EXHIBITS

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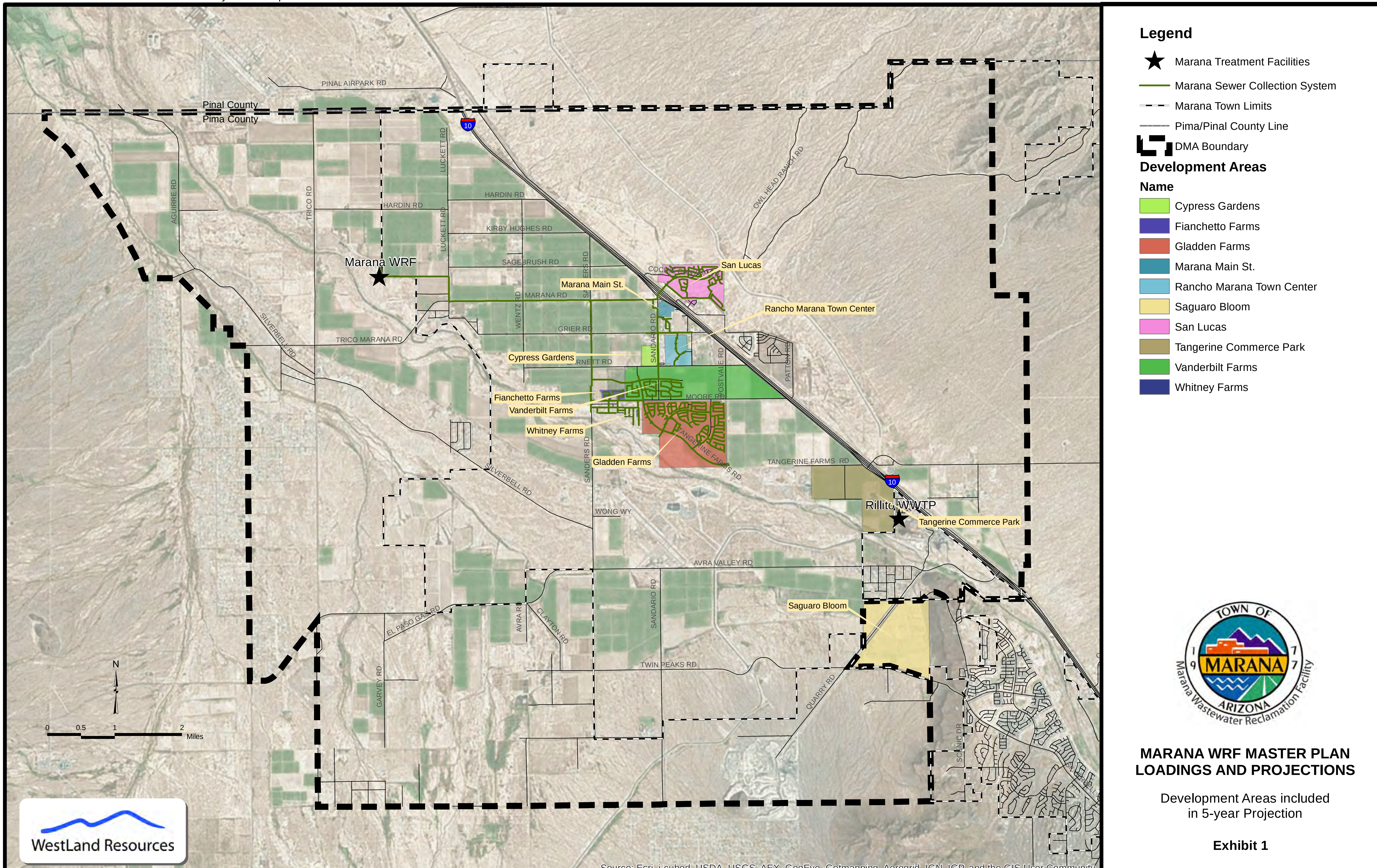


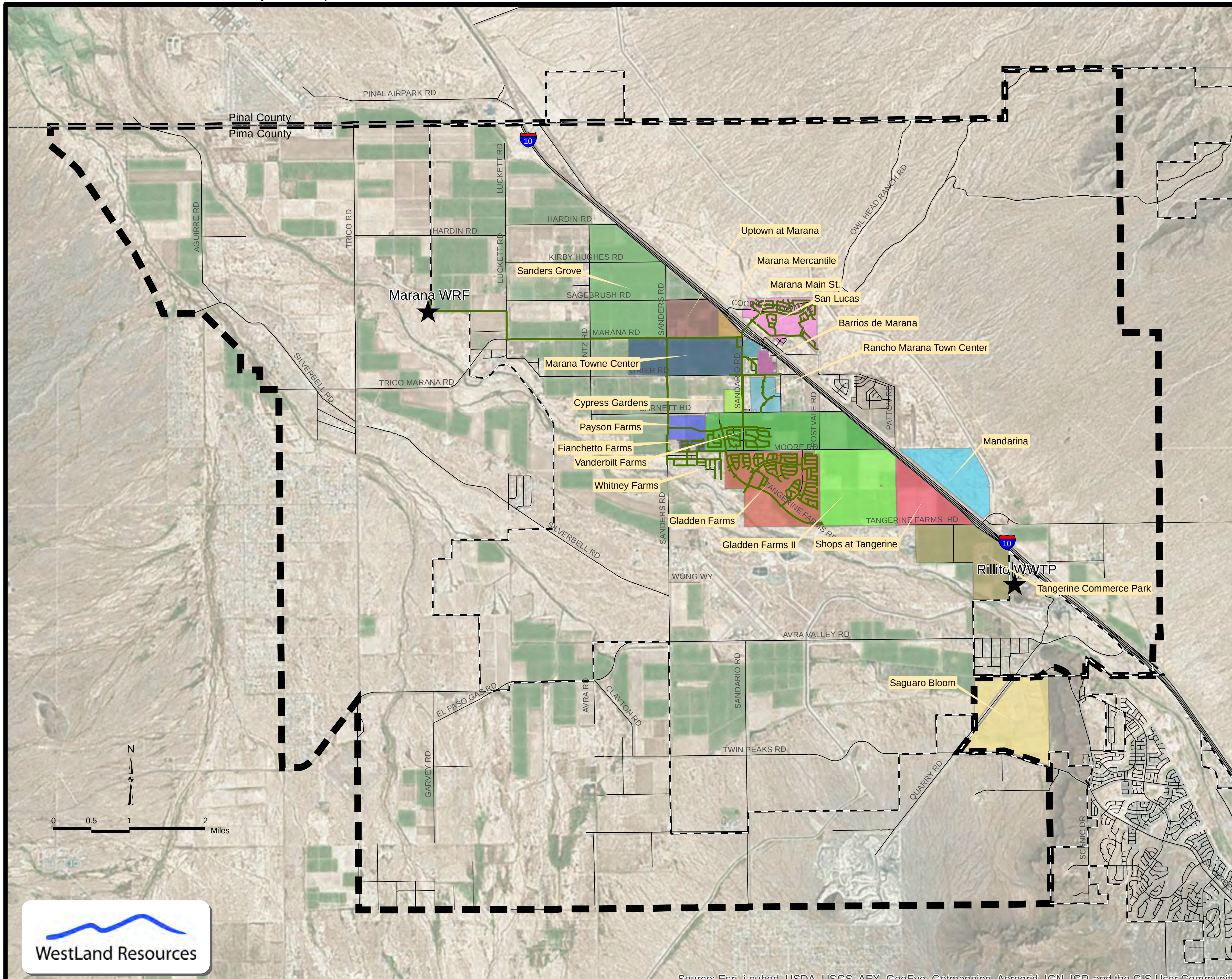
TOWN OF MARANA ARIZONA 1977
Marana Wastewater Reclamation Facility

MARANA WRF MASTER PLAN LOADINGS AND PROJECTIONS

Development Areas included in 5-year Projection

Exhibit 1

M:\Jobs\500's\527.75\ENG\MXD\Exhibit 2. 10-year development.mxd



Legend

- Marana Treatment Facilities
- Marana Sewer Collection System
- Marana Town Limits
- Pima/Pinal County Line
- DMA Boundary

Development Areas

Name

- Barrios de Marana
- Cypress Gardens
- Fianchetto Farms
- Gladden Farms
- Gladden Farms II
- Mandarina
- Marana Main St.
- Marana Mercantile
- Marana Towne Center
- Payson Farms
- Rancho Marana Town Center
- Saguaro Bloom
- San Lucas
- Sanders Grove
- Shops at Tangerine
- Tangerine Commerce Park
- Uptown at Marana
- Vanderbilt Farms
- Whitney Farms

TOWN OF MARANA ARIZONA 1977 Marana Wastewater Reclamation Facility

MARANA WRF MASTER PLAN LOADINGS AND PROJECTIONS

Development Areas included in 10-year Projection

Exhibit 2

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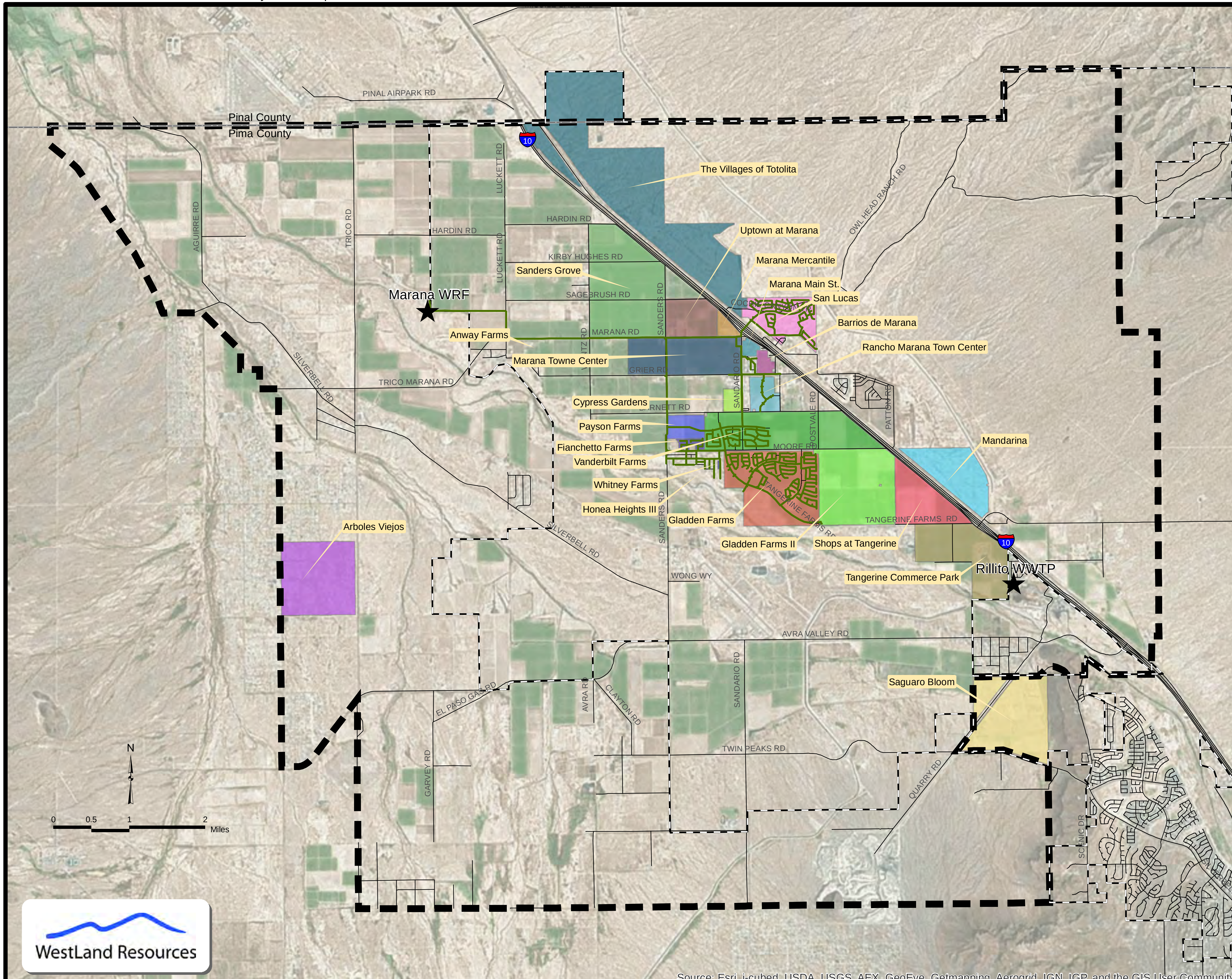

Legend

- ★ Marana Treatment Facilities
- Marana Sewer Collection System
- Marana Town Limits
- Pima/Pinal County Line
- DMA Boundary

Development Areas

Name

- Barrios de Marana
- Cypress Gardens
- Fianchetto Farms
- Gladden Farms
- Gladden Farms II
- Jaguar Lane
- Mandarina
- Marana Main St.
- Marana Mercantile
- Marana Towne Center
- Payson Farms
- Rancho Marana Town Center
- Saguaro Bloom
- San Lucas
- Sanders Grove
- Shops at Tangerine
- Tangerine Commerce Park
- The Villages of Tortolita
- Uptown at Marana
- Vanderbilt Farms
- Whitney Farms

TOWN OF MARANA ARIZONA 1977
Marana Wastewater Reclamation Facility

MARANA WRF MASTER PLAN
LOADINGS AND PROJECTIONS

Development Areas included
in 20-year Projection

Exhibit 3

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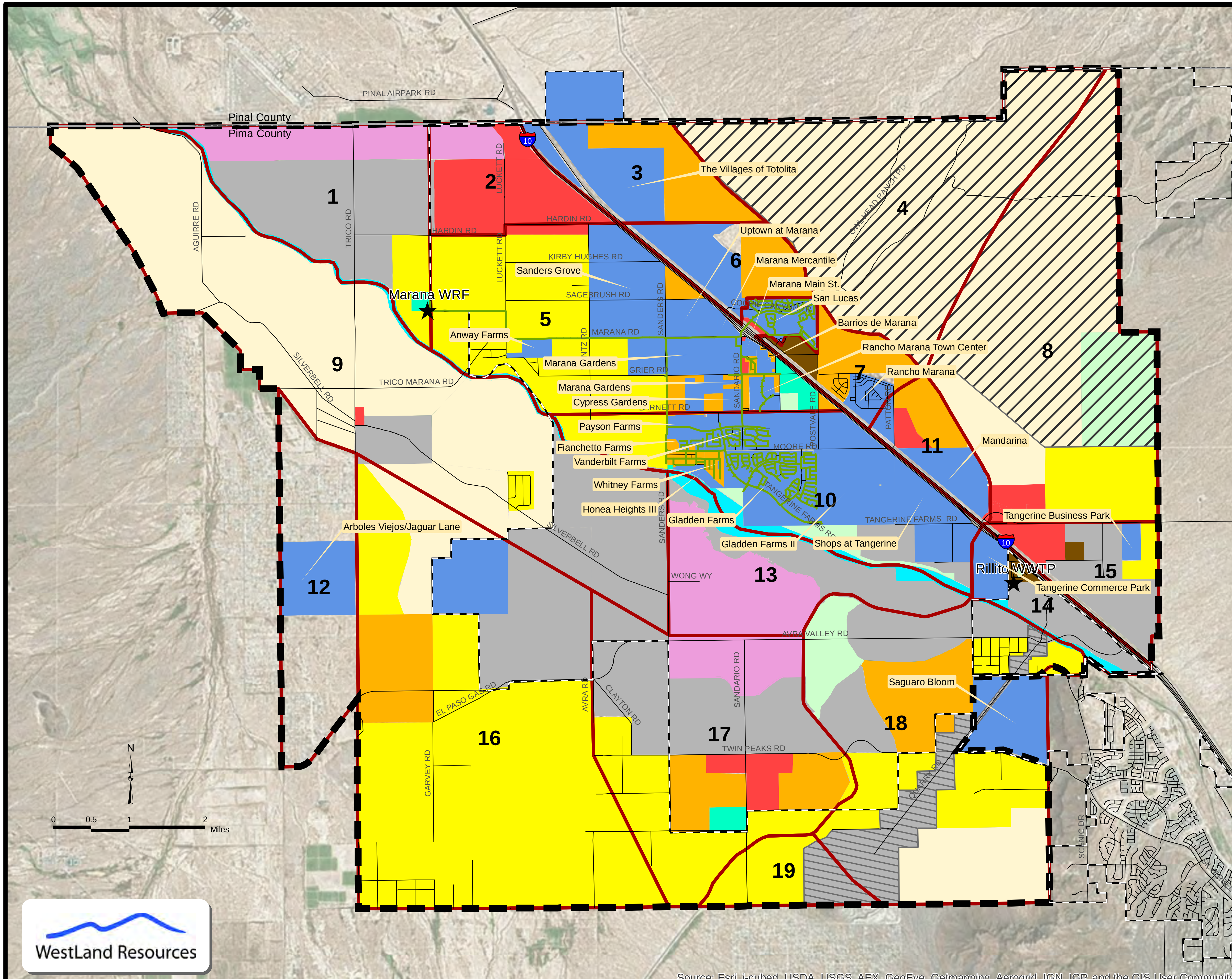


Legend

- Marana Treatment Facilities
- Marana Sewer Collection System
- Marana Town Limits
- Pima/Pinal County Line
- DMA Boundary
- Sewer Basins
- No Expansion of Urban Services

General Plan (2010)

Land_Use

- Airport
- Commercial
- Industrial
- Industrial (Low Density)
- Public\Institutional
- Low Density Residential
- Medium Density Residential
- High Density Residential
- Mixed Rural
- Rural Density Residential
- Specific Plan
- Park
- Riverine Buffer

TOWN OF MARANA ARIZONA 1977
Marana Wastewater Reclamation Facility

MARANA WRF MASTER PLAN LOADINGS AND PROJECTIONS

Marana 2010 General Plan Land Use, Development Areas, and Sewer Basins

Exhibit 4

APPENDIX A

DAILY FLOW DATA
FROM APRIL 2015
THROUGH
SEPTEMBER 2015

Marana WRF 2012 Daily Flow (MGD)

	January	February	March	April	May	June	July	August	September	October	November	December
1				0.285	0.273	0.247		0.310	0.262	0.298	0.282	0.283
2				0.316	0.269	0.261		0.381	0.271	0.288	0.282	0.292
3				0.270	0.217	0.276		0.281	0.272	0.271	0.279	0.315
4				0.276	0.278	0.264		0.266	0.316	0.279	0.293	0.299
5				0.269	0.277	0.254		0.275	0.315	0.271	0.317	0.295
6				0.268	0.303	0.250		0.311	0.284	0.276	0.295	0.293
7				0.281	0.288	0.253		0.340	0.294	0.286	0.283	0.289
8				0.284	0.272	0.261			0.274	0.285	0.282	0.289
9				0.276	0.318	0.266			0.286	0.274	0.290	0.296
10				0.266	0.283	0.275			0.324	0.266	0.284	0.326
11				0.275	0.262	0.250			0.261	0.260	0.315	0.309
12				0.275	0.281	0.256			0.284	0.265	0.320	0.306
13				0.269	0.283	0.265			0.283	0.269	0.315	0.314
14				0.292	0.275	0.268			0.286	0.276	0.295	0.296
15				0.300	0.275	0.265			0.278	0.306	0.298	0.345
16				0.278	0.254	0.274			0.288	0.285	0.287	0.314
17				0.261	0.267	0.261			0.302	0.288	0.282	0.370
18				0.263	0.260	0.264			0.290	0.298	0.304	0.314
19				0.265	0.279	0.258			0.269	0.288	0.325	0.302
20				0.251	0.286	0.260			0.298	0.278	0.300	0.311
21				0.258	0.271	0.248			0.269	0.299	0.291	0.306
22				0.292	0.259	0.265			0.271	0.323	0.293	0.302
23				0.291	0.263	0.278			0.298		0.328	0.308
24				0.275	0.257	0.259			0.306	0.286	0.269	0.310
25				0.275	0.269	0.258		0.290	0.208	0.277	0.289	0.331
26				0.291	0.265	0.270		0.295	0.276	0.282	0.318	0.297
27				0.288	0.258	0.255		0.306	0.272	0.279	0.296	0.293
28				0.292	0.280	0.255		0.294	0.278	0.362	0.291	0.303
29				0.300	0.257	0.257		0.280	0.272		0.284	0.319
30				0.279	0.256	0.262		0.274	0.289	0.290	0.283	0.307
31					0.256			0.274		0.279		0.315
Average				0.279	0.271	0.261		0.298	0.283	0.286	0.296	0.308

Marana WRF 2013 Daily Flow (MGD)

	January	February	March	April	May	June	July	August	September	October	November	December
1	0.345	0.303	0.295	0.313	0.284	0.361	0.361	0.362	0.425	0.393	0.306	0.389
2	0.368	0.282	0.281	0.288	0.311	0.349	0.342	0.389	0.422	0.390	0.297	0.371
3	0.388	0.301	0.294	0.284		0.367	0.349	0.391	0.445	0.395	0.319	0.344
4	0.322	0.321	0.321	0.277	0.161	0.360	0.342	0.452	0.403	0.388	0.340	0.331
5	0.287	0.301	0.285	0.288	0.278	0.362	0.353	0.584	0.392	0.388	0.333	0.349
6	0.311	0.286	0.281	0.276	0.294	0.357	0.325	0.429	0.392	0.387	0.327	0.323
7	0.338	0.276	0.276	0.321	0.336	0.355	0.351	0.435	0.391	0.399	0.295	0.340
8	0.309	0.306	0.278	0.312	0.296	0.356	0.390	0.397	0.404	0.389	0.287	0.351
9	0.3	0.262	0.279	0.299	0.291	0.361	0.349	0.396	0.434	0.388	0.297	0.338
10	0.298	0.32	0.306	0.293	0.291	0.365	0.344	0.389	0.416	0.379	0.308	0.328
11	0.307	0.343	0.323	0.288	0.290	0.358	0.350	0.401	0.423	0.393	0.309	0.324
12	0.316	0.329	0.299	0.292	0.341	0.354	0.366	0.424	0.395	0.378	0.311	0.327
13	0.318	0.304	0.285	0.294	0.379	0.350	0.439	0.411	0.396	0.393	0.296	0.309
14	0.36	0.3	0.273	0.296	0.405	0.351	0.369	0.395	0.379	0.421	0.304	
15	0.321	0.288	0.277	0.313	0.373	0.358	0.375	0.395	0.455	0.409	0.259	
16	0.327	0.279	0.276	0.310	0.363	0.369	0.359	0.397	0.408	0.394	0.300	0.327
17	0.311	0.296	0.268	0.286	0.526	0.376	0.351	0.390	0.408	0.385	0.329	0.319
18	0.293	0.315	0.308	0.292	0.377	0.367	0.363	0.401	0.402	0.413	0.343	0.314
19	0.28	0.302	0.277	0.291	0.412	0.352	0.348	0.415	0.395	0.414	0.213	0.303
20	0.294	0.284	0.268	0.275	0.461	0.354	0.357	0.379	0.389	0.408	0.202	0.331
21	0.308	0.336	0.273	0.298	0.426	0.362	0.371	0.383	0.392	0.429	0.308	0.344
22	0.304	0.286	0.268	0.322	0.395	0.352	0.391	0.381	0.410	0.453	0.316	0.358
23	0.27	0.296	0.261	0.288	0.387	0.370	0.381	0.388	0.425	0.442	0.474	0.329
24	0.28	0.296	0.292	0.277	0.399	0.366	0.389	0.412	0.412	0.285	0.475	0.359
25	0.31	0.325	0.306	0.282	0.389	0.360	0.417	0.413	0.384	0.300	0.388	0.322
26	0.271	0.326	0.292	0.288	0.359	0.352	0.391	0.433	0.384	0.292	0.328	0.317
27	0.334	0.294	0.275	0.277	0.347	0.335	0.360	0.445	0.388	0.312	0.331	0.321
28	0.34	0.291	0.281	0.294	0.361	0.337	0.371	0.382	0.383	0.333	0.356	0.327
29	0.317		0.28	0.302	0.353	0.339	0.384	0.406	0.383	0.312	0.373	0.334
30	0.294		0.272	0.282	0.350	0.353	0.397	0.418	0.412	0.316	0.332	0.327
31	0.307		0.3		0.361		0.318	0.402		0.315		0.340
Average	0.314	0.302	0.285	0.293							0.322	0.334

Data within shaded cells was not used in analysis (5/3/2013 through 10/24/1013); possible problem in headworks or flow meter.

Marana WRF 2014 Daily Flow (MGD)

	January	February	March	April	May	June	July	August	September	October	November	December
1	0.34	0.304	0.32	0.321	0.320	0.305	0.313	0.314	0.305	0.332	0.332	0.348
2	0.322	0.331	0.334	0.316	0.310	0.318	0.297	0.315	0.340	0.327	0.350	0.335
3	0.317	0.355	0.365	0.319	0.310	0.303	0.296	0.323	0.323	0.336	0.377	0.348
4	0.308	0.331	0.322	0.315	0.316	0.287	0.291	0.348	0.319	0.325	0.346	0.327
5	0.337	0.319	0.331	0.308	0.339	0.300	0.308	0.336	0.318	0.331	0.334	0.381
6	0.361	0.327	0.337	0.317	0.323	0.301	0.293	0.318	0.330	0.344	0.335	0.328
7	0.338	0.317	0.322	0.341	0.316	0.291	0.308	0.318	0.333	0.319	0.322	0.349
8	0.322	0.309	0.321	0.341	0.315	0.304	0.310	0.311	0.351	0.315	0.326	0.372
9	0.321	0.325	0.327	0.320	0.316	0.311	0.302	0.310	0.493	0.401	0.332	0.348
10	0.328	0.35	0.368	0.311	0.309	0.300	0.295	0.319	0.348	0.379	0.357	0.328
11	0.32	0.327	0.326	0.304	0.326	0.297	0.296	0.353	0.328	0.320	0.322	0.338
12	0.336	0.313	0.32	0.305	0.338	0.332	0.296	0.333	0.313	0.330	0.339	0.331
13	0.369	0.332	0.318	0.319	0.334	0.298	0.308	0.346	0.317	0.352	0.327	0.323
14	0.342	0.333	0.318	0.342	0.311	0.289	0.329	0.344	0.342	0.345	0.327	0.399
15	0.323	0.317	0.305	0.328	0.316	0.300	0.314	0.323	0.358	0.333	0.317	0.381
16	0.32	0.319	0.318	0.314	0.309	0.303	0.376	0.317	0.344	0.333	0.335	0.349
17	0.316	0.342	0.354	0.345	0.314	0.301	0.304	0.317	0.338	0.329	0.369	0.336
18	0.308	0.33	0.337	0.312	0.316	0.300	0.310	0.343	0.362	0.331	0.344	0.380
19	0.319	0.316	0.3	0.355	0.338	0.296	0.300	0.326	0.338	0.338	0.339	0.350
20	0.341	0.31	0.295	0.342	0.322	0.296	0.310	0.321	0.331	0.372	0.330	0.340
21	0.328	0.305	0.307	0.326	0.322	0.288	0.326	0.318	0.346	0.353	0.333	0.359
22	0.312	0.302	0.292	0.331	0.326	0.299	0.319	0.324	0.365	0.341	0.325	0.353
23	0.307	0.319	0.307	0.349	0.326	0.313	0.310	0.313	0.337	0.334	0.363	0.334
24	0.308	0.348	0.292	0.316	0.326	0.298	0.310	0.325	0.336	0.333	0.373	0.345
25	0.307	0.333	0.327	0.314	0.315	0.296	0.307	0.349	0.323	0.327	0.350	0.384
26	0.319	0.318	0.324	0.308	0.304	0.295	0.313	0.337	0.331	0.340	0.349	0.331
27	0.344	0.318	0.318	0.327	0.319	0.292	0.317	0.361	0.324	0.368	0.384	0.340
28	0.344	0.325	0.35	0.343	0.298	0.297	0.336	0.318	0.338	0.338	0.308	0.358
29	0.314		0.317	0.324	0.298	0.302	0.321	0.328	0.371	0.336	0.338	0.375
30	0.315		0.317	0.320	0.301	0.313	0.319	0.301	0.345	0.330	0.384	0.351
31	0.306		0.35		0.267		0.309	0.302		0.330		0.348
Average	0.326	0.324	0.324	0.324	0.316	0.301	0.311	0.326	0.342	0.339	0.342	0.351

Marana WRF 2015 Daily Flow (MGD)

	January	February	March	April	May	June	July	August	September	October	November	December
1	0.381	0.585	0.334	0.328	0.324	0.351	0.327	0.382	0.347			
2	0.393	0.501	0.371	0.332	0.313	0.332	0.331	0.412	0.342			
3	0.356	0.426	0.37	0.333	0.353	0.326	0.318	0.423	0.332			
4	0.37	0.343	0.344	0.338	0.354	0.322	0.315	0.515	0.337			
5	0.407	0.333	0.346	0.347	0.356	0.324	0.321	0.330	0.350			
6	0.372	0.333	0.341	0.369	0.357	0.323	0.326	0.335	0.336			
7	0.348	0.326	0.347	0.349	0.337	0.333	0.345	0.350	0.332			
8	0.349	0.345	0.357	0.348	0.341	0.345	0.332	0.340	0.372			
9	0.345	0.377	0.377	0.344	0.333	0.329	0.332	0.342	0.367			
10	0.328	0.35	0.359	0.340	0.348	0.320	0.338	0.376	0.346			
11	0.349	0.331	0.342	0.331	0.356	0.325	0.325	0.353	0.359			
12	0.376	0.362	0.342	0.337	0.354	0.217	0.334	0.362	0.351			
13	0.35	0.391	0.335	0.378	0.350	0.312	0.351	0.387	0.348			
14	0.341	0.392	0.336	0.344	0.342	0.311	0.344	0.337	0.372			
15	0.345	0.403	0.347	0.331	0.345	0.334	0.337	0.323	0.368			
16	0.338	0.421	0.35	0.338	0.336	0.321	0.346	0.345	0.346			
17	0.334	0.42	0.327	0.340	0.350	0.311	0.311	0.359	0.346			
18	0.343	0.396	0.308	0.335	0.365	0.321	0.334	0.366	0.345			
19	0.362	0.368	0.324	0.333	0.350	0.326	0.335	0.424	0.333			
20	0.35	0.335	0.323	0.359	0.353	0.313	0.342	0.336	0.333			
21	0.336	0.329	0.315	0.341	0.345	0.320	0.333	0.347	0.369			
22	0.337	0.345	0.325	0.326	0.335	0.332	0.326	0.346	0.376			
23	0.34	0.367	0.359	0.331	0.334	0.333	0.321	0.355	0.366			
24	0.335	0.349	0.346	0.334	0.330	0.324	0.333	0.408	0.347			
25	0.345	0.349	0.342	0.333	0.339	0.336	0.313	0.385	0.334			
26	0.383	0.342	0.332	0.351	0.357	0.333	0.326	0.407	0.337			
27	0.354	0.318	0.335	0.420	0.337	0.320	0.335	0.352	0.344			
28	0.339	0.318	0.32	0.357	0.332	0.343	0.328	0.336	0.366			
29	0.336		0.343	0.336	0.323	0.356	0.316	0.333	0.359			
30	0.333		0.36	0.327	0.324	0.336	0.317	0.345	0.344			
31	0.453		0.343		0.320		0.313	0.372				
Average	0.356	0.373	0.342	0.344	0.342	0.324	0.329	0.367	0.350			

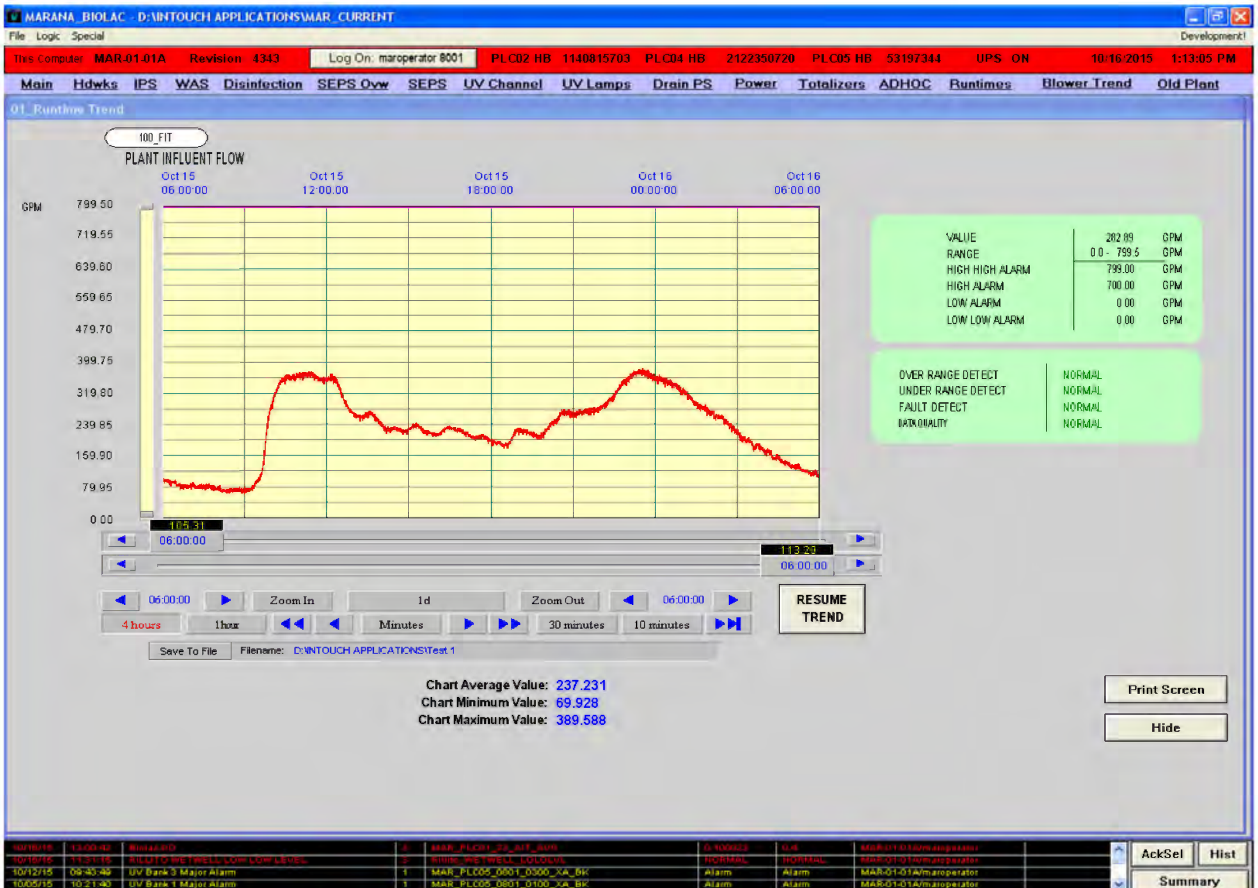
APPENDIX B

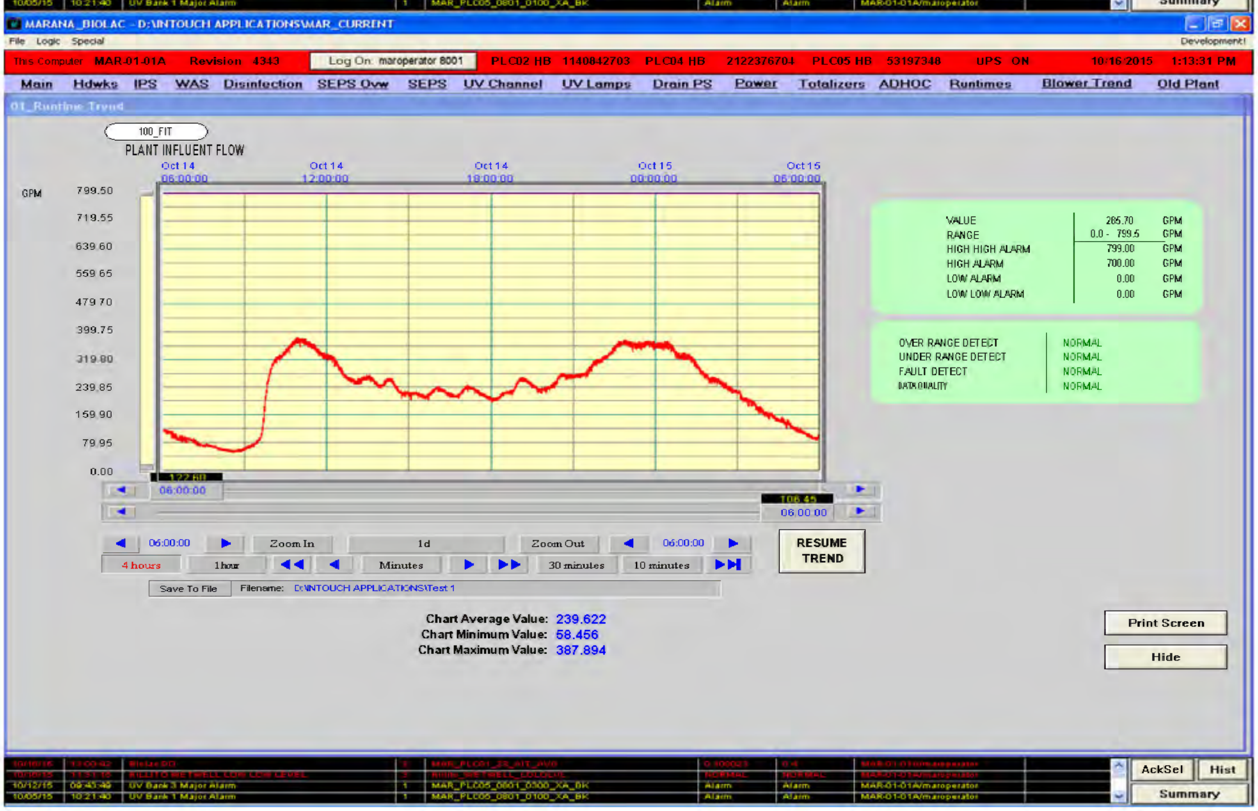
COMMERCIAL CONNECTIONS AND AVERAGE WATER USE FOR 2015

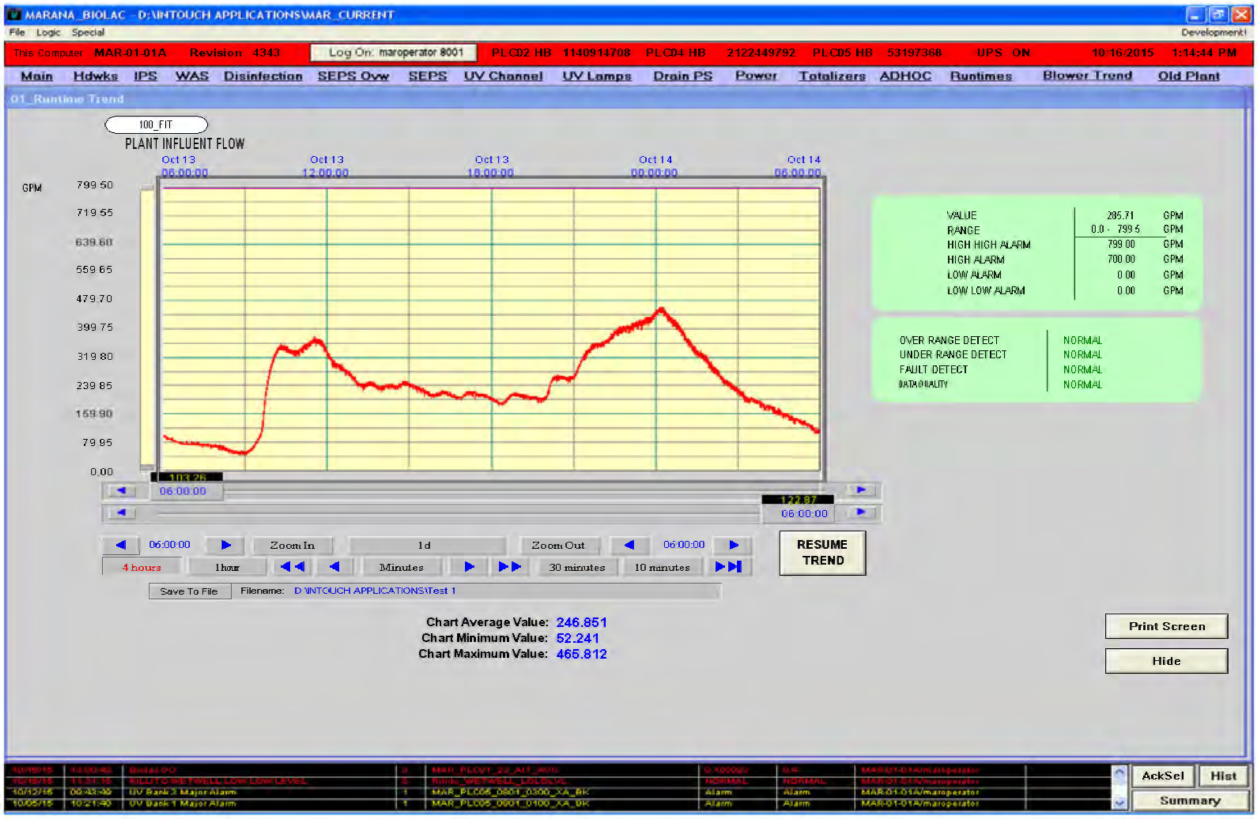
Sewer Rate	Account #	Meter Number	Transponder #	Service Address	Full Name	Consmp for 2014	Avg Gallons/Day
C-Commercial Marana	02000520-01	9066575	81225824	13370 N Lon Adams Rd	TOM - Parks & Rec	70,160	192.22
C-Commercial Marana	02002600-01	10257779	81165221	13961 N Sandario Rd	Circle K Site 2708514	592,370	1622.93
C-Commercial Marana	02002610-01	6590069	80275430	13960 N Sandario Rd	Marana Chevron	211,555	579.60
C-Commercial Marana	02002630-01	19936994	80278088	13915 N Sandario Rd	La Tumbleweed Lounge	102,540	280.93
C-Commercial Marana	02002660-03	12096241	80278814	13865 N Sandario Rd	Sandario Discount Market	58,498	160.27
C-Commercial Marana	02002670-01	19936991	80268332	13780 N Sandario Rd	Pierce Automotive	33,431	91.59
C-Commercial Marana	02002740-03	10257776	80276407	13644 N Sandario Rd	AZ Youth Partnership	125,891	344.91
C-Commercial Marana	02003050-02	12570303	82499446	11780 W Camino Pinos #B	Pascua Yaqui Tribe	40,684	111.46
C-Commercial Marana	02021718-01	14139543	80276404	13475 N Marana Main St	Northwest Fire District	195,950	536.85
C-Commercial Marana	02024587-02	6589999	82590223	11825 W Grier Rd	Family Dollar #7696	355,642	974.36
C-Commercial Marana	02024686-01	10242992	82800171	13395 N Marana Main St	Marana Health Center	1,812,400	4965.48
C-Commercial Marana	02024916-01	11549191	84607314	13395 N Marana Main St-#B	Marana Health Center	106,200	290.96
C-Commercial Marana	02024920-01	11279 W Grier		11279 W Grier Rd	Marana Unified School District	3,768,000	10323.29
C-Commercial Marana	02025725-01	12651269	86155703	13934 N Sandario Rd	McDonald's	121,500	1350.00
MF-Multi-Family Marana	02002820-02	10257771	80276542	13377 N Sandario Rd	Spall Enterprises Inc	2,451,800	6717.26
SC-Laundromat Marana	02005750-04	10257770	80276412	13865 N Sandario Rd	Sandario Discount Laundry	335,590	919.42
SF-Restaurant Fast Food Marana	02002640-01	12570231	80278811	13905 N Sandario Rd	Linda Molitor	210,108	575.64
SG-Self-Serv Car Wash Marana	02002680-01	5006883	80276200	13770 N Sandario Rd	Above All Auto Wash	128,848	353.01
C-Commercial Marana	02011750-02	10294874	80276753	12471 W Moore Rd	Greg Lindsey	71568	196.08

APPENDIX C

Continuous Flow Graphs from September 15 through September 31, 2015


MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLC02 HB 1140815703 PLC04 HB 2122350720 PLC05 HB 53197344 UPS ON
10/16/2015 1:13:05 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
100_FIT
PLANT INFLUENT FLOW
Oct 15 06:00:00
Oct 15 12:00:00
Oct 15 18:00:00
Oct 16 00:00:00
Oct 16 06:00:00
GPM
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 282.89 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
105.31
06:00:00
113.29
06:00:00
06:00:00
Zoom In
1d
Zoom Out
06:00:00
RESUME TREND
4 hours
1hour
Minutes
30 minutes
10 minutes
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 237.231
Chart Minimum Value: 69.928
Chart Maximum Value: 389.588
Print Screen
Hide
10/12/15 09:43:49 UV Bank 3 Major Alarm
10/05/15 10:21:40 UV Bank 1 Major Alarm
AckSel
Hist
Summary


MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLC02 HB 1140842703 PLC04 HB 2122376704 PLC05 HB 53197348 UPS ON
10/16/2015 1:13:31 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
100_FIT
PLANT INFLUENT FLOW
Oct 14 06:00:00
Oct 14 12:00:00
Oct 14 18:00:00
Oct 15 00:00:00
Oct 15 06:00:00
GPM
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 285.70 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
06:00:00
106.45
06:00:00
06:00:00
Zoom In
1d
Zoom Out
06:00:00
RESUME TREND
4 hours
1hour
Minutes
30 minutes
10 minutes
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 239.622
Chart Minimum Value: 58.456
Chart Maximum Value: 387.894
Print Screen
Hide
10/12/15 09:43:49 UV Bank 3 Major Alarm
10/05/15 10:21:40 UV Bank 1 Major Alarm
AckSel
Hist
Summary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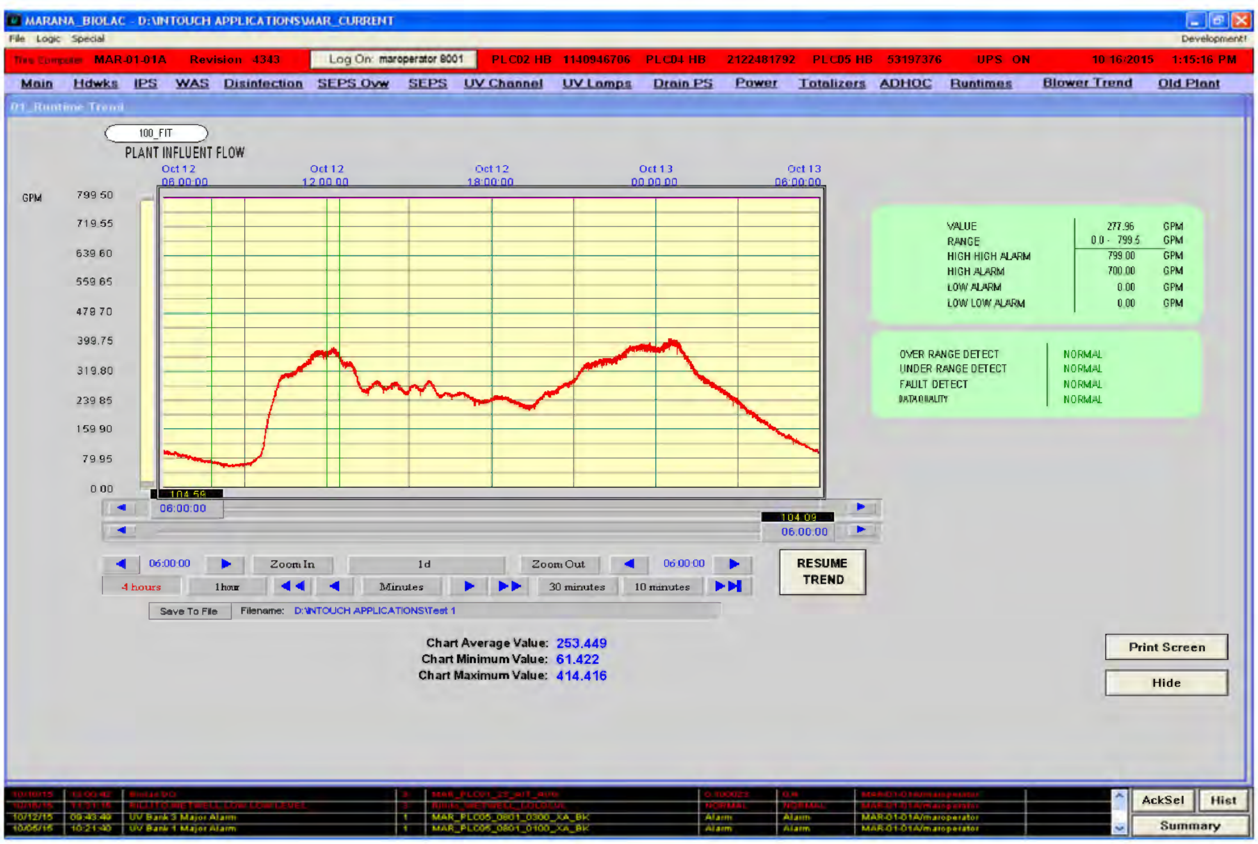
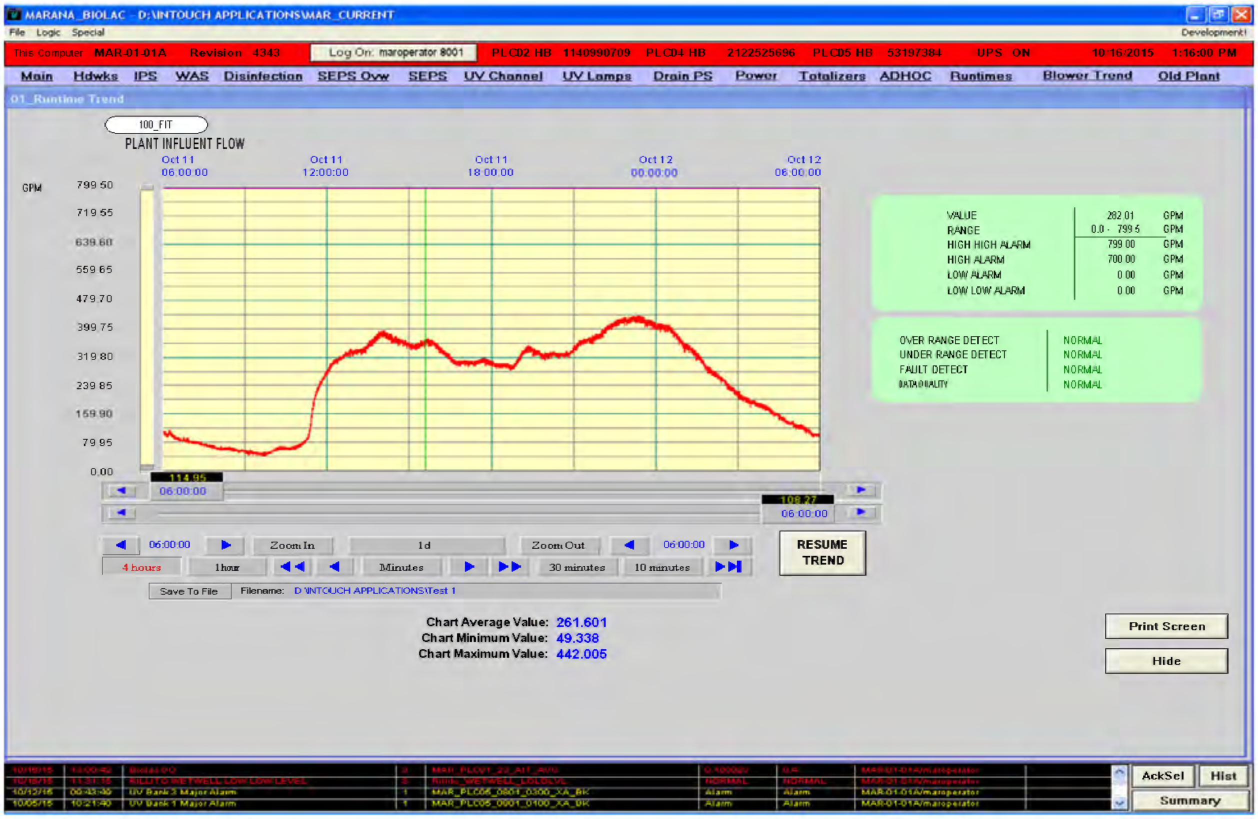
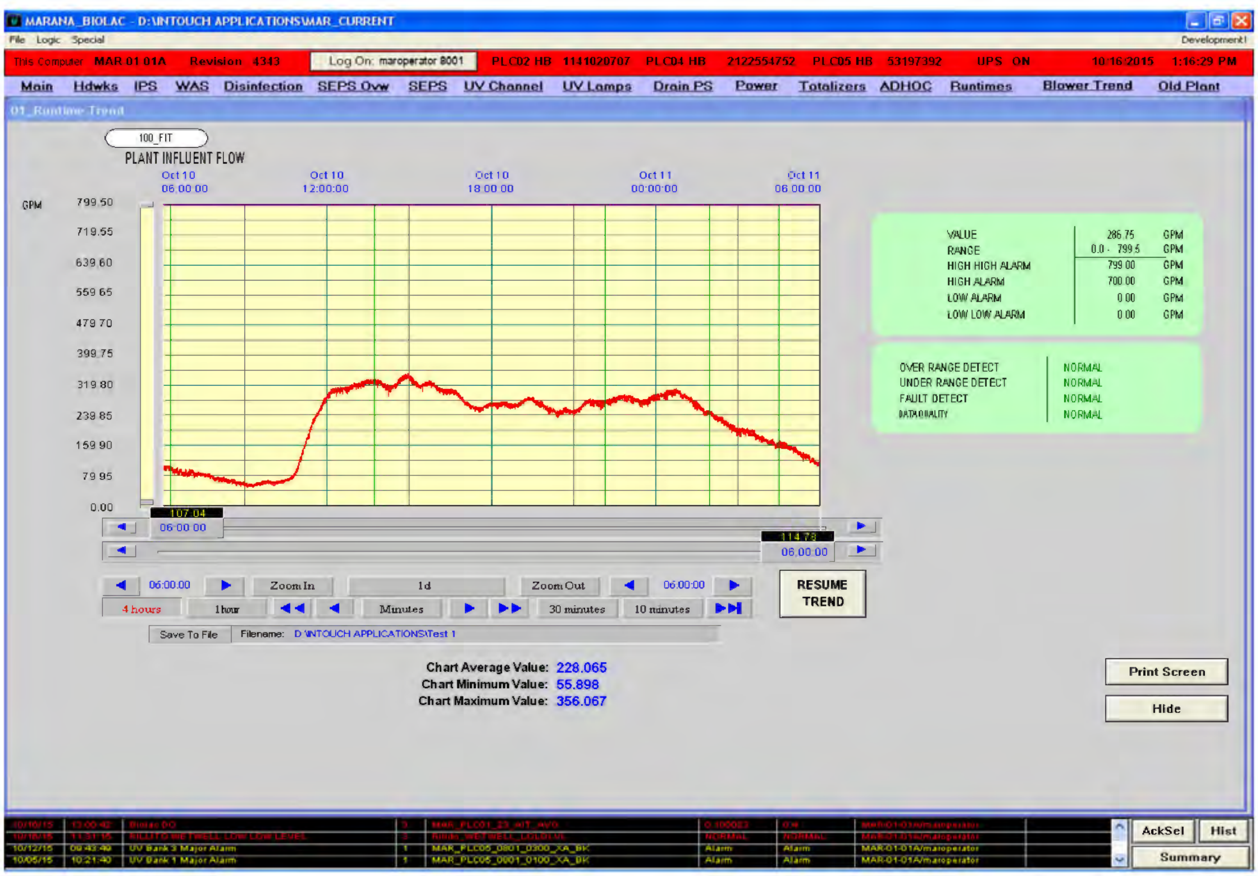
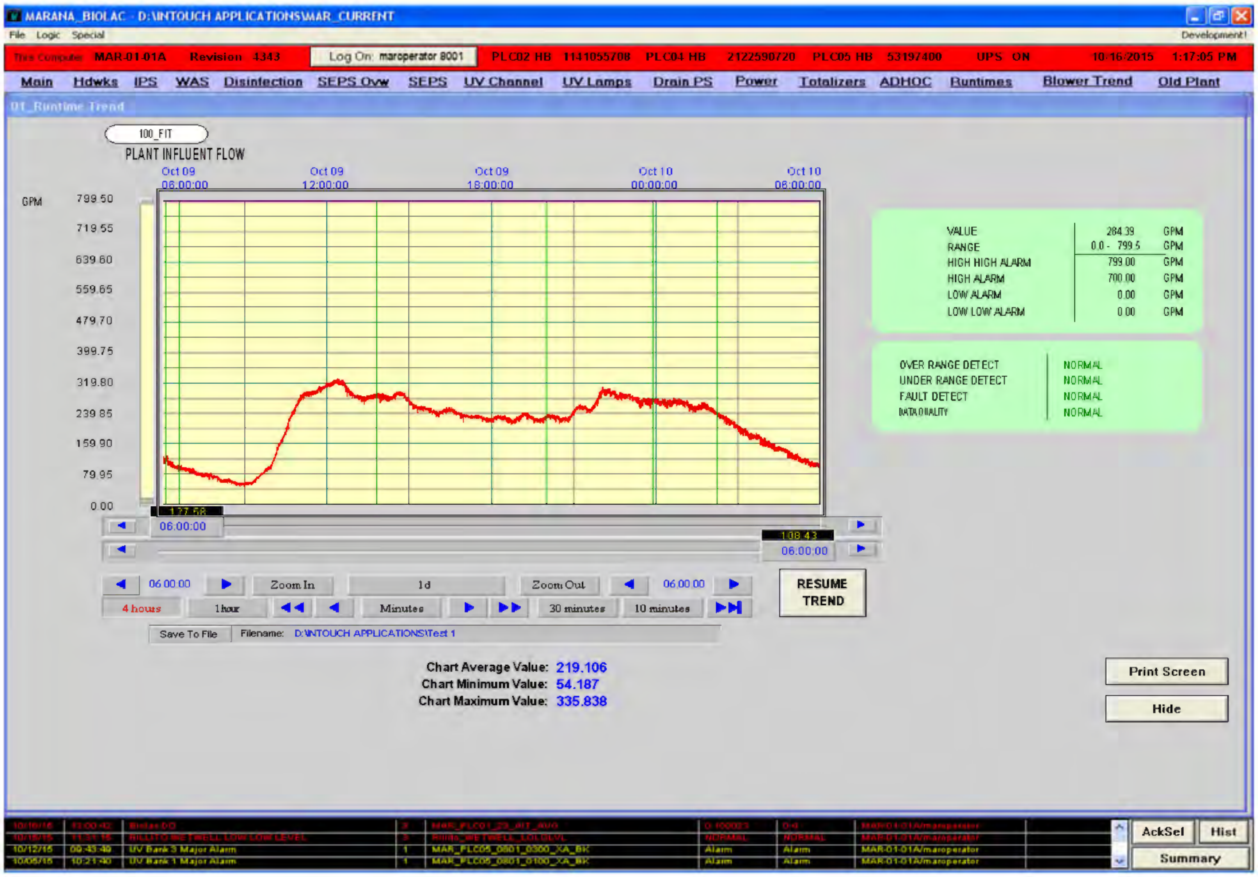
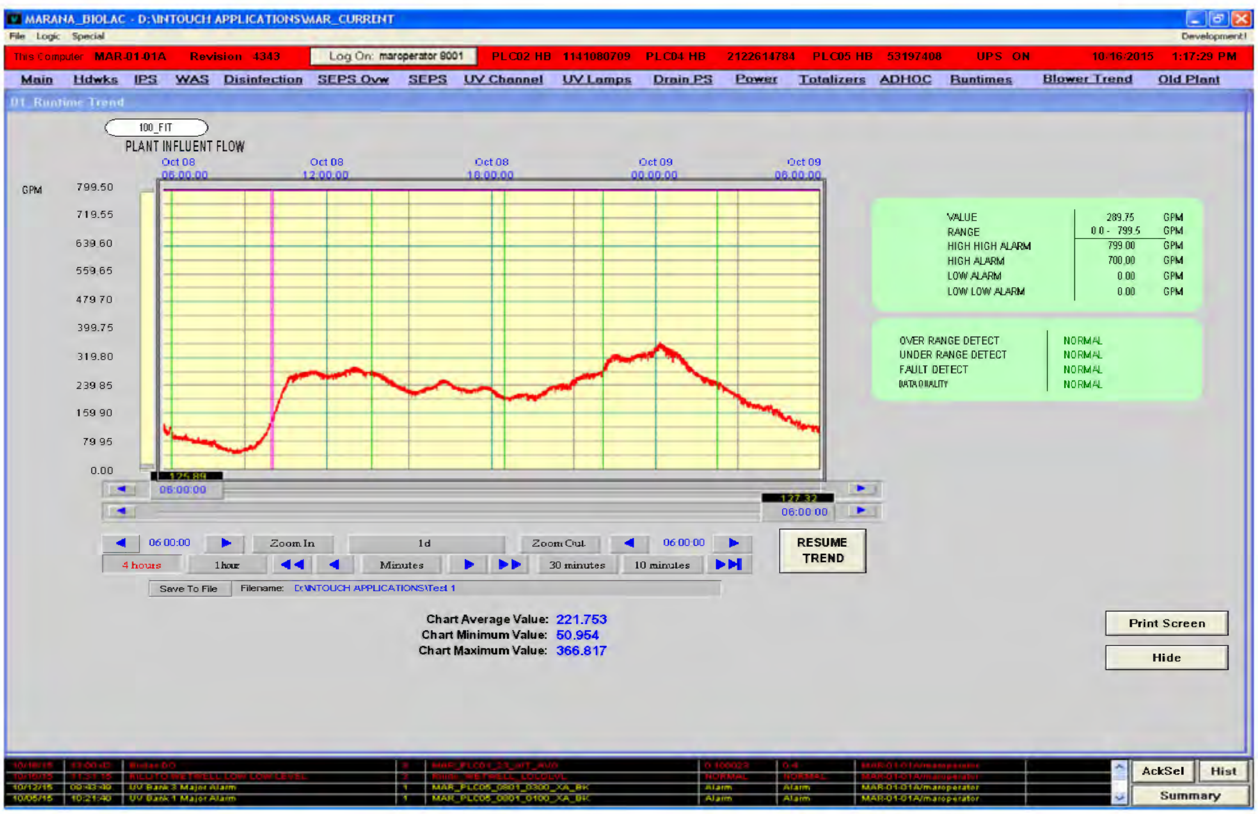
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLC02 HB 1140914708 PLC04 HB 2122449792 PLC05 HB 53197368 UPS ON
10/16/2015 1:14:44 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
01_Runtime Trend
100_FIT
PLANT INFLUENT FLOW
Oct 13 06:00:00
Oct 13 12:00:00
Oct 13 18:00:00
Oct 14 00:00:00
Oct 14 06:00:00
GPM
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 285.71 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
103.26
06:00:00
122.87
06:00:00
06:00:00
Zoom In
1d
Zoom Out
06:00:00
RESUME TREND
4 hours
1 hour
Minutes
30 minutes
10 minutes
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 246.851
Chart Minimum Value: 52.241
Chart Maximum Value: 465.812
Print Screen
Hide
AckSel
Hist
Summary

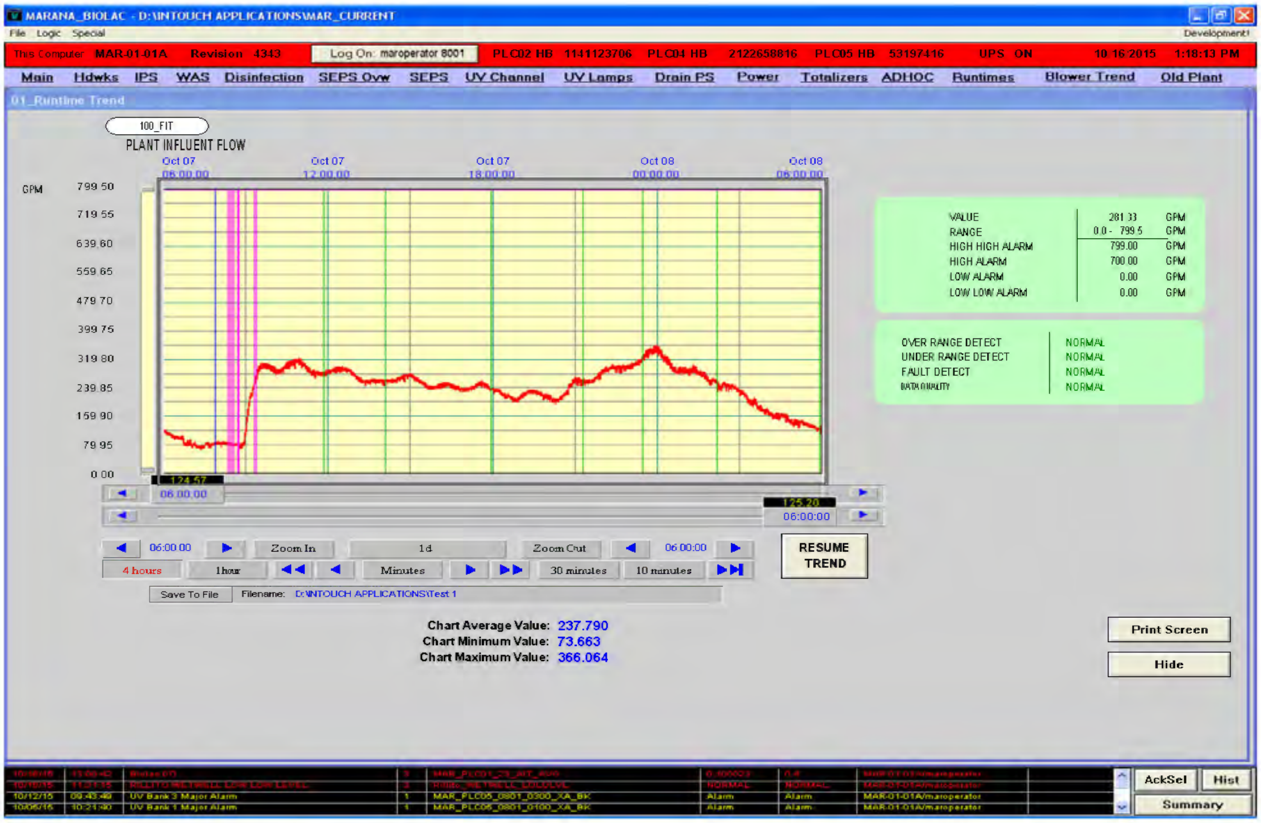

MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLC02 HB 1140946706 PLC04 HB 2122481792 PLC05 HB 53197376 UPS ON
10/16/2015 1:15:16 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
01_Runtime Trend
100_FIT
PLANT INFLUENT FLOW
Oct 12 06:00:00
Oct 12 12:00:00
Oct 12 18:00:00
Oct 13 00:00:00
Oct 13 06:00:00
GPM
799.50
719.55
639.60
559.65
478.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 277.96 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
104.59
06:00:00
104.09
06:00:00
06:00:00
Zoom In
1d
Zoom Out
06:00:00
RESUME TREND
4 hours
1 hour
Minutes
30 minutes
10 minutes
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 253.449
Chart Minimum Value: 61.422
Chart Maximum Value: 414.416
Print Screen
Hide
AckSel
Hist
Summary

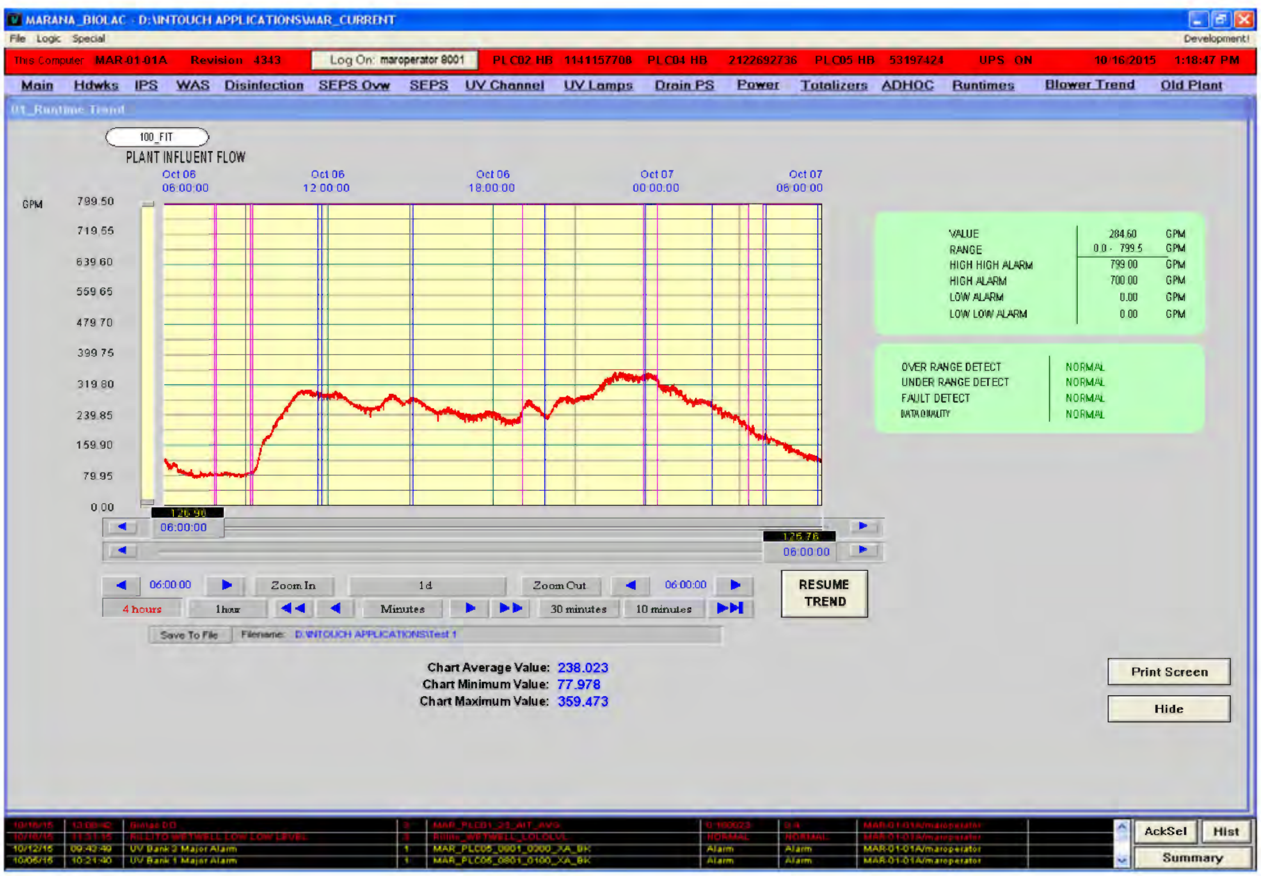

MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLCD2 HB 1140990709 PLCD4 HB 2122525696 PLCD5 HB 53197384 UPS ON 10/16/2015 1:16:00 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
01_Runtime Trend
100_FIT
PLANT INFLUENT FLOW
Oct 11 06:00:00
Oct 11 12:00:00
Oct 11 18:00:00
Oct 12 00:00:00
Oct 12 06:00:00
GPM
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 282.01 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
114.95
06:00:00
108.27
06:00:00
06:00:00
Zoom In
1d
Zoom Out
06:00:00
RESUME TREND
4 hours
1hour
Minutes
30 minutes
10 minutes
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 261.601
Chart Minimum Value: 49.338
Chart Maximum Value: 442.005
Print Screen
Hide
10/12/15 09:43:49 UV Bank 3 Major Alarm 1 MAR_PLC05_0801_0300_XA_BK Alarm Alarm MAR-01-01A/maroperator
10/05/15 10:21:40 UV Bank 1 Major Alarm 1 MAR_PLC05_0801_0100_XA_BK Alarm Alarm MAR-01-01A/maroperator
AckSel
Hist
Summary

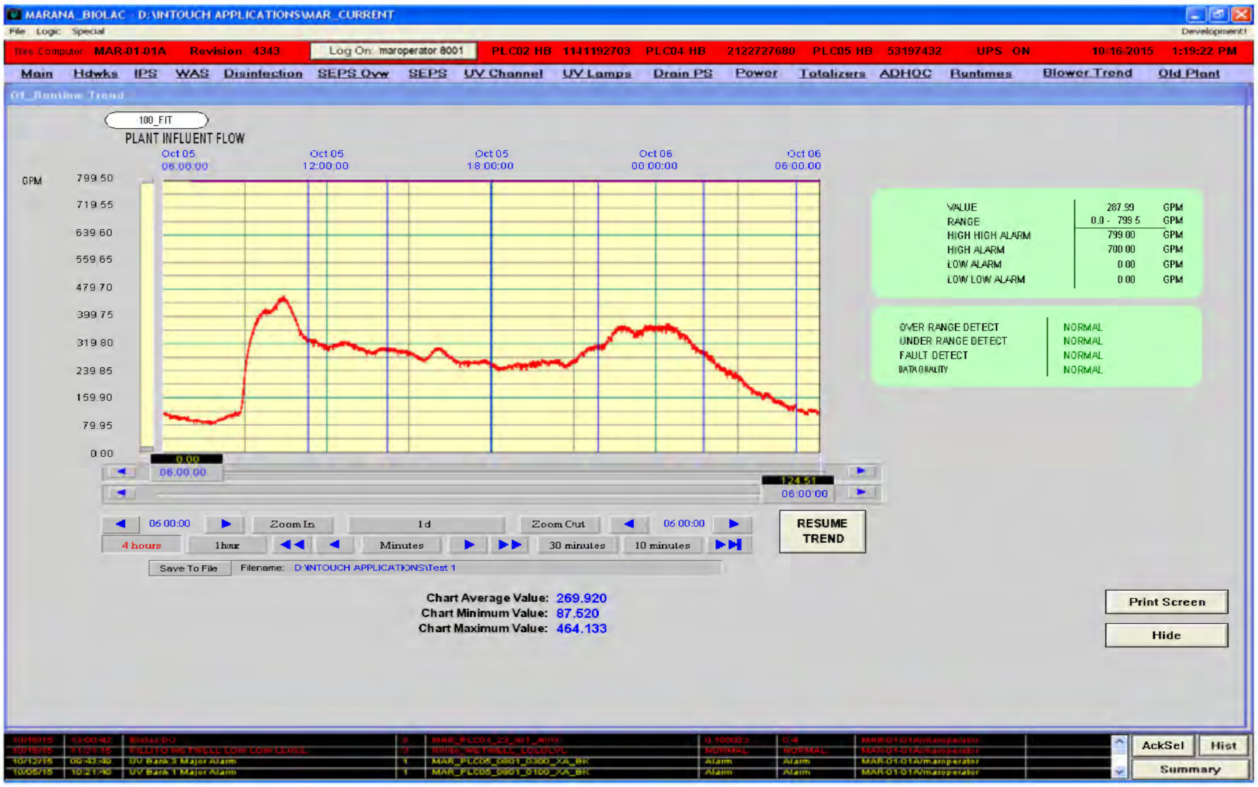

MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLCD2 HB 1141020707 PLCD4 HB 2122554752 PLCD5 HB 53197392 UPS ON 10/16/2015 1:16:29 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
01_Runtime Trend
100_FIT
PLANT INFLUENT FLOW
Oct 10 06:00:00
Oct 10 12:00:00
Oct 10 18:00:00
Oct 11 00:00:00
Oct 11 06:00:00
GPM
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 286.75 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
107.04
06:00:00
114.78
06:00:00
06:00:00
Zoom In
1d
Zoom Out
06:00:00
RESUME TREND
4 hours
1hour
Minutes
30 minutes
10 minutes
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 228.065
Chart Minimum Value: 55.898
Chart Maximum Value: 356.067
Print Screen
Hide
10/12/15 09:43:49 UV Bank 3 Major Alarm 1 MAR_PLC05_0801_0300_XA_BK Alarm Alarm MAR-01-01A/maroperator
10/05/15 10:21:40 UV Bank 1 Major Alarm 1 MAR_PLC05_0801_0100_XA_BK Alarm Alarm MAR-01-01A/maroperator
AckSel
Hist
Summary

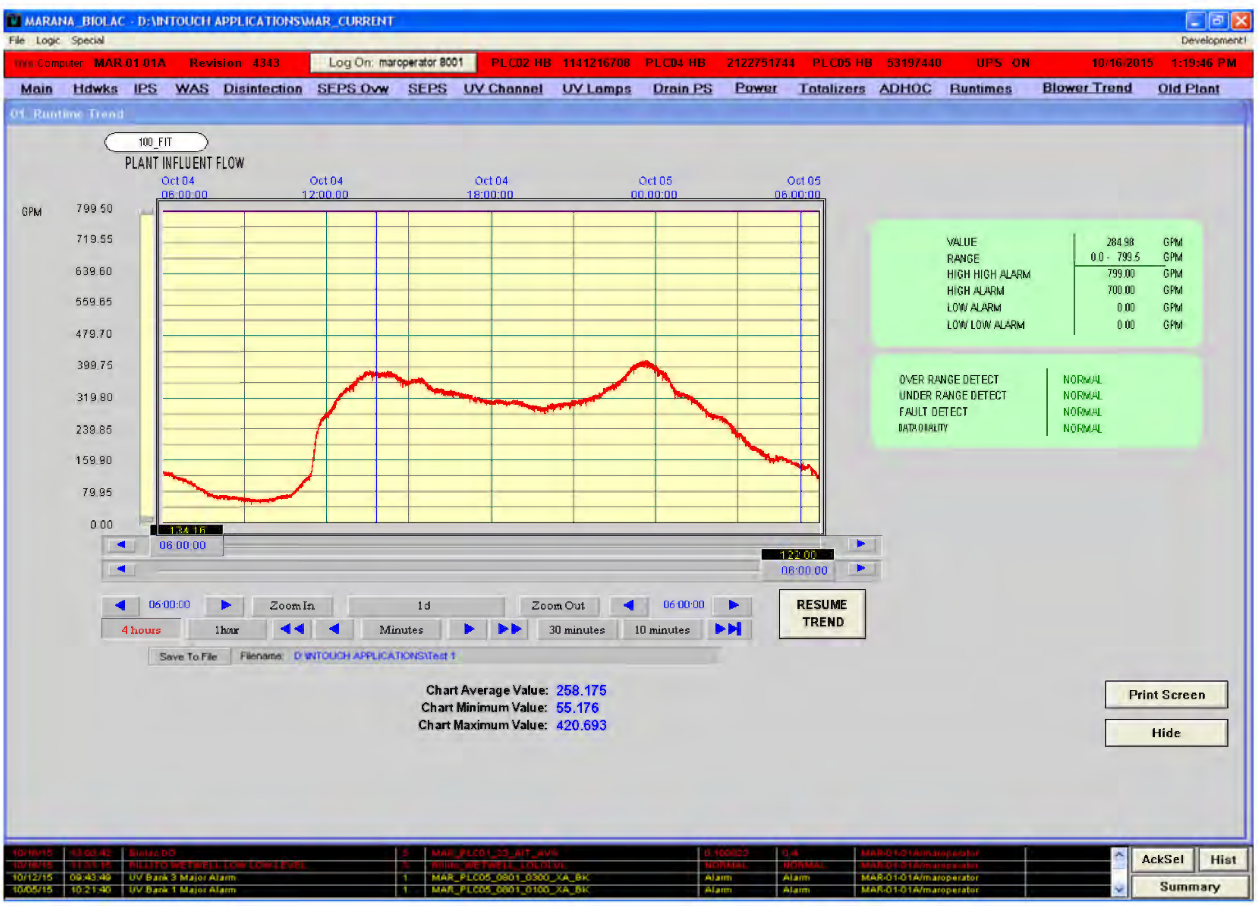

MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
PLANT INFLUENT FLOW
Chart Average Value: 219.106
Chart Minimum Value: 54.187
Chart Maximum Value: 335.838


MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
PLANT INFLUENT FLOW
Chart Average Value: 221.753
Chart Minimum Value: 50.954
Chart Maximum Value: 366.817


MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLC02 HB 1141123706 PLC04 HB 2122658816 PLC05 HB 53197416 UPS ON
10/16/2015 1:18:13 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
100_FIT
PLANT INFLUENT FLOW
Oct 07 06:00:00
Oct 07 12:00:00
Oct 07 18:00:00
Oct 08 00:00:00
Oct 08 06:00:00
GPM
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 281.33 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
06:00:00
06:00:00
06:00:00 Zoom In 1d Zoom Out 06:00:00
RESUME TREND
4 hours 1hour Minutes 30 minutes 10 minutes
Save To File Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 237.790
Chart Minimum Value: 73.663
Chart Maximum Value: 366.064
Print Screen
Hide
AckSel Hist
Summary


MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLC02 HB 1141157708 PLC04 HB 2122692736 PLC05 HB 53197424 UPS ON
10/16/2015 1:18:47 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
100_FIT
PLANT INFLUENT FLOW
Oct 06 06:00:00
Oct 06 12:00:00
Oct 06 18:00:00
Oct 07 00:00:00
Oct 07 06:00:00
GPM
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 284.60 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
06:00:00
06:00:00
06:00:00 Zoom In 1d Zoom Out 06:00:00
RESUME TREND
4 hours 1hour Minutes 30 minutes 10 minutes
Save To File Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 238.023
Chart Minimum Value: 77.978
Chart Maximum Value: 359.473
Print Screen
Hide
AckSel Hist
Summary


MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
PLANT INFLUENT FLOW
Chart Average Value: 269.920
Chart Minimum Value: 87.520
Chart Maximum Value: 464.133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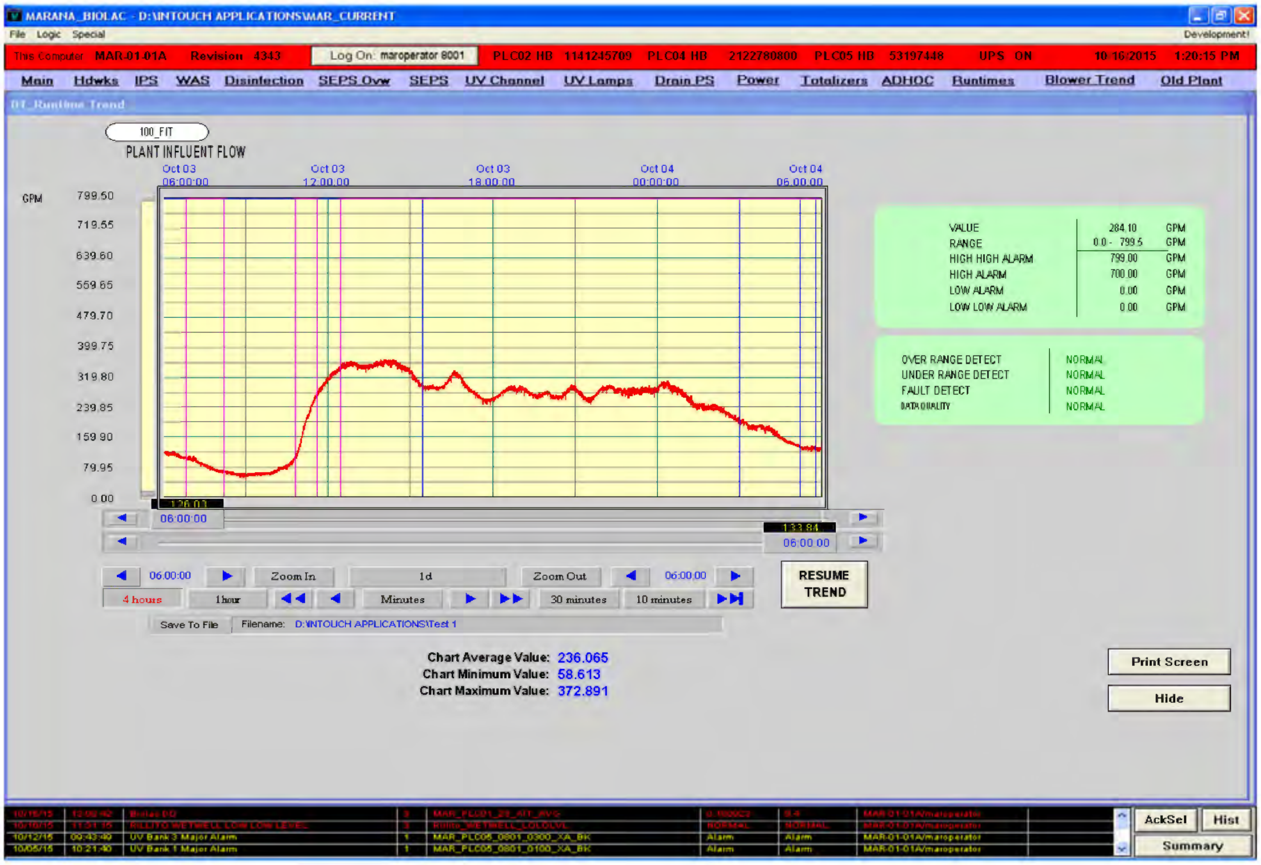
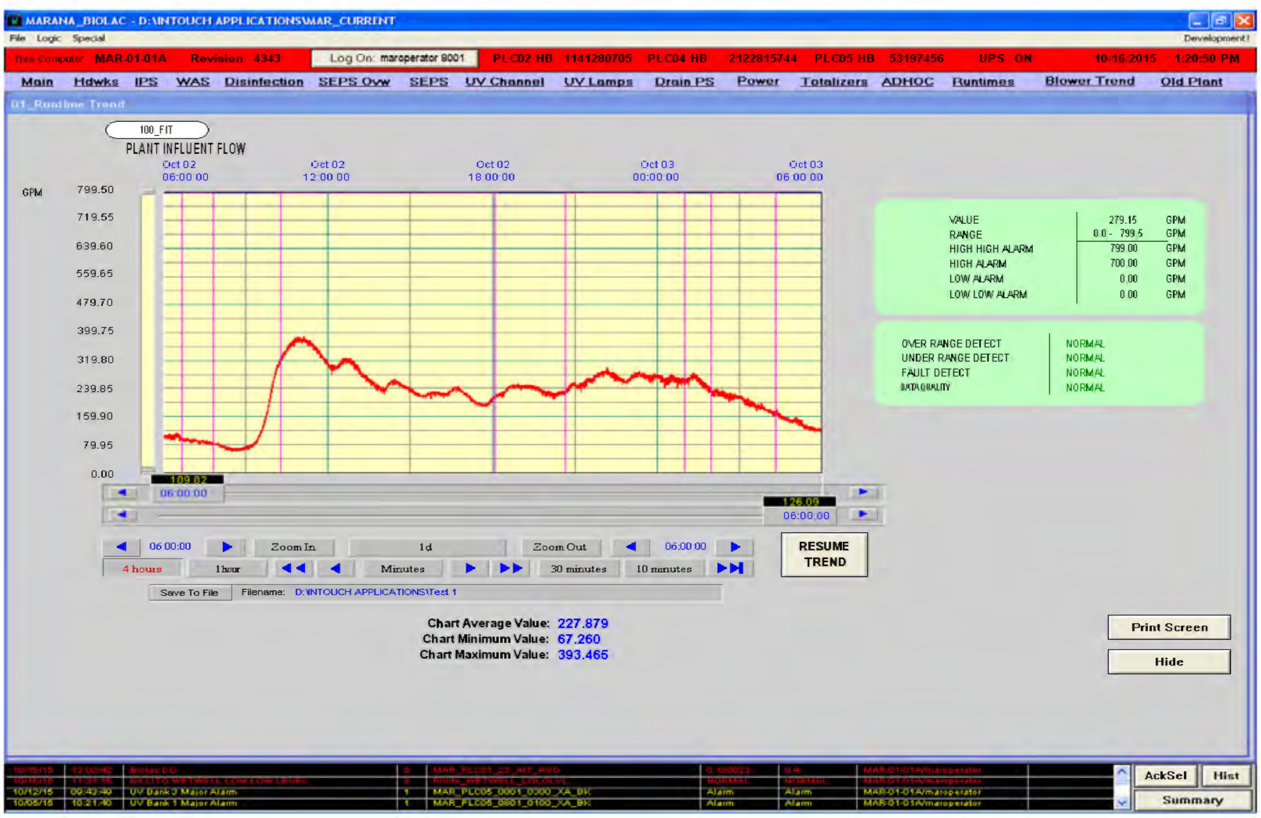
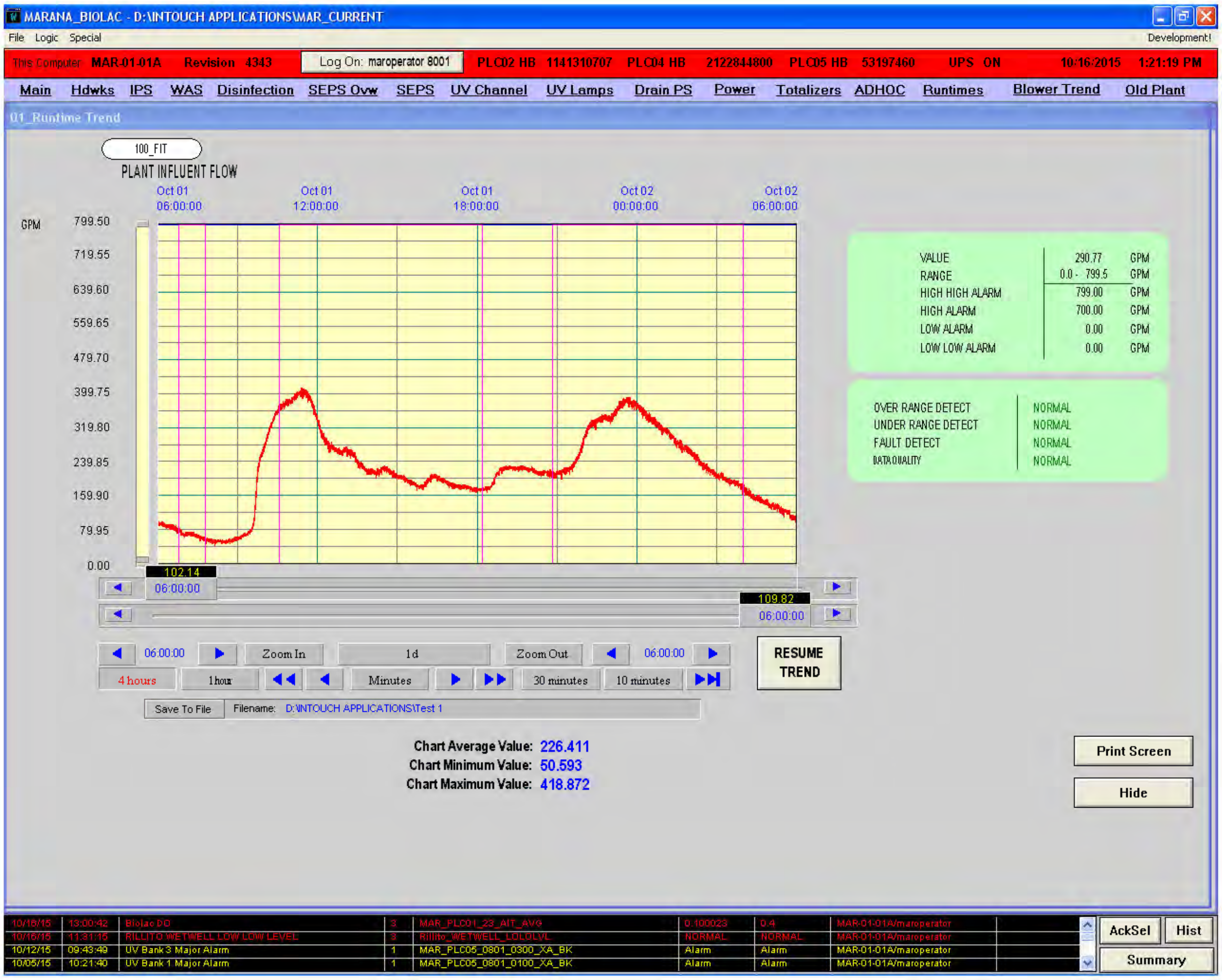
PLANT INFLUENT FLOW
Chart Average Value: 258.175
Chart Minimum Value: 55.176
Chart Maximum Value: 420.693

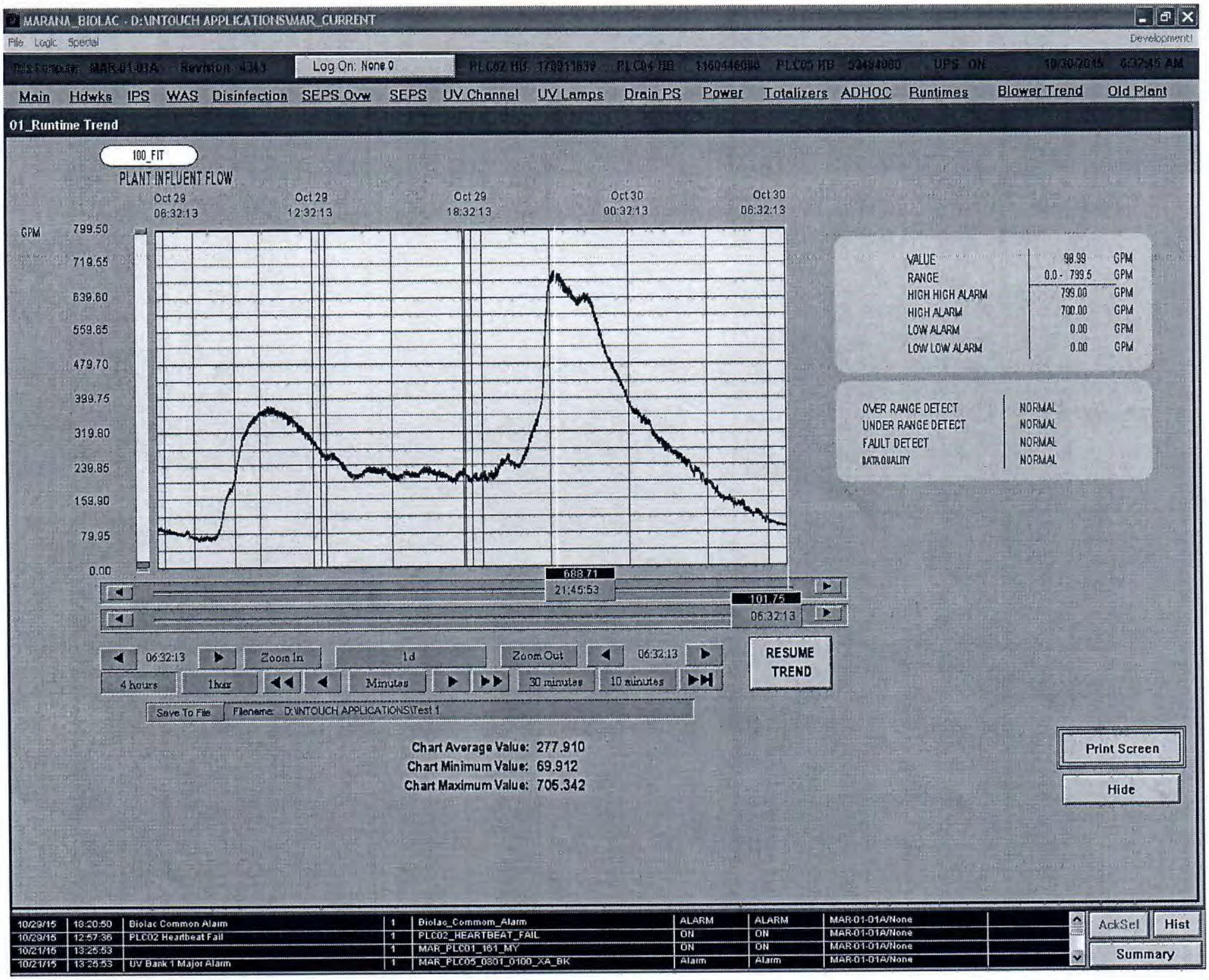

MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
MAR-01-01A
Revision 4343
Log On: maroperator 8001
PLC02 HB 1141245709
PLC04 HB 2122780800
PLC05 HB 53197448
UPS ON
10/16/2015 1:20:15 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
100_FIT
PLANT INFLUENT FLOW
GPM
Oct 03 06:00:00
Oct 03 12:00:00
Oct 03 18:00:00
Oct 04 00:00:00
Oct 04 06:00:00
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 284.10 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
06:00:00
Zoom In
1d
Zoom Out
4 hours
1 hour
Minutes
30 minutes
10 minutes
RESUME TREND
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 236.065
Chart Minimum Value: 58.613
Chart Maximum Value: 372.891
Print Screen
Hide
AckSel
Hist
Summary


MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
MAR-01-01A
Revision 4343
Log On: maroperator 8001
PLC02 HB 1141280705
PLC04 HB 2122815744
PLC05 HB 53197456
UPS ON
10/16/2015 1:20:50 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
100_FIT
PLANT INFLUENT FLOW
GPM
Oct 02 06:00:00
Oct 02 12:00:00
Oct 02 18:00:00
Oct 03 00:00:00
Oct 03 06:00:00
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
VALUE 279.15 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
06:00:00
Zoom In
1d
Zoom Out
4 hours
1 hour
Minutes
30 minutes
10 minutes
RESUME TREND
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 227.879
Chart Minimum Value: 67.260
Chart Maximum Value: 393.465
Print Screen
Hide
AckSel
Hist
Summary


MARANA_BIOLAC - D:\INTOUCH APPLICATIONS\MAR_CURRENT
File Logic Special
Development!
This Computer MAR-01-01A Revision 4343
Log On: maroperator 8001
PLC02 HB 1141310707 PLC04 HB 2122844800 PLC05 HB 53197460 UPS ON 10/16/2015 1:21:19 PM
Main Hdwks IPS WAS Disinfection SEPS Ovw SEPS UV Channel UV Lamps Drain PS Power Totalizers ADHOC Runtimes Blower Trend Old Plant
01_Runtime Trend
100_FIT
PLANT INFLUENT FLOW
Oct 01 06:00:00
Oct 01 12:00:00
Oct 01 18:00:00
Oct 02 00:00:00
Oct 02 06:00:00
GPM
799.50
719.55
639.60
559.65
479.70
399.75
319.80
239.85
159.90
79.95
0.00
102.14
06:00:00
109.82
06:00:00
VALUE 290.77 GPM
RANGE 0.0 - 799.5 GPM
HIGH HIGH ALARM 799.00 GPM
HIGH ALARM 700.00 GPM
LOW ALARM 0.00 GPM
LOW LOW ALARM 0.00 GPM
OVER RANGE DETECT NORMAL
UNDER RANGE DETECT NORMAL
FAULT DETECT NORMAL
DATA QUALITY NORMAL
06:00:00
Zoom In
1d
Zoom Out
06:00:00
RESUME TREND
4 hours
1 hour
Minutes
30 minutes
10 minutes
Save To File
Filename: D:\INTOUCH APPLICATIONS\Test 1
Chart Average Value: 226.411
Chart Minimum Value: 50.593
Chart Maximum Value: 418.872
Print Screen
Hide
10/16/15 13:00:42 Biolac DO 3 MAR_PLC01_23_AIT_AVG 0.100023 0.4 MAR-01-01A/maroperator
10/16/15 11:31:15 RILLITO WETWELL LOW LOW LEVEL 3 Rillito_WETWELL_LOLOLVL NORMAL NORMAL MAR-01-01A/maroperator
10/12/15 09:43:49 UV Bank 3 Major Alarm 1 MAR_PLC05_0801_0300_XA_BK Alarm Alarm MAR-01-01A/maroperator
10/05/15 10:21:40 UV Bank 1 Major Alarm 1 MAR_PLC05_0801_0100_XA_BK Alarm Alarm MAR-01-01A/maroperator
AckSel
Hist
Summary

APPENDIX D

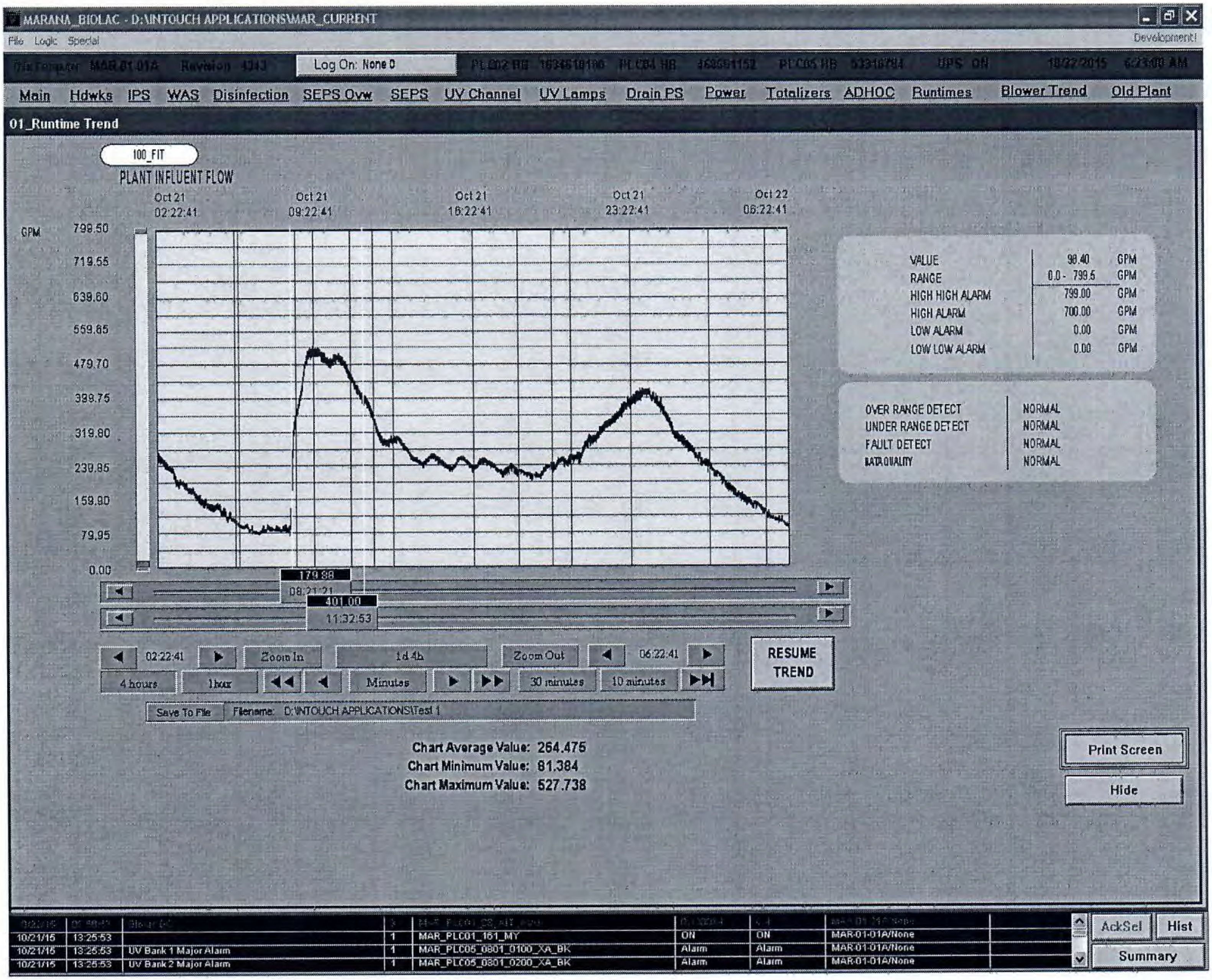
CONTINUOUS FLOW GRAPHS FROM RAINFALL EVENT PEAKS



Rain Event 10/29/15

Start @ 2115
Stop @ 0030
3hrs 15min

flow peak @ 705 gpm
estimated 58,500 gallons of Inflow
overflow to EOB 2315-2350



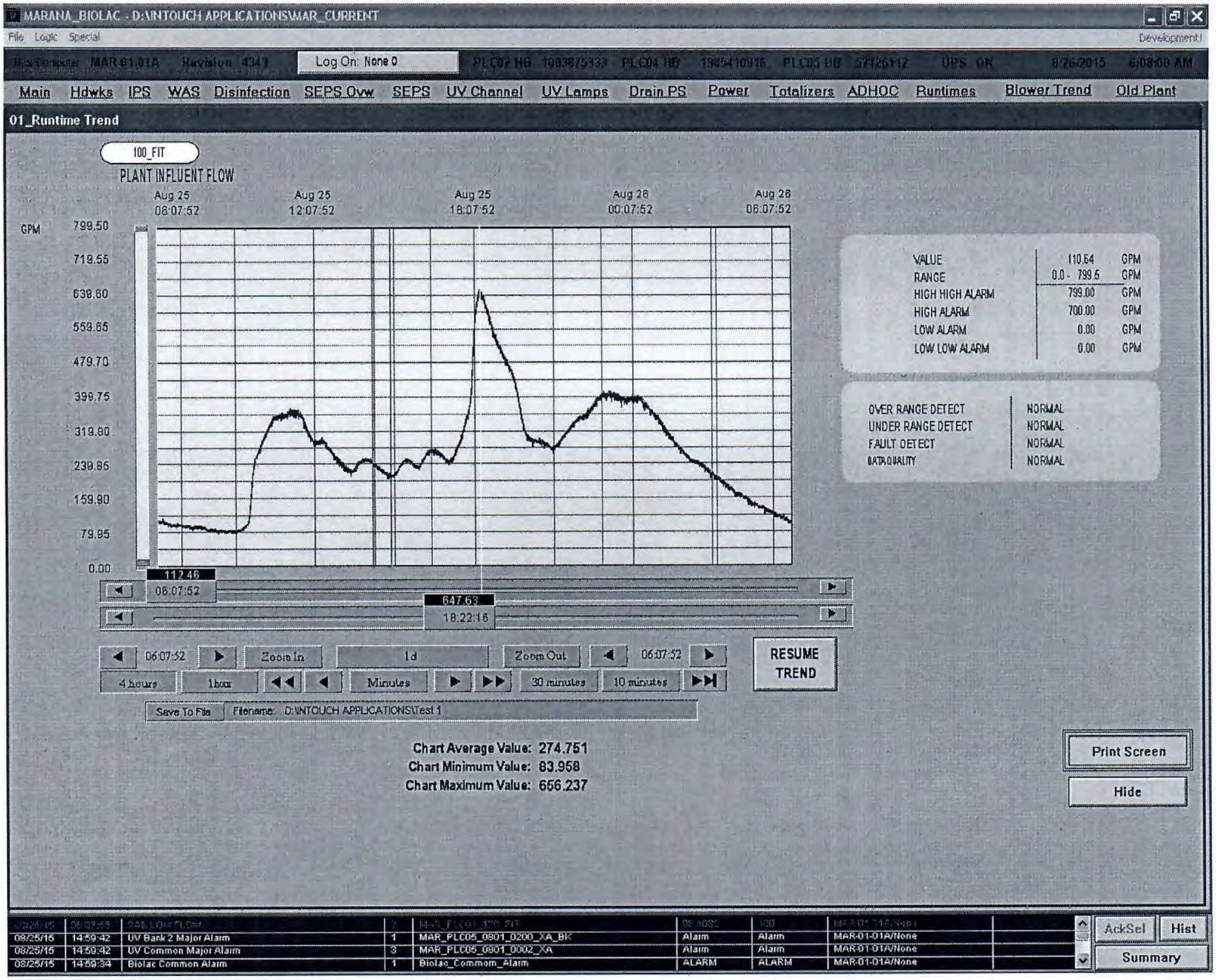
Rain Event

10/21/15

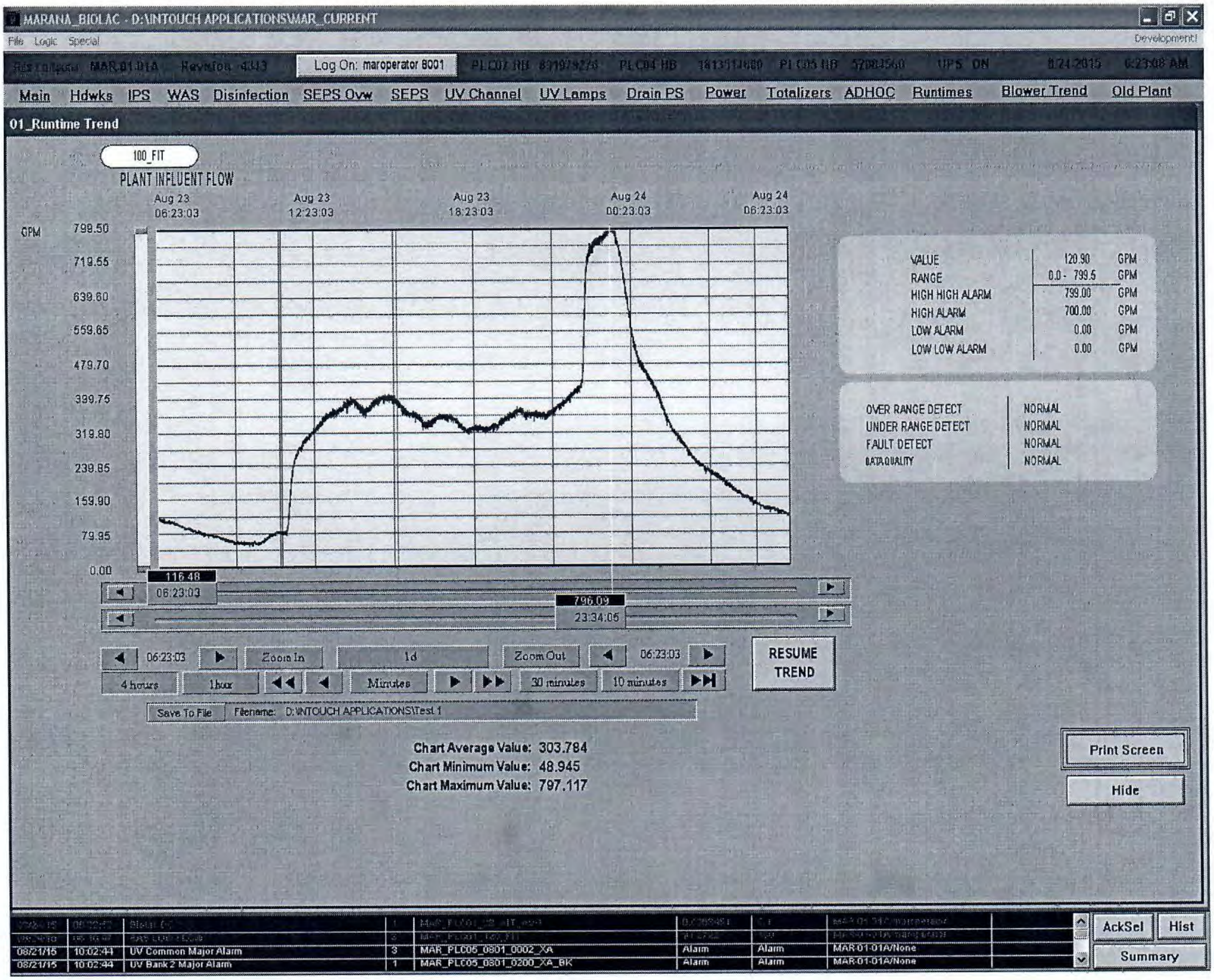
0815 - 1200

127gpm above Average

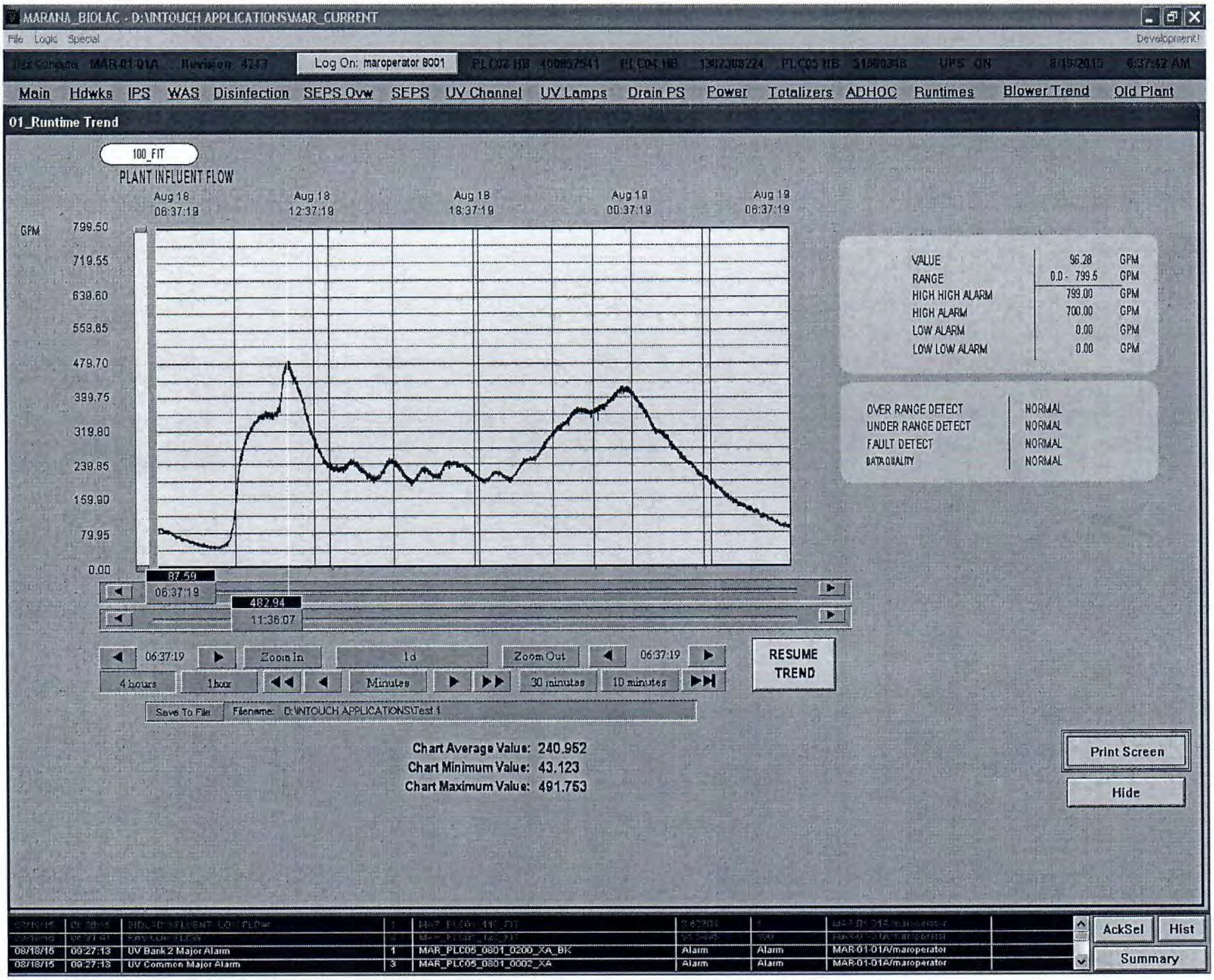
est 30,500 gallons of inflow



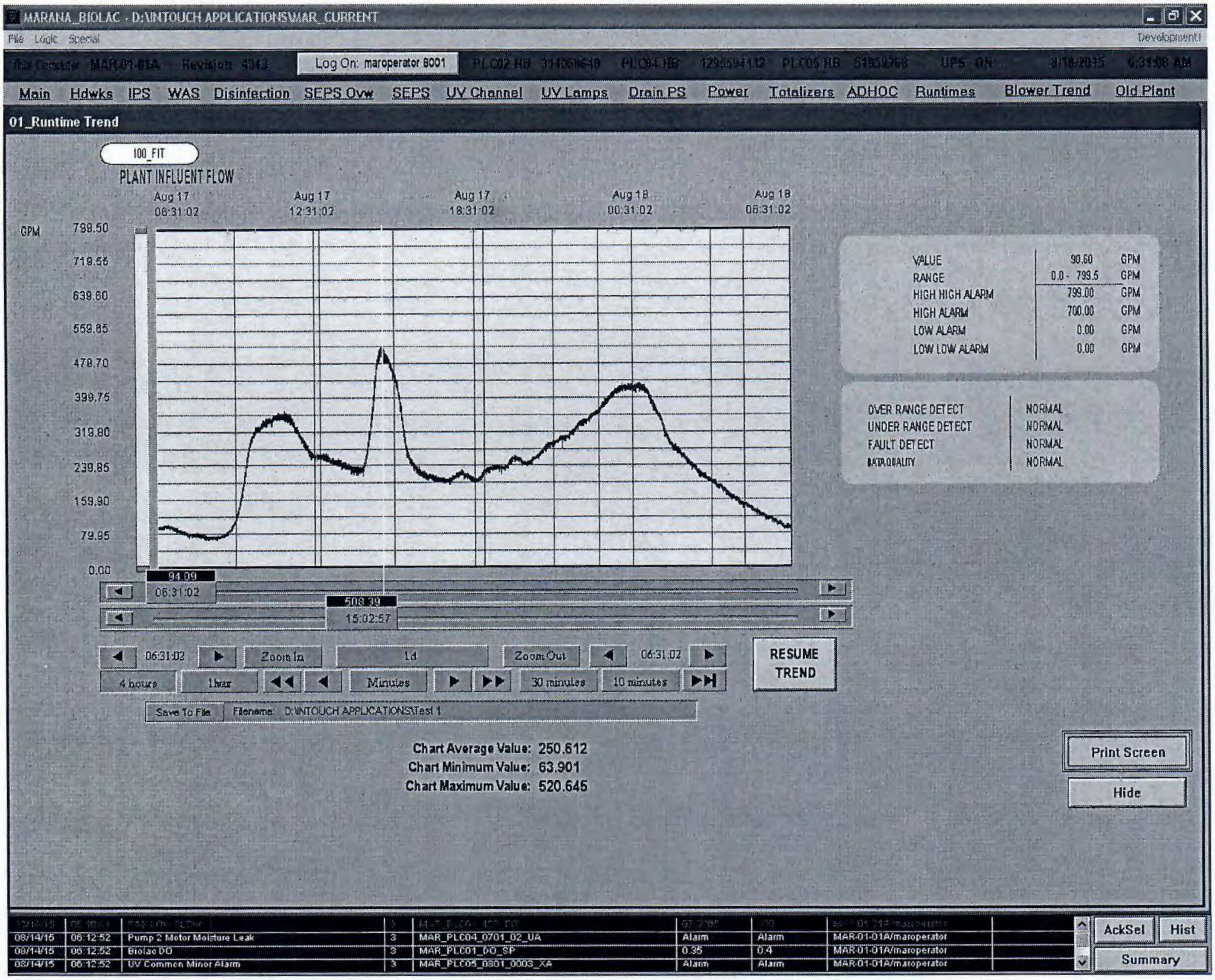
Rain Event 8/25/15



Rain Event 8/23-24/15



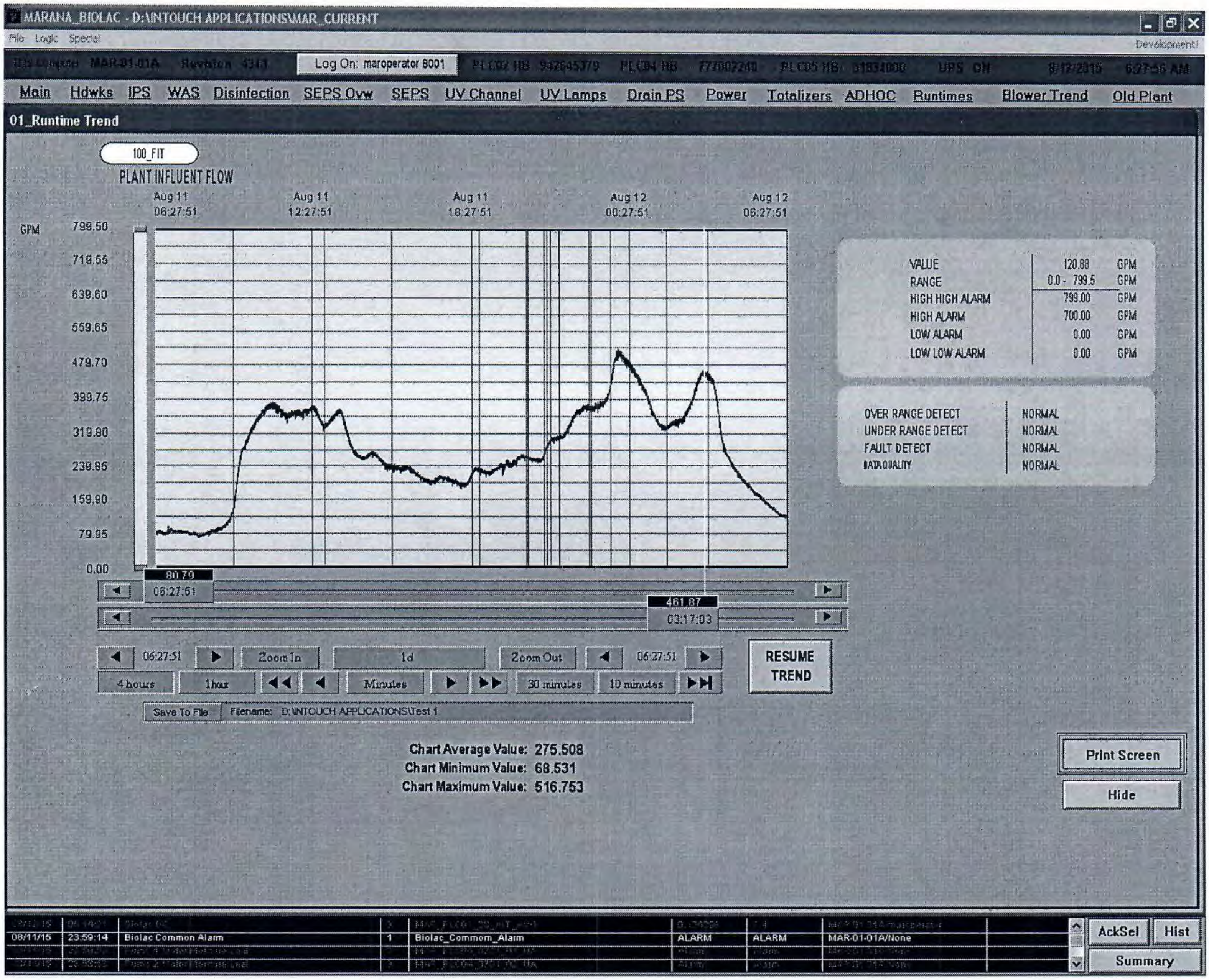
INF Flows from 0637 of 8.18.15 to 0637 of 8.19.15



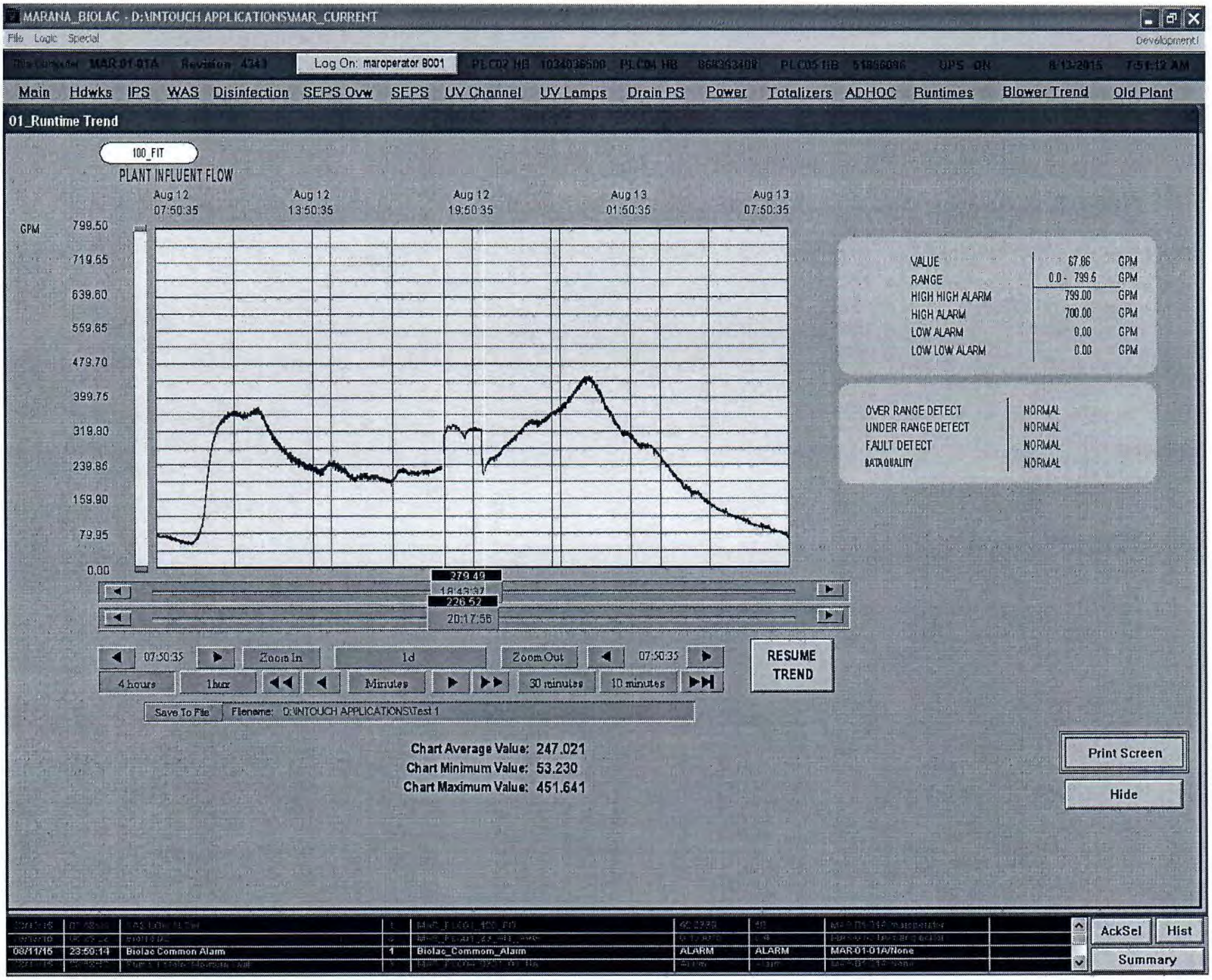
Influent Spike 8/17/15

1420 - 1615

peak @ 1502 @ 510gpm



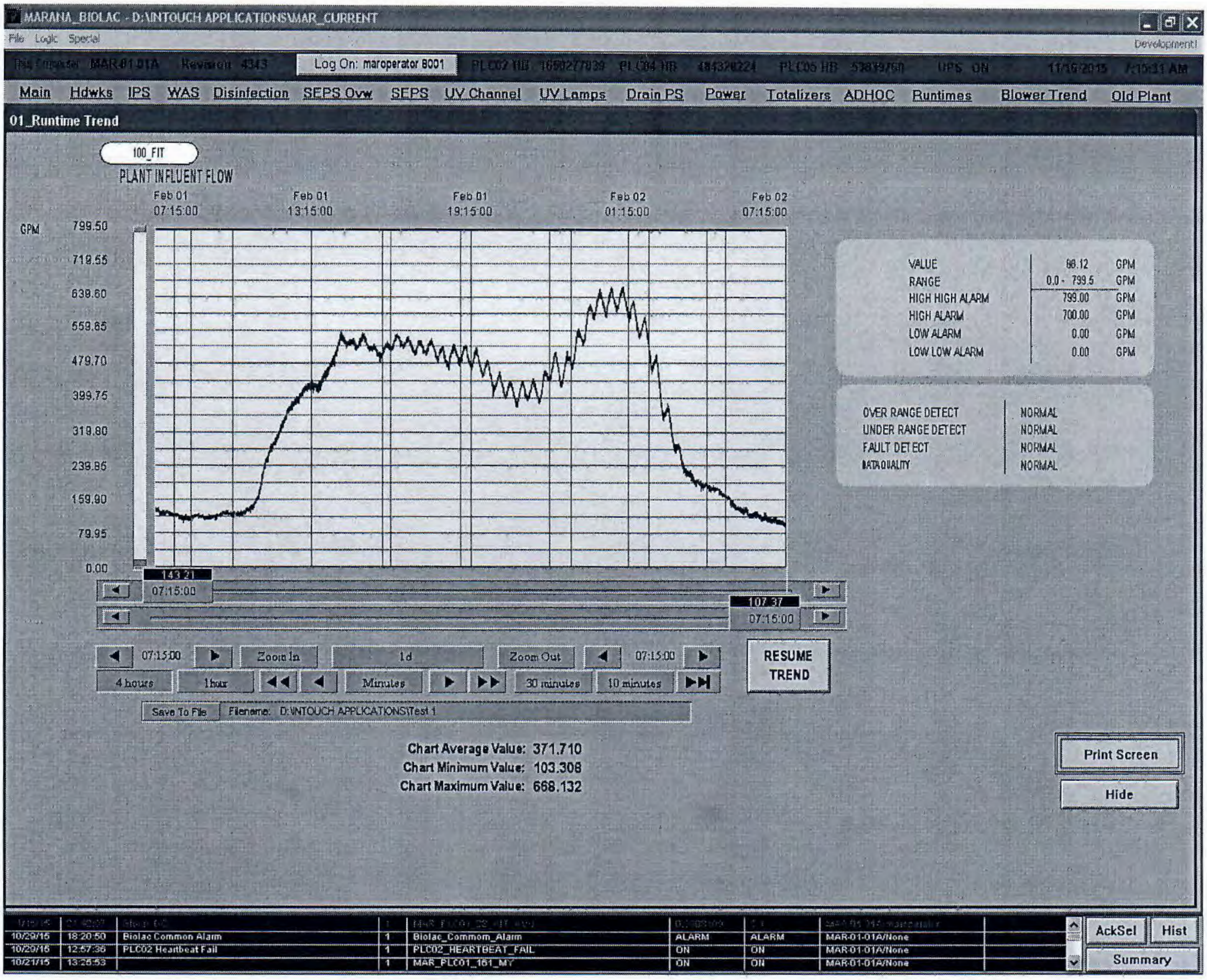
Rain Inflow 8/11-12/15



Object stuck in Plum?

Dump?

Time
8/12/15 1843 - 2017



Rain Event Recorded in log Book

APPENDIX B – TM-2 EXISTING FACILITIES EVALUATION

Registered Professional Engineer (Civil)
CERTIFICATE NO.
41462
ANDREW C.
GILMORE
DATE SIGNED 05/24/2016
ARIZONA, U.S.A.

EXPIRES 09-30-2016

Registered Professional Engineer (Civil)
CERTIFICATE NO.
37702
DANA R.
TROMPKE
DATE SIGNED 05/24/2016
ARIZONA, U.S.A.

EXPIRES 06-30-2017

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

TECHNICAL MEMORANDUM NO. 2
EXISTING FACILITIES EVALUATION

FINAL-FINAL
May 2016

4600 EAST WASHINGTON STREET, SUITE 500 • PHOENIX, ARIZONA 85034 • P. 602.263.9500 • F. 602.265.1422

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

TECHNICAL MEMORANDUM
NO. 2

EXISTING FACILITIES EVALUATION

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A.A.C.	Arizona Administrative Code
AADF	average annual daily flow
AADL	annual average day load
ADEQ	Arizona Department of Environmental Quality
ADF	average daily flow
APP	Aquifer Protection Permit
ATS	auto transfer switch
AZPDES	Arizona Pollution Discharge Elimination System
BNR	biological nutrient removal
BOD	biochemical oxygen demand
cfm	cubic feet per minute
CFR	Code of Federal Regulations
CSF	clarifier safety factor
DO	dissolved oxygen
ea	each
EOB	emergency overflow basin
EPA	Environmental Protection Agency
FRP	fiberglass reinforced plastic
ft	foot/feet
ft^3/MG	cubic feet of screenings per million gallons of wastewater
gal	gallons
gpd	gallons per day
gpd/sq ft	gallons per day per square foot
gpm	gallons per minute
HDPE	high-density polyethylene
hp	horsepower
in	inch/inches
IPS	influent pump station
lb/d	pounds per day
lbs/ft^2 day	pounds per square foot per day
MG	million gallons
mg/L	milligrams per liter
mgd	millions gallons per day
mJ/cm^2	millijoules per centimeter squared
mL/g	milliliter per gram
MLSS	mixed liquor suspended solids
mm	millimeter
MMADF	maximum month average day flow
MMADL	maximum month average day load
NFPA	National Fire Protection Agency
NH_3-N	ammonia nitrogen
NO_2-N	nitrite
NO_3-N	nitrate
NTU	nephelometric turbidity units
OSHA	Occupational Safety and Health Administration
PCRWRD	Pima County Regional Water Reclamation Department
PDF	peak day flow
PF	peaking factor
PHF	peak hour flow

PLC	programmable logic controller
PPE	personnel protective equipment
psig	pounds per square inch gauge
PWWF	Peak Wet Weather Flow
RAS	returned activated sludge
SEPS	Secondary Effluent Pump Station
SES	service entrance switchboard
SOR	surface overflow rate
SRT	solids retention time
SVI	sludge volume index
TDH	total dynamic head
TKN	total Kjeldahl nitrogen
TM	technical memorandum
TN	total nitrogen
Town	Town of Marana
TSS	total suspended solids
UV	ultraviolet
VFD	variable frequency drive
VOCs	volatile organic compounds
WAS	waste activated sludge
WRF	water reclamation facility
WWTP	wastewater treatment plant

EXISTING FACILITIES EVALUATION

1.0 INTRODUCTION

The Town of Marana (Town) owns and operates the Marana Water Reclamation Facility (WRF), which has an operating capacity of 500,000 gallons per day (gpd). At this facility, wastewater receives primary, secondary, and tertiary treatment.

For secondary treatment, the WRF uses a Biolac® treatment system. A separate treatment system consisting of four package biological nutrient removal (BNR) package plants with a combined capacity of 0.2 million gallons per day (mgd) is also located onsite but is not in use. The tertiary treatment system, installed in 2008, consists of sand filtration and ultraviolet (UV) disinfection system and is designed to handle up to 3.5 mgd.

Solids are hauled off site daily and represent the second highest operational cost of the WRF. Effluent is discharged to a tributary of the Santa Cruz River; however, a recharge facility adjacent to the Marana WRF has been designed and will soon be constructed. Once in operation, this facility will recharge all the tertiary effluent produced at the WRF, so that reclaimed water storage credits may be accrued.

Figure 2.1 and Figure 2.2 present process flow diagrams of the liquids and solids flows, respectively.

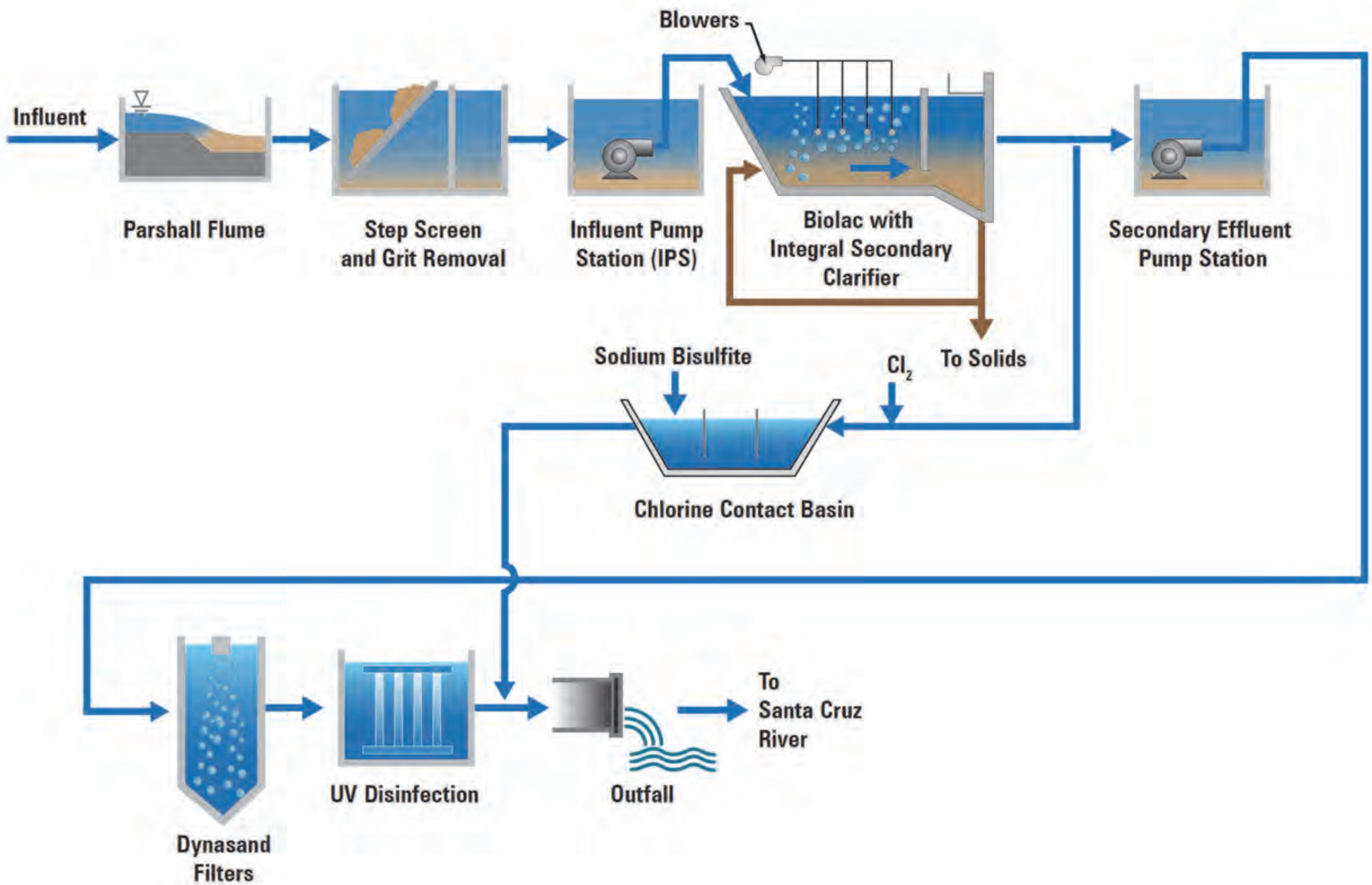
The Marana WRF now operates at an average daily flow rate of approximately 355,000 gpd, which is 71 percent of the secondary treatment system's capacity. The Town projects that within 10 years the Marana WRF will need a capacity of 1.0 to 1.5 mgd.

Prior to starting a plant expansion, the Town desires to complete a Master Plan evaluation of the WRF in order to lay out a methodical plan for future phased expansions to meet the needed capacity and to evaluate the most appropriate treatment process to meet the Town's goals.

1.1 Purpose of Technical Memorandum

This technical memorandum (TM) documents the following existing conditions:

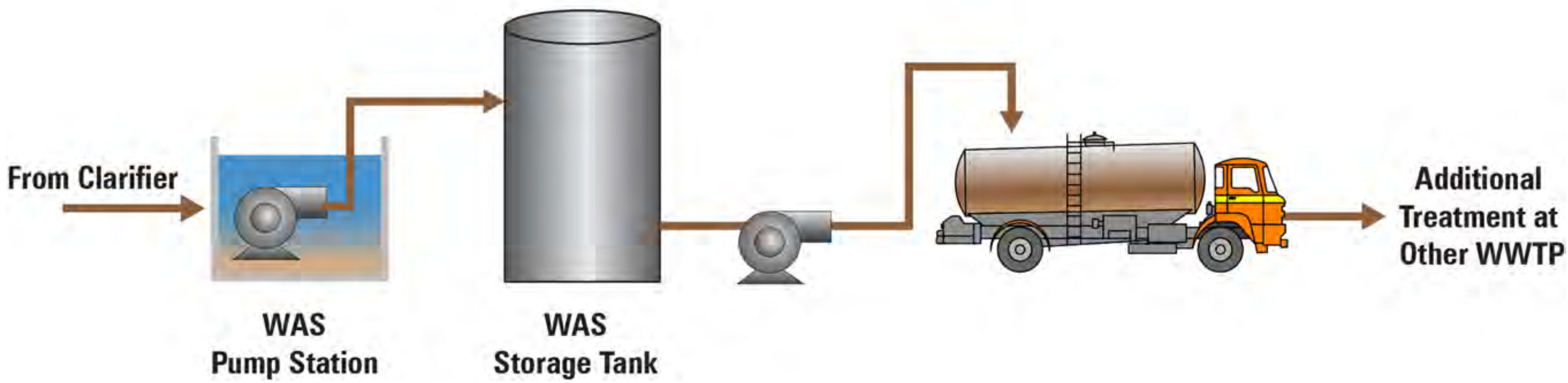
- Treatment plant's processes.
- Capacity of processes.
- Physical conditions of the facilities.



LIQUIDS PROCESS FLOW DIAGRAM

FIGURE 2.1

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN



SOLIDS PROCESS FLOW DIAGRAM

FIGURE 2.2

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

carollo

pw:\\Carollo/Documents\Client/AZ/Marana/10067A00/Deliverables/TM02\Figure 2-2

Specifically, the following tasks were included in this evaluation:

- A one-day plant site visit was conducted in which engineers of various disciplines observed the electrical, mechanical, and structural condition of the existing facilities. They noted deficiencies and evaluated the potential of existing facilities to be used in future phases.
- The existing filter operation was reviewed to determine the volume of backwash and its impact on plant systems.
- The existing hydraulic profile was modeled and evaluated to determine the current and future locations of flow restrictions, if any.
- Computer simulation modeling of the liquid process stream was performed with BioWin software, using plant operating records for calibration.
- A solids balance was performed based on the results of the calibrated modeling effort. The total solids produced at the WRF (dry tons per day) for screenings and waste solids were estimated.
- A review of the 100-year flood elevation and existing mitigation measures was performed.
- An energy assessment was conducted using the current Environmental Protection Agency (EPA) guideline and reference tools.
- The emergency backup system was evaluated for capacity under current plant electrical loads.

1.2 Current Operating Permits

The requirements of current major regulatory and operating permits for the WRF are summarized below.

1.2.1 AZPDES Permit

The WRF discharges treated effluent to an unnamed wash flowing to the Santa Cruz River. These discharges are under legal authority of the current Arizona Pollution Discharge Elimination System (AZPDES) Permit No. AZ0024520. The permit was issued on April 13, 2012, and runs through April 12, 2017. After that date, the permit must be renewed. The permit was last modified to allow a discharge flow up to 3.5 mgd.

According to this permit, effluent must be monitored and reported for the following:

- Flow.
- Biochemical oxygen demand (BOD).
- Total suspended solids.

- *E. coli.*
- Total chlorine residual.
- Cyanide.
- pH.
- Trace substances, including ammonia, temperature, sulfides, mercury, oil, and grease.
- Whole effluent toxicity, which determines the toxicity of effluent on sample selected organisms, specifically green algae, fathead minnow, and water flea.
- The character of the effluent, including its general chemistry, microbiology, metals, and volatile organic compounds, which include several of the constituents listed above as well as total Kjeldahl nitrogen, nitrate/nitrite, dissolved oxygen, and total dissolved solids.

Other conditions of the permit are as follows:

- The discharge shall not cause an increase in the ambient water temperature of more than 3.0 degrees Celsius.
- Unless the percent saturation of oxygen remains equal to or greater than 90 percent, the discharge shall not cause the dissolved oxygen concentration in the receiving water to fall below:
  - 3 milligrams per liter (mg/L) from 3 hours after sunrise to sunset.
  - 1 mg/L from sunset to 3 hours after sunrise.

1.2.1.1 Biosolids

The AZPDES permit also governs the ultimate disposal of biosolids (called non-hazardous sewage sludge as defined in 40 CFR 503.9). This permit states that all sewage sludge generated at the facility shall be stored, dewatered, and hauled off site for disposal at a State approved facility.

Currently, waste activated sludge is settled and decanted to reduce the volume and is then hauled approximately 54 miles to the Casa Grande WRF for further treatment and disposal. The upcoming plant expansion will modify and improve the solids handling process with the goal of disposing dewatered solids at the Marana Regional Landfill 11 miles away. At that time, the AZPDES permit should be modified to reflect the changed operations and disposal location for biosolids.

1.2.2 Aquifer Protection Permit

The WRF produces effluent classified as Class B+ Reclaimed Water as defined in the Arizona Administrative Code (A.A.C.) R18-11-305 and R18-9-206. The Aquifer Protection Permit (APP) No. P-100631 regulates the effluent quality.

The operators regularly monitor the treated and disinfected effluent downstream of the UV disinfection unit and report the results to the Arizona Department of Environmental Quality (ADEQ). The effluent is monitored for the following:

- Daily Flow and Average Monthly Flow for discharge to the Santa Cruz outfall and on-site reuse.
- *E. coli.*
- Total nitrogen (5 sample rolling geometric mean).
- Metals, such as lead and mercury.
- Volatile organic compounds, including total trihalomethanes.

In addition, groundwater quality must be monitored for parameters of concern including:

- Total nitrogen, total Kjeldahl nitrogen, nitrate, and nitrite.
- Total coliform.
- Metals, similar to the effluent monitoring.
- Volatile organic compounds (VOCs), also similar to the effluent monitoring.

The current APP approves multiple phases of the WRF, as described below:

- **Existing WRF:** Permits flow up to 0.7 mgd using the Biolac® treatment system, biological nutrient removal package plants, filtration, UV disinfection, and back-up chlorination and dechlorination.
- **Phase 1:** Permits flows up to 2.0 mgd and includes the addition of:
  - A 1.5 mgd oxidation ditch with clarifiers,
  - New Headworks,
  - New Influent Pump Station, and
  - New solids thickening and storage facilities.
- **Phase 2:** Permits flows up to 3.5 mgd, and includes an additional train of oxidation ditch and clarifier(s). Filters and UV disinfection currently installed at the existing WRF are sized for this Phase 2 flow rate.

The Town has completed the design of recharge basins to be constructed on the property immediately east of the WRF. The APP permit has been revised to allow the treated effluent to be recharged at the new recharge facilities. Once operational, the new recharge basins will be the Town's preferred disposal method of the Class B+ reclaimed water.

If the recharge basins are unavailable for any reason, the next preferred option is to use a reclaimed water spraying field. Although the Town will continue to maintain the AZPDES outfall to the Santa Cruz River, it is the least preferred method of disposal.

As part of the recharge basin project and APP permit expansion, additional monitoring wells have been constructed to monitor the groundwater hydraulically upstream and downstream of the recharge basins.

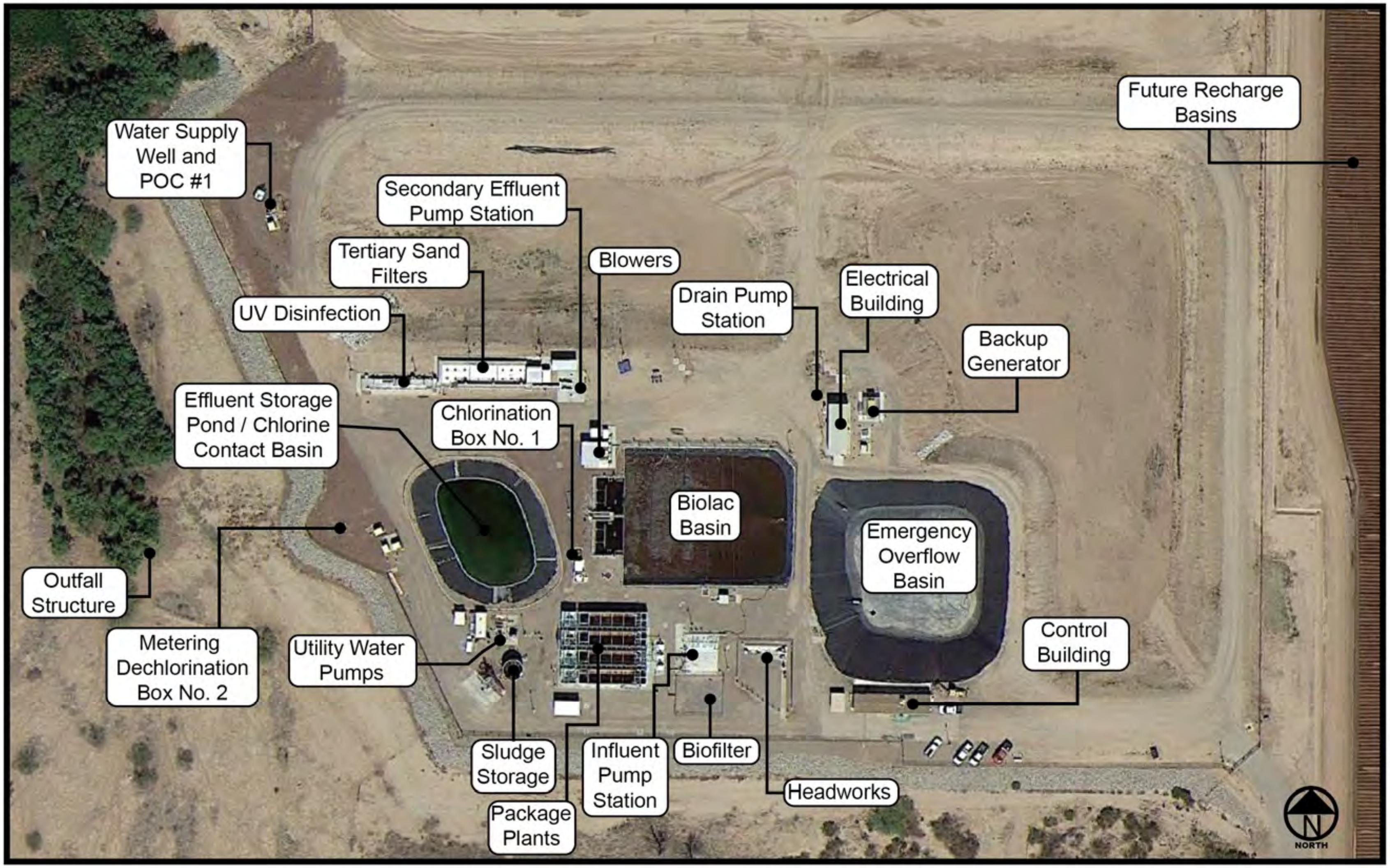
2.0 EXISTING FACILITIES ASSESSMENT

The Marana WRF consists of the following facilities:

- Preliminary treatment (Headworks).
- Influent pumping.
- Secondary treatment.
- Secondary effluent pumping.
- Filters.
- UV disinfection.
- Plant effluent outfall structure.

The WRF also includes backup systems for chlorination/dechlorination and auxiliary systems for odor control, utility water, and standby power generation. The assessment includes equipment sizes and capacities verified during the October 29, 2015, site visit or as shown in the Engineering Report for the Marana Wastewater Treatment Plant (WWTP) Facility Upgrade and Expansion, which was prepared for Pima County Regional Water Reclamation Department (PCRWRD) by Stantec Consulting, Inc. in April of 2009.

Figure 2.3 shows a site plan for the Marana WRF.



MARANA WRF SITE PLAN

FIGURE 2.3

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

carollo

2.1 Preliminary Treatment (Headworks)

During preliminary treatment, the influent wastewater is screened to remove rags and debris. The screened wastewater is then pumped to the secondary treatment process.

The existing Headworks structure is composed of influent flow monitoring, mechanical bar screen, grit removal chamber, and influent sampling. It receives wastewater from a 12-inch diameter sewer connected to Gladden MH-2 just outside the east property line. Manhole MH-2 is fed by a 12-inch diameter sewer pipe from Gladden MH-1, where the 24-inch Gladden interceptor ends.

At the Headworks, a 3-inch Parshall flume monitors influent flow. When properly calibrated, the Parshall flume measures flows ranging from 15 to 830 gallons per minute (gpm) (0.22 to 1.2 mgd).

Downstream from the Parshall flume is an in-channel fine screen unit with manual bypass screen installed in parallel channels. The fine screen is a Comarco 12-inch self-cleaning 3-millimeter (mm) mechanical bar screen constructed of stainless steel. Screenings are conveyed to a washer/compactor and then discharged into a custom screenings bin. The screenings dumpster is emptied twice per week.

A third channel houses a Lakeside Microstrainer (12-inch self-cleaning 2-mm fine screen) that serves as a back-up to the Comarco mechanical bar screen.

Downstream from the mechanical bar screen are two parallel gravity grit removal channels. Adjustable weirs located both upstream and downstream control the flow to each channel. Grit is removed manually from each channel twice per week. A composite auto-sampler collects samples downstream of the grit removal channels, to be analyzed for process control.

Table 2.1 summarizes the existing preliminary treatment facilities.

Table 2.1 Preliminary Treatment Facilities
Marana Water Reclamation Facility Master Plan
Town of Marana

Description	Unit	Value
Flow Metering		
Type of Metering Equipment	--	Parshall Flume
Number of Units	ea	1
Size	in	3.0
Capacity (1)	mgd	1.2

Table 2.1 Preliminary Treatment Facilities Marana Water Reclamation Facility Master Plan Town of Marana		
Description	**Unit**	**Value**
Screening Equipment		
Total Number of Screens	ea	3
Screen No. 1		
Type	--	Manual Bypass Screen
Number of Units	ea	1
Capacity (2)	mgd	1.1
Bar Spacing	mm	Unknown
Screen No. 2		
Type	--	Mechanical Bar Screen
Number of Units	ea	1
Capacity (2)	mgd	1.0
Bar Spacing	mm	3.0
Bar Screen Horsepower, Each	hp	3
Model	--	MS14-500-3
Manufacturer	--	Comarco
Washer Compactor Horsepower, Each	hp	3
Model	--	MSWP15-150
Manufacturer	--	Comarco
Screen No. 3		
Type	--	Mechanical Bar Screen
Number of Units	ea	1
Capacity	mgd	0.68
Bar Spacing	mm	3
Bar Screen Horsepower, Each	hp	2
Model	--	Microstrainer
Manufacturer	--	Lakeside
Grit Removal		
Type of Grit Handling	--	Gravity
Number of Channels	ea	2
Channel Width	ft	1 foot-0 inch
Channel Length	ft	30 feet-0 inch

Notes:
(1) From Stantec Facility Upgrade and Expansion Engineering Report.
(2) From Stantec Interim Plant Upgrade Drawings, Project No. C-343.

2.1.1 Conditions Assessment

The existing Headworks structure was originally constructed in 2006. The current Comarco mechanical bar screen was installed in 2013. Both it and the manual bypass screen are constructed of stainless steel and appear to be in good operating condition. Although the Lakeside Microstrainer with concentric conveyor/dewatering screw is not regularly operated, the Town has noted that it is also in good operating condition.

At the hydraulic entry to the Headworks, concrete corrosion was observed, likely from hydrogen sulfide release where the influent is agitated. Although the damage looks relatively limited, the area should be repaired with a chemical resistant repair mortar. To stop the corrosion, the covers in that area could be replaced with grating or the air space ventilated under a slight negative pressure to an odor control system.

2.1.2 Potential Use for Future Phases

A hydraulic model was completed to evaluate the hydraulic capacity of the existing Headworks. The capacity of the 12-inch diameter sewer piping feeding the existing Headworks is sufficient to handle flows up to 1.4 mgd based on the slope of construction. For flows exceeding 1.4 mgd, the piping should be replaced with a larger pipe sized to meet the capacity of the 24-inch Gladden interceptor.

The Headworks structure has a maximum hydraulic capacity of 1.5 mgd and is limited both by the capacity of the 8-inch line between the Headworks and the Influent Pump Station (IPS) and by the physical size of the Parshall flume. For the future expansion of the plant, upgrading the existing Headworks to accommodate additional capacity is not physically feasible. Therefore, a new Headworks structure should be constructed.

2.2 Influent Pump Station (IPS)

Screened influent is conveyed by gravity to the IPS via an 8-inch diameter pipeline. The wet well dimensions measure 24 feet wide by 24 feet long with a depth of approximately 21 feet (base elevation of 1903 feet). The 8-inch gravity influent pipe from the Headworks structure enters the basin approximately 9.5 feet above the top of slab (12 feet below deck).

The active volume of the pump station is approximately 30,000 gallons, assuming a minimum water level of 3 feet to keep pumps submerged and maximum water below the invert of the 8-inch influent pipe. As such, the volume is not large enough to equalize peak influent flows.

Three submersible pumps are installed at the IPS. Two feed the existing Biolac® system. The third is a mixing pump used to avoid septic conditions and excessive odor release by recirculating the contents of the wet well. Each pump is equipped with an 8-inch discharge isolation plug valve and swing check valve.

A 14-inch magnetic flowmeter meters discharge from the IPS to the Biolac® treatment system. The 14-inch manifold is also connected to the 3-inch manifold that feeds the package plants. There is also a magnetic flowmeter installed in the 3-inch manifold. The IPS will accommodate up to two additional submersible pumps.

A 24-inch overflow pipe connects the existing IPS to the emergency overflow basin (EOB). The high-density polyethylene (HDPE)-lined EOB is 100 feet wide by 100 feet long and has an active depth of 5 feet. The approximate basin volume is 0.6 million gallons (MG). Temporary pumping is required to transfer the contents from the overflow basin back to the IPS.

To control odor in the IPS, a Bohn biofilter is installed adjacent to the wet well. A single blower draws air from the IPS wet well and passes it through the biofilter. At the time of the site visit, the existing blower to the odor control system was not in service.

Table 2.2 describes the composition of the existing Influent Pump Station.

Table 2.2 Existing Influent Pump Station
Marana Water Reclamation Facility Master Plan
Town of Marana

Description	Unit	Value
Package Plant Feed Pumps (1)		
Number of Pumps	ea	3
Type of Pumps	--	Submersible Centrifugal
Speed Control	--	Variable Speed
Horsepower, Each	hp	1.7
Model	--	3067.090
Manufacturer	--	Flygt
Discharge Manifold Size	in	3
Biolac® Feed Pumps		
Number of Pumps	ea	2
Type of Pumps	--	Submersible Centrifugal
Design Capacity, each	gpm	680
Design Total Dynamic Head (TDH), each	ft	36.5
Speed Control	--	Variable Speed
Horsepower, each	hp	10
Model	--	NP3127.090
Manufacturer	--	Flygt
Discharge Manifold Size	in	14

Table 2.2 Existing Influent Pump Station Marana Water Reclamation Facility Master Plan Town of Marana		
Description	**Unit**	**Value**
IPS Mixing Pump		
Number of Pumps	ea	1
Type of Pumps	--	Submersible Centrifugal
Speed Control	--	Constant Speed
Horsepower, Each	hp	20
Model	--	150DLFV6154
Manufacturer	--	Ebara
Odor Control Blower		
Number of Blowers	ea	1
Horsepower, Each	hp	3
Model	--	Size 200 FRP Radial Fume Exhauster
Manufacturer	--	New York Blower
Note: (1) Package Plant Feed Pumps are no longer used.		

2.2.1 Conditions Assessment

The IPS was constructed in early 2001. The current Flygt submersible pumps that feed the Biolac® system and mix the contents of the wet well were installed in 2006 for the interim expansion project, which increased the plant capacity from 0.2 mgd to 0.7 mgd. These pumps have provided reliable service since the Town assumed operation of the WRF in 2012 and appear to be in good operating condition. The submersible pumps that feed the packaged plants are no longer in service.

Since the system was operating during the site visit and access was limited, the internal structure of the IPS could not be inspected. During an outage, a confined space entry can be arranged for a better observation. Although the exterior was in excellent condition, this is not indicative of the interior condition.

Similar to the Headworks, interior concrete corrosion in the IPS can be minimized if the wet well is ventilated under a slight vacuum to an odor control system.

2.2.2 Potential Use for Future Phases

Hydraulically, the capacity of the 8-inch diameter pipeline connecting the existing Headworks to the IPS is sufficient to handle peak flows up to 1.5 mgd.

As indicated above, the IPS can accommodate two additional submersible pumps for an ultimate capacity of 5.4 mgd, matching the capacity of the 24-inch Gladden interceptor. However, that would require replacing the existing pumps with larger ones.

Conceptual plans show several new gravity sewer pipelines conveying raw sewage to the WRF. These new pipelines will be significantly deeper than the existing 24-inch Gladden interceptor, making it impossible for the existing IPS to serve these new pipelines. Unless a new lift station is built to convey flow from the new gravity pipelines, the existing IPS will need to be replaced with a deeper IPS that can accommodate the new gravity sewer pipelines feeding the WRF.

Provisions may be put in place to maintain the functionality of the existing IPS should a future emergency require it. In that case, the existing pump station could be used to pump wastewater from the shallow Gladden interceptor to new treatment facilities.

2.3 Secondary Treatment - Biolac® System

Screened influent is pumped from the IPS to the existing Biolac® basin via a 14-inch diameter pipeline. The Biolac® system is a low organically loaded extended aeration process sized to treat an average daily flow (ADF) of 0.5 mgd. It utilizes an HDPE-lined earthen basin construction with integral clarifiers for secondary clarification.

In the Biolac® system, eight suspended moving fine bubble diffuser aeration chains are suspended approximately 24-inches above the basin bottom for aeration and mixing. The aeration chains span the width of the aeration basin and are fixed with stainless steel cable at both ends.

Each aeration chain consists of a floating polyethylene pipe header with suspended fine bubble diffuser assemblies above the basin floor. Air released from the diffusers causes the chain to oscillate in a regular pattern assisting with basin mixing.

Adjusting the tension in the aeration chains controls the magnitude of the chain oscillation. Low-pressure air for aeration is provided by positive displacement blowers. An air-assist mechanism floats the diffusers up for easier retrieval and maintenance. The aeration and performance of the treatment system are discussed in Section 6.0 of this TM.

As part of the current operation, methanol can be added to the process liquid stream to address the low influent carbon loading. However, the WRF is currently operating without the use of methanol.

At the end of the Biolac® basin, secondary treated effluent passes under a curtain wall to enter the secondary clarifiers. The Biolac® system also includes returned activated sludge (RAS) system, waste activated sludge (WAS) system, and dissolved oxygen (DO) monitoring.

2.3.1 RAS System

The RAS system for the Biolac® runs continuously and provides a maximum sludge return rate of 150 percent of the basin's average design flow. Air from the aeration blowers is used to control the RAS airlift pumps. The return rate is designed to be controlled by a globe valve on the air line that serves each pump; however, WRF operations staff noted that it is very difficult to control the RAS rate with this manual valve.

2.3.2 WAS System

The WAS pump station is located to the west of the Biolac® basin. A 4-inch buried isolation gate valve is used to introduce flow to the WAS pump station. The WAS pump station includes two submersible pumps installed within a manhole. WAS is pumped in a 4-inch pipe from the pump station to an existing sludge storage tank in the southwest corner of the site.

Table 2.3 presents a summary of the existing Biolac® system including the RAS and WAS systems.

Table 2.3 Existing Biolac® System
Marana Water Reclamation Facility Master Plan
Town of Marana

Description	Unit	Value
Aeration Basin		
Number of Basins	ea	1
Basin Volume (1)	MG	0.80
Side Water Depth	ft	11
Design Hydraulic Retention Time	days	1.6
Design MLSS Concentration	mg/L	2,500
Solids Loading (2)	lbs/ft^2 day	35
Aeration Diffuser Chains		
Number of Chains	ea	8
Number of Diffusers/Chain	ea	6 or 7
Total Number of Diffusers	ea	55
Aeration Blowers		
Number of Units	--	4
Type	--	Positive Displacement
Capacity, Each	cfm	690
Horsepower, Each	hp	40
Design Discharge Pressure	psig	5.5

Table 2.3 Existing Biolac® System Marana Water Reclamation Facility Master Plan Town of Marana		
Description	**Unit**	**Value**
Secondary Clarifiers		
Number of Units	ea	2
Type	--	Gravity Settling
Clarifier Width (1)	ft	30'-0"
Clarifier Length (1)	ft	21'-6"
Surface Water Depth (2)	ft	17'-0"
Surface Loading Rate (1)	gpd/sq ft	388
RAS Pumps		
Number	ea	2 (1 per clarifier)
Type of Pumps	--	Air-lift
WAS Pumps (Pumps from Biolac® to Solids Handling System)		
Number of Pumps	ea	1
Type of Pumps	--	Submersible Centrifugal
Capacity, Each (2)	gpm	176
Horsepower, Each	hp	5
Notes: (1) From Biolac® System Design Parameters. (2) From Stantec Interim Plant Upgrade Drawings, Project No. C-343.		

2.3.3 Conditions Assessment

The existing Biolac® system was originally constructed in 2006. The system is functioning; however, several maintenance issues over time have severely impacted the treatment process. These issues are as follows:

- The diffusers have been replaced with BioWorks diffusers to provide additional air; however, the cyclic operation and setting (depth) of the diffusers have been difficult to optimize, resulting in uneven air distribution.
- The air assisted system to raise the diffusers has not functioned in the past.
- Recently, a hole in the underlying HDPE liner was trapping gases and releasing them in eruptions. Divers were engaged to identify and cover the hole with cement bags. Ten additional vents, for a total of 17, have been installed to allow trapped gases, if any, a path for release.

Many components of the secondary clarifiers appeared to be in poor operating condition, including the floating weirs and raking mechanisms. The underflow design into the clarifier can easily disrupt the sludge blanket at peak flow conditions and impede effective settling.

The secondary treatment process lacks redundancy because the aeration basin and integral secondary clarifiers effectively act as a single treatment train. Since neither aeration basin nor clarifiers can be taken out of service, it is not possible to perform reactive and preventative maintenance.

An additional Gardner-Denver positive displacement blower was added to the original Biolac® system for additional redundancy. The current aeration blowers appear to be in good operating condition. However, extending the canopy that currently covers the blower variable frequency drives (VFDs) should be considered. Extending the canopy will help reduce the ambient air temperature at the blower intakes and protect the units from UV deterioration.

WRF Operations staff also said that the existing blowers are difficult to remove from their enclosures and pose safety concerns. Currently, the blowers and motors have to be slid out by hand to attach a crane to complete the removal. To address this, a removable canopy structure could be added to facilitate maintenance of the aeration blowers and mitigate potential safety concerns.

2.3.4 Potential Use for Future Phases

The Biolac® system could be repurposed to serve an emergency storage basin or additional reclaimed water storage basin, once the HDPE liner has been repaired or replaced.

2.4 Secondary Treatment - Package Plants

The integrated package plant consists of four treatment trains, each capable of treating 50,000 gpd for a total treatment capacity of 200,000 gpd. Due to their small treatment capacity (0.2 mgd total or 0.15 mgd firm capacity with one out of service), the package plant treatment system is no longer operational.

As noted above, the existing package plants were originally fed by submersible pumps at the IPS.

Each includes:

- An aeration zone.
- An anoxic zone, with two mixers.
- A clarifier.
- Sludge storage.
- A chlorine contact basin.

The package system used two blowers housed within a single acoustical enclosure to supply air to the diffusers in the aeration basin and sludge storage and to operate the RAS airlift pumps. Each treatment train can operate independently or in parallel. A WAS pump located at the clarifier's scum/WAS box transfers WAS to the sludge storage tank.

Table 2.4 summarizes the existing package plant treatment system.

Table 2.4 Existing Package Plants
Marana Water Reclamation Facility Master Plan
Town of Marana

Description	Unit	Value
Number of Treatment Trains	ea	4
Capacity/Train	gpd	50,000
Capacity, Total	gpd	200,000
Train Length	ft	66'-10"
Train Width	ft	12'-0"
Surface Water Depth	ft	10'-6"
Aeration Zone Volume	gal	34,750
Anoxic Zone Volume	gal	4,150
Sludge Storage Volume	gal	6,000
Chlorine Contact Basin Volume	gal	1,800
Anoxic Mixers		
Number	--	2
Type	--	Vertical Lineshaft
Horsepower, Each	hp	1
Aeration Blowers		
Number of Blower Units	ea	3
Number of Blowers per Unit	ea	2
Total Number of Blowers	ea	6
Type	--	Positive Displacement
Horsepower, Each	hp	15

2.4.1 Conditions Assessment

The existing package plants have not been operated since 2006. The functionality of the blowers, mixers, and pumps is unknown. The diffusers and small diameter piping appeared to be in poor condition and would need to be replaced if the system were to be put into operation again.

The steel shells of the tanks exhibited some minor corrosion. However, none of the damage appears to be structurally significant because there was no net loss of the structural wall sections. These steel tanks could be repurposed after proper surface preparation and coating. Before investing in repurposing the tanks, performing a water leak test is suggested to confirm structural integrity.

WRF operations staff has noticed some settlement of the package plants slab at the east end, closest to the IPS. However, it does not represent a structural issue to the support slab and could be the natural settlement that occurs after construction, which usually lessens over time. Operations staff have identified a service water leak in the vicinity, which may have caused the settling. The compaction and settlement in the vicinity should be measured and monitored closely to determine if it has ceased or is continuing.

2.4.2 Potential Use for Future Phases

The existing package plants have an approximate volume of 186,800 gallons (four trains, each with a capacity of 46,700 gallons). In the future, this volume may be used for WAS storage before thickening. The existing internal components, such as air distribution lines and diffusers, are not in usable condition and would need to be replaced. Additional evaluation of the repurposing of these steel tank structures will be discussed in Technical Memorandum No. 3 - Solids Handling Evaluation.

2.5 Secondary Effluent Pump Station

Secondary effluent is conveyed by gravity to the Secondary Effluent Pump Station (SEPS) via a 12-inch diameter pipeline. The SEPS was needed because there is insufficient hydraulic head between the discharge of the Biolac® clarifiers and the outfall structure to accommodate headloss in the deep bed sand filters and UV disinfection systems.

Two submersible pumps with VFDs lift secondary effluent up to the tertiary sand bed filters. The discharge from each pump includes a 14-inch discharge isolation plug valve and swing check valve. The SEPS can accommodate up to two additional submersible pumps.

Table 2.5 summarizes the existing secondary effluent pump station.

Table 2.5	Existing Secondary Effluent Pump Station Marana Water Reclamation Facility Master Plan Town of Marana	
Description	**Unit**	**Value**
Number of Pumps	--	2
Type of Pumps	--	Submersible Centrifugal
Capacity, Each (1)	gpm	200 - 1050
Design TDH	ft	23 - 26
Horsepower, Each	hp	10
Speed Control	--	Variable Speed
Model	--	NP3127X
Manufacturer	--	Flygt

Note:
(1) From Stantec SE Pump Station, Filtration & UV System, Package One, Project No. 3MAR10.

2.5.1 Conditions Assessment

The existing secondary effluent pump station was originally constructed in 2009. The structure and submersible pumps appear to be in good operating condition.

2.5.2 Potential Use for Future Phases

The SEPS was designed to accommodate up to four submersible pumps, each with a maximum capacity 2,900 gpm (12.5 mgd firm capacity), and can accommodate the pumping demands for the future flow scenarios discussed in this TM. However, the SEPS may not be needed, depending on the hydraulic profile design of future preliminary and secondary treatment facilities.

2.6 Tertiary Filters

Secondary effluent is pumped from the SEPS to the DynaSand® sand bed filters via a 20-inch diameter pipeline. Deep bed sand filters were originally constructed to filter secondary effluent from the package plant and Biolac® facility. Three concrete basins each house six filter modules. In total, the three sand bed filters will accommodate a Peak Wet Weather Flow (PWWF) of 5.4 mgd.

The sand bed filters also include compressed air and polymer feed systems. Backwash from the filters flows by gravity to the Drain Pump Station via an 8-inch diameter drain pipe. The Drain Pump Station houses two constant speed submersible pumps the recycle filter backwash to the IPS. The discharge from each pump includes a discharge isolation valve and check valve.

Table 2.6 summarizes the existing tertiary filters.

Table 2.6	Existing Tertiary Filters Marana Water Reclamation Facility Master Plan Town of Marana	
Description	**Unit**	**Value**
Number of Cells	ea	3
Number of Filter Modules/Cell	ea	6
Surface Area/Module (1)	ft^2	50
Total Filtration Area (1)	ft^2	900
Design Loading Rate (1)	gpm/ft^2	3 - 5
Backwash Rate, Total	gpm	90-100
Air Compressor Horsepower	hp	20
Air Compressor Feed Rate (1)	cfm	24.8 @ 35 psig
Air Receiver Capacity	gal	240
Performance Requirement	--	Less than 5 mg/L TSS or 2 NTU
Model	--	Continuous Backwash Deep Bed DynaSand®
Manufacturer	--	Parkson
Drain Pump Station		
Number of Pumps	--	2
Type of Pumps	--	Submersible Centrifugal
Capacity, Each (2)	gpm	100
Design TDH	ft	25
Horsepower, Each	hp	Unknown (3)
Speed Control	--	Constant Speed
Model	--	CS3068.180
Manufacturer	--	Flygt

Notes:
(1) From Stantec SE Pump Station, Filtration & UV System, Package One, Project No. 3MAR10.
(2) Based on discussions with plant staff.
(3) Minimum of 5 horsepower based on discussions with plant staff.
Abbreviations:
TSS = total suspended solids; NTU = nephelometric turbidity units

2.6.1 Conditions Assessment

The DynaSand® tertiary filters were originally constructed in 2009. One sand filter cell is out of service due to a collapsed module, the cause of which must be investigated after the sand can be removed from the cell. Despite the collapsed cell and except for some minor vertical cracking, the concrete structure is in excellent condition and should serve for many years without repair. The DynaSand® deep bed filters were designed to operate at a backwash rate of 7 to 12 gpm/module. Of the 18 modules of filters installed (3 cells of

6 modules each), only 6 modules are in service at the current plant flows. Based on the backwash design rates, backwash flows between 50,400 and 103,680 gpd would be expected. However, WRF operations staff has noted that backwash rates actually range from 100,000 to 150,000 gpd. The backwash flow rate, which is returned to the Biolac® treatment system, is approximately 42 percent of the average daily flow and can significantly affect the treatment performance and basin volume.

Conversations with the Parkson representative have not determined the cause of the excessive backwash rates. The representative said the only remedy would be to decrease the air rate at which the sand is continuously scoured. WRF Operations staff has indicated that lower air rates have resulted in *E. coli* detection and the air is turned down as low as possible for it still to be able to fluidize the sand bed. Therefore, air flow rates have been established to the minimum allowable to maintain treatment.

Additionally, the WRF operations staff said that the air compressor cycles frequently with only a single sand filter cell in operation. An additional and/or larger air receiver should be considered for future plant expansions.

The Parkson representative proposes the filters be retrofitted with their new filter wash system called EcoWash. The system claims to reduce as much as 85 percent of both the power and volume of returned backwash flows. The retrofit would include a new programmable logic controller (PLC) and control program, new air control panels for each filter cell (three total), and new dual level air lift mechanisms.

Retrofitting one of the three filters is recommended as soon as possible. This would allow the WRF to reduce backwash volumes immediately and provide some time for operational experience to verify its effectiveness before the next plant expansion is constructed. Retrofitting the two remaining cells could be completed and funded as part of the next construction project.

2.6.2 Potential Use for Future Phases

The tertiary filters were designed for a PWWF of 5.4 mgd and can accommodate the filtration demands for the future flow scenarios discussed in this TM. The existing tertiary filters will remain part of the treatment process during future plant expansions.

2.7 UV Disinfection

Filtered effluent flows by gravity to the in-channel Trojan UV disinfection system. The flow is metered as it passes over a suppressed rectangular weir before entering the UV disinfection channel(s). In-channel UV disinfection provides disinfection for the existing package plant and Biolac® facility. Two parallel UV channels were each designed to house three UV banks. Currently, only three UV banks have been installed in one channel. These banks can accommodate a PWWF of 5.4 mgd. Filtered and disinfected effluent flows by gravity from the UV disinfection facility to the existing Metering Dechlorination Box No. 2 via a 24-inch diameter pipeline. Downstream of the UV facility, an ultrasonic level sensor monitors channel level and adjusts an automatic weir to maintain a channel depth of 32 inches.

Table 2.7 summarizes the existing UV disinfection system.

Table 2.7 Existing UV Disinfection System
Marana Water Reclamation Facility Master Plan
Town of Marana

Description	Unit	Value
Number of Channels	ea	2
Number of UV Banks/Channel	ea	3
Number of UV Modules/Bank	ea	9
Total Number of Installed UV Modules	ea	27
Design UV Dose (1)	mJ/cm^2	107
Model	--	3000 Plus
Manufacturer	--	Trojan

Note:
(1) From Stantec SE Pump Station, Filtration & UV System, Package One, Project No. 3MAR10.

2.7.1 Conditions Assessment

The existing Trojan UV disinfection system was originally installed in 2009 and appears to be in good operating condition.

This structure is in excellent condition and should serve for many years without repair.

2.7.2 Potential Use for Future Phases

The existing UV disinfection system was designed for a PWWF of 5.4 mgd and can accommodate the disinfection demands for the future flow scenarios. The existing UV disinfection system will remain part of the treatment process during future plant expansions.

2.8 Plant Effluent Outfall

The plant effluent flow monitoring system consists of an ultrasonic level sensor and V-notch weir installed inside Metering Dechlorination Box No. 2 (downstream from both the Effluent Storage Pond and the UV disinfection system). The total flow leaving the WRF is a function of the depth of flow that passes over the weir. Flow is monitored both for permitting purposes and for flow-pacing dechlorination.

A 30-inch outfall pipe conveys plant effluent from Metering Dechlorination Box No. 2 to the outfall structure. The outfall structure consists of a modified manhole with three 3-foot openings (weir type at the top of the MH ring). The ground surface elevation at the outfall structure is lowered so loose riprap can be installed approximately 10 feet from the outlet. Three outlets at the outfall structure allow discharge to the north, west, and south.

2.8.1 Conditions Assessment

The V-notch weir, located inside Metering Dechlorination Box No. 2, is mounted to the top of redwood boards. The redwood boards, replaced in 2014, leak and make accurate flow measurement difficult. Replacing the redwood boards with an alternative material such as fiberglass reinforced plastic (FRP) or concrete is recommended. The plant effluent outfall structure was not assessed, but the Town believes it to be in good operating condition.

2.8.2 Potential Use for Future Phases

The existing effluent flow monitoring system and the final discharge system were designed for continuous use through the next plant expansion phase. When these systems cannot meet the flow requirement in the future, they will be abandoned or removed. Although the UV structure design allows for a new final effluent metering system and direct discharge line to the Santa Cruz River, they will now be used by the new recharge facility to be located adjacent to the Marana WRF.

2.9 Effluent Storage Pond

The Effluent Storage Pond, located south of the tertiary filters and UV disinfection system, provides a source of water for the utility water and reuse pump stations.

The Effluent Storage Pond may also be used as a chlorine contact chamber for backup chlorination during emergency situations or when the UV disinfection system is not operational. The contact chamber is chlorinated with sodium hypochlorite added upstream from the contact basin in a flow-monitoring chamber. Within the Effluent Storage Pond, directional baffles are used to prevent short-circuiting. With an active volume of approximately 180,000 gallons, the Effluent Storage Pond provides retention time in excess of 8 hours during an average flow of 0.5 mgd and almost 3 hours during peak flow.

When the Effluent Storage Pond is used as a chlorine contact basin, sodium bisulfite is used to dechlorinate the effluent before discharge. It is added as flow passes over the V-notch weir inside Metering Dechlorination Box No. 2. The sodium bisulfite system is housed inside a plastic structure adjacent to the junction box.

Table 2.8 summarizes the existing chlorination/dechlorination system.

Table 2.8	Existing Chlorination/Dechlorination System Marana Water Reclamation Facility Master Plan Town of Marana	
Description	**Unit**	**Value**
Chlorination System (1)		
Type	--	Liquid Sodium Hypochlorite
Storage	gal	1,000
Number of Tanks	--	1
Volume, each	gal	10,000
Feed Pump Type	--	Diaphragm-type metering pumps
Number	ea	2
Dechlorination System (1)		
Type	--	Liquid Sodium Bisulfite
Storage	gal	2 x 55
Number of Tanks	--	1
Volume, each	gal	10,000
Feed Pump Type	--	Diaphragm-type metering pumps
Number	ea	2

Note:
(1) From Stantec SE Pump Station, Filtration & UV System, Package One, Project No. 3MAR10.

2.9.1 Conditions Assessment

The chlorine disinfection system was completely refurbished in 2015. The dechlorination system is being refurbished with an expected completion time of early 2016. At that point, both systems will be in excellent operating condition.

The Town has reported that the seals for the slide gate feeding the Effluent Storage Pond need to be replaced.

2.9.2 Potential Use for Future Phases

This Effluent Storage Pond will continue to provide a source of water for the utility water and reuse pump stations during future plant expansions.

2.10 Sludge Storage

The sludge storage tank has a capacity of approximately 16,500 gallons. Using a single dry-pit submersible pump, the sludge from the storage tank is transferred to trucks and hauled off-site for final disposal. Coarse bubble diffusers mounted inside the sludge tank prevent the sludge from becoming septic and generating odor. The diffusers are fed by a positive displacement blower mounted adjacent to the storage tank.

Table 2.9 summarizes the existing sludge storage system.

Table 2.9 Existing Sludge Storage System
Marana Water Reclamation Facility Master Plan
Town of Marana

Description	Unit	Value
Storage Tank (1)		
Type	--	Vertical, Cylindrical
Number of Tanks	--	1
Volume, Each (2)	gal	16,500
Waste Transfer Pump		
Number of Pumps	ea	1
Type	--	Dry-pit Submersible
Capacity, Each	gpm	250
Horsepower, Each	hp	2.3
Speed Control	--	Constant Speed
Model	--	CT3085
Manufacturer	--	Flygt
Aeration/Mixing Blower		
Number	--	1
Type	--	Positive Displacement
Capacity, Each	cfm	Unknown
Horsepower, Each	hp	5
Design Discharge Pressure	psig	Unknown

Notes:
(1) Decant from the Sludge Holding Tank is recycled to the IPS. The decant pump is operated manually prior to sludge transferring activities.
(2) From Stantec Interim Plant Upgrade Drawings, Project No. C-343.

2.10.1 Conditions Assessment

The sludge holding tank and waste transfer pump were installed in 2006. A new aeration blower was installed in the summer of 2015. Despite manual decanting, the system appears to be in good operating condition.

2.10.2 Potential Use for Future Phases

This sludge holding tank can continue to store sludge or thickened solids during future plant expansions, if necessary.

2.11 Ancillary Systems

2.11.1 Reuse Water System

Without a potable water system available, the WRF relies on two separate systems for reuse water. The Utility Water System is located south of the Effluent Storage Pond and includes three vertical lineshaft pumps and a hydropneumatic tank.

Plant effluent is pumped from the Effluent Storage Pond for various uses around the site. The non-potable well water from Monitoring Well M-1 is currently used for sinks and toilets in the Control Building.

Table 2.10 summarizes the existing reuse water systems.

Table 2.10 Existing Reuse Water System
Marana Water Reclamation Facility Master Plan
Town of Marana

Description	Unit	Value
Utility Water Pumps		
Type	--	Vertical Lineshaft
Number (1)	ea	3
Non-Potable Water Pump (Water Supply Well M-1)		
Type	--	Monitoring Well
Number	--	1

Note:
(1) One utility pump is used for irrigation purposes only.

2.11.2 Conditions Assessment

It is estimated that the existing reuse water systems were part of the original plan construction. The air compressor is no longer in service. The pumps appear to be in good operating condition.

2.11.3 Potential Use for Future Phases

The reuse water systems will continue to serve the plant in their current capacity during future plant expansions.

3.0 FLOODPLAIN EVALUATION

The Marana WRF is located adjacent to the floodplain of the Santa Cruz River. Based on information from the National Flood Insurance Program by the Federal Emergency Management Agency (FEMA), Pima County's Regional Flood Control District (The County) has established 100-year base flood elevations at various cross-sections along the Santa Cruz River. Figure 2.4 illustrates these flood elevations in relation to the WRF site.

After a significant flood event in 2006, the County required that a berm be constructed along the floodplain to protect process basins and structures would during a 100-year flood. Construction plans show the berm elevation to be approximately 1924.00, which is sufficiently higher than the predicted flood elevation (1921.00) adjacent to the plant.

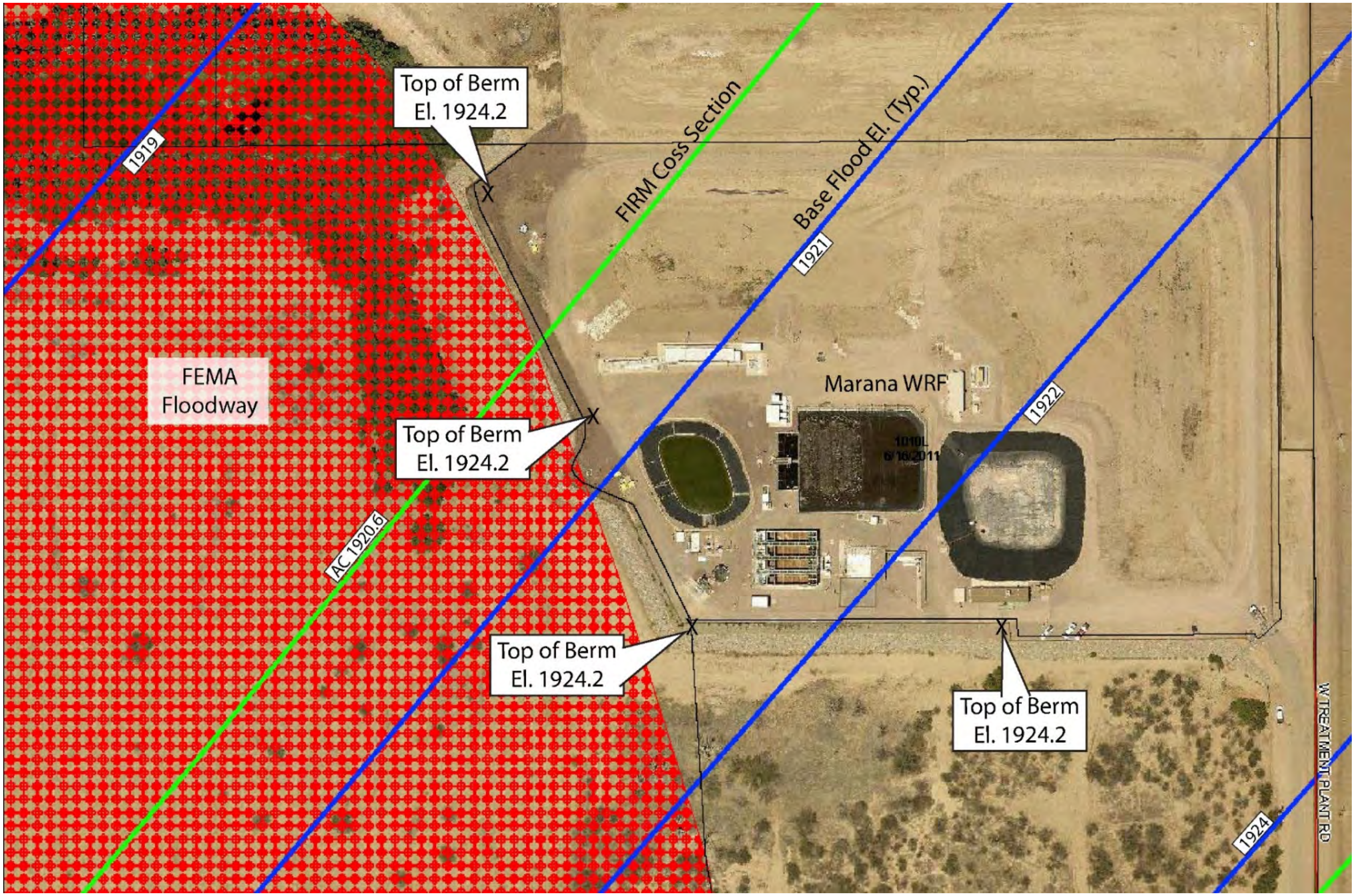
Since completing the berm, the facility has not been flooded. However, for flood events more severe than the 100-year flood, the southeast portions of the WRF may experience flooding. Currently, no critical facilities are located in this area.

4.0 HYDRAULIC CAPACITY EVALUATION

The hydraulic profile for the plant was modeled using *Hydraulix*®, which is a proprietary computer-based hydraulic modeling software developed by Carollo Engineers. The program starts with a downstream elevation, the 100-year flood elevation at the outfall structure, and calculates the head loss, hydraulic grade line, and energy grade line through the upstream elements. For purposes of this TM, the hydraulic model was run at the three flow scenarios listed in Table 2.11.

Table 2.11 Hydraulic Modeling Scenarios
Marana Water Reclamation Facility Master Plan
Town of Marana

Model Run Description	Influent Flow (mgd)
Current Plant Average Annual Daily Flow (AADF)	0.355
Plant Design Average Annual Daily Flow	0.500
Peak Hour Flow of the Design AADF (3.0 x ADF)	1.50



100-YEAR BASE FLOOD ELEVATIONS

FIGURE 2.4

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

4.1 Hydraulic Control Locations

The pump stations lift the flow up to a hydraulic head, where the wastewater flows by gravity through various processes, pipes, and channels. The hydraulic profile of the facility is divided into the following hydraulically isolated sections:

- **Section 1:** Gravity flow from the collection system, into the Headworks facility (Parshall flume, bar screen, and grit channel) to the Influent Pump Station.
- **Section 2:** Lifted from the Influent Pump Station (IPS), through the Biolac® system, over the clarifier weirs, and to the Secondary Effluent Pump Station (SEPS).
- **Section 3:** Lifted from the SEPS, up to the filter influent channel, through the deep bed sand filters, through the UV disinfection system, to Metering Dechlorination Box No. 2, and to the Santa Cruz River via the 30-inch outfall pipe and structure.

The output for the hydraulics model run is included in Appendix A.

4.2 Findings and Recommendations

The modeling results are discussed below.

- **Section 1:** This hydraulic section presents several flow restrictions, as described below.
  a. The Parshall flume is 3 inches wide, and when properly calibrated will measure flows ranging between 15 to 830 gallons per minute (gpm) (0.22 to 1.2 mgd). At the design peak hour flow rate of 1.5 mgd, the water surface in the Parshall flume would be within 0.10 foot of overtopping the structure. The picture below illustrates this point by showing the water surface elevation in the Parshall flume during a typical inflow event is within inches of the top of the structure.

Parshall Flume at Headworks

The 8-inch pipe between the Headworks and the IPS can convey approximately 1.5 mgd before exceeding reasonable velocities (6.7 fps) and creating additional friction losses, which in turn raise the upstream water surface.

b. The depth of the IPS is only 9.5 feet below the invert of the 8-inch inlet pipe. Assuming a minimum water level of 2 feet is required to keep the IPS pumps submerged, the active volume in the IPS is less than 30,000 gallons. At an incoming peak hour rate of 1.5 mgd (1,042 gpm) and assuming only one influent pump is operating, the active volume equates to approximately 20 minutes of flow equalization before flows would begin to backup into the Headworks.

c. The 12-inch sections of gravity sewer inside the WRF are capable of conveying approximately 1.2-1.4 mgd at full pipe flow based on their relatively flat slopes.

- **Section 2:** The piping is sufficiently sized to handle a combined peak hour flow of 1.5 mgd with return flows from the filter backwash drain pump station and pipe velocities ranging from 1.8 to 3.25 fps. There are no major flow restrictions in this hydraulic section for the current plant capacities.

- **Section 3:** There are no major flow restrictions or issues under the three flow scenarios shown in the table above. The FRP V-notch weir in Metering Dechlorination Box No. 2 (the last weir before the outfall to the Santa Cruz River) has a top elevation of 1921.50 and a bottom (discharge) elevation of 1920.50. The 100-year flood elevation is 1921.0. Therefore, the weir opening is slightly submerged if the downstream river is at its 100-year flood elevation. However, the dechlorination manhole structure itself will not be over-topped (top elevation of 1924.50), and the weir at the end of the UV system (elevation 1924.0) will not be submerged. The channels and piping of this hydraulic section are more than sufficiently sized for the current plant design flows.

4.3 Hydraulic Design Considerations in Potential Future Process Selections

The FRP weir inside Metering Dechlorination Box No. 2 could be modified to make the weir's entire length 1921.50, above the 100-year flood plain elevation of 1921.00. This would have no impact on the upstream UV channel and discharge.

The SEPS currently lifts water approximately 12 feet from the level of the Biolac® discharge to the filter inlet channel. Future preliminary and secondary treatment processes could be raised above ground and above 100-year flood elevations. This could eliminate the need for this internal process pump station and reduce pumping costs.

A detailed hydraulic profile of future process scenarios will be evaluated as part of the master planning of future phases.

5.0 TREATMENT PROCESS EVALUATION

Below is a detailed evaluation of the secondary treatment system and its performance to reliably meet permit effluent limitations.

5.1 Existing Process Description

The secondary treatment process is an extended aeration activated sludge system based on a Biolac® system (manufactured by Parkson). This system consists of a lined earthen basin (aeration basin) and integral rectangular secondary clarifiers. Although the air diffuser assemblies have been replaced with the same type of tubular membrane units from a different manufacturer (Bioworks), the original aeration system controls and operating philosophy remain in place.

The treatment process is designed to remove biological nitrogen by cycling aeration on and off in alternating groups of aeration diffusers throughout the aeration basin. This cycling alternates oxic and anoxic conditions in the aeration basin volume within a relatively long solids and hydraulic retention time.

The existing air diffuser system consists of eight aeration "chains." Air delivery is individually controlled by an electrically actuated valve for each chain, which has seven diffuser assemblies with four diffusers per assembly. Chains are numbered starting at the clarifier end with Chain No. 1 and ending with Chain No. 8 at the influent side of the aeration basin.

Due to concerns over the basin's HDPE liner integrity, Chain No. 8 has only 6 diffuser assemblies. To accommodate basin liner repairs, Chain Nos. 5 and 8 were turned off for several weeks around the time of this evaluation.

The standard operating mode of the aeration system is as follows:

- Air to Chain Nos. 1 and 2 is continuously on at a low flow rate, achieving dissolved oxygen concentrations of approximately 0.2 mg/L towards the end of the aeration basin.
- Air is cycled on and off between two groups of chains: 3, 4, and 5 and 6, 7, and 8. Every 60 minutes, the air is alternated from one group to the other, creating alternating oxic and anoxic conditions in the aeration basin. Currently, the air cycle interval has been adjusted to 70 minutes to assist with denitrification.
- A system of positive displacement air blowers sends process air to the activated sludge system. The blower system has a PLC that controls the air delivery to the alternating groups of aeration chains controlled by the automated airflow valves. Blower speed is determined by averaging the dissolved oxygen concentration over two air cycle intervals.

Two integral secondary clarifiers at the end of the aeration basin separate solids from liquids of the activated sludge. However, since no mechanism isolates each clarifier, the entire aeration basin with the two integral clarifiers effectively acts as a single treatment train.

The secondary clarifiers have a single "v-shaped" sludge hopper and use an airlift mechanism to remove the settled activated sludge from the bottom of the clarifiers. Most of the activated sludge is returned to the head of the aeration basins RAS, with a smaller portion wasted to the sludge storage tank WAS.

5.2 Process Data Analysis

Process data between September 2014 and September 2015 were used to evaluate the performance of the secondary treatment process. Table 2.12 summarizes the process data used for this evaluation.

Effluent Nitrogen. The effluent nitrogen data indicate that the activated sludge process is removing nitrogen well below the total nitrogen (TN) alert level of 8 mg/L and the TN permit limit of 10 mg/L.

Effluent ammonia data are usually below 1 mg/L, indicating that the system is performing full nitrification most of the time.

The average effluent nitrate of just below 2 mg/L indicates very good denitrification, as expected for the relatively large system volume and low dissolved oxygen concentrations achieved by the cyclic aeration pattern of the Biolac® system.

There have been periods in which the 30-day average effluent nitrate concentration has increased to approximately 3 mg/L (2 events) and 5 mg/L (1 event), likely due to adjustments to the aeration cycles. While the effluent TN has not reached the alert level of 8 mg/L during these periods, the data indicate that adjustments to the aeration cycles are critical for the performance of the BNR system.

Also, note that as the influent flows increase, the system SRT will drop and the aeration cycle times will require adjustment to maintain the current level of nitrogen removal.

Solids Retention Time (SRT). The activated sludge system SRT for the period analyzed was between 20 and 32 days, averaging 27 days. This is a typical range for extended aeration systems and is consistent with the original design intent of the Biolac® system. As flows increase, the system SRT will decrease unless the operating mixed liquor suspended solids (MLSS) are increased to compensate the increased loads.

Table 2.12 Process Data Evaluation Summary Marana Water Reclamation Facility Master Plan Town of Marana			
Parameter	**Annual Average (1)**	**Minimum 30-day Average (1,2)**	**Maximum 30-day Average (1,2)**
Effluent Total Nitrogen (mg/L)	3.51 (3)	1.30 (4)	4.75 (4)
Effluent Ammonia (mg/L)	0.30 (5)	0.00 (5,6)	2.76 (6)
Effluent Nitrate (mg/L)	1.85	0.87	5.20
MLSS (mg/L)	2,738	2,038	3,199
SRT (days) (7)	26.6	19.8	31.8
RAS TSS (mg/L)	6,835	5,132	8,694
RAS Flow (mgd)	0.383	0.280	0.484
RAS Flow Ratio to Plant Influent	1.11	0.82	1.42
Sludge Volume Index (mL/g)	151	109	241
North Clarifier Clear Water Depth (ft)	8.0	3.0 (8)	10.5 (8)
South Clarifier Clear Water Depth (ft)	7.7	2.0 (8)	10.5 (8)
WAS Flow (gpd)	9,930	6,558	14,158
WAS Solids (lb/d)	551	381	728
Hauled Sludge (gpd)	5,675	5,431	9,533

Notes:
(1) Based on daily sample results, unless otherwise noted.
(2) 30-day running average minimum and maximum values, unless otherwise noted.
(3) Based on bi-weekly sample results.
(4) Minimum and maximum TN values shown are based on the 5-sample rolling geometric mean of bi-weekly sample results. Alert level of 8 mg/L and permit limit of 10 mg/L TN are based on a 5-sample rolling geometric mean.
(5) Ammonia values reported as "<1.00 mg/L" were assumed at a value of zero for the purposes of calculating average and minimum values.
(6) Minimum and maximum ammonia values shown are single-sample bi-weekly sample results.
(7) SRT calculated using an active aeration basin volume of 0.684 MG, assuming 20 mg/L TSS in the secondary effluent, and without including the solids inventory in the secondary clarifiers.
(8) Daily minimum or maximum.

Mixed Liquor Suspended Solids (MLSS). The operating MLSS of the Biolac® system, generally between 2,000 mg/L and 3,200 mg/L, falls within the typical range for aeration activated sludge systems. The most recent data shows MLSS concentrations of approximately 1,700 mg/L, which is slightly below typical MLSS concentrations. WRF Operations staff noted that a lower MLSS target was set to increase the food-to-microorganisms ratio to improve sludge settleability.

Settleability. The sludge volume index (SVI) is a parameter that describes the settleability of the activated sludge. Guidelines for classifying sludge settleability based on SVI are:

- "Well-settling" sludge typically has SVI values below 150 mL/g.
- "Light" sludge has SVI values between 150 and 200 mL/g.
- "Bulking" sludge has SVI values above 200 mL/g.

A typical design value of 150 mL/g is commonly used in the industry when sizing secondary clarifiers.

The SVI values have been generally high at the Marana WRF. While the average SVI is 151 mL/g, the best monthly average SVI values are still higher than 100 mL/g (i.e., 109 mL/g as a minimum 30-day average). On the other hand, the worst SVI values are relatively high, with a maximum 30-day average of 241 mL/g.

Relatively high SVIs are fairly common in extended aeration systems. Conditions that favor the growth of filamentous bacteria in activated sludge systems include long SRTs that lead to low food-to-microorganism ratios in systems without anaerobic or anoxic selectors and low dissolved oxygen concentrations in the presence of available BOD. Because both of these conditions apply to the Marana WRF, relatively high SVIs can be expected. The SVI's impact on the secondary clarifiers' capacity is further discussed in Section 5.3.

The reported clarifier clear water depth was analyzed. While the average clear water depth of approximately 8 feet seems adequate, depths of 2 and 3 feet were reported on several days. Out of one year of daily values in the data set, two periods of 28 days and 34 days the clear water depth was less than 5 feet. These data indicate challenges with achieving good sludge settleability and correlate with the relatively high SVIs mentioned above.

Return Activated Sludge. The RAS total suspended solids concentration falls within typical ranges for activated sludge systems:

- A 30-day averages generally fall between 5,000 and 8,700 mg/L.
- An overall average of 6,800 mg/L.

However, the daily RAS TSS concentrations show significant variability, ranging between 4,000 mg/L and 11,000 mg/L. This relatively wide range of RAS TSS concentrations can be attributed to the difficulty in controlling the RAS flow using the existing airlift mechanism. The RAS airlift system is connected to the process blowers and is very sensitive to small valve adjustments, resulting in a lack of positive control of RAS flow from the clarifiers.

The average RAS flow ratio to the plant influent flow of 1.1 and the 30-day minimum and maximum values of 0.8 and 1.4 fall within typical values for Biolac® extended aeration systems. However, similar to the relatively wide variability of the daily RAS TSS concentration values, the daily RAS flow ratios vary considerably, between 0.6 and 1.8. The difficulties with positive RAS flow control seem to be causing variable RAS flows, which affect the RAS TSS concentration and create more challenges for accurate sludge wasting and SRT control.

Waste Activated Sludge. The average WAS production was compared to the plant influent BOD load, resulting in a unit sludge production of 0.87 pound TSS per pound of influent BOD. This is within the expected sludge production predicted by the process modeling, which is described in the next section.

The ratio of the maximum 30-day average to the annual average WAS solids on a dry solids basis is 1.32. This ratio is in line with monthly influent load variations, validating influent maximum month load design load peaking factors of 1.3.

The WAS volume of almost 10,000 gpd is cut approximately in half by decanting in the sludge storage tank. The hauled sludge is normally 5,500 gpd. The limited storage volume and lack of thickening/dewatering requires daily sludge withdrawal of dilute sludge from the Marana WRF. Sludge dewatering would reduce the volume of sludge disposed and the operational and hauling costs.

5.3 Process Modeling Evaluation of Existing System

Process modeling for the WRF was performed using the Biological Treatment Analysis (Biotran) and BioWin modeling tools.

Biotran is a steady-state, spreadsheet-based modeling tool developed by Carollo for wastewater treatment plant design and process evaluations. It uses mass balances, biological, and physical models to simulate interactions between the different unit processes in a wastewater treatment facility.

BioWin is a commercially available dynamic process simulator developed by EnviroSim Associates.

Both modeling tools allow the user to evaluate "what-if" scenarios for different process alternatives or different wastewater compositions before making any operational changes or capital improvements.

The primary objective for modeling the performance of the Marana WRF was to determine the predicted performance of the existing secondary treatment process under the actual flows and loadings and to identify factors that limit the overall plant capacity.

5.3.1 Influent Wastewater Flows and Loadings

The influent wastewater flows and wastewater characteristics used for this evaluation were defined in Technical Memorandum No. 1 - Wastewater Flows and Loadings. These flows and loads are based on historical data. The facilities were evaluated at the design flow of 0.5 mgd AADF, with the corresponding load peaking factors. Table 2.13 summarizes the design flows and loadings.

Table 2.13	Design Influent Wastewater Characteristics - Existing Facilities Marana Water Reclamation Facility Master Plan Town of Marana	
Parameter	**Unit**	**Value**
Design Flows		
AADF	mgd	0.5
MMADF (PF = 1.1)	mgd	0.55
PDF (PF = 2.0)	mgd	1.0
PHF (PF = 3.0)	mgd	1.5
Design Concentrations at AADF		
BOD	mg/L	228
TSS	mg/L	233
TKN	mg/L	57
NH_3-N	mg/L	42
Design Loads at AADF		
BOD	lb/d	951
TSS	lb/d	972
TKN	lb/d	238
NH_3-N	lb/d	175
"Equivalent" Design Concentrations at MMADF (1)		
BOD	mg/L	269
TSS	mg/L	297
TKN	mg/L	67
NH_3-N	mg/L	50
Design Loads at MMADF		
BOD (PF = 1.3)	lb/d	1,236
TSS (PF = 1.4)	lb/d	1,360
TKN (PF = 1.3)	lb/d	309
NH_3-N (PF = 1.3)	lb/d	228

5.3.2 Activated Sludge System Capacity Evaluation

The secondary treatment system includes the aeration basins and secondary clarifiers.

The aeration basins are designed to maintain a minimum SRT to achieve the desired treatment objectives. As flows and loadings are increased, the system may require either more volume, or a higher operating MLSS to maintain this minimum SRT. Therefore, for a given aeration basin volume, increasing the operating MLSS generally increases the treatment capacity. However, the maximum MLSS in the aeration basins is determined by the capacity of the secondary clarifiers.

The capacity of a given secondary clarifier volume is reduced with an increase in MLSS in the aeration basins.

The existing secondary clarifiers were evaluated to determine the maximum MLSS concentration they can support at the design flows.

The aeration basins were then evaluated at those MLSS concentrations to determine their capacity and BNR performance.

5.3.2.1 Secondary Clarifiers Capacity Evaluation

The clarifier safety factor (CSF) was the primary criteria used to determine the capacity of the secondary clarifiers. This is the ratio between the estimated initial settling velocity of the mixed liquor and the surface overflow rate (SOR) under a given design peak flow condition. CSF values below 1.0 indicate conditions in which the sludge blanket rises. These values should be avoided for the selected design peak flow condition.

The MLSS and the settleability characteristics of the sludge determine the initial settling velocity of the mixed liquor, which is estimated from design MLSS and SVI values. The design SVI should be conservative enough to account for less-than-ideal settling characteristics but should not be overly conservative as to account for the worst possible settling characteristics occurring at peak hydraulic conditions.

As discussed in Section 5.2, the average SVI for the most recent year of operation was 151 mL/g, the 92nd percentile was 192 mL/g, and the highest 30-day average SVI was 241 mL/g. The recommended design SVI is the 92nd percentile value of 192 mL/g.

The design flow condition for the analysis was established as the design PDF, plus an additional safety factor of 15 percent. This condition implies that at the design SVI and MLSS, the sludge blanket may slowly rise only during peak hour flow conditions.

Table 2.14 summarizes the secondary clarifier capacity results. At the recommended design SVI of 192 mL/g, the maximum MLSS at the design flow is 2,000 mg/L. The impact of the design SVI on the maximum MLSS is significant. The MLSS can range between 3,000 mg/L for the average SVI of 151 mL/g, and 1,100 mg/L for the maximum 30-day average SVI of 241 mL/g. For an MLSS of 2,000 mg/L and an SVI of 241 mL/g, the maximum flow that could be supported is 0.58 mgd.

Table 2.14	Effect of SVI on Maximum MLSS at Design Flow - Existing Clarifiers Marana Water Reclamation Facility Master Plan Town of Marana		
Peak Day Flow (mgd) (1)	**SVI (mL/g)**	**MLSS (mg/L)**	**CSF (2)**
1.0	151 (3)	3,000	1.18
1.0	**192 (3)**	**2,000**	**1.17**
1.0	241 (4)	2,000	0.67
1.0	241 (4)	1,100	1.16
0.58	241 (4)	2,000	1.15

Notes:
(1) Corresponds to the design AADF of 0.5 mgd with a PDF PF of 2.0.
(2) The minimum target CSF at the PDF is 1.15 (minimum of 1.0 plus a 15 percent safety factor). The CSF is calculated with the two existing secondary clarifiers in service.
(3) Average SVI between September 2014 and September 2015.
(4) 92nd percentile SVI between September 2014 and September 2015.
(5) Maximum 30-day running average SVI between September 2014 and September 2015.

For capacity analysis, the maximum recommended MLSS with the existing secondary clarifiers is 2,000 mg/L. Based on this analysis we estimate that an MLSS of 2,000 mg/L can be supported by the existing secondary clarifiers for SVIs up to 192 mL/g and peak flows up to 1 mgd (corresponding to an AADF of 0.5 mgd with a PDF PF of 2.0). This condition is identified in bold in the table above.

5.3.2.2 Aeration Basin Capacity Evaluation

The aeration basin capacity was evaluated using the total SRT as the main process capacity criteria (nitrogen removal is also discussed in Section 5.3.3). The total SRT for the existing aeration basin was calculated discounting 50 percent of the aeration basin volume in the side slopes and all the volume in the basin corners because those portions of the basin do not contribute to the treatment process.

Table 2.15 summarizes the capacity evaluation for the aeration basin at the design loads associated with the AADF of 0.5 mgd. At the maximum recommended MLSS of 2,000 mg/L that can be supported by the secondary clarifiers (see Section 5.3.2.1), the total SRT is expected to be between 11 and 14 days at the maximum month average day load (MMADL) and annual average day load (AADL), respectively.

The analysis shows that the aeration basins have sufficient capacity to operate at the design loads associated with the AADF of 0.5 mgd but at SRTs falling below typical values for extended aeration systems (typically 20 to 30 days).

Nitrogen removal performance will be further discussed in Section 5.3.3.

Table 2.15 Capacity Evaluation of Existing Aeration Basin Marana Water Reclamation Facility Master Plan Town of Marana		
Criteria	**Design MLSS (1)**	**High MLSS (2)**
Influent AADL (lb/d) (3)	951	951
Influent MMADL (lb/d) (3)	1,236	1,236
WAS Solids Production at AADL (lb/d)	827	827
WAS Solids Production at MMADL (lb/d)	1,075	1,075
Aeration Basin Effective Volume (MG)	0.684	0.684
Design SVI (mL/g)	192	151
Design MLSS (mg/L)	2,000	3,000
Total SRT at AADL (days)	13.8	20.7
Total SRT at MMADL (days)	10.6	15.9

Notes:
(1) The Design MLSS scenario corresponds to the MLSS that can be supported with the existing integral secondary clarifiers.
(2) The High MLSS scenario corresponds to a higher MLSS that could be operated if the SVI was consistently at or below 150 mL/g, or if additional secondary clarifier were available.
(3) Loadings correspond to the AADF of 0.5 mgd, with a MMADL PF of 1.3.

The aeration basin would operate closer to extended aeration mode at the higher MLSS of 3,000 mg/L since the predicted SRTs are between 16 and 21 days for MMADL and AADL, respectively. However, note that the secondary clarifiers would not be able to support periods of poor sludge settling with SVI values above 150 mL/g.

Also note that the existing system does not provide redundancy. There is a single aeration basin, and the two secondary clarifiers cannot be isolated. This is an important issue for regular plant maintenance and should be addressed as the facility expands.

5.3.3 Nitrogen Removal Performance Evaluation

The existing activated sludge system's nitrogen removal system was evaluated using dynamic process modeling in the BioWin process simulator. The regular operation mode of the aeration chains was implemented in the model. The aeration basin was then generated with three reactors in series to simulate the different sections of the single aeration basin as defined by the grouping of the aeration chains.

Table 2.16 presents a summary of results under AADL conditions. Under the optimum DO concentration, the system is able to remove nitrogen below the alert level of 8 mg/L TN. However, the system is very sensitive to the DO concentration in the aeration basin, as evidenced by the higher TN concentrations at DO concentrations above 1 mg/L. Therefore, aeration system control is critical, and the system requires low DO concentrations to be able to meet the TN goals.

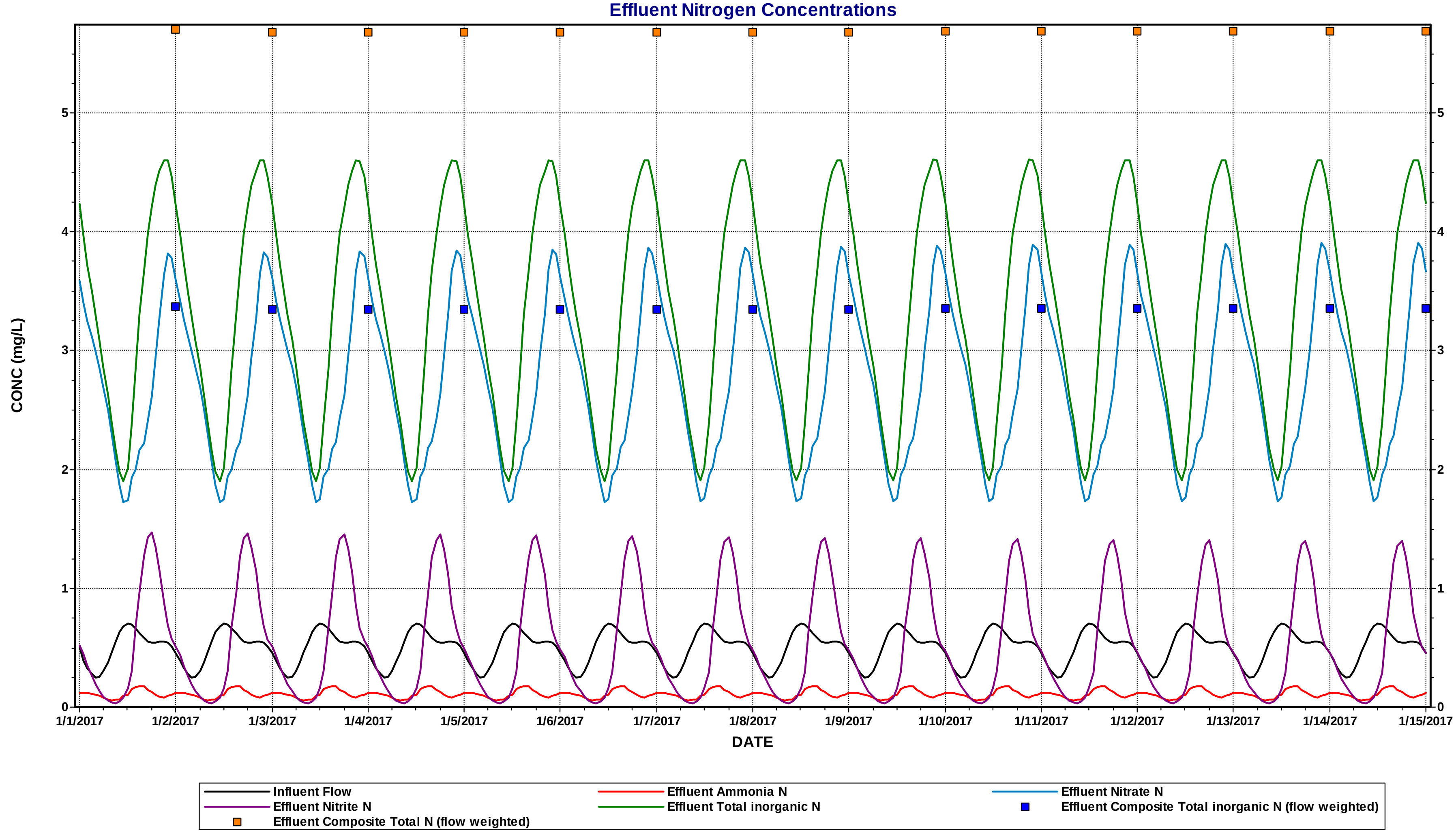
Table 2.16 Nitrogen Removal Evaluation of Existing Aeration Basin
Marana Water Reclamation Facility Master Plan
Town of Marana

Criteria	Scenario 1	Scenario 2	Scenario 3
Average DO in Zone 1 (Chains 6, 7 & 8)	0.3	0.2	0.1
Average DO in Zone 2 (Chains 3, 4 & 5)	1.8	1.2	0.5
Average DO in Zone 3 (Chains 1 & 2)	1.8	2.2	0.8
Average Effluent TN	13.7	9.5	5.7
Average Effluent NH_3-N	0.0	0.0	0.1
Average Effluent NO_3-N	11.3	7.0	2.7
Average Effluent NO_2-N	0.0	0.0	0.6

Note:
(1) Loadings correspond to the AADF of 0.5 mgd.

Figure 2.5 shows the predicted diurnal nitrogen concentrations at the dissolved oxygen concentrations shown in Figure 2.6. A closer examination of the diurnal effluent nitrogen profiles indicates that effluent nitrite concentrations of up to 1.5 mg/L can be expected due to the low DO conditions required for denitrification.

While this is not of concern when using ultraviolet disinfection, nitrite exerts a significant chlorine demand, meaning care should be taken when performing chlorine disinfection. ***In summary, the low DO required to achieve denitrification can also cause incomplete nitrification, which may adversely affect chlorine disinfection and potentially the overall nitrogen removal process.*** As flows continue to increase, the system must be closely monitored and the necessary adjustments made to the aeration cycle times and air flows.



DIURNAL EFFLUENT NITROGEN CONCENTRATIONS

FIGURE 2.5

TOWN OF MARANA

MARANA WATER RECLAMATION FACILITY MASTER PLAN

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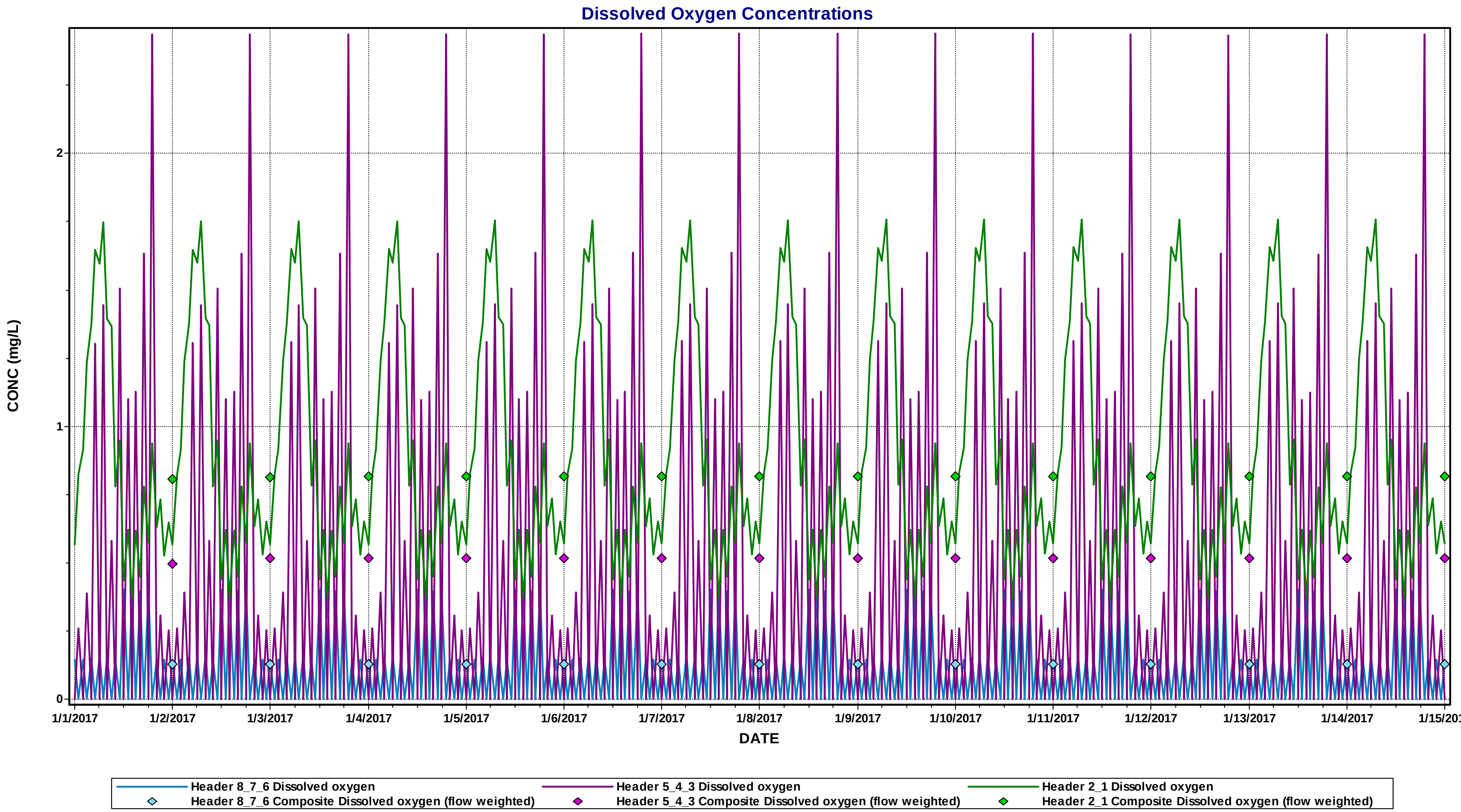

DIURNAL DISSOLVED OXYGEN CONCENTRATIONS

FIGURE 2.6

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

carollo

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5.4 Process Evaluation Summary

The secondary treatment process performs as expected for an extended aeration system. The operating SRT between 20 and 30 days allows influents loadings to be handled relatively well, and the system has been able to produce an effluent TN concentration well below the alert level of 8 mg/L.

As influent flows continue to increase beyond the current flow of 0.355 mgd, the SRT will decrease if the MLSS continues to be operated between 2,000 mg/L and 3,000 mg/L. The process evaluation showed that the system would operate at approximately 11 to 14 days, which is adequate for treatment. However, it is below the typical range for extended aeration and is more representative of conventional activated sludge systems.

Both the size of the existing secondary clarifiers and the historical sludge settleability at the plant (i.e., SVI of 192 mL/g) limit the MLSS concentration at which the aeration basins can be operated to a maximum recommended value of 2,000 mg/L. Operating at higher MLSS at the design flows puts the plant at the risk of sludge blanket overflow and overloading the tertiary filters with suspended solids. The type of extended aeration basins at the Marana WRF (long SRT, low food to microorganism values) tends to favor the growth of filamentous bacteria that result in high SVI values.

Dynamic modeling showed that the system is very sensitive to the DO concentration at the aeration basin and relies on low DO to achieve simultaneous nitrification and denitrification and meet the nitrogen removal goals. As the flows continue to increase, the system will need adjustments to maintain full nitrification (i.e., low effluent ammonia) and adequate denitrification (i.e., effluent nitrates below 5 mg/L) in the system.

The secondary treatment process lacks redundancy because the aeration basin and integral secondary clarifiers effectively act as a single treatment train. Additional process basins are needed as the plant expands, and redundancy should be incorporated in future expansions.

6.0 SOLIDS PRODUCTION

6.1 Headworks Screenings and Grit

The WRF is currently operating within the expected range for screenings and grit volume production.

The Headworks screening equipment currently utilizes a mechanical bar screen with 3-mm openings. For openings greater than 0.5 mm and less than 6 mm, screened materials are classified as fine screenings. These materials include small rags, paper, and plastic materials of various types, grit, undecomposed food waste, feces, grease, and scum.

The Headworks screening equipment produces approximately 8.0 cubic feet of screenings per million gallons of wastewater (ft^3/MG), based on the volume of the storage bin and the frequency at which it is emptied. Average values for fine screenings removed from wastewater range from 5.5 to 11.5 ft^3/MG (WEF, *Manual of Practice No. 8 Design of Municipal Wastewater Treatment Plants*. Fifth Edition. McGraw-Hill. New York. 2010).

Grit consists of sand, gravel, cinders, and other heavy materials that have specific gravities or settling velocities greater than those of organic particles. Wastewater grit quantities will vary greatly based on several factors, including location, sewer system type, industrial waste types, the number of household garbage disposals served, and areas with naturally sandy soils.

Grit is removed downstream of the screening equipment in parallel gravity grit removal channels. By maintaining a constant velocity through the channel with the use of upstream and downstream proportional weirs, these open channels allow sufficient detention time for particles to settle.

The grit removal channels produce approximately 1.6 ft^3/MG, according to WRF Operations staff estimates of grit quantities removed and the frequency. Average grit quantities for separate collection systems range from 0.53 to 5 ft^3/MG (Metcalf & Eddy., George Tchobanoglous, Franklin L. Burton, and H. David Stensel. *Wastewater Engineering: Treatment and Reuse*. 4th ed. Boston: McGraw-Hill, 2003).

6.2 Waste Solids

The Marana WRF sludge production consists of WAS from the secondary treatment process. The WAS stream is separated from the RAS flow by a discharge valve as the RAS stream leaves the secondary clarifiers. WAS flows into a dedicated pump station and is then sent to the aerated sludge storage tank. The sludge is decanted by turning air off and letting the sludge settle before the decanted supernatant (approximately half the volume) is returned to the treatment process to concentrate the solids and reduce the sludge quantities hauled off-site for disposal.

A private contractor currently hauls the sludge from the Marana WRF and disposes of it at the City of Casa Grande WRF. The Town is investigating hauling its solids to the Marana landfill, but the solids would require dewatering to pass the paint filter test required for landfill disposal. A full evaluation of solids handling alternatives is discussed in Technical Memorandum No. 3.

The sludge quantities at both the current and the plant design capacity conditions are summarized in Table 2.17. Under current flows, decanting reduces the WAS volume reduced to approximately half, which requires approximately one truckload per day to haul the sludge off site for disposal. As discussed in Section 5.2, the current average unit WAS production as a function of the plant influent BOD load of 0.87 pound TSS per pound of influent BOD falls within the expected range for municipal wastewater treatment.

Table 2.17 Sludge Production Summary
Marana Water Reclamation Facility Master Plan
Town of Marana

	Current Flow (0.355 mgd AADF)		Design Flow (0.5 mgd AADF)	
Parameter	**Average Conditions**	**Maximum Month**	**Average Conditions**	**Maximum Month**
WAS Solids (lb/d)	551	728	827	1,075
WAS TSS (mg/L)	6,835	6,189	6,500	6,500
WAS Flow (gpd)	9,930	14,158	15,258	19,836
Hauled Sludge (gpd)	5,675	9,533	8,720 (1)	13,356 (2)

Notes:
(1) Based on the current reduction to 57 percent of the sludge volume achieved with decanting, under annual average day conditions.
(2) Based on the current reduction to 67 percent of the sludge volume achieved with decanting, under maximum month average day conditions.

At the design flow of 0.5 mgd, the average quantity of sludge hauled off site is estimated to require more than one truckload per day with the current method of gravity decanting in the sludge storage tank. The estimates shown for the design flow of 0.5 mgd correspond to an operating SRT of approximately 16 to 21 days (at MMADL and AADL, respectively). However, the estimates could be approximately 10 percent higher if the SRT is operated at 11 to 14 days due to the secondary clarifier limitations discussed in Section 5.3.2.2.

7.0 EXISTING FACILITY ELECTRICAL ASSESSMENT

7.1 Main Service

Trico Electric Cooperative provides electric service for the Marana WRF. For electricity, the treatment facility is equipped with:

- A 480-volt, 2,000 amperes, service entrance switchboard (SES) powered from the utility service entrance and a standby generator.
- The SES feeds a 480-volt, 2,000-amperes, auto transfer switch (ATS).
- The ATS feeds two motor control centers, PDC/MCCI and PDC1/MCC, through a bus rated for 480-volt, 2000 amperes.

The maximum demand data recorded through the utility meter for the past 12 months show a peak demand of 197.83 kW recorded in the month of March 2015. NEC Article 220.87 allows the peak demand to be used to calculate the size of an existing service. Based on this calculation, the current load is calculated as 237 amperes. The NEC requires an additional factor of 1.25 to be used to calculate the size of the service. Based on these calculations, the existing 2,000 amperes service has 1,703 amperes available for plant expansion.

7.2 Electrical Gear

During the site visit, the exterior of the electrical equipment was inspected and appeared to be in good condition. Several pieces of electrical equipment have nameplates identifying a date of manufacture within the last 5 to 7 years. Since electrical equipment typically has a useful life of 15 to 20 years, the site's electrical equipment has at least 10 years until it requires replacement.

The dates of manufacture that were found for equipment are the following:

- Switchboard SES was manufactured in August, 2008.
- PDC/MCCI was manufactured in July, 2008.
- PDC1/MCC was manufactured in July, 2010.

While the exterior of the electrical equipment was inspected, the electrical equipment was energized making inspection of the interior of the equipment unsafe. Thus, a periodic visual inspection of the electrical equipment is recommended to view the condition of the internal components of the equipment. In addition, we recommend a thermographic study to ensure the cables are properly torqued at their associated lugs.

7.3 Arc Flash

The majority of the electrical equipment on site has general "Arc Flash Warning" stickers that meet the requirements of NEC Article 110.16. However, additional arc flash analysis may be required to comply with Occupational Safety and Health Organization (OSHA) and National Fire Protection Agency (NFPA) 70E.

A complete electrical system study composed of a fault current analysis, protective device coordination analysis, and an arc flash hazard analysis would determine the arc flash hazard at all locations within the electrical distribution system, protective device settings, and available fault current levels. With this study, it would be possible to define the hazard level, the approach boundary, and the required personnel protective equipment (PPE) to work with the equipment. It is unknown if a complete electrical system study was completed as part of the last plant modification project (*SE Pump Station, Filtration and UV Addition*, 2009). At the time of this report, the Town is soliciting a cost proposal from Sabino Electric for a complete electrical system study.

7.4 Energy Assessment Tool

Using EPA's Energy Assessment Tool, an evaluation was performed to identify the systems at the WRF that use the most energy and thus contribute the most to electric costs. The EPA created the tool to assess a water or wastewater system baseline energy consumption and costs, to identify areas for improved energy efficiency and operational savings. The tool is a free, downloadable spreadsheet-based tool. Though it's useful, the tool was not intended to be a full-scale energy audit, nor can it identify or predict the efficiency of specific equipment.

Approximately one years' worth of electrical billing data was input into the spreadsheet, including monthly total demand in kWh and total charges. In addition, the electrical equipment loads were input by process area, including horsepower or demand, motor efficiencies, and estimated average run times.

Table 2.18 summarizes the findings.

Table 2.18 Summary of EPA Energy Assessment Tool
Marana Water Reclamation Facility Master Plan
Town of Marana

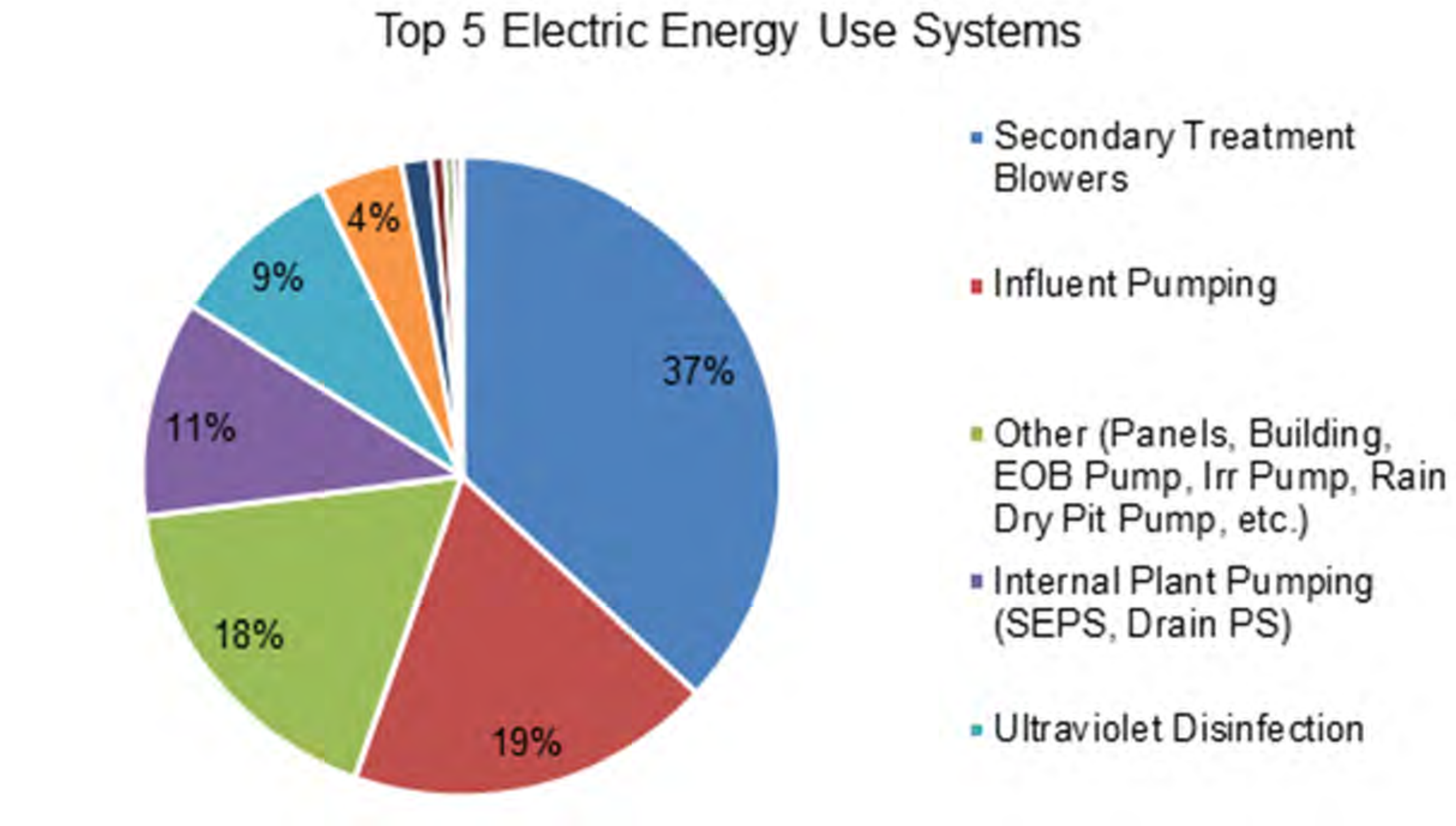
Description	Value
Billing Data Used	Oct 2014 - Sept 2015
Average Monthly Electric Charge	$10,443
Average Monthly Electrical use	89,900 kWh
Average Monthly Volume WW Treated	10.54 MG
WRF Average Energy Use Per MG of WW Treated	8,530 kWh/MG
WRF Average Energy Cost Per MG of WW Treated	$990.93/MG
Highest Electrical Consuming Process	Secondary Treatment (Blowers)

The values above are a baseline for comparing future plant operational data.

Figure 2.7 details the processes that consume the greatest amount of electricity and their contribution to the WRF's total energy use. Note that the highest electricity consuming items are the aeration blowers of the secondary treatment process: 37 percent of the total use. The influent pumping and internal plant process pumping (secondary effluent pump station and drain pump stations) accounted for the second and fourth highest use.

At the facility, aeration blowers and pumping account for a total of 66 percent of the electrical use. Miscellaneous loads, such as air conditioning of panel boards, the main control building use, and small pumps, were combined into one category and account for up to 18 percent of the total usage.

The summary report from the EPA Assessment Tool is found in Appendix B.



HIGHEST ELECTRICAL USE SYSTEMS

FIGURE 2.7

TOWN OF MARANA
MARANA WATER RECLAMATION FACILITY MASTER PLAN

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8.0 STANDBY POWER EVALUATION

The equipment list provided by the Town and data collected during the site visit were used to evaluate the existing generator capacity. Table 2.19 details the loads that are connected to the existing generator.

Table 2.19 Standby Generator Electrical Loads
Marana Water Reclamation Facility Master Plan
Town of Marana

Equipment Description	Motor Size (hp)
Automatic Bar Screen	3
Micro Strainer	2
Compactor	3
IPS Pump 1	10
IPS Pump 2	10
Mixing Pump	18
Odor Control Blower	3
Blower #1	40
Blower #2	40
Blower #3	40
Clarifier #1 Motor	1
Was Pump	5
Sludge Blower Pump	5
Sludge Loading Pump	2.3
Decant Pump	0.5
Loading Station Air Compressor	5
Pump 01	10
Air Compressor #1	20
Air Compressor #2	20
Utility Pump A	3
Utility Pump B	3
Irrigation Pump	10
Drain Pump #1	10
EOB Pump	0.5
Total Horsepower	**263.8**

The capacity of the standby generator is 300 kW. Total calculated operational load at the WRF is approximately 264 hp (197 kW). With the current operating loads, the generator is loaded at approximately 66 percent of its full load capacity.

This evaluation assumes the loads are stepped onto the generator in a sequence that will not allow all loads to start in a single step. Starting all loads in a single step may overload the generator.

Additional loads may be added to the generator in the future; however, further analysis would be needed to determine (1) the amount of load which could be added, (2) the order in which the loads would be "stepped" onto the generator, and (3) the minimum number of steps the generator could handle.

9.0 CONCLUSIONS AND RECOMMENDATIONS

A summary of findings and recommendations from the process treatment and hydraulic evaluations are as follows:

1. The secondary treatment process is performing as expected for an extended aeration system. The operating SRT between 20 and 30 days provides effective handling of the influents loadings, and the system has been able to produce an effluent TN concentration well below the alert level of 8 mg/L.
2. As influent flows continue to increase beyond the current flow of 0.355 mgd, the SRT will decrease if the MLSS continues to be operated between 2,000 mg/L and 3,000 mg/L. The process evaluation showed that the system would operate at approximately 11 to 14 days. This is adequate for treatment but below the typical range for extended aeration and is more representative of conventional activated sludge systems.
3. **The size of the existing secondary clarifiers and the historical sludge settleability at the plant (i.e., SVI of 192 mL/g) limit the MLSS at which the aeration basins can be operated, to a maximum recommended value of 2,000 mg/L.** Operating at higher MLSS at the design flows puts the plant at the risk of sludge blanket overflow and overloading the tertiary filters with suspended solids. The type of extended aeration basins at the Marana WRF (long SRT, low food to microorganism values) tends to favor the growth of filamentous bacteria that lead to high SVI values.
4. Dynamic modeling results showed that the system is very sensitive to the DO concentration at the aeration basin and relies on low DO to achieve simultaneous nitrification and denitrification and meet the nitrogen removal goals. As the flows continue to increase, the system must be adjusted to maintain full nitrification (i.e., low effluent ammonia) and adequate denitrification (i.e., effluent nitrates below 5 mg/L).

5. The secondary treatment process lacks redundancy. The aeration basin and integral secondary clarifiers effectively act as a single treatment train. Additional process basins will be needed as the plant expands, and redundancy should be incorporated in future expansions.

6. Hydraulic modeling of the treatment plant process flow path indicates that the existing systems can hydraulically pass the peak hour flow of the design capacity of the Biolac® system (0.5 mgd).

7. The Headworks cannot convey flows higher than a peak flow rate of 1.5 mgd (1,040 gpm). Flows through the 8-inch pipe between the Headworks and Influent Pump Station would be near 7 fps, creating sufficient friction losses to back up incoming flows.

8. The Parshall flume is within 0.10 foot of overtopping its channel at the 1.5 mgd peak flow rate. The Headworks facility should be replaced in the next plant expansion project.

APPENDIX A – *HYDRAULIX*® HYDRAULIC MODELING SCENARIOS

PROJECT : Town of Marana WRF Master Plan

JOB # : 10067A00 **REVISION:** **CHECKED :** **DATE :** **BY :** WFF **DATE :** 12/8/2015

	Equation Ref.	HGL	EGL

DOWNSTREAM CONTROL

EGL =	1921.00		100-Yr Flood Elev, FEMA		1921.00	1921.00
Flow =	0.355 mgd =	0.55 cfs				

OUTFALL STRUCTURE
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 0.355 mgd = 0.5 cfs

WSE Downstream of Weir 1921.00 ft
Weir Crest Elevation 1916.00 ft
Downstream head, Hd 5.00 ft
Length of Weir, L 9.00 ft

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }
Head on Weir, H NA ft
Upstream WSE NA ft

Submerged Weir Computation { 7 }
K 0.00
M 11.18
Increment 0.10 ft
Upstream Head, Hu1 5.00 ft
F(H1) 0.00
F'(H1) -0.30
Upstream Head, Hu2 5.00 ft
Upstream WSE 1921.00 ft

Head over Weir *5.00 ft*

Condition Upstream of Weir 1921.00 1921.00

OUTFALL PIPE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 0.355 mgd = 0.5 cfs

Pipe Diameter, D 30 inch
Pipe Length, L 219 ft
Absolute Roughness, ε 0.00010 ft
Pipe velocity, v 0.11 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 27970
Friction factor, f 0.0240 Equivalent Hazen-Williams "C" = 142.8232

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q (Average Day) 0.355 mgd = 0.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	0.36	0.55	0.50	----	30	----	0.11	0.00	0.00
1	Outlet Loss - Still Water	0.36	0.55	1.00	30	----	0.11	----	0.00	0.00
									Sum =	0.00

Total Energy Loss = 0.00 ft

Upstream Condition 1921.00 1921.00

WEIR IN OUTFALL/DECHLOR MANHOLE NO. 2
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 0.355 mgd = 0.5 cfs

WSE Downstream of Weir 1921.00 ft
Weir Crest Elevation 1920.49 ft
Downstream head, Hd 0.51 ft
Length of Weir, L 5.00 ft

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }
Head on Weir, H NA ft
Upstream WSE NA ft

Submerged Weir Computation { 7 }
K 0.00
M 0.36
Increment 0.10 ft
Upstream Head, Hu1 0.51 ft
F(H1) 0.00

		Equation Ref.	HGL	EGL
F'(H1)	-2.94			
Upstream Head, Hu2	0.51 ft			
Upstream WSE	1921.00 ft			
Head over Weir	*0.51 ft*			
	Condition Upstream of Weir		1921.00	1921.00

PIPE FROM UV SYSTEM TO DECHLOR MANHOLE NO. 2
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.355 mgd =	0.5 cfs
Pipe Diameter, D	24 inch	
Pipe Length, L	113 ft	
Absolute Roughness, ε	0.00010 ft	
Pipe velocity, v	0.17 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	34962	
Friction factor, f	0.0228	Equivalent Hazen-Williams "C" = 144.3729
Friction Energy Loss, h_L	0.00 ft	

MINOR PIPE LOSS HEADING

Flow, Q 0.355 mgd = 0.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	45 º Bend - Regular Fl.	0.36	0.55	0.23	24	----	0.17	----	0.00	0.00
1	Entrance Loss - Flush	0.36	0.55	0.50	----	24	----	0.17	0.00	0.00
1	Outlet Loss - Still Water	0.36	0.55	1.00	24	----	0.17	----	0.00	0.00
									Sum =	0.00

Total Energy Loss = 0.00 ft

	HGL	EGL
Upstream Condition	1921.00	1921.00

UV SYSTEM DISCHARGE GATE
[SUBMERDED GATE - RECTANGULAR OPENING] { 14 }

Flow, Q	0.355 mgd =	0.5 cfs
Gate Width	2 ft	
Full Height of Opening	2 ft	
Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	0.14 fps	
Energy Loss thru Gate, h_L	0.00 ft	

	HGL	EGL
Condition Upstream of Gate	1921.00	1921.00

UV CHANNEL DOWNSTREAM CONTROL WEIR - MODULATES SO THAT WATER SURFACE DOESN'T VARY MORE THAN 3 INCHES
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	0.355 mgd =	0.5 cfs
WSE Downstream of Weir	1921.00 ft	
Weir Crest Elevation	1924.01 ft	
Downstream head, Hd	-3.01 ft	
Length of Weir, L	6.83 ft	

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }

Head on Weir, H	0.08 ft
Upstream WSE	1924.09 ft

Submerged Weir Computation { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft
Head over Weir	*0.08*	*ft*

	HGL	EGL
Condition Upstream of Weir	1924.09	1924.09

UV CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

			Equation Ref.	HGL	EGL
Flow, Q	0.355 mgd =	0.5 cfs			
Channel Width	3.00 ft				
Total Channel Length	42.83				
Downstream Invert El	1921.47				
Channel Slope	0.10%				
Manning Coeff, n	0.013				

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1921.47	1921.47	2.62	0.070	0.95	0.000	----	----	1924.09	1924.09
8.6	1921.48	1921.47	2.61	0.070	0.95	0.000	0.000	0.00	1924.09	1924.09
17.1	1921.49	1921.48	2.61	0.070	0.95	0.000	0.000	0.00	1924.09	1924.09
25.7	1921.50	1921.49	2.60	0.070	0.95	0.000	0.000	0.00	1924.09	1924.09
34.3	1921.50	1921.50	2.59	0.071	0.95	0.000	0.000	0.00	1924.09	1924.09
42.8	1921.51	1921.50	2.58	0.071	0.95	0.000	0.000	0.00	1924.09	1924.09

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel 1924.09 1924.09

UV SYSTEM
MINOR CHANNEL LOSS HEADING

Flow, Q 0.355 mgd = 0.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance - Sharp Corners	0.355	0.55	0.50	3	3	5	0.04	0.04	0.00	0.00
	UV SYSTEM - 3 BANKS	**0.355**			**SYSTEM VARIES 2 "**						**0.167**
1	Outlet - Sharp Corners	0.355	0.55	1.00	3	3	5	0.04	0.04	0.00	0.00
										Sum =	0.167

Total Energy Loss = 0.17 ft

Upstream Condition 1924.26 1924.26

ENTRANCE TO UV CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	0.36 mgd =	0.5 cfs
Channel Width	3.75 ft	
Total Channel Length	10.00	
Downstream Invert El	1920.00	
Channel Slope	0.10%	
Manning Coeff, n	0.013	

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1920.00	1920.00	4.26	0.034	1.30	0.000	----	----	1924.26	1924.26
2.0	1920.00	1920.00	4.26	0.034	1.30	0.000	0.000	0.00	1924.26	1924.26
4.0	1920.00	1920.00	4.26	0.034	1.30	0.000	0.000	0.00	1924.26	1924.26
6.0	1920.01	1920.00	4.25	0.034	1.30	0.000	0.000	0.00	1924.26	1924.26
8.0	1920.01	1920.01	4.25	0.034	1.30	0.000	0.000	0.00	1924.26	1924.26
10.0	1920.01	1920.01	4.25	0.034	1.30	0.000	0.000	0.00	1924.26	1924.26

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel 1924.26 1924.26

ENTRANCE TO UV CHANNEL
MINOR CHANNEL LOSS HEADING

Flow, Q 0.4 mgd = 0.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Sudden Expansion	0.36	0.55	1.00	3.75	7.10	6.5	0.02	0.01	0.00	0.00
										Sum =	5.7E-06

Total Energy Loss = 0.00 ft

Upstream Condition 1924.26 1924.26

FILTER EFFLUENT CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

			Equation Ref.	HGL	EGL
Flow, Q	0.36 mgd =	0.5 cfs			
Channel Width	1.50 ft				
Total Channel Length	80.00				
Downstream Invert El	1923.28				
Channel Slope	0.00%				
Manning Coeff, n	0.013				

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1923.28	1923.28	0.98	0.374	0.42	0.000	----	----	1924.26	1924.26
16.0	1923.28	1923.28	0.98	0.374	0.42	0.000	0.000	0.00	1924.26	1924.26
32.0	1923.28	1923.28	0.98	0.374	0.42	0.000	0.000	0.00	1924.26	1924.26
48.0	1923.28	1923.28	0.98	0.374	0.42	0.000	0.000	0.00	1924.26	1924.26
64.0	1923.28	1923.28	0.98	0.374	0.42	0.000	0.000	0.00	1924.26	1924.26
80.0	1923.28	1923.28	0.98	0.374	0.42	0.000	0.000	0.00	1924.26	1924.26

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel — HGL 1924.26 — EGL 1924.26

FILTER EFFLUENT CHANNEL
MINOR CHANNEL LOSS HEADING

Flow, Q 0.355 mgd = 0.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	90 Degree Bend - 0º Radius	0.355	0.55	1.30	1.5	----	4.47	0.08	----	0.00	0.00
										Sum =	0.00027

Total Energy Loss = 0.00 ft

Upstream Condition — HGL 1924.26 — EGL 1924.26

FROM FILTER EFFLUENT CHANNEL TO SAND BED FILTERS 1-3

Use tables below for flow through each sand filter bed
Total filter effluent flow = **0.36 mgd**

SFB	Is filter online (enter 0 or 1)	Weir Length, ft	Flow fraction	Flow, MGD
# 1	1	14.70	1.0000	0.36
# 2	0	14.70	0.0000	0.00
# 3	0	14.70	0.0000	0.00

Filter #1 effluent Weir

[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	0.355 mgd =	0.5 cfs
WSE Downstream of Weir	1924.26 ft	
Weir Crest Elevation	1926.63 ft	
Downstream head, Hd	-2.37 ft	
Length of Weir, L	14.70 ft	

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation — { 6 }
Head on Weir, H 0.05 ft
Upstream WSE 1926.68 ft

Submerged Weir Computation — { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.05 ft*

Condition Upstream of Weir — HGL 1926.68 — EGL 1926.68

Dynasand Filter Headloss

12/3/15 Per Jason Vernon design headloss 30-36"
12/3/15 Per Jason Vernon actual headloss 18-20"

Headloss through filter cell #1 = 20 inch
Headloss through filter cell #1 = **1.67 ft**

Upstream Condition in Filter — HGL 1928.35 — EGL 1928.35

18" Pipe connecting the Dynasand to filter influent channel

PROJECT : Town of Marana WRF Master Plan

Hydraulix BY carollo

JOB # : 10067A00	**REVISION:**	**CHECKED :**	**BY :** WFF
		DATE :	**DATE :** 12/8/2015

Equation Ref. | HGL | EGL

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.505 mgd =	0.8 cfs	Influent Flow=	0.355 mgd
			Filtrate Flow=	0.15 mgd
Pipe Diameter, D	18 inch			0.505 mgd
Pipe Length, L	2.67 ft			
Absolute Roughness, ε	0.00015 ft			
Pipe velocity, v	0.44 fps			
Kinematic Viscosity	1.000E-05 ft^2/sec			
Reynold's Number, R	66313			
Friction factor, f	0.0200	Equivalent Hazen-Williams "C" = 147.30		
Friction Energy Loss, h_L	0.00 ft			

MINOR PIPE LOSS HEADING

Flow, Q 0.505 mgd = 0.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Gate Valve (Open)	0.51	0.78	0.19	18	----	0.44	----	0.00	0.00
1	Entrance Loss - Flush	0.51	0.78	0.50	----	18	----	0.44	0.00	0.00
									Sum =	0.00

Total Energy Loss = *0.00* *ft*

Upstream Condition | 1928.35 | 1928.35

FILTER INFLUENT CHANNEL

[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	0.505 mgd =	0.8 cfs	Influent Flow=	0.355 mgd
Channel Width	2.00 ft		Filtrate Flow=	0.15 mgd
Total Channel Length	55.00			0.505 mgd
Downstream Invert El	1924.00			
Channel Slope	0.00%			
Manning Coeff, n	0.013			

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1924.00	1924.00	4.35	0.090	0.81	0.000	----	----	1928.35	1928.35
11.0	1924.00	1924.00	4.35	0.090	0.81	0.000	0.000	0.00	1928.35	1928.35
22.0	1924.00	1924.00	4.35	0.090	0.81	0.000	0.000	0.00	1928.35	1928.35
33.0	1924.00	1924.00	4.35	0.090	0.81	0.000	0.000	0.00	1928.35	1928.35
44.0	1924.00	1924.00	4.35	0.090	0.81	0.000	0.000	0.00	1928.35	1928.35
55.0	1924.00	1924.00	4.35	0.090	0.81	0.000	0.000	0.00	1928.35	1928.35

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel | 1928.35 | 1928.35

SE Pump Design Criteria

Secondary Effluent PS System Headloss

Pump TDH at minimum water level =	**23.00 ft**	Min water level	1908.85		**Flow (mgd)**	**TDH (feet)**
Pump TDH at maximum water level =	**26.00 ft**			Pump design	1000-2900	33
				Initial pump design	200 to 1050	23 to 26

Upstream Condition | 1908.85 | 1931.85

New SE Biolac line to the PS

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.505 mgd =	0.8 cfs	Influent Flow=	0.355 mgd
			Filtrate Flow=	0.15 mgd
Pipe Diameter, D	12 inch			0.505 mgd
Pipe Length, L	66 ft			
Absolute Roughness, ε	0.00010 ft			
Pipe velocity, v	0.99 fps			
Kinematic Viscosity	1.000E-05 ft^2/sec			
Reynold's Number, R	99470			
Friction factor, f	0.0185	Equivalent Hazen-Williams "C" = 149.22		
Friction Energy Loss, h_L	0.02 ft			

MINOR PIPE LOSS HEADING

Flow, Q 0.5 mgd = 0.8 cfs

Dia | Dia | Vel | Vel | Vel | Minor

Equation Ref.	HGL	EGL

No.	Description	Flow (mgd)	Flow (cfs)	K	Up (in)	Down (in)	Up (fps)	Down (fps)	Head (ft)	Loss (ft)
1	Entrance Loss - Flush	0.51	0.78	0.50	----	12	----	0.99	0.02	0.01
1	Entrance Loss - Pipe Ext.	0.51	0.78	1.00	----	12	----	0.99	0.02	0.02
1	Plug Valve (Open)	0.51	0.78	0.77	12	----	0.99	----	0.02	0.01
									Sum =	0.03

Total Energy Loss = 0.05 ft

Upstream Condition 1919.05 1919.05

Exisiting SE Biolac line to the final clarifier #2 discharge line

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.505 mgd =	0.8 cfs
Pipe Diameter, D	12 inch	
Pipe Length, L	50 ft	
Absolute Roughness, ε	0.00010 ft	
Pipe velocity, v	0.99 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	99470	
Friction factor, f	0.0185	Equivalent Hazen-Williams "C" = 149.22
Friction Energy Loss, h_L	0.01 ft	

Influent Flow= 0.355 mgd
Filtrate Flow= 0.15 mgd
0.505 mgd

MINOR PIPE LOSS HEADING

Flow, Q 0.505 mgd = 0.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Plug Valve (Open)	0.51	0.78	0.77	12	----	0.99	----	0.02	0.01
1	Wye - Thru Straight Run	0.51	0.78	0.45	12	----	0.99	----	0.02	0.01
									Sum =	0.02

Total Energy Loss = 0.03 ft

Upstream Condition 1919.09 1919.09

8" SE Biolac line to the final clarifier #2 discharge line

Asumme that 1/4 of the total flow goes through this 8" line

total flow 0.355 mgd

1/4 Influent Flow= 0.08875 mgd 0.25
1/4 Filtrate Flow= 0.0375 mgd 0.25
0.12625 mgd

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)]

Flow	0.126 mgd =	0.2 cfs
Pipe Diameter, D	8 inch	
Pipe Length, L	9 ft	
Absolute Roughness, ε	0.00015 ft	
Pipe velocity, v	0.56 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	37301	
Friction factor, f	0.0230	Equivalent Hazen-Williams "C" = 144.25
Friction Energy Loss, h_L	0.00 ft	

MINOR PIPE LOSS HEADING

Flow, Q 0.126 mgd = 0.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	45 º Bend - Regular Fl.	0.13	0.20	0.23	8	----	0.56	----	0.00	0.00
1	Wye - Thru Side Outlet	0.13	0.20	1.35	8	----	0.56	----	0.00	0.01
2	22.5 º Bend	0.13	0.20	0.15	8	----	0.56	----	0.00	0.00
1	Butterfly Valve (Open)	0.13	0.20	0.50	8	----	0.56	----	0.00	0.00
1	Reducer	0.13	0.20	0.25	8	12	0.56	0.25	0.00	0.00
									Sum =	0.01

Total Energy Loss = 0.01 ft

Upstream Condition 1919.10 1919.10

Final clarifier #2 8" Biolac (PVC) discharge line connection to floating weir
Assume only 1/4 of the flow through the 8" pipe connected to the floating weir

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.126 mgd =	0.2 cfs	1/4 Influent Flow=	0.08875 mgd	0.25
			1/4 Filtrate Flow=	0.0375 mgd	0.25
Pipe Diameter, D	8 inch			0.12625 mgd	
Pipe Length, L	4 ft				
Absolute Roughness, ε	0.00010 ft				
Pipe velocity, v	0.56 fps				
Kinematic Viscosity	1.000E-05 ft^2/sec				
Reynold's Number, R	37301				
Friction factor, f	0.0228	Equivalent Hazen-Williams "C" =	145.01		
Friction Energy Loss, h_L	0.00 ft				

MINOR PIPE LOSS HEADING

Flow, Q 0.126 mgd = 0.2 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Butterfly Valve (Open)	0.126	0.20	0.50	8	----	0.56	----	0.00	0.00
1	Entrance Loss - Flush	0.126	0.20	0.50	----	8	----	0.56	0.00	0.00
2	Mitre Bend - 22.5 º Deflection	0.126	0.20	0.15	8	----	0.56	----	0.00	0.00
									Sum =	0.00

Total Energy Loss = 0.00 ft

	HGL	EGL
Upstream Condition	1919.10	1919.10

Final Clarifier #2 floating weir

Assume only 1/2 of the flow through the 8" floating weir

Assume floating weir is a weigted pipe

Min water level 1922.00 ft
Max water level 1923.00 ft

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.253 mgd =	0.4 cfs	1/2 Influent Flow=	0.1775 mgd	0.5
			1/2 Filtrate Flow=	0.075 mgd	0.5
Pipe Diameter, D	8 inch			0.2525 mgd	
Pipe Length, L	21 ft				
Absolute Roughness, ε	0.00015 ft				
Pipe velocity, v	1.12 fps				
Kinematic Viscosity	1.000E-05 ft^2/sec				
Reynold's Number, R	74602				
Friction factor, f	0.0201	Equivalent Hazen-Williams "C" =	146.6205		
Friction Energy Loss, h_L	0.01 ft				

MINOR PIPE LOSS HEADING

Flow, Q 0.253 mgd = 0.4 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Butterfly Valve (Open)	0.25	0.39	0.50	8	----	1.12	----	0.02	0.01
1	Entrance Loss - Flush	0.25	0.39	0.50	----	8	----	1.12	0.02	0.01
1	90 º Elbow - Regular Fl.	0.25	0.39	0.30	8	----	1.12	----	0.02	0.01
									Sum =	0.03

Total Energy Loss = *0.04* *ft*

	HGL	EGL
Upstream Condition	1922.04	1922.04

Final Clarifier #2 to Biolac System Influent

Biolac min water level 1922.00 ft
Biolac max water level 1923.00 ft

Biolac Channel System Headloss

Headloss through biolac channel = 0.001 ft . Channels normally have minimum headloss

	HGL	EGL
Condition at Upstream End of Channel	1922.04	1922.04

16-inch pipeline from Biolac to Splitter Box

Ras flow	0.355	Per Carlos L assume 100% RAS flow returns to splitter box
Inf flow	0.355	
total flow	**0.710**	

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.710 mgd =	1.1 cfs	
Pipe Diameter, D	16 inch		
Pipe Length, L	140 ft		assumed worst case flow enters biolac system on the east side
Absolute Roughness, ε	0.00150 ft		
Pipe velocity, v	0.79 fps		

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			Equation Ref.	HGL	EGL
Kinematic Viscosity	1.000E-05 ft^2/sec				
Reynold's Number, R	104887				
Friction factor, f	0.0225	Equivalent Hazen-Williams "C" = 133.5013			
Friction Energy Loss, h_L	0.02 ft				

MINOR PIPE LOSS HEADING

Flow, Q 0.710 mgd = 1.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	0.71	1.10	0.50	----	16	----	0.79	0.01	0.00
1	Butterfly Valve (Open)	0.71	1.10	0.50	16	----	0.79	----	0.01	0.00
									Sum =	0.01

Total Energy Loss = 0.03 ft

	HGL	EGL
Upstream Condition	1922.07	1922.07

Isolation gate at splitter box

[SUBMERDED GATE - CIRCULAR OPENING] { 15 }

Flow, Q	0.710 mgd =	1.1 cfs
Diameter of Opening	1.33 ft	
Sluice Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	0.79 fps	
Energy Loss thru Gate, h_L	0.03 ft	

	HGL	EGL
Condition Upstream of Gate	1922.10	1922.10

Interim PS System Headloss

Pump TDH at minimum water level = **18.00 ft**
Pump TDH at maximum water level = **37.00 ft**

Max WSE in Splitter Box	1923.00
Assumed min water level	1905.50
Overflow water level	1913.13
Assumed normal water level	1912.50

Interim PS - 4.0 ADF

	Flow (mgd)	TDH (feet)
Pumping capacity, ea	1555.00	37
Min pumping capacity, ea	200.00	18
25 Hp, ea pump		

	HGL	EGL
Upstream Condition	1912.50	1930.50

8-INCH PIPE FROM PRIMARY INFLUENT PS TO MECHANICAL SCREEN

8-inch Pipeline

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.355 mgd =	0.5 cfs
Pipe Diameter, D	8 inch	
Pipe Length, L	21 ft	
Absolute Roughness, ε	0.00015 ft	
Pipe velocity, v	1.57 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	104887	
Friction factor, f	0.0190	Equivalent Hazen-Williams "C" = 147.183
Friction Energy Loss, h_L	0.02 ft	

MINOR PIPE LOSS HEADING

Flow, Q 0.355 mgd = 0.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	45 º Bend - Regular Fl.	0.36	0.55	0.23	8	----	1.57	----	0.04	0.01
1	90 º Elbow - Regular Fl.	0.36	0.55	0.30	8	----	1.57	----	0.04	0.01
1	Entrance Loss - Flush	0.36	0.55	0.50	----	8	----	1.57	0.04	0.02
1	Wye - Thru Side Outlet	0.36	0.55	1.35	8	----	1.57	----	0.04	0.05
									Sum =	0.09

Total Energy Loss = 0.11 ft

	HGL	EGL
Upstream condition	1912.61	1912.61

Grit dishcarge channel

Minimum headloss in channel 1'-7" wide by 4'-6" deep
Since 8" pipe is full pipe, assume WS elev is above the pipe crown
crown = 1913.25

	Equation Ref.	HGL	EGL
Upstream condition		1913.25	1913.25

Specifications Sections Linear Proportional Weir (Sutro Weir) Plate at the end of grit channel

Assume all flow goes through one channel

$Q=C_dK(\pi/2)*(\text{sqrt}(2gH)$

H= water surface elev
H= 0.0457 inches
H= 0.55 feet

Q= 0.355 mgd
Q= 0.549 ft^3/sec
Cd= 0.62
g= 32.2 ft/sec

K=2x(sqrt y)
x= 3.07 inches
y= 3.00 inches
K= 9.21 inches
0.77 feet

	HGL	EGL
Upstream condition	1913.80	1913.80

Grit channel

Assume all flow goes through one channel

[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	0.36 mgd =	0.5 cfs
Channel Width	1.00 ft	
Total Channel Length	33.00	
Downstream Invert El	1913.00	
Channel Slope	1.50%	slope= (1913.50-1913.00/33)
Manning Coeff, n	0.013	

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1913.00	1913.00	0.79	0.695	0.31	0.000	----	----	1913.79	1913.80
6.6	1913.10	1913.00	0.69	0.795	0.29	0.000	0.000	0.00	1913.79	1913.80
13.2	1913.20	1913.10	0.59	0.931	0.27	0.000	0.000	0.00	1913.79	1913.80
19.8	1913.30	1913.20	0.49	1.125	0.25	0.001	0.000	0.00	1913.79	1913.80
26.4	1913.40	1913.30	0.38	1.435	0.22	0.001	0.001	0.01	1913.78	1913.81
33.0	1913.50	1913.40	0.26	2.083	0.17	0.003	0.002	0.02	1913.76	1913.83

TOTAL ENERGY LOSS *0.03 ft*

	HGL	EGL
Condition at Upstream End of Channel	1913.83	1913.83

Mechanical Screen System Headloss

Headloss through screen = 0.50 ft

Assumed headloss through mechanical screen = .25-.75 ft

	HGL	EGL
Condition at Upstream End of Channel	1914.33	1914.33

FROM MECHANICAL SCREEN TO PARSHALL FLUME

PARSHALL FLUME { 13 }

Flow, Q =	0.355 mgd	
	0.5 cfs	(0.3 < Q < 100)
Downstream WSE =	1914.33 ft	
Downstream EGL =	1914.33 ft	
Throat width =	0.25 ft	(available sizes = 1, 2, 3, 3.5, 4, 5, 6, 7, and 8) (W-2 Contract Drawings)
Flume invert elevation =	1915.11 ft	
Upstream channel width =	1.25 ft	
Downstream depth, Hb =	-0.78 ft	
Upstream depth, Ha =	0.66 ft	
Upstream velocity =	0.66 fps	
Submergence =	**-117.9** %	
Headloss =	1.456 ft	** NOT USED **

	HGL	EGL
WSE Upstream of Flume	1915.77	1915.78

PIPE FROM PARSHALL FLUME TO GLADDIN MH #2

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Q_{full} = 1.27 using manning eqn

Flow	1.270 mgd =	2.0 cfs
Pipe Diameter, D	12 inch	
Pipe Length, L	51 ft	
Absolute Roughness, ε	0.00015 ft	
Pipe velocity, v	2.50 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	250152	
Friction factor, f	0.0162	Equivalent Hazen-Williams "C" = 148.713

Friction Energy Loss, h_L 0.08 ft

	Equation Ref.	HGL	EGL

MINOR PIPE LOSS HEADING

Flow, Q 1.27 mgd = 2.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 º Elbow - Regular Fl.	1.27	1.96	0.30	12	----	2.50	----	0.10	0.03
1	Entrance Loss - Flush	1.27	1.96	0.50	----	12	----	2.50	0.10	0.05
									Sum =	0.08

Total Energy Loss = 0.16 ft

Upstream Condition	HGL	EGL
Upstream Condition	1915.94	1915.94

PIPE FROM GLADDIN MH # 2 to GLADDIN MH #1

Q_{full} = 1.31 using manning eqn

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 1.310 mgd = 2.0 cfs

Pipe Diameter, D	12	inch
Pipe Length, L	388	ft
Absolute Roughness, ε	0.00150	ft
Pipe velocity, v	2.58	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	258031	
Friction factor, f	0.0226	

Equivalent Hazen-Williams "C" = 124.149

Friction Energy Loss, h_L 0.91 ft

MINOR PIPE LOSS HEADING

Flow, Q 1.310 mgd = 2.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	1.31	2.03	0.50	----	12	----	2.58	0.10	0.05
1	Entrance Loss - Pipe Ext.	1.31	2.03	1.00	----	12	----	2.58	0.10	0.10
									Sum =	0.16

Total Energy Loss = 1.06 ft

Upstream Condition	HGL	EGL
Upstream Condition	1917.00	1917.00

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	Equation Ref.	HGL	EGL

DOWNSTREAM CONTROL

EGL =	1921.00		100-Yr Flood Elev, FEMA		1921.00	1921.00
Flow =	0.500 mgd =	0.55 cfs				

OUTFALL STRUCTURE
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	0.500	mgd =	0.8	cfs
WSE Downstream of Weir	1921.00	ft		
Weir Crest Elevation	1916.00	ft		
Downstream head, Hd	5.00	ft		
Length of Weir, L	9.00	ft		

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation { 7 }

K	0.00	
M	11.18	
Increment	0.10	ft
Upstream Head, Hu1	5.00	ft
F(H1)	0.00	
F'(H1)	-0.30	
Upstream Head, Hu2	5.00	ft
Upstream WSE	1921.00	ft

Head over Weir *5.00 ft*

Condition Upstream of Weir 1921.00 1921.00

OUTFALL PIPE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.500	mgd =	0.8	cfs
Pipe Diameter, D	30	inch		
Pipe Length, L	219	ft		
Absolute Roughness, ε	0.00010	ft		
Pipe velocity, v	0.16	fps		
Kinematic Viscosity	1.000E-05	ft^2/sec		
Reynold's Number, R	39394			
Friction factor, f	0.0222		Equivalent Hazen-Williams "C" =	144.9177

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q (Average Day) 0.500 mgd = 0.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	0.50	0.77	0.50	----	30	----	0.16	0.00	0.00
1	Outlet Loss - Still Water	0.50	0.77	1.00	30	----	0.16	----	0.00	0.00
									Sum =	0.00

Total Energy Loss = 0.00 ft

Upstream Condition 1921.00 1921.00

WEIR IN OUTFALL/DECHLOR MANHOLE NO. 2
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	0.500	mgd =	0.8	cfs
WSE Downstream of Weir	1921.00	ft		
Weir Crest Elevation	1920.49	ft		
Downstream head, Hd	0.51	ft		
Length of Weir, L	5.00	ft		

WEIR IS SUBMERGED

Free Discharging Weir Computation { 6 }

Head on Weir, H	NA	ft
Upstream WSE	NA	ft

Submerged Weir Computation { 7 }

K	0.00
M	0.37

Increment	0.10	ft
Upstream Head, Hu1	0.51	ft
F(H1)	0.00	
F'(H1)	-2.95	
Upstream Head, Hu2	0.51	ft
Upstream WSE	1921.00	ft

Head over Weir *0.51 ft*

Condition Upstream of Weir 1921.00 1921.00

PIPE FROM UV SYSTEM TO DECHLOR MANHOLE NO. 2
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.500 mgd =	0.8 cfs
Pipe Diameter, D	24 inch	
Pipe Length, L	113 ft	
Absolute Roughness, ε	0.00010 ft	
Pipe velocity, v	0.25 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	49243	
Friction factor, f	0.0211	Equivalent Hazen-Williams "C" = 146.2975
Friction Energy Loss, h_L	0.00 ft	

MINOR PIPE LOSS HEADING

Flow, Q 0.500 mgd = 0.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	45 º Bend - Regular Fl.	0.50	0.77	0.23	24	----	0.25	----	0.00	0.00
1	Entrance Loss - Flush	0.50	0.77	0.50	----	24	----	0.25	0.00	0.00
1	Outlet Loss - Still Water	0.50	0.77	1.00	24	----	0.25	----	0.00	0.00
									Sum =	0.00

Total Energy Loss = 0.00 ft

Upstream Condition 1921.01 1921.01

UV SYSTEM DISCHARGE GATE
[SUBMERDED GATE - RECTANGULAR OPENING] { 14 }

Flow, Q	0.500 mgd =	0.8 cfs
Gate Width	2 ft	
Full Height of Opening	2 ft	
Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	0.19 fps	
Energy Loss thru Gate, h_L	0.00 ft	

Condition Upstream of Gate 1921.01 1921.01

UV CHANNEL DOWNSTREAM CONTROL WEIR - MODULATES SO THAT WATER SURFACE DOESN'T VARY MORE THAN 3 INCHES
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	0.500 mgd =	0.8 cfs
WSE Downstream of Weir	1921.01 ft	
Weir Crest Elevation	1924.01 ft	
Downstream head, Hd	-3.00 ft	
Length of Weir, L	6.83 ft	

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation { 6 }

Head on Weir, H	0.10 ft
Upstream WSE	1924.11 ft

Submerged Weir Computation { 7 }

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.10 ft*

Condition Upstream of Weir 1924.11 1924.11

UV CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	0.500 mgd =	0.8 cfs
Channel Width	3.00 ft	
Total Channel Length	42.83	
Downstream Invert El	1921.47	
Channel Slope	0.10%	

Manning Coeff, n	0.013

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1921.47	1921.47	2.64	0.097	0.96	0.000	----	----	1924.11	1924.11
8.6	1921.48	1921.47	2.64	0.098	0.96	0.000	0.000	0.00	1924.11	1924.11
17.1	1921.49	1921.48	2.63	0.098	0.95	0.000	0.000	0.00	1924.11	1924.11
25.7	1921.50	1921.49	2.62	0.098	0.95	0.000	0.000	0.00	1924.11	1924.11
34.3	1921.50	1921.50	2.61	0.099	0.95	0.000	0.000	0.00	1924.11	1924.11
42.8	1921.51	1921.50	2.60	0.099	0.95	0.000	0.000	0.00	1924.11	1924.12

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel 1924.11 1924.12

UV SYSTEM
MINOR CHANNEL LOSS HEADING

Flow, Q 0.500 mgd = 0.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance - Sharp Corners	0.500	0.77	0.50	3	3	5	0.05	0.05	0.00	0.00
	UV SYSTEM - 3 BANKS	**0.500**			**SYSTEM VARIES 2 "**						**0.167**
1	Outlet - Sharp Corners	0.500	0.77	1.00	3	3	5	0.05	0.05	0.00	0.00
										Sum =	0.167

Total Energy Loss = 0.17 ft

Upstream Condition 1924.28 1924.28

ENTRANCE TO UV CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	0.50 mgd =	0.8 cfs
Channel Width	3.75 ft	
Total Channel Length	10.00	
Downstream Invert El	1920.00	
Channel Slope	0.10%	
Manning Coeff, n	0.013	

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1920.00	1920.00	4.28	0.048	1.30	0.000	----	----	1924.28	1924.28
2.0	1920.00	1920.00	4.28	0.048	1.30	0.000	0.000	0.00	1924.28	1924.28
4.0	1920.00	1920.00	4.28	0.048	1.30	0.000	0.000	0.00	1924.28	1924.28
6.0	1920.01	1920.00	4.28	0.048	1.30	0.000	0.000	0.00	1924.28	1924.28
8.0	1920.01	1920.01	4.27	0.048	1.30	0.000	0.000	0.00	1924.28	1924.28
10.0	1920.01	1920.01	4.27	0.048	1.30	0.000	0.000	0.00	1924.28	1924.28

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel 1924.28 1924.28

ENTRANCE TO UV CHANNEL
MINOR CHANNEL LOSS HEADING

Flow, Q 0.5 mgd = 0.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Sudden Expansion	0.50	0.77	1.00	3.75	7.10	6.5	0.03	0.02	0.00	0.00
										Sum =	1.1E-05

Total Energy Loss = 0.00 ft

Upstream Condition 1924.28 1924.28

FILTER EFFLUENT CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	0.50 mgd =	0.8 cfs
Channel Width	1.50 ft	
Total Channel Length	80.00	
Downstream Invert El	1923.28	
Channel Slope	0.00%	
Manning Coeff, n	0.013	

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1923.28	1923.28	1.00	0.517	0.43	0.000	----	----	1924.28	1924.28
16.0	1923.28	1923.28	1.00	0.516	0.43	0.000	0.000	0.00	1924.28	1924.28

32.0	1923.28	1923.28	1.00	0.516	0.43	0.000	0.000	0.00	1924.28	1924.28
48.0	1923.28	1923.28	1.00	0.516	0.43	0.000	0.000	0.00	1924.28	1924.28
64.0	1923.28	1923.28	1.00	0.515	0.43	0.000	0.000	0.00	1924.28	1924.29
80.0	1923.28	1923.28	1.00	0.515	0.43	0.000	0.000	0.00	1924.28	1924.29

TOTAL ENERGY LOSS *0.01 ft*

Condition at Upstream End of Channel 1924.28 1924.29

FILTER EFFLUENT CHANNEL
MINOR CHANNEL LOSS HEADING

Flow, Q 0.500 mgd = 0.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	90 Degree Bend - 0º Radius	0.500	0.77	1.30	1.5	----	4.47	0.12	----	0.00	0.00
										Sum =	0.00054

Total Energy Loss = 0.00 ft

Upstream Condition 1924.29 1924.29

FROM FILTER EFFLUENT CHANNEL TO SAND BED FILTERS 1-3

Use tables below for flow through each sand filter bed
Total filter effluent flow = **0.50 mgd**

SFB	Is filter online (enter 0 or 1)	Weir Length, ft	Flow fraction	Flow, MGD
# 1	1	14.70	1.0000	0.50
# 2	0	14.70	0.0000	0.00
# 3	0	14.70	0.0000	0.00

Filter #1 effluent Weir

[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 0.500 mgd = 0.8 cfs

WSE Downstream of Weir 1924.26 ft
Weir Crest Elevation 1926.63 ft
Downstream head, Hd -2.37 ft
Length of Weir, L 14.70 ft

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation **{ 6 }**
Head on Weir, H 0.06 ft
Upstream WSE 1926.69 ft

Submerged Weir Computation **{ 7 }**
K NA
M NA
Increment NA ft
Upstream Head, Hu1 NA ft
F(H1) NA
F'(H1) NA
Upstream Head, Hu2 NA ft
Upstream WSE NA ft

Head over Weir *0.06 ft*

Condition Upstream of Weir 1926.69 1926.69

Dynasand Filter Headloss

12/3/15 Per Jason Vernon design headloss 30-36"
12/3/15 Per Jason Vernon actual headloss 18-20"

Headloss through filter cell #1 = 20 inch
Headloss through filter cell #1 = **1.67 ft**

Upstream Condition in Filter 1928.36 1928.36

18" Pipe connecting the Dynasand to filter influent channel

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow 0.650 mgd = 1.0 cfs

Influent Flow= 0.50 mgd
Filtrate Flow= 0.15 mgd
0.65 mgd

Pipe Diameter, D 18 inch
Pipe Length, L 2.67 ft
Absolute Roughness, ε 0.00015 ft
Pipe velocity, v 0.57 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 85354
Friction factor, f 0.0191

Equivalent Hazen-Williams "C" = 148.24

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q 0.650 mgd = 1.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)

1	Gate Valve (Open)	0.65	1.01	0.19	18	----	0.57	----	0.01	0.00		
1	Entrance Loss - Flush	0.65	1.01	0.50	----	18	----	0.57	0.01	0.00		
									Sum =	0.00		

Total Energy Loss = *0.00* *ft*

Upstream Condition 1928.36 1928.36

FILTER INFLUENT CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	0.650	mgd =	1.0 cfs	Influent Flow=	0.5 mgd
Channel Width	2.00	ft		Filtrate Flow=	0.15 mgd
Total Channel Length	55.00				0.65 mgd
Downstream Invert El	1924.00				
Channel Slope	3.60%				
Manning Coeff, n	0.013				

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1924.00	1924.00	4.36	0.115	0.81	0.000	----	----	1928.36	1928.36
11.0	1924.40	1924.00	3.97	0.127	0.80	0.000	0.000	0.00	1928.36	1928.36
22.0	1924.79	1924.40	3.57	0.141	0.78	0.000	0.000	0.00	1928.36	1928.36
33.0	1925.19	1924.79	3.17	0.158	0.76	0.000	0.000	0.00	1928.36	1928.36
44.0	1925.58	1925.19	2.78	0.181	0.74	0.000	0.000	0.00	1928.36	1928.36
55.0	1925.98	1925.58	2.38	0.211	0.70	0.000	0.000	0.00	1928.36	1928.36

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel 1928.36 1928.36

SE Pump Design Criteria

Secondary Effluent PS System Headloss

Pump TDH at minimum water level = **23.00 ft** Min water level 1908.85
Pump TDH at maximum water level = **26.00 ft**

	Flow (mgd)	TDH (feet)
Pump design	1000-2900	33
Initial pump design	200 to 1050	23 to 26

Upstream Condition 1908.85 1931.85

New SE Biolac line to the PS

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.650	mgd =	1.0 cfs	Influent Flow=	0.5 mgd
				Filtrate Flow=	0.15 mgd
Pipe Diameter, D	12	inch			0.65 mgd
Pipe Length, L	66	ft			
Absolute Roughness, ε	0.00010	ft			
Pipe velocity, v	1.28	fps			
Kinematic Viscosity	1.000E-05	ft^2/sec			
Reynold's Number, R	128031				
Friction factor, f	0.0177		Equivalent Hazen-Williams "C" = 149.89		
Friction Energy Loss, h_L	0.03	ft			

MINOR PIPE LOSS HEADING

Flow, Q 0.7 mgd = 1.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	0.65	1.01	0.50	----	12	----	1.28	0.03	0.01
1	Entrance Loss - Pipe Ext.	0.65	1.01	1.00	----	12	----	1.28	0.03	0.03
1	Plug Valve (Open)	0.65	1.01	0.77	12	----	1.28	----	0.03	0.02
									Sum =	0.06

Total Energy Loss = 0.09 ft

Upstream Condition 1919.09 1919.09

Exisiting SE Biolac line to the final clarifier #2 discharge line

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.650	mgd =	1.0 cfs	Influent Flow=	0.5 mgd
				Filtrate Flow=	0.15 mgd
Pipe Diameter, D	12	inch			0.65 mgd
Pipe Length, L	50	ft			
Absolute Roughness, ε	0.00010	ft			
Pipe velocity, v	1.28	fps			
Kinematic Viscosity	1.000E-05	ft^2/sec			
Reynold's Number, R	128031				
Friction factor, f	0.0177		Equivalent Hazen-Williams "C" = 149.89		
Friction Energy Loss, h_L	0.02	ft			

MINOR PIPE LOSS HEADING

Flow, Q | 0.650 mgd = | 1.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Plug Valve (Open)	0.65	1.01	0.77	12	----	1.28	----	0.03	0.02
1	Wye - Thru Straight Run	0.65	1.01	0.45	12	----	1.28	----	0.03	0.01
									Sum =	0.03

Total Energy Loss = 0.05 ft

Upstream Condition 1919.14 1919.14

8" SE Biolac line to the final clarifier #2 discharge line

Asumme that 1/4 of the total flow goes through this 8" line

total flow 0.500 **mgd**

1/4 Influent Flow= 0.125 mgd 0.25
1/4 Filtrate Flow= 0.0375 mgd 0.25
0.1625 mgd

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)]

Flow 0.163 mgd = 0.3 cfs

Pipe Diameter, D	8	inch
Pipe Length, L	9	ft
Absolute Roughness, ε	0.00015	ft
Pipe velocity, v	0.72	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	48011	
Friction factor, f	0.0219	

Equivalent Hazen-Williams "C" = 145.29

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q 0.163 mgd = 0.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	45 º Bend - Regular Fl.	0.16	0.25	0.23	8	----	0.72	----	0.01	0.00
1	Wye - Thru Side Outlet	0.16	0.25	1.35	8	----	0.72	----	0.01	0.01
2	22.5 º Bend	0.16	0.25	0.15	8	----	0.72	----	0.01	0.00
1	Butterfly Valve (Open)	0.16	0.25	0.50	8	----	0.72	----	0.01	0.00
1	Reducer	0.16	0.25	0.25	8	12	0.72	0.32	0.00	0.00
									Sum =	0.02

Total Energy Loss = 0.02 ft

Upstream Condition 1919.16 1919.16

Final clarifier #2 8" Biolac (PVC) discharge line connection to floating weir
Assume only 1/4 of the flow through the 8" pipe connected to the floating weir

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow 0.163 mgd = 0.3 cfs

1/4 Influent Flow= 0.125 mgd 0.25
1/4 Filtrate Flow= 0.0375 mgd 0.25
0.1625 mgd

Pipe Diameter, D	8	inch
Pipe Length, L	4	ft
Absolute Roughness, ε	0.00010	ft
Pipe velocity, v	0.72	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	48011	
Friction factor, f	0.0216	

Equivalent Hazen-Williams "C" = 146.20

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q 0.163 mgd = 0.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Butterfly Valve (Open)	0.163	0.25	0.50	8	----	0.72	----	0.01	0.00
1	Entrance Loss - Flush	0.163	0.25	0.50	----	8	----	0.72	0.01	0.00
2	Mitre Bend - 22.5 º Deflection	0.163	0.25	0.15	8	----	0.72	----	0.01	0.00
									Sum =	0.01

Total Energy Loss = 0.01 ft

Upstream Condition 1919.17 1919.17

Final Clarifier #2 floating weir

Assume only 1/2 of the flow through the 8" floating weir

Assume floating weir is a weigted pipe
Min water level 1922.00 ft
Max water level 1923.00 ft

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow 0.325 mgd = 0.5 cfs

1/2 Influent Flow= 0.25 mgd 0.5
1/2 Filtrate Flow= 0.075 mgd 0.5
0.325 mgd

Pipe Diameter, D 8 inch

Pipe Length, L	21	ft			
Absolute Roughness, ε	0.00015	ft			
Pipe velocity, v	1.44	fps			
Kinematic Viscosity	1.000E-05	ft^2/sec			
Reynold's Number, R	96023				
Friction factor, f	0.0193		Equivalent Hazen-Williams "C" =	147.0782	
Friction Energy Loss, h_L	0.02	ft			

MINOR PIPE LOSS HEADING

Flow, Q 0.325 mgd = 0.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Butterfly Valve (Open)	0.33	0.50	0.50	8	----	1.44	----	0.03	0.02
1	Entrance Loss - Flush	0.33	0.50	0.50	----	8	----	1.44	0.03	0.02
1	90 º Elbow - Regular Fl.	0.33	0.50	0.30	8	----	1.44	----	0.03	0.01
									Sum =	0.04

Total Energy Loss = *0.06* *ft*

Upstream Condition 1922.06 1922.06

Final Clarifier #2 to Biolac System Influent

Biolac min water level 1922.00 ft
Biolac max water level 1923.00 ft

Biolac Channel System Headloss

Headloss through biolac channel = 0.001 ft Channels normally have minimum headloss

Condition at Upstream End of Channel 1922.06 1922.06

16-inch pipeline from Biolac to Splitter Box

Ras flow 0.500 Per Carlos L assume 100% RAS flow returns to splitter box
Inf flow 0.500
total flow **1.000**

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow 1.000 mgd = 1.5 cfs

Pipe Diameter, D	16	inch			
Pipe Length, L	140	ft	assumed worst case flow enters biolac system on the east side		
Absolute Roughness, ε	0.00150	ft			
Pipe velocity, v	1.11	fps			
Kinematic Viscosity	1.000E-05	ft^2/sec			
Reynold's Number, R	147728				
Friction factor, f	0.0219		Equivalent Hazen-Williams "C" =	131.7032	
Friction Energy Loss, h_L	0.04	ft			

MINOR PIPE LOSS HEADING

Flow, Q 1.000 mgd = 1.5 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	1.00	1.55	0.50	----	16	----	1.11	0.02	0.01
1	Butterfly Valve (Open)	1.00	1.55	0.50	16	----	1.11	----	0.02	0.01
									Sum =	0.02

Total Energy Loss = 0.06 ft

Upstream Condition 1922.13 1922.13

Isolation gate at splitter box

[SUBMERDED GATE - CIRCULAR OPENING] **{ 15 }**

Flow, Q 1.000 mgd = 1.5 cfs

Diameter of Opening	1.33	ft
Sluice Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	1.11	fps
Energy Loss thru Gate, h_L	0.05	ft

Condition Upstream of Gate 1922.18 1922.18

Interim PS System Headloss

Pump TDH at minimum water level = **18.00 ft**
Pump TDH at maximum water level = **37.00 ft**

Max WSE in Splitter Box	1923.00
Assumed min water level	1905.50
Overflow water level	1913.13
Assumed normal water level	1912.50

Interim PS - 4.0 ADF

	Flow (mgd)	TDH (feet)
Pumping capacity, ea	1555.00	37
Min pumping capacity, ea	200.00	18
25 Hp, ea pump		

Upstream Condition 1912.50 1930.50

8-INCH PIPE FROM PRIMARY INFLUENT PS TO MECHANICAL SCREEN

8-inch Pipeline

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	0.500 mgd =	0.8 cfs	
Pipe Diameter, D	8 inch		
Pipe Length, L	21 ft		
Absolute Roughness, ε	0.00015 ft		
Pipe velocity, v	2.22 fps		
Kinematic Viscosity	1.000E-05 ft^2/sec		
Reynold's Number, R	147728		
Friction factor, f	0.0180	Equivalent Hazen-Williams "C" =	147.3048
Friction Energy Loss, h_L	0.04 ft		

MINOR PIPE LOSS HEADING

Flow, Q 0.500 mgd = 0.8 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	45 º Bend - Regular Fl.	0.50	0.77	0.23	8	----	2.22	----	0.08	0.02
1	90 º Elbow - Regular Fl.	0.50	0.77	0.30	8	----	2.22	----	0.08	0.02
1	Entrance Loss - Flush	0.50	0.77	0.50	----	8	----	2.22	0.08	0.04
1	Wye - Thru Side Outlet	0.50	0.77	1.35	8	----	2.22	----	0.08	0.10
									Sum =	0.18

Total Energy Loss = 0.22 ft

Upstream condition 1912.72 1912.72

Grit dishcarge channel

Minimum headloss in channel 1'-7" wide by 4'-6" deep
Since 8" pipe is full pipe, assume WS elev is above the pipe crown
crown = 1913.25

Upstream condition 1913.25 1913.25

Specifications Sections Linear Proportional Weir (Sutro Weir) Plate at the end of grit channel

Assume all flow goes through one channel

$$Q=C_dK\ (\pi/2)*(\mathrm{sqrt}(2gH)$$

K=2x(sqrt y)

H= water surface elev	Q= 0.500 mgd	x= 3.07 inches	
H= 0.0643 inches	Q= 0.774 ft^3/sec	y= 3.00 inches	
H= 0.77 feet	Cd= 0.62	K= 9.21 inches	0.77 feet
	g= 32.2 ft/sec		

Upstream condition 1914.02 1914.02

Grit channel

Assume all flow goes through one channel

[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	0.50 mgd =	0.8 cfs
Channel Width	1.00 ft	
Total Channel Length	33.00	
Downstream Invert El	1913.00	
Channel Slope	1.50%	slope= (1913.50-1913.00/33)
Manning Coeff, n	0.013	

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1913.00	1913.00	1.01	0.764	0.33	0.000	----	----	1914.01	1914.02
6.6	1913.10	1913.00	0.91	0.847	0.32	0.000	0.000	0.00	1914.01	1914.02
13.2	1913.20	1913.10	0.81	0.951	0.31	0.000	0.000	0.00	1914.01	1914.03
19.8	1913.30	1913.20	0.71	1.086	0.29	0.000	0.000	0.00	1914.01	1914.03
26.4	1913.40	1913.30	0.61	1.267	0.27	0.001	0.001	0.00	1914.01	1914.03
33.0	1913.50	1913.40	0.51	1.528	0.25	0.001	0.001	0.01	1914.00	1914.04

TOTAL ENERGY LOSS *0.02 ft*

Condition at Upstream End of Channel 1914.04 1914.04

Mechanical Screen System Headloss

Headloss through screen = 0.50 ft

Assumed headloss through mechanical screen = .25-.75 ft

Condition at Upstream End of Channel 1914.54 1914.54

FROM MECHANICAL SCREEN TO PARSHALL FLUME

PARSHALL FLUME { 13 }

Flow, Q =	0.500 mgd		
	0.8 cfs	(0.3 < Q < 100)	
Downstream WSE =	1914.54 ft		
Downstream EGL =	1914.54 ft		
Throat width =	0.25 ft	(available sizes = 1, 2, 3, 3.5, 4, 5, 6, 7, and 8)	(W-2 Contract Drawings)
Flume invert elevation =	1915.11 ft		
Upstream channel width =	1.25 ft		
Downstream depth, Hb =	-0.57 ft		
Upstream depth, Ha =	0.84 ft		
Upstream velocity =	0.74 fps		
Submergence =	**-68.2** %		
Headloss =	1.421 ft		** NOT USED **

WSE Upstream of Flume 1915.95 1915.96

PIPE FROM PARSHALL FLUME TO GLADDIN MH #2

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Q_{full} = 1.27 using manning eqn

Flow	1.270 mgd =	2.0 cfs
Pipe Diameter, D	12 inch	
Pipe Length, L	51 ft	
Absolute Roughness, ε	0.00015 ft	
Pipe velocity, v	2.50 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	250152	
Friction factor, f	0.0162	Equivalent Hazen-Williams "C" = 148.713
Friction Energy Loss, h_L	0.08 ft	

MINOR PIPE LOSS HEADING

Flow, Q 1.27 mgd = 2.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 º Elbow - Regular Fl.	1.27	1.96	0.30	12	----	2.50	----	0.10	0.03
1	Entrance Loss - Flush	1.27	1.96	0.50	----	12	----	2.50	0.10	0.05
									Sum =	0.08

Total Energy Loss = 0.16 ft

Upstream Condition 1916.12 1916.12

PIPE FROM GLADDIN MH # 2 to GLADDIN MH #1

Q_{full} = 1.31 using manning eqn

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	1.310 mgd =	2.0 cfs
Pipe Diameter, D	12 inch	
Pipe Length, L	388 ft	
Absolute Roughness, ε	0.00150 ft	
Pipe velocity, v	2.58 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	258031	
Friction factor, f	0.0226	Equivalent Hazen-Williams "C" = 124.149
Friction Energy Loss, h_L	0.91 ft	

MINOR PIPE LOSS HEADING

Flow, Q 1.310 mgd = 2.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	1.31	2.03	0.50	----	12	----	2.58	0.10	0.05
1	Entrance Loss - Pipe Ext.	1.31	2.03	1.00	----	12	----	2.58	0.10	0.10
									Sum =	0.16

Total Energy Loss = 1.06 ft

Upstream Condition 1917.18 1917.18

PROJECT : Town of Marana WRF Master Plan

JOB # : 10067A00 **REVISION:** **CHECKED :** **BY :** **DATE :** **DATE :** 12/8/2015

	Equation Ref.	HGL	EGL

DOWNSTREAM CONTROL

EGL =	1921.00		100-Yr Flood Elev, FEMA		1921.00	1921.00
Flow =	1.500 mgd =	0.55 cfs				

OUTFALL STRUCTURE
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	1.500	mgd =	2.3 cfs	
WSE Downstream of Weir	1921.00	ft		
Weir Crest Elevation	1916.00	ft		
Downstream head, Hd	5.00	ft		
Length of Weir, L	9.00	ft		

WEIR IS SUBMERGED

			Equation Ref.
Free Discharging Weir Computation			{ 6 }
Head on Weir, H	NA	ft	
Upstream WSE	NA	ft	
Submerged Weir Computation			{ 7 }
K	0.00		
M	11.18		
Increment	0.10	ft	
Upstream Head, Hu1	5.00	ft	
F(H1)	0.00		
F'(H1)	-0.30		
Upstream Head, Hu2	5.00	ft	
Upstream WSE	1921.00	ft	
Head over Weir	*5.00*	*ft*	

Condition Upstream of Weir 1921.00 1921.00

OUTFALL PIPE
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	1.500	mgd =	2.3 cfs	
Pipe Diameter, D	30	inch		
Pipe Length, L	219	ft		
Absolute Roughness, ε	0.00010	ft		
Pipe velocity, v	0.47	fps		
Kinematic Viscosity	1.000E-05	ft^2/sec		
Reynold's Number, R	118182			
Friction factor, f	0.0176		Equivalent Hazen-Williams "C" =	150.1319
Friction Energy Loss, h_L	0.01	ft		

MINOR PIPE LOSS HEADING

Flow, Q (Average Day) 1.500 mgd = 2.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	1.50	2.32	0.50	----	30	----	0.47	0.00	0.00
1	Outlet Loss - Still Water	1.50	2.32	1.00	30	----	0.47	----	0.00	0.00
									Sum =	0.01

Total Energy Loss = 0.01 ft

Upstream Condition 1921.01 1921.01

WEIR IN OUTFALL/DECHLOR MANHOLE NO. 2
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow	1.500	mgd =	2.3 cfs
WSE Downstream of Weir	1921.01	ft	
Weir Crest Elevation	1920.49	ft	
Downstream head, Hd	0.52	ft	
Length of Weir, L	5.00	ft	

WEIR IS SUBMERGED

			Equation Ref.
Free Discharging Weir Computation			{ 6 }
Head on Weir, H	NA	ft	
Upstream WSE	NA	ft	
Submerged Weir Computation			{ 7 }
K	0.01		
M	0.38		

Increment	0.10	ft
Upstream Head, Hu1	0.54	ft
F(H1)	0.00	
F'(H1)	-3.04	
Upstream Head, Hu2	0.54	ft
Upstream WSE	1921.03	ft

Head over Weir *0.54 ft*

Condition Upstream of Weir 1921.03 1921.03

PIPE FROM UV SYSTEM TO DECHLOR MANHOLE NO. 2
[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow 1.500 mgd = 2.3 cfs

Pipe Diameter, D	24	inch
Pipe Length, L	113	ft
Absolute Roughness, ε	0.00010	ft
Pipe velocity, v	0.74	fps
Kinematic Viscosity	1.000E-05	ft^2/sec
Reynold's Number, R	147728	
Friction factor, f	0.0169	

Equivalent Hazen-Williams "C" = 150.8488

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 1.500 mgd = 2.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	45 º Bend - Regular Fl.	1.50	2.32	0.23	24	----	0.74	----	0.01	0.00
1	Entrance Loss - Flush	1.50	2.32	0.50	----	24	----	0.74	0.01	0.00
1	Outlet Loss - Still Water	1.50	2.32	1.00	24	----	0.74	----	0.01	0.01
									Sum =	0.02

Total Energy Loss = 0.02 ft

Upstream Condition 1921.06 1921.06

UV SYSTEM DISCHARGE GATE
[SUBMERDED GATE - RECTANGULAR OPENING] **{ 14 }**

Flow, Q 1.500 mgd = 2.3 cfs

Gate Width	2	ft
Full Height of Opening	2	ft
Gate Percent Open	100%	
Discharge Coefficient, C	0.61	
Velocity through gate, v	0.58	fps

Energy Loss thru Gate, h_L 0.01 ft

Condition Upstream of Gate 1921.07 1921.07

UV CHANNEL DOWNSTREAM CONTROL WEIR - MODULATES SO THAT WATER SURFACE DOESN'T VARY MORE THAN 3 INCHES
[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 1.500 mgd = 2.3 cfs

WSE Downstream of Weir	1921.07	ft
Weir Crest Elevation	1924.01	ft
Downstream head, Hd	-2.94	ft
Length of Weir, L	6.83	ft

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation **{ 6 }**

Head on Weir, H	0.22	ft
Upstream WSE	1924.23	ft

Submerged Weir Computation **{ 7 }**

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.22 ft*

Condition Upstream of Weir 1924.23 1924.23

UV CHANNEL
[CHANNEL FRICTION LOSSES] **{ 5 }**

Flow, Q	1.500	mgd = 2.3 cfs
Channel Width	3.00	ft
Total Channel Length	42.83	
Downstream Invert El	1921.47	
Channel Slope	0.10%	

Manning Coeff, n	0.013

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1921.47	1921.47	2.76	0.281	0.97	0.000	----	----	1924.23	1924.23
8.6	1921.48	1921.47	2.75	0.281	0.97	0.000	0.000	0.00	1924.23	1924.23
17.1	1921.49	1921.48	2.74	0.282	0.97	0.000	0.000	0.00	1924.23	1924.23
25.7	1921.50	1921.49	2.73	0.283	0.97	0.000	0.000	0.00	1924.23	1924.23
34.3	1921.50	1921.50	2.72	0.284	0.97	0.000	0.000	0.00	1924.23	1924.23
42.8	1921.51	1921.50	2.71	0.285	0.97	0.000	0.000	0.00	1924.23	1924.23

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel 1924.23 1924.23

UV SYSTEM
MINOR CHANNEL LOSS HEADING

Flow, Q 1.500 mgd = 2.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance - Sharp Corners	1.500	2.32	0.50	3	3	5	0.15	0.15	0.00	0.00
	UV SYSTEM - 3 BANKS	**1.500**			**SYSTEM VARIES 2 "**						**0.167**
1	Outlet - Sharp Corners	1.500	2.32	1.00	3	3	5	0.15	0.15	0.00	0.00
										Sum =	0.167

Total Energy Loss = 0.17 ft

Upstream Condition 1924.40 1924.40

ENTRANCE TO UV CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	1.50	mgd = 2.3 cfs
Channel Width	3.75	ft
Total Channel Length	10.00	
Downstream Invert El	1920.00	
Channel Slope	0.10%	
Manning Coeff, n	0.013	

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1920.00	1920.00	4.39	0.141	1.31	0.000	----	----	1924.39	1924.40
2.0	1920.00	1920.00	4.39	0.141	1.31	0.000	0.000	0.00	1924.39	1924.40
4.0	1920.00	1920.00	4.39	0.141	1.31	0.000	0.000	0.00	1924.39	1924.40
6.0	1920.01	1920.00	4.39	0.141	1.31	0.000	0.000	0.00	1924.39	1924.40
8.0	1920.01	1920.01	4.39	0.141	1.31	0.000	0.000	0.00	1924.39	1924.40
10.0	1920.01	1920.01	4.38	0.141	1.31	0.000	0.000	0.00	1924.39	1924.40

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel 1924.39 1924.40

ENTRANCE TO UV CHANNEL
MINOR CHANNEL LOSS HEADING

Flow, Q 1.5 mgd = 2.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Sudden Expansion	1.50	2.32	1.00	3.75	7.10	6.5	0.10	0.05	0.00	0.00
										Sum =	0.0001

Total Energy Loss = 0.00 ft

Upstream Condition 1924.40 1924.40

FILTER EFFLUENT CHANNEL
[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	1.50	mgd = 2.3 cfs
Channel Width	1.50	ft
Total Channel Length	80.00	
Downstream Invert El	1923.28	
Channel Slope	0.00%	
Manning Coeff, n	0.013	

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1923.28	1923.28	1.08	1.427	0.44	0.000	----	----	1924.36	1924.40
16.0	1923.28	1923.28	1.09	1.418	0.44	0.000	0.000	0.01	1924.37	1924.40

32.0	1923.28	1923.28	1.10	1.408	0.45	0.000	0.000	0.01	1924.38	1924.41
48.0	1923.28	1923.28	1.11	1.399	0.45	0.000	0.000	0.01	1924.39	1924.42
64.0	1923.28	1923.28	1.11	1.390	0.45	0.000	0.000	0.01	1924.39	1924.42
80.0	1923.28	1923.28	1.12	1.381	0.45	0.000	0.000	0.01	1924.40	1924.43

TOTAL ENERGY LOSS *0.04 ft*

Condition at Upstream End of Channel 1924.40 1924.43

FILTER EFFLUENT CHANNEL
MINOR CHANNEL LOSS HEADING

Flow, Q 1.500 mgd = 2.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Width Up (ft)	Width Down (ft)	Depth (ft)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
2	90 Degree Bend - 0º Radius	1.500	2.32	1.30	1.5	----	4.47	0.35	----	0.00	0.00
										Sum =	0.00484

Total Energy Loss = 0.00 ft

Upstream Condition 1924.44 1924.44

FROM FILTER EFFLUENT CHANNEL TO SAND BED FILTERS 1-3

Use tables below for flow through each sand filter bed
Total filter effluent flow = **1.50 mgd**

SFB	Is filter online (enter 0 or 1)	Weir Length, ft	Flow fraction	Flow, MGD
# 1	1	14.70	1.0000	1.50
# 2	0	14.70	0.0000	0.00
# 3	0	14.70	0.0000	0.00

Filter #1 effluent Weir

[STRAIGHT EDGED SHARP CRESTED WEIR]

Flow 1.500 mgd = 2.3 cfs

WSE Downstream of Weir 1924.26 ft
Weir Crest Elevation 1926.63 ft
Downstream head, Hd -2.37 ft
Length of Weir, L 14.70 ft

WEIR IS FREE-DISCHARGING

Free Discharging Weir Computation **{ 6 }**
Head on Weir, H 0.13 ft
Upstream WSE 1926.76 ft

Submerged Weir Computation **{ 7 }**

K	NA	
M	NA	
Increment	NA	ft
Upstream Head, Hu1	NA	ft
F(H1)	NA	
F'(H1)	NA	
Upstream Head, Hu2	NA	ft
Upstream WSE	NA	ft

Head over Weir *0.13 ft*

Condition Upstream of Weir 1926.76 1926.76

Dynasand Filter Headloss

12/3/15 Per Jason Vernon design headloss 30-36"
12/3/15 Per Jason Vernon actual headloss 18-20"

Headloss through filter cell #1 = 20 inch
Headloss through filter cell #1 = **1.67 ft**

Upstream Condition in Filter 1928.43 1928.43

18" Pipe connecting the Dynasand to filter influent channel

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow 1.650 mgd = 2.6 cfs

Influent Flow= 1.50 mgd
Filtrate Flow= 0.15 mgd
1.65 mgd

Pipe Diameter, D 18 inch
Pipe Length, L 2.67 ft
Absolute Roughness, ε 0.00015 ft
Pipe velocity, v 1.44 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 216667
Friction factor, f 0.0162

Equivalent Hazen-Williams "C" = 150.15

Friction Energy Loss, h_L 0.00 ft

MINOR PIPE LOSS HEADING

Flow, Q 1.650 mgd = 2.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)

1	Gate Valve (Open)	1.65	2.55	0.19	18	----	1.44	----	0.03	0.01
1	Entrance Loss - Flush	1.65	2.55	0.50	----	18	----	1.44	0.03	0.02
									Sum =	0.02

Total Energy Loss = *0.02* *ft*

Upstream Condition 1928.45 1928.45

FILTER INFLUENT CHANNEL

[CHANNEL FRICTION LOSSES] **{ 5 }**

Flow, Q	1.650 mgd =	2.6 cfs	Influent Flow=	1.50	mgd
Channel Width	2.00 ft		Filtrate Flow=	0.15	mgd
Total Channel Length	55.00			1.65	mgd
Downstream Invert El	1924.00				
Channel Slope	3.60%				
Manning Coeff, n	0.013				

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1924.00	1924.00	4.45	0.287	0.82	0.000	----	----	1928.45	1928.45
11.0	1924.40	1924.00	4.05	0.315	0.80	0.000	0.000	0.00	1928.45	1928.45
22.0	1924.79	1924.40	3.66	0.349	0.79	0.000	0.000	0.00	1928.45	1928.45
33.0	1925.19	1924.79	3.26	0.391	0.77	0.000	0.000	0.00	1928.45	1928.45
44.0	1925.58	1925.19	2.86	0.446	0.74	0.000	0.000	0.00	1928.45	1928.45
55.0	1925.98	1925.58	2.47	0.517	0.71	0.000	0.000	0.00	1928.45	1928.45

TOTAL ENERGY LOSS *0.00 ft*

Condition at Upstream End of Channel 1928.45 1928.45

SE Pump Design Criteria

Secondary Effluent PS System Headloss

Pump TDH at minimum water level =	**23.00 ft**	Min water level	1908.85		**Flow (mgd)**	**TDH (feet)**
Pump TDH at maximum water level =	**26.00 ft**			Pump design	1000-2900	33
				Initial pump design	200 to 1050	23 to 26

Upstream Condition 1908.85 1931.85

New SE Biolac line to the PS

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow	1.650 mgd =	2.6 cfs	Influent Flow=	1.50	mgd
			Filtrate Flow=	0.15	mgd
Pipe Diameter, D	12 inch			1.65	mgd
Pipe Length, L	66 ft				
Absolute Roughness, ε	0.00010 ft				
Pipe velocity, v	3.25 fps				
Kinematic Viscosity	1.000E-05 ft^2/sec				
Reynold's Number, R	325001				
Friction factor, f	0.0153	Equivalent Hazen-Williams "C" = 150.59			
Friction Energy Loss, h_L	0.16 ft				

MINOR PIPE LOSS HEADING

Flow, Q 1.7 mgd = 2.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	1.65	2.55	0.50	----	12	----	3.25	0.16	0.08
1	Entrance Loss - Pipe Ext.	1.65	2.55	1.00	----	12	----	3.25	0.16	0.16
1	Plug Valve (Open)	1.65	2.55	0.77	12	----	3.25	----	0.16	0.13
									Sum =	0.37

Total Energy Loss = 0.54 ft

Upstream Condition 1919.54 1919.54

Exisiting SE Biolac line to the final clarifier #2 discharge line

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow	1.650 mgd =	2.6 cfs	Influent Flow=	1.50	mgd
			Filtrate Flow=	0.15	mgd
Pipe Diameter, D	12 inch			1.65	mgd
Pipe Length, L	50 ft				
Absolute Roughness, ε	0.00010 ft				
Pipe velocity, v	3.25 fps				
Kinematic Viscosity	1.000E-05 ft^2/sec				
Reynold's Number, R	325001				
Friction factor, f	0.0153	Equivalent Hazen-Williams "C" = 150.59			
Friction Energy Loss, h_L	0.12 ft				

MINOR PIPE LOSS HEADING

Flow, Q 1.650 mgd = 2.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Plug Valve (Open)	1.65	2.55	0.77	12	----	3.25	----	0.16	0.13
1	Wye - Thru Straight Run	1.65	2.55	0.45	12	----	3.25	----	0.16	0.07
									Sum =	0.20

Total Energy Loss = 0.32 ft

Upstream Condition 1919.86 1919.86

8" SE Biolac line to the final clarifier #2 discharge line

Asumme that 1/4 of the total flow goes through this 8" line

total flow 1.50 mgd

1/4 Influent Flow= 0.375 mgd 0.25
1/4 Filtrate Flow= 0.0375 mgd 0.25
0.4125 mgd

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)]

Flow 0.413 mgd = 0.6 cfs

Pipe Diameter, D 8 inch
Pipe Length, L 9 ft
Absolute Roughness, ε 0.00015 ft
Pipe velocity, v 1.83 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 121875
Friction factor, f 0.0185

Equivalent Hazen-Williams "C" = 147.29

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 0.413 mgd = 0.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	45 º Bend - Regular Fl.	0.41	0.64	0.23	8	----	1.83	----	0.05	0.01
1	Wye - Thru Side Outlet	0.41	0.64	1.35	8	----	1.83	----	0.05	0.07
2	22.5 º Bend	0.41	0.64	0.15	8	----	1.83	----	0.05	0.02
1	Butterfly Valve (Open)	0.41	0.64	0.50	8	----	1.83	----	0.05	0.03
1	Reducer	0.41	0.64	0.25	8	12	1.83	0.81	0.01	0.00
									Sum =	0.13

Total Energy Loss = 0.14 ft

Upstream Condition 1920.00 1920.00

Final clarifier #2 8" Biolac (PVC) discharge line connection to floating weir
Assume only 1/4 of the flow through the 8" pipe connected to the floating weir

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 0.413 mgd = 0.6 cfs

1/4 Influent Flow= 0.375 mgd 0.25
1/4 Filtrate Flow= 0.0375 mgd 0.25
0.4125 mgd

Pipe Diameter, D 8 inch
Pipe Length, L 4 ft
Absolute Roughness, ε 0.00010 ft
Pipe velocity, v 1.83 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 121875
Friction factor, f 0.0181

Equivalent Hazen-Williams "C" = 149.02

Friction Energy Loss, h_L 0.01 ft

MINOR PIPE LOSS HEADING

Flow, Q 0.413 mgd = 0.6 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Butterfly Valve (Open)	0.413	0.64	0.50	8	----	1.83	----	0.05	0.03
1	Entrance Loss - Flush	0.413	0.64	0.50	----	8	----	1.83	0.05	0.03
2	Mitre Bend - 22.5 º Deflection	0.413	0.64	0.15	8	----	1.83	----	0.05	0.02
									Sum =	0.04

Total Energy Loss = 0.05 ft

Upstream Condition 1920.05 1920.05

Final Clarifier #2 floating weir

Assume only 1/2 of the flow through the 8" floating weir

Assume floating weir is a weigted pipe
Min water level 1922.00 ft
Max water level 1923.00 ft

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow 0.825 mgd = 1.3 cfs

1/2 Influent Flow= 0.75 mgd 0.5
1/2 Filtrate Flow= 0.075 mgd 0.5
0.825 mgd

Pipe Diameter, D 8 inch

Pipe Length, L 21 ft
Absolute Roughness, ε 0.00015 ft
Pipe velocity, v 3.66 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 243751
Friction factor, f 0.0168

Equivalent Hazen-Williams "C" = 146.6306

Friction Energy Loss, h_L 0.11 ft

MINOR PIPE LOSS HEADING

Flow, Q 0.825 mgd = 1.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Butterfly Valve (Open)	0.83	1.28	0.50	8	----	3.66	----	0.21	0.10
1	Entrance Loss - Flush	0.83	1.28	0.50	----	8	----	3.66	0.21	0.10
1	90 º Elbow - Regular Fl.	0.83	1.28	0.30	8	----	3.66	----	0.21	0.06
									Sum =	0.27

Total Energy Loss = 0.38 ft

Upstream Condition 1922.38 1922.38

Final Clarifier #2 to Biolac System Influent

Biolac min water level 1922.00 ft
Biolac max water level 1923.00 ft

Biolac Channel System Headloss

Headloss through biolac channel = 0.001 ft

Channels normally have minimum headloss

Condition at Upstream End of Channel 1922.38 1922.38

16-inch pipeline from Biolac to Splitter Box

Ras flow 0.500 Per Carlos L assume 100% RAS flow returns to splitter box
Inf flow 1.500
total flow **2.000**

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] **{ 4 }**

Flow 2.000 mgd = 3.1 cfs

Pipe Diameter, D 16 inch
Pipe Length, L 140 ft — assumed worst case flow enters biolac system on the east side
Absolute Roughness, ε 0.00150 ft
Pipe velocity, v 2.22 fps
Kinematic Viscosity 1.000E-05 ft^2/sec
Reynold's Number, R 295455
Friction factor, f 0.0211

Equivalent Hazen-Williams "C" = 127.0068

Friction Energy Loss, h_L 0.17 ft

MINOR PIPE LOSS HEADING

Flow, Q 2.000 mgd = 3.1 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	2.00	3.09	0.50	----	16	----	2.22	0.08	0.04
1	Butterfly Valve (Open)	2.00	3.09	0.50	16	----	2.22	----	0.08	0.04
									Sum =	0.08

Total Energy Loss = 0.24 ft

Upstream Condition 1922.63 1922.63

Isolation gate at splitter box

[SUBMERDED GATE - CIRCULAR OPENING] **{ 15 }**

Flow, Q 2.00 mgd = 3.1 cfs

Diameter of Opening 1.33 ft
Sluice Gate Percent Open 100%
Discharge Coefficient, C 0.61
Velocity through gate, v 2.23 fps

Energy Loss thru Gate, h_L 0.21 ft

Condition Upstream of Gate 1922.83 1922.83

Interim PS System Headloss

Pump TDH at minimum water level = **18.00 ft**
Pump TDH at maximum water level = **37.00 ft**

Max WSE in Splitter Box 1923.00
Assumed min water level 1905.50
Overflow water level 1913.13
Assumed normal water level 1912.50

Interim PS - 4.0 ADF

	Flow (mgd)	**TDH (feet)**
Pumping capacity, ea	1555.00	37
Min pumping capacity, ea	200.00	18
25 Hp, ea pump		

Upstream Condition 1912.50 1930.50

8-INCH PIPE FROM PRIMARY INFLUENT PS TO MECHANICAL SCREEN

8-inch Pipeline

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	1.50	mgd =	2.3 cfs
Pipe Diameter, D	8	inch	
Pipe Length, L	21	ft	
Absolute Roughness, ε	0.00015	ft	
Pipe velocity, v	6.65	fps	
Kinematic Viscosity	1.000E-05	ft^2/sec	
Reynold's Number, R	443183		
Friction factor, f	0.0158		Equivalent Hazen-Williams "C" = 144.4801
Friction Energy Loss, h_L	0.34	ft	

MINOR PIPE LOSS HEADING

Flow, Q 1.50 mgd = 2.3 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	45 º Bend - Regular Fl.	1.50	2.32	0.23	8	----	6.65	----	0.69	0.16
1	90 º Elbow - Regular Fl.	1.50	2.32	0.30	8	----	6.65	----	0.69	0.21
1	Entrance Loss - Flush	1.50	2.32	0.50	----	8	----	6.65	0.69	0.34
1	Wye - Thru Side Outlet	1.50	2.32	1.35	8	----	6.65	----	0.69	0.93
									Sum =	1.63

Total Energy Loss = 1.98 ft

Upstream condition 1914.48 1914.48

Grit dishcarge channel

Minimum headloss in channel 1'-7" wide by 4'-6" deep
Since 8" pipe is full pipe, assume WS elev is above the pipe crown
crown = 1913.25

Upstream condition 1913.25 1913.25

Specifications Sections Linear Proportional Weir (Sutro Weir) Plate at the end of grit channel

Assume all flow goes through one channel

$Q=C_dK\ (\Pi/2)*(sqrt(2gH)$

K=2x(sqrt y)

Q= 1.50 mgd
Q= 2.321 ft^3/sec
Cd= 0.62
g= 32.2 ft/sec

x= 3.07 inches
y= 3.00 inches
K= 9.21 inches
0.77 feet

H= water surface elev
H= 0.1929 inches
H= 2.32 feet

Upstream condition 1915.57 1915.57

Grit channel

Assume all flow goes through one channel

[CHANNEL FRICTION LOSSES] { 5 }

Flow, Q	1.50	mgd =	2.3 cfs
Channel Width	1.00	ft	
Total Channel Length	33.00		
Downstream Invert El	1913.00		
Channel Slope	1.50%		slope= (1913.50-1913.00/33)
Manning Coeff, n	0.013		

Station	Invert Up	Invert Down	Depth (ft)	Vel. (fps)	Hydr. Radius (ft)	Sf	Avg. Sf	Friction Loss (ft)	HGL	EGL
0.0	1913.00	1913.00	2.55	0.909	0.42	0.000	----	----	1915.55	1915.57
6.6	1913.10	1913.00	2.45	0.946	0.42	0.000	0.000	0.00	1915.55	1915.57
13.2	1913.20	1913.10	2.35	0.985	0.41	0.000	0.000	0.00	1915.55	1915.57
19.8	1913.30	1913.20	2.26	1.028	0.41	0.000	0.000	0.00	1915.55	1915.57
26.4	1913.40	1913.30	2.16	1.076	0.41	0.000	0.000	0.00	1915.55	1915.57
33.0	1913.50	1913.40	2.06	1.127	0.40	0.000	0.000	0.00	1915.55	1915.57

TOTAL ENERGY LOSS *0.01 ft*

Condition at Upstream End of Channel 1915.57 1915.57

Mechanical Screen System Headloss

Headloss through screen = 0.50 ft

Assumed headloss through mechanical screen = .25-.75 ft

Condition at Upstream End of Channel 1916.07 1916.07

FROM MECHANICAL SCREEN TO PARSHALL FLUME

PARSHALL FLUME { 13 }

Flow, Q =	1.50 mgd		
	2.3 cfs	(0.3 < Q < 100)	
Downstream WSE =	1916.07 ft		
Downstream EGL =	1916.07 ft		
Throat width =	0.25 ft	(available sizes = 1, 2, 3, 3.5, 4, 5, 6, 7, and 8)	(W-2 Contract Drawings)
Flume invert elevation =	1915.11 ft		
Upstream channel width =	1.25 ft		
Downstream depth, Hb =	0.96 ft		
Upstream depth, Ha =	1.77 ft		
Upstream velocity =	1.05 fps		
Submergence =	**54.3** %		
Headloss =	#N/A ft	*** Using empirical solution. ***	**** NOT USED ****

WSE Upstream of Flume 1916.88 1916.90

PIPE FROM PARSHALL FLUME TO GLADDIN MH #2

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Q_{full} = 1.27 using manning eqn

Flow	1.270 mgd =	2.0 cfs
Pipe Diameter, D	12 inch	
Pipe Length, L	51 ft	
Absolute Roughness, ε	0.00015 ft	
Pipe velocity, v	2.50 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	250152	
Friction factor, f	0.0162	Equivalent Hazen-Williams "C" = 148.713
Friction Energy Loss, h_L	0.08 ft	

MINOR PIPE LOSS HEADING

Flow, Q 1.27 mgd = 2.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	90 º Elbow - Regular Fl.	1.27	1.96	0.30	12	----	2.50	----	0.10	0.03
1	Entrance Loss - Flush	1.27	1.96	0.50	----	12	----	2.50	0.10	0.05
									Sum =	0.08

Total Energy Loss = 0.16 ft

Upstream Condition 1917.06 1917.06

PIPE FROM GLADDIN MH # 2 to GLADDIN MH #1

Q_{full} = 1.31 using manning eqn

[PIPE FRICTION LOSSES (DARCY-WEISBACH / COLEBROOK)] { 4 }

Flow	1.310 mgd =	2.0 cfs
Pipe Diameter, D	12 inch	
Pipe Length, L	388 ft	
Absolute Roughness, ε	0.00150 ft	
Pipe velocity, v	2.58 fps	
Kinematic Viscosity	1.000E-05 ft^2/sec	
Reynold's Number, R	258031	
Friction factor, f	0.0226	Equivalent Hazen-Williams "C" = 124.149
Friction Energy Loss, h_L	0.91 ft	

MINOR PIPE LOSS HEADING

Flow, Q 1.310 mgd = 2.0 cfs

No.	Description	Flow (mgd)	Flow (cfs)	K	Dia Up (in)	Dia Down (in)	Vel Up (fps)	Vel Down (fps)	Vel Head (ft)	Minor Loss (ft)
1	Entrance Loss - Flush	1.31	2.03	0.50	----	12	----	2.58	0.10	0.05
1	Entrance Loss - Pipe Ext.	1.31	2.03	1.00	----	12	----	2.58	0.10	0.10
									Sum =	0.16

Total Energy Loss = 1.06 ft

Upstream Condition 1918.12 1918.12