CITY OF BELTON, MISSOURI

WWTF FACILITY PLAN UPDATE

TECHNICAL MEMORANDUM NO. 1

WWTF FLOW PROJECTIONS AND PROJECT PHASING

FINAL
September 2012

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CITY OF BELTON, MISSOURI

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NO. 1

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WWTF FLOW PROJECTIONS AND PROJECT PHASING

1.0 BACKGROUND

1.1 Purpose

The purpose of this Tech Memo (TM) is to evaluate projected population and flow projections, and the capacity of the existing processes at the Belton Wastewater Treatment Facility (WWTF) and identify a phasing plan for addressing capacity needs to meet projected flows through the year 2032. This TM will also evaluate flow triggers and methods for monitoring capacity throughout the planning period.

1.2 Existing Treatment Facilities

1.2.1 General Description

The existing Belton WWTF facilities include influent pumping, preliminary treatment, secondary treatment, effluent disinfection, and solids handling. A general site plan of the facilities at the wastewater treatment plant is shown in Figure 1.1.

The wastewater flows by gravity from the residences and commercial properties within the WWTF service area to the Influent Pump Station. The influent pumps lift the wastewater, from an elevation of 873.0, and pumps it through a 20-inch force main to the Headworks facility, at a discharge elevation of 948.0. The Headworks facilities serve to remove some of the inorganics and larger debris from the waste stream prior to treatment.

Secondary treatment consists of the activated sludge process, which includes an aeration basin with three concentric flow channels followed by secondary clarification with three circular clarifiers. Submerged horizontal axis mechanical aerators supply air to the flow channels in the aeration basin. Effluent from the secondary clarifiers is measured downstream via an effluent Parshall flume, before it continues flowing into the UV Basin for disinfection prior to discharging into East Creek.

The return activated sludge (RAS) that settles out in the bottom of the clarifier basins is collected in a pit at the center of the basin bottom in each clarifier and is routed to a common wet well where it is pumped, using submerged centrifugal pumps, back into the aeration channels. A valve on the discharge header of the RAS pumps also allows the pumps to be used to periodically waste the activated sludge (WAS) to the sludge holding tanks.

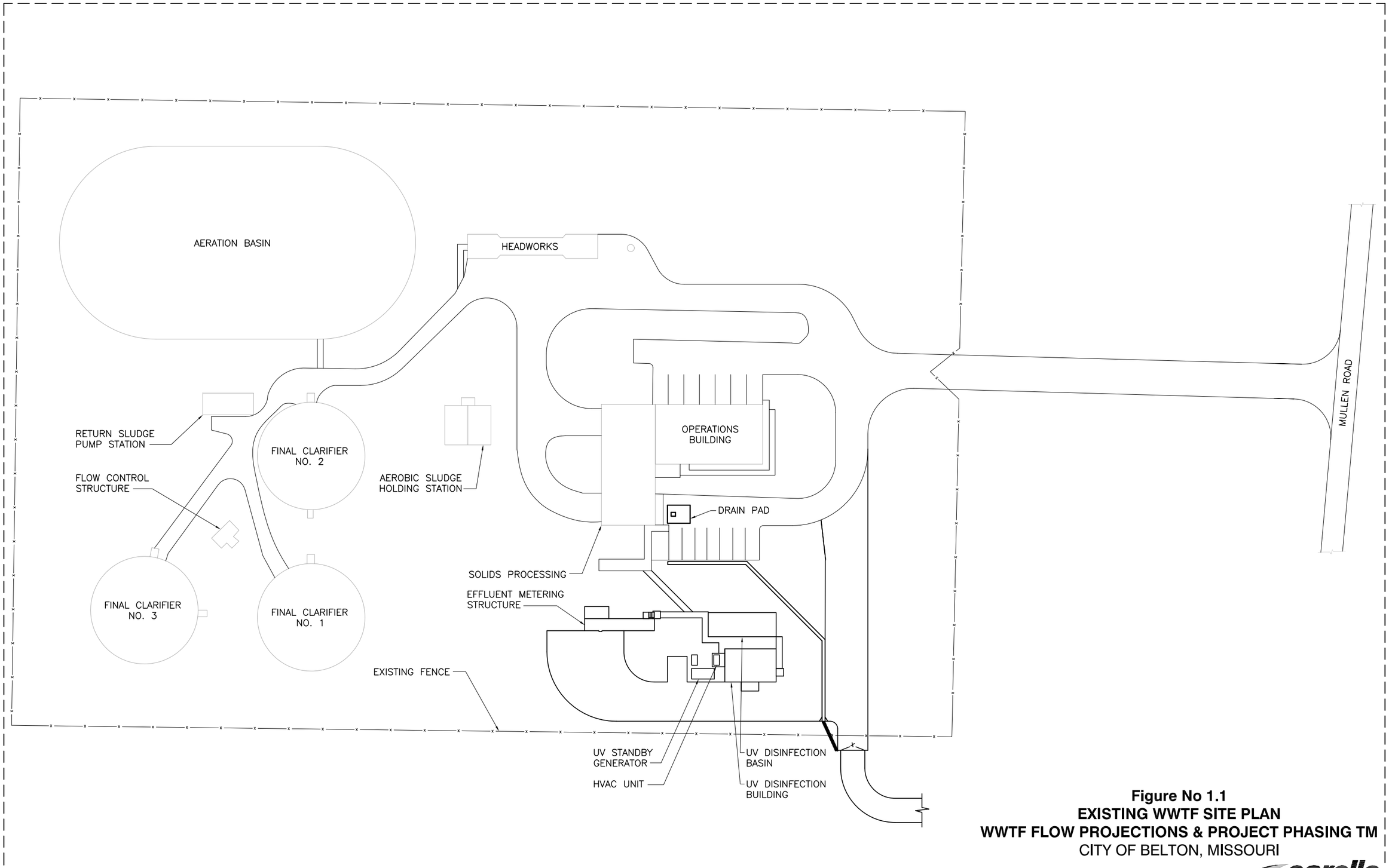

Figure No 1.1
EXISTING WWTF SITE PLAN
WWTF FLOW PROJECTIONS & PROJECT PHASING TM
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Air is supplied, for aeration and mixing of sludge in the sludge holding tanks, from centrifugal blowers. Periodically the air is turned off in the sludge holding tanks to allow the contents to settle and thicken in the basin bottom. The clearer supernatant is decanted off from the top of the sludge basin and discharged to the plant drainage pumping station for return to the head of the treatment system. The thickened biosolids are then pumped from the sludge holding basin to two belt filter presses located in the Operations Building for dewatering. The dewatered biosolids are hauled by truck to the Courtney Ridge Landfill (a division of Allied Waste) in Sugar Creek, Missouri, for ultimate disposal.

1.2.2 Influent Pumping

The Influent Pump Station (IPS) houses a total of 6 submersible pumps, which include five Flygt pumps rated at 1,150 gpm each at a total dynamic head (TDH) of 97 feet, and one ABS pump rated at 1,082 gpm at a TDH of 97 feet. The total rated pump station capacity is 9.84 mgd, with a firm capacity of 8.18 mgd with one of the larger pumps out of service.

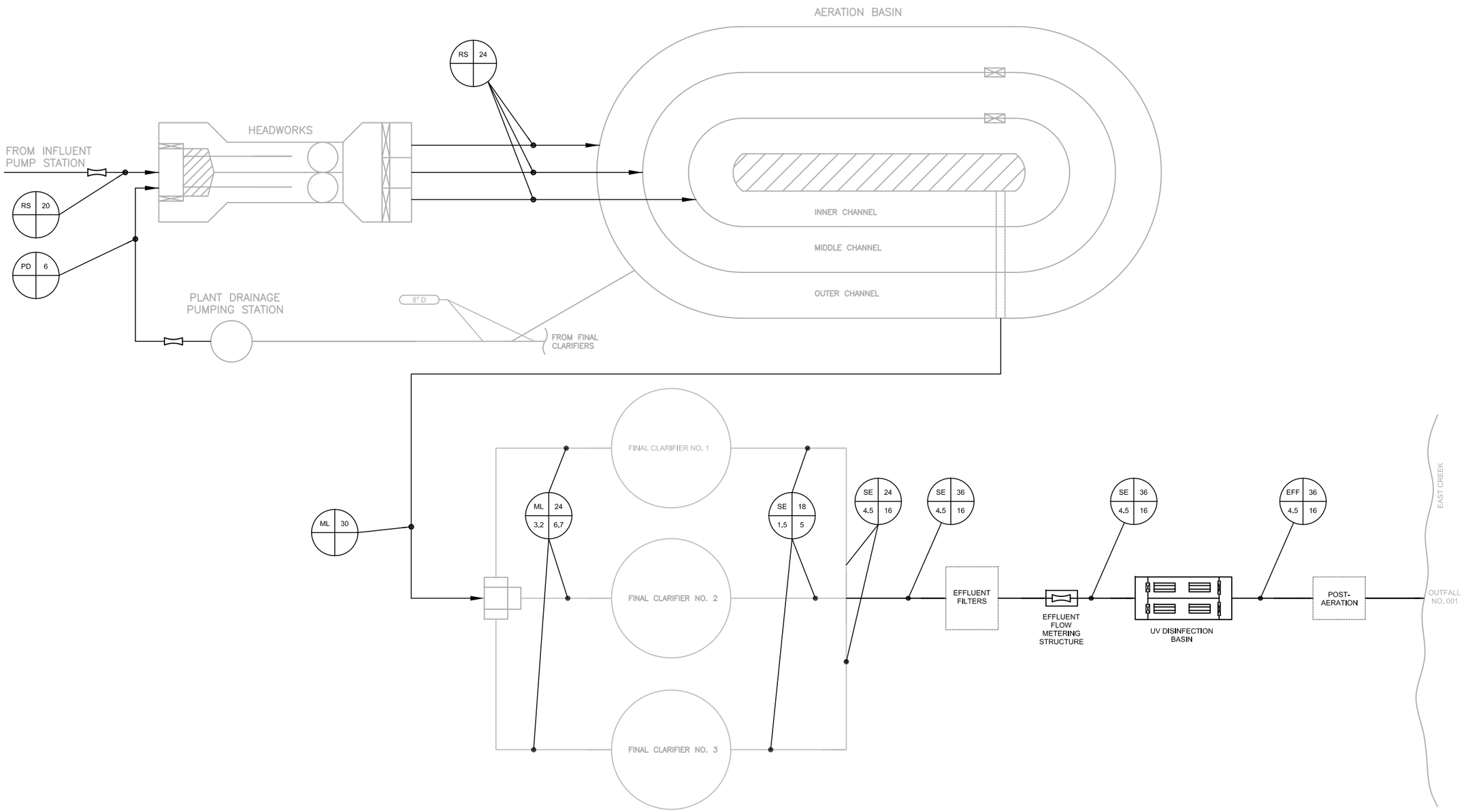
However, peak flows higher than the rated capacity of the IPS have previously been measured through the plant. According to plant staff, during these peak flow events the water level in the pump station wet well increases above the design level, which decreases the pump static head, thereby decreasing the head against which the pumps operate. A decrease in total dynamic head (TDH) allows the pumps to pump a higher flow than designed to the plant. However, an increase of the wet well level also translates to an increase in the water level at the manhole located just upstream of the IPS which can result in flooding of this manhole. Immediate improvements to the IPS are recommended to increase the capacity of the facility to prevent future overflows from the pump station or upstream manholes in the collection system.

As previously mentioned, a 20-inch diameter force main currently conveys flow from the IPS to the Headworks facility on the WWTF site. The maximum velocity recommended through a force main pipe is 8 fps, which limits the capacity of the 20-inch force main to approximately 11.3 mgd. Based on flow monitoring previously collected by others (Wade and Associates, Inc., 2006), peak flows to the WWTF have previously exceeded 11.3 mgd. It is recommended that a parallel force main be installed to reduce the velocity and limit the headlosses at the pumps, which directly relates to increased power costs.

1.2.3 Liquid Treatment

Liquid treatment at the Belton WWTF incorporates both preliminary and secondary treatment. The preliminary treatment consists of grit removal, solids grinding, and bypass screening. As previously mentioned, the secondary treatment process consists of the extended aeration activated sludge process followed by clarification. The liquid treatment schematic for the Belton WWTF is shown in Figure 1.2.

The analysis of the liquid treatment processes will begin with the grit removal process at the Headworks, move through the secondary treatment processes, effluent metering, and on to UV disinfection.



PROCESS FLOW SCHEMATIC

Figure No 1.2
LIQUID PROCESS SCHEMATIC
WWTF FLOW PROJECTIONS & PROJECT PHASING TM
CITY OF BELTON, MISSOURI

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1.2.3.1 Grit Removal

Grit removal is accomplished by gravity using a rectangular horizontal-flow grit chamber at the upstream end of the Headworks facility. The grit removal equipment was installed as part of the 1991 construction project at the WWTF. In this treatment process, the raw sewage flows through the rectangular grit chamber and over a weir into the outlet box where flow discharges from the Headworks facility over one of three proportional weir gates. As flow passes through the grit chamber, heavy solids and gritty particles settle out of the waste stream into the bottom of the channel. Mechanical scraper blades push the settled grit into a nearby dumpster for disposal. The dumpster includes a drain system that is capable of reducing the amount of water in the settled grit slurry by approximately 50% prior to ultimate disposal.

A second grit removal basin, in an adjacent flow channel, is available for redundancy. However, the second grit chamber is not equipped with a grit conveyor. When in-service, plant staff must manually remove the settled grit from the bottom of the rectangular grit chamber.

1.2.3.2 Grinding

Following the grit chambers, the wastewater passes through one of two comminutors located at the downstream end of the Headworks concrete flow channels. Comminuters operate to pulverize or shred the trash and debris in the raw wastewater stream in an effort to simplify treatment and prevent accumulation of the large particles further downstream. The comminutors have a firm nameplate capacity of 3.65 mgd.

One disadvantage with the use of comminutors is that the pulverized solids are not removed from the wastewater stream at the equipment and may present problems further downstream, including clogging of pipes and pump impellers, often resulting in an increase in headloss through the system. The existing comminutors have been in service for over 20 years and are nearing the end of their useful life. Replacement of the comminuters is recommended as part of the immediate improvements at the WWTF.

In the event either or both of the comminutors are out of service, influent flow can be diverted through a manual bar rack installed in a bypass channel between the two comminutors. The bar rack functions as a type of screen to remove larger solids remaining in the flow stream, from the grit chamber, upstream of the secondary treatment processes.

1.2.3.3 Aeration Basin

The Aeration Basin has three concentric channels outfitted with submerged horizontal-shaft mechanical aerators. The basin was originally operated with the influent wastewater entering the inner channel of the basin; with treatment occurring from the “inside-out,” and effluent being withdrawn from the outer channel. However, the basin was also configured with flexibility to allow the basin to be operated in an “outside-in” operation mode, with influent wastewater entering the outer channel and effluent withdrawn from the inner channel.

Plant staff currently uses mixed liquor suspended solids (MLSS) concentration as the parameter for controlling operation of the Aeration Basin. As previously mentioned, sludge is wasted from the final clarifiers on timed intervals to maintain a target MLSS concentration in the flow stream. This target MLSS concentration is adjusted on a weekly basis by plant staff, as needed, based on influent conditions.

The mechanical aerators in the basin are submerged, disk-type, horizontal shaft aerators, designed to efficiently transfer oxygen into the wastewater to meet the biological oxygen demands (BOD). The existing Aeration Basin has 4, 4, and 6 aerator assemblies installed in the inner, middle, and outer flow channels, respectively, and 15 disks per aerator in the inner channel, 27 disks per aerator in the middle channel, and 32 disks per aerator in the outer channel. Currently, the average flows through the plant can be treated with the outer channel out-of-service.

Plant staff also turns aerators on and off as needed to meet the anticipated oxygen demands and to maintain a minimum DO concentration of 2.0 mg/L in the flow stream. No automated control strategy is currently in place to allow the aeration system to automatically react to changes in influent conditions or DO demands. Because of this, there are times when the DO drops below the recommended target value, and other times when the basins are over-aerated and the DO values exceed 5.0 mg/L. Automation of the aeration system in the basin is recommended for more efficient operation.

1.2.3.4 Flow Control Structure

The flow control structure, located downstream of the Aeration Basin, is a T-shaped concrete structure equipped with three weir plates to proportionally split flow between the three downstream clarifier basins. The mixed liquor flows by gravity from the Aeration Basin to the center of the flow control structure and flows out, proportionally, over one of the respective weirs for the clarifiers that are in service. Flow splitting is based on available head over the rectangular weirs. Stop plates are also available at the flow structure for isolation of the clarifiers in the event one of them is out-of-service or is not needed at a particular time. Currently, under normal flow conditions, only two of the three clarifier basins are in operation, with the third basin being brought online during peak wet weather flows.

1.2.3.5 Final Clarification

The final clarifiers are circular concrete basins that provide a quiescent zone where the solids in the mixed liquor are allowed to settle out. The clarified liquid flows over V-notch weirs located around the periphery of the clarifier basin, into an effluent launder flow channel, and on to the downstream treatment facilities. Effluent launder covers and a density current baffle were recently installed on each clarifier basin. The launder covers help to reduce algal growth in the effluent flow channel of the basins. This prevents sloughing of large chunks of algae into the effluent flow stream to the disinfection basins.

Density current baffles are installed along the interior walls of the circular basins at a level midway between the maximum sludge blanket height and the water surface. The current

baffles help to direct solids down toward the center and bottom of the basin. The majority of the solids that are settled out of the mixed liquor are pumped back to the beginning of the aeration zone as RAS. To maintain the target MLSS concentration in the treatment stream, the remaining settled solids are wasted to the aerobic sludge holding basins as WAS.

1.2.3.6 Effluent Metering

The effluent from the three secondary clarifiers combines into a single 24-inch pipe that increases to 36-inches in diameter, upstream of the Effluent Metering Structure (EMS). The EMS includes a Parshall flume for effluent flow measurement. This structure was recently installed in replacement of the original effluent Parhsall flume structure. The effluent flow from the EMS then flows to the new UV Disinfection Basins.

Parshall flumes measure liquid flow in open channels. The Parshall flume is a constriction of the channel that develops a hydraulic head proportional to flow. As with all flow elements, Parshall flumes have a documented accuracy associated with them. These flumes are most accurate during free flow conditions. Free flow occurs when there is no backwater influence that will reduce the discharge or depth of water as it flows through the flume. When the flume is operating under free flow conditions, the flow measuring accuracy is between 95 and 97 percent. During submerged conditions, the accuracy ranges from 80 to 95 percent. Under all conditions, a proper approach that creates uniform flow conditions immediately upstream of the constriction is critical to the proper performance of the flume. The new Metering Structure was designed with a straight approach channel just upstream of the flume to account for better performance of the ultrasonic flow meter, which is suspended over the throat of the flume for flow measurement.

1.2.3.7 Disinfection Treatment

A new UV Disinfection Basin was recently constructed for disinfection of the secondary effluent upstream of the plant outfall. UV disinfection is a physical process in which ultraviolet energy is absorbed in the DNA of the microorganisms, causing structural changes in their DNA that prevents the microorganisms from reproducing. The UV energy is supplied by lamps that are submerged into the flow stream in specially designed flow channels. The hydraulic detention time for UV disinfection to achieve a good quality secondary effluent is just a few minutes.

The UV Basin at the Belton WWTF is designed with two flow channels, each including two banks of UV lamps. The UV system is designed to treat up to a maximum flow rate of 16 mgd (or 8 mgd per channel), and is expandable, with the installation of additional UV lamps in each channel, to an ultimate capacity of 12 mgd per channel, or 24 mgd total.

A UV Disinfection Building was also constructed as part of the disinfection system as an ancillary to the treatment basin. The UV Building houses the electrical equipment associated with operation of the UV equipment, and also serves as a storage facility for maintenance and other supplies for Plant Staff.

1.2.3.8 RAS Pumping

There are five centrifugal, submersible pumps identified as Return Activated Sludge (RAS) Pumps Nos. 1, 2, 3, 4, and 5 installed at the facility. Plant staff uses these pumps alternately to return settled solids (or sludge) from the clarifier basins or to waste sludge to the sludge holding basins. Each RAS Pump has a rated capacity of 1,205 gpm against 16.5 feet of head. RAS is periodically removed from the bottom of the clarifier basins through a 16-inch telescoping valve and 16-inch piping, and flows by gravity to the wet well at the RAS Pump Station. Plant staff manually sets the telescoping valve weir crest elevation to a known set point required to remove enough sludge to maintain the required MLSS concentration in the clarifier basins.

The submersible return sludge pumps are located in the RAS Pump Station wet well and pump the RAS through a manually operated plug valve into a 20-inch common header. The RAS flows through the common 20-inch header to the aeration basin. RAS flow to the Aeration Basin is not measured.

Currently, alternate operation of the return sludge pumps is controlled through the Plant's PLC system, such that an alternate pump is initiated at each start-up of the pumping operation. As required, a RAS flow of approximately 1,205 gpm is maintained, based on original design values. With one pump running, and assuming an average flow of 1.66 mgd to the plant, the RAS return rate is approximately 105% of the influent flow. Ten States Standards recommends a sludge return rate of 50% - 150% for extended aeration processes.

However, the current RAS system does not allow for much process flexibility. Consideration should be given to installing VFD controllers on the RAS pumps and automation for automatic cycling of the pumps. This would allow operating staff to control the RAS rate based on plant influent flow, or on a predetermined operator selectable rate.

1.2.4 Biosolids Treatment

The biosolids treatment at the Belton WWTF consists of WAS pumping, aerobic solids holding basins and a dewatering facility. The solid treatment process schematic for the Belton WWTF is shown in Figure 1.3.

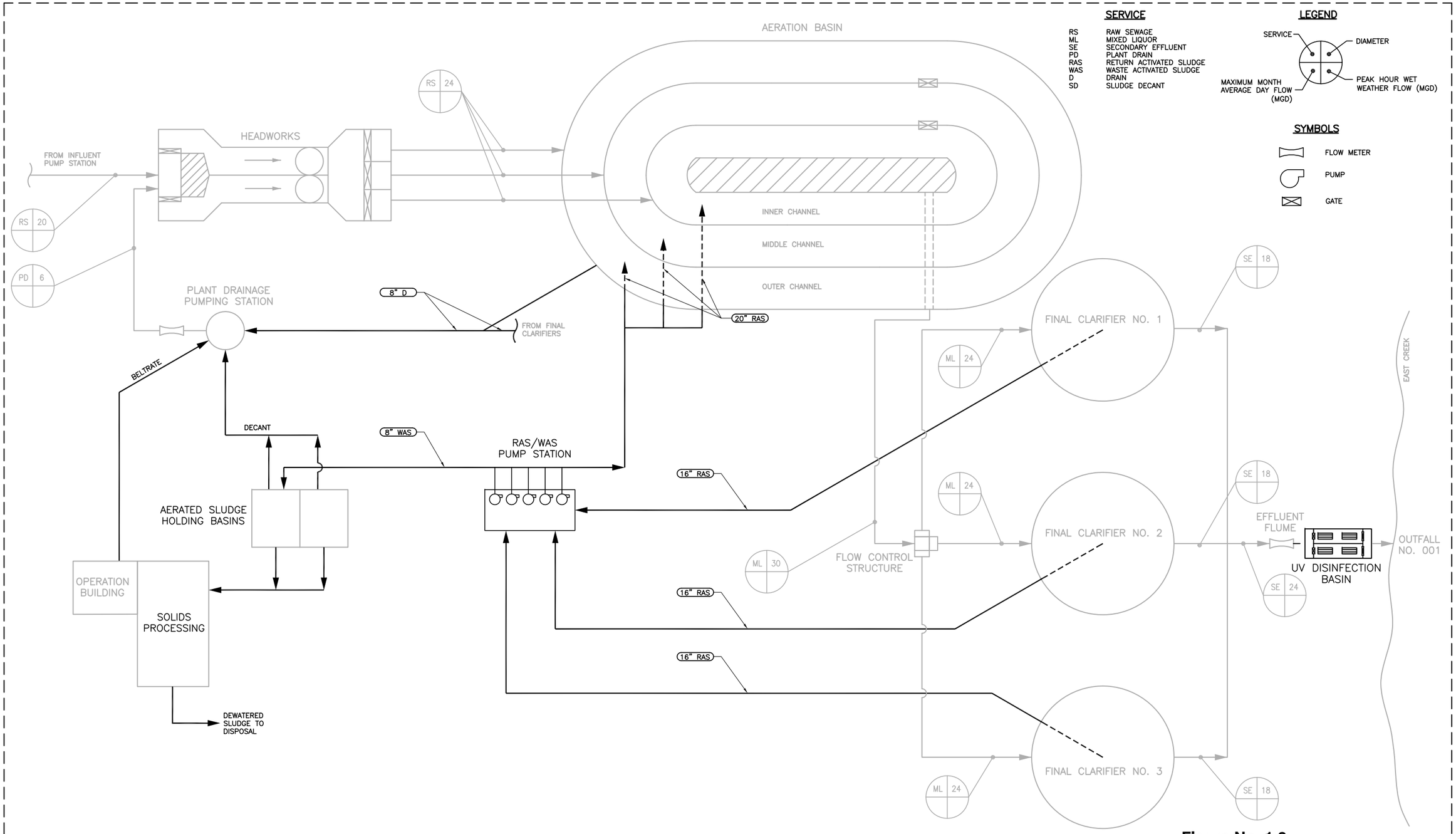

Figure No. 1.3
EXISTING SOLIDS PROCESS FLOW SCHEMATIC
WWTF FLOW PROJECTIONS & PROJECT PHASING TM
CITY OF BELTON, MISSOURI

1.2.4.1 Waste Sludge

To control the MLSS concentration in the biological treatment process, a fraction of the solids settled out in the Final Clarifiers is periodically removed, or wasted, from the system. To waste a portion of the solids, a plug valve, located between the 20-inch header on the RAS pumps and the Aeration Basin, is closed and the discharge from the sludge pumps, or WAS, is diverted through the 20-inch pump header, - which reduces to an 8-inch pipe, to the Aerobic Sludge Holding basins for solids stabilization.

In addition to the modification suggested for the RAS pumps, more flexibility could be obtained by designating one of the RAS pumps specifically for sludge wasting. This would allow that WAS pump to operate continually and provide plant staff the ability to change the daily WAS rate based on influent flow or on a predetermined schedule. This enhancement would also improve the performance of the sludge holding basins by providing a more consistent flow rate and minimizing hydraulic surges as the sludge pump starts and stops and plugs of sludge flow are introduced into the basins.

1.2.4.2 Aerated Sludge Holding Basins

As previously mentioned, WAS from the RAS/WAS pump station is pumped to one of two rectangular concrete aerated sludge holding basins. Each basin has a storage capacity of 5,250 ft^3 (39,270 gal). This equates to a total storage volume of 10,500 ft^3 (78,540 gal) with both basins in service. The piping from the RAS/WAS pump station includes isolation valves that are designed to allow the sludge to be directed to either one or both of the holding basins. The basins are aerated by fine bubble diffusers. Aeration is required to keep the biosolids from going septic and creating odors, as well as for mixing. After a set detention time, the WAS from the aerated sludge holding basins is pumped to the sludge dewatering process. Periodically, the air to each holding basin is shut off for short periods of time to allow the solids in the sludge to settle and thicken. The clear decant liquid, or supernatant, can then be removed from the basins using decant valves installed at each basin. The supernatant flows by gravity to the Plant Drainage Pumping Station. The thickened sludge is then pumped to the sludge dewatering process.

Based on an estimated WAS production of 1,000 lbs solids/d/mgd and an AAD of 1.66 mgd, the sludge holding basins have the capacity to store 2.0 days of 1.0 percent solids (by weight).

1.2.4.2.1 <u>Air Supply</u>

The air needed to meet the biological and mixing requirements of the sludge holding basins is supplied by three centrifugal blowers located outdoors, on the north side of the sludge holding basins on a concrete pad. The blowers are rated at 500 scfm at 7.5 psig each. Assuming that one of the units is out of service, the system would have a firm capacity of 1,000 scfm.

The air requirement for the sludge holding basins at standard operating conditions is 210 scfm. To meet this demand at current AAD conditions only one blower is needed to meet the air requirements.

1.2.5 Sludge Dewatering

Sludge dewatering is accomplished using two belt filter presses.

The belt filter presses and associated equipment are located in a dedicated sludge dewatering room attached to the Operations Building. The room is ventilated to provide a comfortable work atmosphere. However, over the years, the ventilation system has deteriorated and is no longer operating properly.

One of two belt filter feed pumps withdraws thickened sludge from the aerated solids holding basins and feeds the sludge onto one of the belt filter presses. Water is squeezed out of the sludge as it travels through the filter press between wedges and belt rollers prior to its discharge onto the sludge conveyor belt. The conveyor belt transfers the dewatered sludge cakes to a hauling truck. When the truck is loaded, the dewatered cake is hauled to the landfill site.

Assuming a dewatering capacity of 650 lbs/hr/m, the 2-meter sludge presses have the capacity to dewater 1,300 lbs of sludge per hour from 1 to 18 percent solids per cycle. Assuming that one belt filter press is out of service and assuming that three press cycles, - including start up and clean up, can be produced per 8-hour shift, one shift per day equates to a daily sludge dewatering capacity of approximately 3,900 lbs/day. This results in a daily biosolids production of approximately 1,400 lbs of sludge per mgd of influent flow. Based on this range of values, each press has the capacity to dewater sludge produced from an influent flow of 2.8 mgd.

Both belt filter presses are nearing the end of their service life. Consideration for new dewatering equipment is recommended with future expansion of the treatment facilities.

1.2.6 Support Facilities

1.2.6.1 Process Water Pumps

Two (2) submersible pumps are located in the upstream end of the Effluent Metering Structure and supply non-potable water (NPW) to various process areas and hose bibs across the plant site. Each pump is sized at a capacity of 250 gpm at 200 ft TDH to provide adequate water pressure at the various application points. The pumps were designed to operate without clogging of the pumps by debris and algae in the water, as experienced with the previous vertical turbine pumps previously installed in the former metering structure.

1.2.6.2 Plant Drainage Pumping Station

The Plant Drainage Pumping Station consists of one submersible pump located in a manhole on the north side of the WWTF site. The Aeration Basin and Final Clarifiers drain into the manhole as well as drainage from the Operations Building and decant from the sludge holding basins. The collected drain flow is then pumped to the influent channel of the Headworks for treatment.

1.2.7 Age and Condition of Existing WWTF Components

With the exception of the UV Disinfection Facilities and Effluent Flow Metering Structure, which were constructed in 2011, the current treatment facilities at the Belton WWTF were constructed in 1992. No other major expansions have taken place since that time. Typically, the service life of most equipment varies between 15 and 20 years, and the service life of well-maintained structures can extend over 50 years. Some of the equipment in the existing Headworks facility was relocated from the City's original wastewater plant, and has exceeded its useful life. However, the remaining facilities are 20 years old and in relatively fair condition. However, a more detailed structural evaluation may be required if existing structures are planned to be renovated, upgraded or used for purposes other than those originally designed.

1.3 Existing Wastewater Characteristics

The existing capacity of the Belton WWTF will be dependent on the wastewater characteristics, including flow rates and organic loadings, as well as the operating limits of some of the treatment processes. The wastewater contribution to the Belton WWTF is primarily from residential and commercial sources, with little contribution from industrial sources.

A summary of existing wastewater characteristics, as derived from historical plant data for years 2009 – 2011 are presented below.

1.3.1 Historical Wastewater Flows

Historical flow data is presented in Table 1.1 as Annual Average Day (AAD), Maximum Month Average Day (MMAD), and Peak Hour (PH) flows. PH flows shown in the table are based on the peaking factor previously established in a study by Wade & Associates in 2005. Since no infiltration and inflow (I/I) improvements have been implemented since this study, it is assumed that this peaking factor is still realized at the WWTF.

Table 1.1	Historical Influent Flow Rates and Ratios WWTF Flow Projections & Project Phasing TM City of Belton, Missouri				
Year	**AAD Flow (mgd)**	**MMAD Flow (mgd)**	**PH Flow (mgd)**	**MMAD:AAD Ratio**	**PH:AAD Ratio**
2008	1.77	2.79	16.35	1.57	9.24
2009	1.47	2.41	13.58	1.64	9.24
2010	1.66	3.11	15.34	1.87	9.24
2011	1.02	1.85	9.42	1.81	9.24
			Average	**1.72**	**9.24**

In comparison with values presented in the 2006 WWTF Facility Plan (Table 4.6, Belton WWTF Facility Plan, *Carollo Engineers, 2006*), the average MMAD:AAD and PH:AAD ratios are similar to historical flow data at the WWTF for years 2000 – 2005. However, it can be noticed that the flows for 2011 were unusually low due to the dry weather experienced in the Midwest during this year.

1.3.2 Historical Wastewater Characteristics

The summary tables in the following sections were developed for the time period between the years 2008 to 2011.

1.3.2.1 Biochemical Oxygen Demand (BOD_5)

One factor defining wastewater strength is its five-day biochemical oxygen demand, or BOD_5. BOD_5 is described as the amount of oxygen required over a five-day period at 20°C by bacteria while stabilizing decomposable organic matter under aerobic conditions.

For purposes of estimating future BOD_5 loadings (as well as other important mass loadings), it is important to establish BOD_5 parameters that are representative of expected operating conditions so that unit processes are not sized for the rarely occurring peak conditions. As with flow, these key parameters are based on the AAD and MMAD conditions. Mass loadings of BOD_5 provide a more accurate representation of the total contribution to the plant and are used in lieu of concentrations to evaluate process capacities.

Table 1.2 summarizes the influent AAD and MMAD BOD_5 loadings for the years 2008 through 2011. The ratio between the MMAD and AAD values is used in estimating future design conditions. As shown below the historical average MMAD:AAD ratio over this period is 1.45. This is very similar to previously established historical values for BOD_5 at the WWTF.

Table 1.2	Historical Influent BOD_5 Loadings and Ratios WWTF Flow Projections & Project Phasing TM City of Belton, Missouri			
Year	**AAD BOD_5 (mg/L)**	**AAD BOD_5 (lbs/day)**	**MMAD BOD_5 (lbs/day)**	**MMAD:AAD Ratio**
2008	108	1,502	1,951	1.30
2009	107	1,323	1,690	1.28
2010	113	1,326	2,082	1.57
2011	149	1,085	1,774	1.64
			Average	**1.45**

1.3.2.2 Total Suspended Solids (TSS)

Total suspended solids (TSS) is another parameter that is used to define the influent wastewater strength. It is a measure of the amount of suspended material in the influent wastewater. As with BOD_5, TSS mass loading rates provide a more accurate representation of the total TSS contribution to the plant. The AAD and MMAD TSS loadings for the years 2008 through 2011 are summarized in Table 1.3. The historical average MMAD:AAD TSS ratio over this period is 1.74.

Table 1.3	Historical Influent TSS Loadings and Ratios WWTF Flow Projections & Project Phasing TM City of Belton, Missouri			
Year	**AAD TSS (mg/L)**	**AAD TSS (lbs/day)**	**MMAD TSS (lbs/day)**	**MMAD:AAD Ratio**
2008	133	1,726	3,041	1.76
2009	182	2,205	3,154	1.43
2010	179	2,211	4,155	1.88
2011	186	1,318	2,475	1.88
			Average	**1.74**

1.3.2.3 Ammonia (NH_3)

Another parameter used to characterize the strength of wastewater is nitrogen content. Nitrogen can be found in many different forms such as ammonia, organic nitrogen, nitrate, and nitrite. Total Kjehldahl Nitrogen (TKN) is the combination of organic nitrogen and ammonia in wastewater. Organic nitrogen is usually much higher in influent samples than effluent samples. In most domestic wastewater facilities, the biological activity breaks down the organic matter releasing and/or consuming the nitrogen as energy in the process. The TKN analysis is an indictor of the nitrification process efficiency.

Of these parameters, only ammonia concentration values are available for review because this is the nitrogen constituent that is currently regulated by the plant effluent discharge permit. However, influent ammonia concentrations are not currently measured at the

WWTF. Therefore, an ammonia concentration of 30 mg/L and an ammonia MMAD:AAD loading ratio of 1.20, - which are typical values for a residential/commercial type wastewater, will be used as design parameters to evaluate existing and size future treatment units, as necessary.

1.4 Current Operating Conditions

Based on an average of the historical wastewater characteristics measured at the Belton WWTF, and assumptions regarding some of the performance factors and operation of the existing facilities, the following tables provide a summary of the estimated performance of the existing treatment processes in comparison with typical design values for similar facilities or processes.

1.4.1 Preliminary Treatment

As previously mentioned, primary treatment at the Belton WWTF includes grit removal, as well as solids grinding and bypass screening. Of these, only the grit removal system was evaluated for performance. Detention time, or hydraulic residence time (HRT) and velocities through the grit chamber were estimated based on average PH flow and design of the basin as interpreted from existing design drawings. Estimated performance of the grit removal process is shown Table 1.4 below.

Table 1.4 Grit Removal Basin – Existing Operating Conditions
WWTF Flow Projections & Project Phasing TM
City of Belton, Missouri

Parameter	Estimated Value(1)	Suggested Design Values
PH Flow, mgd	15.34	-
Surface Area, sq. ft.	210	-
Side Water Depth, ft	2.9	-
Volume, gal.	4,555	-
HRT, seconds	30.6	45 - 90
Settling Velocity, ft/min	5.72	3.2 – 4.2
Horizontal Velocity, ft/sec	1.38	0.8 – 1.3

Notes:
(1) Based on PH flows.

1.4.2 Secondary Treatment

The activated sludge process, which is the liquid treatment process used at the Belton WWTF, is affected by several factors, including: hydraulic detention time (HRT); hydraulic loading; organic loading; aeration capacity; mean cell-residence time (MCRT); food/microorganisms ratio (F/M); RAS recirculation rate; and environmental factors, such as barometric pressure and temperature.

As previously mentioned, the secondary treatment process consists of the Aeration Basin followed by the Clarifiers. Estimated operating values for the Aeration Basin, as shown in Table 1.5, are based on a historical data recorded at the WWTF at average (MMAD) flow conditions.

Table 1.5 Aeration Basin – Existing Operating Conditions
WWTF Flow Projections & Project Phasing TM
City of Belton, Missouri

Parameter	Estimated Value	Suggested Design Values
MMAD Flow (mgd)	2.86	-
Number of Basins	1	-
Volume (MG)	1.42	-
Volume (ft^3)	189,710	-
BOD Loading (lb / day)	3,071	-
Unit Loading (lb BOD_5 / 1,000 ft^3 / day)	16.2	15 (maximum)
F:M (1,2)	0.11	0.05 - 0.10
HRT (hours)	11.9	18 - 36
SRT (2) (days)	13.1	20 - 30

Notes:
(1) Based on an MLSS concentration of 3,151 mg/L.
(2) Based on an assumed MLVSS:MLSS ratio of 0.75.

As shown in the table, several operating parameters of the Aeration Basin are outside of the recommended design limits. Performance and capacity of the extended aeration process could be optimized by adjusting these parameters accordingly.

Existing operating parameters for the Secondary Clarifiers (Table 1.6) are based on historical average and PH design characteristics.

Table 1.6	Secondary Clarifiers – Existing Operating Conditions WWTF Flow Projections & Project Phasing TM City of Belton, Missouri	
Parameter	**Estimated Value**	**Suggested Design Values**
Number of Clarifiers	2	
Diameter each (ft)	70	
MMAD Flow (mgd)	2.86	-
PH Flow (mgd)	15.34	-
Overflow rate, MMAD (gpd/sq ft)	371(3)	400 (2)
Overflow rate, PH (gpd/sq ft)	1,329(3)	1,000 max (1)
Weir Loading, MMAD (gpd/ft)	6,492(3)	15,000 (2)
Solids Loading, MMAD (lb/d/sq ft)	18.2(3)	50 max (1) 24 - 35 max (2)

Notes:
(1) Missouri DNR Criteria.
(2)Generally accepted design values.
(3)Assumes one clarifier is out of service at MMAD conditions. At PH flow, all clarifiers are in service.

During projected PH flows, the overflow rate to the clarifiers is estimated to in excess of the recommended design value for this parameter. A high overflow rate indicates that too much flow is being pushed through the basin, which reduces the capability of the activated sludge to settle out effectively and often forces solids to flow out over the effluent weirs.

2.0 PROJECTIONS AND EXISTING CAPACITY

2.1 Planning Period

For this technical memo, a 20-year planning period is appropriate. Based on the development of this memo in 2012, the end of the planning period for this project will be in the year 2032.

2.2 Service Area

Prior to development of this technical memo, another consultant worked with the City to develop a model of the existing collection system and service area boundaries that could be used to estimate flows to the WWTF. Figure 2.1 shows the various service area boundaries within the Kansas City Metropolitan area and the projected future boundaries, including those tributary to the Belton WWTF.

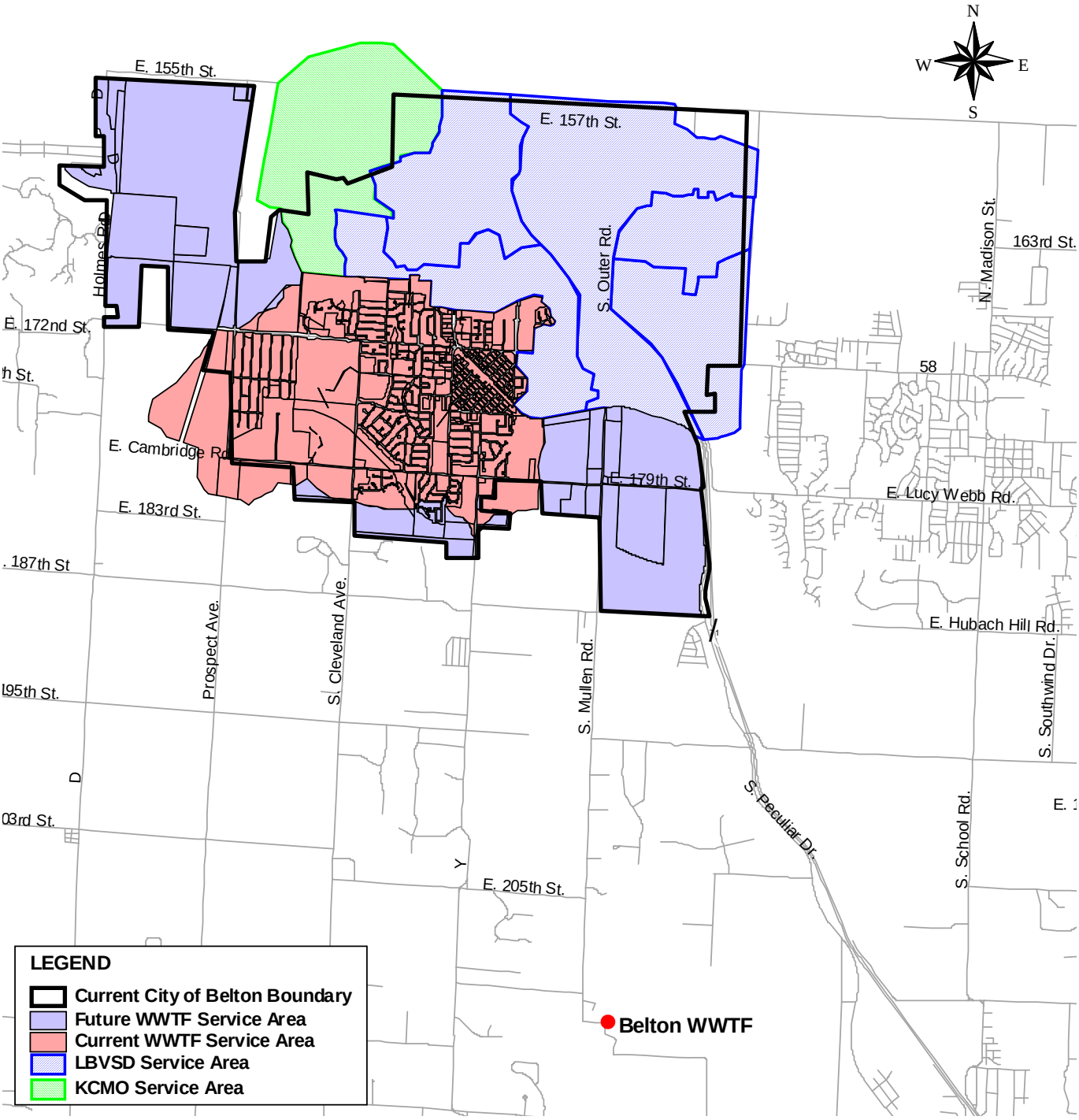

FIGURE 2.1 – SERVICE AREAS
BELTON WWTF FACILITY PLAN
CITY OF BELTON, MISSOURI

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2.3 Population Projections

Based on previous evaluations by the City, the population for the City of Belton is expected to grow gradually at a rate of 1% per year throughout the planning period, - including the Stiles retirement village proposed for development on the west end of the City. Based on current land use for the City, areas available for growth and development are on the west and south sides of the City area. For this reason, it is anticipated that future growth in the City will be evident in these areas. Thus, it is assumed that all future growth will occur within the WWTF service area.

Based on land use parcels currently within the WWTF service area and the City population from the 2010 U.S. Census, the percentage of the City's population serviced by the WWTF in 2010 was estimated to be 43.3%. The service area population is then expected to increase in direct correlation with the 1% annual growth rate for the City.

Table 2.1 shows the City of Belton's estimated population growth projections, as well as those projected for the WWTF service area.

Table 2.1 Estimated Population Projections WWTF Flow Projections & Project Phasing TM City of Belton, Missouri			
Year	**Belton Population (1)**	**Percent Served by Belton WWTF**	**Population Served by Belton WWTF (2)**
2010	23,116	43.3%	10,009
2011	23,347	43.9%	10,240
2012	23,581	44.4%	10,474
2013	23,816	45.0%	10,710
2014	24,055	45.5%	10,948
2015	24,295	46.1%	11,188
2016	24,538	46.6%	11,431
2017	24,783	47.1%	11,677
2018	25,031	47.6%	11,925
2019	25,282	48.2%	12,175
2020	25,534	48.7%	12,428
2021	25,790	49.2%	12,683
2022	26,048	49.7%	12,941
2023	26,308	50.2%	13,201
2024	26,571	50.7%	13,464
2025	26,837	51.2%	13,730
2026	27,105	51.6%	13,999
2027	27,376	52.1%	14,270
2028	27,650	52.6%	14,543
2029	27,927	53.1%	14,820
2030	28,206	53.5%	15,099
2031	28,488	54.0%	15,381
2032	28,773	54.4%	15,666

Notes:

(1) Population based on data from the 2010 U.S. Census, and annual growth rates as established by City Staff of 1%/year.

(2) Initial service area population established by review of land parcels in the service area for 2010. Projections based on input from City staff with increase directly proportional to increase in City population.

2.4 Flow and Loading Projections

2.4.1 Flow Projections

As discussed above, the estimated population served by the Belton WWTF is expected to increase from 10,009 in the year 2010 to approximately 15,666 at the end of the planning period in the year 2032. The projected dry weather, or annual average day (AAD), flow rates were determined by applying the per capita flow rate (gallons of flow contributed per person per day) estimated of the population served by Belton WWTF in the year 2010 to the year 2012 dry weather flow. (Flow data for the year 2011 was not used because of the unusually dry weather year and low flow values.) Based on historical flow data and estimated population for 2010, the per capita flow rate for the Belton WWTF service area is

estimated to be 166 gal/capita/day. This value is consistent with textbook design values for per capita flow rates (165 gal/c/d, *Metcalf & Eddy, Tbl 2*-1), which includes allowances for domestic, as well as nondomestic, public service, and unaccounted for losses and I/I. However, this value is higher than the average per capita flow of 100 gal/c/d as used by Missouri DNR. This can be attributed to the excess I/I currently in the collection system for the Belton WWTF.

Table 2.2 summarizes the estimated dry weather (AAD) flow rates expected at the WWTF during each year of the planning period. As shown in the table, the year 2032 estimated AAD flow rate is approximately 2.60 mgd.

These AAD flow rates can then be used to estimate maximum month average day (MMAD) and PH (peak hour) flow rates. In 2005, Wade & Associates, Inc. estimated an existing peak flow to the plant of 13.95 mgd. Based on the recorded plant flow for 2005, this resulted in a wet weather peaking factor (PH:AAD) of 9.24 for 2005. Since no improvements in the collection system had been reported by the City to reduce a significant amount of the I/I since the study by Wade & Associates in 2005, it is assumed that this peaking factor still holds true for 2010. In addition, the historical MMAD:AAD ratios, based on 2008 – 2011 flow data, were averaged to be 1.72. Both of these peaking factors are high in comparison to typical peaking factors for treatment plants of this size, and are likely associated with the high I/I rates experienced in the collection system. Typical MMAD:AAD peaking factors range from 1.25 to 1.33, while typical PH:AAD peaking factors can be determined from an equation used by Missouri DNR to estimate this peaking factor based on population. However, based on a study performed by others on behalf of the City, I/I reduction improvements are planned for implementation beginning in 2012 through 2017, and are anticipated to reduce I/I by 30%. Based on this assumption, it is assumed that the PH:AAD peaking factor will decrease gradually by 30%, to 6.47, through the year 2017, and will sustain at this value throughout the duration of the planning period.

To provide a conservative basis for design, MMAD:AAD and PH:AAD peaking factors of 1.33 and 6.47, respectively, are recommended. These peaking factors should be realized by the year 2017 when improvements to the collection system are anticipated to be complete. A linear reduction from the current values to these values can be used to estimate flows during the time period when collection system improvements are occurring. Using these factors and the projected AAD flow rates, MMAD and PH flow rates can be estimated. Table 2.2 provides a tabular summary of the results, while Figure 2.2 presents the results graphically.

Table 2.2 Flow Projections
WWTF Flow Projections & Project Phasing TM
City of Belton, Missouri

Year	Population Served by WWTF (1)	AAD Flow (2)	MMAD Flow (3)	PH Flow (4)
2010	10,009	1.66	2.86	15.34
2011(2)	10,240	--	--	--
2012	10,474	1.74	2.89	16.05
2013	10,710	1.78	2.86	15.42
2014	10,948	1.82	2.82	14.76
2015	11,188	1.86	2.78	14.06
2016	11,431	1.90	2.73	13.31
2017	11,677	1.94	2.58	12.52
2018	11,925	1.98	2.63	12.79
2019	12,175	2.02	2.69	13.06
2020	12,428	2.06	2.74	13.33
2021	12,683	2.10	2.80	13.60
2022	12,941	2.15	2.85	13.88
2023	13,201	2.19	2.91	14.16
2024	13,464	2.23	2.97	14.44
2025	13,730	2.28	3.03	14.73
2026	13,999	2.32	3.09	15.01
2027	14,270	2.37	3.15	15.30
2028	14,543	2.41	3.21	15.60
2029	14,820	2.46	3.27	15.89
2030	15,099	2.50	3.33	16.19
2031	15,381	2.55	3.39	16.50
2032	15,666	2.60	3.46	16.80

Notes:

(1) Population based on input from City Staff for an annual increase in population of 1%. WWTF service area population is anticipated to increase in direct correlation with population increase for the City.

(2) Years 2010 and 2011 dry weather flow based on historical data. However, flow data for Year 2011 was unusually low due to extreme dry weather conditions. For this reason, subsequent flow values were determined by applying per capita flow rate of 166 gal/capita/day to the population served by Belton WWTF to the Year 2010 dry weather flow.

(3) Historical MMAD:AAD flow peaking factor of 1.72 used to estimate Years 2010 and 2011 MMAD flows. Subsequent values based on linear reduction of MMAD:AAD peaking factor to 1.33 in Year 2017 to coincide with anticipated I/I reductions.

(4) Year 2010 wet weather peaking factor of 9.24 previously derived from data provided by Wade & Associates, Inc (2005). Subsequent values based on linear reduction of wet weather peaking factor to 6.47 in Year 2017 to coincide with anticipated I/I reductions of 30%.

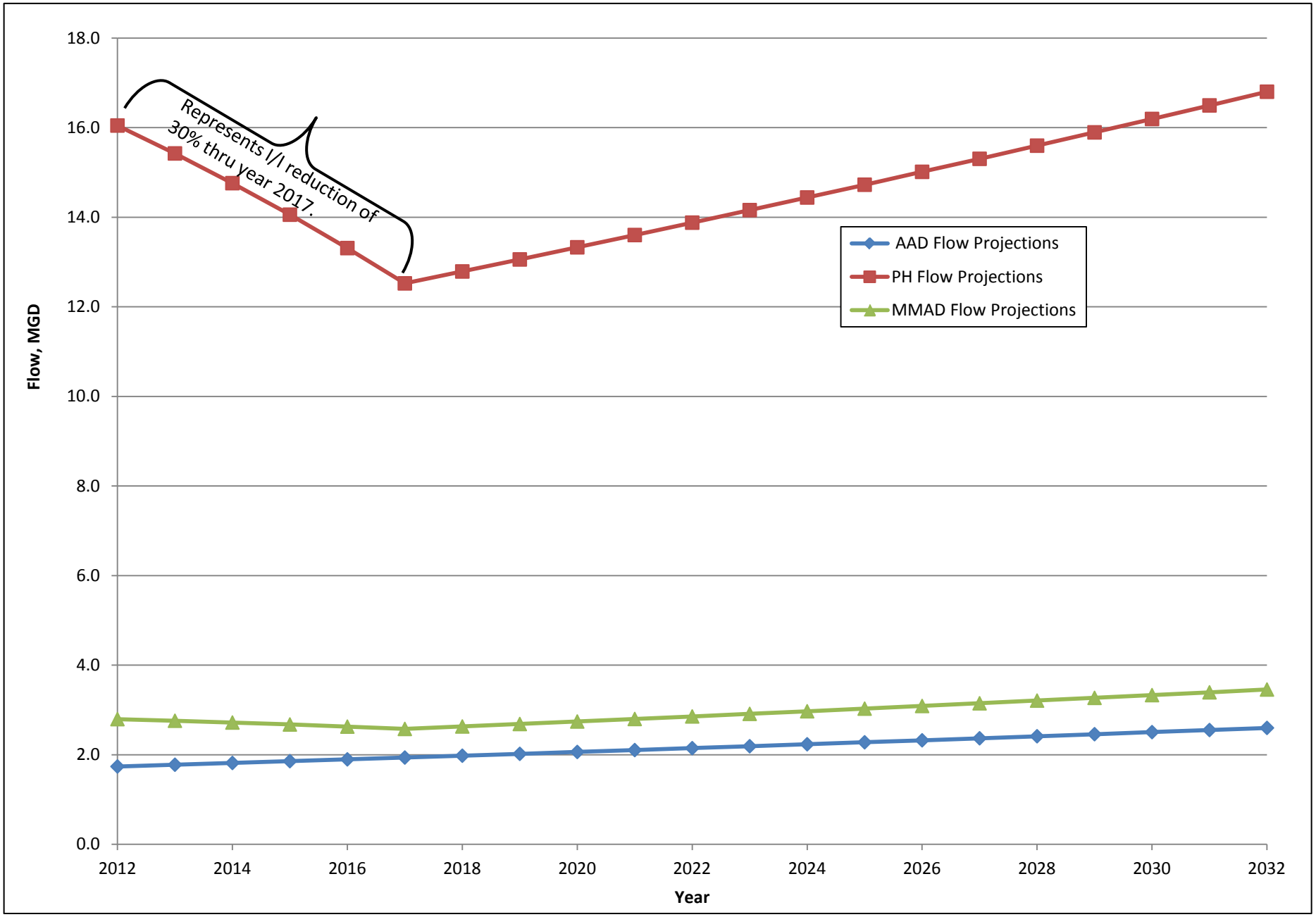

FIGURE 2.2 – WWTF FLOW PROJECTIONS

WWTF FLOW PROJECTIONS AND PROJECT PHASING TM
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2.4.2 Projected Influent Loadings

As discussed in Section 1.3.2, wastewater strength is generally defined by its five-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), and nitrogen content.

2.4.2.1 BOD_5 Loadings

Missouri DNR guidelines state that the minimum design loading factor for residential single-family dwelling units that include grinders, or garbage disposals, shall be at least 0.22 lbs BOD_5/capita/day. Based on an average daily wastewater flow factor of 166 gal/capita/day, the resulting average BOD_5 concentration is approximately 159 mg/L. Using this concentration and a year 2026 AAD flow rate of 2.60 mgd, the estimated year 2026 influent BOD_5 loading is approximately 3,446 lbs BOD_5 /day.

As shown in Table 1.2, the historical average MMAD:AAD loading ratio is 1.45. This ratio is consistent with historical MMAD:AAD loading ratios for BOD for years 2000 – 2005 (refer to Belton WWTF Facility Plan, *Carollo, 2010*). Although typical values for similar treatment facilities in the Midwest are approximately 1.25, historical data shows that this ratio of 1.45 is actually higher for the Belton facility and will be used to determine the MMAD BOD_5 loading value and associated MMAD BOD_5 concentration for the planning period.

Applying the MMAD:AAD ratio of 1.45 results in a year 2032 MMAD BOD_5 loading of approximately 4,998 lbs BOD_5 /day. Based on the MMAD flow of 3.46 mgd determined earlier, the estimated MMAD BOD_5 design concentration is approximately 173 mg/L.

2.4.2.2 TSS Loadings

Missouri DNR guidelines state that the minimum design loading factor for residential single-family dwelling units shall be at least 0.20 lbs TSS/capita/day. Based on an average daily wastewater flow factor of 166 gal/capita/day, the resulting average TSS concentration is approximately 144 mg/L. Using this concentration and a year 2026 AAD flow rate of 2.60 mgd, the estimated year 2026 TSS loading rate is approximately 3,122 lbs TSS/day.

However, this value is considerably lower than what has historically been measured in the influent at the WWTF. For this reason, average historical values of 220 mg/L for TSS concentration will be assumed. Using this concentration and a year 2026 AAD flow rate of 2.60 mgd, the estimated year 2026 AAD influent TSS loading is approximately 4,770 lbs TSS /day. (This equates to 0.30 lbs TSS/capita/day.)

The historical average MMAD:AAD loading ratio is 1.74, as shown in Table 1.3. Although this is considerably higher than the typical value for similar treatment facilities in the Midwest whose MMAD:AAD loading ratio for TSS is typically 1.25, historical data at the Belton facility has consistently been measured at a value greater than this. Therefore, an average value of 1.50, between the typical and measured value, will be used to determine the MMAD TSS loading value and associated MMAD TSS concentration.

Applying the MMAD:AAD ratio of 1.50 results in a year 2026 MMAD TSS loading of approximately 7,155 lbs TSS/day. Based on the MMAD flow of 3.46 mgd determined earlier, the estimated MMAD TSS design concentration is approximately 248 mg/L.

2.4.3 Ammonia Loading

As previously stated, no historical influent ammonia measurements are available for the Belton WWTF. For the purpose of this evaluation, an average ammonia concentration of 30 mg/L and an MMAD:AAD loading factor of 1.20, - typical values for a residential/commercial type wastewater, will be used as design parameters to evaluate and size the treatment units.

Based on these design criteria, the AAD NH_3-N loading for the year 2032 is projected to be 651 lbs/day and the MMAD NH_3-N loading for the year 2032 is calculated as 781 lbs/day. Based on the MMAD flow of 3.46 mgd determined earlier, the estimated MMAD NH_3-N design concentration is approximately 27 mg/L.

2.4.4 Proposed MMAD Design Criteria

For average design conditions, MMAD flow and loadings are generally used to ensure the facilities have the capacity to handle average flows up to the maximum flow/demand for each month. For some processes, it is necessary to design to meet PH flows and loadings.

MMAD design flow and loadings for the design year 2032 are outlined in Table 2.3. The loadings are calculated using the MMAD concentrations as previously identified for the flow. Total nitrogen (TN) and total phosphorus (TP) values are listed as typical values for domestic wastewater. It is recommended that future influent testing be conducted to confirm actual TN and TP values in the influent stream.

Table 2.3 Design Year Criteria - Maximum Month Average Day (MMAD)
WWTF Flow Projections & Project Phasing TM
City of Belton, Missouri

Parameter	Concentration	Loading/Flow
Flow	N/A	3.46 mgd
BOD_5	173 mg/L	4,998 lbs/day
TSS	136 mg/L	3,916 lbs/day
NH_3-N	27 mg/L	781 lbs/day
TN (1)	40 mg/L	1,154 lbs/day
TP (1)	8 mg/L	231 lbs/day

Notes:
(1) Adapted from Table 3-16, *Wastewater Engineering Treatment, Disposal, and Reuse,* Metcalf & Eddy, 3rd edition.

2.5 WWTF Effluent Limits

The City of Belton was issued an updated State Operating Permit (MO-0117412) from the Missouri Department of Natural Resources (DNR) for operation of the Belton WWTF in February 2012. A summary of the discharge requirements listed in the permit is presented in Table 2.4. This permit is scheduled to expire on February 20, 2017.

Table 2.4 Plant Effluent Discharge Limits - Outfall #001
WWTF Flow Projections & Project Phasing TM
City of Belton, Missouri

		Effluent Limitations[1]		
Effluent Parameter	**Unit**	**Daily Maximum**	**Weekly Average**	**Monthly Average**
BOD_5 [2]	mg/L			
(Apr 1 - Sep 30)		20	--	20
(Oct 1 - Mar 31)		25	--	25
TSS [2]	mg/L	45	--	30
Ammonia as N	mg/L			
(Apr 1 - Sep 30)		6.1	--	1.2
(Oct 1 - Mar 31)[3]		--	--	--
pH	Standard Units	6 to 9	--	--
Oil and Grease	mg/L	15	--	10
E. coli	#/100 mL	-	1,030	206

Notes:
(1) Whole Effluent Toxicity (WET) testing is also required on an annual basis.
(2) A minimum removal efficiency of 85% is required for these parameters.
(3) Monitoring only is required.

2.6 Anticipated Discharge Permit Limits

Missouri DNR is currently evaluating additional regulations that could affect the discharge limits listed in the current operating permit. Although changes are typically implemented only during permit renewals, changes to the permit can be made at any time. The following section discusses regulations under development and existing regulations that could affect future permit limits.

2.6.1 Nutrient Limits

Although regulated standards have not yet been established for total nitrogen (TN) and total phosphorus (TP) limits, it is anticipated that new regulations on these nutrients could be established within the next few years. In fact, in the State of Kansas, KDHE has established TN and TP effluent limits of 8 and 1.5 mg/L, respectively, and it is likely that similar requirements will be established for the State of Missouri.

Missouri DNR is currently in the process of collecting and analyzing data to develop the nutrient criteria, - including nitrogen and phosphorus limits, for all water bodies in the state of Missouri. Once established, the regulated limits will be required to be achieved at the effluent discharge location into the water body (i.e., “end-of-pipe”) for facilities that discharge into a Class C stream, such as East Creek. Therefore, it is recommended that any planned expansion of the WWTF include provisions for TN and TP removal.

2.6.2 Dissolved Oxygen

For streams where protection of aquatic life is designated, such as in East Creek, the water quality standards indicate that water contaminants shall not cause the dissolved oxygen in the stream to be lower than 5 mg/L. [Ref 10 CSR 20-7.031(4)(J) and 10 CSR 20-7.031, Table A]. As with nitrogen and phosphorus limits, this dissolved oxygen limit must be achieved at the effluent discharge location into the water body (i.e., “end-of-pipe”) for facilities that discharge into a Class C stream. Although the current operating permit does not include dissolved oxygen limits, this effluent limit could be added to the operating permit at any time.

3.0 RECOMMENDATIONS & PROJECT PHASING

Now that the design criteria has been established for the projected flows and loadings to the WWTF, the operating capacity of the existing facilities were evaluated to determine if/when expansion of the facilities will be required to maintain operating capacity through the end of the planning period.

3.1 Existing Treatment Capacity

As previously mentioned, the different unit operations in the activated sludge process are impacted by different variations in flowrate and mass loading. Thus, each process is designed based on a specific design factor that is critical to its successful performance. Table 3.1 lists the different unit processes at the Belton WWTF and the respective critical design factor and criteria used to size the process.

Table 3.1 Unit Process Sizing Criteria WWTF Flow Projections & Project Phasing TM City of Belton, Missouri		
Unit Operation/Process	**Critical Design Factor(1)**	**Sizing Criteria(1)**
Influent Pumping	PH Flow	Flowrate
Influent Screening	PH Flow/Min. Flow	Flowrate/Channel Approach Velocity
Grit Removal	PH Flow	Detention Time, Settling Velocity & Horizontal Velocity
Activated Sludge (Aeration Basins)	MMAD Flow & Organic Loading	HRT & F:M
Secondary Clarification	PH Flow & Organic Loading	Surface Overflow Rate & Solids Loading Rate
Notes: (1) From Table 5-10, *Metcalf & Eddy*.		

3.1.1 Treatment Capacity Based on Recommended Values

As noted in Section 1, several of the current operating conditions for the treatment units are outside of the recommended range for process operation, particularly those of the biological treatment units. Some of these operating parameters, such as BOD_5 or TSS concentrations and velocity, are based on the characteristics of the wastewater or the design of the treatment unit, and are factors over which the plant operators would have little to no control. However, operating parameters such as loading rates, F:M ratios, and retention time, are factors that can be modified by an adjustment in one or more operating parameters.

Table 3.2 shows the existing, suggested, and recommended parameters for the biological treatment process at the Belton WWTF. The current and suggested values are as presented in Section 1, and are based on historical data, and regulatory requirements and text book values. The recommended values listed are values that would be used for the design of new or expansion facilities for the WWTF.

Table 3.2	**Summary of Existing & Recommended Operating Conditions WWTF Flow Projections & Project Phasing TM City of Belton, Missouri**		
Parameter	**Existing Value(1)**	**Suggested Values**	**Recommended Design Values**
MLSS, mg/L	3,151	3,000 – 5,000	3,000
Organic Loading, lbs BOD_5/1000 cf/d	16.2	15	16(3)
F:M Ratio, (3) lbs BOD_5/lbs MLVSS/d	0.11	0.05 – 0.10	0.10
SRT, days	13.1	20 – 30	13(3)
HRT, hours	11.9	18 – 36	12
Surface Overflow Rate, gpd/ft^2 (2)	216 / 1,329	400 / 1,000 (max)	400 / 1,000 (max)
Peak Solids Loading Rate, lb/d/ft^2 (2)	14.2 / 40.6	24 – 50, max	24 / 35
Weir Loading Rate, gpd/lf (2)	3,774 / 23,249	10,000 – 15,000	10,000 / 15,000

Notes:

(1) Based on historical data from 2008 - 2011.

(2) At AAD/PH flow conditions.

(3) Although outside of the suggested range, Plant Staff has been able to operate effectively under this condition. However, a value within the suggested range is recommended for design of new facilities.

The Aeration Basin is operated under conditions that are outside of the range of recommended design values. However, if the values for organic loading, F:M ratio, HRT and SRT were adjusted to the fall within the range of recommended design values, treatment capacity would be reduced in the Aeration Basin. Although these values are currently exceeded, Plant Staff has been able to maintain effective operation of the facilities while still meeting permit effluent limits. Thus, no modifications to the operation of the Aeration Basin are suggested, and a slight variation of the current operating values will be used in the assessment of the existing treatment capacity, as well as the current measured value for MLSS.

Contrarily, modifications to the manner in which the Secondary Clarifiers are operated could result in an increase in treatment capacity. The Secondary Clarifiers are currently operated below their potential capacity during average flow conditions. Recommended design values for solids loading and surface overflow rate for secondary clarifiers under average operating conditions are 24 lb/d/ft^2 and 400 gpd/ft^2, respectively. Under peak flow conditions, the recommended values are 35 lb/d/ft^2 and 1,000 gpd/ft^2. Although DNR allows for a maximum of 50 lb/d/ft^2 for solids loading, it is not recommended that the 1,000 gpd/ft^2 be exceeded for

surface overflow. Pushing the Clarifiers beyond this value could potentially result in an overflow of solids out of the Clarifiers. The recommended design values will be used in the assessment of the existing treatment capacity of the Clarifiers under average flow conditions.

3.1.2 Grit Removal Process Capacity

The capacity of the gravity grit removal process is based on peak hour flow. According to published data, the capacity of this type of system is based on three main criteria: detention time, settling velocity, and horizontal velocity. A typical range for detention time is 45 - 90 seconds. Likewise, the typical range for settling velocity and horizontal velocity are 3.2 - 4.2 ft/min and 0.8 -1.3 ft/sec, respectively. The settling velocity values assume 100-percent removal of material passing a 65-mesh screen (a screen having 65 openings per lineal inch to capture smaller grit particles). Assuming 1 ft/sec for the horizontal velocity, 3.8 ft/min for the settling velocity, and 60 sec of detention time, the minimum allowable peak flow capacity of the grit removal system is estimated to be 6.59 mgd (limited by the detention time). At this flow rate the hydraulic detention time is approximately 60 seconds, the settling velocity is 2.91 ft/min, and the horizontal velocity is 0.70 ft/s.

3.1.3 Extended Aeration Process Capacity

The biological treatment capacity of an aeration basin defines its ability to biologically treat the pollutant loads in the wastewater, and is primarily a function of the biomass inventory in the aeration basins.

To determine the biological treatment capacity of the existing Aeration Basin, the current operating conditions used by Plant Staff to operate the basin were used rather than the values that would be used for the design of new facilities. These parameters are shown in Table 3.3 below. As shown in this table, the estimated biological treatment capacity of the Aeration Basin is controlled by F:M, which is the limiting factor for evaluation of the Aeration Basin. Based on the current F:M ratio of the existing treatment stream, the Aeration Basin has a rated capacity of approximately 4.47 mgd when operated under the specified conditions, as described in the table.

Table 3.3	**Extended Aeration Treatment Capacity Evaluation WWTF Flow Projections & Project Phasing TM City of Belton, Missouri**
Parameter	**Value**
MLSS	3,151 mg/L
MLVSS:MLSS ratio	0.75
Sludge Yield, Observed	0.78 lbs MLSS / lb BOD_5
Maximum Allowable Flow Rate(1)	
Based on Organic Loading (2)	4.85 mgd
Based on F:M Ratio (3)	4.47 mgd
Based on SRT (4)	5.87 mgd
Based on HRT (5)	4.88 mgd

Notes:

(1) Assumes all three channels of basin in service.

(2) Based on current organic loading rate of 16 lbs BOD_5/1,000 ft^3/day.

(3) Assumes current F:M ratio of 0.11 lbs BOD_5/lbs MLVSS/day; based on 75% volatile solids in the total solids.

(4) Assumes current SRT of 13 days; based on current MLSS concentration of 3,151 mg/L, volume of 2 inner channels of the Aeration Basin, and an assumption of 1,000 lbs/d of solids produced.

(5) Assumes current HRT of 12 hours.

However, when a new treatment facility is designed a more conservative approach is taken and Missouri DNR guidelines for extended aeration are followed. These guidelines recommend a maximum design organic loading rate of 15 lbs BOD_5/1,000 ft^3/day and a design F:M ratio of 0.05 to 0.10 lbs BOD_5/lbs MLVSS/day. Additional generally accepted design criteria, including a solids retention time (SRT) of 20 to 30 days and a hydraulic residence time (HRT) of 18 to 36 hours, are also used. If these values were used, the capacity of the existing Aeration Basin would be less than those shown in Table 3.3.

3.1.4 Clarifier Treatment Capacity

To determine the treatment capacity of the existing secondary clarifiers, recommended design operating parameters were assumed for the average day conditions. At peak flow conditions, although outside of the recommended range, the existing operating parameters were assumed. Table 3.4 summarizes the results of the evaluation and other key assumptions used for the purpose of this analysis. As shown in this table, the estimated secondary clarifier treatment capacity is 2.81 mgd and is limited by the solids loading rate. At PH conditions, the treatment capacity is 13.27 mgd, also limited by solids loading rate.

Table 3.4	**Secondary Clarifier Capacity Evaluation WWTF Flow Projections & Project Phasing TM City of Belton, Missouri**	
Parameter	**AAD/MMAD Value**	**PH Value**
MLSS	3,151 mg/L	3,151 mg/L
Maximum Allowable Flow Rate (1)		
Based on Surface Overflow Rate (SOR)(2)	3.08 mgd	11.55 mgd
Based on Solids Loading Rate (SLR)(3)	2.81 mgd	13.23 mgd

Notes:

(1) Assumes flow is split equally between two (2) final clarifiers at AAD/MMAD flow conditions and three (3) final clarifiers at PH flow conditions.

(2) Based on an SOR of 400 gpd/ft^2 and 1,000 gpd/ft^2, at AAD and PH flow conditions, respectively.

(3) Based on SLR of 24 lb/d/ft^2 and 35 lb/d/ft^2, at AAD and PH flow conditions, respectively.

By increasing the loading rates to the clarifiers to the recommended design values, the limiting capacity to the Clarifiers under average design conditions is increased from 1.66 mgd to 2.81 mgd. Changing this criteria allows RAS to be increased and an increase in flow through each Clarifier.

3.1.5 Capacity Summary of the Liquid Treatment Processes

The liquid treatment processes at the Belton WWTF each have a different capacity, as shown in Table 3.5. The treatment process with the lowest capacity is the Secondary Clarifiers with an AAD capacity of 2.81 mgd.

The clarifiers are the limiting process at the WWTF, but are shown to have adequate process capacity to meet projected average flows beyond the planning period. However, the Clarifiers are limited to a capacity at peak hour flows of 11.55 mgd, and would not have enough available capacity to meet projected peak flows throughout the planning period. It should also be noted that hydraulic conditions might also limit the treatment processes below the “rated” capacities shown here.

Table 3.5	**Existing Treatment Process Capacities WWTF Flow Projections & Project Phasing TM City of Belton, Missouri**		
	Estimated Capacity (mgd)		
Parameter	**Average Conditions**	**Peak Conditions**	**Limiting Parameter**
Influent Pump Station (1)	-----	8.18	Equipment Capacity
Grit Removal	-----	6.59	Settling Velocity(4)
Grinding	-----	3.65	Equipment Capacity
Aeration Basin (2)	Existing – 4.47 Future – 3.33	-----	F:M(5)
Secondary Clarifiers(7)	2.81 (3)	11.55	Solids Loading/Surface Overflow(6)
Effluent Flume	-----	27.00	Throat Velocity
UV Disinfection Basin(8)	4.50	16.00	Equipment Capacity
RAS Pump Station(1)	6.94	--	Equipment Capacity
Sludge Holding Tanks	2.86	--	Storage Capacity

Notes:

(1) Assumes one of the larger pumps out-of-service.

(2) Assumes all three flow channels in service. Capacity of existing aeration basin decreases from 4.47 mgd to 3.33 mgd as the influent BOD_5 concentration increases from 129 mg/L to 173 mg/L due to reduced inflow/infiltration from collection system improvements.

(3) Assumes one of the clarifiers is out-of-service.

(4) Recommended settling velocity is 3.2 - 4.2 fps.

(5) It is assumed the Aeration Basin will be operated at an F:M of 0.10.

(6) Clarifiers are limited by solids loading under average conditions and surface overflow rate at peak conditions. Assumes the Clarifiers are operated at the existing SOR of 1,000 gpd/ft^2 and an SLR of 35 lb/d/ft^2 at peak flows.

(7) Assumes two clarifiers in service during AAD and MMAD flow conditions, and all three in service during PH flows.

(8) Assumes both UV channels in service. UV flow channels are expandable to accommodate future capacities up to 12.0 mgd per channel.

3.2 Recommended Improvements and Project Phasing

3.2.1 Projected Flows vs. Existing Capacity

Figure 3.1 presents a summary of projected flows and existing process capacity for the various process units. As depicted by the graph, the existing Secondary Clarifiers and Aeration Basin have adequate capacity to sustain projected average day flows throughout the planning period. There are, however, a several things to note about the existing process capacity.

First, as depicted by the graph, beyond the Year 2030, projected flows are expected to exceed the estimated capacity of the Aeration Basin. The additional basin capacity needed to meet the Year 2032 projected flow is less than 10% of the existing capacity. However, because Plant Staff is able to currently operate the basin above the recommended design values, if the basin continues to operate at an F:M of 0.11 (as shown by historical data) instead of the recommended 0.10, the maximum capacity of the basins would increase to 3.62 mgd (at a future BOD_5 concentration of 173 mg/L) and would provide adequate capacity through the planning period.

Secondly, as previously shown, the Secondary Clarifiers are limited first by the recommended surface overflow rate which limits the capacity of the Clarifiers to 11.55 mgd under peak flow conditions; second, a hydraulic bottleneck was discovered in the piping downstream of the Clarifiers that would cause the Clarifier effluent weirs to be submerged above flows of 12.88 mgd. At the recommended surface overflow rate of 1,000 gpd/ft^2 , the Clarifiers are limited to a capacity of 11.55 mgd. Although the Clarifiers may have been operated above this overflow rate in the past, it is recommended that the overflow rate be limited to the maximum recommended values to reduce the potential for washing solids out over the effluent weirs.

A preliminary hydraulic analysis performed on the existing WWTF showed that the piping directly downstream of the Clarifiers creates a hydraulic bottleneck at peak flows above 12.88 mgd. Above this flow, the effluent weirs on the Clarifiers become submerged. There are essentially three options to handling the hydraulic bottleneck: (1) operate the effluent weirs under submerged conditions during peak flow events; (2) replace the piping downstream of each Clarifier with larger diameter piping; or (3) construct a fourth clarifier basin to relieve some of the flow through the effluent piping of each of the existing Clarifiers. A wye-fitting with a blind flange was installed on the combined clarifier effluent piping, just upstream of the Effluent Metering Structure for tie-in of future clarifier basins.

Finally, another very important fact to consider in evaluation of existing plant capacity is compliance with the operating permit, which limits the rated capacity of the WWTF to 2.26 mgd. While the biological treatment process has the available capacity to handle flows greater than 2.26 mgd, Missouri DNR has only permitted the facility up to the 2.26 mgd. A request to re-rate the facility up to 2.81 mgd could be made to Missouri DNR to allow the plant to operate to this limit.

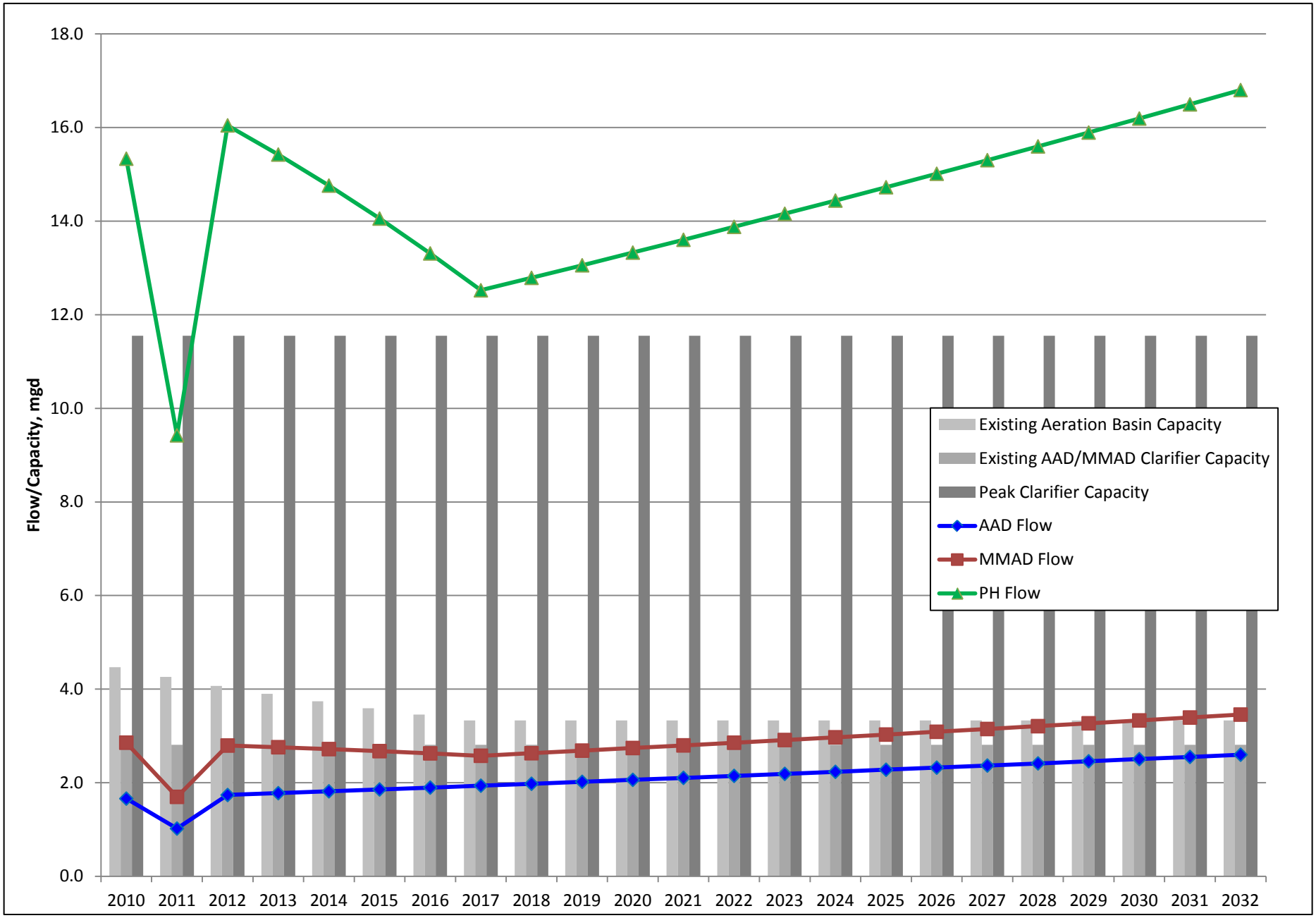

FIGURE 3.1 – PROJECTED FLOWS VS. PROCESS CAPACITIES

WWTF FLOW PROJECTIONS AND PROJECT PHASING TM

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3.2.2 Recommended Improvements

3.2.2.1 Parallel Influent Pump Station and Force Main

As previously mentioned, immediate improvements are recommended for the existing Influent Pump Station to increase pumping capacity to prevent overflows at the upstream manhole(s) during peak flow conditions. The recommended improvements to increase influent pumping capacity include the addition of a parallel pump station facility rated at 9.2 mgd (minimum) to provide supplemental capacity to the existing pump station, and a 20-inch diameter parallel force main to help convey peak flows to the WWTF while reducing headloss.

A building to house the electrical equipment above the 100-yr flood elevation is also recommended as part of the new pump station facility.

The addition of VFDs to the controls of the existing influent pumps is also recommended as part of the improvements. By adding VFDs on the existing pumps, the speed of the pumps can be adjusted as necessary to maintain a target water level in the wet well. This will allow the pumps to operate more efficiently based on actual incoming flow, and will help minimize the potential for the upstream manhole(s) to overflow.

3.2.2.2 New Headworks Facility

The preliminary treatment equipment in the existing Headworks facility has reached the end of its useful life, and it is recommended that the facility be replaced as part of the immediate improvements at the WWTF.

The Belton WWTF staff has observed that plastics and other undesirable material are often present in the influent following the grit removal and grinding operations. These materials, particularly the rags and plastics, are seen throughout downstream processes at the treatment plant. Not only is this material an eyesore, it creates extra maintenance for plant staff relating to clogging of pumps and material getting hung up on slide gates, sluice gates, and weirs. In an effort to reduce or eliminate this nuisance, a new Headworks facility with the following preliminary equipment is recommended.

The new Headworks facility would consist of at least two process flow channel including a bypass channel, for operational flexibility. An automatic step screen with 1/4-inch openings is recommended in each flow channel to remove the majority of the undesirable materials. A washer compactor is also recommended to liquefy the fecal material removed by the screens and separate it from the trash and rags, resulting in a relatively non-offensive waste product. The washed screenings are then compacted to remove additional liquids and organic material from the trash prior to being deposited into a dumpster. The washed and dewatered screenings would then be deposited at a landfill.

A bypass channel with a manual bar rack is also recommended for installation parallel to the bar screen flow channels to allow flow to bypass around the mechanical screens in the

event they are out of service. The manual bar rack would allow screenings to continue to be removed in bypass operation.

To provide a reliable means for removing the grit from the waste stream, new vortex-type grit removal facilities are proposed. Grit collected in the grit chambers will be pumped to grit separation and washing units. These units will remove putrescible organic material from the grit particles and return these organics back into the process flow stream while producing a dewatered grit with minimal odors. The dewatered grit will be collected in a dumpster for landfill disposal.

The 20-inch parallel force main system from the Influent Pump Stations will be directed to the new Headworks. Upstream of the bar screens, a Parshall flume is proposed to measure influent flows to the WWTF. This will allow for accurate measurement of peak flows into the plant so that WWTF Staff can make adjustments, as required, to the operating conditions based on influent flows.

In the interim, it is recommended that a temporary flow meter be installed in the existing manhole upstream of the Headworks facility to capture current influent flows to the WWTF for a more accurate representation of flow trends to the plant. It may be possible to install a clamp-on ultrasonic flow meter on the influent force main in the former metering manhole upstream of the Headworks. For an accurate representation of actual flow trends, flow data should be collected for a minimum of two-years, though a one-year cycle is acceptable. The recorded flow trend data will allow the City to evaluate the diurnal pattern of flow actually being conveyed to the WWTF by the existing influent pumps.

The new screening, grit removal, and flow metering facilities will be housed in an enclosed structure. Air handling equipment will provide a comfortable working environment in the new structure and will be designed to accommodate future odor control.

3.2.2.3 New Final Clarifier

Based on the rate at which influent flows to the WWTF are projected to increase, a fourth Clarifier is recommended for construction at the WWTF. The capacity of the existing Clarifiers is limited to 11.55 mgd and PH flows are estimated above this value throughout the planning period. A fourth Clarifier will not only provide additional capacity to handle projected PH flows, but will also help to relieve the head downstream of the existing Clarifiers at flows above 12.88 mgd.

A fourth Clarifier would also allow for several additional benefits, such as: the ability to take a Clarifier out of service for long-term maintenance without the worry of operating with limited capacity in the event of a wet weather event; ability to operate the Clarifiers under recommended conditions at peak flows; as well as additional capacity for future growth.

3.2.2.4 Additional Improvements

3.2.2.4.1 Re-rating of the WWTF Capacity

As previously noted, the permitted capacity of the WWTF is 2.26 mgd. As influent flows begin to approach this limit, an expansion of the facilities or a re-rating of the permitted capacity for the WWTF may be warranted, depending on the rate at which flows are increasing. Missouri DNR generally requires an expansion of treatment facilities once influent flows reach 80% of the rated capacity of the facilities. In the case of the Belton WWTF, this would mean that once influent flows reach 1.81 mgd (based on 80% of the 2.26 mgd capacity), plans to expand the existing facilities should be implemented. However, if it may be possible to demonstrate that the anticipated rate at which plant flows are projected to increase above the 80% value is unusually slow, and Missouri DNR may allow the City to delay an expansion until a different pre-determined set point is met.

3.2.2.4.2 SCADA System

Most functions of the existing plant are not automated. Those processes that are automated use low-end PLCs, control relays, and hard-wired timers. Monitoring of the various systems is hard-wired to the main control board in the operations building, and emergency alarms are dialed out through a hardware automatic telephone dialer. Some of the new processes will require automation and monitoring that are more sophisticated.

The existing monitoring through the main control board is very limited, and somewhat vulnerable. The hard-wired connections from remote locations (influent pump station and MCC-3) are susceptible to damage from lightning surges, and separate conductors are required for every item to be monitored. Using communications between PLCs, significantly more information can be brought to the operations building. Fiber optic media for the communications will not be susceptible to lightning noise or damage.

To provide more sophisticated displays, manual control and monitoring, a Supervisory Control and Data Acquisition (SCADA) system is recommended. The SCADA system would operate on dedicated personal computer workstations to monitor and log process conditions, provide supervised manual control, manage alarms, and dial out each critical alarm. Hard-wired manual controls would still be provided, but normal manual operations would be through the SCADA, to allow hard-wired controls to be local at the equipment, or at the MCC to reduce field wiring.

3.2.2.4.3 Miscellaneous Improvements

As part of the original construction, a wet weather bypass line was installed upstream of the Aeration Basin to bypass flows directly to the plant outfall during peak wet weather events. However, the practice of bypassing untreated flows is no longer acceptable nor practiced by the City of Belton. For this reason, it is recommended that any valves located on the bypass line be permanently disconnected and removed from service.

Based on recent operating information, peak flows, - up to 6.6 mgd, have been pushed through the WWTF with only a single Clarifier in operation. Based on the recommended parameters for operation of the Final Clarifiers at limited overflow and loading rates, it is recommended that a second clarifier be brought online at peak flows above 3.5 mgd. As a

general practice, it is recommended that each Clarifier be limited to an operating capacity of 3.5 mgd, with a subsequent basin being brought online as flows to each operating basin approach this limit.

3.2.3 Project Phasing

To provide a balance between treatment requirements and project funding, a phased expansion approach is recommended.

The first phase of work, which should be implemented immediately, includes improvements to replace equipment that has reached the end of its useful life and to provide additional capacity at the Influent Pump Station to relieve system overflows. The second phase of improvements is recommended to increase treatment capacity as influent flows approach the hydraulic capacity of the existing system.

3.2.3.1 Phase 1 Project Components

The recommended Phase 1 project consists of construction of a new parallel influent pump station and force main, new Headworks facility, new Final Clarifier, and SCADA system. The miscellaneous improvements, as described above including re-rating of the WWTF and de-commissioning of the existing bypass valve, are also recommended as part of this first phase on construction. The rated capacity of the WWTF will remain at its current capacity.

The estimated cost for construction of these facilities is presented in Table 3.6 below.

Table 3.6 Cost Summary for Recommended Improvements - Phase 1
WWTF Flow Projections & Project Phasing TM
City of Belton, Missouri

Description	Project Costs[1, 2]
New Parallel Pump Station and Force Main	$4,817,000
New Preliminary Treatment Facility	$4,637,000
New Final Clarifier No. 4	$1,985,000
SCADA System	$727,000
Total	**$12,166,000**

Notes:
(1) Opinion of probable construction and project costs. Costs presented in 2012 dollars.
(2) Includes estimating contingency of 20%, general condition contingency of 10%, and engineering, legal, and administration contingency of 20%.

3.2.3.2 Phase 2 Project Components

Phase 2 project components would be implemented to expand treatment capacity at the WWTF as growth continues in the City and the design capacity of the treatment facilities is approached. Phase 2 project components should be assessed based on anticipated growth

for the City, but would generally include expansion facilities as recommended in the most recent Facility Plan (*Carollo,* 2010).

Additional improvements, such as sludge handling and electrical improvements will need to be evaluated at the time of design for the fourth Clarifier to determine if additional improvements are also warranted, based on current plant data.

4.0 FUTURE PLANNING

As City Staff looks forward to future growth in the City of Belton, several key factors can be monitored to evaluate how growth will impact capacity and performance at the WWTF.

There are essentially two approaches to evaluating future growth needs at the WWTF. One approach is to follow Missouri DNR guidelines for planned expansion when 80% of the rated capacity is realized. In this case, design and construction of expansion facilities would be planned to begin when influent flows, for example, reach 1.81 mgd (currently projected prior to Year 2014). It is assumed that the planning, design and construction of expansion facilities will take place over a 4-yr period.

By planning ahead and having the capacity already in place, the City will have the resources available to readily accept future developments up to the expanded capacity. On the downside, the expansion facilities could be in place for years before they are actually needed.

A second approach is to delay expansion until four years prior to influent flows or loads reaching 95% of the rated capacity. In this scenario, planning, design, and construction of expansion facilities should begin four years before influent flows, for example, are projected to reach 2.15 mgd (currently projected at Year 2022). However, the City runs the potential risk of not having the capacity readily available to meet the needs of potential future development ventures.

4.1 Flow & Load Triggers

Several parameters can be monitored as triggers for determining when expansion of the existing facilities is warranted.

4.1.1 <u>Influent Flow</u>

Influent flow is the main parameter that is most easily recognized for evaluating expansion needs, and it is also easily monitored with flow meters at the WWTF. Existing capacity of the current treatment facilities has already been determined and is shown in Table 4.1 below. Thus, as flows begin to approach the rated capacity of each treatment process, the City can evaluate when to begin the process of expanding the necessary facilities to keep up with increasing flows.

Table 4.1 Existing Treatment Process Capacities
WWTF Flow Projections & Project Phasing TM
City of Belton, Missouri

Parameter	Estimated Capacity (mgd) Average Conditions	Peak Conditions	Limiting Parameter
Influent Pump Station (1)	-----	8.18	Equipment Capacity
Grit Removal	-----	6.59	Settling Velocity(4)
Grinding	-----	3.65	Equipment Capacity
Aeration Basin (2)	Existing – 4.47 Future – 3.33	-----	F:M(5)
Secondary Clarifiers(7)	2.81 (3)	11.55	Solids Loading/Surface Overflow(6)
Effluent Flume	-----	27.00	Throat Velocity
UV Disinfection Basin(8)	4.50	16.00	Equipment Capacity
RAS Pump Station(1)	6.94	--	Equipment Capacity
Sludge Holding Tanks	2.86	--	Storage Capacity

Notes:

(1) Assumes largest pump is out-of-service.

(2) Assumes all three flow channels in service. Capacity of existing aeration basin decreases from 4.47 mgd to 3.33 mgd as the influent BOD_5 concentration increases from 129 mg/L to 173 mg/L due to reduced inflow/infiltration from collection system improvements.

(3) Assumes largest clarifier is out-of-service.

(4) Recommended settling velocity is 3.2 - 4.2 fps.

(5) It is assumed the Aeration Basin will be operated at an F:M of 0.10.

(6) Clarifiers are limited by solids loading under average conditions and surface overflow rate at peak conditions. Assumes the Clarifiers are operated at the existing SOR of 1,000 gpd/ft^2 and an SLR of 35 lb/d/ft^2 at peak flows.

(7) Assumes two clarifiers in service during average flow conditions, and all three in service during PH flows.

(8) Assumes both UV channels in service. UV flow channels are expandable to accommodate future capacities up to 12.0 mgd per channel.

Added flows from new developments can be estimated, as previously discussed, based on Missouri DNR design values for specific types of developments. For example, Missouri DNR recommends a design value of 75 – 100 gal/capita/day for residential developments. Although a per capita flow rate of 166 gal/capita/day was used to estimate projected flows throughout the planning period, a design value of 100 gal/capita/day is recommended for estimating flows from new residential developments. Once a proposal for a certain number of residences has been presented, an estimate of the number of persons that will occupy those residences can be assumed, and the flow contribution from the development assessed.

4.1.2 BOD_5 Loading

Biochemical oxygen demand, or BOD_5, is another widely used parameter that is regularly monitored for wastewater. The measurement of BOD_5 concentration at the wastewater facility serves as an indicator of the amount of organic content in the waste stream. BOD_5 concentration is generally evaluated as a mass loading, and can vary depending on the type of contributors to the waste stream at the Belton WWTF.

As previously mentioned, Missouri DNR recommends a per capita value for BOD_5 loading by residential contributors with garbage disposals of 0.22 lbs/capita/day. However, if an industry that contributes waste high in organic content, such as a dairy or food producer, were to connect to the City's collection system, the characteristics of the WWTF influent could vary greatly from what is treated at the WWTF today.

The extended aeration process is impacted by BOD_5 loading. Figure 4.1 illustrates how an increase in BOD_5 loading results in a reduction in Aeration Basin capacity. BOD_5 loading is integral to the organic loading, F:M ratio, and SRT of the extended aeration process.

Based on the design value, as regulated by Missouri DNR guidelines, extended aeration processes should be designed for an organic loading of 15 lbs BOD_5/1000-ft^3/d. With a total volume of 326,000 ft^3 in all three channels of the existing Aeration Basin, the estimated BOD_5 capacity is 4,890 lbs/d. Influent to the WWTF currently averages an MMAD BOD_5 loading of 3,714 lbs BOD_5/d. It is recommended that as BOD_5 loading to the WWTF approaches 95% of the estimated capacity (or 4,646 lbs BOD_5/d), additional aeration capacity should be planned.

Projected flows to the WWTF are estimated based on expansion of residential developments, and accounts for BOD_5 concentrations from additional residential contributors. However, if any other types of developments, - particularly industries producing a high content of organic waste, is proposed for development within the WWTF service area, an estimate of the BOD_5 concentration in their contributing waste should be assessed prior to connecting to the City's collection system. Missouri DNR guidelines can also be consulted for an estimate of BOD loading contribution from schools, institutions, commercial developments. Based on the estimated concentration, the mass loading of BOD can be determined by the following equation:

$$\text{Mass Loading, lbs/d} = BOD_5\,(\text{mg/L}) \times Q\,(\text{mgd}) \times 8.34$$

where:

BOD_5 = Estimated BOD_5 concentration;
Q = influent flowrate to the WWTF;
8.34 is a conversion factor.

Compare the estimated loading from the proposed development to the estimated BOD_5 loading capacity discussed above to evaluate the need for additional aeration capacity based on mass loading.

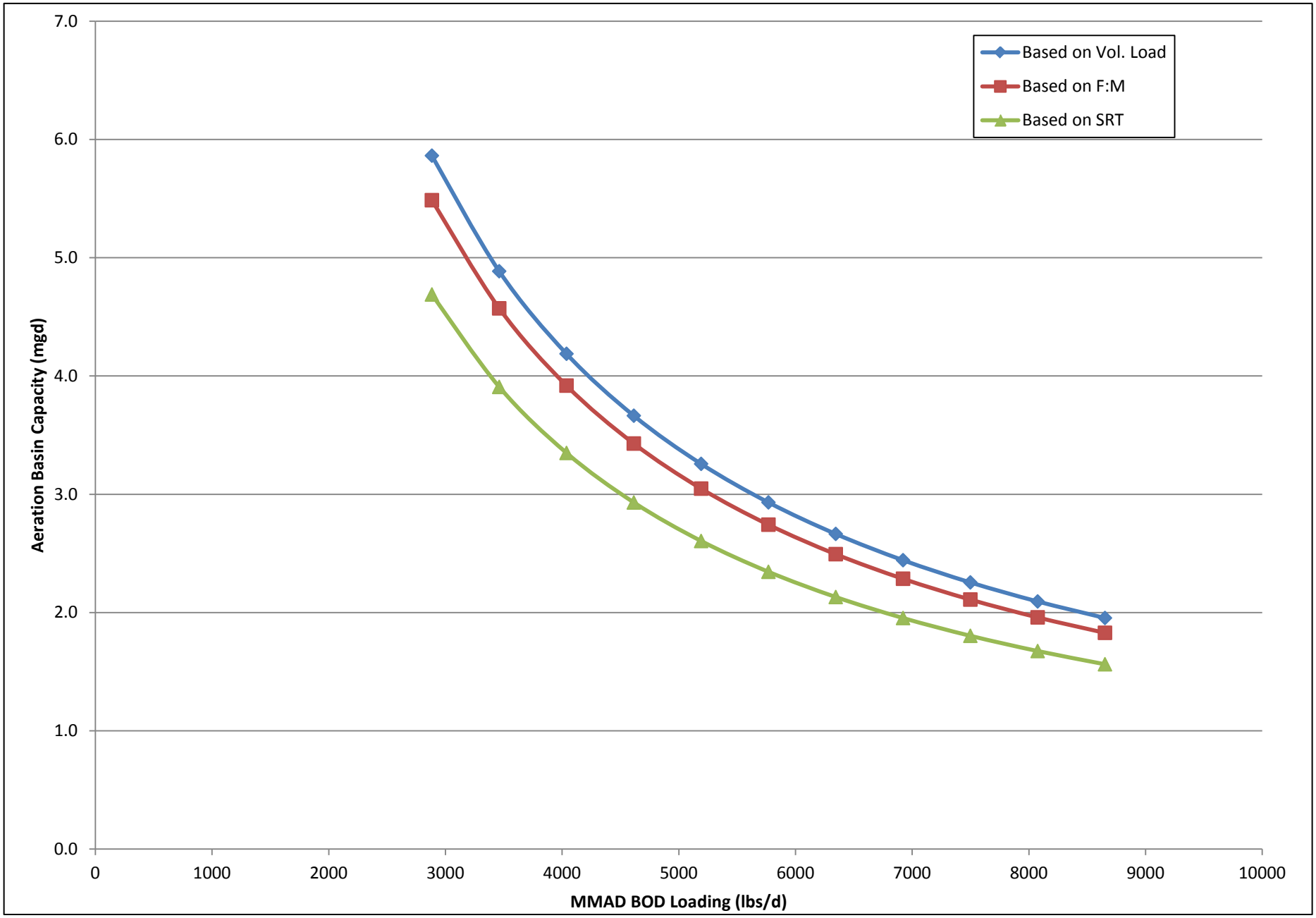

FIGURE 4.1 – BOD_5 LOADING VS. AERATION CAPACITY

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4.1.3 Other Parameters

Other parameters that could be evaluated to assess the impact of new developments on the existing treatment capacity include those used to design new facilities, such as TSS and ammonia concentrations. However, since these values are not currently regulated by Missouri DNR, and are not directly used to size the current treatment facilities, these parameters are difficult to evaluate as triggers for expansion.

However, if TSS and ammonia concentrations at the WWTF begin to exceed normal historical values, an assessment of the treatment capacity is recommended.