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Technical Memorandum #1
Conceptual Design

To:	David Pergrin, Harford County Division of Water & Sewer Chris Skaggs, Northeast Maryland Waste Disposal Authority		
From:	Scott Davis/Wiliam Lai, HDR	Project:	Pumping of Reclaimed Water from Joppatowne WWTP to the NMWDA Waste to Energy Facility
CC:			
Date:	October 19, 2007	Job No:	147-67242

RE: Pumping of Reclaimed Water from Joppatowne Wastewater Treatment Plant to the NMWDA Waste to Energy Facility - Conceptual Design Evaluation

1. Objective

The proposed expansion of the Harford County Waste to Energy (WTE) facility will require a source of water for cooling towers and quenching of ash. It is anticipated that the WTE will require 0.8 to 1.5 MGD of water. A feasibility study is currently underway to evaluate re-using effluent from the Joppatowne wastewater treatment plant (WWTP) as cooling water make up for the proposed WTE. **See Figure 1: Location Map (Attached).**

This technical memo presents data and calculations to determine the design discharge flow rate from the WWTP to the WTE. The design discharge rate will be used to provide preliminary sizing of the pumping system and the force main diameter. The force main will travel approximately 3.5 miles from the WWTP to the proposed WTE, as shown on Figure 1.

The May 30, 2006 version of the Harford County Water and Sewer Design Guidelines was reviewed. Chapter 6, Wastewater Pumping Station Design, was followed for the pump station sizing, force main design, and pump selection.

2. WWTP Flow Data

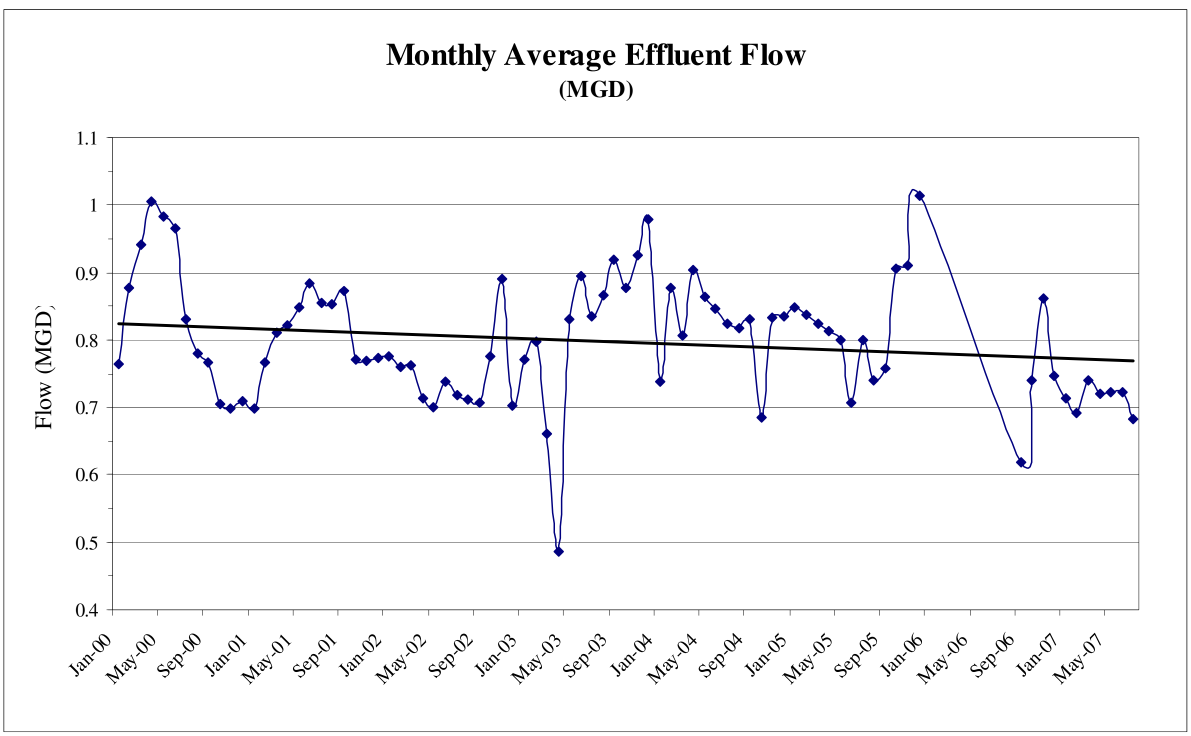
The WWTP service area is largely residential, with some commercial development. Commercial flow is from small shopping centers and fast food restaurants. A pump station within the service area transfers sanitary flow to the Sod Run WWTP service area to reduce influent flow when the Joppatowne WWTP is under maintenance or repair. The plant has an average capacity of 0.95

MGD. The plant monitors flow at two locations. Influent flow is measured with a mag meter on the effluent from the equalization tank. Effluent flow is measured with a Doppler meter at the effluent weir, which determines flow from the height of water over the weir. It should be noted that conversations with the County indicate that the plant has had calibration issues with the effluent Doppler meter and believes the flow information from the influent mag meter may be more reliable. For the purposes of this preliminary study, the data from both meters was reviewed and are acceptable for analysis.

2.1. Historical Effluent Data

Effluent monthly average flow data has been provided from January 2000 through present and is shown in Figure 2 below. The effluent data shows a very slight trend down, although this is attributed to the lower values seen in 2007. The highest effluent flow observed is 2.761 MGD for one day, which occurred in February 2003. The max month peak effluent flow is 1.014 MGD, for December 2005. It is noted that there are higher maximum monthly values in early 2006, however operator notes indicate that the flow meter was not calibrated. From this data, it appears that there is very little change in the population growth within the service area, and that the available flow data represents future flow conditions at the WWTP.

Figure 2: Monthly Average Effluent Data



2.2. *Historical Influent/Effluent Data*

Daily influent and effluent flow data with rainfall have been provided from May 2006 through August 4, 2007, with the exception of December 2006. This daily flow is summarized in Table 1 below. The effluent flow meter data is not reliable from May 2006 through August 2006 due to poor calibration.

Key points observed in the daily flow data are:

1. The influent flow data is significantly lower in 2007 compared to data in 2006. From May 2006 through November 2006, the influent flow averaged 0.913 MGD, compared to an average of 0.583 from January 2007 through July 2007. This flow decrease will not impact the pump station design, as the pump station will be sized to accommodate higher flows from the plant. However, the decrease in flow will impact the amount of water that can be provided to the WTE.
2. The influent flow is also considerably lower than the effluent data. For instance, the July 2007 influent flow is only 60% of the effluent flow.
3. These inconsistencies in the data should be investigated prior to final design of the pump station. It should be determined if the flow decrease is a result of flow being bypassed to the regional WWTP or if the mag meter is out of calibration or malfunctioning.

Table 1: Flow summary (May 1, 2006 through July 31, 2007)

Month	Rainfall (inch)	Influent Flow (MGD)	Max Day (MGD)	Effluent Flow (MGD)	Max Day (MGD)
May 2006	0.29	0.907	1.08	0.979	1.24
June 2006	0.95	0.950	1.69	1.230	2.23
July 2006	0.72	0.915	1.39	1.095	1.62
August 2006	0.14	0.979	1.30	0.856	1.01
September 2006	0.81	0.873	1.43	0.618	1.10
October 2006	0.61	0.911	1.43	0.740	1.18
November 2006	0.65	0.857	1.25	0.862	1.55
December 2006					
January 2007	0.17	0.979	1.49	0.714	1.07
February 2007	0.68	0.791	1.23	0.691	0.84
March 2007	0.72	0.586	0.86	0.739	1.25
April 2007	0.53	0.510	1.33	0.721	1.67
May 07	0.43	0.433	0.89	0.722	0.88
June 07	0.33	0.407	0.52	0.722	1.00
July 07	0.41	0.409	0.56	0.682	0.88
Average		**0.751**	**1.18**	**0.813**	**1.14**
Min Day		**0.160 (April 24, 2007)**		**0.345 (Sept. 22, 2006)**	
Max Day	**3.90 (July 5, 2006)**	**1.687 (June 25, 2006)**		**1.670 (April 16, 2007)**	

The max influent and effluent values in Table 1 both correspond to rain events. As noted above, the effluent data from May 2006 through August 2006 is suspect so higher daily flows in this time period were not considered.

Table 1 includes the peak daily flow per month, and the average of these peak days. Options for sizing the pump station include:

1. Use the peak day observed over the entire flow range. This method will result in the largest pump station.
2. Use an average of the monthly peak day values. This method does not account for numerous high daily flows that occurred within the same month.
3. Use an average of the five highest peak day values. This method will provide a value between options 1 and 2.
4. Use the peak 7-day average daily value. This value represents the highest daily average flow observed over a seven day period.

Figure 3A: WWTP Influent Flow

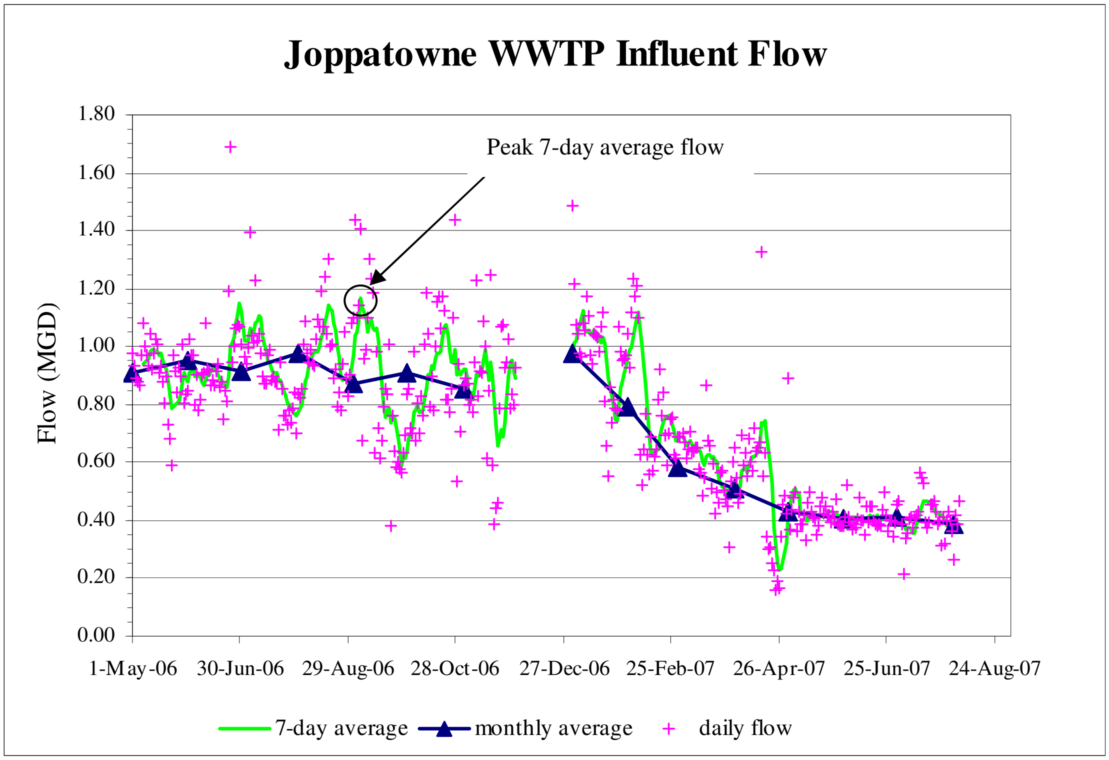
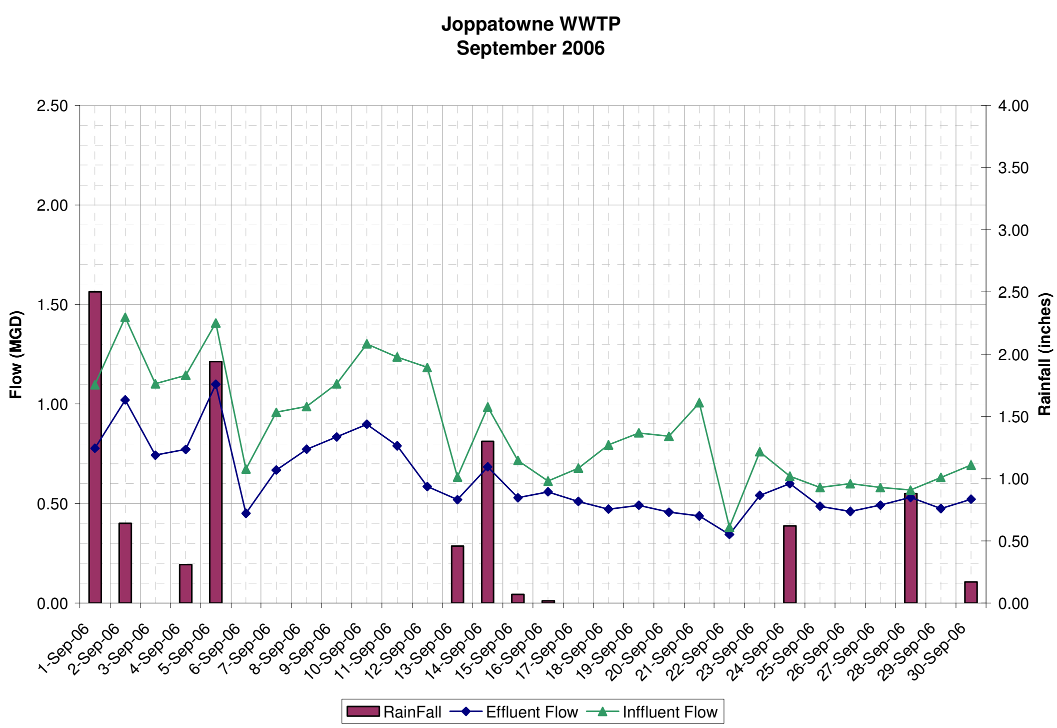


Figure 3B: WWTP Influent Flow – September 2006



Using the data from May 2006 through present, it would appear that a reasonable maximum flow for sizing the pump station is 1.2 MGD. This value is based on the highest seven day average flow observed at the WWTP, which is 1.17 MGD, occurring the last days of October 2006 and beginning of September 2006. Higher single day flows are likely due to extreme rain events, but sizing the pump station to accommodate these rare high flow events will result in a larger pumps and a bigger facility to account for a peak flow rate that may occur once every few months. Using an average of the five highest flow events would result in a 1.5 MGD pump station, which is 25% larger than the proposed 1.2 MGD pump station.

The minimum daily flow into the plant is 0.16 MGD on the influent side, with a weekly minimum average daily flow of 0.23 MGD. The minimum daily flow on the effluent side is 0.35 MGD, with a minimum weekly average of 0.39 MGD. The minimum flows must be verified prior to committing a definitive flow to the proposed WTE.

Based on the effluent diurnal flow data, the minimal flow typically occurs during the early morning hours when human activity is very low. Seven diurnal flow charts were reviewed and are from weekly periods in June and July 2006 and January, April, June, and July 2007. The diurnal flow curves show peaks in the morning and evening, which are typical of largely residential areas. Hourly peaks reach 1.5 MGD, while the low flow periods are approximately 0.25 MGD. There are higher effluent hourly peaks in the 2006 data, however this was the time period during which the effluent flow meter was not calibrated properly. The low flow period occurs for a period of 3 to 6 hours overnight. Therefore, the available effluent flow for reuse will not be able to match the demand for cooling water make up during this period of time without installation of a holding tank at the WTE and another water source to supplement the effluent flow from the WWTP. Hourly effluent data is required to determine the extent to which the equalization tank reduces these flow peaks.

2.3. Flow Summary

The pump station should be sized to accommodate 1.2 MGD of flow, which can convey the peak seven day average observed in the daily flow data from May 2006 through July 31, 2007. Average flow seen in the influent during this time period is 0.75 MGD with a peak of 1.67 MGD and a minimum of 0.16 MGD, although there is a noticeable downward trend in the 2007 data. From May 2006 through November 2006, the influent flow averaged 0.913 MGD, compared to an average of 0.583 from January 2007 through July 2007. The diurnal flow charts show peaks reaching 1.5, however these peaks are of short duration.

3. WTE Influent Requirements

It is anticipated that the WTE will require 0.8 to 1.5 MGD of cooling tower makeup water, and that the demand for the cooling water will likely be continuous day and night. For that reason, a storage tank may be required at the WTE to supply cooling water makeup during low flow periods. A 500,000 gallon tank with a 50 foot diameter would be approximately 35 feet tall. At 1.0 MGD of cooling water demand and effluent flow, the tank would provide 12 hours, or half a day, of cooling water. Proposals with various sizes and flow requirements for the WTE expansion have been submitted. Key issues for the WTE include:

1. Corrosion, which occurs due to dissolved metals in the cooling water;
2. Scaling, which is caused by deposition of minerals such as calcium;
3. Biological fouling, which can lead to corrosion or plugging of heat exchangers or cooling towers;

Table 2 contains the WTE water quality requirements of a similar facility. In addition to these parameters, suspended and dissolved solids can impact the WTE plant performance. Complete water quality standards must be provided by the Harford County prior to determining what pre-treatment would be needed at the proposed WTE, if effluent reuse was deemed feasible.

Table 2: WTE water quality requirements

parameter	min	max
pH	5	8
(ideal pH)	7.0	7.3
Phosphates (mg/L)		65
Chlorine (mg/L)		0.1 to 0.3

Although the WTE requires a low chlorine concentration, the WWTP effluent must be chlorinated upon entering the reuse pump station to prevent microbial growth in the pump station and force main. Dechlorination, and additional pre-treatment, may be required at the WTE if the residual chlorine concentration is too high.

A sampling plan has been provided to the County. HDR recommends additional WWTP effluent sampling, over a two week period, of the following parameters: TDS, conductivity, hardness, turbidity, orthophosphates, ammonia, pH, chlorine, sulfate, and alkalinity. This sampling will ensure that the WWTP effluent can meet the anticipated WTE influent requirements. It should be noted that WWTP effluent quality could decrease at high flows.

Operational sampling of the WWTP effluent will be required upon startup. Sample frequency and analysis parameters will be negotiated with the WTE.

4. Pump Station Location Alternatives

The effluent reuse pump station must be kept within the boundaries of the existing treatment plant. The WWTP may be required to achieve enhanced nutrient removal (ENR) in the future, which will require upgraded treatment processes and will require open space.

The WWTP currently has screening and grit removal, followed by an equalization tank. Flow from the EQ tank is directed to three reactor/clarifiers. The clarifier supernatant is directed to the new chlorine contact tank. The new chlorine contact tank is located in the western corner of the site. Effluent from the new chlorine tank is directed to the old chlorine contact tank, located beneath the utility water building. Dechlorination is provided at the end of the old chlorine contact tank prior to discharge to the outfall. An effluent weir maintains the water level in the older contact tank.

Key issues for the final effluent pump station design are:

1. Ability for the contact tank effluent to flow by gravity from the chlorine contact tank to the pump station;
2. Maintaining the required chlorine contact time, specifically if the second contact tank is converted to a pump station or if it must be bypassed to achieve gravity flow to a new pump station. This could be achieved by increasing the height of the walls or by adding additional channels to the newer contact tank. If the wall height is increased, a hydraulic profile would be required to ensure there is adequate head upstream of the contact tank.
3. Ability for the treated effluent/stored water to flow by gravity to the outfall in events of high flow, when either the WTE is not calling for water or the level in the pump station reaches a high level.
4. Ability to provide dechlorination of the treated effluent/stored water prior to discharge via the outfall. The older contact tank currently has the plant's primary dechlorination point.

The options for installing an effluent reuse pump station are discussed below. The screening evaluation for the best alternative will be performed subsequent to County review and approval of these options. The pump station locations are shown in **Figure 4 (attached).**

4.1. Pump Station Alternative 1

The WWTP currently has two chlorine contact tanks operating in series to provide the appropriate contact time. The older contact tank is located beneath the Chlorine/Utility Water Building in the western portion of the site. This tank would make an ideal wet well for collecting the treated effluent as the tank is existing and would not require any land use. Minimizing the land use for the pump station is important to ensure there is adequate space for ENR expansion. In addition, the contact tank has an existing influent line from the new contact tank and an outfall to the river for high flow conditions. The newer chlorine contact tank would require modifications to provide the required contact time. Figure 5 below shows the chlorine contact tank building.

4.2. Pump Station Alternative 2

A new pump station/wet well could be constructed on the site. The pump station will require an inlet line from the chlorine contact tank and an effluent pipe to the outfall for cases of high flow. This is not an ideal option as it would use land that may be required for ENR expansion. It is estimated that this new configuration will occupy approximately a footprint of 15' x 15'. In addition, the site has high groundwater levels, which will require dewatering during construction of a new pump station/wet well.

4.3. Pump Station Alternative 3

A wet well with submersible pumps could be added adjacent to the effluent box of the older chlorine contact tank. The site area is limited in this corner, but would not likely affect the ENR expansion. The County has indicated flooding in this corner of the site up to the fenceline.

Figure 5: Chlorine Contact/Utility Water Building, with new chlorine contact tank on the right

5. Pump Station Sizing and Pump Selection

As noted above, the pump station should be sized to transfer a maximum flow of 1.2 MGD. The minimum pump cycling time, per the Harford County guidelines, is 10 minutes. With VFDs, the pump will not cycle on and off with level but will run at varying speeds to maintain the level in the tank. Therefore, the minimum cycle time is not applicable. To provide a 10 minute holding time, the pump station must have an operating volume of 8,350 gallons.

In order to convey the WWTP effluent to the proposed WTE, the effluent will need to be pumped approximately 3.5 to 4 miles, depending on the piping route selected. There is an elevation increase of approximately 37 ft from the WWTP to the WTE. The WWTP is located at 14.3 ft above sea level, and the WTE is at 51.6 ft. In addition, there will be increased static head if there is a holding tank at the MTE. For instance, a 500,000 gal storage tank with a 50 ft diameter would be approximately 35 ft tall, with freeboard. This would increase the static head on the pump from 37 ft to 72 ft. The storage tank sizing at the WTE is required to complete the pump sizing.

The pump station will have two pumps, one operating and one on standby. Each pump will be capable of handling the maximum flow. Suction lift pumps, located above the wet well in the utility building, are suggested regardless of the motor size. Harford County sewer guidelines indicate that for motor sizes greater than 40 hp, a wet well/dry well pumping station is required. However, due to the water quality of the effluent flow and the available options for pump station installation at the WWTP, it is recommended that the best pump station design relative to cost and operation be selected for effluent reuse pumping, as opposed to County design guidelines. A pump curve, multi-speed performance curve, and dimensions sheet for the proposed pump are presented in Figures 6, 7, & 8 below. The proposed pump has a 50 hp motor and 7.63 inch diameter impeller and can pump 1.2 MGD at 156 ft of head. The specified pumps can operate within a range approximately 800 rpm to 1,700 rpm, translating into a minimum flow of approx. 300 gpm to a maximum flow of 850 gpm.

The pump station will have level indicating transmitters. The level controls will operate the pumps. A high level setpoint will start the lead pump. A high-high level will stop the lead pump and start the lag pump. During pump operation, a low level setpoint will stop the pump. A separate, redundant low-low level switch hardwired to the pumps will stop them in the event the low level setpoint does not. If flow exceeds the high-high setpoint, it will overflow to the outfall. The control system will activate the dechlorination system during overflow conditions to ensure the residual Cl level is not exceeded in the effluent. In addition to these effluent reuse pump station controls, there will be a level indicator at the WTE facility storage tank indicating the volume in the tank. If the water level is above an operator selectable high level setpoint, the WWTP effluent pumps will be locked out, stopping water supply. If the water level is below an operator selectable setpoint, the WWTP effluent pumps will be called on. These two separate control systems will be integrated to best suit the needs of the WTE facility.

Figure 6: Proposed Pump Curve

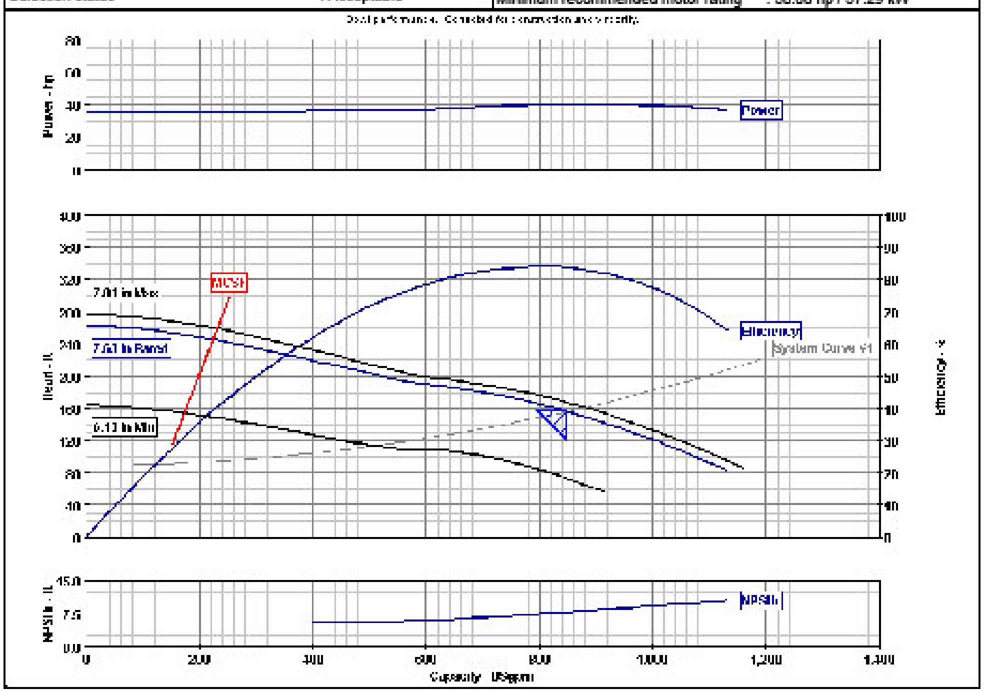
Reiner Pump Systems, Inc.

Pump Performance Datasheet

Customer	: REINER PUMP SYSTEMS, INC.	Quote number	: 42464
Customer reference	:	Pump size	: 12DKH
Item number	: 003	Stages	: 7
Service	: 850 GPM @ 156 FT TDH	Based on curve number	: 12DKH 1170 Rev. 0
Quantity of pumps	: 1	Date last saved	: 04 Oct 2007 12:17 PM

Operating Conditions		Liquid	
Flow, rated	: 850.0 USgpm	Liquid type	: Water - Clean
Head, rated (requested)	: 156.0 ft	Additional liquid description	:
Head, rated (actual)	: 156.4 ft	Solids diameter, max	: 0.00 in
Suction pressure, rated / max	: 0.00 / 0.00 psi.g	Temperature, max	: 68.00 deg F
NPSH available, rated	: Ample	Fluid density, rated / max	: 0.998 / 0.998 SG
Frequency	: 60 Hz	Viscosity, rated	: 1.00 cP
		-	:

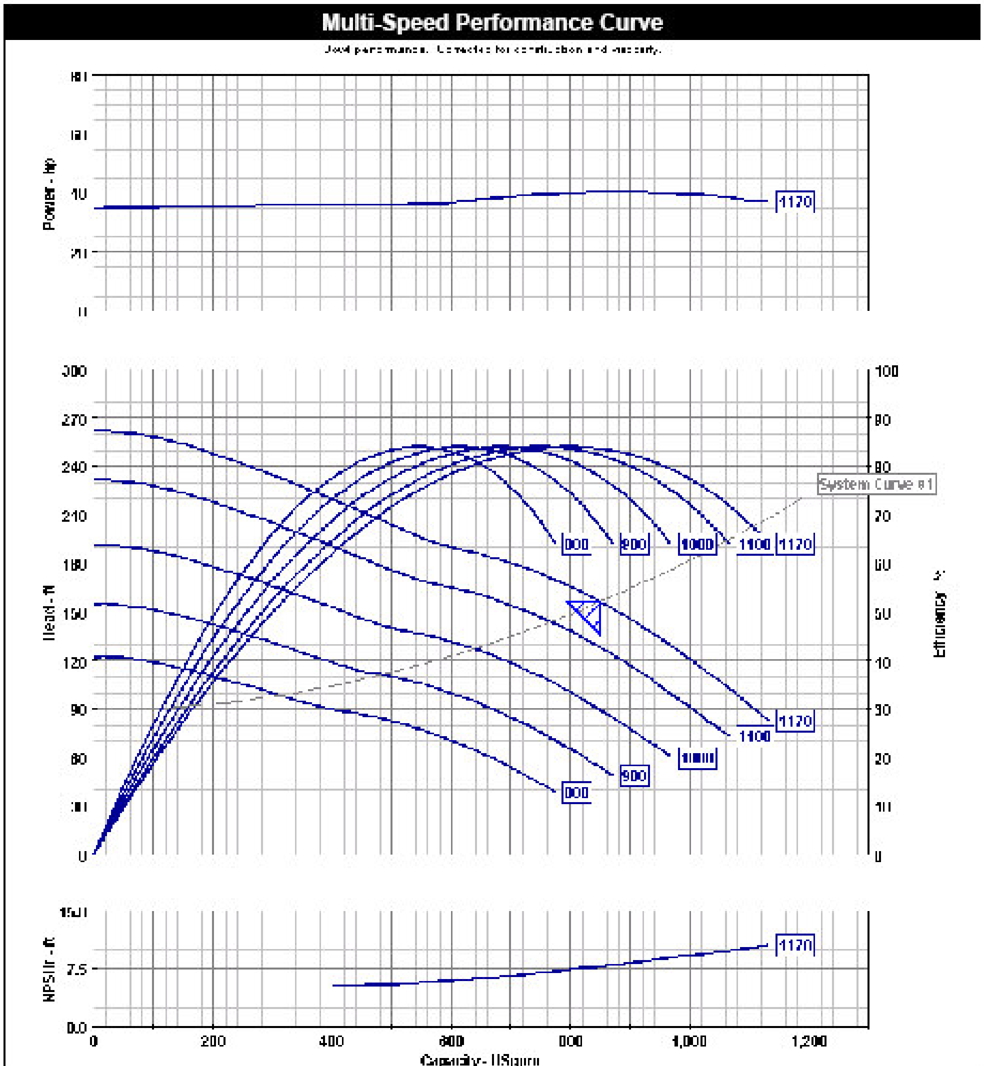
Performance		Material	
Pump speed, rated	: 1,170 rpm	Material requested	: Auto
Impeller diameter, rated	: 7.63 in	Material selected	: Cast Iron/Bronze
Impeller diameter, maximum	: 7.81 in	**Pressure Data**	
Impeller diameter, minimum	: 6.13 in	Maximum working pressure	: 113.1 psi.g
Efficiency (bowl / pump)	: 83.53 / 82.91 %	Maximum allowable working pressure	: 380.0 psi.g
NPSH required / margin required	: 7.75 / 0.00 ft	Maximum allowable suction pressure	: N/A
Specific speed / Suction specific speed	: 3,045 / 7,395 US units	Hydrostatic test pressure	: N/A
MCSF	: 225.3 USgpm	**Driver & Power Data**	
Head, maximum, rated diameter	: 261.3 ft	Driver sizing specification	: Maximum power
Head rise to shutoff	: 67.40 %	Margin over specification	: 0.00 %
Flow, best eff. point (BEP)	: 801.0 USgpm	Service factor	: 1.15
Flow ratio (rated / BEP)	: 106.11 %	Power, hydraulic	: 33.42 hp
Diameter ratio (rated / max)	: 97.59 %	Power (bowl / pump)	: 40.01 / 40.20 hp
Head ratio (rated dia / max dia)	: 93.40 %	Power, maximum, rated diameter	: 40.26 hp
Viscous coefficients (CQ / CH / CE)	: 1.00 / 1.00 / 1.00	Minimum recommended motor rating	: 50.00 hp / 37.29 kW
Selection status	: Acceptable		



Reiner Pump Systems, Inc. Quote No. 42464 — 04 Oct 2007 12:17 PM

Figure 7: Multi-Speed Performance Curve

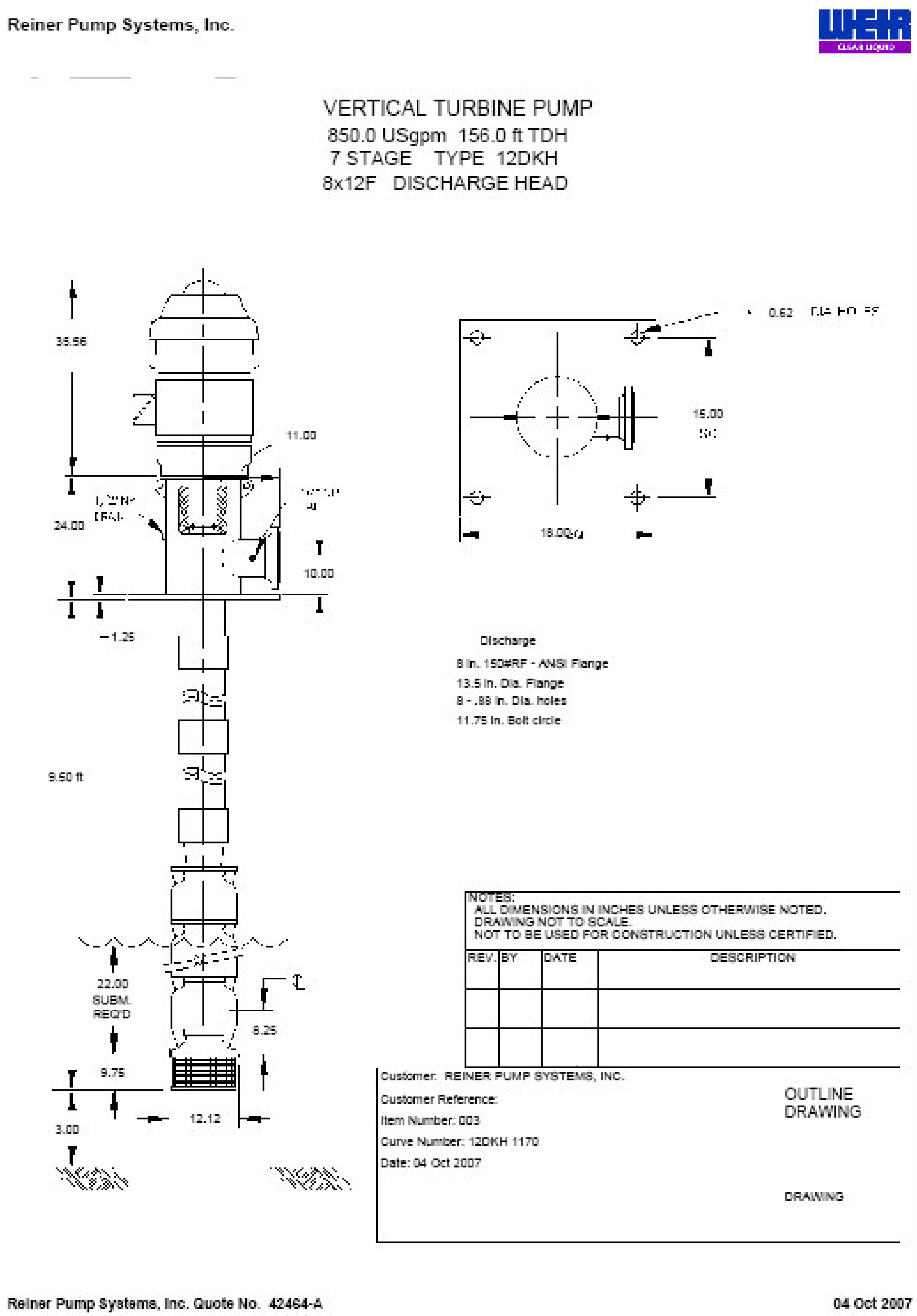
Reiner Pump Systems, Inc.



Customer	: REINER PUMP SYSTEMS, INC.	Pump size	: 12DKH
Customer reference	:	Stages	: 7
Item number	: 003	Pump speed, rated	: 1,170 rpm
Service	: 850 GPM @ 156 FT TDH	Based on curve number	: 12DKH 1170 Rev. 0
Quantity of pumps	: 1	Flow, rated	: 850.0 USgpm
Quote number	: 42464	Head, rated	: 156.0 ft
Date last saved	: 04 Oct 2007 12:17 PM	Fluid density, rated / max	: 0.998 / 0.998 SG
-	:	Viscosity	: 1.00 cP
-	:	CQ / CH / CE	: 1.00 / 1.00 / 1.00
-	:	Impeller diameter, rated	: 7.63 in

Reiner Pump Systems, Inc. Quote No. 42464 04 Oct 2007 12:17 PM

Figure 8: Proposed Pump Dimensions



6. Preliminary Force Main Sizing

The force main velocity should be between 3 to 6 ft/sec per the Harford County guidelines. While the Harford County guidelines indicate that force mains greater than 4 inch diameter must be ductile iron pipe (DIP), it is recommended that HDPE be used for the force main, due to the effluent quality, ease of installation, lower roughness coefficient and lower cost compared to a DIP main. Table 3 below shows the anticipated velocity, at 1.2 MGD and 0.8 MGD in a variety of available pipe diameters. Since VFDs will be used for the pumps, the velocity will be lower than 3 ft/sec during low plant flows. Although lower than the Harford County guidelines, the low velocity is suggested to prevent solids from settling in the pipe. There should be minimal solids in the WWTP treated effluent, so the lower velocity should be acceptable.

At the peak sizing flow of 1.2 MGD from the WWTP, an 8 inch diameter force main is required to keep the velocity below 6 ft/sec. At the lower flow rate of 0.8 MGD, the velocity is 3.55 ft/sec in the 8 inch pipe. During low flow periods, such as late at night, the velocity in the pipe will be less than 3 ft/sec. Below 0.68 MGD, the velocity in an 8 inch diameter pipe will drop below 3 ft/sec. To provide capacity if future additional flow is to be conveyed in the force main, a 10 inch diameter pipe is suggested.

Table 3: Pipe velocity

Pipe Diameter (inch)	**Velocity at 1.2 MGD (ft/s)**	**Velocity at 0.8 MGD (ft/s)**
6	9.46	6.30
8	5.32	3.55
10	3.40	2.27
12	2.36	1.58

In order to best analyze possible force main routes, a screening matrix was developed to best compare routes against one another. Many factors are taken into account in this matrix, primarily community impacts, construction difficulty, land acquisition implications, environmental restraints, and eventually, cost. Force main route alternatives will be presented in a future task that will commence shortly. The force main in all alternatives will be equipped with inspection ports/manholes, blow offs, and flushing hydrants as required. Figure 9 shows the utility corridor, which could be utilized for the new force main.

Figure 9: Utility corridor

7. Summary

The Joppatowne WWTP is rated for 0.95 MGD. Flow data provided by the WWTP shows a peak seven day average flow rate of 1.2 MGD, which was used to size the pump station. The pump station location will be investigated further. Dual 50 hp pumps will be installed to convey the treated effluent through a 10" diameter HDPE pipe. The pumps will be suction lift and will operate with VFDs. Each pump will be rated for the entire 1.2 MGD at the design head of approximately 156 ft. The force main will travel approximately 3.5 miles and gain a minimum of 37 ft in elevation to the WTE. The discharge elevation at the WTE is required prior to sizing the pumps, as the head could increase if a larger storage tank is installed at the WTE.