp disinfects faster and has greater penetration capability because of its higher intensity. However, these lamps operate at higher temperatures with a higher energy consumption. There are three (3) critical areas that must be considered when choosing a UV disinfection unit:

1. Hydraulic properties of the reactor: Ideally, a UV disinfection system should have a uniform flow with enough axial motion (radial mixing) to maximize exposure to UV radiation. The path that an organism takes in the reactor determines the amount of UV radiation it will be exposed to before inactivation. A reactor must be designed to eliminate short-circuiting and/or dead zones, which can result in inefficient use of power and reduced contact time.

2. Intensity of the UV radiation: Factors affecting the intensity are the age of the lamps, lamp fouling, and the configuration and placement of lamps in the reactor.

3. Wastewater characteristics: These include the flow rate, suspended and colloidal solids, initial bacterial density, and other physical and chemical parameters. Both the concentration of TSS and the concentration of particle-associated microorganisms determine how much UV radiation ultimately reaches the target organism. The higher these concentrations, the lower the UV radiation absorbed by the organisms.

UV disinfection for this facility will occur after filtration of the wastewater which will lower the suspended solids (< 5 mg/l) and the colloidal solids (chemical precipitation using aluminum salts will occur before the filters). The flow through the unit will vary from 219 GPM at design capacity to 295 GPM at peak flow conditions. Low pressure bulbs will be used, operating at a wavelength of 254 nm (Newton-meter) to provide the disinfection of the wastewater. The intensity will be measured via a calibrated on-line meter.

The unit chamber for this design is 20 inches in diameter, and is specified with 150mm PN16 (equivalent 6-inch) flanged inlet and outlet ports installed perpendicularly on either end of the chamber body, in a biased alignment along the reactor's horizontal axis. Water enters and exits through these ports, and flows parallel to the two quartz sleeves. Baffles within the reactor cause a circular pattern to the flow as it moves to the effluent port. Each quartz sleeve holds a "lamp," which for this reactor comprises a bundle of four conventional low-pressure, mercury vapor lamps. Only one lamp (4 low pressure bulbs) will be operated at a time to achieve the needed disinfection. The second lamp will be for backup and alternately operated by the operator to extend the life of the unit.

The unit is equipped with an automatic wiper for cleaning the quartz surfaces. The two open-ended quartz sleeves are supported at both ends of the reactor by compression fittings inserted in the reactor face plates. The control panel monitors unit operation, with access to UV intensity, lamp LED's, running time, etc."

The closed vessel UV unit has been sized to provide greater than a 3 log reduction of fecal and total coliforms for the peak flow rate of 295 GPM. The UV system dose was confirmed through a bioassay. The design UV transmittance was 65% and the maximum total suspended solids (TSS) of 30 mg/l with solids no greater than 20 microns.

All the appropriate alarms and control features are included to assure proper UV dosage"

VIII. POST AERATION OF WASTEWATER

The effluent standards placed on this facility by the TN Division of Water Pollution Control requires a minimum DO limit of 6.0 mg/l. As the wastewater passes through the up-flow filters and then the closed vessel UV unit, it will be limited to contact with the atmosphere. DO levels in the wastewater will be low and must be raised prior to discharge into Mill Creek. There is sufficient fall from the location of the treatment plant and the discharge point for the use of a cascade (step) aerator. This type of aeration employs a series of steps over which the flow moves in fairly thin layers and drops to the next step to provide turbulence to increase oxygen transfer from the atmosphere to the wastewater.

There are two different approaches to determine the design of a cascade aerator. One determines the height needed for the cascade aerator and the second approach considers the height of the steps as it calculates the number of steps needed. We have based this design of the second approach using the empirical formula:

$$n = \frac{\ln(D_o/D_n)}{\ln[1 + 0.33(1 + 0.046T)\,Z]}$$

[taken from "Low Maintenance Mechanically Simple Wastewater Treatment Systems, " by Linvil G. Rich.]

where;

n = number of steps required

D_o = oxygen saturation deficit of the wastewater at the top of cascade

D_n = oxygen saturation deficit of the wastewater at the bottom of cascade

T = temperature of wastewater in °C

Z = height of step in meters

Based on an elevation 1700 feet above sea level the atmospheric pressure will be approximately 716 mm. Assuming a temperature of 22°C the oxygen saturation of water is 8.2 mg/l. Considering the DO concentration will be low at the top of the cascade, we have assumed a level of 1.0 mg/l and 6.0 mg/l at the bottom. Using an 18" height between steps, the number of steps needed can be determined to be approximately:

$$n = \frac{\ln(D_o/D_n)}{\ln[1 + 0.33(1 + 0.046T)\ Z]}$$

$D_o = 8.2 - 1 = 7.2$
$D_n = 8.2 - 6.0 = 2.2$
$T = 22°C$
$Z = 18$ inches $= 0.46$ meters

$$n = \frac{\ln(7.2/2.2)}{\ln[1 + 0.33(1 + 0.046(22))\ 0.46]}$$

$n = \frac{1.61}{0.267}$ => $n = 6.03$ steps (as a minimum)

Adding a seventh step as a safety factor, the total drop in the cascade aerator will be approximately (7 x 1.5) 10.5 feet. As stated above the cascade aerator requires a thin layer of wastewater flowing over the steps. At the peak design flow rate of 295 GPM (0.657 CFS) and a water layer of less than 1.0 inch (0.0833 ft), the required surface area of each step will need to be a minimum of 7.88 square feet. For easy of construction the steps will be 3.0 feet wide with a 3.0 feet tread (9.0 sq ft). A 6-inch overhang will be over each step to insure that the water drops and does not flow down the wall (rise) of the step.

HYDRAULICS
OF
TREATMENT PLANT DESIGN, TAFF
PUMP STATION
AND
GRAVITY SEWERS

Hydraulics for Bledsoe County Prison (Southeast Regional Correctional Facility)
Feb-09

1 Main pump station and Headworks

Influent invert 1692 Elevation

n (roughness coef): 0.013

Channel - Rectangular

Invert out in feet	Invert in, In feet	Length of channel in feet	Width of Channel in feet	Water depth in inches	Slope	Area in Square Feet	Hydraulic Radius	Flow in cfs	Flow in MGD	Flow in gpm	Velocity in ft per sec	Water level Elevation (ft)
1692	1691.75	15	2	1.74	0.0167	0.29	0.1266376	1.08	0.70	485.32	3.728864	1692.15
1692	1691.75	15	2	2.05	0.0167	0.342	0.1459075	1.40	0.90	628.44	4.098331	1692.17

2 Course Bar Screen

HL = B (W/b) ^(4/3) x Hv x sin θ

HL = Headloss (in feet)

Bf = bar shape factor (2.42 for sharp edge rectangular bar, 1.83 for rectangular bar with semicircle upstream, 1.79 for circular bar and 1.67 for rectangular bar with both u/s and d/s face as semicircular).

W = Maximum width of bar (in feet)

b = Minimum space between bars (in feet)

Hv = velocity head of flow approaching bar screen - $(V^2/2g)$ - ft/sec

θ = angle of inclination of rack with horizontal

Width of Bars (inches)	Spacing Between Bars (inches)	Channel Width (in feet)	θ (Degrees)	Flow in MGD	Approach Velocity in fps	Distance (in) between Cline of bars	Number of Bars in Channel	W (in feet)	b (in feet)	Hv (fps)	Bf	Head Loss (feet)
0.375	1.25	1.5	60	0.32	2.406544	1.625	10	0.3125	1.1875	0.089929	1.83	0.024035
0.375	1.25	1.5	60	0.425	2.406544	1.625	10	0.3125	1.1875	0.089929	1.83	0.024035

3 Fine Screen

HL = 0.01553 (Q/CA)

HL = Headloss (in feet)

Q = Flow in cubic feet per sec

C = coefficient of discharge (typical value 0.6)

A = effective submerged open area (in sq ft)

Channel Width (in feet)	θ (Degrees)	Water Depth (in inches)	A = area (in sq ft)	Flow (in cfs)	C (coefficient) of discharg.	HL (Headloss) in feet
1	50	1.98	0.215392203	0.5	0.6	0.060084193
1	50	2.38	0.258905779	0.66	0.6	0.065981532

4 Parshall Flume (3" - Parshall Flume)

$HL = (Qx1.00806)^{0.6464}$

HL = Headloss (in feet)

Q = Flow in cfs

Flow (Q) in MGD	Flow (Q) in gpm	Flow (Q) in cfs	HL Headloss (in feet)
0.25	173.6	0.39	0.5440

0.322	223.6	0.50	0.6407
0.425	295.1	0.66	0.7666

Total headloss through headworks = Course Bar Screen + Fine Screen + Parshall Flume
Total Head = 0.7249 feet at design flow
Total Head = 0.8567 feet at peak design flow

5 Wetwell

The wet well influent pipe invert will be set below the influent invert elevation of the headworks by a value equal to the total headloss at the peak flow plus 10%

Influent Invert elevation: 1692
Headloss w/10%: 0.942322
Wetwell influent elevation: 1691.00
Average daily flow 0.322 MGD 223.6111 GPM
Peak daily flow: 0.425 295.1389 GPM

Wet well size will be set for maximum capacity equal to 30 minutes of average daily flow

Wet well capacity = 6708.333 gallons
896.836 cubic feet
depth of wet well below inver. 8 feet
Surface area: 128.1194 square feet
Wet well width: 10 feet
Wet well length: 13 feet (rounded up)
Bottom of Wetwell: 1682.00 1 ft safety factor

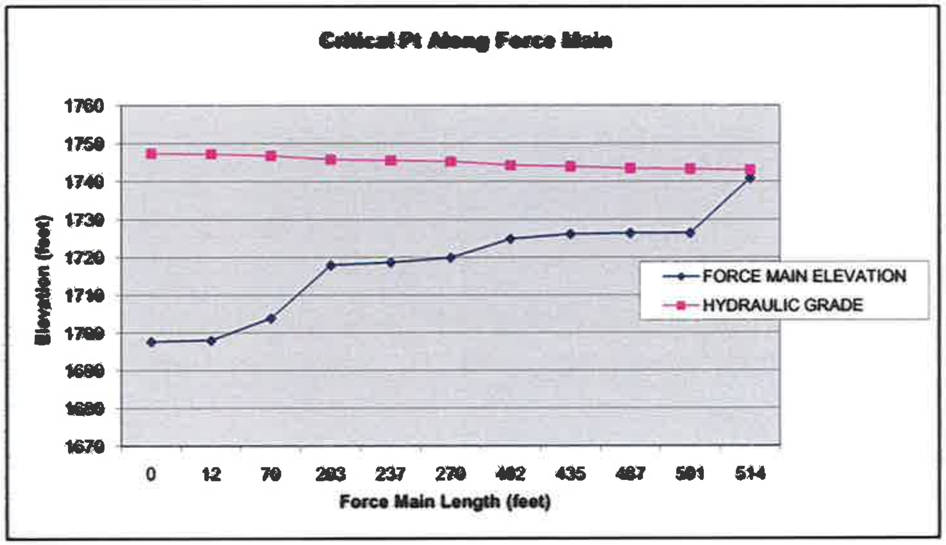
MAIN PUMP STATION

Influent invert to wet well: 1690.50
bottom elevation of wet well: 1682.00

FLOW (gpm)	295	PIPE ROUGH.COEFF.for PUMP STATION	120
PIPE DIAMETER of Pump Sta. (inches)	4	PIPE ROUGH.COEFF.for FORCE MAIN	130
LEAD PUMP CUT OFF ELEVATION:	1685.00	HEADLOSS PER 100 FEET AT FLOW (P.S.)	6.49
C.LINE OF PUMP INLET ELEVATION:	1706.42	HEADLOSS PER 100 FEET AT FLOW (F.M.)	0.78
PIPE DIAMETER of Force Main (inches)	6	GROUND ELEVATION at MAIN P.S.	1708
HEAD FOR PUMP AT FLOW	71.5	ATMOSPHERIC PRESS. AVAILABLE (ft.)	31.99

DESCRIPTION	EQUIV. OR ACTUAL LENGTH	ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	ACCUM. PIPE FRICTION HEADLOSS	HYDRAULIC GRADE
PUMP STATION								
PUMP SUCTION INTAKE	0.54	1682.42	0.5	0.44	0.44	0.04	0.04	1684.52
ELBOW - 90 DEG.	0.58	1682.96	0.4	0.35	0.79	0.04	0.07	1684.13
RISER PIPE	22.5	1705.46		0.00	0.79	1.46	1.53	1682.67
ELBOW - 90 DEG.(turn into pump)	0.58	1706	0.4	0.35	1.15	0.04	1.57	1682.28
STRAIGHT PIPE	1.5	1706		0.00	1.15	0.10	1.67	1682.19
Pump Discharge (top)	0.75	1706.75	2	1.76	2.91	0.05	1.72	1751.88
CHECK VALVE	1	1707.75	1	0.88	3.79	0.06	1.78	1750.93
PLUG VALVE	0.75	1708.5	2.5	2.20	5.99	0.05	1.83	1748.68
ELBOW - 90 DEG.(turn)	0.58	1709.04	0.4	0.35	6.34	0.06	1.89	1748.26
STRAIGHT PIPE	0.75	1709.04		0.00	6.34	0.05	1.94	1748.21
TEE - SIDE (4" x 4" x 6")	0.58	1708.37	0.4	0.35	6.69	0.04	1.98	1747.82
STRAIGHT PIPE (6")	10	1698.37		0.00	6.69	0.08	2.06	1747.75
ELBOW - 90 DEG. (turn - 6")	0.79	1697.7	0.4	0.07	6.76	0.01	2.06	1747.67
STRAIGHT PIPE (6")	9	1697.7		0.00	6.76	0.07	2.13	1747.60
				0.00	6.76	0.00	2.13	1747.60
suction line	24.2							
discharge	26.28							

DESCRIPTION	STATION	LENGTH	FORCE MAIN ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	HYDRAULIC GRADE	
5 ft. outside station	0+00	0	1697.7		0.00	0.00	0.00	1747.60	
45 deg. bend	0+12	12	1698	0.22	0.04	0.04	0.09	1747.47	
	0+70	70	1704		0.00	0.04	0.54	1747.02	
	2+03	203	1718		0.00	0.04	1.58	1745.98	
45 deg. bend	2+37	237	1718.75	0.22	0.04	0.08	1.84	1745.68	
	2+70	270	1720		0.00	0.08	2.10	1745.42	
	4+02	402	1725		0.00	0.08	3.13	1744.40	
45 deg. bend	4+35	435	1726.25	0.22	0.04	0.11	3.38	1744.10	
90 deg. bend	4+87	487	1726.5	0.45	0.08	0.19	3.79	1743.62	
90 deg. Bend	5+01	501	1726.5	0.45	0.08	0.27	3.90	1743.43	
Outlet into splitter box	5+14	514	1741	1	0.17	0.45	4.00	1743.16	Critical point

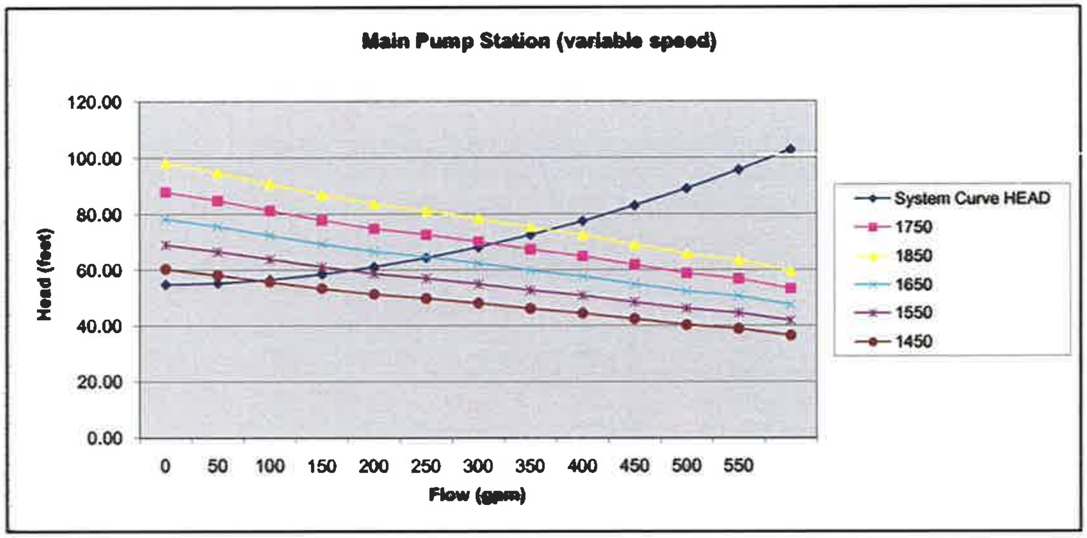


Owner:	**SERCF**
Project Title:	**MAIN PUMP STATION**
Type of Pump:	**GORMAN RUPP - SELF PRIMING**
Model No.:	**MODEL - T4A-B**
PUMP CURVE:	9 INCH IMPELLER
PUMP SPEED:	1750 RPM
IMPELLER NO.:	
Minimum Flow (gpm):	295

Assumed water level in pump station:	1686	
Critical force main elevation:	1741	
Force main length to critical elevation:	514	including fittings - equivalent pipe
Equivalent ft of pipe at pump station:	1143.85	
Diameter (in) of pump staion piping:	4	
Diameter (in) of force main piping:	6	
Static Head:	55	

PIPE ROUGH.COEFF.for PUMP STATION	120
PIPE ROUGH.COEFF.for FORCE MAIN	130
HEADLOSS PER 100 FEET AT FLOW (P.S.)	6.49
HEADLOSS PER 100 FEET AT FLOW (F.M.)	0.78

	PUMP SPEED - RPM							System Curve HEAD (FT)
	1750		1850	1650	1550	1450	1350	
	FLOW (GPM)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	
Shutoff	0	88.1	98.5	78.3	69.1	60.5	52.4	55.00
	50	85	95.0	75.6	66.7	58.4	50.6	55.48
	100	81.5	91.1	72.5	63.9	56.0	48.5	56.74
	150	78	87.2	69.3	61.2	53.5	46.4	58.69
	200	75	83.8	66.7	58.8	51.5	44.6	61.26
	250	72.8	81.4	64.7	57.1	50.0	43.3	64.50
	300	70.3	78.6	62.5	55.1	48.3	41.8	68.31
	350	67.5	75.4	60.0	53.0	46.3	40.2	72.70
	400	65	72.6	57.8	51.0	44.6	38.7	77.65
	450	62	69.3	55.1	48.6	42.6	36.9	83.17
	500	59	65.9	52.4	46.3	40.5	35.1	89.23
	550	57	63.7	50.7	44.7	39.1	33.9	95.83
	600	53.5	59.8	47.56	41.97	36.73	31.84	102.96


Main Pump Station (variable speed)

SPLITTER BOX

This structure will be used to split the incoming flow to the SBR units.
The flow will be split evenly by the use of rectangular weirs on either side of the inlet to the box.

The flow will enter the basin (splitter box) from the main pump station force main. The influent weir elevation will be set at 1741.0 feet. The ground elevation at the structure is approximately 1726.5.
Rock was found at approximately 1722.5 The SBR locations showed rock at elevations of approximately 1718.83 and 1721 feet.
Some rock excavation will be required to construct SBR basins at the same elevation (1720), so set splitter box elevation at 1720.

ADF = 322,000 GPD (224 GPM)
PDF = 425,000 GPD (295 GPM)

Setting two rectangular weirs at an elevation of 1741 feet and the top of the structure at 1744, approximately 112 GPM would flow over both of them at the average daily flow rate and 147.5 GPM at the peak daily flow rate.

The empircal equation $Q = KLH^{1.5}$ can be used to determine an approximate weir length to use on the splitter box
where Q = flow in gpm
K = 1495
L = length of weir in feet
H = height of water over the weir in feet

weir length (in feet)	Water level over weir (in inches)	Flow (in GPM)	Flow in CFS	Velocity over weir fps
1	2	101.72	0.23	1.36
1	2.5	142.16	0.32	1.52
1	3	186.88	0.42	1.67
1	4	287.71	0.64	1.92
1.5	1.5	99.11	0.22	1.18
1.5	2	152.58	0.34	1.36
1.5	2.5	213.24	0.48	1.52
1.5	3	280.31	0.63	1.67
2	2	203.44	0.45	1.36
2	2.5	284.32	0.63	1.52
2	3	373.75	0.83	1.67

To keep the organics suspended and not settle out, the velocity needs to be maintained over 1.0 fps. The closer to 2.0 fps the better to keep the inorganics suspended.

Thus use 1.5 ft weirs set at: 1741.00 feet

weir length (in feet)	Water level over weir (in inches)	Flow (in GPM)	Flow in CFS	Velocity over weir fps	Water Elevation (feet)
1.5	1.63	112.26	0.25	1.23	1741.14
1.5	1.96	148.03	0.33	1.35	1741.16
1.5	3.11	295.87	0.66	1.70	1741.26

At the ADF = 224 gpm (112 gpm per weir) the water level will be: 1741.14
At the PDF = 295 gpm (147.5 gpm per weir) the water level will be: 1741.16
At the maintenance flow of 295 gpm over one weir the water level will be: 1741.26

Lines to SBR tanks will be 6-inch in size. When both basins are in operation the ADF in each line will be 161,000 GPD or 112 gpm. At the PDF rate of 425,000 GPD, the flow through each pipe will be approximately 212,500 GPD or 295 gpm.

The pipe length and/or equivalent pipe length to the SBR's is found as follows:

Roughness factor "C" : 120

Hdf = friction head in feet of liquid per 100 feet of pipe

Fhd = Headloss in fitting = $k(V^2/2g)$

Pipe fittings	"K" values	Pipe Diameter (inches)	Flow (GPM)	Pipe Fric. Headloss Hdf	Headloss in fittings Fhd	Equivalent length of pipe (feet)
Pipe inlet	0.5	6	295	0.90215214	0.0871522	10
90° bends	0.42	6	295	0.90215214	0.0732078	9
45° bends	0.22	6	295	0.90215214	0.0383469	5
Pipe outlet	1	6	295	0.90215214	0.1743043	20

total piping between splitter box and SBR tank

Type of fitting or pipe	Equivalent pipe
inlet to pipe	10
straight pipe	22
90° bend	9
straight pipe	34
45° bend	5
straight pipe	19
90° bend	9
straight pipe	22
90° bend	9
straight pipe	3
Total pipe	**142**

Headloss in pipe to SBR unit

Invert pipe elevation into SBR units 1739.08

Pipe Diameter (inches)	Flow (GPM)	Pipe Fric. Headloss Hdf/100 ft	Total Equivalent Pipe	Total Headloss in Pipe Seg	Elevation of Water in Splitter	
6	295	0.9021521	142	1.28	1740.36	below concrete weir opening at splitter box (1740.8)
6	112	0.1503704	142	0.21	1739.29	below pipe inlet at splitter box (1739.55)
6	148	0.2518217	142	0.36	1739.44	below pipe inlet at splitter box (1739.55)

SBR treatment units
Sequencing Batch Reactors

These two structures will be used to biologically treat the organic content of the wastewater.
The basins will also allow physical settling of the wastewater and decant to the treated contents.

The maximum water depth in each basin will be 20 feet.
The bottom of this structure has been set at Elevation 1719.0, thus the top water level will be El 1739.0

The 6" influent line coming into each basin will be set with a center line elevation of 1939.33 (invert El approx. 1939.06)

The basins will be operated in batch cycles that consists of filling, aeration, settling and decanting.
The decanter in each basin will be lowered into the basin automatically by an electric operator controlled by the SBR PLC.
The decant cycle will take approximately 72 minutes (1.2 hrs) during normal average daily flows.
During maintenance cycling the decant rate will be reduced to 36 minutes (0.6 hrs).
The flow rates expected are as follows:

		Flow per Basin		Cycles per day	Flow per Cycle	Cycle Time/day	decant Cycle time	Flow rate GPM
Average daily flow:	322,000	161,000	GPD	5	32,200	4.8	72	447.2
Peak daily flow:	425,000	212,500	GPD	5	42,500	4.8	72	590.3
Maintenance flow:	322,000	322,000	GPD	10	32,200	2.4	36	894.4
Maintenance peak flow:	425,000	425,000	GPD	10	42,500	2.4	36	1180.6

The decanter outlet pipe has been set at an elevation of 1735.3 ft. A swivel joint will to placed on the pipe at this point allowing the decanter to pivot up to an elevation of 1739.75 (above top water level).
The outlet pipe will be sloped to the post equalization basin where the decant will be stored until it is pumped to the effluent filters.
The invert elevation of the influent line to the post equalization basin has set at 1734.0 feet. The distance from the decant outlet pipe to the post equalization basin is approximately 24 feet.
The outlet pipe is 8" diameter ductile iron pipe. This pipe has an outside diameter of 9.05 inches and a wall thickness of approximately 0.333 inches. Thus, the inside diameter is approximately 8.38".
The invert elevation of the outlet pipe is therefore approximately 1735.3 - (8.38/24) = 1734.95 feet
The difference in elevation of these two pipes is (1734.95 - 1734.0) = 0.95 ft. The slope of the line is therefore (0.95/24) 0.3958 or 39.58%.

The flow through the 8" pipe at this slope is found as follows:
The capacity of a PVC sewer line can be calculated using the empirical formula:

Q = (1.49/n) A R^2/3 S^1/2
where n = roughness coefficient (0.0115 for PVC pipe per State of TN design criteria)
A = cross sectional area of the water in the pipe
R = the hydraulic radius (cross sectional area of the water/wetted perimeter of the pipe)
S = slope of the pipe
Q = flow in cubic feet per sec
n = 0.0115 thus, (1.49/n) = 129.5652

Pipe Segment No. and MH No.	Pipe Diameter (inches)	Water Depth in pipe (inches)	Upstream Invert Elev. (ft.)	Downstream Invert Elev. (ft.)	Distance Between Invert (ft.)	Slope in Pipe (vert ft./ horizft.)	Angle θ (radians)	Wetted Perimeter (ft.)	Cross Sectional Area (sq ft)	Hydraulic Radius (ft.)	Flow (Q) in pipe (cu. ft./sec.)	Flow (Q) in pipe (gpm)	Flow (Q) in pipe (MGD)
SBR outlet	8	3.35	1734.95	1734	24	0.03958	2.81515	0.93838	0.13858	0.14768	0.997	447.65	0.645
SBR outlet	8	3.92	1734.95	1734	24	0.03958	3.10159	1.03386	0.17009	0.16452	1.316	590.46	0.850
SBR outlet	8	6.33	1734.95	1734	24	0.03958	4.38519	1.46173	0.29623	0.20266	2.633	1181.78	1.702
SBR outlet	8	8	1734.95	1734	24	0.03958	6.28319	2.09440	0.34907	0.16667	2.723	1222.30	1.760

As can be seen the capacity needed for the decant flow rate of 1180 gpm can be handled by the decant outlet pipe at approximately 6.33 inches of depth in the pipe.
Therefore water will not backup in the outlet pipe past elevation 1734.95 + (6.33/12) = 1735.48 feet unless the water level in the post equalization basin goes over the invert inlet.

POST EQUALIZATION BASIN

This basin will be used to receive the decant flows from the two SBR units. The volume of this tank needs to be sufficient to handle the flow released from the SBR tanks during the decant cycle.

The flow rates expected are as follows:

		Flow per Basin		Cycles per day	Flow per Cycle	Cycle Time/day (hours)	decant Cycle time (minutes)	Flow rate GPM	Time Between Decants (hours)	Design Flow rate to Filter GPM	Flow rate Added to EQ Basin	Volume Added to Basin (Gallons)	Volume Pumped to filters between Decants
Average daily flow:	322,000	161,000	GPD	5	32,200	4.8	72	447.2	1.8	300	147.2	10,600	32,400
Peak daily flow:	425,000	212,500	GPD	5	42,500	4.8	72	590.3	1.8	300	290.3	20,900	32,400
Maintenance flow:	322,000	322,000	GPD	10	32,200	2.4	36	894.4	1.8	300	594.4	21,400	32,400
Maintenance peak flow:	425,000	425,000	GPD	10	42,500	2.4	36	1180.6	1.8	300	880.6	31,700	32,400

Based on the above calculations, the equalization basin needs to have a volume of approximately 31,700 gallons.

The size of the equalization basin can be determined as follows:

Size of basin: 31,700 gallons
Size of basin: 4,238 cubic feet
Setting basin bottom Elev.: 1720.00 feet
Inlet invert Elev.: 1734.00 feet
Depth: 14.00 feet
Surface Area: 302.71 square feet
Diameter: 19.632241 feet

The diameter of this tank was set for 19.5 feet since the maintenance peak flow would only occur if one of the basins is not operated for maintenance purposes and the peak flow occurs due to a high flow rain event. Maintenance will normally be done during dry weather.

Set Diameter: 19.5 feet
Surface Area: 298.65 square feet
Depth to Invert Inlet: 14.00 feet
Basin Volume: 4,181.07 cubic feet
Basin Volume: 31,274.38 gallons

Expected water levels in the Equalization basin immediately after decant cycle:

				Volume Added to Basin (Gallons)	Water Level after decant (elevation)	
Average daily flow:	322,000	161,000	GPD	10,600	1724.75	below EQ inlet invert
Peak daily flow:	425,000	212,500	GPD	20,900	1729.36	below EQ inlet invert
Maintenance flow:	322,000	322,000	GPD	21,400	1729.58	below EQ inlet invert
Maintenance peak flow:	425,000	425,000	GPD	31,700	1734.19	below SBR outlet invert (1734.95) and water would be pumped to filters before next decant cycle

EFFLUENT FILTER PUMPS

The pumps used to transport the decant from the post equalization basin to the filters will be housed in the chemical building.
The floor slab for this building is set at 1720.0 feet. The pumps have been set on a concrete pad at an influent
center line elevation of 1722.46 ft. This is 2.45 feet above the bottom of the equalization basin.
Thus, the pumps used for this will be suction lift pumps that do not require a flooded suction.
The water level in the filter channel will be at a minimum at the effluent weir elevation of 1738.42 feet.
Normally the water level will be at an elevation of 1740.42 feet which would put a head of approximately
[1740.42 - 1722.46] = 17.96 ft on the pump.

Minimum water in EQ basin:	1720.50	feet
Pump intake elevation	1722.46	feet

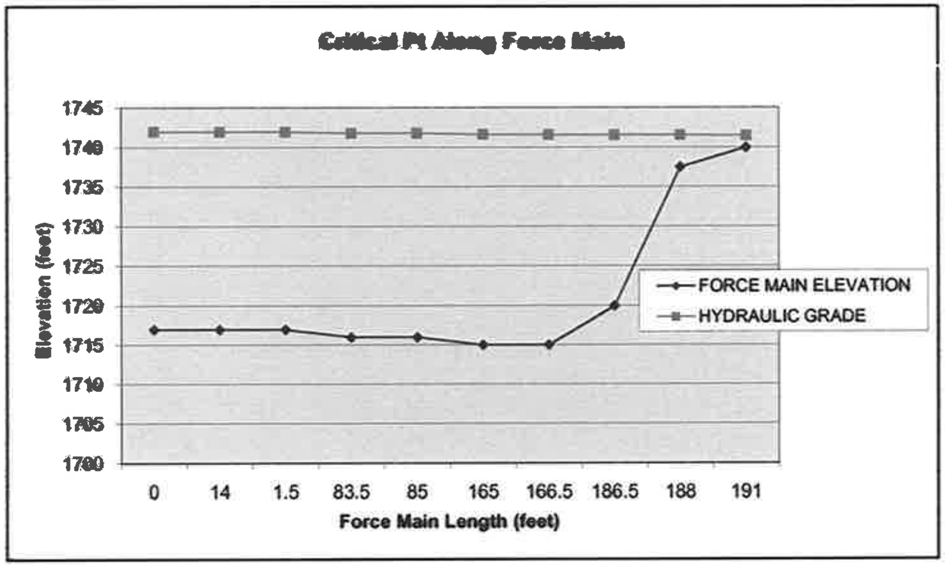
FLOW (gpm)	300	PIPE ROUGH.COEFF.for PUMP STATION	120
PIPE DIAMETER of Pump Sta. (inches)	4	PIPE ROUGH.COEFF.for FORCE MAIN	130
LEAD PUMP CUT OFF ELEVATION:	1720.50	HEADLOSS PER 100 FEET AT FLOW (P.S.)	**6.69**
C.LINE OF PUMP INLET ELEVATION:	1722.46	HEADLOSS PER 100 FEET AT FLOW (F.M.)	**0.20**
PIPE DIAMETER of Force Main (inches)	8	GROUND ELEVATION at Pumps	1720
HEAD FOR PUMP AT FLOW	29	ATMOSPHERIC PRESS. AVAILABLE (ft.)	**31.94**

DESCRIPTION		EQUIV. OR ACTUAL LENGTH	ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	ACCUM. PIPE FRICTION HEADLOSS	HYDRAULIC GRADE
PUMP STATION									
PUMP SUCTION INTAKE	Vdown	3.29	1720	0.5	**0.03**	**0.03**	**0.01**	**0.01**	**1720.47**
ELBOW - 90 DEG.		1.5	1716.71	0.4	**0.02**	**0.05**	**0.00**	**0.01**	**1720.44**
STRAIGHT PIPE		92			**0.00**	**0.05**	**0.18**	**0.19**	**1720.26**
ELBOW - 45 DEG.		1.1		0.22	**0.01**	**0.06**	**0.00**	**0.19**	**1720.24**
STRAIGHT PIPE		15			**0.00**	**0.06**	**0.03**	**0.22**	**1720.21**
STRAIGHT PIPE		6.5			**0.00**	**0.06**	**0.01**	**0.24**	**1720.20**
ELBOW - 90 DEG.		1.5	1716.71	0.4	**0.02**	**0.09**	**0.00**	**0.24**	**1720.17**
STRAIGHT PIPE	Vup	5			**0.00**	**0.09**	**0.01**	**0.25**	**1720.16**
TEE - SIDE		1.33	1722.29	1.8	**0.10**	**0.19**	**0.00**	**0.25**	**1720.06**
STRAIGHT PIPE		1.33			**0.00**	**0.19**	**0.00**	**0.25**	**1720.06**
ELBOW - 90 DEG.		1.5		0.4	**0.02**	**0.21**	**0.00**	**0.26**	**1720.03**
PLUG VALVE		0.54		1	**0.06**	**0.27**	**0.00**	**0.26**	**1719.97**
STRAIGHT PIPE		3	1722.29		**0.00**	**0.27**	**0.01**	**0.26**	**1719.97**
ELBOW - 90 DEG.		1		0.4	**0.02**	**0.29**	**0.00**	**0.27**	**1719.94**
8" x 4" reducer		1.42		0.4	**0.36**	**0.66**	**0.10**	**0.36**	**1719.48**
PUMP INTAKE			1722.29		**0.00**	**0.66**	**0.00**	**0.36**	**1719.48**
PUMP DISCHARGE			1723.11		**0.00**	**0.66**	**0.00**	**0.36**	**1748.48**
CHECK VALVE		1		2.5	**2.28**	**2.93**	**0.07**	**0.43**	**1746.14**
PLUG VALVE		0.54		1	**0.91**	**3.84**	**0.04**	**0.46**	**1745.19**
TEE - SIDE		1.33		1.8	**1.64**	**5.48**	**0.09**	**0.55**	**1743.46**
STRAIGHT PIPE		1.33			**0.00**	**5.48**	**0.09**	**0.64**	**1743.37**
ELBOW - 90 DEG.	Vdown	1.5		0.4	**0.36**	**5.85**	**0.10**	**0.74**	**1742.91**
STRAIGHT PIPE		1.5			**0.00**	**5.85**	**0.10**	**0.84**	**1742.81**
ELBOW - 90 DEG.	Horizontal	1.5		0.4	**0.36**	**6.21**	**0.10**	**0.94**	**1742.34**
STRAIGHT PIPE		2			**0.00**	**6.21**	**0.13**	**1.08**	**1742.21**

4" Magmeter		0.75	1721.46	0.01	0.01	6.22	0.05	1.13	1742.15
STRAIGHT PIPE		1.25			0.00	6.22	0.08	1.21	1742.07
ELBOW - 90 DEG. increaser	Vdown	1.5		0.35	0.02	6.24	0.00	1.21	1742.04
STRAIGHT PIPE	Vdown	4			0.00	6.24	0.01	1.22	1742.04
ELBOW - 90 DEG.	Horizontal	1.5	1716.71	0.4	0.02	6.26	0.00	1.23	1742.01
STRAIGHT PIPE		5			0.00	6.26	0.01	1.24	1742.00

DESCRIPTION	STATION	LENGTH	FORCE MAIN ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	HYDRAULIC GRADE	
Starting at increaser	0+00	0	1717		0.00	0.00	0.00	1742.00	
Straight pipe - 14 ft	0+14	14	1717		0.00	0.00	0.03	1741.97	
90 deg. bend	0+15.5	1.5	1717	0.4	0.02	0.02	0.00	1741.97	
Straight pipe - 68 ft.	0+83.5	83.5	1716		0.00	0.02	0.17	1741.81	
90 deg. bend	0+85	85	1716	0.4	0.02	0.05	0.17	1741.79	
Straight pipe - 80 ft.	1+65	165	1715		0.00	0.05	0.33	1741.63	
90 deg. Bend	1+66.5	166.5	1715	0.4	0.02	0.07	0.33	1741.60	
Vertical pipe - 20 ft.	1+86.5	186.5	1720		0.00	0.07	0.37	1741.56	
90 deg. Bend	1+88	188	1737.5	0.4	0.02	0.09	0.37	1741.54	
Discharge into influent channel	1+91	191	1740	1	0.06	0.15	0.38	1741.47	Critical point

Water level is approximately
1740



Maximum water level in EQ basin is 1734 and lowest 1721.

Owner: **SERCF**
Project Title: **FILTER PUMPS**
Type of Pump: **GORMAN RUPP - SELF PRIMING**

Water level in Post EQ Basin: 1730

Model No.: **MODEL - T4A-B**

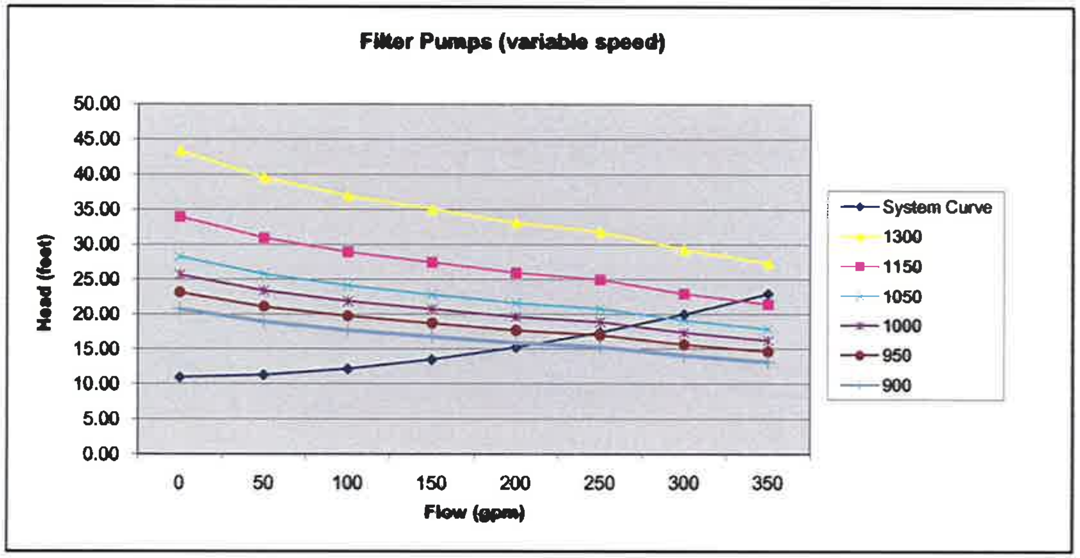
PUMP CURVE: 8.5 INCH IMPELLER
PUMP SPEED: 1750 RPM
IMPELLER NO.:

Minimum Flow (gpm): 300

Critical force main elevation:	1741	
Force main length to critical elevatio	191	including fittings - equivalent pipe
Equivalent ft of pipe at pump station:	112.08	
Diameter (in) of pump staion piping:	4	
Diameter (in) of force main piping:	6	
Static Head:	11	

PIPE ROUGH.COEFF.for PUMP STATION	120
PIPE ROUGH.COEFF.for FORCE MAIN	130
HEADLOSS PER 100 FEET AT FLOW (P.S.)	**6.69**
HEADLOSS PER 100 FEET AT FLOW (F.M.)	**0.80**

	PUMP SPEED - RPM							System Curve HEAD (FT)
	1150		1300	1050	1000	950	900	
	FLOW (GPM)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	
Shutoff	0	34	43.4	28.3	25.7	23.2	20.8	**11.00**
	50	31	39.6	25.8	23.4	21.2	19.0	**11.33**
	100	29	37.1	24.2	21.9	19.8	17.8	**12.18**
	150	27.5	35.1	22.9	20.8	18.8	16.8	**13.51**
	200	26	33.2	21.7	19.7	17.7	15.9	**15.27**
	250	25	31.9	20.8	18.9	17.1	15.3	**17.45**
	300	23	29.4	19.2	17.4	15.7	14.1	**20.03**
	350	21.5	27.5	17.9	16.3	14.7	13.2	**23.01**
	400	19.25	24.6	16.0	14.6	13.1	11.8	**26.38**
	450	17	21.7	14.2	12.9	11.6	10.4	**30.12**
	500	14	17.9	11.7	10.6	9.6	8.6	**34.24**
	550		0.0	0.0	0.0	0.0	0.0	**38.72**
	600		0.0	0.0	0.0	0.0	0.0	**43.56**



Final Upflow Effluent Sand Filters

This treatment unit will be used to remove suspended solids in the effluent, phosphorus in the form of solids and total nitrogen. Three cells will be provided, each with 50 square feet of surface area and operate at a design flow rate of 2.0 gallons per minute per square foot .

The design of the filters recommends a total height of 23.5 feet from the top of the bottom slab. Flow is pumped to an influent channel that is needed to provide the head to push the wastewater through the sand filter.

To allow the possible use of the sand filter as a biological process to remove nitrogen in the form of nitrates and nitrites the filter is given an extra depth of 3 to 4 feet to allow the growth of microorganisms in a portion of the filter. The manufacter recommends that the effluent weir be set at a level approximately 18.42 feet above the bottom slab floor. The influent pipe to the filter is set at a level that is approximately 17.27 feet above the bottom slab. The influent piping is connected to a channel that supplies the water to the filters. The channel walls must be sufficient in height to provide a water head over the filters. The normal headloss through the filter is approximately 18" to 24", thus the influent channel should be at least 2 feet higher than the effluent weir (18.42). That would raise the water level in the influent channel to 20.42 feet above the floor slab. If one cell is taken out of service for maintenance and the other two cells left to filter all of the flow the flow rate will increase to approximately 3 gpm per square foot and increase the headloss through the filters by another 9 to 12 inches. Adding 2 feet of freeboard increases the structure to approximately [20.42 + 1(additional headloss for maintenance) + 2(freeboard)] 23.42 feet above the floor slab. Thus the reasoning for the 23.5 feet height. Settling the bottom floor slab to 1720.00 feet will make the top of the structure 1743.5 feet. The bottom of the influent channel must be below the 8" influent pipe to the filter. A valve will be used to turn off the flow going to each filter. The floor of the influent channel must also be below the valve. Settling the center line of the influent pipe at an elevation of 1737.27 feet requires that the channel bottom be approximately 8.5-inches (0.71 feet) lower or approximately 1736.56 feet. Under normal operating flows the water level in the channel will be at a minimum level of approximately [1720.00 + 18.42 + 1.5] 1739.92

CASCADE AERATOR AND UV DISINFECTION

The flow leaving the filters will be transported to the inline UV unit located in the new chemical building. This UV unit will be mounted to the floor (elevation 1720.0 ft) and have an approximate elevation of 1722.76 on the influent and effluent piping.

The water leaving the filters will come from the effluent weirs and the supporting piping. The center line of the effluent pipe is set for an elevation of 1736.28 feet. The water line will convey the filter water to the UV unit in the chemical building by an 8" DIP. As the wastewater flows through the UV unit, the effluent line will transport the water to the cascade aerator that has a 45° V-notch set an an elevation of 1727.0 feet. This will keep the line full at all times (below 1727) and allow a small pump to be connected to the line to provide a non-potable water supply.

Cascade aerator effluent weir (45° V-notch) set at an elevation of 1727.0 ft.
Empirical equation to determine flow over this weir: GPM = 464.5 $H^{2.5}$

Height over weir Inches	**Flow over V-Notch GPM**	**Height over weir (feet)**	**Elevation (feet)**	
0.5	0.16	0.04	1727.04	
1	0.93	0.08	1727.08	
2	5.27	0.17	1727.17	
3	14.52	0.25	1727.25	
4	29.80	0.33	1727.33	
5	52.05	0.42	1727.42	
6	82.11	0.50	1727.50	
7	120.72	0.58	1727.58	
8	168.56	0.67	1727.67	
9	226.28	0.75	1727.75	
10.01	295.20	0.83	1727.83	Maximum water level at the peak design flow
10.0748	300.00	0.84	1727.84	

Piping layout from filters to chemical building and then to cascade aerator.

Filters effluent center line elevation: 1736.28
Cascade aerator water elevation:

FLOW (gpm)	300	PIPE ROUGH.COEFF.for LINE No. 1	120	0.67	FLOW (in
PIPE DIAMETER (inches)	8	PIPE ROUGH.COEFF.for LINE No. 2	130	0.35	PIPE ARE
WATER LEVEL ELEVATION:	1730.81	HEADLOSS PER 100 FEET AT FLOW (Line 1)	0.20	1.91	VELOCITY
C.LINE OF PIPE OUTLET ELEVATION:	1736.28	HEADLOSS PER 100 FEET AT FLOW Line 2)	0.80		
CHANGE IN PIPE DIAMETER of LINE (inches)	6				
HEAD FOR PIPE AT FLOW				0.20	PIPE ARE
				3.40	VELOCITY

DESCRIPTION	EQUIV. OR ACTUAL LENGTH	ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	ACCUM. PIPE FRICTION HEADLOSS	HYDRAULIC GRADE
FILTERS								
PIPE OUTLET	2	1736.28	0.5	0.03	0.03	0.00	0.00	1730.78
8" ELBOW - 90 DEG.	0.91	1735.53	0.4	0.02	0.05	0.00	0.01	1730.75

VERTICAL PIPE DOWN	19	1716.53		0.00	0.05	0.04	0.04	1730.72	
8" ELBOW - 90 DEG.	0.91	1715.78	0.4	0.02	0.07	0.00	0.05	1730.69	
STRAIGHT PIPE (horiz.)	1.21	1715.78		0.00	0.07	0.00	0.05	1730.69	
8" x 8" x 4" Tee (run)	1.5	1715.78	0.3	0.02	0.09	0.00	0.05	1730.67	
STRAIGHT PIPE (horiz.)	1.21	1715.78		0.00	0.09	0.00	0.05	1730.67	
8" x 8" x 4" Tee (run)	1.5	1715.78		0.00	0.09	0.00	0.06	1730.66	
STRAIGHT PIPE (horiz.)	5.5	1715.78	0.3	0.02	0.11	0.01	0.07	1730.64	
8 ELBOW - 90 DEG.(turn)	0.91	1715.78	0.4	0.02	0.13	0.00	0.07	1730.61	
STRAIGHT PIPE (horiz.)	51	1715.78		0.00	0.13	0.10	0.17	1730.51	
8" ELBOW - 90 DEG.(turn)	0.91	1715.78	0.4	0.02	0.15	0.00	0.17	1730.48	
STRAIGHT PIPE (horiz.)	40	1715.78		0.00	0.15	0.08	0.25	1730.41	
8" x 8" x 6" Tee (run)	1.5	1715.78	0.3	0.02	0.17	0.00	0.25	1730.39	
STRAIGHT PIPE (horiz.)	11.5	1715.78		0.00	0.15	0.02	0.19	1730.46	
8" ELBOW - 90 DEG.(UP)	0.91	1716.53	0.4	0.02	0.18	0.00	0.20	1730.44	
STRAIGHT PIPE	3.97	1720.5		0.00	0.18	0.01	0.20	1730.43	
REDUCER (8" x 6")	0.92	1721.42	0.4	0.07	0.25	0.01	0.21	1730.35	
STRAIGHT PIPE (up)	3.33	1724.75		0.00	0.25	0.03	0.24	1730.32	
6" ELBOW - 90 DEG.(turn)	0.88	1725.41	0.4	0.07	0.32	0.01	0.24	1730.24	
STRAIGHT PIPE	7.66	1725.41		0.00	0.32	0.06	0.31	1730.18	
6" ELBOW - 90 DEG.(turn)	0.78	1725.41	0.4	0.07	0.39	0.01	0.31	1730.10	
6" Plug Valve	0.88	1725.41	2.5	0.45	0.84	0.01	0.32	1729.65	
STRAIGHT PIPE	2.45	1725.41		0.00	0.84	0.02	0.34	1729.63	
6" ELBOW - 90 DEG.(down)	0.78	1724.75	0.4	0.07	0.91	0.01	0.35	1729.55	
STRAIGHT PIPE	2	1722.75		0.00	0.91	0.02	0.36	1729.53	
UV Disinfection Unit	3.5	1722.75	2.2	0.40	1.31	0.03	0.39	1729.11	
STRAIGHT PIPE	2	1724.75		0.00	1.31	0.02	0.41	1729.09	
6" ELBOW - 90 DEG.(horz)	0.78	1725.41	0.4	0.07	1.38	0.01	0.41	1729.02	
6" Plug Valve	0.88	1725.41	2.5	0.45	1.83	0.01	0.42	1728.56	
STRAIGHT PIPE	2.55	1725.41		0.00	1.83	0.02	0.44	1728.54	
6" ELBOW - 90 DEG.(turn)	0.78	1725.41	0.4	0.07	1.90	0.01	0.45	1728.46	
STRAIGHT PIPE	3.26	1725.41		0.00	1.90	0.03	0.47	1728.43	
6" ELBOW - 90 DEG.(down)	0.78	1724.75	0.4	0.07	1.98	0.01	0.48	1728.36	
INCREASER (6" x 8")	0.92	1723.83	0.1	0.01	1.98	0.00	0.48	1728.35	
8" STRAIGHT PIPE (down)	1.25	1722.58		0.00	1.98	0.00	0.48	1728.35	
8" ELBOW - 90 DEG.(horiz)	0.91	1721.83	0.4	0.02	2.00	0.00	0.48	1728.32	
8" STRAIGHT PIPE	8.25	1721.83		0.00	2.00	0.02	0.50	1728.30	
8" x 8" x 4" Tee (run)	1.5	1721.83	0.3	0.02	2.02	0.00	0.50	1728.28	
8" STRAIGHT PIPE	4.75	1721.83		0.00	2.02	0.01	0.51	1728.28	
8" ELBOW - 90 DEG.(down)	0.91	1721.08	0.4	0.02	2.04	0.00	0.51	1728.25	
STRAIGHT PIPE (below grade)	3.97	1718.61		0.00	2.04	0.01	0.52	1728.24	
8" ELBOW - 90 DEG.(horiz)	0.91	1717.86	0.4	0.02	2.07	0.00	0.52	1728.22	
8" STRAIGHT PIPE	11.5	1717.86		0.00	2.07	0.02	0.55	1728.20	
8" x 8" x 6" Tee (run)	1.5	1717.86	0.3	0.02	2.08	0.00	0.55	1728.18	
8" STRAIGHT PIPE	50	1717.86		0.00	2.07	0.10	0.62	1728.12	
8" ELBOW - 90 DEG.(turn)	0.91	1717.86	0.4	0.02	2.09	0.00	0.63	1728.09	
8" STRAIGHT PIPE	52	1717.86		0.00	2.09	0.10	0.73	1727.99	
8" ELBOW - 90 DEG.(turn)	0.91	1717.86	0.4	0.02	2.11	0.00	0.73	1727.97	
8" STRAIGHT PIPE (on slope)	32	1722.64		0.00	2.11	0.06	0.79	1727.90	
8" OUTLET INTO CASCADE	1	1722.64	1	0.06	2.17	0.00	0.80	1727.84	:Water elevation at V-notch
								2.97	total headloss at flow of: 300 GPM

Bottom of V-notch weir: 1727.00

height over weir: 0.84 feet

NON-POTABLE WATER PUMPING SYSTEM

This water system will supply washdown water around the plant site. The caustic soda added to the influent at the splitter box will be transported by the non-potable water. During operation of the sludge press, the non-potable water will be used to wash the belt filter of the press. Yard hydrants will also be located at the SBR tanks and sludge holding tank for the purpose of washdown these treatment units.

EVALUATION OF WATER USAGE AND NEED:

Item No.	Treatment unit Description:	Water Usage Demand	Amount of water used (GPM)	
1	Belt press	Periodic - approx 5 days per month	42	
2	Sludge FM Line to Sludge Holding Basin	Periodic - approx 10 times per day	80	
3	Polymer Dilution Water	Periodic - approx 5 days per month	5	
4	Caustic Soda to influent	Continuous - during pumping from Main PS	1	
5	Yard hydrant	Periodic	10	
6	Main PS	Continuous - during screening washing	15	
		Totals	**153**	**Maximum demand**
			16	**Minimum demand**
			96	**Average Daily demand**

Assume that there will be 2 pumps alternating in operation with no more than 10 starts per hour each during average daily demand

Average daily demand was determined to be:	**96**	**GPM**
Number of pumps in system:	2	
Number of starts per hour per pump:	10	
Total number of starts per hour:	20	
Minimum duration between pump starts:	3	minutes
Total flow expected:	288	gallons
Number of pressure tanks in use:	2	
Maximum pressure (pump shut off):	60	psi
Minimum pressure (pump turn on):	40	psi
Acceptance factor per Wessels chart:	0.268	
Tank size required:	537	gallons

Piping from UV unit to cascade aeraor

					Line		
FLOW FROM UV UNIT (gpm)	300		PIPE ROUGH.COEFF.for LINE No. 1	120	1	0.67	FLOW (in cfs)
PIPE DIAMETER (inches)	6		PIPE ROUGH.COEFF.for LINE No. 2	120	1	0.20	PIPE AREA (in ft)
WATER LEVEL ELEVATION:	1730.81		PIPE ROUGH.COEFF.for LINE No.3	120	1	3.40	VELOCITY (in fps)
C.LINE OF PIPE OUTLET ELEVATION:	1736.28		PIPE ROUGH.COEFF.for LINE No.4	120			
CHANGE IN PIPE DIAMETER of LINE 2 (inches)	8		PIPE ROUGH.COEFF.for LINE No.5	140	2	0.35	PIPE AREA (in ft)
CHANGE IN PIPE DIAMETER of LINE 3 (inches)	4		HEADLOSS PER 100 FEET AT FLOW (Line 1)	0.93	2	1.91	VELOCITY (in fps)
CHANGE IN PIPE DIAMETER of LINE 4 (inches)	3		HEADLOSS PER 100 FEET AT FLOW (Line 2)	0.23	3	0.09	PIPE AREA (in ft)
CHANGE IN PIPE DIAMETER of LINE 5 (inches)	2		HEADLOSS PER 100 FEET AT FLOW (Line 3)	0.16	3	1.02	VELOCITY (in fps)
FLOW TO NON-POTABLE WATER PUMPS	40	GPM	HEADLOSS PER 100 FEET AT FLOW (Line 4)	0.65	4	0.05	PIPE AREA (in ft)
	0.09	cfs	HEADLOSS PER 100 FEET AT FLOW (Line 4)	3.53	4	1.82	VELOCITY (in fps)

5	0.02	PIPE AREA (in ft)
5	4.09	VELOCITY (in fps)

Hydraulic Grade at entrance to UV unit: 1728.5 feet

Head of pump at flow: 161

DESCRIPTION	EQUIV. OR ACTUAL LENGTH	ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	ACCUM. PIPE FRICTION HEADLOSS	HYDRAULIC GRADE
UV Disinfection Unit	3.5	1722.75	2.2	0.40	0.40	0.03	0.03	1728.07
STRAIGHT PIPE	2	1724.75		0.00	0.40	0.02	0.05	1728.05
6" ELBOW - 90 DEG.(horz)	0.78	1725.41	0.4	0.07	0.47	0.01	0.06	1727.97
6" Plug Valve	0.88	1725.41	1.1	0.20	0.67	0.01	0.07	1727.77
STRAIGHT PIPE	2.55	1725.41		0.00	0.67	0.02	0.09	1727.74
6" ELBOW - 90 DEG.(turn)	0.78	1725.41	0.4	0.07	0.74	0.01	0.10	1727.66
STRAIGHT PIPE	3.26	1725.41		0.00	0.74	0.03	0.13	1727.63
6" ELBOW - 90 DEG.(down)	0.78	1724.75	0.4	0.07	0.81	0.01	0.14	1727.55
INCREASER (6" x 8")	0.92	1723.83	0.1	0.01	0.82	0.00	0.14	1727.55
8" STRAIGHT PIPE (down)	1.25	1722.58		0.00	0.82	0.00	0.14	1727.54
8" ELBOW - 90 DEG.(horiz)	0.91	1721.83	0.4	0.02	0.84	0.00	0.14	1727.52
8" STRAIGHT PIPE	8.25	1721.83		0.00	0.84	0.02	0.16	1727.50
8" x 8" x 4" Tee (side)	1.5	1721.83	1.8	0.03	0.87	0.00	0.16	1727.47
4" STRAIGHT PIPE (horiz)	2	1721.83		0.00	0.87	0.00	0.17	1727.47
4" x 4" x 4" Tee (run)	1.1	1721.83	0.3	0.00	0.87	0.00	0.17	1727.46
4" STRAIGHT PIPE (horiz)	5	1721.83		0.00	0.87	0.01	0.18	1727.45
4" ELBOW - 90 DEG. (turn)	0.58	1721.83	0.4	0.01	0.88	0.00	0.18	1727.44
4" Gate Valve	0.75	1721.83	0.2	0.00	0.88	0.00	0.18	1727.44
4" STRAIGHT PIPE (horiz)	1.5	1721.83		0.00	0.88	0.00	0.18	1727.44
Pump for non-potable water supply	1.75	1721.83		0.00	0.88	0.00	0.18	1888.43
3" Check Valve	0.78	1721.83	2.5	0.13	1.01	0.01	0.19	1888.30
3" Gate Valve	0.67	1721.83	0.2	0.01	1.02	0.00	0.19	1888.29
3" STRAIGHT PIPE (horiz)	1.05	1721.83		0.00	1.02	0.01	0.20	1888.28
3" ELBOW - 90 DEG. (turn)	0.52	1721.83	0.4	0.02	1.04	0.00	0.20	1888.26
3" STRAIGHT PIPE (horiz)	2	1721.83		0.00	1.04	0.01	0.22	1888.24
3" x 3" x 3" Tee (side)	1.1	1721.83	1.8	0.09	1.13	0.01	0.22	1888.14
3" STRAIGHT PIPE (horiz)	2	1721.83		0.00	1.13	0.01	0.24	1888.13
3" x 3" x 3" Tee (side)	1.1	1721.83	1.8	0.09	1.22	0.01	0.24	1888.03
3" STRAIGHT PIPE (horiz)	6	1721.83		0.00	1.22	0.04	0.28	1887.99
3" ELBOW - 90 DEG. (down)	0.52	1721.37	0.4	0.02	1.25	0.00	0.29	1887.97
3" STRAIGHT PIPE (vert)	4.5	1716.87		0.00	1.25	0.03	0.32	1887.94
3" ELBOW - 90 DEG. (horiz)	0.52	1716.41	0.4	0.02	1.27	0.00	0.32	1887.91
3" STRAIGHT PIPE (horiz)	10.75	1716.41		0.00	1.27	0.07	0.39	1887.84
3" x 3" x 3" Tee (side)	1.1	1716.41	1.8	0.09	1.36	0.01	0.40	1887.75
3" STRAIGHT PIPE (horiz)	40	1716.41		0.00	1.36	0.26	0.66	1887.48
3" ELBOW - 90 DEG. (turn)	0.52	1716.41	0.4	0.02	1.38	0.00	0.66	1887.46
3" STRAIGHT PIPE (horiz)	110	1716.41		0.00	1.38	0.72	1.38	1886.74
3" ELBOW - 45 DEG. (turn)	0.63	1720.41	0.22	0.01	1.39	0.00	1.38	1886.73
3" STRAIGHT PIPE (horiz)	50	1722.41		0.00	1.39	0.33	1.71	1886.40
3" ELBOW - 45 DEG. (turn)	0.63	1722.41	0.22	0.01	1.40	0.00	1.71	1886.39
3" STRAIGHT PIPE (horiz)	120	1723.41		0.00	1.40	0.78	2.50	1885.60
3" ELBOW - 90 DEG. (turn)	0.63	1723.41	0.4	0.02	1.42	0.00	2.50	1885.58
3" STRAIGHT PIPE (horiz)	105	1720.41		0.00	1.42	0.69	3.19	1884.89
3" x 3" x 2" Tee (side)	0.56	1720.41	1.1	0.29	1.71	0.02	3.21	1884.59
2" Connection/conversion to PVC	0.5	1720.41		0.00	1.71	0.02	3.22	1884.57

2" PVC STRAIGHT PIPE (horiz)	25	1720.41		0.00	1.71	0.88	4.11	1883.69	
1 1/2" yard hydrant	5	1725.41	2.5	0.65	2.35	0.18	4.28	1882.86	
					Water flow rate gpm:		40.00	68.18	pressure at yard hydrant with pump running
Pressure Tank (assume pump not running) Tank at elevation 1722.0 ft and set pressure:					50.00 psi				
3" Tank outlet	0.5	1721.83	0.5	0.03	0.03	0.02	0.02	1837.26	
3" Gate Valve	0.67	1721.83	0.2	0.01	0.04	0.02	0.04	1837.23	
3" ELBOW - 90 DEG. (turn)	0.63	1721.83	0.4	0.02	0.06	0.02	0.06	1837.18	
3" STRAIGHT PIPE (horiz)	2.5	1721.83		0.00	0.06	0.09	0.15	1837.10	
3" x 3" x 3" Tee (side)	1.1	1721.83	1.8	0.09	0.15	0.04	0.19	1836.96	
3" STRAIGHT PIPE (horiz)	1.33	1721.83		0.00	0.15	0.05	0.24	1836.92	
3" x 3" x 3" Tee (side)	1.1	1721.83	1.8	0.09	0.24	0.04	0.28	1836.79	
3" STRAIGHT PIPE (horiz)	6	1721.83		0.00	0.24	0.21	0.49	1836.57	
3" ELBOW - 90 DEG. (down)	0.52	1721.37	0.4	0.02	0.26	0.02	0.51	1836.54	
3" STRAIGHT PIPE (vert)	4.5	1716.87		0.00	0.26	0.16	0.66	1836.38	
3" ELBOW - 90 DEG. (horiz)	0.52	1716.41	0.4	0.02	0.28	0.02	0.68	1836.34	
3" STRAIGHT PIPE (horiz)	10.75	1716.41		0.00	0.28	0.38	1.06	1835.96	
3" x 3" x 3" Tee (side)	1.1	1716.41	1.8	0.09	0.37	0.04	1.10	1835.83	
3" STRAIGHT PIPE (horiz)	40	1716.41		0.00	0.37	1.41	2.51	1834.42	
3" ELBOW - 90 DEG. (turn)	0.52	1716.41	0.4	0.02	0.39	0.02	2.53	1834.38	
3" STRAIGHT PIPE (horiz)	110	1716.41		0.00	0.39	3.88	6.41	1830.50	
3" ELBOW - 45 DEG. (turn)	0.63	1720.41	0.22	0.01	0.41	0.02	6.43	1830.46	
3" STRAIGHT PIPE (horiz)	50	1722.41		0.00	0.41	1.76	8.20	1828.70	
3" ELBOW - 45 DEG. (turn)	0.63	1722.41	0.22	0.01	0.42	0.02	8.22	1828.67	
3" STRAIGHT PIPE (horiz)	120	1723.41		0.00	0.42	4.23	12.45	1824.43	
3" ELBOW - 90 DEG. (turn)	0.63	1720.41	0.4	0.02	0.44	0.02	12.48	1824.39	
3" STRAIGHT PIPE (horiz)	105	1720.41		0.00	0.44	3.70	16.18	1820.69	
3" x 3" x 2" Tee (side)	0.56	1720.41	1.8	0.47	0.90	0.02	16.20	1820.20	
2" Connection/conversion to PVC	0.5	1720.41		0.00	0.90	0.02	16.22	1820.18	
2" PVC STRAIGHT PIPE (horiz)	25	1720.41		0.00	0.90	0.88	17.10	1819.30	
1 1/2" yard hydrant	5	1725.41	2.5	0.65	1.55	0.18	17.28	1818.48	
					Water flow rate gpm:		40.00	40.30	pressure at yard hydrant with no pump

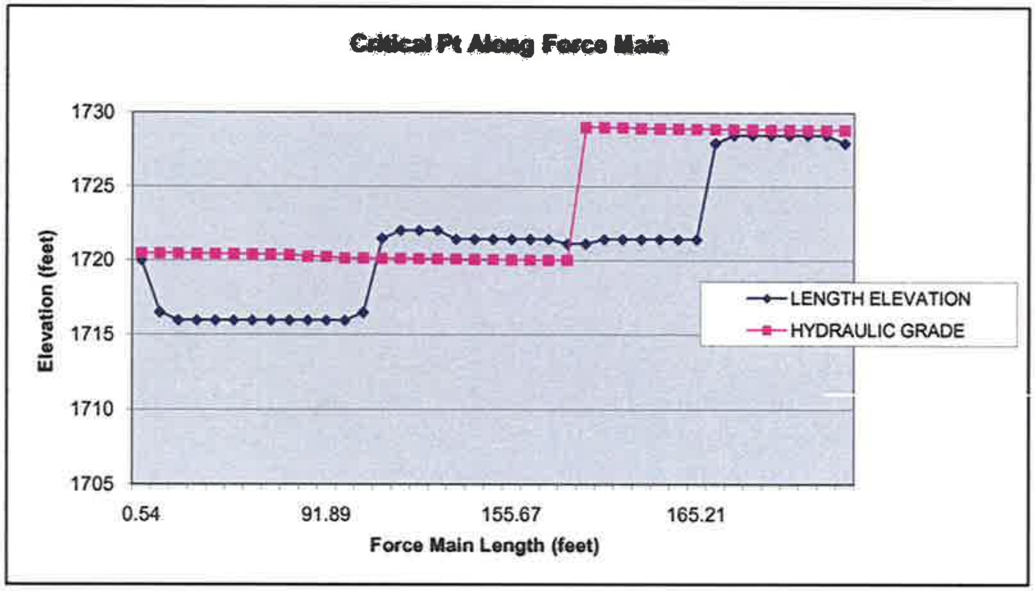
SLUDGE PUMPS TO BELT PRESS

Influent invert to wet well: 1691.75
bottom elevation of wet well: 1681.50

FLOW (gpm)	45	PIPE ROUGH.COEFF.for PUMP STATION	120
PIPE DIAMETER of Pump Sta. (inches)	4	PIPE ROUGH.COEFF.for FORCE MAIN	120
LEAD PUMP CUT OFF ELEVATION:	1720.50	HEADLOSS PER 100 FEET AT FLOW (P.S.)	**0.20**
C.LINE OF PUMP INLET ELEVATION:	1721.50	HEADLOSS PER 100 FEET AT FLOW (F.M.)	**0.20**
PIPE DIAMETER of Force Main (inches)	4	GROUND ELEVATION at MAIN P.S.	1708
HEAD FOR PUMP AT FLOW	9	ATMOSPHERIC PRESS. AVAILABLE (ft.)	**31.94**

DESCRIPTION	EQUIV. OR ACTUAL LENGTH	ACCUM LENGTH FEET	ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	ACCUM. PIPE FRICTION HEADLOSS	HYDRAULIC GRADE
PUMP STATION									
SUCTION INTAKE	0.54	0.54	1720	0.5	**0.01**	**0.01**	**0.00**	**0.00**	**1720.49**
VERTICAL DROP PIPE	3.5	4.04	1716.5		**0.00**	**0.01**	**0.01**	**0.01**	**1720.48**
ELBOW - 90 DEG.(horiz)	0.59	4.63	1715.96	0.4	**0.01**	**0.02**	**0.00**	**0.01**	**1720.47**
STRAIGHT PIPE	4	8.63	1715.96		**0.00**	**0.02**	**0.01**	**0.02**	**1720.46**
PLUG VALVE	0.75	9.38	1715.96	0.6	**0.01**	**0.03**	**0.00**	**0.02**	**1720.45**
STRAIGHT PIPE	7.12	16.5	1715.96		**0.00**	**0.03**	**0.01**	**0.03**	**1720.44**
ELBOW - 90 DEG.(turn)	0.59	17.09	1715.96	0.4	**0.01**	**0.04**	**0.00**	**0.03**	**1720.43**
STRAIGHT PIPE	2.5	19.59	1715.96		**0.00**	**0.04**	**0.01**	**0.05**	**1720.41**
TEE (run)	1	20.59	1715.96	0.6	**0.01**	**0.05**	**0.00**	**0.05**	**1720.40**
STRAIGHT PIPE	71	91.59	1715.96		**0.00**	**0.05**	**0.14**	**0.19**	**1720.26**
ELBOW - 45 DEG. (turn)	0.3	91.89	1715.96	0.2	**0.00**	**0.06**	**0.00**	**0.19**	**1720.25**
STRAIGHT PIPE	47	138.89	1715.96		**0.00**	**0.06**	**0.09**	**0.29**	**1720.16**
ELBOW - 90 DEG.(vertical up)	0.59	139.48	1716.5	0.4	**0.01**	**0.06**	**0.00**	**0.29**	**1720.15**
VERTICAL RISER PIPE	5	144.48	1721.5		**0.00**	**0.06**	**0.01**	**0.30**	**1720.14**
ELBOW - 90 DEG.(horiz)	0.59	145.07	1722.04	0.4	**0.01**	**0.07**	**0.00**	**0.30**	**1720.13**
STRAIGHT PIPE	2.61	147.68	1722.04		**0.00**	**0.07**	**0.01**	**0.30**	**1720.12**
ROTA-CUTTER	1.8	149.48	1722.04	0.8	**0.02**	**0.09**	**0.00**	**0.31**	**1720.10**
STRAIGHT PIPE	2.75	152.23	1721.44		**0.00**	**0.09**	**0.01**	**0.31**	**1720.10**
TEE (side)	0.85	153.08	1721.44	1.8	**0.04**	**0.13**	**0.00**	**0.31**	**1720.06**
STRAIGHT PIPE	2	155.08	1721.44		**0.00**	**0.13**	**0.00**	**0.32**	**1720.06**
ELBOW - 90 DEG.(horiz)	0.59	155.67	1721.44	0.4	**0.01**	**0.13**	**0.00**	**0.32**	**1720.05**
PLUG VALVE	0.75	156.42	1721.44	0.6	**0.01**	**0.15**	**0.00**	**0.32**	**1720.03**
STRAIGHT PIPE	1.5	157.92	1721.44		**0.00**	**0.15**	**0.00**	**0.32**	**1720.03**
GOOSENECK PUMP INTAKE	1.1	159.02	1721.14	0.4	**0.01**	**0.15**	**0.00**	**0.33**	**1720.02**
VOGELSANG PUMP	0.75	159.77	1721.14		**0.00**	**0.15**	**0.00**	**0.33**	**1729.02**
GOOSENECK PUMP DISCHARGE	1.1	160.87	1721.44	0.4	**0.01**	**0.16**	**0.00**	**0.33**	**1729.01**
STRAIGHT PIPE	1	161.87	1721.44		**0.00**	**0.16**	**0.00**	**0.33**	**1729.01**
CHECK VALVE	1	162.87	1721.44	2.5	**0.05**	**0.21**	**0.00**	**0.33**	**1728.95**
PLUG VALVE	0.75	163.62	1721.44	1	**0.02**	**0.23**	**0.00**	**0.34**	**1728.93**
STRAIGHT PIPE	1	164.62	1721.44		**0.00**	**0.23**	**0.00**	**0.34**	**1728.93**
ELBOW - 90 DEG.(up)	0.59	165.21	1721.44	0.4	**0.01**	**0.24**	**0.00**	**0.34**	**1728.92**
VERTICAL RISER PIPE	6.5	171.71	1727.94		**0.00**	**0.24**	**0.01**	**0.35**	**1728.91**
ELBOW - 90 DEG.(horiz)	0.59	172.3	1728.48	0.4	**0.01**	**0.25**	**0.00**	**0.35**	**1728.90**
STRAIGHT PIPE	5.5	177.8	1728.48		**0.00**	**0.25**	**0.01**	**0.36**	**1728.89**
TEE (run)	1	178.8	1728.48	0.4	**0.01**	**0.26**	**0.00**	**0.37**	**1728.88**

STRAIGHT PIPE	8.83	187.63	1728.48		0.00	0.26	0.02	0.38	**1728.86**	
ELBOW - 90 DEG.(horiz)	0.59	188.22	1728.48	0.4	0.01	0.27	0.00	0.39	**1728.85**	
STRAIGHT PIPE	1	189.22	1728.48		0.00	0.27	0.00	0.39	**1728.85**	
ELBOW - 90 DEG.(down)	0.59	189.81	1727.94	0.4	0.01	0.27	0.00	0.39	**1728.84**	Critical point



Owner: **SERCF**
Project Title: **SLUDGE PUMPS**
Type of Pump: **VOGELSANG - POSITIVE DISPLACEMENT**
Model No.: **MODEL - V100-45Q**

PUMP CURVE:
PUMP SPEED: **0-900** RPM

Assumed water level in sludge tank: 1721
Critical force main elevation: 1728.84
Force main length to critical elevation: 190 including fittings - equivalent pipe
Equivalent ft of pipe at pump station: 0.00
Diameter (in) of pump staion piping: 4

IMPELLER NO.:

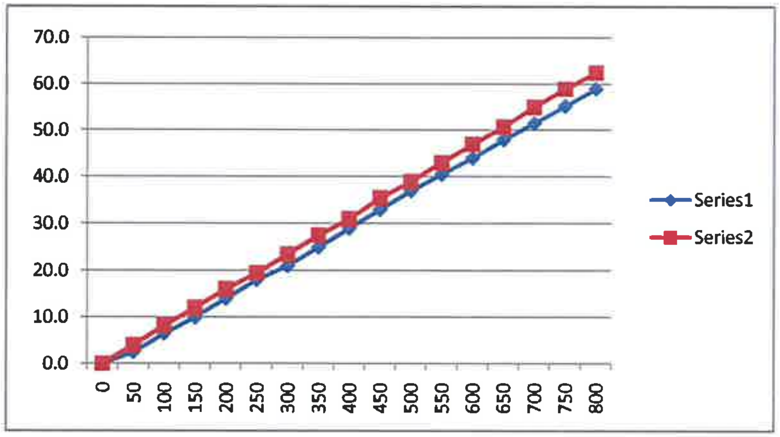
Minimum Flow (gpm): 30
Maximum Flow (gpm): 45

Diameter (in) of force main piping: 4
Static Head: 7.84

PIPE ROUGH.COEFF.for PUMP STATION 120
PIPE ROUGH.COEFF.for FORCE MAIN 120
HEADLOSS PER 100 FEET AT FLOW (P.S.) **0.09**
HEADLOSS PER 100 FEET AT FLOW (F.M.) **0.09**

SPEED RPM	FLOW at 1 psi (GPM)	FLOW at 5 psi (GPM)	FLOW at 25 psi (GPM)
0	0	0.0	
50	4	2.5	
100	8.25	6.5	
150	12	10.0	
200	16	14.0	
250	19.5	18.0	
300	23.5	21.0	
350	27.5	25.0	
400	31	29.0	
450	35.5	33.0	
500	39	37.0	
550	43	40.5	
600	47	**44.00**	
650	50.75	**48.00**	
700	55	**51.50**	
750	59	**55.25**	
800	62.5	**59.00**	

System Curve HEAD (FT)
7.84
7.84
7.85
7.86
7.88
7.91
7.93
7.97
8.01
8.05
8.10
8.15
8.20
8.27
8.33
8.40
8.47



TAFF PUMP STATION

Influent invert to wet well:	1666.00
bottom elevation of wet well:	1655.00

FLOW (gpm)	180
PIPE DIAMETER of Pump Sta. (inches)	4
LEAD PUMP CUT OFF ELEVATION:	1657.50
C.LINE OF PUMP INLET ELEVATION:	1674.37
PIPE DIAMETER of Force Main (inches)	6
TD HEAD FOR PUMP AT FLOW	120

PIPE ROUGH.COEFF.for PUMP STATION		120
PIPE ROUGH.COEFF.for FORCE MAIN		130
HEADLOSS PER 100 FEET AT FLOW (P.S.)		**2.60**
HEADLOSS PER 100 FEET AT FLOW (F.M.)		**0.31**
GROUND ELEVATION at MAIN P.S.		1671
ATMOSPHERIC PRESS. AVAILABLE (ft.)		**32.02**
STATIC SUCTION (ft)	16.87	
Deduction from Atmp Press. (Safe Fact)		2
Deduct. from Atmp Press. (vapor Pres.)		1

DESCRIPTION	EQUIV. OR ACTUAL LENGTH	ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	ACCUM. PIPE FRICTION HEADLOSS	HYDRAULIC GRADE	
PUMP STATION									
PUMP SUCTION INTAKE	0.54	1656	0.5	0.16	0.16	0.01	0.01	1657.32	
ELBOW - 90 DEG. (6" x 4" base)	0.58	1656.54	0.4	0.13	0.30	0.02	0.03	1657.18	
RISER PIPE	17.3	1673.84		0.00	0.30	0.45	0.48	1656.73	
ELBOW - 90 DEG.(turn into pump)	0.58	1674.38	0.4	0.13	0.43	0.02	0.49	1656.58	
STRAIGHT PIPE	1.5	1674.38		0.00	0.43	0.04	0.53	1656.54	17.41 TDSL
Pump Discharge (top)	0.75	1675.13	2	0.66	1.08	0.02	0.55	1775.87	
CHECK VALVE	1	1676.13	1	0.33	1.41	0.03	0.58	1775.51	
PLUG VALVE	0.75	1676.88	2.5	0.82	2.23	0.02	0.60	1774.67	
ELBOW - 90 DEG.(turn)	0.58	1677.42	0.4	0.13	2.36	0.03	0.62	1774.51	
STRAIGHT PIPE	0.75	1677.42		0.00	2.36	0.02	0.64	1774.49	
TEE - SIDE (4" x 4" x 6")	0.58	1676.75	0.4	0.13	2.49	0.02	0.66	1774.35	
STRAIGHT PIPE (6")	6	1670.75		0.00	2.49	0.02	0.68	1774.33	
ELBOW - 90 DEG. (turn - 6")	0.79	1670.08	0.4	0.03	2.52	0.00	0.68	1774.30	
STRAIGHT PIPE (6")	10	1670.08		0.00	2.52	0.03	0.71	1774.27	
				0.00	2.52	0.00	0.71	1774.27	

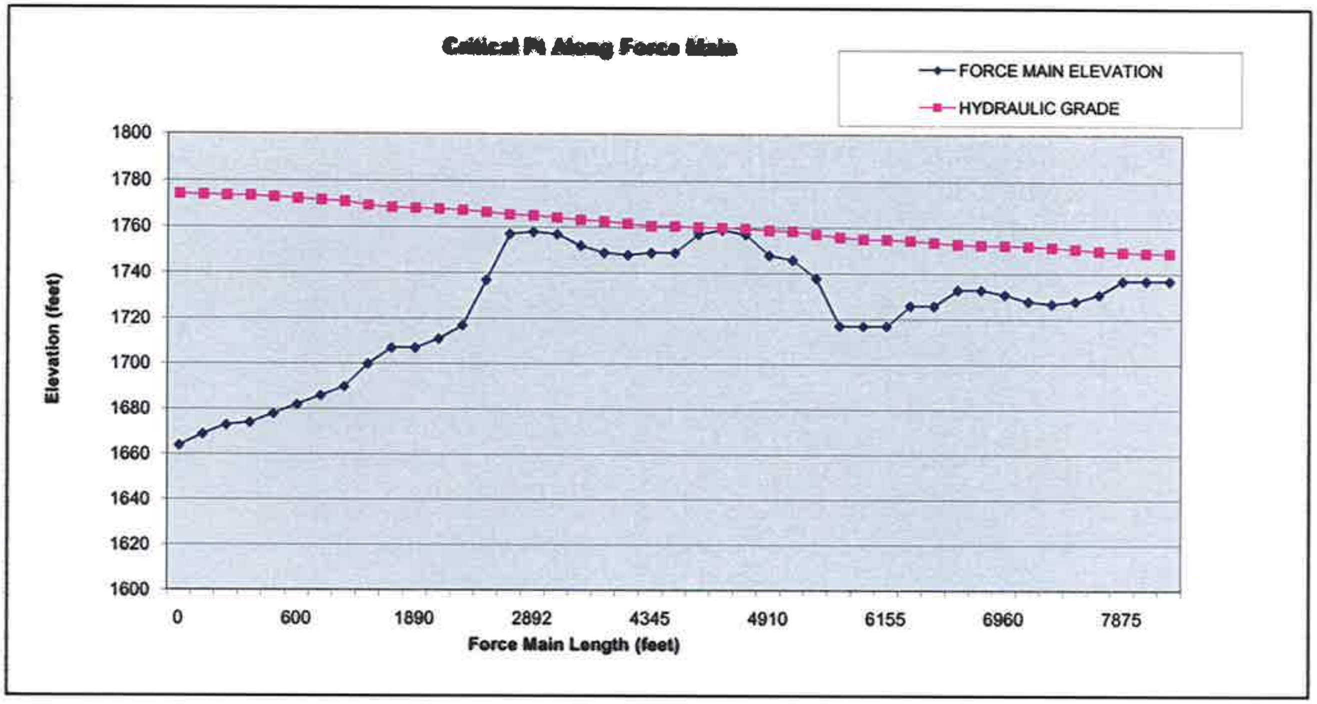
suction piping line length	20.5
discharge piping length	21.2

Total Net Deductions from Atm Pressure:	20.4132
NPSH (Net Positive Suction Head) Available:	11.60

DESCRIPTION	STATION	LENGTH	FORCE MAIN ELEVATION	FITTINGS "K" COEF.	MINOR HEAD-LOSS	ACCUM. MINOR HEAD.	PIPE FRICTION HEADLOSS	HYDRAULIC GRADE
5 ft. outside station	0+00	0	1664		0.00	0.00	0.00	1774.27
	1+00	100	1669		0.00	0.00	0.31	1773.96
45 deg. bend	1+70	170	1673	0.22	0.01	0.01	0.53	1773.73
	2+00	200	1674		0.00	0.01	0.62	1773.63
	4+00	400	1678		0.00	0.01	1.25	1773.01
	6+00	600	1682		0.00	0.01	1.87	1772.38

	8+00	800	1686		0.00	0.01	2.50	1771.76	
	10+00	1000	1690		0.00	0.01	3.12	1771.14	
	15+00	1500	1700	0.45	0.03	0.04	4.68	1769.55	
change in grade	18+00	1800	1707	0.45	0.03	0.07	5.61	1768.58	
Ditch center line	18+90	1890	1707	1	0.06	0.14	5.90	1768.24	
	20+00	2000	1711		0.00	0.14	6.24	1767.89	
22.5 deg. Bend (change in grade)	21+70	2170	1717	0.12	0.01	0.15	6.77	1767.36	
change in grade	24+10	2410	1737		0.00	0.15	7.52	1766.61	
change in grade	28+00	2800	1757		0.00	0.15	8.73	1765.39	
Air Release valve (high point) & 45 de	28+92	2892	1758	0.3	0.02	0.16	9.02	1765.08	
change in grade	31+80	3180	1757		0.00	0.16	9.92	1764.19	
	35+00	3500	1752		0.00	0.16	10.92	1763.19	
change in grade	37+00	3700	1749		0.00	0.16	11.54	1762.56	
	40+00	4000	1748		0.00	0.16	12.48	1761.63	
Center line of HWY 285	43+45	4345	1749		0.00	0.16	13.55	1760.55	
change in grade	43+70	4370	1749		0.00	0.16	13.63	1760.47	
change in grade	44+80	4480	1757		0.00	0.16	13.97	1760.13	
Air Release valve (high point)	45+80	4580	1759	0.25	0.02	0.18	14.29	1759.80	Critical point
change in grade	46+80	4680	1757		0.00	0.18	14.60	1759.49	
change in grade	49+10	4910	1748		0.00	0.18	15.32	1758.77	
Farm Rd center line	50+25	5025	1746		0.00	0.18	15.67	1758.42	
change in grade	54+35	5435	1738		0.00	0.18	16.95	1757.14	
change in grade	58+55	5855	1717		0.00	0.18	18.26	1755.83	
Ditch center line	60+85	6085	1717		0.00	0.18	18.98	1755.11	
change in grade	61+55	6155	1717		0.00	0.18	19.20	1754.89	
change in grade	63+05	6305	1726		0.00	0.18	19.67	1754.42	
change in grade	65+45	6545	1726		0.00	0.18	20.42	1753.67	
change in grade	68+05	6805	1733		0.00	0.18	21.23	1752.86	
change in grade	69+10	6910	1733		0.00	0.18	21.55	1752.54	
Farm Rd center line	69+60	6960	1731		0.00	0.18	21.71	1752.38	
change in grade	70+70	7070	1728		0.00	0.18	22.05	1752.04	
Ditch center line	72+30	7230	1727		0.00	0.18	22.55	1751.54	
change in grade	74+75	7475	1728		0.00	0.18	23.32	1750.77	
change in grade	77+30	7730	1731		0.00	0.18	24.11	1749.98	
change in grade	78+75	7875	1737		0.00	0.18	24.56	1749.53	
New prison Rd center line	80+00	8000	1737		0.00	0.18	24.95	1749.14	
Discharge into new manhole	80+20	8020	1737	1	0.06	0.25	25.02	1749.01	

Static Discharge (at critical point)	84.63
Total Static Head	101.50
Total Friction Head (at critical point)	17.52
TOTAL DYNAMIC HEAD	119.02

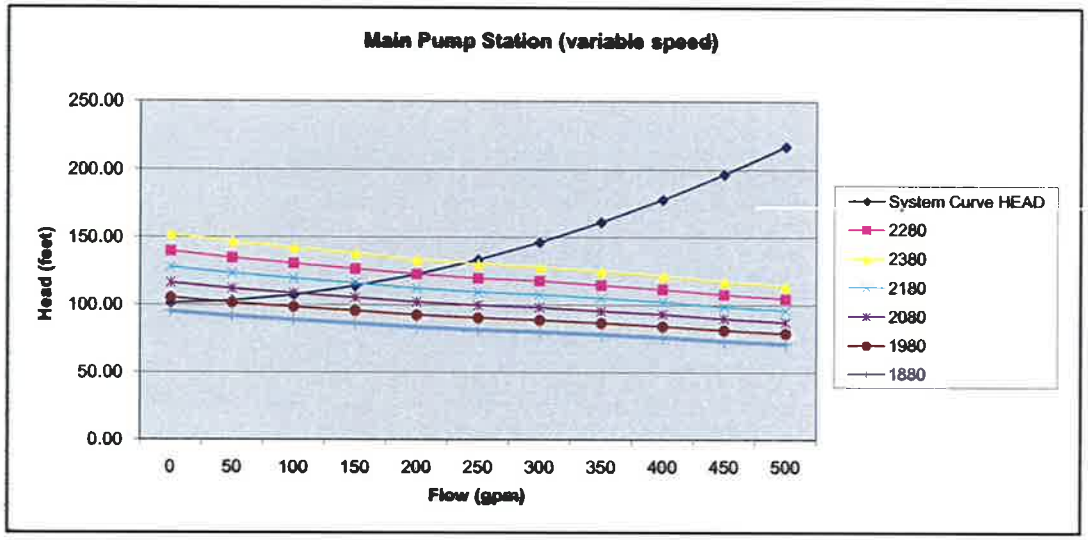


Owner:	**SERCF**
Project Title:	**MAIN PUMP STATION**
Type of Pump:	**GORMAN RUPP - SELF PRIMING**
Model No.:	**MODEL - V3-A-B-2**
PUMP CURVE:	9 INCH IMPELLER
PUMP SPEED:	2280 RPM
IMPELLER NO.:	
Minimum Flow (gpm):	180

Assumed water level in pump station:	1657.5	
Critical force main elevation:	1759	
Force main length to critical elevation:	4580	including fittings - equivalent pipe
Equivalent ft of pipe at pump station:	1035.39	
Diameter (in) of pump staion piping:	4	
Diameter (in) of force main piping:	6	
Static Head:	101.5	

PIPE ROUGH.COEFF.for PUMP STATION	120
PIPE ROUGH.COEFF.for FORCE MAIN	130
HEADLOSS PER 100 FEET AT FLOW (P.S.)	2.60
HEADLOSS PER 100 FEET AT FLOW (F.M.)	0.31

	PUMP SPEED - RPM							System Curve HEAD (FT)
	2280		2380	2180	2080	1980	1880	
	FLOW (GPM)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	HEAD (FT)	
Shutoff	0	140	152.6	128.0	116.5	105.6	95.2	101.50
	50	135	147.1	123.4	112.4	101.8	91.8	103.14
	100	131	142.7	119.8	109.0	98.8	89.1	107.40
	150	127	138.4	116.1	105.7	95.8	86.3	114.00
	200	123	134.0	112.4	102.4	92.8	83.6	122.79
	250	120	130.8	109.7	99.9	90.5	81.6	133.66
	300	118	128.6	107.9	98.2	89.0	80.2	146.57
	350	115	125.3	105.1	95.7	86.7	78.2	161.44
	400	112	122.0	102.4	93.2	84.5	76.1	178.23
	450	108	117.7	98.7	89.9	81.4	73.4	196.92
	500	105	114.4	96.0	87.4	79.2	71.4	217.45



Main Pump Station (variable speed)

The capacity of a PVC sewer line can be calculated using the empirical formula:

Q = (1.49/n) A R^2/3 S^1/2
where n = roughness coefficient (0.0115 for PVC pipe per State of TN design criteria)
A = cross sectional area of the water in the pipe
R = the hydraulic radius (cross sectional area of the water/wetted perimeter of the pipe)
S = slope of the pipe
Q = flow in cubic feet per sec
n = 0.0115 thus, (1.49/n) = 129.5652174

Taff Center flow enters at MH A-11	180 GPM	Peak flow	From force main approximately 111 minutes per day
New BCX Prison flow enters at MH A-10	150 GPM	Peak flow	
Old SERCF enters at MH B-2 and C-2	115 GPM	Peak flow	

Pipe Segment No. and MH No.	Pipe Diameter (inches)	Water Depth in pipe (inches)	Manhole Ground Elev.	Manhole Invert Elev. (ft.)	Downstream Manhole Ground Elev. (ft)	Downstream Manhole Invert Elev. (ft.)	Distance Between MH (ft.)	Slope in Pipe (vert ft./ horizft.)	Angle θ (radians)	Wetted Perimeter (ft.)	Cross Sectional Area (sq ft)	Hydraulic Radius (ft.)	Flow (Q) in pipe (cu. ft./sec.)	Flow (Q) in pipe (gpm)	Flow (Q) in pipe (MGD)
A-11 to A-10	8	2.25	1739	1730.5	1725.5	1719.1	392.4	0.02905	2.23596	0.74532	0.08051	0.10802	0.403	180.85	0.260
A-10 to A-9	8	4.3	1725.5	1719	1724	1717.01	216.9	0.00917	3.29173	1.09724	0.19118	0.17424	0.740	332.00	0.478
A-9 to A-8	8	3.81	1724	1716.91	1721	1713.97	214.6	0.01370	3.04656	1.01552	0.16398	0.16148	0.737	330.75	0.476
A-8 to A-7	8	3.86	1721	1713.87	1717	1709.99	296	0.01311	3.07158	1.02386	0.16676	0.16287	0.737	330.90	0.476
A-7 to A-6	8	3.85	1717	1709.89	1712.7	1705.98	296	0.01321	3.06658	1.02219	0.16620	0.16259	0.737	330.69	0.476
A-6 to A-5	8	3.43	1712.7	1705.88	1707.9	1700.35	278.2	0.01988	2.85562	0.95187	0.14297	0.15020	0.738	331.00	0.477
A-5 to A-4	10	3.9	1707.9	1700.25	1706	1697.59	312	0.00853	2.69796	1.12415	0.19694	0.17519	0.737	330.87	0.476
A-4 to A-3	10	4.59	1706	1697.49	1704.9	1696.86	132.8	0.00474	2.97741	1.24059	0.24427	0.19690	0.737	330.93	0.477
A-3 to A-2	10	4.17	1704.9	1696.86	1702	1694.8	308.5	0.00668	2.80805	1.17002	0.21533	0.18404	0.737	330.87	0.476
A-2 to A-1	10	5.85	1702	1694.7	1701	1693.4	325.4	0.00400	3.48325	1.45135	0.33145	0.22837	1.014	454.92	0.655
A-1 to MH 1	10	6.57	1701	1693.3	1706.5	1693.12	63.4	0.00284	3.78040	1.57517	0.37992	0.24119	1.016	455.89	0.656
MH1 to MPS	10	3.22	1706.5	1692.5	1709.4	1692	15	0.03333	2.41362	1.00568	0.15176	0.15090	1.017	456.38	0.657
Ex MH to C-2	8	1.1	1709	1706.44	1707	1702.11	107.3	0.04035	1.51952	0.50651	0.02894	0.05713	0.112	50.09	0.072
C-2 to C-1	8	1.75	1707	1702.01	1705.3	1700.84	188.6	0.00620	1.94678	0.64893	0.05648	0.08704	0.113	50.76	0.073
C-1 to B-1	8	1.94	1705.3	1700.74	1704.5	1700.23	127.2	0.00401	2.05958	0.68653	0.06537	0.09522	0.112	50.15	0.072
Connection of old plant to collection system starting with MH B-2 (180,000 GPD)															0.000
B-2 to B-1	8	1.36	1706.8	1704.17	1704.5	1701	110.9	0.02858	1.69996	0.56665	0.03935	0.06944	0.145	65.30	0.094
B-1 to Screen	8	2.04	1704.5	1700.13	1703.5	1698.56	90	0.01744	2.11741	0.70580	0.07017	0.09942	0.258	115.58	0.166
Screen to A-2	8	1.78	1703.5	1698.31	1702	1697.2	36.7	0.03025	1.96487	0.65496	0.05786	0.08834	0.258	115.97	0.167

Appendix D

Proposed Post Equalization Basin Volume Calculations

Inputs and Outputs for EQ Tank Design

Project Name: **Pikeville, TN ICEAS**
Sanitaire Number: **28087-17a**
Date: **12/21/2017**
Created By: **JTB**

Inputs	**Value**	**Units**	**Notes**
Normal Cycle Information			
Total Normal Cycle Time	**288**	min	Input from process sheet.
Normal Cycle Decant Time	**72**	min	Input from process sheet.
Normal Cycle Decant Flow	**590**	gpm	Input from process sheet.
Basin #1 Decant Start Time	**0**	min	Use Cycle time "0" and offset other decant times from "0".
Basin #2 Decant Start Time	**72**	min	Use # of minutes the start of the decant cycle is offset from "0" (start of Basin #1 decant cycle).
Basin #3 Decant Start Time	**144**	min	Leave blank (delete entry) if only 2 basins.
Basin #4 Decant Start Time	**216**	min	Leave blank (delete entry) if only 2 or 3 basins.
Storm Cycle Information			
Total Storm Cycle Time	**180**	min	Input from process sheet.
Storm Cycle Decant Time	**45**	min	Input from process sheet.
Storm Cycle Decant Flow	**906**	gpm	Input from process sheet.
Basin #1 Decant Time Start	**0**	min	Use Cycle time "0" and offset other decant times from "0".
Basin #2 Decant Time Start	**90**	min	Use # of minutes the start of the decant cycle is offset from "0" (start of Basin #1 decant cycle).
Basin #3 Decant Time Start	**135**	min	Leave blank (delete entry) if only 2 basins.
Basin #4 Decant Time Start	**180**	min	Leave blank (delete entry) if only 2 or 3 basins.
Second Storm Cycle Information			
Total Second Storm Cycle Time		min	Input from process sheet.
Second Storm Cycle Decant Time		min	Input from process sheet.
Second Storm Cycle Decant Flow		gpm	Input from process sheet.
Basin #1 Decant Time Start	**0**	min	Use Cycle time "0" and offset other decant times from "0".
Basin #2 Decant Time Start		min	Use # of minutes the start of the decant cycle is offset from "0" (start of Basin #1 decant cycle).
Basin #3 Decant Time Start		min	Leave blank (delete entry) if only 2 basins.
Basin #4 Decant Time Start		min	Leave blank (delete entry) if only 2 or 3 basins.
EQ Tank Information			
Target EQ Tank Discharge Rate	**800**	gpm	Leave blank if unknown, see EQ Tank Discharge Rate below, will turn red if too low.
Target Max. Depth in EQ Tank	**10.00**	ft	Must enter, check discharge elevation or max water level, does not include tank free board.
Target Min. Depth in EQ Tank	**2.00**	ft	Must enter, check pump submergence, use "0" if tank can be drained completely.

Outputs	**Value**	**Units**	**Notes**
EQ Tank Discharge Rate		gpm	See Target EQ Tank Discharge Rate above.
Target Max. EQ Tank Volume	**14,310**	**gal**	Based on Target EQ Tank Discharge Rate above.
Target EQ Tank Area	**239.1**	ft^2	Based on Target EQ Tank Discharge Rate above.
Target EQ Tank Length	**15.46**	ft	Based on Target EQ Tank Discharge Rate above, assumes square tank.
Target EQ Tank Width	**15.46**	ft	Based on Target EQ Tank Discharge Rate above, assumes square tank.

EQ Tank Calculations

Project Name: **Pikeville, TN ICEAS**
Sanitaire Number: **28087-17a**
Date: **12/21/2017**

Parameter	**Symbol**	**Value**	**Units**	**Calculation**
Normal Cycle Information				
Total Normal Cycle Time	NCT	288	min	input
Normal Cycle Decant Time	NDT	72	min	input
Normal Cycle Decant Flow	NDF	590	gpm	input
Basin #1 Decant Start Time	$NDST_{B1}$	0	min	assume "0"
Basin #2 Decant Start Time	$NDST_{B2}$	72	min	input
Basin #3 Decant Start Time	$NDST_{B3}$	144	min	input
Basin #4 Decant Start Time	$NDST_{B4}$	216	min	input
Normal Cycle One Decant Volume	NDV_{one}	42,480	gal	=NDT * NDF
Shortest Time Between Decants	$NTBD_{min}$	0	min	=min(nondecant segment durations)
Storm Cycle Information				
Total Storm Cycle Time	SCT	180	min	input
Storm Cycle Decant Time	SDT	45	min	input
Storm Cycle Decant Flow	SDF	906	gpm	input
Basin #1 Decant Time Start	$SDST_{B1}$	0	min	assume "0"
Basin #2 Decant Time Start	$SDST_{B2}$	90	min	input
Basin #3 Decant Time Start	$SDST_{B3}$	135	min	input
Basin #4 Decant Time Start	$SDST_{B4}$	180	min	input
Storm Cycle One Decant Volume	SDV_{one}	40,770	gal	=SDT * SDF
Shortest Time Between Decants	$STBD_{min}$	45	min	=min(nondecant segment durations)
Second Storm Cycle Information				
Total Second Storm Cycle Time	SSCT		min	input
Second Storm Cycle Decant Time	SSDT		min	input
Second Storm Cycle Decant Flow	SSDF		gpm	input
Basin #1 Decant Time Start	$SSDST_{B1}$		min	assume "0"
Basin #2 Decant Time Start	$SSDST_{B2}$		min	input
Basin #3 Decant Time Start	$SSDST_{B3}$		min	input
Basin #4 Decant Time Start	$SSDST_{B4}$		min	input
SS Cycle One Decant Volume	$SSDV_{one}$		gal	=SSDT * SSDF
Shortest Time Between Decants	$STBD_{min}$		min	=min(nondecant segment durations)
EQ Tank Design at Target Discharge Rate				
Target EQ Tank Discharge Rate	TQ_{out}	800	gpm	input, must be = or > EQQ_{max}
Target Max. Depth in EQ Tank	TD_{max}	10.0	ft	input, does not include free board
Target Min. Depth in EQ Tank	TD_{min}	2.0	ft	input, based on pump submergence
Target Max. EQ Tank Volume	$\mathbf{TV_{max}}$	**14,310**	**gal**	=max(target EQ tank volumes)
Target EQ Tank Discharge Depth	EQDD	8.0	ft	$=TD_{max} - TD_{min}$
Target EQ Tank Area	TEQA	239.1	ft^2	$=TV_{max}$ / 7.4805 / EQDD
Target EQ Tank Length	TEQL	15.5	ft	=√(TEQA)
Target EQ Tank Width	TEQW	15.5	ft	=√(TEQA)
EQ Tank Design at Design Discharge Rate				
Normal Cycle Min. Discharge Rate	NQ_{out}	590	gpm	$=NDV_{one}$ / (NDT + $NTBD_{min}$)
Storm Cycle Min. Discharge Rate	SQ_{out}	453	gpm	$=SDV_{one}$ / (SDT + $STBD_{min}$)
2nd Storm Cycle Min. Dis. Rate	SSQ_{out}	0	gpm	$=SSDV_{one}$ / (SSDT + $SSTBD_{min}$)
Min. EQ Tank Discharge Rate	$\mathbf{EQQ_{max}}$	**590**	**gpm**	=max(Q_{out} flows)
Normal Cycle EQ Tank Volume	$NEQV_{max}$	0	gal	=(NDV_{one} * 2) - [NQ_{out} * (NDT * 2 + $NTBD_{min}$)]
Storm Cycle EQ Tank Volume	$SEQV_{max}$	20,385	gal	=(SDV_{one} * 2) - [SQ_{out} * (SDT * 2 + $STBD_{min}$)]
2nd Storm Cycle EQ Tank Vol.	$SSEQV_{max}$	0	gal	=($SSDV_{one}$ * 2) - [SSQ_{out} * (SSDT * 2 + $SSTBD_{min}$)]
Min. EQ Tank Volume Required	$\mathbf{EQV_{max}}$	**20,385**	**gal**	=max(EQV_{max} volumes)
Min. EQ Tank Area	EQA	340.6	ft^2	$=EQV_{max}$ / 7.4805 / EQDD
Min. EQ Tank Length	EQL	18.5	ft	=√(EQA)
Min. EQ Tank Width	EQW	18.5	ft	=√(EQA)

						(+)		**(-)**	
	Normal Cycle Time Segment (min)			**Segment Duration (min)**	**Decant Flow, NDF (gpm)**	**Decanted Vol. (gal)**	**Target EQ Discharge Rate, TQ_{out} (gpm)**	**Target Discharge Vol. (gal)**	**Target EQ Tank Vol. Needed (gal)**
B1	0	to	72	72	590	42480	800	57600	0
	72	to	72	0	0	0	800	0	0
B2	72	to	144	72	590	42480	800	57600	0
	144	to	144	0	0	0	800	0	**0**
B3	144	to	216	72	590	42480	800	57600	0
	216	to	216	0	0	0	800	0	**0**
B4	216	to	288	72	590	42480	800	57600	0
	288	to	288	0	0	0	800	0	**0**
	Storm Cycle Time Segment (min)			**Segment Duration (min)**	**Decant Flow, SDF (gpm)**	**Decanted Vol. (gal)**	**Target EQ Discharge Rate, TQ_{out} (gpm)**	**Target Discharge Vol. (gal)**	**Target EQ Tank Vol. Needed (gal)**
B1	0	to	45	45	906	40770	800	36000	4770
	45	to	90	45	0	0	800	36000	0
B2	90	to	135	45	906	40770	800	36000	4770
	135	to	135	0	0	0	800	0	***4770***
B3	135	to	180	45	906	40770	800	36000	9540
	180	to	180	0	0	0	800	0	***9540***
B4	180	to	225	45	906	40770	800	36000	14310
	225	to	180	0	0	0	800	0	***14310***
	2nd Storm Cycle Time Segment (min)			**Segment Duration (min)**	**Decant Flow, SSDF (gpm)**	**Decanted Vol. (gal)**	**Target EQ Discharge Rate, TQ_{out} (gpm)**	**Target Discharge Vol. (gal)**	**Target EQ Tank Vol. Needed (gal)**
B1									
B2									
B3									
B4									

Low Flow Calcs

Flow to Filters	265	gpm
Flow from ICEAS	32000	gpd
	22	gpm
Time of Flow	6	hrs
	360	min
Existing Tank	25000	gal

Total Volume Needed	87313	gal
Volume New Tank	62313	gal
Max SWD	12	ft
Min SWD	2	ft
Diameter New Tank	32.5680569	ft

Average Flow Calcs

Flow to Filters	355	gpm
Flow from ICEAS	630000	gpd
	438	gpm
Time of Flow	18	hrs
	1080	min
Existing Tank	25000	gal

Total Volume Needed	89362	gal
Volume New Tank	64362	gal
Max SWD	12	ft
Min SWD	2	ft
Diameter New Tank	33.0994451	ft

Max Flow Calcs

Flow to Filters	445	gpm
Flow from ICEAS	850000	gpd
	590	gpm
Time of Flow	10	hrs
	600	min
Existing Tank	25000	gal

Total Volume Needed	87313	gal
Volume New Tank	62313	gal
Max SWD	12	ft
Min SWD	2	ft
Diameter New Tank	32.56805691	ft

Peak Flow Calcs

Flow to Filters	535	gpm
Flow from ICEAS		gpd
	906	gpm
Time of Flow	1	hrs
	60	min
Existing Tank	25000	gal

Total Volume Needed	22275	gal
Volume New Tank	0	gal
Max SWD	12	ft
Min SWD	2	ft
Diameter New Tank	0	ft

Three Basin Flow Calcs

Flow to Filters	625	gpm
Flow from ICEAS		gpd
	1208	gpm
Time of Flow	1	hrs
	60	min
Existing Tank	25000	gal

Total Volume Needed	34995	gal
Volume New Tank	9995	gal
Max SWD	12	ft
Min SWD	2	ft
Diameter New Tank	13.04327368	ft

0

GPM between flows

90

Volume New Tank	**64362**

Appendix E

Proposed Effluent Filter Pumps

Design Calculations

Appendix E
Effluent Filter Pumps and Force Main Calculations

The existing effluent pumps and piping will be reused to accommodate the expanded capacity. The following calculations demonstrate the suitability of these facilities at the expanded capacity.

FLOW: Average Daily Flow = 0.630 mgd (440 gpm)
Peak Daily Flow = 0.850 mgd (590 gpm)
For any flows in excess of 590 gpm, allow both pumps to operate simultaneously.

MAXIMUM STATIC HEAD: Filter Overflow Weir 1738.5
Post EQ Basin LWL 1721.5
17.0 feet

SUCTION PIPING: 135 feet of 8” (C = 120)
0.630 mgd H_f = (4.6’/1000’)(135’) = 0.62 feet
0.850 mgd H_f = (8.1’/1000’)(135’) = 1.09 feet

DISCHARGE PIPING: 195 feet of 8” (C = 120)
0.630 mgd H_f = (4.6’/1000’)(195’) = 0.90 feet
0.850 mgd H_f = (8.1’/1000’)(195’) = 1.58 feet

MINOR LOSSES: Assume minor losses = 10 feet (Conservative)

TOTAL DYNAMIC HEAD: 0.630 mgd 440 gpm @ 28.5 feet
0.850 mgd 590 gpm @ 29.7 feet

The Pump Curve for the existing pumps is attached showing these design conditions. The 10 horsepower motors may also continue to be used.

Appendix F

Proposed Effluent Pump Station

Design Calculations

Appendix F
Proposed Effluent Pumps and Piping Calculations

The proposed effluent pumps will be used to transfer half the effluent flow (after disinfection) to the effluent storage tank. Pumps to be used will be similar to the self priming pumps to be used for the influent equalization pumps, SBR influent pumps, and the existing filter pumps.

FLOW: Average Daily Flow = 0.630 mgd (440 gpm) Q/2 = 0.315 mgd (220 gpm)
Peak Daily Flow = 0.850 mgd (590 gpm) Q/2 = 0.425 mgd (295 gpm)
If it should ever be necessary to pump in excess of 295 gpm, allow both pumps to operate simultaneously.

MAXIMUM STATIC HEAD:	Effluent Tank HWL	1758.6
	Wetwell FFE (Estimated)	<u>1710.0</u>
		48.6 feet

Suction Piping: Gravity Discharge
Discharge Piping 650 feet ~ 6”

FRICTION LOSSES: 650 feet ~ 6” (C = 120)
0.315 mgd H_f = (5.2’/1000’)(650’) = 3.4 feet
0.425 mgd H_f = (9.0’/1000’)(650’) = 5.9 feet

MINOR LOSSES: Assume minor losses = 10 feet (Conservative)

TOTAL DYNAMIC HEAD: 0.315 mgd 220 gpm @ 62.0 feet
0.425 mgd 295 gpm @ 64.5 feet

Typical Pump Curve attached for these design conditions. 15 horsepower motors required.

Appendix G

Soils Investigation by Innovative Wastewater Solutions, Inc.

Pedon Sheets Area 2

Extra High Intensity Soils Map Area 2

Pedon Sheets Area 3

Extra High Intensity Soils Map Area 3

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach

Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: B5

SOP# (office use only:

Soil Series: Atkins

Drainage Class: poorly

Soil Classification (control section): Fine-loamy

Ground Water:

Parent Material: shale and sandstone

Erosion:

Climate:

Land Cover: Forest

Slope of Map Unit: 0-5

Slope of Pit: 3

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to Redoximorphic features

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach

Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: B8

SOP# (office use only:

Soil Series: Lily

Drainage Class: well

Soil Classification (control section): Fine-loamy

Ground Water:

Parent Material: sandstone

Erosion:

Climate:

Land Cover: Forest

Slope of Map Unit: 0-5

Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-8	10YR 4/3		Loam	1	F	Gr	
Bt1	8-16	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	16-32	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	32+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: B11 SOP# (office use only:

Soil Series: Lonewood Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 3/2		Loam	1	F	Gr	
Bt1	4-18	10YR 5/4		Silt Loam	2	F	SBK	
Bt2	18-25	10YR 5/6		Silt Loam	2	F	SBK	
Bt3	25-32	10YR 5/6		Clay Loam	2	F	SBK	
Bt4	32+	7.5YR 5/6		Clay Loam	2	F	SBK	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: B14 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-14	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	14-30	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	30+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: B17 SOP# (office use only:

Soil Series: Hendon Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone, siltstone and shale Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-3	10YR 3/2		Loam	1	M	Gr	
Bt1	3-17	10YR 5/6		Silty Loam	2	M	SBK	
Bt2	17-24	10YR 5/8		Silt Loam	2	M	SBK	
Btx	24+	10YR 5/6	5YR 5/8, 10YR 7/3					

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: E3 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 2

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-18	7.5YR 5/6		Clay Loam	2	M	SBK	
Bt2	18-22	7.5YR 5/6		Sandy Clay Loam	2	M	SBK	
R	22+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: E5 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 2

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-6	10YR 4/3		Loam	1	F	Gr	
Bt1	6-17	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	17-30	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	30+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: E8 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 7

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-15	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	15-26	7.5YR 5/5		Clay Loam	2	F	SBK	
Bt3	26-36	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	36+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: E11 SOP# (office use only:

Soil Series: Hendon Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone, siltstone and shale Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-3	10YR 3/2		Loam	1	M	Gr	
Bt1	3-17	10YR 5/6		Silty Loam	2	M	SBK	
Bt2	17-26	10YR 5/8		Silt Loam	2	M	SBK	
Btx	26+	10YR 5/6	5YR 5/8, 10YR 7/3					

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: E14 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 4

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-12	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	12-24	7.5YR 5/5		Clay Loam	2	F	SBK	
Bt3	24-36	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	36+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach - Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: E17 SOP# (office use only:

Soil Series: Lonewood Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 4

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 3/2		Silty Clay Loam	1	F	Gr	
Bt1	4-14	10YR 5/4		Silt Loam	2	F	SBK	
Bt2	14-23	10YR 5/6		SSilty Clay Loam	2	F	SBK	
Bt3	23-36	10YR 5/6		Silty Clay	2	F	SBK	
Bt4	36+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach
Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H3
SOP# (office use only:

Soil Series: Ramsey
Drainage Class: somewhat excessively

Soil Classification (control section): Loamy
Ground Water:

Parent Material: sandstone or quartzite
Erosion:

Climate:
Land Cover:

Slope of Map Unit: 5-15
Slope of Pit: 9

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to R

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H5 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 3

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-6	10YR 4/3		Loam	1	F	Gr	
Bt1	6-17	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	17-30	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	30+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H8 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-10 Slope of Pit: 8

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-12	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	12-24	7.5YR 5/5		Clay Loam	2	F	SBK	
Bt3	24-34	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	34+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H11 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-10 Slope of Pit: 8

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-12	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	12-24	7.5YR 5/5		Clay Loam	2	F	SBK	
Bt3	24-34	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	34+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H14 SOP# (office use only:

Soil Series: Ramsey Drainage Class: somewhat excessively

Soil Classification (control section): Loamy Ground Water:

Parent Material: sandstone or quartzite Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-15 Slope of Pit: 7

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to R

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H17 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-10 Slope of Pit: 8

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-20	7.5YR 5/6		Clay Loam	2	M	SBK	
Bt2	20-24	7.5YR 5/6		Sandy Clay Loam	2	M	SBK	
R	24+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H20 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-10 Slope of Pit: 7

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-3	10YR 4/3		Loam	1	F	Gr	
Bt1	3-20	7.5YR 5/6		Loam	2	F	SBK	
Bt2	20-36	7.5YR 5/5		Clay Loam	2	F	SBK	
R	36+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: J14 SOP# (office use only:

Soil Series: Ramsey Drainage Class: somewhat excessively

Soil Classification (control section): Loamy Ground Water:

Parent Material: sandstone or quartzite Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-15 Slope of Pit: 9

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to R

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: J17 SOP# (office use only:

Soil Series: Ramsey Drainage Class: somewhat excessively

Soil Classification (control section): Loamy Ground Water:

Parent Material: sandstone or quartzite Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-15 Slope of Pit: 10

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to R

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach — Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: J20 — SOP# (office use only:

Soil Series: Lily — Drainage Class: well

Soil Classification (control section): Fine-loamy — Ground Water:

Parent Material: sandstone — Erosion:

Climate: — Land Cover: Forest

Slope of Map Unit: 5-10 — Slope of Pit: 7

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-12	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	12-24	7.5YR 5/5		Clay Loam	2	F	SBK	
Bt3	24-34	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	34+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 9-27-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: L20 SOP# (office use only:

Soil Series: Ramsey Drainage Class: somewhat excessively

Soil Classification (control section): Loamy Ground Water:

Parent Material: sandstone or quartzite Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-15 Slope of Pit: 9

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to R

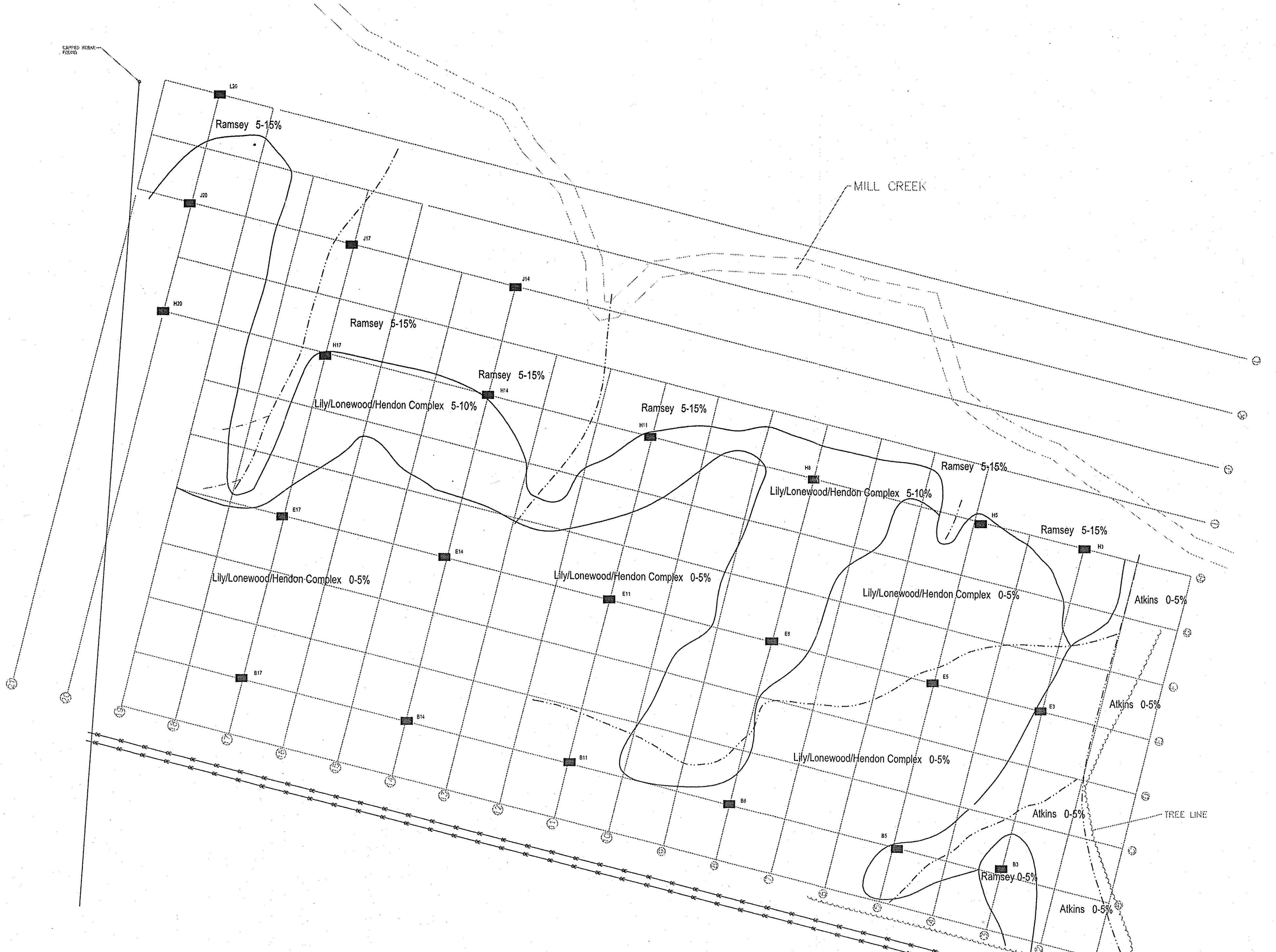
Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

EXTRA HIGH INTENSITY SOILS MAP FOR SPRAY IRRIGATION SYSTEMS

Bledsoe County Correctional Complex
1045 Horsehead Road
Bledsoe County, TN

SOIL NAME SLOPE CLASS	LOADING RATE (G/D/FT^2)	DEPTH TO RESTRICTING LAYERS (INCHES)	SOIL IMPROVEMENT PRACTICES/ NOTES/ PERC STATUS
Lily, Hendon, Lonewood Complex, 0-5%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 0 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.
Lily, Hendon, Lonewood Complex, 5-10%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 5 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.
Ramsey, 0-5%	0.0	<20""	The Ramsey series consists of shallow and very shallow, somewhat excessively drained soils that formed in residuum or colluvium weathered from sandstone or quartzite. They are dominantly on plateaus and upper slopes of mountains. Runoff is medium to rapid and permeability is rapid. Slopes range from 3 to 70 percent.
Atkins, 0-5%	0.0	<20" TO REDOXIMORPHIC FEATURES	The Atkins series consists of very deep, poorly drained soils formed in acid alluvium washed from upland soils that formed in shale and sandstone. Permeability is slow to moderate. Slope ranges from 0 to 3 percent. Mean annual precipitation is about 46 inches and the mean annual air temperature is about 54 degrees F.



Map Legend
lot corner ○
control flag ● A1
drain (25' setbacks) — · · — · · —
drain (15' setbacks) ---- · ---- · ----
gully (15' setbacks)
cut bank
drive — — — — — —
rock outcrop Vv
doline bottom
culvert
fence
soil pit ■

scale
1"=100'

IF THESE SOILS ARE DISTURBED (CUT, FILLED OR COMPACTED) AFTER THE DATE SHOWN, THIS SOILS MAP WILL BE VOID!

I, Kevin Davis, affirm that this Division of Water Resources Soil Map has been prepared in accordance with accepted standards of soil science practice and the standards and methodologies established in the NRCS Soil Survey Manual and USDA Soil Taxonomy. No other warranties are made or implied.

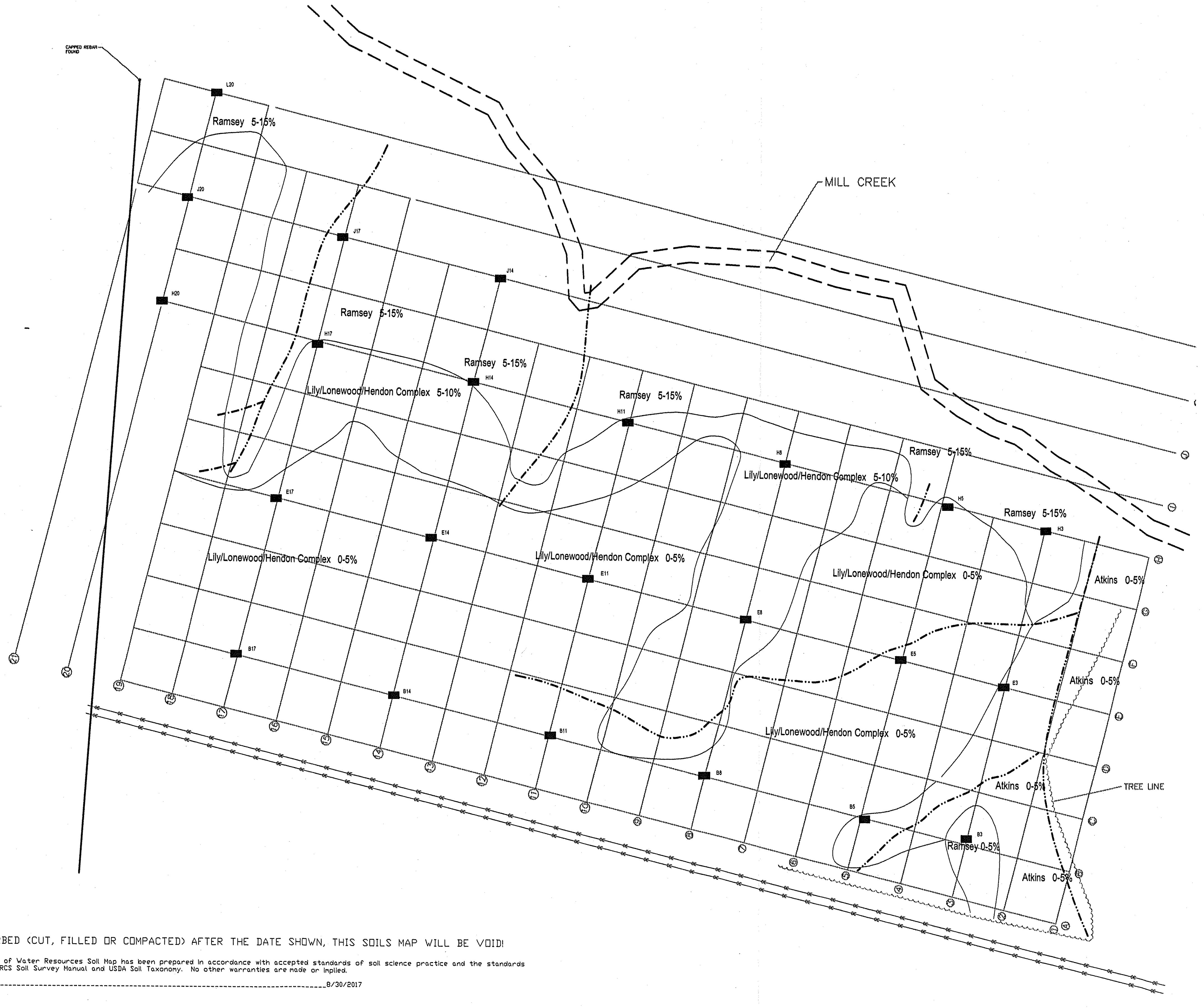
______________________________8/30/2017

EXTRA- HIGH INTENSITY SOILS MAP BY:
IWS, INC.
3675 Brown Springs Road
Greeneville, TN 37743
(423) 620-0036
(865) 776-8771

EXTRA HIGH INTENSITY SOILS MAP FOR SPRAY IRRIGATION SYSTEMS

Bledsoe County Correctional Complex
1045 Horsehead Road
Bledsoe County, TN

SOIL NAME SLOPE CLASS	**LOADING RATE (G/D/FT^2)**	**DEPTH TO RESTRICTING LAYERS (INCHES)**	**SOIL IMPROVEMENT PRACTICES/ NOTES/ PERC STATUS**
Lily, Hendon, Lonewood Complex, 0-5%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 0 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.
Lily, Hendon, Lonewood Complex, 5-10%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 5 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.
Ramsey, 0-5%	0.0	<20""	The Ramsey series consists of shallow and very shallow, somewhat excessively drained soils that formed in residuum or colluvium weathered from sandstone or quartzite. They are dominantly on plateaus and upper slopes of mountains. Runoff is medium to rapid and permeability is rapid. Slopes range from 3 to 70 percent.
Atkins, 0-5%	0.0	<20" TO REDOXIMORPHIC FEATURES	The Atkins series consists of very deep, poorly drained soils formed in acid alluvium washed from upland soils that formed in shale and sandstone. Permeability is slow to moderate. Slope ranges from 0 to 3 percent. Mean annual precipitation is about 46 inches and the mean annual air temperature is about 54 degrees F.



Map Legend
lot corner ○
control flag ● A1
drain (25' setbacks)
drain (15' setbacks)
gully (15' setbacks)
cut bank
drive
rock outcrop Vv
doline bottom
culvert
fence
soil pit ■

scale
1"=100'

EXTRA- HIGH INTENSITY SOILS MAP BY:
IWS, INC.
3675 Brown Springs Road
Greeneville, TN 37743
(423) 620-0036
(865) 776-8771

IF THESE SOILS ARE DISTURBED (CUT, FILLED OR COMPACTED) AFTER THE DATE SHOWN, THIS SOILS MAP WILL BE VOID!

I, Kevin Davis, affirm that this Division of Water Resources Soil Map has been prepared in accordance with accepted standards of soil science practice and the standards and methodologies established in the NRCS Soil Survey Manual and USDA Soil Taxonomy. No other warranties are made or implied.

____8/30/2017

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach   Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: E16-2   SOP# (office use only:

Soil Series: Lily   Drainage Class: well

Soil Classification (control section): Fine-loamy   Ground Water:

Parent Material: sandstone   Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-10   Slope of Pit: 7

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-3	10YR 4/3		Loam	1	F	Gr	
Bt1	3-20	7.5YR 5/6		Loam	2	F	SBK	
Bt2	20-32	7.5YR 5/5		Clay Loam	2	F	SBK	
R	32+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: E2-2 SOP# (office use only:

Soil Series: Lonewood Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 4

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 3/2		Silty Clay Loam	1	F	Gr	
Bt1	4-15	10YR 5/4		Silt Loam	2	F	SBK	
Bt2	15-24	10YR 5/6		SSilty Clay Loam	2	F	SBK	
Bt3	24-34	10YR 5/6		Silty Clay	2	F	SBK	
Bt4	34+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: D25-2 SOP# (office use only:

Soil Series: Sequoia Drainage Class: well

Soil Classification (control section): Fine Ground Water:

Parent Material: shale and siltstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 2

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-5	10YR 4/2		Loam	1	F	Gr	
Bt1	5-10	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	10-24	5YR 5/8	10YR 7/8, 10YR 6/3	Clay	2	F	SBK	
BC	24+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach

Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: D7-2

SOP# (office use only:

Soil Series: Lonewood

Drainage Class: well

Soil Classification (control section): Fine-loamy

Ground Water:

Parent Material: sandstone

Erosion:

Climate:

Land Cover: Forest

Slope of Map Unit: 0-5

Slope of Pit: 4

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles.etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 3/2		Silty Clay Loam	1	F	Gr	
Bt1	4-14	10YR 5/4		Silt Loam	2	F	SBK	
Bt2	14-23	10YR 5/6		SSilty Clay Loam	2	F	SBK	
Bt3	23-32	10YR 5/6		Silty Clay	2	F	SBK	
Bt4	32+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach
Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: C23-2
SOP# (office use only:

Soil Series: Sequoia
Drainage Class: well

Soil Classification (control section): Fine
Ground Water:

Parent Material: shale and siltstone
Erosion:

Climate:
Land Cover: Forest

Slope of Map Unit: 0-5
Slope of Pit: 2

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-5	10YR 4/2		Loam	1	F	Gr	
Bt1	5-10	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	10-24	5YR 5/8		Clay	2	F	SBK	
Bt3	24-34	5YR 5/8	10YR 7/8, 10YR 6/3	Clay	2	F	SBK	
BC	34+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: C18-2 SOP# (office use only:

Soil Series: Atkins Drainage Class: poorly

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: shale and sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 3

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to Redoximorphic features

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: C5-2 SOP# (office use only:

Soil Series: Hendon Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone, siltstone and shale Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-3	10YR 3/2		Loam	1	M	Gr	
Bt1	3-14	10YR 5/6		Silty Loam	2	M	SBK	
Bt2	14-24	10YR 5/8		Silt Loam	2	M	SBK	
Btx	24+	10YR 5/6	5YR 5/8, 10YR 7/3					

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach

Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: B21-2

SOP# (office use only:

Soil Series: Hendon

Drainage Class: well

Soil Classification (control section): Fine-loamy

Ground Water:

Parent Material: sandstone, siltstone and shale

Erosion:

Climate:

Land Cover: Forest

Slope of Map Unit: 0-5

Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-3	10YR 3/2		Loam	1	M	Gr	
Bt1	3-17	10YR 5/6		Silty Loam	2	M	SBK	
Bt2	17-24	10YR 5/8		Silt Loam	2	M	SBK	
Btx	24+	10YR 5/6	5YR 5/8, 10YR 7/3					

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H25-2 SOP# (office use only:

Soil Series: Sequoia Drainage Class: well

Soil Classification (control section): Fine Ground Water:

Parent Material: shale and siltstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 2

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-5	10YR 4/2		Loam	1	F	Gr	
Bt1	5-10	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	10-26	5YR 5/8	10YR 7/8, 10YR 6/3	Clay	2	F	SBK	
BC	26+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach

Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H6-2

SOP# (office use only:

Soil Series: Lily

Drainage Class: well

Soil Classification (control section): Fine-loamy

Ground Water:

Parent Material: sandstone

Erosion:

Climate:

Land Cover: Forest

Slope of Map Unit: 5-10

Slope of Pit: 8

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-12	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	12-24	7.5YR 5/5		Clay Loam	2	F	SBK	
Bt3	24-34	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	34+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: H2-2 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 3

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-6	10YR 4/3		Loam	1	F	Gr	
Bt1	6-14	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	14-28	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	28+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: G16-2 SOP# (office use only:

Soil Series: Atkins Drainage Class: poorly

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: shale and sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 3

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to Redoximorphic features

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: F26-2 SOP# (office use only:

Soil Series: Lonewood Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 4

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 3/2		Silty Clay Loam	1	F	Gr	
Bt1	4-14	10YR 5/4		Silt Loam	2	F	SBK	
Bt2	14-23	10YR 5/6		SSilty Clay Loam	2	F	SBK	
Bt3	23-36	10YR 5/6		Silty Clay	2	F	SBK	
Bt4	36+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: F7-2 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-10 Slope of Pit: 8

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-12	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	12-24	7.5YR 5/5		Clay Loam	2	F	SBK	
Bt3	24-34	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	34+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: L4-2 SOP# (office use only:

Soil Series: Hendon Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone, siltstone and shale Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-3	10YR 3/2		Loam	1	M	Gr	
Bt1	3-17	10YR 5/6		Silty Loam	2	M	SBK	
Bt2	17-34	10YR 5/8		Silt Loam	2	M	SBK	
Btx	34+	10YR 5/6	5YR 5/8, 10YR 7/3					

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: K23-2 SOP# (office use only:

Soil Series: Sequoia Drainage Class: well

Soil Classification (control section): Fine Ground Water:

Parent Material: shale and siltstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 2

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-5	10YR 4/2		Loam	1	F	Gr	
Bt1	5-10	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	10-24	5YR 5/8		Clay	2	F	SBK	
Bt3	24-36	5YR 5/8	10YR 7/8, 10YR 6/3	Clay	2	F	SBK	
BC	36+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: K16-2 SOP# (office use only:

Soil Series: Ramsey Drainage Class: somewhat excessively

Soil Classification (control section): Loamy Ground Water:

Parent Material: sandstone or quartzite Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-15 Slope of Pit: 10

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to R

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: J5-2 SOP# (office use only:

Soil Series: Lonewood Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 5

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles.etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 3/2		Loam	1	F	Gr	
Bt1	4-18	10YR 5/4		Silt Loam	2	F	SBK	
Bt2	18-25	10YR 5/6		Silt Loam	2	F	SBK	
Bt3	25-32	10YR 5/6		Clay Loam	2	F	SBK	
Bt4	32+	7.5YR 5/6		Clay Loam	2	F	SBK	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach   Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: J3-2   SOP# (office use only:

Soil Series: Lily   Drainage Class: well

Soil Classification (control section): Fine-loamy   Ground Water:

Parent Material: sandstone   Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5   Slope of Pit: 2

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles.etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-15	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	15-33	7.5YR 5/6		Sandy Clay Loam	2	F	SBK	
R	33+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: I15-2 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-10 Slope of Pit: 7

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-12	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	12-28	7.5YR 5/5		Clay Loam	2	F	SBK	
R	28+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach   Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: L13-2   SOP# (office use only:

Soil Series: Atkins   Drainage Class: poorly

Soil Classification (control section): Fine-loamy   Ground Water:

Parent Material: shale and sandstone   Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5   Slope of Pit: 3

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to Redoximorphic features

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: L10-2 SOP# (office use only:

Soil Series: Lily Drainage Class: well

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 5-10 Slope of Pit: 8

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-4	10YR 4/3		Loam	1	F	Gr	
Bt1	4-20	7.5YR 5/6		Clay Loam	2	M	SBK	
Bt2	20-24	7.5YR 5/6		Sandy Clay Loam	2	M	SBK	
R	34+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: L6-2 SOP# (office use only:

Soil Series: Atkins Drainage Class: poorly

Soil Classification (control section): Fine-loamy Ground Water:

Parent Material: shale and sandstone Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5 Slope of Pit: 3

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to Redoximorphic features

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach   Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: B19-2   SOP# (office use only:

Soil Series: Atkins   Drainage Class: poorly

Soil Classification (control section): Fine-loamy   Ground Water:

Parent Material: shale and sandstone   Erosion:

Climate: Land Cover: Forest

Slope of Map Unit: 0-5   Slope of Pit: 3

Geomorphic Description:

Physiographic Location:

Additional Notes: <20" to Redoximorphic features

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach

Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: O24-2

SOP# (office use only:

Soil Series: Sequoia

Drainage Class: well

Soil Classification (control section): Fine

Ground Water:

Parent Material: shale and siltstone

Erosion:

Climate:

Land Cover: Forest

Slope of Map Unit: 0-5

Slope of Pit: 2

Geomorphic Description:

Physiographic Location:

Additional Notes:

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	
Ap	0-5	10YR 4/2		Loam	1	F	Gr	
Bt1	5-8	7.5YR 5/6		Clay Loam	2	F	SBK	
Bt2	8-22	5YR 5/8		Clay	2	F	SBK	
Bt3	22-35	5YR 5/8	10YR 7/8, 10YR 6/3	Clay	2	F	SBK	
BC	35+							

DWR Soil Pedon Description (Field) Form

Described By: Kenton Brotherton/ Billy Roach

Date: 12-19-2017

Site Location: BCCX WWTP Expansion, Bledsoe County, TN

Pit #: L16-2

SOP# (office use only:

Soil Series: Ramsey

Drainage Class: somewhat excessively

Soil Classification (control section): Loamy

Ground Water:

Parent Material: sandstone or quartzite

Erosion:

Climate:

Land Cover: Forest

Slope of Map Unit: 5-15

Slope of Pit: 10

Geomorphic Description:

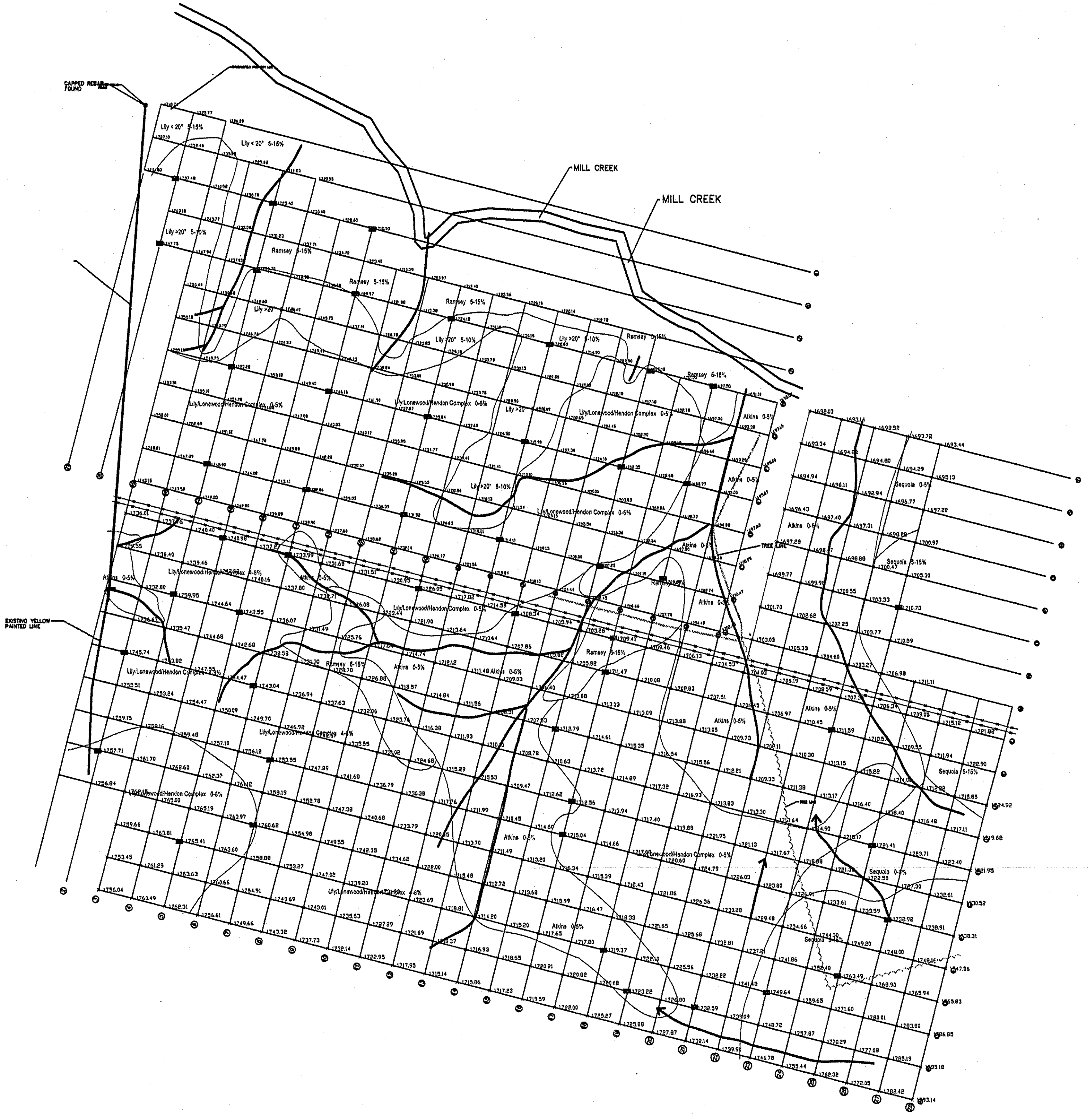
Physiographic Location:

Additional Notes: <20" to R

Soil Pedon Description

Horizon	Depths	Matrix Color	Depletions/Concentrations Redox/Mottles,etc.	Texture	Structure			Soil Horizon Notes
					Grade	Size	Type	

SOIL NAME SLOPE CLASS	LOADING RATE (G/D/FT^2)	DEPTH TO RESTRICTING LAYERS (INCHES)	SOIL IMPROVEMENT PRACTICES/ NOTES/ PERC STATUS
Lily, Hendon, Lonewood Complex, 0-5%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 0 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.
Lily, Hendon, Lonewood Complex, 5-10%	0.25	>20"	This complex consists three similar map units that are so intermingled that they cannot be separated. The Lily series consists of moderately deep, well drained soils formed in residuum weathered primarily from sandstone. Permeability is moderately rapid. These nearly level to very steep soils are on ridge tops and hill sides. Slopes range from 5 to 10 percent. The Hendon series consists of deep, well drained, nearly level to sloping soils that formed in a loamy mantle 18 to 30 inches thick that is higher in silt content than the underlying loamy residuum that weathered from interbedded flat-lying sandstone, siltstone and shale. These soils are on broad interfluves of the Cumberland Plateau and have a fragic layer in the subsoil. Slopes range from 0 to 10 percent. The Lonewood series consists of deep and very deep, well drained, moderately permeable soils. They formed in a silty mantle 1 to 3 feet thick and the underlying residuum of weathered shale and sandstone. These soils are on broad undulating and rolling plateaus of the Cumberland Mountains. Slopes range from 0 to 20 percent, but commonly range from 2 to 12 percent.
Ramsey, 0-5%	0.0	<20""	The Ramsey series consists of shallow and very shallow, somewhat excessively drained soils that formed in residuum or colluvium weathered from sandstone or quartzite. They are dominantly on plateaus and upper slopes of mountains. Runoff is medium to rapid and permeability is rapid. Slopes range from 3 to 70 percent.
Atkins, 0-5%	0.0	<20" TO REDOXIMORPHIC FEATURES	The Atkins series consists of very deep, poorly drained soils formed in acid alluvium washed from upland soils that formed in shale and sandstone. Permeability is slow to moderate. Slope ranges from 0 to 3 percent. Mean annual precipitation is about 46 inches and the mean annual air temperature is about 54 degrees F.
Sequoia, 0-15%	0.25	>20"	Sequoia soils have formed in residuum from shale and are typically 40 inches or more to auger refusal. Subsoils are clayey and somewhat poorly structured with red and yellow mottling at depths below 24 inches.


CAPPED REBAR FOUND
MILL CREEK
MILL CREEK
Lily < 20" 5-15%
Lily < 20" 5-15%
Lily >20" 5-15%
Ramsey 5-15%
Ramsey 5-15%
Ramsey 5-15%
Lily >20" 5-10%
Lily/20" 5-10%
Lily >20" 5-10%
Ramsey 5-15%
Lily/Lonewood/Hendon Complex 0-5%
Lily/Lonewood/Hendon Complex 0-5%
Lily >20" 5-10%
Lily/Lonewood/Hendon Complex 0-5%
Atkins 0-5%
Lily >20" 5-10%
Lily/Lonewood/Hendon Complex 0-5%
Atkins 0-5%
TREE LINE
Atkins 0-5%
Sequoia 0-5%
Sequoia 5-15%
Atkins 0-5%
EXISTING YELLOW PAINTED LINE
Lily/Lonewood/Hendon Complex 4-8%
Lily/Lonewood/Hendon Complex 0-5%
Ramsey 5-15%
Atkins 0-5%
Atkins 0-5%
Ramsey 5-15%
Lily/Lonewood/Hendon Complex 4-8%
Atkins 0-5%
Atkins 0-5%
Lily/Lonewood/Hendon Complex 4-8%
Sequoia 5-15%
Lily/Lonewood/Hendon Complex 0-5%
Atkins 0-5%
Lily/Lonewood/Hendon Complex 0-5%
Sequoia 0-5%
Lily/Lonewood/Hendon Complex 4-8%
Atkins 0-5%
Sequoia 5-15%