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Investing in clean water resources

Volume 4
Final Facility Plan for the West and East Wastewater Treatment Plants

Submitted

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Prepared for

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Acronyms and Abbreviations

°C	degrees Celsius
°F	degrees Fahrenheit
BAF	biological aerated filtration
BioCEC	biological and chemically enhanced clarification
BNR	biological nutrient removal
BOD	biochemical oxygen demand
$CBOD_5$	5-day carbonaceous biochemical oxygen demand
cfm	cubic feet per minute
CFR	*Code of Federal Regulations*
CSO	combined sewer overflow
Decree	Consent Decree between Evansville Water and Sewer Utility and the United States and State of Indiana
EPA	U.S. Environmental Protection Agency
Facility Plan	Facility Plan for the West and East Wastewater Treatment Plants
FRP	fiberglass-reinforced plastic
ft^2	square foot/feet
gpm	gallons per minute
HDPE	high-density polyethylene
hp	horsepower
IOCP	Integrated Overflow Control Plan
lb/day	pounds per day
LF	linear feet
LTCP	Long-term Control Plan
mg/L	milligrams per liter
MG	million gallons
mgd	million gallons per day
MHI	median household income

mL	milliliters
mm	millimeters
NFA	no feasible alternatives
NPDES	National Pollutant Discharge Elimination System
NPV	net present value
O&M	operation and maintenance
PE	primary effluent
RAS	return-activated sludge
scfm	standard cubic feet per minute
SSO	sanitary sewer overflow
SSRMP	Sanitary Sewer Remedial Measures Plan
SVI	sludge volume index
SWD	sidewater depth
TDH	total dynamic head
TEFC	totally enclosed, fan cooled
TSS	total suspended solids
Utility	Evansville Water & Sewer Utility
UV	ultraviolet
WAS	waste-activated sludge
WSE	water-surface elevation
WWTP	wastewater treatment plant

Executive Summary

This document includes a Recommended Plan set forth in Section 5 below, which was submitted to EPA and IDEM for review on May 31, 2013, and rejected by the agencies on June 16, 2014. A final Negotiated Plan set forth in Section 6 below is based upon an agreement between Evansville and EPA and IDEM for increased combined sewer overflow control. The final Negotiated Plan supercedes the Recommended Plan.

In June 2011, the City of Evansville Water & Sewer Utility (Utility) entered into a Consent Decree with the United States and the State of Indiana (the Decree) that requires the Utility to develop and implement a capital plan to control combined overflows (CSOs). The capital plan includes improvements to increase the capacities of the East and West wastewater treatment plants (WWTPs), which are summarized in this Facility Plan (Volume 4 of the Integrated Control Plan).

The existing WWTPs include preliminary treatment, primary clarification, conventional activated sludge, secondary clarification, and disinfection. The East WWTP has an average day capacity of 18 million gallons per day (mgd). Stress testing showed the that East WWTP could treat up to 28 mgd; however, this would require operating disinfection facilities at less than 15 minutes of contact time. For the purposes of facility planning, peak wet-weather capacity was determined to be 26 mgd, which achieves 15 minutes of contact time. The West WWTP is similarly configured but also includes a biological aerated filtration (BAF) facility that operates in parallel with the conventional activated sludge WWTP. The West WWTP has an average day capacity of 30.6 mgd and peak wet-weather capacity of 37 mgd. During dry-weather conditions, the wastewater flows to the WWTPs are well below their respective capacities. However, during wet-weather conditions, flows reaching the WWTPs often exceed their existing capacities.

Various technologies were evaluated to increase the secondary treatment capacity of WWTPs, including biological and chemically enhanced clarification (BioCEC), BAF, and conventional activated sludge for flow rates of 60 mgd and 80 mgd. The 40-mgd secondary treatment option for the East WWTP considered only conventional treatment because the original WWTP design was configured to accommodate a fourth aeration basin and clarifier. The various treatment technologies were evaluated based on capital and lifecycle costs, size and space requirements, operational features, and effluent quality.

BioCEC has the lowest capital and lifecycle costs because the process does not require large tank volumes. However, BioCEC has no supplemental benefits during normal dry-weather conditions, is unfamiliar to existing operators, requires different operating parameters, requires additional staffing while in operation, produces lower-quality effluent, and is a relatively new technology without a history of proven performance.

BAF has been in operation at the West WWTP since 2009. This process has the benefit of combining a biological reactor and solids separation unit in a single structure, which provide a compact system for secondary treatment. However, based on the Utility's experience with BAF, this technology is not well suited to treat peak wet-weather flows for a sustained period of time. During wet-weather conditions, the water velocity though the reactor causes the fixed film bacteria to be removed from the filter media, resulting in high effluent total suspended solids (TSS) concentrations and slow biological recovery.

Conventional treatment has the lowest operation and maintenance costs due to the simplicity of operation and familiarity with the existing operators. The expanded facilities that are constructed to treat wet-weather flows could also be configured to provide redundancy to the existing equipment, which would improve reliability and maintenance flexibility. The additional tank volumes could also be configured to provide biological nutrient removal to meet the anticipated phosphorous limit of 1 milligram per liter. Conventional treatment also provides the highest quality effluent of the options evaluated and would increase the dry-weather capacity of the WWTP.

The East WWTP analysis also evaluated the option of reactivating the abandoned chlorine contact tank and primary effluent (PE) outfall, which were used prior to the WWTP upgrades in the mid-1970s. Activation of the PE outfall would allow the Utility to meet the CSO Policy goals of primary equivalency and disinfection with a savings of $34.3 to $54.9 million compared to the capital cost to provide full secondary treatment.

A range of peak wet-weather capacities (40 mgd to 80 mgd) were evaluated at each WWTP as a means to mitigate CSOs. The lower range (40 mgd) represents the nominal capacity that can be hydraulically conveyed through the facilities without significant piping modifications. The upper range (80 mgd) is limited by the biological mass that can be sustained during current dry-weather conditions, which can then be capable of providing secondary treatment during wet-weather conditions. Originally, the intent was to evaluate three capacities within the range (40 mgd, 60 mgd, and 80 mgd). However, during the process and hydraulic evaluations of the West WWTP, it became evident that the West WWTP could be expanded from 40 mgd to 45 mgd with minimal capital investments. Moreover, similar evaluations of the East WWTP showed that 68 mgd could be achieved using a primary effluent bypass without significant capital investment.

The alternatives to expand the East WWTP to 60 mgd and 80 mgd also require the demolition of the existing solids storage facility. The storage facility is currently used to store dewatered solids prior to landfill disposal or land application. Removal of the solids storage facility would require changes to the Utility's current approach to sludge management. A summary of the capital costs for the various flow ranges and treatment technologies are summarized in Table ES-1.

Table ES-1 Total Capital Costs for WWTP Expansion by Treatment Technology

	Volume Treated in Typical Year (in millions of gallons)	PE Bypass[a,b]	Conventional Treatment	BioCEC[c]
West WWTP				
40[d]	4,660	-	$21,910,000	-
45	6,594	-	$22,163,000	-
60	6,650	-	$57,930,000	$48,470,000
80	6,725[e]	-	$126,820,000	$108,610,000
East WWTP				
40	5,883	$12,730,000	$47,001,000	-
60[f]	5,950	-	$97,610,000	$82,430,000
68[g]	5,958	$21,182,000	-	-
80[f]	5,950	$58,696,000	$113,596,000	$107,576,00

[a] The PE bypass options include the following treatment capacities: 40-PE bypass = 26 mgd secondary treatment/ 14 mgd primary treatment; 68-PE bypass = 28 mgd secondary treatment/40 mgd primary treatment; 80-PE bypass = 40 mgd secondary treatment/40 mgd primary treatment.

[b] Capital costs do not include influent pumping. Pumping facilities are accounted for in Bee Slough Alternatives Analysis estimates.

[c] BioCEC was not considered for expanding treatment capacity to 40 mgd.

Table ES-1 Total Capital Costs for WWTP Expansion by Treatment Technology

[d] The majority of costs to expand capacity to 40 mgd at the West WWTP are due to new headworks facilities.

[e] Volume treated in the typical year with WWTP capacity at 80 mgd was estimated by linear extrapolation of West WWTP 45 mgd and 60 mgd data points.

[f] Capital costs do not reflect additional costs associated with altering the Utility's solids handling process which, due the WWTPs' interconnectedness, could affect both WWTPs. A planning-level evaluation of the Utility's solids handling process can be found in Appendix D of the Facility Plan.

[g] Volume treated in the typical year with WWTP capacity at 68 mgd was estimated by linear interpolation of East WWTP 60 mgd and 80 mgd data points.

As the capacity of the WWTP increases, the frequency of operation at its peak capacity typically decreases due to the variability in the size and duration of wet-weather events. General collection system attributes may also affect this frequency. Therefore, the costs for the various WWTP expansion options were compared to the benefits of additional CSO volume treated in the typical year, as determined by West and East system hydraulic modeling. Figures ES-1 and ES-2 depict this comparison for the West and East WWTPs, respectively. This analysis identified a "knee of the curve" at each WWTP or the inflection point at which investment of capital begins to result in less benefit (CSO volume treated in the typical year).

At the West WWTP, the analysis showed that at a given capacity, neither treatment technology provides significant cost savings over the other from a cost-per-gallon-treated perspective. It also clearly shows that, regardless of treatment technology, the knee of the curve corresponds to a WWTP capacity of 45 mgd. At the East WWTP, the analysis showed that treatment technology can provide significant cost savings, and that the knee of the curve is dependent upon the selected technology. Similar to the West WWTP, there is little difference between conventional treatment and BioCEC from a cost-per-gallon-treated perspective. The cost-benefit curves associated with these two technologies are approximately linear. However, the PE bypass option provides significant cost savings over conventional treatment and BioCEC and has a clear knee of the curve corresponding to WWTP capacity of 68 mgd.

Figure ES-1 West WWTP Cost-Benefit Curve by Treatment Technology

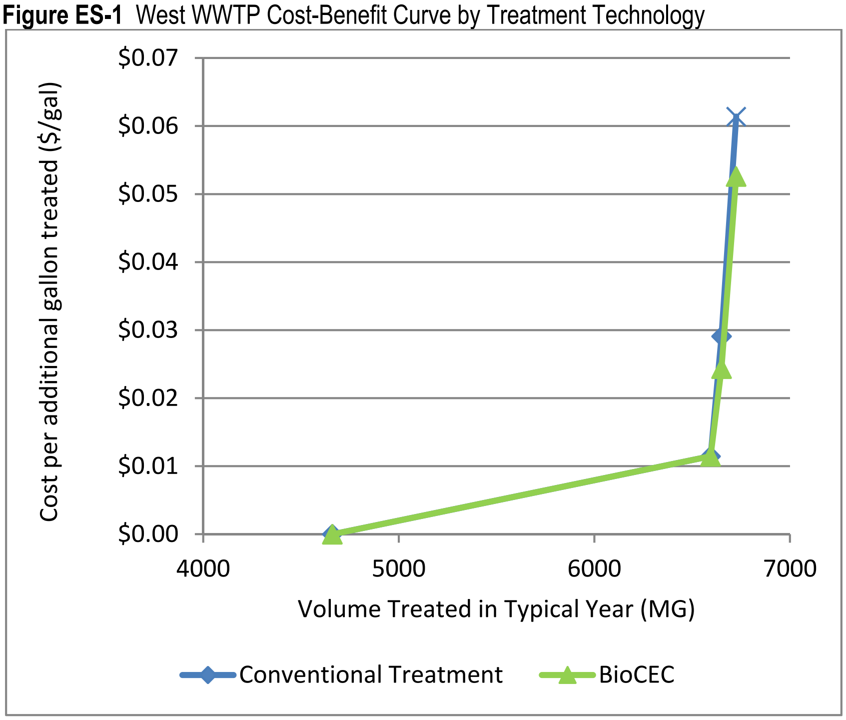
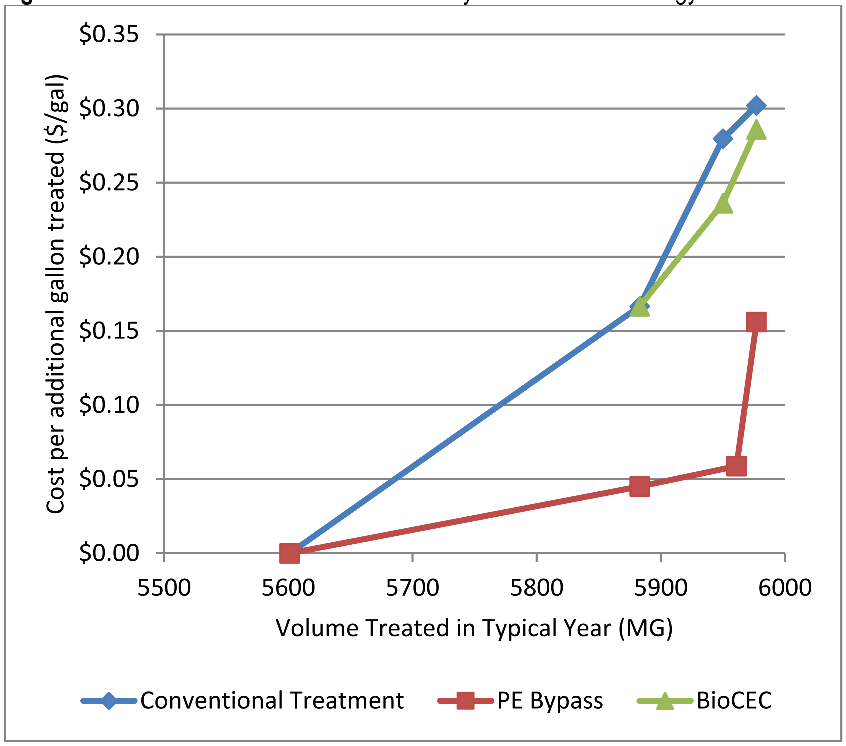


Figure ES-2 East WWTP Cost-Benefit Curve by Treatment Technology



For each flow scenario incorporating the primary effluent bypass, the concentrations of biological oxygen demand (BOD), TSS, and ammonia were estimated for the blended primary and secondary effluent. This analysis indicates that the blended effluent would routinely exceed the weekly and monthly National Pollutant Discharge Elimination System requirements for TSS and BOD. While it would be possible for the primary effluent to meet the disinfection standards, it would not be able to achieve the secondary treatment standards for BOD and TSS concentrations, even when blended with secondary effluent. The Utility could improve the quality of the primary effluent during wet-weather events by storing or relocating the discharge of WWTP recycle flows and adding a coagulant and polymer to primary clarifiers. Therefore, alternatives that incorporate the primary effluent bypass should seek to comply with bacterial water quality standards but not TSS and BOD requirements. The Utility should continue to monitor the primary effluent concentrations of TSS and BOD and should implement reasonable measures to improve primary effluent quality.

As a result of the analysis described above, the Recommended Plan for the East WWTP includes expanding to 68 MGD by means of PE bypass. The Recommended Plan for the West WWTP includes expanding to 45 MGD by means of conventional treatment. During negotiations with EPA, the following are changes were made which make up the Negotiated Plan:

- Elimination of the Primary Bypass features at the West WWTP; instead, modification to the existing primary clarifiers are incorporated to increase hydraulic capacity by 5 MGD.
- Elimination of the PE Bypass features at the East WWTP. Instead, expand capacity to 40 MGD by means of conventional activated sludge; and

- Construction of a wetland treatment system to treat volume from CSO 001, but eliminate infrastructure allowing the Utility to route wetland influent to the secondary treatment process at the East WWTP

Table ES-2 summarizes the recommended and selected plans.

TABLE ES-2: Summary of Recommended and Selected Plans

	Description	Capital Cost (Million $)
West WWTP		
Recommended Plan	New headworks facility (screening, pumping, grit removal) with firm capacity of 45 MGD. Plan also includes 5 MGD bypass around primary clarifiers, new RAS pumps, and new chemical feed/storage building for disinfection.	22.16
Negotiated Plan	Do not utilize the 5 MGD bypass around the primary clarifiers. Otherwise, the negotiated Plan is the same as the Recommended Plan	22.36
East WWTP		
Recommended Plan	Utilize PE Bypass to expand capacity to 68 MGD. Rehab and expansion of existing chlorine contact tanks, along with new chemical feed/storage building and effluent sewer segment for disinfection. Also includes new 65 MGD effluent pump station, and new forcemain from the CSO 001 wetland pump station to secondary treatment facilities at the East WWTP	21.18
Negotiated Plan	Do not utilize PE Bypass. Expand secondary treatment capacity to 40 MGD (new aeration basin, clarifier, two blowers, RAS/WAS system pumping system). Also includes new UV disinfection system and modifications to existing contact tank/effluent sewer to accommodate UV hydraulics. New 40 MGD effluent pump station.	47.00

SECTION 1

Introduction

1.1 Objective

In June 2011, the City of Evansville Water & Sewer Utility (Utility) entered into a Consent Decree with the United States and the State of Indiana (the Decree) that requires the Utility to develop and implement a capital plan to control sewer system overflows and to develop and implement measures to properly operate and maintain the sewer systems and wastewater treatment plants (WWTPs). The required capital plan is referred to as the Integrated Overflow Control Plan (IOCP), which will include the Utility's combined sewer overflow (CSO) Long-term Control Plan (LTCP) and a Sanitary Sewer Remedial Measures Plan (SSRMP). The objectives of the SSRMP and LTCP are to present remedies for sanitary sewer overflows (SSOs) and to reduce the frequency and duration of CSOs, respectively. The IOCP, LTCP, and SSRMP can be found in Volumes 1, 2, and 3, respectively, of this report.

IOCP analysis will identify the optimal combination of conveyance, storage, and treatment projects to mitigate CSOs and SSOs. The treatment component of this evaluation is presented in this document, the Facility Plan for the West and East Wastewater Treatment Plants (Facility Plan). The primary intent of this document is to identify the recommended improvements to increase the capacities of the West and East WWTPs and to provide a level of detail appropriate for comparison of alternatives and preparation of planning-level cost estimates. The Facility Plan also evaluates the lifecycle costs for the various alternatives, incorporating operation and maintenance (O&M) activities over the 40-year planning horizon. Preliminary engineering, which would include a validation of influent flows and loadings, optimization of concepts presented in this report, and a detailed survey of existing site conditions, should be conducted prior to implementation of any projects described in this report.

Overall, this Facility Plan is intended to develop representative costs for expanding the WWTPs for three flow rate scenarios: 40, 60, and 80 million gallons per day (mgd). The costs and benefits associated with these scenarios are incorporated into the overall system West and East Alternatives Analysis, which identifies the optimum combination of treatment and conveyance improvements for the West and East systems. Additional expansion scenarios will be developed as needed to support respective alternatives analysis.

1.2 Facility Plan Document Organization

This Facility Plan is Volume 4 of the IOCP and includes the following sections:

- **Section 1, Introduction.** Section 1 summarizes the Utility's requirements and objectives in compiling this document
- **Section 2, Approach.** Section 2 documents the basis of planning and includes considerations made regarding effluent limits, hydraulic and process modeling, cost estimation, and technology screening.
- **Section 3, West WWTP.** Section 3 describes the existing facilities at the West WWTP and documents the requirements for expanding each unit process at the West WWTP.

- **Section 4, East WWTP.** Section 4 describes the existing facilities at the East WWTP and documents the requirements for expanding each unit process at the East WWTP.
- **Section 5, Recommended Plan – May 31, 2012.** Section 5 describes the recommended plan for each WWTP, as developed for submission to and review by EPA and IDEM on May 31, 2012.
- **Section 6, Final Negotiated Plan – December 4, 2015.** Section 6 describes the final negotiated plan for each WWTP as agreed to by Evansville and EPA and IDEM.
- **Section 7, References.** Section 5 contains bibliographic references for the documents cited in this Facility Plan.

1.3 Related Deliverables

Appendixes B and C of the Decree list the tasks the Utility is required to complete to develop the IOCP and the associated completion schedule. Table 1-1 is an excerpt of Appendix B that lists the deliverables relevant to facility planning for the IOCP development, the deliverable status, and its relationship to facility planning.

Table 1-1 Appendix B Excerpt for Facility Planning

	Deliverable	Due Date	Status	Relationship to Facility Planning
6	Install baffles in the secondary treatment system clarifiers at the West WWTP (Consent Decree paragraph 20.b).	April 30, 2011	Complete	Intended to increase capacity of existing secondary clarifiers, which is a basis for planning.
11	Submit report on capacity of clarifiers.	November 1, 2011	Complete	Documents current operational attributes of the West WWTP.
12	Submit report evaluating the effectiveness of step feed and/or contact stabilization in the secondary aeration basins to maximize wet-weather flow through the secondary treatment at East WWTP and West WWTP (Consent Decree paragraph 19.a and 20.c).	November 1, 2011	Complete	Documents current operational attributes of the WWTPs.
13	Submit stress test protocols that will identify the proposed revision to the Maximum Treatable Flow of the East WWTP and West WWTP. (Consent Decree paragraph 19.d and 20.d).	November 1, 2011	Complete	Documents a plan to identify revised capacities of unit process at the WWTPs.
18	Submit Update to West CSS and East CSS characterization and hydraulic model including development of SSS Hydraulic model.	November 30, 2011	Complete	Documents the current condition and capacity of the unit processes at the WWTPs.
22	Install East WWTP Early Action Upgrades including a second bar screen and fourth influent pump. (Consent Decree paragraph 19.b and c).	March 1, 2012	Complete	Intended to increase existing screening and pumping capacity, which is a basis for planning.

Table 1-1 Appendix B Excerpt for Facility Planning

	Deliverable	Due Date	Status	Relationship to Facility Planning
24	Conduct stress test of East and West WWTP and identify revised Maximum Treatable Flow. (Consent Decree paragraph 19.d and 20.d).	July 31, 2012	Complete	Documents revised capacities of unit processes at the WWTPs.
26	Submit the Draft IOCP to plaintiffs and to public, including but not limited to, CSO/SSS capacity alternatives analysis; the alternatives analysis for the LTCP; the alternatives analysis for the SSRMP; the facility plans for expansions of the East and West WWTPs; and proposed implementation schedules for the SSRMP and LTCP.	July 31, 2012	Complete	Results of facility planning will be incorporated into the IOCP.
29	Submit Final IOCP, including post-construction monitoring plan.	November 30, 2012	Based on Draft IOCP	Any revisions resulting from public and agency comment period will be incorporated.

SECTION 2

Approach

2.1 Basis for Planning

2.1.1 Effluent Requirements

2.1.1.1 Current Limits

Evansville's current effluent requirements for the East and West WWTPs were considered in facility planning. Tables 2-1 and 2-2 summarize the effluent limitations, as required by the WWTPs' associated National Pollutant Discharge Elimination System (NPDES) permits, effective February 1, 2012.

Table 2-1 West WWTP NPDES Effluent Limits

	Quantity or Loading			Quality or Concentration		
Parameter	**Monthly Average**	**Weekly Average**	**Units**	**Monthly Average**	**Weekly Average**	**Units**
5-day carbonaceous biochemical oxygen demand	5,107	7,661	lb/day	20	30	mg/L
Total suspended solids	7,661	11,491	lb/day	30	45	mg/L
Ammonia-nitrogen	2,375	3,575	lb/day	9.3	14	mg/L
Fecal	2,000	---	count/100 mL	---	---	---
Escherichia coliform[a]	125	---	count/100 mL	---	---	---

[a] Daily maximum limits for *Escherichia* coliform are 235/100 mL

lb/day = pounds per day
mL = milliliter
mg/L = milligrams per liter

Table 2-2 East WWTP NPDES Effluent Limits

	Quantity or Loading			Quality or Concentration		
Parameter	**Monthly Average**	**Weekly Average**	**Units**	**Monthly Average**	**Weekly Average**	**Units**
5-day carbonaceous biochemical oxygen demand	3,004	4,506	lb/day	16	24	mg/L
Total suspended solids	5,633	8,449	lb/day	30	45	mg/L
Ammonia-nitrogen	1,746	2,629	lb/day	9.3	14	mg/L
Fecal	2,000	---	count/100 mL	---	---	---
Escherichia coliform[a]	125	---	count/100 mL	---	---	---

[a] Daily maximum limits for *Escherichia* coliform are 235/100 mL.

2.1.1.2 Anticipated Future Requirements

Effluent requirements limiting phosphorus discharge are anticipated in the next permitting cycle, which will occur in 2017. However, the anticipated phosphorus limit is 1 mg/L, which can be achieved by modifying the existing aeration basins to include unaerated zones to promote biological nutrient removal (BNR). Although the costs to provide these modifications are not included in this Facility Plan, alternatives that would support future BNR configurations were noted in the evaluation process. Furthermore, biological phosphorus removal could be incorporated into this Facility Plan at a relatively small cost and could be constructed within the time required by a new permit. If the new permit limits for phosphorus are below 1 mg/L, detailed process modeling would need to be conducted and additional facilities would likely be required.

2.1.2 WWTP Data Analysis

From January 2007 to June 2011, daily plant data for the West and East WWTPs were gathered, including influent, primary effluent, and secondary effluent flow; 5-day carbonaceous biochemical oxygen demand ($CBOD_5$); total suspended solids (TSS); and ammonia concentrations. In addition, information about process conditions at the aeration basins, such as dissolved oxygen concentrations, mixed liquor suspended solids concentrations, temperature, and sludge volume index (SVI), was also gathered.

Influent data were analyzed to estimate daily, monthly, and weekly average and maximum flows and loads to the East and West WWTPs for each year. Daily averages and maximums were obtained using the daily data, while monthly and weekly averages and maximums were obtained using 30-day rolling and 7-day rolling average values, respectively. The historical average annual flows and loads, historical maximum and minimum annual flows and loads, historical maximum monthly loads, and historical maximum weekly loads are summarized in Tables 2-3 through 2-6, respectively.

Table 2-3 Historical Annual Average Flows and Loads to West and East WWTPs[a]

	2007	2008	2009	2010	2011	5-year Average
East Plant						
Flow (mgd)	12.5	12.8	12.2	14.1	16.6	14.0
$CBOD_5$ (lb/day)	18,636	20,419	20,380	23,753	27,105	22,059
TSS (lb/day)	15,396	16,504	15,350	21,944	26,392	19,117
Ammonia-N (lb/day)	1,661	1,480	1,710	1,822	1,954	1,726
West Plant						
Flow (mgd)	15.0	15.0	15.0	13.0	17.0	15.0
$CBOD_5$ (lb/day)	16,557	16,557	15177	16,414	16,417	16,141
TSS (lb/day)	17,460	17,460	17,953	17,294	19,585	18,074
Ammonia (lb/day)	1,127	1,127	1,333	1,416	1,367	1,311

[a] Analysis was conducted on data gathered from January 2007 through June 2011.

Table 2-4 Historical Annual Maximum and Minimum Flows to West and East WWTPs[a]

	2007	2008	2009	2010	2011
East Plant					
Maximum Flow (mgd)	19.2	19.4	19.2	22.1	27
Minimum Flow (mgd)	7.5	7.3	7.2	1.4	9.3
West Plant					
Maximum Flow (mgd)	23.6	21.1	35.1	32.9	36.1
Minimum Flow (mgd)	8.8	8.7	8.4	6.7	9.1

[a] Analysis was conducted on data gathered from January 2007 through June 2011.

Table 2-5 Historical Maximum Monthly Loads to West and East WWTPs[a]

	2007	2008	2009	2010	2011	5-year Maximum
East Plant						
$CBOD_5$ (lb/day)	25,548	28,705	31,703	32,600	31,788	32,600
TSS (lb/day)	27,239	26,365	27,786	32,021	35,052	35,052
Ammonia-N (lb/day)	2,029	1,861	2,205	2,322	2,403	2,403
West Plant						
$CBOD_5$ (lb/day)	22,474	22,474	21,746	21,538	22,536	22,536
TSS (lb/day)	31,101	31,101	30,602	26,831	26,049	31,101
Ammonia (lb/day)	2,361	2,361	2,893	1,774	1,703	2,893

[a] Analysis was conducted on data gathered from January 2007 through June 2011.

lb/day = pounds per day

Table 2-6 Historical Maximum Weekly Loads to West and East WWTPs[a]

	2007	2008	2009	2010	2011	5-year Maximum
East Plant						
$CBOD_5$ (lb/day)	30,491	38,322	44,357	48,723	38,786	48,723
TSS (lb/day)	37629	35,775	44,498	39,983	50,135	50,135
Ammonia-N (lb/day)	2185	2,218	3,085	3,757	2,599	3,757
West Plant						
$CBOD_5$ (lb/day)	34,385	34,385	32,240	26,993	29,642	34,385
TSS (lb/day)	64,250	64,250	41,890	30,794	29,624	64,250
Ammonia (lb/day)	3,483	3,483	5,025	2,162	2,783	5,025

[a] Analysis was conducted on data gathered from January 2007 through June 2011.

Using the data presented in Tables 2-3 through 2-6, peaking factors were determined for the peak monthly and peak weekly conditions. Table 2-7 summarizes these values. These values appear to be high relative to typical municipal WWTPs. Potential causes for the high peaking factors could be variable load inputs from local industries or high loadings from "first flush" events. Therefore, these values should be validated prior to design of any major capital expenditures.

Table 2-7 Monthly and Weekly Peaking Factors for West and East WWTPs

	$CBOD_5$	TSS	Ammonia
East Plant			
Peak Month to Average	1.48	1.83	1.39
Peak Week to Average	2.21	2.62	2.18
West Plant			
Peak Month to Average	1.40	1.72	2.21
Peak Week to Average	2.13	3.55	3.83

2.1.3 Wet-weather Flows and Loads

At the onset of a wet-weather event, influent TSS and $CBOD_5$ loads generally increase because the solids that have been deposited in the collection system are flushed to the WWTP. Since the Utility has only limited discrete wet-weather sampling data, the following procedure was implemented to estimate the current wet-weather loading using the daily composite samples collected at the WWTPs:

- For each year, daily flows and loads to the plants were plotted to develop an understanding of which months were representing dry- and wet-weather events. Generally, low flows and loads were observed between July and October for both West and East WWTPs. Thus, dry-weather flow data were obtained by averaging the flows to the plants between July and October. Any flow in excess of that was assumed to be wet-weather flow.
- $CBOD_5$, TSS, and ammonia loads for the entire dataset were averaged for flows below and above the average dry-weather flow. The difference in the average loads between these two scenarios was converted to a concentration value to determine whether (and to what extent) excess $CBOD_5$, TSS, and ammonia were being flushed into the WWTPs during a storm event.
- If a significantly high difference in concentration between the dry- and wet-weather flows was observed, then loads were plotted against wet-weather flows to the plants to determine additional pounds of load contribution per mgd flow during a storm event through a linear regression analysis.

For the West WWTP, an average dry-weather flow from 5 years of historical data was found to be 9 mgd. Any flow in excess of 9 mgd was assumed to be a contribution from wet-weather events. A significant difference in $CBOD_5$ and TSS loads for flows below and above 9 mgd was observed. However, for ammonia, no significant increase was found during wet-weather conditions. Thus, TSS and $CBOD_5$ loads were plotted for all flow data above 9 mgd, and a linear regression was used to determine additional pounds of load contribution per mgd flow during a storm event. It was estimated that an additional 296 pounds of $CBOD_5$ and 556 pounds of TSS were being flushed into the WWTPs per mgd of flow during wet-weather events.

For the East WWTP, an average dry-weather flow from 5 years of historical data was found to be 11 mgd. Any flow in excess of 11 mgd was assumed to be a contribution from wet-weather events. A significant difference in $CBOD_5$ and TSS loads for flows below and above 11 mgd was observed. However, for ammonia, no significant increase in concentration was found for wet-weather conditions. Thus, TSS and $CBOD_5$ loads were plotted for the East WWTP against all flow data above 11 mgd, and a linear regression was used to determine additional pounds of load contribution per mgd flow during a storm event. It was estimated that an additional 317 pounds of $CBOD_5$ and 513 pounds of TSS were being flushed into the WWTP per mgd flow during wet-weather events. The additional loadings caused by these flushing events typically occur during the beginning of the wet season and have durations of 1 day.

2.1.4 Estimating Wet-weather Loads for Facility Planning

As part of the facility planning effort, three wet-weather flow scenarios were considered: 40, 60, and 80 mgd. To determine the $CBOD_5$, TSS, and ammonia loads at these flow scenarios, the following steps were implemented:

- According to 2000 and 2008 Evansville Metro area census population data, a 0.27 percent population growth occurred in this 8-year period. In the absence of any other census data, similar growth was assumed for Evansville for the next 20 years. A 20-year planning horizon is typically used to estimate future flows and loads to the WWTPs even though a 40-year period was used for the present worth comparisons of alternatives for the IOCP. This resulted in a growth factor of 1.0554, using the following equation:

  $$Growth\ Factor = (1 + 0.27\%)^{20} \qquad (1)$$

  This growth factor was added to the 5-year historical average $CBOD_5$, TSS, and ammonia values to determine 20-year growth adjusted average loads.

- To the growth-adjusted average loads, the peaking factors summarized in Table 2-7 were added to determine design peak monthly and weekly loads to each plant.

- Finally, to incorporate additional $CBOD_5$ and TSS loads contributed only during wet-weather events, the pounds-per-mgd flow, as explained above, was added to any flows above 15.5 mgd for the West WWTP and 14 mgd for the East WWTP because these flows represented the growth-adjusted average dry-weather flow to the plants. For this analysis, only a 1-day storm event was assumed.

- Tables 2-8 and 2-9 summarize the peak monthly and peak weekly $CBOD_5$, TSS, and ammonia loads to the West and East WWTPs, respectively, for dry- and wet-weather conditions.

Table 2-8 West WWTPs Peak Monthly and Peak Weekly $CBOD_5$, TSS, and Ammonia Loads

	$CBOD_5$ (lb/day)	TSS (lb/day)	Ammonia (lb/day)
Growth-adjusted Scenario (not including wet-weather loads)			
Peak Month	27,813	32,824	3,053
Peak Week	36,290	67,809	5,303
40-mgd Flow Scenario			
Peak Month	31,078	46,470	3,053
Peak Week	43,555	81,456	5,303
60-mgd Flow Scenario			
Peak Month	36,998	57,590	3,053
Peak Week	49,475	92,576	5,303
80-mgd Flow Scenario			
Peak Month	42,918	68,710	3,053
Peak Week	55,395	103,696	5,303

Table 2-9 East WWTPs Peak Monthly and Peak Weekly $CBOD_5$, TSS, and Ammonia Loads

	$CBOD_5$ (lb/day)	TSS (lb/day)	Ammonia (lb/day)
Growth-adjusted Scenario (not including wet-weather loads)			
Peak Month	34,406	36,993	2,536
Peak Week	51,422	52,912	5,522
40-mgd Flow Scenario			
Peak Month	42,636	50,312	2,536
Peak Week	59,652	66,231	5,522
60-mgd Flow Scenario			
Peak Month	48,976	60,572	2,536
Peak Week	65,992	76,491	5,522
80-mgd Flow Scenario			
Peak Month	55,316	70,832	2,536
Peak Week	72,332	86,751	5,522

2.1.5 Modeling

2.1.5.1 Hydraulic Modeling

All hydraulic modeling to assess required upgrades for the three flow scenarios (40, 60, and 80 mgd) at the Evansville WWTPs was performed using CH2M HILL's WinHydro software.

Modeling began at the Ohio River and continued upstream through the Headworks Facility. The models accounted for WWTP influent from the collection system as well as return-activated sludge (RAS), which circulates between the secondary clarifiers and the aeration tanks. A Hazen-Williams roughness coefficient of C=100 was assumed initially for all reinforced concrete pipe. This factor is approximately equal to a Manning's n=0.013. Engineering drawings provided the information (configuration, geometry, elevation, etc.) necessary to develop the hydraulic models. These drawings originated between 1955 and 2007. The most hydraulically conservative flow path was modeled, and all elevations were appropriately adjusted to the North American Vertical Datum of 1988.

A series of surveys were conducted at each WWTP to verify elevations of specific channels, structures, and weirs. The models also were calibrated using the wet-weather survey data by modeling the observed wet-weather event and comparing predicted water-surface elevations (WSEs) with observed WSEs. Discrepancies were addressed primarily by verifying and adjusting model elements based on engineering drawings, WWTP operations, and field observations. Minor losses were also adjusted where practical.

Running the calibrated models for each flow scenario identified existing hydraulic bottlenecks and predicted WSEs resulting from modifications of existing infrastructure and proposed additions to the treatment processes. The primary output of the models was incorporated into hydraulic profiles of the recommended flow scenario for each WWTP, as discussed in Section 5.

2.1.5.2 Process Modeling

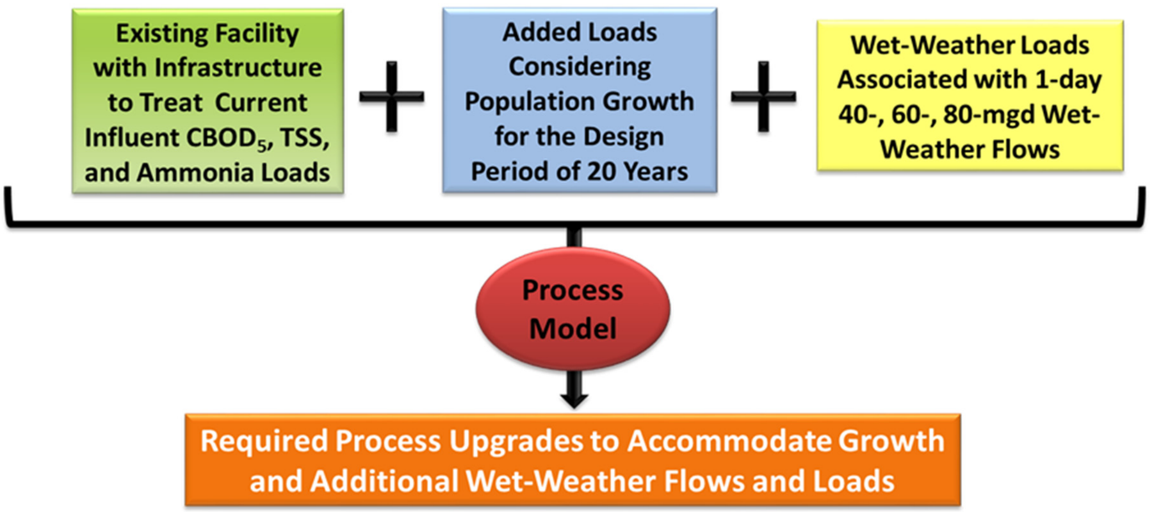
All process modeling to assess required upgrades for the three flow scenarios (40, 60, and 80 mgd) was performed using CH2M HILL's Pro2D whole-plant simulator. This tool provided a flexible and robust modeling approach for characterizing, sizing, and predicting WWTP performance. Pro2D was used to calculate and document process sizing and operational information related to the evaluation of WWTP upgrades. The Pro2D activated-sludge model uses the International Water Association's ASM2d model. A second model, PClarifier, was used to provide detailed modeling of secondary clarifier performance and capacity using state-point analysis.

Process modeling was conducted using traditional capacity analysis assumptions, such as the maximum 30-day loadings (based on January 2007 through June 2011 data) during minimum month cold-weather conditions. Maximum 30-day loadings and minimum month temperatures were assumed because the WWTP must meet monthly permit limits, not only during average loadings, but also during peak month loadings and winter temperatures when biological treatment rates are slowest. Wastewater temperature data were not available; therefore, based on judgment at other WWTPs at the same approximate latitude, 12° Celsius [C] (54 Fahrenheit [F]) was assumed.

Ammonia limits were added to the most recent NPDES permit. Therefore, all model runs were performed to estimate capacity with nitrification. Based on the assumed minimum-month temperature of 12° C and existing ammonia limits, a solids retention time that would provide a 1.75 nitrification safety factor was used to estimate capacity with nitrification for both plants. In addition, based on historical SVI data, as well as communications with plant staff, it was believed that both plants could maintain an SVI of 150 or less for purposes of maximizing capacity during wet-weather events. Thus, for all model runs an SVI of 150 was used.

Figure 2-1 illustrates the approach used to model the process upgrades for both West and East WWTPs. Process models were initially developed and calibrated based on historical data obtained from the WWTPs. The calibrated model was then updated to accommodate the flows and loads for the 20-year design life of the facilities, as well as additional contributions from wet-weather events.

Figure 2-1 Process Modeling Approach for West and East WWTPs



2.2 Basis for Cost Estimating

The *Facility Plan Cost Estimating Guidelines* (CH2M HILL, 2012a) was used to complete the estimates for each unit process alternative in a uniform manner. Appendix A presents more detailed information about assumptions made and specific markups included.

The costs presented in this report are total capital costs, which include construction costs with contractor capital cost markups (client cost) markups. Forty-year lifecycle costs are also presented. All costs are shown in January 2012 dollars. Escalation was only accounted for in development of lifecycle costs. Estimates provided are considered planning-level estimates, with an expected accuracy of +50 percent to -30 percent.

2.3 Technology Screening

The primary intent of this report is to develop representative cost estimates for expanding the wet-weather treatment capacities at the West and East WWTPs. Therefore, flow equalization alternatives are not considered in this report but, rather, are included in the development of the systemwide alternatives.

2.3.1 Primary Treatment

Primary treatment is the method used for removing TSS in the influent wastewater by providing sufficient detention time to allow the particles to settle. This process was the predominant treatment method at the West WWTP for the first 20 years of operation. When the Utility added the biological secondary treatment system in the 1970s, it retained the primary treatment process because it reduced the $CBOD_5$ in the wastewater prior to secondary treatment, thus lowering both the capital and O&M costs of the secondary system.

Chemically enhanced primary treatment is a very common approach to increasing primary treatment capacity during wet-weather events because it produces a well settling sludge, allowing the clarifier to maintain good solids removal at much higher overflow rates. However, this option is not well suited for the West and East WWTPs because of hydraulic restrictions that would require costly modifications to correct. Also, the stress tests showed that the clarifiers provide adequate removal during peak flow conditions. Therefore, chemically enhanced primary treatment was not considered for expanding primary treatment facilities.

Most wet-weather treatment expansions that incorporate secondary treatment do not include expanding primary treatment because the cost (capital and lifecycle) associated with primary clarifiers is not offset by the savings in the secondary treatment system. Also, removal efficiencies under dilute influent conditions (typical of wet-weather events after the first flush has occurred) are relatively low. Furthermore, the WWTPs—particularly the West WWTP—have limited space for expansion of primary clarifiers. Therefore, expansion of primary clarification was not evaluated as part of facility planning. However, modeling and stress testing identified multiple improvements that could be made to improve wet-weather hydraulics. The existing primary clarifiers and associated influent and effluent channels have capacity to accommodate 40 mgd if simple modifications are made (for example, adjusting existing primary influent gates to sufficiently reduce head loss). Hydraulics could further be improved using a number of relatively simple modifications, such as:

- Removing the primary effluent Parshall flumes
- Raising the primary effluent weirs
- Upsizing the primary influent gates

2.3.2 Secondary Treatment

A cursory evaluation of secondary treatment options for the West and East WWTPs was conducted with the intent of developing cost estimates for the additional secondary treatment facilities that are needed to treat primary effluent and/or raw influent flow as described in this Facility Plan. This section provides a summary of the evaluation. Appendix B provides the technical memorandum further describing secondary treatment system.

Three alternative technologies were considered: biological aerated filtration (BAF), activated sludge, and biological and chemically enhanced clarification (BioCEC). The first alternative, BAF, was evaluated only as an option in the expansion of the East WWTP. However, from the Utility's experience with BAF at the West WWTP, it is clear that this technology is not well suited to treat wet-weather flows at sustained peak flows because it has high construction and O&M costs, adds an additional level of unnecessary complexity to operations, and lacks operational flexibility compared to other technologies. Therefore, BAF is not recommended for further evaluation.

The activated sludge alternative was evaluated for both the West and East WWTPs. Currently, the WWTPs operate in plug flow mode. However, wet-weather modeling was conducted with the assumption that step feed or contact stabilization operating modes would be used. In summary, expansion would involve adding new aeration tanks, secondary clarifiers, and supporting equipment. Expansions were planned such that the newly expanded system would treat up to 40 mgd of primary effluent, while flows in excess of 40 mgd would bypass primary treatment and would be conveyed directly to aeration facilities. To match the hydraulic profile of the existing facilities, the new aeration tanks would require a dedicated clarifier. The new aeration tanks would provide additional operational benefits during dry-weather conditions.

BioCEC was evaluated as a high-rate treatment alternative for both the West and East WWTPs. It was considered an option, primarily due to its small footprint and the constrained site conditions. In summary, a 20-mgd BioCEC treatment train would involve constructing a short detention time biological contact tank ahead of a ballasted flocculation high-rate clarification system (Actiflo by Kruger), along with a small equipment/chemical storage building. RAS from the existing activated sludge process would be diverted to the biological contact tank, where it would mix with the excess wet-weather flow. A 40-mgd BioCEC system would require two of the trains described above. This system would be intended only to treat wet-weather flows in excess of 40 mgd. It would remain offline during all other times.

The alternatives were evaluated based on size and space requirements, operational features, present worth costs, and effluent quality. During the development of the alternatives, the following items were identified:

- Conventional treatment has the lowest O&M costs but has the highest capital and present worth costs.
- Conventional treatment provides the best quality effluent, operational reliability, and flexibility.
- Conventional treatment improvements also provide expanded dry-weather capacity of the WWTP to accommodate future growth.
- Conventional treatment could be used year-round to provide added redundancy to existing process units and could be configured to provide BNR to meet the anticipated phosphorus limits of 1 mg/L.
- BioCEC provides no supplemental benefits during dry-weather conditions.
- BioCEC requires greater attention during operations, and the current staff has no experience operating this type of system.

Therefore, it was assumed the Utility would use the budgetary cost estimates developed for the conventional treatment alternative in the IOCP. During preliminary design, the influent flows and loads should be validated, and opportunities for optimizing the WWTP configurations should be identified.

2.3.3 Disinfection

A cursory evaluation of disinfection options for the West and East WWTPs was conducted with the intent of developing cost estimates for the disinfection facilities that are needed to treat effluent produced in the liquid treatment units described in this Facility Plan. This section provides a summary of the evaluation. Appendix C provides the technical memorandum further describing disinfection.

Cost and non-cost factors were considered for two alternatives, which employed two different disinfection technologies. The first alternative would use liquid sodium hypochlorite to disinfect flows. Existing contact tanks would be expanded as necessary to provide 15 minutes of chlorine contact time at peak flow conditions, and new chemical feed/storage facilities would be constructed. Minor modifications to the existing tanks would be required to accommodate hydraulics.

The second alternative would use a combination of ultraviolet (UV) and liquid sodium hypochlorite for disinfection. UV would provide disinfection for flows up to 40 mgd, and sodium hypochlorite would be used for flows in excess of 40 mgd (wet weather). Depending on the flow scenario, the existing contact tanks would either be repurposed for UV disinfection or expanded as necessary to provide 15 minutes of chlorine contact time during peak flow conditions. Minor modifications to the existing tanks would be required to accommodate hydraulics. Some scenarios at the East WWTP would require a stand-alone UV structure. Scenarios using liquid sodium hypochlorite would require new chemical feed/storage facilities.

The alternatives were evaluated based on present worth costs, effluent water quality, operational considerations, and environmental considerations. During the development of the alternatives, the following items were identified:

- At the West WWTP, capital and present worth costs for the sodium hypochlorite alternative are significantly lower than those of the UV alternative for the 40-mgd scenario. However, the present worth costs for the 60- and 80-mgd scenarios are so similar that non-cost factors should have a greater influence on the final recommendation.
- At the East WWTP, the capital and present worth costs for the UV alternative are significantly lower than the corresponding costs for the sodium hypochlorite alternative in the 40-mgd scenario. However, the present worth costs for the 60- and 80-mgd scenarios are so similar that non-cost factors should have a greater influence on the final recommendation.

Therefore, if the West WWTP is expanded to 40 mgd, it was assumed the Utility would use the sodium hypochlorite alternative as a basis for facility planning. If the West WWTP is expanded to 60 or 80 mgd, it was assumed the Utility would use the UV + sodium hypochlorite alternative as a basis for facility planning.

If the East WWTP uses the primary effluent bypass, then it is assumed that the Utility would rehabilitate the old chlorine contact tanks and disinfect both the primary effluent and secondary effluent with sodium hypochlorite. However, for all three secondary treatment expansion scenarios, it is assumed that the Utility would use UV + sodium hypochlorite for disinfection.

These assumptions are intended only for this wet-weather Facility Plan. Alternatives should be considered further prior to implementation.

2.3.4 Solids Handling

A cursory evaluation of solids-handling options for the West and East WWTPs was conducted with the intent of identifying and developing representative cost estimates for the solids-handling facilities that are needed to process the primary sludge and waste-activated sludge (WAS) produced in the liquid treatment units. This section provides a summary of the evaluation. Appendix D provides the technical memorandum further describing solids handling. This evaluation was not conducted with the goal of developing a solids-handling master plan.

Three alternatives were considered. The first would eliminate the digestion process at both WWTPs. Each WWTP would have onsite facilities to dewater undigested sludge, which would be hauled to the landfill. Hauling undigested sludge is currently an approved practice in Indiana. In many instances, this approach is the least costly and easiest to operate. However, it would not allow for beneficial use of the sludge, either as a source for methane production or to use its nutrient value for land application. Therefore, the thickening and dewatering facilities developed as part of this alternative could be integrated into a future solids handling system that would provide beneficial reuse of biosolids.

The second alternative is continuation of the current operation of anaerobic digestion at both WWTPs, sludge transfer between the two WWTPs, sludge dewatering at the East WWTP, and landfill disposal. This alternative also includes evaluating the option of installing dewatering facilities at the West WWTP and decommissioning the transfer sludge line between the West and East WWTPs, which is currently in poor condition.

The third alternative would be a combination of the first two alternatives. The digestion process at the East WWTP would continue, along with the potential for cogeneration of power, and the West WWTP digesters would be decommissioned. Undigested sludge would be pumped from the West WWTP to the East WWTP. The West WWTP sludge would either be introduced into the East WWTP digesters or would be dewatered without digestion, depending on available capacity at the East WWTP. This alternative also includes evaluating the option of installing dewatering facilities at the West WWTP and decommissioning the transfer sludge line between the West and East WWTPs.

During the development of the alternatives, the following items were identified:

- The existing digesters at the East WWTP have marginally insufficient capacity to process future peak month loadings. Given the slight difference between the solids loading requirements and digester capacity, the Utility should defer any decision to expand the digesters, continue to closely monitor the solids loadings, and validate the future digester capacity requirements with additional data.
- With the exception of the previously noted East WWTP digester capacity concern, the existing solids-handling facilities have sufficient capacity to treat the additional wastewater loads from the projects associated with the IOCP. However, many components of the solids-handling facilities are in need of rehabilitation due to their age and condition.
- Options that require a new digester have the highest capital and lifecycle costs.
- Replacement of the sludge transfer line has a higher capital and lifecycle cost than building a new dewatering facility at the West WWTP. The transfer line may also include more inherent risks for construction.
- Options that eliminate digestion at the West WWTP have the lowest capital and lifecycle costs.

There are a number of sludge-handling options available, with varying complexities, costs, and benefits (economic and non-economic), that the Utility should consider prior to implementation of any major solids handling improvement. A significant non-economic factor of concern to the Utility is the potential for odor control issues associated with processing undigested raw sludge at the West WWTP; especially in regards to the surrounding community. In addition, raw sludge is generally limited to being landfilled. Consequently, any future restrictions on landfilling raw sludge from the West WWTP would be problematic. The lowest cost alternative that results in a stabilized sludge at both WWTPs is Alternative 2B, which would include continued digestion at both WWTPs, rehabilitation of the West WWTP digester system, abandoning the sludge transfer line, and new dewatering at each WWTP. Properly digested and stabilized sludge has less potential for producing odors. It also has the benefit of being a Class B biosolid which can be land applied should landfilling become restricted or discontinued, or a beneficial land application opportunity becomes available.

Therefore for wet-weather planning purposes, this evaluation assumes the Utility would plan for a capital cost of $27.8 million, which is associated with Alternative 2B.

2.3.5 Effluent Pumping – East WWTP

A cursory evaluation of effluent pumping options for the East WWTP was conducted with the intent of identifying and developing cost estimates for effluent pumping facilities needed to convey effluent from the liquid treatment units described in this Facility Plan. This section provides a summary of the evaluation. Appendix E provides the technical memorandum further describing effluent pumping.

Effluent pumping options were evaluated at two locations. Alternative 1 would involve constructing the effluent pump station on the north side of the East WWTP and repurposing approximately 2,000 linear feet (LF) of the existing interceptor sewer as a force main. A new force main would be installed from the pump station to the existing interceptor sewer, and the existing interceptor chamber would be sealed. In addition, a 60-inch sluice gate would be installed at the Levee Authority's K-4 Pump Station bypass line. This alternative would require relocating the existing storm sewer at Veterans Memorial and Waterworks Roads.

Alternative 2 would involve constructing the effluent pump station at Sunset Park, overtopping the existing interceptor sewer, and repurposing approximately 600 LF of the downstream end of the existing interceptor sewer as a force main. The upstream segments of the interceptor would be reused in their present condition and would remain a gravity system. This alternative would also require installing a 60-inch sluice gate at the existing K-4 Pump Station bypass line.

The alternatives were evaluated based on constructability, operational considerations, environmental considerations, customer/public relations issues, and present-worth costs. During the development of the alternatives, the following findings were identified:

- The capital and total combined costs for Alternative 2 were significantly lower than the costs associated with Alternative 1.
- Alternative 2 is more beneficial to the Utility from constructability and environmental standpoints.
- The Utility has extensive experience operating pump stations remotely, so operational considerations can be adequately addressed with Alternative 2.

An aesthetically attractive pump station could be constructed to mitigate public concerns, so customer/public relations issues can be addressed with Alternative 2. Therefore, it was assumed the Utility would use the budgetary cost estimates developed for Alternative 2—Pump Station at Sunset Park—in the IOCP.

SECTION 3

West WWTP

3.1 System Characterization

A summary of the West WWTP's existing facilities is provided below. The following documents provide more details regarding the condition, capacity, and operational attributes of the West WWTP:

- *West WWTP Wet-Weather SOPs* (CH2M HILL, 2012b)
- *West WWTP Step Feed and Contact Stabilization Study* (CH2M HILL, 2011a)
- *West WWTP Stress Testing Report* (CH2M HILL, 2012c)

3.1.1 WWTP Overview

The West WWTP was originally constructed in the mid-1950s. Activated-sludge secondary treatment was added in the early 1970s, and BAF secondary treatment, which operates in parallel with the activated-sludge process, was added in 2009. Currently, the West WWTP provides preliminary treatment, primary clarification, secondary treatment, and disinfection.

The NPDES permit states that the West WWTP has an average design flow of 21.7 mgd and peak sustained wet-weather flow of 30.6 mgd. The weekly and monthly mass limits in the permit are based on the sustained peak wet-weather flow. Under favorable operating conditions, the West WWTP's current peak wet-weather capacity is 37 mgd.

Raw wastewater enters the WWTP via a gravity sewer. The wastewater flows through a single mechanically cleaned bar screen before being pumped by the influent pumps. The influent pumps discharge upstream of a single vortex grit removal unit. After grit removal, the wastewater flows to the primary clarifiers. Primary effluent is split between the aeration tanks and the BAF for secondary treatment. Mixed liquor from the aeration tanks flows to the secondary clarifiers for liquid/solids separation. Secondary effluent from the clarifiers and from the BAF flows into the chlorine contact tank for disinfection. The disinfected effluent is dechlorinated prior to discharge into the Ohio River. Table 3-1 summarizes the unit processes at the West WWTP.

Table 3-1 Summary of West WWTP Unit Processes and Design Capacities

Unit Process	Type	Quantity	Size	Design Capacity
Preliminary Treatment				
Influent Screening	Mechanical	1	0.25-inch openings	40 mgd
Influent Pumping	Engine Driven	2		20 mgd
	Motor Driven	1		20 mgd
Grit Removal	Vortex Unit – 360° design Pista Grit	1	19-foot diameter	50 mgd
Primary Treatment				
Primary Clarification	Rectangular, chain and flight system	6	137-foot-long by 32-foot-wide by 8-foot, 8-inch SWD	39.6 mgd[a]

Table 3-1 Summary of West WWTP Unit Processes and Design Capacities

Unit Process	Type	Quantity	Size	Design Capacity
Secondary Treatment: Activated Sludge				
Aeration Tanks	Plug-flow tanks with ceramic grid fine bubble aeration system	3	342-foot-long by 30-foot-wide by 15-foot SWD	1.15 MG, each
Blowers	Positive displacement	4	5,050 cfm, each	15,150 cfm with largest unit out of service
Clarification	Circular, flat-bottom, peripherally fed	3	105-foot-diameter, 12-foot SWD	33 mgd[a]
Return Sludge Pumping	Non-clog, horizontally mounted	3	6-inch-diameter suction by 8-inch-diameter discharge; 50-hp TEFC motors	1,500 gpm at 64.6-foot TDH
Waste Sludge Pumping	Non-clog, horizontally mounted	3	6-inch-diameter suction by 6-inch-diameter discharge; 5-hp TEFC motors	625 gpm at 10.7-foot TDH
Secondary Treatment: BAF				
BAF	Aerated upflow w/ polystyrene beads	1	Six 11.5-foot-deep cells with bed of 4.5-mm polystyrene beads	20-mgd peak hour, 12.5-mgd sustained peak; operated at 14 mgd
Disinfection				
Chlorination	Gas chlorinators	2	2,000 lb/day	45.2 mgd
	12-pass serpentine contact tank	1	471,022 gallons	43 mgd[a]
Dechlorination	Sodium bisulfate pumps	2	20.8 gallons per hour	173 mgd
Other				
Effluent Pumping	Wet/dry well	3	20 mgd each	20 mgd each

[a] Capacity based on stress testing conducted April 2012. Stress Testing Report (CH2M HILL, 2012c) provides more information.

cfm = cubic feet per minute
gpm = gallons per minute
hp = horsepower
MG = million gallons
mm = millimeter
SWD = sidewater depth
TDH = total dynamic head
TEFC = totally enclosed, fan cooled

WAS is thickened by gravity belt thickeners. Primary sludge and thickened secondary WAS are then pumped to the primary anaerobic digesters. The digested sludge is thickened by decanting in a secondary digester and is pumped through an interplant pipeline to the East WWTP, where it is dewatered with belt filter presses and trucked to an onsite storage pad. From there, dewatered solids are hauled to a landfill for disposal.

3.2 Unit Process Evaluation

3.2.1 Headworks

The existing Headworks Facility includes influent screening, pumping, and grit removal. However, there are many problems with the existing facility. For example, it does not have redundant screenings capability, is in need of major rehabilitation, and cannot operate at the lower wet-well level that is needed for future IOCP projects. Therefore, this Facility Plan includes a complete replacement of the existing Headworks Facility. Table 3-2 presents a summary of requirement for the new Headworks Facility for the three flow scenarios.

Table 3-2 Summary of Requirements for New Headworks Facility

	40 mgd	60 mgd	80 mgd
Screening Required	Two 40-mgd, 0.25-inch fine screens with washer/compactor	Two 60-mgd, 0.25-inch fine screens with washer/ compactor	Four 40-mgd, 0.35-inch fine screens with washer/ compactor
Grit Removal Required	Two 20-mgd vortex grit units with classifying, washing, and dewatering equipment	Two 30-mgd vortex grit units with classifying, washing, and dewatering equipment	Four 20-mgd vortex grit units with classifying, washing, and dewatering equipment
Pumping Required	Three 20-mgd pumps	Four 20-mgd pumps	Five 20-mgd pumps
Other	Headworks Facility with wet well/dry well ~35 feet deep Influent diversion structure (~15 feet by 15 feet) with actuated sluice gates	Headworks Facility with wet well/dry well ~35 feet deep Influent diversion structure (~15 feet by 15 feet) with actuated sluice gates New flow splitter structure with automated plug valves	Headworks Facility with wet well/dry well ~35 feet deep Influent diversion structure (~15 feet by 15 feet) with actuated sluice gates New flow splitter structure with automated plug valves

As shown in Table 3-2, the new Headworks Facility at the West WWTP would include screening, grit removal, and influent pumping. The screening channel depth and wet-well depth are expected to be approximately 25 feet and 35 feet, respectively. By raising the water level in the influent channel, 60-mgd screening capacity could be achieved with 40-mgd screening equipment. In addition, all scenarios would involve constructing an influent diversion structure to route flow to the new facility. The 60- and 80-mgd scenarios would also include an effluent splitter structure to bypass flows in excess of 40 mgd around primary treatment facilities. The bypassed flow would be conveyed directly to new aeration basins.

3.2.1.1 Cost Estimates

Cost estimates were prepared for all alternatives in accordance with the basis for cost estimating guidelines discussed in Section 2.2. Total capital costs, as well as net present value (NPV) 40-year O&M costs, are summarized for each flow scenario in Table 3-3. Appendix A provides more detailed estimates.

Table 3-3 Cost Summary of New Headworks Facility

Flow	Total Capital Cost ($)	NPV 40-year O&M Cost ($)
40 mgd	19,780,000	1,700,000
60 mgd	21,348,00	1,900,000
80 mgd	41,228,000	2,000,000

3.2.2 Secondary Treatment

An evaluation of secondary treatment options was performed with the intent of identifying and developing cost estimates for the additional secondary treatment facilities. This evaluation identified conventional treatment as the most beneficial to the Utility. Therefore, this section describes alternatives for expanding the West WWTP's secondary treatment capacity to 40, 60, and 80 mgd, via expansion of the activated-sludge system. Appendix B provides more details regarding the secondary treatment evaluation.

The existing secondary treatment system at the West WWTP includes conventional activated-sludge treatment along with a BAF system. The two treatment systems typically operate in parallel, with effluent flow streams combining prior to disinfection. Hydraulic modeling identified that a bottleneck downstream of the chlorine contact tank limits the West WWTP to 37 mgd and the activated sludge system to 25 mgd. If the bottleneck is addressed, the West WWTP could treat 45 mgd, with freeboard being the new hydraulic limitation. The Utility corrected this bottleneck in August 2012. However, adequate wet-weather flow has not occurred since the adjustment; therefore, the new maximum capacity has not been confirmed.

The existing activated-sludge system has three parallel trains of aeration basins with dedicated clarifiers. The WWTP hydraulic profile will not allow a splitter box to be constructed between the aeration basin and clarifier; therefore, alternatives that require construction of a new clarifier would also require construction of a new aeration basin.

Table 3-4 summarizes the requirements for expanding secondary treatment capacity to 40, 60, and 80 mgd.

Table 3-4 Summary of Secondary Treatment Expansion Requirements

	40 mgd	60 mgd	80 mgd
Additional Clarifier Capacity Required	None	One 150-foot-diameter secondary clarifier and piping	Two 150-foot-diameter secondary clarifier and piping
Additional Aeration Capacity Required	None	One 0.625-MG aeration basin and piping	Two 0.52-MG aeration basins and piping
Additional Blower Capacity Required	None	Two blowers with 9,300 scfm each (one duty/one standby)	Two blowers with 10,480 scfm each (one duty/one standby)
Additional RAS/WAS Pumping Capacity Required	Two centrifugal RAS pumps; 2,100 gpm each	Two centrifugal RAS pumps; 2,100 gpm each (for existing trains) Three RAS pumps; 4,000 gpm each (for new trains)	Two centrifugal RAS pumps; 2,100 gpm each (for existing trains) Three RAS pumps; 4,000 gpm each (for new trains)
Other	None	None	Aeration basin influent splitter box

scfm = standard cubic feet per minute

3.2.2.1 40-mgd Expansion

Stress testing of the secondary clarifiers at the West WWTP showed that under existing operating conditions (April 2012), the secondary treatment system has a capacity of 33 mgd. However, process modeling indicated that an additional 3 mgd of RAS flow would be needed to meet the requirements under conditions with higher solids loadings; therefore, additional RAS pumping is required.

3.2.2.2 60-mgd Expansion

Hydraulic and process modeling indicated that expanding the West WWTP to 60 mgd would require additional secondary clarifier capacity. In addition to the modifications and expansions required for the 40-mgd scenario, a 150-foot-diameter secondary clarifier and a 0.625-MG aeration tank would also be needed. The new treatment train would be located west of the existing aeration tanks and could be connected to existing basins for flexibility of operations. Wet-weather flows over 40 mgd would bypass primary treatment and would be conveyed via new influent piping directly to this treatment train. Secondary effluent would be conveyed to disinfection and effluent pumping facilities.

3.2.2.3 80-mgd Expansion

Hydraulic and process modeling indicated that expanding the West WWTP to 80 mgd would require additional aeration and secondary clarifier capacity. In addition to the modifications and expansions required for the 40-mgd scenario, two 150-foot-diameter secondary clarifiers and two 0.625-MG aeration tanks would also be needed. A raw wastewater splitter box is also required to split flow between the two aeration tanks.

The role of the new treatment train in this scenario is the same as that presented in the 60-mgd scenario (that is, flows in excess of 40 mgd would bypass primary treatment and would be conveyed via new influent piping directly to the new aeration basin). Secondary effluent would be conveyed to disinfection and effluent pumping facilities.

3.2.2.4 Cost Estimates

Cost estimates were prepared for all alternatives in accordance with the basis for cost estimating discussed in Section 2.2. Total capital costs and NPV 40-year O&M costs are summarized for each flow scenario in Table 3-5. More detailed estimates are presented in Appendix A.

Table 3-5 Cost Summary of Secondary Treatment Expansion

Flow	Total Capital Cost ($)	NPV 40-year O&M Cost ($)
40 mgd	740,000	244,000
60 mgd	29,722,000	2,058,000
80 mgd	49,559,000	6,334,000

3.2.3 Disinfection

This existing WWTP uses chlorinators to introduce chlorine gas into the wastewater. There is one 12-pass contact tank, having approximately 471,000-gallon capacity, which provides a minimum detention time of 15 minutes during peak flows. Although this is a proven and effective method to disinfect the wastewater, handling and storage of chlorine gas onsite poses health and safety risks to plant staff and the surrounding community. Due to such concerns, most wastewater facilities have replaced chlorine gas systems with liquid sodium hypochlorite or UV disinfection. Appendix C provides more details regarding disinfection.

Two chlorination alternatives (sodium hypochlorite and UV) were evaluated based on economic and non-economic factors. Although replacement of the gaseous chlorine system is not required to increase the WWTP capacity to 40 mgd, conversion to sodium hypochlorite is recommended due to safety concerns. However, for the 60- and 80-mgd scenarios, the evaluation indicated that a combination of UV and liquid sodium hypochlorite is most beneficial to the Utility (UV for flows up to 40 mgd; liquid sodium hypochlorite for flows in excess of 40 mgd).

Table 3-6 summarizes the requirements for converting and expanding disinfection capacity to 40, 60, and 80 mgd.

Table 3-6 Summary of Disinfection Conversion/Expansion Requirements

	40 mgd	60 mgd	80 mgd
UV System	None	Modify existing chlorine contact tank to accommodate 40-mgd UV system	Modify existing chlorine contact tank to accommodate 40-mgd UV system
Additional Contact Tankage Required	None	Construct new tank: 209,000 gallons with effluent piping to existing effluent pump station	Construct new tank: 417,000 gallons with effluent piping to new and existing effluent pump station
Chemical Feed/ Storage Required	Construct new Chemical Feed Building with three 7,500-gallon FRP storage tanks	Construct new Chemical Feed Building with two 5,000-gallon FRP storage tanks	Construct new Chemical Feed Building with three 7,500-gallon FRP storage tanks
Existing Contact Tank Modifications Required	None	Modify influent channel and effluent weir to accommodate UV hydraulics	Modify influent channel and effluent weir to accommodate UV hydraulics
Other	None	Construct new secondary effluent diversion structure	Construct new secondary effluent diversion structure Construct new chlorinated effluent diversion structure

FRP = fiberglass-reinforced plastic

3.2.3.1 40-mgd Expansion

Hydraulic modeling indicates that the existing chlorine contact tank has sufficient capacity for 40 mgd; therefore, no expansions are required.

3.2.3.2 60- and 80-mgd Expansions

Expanding the West WWTP disinfection capacity to 60 or 80 mgd would involve constructing a 40-mgd UV system within the existing contact tank. Disinfected effluent would be conveyed to effluent pumping facilities as it is under current operations. These scenarios would also involve constructing an additional secondary effluent diversion structure and liquid sodium hypochlorite facilities. The new diversion structure would route 40 mgd to the UV system and remaining flow to the chlorine contact tanks. The 60- and 80-mgd options would require a 209,000-gallon and 417,000-gallon contact tank, respectively, along with a chemical storage building. In addition, the 80-mgd option would require a chlorinated effluent diversion structure because the existing effluent pumping facilities only have capacity for 60 mgd. This new diversion structure would convey flow greater than 20 mgd to new effluent pumping facilities.

3.2.3.3 Cost Estimates

Cost estimates were prepared for all alternatives in accordance with the basis for cost estimating discussed in Section 2.2. Total capital costs and NPV 40-year O&M costs are summarized for each flow scenario in Table 3-7. More detailed estimates are presented in Appendix A.

Table 3-7 Cost Summary of Disinfection Expansion

Flow	Total Capital Cost ($)	NPV 40-year O&M Cost ($)
40 mgd	1,390,000	3,337,000
60 mgd	6,860,000	2,584,000
80 mgd	7,860,000	2,903,000

3.2.4 Final Effluent Conveyance Alternatives

At flow rates less than 27 mgd and low-river stages, effluent from the West WWTP is conveyed through a 48-inch effluent sewer via gravity to the Ohio River. When either the flows are above 27 mgd or during high-river stages, the effluent pumps pressurize the existing effluent sewer to the Ohio River. The maximum river stage (approximately 42 feet) in a typical year was assumed for evaluating effluent conveyance expansions.

Table 3-8 summarizes the requirements for expanding final effluent conveyance to 40, 60, and 80 mgd.

Table 3-8 Effluent Pumping Expansion Requirements

	40 mgd	60 mgd	80 mgd
Additional Pumping Required	None	One 20-mgd pump (for redundancy)	One 20-mgd pump (for redundancy) Two new 20-mgd pumps at ~46 total dynamic head, each
Additional Force Main Required	None	None	Approximately 4,700 LF of 36-inch force main
Other	None	Modify existing influent pump discharge line	Modify existing influent pump discharge line Construct new effluent pump station with wet well and dry well

The West WWTP currently has a firm pumping capacity of 40 mgd, and the existing force main is regularly subjected to flows and pressures associated with 37 mgd. An initial capacity assessment was conducted on the effluent sewer, and it was determined that the sewer had a capacity of approximately 68 mgd. Therefore, no conveyance improvements or expansions are needed for the 40-mgd or 60-mgd scenarios.

Expanding effluent pumping capacity to 60 mgd would involve adding a redundant pump. Because the existing Headworks Facility is planned to be abandoned, the existing influent pump(s) could be repurposed for effluent pumping. Therefore, minor piping modifications would be required to connect the existing influent pump(s) to the effluent force main. In addition, the slide gate separating the existing influent and effluent wet wells would be opened, allowing disinfected effluent to enter both wet wells.

Expanding effluent pumping capacity to 80 mgd would involve all modifications required for the 60-mgd expansion. In addition, it would include constructing a new 20-mgd effluent pump station and approximately 4,700 LF of force main.

3.2.4.1 Cost Estimates

Cost estimates were prepared for all alternatives in accordance with the basis for cost estimating discussed in Section 2.2. Total capital costs and NPV 40-year O&M costs are summarized for each flow scenario in Table 3-9. More detailed estimates are presented in Appendix A.

Table 3-9 Cost Summary of Effluent Pumping Expansion

Flow	Total Capital Cost ($)	NPV 40-year O&M ($)
40 mgd	-	-
60 mgd	-	-
80 mgd	28,173,000	1,308,000

3.3 Alternatives Development

Table 3-10 summarizes the requirements and estimated costs to expand the West WWTP to 40, 60, and 80 mgd. The costs associated with these scenarios are incorporated into the overall West System Alternatives Analysis, which will identify the optimum WWTP capacity for the West System. Preliminary site plans for the proposed improvements are shown in Figures 3-1 through 3-3 for the three different flow scenarios.

Table 3-10 West WWTP Expansion Summary

	40 mgd	60 mgd	80 mgd
Headworks			
Screening Required	Two 40-mgd, 0.25-inch fine screens with washer/compactor	Two 60-mgd, 0.25-inch fine screens with washer/compactor	Four 40-mgd, 0.25-inch fine screens with washer/compactor
Grit Removal Required	Two 20-mgd vortex grit units with classifying, washing, and dewatering equipment	Two 30-mgd vortex grit units with classifying, washing, and dewatering equipment	Four 20-mgd vortex grit units with classifying, washing, and dewatering equipment

Table 3-10 West WWTP Expansion Summary

	40 mgd	60 mgd	80 mgd
Pumping Required	Three 20-mgd pumps	Four 20-mgd pumps	Five 20-mgd pumps
Other	Headworks Facility with wet-well/dry-well ~35 feet deep Influent diversion structure (~15 feet by 15 feet) with actuated sluice gates	Headworks Facility with wet well/dry well ~35 feet deep Influent diversion structure (~15 feet by 15 feet) with actuated sluice gates Effluent splitter structure (~20 feet by 20 feet) with actuated plug valves	Headworks Facility with wet well/dry well ~35 feet deep Influent diversion structure (~15 feet by 15 feet) with actuated sluice gates Effluent splitter structure (~20 feet by 20 feet) with actuated plug valves
Secondary Treatment			
Additional Clarifier Capacity Required	None	One 150-foot-diameter secondary clarifier and piping	Two 150-foot-diameter secondary clarifiers and piping
Additional Aeration Capacity Required	None	One 0.625-MG aeration basin and piping	Two 0.625-MG aeration basins and piping
Additional Blower Capacity Required	None	Two blowers with 9,300-scfm capacity, each (one duty/one standby)	Two blowers with 10,480-scfm capacity, each (one duty/one standby)
Additional RAS/WAS Pumping Capacity Required	Two centrifugal RAS pumps; 2,100 gpm ea.	Two centrifugal RAS pumps; 2,100 gpm each (for existing trains) Three RAS pumps; 4,000 gpm, each (for new trains)	Two centrifugal RAS pumps; 2,100 gpm each (for existing trains) Three RAS pumps; 4,000 gpm, each (for new trains)
Other	None	None	Construct aeration basin influent splitter box
Disinfection			
UV System	None	Modify existing chlorine contact tank to accommodate 40-mgd UV system	Modify existing chlorine contact tank to accommodate 40-mgd UV system
Additional Contact Tankage Required	None	Construct new tank: 209,000 gallons with effluent piping to existing effluent pump station	Construct new tank: 417,000 gallons with effluent piping to new and existing effluent pump station
Chemical Feed/ Storage Required	Construct new Chemical Feed Building with three 7,500-gallon FRP storage tanks	Construct new Chemical Feed Building with two 5,000-gallon FRP storage tanks	Construct new Chemical Feed Building with three 7,500-gallon FRP storage tanks
Existing Contact Tank Modifications Required	None	Modify influent channel and effluent weir to accommodate UV hydraulics	Modify influent channel and effluent weir to accommodate UV hydraulics
Other	None	Construct new secondary effluent diversion structure	Construct new secondary effluent diversion structure Construct new chlorinated effluent diversion structure

Table 3-10 West WWTP Expansion Summary

	40 mgd	60 mgd	80 mgd
Effluent Pumping			
Additional Pumping Required	None	One 20-mgd pump (for redundancy)	One, 20-mgd pump (for redundancy) Two new 20-mgd pumps at ~46 total dynamic head, each
Additional Force Main Required	None	None	Approximately 4,700 LF of 36-inch force main
Other	None	Modify existing influent pump discharge line	Modify existing influent pump discharge line Construct new effluent pump station with wet well and dry well
Total Capital Cost	$21,910,000	$57,930,000	$126,820,000
NPV 40-year O&M Cost	$5,281,000	$6,542,000	$12,545,000

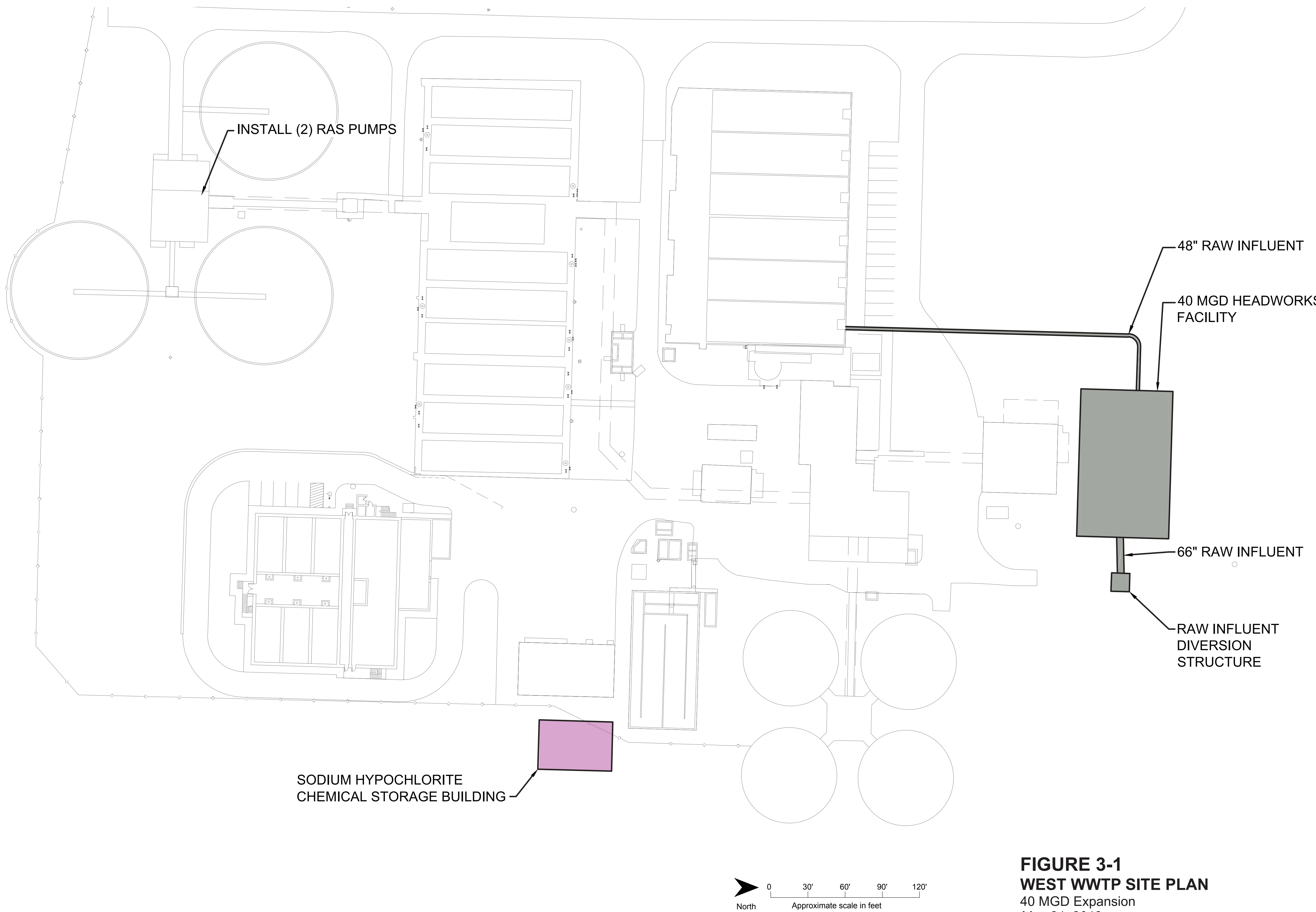


FIGURE 3-1
WEST WWTP SITE PLAN
40 MGD Expansion
May 31, 2013

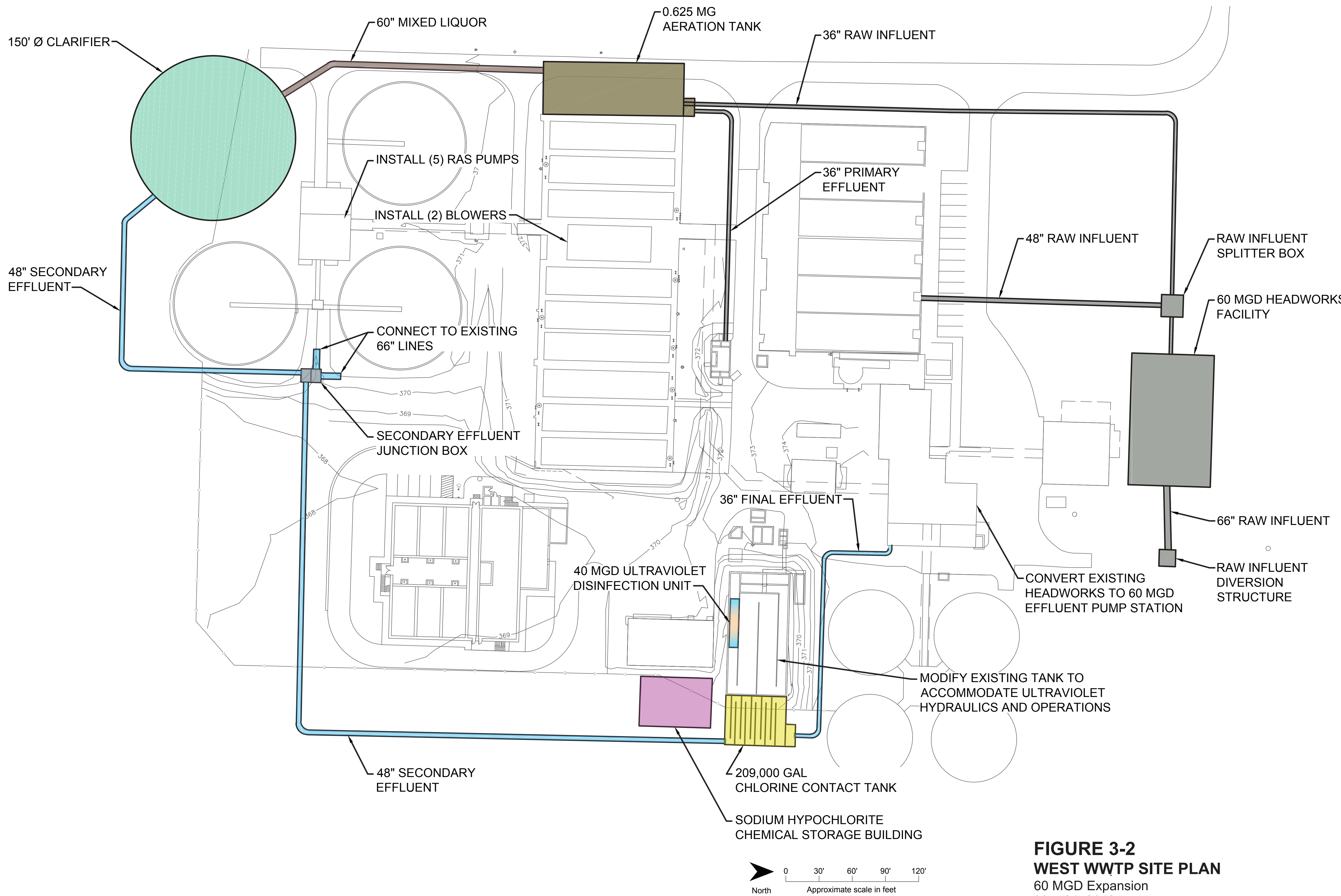


FIGURE 3-2
WEST WWTP SITE PLAN
60 MGD Expansion
May 31, 2013

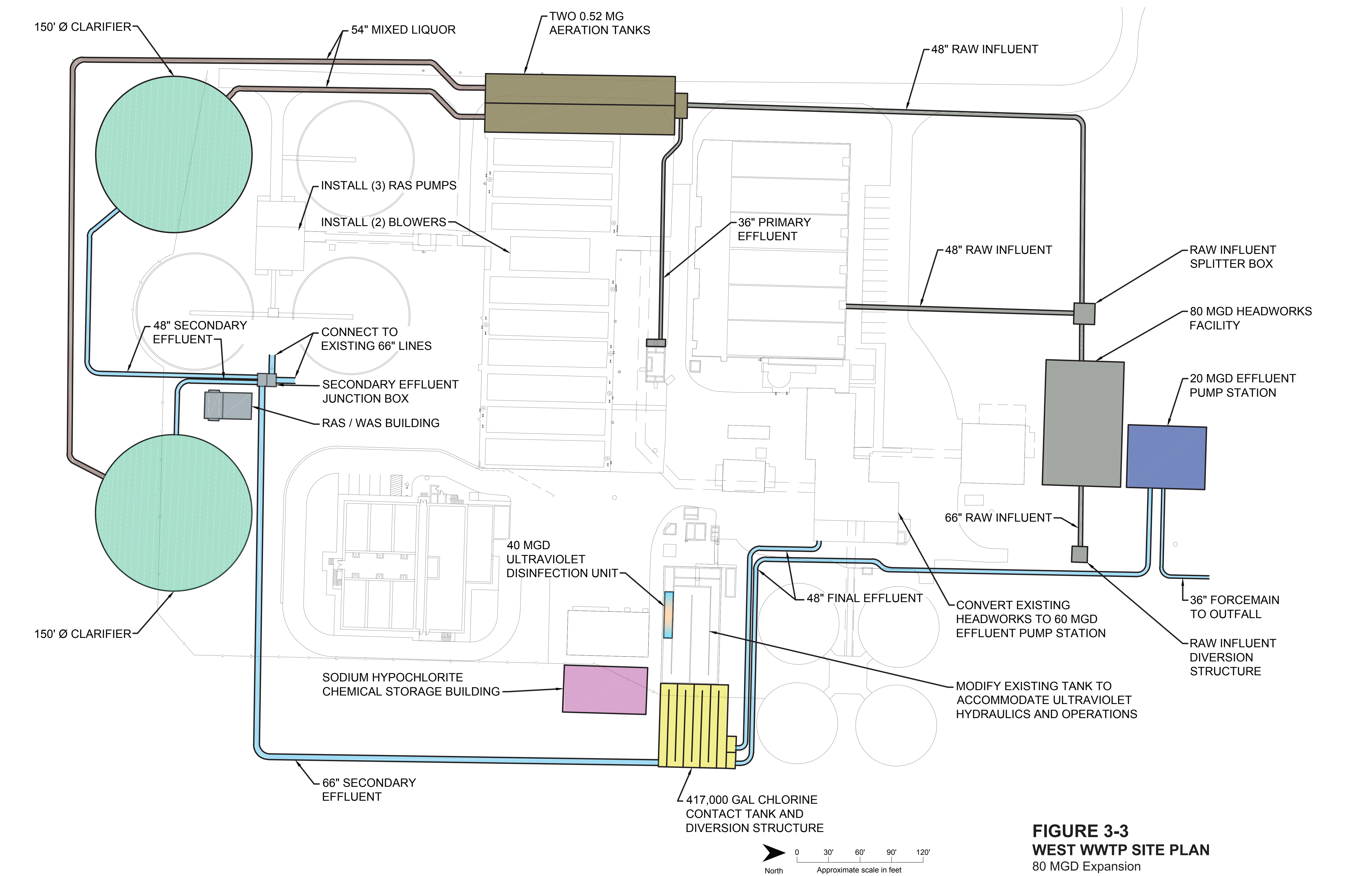


FIGURE 3-3
WEST WWTP SITE PLAN
80 MGD Expansion
May 31, 2013

WBG073012083042IDA EVANSVILLE_IOCP FacilityPlan_Figure_3-3

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SECTION 4

East WWTP

4.1 System Characterization

A brief summary of the East WWTP's existing facilities is provided below. The following documents provide more details regarding the condition, capacity, and operational attributes of the East WWTP:

- *East WWTP Wet-Weather SOPs* (CH2M HILL, 2010)
- *East WWTP Step Feed and Contact Stabilization Study* (CH2M HILL, 2011b)
- *East WWTP Stress Testing Report* (CH2M HILL, 2012d)

4.1.1 WWTP Overview

The East WWTP was originally constructed in the late 1950s and included preliminary and primary treatment. The primary effluent was disinfected using chlorine contact tanks that were adjacent to the primary clarifiers. The chlorine contact tanks provided 15 minutes of detention time at 14 mgd. When the activated-sludge system was added in the 1970s, a new chlorine contact tank was constructed to accommodate the change in the WWTP's hydraulic profile. The new contact tank was constructed to process a peak flow of 26 mgd with 15 minutes of detention time. Currently, the East WWTP provides preliminary treatment, primary clarification, secondary treatment, and disinfection. The NPDES permit states that the average-day design capacity is 18 mgd, and the current peak design flow is 22.5 mgd. Recent stress tests show that the WWTP can process up to 28 mgd under favorable operating conditions. The monthly and weekly mass limits for $CBOD_5$, TSS, and ammonia have been calculated using peak design flow.

Raw wastewater flows by gravity to the WWTP. Two mechanically cleaned bar screens and two vortex grit removal units provide preliminary treatment upstream of the influent pumps. The preliminary treated wastewater is discharged from the influent pumps into the primary clarifiers. Flow is by gravity through the plant once it enters primary treatment. Effluent flows from the primary clarifiers to the aeration tanks for secondary treatment and then to the secondary clarifiers for liquid/solids separation. Secondary effluent from the clarifiers flows into the chlorine contact tank for disinfection. The disinfected effluent is dechlorinated prior to discharge into the Ohio River. During high-river stages, the final effluent is discharged into the drainage area of the K-4 Pump Station. The K-4 Pump Station pumps water from the drainage area over the levee and into the Ohio River.

WAS from the secondary clarifiers is thickened by gravity belt thickeners. Primary sludge and thickened secondary WAS are then pumped to the primary anaerobic digesters and fed to secondary digesters. Digested sludge is pumped to conditioning tanks, where it is blended with digested sludge from the West WWTP and dewatered by belt filter presses. Trucks then haul the dewatered sludge to the onsite storage building until a local trucking company hauls the dewatered sludge to a landfill for disposal.

Table 4-1 provides a summary of the unit processes and design capacities at the East WWTP.

Table 4-1 East WWTP Unit Process and Design Capacities Summary

Unit Process	Type	Quantity	Size	Design Capacity
Preliminary Treatment				
Influent Screening	Mechanical	2	0.25-inch openings	20 mgd each
Influent Pumping	Flygt - submersible non-clog centrifugal	4	Flygt motor -170 hp	16.5 mgd @ 47.5-foot TDH (each)
Grit Removal	Vortex unit – 270° design Pista Grit	2	18-foot diameter	30 mgd each[a]
Primary Treatment				
Primary Clarification	Rectangular with chain and flight system	7	103-foot-long by 32-foot-wide by 8-foot, 8-inch SWD	50 mgd[b]
Secondary Treatment: Activated Sludge				
Aeration Tanks	Plug-flow tanks with ceramic grid fine bubble aeration system	3	300-foot-long by 30-foot-wide by 15-foot SWD	3.03 MG[c]
Blowers	Positive displacement	4	5,050 cfm, each	40.6 mgd
Clarification	Circular, flat-bottom, peripherally fed	3	100-foot-diameter, 12-foot SWD	28 mgd[b]
Return Sludge Pumping	Non-clog, horizontally mounted	3	6-inch diameter suction by 6-inch-diameter discharge; 25-hp TEFC motors	1,480 gpm at 39.5-foot TDH
Waste Sludge Pumping	Non-clog, horizontally mounted	3	6-inch diameter suction by 6-inch diameter discharge; 5-hp TEFC motors	550 gpm at 13.6-foot TDH
Disinfection				
Contact Tank	Two-pass, serpentine	1	315,000 gallons	32.4 mgd[b]
Chlorination	Gas chlorinators	2	2,000 lb/day	64.8 mgd
Dechlorination	Sodium bisulfate pumps	2	20.8 gallons/hour	173 mgd
Other				
Flow Metering	Magmeter	2	16 inch	25 mgd each
Effluent Pumping	U.S. Army Corps of Engineers Pump Station	1		110 mgd

[a] At peak-hourly flow.

[b] Capacity based on stress testing conducted April 2012. Stress Testing Report (CH2M HILL, 2012d) provides more information

[c] Existing aeration tanks are permitted for 18-mgd average and 22.5-mgd peak by the City's NPDES Permit, No. IN 0033073, dated February 1, 2012.

4.1.2 Primary Effluent Bypass

For combined sewer systems, the CSO policy allows wet-weather flows that pass the WWTP headworks to receive at least the equivalent of primary clarification, solids and floatables removal, and disinfection. With relatively minor improvements, the East WWTP could be configured to provide 14 mgd of disinfected primary effluent that would bypass the secondary treatment system. Consistent with Section II.C.7 of the CSO Policy, the primary effluent bypass discharge would receive screening, primary treatment, and disinfection. Under this scenario, the headworks and primarily clarifiers would treat 40 mgd, with 26 mgd receiving secondary treatment and 14 mgd receiving primary treatment. If operating conditions allow the WWTP to process 28 mgd with secondary treatment, primary effluent bypass would be reduced to 12 mgd. The old chlorine contact tanks would be rehabilitated and the primary effluent bypass would be conveyed through the existing 36-inch discharge pipe. The secondary effluent and primary effluent bypass would be combined downstream of the existing Parshall flume; therefore, the flow rate of the primary effluent bypass would be measured at the discharge weir in the chlorine contact tank. The disinfection expansion requirements and costs are discussed in Section 4.2.3.

4.2 Unit Process Evaluation

4.2.1 Preliminary Treatment

Preliminary treatment at the East WWTP includes influent screening, grit removal, and influent pumping. The sections below describe means for expanding the East WWTP preliminary treatment capacity to 40, 60, and 80 mgd. The scope of this analysis was limited to evaluating a single better alternative for each flow scenario. Table 4-2 summarizes the requirements for expanding the preliminary treatment process to 40, 60, and 80 mgd.

Table 4-2 Headworks Expansion Requirements

	40 mgd	60 mgd	80 mgd
Screening	None	New bypass channel and manual bar screen Replace two existing screens with two 30-mgd screens	New bypass channel and manual bar screen Replace two existing screens with two 40-mgd screens
Grit Removal	None	None	None
Pumping	None	Replace four existing pumps with four 20-mgd submersible pumps	Replace four existing pumps with four 20-mgd submersible pumps Expand wet-well and install one additional, 20-mgd influent pump.
Other	None	Modify existing H-flume Construct new flow splitter structure with automated plug valves	Modify existing H-flume Construct new flow splitter structure with automated plug valves

4.2.1.1 40-mgd Expansion

The Headworks Facility currently has a capacity of 40 mgd; therefore, no expansion is required.

4.2.1.2 60-mgd Expansion

Expansion of the Headworks Facility to 60 mgd would require replacing the existing screens and pumping equipment with larger units. The existing flume upstream of the wet well would require modification to improve hydraulics. Furthermore, expanding to 60 mgd would require a new flow splitter structure to divert flows over 40 mgd to the new wet-treatment facilities.

4.2.1.3 80-mgd Expansion

Expansion of the Headworks Facility to 80 mgd would require replacing the existing screens with larger units. In addition to replacing the four existing pumps with larger pumps, the wet well would need to be expanded to add a fifth submersible pump.

No modifications for grit removal are proposed because the existing hydraulic capacity of units is 80 mgd. Although enlarging the grit tanks would improve their performance during wet-weather events, it would also likely result in lowered daily performance due to being oversized. This is the current situation at the West WWTP. Furthermore, the duration of wet-weather events of this magnitude is expected to be short, and TSS concentration would likely be lower.

4.2.1.4 Cost Estimates

Cost estimates were prepared for all alternatives in accordance with the discussion of cost estimating in Section 2.2. Total capital costs and NPV 40-year O&M costs are summarized for each flow scenario in Table 4-3. More detailed estimates are presented in Appendix A.

Table 4-3 Summary of Headworks Expansion Costs

Flow	Total Capital Cost ($)	NPV 40-year O&M Cost ($)
40 mgd	-	-
60 mgd	9,524,000	6,400,000
80 mgd	11,963,000	12,000,000

4.2.2 Secondary Treatment

An evaluation of secondary treatment options was performed with the intent of identifying and developing cost estimates for the additional secondary treatment facilities. This evaluation identified conventional treatment as the most beneficial to the Utility. Therefore, this section describes alternatives for expanding the East WWTP secondary treatment capacity to 40, 60, and 80 mgd, via expansion of the activated-sludge system. Appendix B presents more details regarding the secondary treatment evaluation.

Secondary treatment at the East WWTP currently consists of a conventional activated-sludge process, which includes three identical secondary clarifier/aeration tank trains. Each clarifier is paired with a dedicated aeration tank; there is currently no means of conveying mixed liquor to any other clarifier; primary effluent is split proportionately among the three trains; secondary effluent recombines prior to disinfection. The WWTP hydraulic profile will not allow a splitter box to be constructed between the aeration basin and clarifier. Therefore, alternatives that require construction of a new clarifier would also require construction of a new aeration basin. Table 4-4 summarizes the requirements for expanding secondary treatment capacity to 40, 60, and 80 mgd.

The expansion to 40 mgd would include a new aeration basin and clarifier downstream of the primary clarifiers at the locations designated in the original plant design. Flow would be routed to the new train via the existing primary effluent splitter box. Secondary effluent would recombine at the existing junction structure downstream of the existing secondary clarifiers, where it would then be conveyed to disinfection facilities via the existing 60-inch line. The aeration basin and clarifier would be larger than the existing units to provide sufficient capacity to treat future influent loads with full nitrification.

Table 4-4 Secondary Treatment Expansion Requirements

	40 mgd	60 mgd	80 mgd
Additional Clarifier Capacity Required	One 150-foot-diameter clarifier and piping	One 150-foot-diameter clarifier and piping Two 123-foot-diameter clarifiers and piping	One 150-foot-diameter clarifier and piping Two 141-foot-diameter clarifiers and piping
Additional Aeration Capacity Required	One 1.4-MG tank	One 1.4-MG tank Two 0.625-MG tanks	One 1.4-MG tank Two 1.04-MG tanks
Additional Blower Capacity Required	Two turbo blowers, each at 11,300 scfm	Two turbo blowers, each at 11,300 scfm Two turbo blowers, each at 12,000 scfm	Two turbo blowers, each at 11,300 scfm Two turbo blowers, each at 13,300 scfm
Additional RAS/WAS Pumping Capacity Required	Three centrifugal pumps, 4,000 gpm at 40-foot TDH each	Six centrifugal pumps, 4,000 gpm at 40-foot TDH each	Six centrifugal pumps, 4,000 gpm at 40-foot TDH each
Other	None	Influent splitter box	Influent splitter box

Similar to the West WWTP, the alternatives to expand the East WWTP to 60 and 80 mgd would include new aeration basins and clarifiers without primary treatment. The additional treatment units could also be interconnected with the existing treatment units to provide additional redundancy during dry weather conditions.

4.2.2.1 Cost Estimates

Cost estimates were prepared for all alternatives in accordance with the basis for cost estimating discussed in Section 2.2. Total capital costs and NPV 40-year O&M costs are summarized for each flow scenario in Table 4-5. More detailed estimates are presented in Appendix A.

Table 4-5 Cost Summary of Secondary Treatment Expansion

Flow	Total Capital Cost ($)	NPV 40-year O&M Cost ($)
40 mgd	31,334,000	1,666,000
60 mgd	66,203,000	3,301,000
80 mgd	74,051,000	3,360,000

4.2.3 Disinfection Alternatives

An evaluation of disinfection options was conducted with the intent of identifying and developing cost estimates for the additional disinfection facilities. The evaluation compared the cost and non-economic benefits of UV disinfection for flows up to 40 mgd versus a liquid sodium hypochlorite disinfection system. For wet-weather flows above 40 mgd, only liquid sodium hypochlorite was considered. The evaluation identified that the liquid sodium hypochlorite alternative would be more beneficial if a primary effluent bypass is used, and the secondary treatment system is not expanded. If the secondary treatment system is expanded, the UV alternative would be more beneficial to the Utility. Appendix C provides more details regarding the disinfection evaluation.

Table 4-6 summarizes the requirements for converting and expanding disinfection capacity to 40, 60, and 80 mgd.

Table 4-6 Disinfection Expansion Requirements

	40 mgd with 14 mgd Bypass	40 mgd	60 mgd	80 mgd
UV System	None	Install 40-mgd UV unit in existing contact tank	Construct new 40-mgd UV system; and new effluent piping to new junction structure	Construct new 40-mgd UV system; and new effluent piping to new junction structure
Additional Contact Tankage Required	None	None	None	One new tank: 176,000 gallons
Chemical Feed/ Storage Required	New Chemical Feed Building with three 7,500-gallon FRP storage tanks	None	New Chemical Feed Building with two 5,000-gallon FRP storage tanks	New Chemical Feed Building with three 7,500-gallon FRP storage tanks
Existing Contact Tank Modifications Required	Rehab contact tank walls Modify contact tank effluent weir Replace existing 48-inch sluice gate with automated gate and level sensor	Modify influent channel and effluent weir to accommodate UV hydraulics	None	Lower existing contact tank weir approximately 0.2 foot Add a second 48-inch sluice gate to existing influent channel
Other	Seal existing final effluent manhole		Construct new UV diversion structure (~10 feet by 10 feet) with gates	Upsize existing effluent piping Construct new UV diversion structure (~10 feet by 10 feet) with gates Construct new chlorinated effluent diversion structure to bypass existing contact tank

4.2.3.1 40-mgd Expansion

Expanding the East WWTP's disinfection capacity to 40 mgd could involve two different scenarios. If the existing secondary treatment process is not expanded, a primary effluent bypass could be used to divert approximately 14 mgd to the two existing, older contact tanks for disinfection. These two tanks would operate in series and would be hydraulically separate from the southern tank (which would continue to disinfect approximately 26 mgd of secondary effluent). This expansion would involve rehabilitating the existing chlorine contact tanks and replacing the effluent weir. It would also involve replacing the existing 48-inch primary effluent bypass sluice gate with an automated gate and level sensor, thereby sealing off the existing final effluent manhole at the north end of the contact tank. Final effluent would be conveyed via the WWTP's original effluent sewer.

If secondary treatment is expanded to 40 mgd, disinfection would be expanded by retrofitting the southern chlorine contact tank by installing a new 40-mgd UV unit. It would require modification of the influent channel and effluent weir to accommodate UV hydraulics.

4.2.3.2 60-mgd Expansion

Expanding the East WWTP disinfection capacity to 60 mgd would involve constructing a stand-alone 40-mgd UV unit just downstream of the existing secondary clarifiers. A new diversion structure would route 40 mgd to the UV unit, and the remaining 20 mgd would be routed to the existing contact tank for liquid chlorine disinfection, bringing the total disinfection capacity to 60 mgd. A new Chemical Feed Building would also be constructed to store and handle liquid sodium hypochlorite.

4.2.3.3 80-mgd Expansion

Expanding the East WWTP disinfection capacity to 80 mgd would involve constructing a new 40-mgd UV system and diversion structure, as described in the 60-mgd expansion. The remaining 40 mgd of disinfection capacity would consist of the existing contact tank and a newly constructed 20-mgd contact tank. The two contact tanks would operate in series, thereby bringing the total disinfection capacity to 80 mgd. This scenario would also require infrastructure to allow bypass of the existing contact tank when needed. A new Chemical Feed Building would also be constructed to store sodium hypochlorite.

4.2.3.4 Cost Estimates

Cost estimates were prepared for all alternatives in accordance with the basis for cost estimating discussed in Section 2.2. Total capital costs and NPV 40-year O&M costs are summarized for each flow scenario in Table 4-7. More detailed estimates are presented in Appendix A.

Table 4-7 Cost Summary of Disinfection Expansion

Flow	Total Capital Cost ($)	NPV 40-year O&M Cost ($)
40 mgd – Primary Effluent Bypass	2,160,000	2,719,000
40 mgd	3,890,000	1,778,000
60 mgd	9,420,000	1,946,000
80 mgd	12,960,000	1,951,000

4.2.4 Final Effluent Conveyance Alternatives

Final effluent from the East WWTP disinfection facilities currently flows by gravity to a Parshall flume and is then conveyed downstream to the interceptor chamber that houses CSO Diversion Structure 103. From this point, WWTP effluent is conveyed through a 72-inch/84-inch gravity sewer to the Ohio River. During high-river stages, effluent from the East WWTP is pumped via the K-4 Pump Station.

Effluent pumping options for the East WWTP were evaluated with the intent of identifying and developing cost estimates for effluent pumping facilities. That evaluation indicated that constructing an effluent pump station at Sunset Park was the most beneficial option for the Utility. Appendix E provides more details regarding the effluent pumping evaluation.

Table 4-8 summarizes the requirements for expanding effluent conveyance and pumping capacity to 40, 60, and 80 mgd. All flow scenarios would include constructing a new effluent pump station at Sunset Park. Other elements common to all flow scenarios include constructing a new pump station, relining approximately 600 LF of the existing interceptor sewer, and sealing the existing K-4 bypass line.

Table 4-8 Effluent Conveyance Expansion Requirements

	40 mgd – Primary Effluent Bypass	40 mgd	60 mgd	80 mgd
Effluent Conveyance (from WWTP to Pump Station)				
Disinfected Effluent Modifications Required	None	Replace existing 3-inch effluent line downstream of Parshall flume with a 54-inch line	Replace existing 36-inch effluent line downstream of Parshall flume with a 54-inch line	Replace existing 3-inch effluent line downstream of Parshall flume with a 54-inch line
		Replace existing 5-inch effluent line upstream of Parshall flume with a 54-inch line, ~2 feet deeper		Replace existing 5-inch effluent line upstream of Parshall flume with a 54-inch line, ~2 feet deeper
Effluent Metering Required	None	Replace existing structure with deeper structure (~2 feet deeper)	Modify existing flume to improve hydraulics/ functionality	Replace existing structure with deeper structure (~2 feet deeper)
		Upsize existing Parshall flume to 48-inch flume		Upsize existing Parshall flume to 48-inch flume
Effluent Pumping				
Pump Station Building Required	Construct new effluent pump station (~20 feet tall/ 460 ft^2); wet well ~32 feet deep	Construct new effluent pump station (~20 feet tall/460 ft^2); wet well ~32 feet deep	Construct new effluent pump station (~20 feet tall/560 ft^2); wet well ~32 feet deep	Construct new effluent pump station (~20 feet tall/ 670 ft^2); wet well ~32 feet deep
Effluent Pumping Required	Install three submersible pumps, 13,900 gpm each	Install three submersible pumps, 13,900 gpm each	Install four submersible pumps, 13,900 gpm each	Install five submersible pumps, 13,900 gpm each
Effluent Force main Required	Slip-line existing 84-inch sewer with ~600 LF of 54-inch HDPE	Slip-line existing 84-inch sewer with ~600 LF of 54-inch HDPE	Slip-line existing 84-inch sewer with ~600 LF of 60-inch HDPE	Reline existing 84-inch sewer with ~600 LF of cured-in-place pipe
Other	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station

ft^2 = square feet
HDPE = high-density polyethylene

In addition to constructing new pumping facilities at Sunset Park, the items discussed in Sections 4.2.4.1 through 4.2.4.3 would be required for each flow scenario.

4.2.4.1 40-mgd Expansion

To increase WWTP secondary capacity to 40 mgd, the final effluent lines upstream and downstream of the existing Parshall flume would need to be upsized and lowered approximately 2 feet to maintain acceptable hydraulic grade lines upstream. The existing Parshall flume would also need to be upsized to 48 inches to accommodate the increased flow.

If the primary effluent bypass alternative is selected, no modifications to the final effluent piping are required. Disinfected secondary effluent would be conveyed to the existing Parshall flume and by the existing effluent sewer. Disinfected primary effluent would be conveyed to the outfall by the WWTP's original effluent sewer.

4.2.4.2 60-mgd Expansion

In the 60-mgd scenario, the existing chlorine contact tank would be used to disinfect a peak flow of 20 mgd using liquid sodium hypochlorite, as is the current practice. Therefore, the final effluent line upstream of the Parshall flume requires no modification. However, the Parshall flume and supporting structure would be modified to correct hydraulic/operational issues observed by WWTP staff. In addition, the 36-inch line downstream of the Parshall flume would be upsized to a 54-inch line.

4.2.4.3 80-mgd Expansion

In the 80-mgd scenario, the existing chlorine contact tank would be expanded to disinfect 40 mgd using liquid sodium hypochlorite. Therefore, the final effluent lines upstream and downstream of the existing Parshall flume would need to be upsized and lowered approximately 2 feet to maintain acceptable hydraulic grade lines upstream. The existing Parshall flume would also need to be upsized to a 48-inch flume to accommodate the increased flow.

4.2.4.4 Cost Estimates

Cost estimates were prepared for all alternatives in accordance with the basis for cost estimating discussed in Section 2.2. Total capital costs and NPV 40-year O&M costs are summarized for each flow scenario in Table 4-9. More detailed estimates are presented in Appendix A.

Table 4-9 Cost Summary of Effluent Conveyance Expansion

Flow	Total Capital Cost ($)	NPV 40-year O&M Cost ($)
40 mgd – Primary Effluent Bypass	10,570,000	766,000
40 mgd	11,777,000	766,000
60 mgd	12,463,000	1,066,000
80 mgd	14,622,000	1,366,000

4.3 Alternatives Development

Table 4-10 summarizes the requirements and estimated costs to expand the East WWTP to 40, 60, and 80 mgd. The costs associated with these scenarios are incorporated into the overall East System Alternatives Analysis, which identified the optimum WWTP capacity for the East system. Preliminary site plans for the proposed improvements are shown in Figures 4-1 through 4-3 for the three different flow scenarios.

Table 4-10 East WWTP Expansion Summary

	40 mgd – Primary Effluent Bypass	40 mgd	60 mgd[a]	80 mgd[a]
Preliminary Treatment				
Screening	None	None	New bypass channel and manual bar screen Replace two existing screens with two 30-mgd screens	New bypass channel and manual bar screen Replace two existing screens with two 40-mgd screens
Grit Removal	None	None	None	None
Pumping	None	None	Replace four existing pumps with four 20-mgd submersible pumps	Replace four existing pumps with four 20-mgd submersible pumps Expand wet well and install one additional 20-mgd influent pump
Other	None	None	Modify existing H-flume Construct new flow splitter structure with automated plug valves	Modify existing H-flume Construct new flow splitter structure with automated plug valves
Secondary Treatment				
Additional Clarifier Capacity Required	None	One 150-foot-diameter clarifier, required ancillary equipment, and piping	One 150-foot-diameter clarifier and piping Two 123-foot-diameter clarifiers and piping	One 150-foot-diameter clarifier and piping Two 141-foot-diameter clarifiers and piping
Additional Aeration Capacity Required	None	One 1.4-MG tank	One 1.4-MG tank Two 0.625-MG tanks	One 1.4-MG tank Two 1.04-MG tanks
Additional Blower Capacity Required	None	Two turbo blowers, each at 11,300 scfm	Two turbo blowers, each at 11,300 scfm Two turbo blowers, each at 12,000 scfm	Two turbo blowers, each at 11,300 scfm Two turbo blowers, each at 13,300 scfm
Additional RAS/WAS Pumping Capacity Required	None	Three centrifugal pumps, 4,000 gpm at 40-foot TDH each	Six centrifugal pumps, 4,000 gpm at 40-foot TDH each	Six centrifugal pumps, 4,000 gpm at 40-foot TDH each
Other		None	Influent splitter box	Influent splitter box

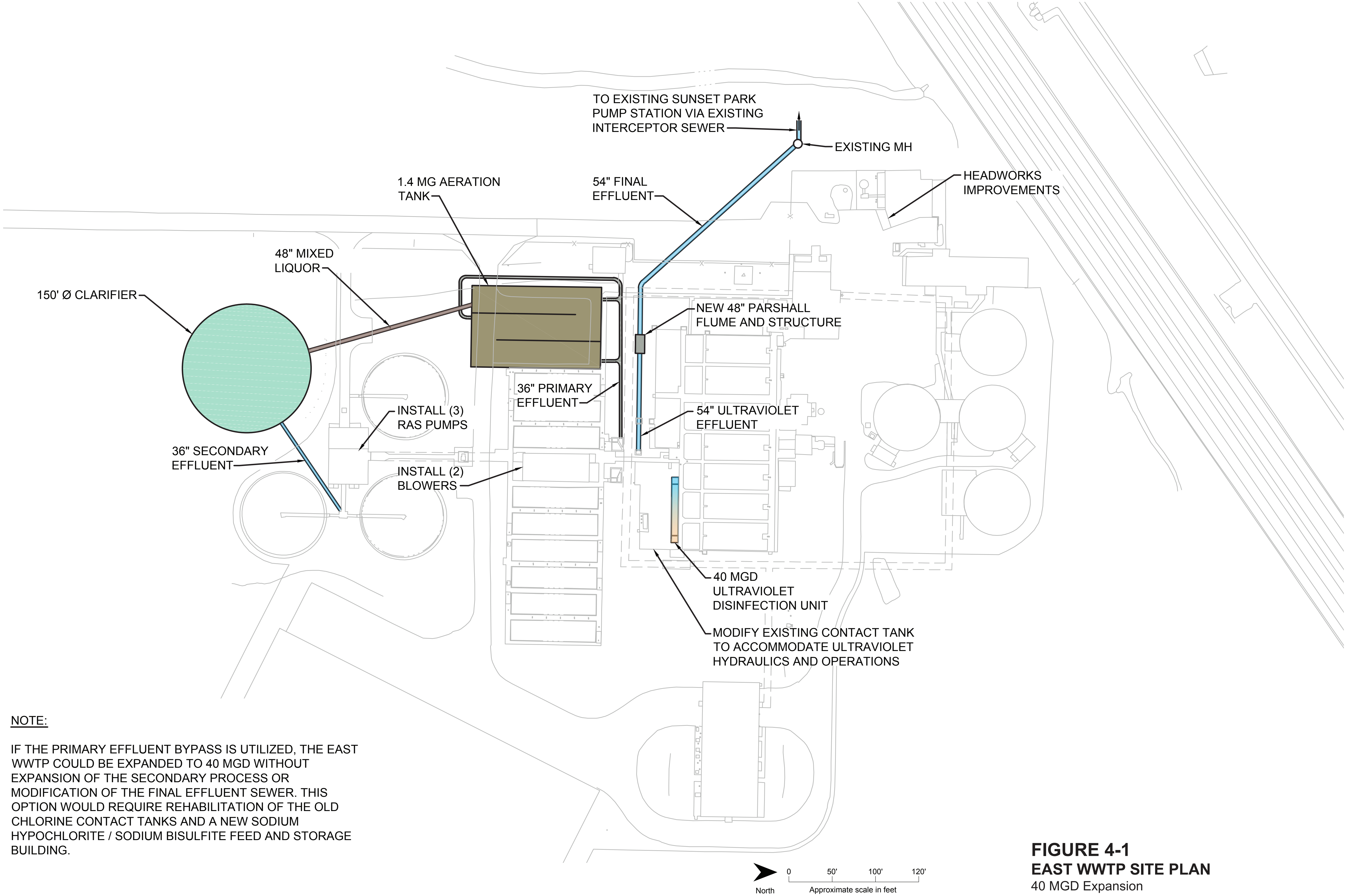
Table 4-10 East WWTP Expansion Summary

	40 mgd – Primary Effluent Bypass	40 mgd	60 mgd[a]	80 mgd[a]
Disinfection				
UV System	None	Modify existing chlorine contact tank to accommodate 40-mgd UV system; upsize existing effluent piping	Construct new 40-mgd UV system; install new effluent piping to new junction structure	Construct new 40-mgd UV system; install effluent piping to new junction structure
Additional Contact Tankage Required	None	None	None	One new 176,000-gallon tank
Chemical Feed/ Storage Required	New Chemical Feed Building with three 7,500-gallon FRP storage tanks	None	New Chemical Feed Building with two 5,000-gallon FRP storage tanks	New Chemical Feed Building with three 7,500-gallon FRP storage tanks
Existing Contact Tank Modifications Required	Rehab contact tank walls Modify contact tank effluent weir Replace existing 48-inch sluice gate with automated gate and level sensor	Modify influent channel and effluent weir to accommodate UV hydraulics	None	Lower existing contact tank weir ~0.2 foot Add a second 48-inch sluice gate to existing influent channel
Other	Seal existing final effluent manhole		Construct new UV diversion structure (~10 feet by 10 feet) with gates	Upsize existing effluent piping New UV diversion structure (~10 feet by 10 feet) with gates Construct new chlorinated effluent diversion structure to bypass existing contact tank
Effluent Conveyance (from WWTP to Pump Station)				
Disinfected Effluent Modifications Required	None	Replace existing 36-inch effluent line downstream of Parshall flume with a 54-inch line Replace existing 54-inch effluent line upstream of Parshall flume with a 54-inch line, ~2 feet deeper	Replace existing 36-inch effluent line downstream of Parshall flume with a 54-inch line	Replace existing 36-inch effluent line downstream of Parshall flume with a 54-inch line Replace existing 54-inch effluent line upstream of Parshall flume with a 54-inch line, ~2 feet deeper
Effluent Metering Required	None	Replace existing structure with deeper structure (~2 feet deeper) Upsize existing Parshall flume to 48-inch flume	Modify existing flume to improve hydraulics/ functionality	Replace existing structure with deeper structure (~2 feet deeper) Upsize existing Parshall flume to 48-inch flume

Table 4-10 East WWTP Expansion Summary

	40 mgd – Primary Effluent Bypass	40 mgd	60 mgd[a]	80 mgd[a]
Effluent Pumping				
Pump Station Building Required	Construct new effluent pump station (~20 feet tall/460 ft^2); wet-well ~32 feet deep	Construct new effluent pump station (~20 feet tall/460 ft^2); wet-well ~32 feet deep	Construct new effluent pump station (approx. 20 feet tall/560 ft^2); wet-well ~32 feet deep	Construct new effluent pump station (~20 feet tall/670 ft^2); wet well ~32 feet deep
Effluent Pumping Required	Install three submersible pumps, 13,900 gpm each	Install three submersible pumps, 13,900 gpm each	Install four submersible pumps, 13,900 gpm each	Install five submersible pumps, 13,900 gpm each
Effluent Force Main Required	Slip-line existing 84-inch sewer with ~600 LF of 54-inch HDPE	Slip-line existing 84-inch sewer with ~600 LF of 54-inch HDPE	Slip-line existing 84-inch sewer with ~600 LF of 60-inch HDPE	Reline existing 84-inch sewer with ~600 LF of cured-in-place pipe
Other	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station
Total Capital Cost	**$12,730,000**	**$47,001,000**	**$97,610,000**	**$113,596,000**
NPV 40-year O&M Cost	**$3,485,000**	**$4,210,000**	**$12,713,000**	**$18,677,000**

[a] The 60- and 80-mgd expansion alternatives would require demolition of the existing sludge storage facility and therefore could require modification of the Utility's solids-handling operation. Table 4-10 does not include these associated costs. Refer to Appendix D for more information about potential modifications to the Utility's solids-handling process.



NOTE:

IF THE PRIMARY EFFLUENT BYPASS IS UTILIZED, THE EAST WWTP COULD BE EXPANDED TO 40 MGD WITHOUT EXPANSION OF THE SECONDARY PROCESS OR MODIFICATION OF THE FINAL EFFLUENT SEWER. THIS OPTION WOULD REQUIRE REHABILITATION OF THE OLD CHLORINE CONTACT TANKS AND A NEW SODIUM HYPOCHLORITE / SODIUM BISULFITE FEED AND STORAGE BUILDING.

FIGURE 4-1
EAST WWTP SITE PLAN
40 MGD Expansion
May 31, 2013

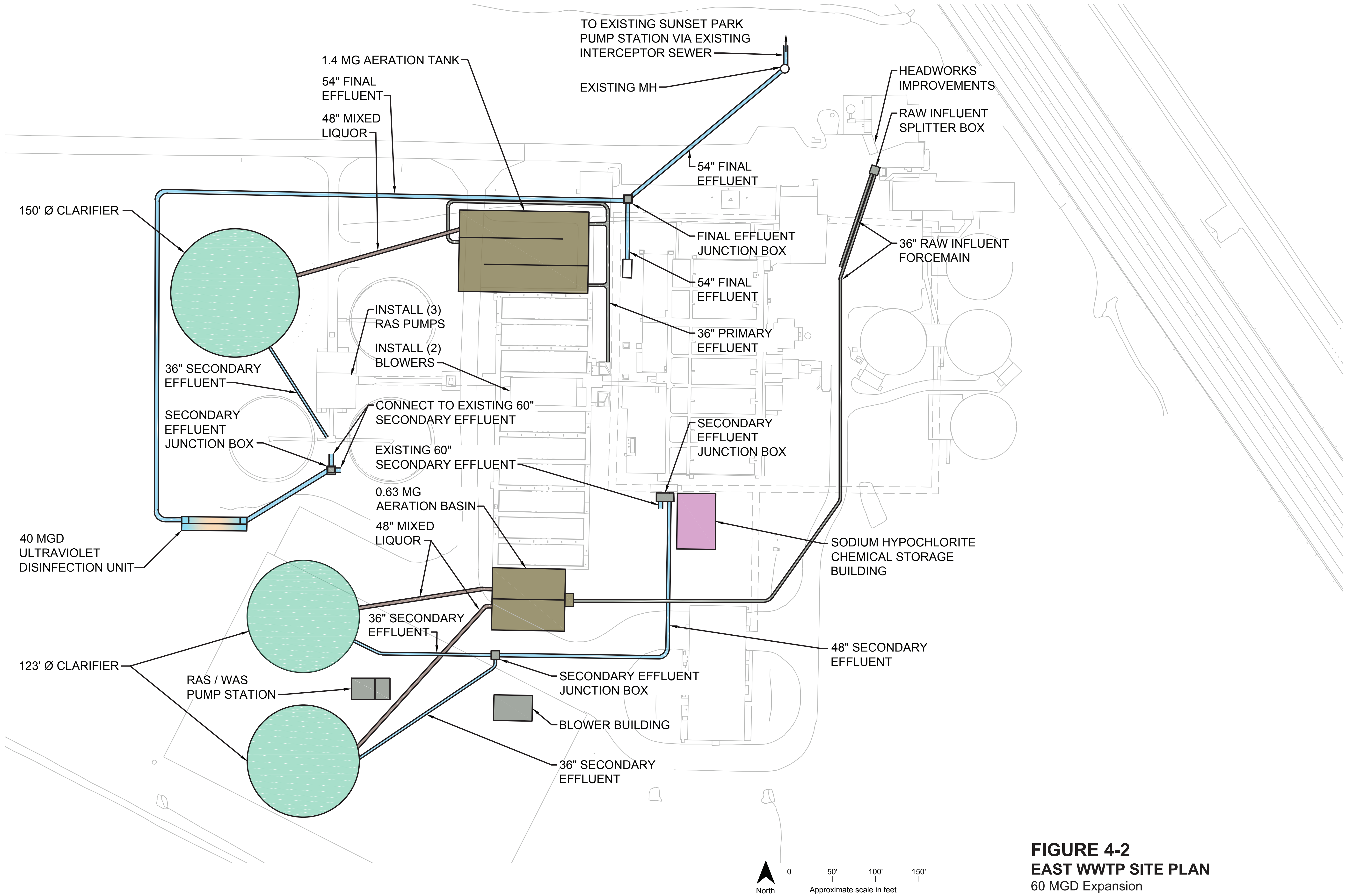


FIGURE 4-2
EAST WWTP SITE PLAN
60 MGD Expansion
May 31, 2013

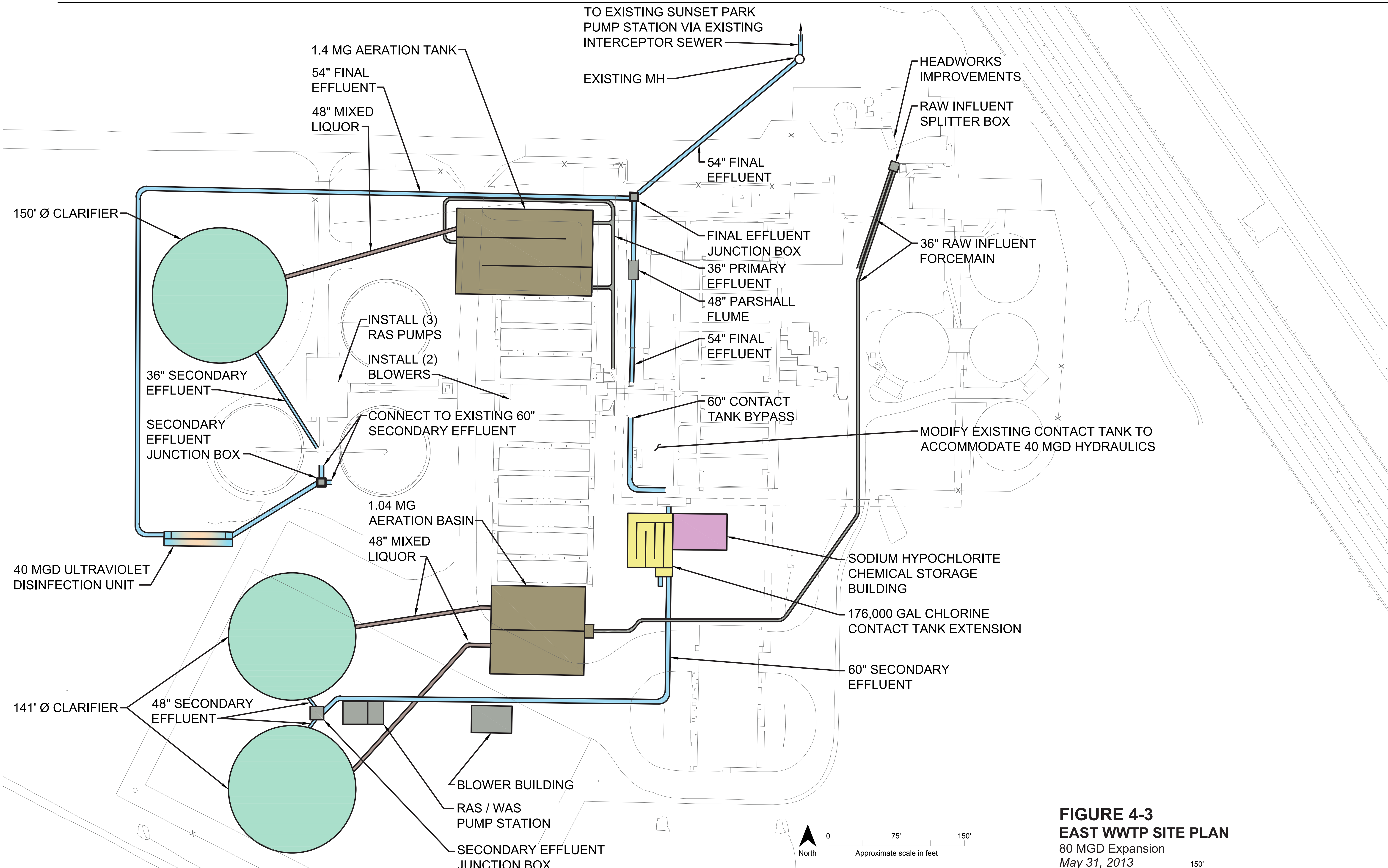


FIGURE 4-3
EAST WWTP SITE PLAN
80 MGD Expansion
May 31, 2013

150'

4.4 Bee Slough Considerations

The Bee Slough System is one of three major sewer systems in the East Service Area consisting of four CSOs that collectively discharge approximately 1,053 MG in the typical year. In terms of infrastructure, the system's most notable feature is Bee Slough, a concrete-lined drainage channel that runs parallel to Veterans Memorial Parkway, passes adjacent to the East WWTP, and ultimately discharges into the Ohio River. It is specifically referenced in the Decree, and remedying the associated health, odor, and aesthetic issues is the Utility's highest priority. As documented in Section 5.5 of the LTCP, numerous control measures were developed to address Bee Slough. To facilitate assessment and refinement of the Bee Slough control measure alternatives, two East WWTP expansion scenarios were developed supplementary to those presented earlier in this report.

The additional expansion alternatives involved reactivating the East WWTP's primary effluent (PE) bypass. Doing so would allow the WWTP to operate primary and secondary processes in parallel during wet weather, which could significantly increase treatment capacity at a low capital cost. During dry weather, the WWTP would maintain existing operations (that is, raw influent entering the headworks would receive primary and secondary treatment in series along with disinfection). However, during certain wet-weather events, primary and secondary processes would transition to a parallel operating scheme. In these instances, flow entering the WWTP headworks would receive primary treatment and disinfection only; the secondary treatment facilities would receive and treat CSO overflow volumes from Bee Slough by means of collection and pumping systems developed as part of Bee Slough control measure alternatives. Secondary effluent would also receive disinfection. Refer to Section 5.5 in the LTCP for more detail on the East WWTP's proposed interaction with the Bee Slough Treatment System.

Two scenarios were developed with the PE Bypass configuration in mind:

- 68-mgd PE bypass (40-mgd primary treatment/28-mgd secondary treatment)
- 80-mgd PE bypass (40-mgd primary treatment/40-mgd secondary treatment)

The required WWTP infrastructure and associated capital and O&M costs for these scenarios are presented in Sections 4.4.1 and 4.4.2.

4.4.1 68-mgd PE Bypass

The 68-mgd PE bypass scenario would involve rehabilitating the existing north chlorine contact tanks as well as constructing a 0.262-MG contact tank extension along the north side of the primary clarifiers. A 54-inch effluent sewer would convey chlorinated effluent to an existing manhole just north of the headworks facility. In addition, an effluent pump station would be constructed at Sunset Park to pump final effluent during high river conditions. As described in Section 4.2.4, a pump station at this location would be most beneficial option for the Utility. A new chemical feed and storage building would also be required for storage and distribution of liquid sodium hypochlorite.

For the activated sludge system to receive flow collected by the Bee Slough control measure, a junction structure would be constructed and connected to the existing PE splitter boxes. Flow to this junction structure would be conveyed by an aboveground force main. The force main is proposed to be above ground due to the significant yard piping known to be present in this area. Table 4-11 summarizes the requirements and estimated costs to expand the East WWTP to

68 mgd by means of a PE bypass. Figure 4-4 illustrates the proposed site plan for the 68-mgd PE bypass.

Table 4-11 East WWTP PE Bypass Expansion Summary

	68-mgd PE Bypass	80-mgd PE Bypass
Preliminary Treatment		
Screening	None	None
Grit Removal	None	None
Pumping	None	None
Secondary Treatment		
Additional Clarifier Capacity Required	None	One 150-foot-diameter clarifier, required ancillary equipment, and piping
Additional Aeration Capacity Required	None	One 1.4-MG tank
Additional Blower Capacity Required	None	Two turbo blowers, each at 11,300 scfm
Additional RAS/WAS Pumping Capacity Required	None	Three centrifugal pumps, 4,000 gpm at 40-foot total dynamic head, each
Disinfection		
UV System	None	Modify south chlorine contact tank to accommodate 40-mgd UV system; upsize existing effluent piping
Additional Contact Tankage Required	New 0.262-MG contact tank	New 0.262-MG contact tank
Chemical Feed/ Storage Required	New Chemical Feed Building with three 7,500-gallon FRP storage tanks	New Chemical Feed Building with three 7,500-gallon FRP storage tanks
Existing Contact Tank Modifications Required	Rehab contact tank walls	Modify influent channel and effluent weir to accommodate UV hydraulics; rehab north contact tank walls
	Modify contact tank effluent weir	Modify contact tank effluent weir
	Replace existing 48-inch sluice gate with automated gate and level sensor	Replace existing 48-inch sluice gate with automated gate and level sensor
Effluent Conveyance (from WWTP to Pump Station)		
Disinfected Effluent Modifications Required	New 54-inch final effluent sewer	Replace existing 36-inch effluent line downstream of Parshall flume with a 54-inch line; install a second new 54-inch final effluent sewer from new contact tank
		Replace existing 54-inch effluent line upstream of Parshall flume with a 54-inch line, ~2 feet deeper
Effluent Metering Required	None	Replace existing structure with deeper structure (~2 feet deeper)
		Upsize existing Parshall flume to 48-inch flume

Table 4-11 East WWTP PE Bypass Expansion Summary

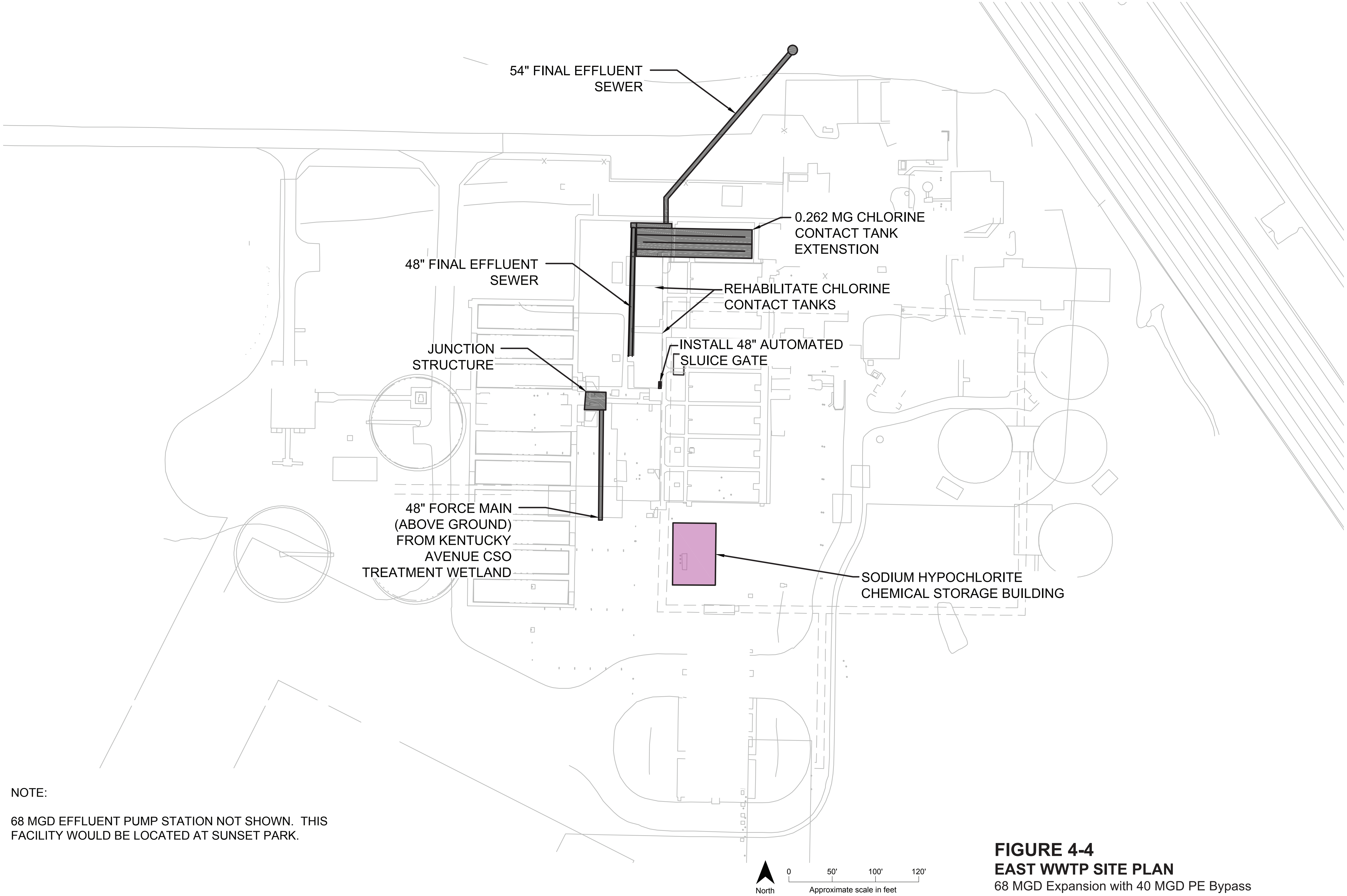
	68-mgd PE Bypass	80-mgd PE Bypass
Effluent Pumping		
Pump Station Building Required	Construct new effluent pump station (~20 feet tall/460 ft^2); wet well ~32 feet deep	Construct new effluent pump station (~20 feet tall/670 ft^2); wet well ~32 feet deep
Effluent Pumping Required	Install three submersible pumps, 13,900 gpm each	Install five submersible pumps, 13,900 gpm each
Effluent Force Main Required	Slip-line existing 84-inch sewer with ~600 LF of 54-inch HDPE	Reline existing 84-inch sewer with ~600 LF of cured-in-place pipe
Other	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station
Total Capital Cost	**$21,182,000**	**$58,696,000**
NPV 40-year O&M Cost	**$3,994,000**	**$10,183,000**

4.4.2 80-mgd PE Bypass

The 80-mgd PE bypass scenario would involve a combination of the following two expansion scenarios previously developed:

- 40-mgd conventional expansion scenario; and
- 68-mgd PE bypass scenario

All features identified in both of the abovementioned scenarios would be included. The secondary treatment facilities would be expanded to 40 mgd by adding a fourth aeration basin and clarifier. Secondary effluent would be disinfected by a UV system retrofitted into the existing southern chlorine contact tank, and the existing effluent lines and Parshall flume would be appropriately upsized. Furthermore, the northern chlorine contact tanks would be rehabilitated and expanded as described in Section 4.4.1. The only modification to the two scenarios previously developed would involve the final effluent pump station located at Sunset Park. In each scenario, this pump station would be sized for 40 and 68 mgd, respectively. The 80-mgd PE bypass scenario would require an 80-mgd pump station. Table 4-11 summarizes the requirements and estimated costs to expand the East WWTP to 80 mgd by means of a PE Bypass. Figure 4-5 illustrates the proposed site plan for the 80-mgd PE bypass.



NOTE:

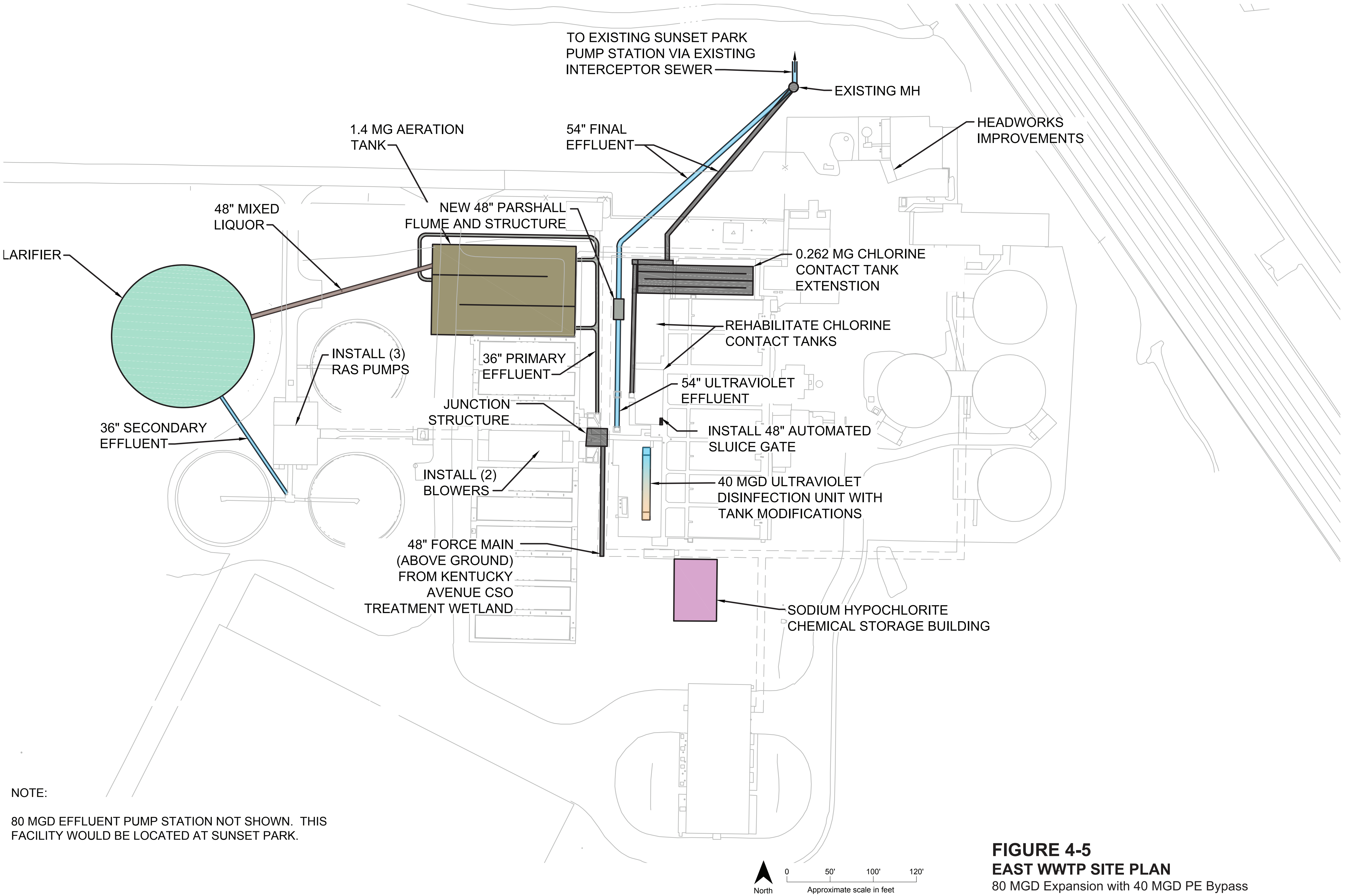
68 MGD EFFLUENT PUMP STATION NOT SHOWN. THIS FACILITY WOULD BE LOCATED AT SUNSET PARK.

FIGURE 4-4

EAST WWTP SITE PLAN

68 MGD Expansion with 40 MGD PE Bypass

May 31, 2013



NOTE:

80 MGD EFFLUENT PUMP STATION NOT SHOWN. THIS FACILITY WOULD BE LOCATED AT SUNSET PARK.

FIGURE 4-5
EAST WWTP SITE PLAN
80 MGD Expansion with 40 MGD PE Bypass
May 31, 2013

SECTION 5

Recommended Plan – May 31, 2013

5.1 System-wide Alternatives Analysis Recommendations

Scenarios to expand the wet-weather treatment capacities for the West and East WWTPs were evaluated for various flow rates, as presented in Sections 3 and 4. The costs for these improvements were incorporated into the respective systemwide Alternatives Analysis, which identified the optimal combination of improvements to mitigate CSOs and SSOs. The systemwide improvements include a combination of storage, satellite treatment, natural wetlands, and conveyance projects. Refer to the LTCP for details regarding systemwide alternatives analysis. The resulting analysis indicates that the optimal capacity for the West and East WWTPs is 40 mgd and 68 mgd, respectfully.

5.2 West WWTP

Currently, the peak treatment capacity of the West WWTP is 40 mgd. Stress testing indicates that both influent screening and primary clarification are limiting unit processes. However, the Headworks Facility should be replaced for the following reasons:

- In order to pump 40 mgd, flooding the wet well is required, which causes sewage to bypass the screen and surcharging and potential overflows in the influent sewers upstream of the WWTP.
- Continuing to operate the wet well in flooded conditions could render any upstream capacity improvements ineffective due to the backwater conditions created by wet-well flooding.
- There is no space to install a redundant screen, which is needed for process reliability.

The second limiting unit process at the West WWTP is primary clarification, which also has a capacity of 40 mgd. However, during wet-weather conditions, it would be possible to divert effluent from the grit chambers directly into the activated sludge system. The process models indicate that in rare instances (future peak month loading during low temperatures), the activated-sludge system would not have sufficient capacity to provide nitrification during a primary clarifier bypass condition. However, the Utility would still benefit from increased treatment capacity during most wet-weather conditions. The estimated cost of a 24-inch bypass line, junction structure, and fixed weir is approximately $45,000.

In reviewing the capacities for each unit process, and the incremental cost to expand the Headworks Facility and install a primary effluent bypass line, it became evident that with a $0.26 million investment (1.3 percent cost increase), the wet-weather capacity of the West WWTP could be increased from 40 mgd to 45 mgd. Therefore, the recommended improvements presented in Table 5-1 are based on an expansion of the West WWTP to 45 mgd. A site plan, process flow diagram, and hydraulic profile of the recommend plan are shown on Figures 5-1, 5-2, and 5-3, respectively.

Table 5-1 West WWTP Recommended Process Improvements

Location	Recommendation	Capital Cost (Million $)
Headworks		**19.99**
Screening Required	Two 45-mgd, 0.25-inch fine screens with washer/compactor	
Grit Removal Required	Two 22.5-mgd vortex grit units with classifying, washing, and dewatering equipment	
Pumping Required	Three 22.5-mgd pumps	
Other	Construct new Headworks Facility with wet well/dry well ~35 feet deep Construct new influent diversion structure (~15 feet by 15 feet) with actuated sluice gates	
Primary Treatment		**0.04**
Bypass Line	Install 24-inch pipe between existing grit effluent and primary effluent channels to bypass 5 mgd	
Secondary Treatment		**0.74**
Additional RAS/WAS Pumping Capacity Required	Two centrifugal RAS pumps; 2,100 gallons per minute each	
Disinfection		**1.39**
Chemical Feed/ Storage Required	Construct new Chemical Feed Building with three 7,500-gallon fiberglass-reinforced plastic storage tanks	
Total		**22.16**

5.3 East WWTP

The current design peak capacity of the East WWTP is 22.5 mgd. Stress testing determined that under favorable operating conditions, the East WWTP has the potential to treat peak flows up to 28 mgd. However, it should be noted that stress testing was conducted under controlled, dry-weather conditions and only offers an estimate of capacity in that “snap-shot” of time. Therefore, the results may not represent actual performance during all wet-weather events.

The 68 mgd PE bypass alternative is recommended to expand the wet-weather capacity to 68 mgd. As described in Section 4.4.1, this configuration would involve reactivating the East WWTP’s PE bypass to allow the WWTP to operate primary and secondary processes in parallel during certain wet-weather events. During those events, 40 mgd from the existing headworks facility would receive primary equivalency; 28 mgd conveyed from Bee Slough control measure facilities would receive secondary equivalency. All flow would be disinfected.

This East WWTP treatment configuration is intended to be operated closely with the Bee Slough Control Measure, which addresses CSO volumes associated with CSOs 001, 002, and 004. During wet-weather events, flow from those three CSOs, as well as flow from the WWTP influent sewer, will be collected and routed to proposed infrastructure. Routing will be determined by the duration and volume of individual storm events. As such, a robust control system is anticipated. Refer to Section 5.5 of the LTCP for more details on the recommended Bee Slough control measure and its interaction with the East WWTP.

Operating the East WWTP's primary and secondary process in parallel would result in a blended effluent. Analysis indicates that the blended WWTP effluent would routinely exceed the weekly and monthly NPDES requirements for TSS and BOD. While it would be possible for the primary effluent to meet the disinfection standards, it would not be able to achieve the secondary treatment standards for BOD and TSS concentrations, even when blended with secondary effluent. Therefore, when utilizing the PE Bypass the Utility should seek to comply with bacterial water quality standards, but not TSS and BOD requirements. Furthermore, the Utility should continue to monitor the primary effluent concentrations of TSS and $CBOD_5$ and implement reasonable measures (such as storing or relocating the discharge of the WWTP's recycle flows and or adding a coagulant and polymer to primary clarifiers) to improve primary effluent quality.

In addition to expanded disinfection facilities at the East WWTP, an effluent pump station would be required to convey final effluent to the Ohio River during high river stages, instead of using the K-4 Levee Pumping Station. Table 5-2 summarizes the recommended improvements required to expand the East WWTP to 68 mgd and to pump effluent directly to the Ohio River without reliance on the K-4 Levee Pumping Station. A site plan, process flow diagram, and hydraulic profile are shown on Figures 5-4, 5-5, and 5-6, respectively.

Table 5-2 East WWTP Recommended Process Improvements

Location	Recommendation	Capital Cost (Million $)
Disinfection		**7.53**
Existing Contact Tank Modifications Required	Rehab old chlorine contact tanks	
	Construct 0.262-MG contact tank extension	
	Replace existing 48-inch sluice gate with automated gate and level sensor	
	New 54-inch chlorinated effluent sewer	
Chemical Feed/ Storage Required	New Chemical Feed Building with three 7,500-gallon fiberglass-reinforced plastic storage tanks	
Conveyance from Bee Slough Control Measure		**1.37**
Conveyance System[a]	Construct new junction box and install force main	
WWTP Effluent Pumping/ Conveyance		**12.28**
Pump Station Building Required	Construct new effluent pump station (~ 20 feet tall/ 460 ft^2); wet well ~32 feet deep	
Effluent Pumping Required	Install three submersible pumps, 13,900 gallons per minute each	
Effluent Force main Required	Slip-line existing 84-inch sewer with ~600 LF of 54-inch high-density polyurethane	
Other	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	
Total		**21.18**

[a] Pump station costs accounted for in Section 5.5 of the LTCP, Bee Slough Control Measure

5.3.1 No Feasible Alternative Discussion

The Utility has developed this IOCP to address both combined and sanitary sewer overflows. The plan, as described in Volume 1, Integrated Overflow Control Plan, anticipates total capital expenditures of approximately $800 million over a 28 year period ($540 million in 2013 dollars). To fund the IOCP, Utility's sewer customers are anticipated to bear a doubling of sewer bills over the next 7 years with increases of 25 percent in 2013, 8 percent, in each of 2014 and 2015, 18 percent in 2016, and 15 percent in each of 2017 through 2019, with an average annual increase from 2012 to 2019 of 14.8 percent. The Utility is concerned about the significant adverse impacts that such increases will have on it commercial and industrial customers as well the incredible burden on residential customers. The residential burden is projected to increase from 1.1 percent of median household income (MHI) to over 2 percent in just 7 years for the 74 percent of residential customers that reside within the City of Evansville. The Utility fully anticipates that both residential and non-residential customers will implement strategies and programs that will seek to mitigate the impact of the anticipated rate increases, which will force the Utility to increase rates even further to collect the necessary revenues. Those least able to modify behaviors will pay proportionately more. Those residential customers with incomes of less than $25,000 will be facing sewer bills that are approaching 3 percent of MHI.

The Utility's final plan for the East WWTP, as described in Section 5.3 above, would expand the facility's wet-weather capacity to 68 mgd by reactivating the PE bypass to allow the WWTP to operate primary and secondary processes in parallel. The final plan would provide 28 mgd to secondary equivalent treatment and 40 mgd of primary equivalent treatment before disinfection and discharge of the blended effluent. When operating in parallel, the blended effluent will achieve applicable water quality-based effluent limits for bacteria, but may not always meet secondary treatment requirements related to TSS and BOD.

The U.S. Environmental Protection Agency (EPA) CSO Control Policy (EPA, 1994) acknowledges that maximizing use of WWTP facilities in this manner can provide substantial benefits in treating wet-weather flows:

> In some communities, POTW treatment plants may have primary treatment capacity in excess of their secondary treatment capacity. One effective strategy to abate pollution resulting from CSOs is to maximize the delivery of flows during wet weather to the POTW treatment plant for treatment. Delivering these flows can have two significant water quality benefits: First, increased flows during wet weather to the POTW treatment plant may enable the permittee to eliminate or minimize overflows to sensitive areas; second, this would maximize the use of available POTW facilities for wet weather flows and would ensure that combined sewer flows receive at least primary treatment prior to discharge.[1]

The Utility believes that the final plan for the East WWTP takes best advantage of these benefits by providing greater wet-weather treatment capacity, at a lower cost than expansion of secondary capacity. EPA has indicated that such practices can be justified only after conducting a "no feasible alternatives" (NFA) analysis under federal bypass regulations at 40 *Code of Federal Regulations* (CFR) 122.41(m). If the blended effluent consistently complied with all applicable

[1] CSO Policy, 59 Fed. Reg. at 18,693.

secondary treatment requirements at the point of discharge, an NFA should not be required.[2] However, in this case, there may be times when the blended flows from the East WWTP will not comply with all secondary treatment requirements governing BOD and TSS. As a result, the Utility is providing this NFA.

The CSO Control Policy makes it clear that the alternatives analysis contained in a permittee's CSO long-term control plan can serve as an independent basis for authorization of peak wet-weather discharges:

> For some CSO-related permits, the study of feasible alternatives in the control plan may provide sufficient support for the permit record and for approval of a CSO-related bypass in the permit itself, and to define the specific parameters under which a bypass can legally occur.[3]

The CSO Control Policy also describes how NFA requirements can be met:

> EPA further believes that the feasible alternatives requirement of the regulation can be met if the record shows that the secondary treatment system is properly operated and maintained, that the system has been designed to meet secondary limits for flows greater than the peak dry weather flow, plus an appropriate quantity of wet weather flow, and that it is either technically or financially infeasible to provide secondary treatment at the existing facilities for greater amounts of wet weather flow.[4]

The Utility has demonstrated that the secondary treatment system is and will continue to be properly operated and maintained, and that by providing 28 mgd of secondary equivalent treatment, the East WWTP has been designed to meet secondary limits for flows greater than the peak dry weather flow plus an appropriate quantity of wet-weather flow. The analysis here demonstrates that it is financially infeasible to provide secondary treatment at the existing facilities for greater amounts of wet-weather flow. As a result, the final plan, including the PE bypass and discharge of blended flows, is justified under 40 CFR 122.41(m), and can be authorized in the Utility's NPDES permit.

The Utility has evaluated a number of alternatives to the final plan to understand the technical and financial implications of various approaches that would eliminate the need to discharge blended primary and secondary effluent flows at the East WWTP and the treatment of flows from the Kentucky Ave CSO. These alternatives are described in Section 4 of this Volume 4, and would more than double the costs of improvements at the treatment plant and in some cases increase the costs by more than 400 percent. The key alternatives being evaluated are as follows:

- Expansion of secondary capacity to provide total capacity of 40 mgd with no separate sidestream treatment. The capital costs for this alternative, which would simply increase secondary capacity to match existing primary capacity, is approximately $47.0 million, more

[2] *See, Iowa League of Cities v. EPA,* 8th Cir., No. 11-3412 (March 25, 2013) ("insofar as the blending rule imposes secondary treatment regulations on flows within facilities, we vacate it as exceeding the EPA's statutory authority").

[3] CSO Policy at 18,693.

[4] CSO Policy at 18,694.

than double the costs for the comparable improvements under the final plan (so called Scenario 2).

- Construct 32 mgd of additional secondary treatment; utilize 20 mgd of new secondary capacity for Kentucky Ave CSO. The capital costs for this alternative are over $88 million, four times greater than those for the comparable improvements under the final plan (so called Scenario 5).

The purpose of this analysis is to assess the additional burden on the City of Evansville and its ratepayers of implementing an alternative approach at the East WWTP comparable to Scenario 2 or 5.

5.3.1.1 Financial Analysis

To assess the additional burden resulting from implementing one of the alternative approaches, the financial and affordability analysis completed for the final plan was redone substituting Scenarios 2 and 5 for the East WWTP and Kentucky Ave CSO approach in the final plan. As noted previously, both Scenario 2 and 5 will impose additional capital costs increasing the total capital costs to $817 and $865 million respectively. Given the overall project sequencing and the need to address plant capacity, the additional cost will be incurred by 2021; this is the time frame when projected rate increases are at their highest and where the economic hardship resulting from the final plan will likely be the greatest. This underscores the Utility's concerns over additional requirements.

The following table compares the projected household bills for in-city residents of the final plan to Scenarios 2 and 5. As noted earlier, these projections are based on the same set of assumptions and the same methodology that has been prepared for the final plan. The only modification is the change in the proposed improvements at the East WWTP.

NFA-1 Monthly Projected Residential Bills

	2012	2013	2014	2015	2016	2017	2018	2019
Recommended	$26.32	$32.90	$35.55	$38.40	$45.35	$52.20	$60.05	$69.10
Scenario 2	$26.32	$32.90	$35.55	$38.40	$45.35	$52.20	$60.05	$69.10
Scenario 5	$26.32	$32.90	$35.55	$38.40	$45.35	$52.20	$60.05	$69.10
	2020	**2021**	**2022**	**2023**	**2024**	**2025**	**2026**	
Recommended	$69.80	$70.50	$71.95	$72.70	$74.90	$75.65	$76.45	
Scenario 2	$69.80	$70.50	$74.05	$75.55	$77.85	$78.65	$79.45	
Scenario 5	$72.60	$74.10	$79.30	$80.90	$83.35	$84.20	$85.05	

As can be seen from the table, projected household bills are the same from 2013 to 2020, when the projected costs of Scenario 5 increase begins to diverge from the final case. Scenario 5 eventually peaks at a projected household bill over 11 percent greater than the final plan. Scenario 2 costs begin to exceed the final plan in 2022, eventually peaking at a cost approximately 4 percent higher. These cost differentials, although they vary slightly from year to year are permanent for the entire forecast period and until all debt associated while the treatment plant upgrade work is outstanding.

The additional projected annual costs directly translate into an additional ongoing burden. The Utility's final plan with its 28-year schedule was designed to keep the residential burden at approximately 2 percent for the in-city residents that make up the bulk of the Utility's customer base and whom have a household income of approximately 60 percent of the residential customers who reside outside the City of Evansville. As noted, the Utility is concerned about imposing a burden of 2 percent on MHI on its customers, and going beyond that is not tenable from the Utility's perspective. The following table compares the residential burden index under the final plan to Scenarios 2 and 5.

NFA-2 Projected Residential Burden, In City Customers—In-City Burden

	2012	2013	2014	2015	2016	2017	2018	2019
Recommended	1.10%	1.15%	1.20%	1.39%	1.58%	1.78%	2.02%	2.02%
Scenario 2	1.10%	1.15%	1.20%	1.39%	1.58%	1.78%	2.02%	2.02%
Scenario 5	1.10%	1.15%	1.20%	1.39%	1.58%	1.78%	2.02%	2.02%
	2020	**2021**	**2022**	**2023**	**2024**	**2025**	**2026**	
Recommended	2.01%	2.00%	2.01%	2.00%	2.02%	2.01%	2.00%	
Scenario 2	2.01%	2.00%	2.07%	2.07%	2.10%	2.09%	2.08%	
Scenario 5	2.09%	2.10%	2.21%	2.22%	2.25%	2.24%	2.23%	

As with the projected household bills, the impacts of Scenario 2 and Scenario 5 are not seen until 2022 and 2020, respectively. The burden under Scenario 5 is more than 10 percent greater than the final plan and Scenario 2 is approximately 4 percent greater. As has been noted, this level of burden is likely to be the straw that breaks the camel's back, and the Utility is concerned that the economic impacts resulting from this, including reduce billable flow caused by retrofitting and businesses and households leaving the service area, will result in bills and burdens much higher than are currently being projected and will undermine the ability of the Utility to ensure the integrity of its sewer infrastructure.

The Utility believes that the economic shocks resulting from the IOCP are partially captured by the residential burden index, but also believes that the rate of increase in bills is also a valid and useful indicator. All customers will react to how much they see those bills increasing and what the anticipated pattern is. This measure also has the advantage of addressing the impacts on all customers and not ignoring industrial and commercial customers who are critical in ensuring the viability of the overall sewer utility. The following table compares the projected rate increases between the final plan and Scenarios 2 and 5.

NFA-3 Projected Rate of Rate Increases—% change/ yr by yr

	2012	2013	2014	2015	2016	2017	2018	2019
Recommended	25.00%	8.05%	8.02%	18.10%	15.10%	15.04%	15.07%	25.00%
Scenario 2	25.00%	8.05%	8.02%	18.10%	15.10%	15.04%	15.07%	25.00%
Scenario 5	25.00%	8.05%	8.02%	18.10%	15.10%	15.04%	15.07%	25.00%
	2020	**2021**	**2022**	**2023**	**2024**	**2025**	**2026**	
Recommended	1.01%	1.00%	2.06%	1.04%	3.03%	1.00%	1.06%	
Scenario 2	1.01%	1.00%	5.04%	2.03%	3.04%	1.03%	1.02%	
Scenario 5	5.07%	2.07%	7.02%	2.02%	3.03%	1.02%	1.01%	

Table NFA-3 shows that to undertake this program over the next 10 years, the Utility and its customers will face a steady pattern of double digit rate increases. Even under the final plan the ability of the Utility's customers to absorb a consistent pattern of such increases is questionable. It is likely and entirely possible that all of the Utility's customers will take actions to mitigate the impacts reducing their billable volumes and increasing the rate of increase and costs on remaining customers. Under all three cases, the average annual rate increase between 2013 and 2019 will be nearly 15 percent, causing rates to more than double in that time period. From 2019 to 2025, under the final plan, the rate increases are projected to moderate to rates approximately equal to the anticipated increase in household income. Under Scenario 5, the projected increases in that same time period (2019 to 2025) are anticipated to be twice as great as the projected increase in household income.

5.3.1.2 Conclusions

Given the aggressive rate increases necessary to fund the final plan, and based on the additional financial evaluation conducted for Scenarios 2 and 5, those alternatives (and others similar to them) are not financially feasible and would likely impose irreparable harm on the Utility. In addition, the final plan provides substantially greater benefits in terms of wet-weather flow treatment, at a substantially reduced cost. The analysis here demonstrates that it is financially infeasible to provide secondary treatment at the existing facilities for greater amounts of wet-weather flow. As a result, the final plan, including the PE bypass and discharge of blended flows, is justified under 40 CFR 122.41(m), and can be authorized in the Utility's NPDES permit.

Additionally, the blending approach at the East WWTP set forth in Evansville's plan and further explained in the No Feasible Alternative discussion above is projected to meet permit limits for bacteria. TSS and BOD are challenging metrics for permit compliance during certain wet-weather conditions when primary and secondary flows are blended and discharged. Consequently, in addition to a NFA determination to the extent it is necessary, an NPDES permit modification that accommodates higher levels of TSS and BOD during certain wet-weather events is a feasible approach to authorizing bypassing full treatment and blending primary and secondary flows before discharge from the East WWTP.

5.4 Project Sequencing

This evaluation provided a representative approach and cost estimates for expanding the WWTPs. Preliminary engineering should include validation of the design flow and loading conditions, optimization of the concepts presented in this report, and a detailed survey of existing site conditions prior to implementing any projects described in this report. At the East WWTP, preliminary engineering should place specific emphasis on the WWTP's interaction with the selected Bee Slough Control Measure (Kentucky Avenue Treatment Wetland). Refer to Chapter 3 of the *Bee Slough Alternatives Analysis* Report (CH2M HILL, 2013) and Section 5.5 of the LTCP for more details on the Bee Slough control measure.

The specific schedule for implementation of the WWTP projects can be found in the Volume 1 of IOCP.

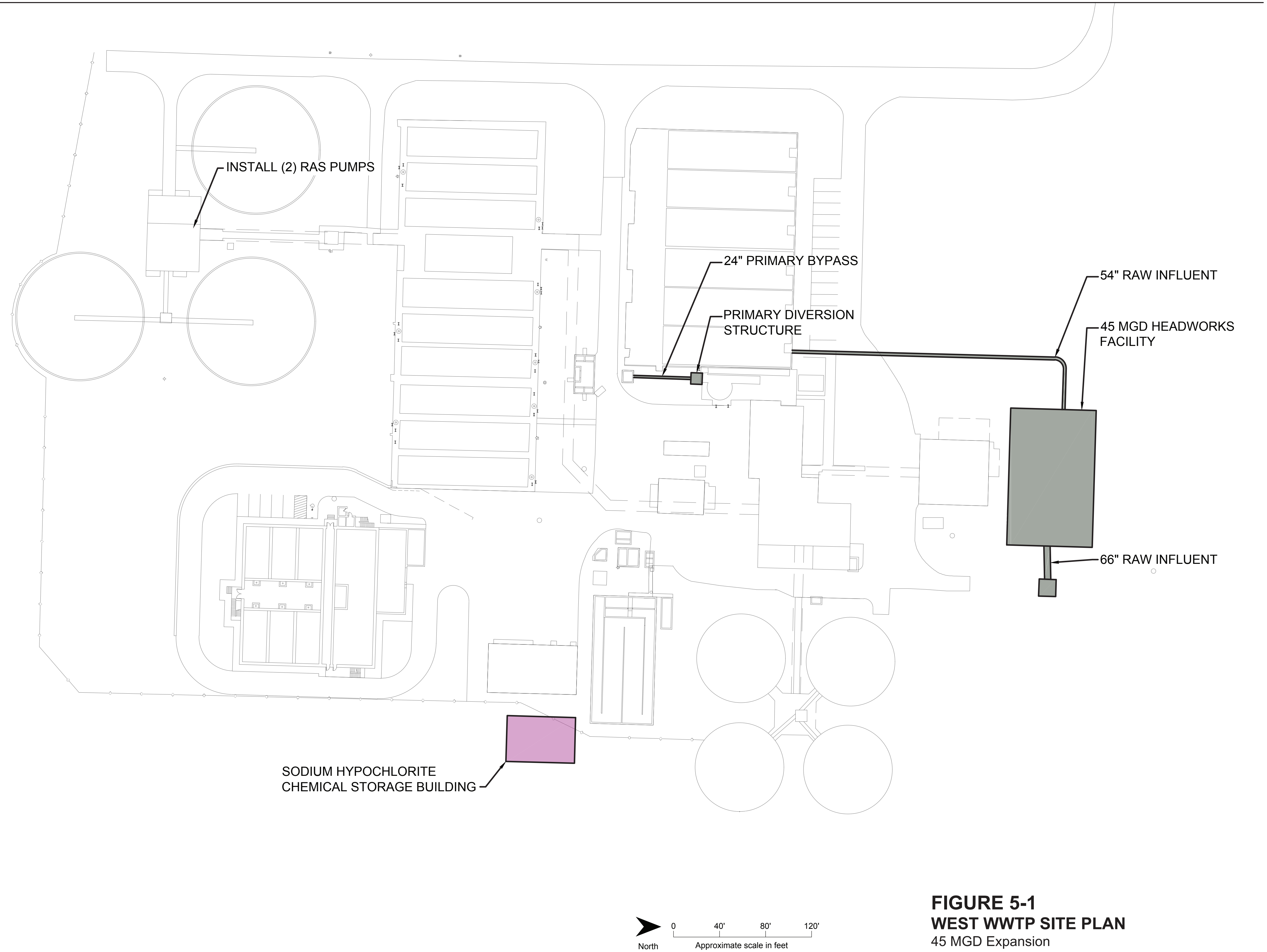


FIGURE 5-1
WEST WWTP SITE PLAN
45 MGD Expansion
May 31, 2013

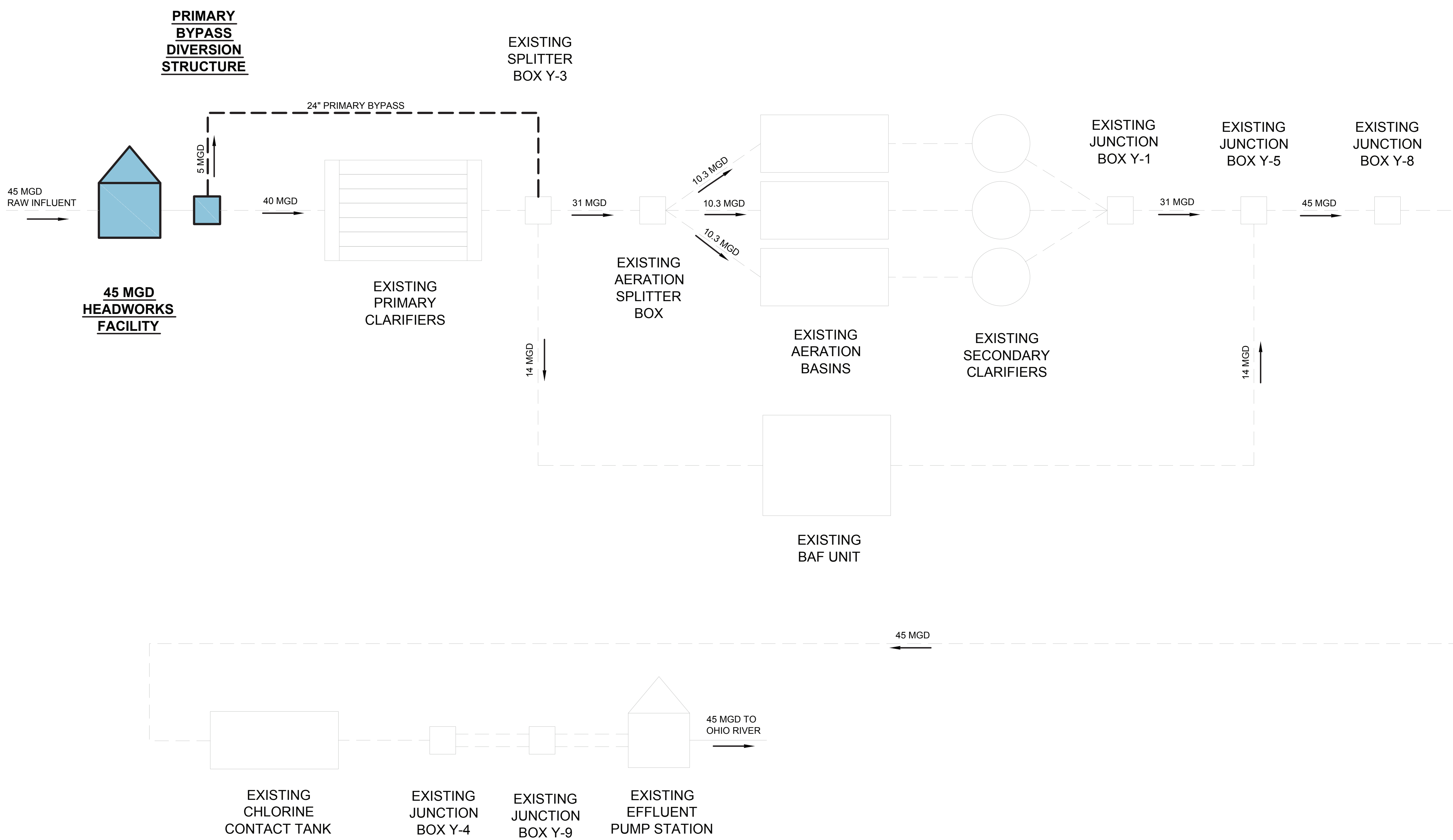
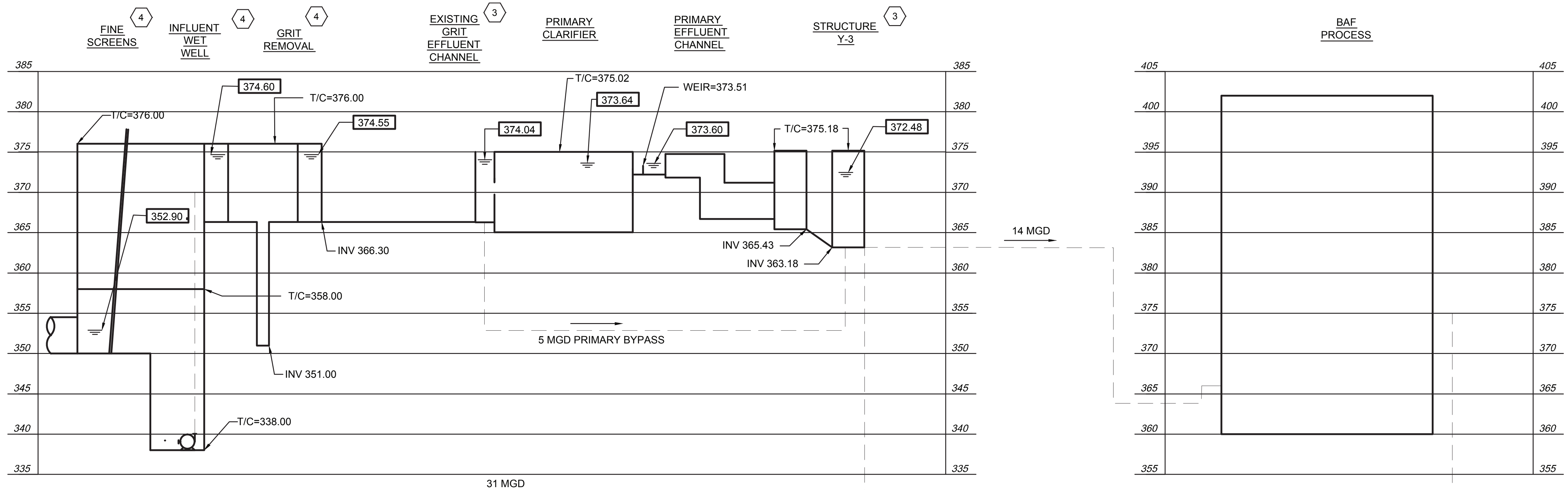
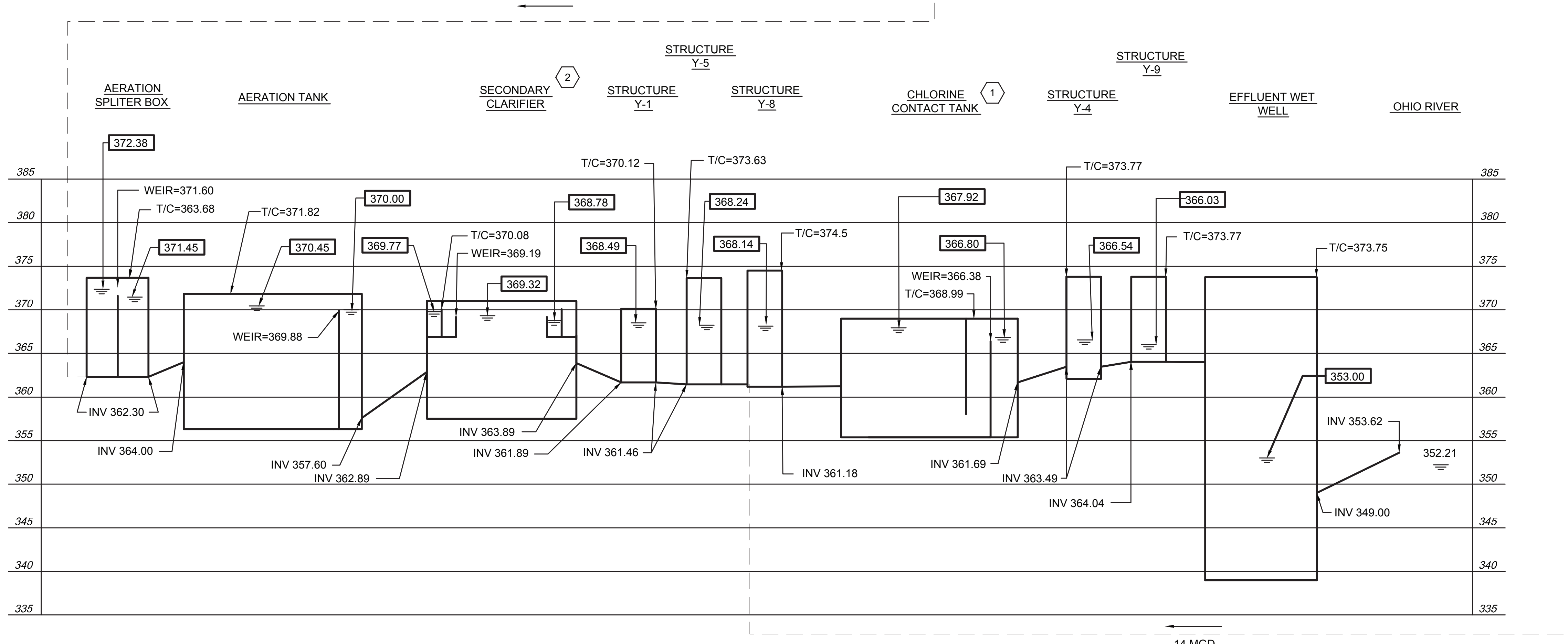


FIGURE 5-2
WEST WWTP PROCESS FLOW DIAGRAM
45 MGD Expansion
May 31, 2013





LEGEND

XXX.XX HGL AT 45 MGD

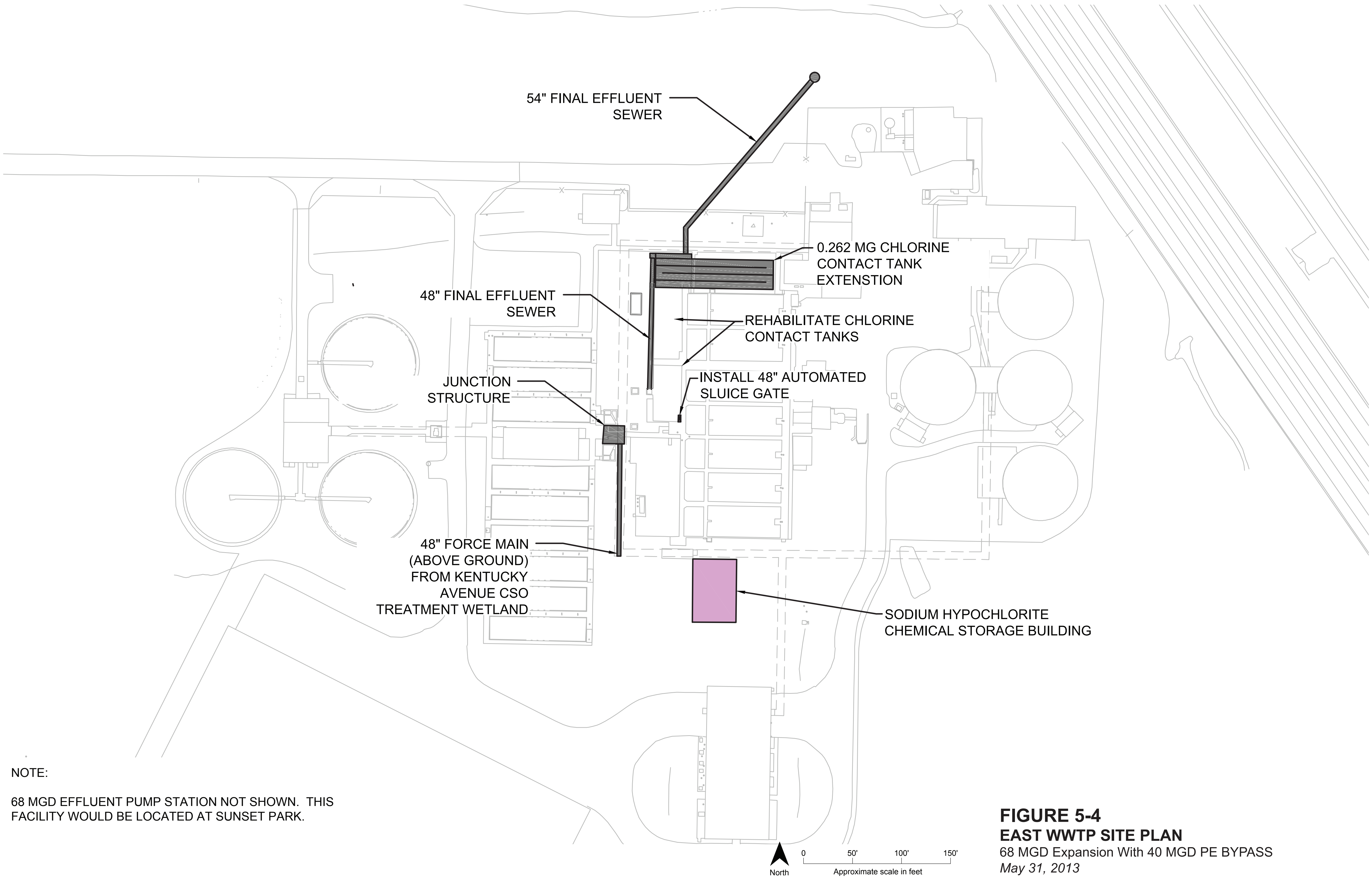
SHEET KEYNOTES

1. RETURN ACTIVATED SLUDGE FLOW MODELED AT 50% OF INFLUENT FLOW
2. 24" PRIMARY BYPASS AND DIVERSION STRUCTURE CONSTRUCTED AT EXISTING GRIT REMOVAL EFFLUENT CHANNEL, CONNECTS TO EXISTING STRUCTURE Y-3.
3. UNIT PROCESS LOCATED IN NEW 45 MGD HEADWORKS FACILITY

FIGURE 5-3
WEST WWTP HYDRAULIC PROFILE

45 MGD Expansion

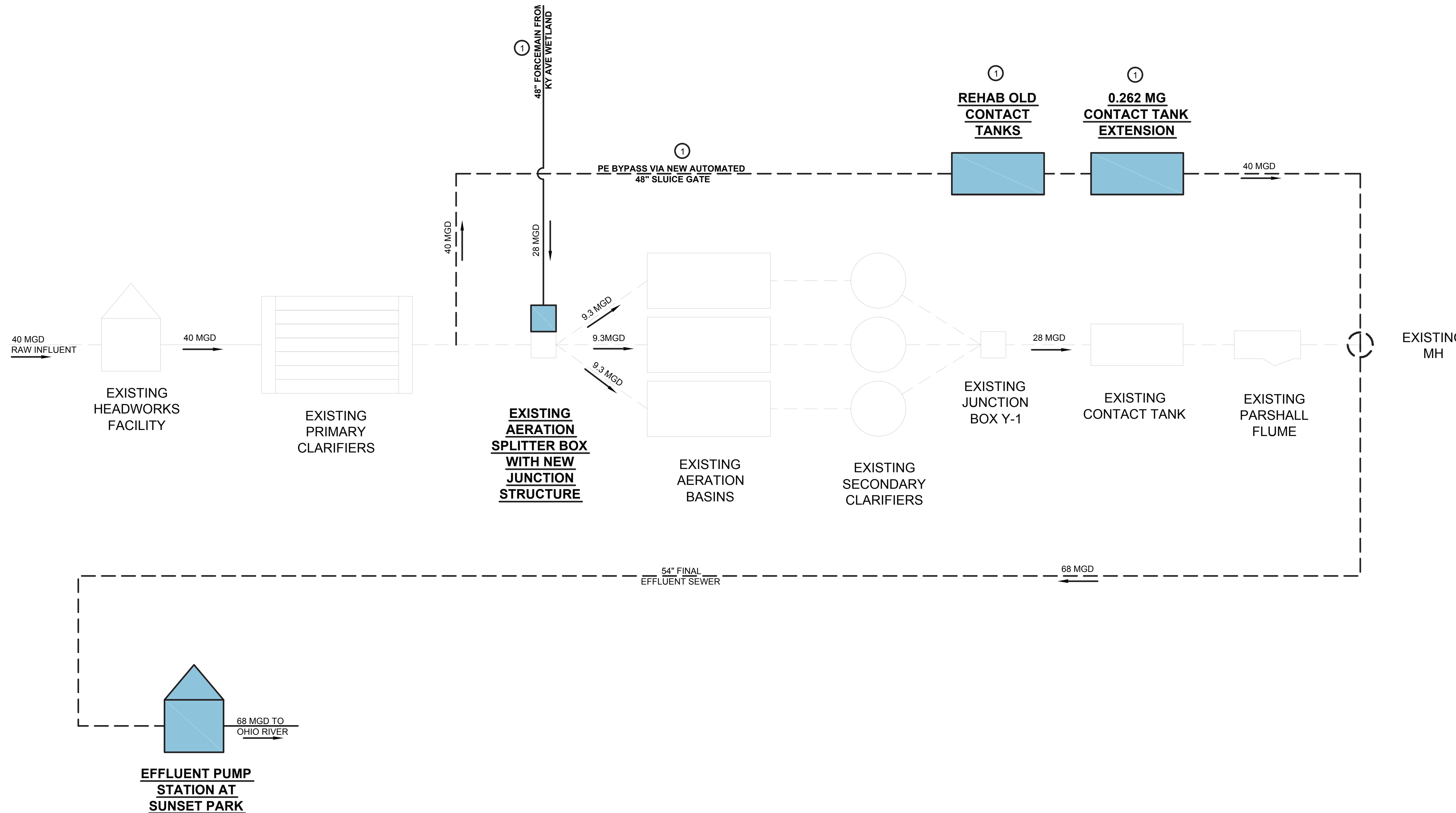
May 31, 2013



NOTE:

68 MGD EFFLUENT PUMP STATION NOT SHOWN. THIS FACILITY WOULD BE LOCATED AT SUNSET PARK.

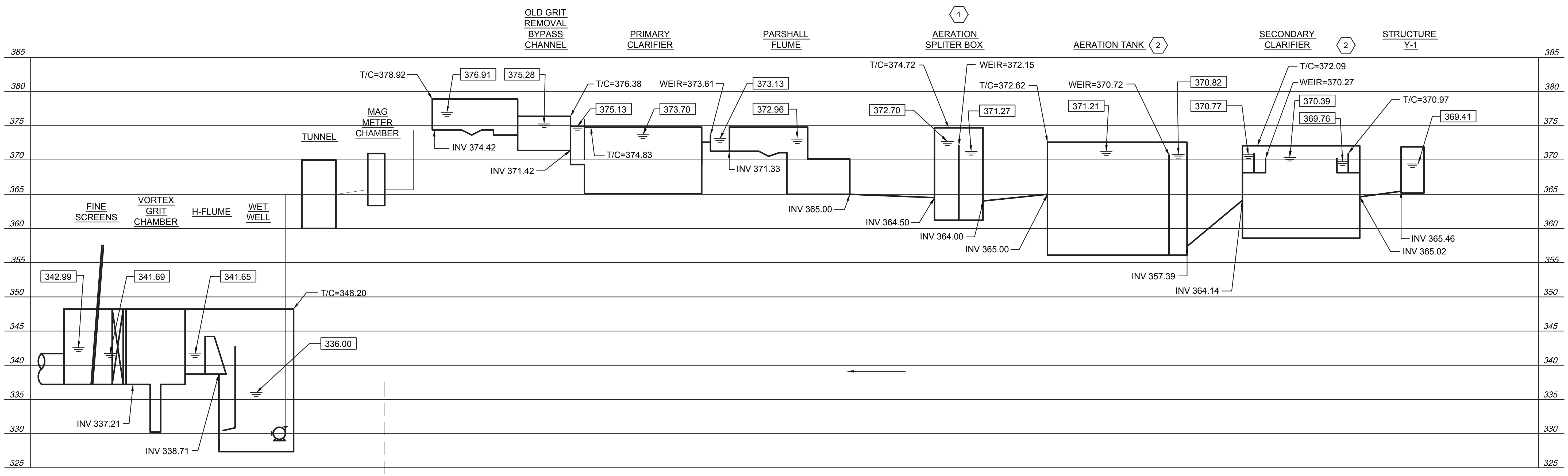
FIGURE 5-4
EAST WWTP SITE PLAN
68 MGD Expansion With 40 MGD PE BYPASS
May 31, 2013

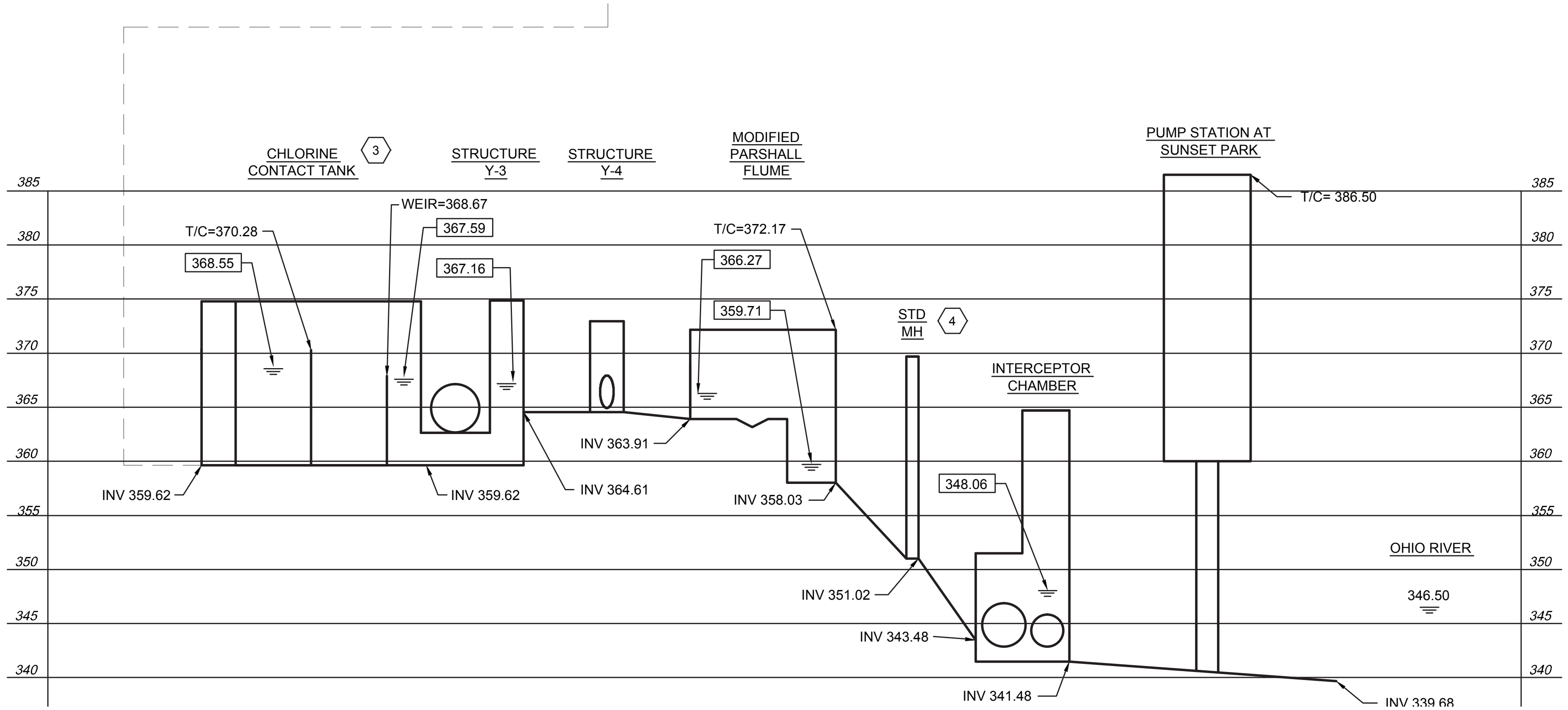


NOTE:

1. INFRASTUCTURE NOT OPERATED IN DRY WEATHER

FIGURE 5-5
EAST WWTP PROCESS FLOW DIAGRAM
68 MGD Expansion
May 31, 2013





LEGEND

XXX.XX HGL AT 68 MGD

SHEET KEYNOTES

1. 40 MGD FROM PRIMARIES DIVERTED TO NORTH CHLORINE CONTACT TANKS. 28 MGD THRU SECONDARY PROCESS CONVEYED FROM KY AVE WETLAND FACILITIES
2. RETURN ACTIVATED SLUDGE MODELED AS 50% OF INFLUENT FLOW.
3. NORTH CHLORINE CONTACT TANKS NOT SHOWN. HGL IN THOSE TANK IS APPROXIMATELY 370.30 @ 40 MGD.
4. DISINFECTED P.E. RECOMBINES WITH DISINFECTED SECONDARY EFFLUENT. FLOW IS 68 MGD FROM THIS POINT DOWNSTREAM.

FIGURE 5-6
EAST WWTP HYDRAULIC PROFILE
May 31, 2013

renewEVANSVILLE

SECTION 6

Final Negotiated Plan – December 4, 2015

6.1 Final Negotiated Plan

Following extensive discussions with EPA and IDEM following the submission of the Recommended Plan set forth above, Evansville and the agencies agreed to a Final Negotiated Plan that includes the improvements to the treatment plants set forth below. This plan supercedes the Recommended Plan.

6.2 West WWTP

Changes to the West WWTP included elimination of the primary bypass features. Two alternatives for this change were developed, the details of which can be seen in Appendix F. The Utility agreed to implement Alternative 2, which involves raising primary clarifier weirs and constructing curb walls for the primary clarifier influent channel. The Negotiated Plan therefore includes these changes along with upgrades originally recommended to bring treatment capacity to 45 MGD (new headworks facilitiy, conversion to liquid chlorine for disinfection, and upgrades to the existing RAS system). Table 6-1 summarizes the cost breakdown to expand the West WWT. Refer to Figure 6-1 for a site plan of the facility improvements.

Table 6-1 West WWTP Recommended Process Improvements

Location	Recommendation	Capital Cost (Million $)
Headworks		**19.99**
Screening Required	Two 45-mgd, 0.25-inch fine screens with washer/compactor	
Grit Removal Required	Two 22.5-mgd vortex grit units with classifying, washing, and dewatering equipment	
Pumping Required	Three 22.5-mgd pumps	
Other	Construct new Headworks Facility with wet well/dry well ~35 feet deep	
	Construct new influent diversion structure (~15 feet by 15 feet) with actuated sluice gates	
Primary Treatment		**0.24**
Tank Modifications	Raise weirs approximately 5 inches; construct curb walls atop influent channel walls	
Secondary Treatment		**0.74**
Additional RAS/WAS Pumping Capacity Required	Two centrifugal RAS pumps; 2,100 gallons per minute each	
Disinfection		**1.39**
Chemical Feed/ Storage Required	Construct new Chemical Feed Building with three 7,500-gallon fiberglass-reinforced plastic storage tanks	
Total		**22.36**

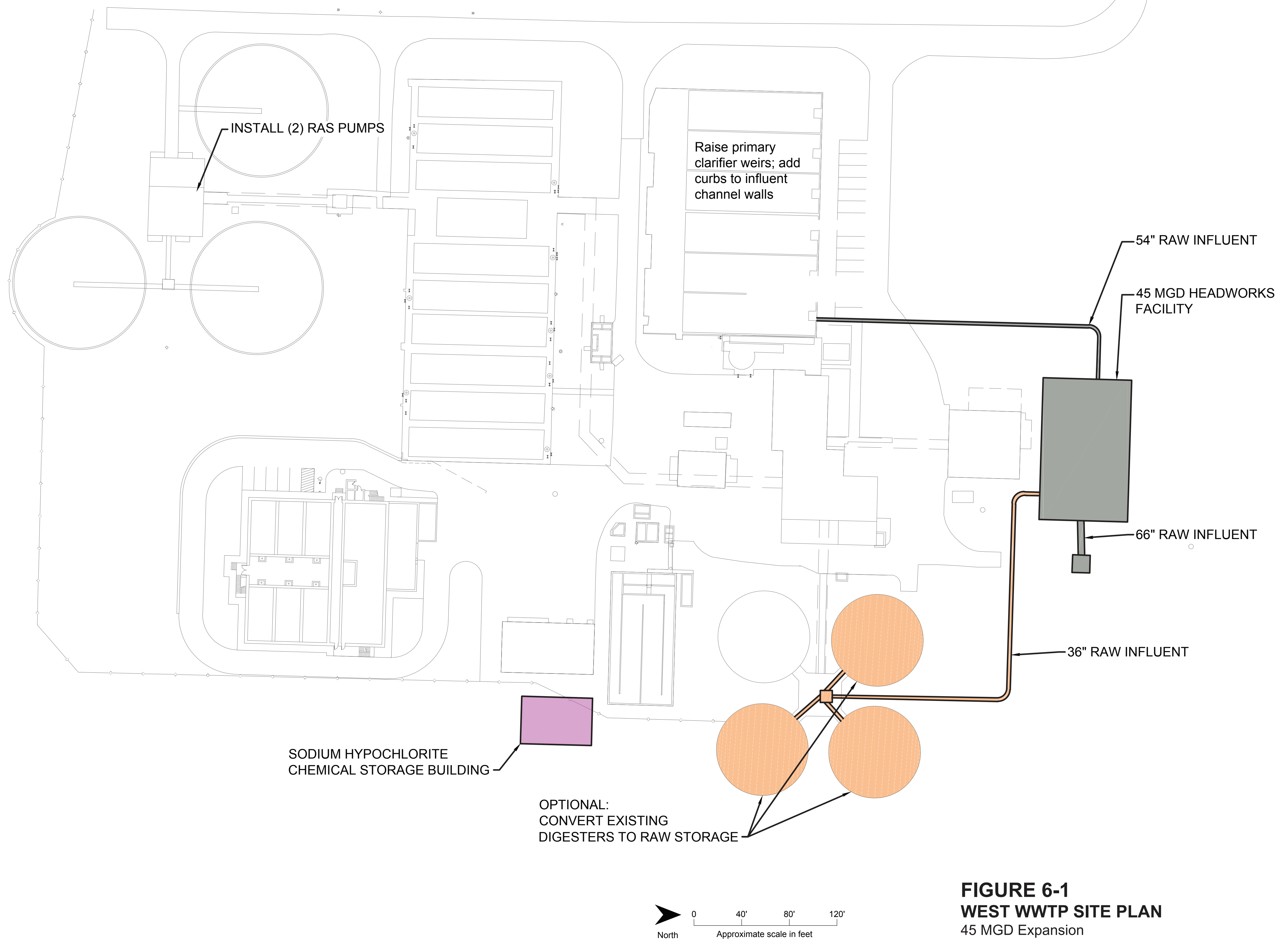


FIGURE 6-1
WEST WWTP SITE PLAN
45 MGD Expansion
January 15, 2016

6.3 East WWTP

Changes to the East WWTP Recommended Plan included elimination of the PE bypass features, decoupling of the wetland treatment system (for CSO 001) from the secondary treatment process, and expansion of total treatment capacity to 40 MGD by conventional means. As described in previous sections of this report, expanding to 40 MGD in this manner involves adding a fourth activated sludge process unit with associated equipment (blowers, diffusers, RAS/WAS pumping, etc.). In addition, the disinfection system is converted to a UV system and downstream infrastructure is upsized to accommodate the increased flow. Table 6-2 summarizes the cost breakdown for the Negotiated Plan to expand the East WWTP to 40 mgd. Refer to Figure 6-2 for a site plan of the facility improvements.

Table 6-2 East WWTP Summary of Selected Improvements

Location	Recommendation	Capital Cost (Million $)
Secondary Treatment		**31.33**
Additional Clarifier Capacity Required	One 150-foot-diameter clarifier, required ancillary equipment, and piping	
Additional Aeration Capacity Required	One 1.4-MG tank with ancillary equipment/piping	
Additional Blower Capacity Required	Two turbo blowers, each at 11,300 scfm	
Additional RAS/WAS Pumping Capacity Required	Three centrifugal pumps, 4,000 gpm at 40-foot TDH each	
Disinfection		**3.89**
UV System	Modify existing chlorine contact tank to accommodate 40-MGD UV system; upsize existing effluent piping. Modify influent channel and effluent weir to accommodate UV hydraulics	
Effluent Pumping/Conveyance		**11.78**
Disinfected Effluent Modifications Required	Replace existing 36-inch effluent line downstream of Parshall flume with a 54-inch line. Replace existing 54-inch effluent line upstream of Parshall flume with a 54-inch line, ~2 feet deeper	
Effluent Metering Required	Replace existing structure with deeper structure (~2 feet deeper). Upsize existing Parshall flume to 48-inch flume	
Pump Station Building Required	Construct new effluent pump station. Install three submersible pumps, 13,900 gpm each.	
Effluent Force Main Required	Slip-line existing 84-inch sewer with ~600 LF of 54-inch HDPE	
Other	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	
Total		**47.00**

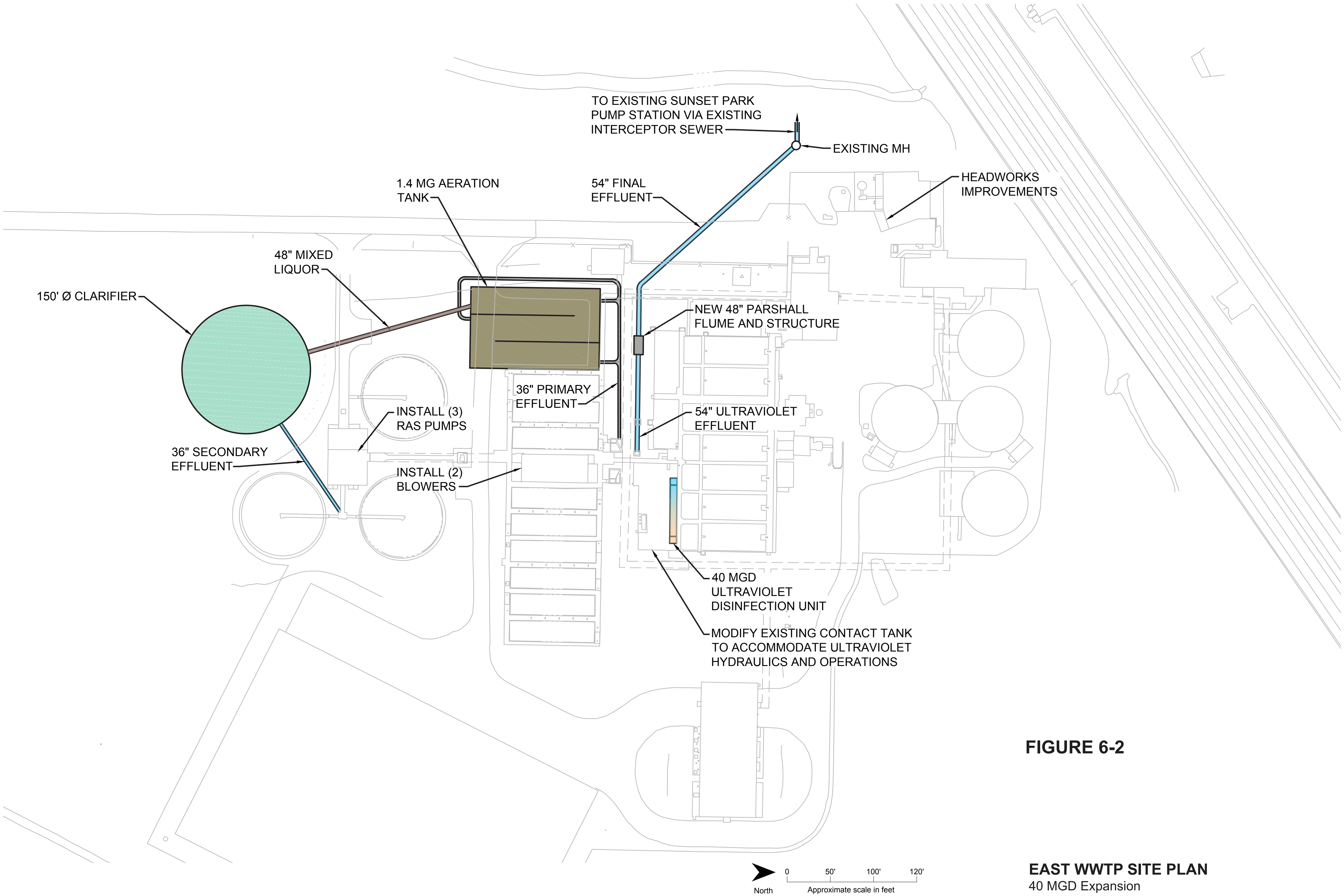


FIGURE 6-2

EAST WWTP SITE PLAN
40 MGD Expansion
January 15, 2016

SECTION 7

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APPENDIX A

Cost Estimating Guidance

Submitted 01/24/2012

Facility Plan Cost Estimating Guidelines

Prepared for

Evansville Water and Sewer Utility

City-County Administration Building
1 Martin Luther King Blvd.
Evansville, Indiana 47708

Prepared by

CH2MHILL

915 South Main Street
Suite 406
Evansville, IN 47708

January 2012

Submittal Authorization

Approved By: ______________________________ ____________________

Date

I certify under penalty of law that I have examined and am familiar with the information submitted in this document and all attachments and that this document and its attachments were prepared under my direction or supervision in a manner designed to ensure that qualified and knowledgeable personnel properly gather and present the information contained therein. I further certify, based on my inquiry of those individuals immediately responsible for obtaining the information, that I believe that the information is true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fines and imprisonment.

Contents

Tables

Figure

SECTION 1

Opinion of Probable Construction Cost

This document was prepared as guidance for design engineers in developing Opinions of Probable Construction Cost (OPCC) and includes an overview of:

- Cost Estimate Methodology and Format
- Overview of Markups
- Major Considerations; and
- Cost Estimate Class

It is intended to be used in conjunction with the cost estimating spreadsheet template, which consists of calculation sheets for construction, operations and maintenance (O&M), and client capital costs. The spreadsheet includes individual estimating sheets for each flow scenario and associated alternative.

1.1 Cost Estimate Methodology and Format

The OPCC was prepared using a cost-based method. This is a "bottom-up" pricing of contractor costs for crew sizes, associated labor, equipment, materials, rates of construction, and overhead and profit. Although sufficient detail may only be at less than 10% for some facilities, engineering judgment is used to make considerations when sufficient information is unavailable.

1.1.1 Base Unit Cost

Base unit cost, or Unit Cost, includes material, labor, equipment, and sub consultant cost. Design Engineers shall include a separate Equipment Installation Cost for all equipment related to the design alternate. Considerations for equipment costs shall be included in cost estimate for review. Base unit cost should not include any anticipated contractor mark-ups, as all markups are applied later. For more details on markups, refer to Section 1.2.

Base unit costs shall be determined using RS Means 2012, past construction project bids, material quotes and equipment manufacturer quotes. Unit costs not included in template are to be calculated by design engineering firms. Design firms shall provide source for unit cost. Table 1-1 displays considerations made for associated unit costs. Figure 1-1 displays an example construction cost template. The provided unit costs are meant as guidance and should only be included when appropriate; justification should be provided for any deviations.

TABLE 1-1
Base Unit Cost Considerations

Item	Description	Considerations:
1	Concrete Coring	48" diameter, 24" thick
2	Clear & Grub	Clearing of brush and trees up to 12" diameter
4	Haul Excess Excavation	Up to 10 miles
6	Backfill And Compaction	Heavy soil, 12" layers, hand tamp
7	Sheeting	Steel sheeting, up to 15' depth, includes extraction
9	Junction Boxes	10' deep
10	Asphalt Concrete Pavement	For parking area / not roadway

TABLE 1-1
Base Unit Cost Considerations

Item	Description	Considerations:
12	Concrete Pavement	For parking area / not roadway
13	Fencing	8' tall, 3-wire barb, incl. gates
14	Site Restoration	Fine grading, incl 1" topsoil
15	Turf	1" blue grass, sloped ground 1,000 or less
17	Concrete 4000 Psi Structural	Does not include steel reinforcement
18	Concrete 2000 Psi Non Structural	Does not include steel reinforcement
19	Reinforcement	Used with concrete
20	Miscellaneous Metals	User defined entry
21	Hand Railing	3-rail aluminum, incl gates
22	Aluminum Stairs	User defined entry
24	Access Hatch Covers	User defined entry
25	Water Repellent Treatment - Concrete	$0.20/ sf pressure wash, $2.50/sf epoxy coating for corrosive environments.
26	Water Repellent Treatment - Masonry	$0.20/ sf pressure wash, $1.20/sf two coat water repellant
28	Caulking & Sealants	1/2" x 1/2" latex
29	Doors & Frames	3'-6" x 7' with glass
30	Painting	$0.16/ sf surface prep, $.74/sf two coats
32	Temporary Pumping	User defined entry
33	Dewatering Pumps	User defined entry
34	Ductile Iron Pipe - External	RECOMMEND THIS BE INDIVIDUAL TO EACH PROJECT - Costs here represent 12" DIP, 8-ft deep, in open space (no pavement opening or restoration).
35	Ductile Iron Pipe - Internal	RECOMMEND THIS BE INDIVIDUAL TO EACH PROJECT - Costs here represent 12" DIP installed with pipe hangers
35	Copper Pipe - External	RECOMMEND THIS BE INDIVIDUAL TO EACH PROJECT - Costs here represent exterior 2" 4' depth buried
36	Copper Pipe - Internal	RECOMMEND THIS BE INDIVIDUAL TO EACH PROJECT - Costs here represent interior 2"
36	PVC Pipe - External	RECOMMEND THIS BE INDIVIDUAL TO EACH PROJECT - Costs here represent exterior 2" 4' depth buried
36	PVC Pipe - Internal	RECOMMEND THIS BE INDIVIDUAL TO EACH PROJECT - Costs here represent interior 2"
37	Yard Piping - External	RECOMMEND THIS BE INDIVIDUAL TO EACH PROJECT - Costs here represent 48" PCCP, 8-ft deep, in open space (no pavement opening or restoration).
38	HVAC	Use estimators judgment, 1-3% of Base Construction Cost (if any required)
39	Instrumentation	Use estimators judgment, 2-8% of Base Construction Cost (if any required)
40	Electrical	Use estimators judgment, 5-15% of Base Construction Cost (if any required)

FIGURE 1-1
Example Construction Cost Template

	Evansville Water and Sewer Utility			***Estimate Date***		**<Estimate Date>**
	Wastewater Treatment Plant			***Print Date***		**January 24, 2012**
	Facility planning xx MGD Treatment Capacity					
	Construction Cost Estimate			***ENRCCI 8952 - NOV 10***		
Item	***Description***	***Quantity***	***Unit***	***Unit Cost***		***Extended Cost***
	Evansville Water and Sewer Utility			***Estimate Date***	**<Estimate Date>**	
	Wastewater Treatment Plant			***Print Date***	**January 24, 2012**	**Estimator name an**
	Facility planning 40 MGD Treatment Capacity					Johny Cash - 01/0
	Construction Cost Estimate			***ENRCCI 9172 - JAN 2012***		**QC Reviewer and c**
						Richy Rich - 01/08
Item	***Description***	***Quantity***	***Unit***	***Unit Cost***	***Equipment Installation***	***Extended Cost***
1	Concrete Coring		EA	**$5,000**		$0
2	Clear & Grub		SF	**$0.25**		$0
3	Excavation		CY	**$24**		$0
4	Haul Excess Excavation		CY	**$15**		$0
5	Rock Excavation		CY	**$75**		$0
6	Backfill and Compaction		CY	**$53**		$0
7	Sheeting		SF	**$15**		$0
8	Stone & Gravel		CY	**$25**		$0
9	Junction Boxes		EA	**$128,000**		$0
10	Asphalt Concrete Pavement		SY	**$26**		$0
11	Concrete Sidewalk		SF	**$10**		$0
12	Concrete Pavement		SY	**$45**		$0
13	Fencing		LF	**$20**		$0
14	Site Restoration		SY	**$1.75**		$0
15	Turf		SF	**$0.71**		$0
16	Trees, Shrubs & Groundcover		SF	**$7.50**		$0
17	Concrete - 4000 psi structural		CY	**$530**		$0
18	Concrete - 2000 psi non-structural		CY	**$310**		$0
19	Reinforcement		TN	**$2,200**		$0
20	Miscellaneous Metals		LS	**$100,000**		$0
21	Hand railing		LF	**$95**		$0
22	Aluminum Stairs		LS	**$10,000**		$0
23	Aluminum Grating		SF	**$55**		$0
24	Access Hatch Covers		LS	**$10,000**		$0
25	Water Repellent Treatment - Concrete		SF	**$2.70**		$0
26	Water Repellent Treatment - Masonry		SF	**$1.40**		$0
27	Roofing		SF	**$30**		$0
28	Caulking & Sealants		LF	**$1.70**		$0
29	Doors & Frames		EA	**$2,100**		$0
30	Painting		SF	**$0.90**		$0
31	Fire Extinguishers/Safety Equipment		LS	**$2,000**		$0
32	Temporary Pumping		LS	**$100,000**		$0
33	Dewatering Pumps		LS	**$100,000**		$0
34	Ductile Iron Pipe - External		LF	**$175**		$0
35	Ductile Iron Pipe - Internal		LF	**$85**		$0
35	Copper Pipe - External		LF	**$40**		$0
36	Copper Pipe - Internal		LF	**$28**		$0
36	PVC Pipe - External		LF	**$35**		$0
36	PVC Pipe - Internal		LF	**$15**		$0
37	Yard Piping - External		LF	**$235**		$0
38	HVAC		LS	**$90,000**		$0
39	Instrumentation		LS	**$800,000**		$0
40	Electrical		LS	**$1,200,000**		$0
41						
42						
43						
44						
45						
46						
47						
48						
49						
50						
51						
		Total Estimated Construction Cost Before Markups =				$0

1.2 Overview of Markups

This section highlights the various markups applied to the unit costs after raw construction costs are estimated, as shown in Table 1-1. The following markups were considered and are described in greater detail in the following sections:

- Project Construction Cost Contractor Markups
- Project Construction Contingency
- Escalation Rate
- Capital Cost; and
- Excluded Costs

1.2.1 Project Construction Cost Contractor Markups

Table 1-2 summarizes the first layer of markups, which represent the standard markups applied to a detail construction cost estimate based on base unit costs to the contractor. The General Conditions mark-up includes temporary facilities and other construction requirements not typically included in contractor overhead costs. Project construction cost contractor markups are applied in the summary tab of the included excel file.

TABLE 1-2
Summary of Contractor Markups

Cost Item	Percent of Construction Cost
Contractor Overhead (office expenses)	8%
Contractor Profit	5%
General Conditions (on-site expenses)	10%
Mobilization/Demobilization	5%
Insurance	2%
Performance Bond – General	2%
Performance Bond – Electrical	1%

1.2.2 Project Construction Contingency

Table 1-3 summarizes the second layer of markups, which is comprised of the Project Construction Contingencies. These percentages represent unknown elements based on the level of design and the size of the project.

TABLE 1-3
Summary of Construction Contingency Markups

Cost Item	Percent of Construction Cost

TABLE 1-3
Summary of Construction Contingency Markups

Design Contingency	30%
Special Conditions Contingency	0%

The design contingency is the only factor in the second layer of markups and is set at 30% for planning level estimates. None of the construction techniques are assumed to utilize new technologies. Therefore, the special conditions contingency is set at 0% and left off the estimate. If special construction or new construction techniques are anticipated design engineer shall provide markup percentage with explanation of factor.

1.2.3 Escalation Rate

All estimates will be performed for a set month and year for costs. All costs developed for Facility Planning will be developed in January 2012 dollars. Escalation should only be accounted for in the development of life cycle costs, which should be calculated over the 40-year planning horizon.

1.2.4 Capital Cost Items

Table 1-4 summarizes the third layer of markups, which is Capital Cost Items. Capital cost markups are above and beyond what is included in the Contractor Markups and Project Construction Contingencies presented above. These items are considered indirectly related to the construction itself.

TABLE 1-4
Summary of Capital Cost Markups

Cost Item	**Percent of Construction Cost**
Right-of-Way Costs	0.00%
Planning & Preliminary	3.00%
Design Services	8.00%
Administration Costs	3.00%
Miscellaneous	2.00%
Capitalized Interest (1 year assumed)	3.25%
Field Engineering & Inspection	10.00%
Project Contingencies	15.00%
Total Non-Project Costs	**44.25%**

Right-of-Way costs can be provided from Client Property Division, but if unavailable a standard estimate at planning level is 1% of the construction cost estimate. If land is currently owned by the client then this can be set at 0%. The budgeted amount for client to perform Planning and Preliminary Design Services for the Utility is approximately 3% of the estimated Construction Cost; Design Services for the Utility are approximately 8%.

The miscellaneous markup is for cost items to the client that may surface later in the project that are unaccounted for at present, because they are undisclosed or unknown at this time. This markup is set at 2.00% of total construction cost.

The Capital Interest percentage is from the most recent bond issue following the *Guidelines and Discount Rates for Benefit-cost Analysis of Federal Programs* from the Office of Management and Budget of 2.7% and assumed 52 weeks (1-year) of construction. Field Engineering and Inspection is set at 10% for this project. It is anticipated that the services required during construction will not be provided by client. Lastly, the OPCC has many project elements that contain potential for changes and unknown details. Thus a Project Contingencies line item is needed for the OPCC and is set at 15%. As design is further progressed this contingency should decrease.

1.2.5 Excluded Costs

The OPCC excludes the following costs:

- Material adjustment allowances above and beyond what is included at the time of the cost opinion;
- Operations and maintenance costs; and
- Easements already acquired.

Design engineers shall perform a 40 year life cycle cost analysis to serve as the O&M cost. O&M cost considerations shall be clearly explained with submitted life cycle cost analysis.

1.3 Major Considerations

For all facilities, estimates are based on the assumption that the work will be done on a competitive bid basis and contractors will have an appropriate amount of time to complete the work. Additionally, it is assumed that all contractors are equally qualified within their discipline, with a reasonable project schedule, no overtime, constructed under multiple contracts for the separate components within the project, such as shaft construction or the consolidation sewers, and no liquidated damages. All estimates are in regards to the descriptions and drawings that should be provided. Other considerations by the estimator not provided in earlier sections should be listed out and included with the submitted OPCC.

1.4 Cost Estimate Class

Because the facilities are still at a 10% or less level of engineering design detail, the costs developed at this phase are planning level costs. Unit costs are used from RSMeans index, local labor rates but many considerations are made based on multiple cities historical project data, USEPA documentation, and researched similar project data including hard bid information. Estimates are prepared based on best available data and judgments at the time they were developed. As designated under AACE 17R-97 or ASTM E 2516-06, the current project definition aligns with a Class 4 Planning Level estimate. A Class 4 estimate includes an accuracy of +50% to -30% and is most often used to compare multiple alternatives. This cost range is given in the template for the Construction Cost and for the Capital Cost and should be presented with the estimate.

APPENDIX B

Secondary Treatment Technical Memorandum

West and East Wastewater Treatment Plants Secondary Treatment Expansion Alternatives

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DATE: July 25, 2012

Introduction

The Evansville Water and Sewer Utility (Utility) is currently developing a Facility Plan for the West and East Wastewater Treatment Plants (WWTPs). The purpose of this technical memorandum (TM) is to provide planning-level cost estimates to increase the secondary treatment capacities of the West and East WWTPs during wet weather events. The increased WWTP capacities are needed to mitigate sanitary sewer overflows and combined sewer overflows in the collection system.

This TM presents the representative capital and life-cycle cost estimates to increase the wet weather capacity of the WWTPs. These costs will be combined with other plant improvements to develop budgetary cost estimates for various treatment capacities, which will be compared with other options, such as storage and satellite treatment, in the Integrated Overflow Control Plan (IOCP).

Effluent Limits

The Indiana Department of Environmental Management has issued National Pollutant Discharge Elimination System (NPDES) permits for the West and East WWTPs. The NPDES permits establish effluent concentration limits for 5-day carbonaceous biochemical oxygen demand ($CBOD_5$), total suspended solids (TSS), ammonia (NH_3), fecal coliform, pH, total residual chlorine and E. coli. The parameters controlled by the secondary treatment process are CBOD, TSS and NH_3, as shown in Table 1.

TABLE 1
WWTP Effluent Quality

	Permit		Design (mg/L)	
	Monthly	**Weekly**	**Conventional**	**BioCEC/BAF**
West WWTP				
$CBOD_5$	20	30	<7	<30
TSS	30	45	<15	<30
NH_3	9.3	14	<1	NA
East WWTP				
$CBOD_5$	16	24	<7	<30
TSS	30	45	<15	<30
NH_3	9.3	14	<1	NA

Notes:
BAF = biological aerated filter
HRT = high rate treatment
mg/L = milligrams per liter

The NPDES permits include both concentration limits and mass limits. The weekly and monthly mass limits for CBOD, TSS, and NH_3 are based on sustained flows of 30.6 million gallons per day (mgd) for the West WWTP and 22.5 mgd for the East WWTP. In most cases, the sustained flows are below these values during a weekly or monthly time period. If the sustained flows exceed these values, the plants will need to produce a higher quality effluent to comply with the mass loadings. However, any improvements to the facilities that would expand the treatment plant capacity would require a NPDES construction permit. During the NPDES application process, the Utility can request higher mass loadings to correspond with the higher treatment plant capacities.

The design effluent concentration for various treatment options are also shown in Table 1. Conventional activated sludge produces higher quality effluent than biologically and chemically enhanced clarification (BioCEC) or Biological Aerated Filters (BAF). The predicted effluent quality for the activated sludge alternative was determined by process models assuming a sustained peak flow lasting 24 hours. Although the effluent quality may degrade for sustained flows longer than 24 hours, the effluent would still be within the permit limits provided; hence the necessity to produce effluent having a higher quality than required.

For this evaluation, the BioCEC and BAF alternatives are not sized to provide nitrification (ammonia removal) and have higher effluent concentrations of CBOD and TSS. However, when combined with the effluent from the conventional treatment process operating with a solids retention time of at least 8 days, the blended effluent should be in compliance with the permit limits.

Existing Facilities

The West WWTP is designed for an average flow of 21.7 mgd. The plant uses both conventional activated sludge and BAF for secondary treatment, and these processes are operated in parallel. The activated sludge system is designed to operate in plug flow, step feed, or contact stabilization mode. The treatment units at the West WWTP consist of fine bar screens, grit removal, six primary clarifiers, three aeration basins, three final clarifiers, BAF (6 cells), and effluent chlorination/dechlorination. Even though the NPDES permit states that the peak 24-hour sustained wet weather flow is 30.6 mgd, recent stress testing and process modeling indicate that the West WWTP can treat a sustained peak of 40 mgd (26 mgd conventional and 14 mgd BAF).

The East WWTP is designed for an average flow of 18 mgd. The treatment facility consists of fine bar screens, vortex grit system, seven primary clarifiers, three aeration basins, three final clarifiers, and effluent chlorination/dechlorination. Even though the NPDES permit states that the peak sustained wet weather flow is 22.5 mgd, recent stress testing and process modeling indicate that the East WWTP can treat a 24-hour sustained peak of 26 mgd.

Biological Aerated Filtration

BAF was evaluated as a treatment option in the expansion of the East WWTP. The BAF technology is not well-suited to treating wet weather flows at sustained peak flows. For example, the BAF at the West WWTP is designed for an average flow of 10 mgd and can only treat a sustained flow of 14 mgd. At higher flows, the water velocity in the BAF cells increases and shears the attached growth organisms off the media, causing both an exceedence in effluent TSS and a reduction in dry weather treatment capacity following the event.

The cost to expand the conventional secondary treatment system at the East WWTP was compared to the estimated cost to construct a BAF facility. The estimated construction cost for the secondary treatment facility, including design contingencies, is approximately $22 million. The cost to construct the BAF system at the West WWTP in 2007 was $24 million, which is approximately $28 million with escalation to today's dollars (*Engineering News-Record* Construction Cost Index, 2012 / *Engineering News-Record* Construction Cost Index, 2007). Also, the BAF system produces a lower quality effluent than conventional treatment based on the experience at the WWTP, has higher operating costs, and adds another level of complexity to the WWTP operations. Therefore, BAF was not recommended for further evaluation.

Alternatives

Two secondary treatment technologies were evaluated for the West and East WWTPs—activated sludge and BioCEC. As previously stated, the purpose of the evaluation was to provide budgetary estimates in support of the

IOCP. However, given the constrained site conditions, BioCEC was considered as an alternative to activated sludge because it has a much smaller footprint. Once the IOCP identifies the optimal plant capacity to mitigate overflows in the collection system, the Utility can begin advanced facility planning for the improvements at the WWTPs. The advanced plan should include a review of additional treatment technologies that can be implemented within the budget established in the Facility Plan.

The West WWTP can currently treat a sustained peak flow of 40 mgd under current influent loading conditions. However, additional return activated sludge pumps are required to meet the system requirements when the WWTP is operating at its dry weather design loads.

The East WWTP can currently treat a sustained peak flow of 26 mgd. Expansion of the existing activated sludge system was the only alternative considered for the secondary expansion of the East WWTP to 40 mgd because the WWTP was originally configured to include a fourth aeration basin and clarifier. Additionally, the expanded activated sludge treatment facilities would provide needed redundancy to the existing treatment units, provide higher quality effluent than BioCEC, and allow greater flexibility to meet future water quality requirements. The additional secondary treatment facilities needed to expand the West and East WWTPs are shown in Table 2.

TABLE 2
Additional Secondary Treatment Facilities

	Conventional Improvements				BioCEC		
	Aeration Basin Volume (MG)	**Clarifiers (number/ diameter in feet)**	**Clarifier Surface Area (ft^2)**	**Additional RAS Pumping (mgd)**	**Treatment Basins (MG)**	**Clarifier Surface Area (ft^2)**	**Sand Recirc Pumping (mgd)**
West WWTP							
40 mgd	--	--	--	3	--	--	--
60 mgd	0.63	1/150	17,700	13	0.37	500	3.3
80 mgd	1.04	2/150	35,400	13	0.74	1,000	6.7
East WWTP							
40 mgd	1.40	1/150	17,700	8	NA	NA	NA
60 mgd	2.03	1/150 and 2/123	41,400	18	0.37	500	3.3
80 mgd	2.44	1/150 and 2/141	48,900	18	0.74	1,000	6.7

Notes:
ft^2 = square feet
MG = million gallons

Conventional Treatment

During wet weather events, the influent wastewater is diluted with stormwater. As a result, the aeration basins do not have to be as large as they would need to be to treat typical dry weather flows. For example, each of the existing aeration basins can accommodate 1.15 MG and are sized for an average flow of 7.2 mgd. Based on the results of the process models during wet weather conditions, a new 1.04-MG basin at the West WWTP will have sufficient capacity to treat an additional 40 mgd of flow. Furthermore, the new aeration basins would be configured to operate in plug flow, step feed, or contact stabilization.

A conceptual site plan showing the conventional treatment alternative that would increase the West WWTP capacity to 60 or 80 mgd is shown in Figure 1. The 60-mgd option would have an additional aeration basin and clarifier and the 80-mgd option would have two additional aeration basins and clarifiers. A new headwork facility, which would include screening and grit removal, would convey flow directly into the new aeration basin(s) without primary treatment. The new aeration basins and clarifiers would have the same hydraulic profile as the existing aeration basins and clarifiers. A pipe would be constructed between the existing primary effluent splitter box and the new aeration basins to allow the new aeration basins and clarifiers to be used as spare units during dry weather flow conditions. Also, the pipe exiting the aeration basin would be interconnected with the adjoining aeration basin to allow the new clarifiers to serve as standby units for the existing clarifiers without putting the new aeration basins into service.

A conceptual site plan showing the conventional treatment alternative that would increase the East WWTP capacity to 40, 60 or 80 mgd is shown in Figure 2. The expansion to 40 mgd would include a new aeration basin and clarifier downstream of the primary clarifiers at the locations designated in the original plant design.

Similar to the West WWTP, the alternatives to expand the East WWTP to 60 and 80 mgd would include new aeration basins and clarifiers without primary treatment. The additional treatment units could also be interconnected with the existing treatment units to provide additional redundancy during dry weather conditions.

High Rate Treatment

The HRT system evaluated in this alternative is BioCEC. BioCEC incorporates Actiflo, which is a patented process that uses microsand to increase the settling velocity of the particles in the water and thereby reduce the required size of the clarifiers. The Actiflo process includes a series of basins (coagulation, injection, and maturation) to create a floc particle around a sand nucleus. Once the particle settles to the bottom the clarifier, it is pumped to a hydrocyclone where the sand is separated from the sludge. The sand is injected back into the process and the waste sludge is sent to a downstream treatment process.

A conceptual site plan for the conventional treatment alternative that would increase the West WWTP capacity to 60 or 80 mgd is shown in Figure 3. The BioCEC process includes a small biological contact unit (aeration basin) upstream of the coagulation basin. Return activated sludge is mixed with the influent in the biological contact unit to achieve a mixed liquor suspended solids concentration in the aeration basin of 800 mg/L. The waste sludge from the hydrocyclone is conveyed back to the conventional treatment basins; therefore, the BioCEC system would work in conjunction with the existing activated sludge system to provide the necessary biological treatment.

A conceptual site plan for the conventional treatment alternative that would increase the East WWTP capacity to 60 or 80 mgd is shown in Figure 4. The BioCEC system would have the same treatment units described for the West WWTP. However, the influent and effluent piping configurations would be different for the two WWTP sites.

Alternatives Evaluation

The alternatives were evaluated based on the following consideration:

- Size and space requirements
- Operational features
- Present-worth costs
- Effluent quality

Size and Space Requirements

As shown in Figures 1 through 4, the BioCEC process has a smaller footprint than conventional activated sludge treatment. The smaller footprint is advantageous because of the limited space on the existing WWTP sites for new construction. The BioCEC system has an additional advantage is that it can be constructed with limited interconnections with the existing process units. Therefore, the BioCEC system would have the lowest negative impact from a siting and construction perceptive.

Operational Features

The additional basin volumes provided under the conventional treatment alternatives at both plants could allow the Utility to reconfigure the new and existing basins to provide anoxic swing zones. A swing zone would allow the Utility to operate a portion of the basin in an anoxic condition under normal flow conditions to promote biological nutrient removal through nitrification and denitrification processes. This would not only provide adequate ammonia removal, but also result in considerable power cost savings as a result of the reduced aeration requirements. During high-flow conditions, the swing zone could be converted to aerobic zones to provide CBOD and TSS removal.

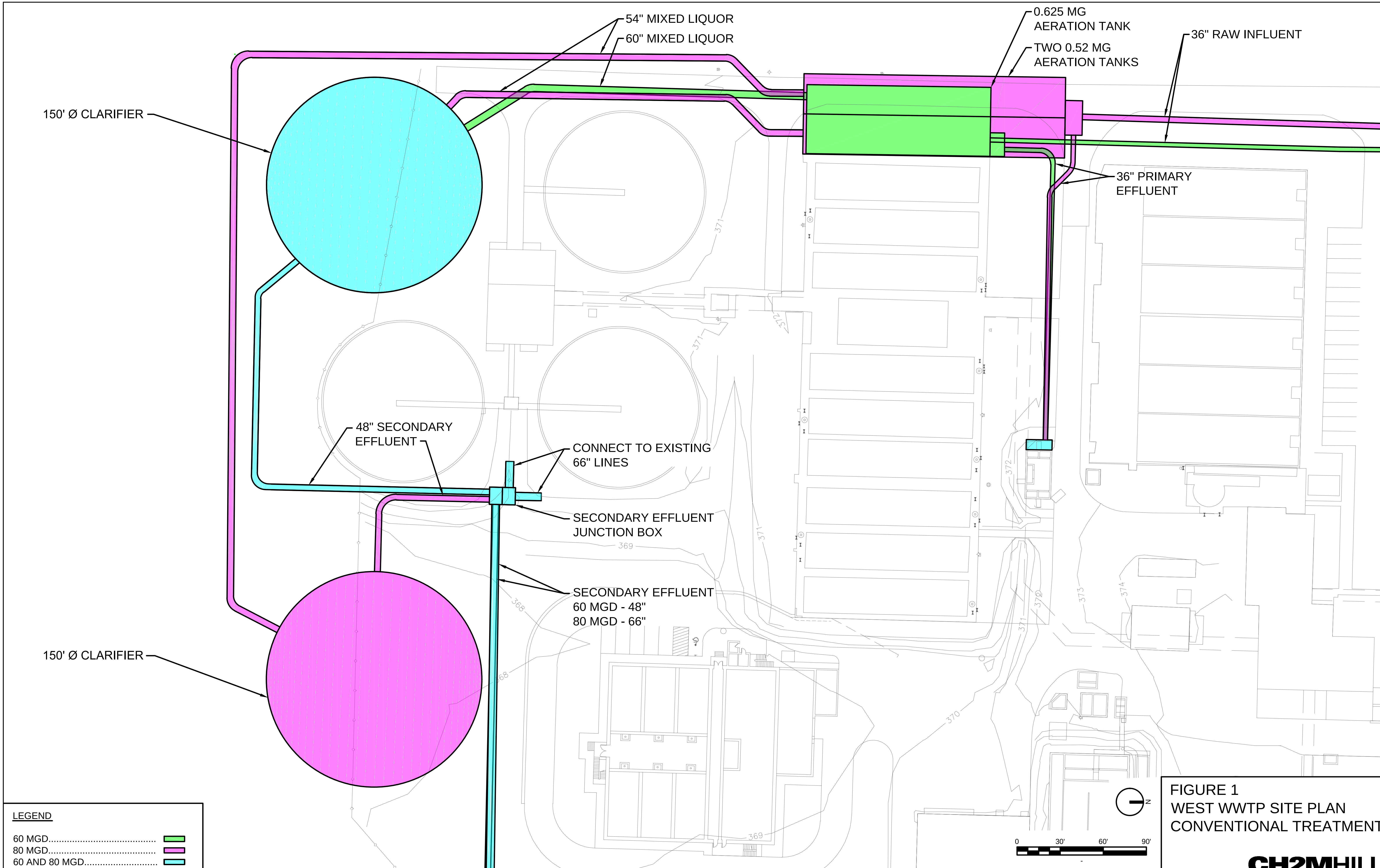


FIGURE 1
WEST WWTP SITE PLAN
CONVENTIONAL TREATMENT

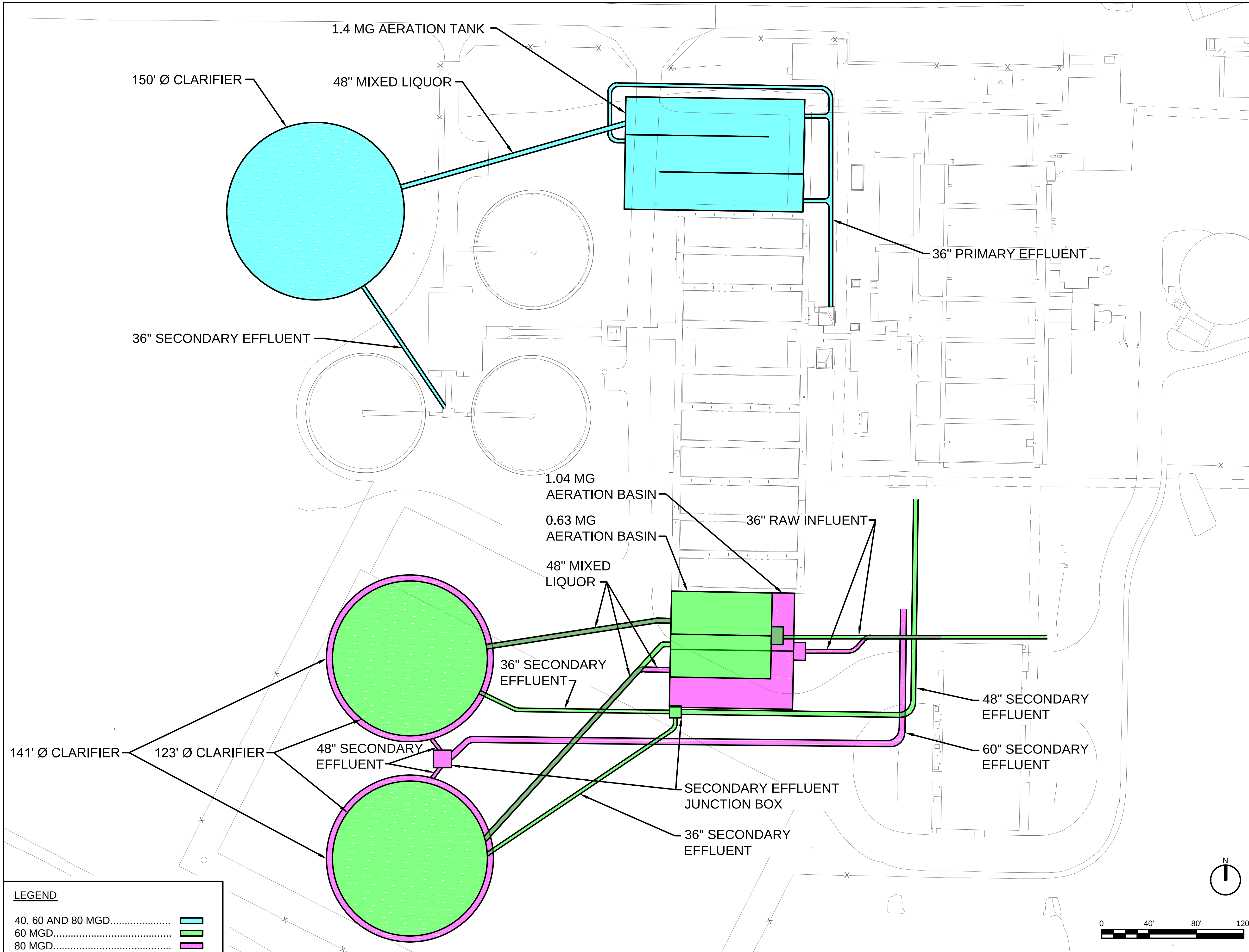
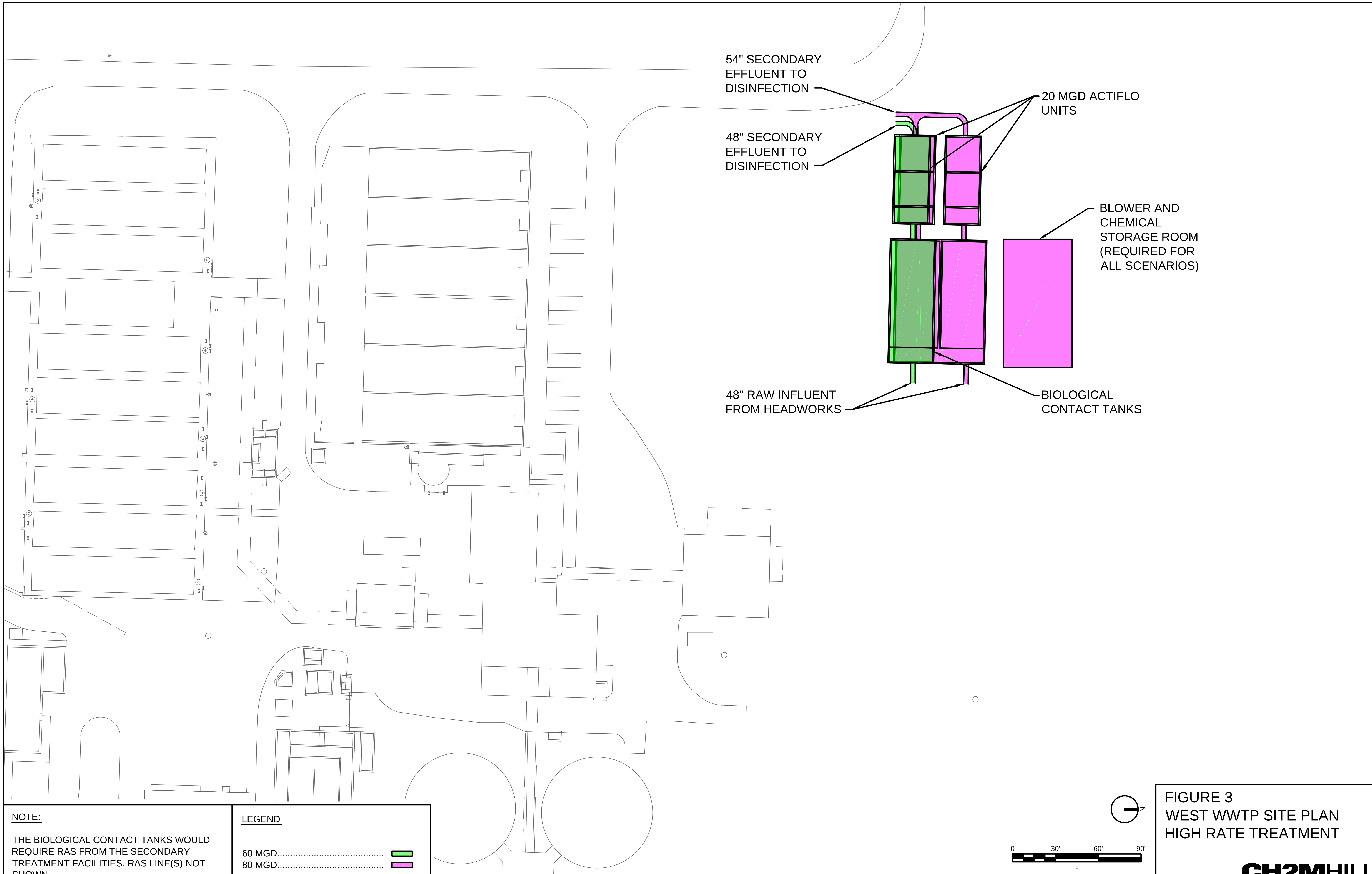


FIGURE 2
EAST WWTP SITE PLAN
CONVENTIONAL TREATMENT

CH2MHILL.



NOTE:

THE BIOLOGICAL CONTACT TANKS WOULD REQUIRE RAS FROM THE SECONDARY TREATMENT FACILITIES. RAS LINE(S) NOT SHOWN.

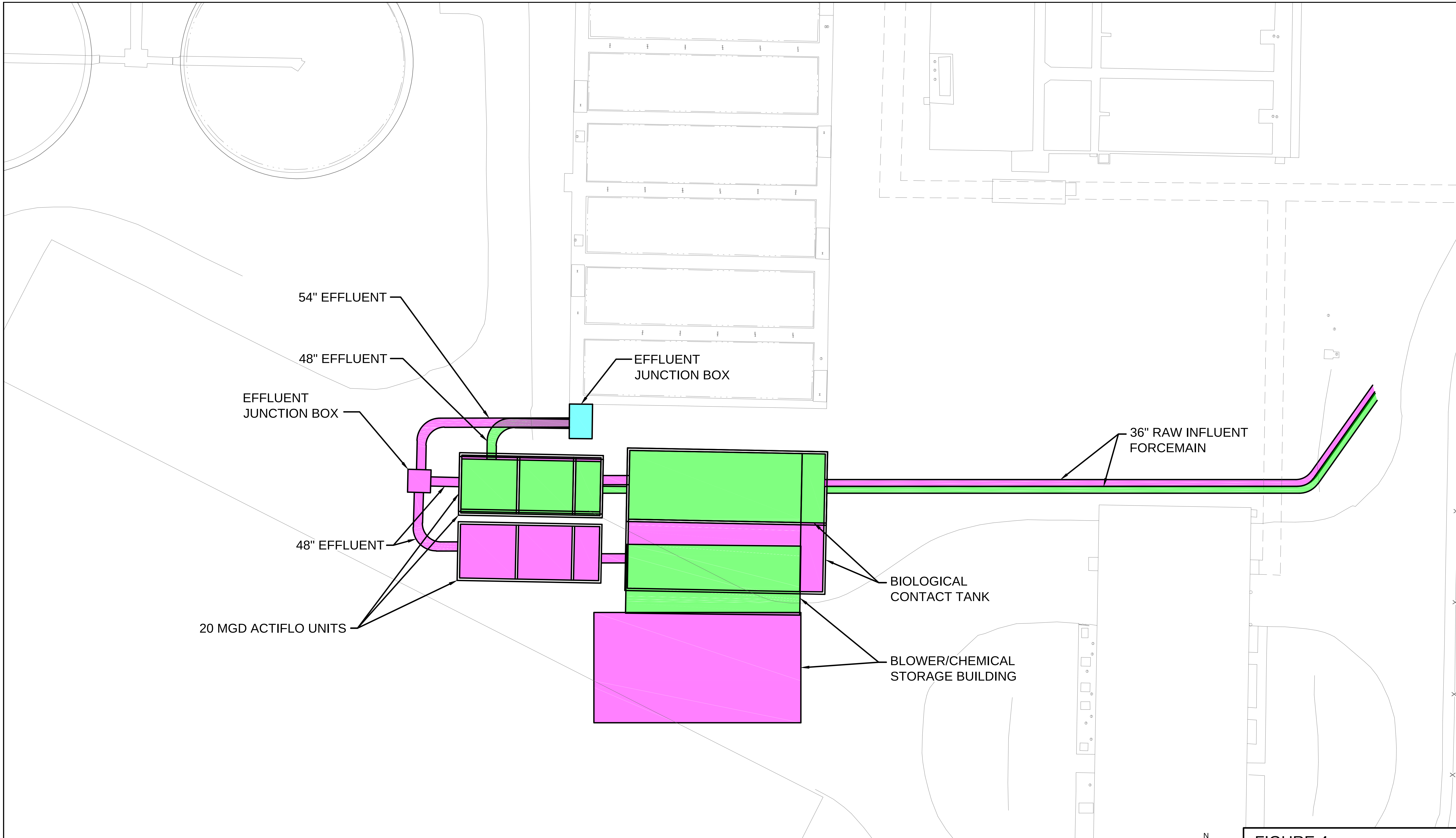
LEGEND

60 MGD..

80 MGD..

FIGURE 3
WEST WWTP SITE PLAN
HIGH RATE TREATMENT

CH2MHILL.



NOTE:

THE BIOLOGICAL CONTACT TANKS WOULD REQUIRE RAS FROM THE SECONDARY TREATMENT FACILITIES. RAS LINE(S) NOT SHOWN.

LEGEND

60 AND 80 MGD............................
60 MGD...
80 MGD...

FIGURE 4
EAST WWTP SITE PLAN
HIGH RATE TREATMENT

CH2MHILL.

New aeration basins and clarifiers would also provide additional redundancy to the existing WWTP. During dry weather conditions, the new basins and clarifiers could operate in parallel with the existing units. During a wet weather event, the isolation gate at the primary effluent splitter box would close and the new headworks would convey wet weather flow directly into the aeration basins.

Although the Utility has operated a conventional treatment system for many years, BioCEC is an unfamiliar treatment process to the Utility. Several feed systems (ferric, polymer, sand) and mechanical equipment (mixers, sand recirculation pumps) make BioCEC a more complex system to operate. Although the BioCEC system could be automated to some extent, it would most likely require significant operator attention during startup of the facility. Therefore, conventional treatment would have the lowest negative impact from an operational perspective.

Present-worth Costs

The present-worth costs were calculated using the cost guidance prepared for the IOCP. These costs are a combination of capital and operation and maintenance costs, as shown in Table 3. As previously stated, only conventional treatment was considered to increase the WWTP capacity to 40 mgd. The BioCEC alternative has significantly lower capital and present-worth costs, even though the operational costs are much higher. The higher operational costs are associated with the additional staffing, power, and chemicals required to operate the BioCEC system.

The estimated costs for the conventional treatment plant are greatly affected by the poor soil conditions experienced at the WWTPs on past projects. Due to requirements for piles, over-excavation and backfill, the conventional treatment alternative has much higher construction costs than the HRT alternative with its smaller footprint.

TABLE 3
Present-worth Costs

	Conventional Treatment			**High-rate Treatment**		
	Total Capital	**40-year O&M**	**Present Worth**	**Total Capital**	**40-year O&M**	**Present Worth**
West WWTP						
40 mgd	740,000	244,000	984,000	--	--	--
60 mgd	29,722,000	2,058,000	31,780,000	20,272,000	8,244,000	28,516,000
80 mgd	49,559,000	6,334,000	55,893,000	31,359,000	9,844,000	41,203,000
East WWTP						
40 mgd	31,334,000	1,666,000	33,000,000	--	--	--
60 mgd	66,203,000	3,301,000	69,504,000	49,980,000	10,719,000	61,742,000
80 mgd	74,051,000	3,360,000	77,411,000	58,830,000	12,319,000	72,220,000

Effluent Quality

Although the Actiflo process has an established performance record in the water treatment industry, its application for activated sludge, BioCEC, is new and has never been permitted in Indiana. The BioCEC process operates at a lower mixed liquor suspended solids concentration than activated sludge and has a lower detention time, resulting in a lower-quality effluent than conventional treatment. Additionally, this system requires the continuous operation of mechanical mixers, pumps, and chemical feed systems. Failure of any of these components, or lack of operator attention, could result in an effluent permit violation.

Conventional treatment has a proven history of meeting the effluent requirements during dry and wet weather conditions. Expansion of these facilities will provide even greater reliability during normal flow conditions and the opportunity to improve effluent quality by providing additional basin volume for biological nutrient removal. Therefore, conventional treatment poses the least risk of non-compliance with effluent quality requirements

Summary

The conventional treatment alternative offers the following advantages to the Utility:

- Conventional treatment provides the best quality effluent, operational reliability and flexibility.
- The conventional treatment facilities could be used year round to provide added redundancy to existing process units.
- The conventional treatment facilities could be configured to provide biological nutrient removal to meet the anticipated phosphorus requirement of 1 mg/L.
- The conventional treatment will require fewer operators and will use a process familiar to the staff.

Therefore, for the purpose of Facility Plan budgeting, it is assumed that the Utility would use the conventional treatment alternative. The conventional treatment alternative, as described herein, provides a representative budgetary cost estimate that can be used for the IOCP development. During preliminary design, the influent flows and loadings should be validated, and opportunities for optimizing the treatment plant configuration should be performed.

APPENDIX C

Disinfection Technical Memorandum

West and East Wastewater Treatment Plants Disinfection Expansion Alternatives

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Introduction

The Evansville Water and Sewer Utility (Utility) is currently developing a Facility Plan for the West and East Wastewater Treatment Plants (WWTPs). The Facility Plan will provide planning level cost estimates to increase the wet weather treatment capacities of the West and East WWTPs. The increased WWTP capacities are needed to mitigate sanitary and combined sewer overflows in the collection system.

The purpose of this technical memorandum (TM) is to provide capital and life-cycle cost estimates for the disinfection facilities, which will be combined with other WWTP improvements to support the development of the Facility Plan and Integrated Overflow Control Plan (IOCP). This TM also describes the evaluation of the economic and non-economic factors for two different disinfection technologies: ultraviolet (UV) and sodium hypochlorite.

Effluent Limits

The Indiana Department of Environmental Management has issued National Pollutant Discharge Elimination System (NPDES) permits for the West and East WWTPs. The NPDES permits establish effluent concentration limits for carbonaceous biochemical oxygen demand, total suspended solids, ammonia, fecal coliform, pH, total residual chlorine and E. coli. The parameters that are controlled by the disinfection system are shown in Table 1.

TABLE 1
West and East WWTP Effluent Limits[a]

	Quantity or Loading		
Parameter	**Monthly Average**	**Daily Maximum**	**Units**
Fecal Coliform	2,000	---	count/100 ml
E. coli	125	235	count/100 ml

[a] Fecal coliform and E. coli limits set by the NPDES permits are the same for the West and East WWTPs.
Note:

ml = milliliters

Existing Facilities

The West WWTP is designed for an average flow of 21.7 million gallons per day (mgd). The WWTP has both conventional activated sludge and biological aerated filtration (BAF) for secondary treatment, and these processes are operated in parallel. The treatment units at the WWTP consist of fine bar screens, grit removal, six primary clarifiers, three aeration basins, three final clarifiers, BAF (six cells), and effluent chlorination using gaseous chlorine. The final effluent is dechlorinated prior to discharge. Even though the NPDES permit states that the peak sustained wet-weather flow is 30.6 mgd, recent stress testing and process modeling indicate that the West WWTP can treat a 24-hour sustained peak of 40 mgd (26 mgd conventional treatment and 14 mgd BAF).

The East WWTP is designed for an average flow of 18 mgd. The treatment facility consists of fine bar screens, a vortex grit system, seven primary clarifiers, three aeration basins, three final clarifiers, and effluent chlorination and dechlorination. Even though the NPDES permit states that the peak sustained wet-weather flow is 22.5 mgd, recent stress testing and process modeling indicate that the East WWTP can treat a 24-hour sustained peak of 26 mgd with 15 minutes of contact time during peak flows.

Preliminary hydraulic modeling indicated that the two older chlorine contact tanks at the East WWTP do not have sufficient hydraulic capacity without submerging the secondary clarifier effluent weirs. The two older contact tanks were designed prior to the activated sludge system and are approximately 6 feet higher than the new chlorine contact tank (which was designed along with the activated sludge process). To disinfect secondary effluent with the older contact tanks, the contact tank weir must be set sufficiently lower than secondary clarifier weirs, resulting in a shallow contact tank side-water depth of roughly 4 feet. This configuration would have a corresponding capacity of approximately 4.6 mgd with 15 minutes of contact time. Therefore, due to the hydraulic restrictions associated with the two old contact tanks, they were not considered in this evaluation unless they are used solely for the purpose of disinfecting primary effluent.

Alternatives Development

Two alternative disinfection technologies were evaluated for the West and East WWTPs:

- Alternative 1 – Disinfect all flow using liquid sodium hypochlorite.
- Alternative 2 – Disinfect flows up to 40 mgd with ultraviolet disinfection, and disinfect wet-weather flows greater than 40 mgd with sodium hypochlorite.

Alternative 1 – Sodium Hypochlorite

The requirements to convert the West WWTP disinfection system from the existing gaseous chlorine to sodium hypochlorite are shown in Table 2.

TABLE 2
West WWTP Sodium Hypochlorite Facilities

	40-mgd Capacity	60-mgd Capacity	80-mgd Capacity
Additional Contact Tank Required	None	One new 209,000-gallon tank with effluent piping to existing effluent pump station	One new 417,000-gallon tank with effluent piping to new and existing effluent pump stations
Chemical Feed and Storage Required	New Chemical Feed Building Three 7,500-gallon FRP storage tanks	New Chemical Feed Building Three 10,000-gallon FRP storage tanks	New Chemical Feed Building Four 10,000-gallon FRP storage tanks
Other	None	None	Install a new chlorinated effluent diversion structure

Note:

FRP = fiberglass-reinforced plastic

A conceptual site plan for the sodium hypochlorite alternative that would provide 40, 60, or 80 mgd disinfection capacity at the West WWTP is shown in Figure 1. No modifications of the existing contact tank are required for the 40-mgd scenario. However, hydraulic modeling indicated that neither the existing contact tank nor the upstream infrastructure have hydraulic capacity for an additional 20 or 40 mgd. Therefore, a new 209,000-gallon contact tank would be required to expand to 60 mgd. The new contact tank would be hydraulically separate from the existing tank, and would be a dedicated for wet-weather flows. Final effluent would be conveyed to existing effluent pumping facilities.

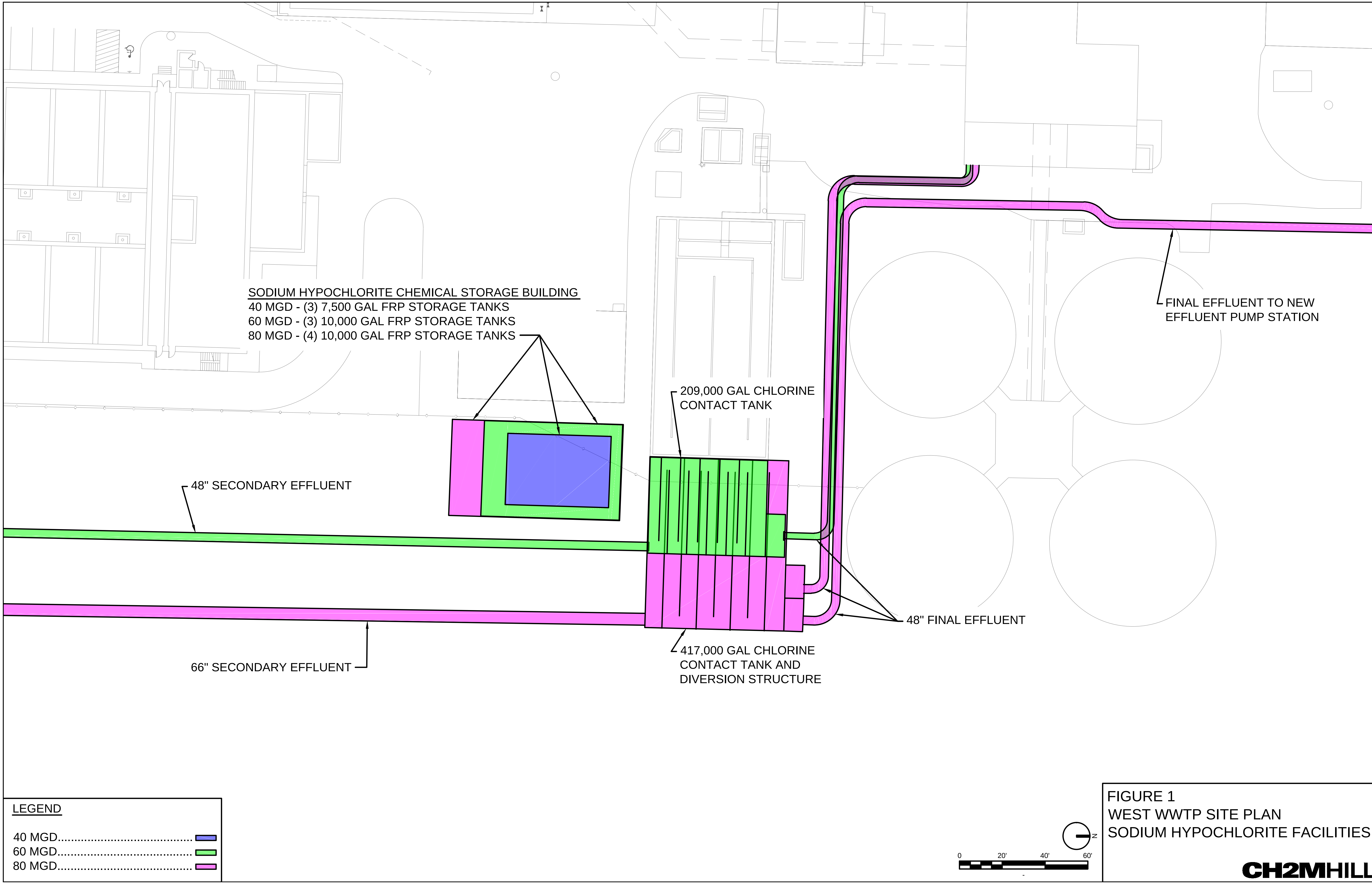

FIGURE 1
WEST WWTP SITE PLAN
SODIUM HYPOCHLORITE FACILITIES

The 80-mgd option would be very similar to the expansion proposed for 60 mgd. The same modification to the existing contact tank is required. In addition to this, a new 417,000-gallon contact tank would be required and would be similar to the tank described for the 60-mgd option. Additionally, a new final effluent diversion structure would be required because the existing effluent pump station only has a capacity of only 60 mgd. The diversion structure would route wet-weather flow over 20 mgd to a new wet weather effluent pump station. Flows less than 20 mgd would be conveyed to the existing effluent pumping facilities. Each of the three expansion options would require new chemical storage facilities.

The requirements to convert the East WWTP disinfection system from the existing gaseous chlorine to sodium hypochlorite are shown in Table 3.

TABLE 3
East WWTP Sodium Hypochlorite Facilities

	40-mgd Capacity with 14-mgd PE Bypass	40-mgd Capacity	60-mgd Capacity	80-mgd Capacity
Additional Contact Tank Required	None	One 176,000-gallon contact tank extension; modify or upsize the existing disinfected effluent line	One 372,000-gallon contact tank extension; upsize the existing disinfected effluent line	One 372,000-gallon contact tank extension; upsize the existing disinfected effluent line One 150,000-gallon contact tank; construct a new effluent line
Chemical Feed and Storage Required	One Chemical Feed Building with three 7,500-gallon FRP storage tanks	One Chemical Feed Building with three 7,500-gallon FRP storage tanks	One Chemical Feed Building with three 10,000-gallon FRP storage tanks	One Chemical Feed Building with four 10,000-gallon FRP storage tanks
Existing Contact Tank Modifications Required	Rehab contact tank walls Modify contact tank effluent weir Replace existing 48-inch sluice gate with automated gate and level sensor	Lower the existing contact tank weir approximately 0.2 foot Add a second 48-inch sluice gate to the existing influent channel	Lower the existing contact tank weir approximately 0.2 foot Remove the 48-inch sluice gate in the existing influent channel and install two 60-inch gates	Lower the existing contact tank weir approximately 0.2 foot Remove the 48-inch sluice gate in the existing influent channel and install two 60-inch gates
Other	Seal existing final effluent MH	Construct a new diversion structure to allow the existing tank to be bypassed	Construct a new diversion structure to allow the existing tank to be bypassed	Construct new diversion structure to allow the existing tank to be bypassed Construct a new diversion structure to convey 20 mgd to the 150,000-gallon contact tank Construct a new junction box on the north side of the East WWTP to combine the chlorinated effluent streams

A conceptual site plan for the sodium hypochlorite alternative that would provide 40, 60, or 80 mgd disinfection capacity at the East WWTP is shown in Figure 2. Expanding the East WWTP's disinfection capacity to 40 mgd could involve two different scenarios. If the existing secondary treatment system is not expanded, then primary effluent could be diverted to the two existing, older contact tanks, providing approximately 14 mgd of disinfected primary effluent. These two tanks would operate in series and would be hydraulically separate from the southern tank (which would continue to disinfect approximately 26 mgd of secondary effluent). This expansion would involve rehabilitating the existing chlorine contact tanks, and replacing the effluent weir. It would also involve replacing the existing 48-inch primary effluent bypass sluice gate with an automated gate and level sensor, sealing of the existing final effluent man hole. Final effluent would be conveyed by the WWTP's original 36-inch final effluent line.

If secondary treatment is expanded to 40 mgd, then disinfection would be expanded by constructing a new 176,000-gallon chlorine contact tank that would be operated in series with the existing tank to achieve a cumulative 15 minutes of contact time during peak flows. A new diversion structure would also be constructed to allow bypass of the existing tank when needed; flow would be diverted around to the existing contact tank effluent channel. A number of minor modifications to the existing tank would also be required to accommodate hydraulics. The 60- and 80-mgd options would be very similar to the 40-mgd option. The major difference is larger contact tanks are necessary to reach 15 minutes of contact time. Due to site and hydraulic constraints, the 80-mgd option would require two new contact tanks. This additional 150,000-gallon tank would disinfect 20 mgd in parallel with the 60-mgd train proposed in the 60-mgd option. Final effluent would recombine at a downstream junction box. All expansion options would require new chemical storage facilities.

Alternative 2 – Ultraviolet and Sodium Hypochlorite Disinfection

The requirements to convert the West WWTP disinfection system from gaseous chlorine to a combination of UV and sodium hypochlorite are shown in Table 4.

TABLE 4
West WWTP Ultraviolet and Sodium Hypochlorite Disinfection Facilities

	40-mgd Capacity	60-mgd Capacity	80-mgd Capacity
UV System	Modify the existing chlorine contact tank to accommodate a 40-mgd peak-flow UV system	Modify the existing chlorine contact tank to accommodate a 40-mgd pea- flow UV system	Modify the existing chlorine contact tank to accommodate a 40-mgd peak-flow UV system
Additional Contact Tank Required	None	Construct a new 209,000-gallon tank with effluent piping to the existing effluent pump station	Construct a new 417,000-gallon tank with effluent piping to the new and existing effluent pump stations
Chemical Feed and Storage Required	None	Construct new Chemical Feed Building with two 5,000-gallon FRP storage tanks	Construct new Chemical Feed Building with three 7,500-gallon FRP storage tanks
Existing Contact Tank Modifications Required	Modify the influent channel and effluent weir to accommodate hydraulics	Modify the influent channel and effluent weir to accommodate hydraulics	Modify the influent channel and effluent weir to accommodate hydraulics
Other	None	Construct a new secondary effluent diversion structure	Construct a new secondary effluent diversion structure Construct a new chlorinated effluent diversion structure

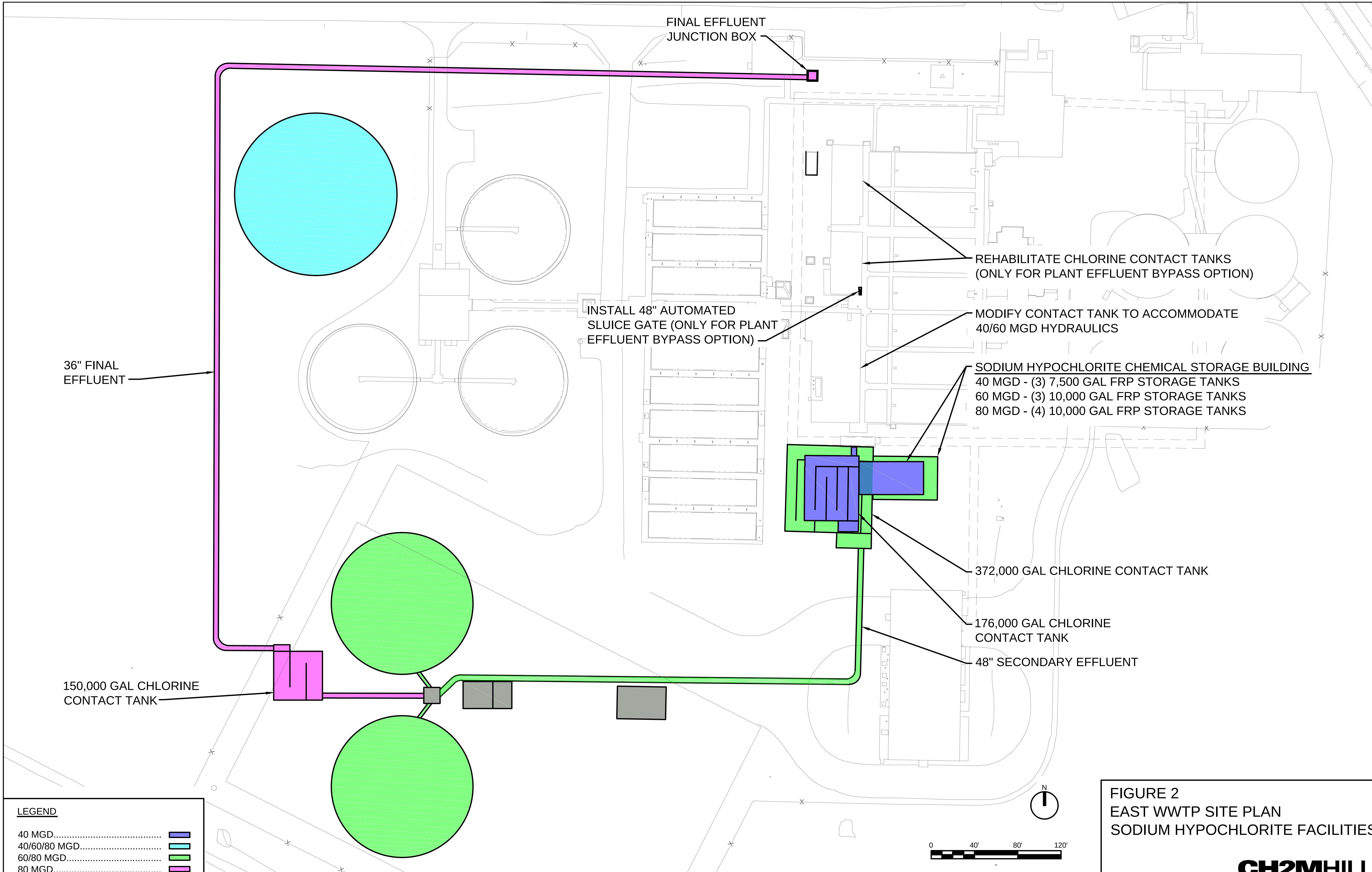

FIGURE 2
EAST WWTP SITE PLAN
SODIUM HYPOCHLORITE FACILITIES

A conceptual site plan for the UV alternative that would provide 40, 60, or 80 mgd disinfection capacity at the West WWTP is shown in Figure 3. For the 40 mgd option, this alternative would construct a 40-mgd UV system within the existing contact tank. Disinfected effluent would be conveyed to the existing effluent pumping facility. The 60- and 80-mgd options would involve the same UV system configuration and effluent line, but would also involve constructing an additional secondary effluent diversion structure and liquid sodium hypochlorite facility. The new diversion structure would route 40 mgd to the UV system and the remaining flow would be routed to the chlorine contact tank. The 60- and 80-mgd options would require 209,000- and 417,000-gallon contact tanks, respectively, along with a chemical storage building. In addition, the 80-mgd option would require a final effluent diversion structure because the exiting effluent pumping facilities only have capacity for 60 mgd. This new diversion structure would convey flow greater than 20 mgd to new effluent pumping facilities.

The requirements to convert the East WWTP disinfection system from gaseous chlorine to a combination of UV and sodium hypochlorite are shown in Table 5.

TABLE 5
East WWTP Ultraviolet and Sodium Hypochlorite Disinfection Facilities

	40-mgd Capacity	60-mgd Capacity	80-mgd Capacity
UV System	Modify the existing chlorine contact tank to accommodate 40-mgd peak-flow UV system; upsize the existing effluent piping	Construct a new 40-mgd peak-flow UV system; install new effluent piping to new junction structure	Construct a new 40-mgd peak-flow UV system; install new effluent piping to new junction structure
Additional Contact Tank Required	None	None	One new 176,000-gallon tank
Chemical Feed and Storage Required	None	New Chemical Feed Building with two 5,000-gallon FRP storage tanks	New Chemical Feed Building with three 7,500-gallon FRP storage tanks
Existing Contact Tank Modifications Required	Modify the influent channel and effluent weir to accommodate hydraulics	None	Lower the existing contact tank weir approximately 0.2 foot Add a second 48-inch sluice gate to the existing influent channel
Other		Construct a new UV diversion structure (approximately 10 feet by 10 feet) with gates	Upsize the existing effluent piping Construct a new UV diversion structure (approximately 10 feet by 10 feet) with gates Construct a new chlorinated effluent diversion structure to bypass existing contact tank

A conceptual site plan for the UV alternative that would provide 40, 60, or 80 mgd disinfection capacity at the East WWTP is shown in Figure 4. For the 40 mgd option, this alternative would construct a new 40-mgd UV unit inside the existing chlorine contact tank and increase the size of the existing final effluent line. The 60- and 80-mgd options would construct a stand-alone UV system, new effluent line, and use the existing chlorine contact tank as needed to obtain 15 minutes of contact time. The 60-mgd option would not require an additional chlorine contact tank. However, the 80-mgd option would require an additional 176,000-gallon contact tank. Therefore, a new tank would be constructed and would operate in series with the existing contact tank. A new chlorinated effluent diversion structure would be constructed for the 80-mgd option to bypass flows around the existing contact tank when needed. The 80-mgd option would also require minor modifications to the existing contact tank. Final effluent would be conveyed around the WWTP and to a junction box on the north side of the WWTP.

Alternatives Evaluation

The alternatives were evaluated based on the following parameters:

- Present-worth costs
- Effluent water quality
- Operational considerations
- Environmental considerations

Present-worth Costs

The present-worth costs were calculated using the cost guidance prepared for the IOCP, and are a combination of capital and operation and maintenance (O&M) costs, as shown in Table 6 and Table 7.

At the West WWTP, Alternative 1 (Sodium Hypochlorite) has significantly lower capital and total present worth costs than Alternative 2 (UV+ Sodium Hypochlorite) for the 40-mgd scenario, even though the operational costs are higher. The higher operational costs are associated with the chemicals that are required for chlorination/dechlorination. However, the present worth cost differential between Alternative 1 and 2 decreases as the treatment capacity is expanded to 80 mgd.

At the East WWTP, Alternative 2 (UV and Sodium Hypochlorite) has much lower capital, operational and present worth costs for the 40-mgd scenario with full secondary treatment. However, the capital cost is much lower for Alternative 1, the primary effluent bypass scenario. For the 60- and 80-mgd scenarios, Alternatives 1 and 2 have very similar present worth costs but Alternative 2 has significantly higher capital costs than Alternative 1.

TABLE 6
West WWTP Costs

Scenario	Alternative	Total Capital Cost ($)	40-Year O&M ($)	Present-Worth Cost ($)
40 mgd	1	1,390,000	3,337,000	4,727,000
	2	4,450,000	1,902,000	6,352,000
60 mgd	1	4,890,000	4,084,000	8,974,000
	2	6,860,000	2,584,000	9,444,000
80 mgd	1	8,020,000	4,359,000	12,379,000
	2	7,860,000	2,903,000	10,733,000

TABLE 7
East WWTP Costs

Scenario	Alternative	Total Capital Cost ($)	40 Year O&M ($)	Present-Worth Cost ($)
40 mgd	1 – PE Bypass	2,160,000	2,719,000	4,879,000
	1	4,960,000	2,719,000	7,679,000
	2	3,890,000	1,778,000	5,668,000
60 mgd	1	7,080,000	2,860,000	9,940,000
	2	9,420,000	1,946,000	11,336,000
80 mgd	1	9,720,000	2,090,000	12,620,000
	2	12,960,000	1,951,000	14,911,000

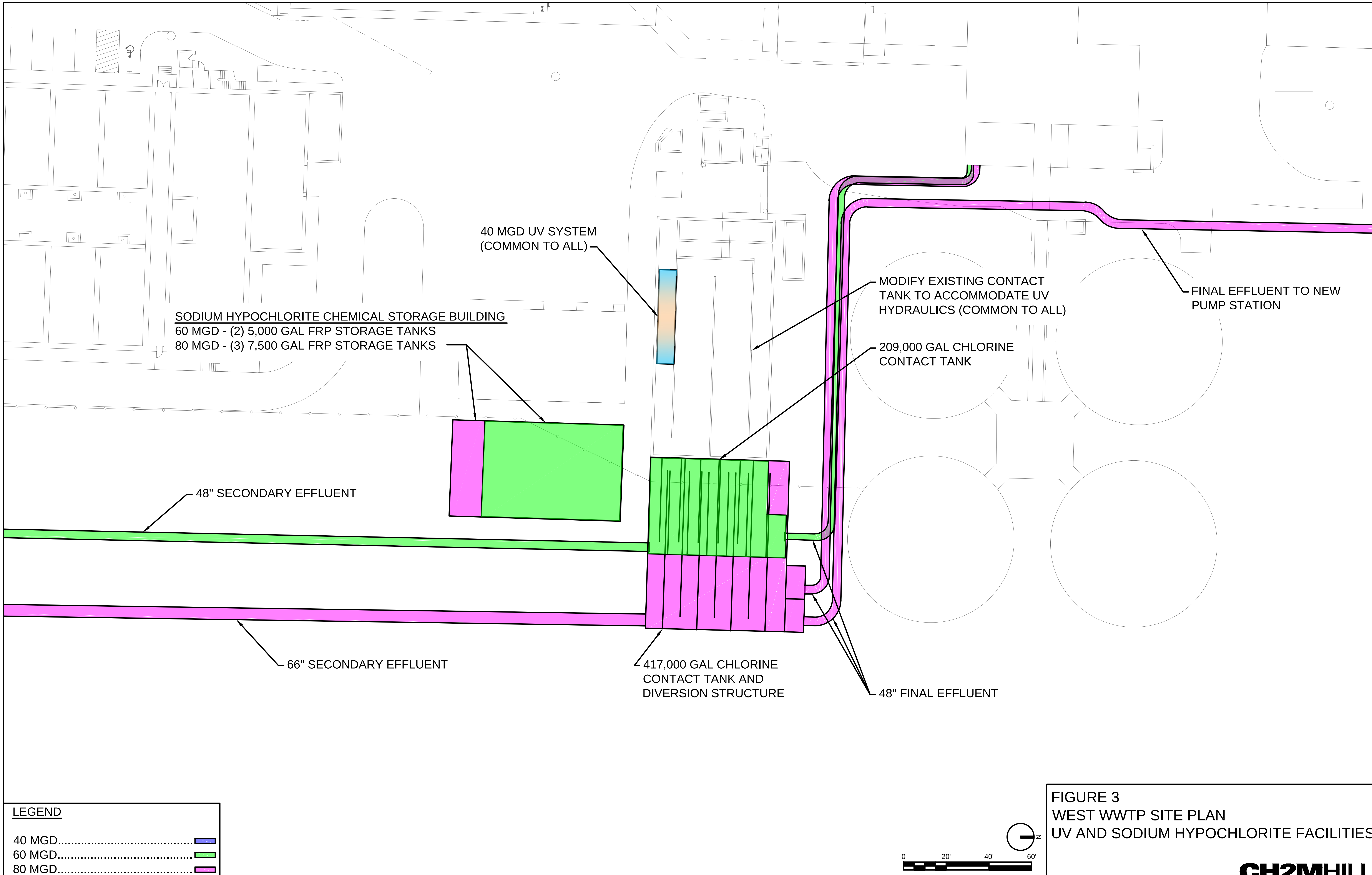

FIGURE 3
WEST WWTP SITE PLAN
UV AND SODIUM HYPOCHLORITE FACILITIES

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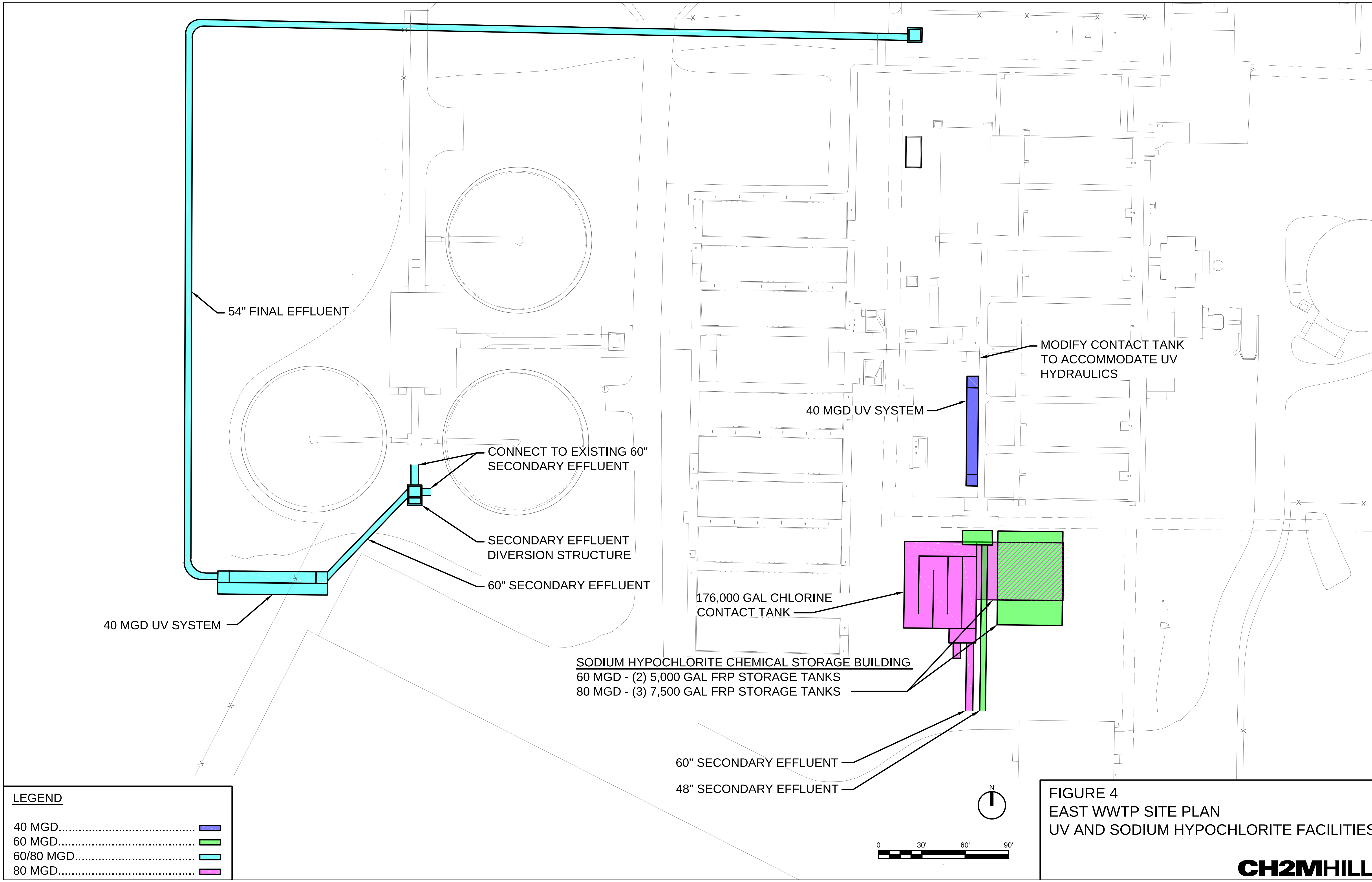

FIGURE 4
EAST WWTP SITE PLAN
UV AND SODIUM HYPOCHLORITE FACILITIES

Effluent Water Quality

Both alternatives are capable of meeting current and proposed regulations. Sodium hypochlorite is a proven technology and is very similar to the current practice of using gaseous chlorine. UV has the added benefit of meeting existing microbial standards without dechlorination. However, its performance is highly dependent on maintaining the effluent total suspended solids less than 30 mg/L. WWTP upsets can decrease its effectiveness. Therefore, sodium hypochlorite poses the least risk to non-compliance with effluent quality requirements

Operational Considerations

A sodium hypochlorite system has fewer components and maintenance requirements than a UV system. In addition, purchased hypochlorite eliminates most of the risk to operators compared to gaseous chlorine, but there is some risk to employees if proper protection is not worn when handling the 15% hypochlorite solution.

A UV is highly automated and would require little operator intervention or maintenance. UV would also eliminate most of the risk associated with chemical feed systems. Therefore, UV poses the least risk from an operational perspective.

Environmental Considerations

Sodium hypochlorite would require weekly truck traffic through nearby neighborhoods. While, a liquid feed system would have a much lower security risk compared to gaseous chlorine, an unintentional spill does present some risk of localized exposure to a corrosive chemical.

Unlike sodium hypochlorite, UV would require minimal additional truck traffic, and does not use large volumes chemicals that could be released. In addition, the facilities are generally compact, which is a benefit at both WWTPs due to significant site constraints. Therefore, UV poses the least risk from an environmental perspective.

Summary

At the West WWTP, the capital and present worth costs for the sodium hypochlorite alternative are significantly lower than the costs associated with the UV alternative in the 40-mgd scenario. However, for the 60- and 80-mgd scenarios the present worth costs are so similar that non-cost factors have a greater influence on the final technology selection. Therefore, for the purpose of Facility Plan budgeting, it was assumed that the Utility would use the sodium hypochlorite alternative if expanding to 40 mgd, but would use the UV alternative if expanding to 60- or 80-mgd.

At the East WWTP, the capital and present worth costs for the UV alternative are significantly lower than the costs associated with the sodium hypochlorite alternative for the 40-mgd scenario, if bypassing secondary treatment is not considered. If the primary effluent bypass is implemented, rehabilitation of the existing chlorine contact tanks has the lowest capital cost. For the 60- and 80-mgd scenarios, the present worth costs of the alternatives are so similar that non-cost factors have a greater influence on the final technology selection. Therefore, for the purpose of Facility Plan budgeting, it was assumed that the Utility would use the sodium hypochlorite alternative if the primary effluent bypass was implemented, but would use the UV alternative if the primary effluent bypass was not implemented.

APPENDIX D

Solids-handling Technical Memorandum

West and East Wastewater Treatment Plants Solids Handing Evaluation

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Introduction

The Evansville Water and Sewer Utility (Utility) is currently developing a Wet Weather Facility Plan for the West and East Wastewater Treatment Plants (WWTPs). The purpose of this technical memorandum (TM) is to provide planning-level cost estimates to increase the wet weather treatment capacities of the West and East WWTPs. The increased WWTP capacities are needed to mitigate sanitary sewer overflows (SSOs) and combined sewer overflows (CSOs) in the collection system.

The Facility Plan evaluates the influent solids loading to the WWTPs, including the effects of future growth over a 20-year planning horizon and capture and treatment of SSOs and CSOs. During the facility planning process, it became evident that the existing facilities should have sufficient capacity to handle the increased solids loading from SSO and CSO abatement projects. Although the projected solids loading is marginally higher than the current capacity of the existing digesters, there is still significant available capacity now and a likelihood that the digesters will not need to be expanded during the 20-year planning horizon. Therefore, the alternatives evaluate the cost implications with and without expanded digester facilities.

Existing Facilities

The solids handling process equipment and design capacities of the West and East WWTPs are summarized in Tables 1 and 2, respectively. Digested primary and waste activated sludge (WAS) from the West WWTP is pumped approximately five miles to the East WWTP for ultimate dewatering and disposal. The East WWTP also has anaerobic digesters and uses belt presses to dewater the combined solids from both WWTPs. The East WWTP has a covered staging area to stockpile the dewatered sludge for land application; however, the Utility currently disposes the dewatered solids at a landfill and does not plan to land apply WWTP biosolids in the future. Therefore, the area that is currently designated for the stockpiled biosolids will be available for expansion of the liquid treatment processes described in the Facility Plan.

The anaerobic digesters produce methane gas, which is primarily used as a fuel source for the boilers that heat the digesters. The West WWTP also uses digester gas as a supplemental fuel for the natural gas-driven influent pumps. The Utility currently plans to install a cogeneration system at the East WWTP to generate power onsite with the excess digester gas.

The Utility performed a condition assessment of the existing solids handling facilities. The digesters at the West WWTP are in poor condition and need to be rehabilitated. One belt press at the East WWTP will be replaced with a centrifuge as part of an energy savings performance contract. Also, the sludge transfer line between the West and East WWTPs is in poor condition and needs to be replaced.

TABLE 1
West WWTP Solids Handling Process and Design Capacities Summary

Unit Process	Type	Quantity	Size	Design Capacity
Thickening	Gravity belt thickener	2	2-meter units	20.2+ mgd [a]
				635 lbs dry solids/meter/hour minimum, hydraulic loading 100 to 500 gpm
Digestion	Primary: circular anaerobic	3	75-foot-diameter, 28.5-foot depth operated to 26-foot SWD	Average monthly loading: 29 lbs VSS/1,000 ft^3/day [b]
			Volume: 922,000 gallons each	HRT: 28 days[b] average
	Secondary: circular anaerobic	1	75-foot-diameter, 28.5-foot depth operated to 26-foot SWD	---
			Volume: 922,000 gallons each	
Sludge Transfer Pumps (to East WWTP)	Centrifugal vortex impeller	2	40 horsepower	300 gpm

[a] 7-hour shift, 5 days per week; maximum 96 million gallons per day (mgd).
[b] Average based on data from January 2010 through September 2011.

Notes:
ft^3 = cubic feet
HRT = hydraulic residence time
gpm = gallons per minute
lbs = pounds
SWD = side water depth
VSS = volatile suspended solids

TABLE 2
East WWTP Solids Handling Process and Design Capacities Summary

Unit Process	Type	Quantity	Size	Design Capacity
Thickening	Gravity belt thickener	2	2-meter units	635 lbs/meter/hour minimum
Digestion	Primary: circular anaerobic	3	75–foot-diameter, 25-foot depth, operated to 23 feet SWD	Average monthly loading: 54 lbs VSS/1,000 ft^3 [c,d]
			Volume: 826,000 gallons each	HRT: 32 days average [a,c]
	Secondary: circular anaerobic	1	75-foot-diameter, 25-foot depth, operated to 23 feet SWD	---
			Volume: 826,000 gallons	
Dewatering	Belt filter press	4	2-meter units	10.7 mgd[b]
Polymer System	Dry polymer	1	---	---
Storage	Covered storage pad	1	100,000 ft^2	13.3 mgd
				Truck to landfill: 4 trucks/day, 3 days each week

[a] At average flow of 18 mgd, the detention time is 29 days. Sludge is stored for further drying and land filling.
[b] 7-hour shift, 5 days per week – maximum 51 mgd.
[c] Average based on data from January 2010 through October 2011.
[d] Includes 45 lbs VSS + 9 lbs VSS expected from current fat, oil, and grease program.

Note:
N/A = not available

Solids Loading

Process models were used to estimate the primary, waste activated, and anaerobic digested sludge production, as summarized in Table 3. The annual average sludge production, which will not significantly vary for the different flow capacities, is used to estimate the operation and maintenance costs. The peak week sludge production is used to size the gravity thickeners. The peak month sludge production is used to size the anaerobic digesters and dewatering equipment. All of the alternatives provide sufficient onsite storage capacity to allow the dewatering equipment to be sized for peak month solids loadings, rather than peak week solids loading.

TABLE 3
Sludge Production

	Units	West WWTP	East WWTP
Primary Sludge			
Average	lb/day	12,100	10,700
Peak Week	lb/day	45,700	36,200
Peak Month	lb/day	25,700	27,000
Waste Activated Sludge			
Average	lb/day	5,400	11,800
Peak Week	lb/day	16,400	29,900
Peak Month	lb/day	9,400	17,600
Thickened Sludge to Digesters			
Average	lb/day	17,300	21,900
Peak Month	lb/day	32,100	41,100
Anaerobic Digested Sludge			
Average	lb/day	10,300	13,100
Peak Month	lb/day	18,900	25,900

Alternatives Analysis

This TM provides a cursory review of solids handling options for the Utility and considers three alternatives to provide a range of costs for planning. The first alternative would eliminate the digestion process at both WWTPs. Each WWTP would have onsite facilities to dewater undigested sludge, which would be hauled to the landfill. In many instances, this approach is the least costly and easiest to operate. However, it would not allow beneficial use of the sludge either as a source for methane production or a nutrient value for land application.

The second alternative is to continue the current operation of anaerobic digestion at both WWTPs, sludge transfer between the two WWTPs, sludge dewatering at the East WWTP, and landfill disposal. This alternative also features the option of installing dewatering facilities at the West WWTP and decommissioning the transfer sludge line between the West and East WWTPs because the sludge transfer line is in poor condition.

The third alternative would be a combination of the first two alternatives. The digestion process at the East WWTP would continue, along with cogeneration of power, and the West digesters would be decommissioned. Undigested sludge would be pumped from the West WWTP to the East WWTP. The West WWTP sludge would either be introduced into the East WWTP digesters or would be dewatered without digestion, depending whether there is available capacity at the East WWTP. This alternative also features the option of installing dewatering facilities at the West WWTP and decommissioning the transfer sludge line between the West and East WWTPs because the sludge transfer line is in poor condition.

Alternative 1 – No Digestion

Alternative 1 would eliminate the current practices of digesting the WWTP sludge and transferring sludge between the WWTPs, as shown in Figure 1. The primary sludge and WAS would be thickened separately in gravity tank thickeners and blended before dewatering. Each WWTP would have the capability to haul dewatered sludge to the landfill. Currently, Indiana allows the disposal of undigested sludge at landfills. While it is recognized that landfill disposal options may be more limited in the future, the facilities required for this alternative could be easily integrated into other solids handling options.

The East WWTP would convert one existing digester (75-foot diameter) to a WAS gravity thickener. A smaller (45-foot diameter) gravity thickener would be constructed for the primary sludge. Building 88 would be retrofitted with new dewatering equipment, including three centrifuges, a polymer feed system, progressive cavity pumps, and dewatered sludge conveyors. This alternative also accounts for the lost revenue associated with the proposed digester gas cogeneration system.

The West WWTP would also convert an existing digester (75-foot diameter) into a gravity thickener for WAS and construct a new thickener (50-foot diameter) for primary sludge. The existing sludge day tank would be expanded to provide approximately 180,000 gallons of additional storage. Building 32 would be retrofitted with new dewatering equipment, including three centrifuges, a polymer feed system, progressive cavity pumps, and dewatered sludge conveyors. Also, a new truck loading facility would be constructed. This alternative accounts for the loss of supplemental digester gas that is used for the influent pumps.

Alternative 2 – Digestion at Both WWTPs

The second alternative would be to continue the current operation of anaerobic digestion at both WWTPs, sludge transfer between the two WWTPs, sludge dewatering at the East WWTP, and landfill disposal. This alternative, shown in Figure 2, would completely refurbish the existing digesters at the West WWTP, with new mixing equipment, gas recovery, boilers, and piping. Also, the primary and WAS pump would be replaced. The digesters at the East WWTP are in considerably better operating condition and have had a recent upgrade of the boilers and mixing system. However, an additional digester may be needed at the East WWTP to handle future peak solids loads.

The digester capacities are based on maintaining a 15-day HRT. The analysis shows that the existing digesters at the West WWTP have sufficient capacity, whereas the digesters at the East WWTP are marginally too small. Therefore, sub-alternatives (2A and 2B) were developed to estimate the cost without constructing a fifth digester (75-foot diameter) at the East WWTP.

Alternative 2 includes a 180,000-gallon storage tank at the East WWTP that would receive the pumped sludge from the WWTP. The storage tank would include provisions for decanting and aeration.

Alternative 2 also includes replacement of the sludge transfer line. The replacement line would be high-density polyethylene and would be installed using a pipe bursting technique. Alternative 2A would incorporate a lower-cost option, which would be a cured-in-place lining repair of the sludge transfer force main instead of pipe bursting. Alternative 2B would involve abandoning the sludge transfer piping, building a dewatering building at the West WWTP for the digested sludge, and hauling the dewatered sludge directly from the West WWTP. The abandoned sludge line would be filled with controlled low-strength material.

Alternative 3 – Digestion at East WWTP

The third alternative is a hybrid of Alternatives 1 and 2. The East WWTP would continue the current operation of anaerobic digestion, sludge dewatering, and landfill disposal. An additional digester may be needed at the East WWTP to handle future peak solids loads. The West WWTP would pump its raw, undigested sludge to the East WWTP. Solids loadings from the West WWTP that exceed the capacity of the East WWTP digesters would be dewatered and hauled to a landfill without digestion. This alternative accounts for the loss of supplemental digester gas that is for the influent pumps at the West WWTP.

Shown in Figure 3, Alternative 3 would convert the existing secondary digester at the West WWTP to a solids storage tank, which would have provisions for mixing. Two of the primary digesters would be cleaned and the covers rehabilitated so they can be used to store peak sludge flows. These two tanks would not be thickeners because the intent is to pump diluted sludge to the East WWTP for processing.

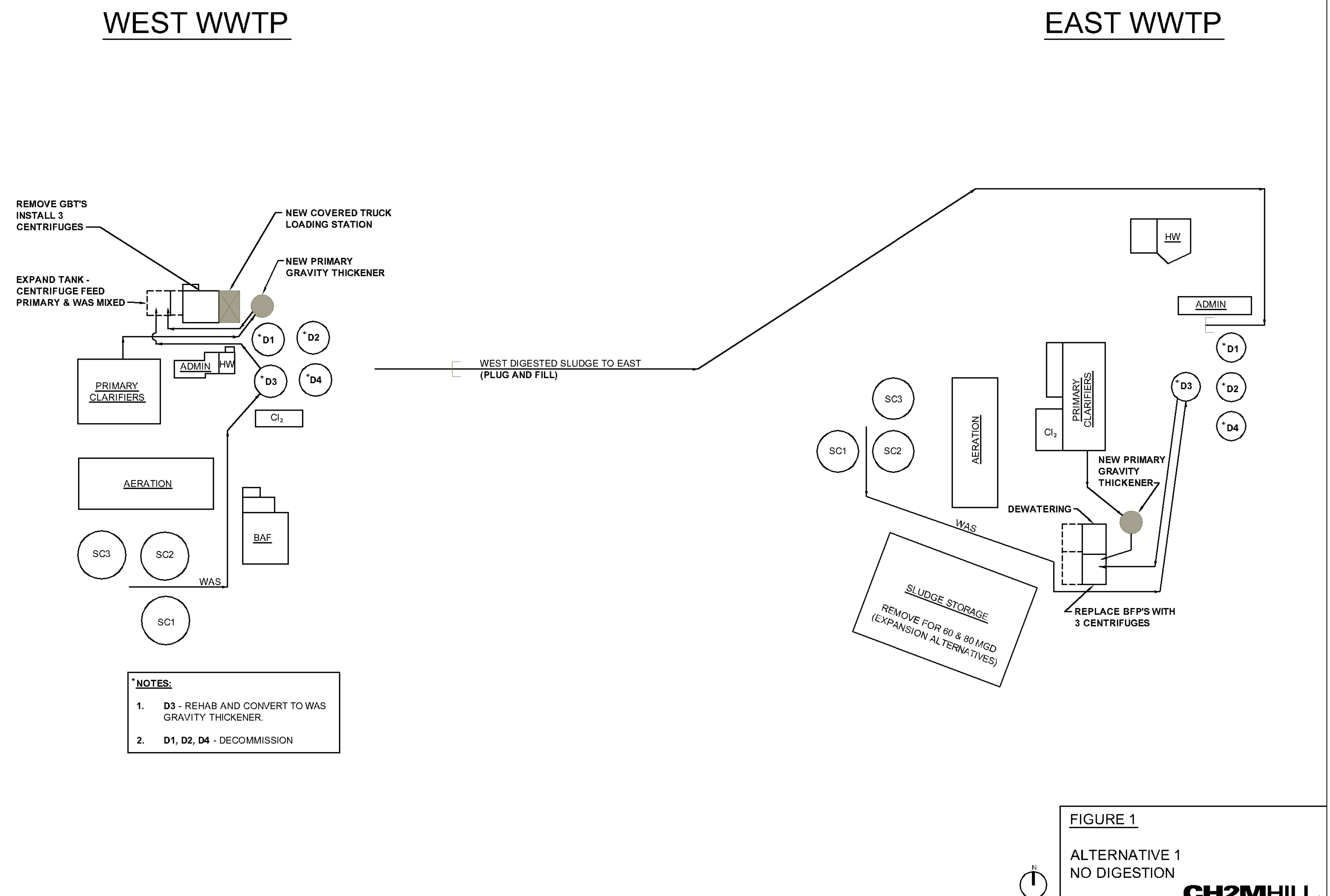
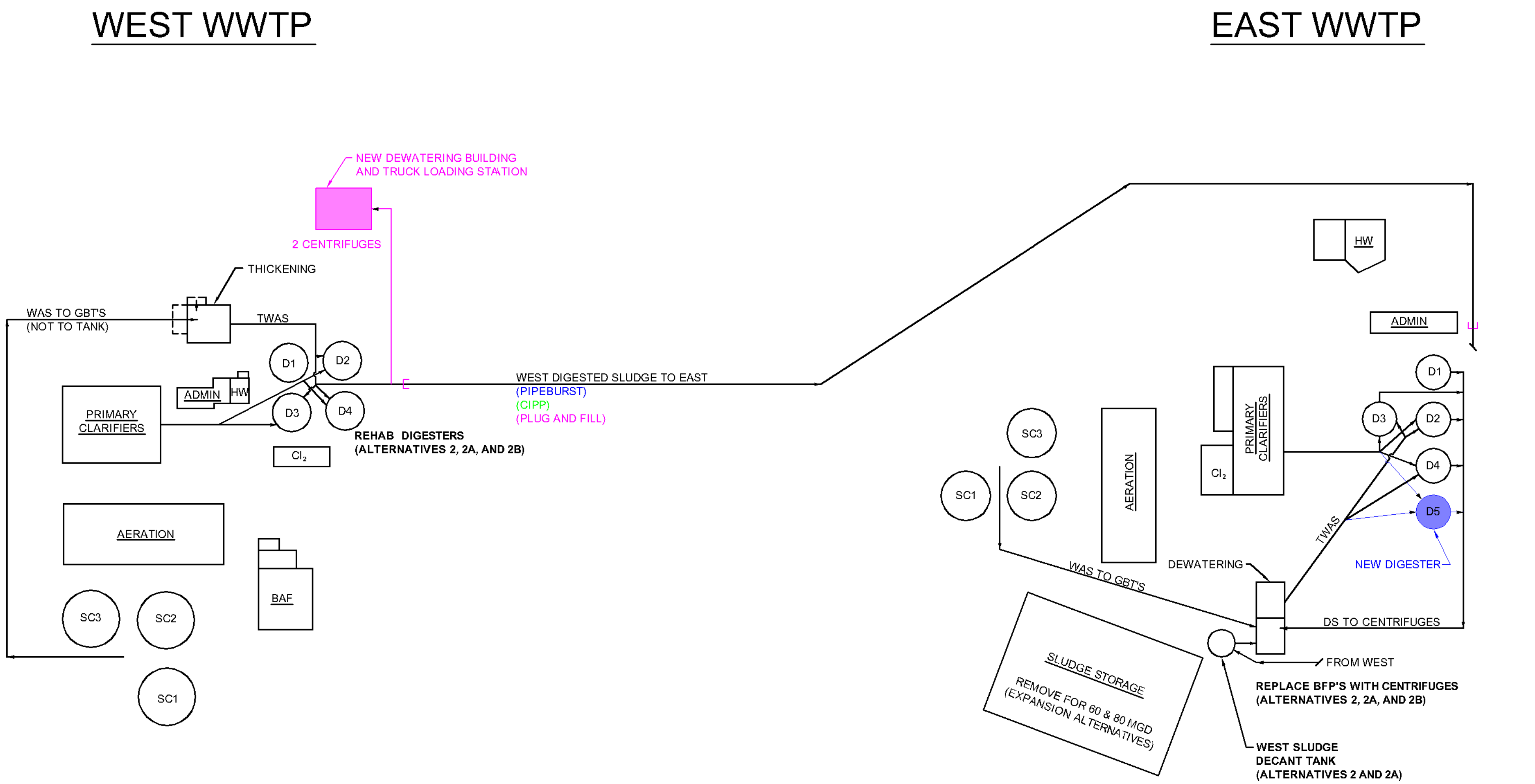


FIGURE 1

ALTERNATIVE 1
NO DIGESTION



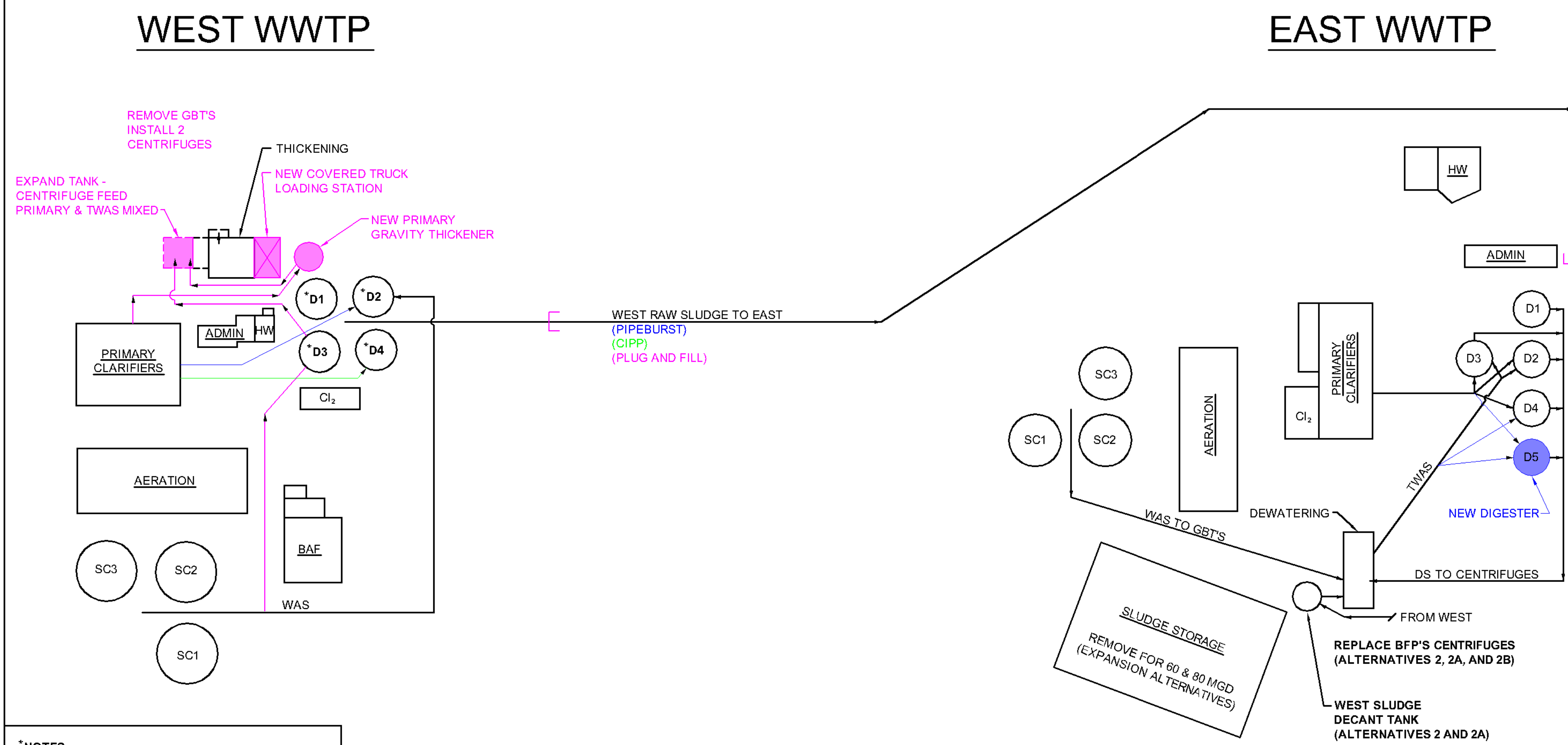
LEGEND

ALTERNATIVE 2..........................
ALTERNATIVE 2A........................
ALTERNATIVE 2B........................

FIGURE 2

ALTERNATIVE 2
DIGESTION AT BOTH WWTPs

CH2MHILL



*NOTES:

1. **D2 & D4** - REHAB AND USE FOR RAW PRIMARY AND WAS STORAGE.
(ALTERNATIVES 2 AND 2A)

2. **D1 & D3** - DECOMMISSION
ALTERNATIVES 2 AND 2A)

3. **D3** - REHAB AND CONVERT TO WAS GRAVITY THICKENER.
(ALTERNATIVE 2B)

4. **D1, D2, D3** - DECOMMISSION
(ALTERNATIVE 2B)

LEGEND

ALTERNATIVE 2..........................
ALTERNATIVE 2A........................
ALTERNATIVE 2B........................

FIGURE 3

ALTERNATIVE 3
DIGESTION AT EAST WWTP ONLY

CH2MHILL

Similar to the analysis for Alternative 2, the analysis shows that the existing digesters at the East WWTP are marginally too small. Therefore, sub-alternatives (3A and 3B) were developed to estimate the cost without constructing a fifth digester (75-foot diameter) at the East WWTP. Alternative 3A also includes a cured-in-place lining repair of the sludge transfer force main instead of pipe bursting.

Alternative 3B would involve abandoning the sludge transfer piping, installing dewatering equipment at the West WWTP for the undigested sludge, and hauling the dewatered sludge directly from the West WWTP. The abandoned sludge line would be filled with controlled low strength material. The new dewatering equipment would be installed in Building 32 and would include three centrifuges, a polymer feed system, progressive cavity pumps, and dewatered sludge conveyors. Also, a new truck loading facility would be constructed.

Cost Estimates

Cost estimates were developed for the solids handling facilities needed to process the primary sludge and WAS produced in the liquid treatment units described in Facility Plan. The capital and life-cycle cost estimates for the various alternatives are shown in Table 4. The capital costs for the alternatives ranged from $23 million (Alternative 3B) to $43 million (Alternative 2). Alternative 3B had the lowest life cycle cost, and Alternative 2 had the highest life cycle cost.

TABLE 4
Solids Handling Life-Cycle Cost Summary

Alternative	Total Capital ($)	20-year O&M ($)	Combined Cost ($)
1	32,900,000	15,700,000	48,600,000
2	43,300,000	11,600,000	54,900,000
2A	32,200,000	11,600,000	43,800,000
2B	27,800,000	11,500,000	39,300,000
3	36,500,000	13,500,000	50,000,000
3A	25,400,000	13,500,000	38,900,000
3B	23,200,000	13,200,000	36,400,000

During the development of the alternatives, the following findings were identified:

- Options that require a new digester have the highest capital and life-cycle costs.
- The sludge transfer line has a higher capital and life-cycle cost than building a new dewatering facility at the West WWTP. Replacement of the sludge transfer line also may include more inherent risks for construction.
- Options that eliminate digestion at the West WWTP have the lowest capital and life-cycle costs.

In summary, for the purpose of Facility Plan budgeting it was assumed that the Utility would plan for a capital cost of $23.2 million, which is based on the lowest cost alternative. However, there are a wide range of sludge handling options that are available, with varying complexities, costs and benefits (economic and non-economic), that the Utility should consider before implementing any major solids handling improvement.

APPENDIX E

East WWTP Effluent Pumping Technical Memorandum

East Wastewater Treatment Plant Effluent Pumping Evaluation

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DATE: July 25, 2012

Introduction

The Evansville Water and Sewer Utility (Utility) is currently developing a Facility Plan for the West and East Wastewater Treatment Plants (WWTPs). The Facility Plan will provide planning level cost estimates to increase the wet weather treatment capacities of the West and East WWTPs. The increased WWTP capacities are needed to mitigate sanitary sewer overflows and combined sewer overflows in the collection system.

The purpose of this technical memorandum (TM) is to provide capital and life-cycle cost estimates for the effluent pumping facilities at the East WWTP, which will be combined with other WWTP improvements to support the development of the Facility Plan and Integrated Overflow Control Plan (IOCP). This TM also describes the evaluation of the economic and non-economic factors for constructing effluent pumping facilities at two different locations.

Existing Facilities

The East WWTP is designed for an average flow of 18 million gallons per day (mgd). The treatment facility consists of fine bar screens, a vortex grit system, seven primary clarifiers, three aeration basins, three final clarifiers, and effluent chlorination/dechlorination. Final effluent from East WWTP flows by gravity to a Parshall flume and is then conveyed downstream to the interceptor chamber that currently houses combined sewer overflow Diversion Structure 103. From this point, WWTP effluent is conveyed through a 72-inch and 84-inch gravity sewer to the Ohio River. During high river stages, effluent from the East WWTP is pumped using the Levee Authority's K-4 Pump Station.

Alternatives Development

Effluent pumping options were evaluated at two locations:

- Alternative 1 –At the East WWTP, north of the access drive
- Alternative 2 – At Sunset Park, between the existing screening chamber and tennis courts

Refer to Figure 1 for the relative locations of these alternatives. The maximum river stage in a typical year was assumed in all effluent pumping alternatives.

Alternative 1

The effluent pump station requirements to convey 40, 60, and 80 mgd from the East WWTP are summarized in Table 1 and Figure 1.

TABLE 1
Alternative 1 – East WWTP Effluent Pumping Requirements

	40 mgd	60 mgd	80 mgd
Pump Station Building	Construct new effluent pump station (Approximately 20 ft tall/ 460 ft^2); wet well approximately 29 ft deep	Construct new effluent pump station (Approximately 20 ft tall/ 560 ft^2); wet well approximately 29 ft deep	Construct new effluent pump station (Approximately 20 ft tall/ 670 ft^2); wet well approximately 29 ft deep
Effluent Pumping	Install three submersible pumps, 13,900 gpm each	Install four submersible pumps, 13,900 gpm each	Install five submersible pumps, 13,900 gpm each
Effluent Force Main	Install approximately 930 LF of 54-inch force main Slip-line existing 72-inch/84-inch sewer with approximately 2,000 LF of 54-inch HDPE	Install approximately 930 LF of 60-inch force main Slip-line existing 72-inch/84-inch sewer with approximately 2,000 LF of 60-inch HDPE	Install approximately 930 LF of 72-inch force main Reline existing 72-inch/84-inch sewer with approximately 2,000 LF of CIPP
Other	Modify Interceptor chamber(Diversion Structure 103); seal existing flap gates and influent pipes Add 60-inch sluice gate to existing bypass line to K-4 Pump Station Relocate existing storm sewer at Veterans Memorial & Waterworks roads	Modify Interceptor chamber(Diversion Structure 103); seal existing flap gates and influent pipes Add 60-inch sluice gate to existing bypass line to K-4 Pump Station Relocate existing storm sewer at Veterans Memorial & Waterworks roads	Modify Interceptor chamber(Diversion Structure 103); seal existing flap gates and influent pipes Add 60-inch sluice gate to existing bypass line to K-4 Pump Station Relocate existing storm sewer at Veterans Memorial & Waterworks roads

Notes:

CIPP = cured-in-place pipe

ft = feet

ft^2 = square feet

gpm = gallons per minute

HDPE = high-density polyethylene

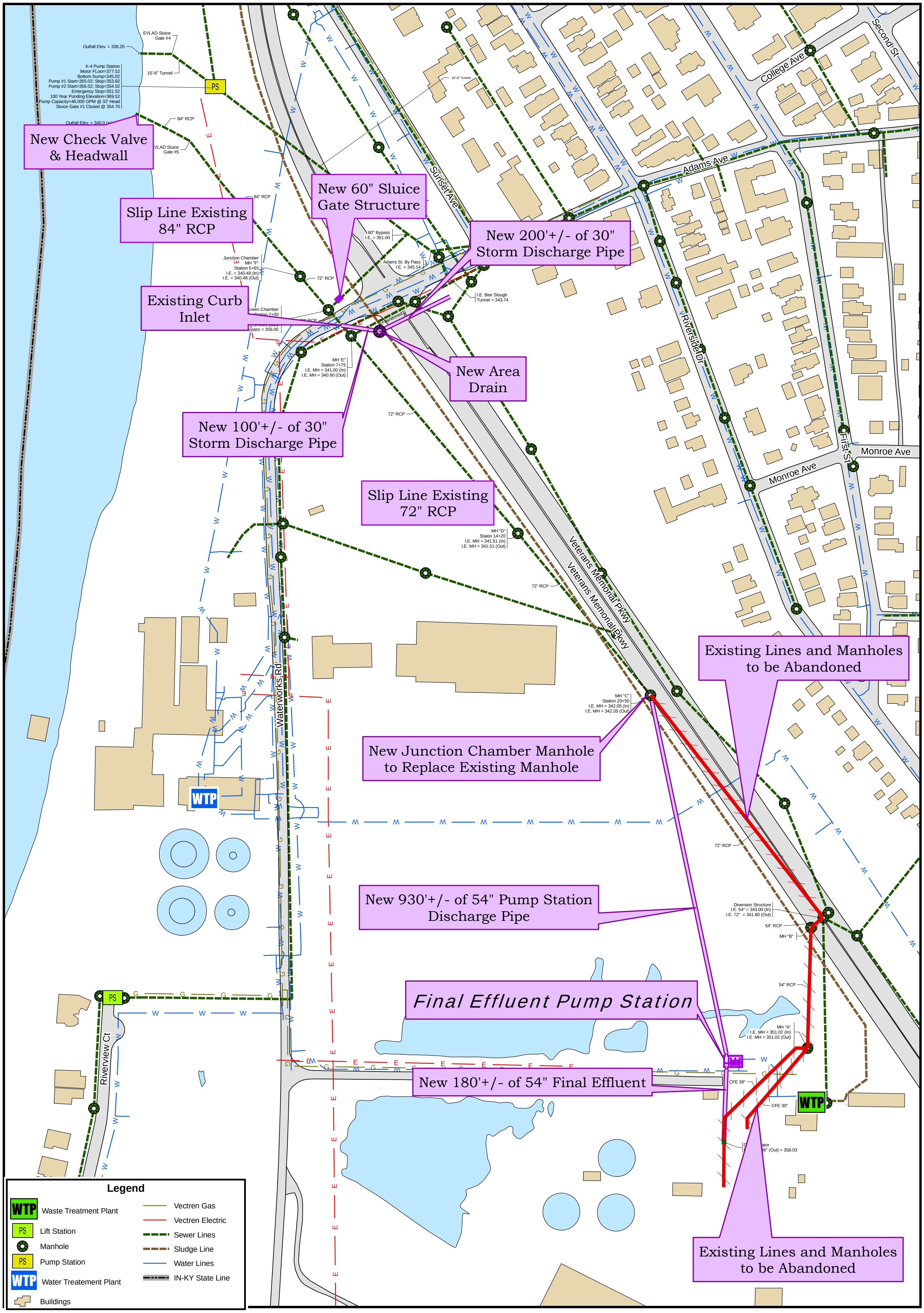
LF = linear feet

Alternative 1 would construct the effluent pump station on the north side of the East WWTP and repurpose approximately 2,000 LF of the existing interceptor sewer as a force main. All scenarios would install a new force main from the pump station to the interceptor sewer and seal the existing interceptor chamber. For the 40 and 60 mgd scenarios, the interceptor sewer would be slip-lined with HDPE pipe. The 80 mgd scenario would require CIPP in order to maximize the diameter of the force main. In addition, all scenarios would install a 60-inch sluice gate at the existing K-4 Pump Station bypass line as well as relocate the existing storm sewer at Veterans Memorial and Waterworks roads.

The existing 60-inch sluice gate would normally be closed to allow the new pump station to operate independently of the K-4 Pump Station. When the new pump station is not in service, the 60-inch sluice gate could be opened to allow use of the K-4 Pump Station.

Alternative 2

The effluent pump station requirements to convey 40, 60, and 80 mgd at Sunset Park are summarized in Table 2 and Figure 2.



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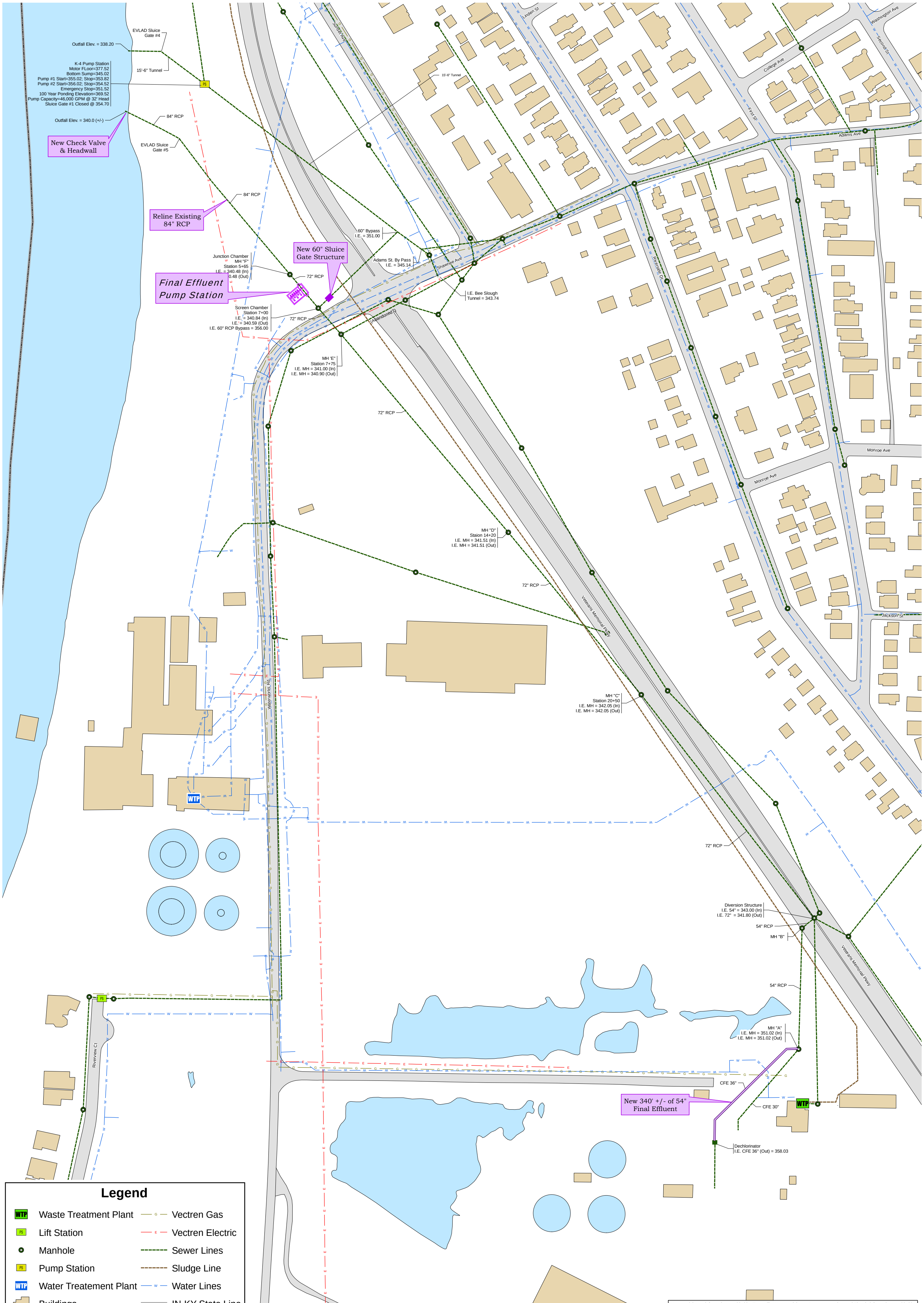
Revisions		
No.	By	Date

0 45 90 180 270 360 450 540 630 Feet

Sheet Title:

Figure 1
Pump Station at East WWTP

Scale: 1 " = 215 '	
Designed By: L.A.M.	Job Number: 8440
Drawn By: J.E.V.	Date: 5/11/2012
Filename: Pump Station Loc 1 - 40.mxd	
Sheet Number: Of:	



Water and Sewer information provided by Evansville Water and Sewer Department. Gas and Electric information provided by Vectren.

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No.	By	Date

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0 45 90 180 270 360 450 540 630 Feet

Sheet Title:

Figure 2
Pump Station at Sunset Park

Scale: 1 " = 100 '	
Designed By: L.A.M.	Job Number: 8440
Drawn By: J.E.V.	Date: 4/25/2012
Filename: Pump Station Loc 2.mxd	
Sheet Number:	Of:

TABLE 2
Alternative 2 – Sunset Park Effluent Pumping Requirements

	40 mgd	60 mgd	80 mgd
Pump Station Building	Construct new effluent pump station (approximately 20 ft tall/ 460 ft^2); wet well approximately 32 ft deep	Construct new effluent pump station (approximately 20 ft tall/ 560 ft^2); wet well approximately 32 ft deep	Construct new effluent pump station (approximately 20 ft tall/ 670 ft^2); wet well approximately 32 ft deep
Effluent Pumping	Install three submersible pumps, 13,900 gpm each	Install four submersible pumps, 13,900 gpm each	Install five submersible pumps, 13,900 gpm each
Effluent Force main	Slip-line existing 84-inch sewer with approximately 600 LF of 54-inch HDPE	Slip-line existing 84-inch sewer with approximately 600 LF of 60-inch HDPE	Reline existing 84-inch sewer with approximately 600 LF of CIPP
Other	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station	Add 60-inch sluice gate to existing bypass line to K-4 Pump Station

Alternative 2 would construct the effluent pump station at Sunset Park on top of the existing interceptor sewer. It would also repurpose approximately 600 LF of the downstream end of the interceptor sewer as a force main. The upstream segments of the interceptor would remain a gravity system. For the 40 and 60 mgd scenarios, the downstream portions of the interceptor sewer would be slip-lined with HDPE pipe. For the 80 mgd scenario, these portions would require CIPP. In addition, all scenarios would install a 60-inch sluice gate at the existing K-4 Pump Station bypass line.

Alternatives Evaluation

The alternatives were evaluated based on the following parameters:

- Constructability
- Operational considerations
- Environmental considerations
- Customer or public relations issues
- Present-worth costs

Constructability

Alternative 1 would be constructed within the confined, more-secured boundary of the existing WWTP, and could be constructed with little disruption to WWTP operations. The depth of construction would be approximately three feet shallower than Alternative 2 because of hydraulic grade line differences. However, this alternative would require relocation or encasement of the main potable waterline, main gas line feed, and main electrical feed at the East WWTP. The structural condition of the existing sewer interceptor and soil conditions for the new force main were assumed to be adequate; remediating actual conditions would drive up construction costs significantly. Relocation of the storm sewer at Sunset Park and Waterworks Road would be required as well.

Construction of Alternative 2 would likely require the closing of a portion of the Sunset Park parking lot for the contractor's construction equipment, laydown area, and employee vehicles. Additional costs would be required to secure the work area. However, Alternative 2 would not require relocation of the storm sewer at Sunset Park and Waterworks Road. Furthermore, most of the interceptor sewer piping upstream of Sunset Park could remain as a gravity line and may not be required to be re-lined for use as a pressure line. Therefore, Alternative 2 is more beneficial from a constructability standpoint.

Operational Considerations

For Alternative 1, all operation and maintenance (O&M) would be conducted within the secured location of the East WWTP, away from the general public and at a location where personnel are already centrally located. If the remote control and monitoring systems of the pumping station were ever lost due to inclement weather or equipment failure, the pumping station could still be operated in the manual mode by WWTP personnel who are already operating the WWTP. In addition, backup power and electrical feed to the pumping station would be readily available because of its proximity to the existing WWTP facility.

Alternative 2 would require remote operation of the pump station. However, the Utility has extensive experience operating remote pump stations. Nonetheless, Alternative 1 is more beneficial from an operational standpoint.

Environmental Considerations

Alternative 1 would require wetland mitigation for the construction of the pump station and force main. This alternative could also reduce the K-4 ponding basin. A stage-storage/pumping analysis would be necessary to ensure that the existing K-4 pump, new effluent pump station, and smaller storage area in the ponding basin would still be able to maintain the 100-year protection elevation.

Alternative 2 would not disturb existing forested wetlands north of the existing WWTP or K-4 ponding area. Therefore, Alternative 2 is more beneficial from an environmental standpoint.

Customer or Public Relations issues

In Alternative 1, the pump station would be less obtrusive to the general public because it would be screened by the existing woods on the west, north, and east sides, with only the south side open to view from the existing service by WWTP personnel. Because screening is already present, the need for landscaping/screening around the structure would not be required. In addition, the exterior aesthetics of the structure would not be a concern. Working conditions for personnel would be favorable because of the proximity to the WWTP.

In Alternative 2, the pumping station would be constructed within Sunset Park and would therefore be more obtrusive to the general public. The pumping station may also need to be enclosed within a fenced area for security. In addition, the structure would need to be made more visually attractive to the general public by providing exterior architectural amenities and/or landscaping to screen the building. Working conditions for personnel would be less favorable because of the greater distance from the WWTP. Therefore, Alternative 1 is more beneficial from a customer and public relations perspective.

Present-worth Costs

The present-worth costs were calculated using the cost guidance prepared for the IOCP, and are a combination of capital and O&M costs, as shown in Table 3. Alternative 1 has significantly higher total capital costs than Alternative 2 for all flow scenarios. This is primarily because of the additional construction required (more force main, relocation of existing storm sewer, etc.). The net-present value 40-year O&M cost was the same for each alternative for a given flow scenario.

TABLE 3
Effluent Pump Station Cost Summary

Flow	Alternative	Total Capital Cost ($)	Net Present Value 40-Year O&M ($)	Total Combined Costs ($)
40 mgd	1	16,150,000	766,000	16,916,100
	2	10,570,000	766,000	11,336,100
60 mgd	1	17,522,000	1,066,000	18,587,600
	2	11,572,000	1,066,000	12,637,600
80 mgd	1	21,575,000	1,366,000	22,940,500
	2	13,415,000	1,366,000	14,780,500

Summary

The capital and total combined costs for Alternative 2 are significantly lower than the costs for Alternative 1. As previously discussed, Alternative 2 is also more beneficial to the Utility from the constructability and environmental standpoints. Furthermore, operational and public relations issues can be addressed with Alternative 2 because the Utility has extensive experience operating pump stations remotely. Also, the Utility can construct an aesthetically attractive pump station to mitigate public concerns. Therefore, for the purpose of Facility Plan budgeting, it is assumed that the Utility would construct the pump station at Sunset Park (Alternative 2). During preliminary design, field conditions should be validated and opportunities for optimizing the effluent pumping configuration should be identified.

APPENDIX F

Elimination of Primary Bypass at West WWTP Technical Memorandum

Evaluation of Low-Cost Alternatives to Eliminate Primary Clarification Bypasses at the West WWTP

PREPARED FOR: EWSU

COPY TO: File

PREPARED BY: Chris Andres/CH2M

DATE: May 20, 2015

PROJECT NUMBER:

REVISION NO.:

APPROVED BY:

The technical memo is intended to summarize analysis conducted in response to action items agreed to during the follow-up meeting between Evansville Water and Sewer Utility (EWSU) and EPA on Monday 5/18/15. The results of this analysis will be used to support ongoing discussions between EWSU and EPA related to the City's Integrated Overflow Control Plan (IOCP). This TM specifically addresses EPA's request to evaluate low-cost alternatives that would result in the elimination of the primary treatment bypasses during wet weather.

Background and Approach

As documented the Facility Plan (submitted to EPA May 31, 2013), the recommended plan for the West WWTP included constructing a new headworks facility with screening, grit removal, and influent pumping, all of which would be rated at a firm capacity of 45 MGD. For a number of reasons, expanding primary treatment capacity was ruled out during the technology screening phase of facility planning. Refer to the Facility Plan for more details on that analysis. As the current capacity of primary clarification is limited at 40 MGD, the recommended plan also included a 5 MGD primary bypass diversion structure to route excess flow around the primaries during wet-weather. The estimated total capital cost for the 5 MGD bypass was $45,000 (2012 dollars). Analysis conducted during facility planning showed the primary bypass would have nominal negative impacts on downstream processes.

In response to EPA's requests, CH2M's WinHydro hydraulic modeling software was utilized to assess existing hydraulics and to evaluate the effectiveness of potential improvements. The final hydraulic model developed in facility planning (45 MGD through the West WWTP) was utilized in this evaluation. Data collected during stress testing as well as findings from facility planning effort were also utilized to develop solutions. The same bottom-up cost estimating guidelines/approach developed for Facility Planning were utilized to determine capital costs for each alternative in this evaluation. Therefore, the estimates developed during this evaluation are directly comparable to those developed as part of the facility planning effort. Details of the cost estimating guidelines can be found in Appendix A of the Facility Plan report.

Due to the desire for low-cost alternatives, options such as adding additional process units were not considered.

Evaluation

Stress testing of the West WWTP conducted in spring of 2012 showed that the six primary clarifiers were hydraulically limited. Data collected from stress testing indicated the root cause of this limit is downstream hydraulics, i.e. tail-water conditions in the effluent launders and common effluent channel. Upon further analysis, the following observations were made:

- The existing 48-inch piping connecting the primary clarifier effluent conduit with Structure Y-3 is a hydraulic bottleneck at peak flow. Replacing this pipe with a channel of similar width as Structure Y-3 would allow the additional 5 MGD to be conveyed over the effluent weirs while maintaining similar gradelines that the WWTP currently sees at 40 MGD. However, it should be noted that this results in submerged weir conditions during peak flow.

- According to the model, it is feasible to raise effluent weirs 4.8 inches while maintaining 12-inches of freeboard in the primary tanks. Depending on influent gate position, the resulting freeboard in the primary clarifier influent channel ranges from 9.5-inches (at 50% open) to 12-inches (100% open).
- If clarifier weirs are raised 4.8 inches, the weirs will be high enough to maintain freefall without implementing modifications at Structure Y-3.

As a result of these findings two options were developed. Table 1 describes each along with the estimated total capital costs. For reference, the Base Option 5 MGD primary bypass is included as well. Figure 1 illustrates the locations of the proposed options. During discussions with EWSU, EPA specifically mentioned evaluating chemical addition as one of the potential options. Chemical addition is a well-established means of improving the removal efficiency of clarifiers, thereby increasing process capacity. However, because the West WWTP primary clarifiers are hydraulically limited, chemical addition will not help increase hydraulic capacity. Therefore, this option was not evaluated further.

TABLE 1
Summary of Low-Cost Alternatives

	Description	Capital Cost	Comments
Base Option	5 MGD Primary Bypass	$45,000	Original recommendation per Facility Plan
Alternative 1	Extend Structure Y-3 to the primary clarifier effluent conduit	$42,000	Additional 5 MGD conveyed through primaries, but results in submerged weir conditions during peak flow
Alternative 2	Raise clarifier weirs	$170,000 - $235,000	Depending on actual field/operating conditions, an additional $65K may be required to raise influent curb walls. No weir submergence observed during peak flow.

As shown in Table 1, Alternative 1 has a very similar capital cost with respect to the primary bypass base option. This modification will allow an additional 5 MGD to be conveyed through the primaries, and could have the added benefit of improving parshall flume operation/accuracy by reducing tailwater conditions in the channel. However, implementing Alternative 1 will result in submerged weir conditions. EWSU operator preference is to maintain freefall over the weirs.

Implementing Alternative 2 would have a more hydraulically desirable impact on clarifier operation by maintaining freefall over effluent weirs during wet-weather events, but would cost approximately four times more in capital cost than the other two options. Moreover, depending on actual field/operating conditions, raising weirs could result in low freeboard conditions in the influent channel. For this reason, costs associated with constructing 6-inch curb walls around the influent channel were determined as a potential “adder” to Alternative 2. Because high water levels in this channel are well documented, it is recommended that the root cause(s) of the high water levels be identified prior to constructing curbs. Documentation from previous site visits indicates that significant throttling of the influent gates could be contributing to the issue.

Recommendations

If increasing primary treatment capacity 5 MGD is a priority, and occasionally operating the primary clarifiers under submerged weir conditions is a concern, it is recommended that the Utility implement Alternative 2. However, if the Utility is comfortable operating with weirs submerged during peak flow events, it is recommended that Alternative 1 be implemented; the cost savings could then be applied toward other WWTP or collection system improvements.

FIGURE 1
West WWTP Site Plan with Low-Cost Options